CARBON FIBER REINFORCED POLYMER (CFRP) SYSTEM FOR EXTERNAL STRENGTHENING OF CONCRETE BRIDGE GIRDER

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Abstract

There is a need for Malaysia to strengthen her bridges before she can relax the weight restriction limits on her bridges. Several strengthening techniques are available and Fiber Reinforced Polymer (FRP), in particular Carbon Fiber Reinforced Polymer (CFRP) appears to be promising. JKR does not have any experience in FRP despite many studies around the world in this new material. A committee has been set up by JKR in association with UTM and some other interested parties from the industries to gain more knowledge in the use of CFRP in bridge strengthening. This paper reports the results of the first phase of the study, which involves measuring the performance of CFRP on a bridge.

INTRODUCTION

The Need for Bridge Strengthening in Malaysia

The Public Works Department of Malaysia (JKR) has, in its custody, some 90 percent of the nation's bridges. A recent inspection program recorded more than 2,370 bridges and 4,500 culverts along Federal Trunk Routes in Peninsular Malaysia. Reinforced concrete "slab on girder" bridges constitute about 70 percent of the existing bridges.

The National Axle Load Study (GOM 1989) conducted in 1986-88 had identified that bridges and culverts pose as the weakest links in the national road network. As a result, the Weight Restriction Orders 1989 and its amendments impose much restriction to the vehicles. Many of the identified 'weak' bridges are actually in good physical condition. Their low load-carrying capacities were largely due to their being designed to a lower loading specification.

Unnecessarily restrictive control of the vehicles imposes much constraint to the national economy. However, before the legal load limit may be raised these bridges and culvert have to be strengthened. A number of bridge strengthening techniques are available. Externally bonding Fiber Reinforced Polymer (FRP) onto existing bridge beams is one of them. This technique appears to be a promising technique due to the fact that FRP has a high strength to weight and stiffness to weight ratios, excellent fatigue properties and outstanding corrosion resistance. These properties have made FRP more favorable than steel plates in flexural strength enhancement by external bonding.

Use of Fiber Reinforced Polymers in Malaysia

FRPs are composites that essentially consist of high strength fibers bonded to a continuous polymer or resin matrix. Three types of FRP fibers are commercially available: Carbon, Aramide and Glass. Each fiber has benefits and drawbacks in relation to its performance for different applications. Early research in FRP were conducted in Japan, Switzerland and Germany; joined by the U. S., Canada and Saudi Arabia in the early 1990s. In Japan, Carbon FRP or CFRP is mainly used for seismic retrofitting of bridges, buildings, tunnels and other structures. In Europe, research in the use of FRP focus on beams and slabs.

Use of FRP in Malaysia is very new and has been restricted to strengthening work on buildings. JKR has hitherto not been strengthening structures due mainly to more pressing need in restoring deteriorated structures rather than increasing the load-carrying capacity of these structures. In rare occasions where structures were to be strengthened JKR was hesitant to use FRP because of the lack of experience in this material. Cost was also a major consideration.

Prices of FRP are dropping each year. JKR has felt the need to investigate and promote the use of this technology in Malaysia. A committee was thus set up in association with Universiti Teknologi Malaysia (UTM) and other interested parties from the industries to gain more knowledge in the use of FRP in bridge strengthening. Besides serving as a forum for its members to exchange experience and literature the committee also embarks on a study funded by Ministry of Science, Technology & Environment and contributions from the industries. The study comprises two phases; the first phase involves a field test on a JKR bridge and the second phase involves laboratory tests to be carried out in UTM. In the field study, only the performance of CFRP in flexural strengthening of R. C. girders is investigated. In the laboratory, both shear and flexural strengthening by CFRP are studied.

This paper will report the purpose, procedure and results of the load tests carried out on bridge FT009/035/80 in Kuala Pilah before and after the installation of CFRP. Its main aim is to publicize the Committee's efforts and promote the FRP technology in Malaysia. The second phase of the study is underway at the time of writing this paper and the results will be reported separately elsewhere.

BRIDGE LOAD TESTS

A seven-span simple R. C. bridge in Kuala Pilah Town was selected for the load test (Fig. 1). The deck comprises two independent superstructure of 3 rectangular R.C. beams cast monolithic with an R.C. slab (Fig 2). Estimated in-situ concrete cube strength based on seven cores taken from a beam of each span gave a range of between 20.5 N/mm^2 and 33 N/mm^2. Rebound hammer tests showed that concrete strength for each span are rather uniform.

Each span of the bridge was load tested before and after installation of CFRP. The first load test was done on 2.5.2000 before installation of CFRP. After the load test, six products were installed separately by their agents on beams B3 and B4 of spans S1-S6. These products were equally divided between CFRP sheets and CFRP laminates. Span S7 was not installed with any CFRP and offered an opportunity later to verify the results of the first load test by a re-test. The second load test was done on 13.5.2000.

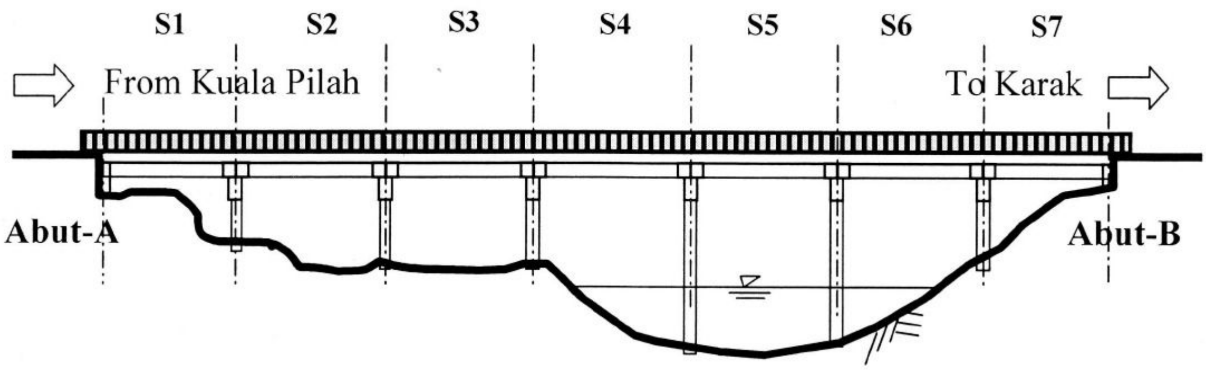


Fig. 1: Longitudinal section of the bridge

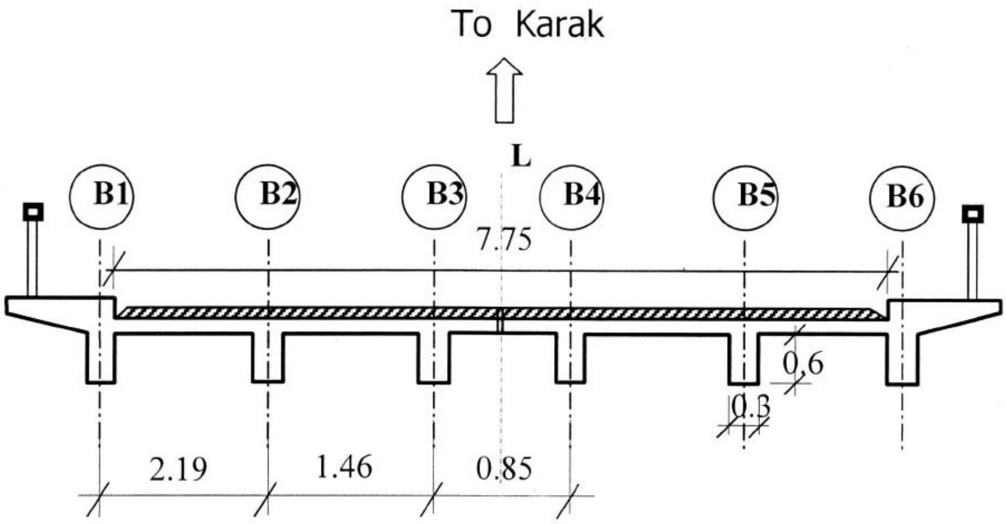


Fig. 2: Cross-section of the bridge

Instrumentation

In the first load test, strain gauges were installed at three locations at the vertical side of beams 3 and 4 as shown in Fig 3. Additional strain gauges were installed on the CFRP in the second load test. Linear Variable Displacement Transducers (LVDTs) were installed at the mid-span of each of the two beams on independent supports to measure maximum deflection of the beams due to each load case. The readings from the strain gauges and LVDTs were recorded via connections to two portable data loggers with multiplexors on each side of the river. Instrumentation on Span 1, 2, 3 and 4 were connected to the data logger stationed at the Kuala Pilah riverbank while instrumentation on Span 5, 6, and 7 were connected to the data logger stationed at the Karak riverbank. The arrangement of two data loggers was to avoid errors in readings due to long wires. Concurrently, precise level measurements were also taken to check for settlement at the supports as well as a counter check for the LVDT readings.

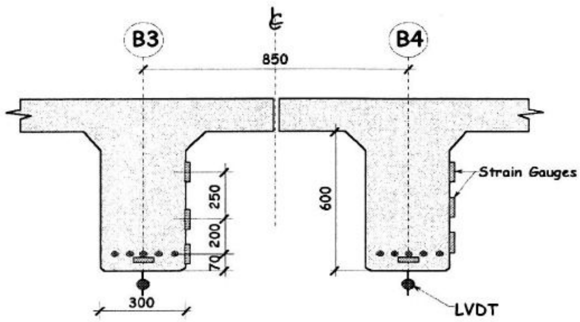


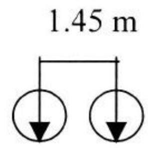
Fig. 3: Location of strain gauges and LVDT

Applied Loading

A low-loader with concrete blocks loading on it was used as a test vehicle. The low-loader was so positioned during the load testing that only the tandem axles of the low-loader were loaded on the span of interest. Each tandem axle has two wheels 1.83 m apart. Four levels of loading were carried out and the weight on each axle for each load level is given in Table 1. The loading procedures are discussed in (Ku Mohd. Sani & Ng 2000).

Table 1: Axle Loads and Vehicle Loads

Load	Axle Load, kN	
Level	Front axle	Rear axle
I	112	112
II	152	155
III	176	179
IV*	190	195



* Load Level IV was applied in the second load test only.

RESULTS AND DISCUSSIONS

Peculiar results were observed from the first load test. Table 2 shows results for span S7. Strain readings for Load Level III were found to be lower than that for Load Level II, which defied logic. Also, strains measured at the steel differed from that in the concrete at the same level, that is, 70 mm from the beam soffit. The deflections, however, are consistent with theoretical calculations, albeit, with a margin.

Table 2: Results of first load test in span S7

Load Level	**Strain (x 10^{-6})** At steel	At 70	At 270	At 520	**Deflection At mid-span (mm)**
I	269	374	369	216	-0.96
II	554	682	664	484	-1.23
III	525	673	665	449	-1.34

It is important to rationalize the results. Two possible explanations were considered: 1) Bearings were frozen and as a result contribute to the load-bearing capacity of the beams, as reported in Bakht & Jaeger (1988); 2) Loading procedure that involves loading each span in turn amounts to cyclic loading. 3) Presence of hairline shrinkage cracks at the beams prior to loading might have widened during higher loading at Load Level III.

It was thus decided that span S7 be re-tested with special arrangement to pinpoint the causes of the anomaly. First, additional strain gauges were installed at the bottom of beam B3 close to support P1. One gauge was installed at the underside of the beam while the other was on the vertical face at an angle. Second, load increment was made by adding concrete blocks over those already on the low loader without first removing the loading from the bridge. The results are presented in Table 3.

Table 3: Additional tests on span S7 to check anomaly of first load test results

			Strain (x 10^{-6})				**Deflection**
Load Level	**At steel**	**At 70**	**At 270**	**At 520**	**At support (bottom fiber)**	**At support (at an angle near soffit)**	**At mid-span (mm)**
I	123	244	252	63	87	71	-0.98
II	234	397	403	290	171	142	-1.36
III	230	420	422	269	126	113	-1.57

From the results presented in Table 3 it was noted that:

i. The strains measured at the support P1 are tensile indicating that the arching effect expected of frozen bearings are not present.

ii. The phenomenon of reduction in strain measurement from Load Level II to Load Level III persists for strain measurements at the steel reinforcement and on concrete surface at level 520 mm from the beam soffit.

iii. Repeatability for strain measurement comparing Table 2 and Table 3 is poor.

iv. Repeatability for deflection measurement is good.

Obviously, strains were not quite correctly measured during the test, apparently due to presence of shrinkage cracks prior to application of test loads. Notwithstanding, spans S1-S6 were subsequently test loaded following the original loading sequence to check any improvement in flexural strength by installation of CFRP.

Comparisons of Strain Readings Before and After CFRP Installation

The results of the strain readings and deflections with and without CFRP for selected spans are presented in Tables 4-7. Strains and deflections for spans S2 and S6 are plotted in Figures 4-13 to show differences between beams with and without CFRP. Comparisons between CFRP sheets and laminates can also be made.

Table 4: Strain results for span S2 (Before and after CFRP installation)

	Span 2 Beam 3 (CFRP **sheets**)					
	Load Level I		Load Level II		Load Level III	
	Before	After	Before	After	Before	After
At steel	101 mμ	81 mμ	537 mμ	153 mμ	431 mμ	90 mμ
At 70	172 mμ	164 mμ	532 mμ	254 mμ	495 mμ	227 mμ
At 270	159 mμ	111 mμ	601 mμ	210 mμ	536 mμ	175 mμ
At 520	34 mμ	-11 mμ	368 mμ	42 mμ	297 mμ	-13 mμ
Deflection	-0.75 mm	-0.73 mm	-1.14 mm	-0.97 mm	-1.23 mm	-1.04 mm
At CFRP (ave.)	-	159 mμ	-	112 mμ	-	92 mμ

Table 5: Strain results for span S3 (Before and after CFRP installation)

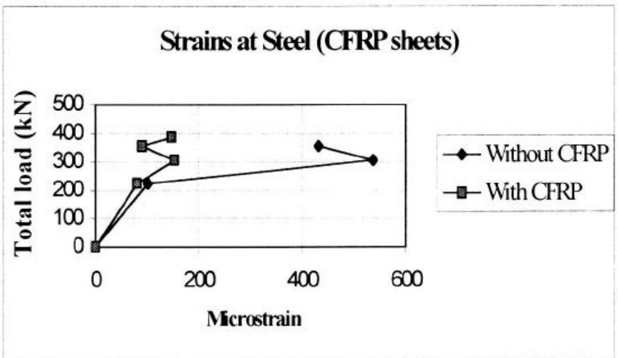
	Span 3 Beam 3 (CFRP **fabrics**)					
	Load Level I		Load Level II		Load Level III	
	Before	After	Before	After	Before	After
At steel	137 mμ	121 mμ	577 mμ	67 mμ	515 mμ	37 mμ
At 70	97 mμ	-6 mμ	475 mμ	5 mμ	423 mμ	7 mμ
At 270	79 mμ	69 mμ	448 mμ	12 mμ	388 mμ	-15 mμ
At 520	81 mμ	37 mμ	467 mμ	-23 mμ	403 m@	-53 mμ
Deflection	-0.96 mm	-0.93 mm	-1.26 mm	-1.31 mm	-1.21 mm	-1.4 mm
At CFRP (ave.)	-	197 mμ	-	144 mμ	-	56 mμ

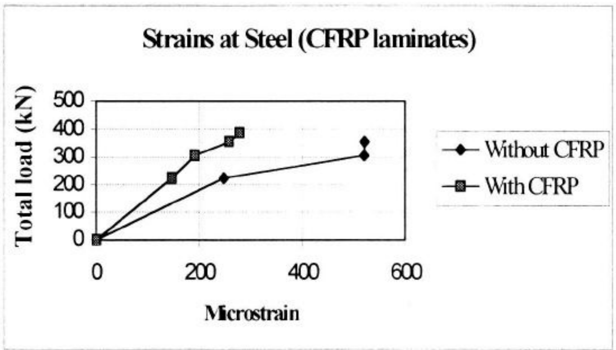
Table 6: Strain results for span S4 (Before and after CFRP installation)

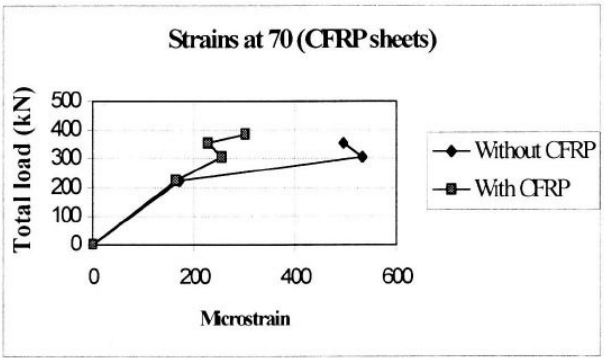
	Span 4 Beam 3 (CFRP **laminates**)					
	Load Level I		Load Level II		Load Level III	
	Before	After	Before	After	Before	After
At steel	151 mμ	27 mμ	539 mμ	42 mμ	448 mμ	58 mμ
At 70	106 mμ	-56 mμ	487 mμ	-73 mμ	373 mμ	-48 mμ
At 270	107 mμ	-54 mμ	508 mμ	-74 mμ	375 mμ	-47 mμ
At 520	93 mμ	-72 mμ	492 mμ	-74 mμ	394 mμ	-50 mμ
Deflection	-1.01 mm	-0.95 mm	-1.29 mm	-1.28 mm	-1.31 mm	-1.4 mm
At CFRP (ave.)	-	22 mμ	-	-4 mμ	-	22 mμ

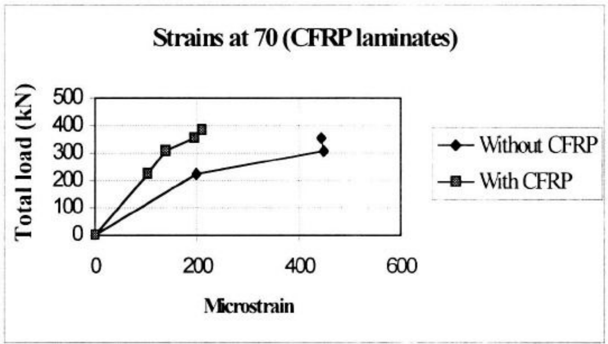
Table 7: Strain results for span S6 (Before and after CFRP installation)

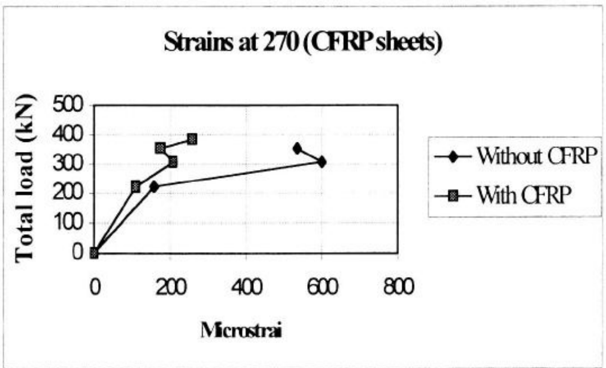
	Span 6 Beam 3 (CFRP **laminates**)					
	Load Level I		Load Level II		Load Level III	
	Before	After	Before	After	Before	After
At steel	247 mμ	147 mμ	520 mμ	191 mμ	521 mμ	258 mμ
At 70	198 mμ	103 mμ	448 mμ	138 mμ	443 mμ	194 mμ
At 270	190 mμ	99 mμ	442 mμ	128 mμ	434 mμ	185 mμ
At 520	198 mμ	101 mμ	452 mμ	122 mμ	440 mμ	194 mμ
Deflection	-1.11 mm	-1.0 mm	-1.39 mm	-1.3 mm	-1.4 mm	-1.5 mm
At CFRP (ave.)	-	150 mμ	-	191 mμ	-	263 mμ

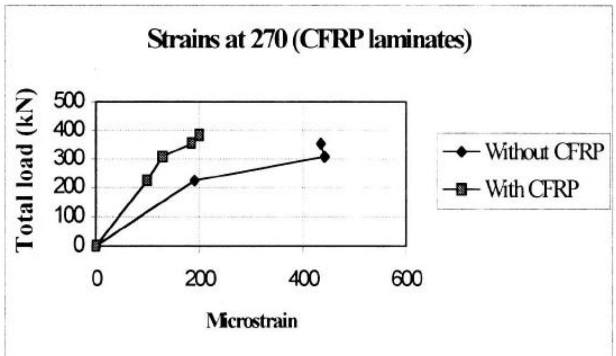




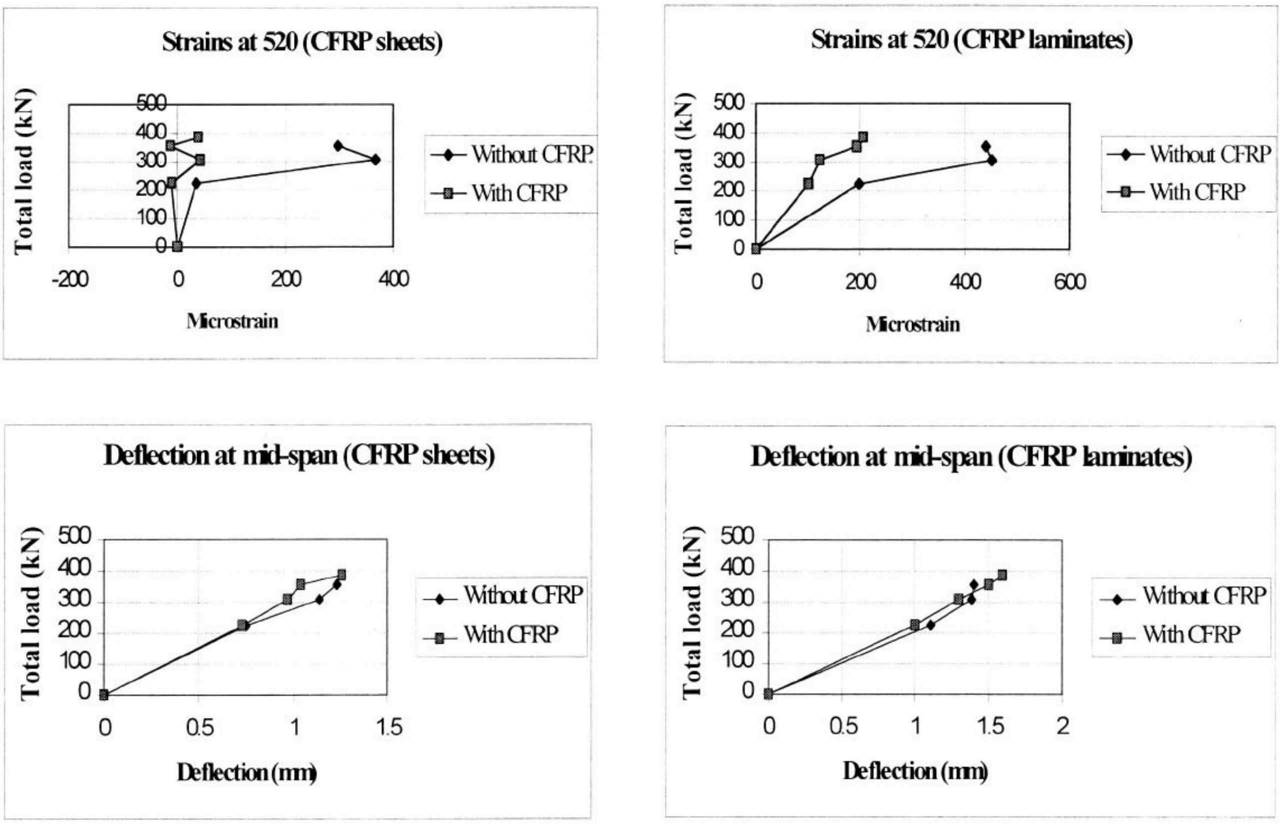








Figures 4-9: Comparison of results



Figures 10-13: Comparison of results

CONCLUSIONS

Subject to errors due to strain measurements the results as shown in Figures 4-13 do indicate that bonding of CFRP sheets or laminates to the underside of the R. C. beams would increase the stiffness of the beams. Flexural capacity of the beams would thus be enhanced. The test, however, does not indicate any improvement in deflection behavior of the beams by the use of CFRP bonding. In the load test the bridge was loaded within the working load condition only. The behavior of beams at higher loads would definitely differ from that at lower load. Performance of bonded CFRP on R. C. beams at ultimate limit is presently under study in the UTM laboratory. Further observation of the applied CFRP on the Kuala Pilah Bridge will be made to study the long-term behavior of the material. The general conclusion that can be made from the load study is CFRP bonding has a potential to become an effective technique of bridge strengthening.

It is important to take heed of cautions given by Robery and Duckett (1998) and Kelly et al (2000) in regard to the need of understanding by designers the properties of resins and the structural performance of resin composites. Particular attention should be given to durability and fire-resistant properties of the FRPs. It is imperative to limit on strength enhancement to maintain ductile behavior of the strengthened members.

ACKNOWLEDGEMENT

The authors would like to acknowledge the contributions of all the members of the JKR-UTM Committee on CFRP, in particular Dr. Abdul Rahman Mohd. Sam of UTM. Financial contributions from Ministry of Science, Technology & Environment (IRPA Program) and BBR Construction Systems, Biaxis, ORBTECH Composites, RNC Technologies/MBT, Sika Kimia, Vertitech System are gratefully acknowledged. Assistance provided by staff of JKR Bridge Unit, JKR Kuala Pilah and JKR Federal Workshop are acknowledged. Last but not least, encouragement and supports from Senior Superintending Engineer of Bridge Unit, Ir. Chin Wei Choong are also acknowledged. Opinions expressed here may not necessary constitute the policies of JKR or UTM.

REFERENCES

1. Bakht, B., Jaeger, L.G., (1988), 'Bearing Restraint in Slab-on-girder Bridges," ASCE Journal of Structural Engineering, 114 (12), pp. 2724-2740.
2. Government Of Malaysia (GOM 1989), '*The National Axle Study*,' Final report.
3. Kelley, Paul L., Micheal L. Brainerd and Vatovec, Milan (2000), "Dersign Philosophy for Structural Strengthening with FRP," Concrete International, February, 2000.
4. Ku Mohd Sani, K.M. & Ng, See King (2000), 'Load Testing of Bridge FT009/035/80 Over Muar River in Kuala Pilah," APSEC 2000, Kuala Lumpur.
5. Robery, Peter and Duckett, Will (1998), "Carbon fibre strengthening of existing structures," The Consulting Engineer, Issue 11, Spring 1998.