An Experimental Study on Seismic Performance Using GRP Wrapping around the Lap-Spliced Bridge Piers

T.G. Kwon *, K.S. Youm **, Y.H. Lee **, Y.K. Hwang **

* M. of Eng., Structure Research Dept., Korea Institute of Construction Technology, 2311, Daehwa-Dong, Ilsanseo-Gu, Goyang-Si, Gyeonggi-Do 411-712, Korea
** Dr. of Eng., Structure Research Dept., Korea Institute of Construction Technology, 2311, Daehwa-Dong, Ilsanseo-Gu, Goyang-Si, Gyeonggi-Do 411-712, Korea

This paper addresses experimental studies on the seismic retrofit performance of reinforced concrete circular columns with poor lap-splice details using GFRP wrapping. Five full-scaled model columns of 1.2 m diameter and 6.55 m height were classified as one column with no lap-splice, another column with lap-splice, and the other three GFRP wrapped retrofitted columns with lap-splice and tested using quasi-static cyclic loading test. The as-built column suffered a brittle failure due to the bond failure of lap-spliced longitudinal reinforcement. The retrofitted columns using GFRP wrapping showed significant improvement of seismic performance. However, the predicted flexural failure mode was not achieved and the longitudinal bars were not yielded. Failure modes of the retrofitted columns are considered to be the gradually delayed bond slip in lap-spliced longitudinal reinforcement.

Key Words: Full-scale column test, Lap-splice, FRP wrapping, Retrofit, Bond failure

1. INTRODUCTION

Recent earthquakes such as in South Asia in 2004, and in Fukuoka in 2005 have demanded the prevention of disasters. Especially, the existing bridge columns which have built prior to current seismic design codes and construction practices urgently should be retrofitted and upgraded to current seismic standards.

Based on the evaluation on the data of the bridge management systems in Korea Institute of Construction Technology, 77 % of all the bridges in South Korea were estimated to have insufficient transverse reinforcement, lap splices in the plastic hinge region, and seismic details. The columns with lap-spliced longitudinal bar, which were located at the lower column end to form the connection between the footing and the column for easer construction, will experience the brittle failure attributed to lap-splice bond failure.

Several composite jacket systems have been developed and validated in laboratory or field conditions to prevent bond-slip failure of the existing bridge piers and to improve the seismic performance. Chai et al. (1991) proved that steel jacketing was an effective method to retrofit bridge columns. Matsuda et al. (1990) tested a system for bridge pier retrofit using unidirectional carbon fiber sheets wrapped in the potential plastic hinge region. The carbon fiber sheets were bonded using epoxy resin. Priestley et al. (1991) conducted experimental testing on 40 % scale-down bridge piers retrofitted by glass fiber jackets, which were more economical than carbon fiber jackets. Test demonstrated significant improvement of seismic performance with increased strength and ductility. Many researchers proposed a retrofit measure using GFRP and CFRP wrapping (Saadatmanesh et al. 1994; Seible et al. 1995). Xiao et al. (1997) conducted a retrofit column test using prefabricated GFRP composite jackets. Ma et al. (2000) experimentally proved the effectiveness of carbon fiber composite jacketing system. Priestley et al. (1996) and Seible et al. (1995) proposed design methods to retrofit the lap-spliced column using FRP, respectively.

Methods of construction can be differentiated into two general classes of wrapping processes, hand lay-up sheets and prefabricated FRP panel. Although a prefabricated FRP jacket system has a simple retrofit procedure, the discontinuity of composite fiber and low-bonding capacity caused the rehabilitated columns to develop lower improved performance. Wet lay-up, sometimes referred to as hand lay-up, is an FRP technique often used in structural rehabilitation applications, where FRP sheets or fabrics are bonded to the exterior of reinforced concrete. This technique has advantages; it is easily and rapidly performed in the field. However, quality control is extremely important in this procedure, and skilled labor is often required.

In this study, a retrofit method using continuous GFRP wrapping has been proposed and experimentally validated. This can prevent the existing bridge piers with lap-spliced longitudinal bars in plastic region from failing. The Continuous GFRP wrapping method guarantees uniform quality as well as solves the disadvantages of fiber discontinuity.

The strains of lap-spliced longitudinal bar were measured to verify whether the longitudinal bar yielded or not. Also, to estimate the confining stress of the GRP and to evaluate the confined effect of the hoop bars, the dilation strains were recorded. The prototype is an existing circular reinforced concrete bridge pier designed and constructed in 1979 following the pre-seismic codes. The total of five full-scaled column specimens were constructed. After they were retrofitted with GFRP, the seismic performance was estimated by cyclic loading test with drift ratio increasing instead of displacement ductility.

2. FULL-SCALE SPECIMEN DESIGN

2.1 Specimen Preparation

The prototype column designed following the pre-seismic codes has been served since it constructed in 1979. It is a simple eight-span bridge with seven piers of rather irregular height and two abutments. The model columns were designed based on a full-scale of prototype column. The total of five columns of 6550 mm tall and 1200 mm diameter were tested.

As shown in Figure 1, the circular column section was reinforced with thirty seven No. 8 longitudinal bars (nominal diameter = 25.4 mm), which constituted the total longitudinal steel ratio of 1.67 % of the gross sectional area. All longitudinal bars were lap-spliced in the lower end of the column with a lap-splice length of 870 mm. The specimen was transversely reinforced with No. 4 hoop bars (nominal diameter = 12.7 mm) with a spacing of 300 mm. The design of the column was based on a steel bar of actual yield strength of 294.1 MPa and a concrete strength of 23.5 MPa. The aging effect of old concrete was ignored. The design axial load was 1520 kN, which corresponded to 5.7 % of column axial load capacity based on nominal concrete strength and gross sectional area.

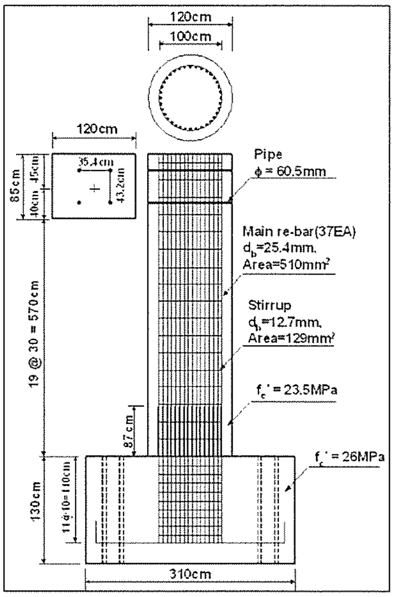


Figure 1. Details of Full-scale Model Column

Figure 2. GFRP wrapping equipment

As shown in Fig. 2, 1.2m width FRP wrapping in the lap-spliced region was conducted using automated FRP wrapping equipment in KICT (Korea Institute of Construction Technology) structures laboratory. This equipment was designed to control fiber orientation angle, width, and thickness of FRP wrapping. Since this equipment wraps wetted fiber bundles continuously, better confinement effect can be achieved than that of jacketing or sheet bonding. In the FRP wrapping fiber orientation angle is limited to 5° to provide better confinement effect and uniform continuous wrapping. The details of specimens are shown in Table 1. The mechanical properties of FRP were determined according to ISO 527-5 [5] and ISO 14129 [6]. The mechanical properties of GFRP from the coupon test are summarized at Table 2. The wrapping length of GFRP was chosen to be 1200 mm, which was 1.4 times of the lap-splice length.

Table 1. The details of FRP wrapping specimens

Specimens Labels	Details	GFRP thickness
As-built column	870 mm lap-splice	-
Continuous bar column	Continuous bars	
6 mm GFRP	870 mm lap-splice	6 mm
8 mm GFRP		8 mm
10 mm GFRP		10 mm

Table 2. Material properties of the GFRP

Elastic modulus	E_j	28944 MPa
Ultimate strength	f_{uj}	373.3 MPa
Ultimate strain	ε_{uj}	0.0129

2.3 Experimental Program

To evaluate the seismic performance, the cyclic loading tests were conducted. Axial load of 1520 kN was applied to the test column using a ±500 mm stroke actuator with the capacity of 3500 kN before imposing lateral displacement. The lateral seismic force was applied by the horizontally positioned actuator at the top of the column. The lateral loading scheme used in the tests was based on the drift ratio which is defined by the lateral displacement divided by the column height. As shown in Figure 3, the drift ratio was increased every 0.25% with two loading cycles conducting. Because the longitudinal bars don't yield due to the bond slip in the lap-spliced, the cyclic loading is controlled by the drift ratio instead of the displacement ductility factor. The ultimate displacement is defined as a displacement measured at 20% reduction of the maximum lateral load. Figure 4 schematically shows test setup.

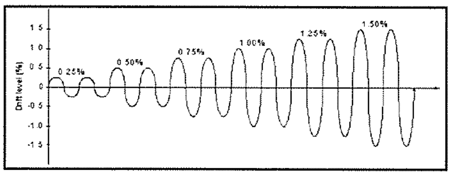


Figure 3. Loading history for test columns

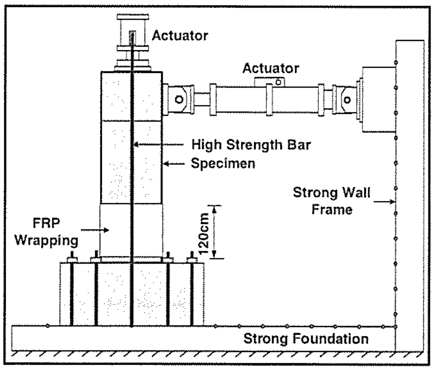


Figure 4. Column test setup

The lateral loading was measured by calibrated load cells. The lateral displacement at the point of lateral loading position was recorded by a linear variable differential transformer (LVDT). Strain gauges were attached on the surfaces of starter bars and main bars to measure the longitudinal deformation. Dilation strains of GFRP and hoop bars were measured by strain gauges mounted on the jacket and the surface of hoop bars.

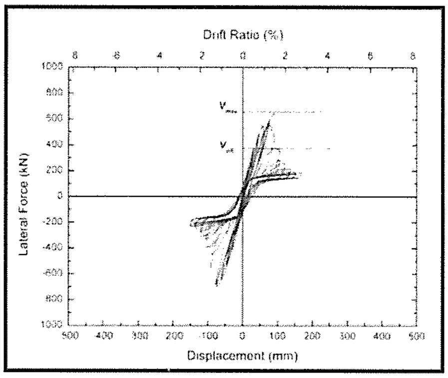
3. TEST RESULTS AND DISCUSSION

3.1 Test Results

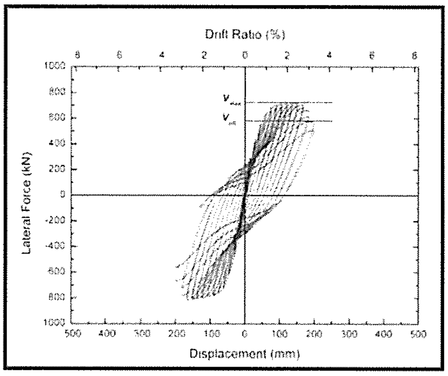
As shown in Figure 5(a), the as-built column developed an unstable response due to lap-spliced bond failure between the longitudinal main and starter bars. The first flexural crack was observed at a drift ratio of 0.5 % or at a lateral displacement of 30.5 mm. The first vertical crack was occurred at a drift ratio of 1.25 % (76.25 mm). As the drift ratio increased to 1.5 % (91.5 mm), cracks were spread within the lap-spliced region and the load-carrying capacity rapidly dropped due to the bond failure, as shown in Figure 6(a). The test was terminated at a drift ratio of 1.65 % or a displacement of 101.0 mm. The maximum lateral force of the as-built column was recorded as 643 kN corresponding a displacement of 91.5 mm. After the lap-spliced bond failure, its load-carrying capacity settled at 200 kN.

As observed in Figure 5(b), the performance of the continuous bar column, which did not have lap-spliced starter bars in the plastic hinge region, was better than that of lap-spliced column. The failure of the continuous bar column occurred at a drift ratio of 3.05 % or a lateral displacement of 185.8 mm, when the compression buckling of longitudinal bar was observed, as shown in Figure 6(b).

Figure 7(a) depicts the retrofitted columns with GFRP. A gap of 30 mm was provided to avoid additional strength enhancement and damage of footing. The FRP thickness of retrofitted columns was 6 mm, 8 mm and 10 mm, respectively. Compared to the lap-spliced column, the retrofitted columns developed very stable hysteretic loops and exhibited significantly improved seismic performance, as observed in Figure 8(a), (b), and (c). Flexural cracks were observed at the column base, as shown in Figure 7(b). The retrofitted columns reached 3.1 to 3.2 times the ultimate displacements of the as-built column. Throughout the testing, no rupture of GFRP was observed. As the drift ratio increased, the degradation of the load-carrying capacity and deterioration of the bonding within the retrofitted region were observed. Figure 9 compares the lateral load-displacement envelope curves for all the tested columns. The maximum lateral force of the retrofitted columns was noted as 763 to 776 kN, corresponding to a displacement of 315 to 318 mm.



(a) Specimen 1 – As-built column



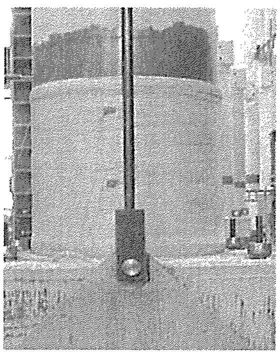
(b) Specimen 2 – Continuous bar column

Figure 5. Hysteretic response of column

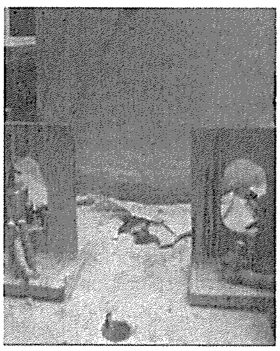
(a) Bond failure

(b) Flexural failure

Figure 6. Column failure modes

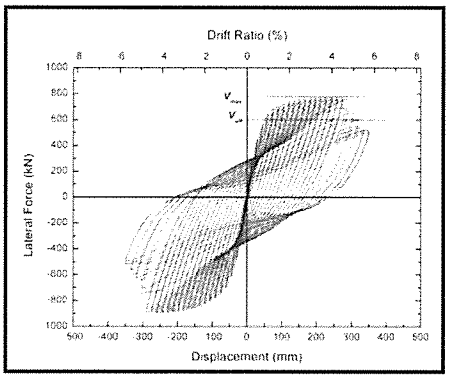


(a) Retrofitted section

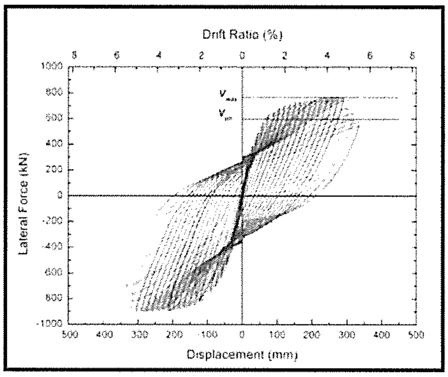


(b) Crack pattern

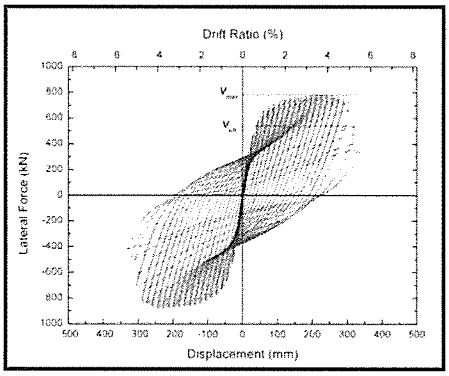
Figure 7. GFRP retrofitted column



(a) Specimen 3 – 6 mm GFRP



(b) Specimen 4 – 8 mm GFRP



(c) Specimen 5 – 10 mm GFRP

Figure 8. Hysteretic response of retrofitted columns

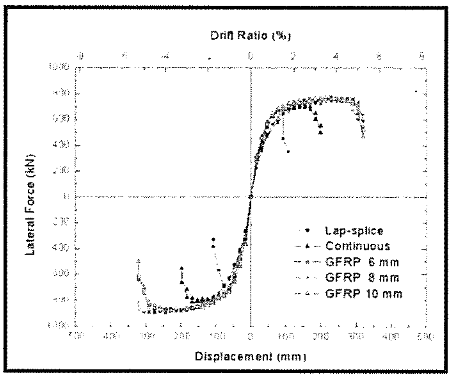


Figure 9. Comparison of load-displacement envelopes

3.2 Discussion

The reason why the displacement ductility cannot define is that the yielding of the longitudinal bar of the lap-spliced column never occurs. Therefore, it is impossible to calculate the yield displacement of the lap-spliced column and to define the displacement ductility factor (μ), which is defined as the ratio of the maximum displacement to the yield displacement at the theoretical first yield of tension bar. Therefore the estimated displacement ductility factor ($\bar{\mu}_{est}$), instead of the displacement ductility factor (μ), is defined as, using the theoretical yield displacement of the column with continuous bars (Δ_y).

$$\bar{\mu}_{est} = \frac{\Delta_{ult}}{\Delta_y} \quad (1)$$

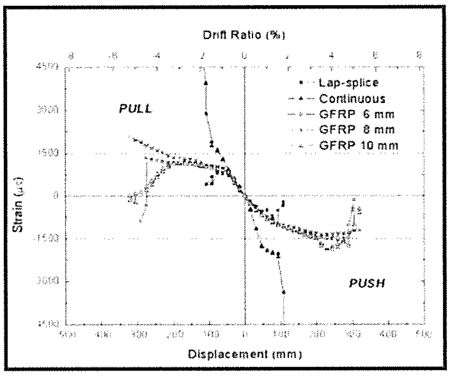
The estimated displacement ductility of the as-built column is calculated as 2.6, whereas that of the continuous bar column is 4.8. The estimated displacement ductility of the retrofitted columns was evaluated as 8.2 to 8.3. This showed that the retrofitted columns developed higher ductility capacity compared to the lap-spliced column and even the column with continuous bars. Test results of all the columns are summarized in Table 3.

Table 3. Test results of all columns

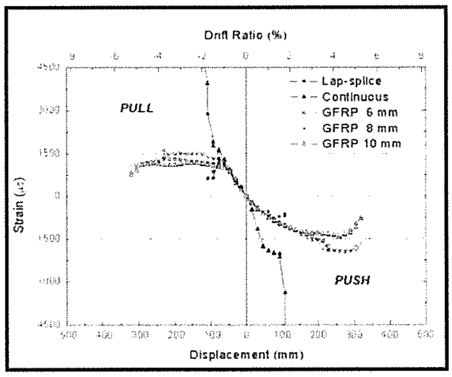
Specimen	V_{max} (kN)	Δ_{max} (mm)	V_{ult} (kN)	Δ_{ult} (mm)	$\bar{\mu}_{est}$
As-built	643.0	91.5	367.0	101.0	2.6
Continuous bar	720.0	147.5	575.2	185.8	4.8
6 mm GFRP	774.9	256.3	597.7	315.8	8.2
8 mm GFRP	762.8	256.9	594.6	318.3	8.3
10 mm GFRP	775.6	257.7	529.7	314.8	8.2

V_{max} : Maximum lateral load (kN)
Δ_{max} : Displacement at maximum lateral load (mm)
V_{ult} : Failure lateral load (kN)
Δ_{ult} : Displacement at failure (mm)
$\bar{\mu}_{est}$: Estimated displacement ductility factor

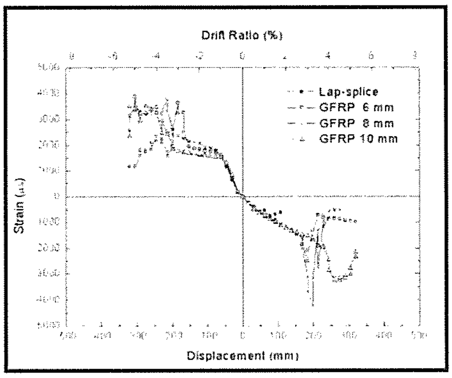
Strain gauges were attached to the surface of GFRP at the bottom of the column in the push and pull faces and mounted on the main and starter bar in the lap-spliced region. The strain distributions of the longitudinal bar of the tested columns were shown in Figure 10. In case of the column with continuous bar, the strain gauge readings confirmed the yielding of the longitudinal bars, as shown in Figure 10(a) and (b). However, the strain gauge readings of the lap-spliced column confirmed the lap-spliced bond failure between the main and starter bars. Although the main and starter bar strain slightly exceeded the yield strain value of 1500 $\mu\varepsilon$, the unyielding of the lap-spliced longitudinal bar was confirmed by the recorded strains of the gauges, regardless of the GFRP thickness. If GFRP perfectly confined concrete to ensure plastic hinge rotation and effectively prevented the lap-spliced bond failure, the lap-spliced longitudinal bars had to be yielded. It is estimated that the behavior of the retrofitted columns at large displacements would be gradually delayed bond-slip of the lap-spliced longitudinal bars. The strain distribution of the main bar at 1.2 m height from the bottom, as shown in Figure 10(c), verified the yielding of the main bar. It is suggested that the plastic hinge was located at the end of the GFRP wrapping region.



(a) Starter bar at 0.5 m height from the bottom



(b) Main bar at 0.5 m height from the bottom

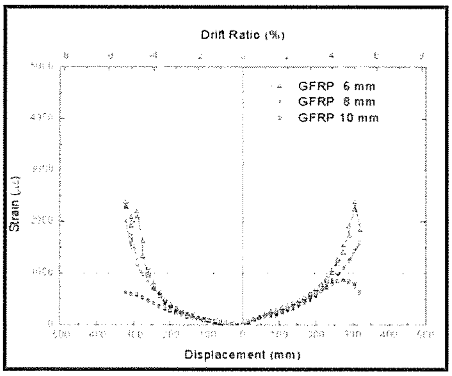


(c) Main bar at 1.2 m height from the bottom

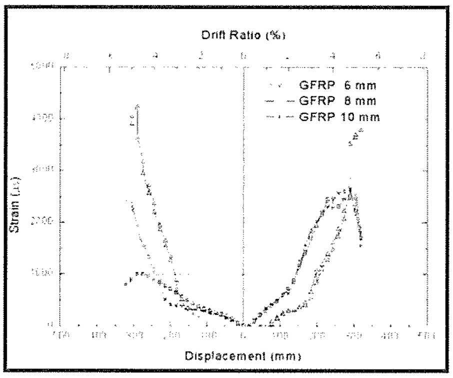
Figure 10. Strain distribution of longitudinal bar

The dilation strain distribution of GFRP recorded for all the retrofitted columns was from 900 to 2000 $\mu\varepsilon$ at 0.5 m height and from 2000 to 4000 $\mu\varepsilon$ at 1.2 m height, as shown in Figure 11(a) and (b). Especially, the strain dispersion of 8 mm thick retrofitted column with increasing the height was the smallest of all retrofitted columns and maintained approximately 2000 $\mu\varepsilon$. The maximum recorded GFRP strain value was about 4300 $\mu\varepsilon$, which was about 33 % of the ultimate GFRP strain used in the test. This means that the GFRP of the retrofitted columns didn't damaged at all. According to the thickness of GFRP wrapping, the dilation strain of GFRP varied. Therefore, To retrofit the lap-spliced column with GFRP, the retrofit design equation is suggested that the dilation strain of GFRP should be estimated according to the thickness of GFRP.

In case of bridge columns constructed before 1991 in South Korea, it was common to contain insufficient transverse reinforcement. In case of all the tested columns, the hoop bars were closed by lap splices placed at 300 mm centers. The measured strain of the hoop bar verified the insufficient amount of confining pressure provided by the hoop bars and confirmed the unyielding of hoop bars, as shown in Figure 12(a) and (b). Even though the columns were retrofitted with GFRP, the strain of hoop bar didn't varied. Therefore, it is conservative that the confining effect of hoop bar should be ignored.

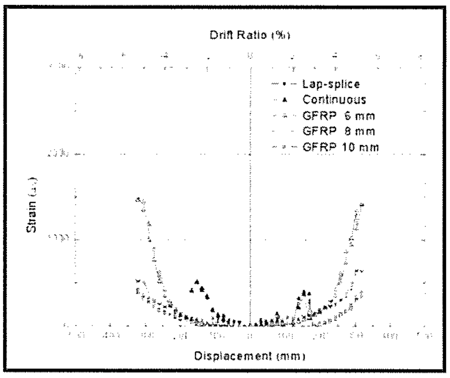


(a) Lateral strain at 0.5 m height from the bottom

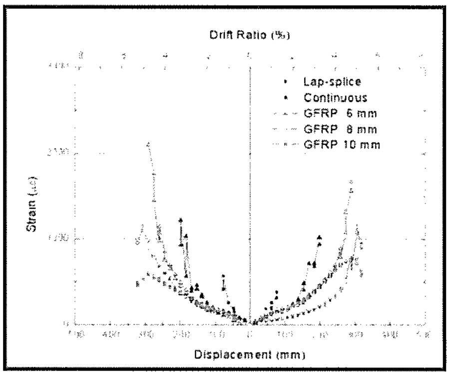


(b) Lateral strain at 1.2 m height from the bottom

Figure 11. Strain distribution of GFRP



(a) Lateral strain at 0.3 m height from the bottom



(b) Lateral strain at 0.6 m height from the bottom

Figure 12. Comparison of hoop bar strain

4. CONCLUSION

The five prototype column constructed in 1979 and four retrofitted columns with GFRP were conducted by cyclic loading test increasing the drift ratio. From the test results, the following conclusion can be drawn:

1) Compared to the as-built column developed an unstable response due to lap-splice bond failure, the retrofitted columns developed very stable hysteretic loops and exhibited significantly improved seismic performance. The estimated displacement ductility of the as-built column is calculated as 2.6, whereas that of the retrofitted columns was evaluated as 8.2 to 8.3.

2) The strain gauge readings of the retrofitted columns confirmed the unyielding of the lap-spliced longitudinal bars within lap-spliced region (870 mm), regardless of the GFRP thickness. The failure behavior of the retrofitted columns was estimated gradually delayed bond-slip of the lap-spliced longitudinal bars. Therefore, it is impossible to define the displacement ductility factor and to conduct the cyclic loading test using it. Instead of it, it is concluded that applying the drift ratio to the lap-spliced column test is more reasonable as our experiments.

3) To retrofit the lap-spliced column with GFRP, the retrofit design equation is suggested that the dilation strain of GFRP should be estimated according to the thickness of GFRP and the confining effect of hoop bar ignored.

References

1) Korea Institute of Construction Technology, *Bridge Report*, 2001.

2) Chai, Y.H., Priestley, M.J.N. and Seible, F., Seismic retrofit of circular bridge columns for enhanced flexural performance. ACI Structural Journal, Vol. 88, No. 5, pp. 572-584, 1991.

3) Ma, R., Xiao, Y. and Li, K.N., Full-scale testing of a parking structure column retrofitted with carbon fiber reinforced composites. Construction and Building Materials, Vol. 14, pp. 63-71, 2000.

4) Matsuda, T., Sato, H., Fujiwara, H. and Higashira, N., Effect of carbon fiber reinforcement as a strengthening measure for reinforced concrete bridge piers. *Proceedings of the 1st US-Japan Workshop on seismic retrofit of bridges, Public Works Research Institute*, Tsukuba Science city, Japan, pp.17-18, 1990.

5) Priestley, M.J.N. and Seible, F., Seismic assessment and retrofit of bridges, *Proceedings of the 2nd International Workshop on the Seismic Design of Bridges*, Vol. 2, pp. 46-70, 1991.

6) Priestley, M.J.N., Seible, F. and Calvi, G.M., *Seismic design and retrofit of bridges*, John Wiley & Sons, New York, N.Y., 1996.

7) Saadatmanesh, H., Ehsani, M.R. and Li, M.W., Strength and ductility of concrete columns externally reinforced with fiber composite straps. **ACI Structural Journal**, pp.434, 1994.

8) Seible, F., Priestley, M.N.J., Hegemier, G.A. and Innamorato, D., Seismic retrofit of RC columns with continuous carbon fiber jackets. **Journal of Composites for Construction, ASCE**, Vol. 1, No. 2, pp. 52-62, 1997.

9) Xiao, Y. and Ma, R., Seismic retrofit of RC circular columns using prefabricated composite jacketing. **Journal of Structural Engineering, ASCE**, Vol. 123, No. 10, pp. 1357-1364, 1997.