Recoverability of FRP retrofitted columns with lap splice and plastic hinge deficiency

M. Fahmy & Z. Wu
Urban & Civil Engineering Department, Ibaraki University, Hitachi, Ibaraki, Japan

ABSTRACT: In reinforced concrete (RC) bridge columns that were constructed before the 1970s, numerous experimental investigations have highlighted the success of the fiber-reinforced plastic (FRP) retrofitting to control lap splice and plastic hinge deficiency from the standpoint of column ductility. However, the primary objective of the current seismic codes is that bridges in important routes, under a moderate or strong earthquake, should not suffer damage to the extent requiring immediate repair after an earthquake. Consequently, the novel concept of this paper is an examination of the capability for quick recovery of the original functions of retrofitted existing RC bridge columns using FRP jacketing. Based on this concept, the discussion is mainly focused on the seismic performance of the FRP wrapped bridge columns from the perspective of the column stiffness in the inelastic stage. Hence, an up-to-date literature search concerning the inelastic performance of retrofitted columns with FRP is first represented. From the envelope curve of the hysteretic response of FRP retrofitted columns, sixteen columns exhibited the idealized lateral load-displacement relations with stable post-yield stiffness, namely, a second stiffness. An empirical model has been proposed to evaluate the second stiffness of FRP wrapped bridge columns with lap-splice and plastic hinge deficiency.

1 INTRODUCTION

Most concrete bridges in earthquake-prone regions of the world were constructed before the enforcement of modern seismic design codes and thus are inherently vulnerable to earthquakes. Short lap splices in columns are a typical deficiency in such bridges, especially when they are located in the plastic hinge region. Also, one of the failure modes of these bridges is the confinement failure of the flexural plastic hinge region, where subsequent to flexural cracking, cover-concrete crushing and spalling, buckling of the longitudinal reinforcement, or compression failure of the core concrete initiates the deterioration of the plastic hinge zone. Experimental investigations have been carried out to control the deficit in the seismic performance of bridge columns with these deficiencies. Numerous findings have shown the success of FRP retrofitting for the existing RC columns, but the highlighted success of the retrofitted columns is the augmentation of the column ductility. Nevertheless, pseudo-dynamic test results revealed that a ductile member may have no chance to entirely develop its ductile behavior in order to dissipate seismic energy because it may be suddenly destroyed by a significant pulse-like wave (Chang et al. 2004).

With the rapid development of material, design, and construction techniques, increasing attention has been paid to improving the seismic performance and recoverability of RC structures. In seismic design of structures, it is important to have a clear vision of the desired seismic performance. Decisions which help to answer the question, “What is the required performance for the structure during and after an earthquake?” are undoubtedly important. According to the Earthquake Resistant Design Codes in Japan, seismic performance can be categorized into three levels as follows: 1-Seismic Performance I, capability of maintaining the original functions
without any repair and no excessive displacement occurring during an earthquake; 2- Seismic Performance II, capability of making a quick recovery of the original functions with repairs after an earthquake; 3- Seismic Performance III, capability of keeping the overall structure in place without collapse during an earthquake. These performance levels are mainly defined by the degree of structural recovery after an earthquake (Akihiko 2004). Furthermore, seismic performance levels are also connected with the state of damage of members. Since the damage level of a member will strongly influence the structural seismic performance, proper determination of this quality is important. It is considered appropriate to determine damage levels by considering the relationship among member properties, damage state, and repair method, (Fig.1).

![Diagram](image_url)

Figure 1: Lateral load-deformation relation for reinforced concrete member, with general level of compressive axial force (Modified based on Earthquake Resistant Design Codes in Japan, 2000).

2 ENHANCEMENT OF RC STRUCTURE RECOVERABILITY

Currently, important infrastructures are required to have not only high strength and high ductility, but also usability and reparability after earthquakes. Hence, it is obvious that elastic design of bridges in important routes, in order to withstand the greatest likely earthquake without damage, would be uneconomical. Investigation revealed that the seismic recoverability of RC structures might be improved by weakening the debonding between rebar and concrete (Mutsuyoshi et al. 2005 & Iemura et al 2002). Iemura et al. (2003) and Wu et al. (2004 &2007) propose a secondary stiffness of civil structures. Recently, a new seismic design methodology has been proposed with inland earthquakes taken into account. Consequently, it is significant to realize the importance of having a reliable fusion in the inelastic stage.

3 IMPORTANCE OF USING POST-YIELD STIFFNESS IN SEISMIC DESIGN

One of the important proposed changes to the current bridge design specifications is the consideration of two earthquake designs under specific circumstances: Safety Evaluation Earthquake (SEE) and Functional Evaluation Earthquake (FEE). For the SEE, the bridge should behave in an elastic manner without any significant structural damage. For the FEE, standard bridges should prevent critical failure, while important bridges should perform with limited damages. Since the strength requirement for the FEE is much higher than the SEE, the existing RC bridge columns which satisfy the SEE must be enlarged and/or increased in reinforcement to meet the new requirements. However, if a suitable retrofitting is used effectively to assure post-yield stiffness, the FEE design criteria may be met without dramatically increasing the column section size or the amount of reinforcement. Another disadvantage of having a small post-yield stiffness is that it results in a large residual displacement response from the FEE. This large displacement significantly complicates repair work after the earthquake. Earthquake Resistant Design Codes in Japan specify that the residual displacement should not be greater than 1% of the pier height.

4 SEISMIC PERFORMANCE OF FRP RETROFITTED COLUMNS:

Since the main aim of this paper is to check the recoverability of FRP retrofitted columns, the following discussion mainly applies to the inelastic performance of the collected tested columns in terms of finding successful FRP retrofitting for columns having clear post-yield stiffness.
4.1 Columns with lap splice deficiency

There are considerable research efforts being directed at developing and applying retrofit strategies to upgrade the seismic performance of deficient lap spliced columns (Xiao et al. 1997; Haroun et al. 2005; Frieder et al. 1997; Chang et al. 2001; Kumar et al. 2007; Harries et al. 2006).

Two retrofitted columns and one repaired circular column were tested by (Xiao et al. 1997). The tested retrofitted circular column C2-RT4 successfully reached the ideal flexural capacity, showing a stable hysteretic response up to the first cycle, corresponding to a ductility factor of about 6. The increase in FRP layers by one layer engaged the hoop strain to 0.001, and the entirely retrofitted C3-RT5 circular column gained strength over the ideal flexural capacity, which appears to be an ascending straight line, (Fig.2). However, the repaired C4-RP4 developed a maximum carrying capacity approximately equal to the first yield strength as calculated on the basis of the virgin section condition.

Figure 2. Load-displacement envelope for circular lap spliced column (Specimen C3-RT5) (Xiao 1997).

Frieder et al. (1997) tested two circular lap-spliced columns; (Figs 3.a, b). Post-test measurements on the primary confinement jacket thickness of the first carbon test column revealed that the actual jacket thickness was 20% less than the required design thickness; meanwhile, this design showed a successful stable hysteretic response over the ideal lateral strength, and the lap splice debonding occurred at a displacement ductility factor > 5. The second carbon jacket test achieved the main aim of this paper by inducing a gradual increase in the lateral strength over the ideal flexural capacity, and the displacement ductility factor prior to starter bar rupture was eight.

Figure 3.a. Load-displacement envelope for circular column (test 1) (Frieder 1997)  
Figure 3.b. Load-displacement envelope for circular column (test 2) (Frieder 1997)

(Haroun et al. 2005) tested columns of circular and square cross-sections. For the FRP retrofitted circular columns, the two samples CF-R1 and CF-R2 successfully achieved the theoretical strength, but the FRP retrofitting was not sufficient to activate the existence of the post-yield stiffness. Meanwhile, the design considered for the remainder of the retrofitted circular columns CF-R3, CF-R4, and CF-R5 (Fig.4), behaved similarly and demonstrated a significant improvement in their cyclic performance with a gradual increase in the lateral capacity over the theoretical flexural strength until a ductility of about 5, which evidenced the appearance of the second stiffness. The hysteretic response of all square jacketed columns (one of these columns was a quasi-circular section with continuous confinement) had a very limited improvement in clamping on the lap-splice region (Fig.5).
4.2 Columns with plastic hinge deficiency

The wide spread of FRP applications has triggered researchers (Frieder et al. 1997; Chang et al. (NCRRE Report); Shamim et al. 2002) to develop and apply retrofit strategies to upgrade the seismic performance of deficient plastic hinge regions of columns.

The design scenario considered by (Frieder et al. 1997) is to calculate the appropriate composite jacket thickness for upgrading the limited inelastic deformation at the plastic hinge region under the effect of seismic action; this action is also appropriate for inducing a second stiffness after the elastic one until a displacement ductility \( \geq 7 \); (Fig. 6).

Rectangular and circular columns with a weak confinement at plastic zones were tested (Chang et al. NCREE’s program). For the tested rectangular column FR1, the retrofit efficiency of force-based design was expedient; where the lateral strength during the pulling cycles of the tested column increased steadily after the global yielding of the columns, the displacement ductility equaled 5, (Fig. 7.a). With respect to the displacement-based design only, the CFRP thickness was appropriate for improving the seismic performance of the rectangular columns. For circular tested columns, however, the retrofit efficiencies of force-based design and of displacement-based design were nearly same. The pulling and pushing hysteretic response of retrofitted circular column FCF3 demonstrated no deterioration for the column stiffness after the theoretical lateral capacity of the column (Fig.7.b). When column FCF2 was tested with lateral force until a 0.5 mm crack width was initiated and then was repaired with 4-layer CFR, the hysteretic response of the repaired column was almost the same as the bridge columns without cracks (Fig. 7.c).

5. RESIDUAL DISPLACEMENT RESPONSE OF FRP RETROFITTED COLUMNS

After an earthquake, reconstruction and repair of the bridge structures should begin for the restoration of necessary routs. One measurement which must be checked during the investigation and reconstruction/repair is the level of inclination of the columns. In some cases, large residual inclination makes placing of the bridge girders difficult and causes visual uneasiness even if repair is possible (Fujino et. al. 2005). Figure 8 shows the effect of post-yield stiffness increase on the residual inclination ratio, where the increase in the second stiffness of retrofitted columns is accompanied by a decrease in the residual inclination, an important indication of how to control the second stiffness value.
Figure 8. The effect of the post-yield stiffness on the residual inclination of FRP retrofitted columns.

6. EMPIRICAL MODEL FOR CALCULATION OF THE SECOND STIFFNESS VALUE OF FRP RETROFITTED COLUMNS WITH LAP SPLICE OR PLASTIC HINGE DIFFICIENCY

The collected and tested FRP retrofitted columns are 54 circular, square, and rectangular columns with aspect ratio \((L/D) > 4\). Based on the aim of FRP retrofitted columns with quick recoverability for the original bridge functions after the earthquake, sixteen FRP retrofitted columns with circular and rectangular cross-sections have successfully been shown to have clear post-yield stiffness. To calculate the second stiffness, a model is established based on a comprehensive trial model that includes as many potentially influential parameters as possible. It is suggested that the longitudinal reinforcement ratio, the axial load ratio, the FRP confinement ratio, the aspect ratio, and the hoping spacing ratio are the important factors influencing the second stiffness value. Using regression analysis, the inclination of the inelastic branch can be calculated from proposed equations (1, 2).

\[
K_2 = 0.3038X + 0.024
\]  

\[
X = 1.6\rho f_y + 1.2\frac{P}{A_g f_y} - 0.5\left(\frac{f_c}{f_y} + \frac{L}{D}\right) - 6\frac{S}{D} + 5.0
\]

where \(K_2\) is the second stiffness, \(\rho\) is the longitudinal steel reinforcement ratio, \(f_y\) is the yield strength of the longitudinal steel, \(f_c\) is the compressive concrete strength, \(P\) is the constant axial force, \(A_g\) is the gross area of the cross-section, \(f_l\) is the FRP confinement ratio, \(D\) is the width of the cross-section, \(L\) is the column length, and \(S\) is the hoping steel spacing. The FRP confinement ratio \(f_l\) equals \(0.5\rho f_l\), where \(f_l\) is the ultimate tensile strength of FRP, and \(\rho\) is the volumetric ratio of FRP to concrete, which can be determined as follows: \(\rho = 4t_f/D\) for circular continuous wrapping and \((2t_f(b+h))/ (b.h)\) for continuous rectangular wrapping, where \(t_f\) is the thickness of the FRP jacket at the retrofitted deficient zone, and \(b\) and \(h\) are the rectangular cross-section dimensions. A comparison between the predicted and the experimental values of the post-yield stiffness is shown in (Table1). The static correlation coefficient \(R^2\) of equation (1) is 0.934. The average ratio of the calculated values of equation (1) to the test results is 1.008 with a standard deviation of 0.093. Units of the equation parameters are in (mm) for dimensions and (N/mm²) for stresses.

<table>
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<tr>
<th>Author</th>
<th>Sample</th>
<th>Deficiency</th>
<th>Cross-Section</th>
<th>b/D</th>
<th>l/D</th>
<th>(f/f_y)</th>
<th>(P/f_b A_g)</th>
<th>(A_g f_b / A_f l)</th>
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<th>Error/Ratio</th>
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<td>0.067</td>
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* the experimental second stiffness value, ** the calculated second stiffness value
CONCLUSION:
The existence of a post-yield stiffness in the column hysteretic response is the measure of success of FRP retrofitted or repaired columns, as stressed here. The following conclusions are derived from the inclusive data conducted on columns with lap splice or plastic hinge deficiency:
1. Sixteen FRP retrofitted circular and rectangular columns showed that the idealized hysteretic lateral load-displacement response has a post-yield stiffness; consequently, the FRP-RC columns of the important bridges can utilize a seismic design methodology of “no damage during a small earthquake, and prompt recoverability during a medium to large earthquake”.
2. For deficient columns retrofitted with FRP, the higher the value of the post-yield stiffness, the smaller the residual displacement response results.
3. Although many parameters varied among the considered specimens, the proposed empirical formula to predict the second stiffness of FRP wrapped RC bridge columns with lap splice or plastic hinge deficiency shows acceptable accuracy in comparison to the experimental results.

REFERENCES