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Restoration of Pre-damaged RC Bridge Columns with Deficient Plastic Hinge Regions Using Basalt FRP

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ABSTRACT

This study aims to examine the applicability of using basalt fiber reinforced polymer (BFRP) jackets to restore the functionality of pre-damaged reinforced concrete square columns. Firstly, two reinforced concrete bridge columns with improper details of longitudinal and/or transverse reinforcement were tested under the effect of constant axial compressive load and increasing lateral cyclic load. Test results showed that column with short lap-splice reinforcement exhibited inferior performance where column strength showed rapid degradation before achieving the theoretical strength, and its deformation capacity was limited; however, quick restoration is possible through a suitable rehabilitation technique. On the other hand, expensive repairs or even complete replacement of damaged column could be the decision for the column with confinement failure of the flexural plastic hinge region. In the second part of the study, a rehabilitation technique using BFRP jacketing technique was adopted. Design details of external jackets, guaranteeing the enhancement in the inelastic performance of both damaged columns, were addressed and defined. Test results of repaired columns confirmed that post-damage reparability and restorability of structure are dependent on the reinforcement details within the plastic hinge zone. Furthermore, lap-splice of longitudinal reinforcement would be applied as a design tool that control the structure reparability and restorability after a massive earthquake action.

1. INTRODUCTION

Splices in reinforced concrete columns of old structures were commonly designed as compression lap-splices, which are typically only 20 to 24 bar diameters (d_b) long and enclosed within light transverse reinforcement. Observations of column damage following earthquakes have revealed that these splices perform poorly. Even if columns

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were with continuous longitudinal reinforcement, failures typically occur with some displacement ductility and early degradation of column resistance to lateral load due to inadequate horizontal confinement. Hence, extensive research on retrofitting of deficient columns has been conducted in which concrete, steel, or fiber reinforced polymer (FRP) jacketing was applied to restore the strength and ductility of the bridges locate in seismic zone. Few studies, however, have focused on the performance of repaired RC bridge columns (Saadatmanesh et al. (1997) and He et al. (2013)), especially those with premature failure modes due to improper details of longitudinal reinforcement and/or transverse ties.

To restore an earthquake-damaged community as quickly as possible, a well-prepared repair, reconstruction, or replacement strategy is most essential. Replacing the entire damaged bridge is cumbersome, time consuming, and expensive. Therefore, appropriate bridge repair needs to be carried out to restore the bridge. In view of the successful performance of FRP-retrofitted bridge columns using FRP wrap, the aim of the current study is to identify the performance of deficient bridge columns before and after rehabilitation with FRP jacketing. Sim et al. (2005) reported that basalt fibers have greater tensile strength than E-glass fibers, greater failure strain than carbon fibers and good resistance to chemical attack, impact load and fire while giving off fewer poisonous fumes. Therefore, basalt FRP (BFRP) sheets were applied as external confinement. Effect of reinforcement details on the required recoverability is considered. The reported results in this study about two column specimens with light internal transverse ties: one column was with longitudinal reinforcement of a relatively small splice length at the plastic hinge zone, and the second column was with continuous longitudinal reinforcement. Both columns were subjected to constant axial compressive load and increasing lateral cyclic loading till failure. Then damaged zones of both columns were rehabilitated with external BFRP jacket and the columns were subjected once more to a constant axial compressive load combined with lateral cyclic loading up to failure.

2. EXPERIMENTAL DETAILS AND MATERIALS

Two concrete column specimens had a height of 1.0 m and a cross section of 200 x 200 mm. An approximate longitudinal reinforcement ratio of 2%, provided with 13-mm-diameter deformed rebar was used. The column longitudinal bars were continuous reinforcement or lap spliced at the base of the column to starter bars extending up from the column base. The length of the lap splice was $20d_b$ (d_b is bar diameter), i.e. 260 mm. The footings were oversized to force failure of the specimen into the columns. Both columns had wide spaced stirrups in the plastic hinge zone, i.e. stirrups spaced at 100mm which was double the maximum spacing permitted according to the current code provision (AASHTO LRFD Bridge Design Specifications [AASHTO (2014)]). Each column was fixed into a heavily reinforced 0.45 m deep base block, 1.0 x 0.5 m in plan. The tested specimens were attached to a strong steel floor using pre-stressed high-strength steel rods.

Tensile tests were performed to determine the yield strengths of these rebars. The longitudinal bars had a yield stress of 349 MPa, a tensile strength of 541 MPa. The corresponding values for the steel used for the 6-mm stirrups were 416 MPa (proof stress), and 625 MPa. To simulate the field conditions, the base blocks were firstly cast and after two days columns were cast. The concrete compressive strength on the day of testing the columns was almost 30 MPa based on the average of three tested concrete cylinders.

3. EXPERIMENTAL PROGRAM

The experimental program included two columns; where the first column (CF) was reinforced with continuous longitudinal steel bars and the other column (CL) was reinforced with short lap-splice steel bars. These specimens were tested under reversed inelastic cyclic loading with gradual increasing levels of lateral displacements under a constant superimposed axial load till failure occurred. After that, they were repaired with the BFRP composite wrap and designed as R-CF and R-CL, "R" indicating a repaired column. The test procedure was identical for the test specimens.

4. TEST SETUP AND LOADING SYSTEM

All specimens were tested to failure under reversed inelastic cyclic loading with increasing levels of lateral displacements under a constant superimposed axial load of 40 kN (1 MPa), which corresponds to that of actual highway bridge piers in Japan. The axial force was applied through prestressed high-strength steel rods using a hydraulic jack with a capacity of 200 kN mounted on the top of the tested column, Fig. 1. The lateral load was applied using a horizontally-aligned 700 KN of push and pull capacity hydraulic actuator. The foundations of the test specimens were post-tensioned to the laboratory strong floor, Fig. 1. Lateral loads were applied under displacement control based on a pattern of progressively increasing displacements, referenced to the column drift ratio. The lateral load sequence started with one complete cycle to 0.25% drift ratio. Then, the loading pattern for the specimens consisted of three cycles at displacement levels of 0.5, 1.0, 1.5, 2.0, 2.5, 3.0, 3.5, 4.0, 4.5 % etc., unless failure occurred first. Failure of a test specimen was defined by a 20% drop in lateral strength from the peak of this specimen.

5. MEASUREMENTS

All specimens were instrumented during testing to monitor strains at several locations. LVDTs were used to record the horizontal displacements of the column as well as any vertical or horizontal movement of the base block, see Fig. 1. The applied normal force and local strains in the main reinforcement were also measured.

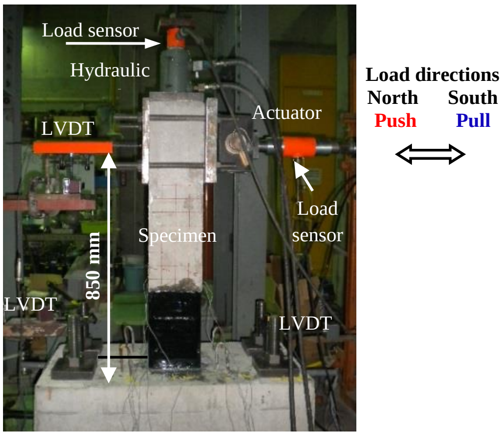


Fig. 1- Specimen during testing

6. Test Results of Reference Columns

6.1 CF Column

The hysteretic response of the CF column is plotted in Fig. 2.a and the theoretical lateral strength of the column is superimposed on the same figure. The first yielding of the longitudinal steel bars was observed at a drift of 0.68% while pulling the column to the first cycle of a drift of 1.0%, at which the column was able to attain the theoretical strength. With further loading of the column, the column was able to continue carrying load up to lateral drifts of -2% and 2.5% to achieve lateral strengths of -33.5 kN and 36.5 kN, respectively. The measured response of CF column showed 4.5% lateral drift; however, column could not continue withstand lateral load after drifts of 2.0 and 2.5 % in the push and pull loading directions, respectively, (Fig. 2.a). The concrete damage concentrated at the base within a height of approximately 200-mm above the footing. Large displacement reversals caused fracturing of the concrete core, and slight buckling of the steel bars by forcing the transverse steel to unwrap, (Figs. 3 (a & b)). The test was stopped at a lateral drift of 4.5%, at which the column strength was 29 kN corresponding almost to 80% of the achieved maximum strength.

6.2 CL Column

Figure (2.b) presents the applied load-displacement response of CL column. Cracking was first observed near the top of the lap-splice region at 0.25% drift. During the cycles at 1% drift, increased transverse and vertical cracking starting from column-footing interface occurred on all faces of the column (Figs.3 (c & d)), and the responses to the second and third cycles showed strength deterioration. This specimen failed at drift of 1.50%, at which a 20% drop in load carrying capacity was observed. Although the CL column barely achieved the theoretical flexural capacity of the cross-section in the first cycle pulling to a 1.0% drift, it was unable to do so upon reversal to the push direction. At a drift of 1.50%, a decrease of 20% from the peak load was observed. At this point in the testing, additional transverse, vertical and inclined cracks appeared, and the number of cracks in the lap-splice region increased due to significant slipping of the splice that occurred at that time. With further loading of the column to drift of 2.5%, the lateral strength of the CL column decreased to 67% of the theoretical strength with a significant widening of the longitudinal cracks, as shown in the northeast corner (Figs. 3 (c & d)).

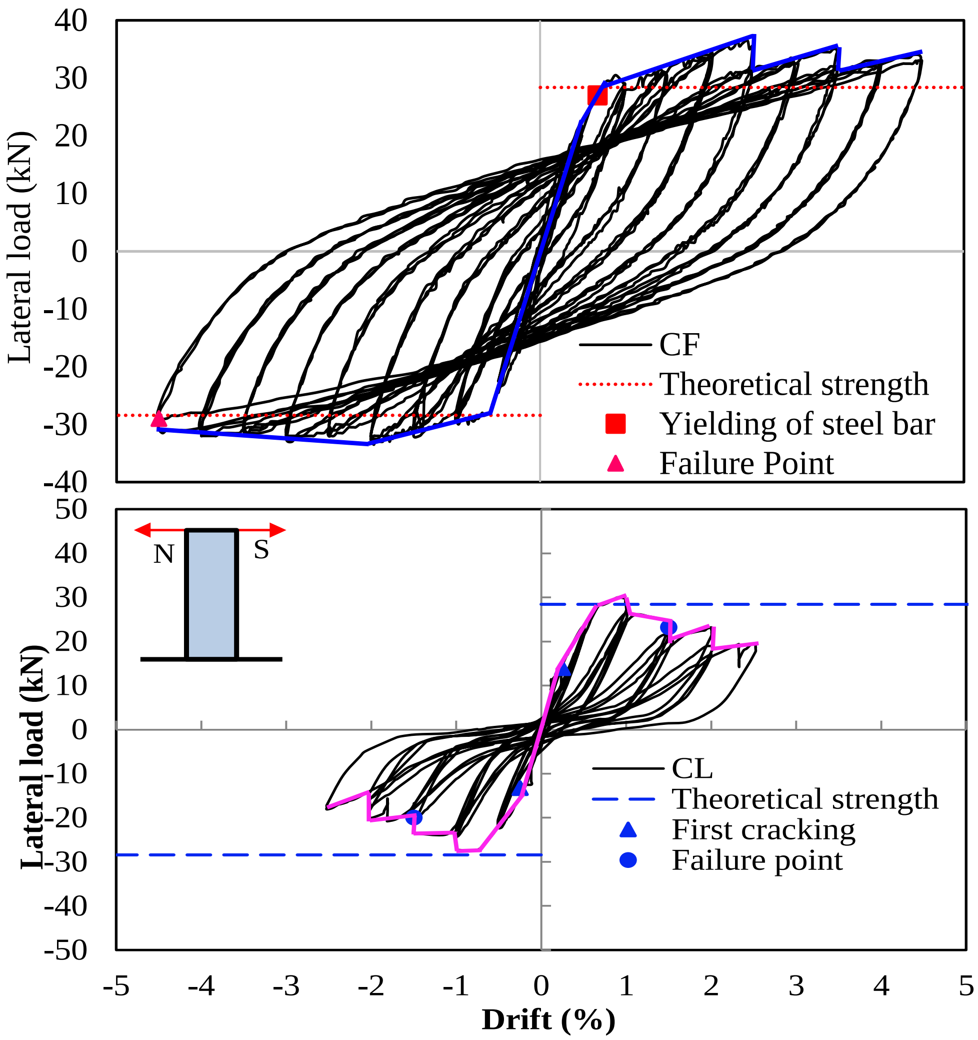


Fig. 2- Load-versus drift response of CL and CF

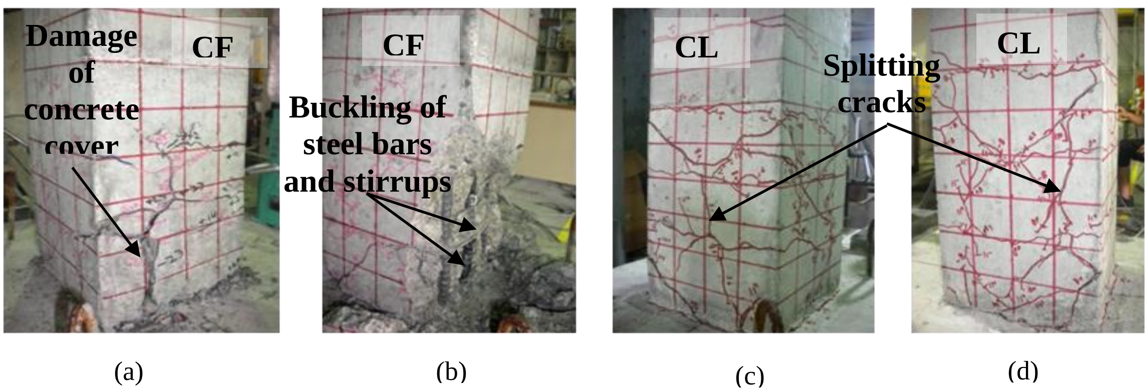


Fig. 3– Final failure pattern of (a & b) CF column and (c & d) CL column

7. REPAIRING OF PRE-DAMAGED COLUMNS

In modern seismic design codes, importance level of a RC bridge and the level of design earthquake define the required post-earthquake recoverability. For instance, critical bridges should be opened for all traffic after a massive earthquake. Hence, quick repairing of damaged columns would be necessary to restore the functionality of the bridge. The term ‘‘rapid’’ in the context of this study refers to a seven-day time period based on a direct interview and questionnaire survey conducted by Kawashima and Miyaji (2006) to the public who are nothing to do with construction industry on their understanding and demand for the seismic performance goals of bridges. The question was on how shortly they want bridges which would suffer damage after a significant earthquake should be repaired no matter how temporarily (emergency repair). One of the important lessons from the 1995 Kobe earthquake in Japan was the examination of column residual inclination (residual drift ratio) to make a decision about the probability of repairing or demolishing of the column. As reported by Fujino et al. (2005), the residual inclination tends to be large in severely damaged piers, and it also exists in many lightly damaged or even non-damaged piers. Hence, piers with an inclination larger than a specific limit will be demolished even if the visually judged damage is mild. Seismic Design Code for Railway Structures in Japan [JSCE (2000)] reflects recent advances in earthquake engineering; in which the residual deformation is limited to not exceed 1% of the column height for a quick recovery after an earthquake (recoverability limit).

7.1 Repositioning of the Column

In the light of the above perspective, the first measurement check to repair pre-damaged bridge columns is to define the residual inclination (residual drift) of each column and compare to the recoverability limit. Fig. 4 presents the relationship between column drift and residual drift and column drift versus the normalized recovery force for both test columns. The residual drift is the column displacement at zero loading, while unloading the column after pushing or pulling to a certain displacement, divided by the column height (850 mm). The recovery force is the load required to pushing or pulling the column to the original position: its value is dependent on the value of the column lateral load at each cycle while the column displacement was zero. The normalized recovery force is the ratio of the recovery force to the column theoretical strength (28.4 kN). For the CF column, the residual inclination was 3.0% at drift of 4.5%, and the corresponding normalized recovery force was 0.55. That is, a force of 15.6 kN was required to push the column back to its original vertical position. Under the effect of a moderate seismic excitation, if CF would achieve its full deformation capacity (4.5% lateral drift), the probability of quick recovery of the functional bridge after the earthquake could not be assured. In this case the structural system of the bridge would be a key-parameter in making a decision whether expensive repair or complete replacement of damaged columns is required. For instance, necessary repositioning of the column to its original place (pushing back the column to the position of zero lateral displacement) would be impossible, and so demolishing of this part of the bridge is required. For the column CL, column residual drift was 0.9% at drift of 2.5%, which is lower than the recoverability limit. In addition, the normalized recover force was around 0.05. Hence, due to the limited residual deformations of lap-splice column, a quick repairing technique could be applied. When pushing the column back to its original position is not possible, repairing works could be achieved to quickly restore the bridge functions. Moreover, if pushing or pulling the column to its original vertical position is necessary, the recovery force is small in comparison with that of the column CF. It should be noted that short lap-splice would not guarantee the stability of the bridge parts, and so the reparability of the column is still dependent on the bridge system and integrity. In this work, the pre-damaged specimens (CF and CL) were initially pushed back to the original position (i.e., zero lateral displacement), after that the proposed repairing technique was applied.

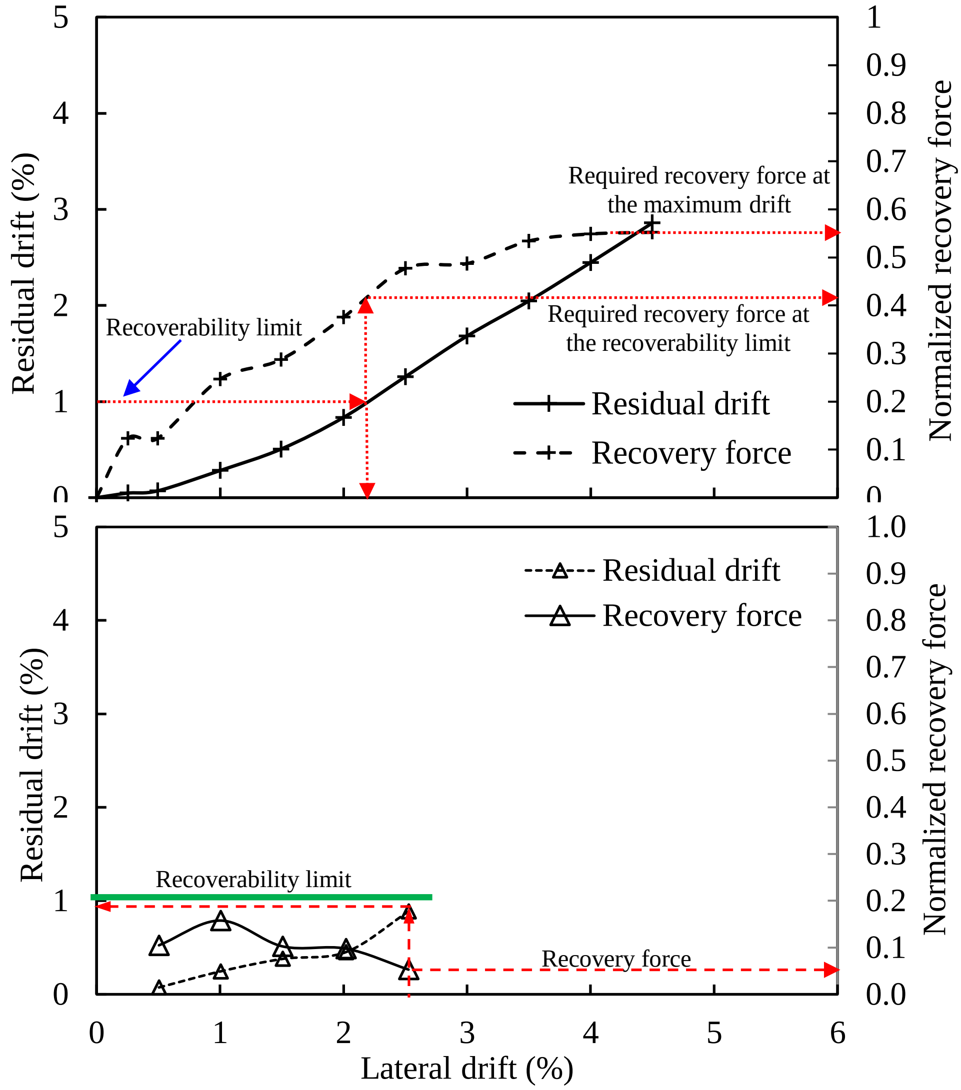


Fig. 4 Column lateral load versus residual drift and the normalized recovery force of (a) CF column and (b) CL column

7.2 Application of Polymer Cement

There is currently great interest in using polymer-modified mortar and concrete as repair materials for deteriorated reinforced concrete structures. Polymer-modified concrete and mortar are promising construction materials for the future because of the good balance between their performance and cost compared to other composites. Therefore, the second step in the repairing process was to chip out loose concrete in the failure zones, fill the gap with polymer cement (Fig. 5.b), with the given characteristic values in Table 1.

Table 1 – Characteristic values of polymer cement

Material name	Characteristics	Characteristic values
Polymer cement	Mortar flow value	165 (mm)
	Compressive strength (7 days)	30.4 (MPa)
	Bending strength (7 days)	7.3 (MPa)
	Bond strength (7 days)	1.8 (MPa)

7.3 Application of External BFRP Jacket

Based on the previous test results of the columns CF and CL, the average of the maximum flexural capacity was 35 kN of the unretrofitted column CF. The nominal shear strength of the unretrofitted column was 45.9 kN. In other words, the shear strength exceeded the actual shear applied at flexural hinging of the column; thus, flexural-dominated failure was observed for CF column. For CL column, actually, in the case considered in this work, the splices of all bars were at the same location; see Fig. 1.a. Hence, the required lap-splice length based on the current AASHTO (2014) provisions should not be less than 342.1 mm, which was not achieved in the given details (lap-splice length = 260 mm). In addition, the required vertical spacing between steel stirrups in the splice region (50 mm) was violated in that the spacing of the ties was 100 mm. Thus, the control column (CL) displayed early degradation of lateral strength and limited ductility capacity, as discussed above. Based on these values and the objectives of damage-controllable structure, it appears that a retrofit system ensuring the enhancement of both column flexural strength and ductility was required.

In a study of the inelastic performance of 106 FRP-retrofitted columns by Fahmy et al. (2010), sixty-one scale-model columns demonstrated that the idealized hysteretic lateral response displays the required performance for damage-controllable structures. Accordingly, the authors recommended several FRP design assumptions and concepts providing adequate post-yield stiffness and ductility based on both column deficiency and cross-sectional shape. For example, the minimum design lateral confinement ratio f_l / f'_c should be ≥ 0.12 in columns with continuous reinforcement and ≥ 0.25 in columns with lap-splice reinforcement, where f_l is the lateral confinement strength and f'_c is the nominal concrete compressive strength. However, to develop and maintain the flexural strength of the column, debonding of reinforcement lap-splice in the plastic hinge zones should be prevented. Thus, for columns with a short-lap-splice deficiency, a lateral clamping pressure f_l *(lap-splice)* over the lap-splice L_s should be calculated using Eq. (1).

$$f_{l,lap-splice} = \frac{1.25A_s f_y}{\left[\frac{p}{2n} + 2(d_b + cc)\right] L_s} \geq 2.0MPa \ (1)$$

where p = the cross-sectional column perimeter at the lap-spliced bar location, n = the number of spliced bars along p, A_s = the total area of the main column reinforcing bars, f_y = the yield strength of the longitudinal steel, and cc = the concrete cover over the main column reinforcement with diameter db. Furthermore, a hoop strain of 1,000 $\mu\varepsilon$ is defined as appropriate for the design of the FRP jacket for both circular and rectangular columns. In addition, steel plates should be used in the wrapper region of rectangular columns, as described by Fahmy et al. (2010).

In view of the details of transverse reinforcement used in the plastic hinge zone of CF column, the horizontal stress provided by the existing stirrups at yielding of steel stirrups could be 1.15 MPa, which is smaller than the minimum defined lateral confinement pressure of $0.12f'_c$ = 3.6 MPa. In addition, this horizontal stress provided by the existing stirrups of CL column, when stirrups would achieve the yielding capacity, is lower than the minimum required lateral pressure (2.0 MPa, Eq. [1]) to clamp lap-spliced bars. Furthermore, the maximum lateral confinement ratio would be 0.04, which is much less than the design level of 0.25 established by Fahmy et al. (2010). Accordingly, to address both lap-splice and flexural deficiencies in the scale-model columns, the proposed composite jacket design guideline by Seible et al. (1997) was adopted along with the FRP design assumptions by Fahmy et al. (2010).

To determine the thickness of the BFRP jacket as a retrofit method to improve the performance of under-designed columns, the horizontal pressure of steel stirrups was determined using Eq. (2) to be 0.56 MPa (at 0.001 steel strain). For rectangular column, the required jacket thickness to improve the inelastic deformation capacity can be evaluated from Eq.3.

$$f_h = \frac{A_{st} f_s}{bs} \ (2)$$

where A_{st} is the cross-section area of steel stirrups; f_s is the steel stress; b the column width; and S is the spacing between transverse reinforcement.

$$t_j = 0.09 \frac{D(\varepsilon_{cu} - 0.004) f'_{cc}}{\phi_f f_{ju} \varepsilon_{ju}} \ (3)$$

where t_j is the thickness of FRP jacket; f'_{cc} is the confined concrete compression strength and it was assumed as 1.5 f'_c; ε_{cu} is the ultimate axial strain of FRP-confined concrete which can be obtained from Eq. (4) based on the ultimate section curvature Φ_u and the corresponding neutral axis depth c_u; f_{ju} and ε_{ju} = ultimate strength and strain of the composite jacket in the hoop direction; and ϕ_f is a flexural capacity reduction factor (typically taken equal to 0.9).

$$\varepsilon_{cu} = \Phi_u c_u \quad (4)$$

Ultimate section curvature can be defined from the following form based on the required column displacement ductility μ_Δ.

$$\frac{\Phi_u}{\Phi_y} = 1 + \left(\frac{\mu_\Delta - 1}{3}\right)\frac{L^2}{L_p\left(L - 0.5L_p\right)} \quad (5)$$

where L represents the column shear span to the plastic hinge; Φ_y is the cross-section yield curvature; and L_p is the plastic-hinge length and can be predicted from Eq. (6)

$$L_p = 0.08L + 0.022 f_{sy} d_b \quad (6)$$

For CF column, displacement ductility was assumed to be 8, and hence the required curvature ductility (Φ_u/Φ_y) was 14.1 resulting in a required ultimate confined concrete strain of 0.01 (mm/mm). Mechanical properties of the BFRP sheet used were experimentally determined where nine samples were tested under the effect of axial tension force and the average tensile strength and tensile modulus of elasticity were 2825 MPa and 111GPa, respectively. Consequently, the required FRP jacket thickness was 0.208mm, i.e. two layers of BFRP sheets of 0.111mm thickness were required to improve the column flexural deficiency. The horizontal stress provided by the existing stirrups at yielding of steel stirrups could be 1.15 MPa in addition to the lateral pressure provided by the external FRP jacket which would achieve ultimate value of 6.1 MPa (Eq. (7)).

$$f_{lu} = 0.5\rho_j f_{ju} \quad (7)$$

where $\rho_j = \frac{2t_j(b+h)}{bh}$; and h is column depth in the loading direction.

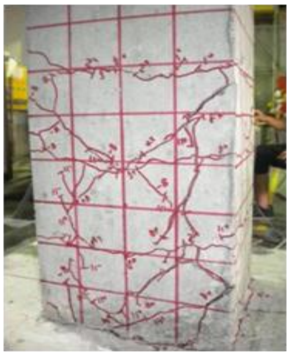
For CL column, similar to CF column a 0.208-mm jacket thickness would be sufficient to achieve a ductility of 8, however a lateral clamping of pressure ($f_{l,\ lap\text{-}splice}$) of 2.0 MPa based on Eq. (1) was required to prevent slippage of short-lap spliced bars in the plastic hinge zone. Limiting the dilation strain of external FRP to a design hoop strain $\varepsilon_{j,des}$ of 1,000 $\mu\varepsilon$ a thickness of 1.83 mm could provide a lateral clamping pressure of 2.0 MPa based on the following form proposed by Seible et al. (1997).

$$t_j = 500\frac{D\left(f_{l,lap-splice} - f_h\right)}{E_j} \quad (7)$$

Where D is the equivalent circular column diameter, see Seible et al. (1995) for evaluation and schematic representation. Thus, with nominal thickness of the basalt FRP sheet (0.111 mm), two layers would be sufficient to enhance the column ductility, and 17 layers would be necessary to ensure clamping of the lap-splice with a lateral pressure of 2 MPa. Furthermore, to meet the requirement of $f_l/f'_c \geq 0.25$, the required thickness should be 8.84 mm to provide a clamping pressure of 7.5 MPa in the lap-splice zone. It can be inferred that intensive work would be required to enhance the performance of short lap-splice columns using an FRP-confinement system. In this study, a twelve BFRP sheets were applied as a retrofitting for the plastic hinge zone in the light of Eq. (8)

$$\left(f_{l,lap-splice} - f_h\right) = 0.5\left(\rho_j \varepsilon_{j,des} E_j\right) \quad (8)$$

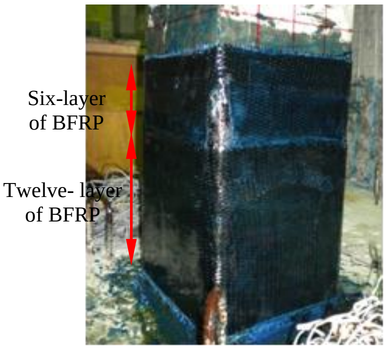
To avoid stress concentrations on the BFRP, the corners of the columns were rounded to a 15 mm. The surface of the column, to which the retrofitting was to be installed, was then abraded to smooth out irregularities and to provide more surface area for adhesion. In order to promote bonding and prevent the surface from drawing resin away from the BFRP, a low viscosity epoxy primer was applied with a roller until the column surface was saturated; see Fig. (5.c). One day after, the epoxy was mixed using two components (A and B) at a volumetric ratio of 100 parts A to 50 parts B. Lengths of BFRP sheets (300-mm and 200-mm) were placed on plastic sheets and saturated with epoxy using a rolling brush. For the CL column, the surface was thoroughly coated with epoxy to improve the concrete fabric bond and the impregnated BFRP fabric was hand wrapped around the column over a length of 300-mm with fiber orientation in the lateral direction. One more coat of epoxy was applied on the repaired zone, and then the consecutive layer covered the bottom 300-mm. All previous steps were repeated for additional four-layers of 300-mm and six-layer of 200-mm from the column footing interface, as shown in Fig. 5.d and Fig. 6. A gap of 10-mm was provided between the BFRP jacket and the footing for both columns. This gap needs to be provided to assure that BFRP jacket is not directly responsible for any strength and stiffness increase from the retrofit measure.



(a) Pre-damaged CL

(b) CL column after chipping out loose concrete in the failure zone and filling the gap with polymer cement

(c) Applying epoxy primer



(d) 12 layers of BFRP in the plastic hinge zone

Fig.5 - Repairing procedure of CL column using polymer cement and BFRP Jacket

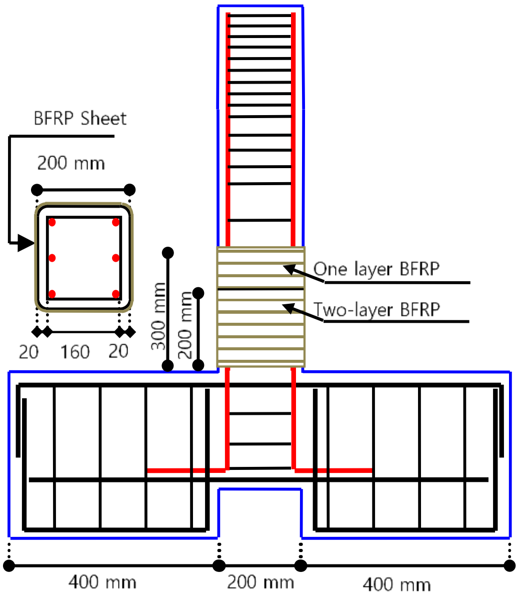


Fig. 6- Details of BFRP repaired columns R-CL and R-CL

8. TEST RESULTS OF REPAIRED COLUMNS

In general, both repaired columns performed extremely well under the simulated earthquake loading. Fig. 7.a shows a clear improvements in the hysteresis loops of R-CL. Although main longitudinal reinforcement of R-CF column as well as the steel stirrups in the plastic hinge zone was buckled, the column was able to attain the theoretical capacity at a drift of $\pm$1%. With further loading, the column could continue carrying load up to drifts of $\pm$4.5% to achieve a lateral strength of +34.5 and -35.5 during loading to south and north directions, respectively. Furthermore, its maximum deformation capacity increased to 7% drift at which the loading was stopped, as the column attained the failure point. No rupture was observed on the surface of BFRP jacket at this level of lateral deformation.

Fig.7.b shows also the hysteretic response of R-CL column, and a clear enhancement in both column deformation capacity and lateral strength could be realized with the applied repairing technique. The column was able to attain the theoretical strength at drifts of $\pm$2.0%. At drift of 2.5% for both loading directions, a sudden drop in strength was observed during loading to the second and third cycles which could be attributed to a sudden slippage in steel reinforcement. It is noteworthy that when CL column achieved this level of lateral deformation, loading was stopped due to excessive cracks observed on the plastic hinge zone as shown in Figs. (c & d). at the same level However by further increasing the lateral displacement, the column was able to continue carrying load up to lateral drifts $\pm$5% to achieve an average lateral strength of 30.4 kN exceeding the theoretical lateral strength.

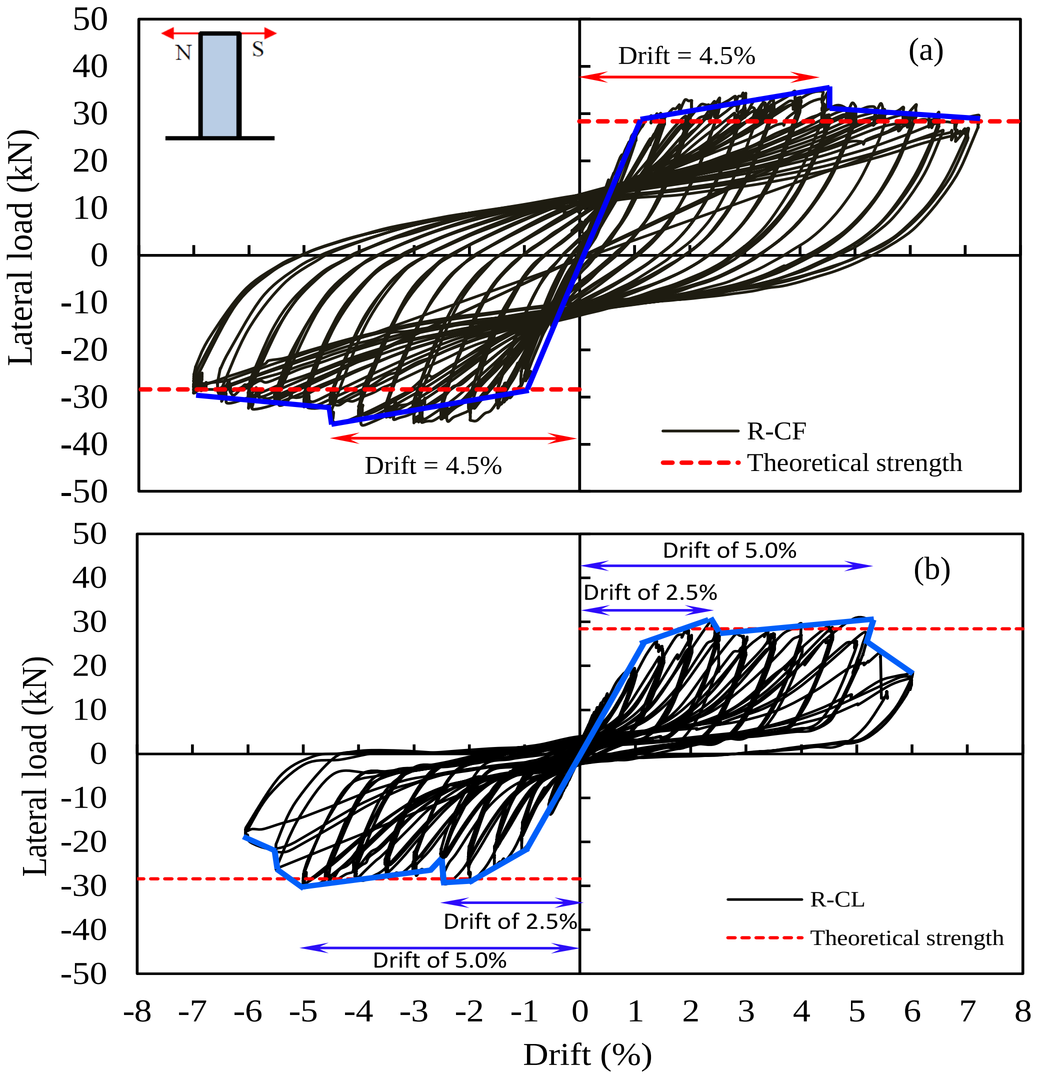


Fig. 7- Load-versus drift response of CL and CF after repair (a) R-CL and (b) R-CF

From the perspective of quick recovery, Fig. 8 shows the column drift versus both column residual drift ratio and normalized recovery force for both repaired columns. For the column R-CF, the recoverability limit (residual drift ratio = 1%) was corresponding to a column drift of 2.4% and the recovery force would be 31% of the theoretical capacity. At the maximum drift capacity of 7%, column residual drift increased to 5.3% and recovery force would be more or less 40% of the theoretical capacity; see Fig. 8.a. On the other hand, when R-CL achieved a deformation capacity of 4.5% drift, the column residual drift was 1%. At a lateral drift of 5%, the corresponding residual drifts were 1.11% and 1.55% in the pull and push directions of loading, respectively. Fig.8.b. shows that the maximum recovery force was less than 10% of the column theoretical strength.

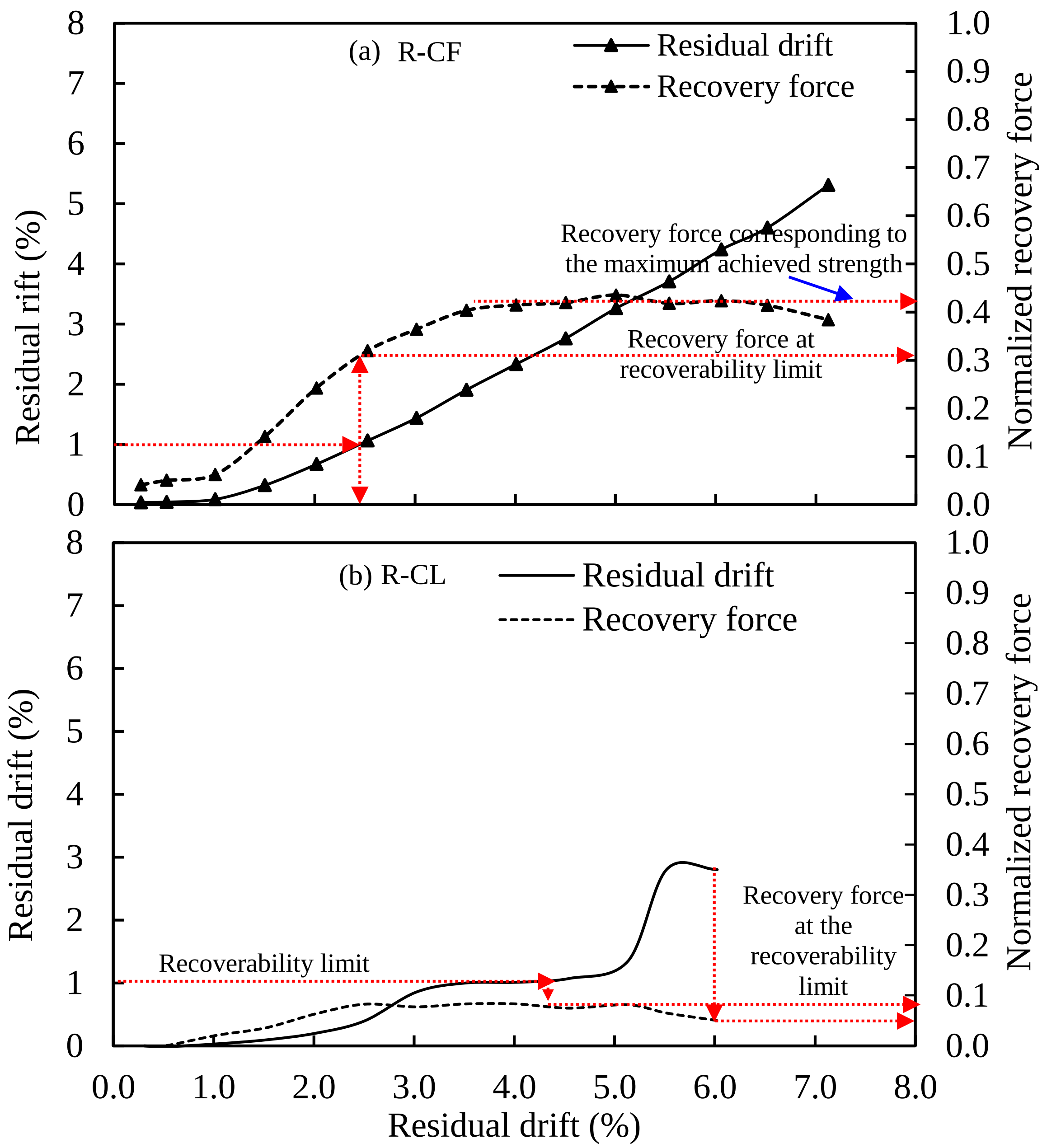


Fig. 8- Load-versus drift response of CL and CF after repair (a) R-CL and (b) R-CF

Figure 9 presents the cumulative dissipated energy versus the column drift ratio for both repaired columns. The cumulative dissipated energy was computed by summing up the areas enclosed by the hysteretic loops in the lateral load-displacement relationships of the structure up to failure. Up to a drift of 2.5% both columns had a comparable ability of energy dissipation, however from this point forward R-CF column showed a higher ability for dissipating energy than R-CL column. Apparent from Fig.9, for R-CL column, that the rate of increase in dissipating energy was almost constant up to a drift of 5% after which the rate slightly increased due to the incorporated damage. On the other hand, R-CF column showed a steeper increase up to a drift of 4.5% and a higher increase could be noticed up to the maximum achieved drift ratio. At a drift of 5%, the cumulative dissipated energy by R-CF column was 8.4 kN.m, however R-CL had 42.8% of this value (3.6 kN.m).

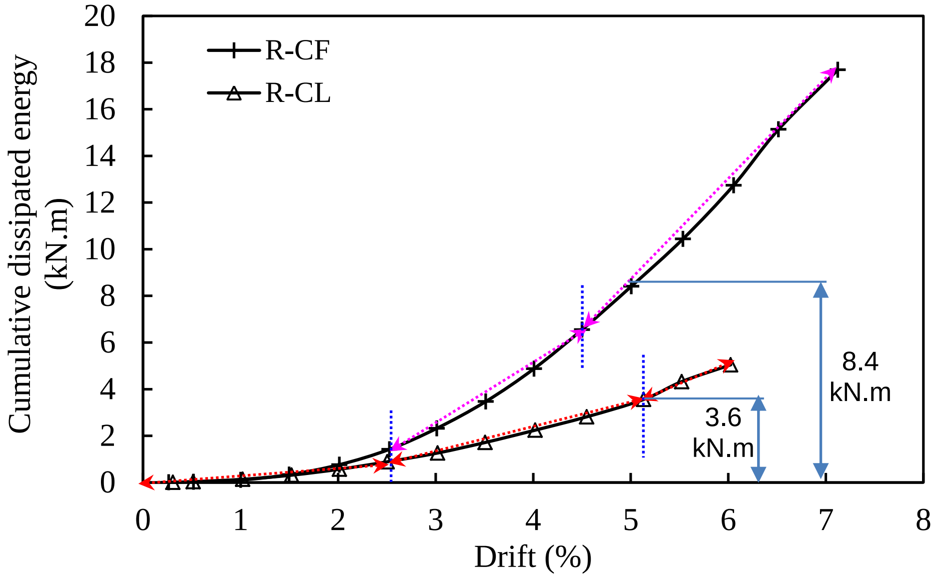


Fig. 9- Cumulative dissipated energy versus column drift of R-CF and R-CL columns

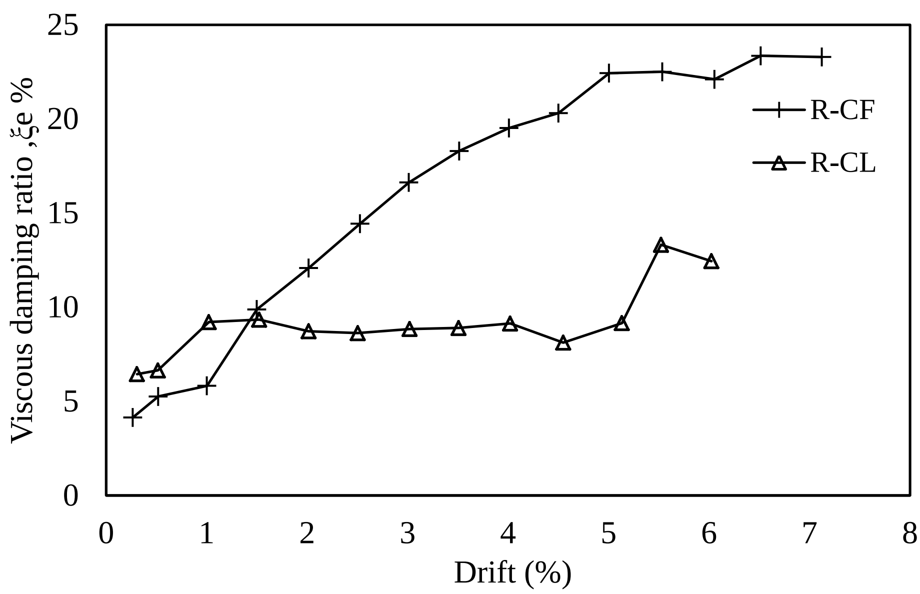


Fig. 10- Viscous damping ratio versus column drift of R-CF and R-CL columns

Energy dissipation capacity was also studied in terms of a viscous damping, which is another measure reflecting the capacity of seismic-critical elements in dissipating earthquake energy. Damping mechanisms of a structure can be represented by a viscous damping ratio, as reported by Chopra (2011). Using the hysteretic responses of the columns under study, the damping viscous ratio was determined as the ratio between the hysteretic energy dissipated during one full cycle of loading and the corresponding energy dissipated in the equivalent elastic system at the same achieved displacement. Thus, for any cycle *i*, the equivalent viscous damping ξ_e can be estimated using the following form:

$$\xi_{ei} = \frac{1}{4\pi}\frac{E_i}{E_{si}}$$

where ξ_{ei} is the equivalent viscous damping ratio for the cycle *i*, and E_i and E_{si} are the dissipated energy and the elastic energy in the cycle, respectively. The damping ratio was determined for all cycles up to the ultimate drift capacity. Fig. 10 shows measured damping ratios plotted against the drift ratio of both repaired columns. Up to a drift ratio of 1.5%, the repaired lap-splice columns displayed a higher viscous damping ratio than did the R-CF column. During the subsequent excursions up to a drift of 5%, the R-CF column displayed a steeper increase in the viscous damping ratio; e.g., ξ_e = 22.4% at a drift of 5%. The lap-splice columns, however, displayed a slower rate of increase in the viscous damping ratio. When displacing the columns to greater drifts, three distinct behaviors were observed: at a very slow rate of increase, the R-CF column exhibited a damping ratio of 23.3% at a drift of 7%; column R-CL displayed an approximately constant viscous damping ratio more or less 9% up to a drift of 4.0%; however, a sudden increase in the viscous damping ratio was noticed up to a drift of 5.5%, developing ξ_{ei} of 13.3%. Priestley (1996) reported that typical equivalent viscous damping levels of ductile systems are in the range 15 to 30% of critical damping.

In conclusion, the applied repairing technique using BFRP sheets was able to enhance the performance of the RC columns with deficient plastic hinge zone, e.g. columns with flexural deficiency and/or lap-splice deficiency. However, columns with short lap-splice would be in need for more future studies to investigate the possibility to increase both its deformability and post-yield stiffness. In addition adoption of lap-slice as a damage-controllable tool for modern structures should be examined to generate an advanced design-guideline satisfying seismic design provisions of the modern seismic design codes.

9. CONCLUSIONS

The following conclusions are drawn from the test results of seismically deficient columns before and after repair:

(1) External confinement using BFRP composite is effective in restoring the flexural strength and ductility capacity of pre-damaged concrete columns under the effect of cyclic loading.

(2) Post-yield performance of repaired columns using BFRP external jacket is dependent on the details of column main reinforcement rather than the amount of BFRP used. Although amount of BFRP used to retrofit lap-splice column was six times that required for the column with continuous reinforcement, lap-splice column could barely restore its flexural strength without any appearance of positive post-yield stiffness. In addition, at the same lateral deformation, residual drifts of the lap-splice

column were less than those of the column with continuous reinforcement.
(3) Rehabilitation strategy to restore pre-damaged columns are dependent on the details of longitudinal reinforcement in the plastic hinge zone. Rapid repair technique could be applied for columns with short lap-splice, on the other hand when column main reinforcement is continuous verification of the possibility to pushing the column back to its original position is an obligatory stage to make a decision on repairing or demolishing of the damaged column.
(4) In future research studies, lap-splice of the longitudinal reinforcement in the plastic hinge regions could be adopted as a design-damage-controllable tool to ensure the reparability and restorability of structures locating in seismic zones.

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