SEISMIC BEHAVIOUR OF GRAVITY-DESIGNED RC FRAME RETROFITTED BY FRP

Y. Lu [1] and J.W. Wei [2]

[1] *Professor, School of Engineering and Electronics, The University of Edinburgh, UK*
[2] *PhD, School of Civil and Environmental Engineering, Nanyang Technological University, Singapore*
Email: yong.lu@ed.ac.uk

ABSTRACT:
Fibre reinforced polymer (FRP) is being used widely in repairing and retrofitting structures which suffer from damage or some design deficiencies. For gravity-designed (or generally nonseismically designed) structures, FRP strengthening also appears to be an obvious choice when comes to the need of upgrading for improved seismic resistance. However, typical FRP strengthening is known to be susceptible to debonding and such problem is expected to become more serious under cyclic deformations. This paper investigates the effectiveness of FRP retrofitting for enhancing the seismic performance of RC frames. A half-scale reinforced concrete test frame, originally designed without considering any seismic requirements, is constructed and subsequently subjected to simulated earthquakes until a significant degree of damage is generated. The damaged frame is then repaired using glass fibre reinforced polymer (GFRP) following a typical repairing procedure. The repaired frame is subsequently tested until final failure. The observed responses before and after the repairing are compared. The effectiveness of the retrofitting scheme is discussed in terms of the restoration of the initial stiffness, the evaluation of damage, and the base shear strength and ductility. The development and pattern of debonding during the course of tests are highlighted.

KEYWORDS:
RC frame, fibre reinforced polymer, shake table test, repairing, debonding

1. INTRODUCTION

The need for repairing and strengthening of structures arises following a damaging event such as an earthquake, or when upgrading of an existing structure becomes necessary in order to meet the updated loading or safety requirements. Among various repairing and retrofitting strategies, the Fiber Reinforced Polymer (FRP) is perhaps the most popular choice because of its merits such as a low weight-to-strength ratio. However, typical FRP strengthening is known to be susceptible to debonding and this problem is expected to become more serious under cyclic deformations.

Comparing to the numerous studies on FRP strengthening of structures under monotonic loads (e.g. Sheikh 2002, Antonopoulos and Triantafillou 2003, Prota et al. 2004, Balsamo et al. 2005), the research on the performance of FRP strengthened structures under seismic loading is much less intensive, and systematic information on the criteria for debonding under cyclic load, especially at the joint and connection regions, needs to be accumulated.

This paper is aimed to investigate the effectiveness of FRP strengthening on the seismic performance of RC frames. For this purpose, a reinforced concrete test frame was constructed at half scale and tested under simulated earthquakes until a significant degree of damage was generated. The damaged frame was then repaired using glass fibre reinforced polymer (GFRP) following a typical repairing procedure. The repaired frame was subsequently tested until final failure. The test results are analyzed to examine the effectiveness of the retrofitting in terms of the restoration of stiffness, strength and deformation capabilities, as well as the failure mode. In particular, the process of debonding at the column-foundation interface and at some joints is scrutinized.

The experimental programme was in association with a study on the behaviour of gravity-designed frames under seismic loading. The test results also provide useful insights with regard to the potential seismic resistance of this category of RC frames.

2. TEST STRUCTURE AND TEST PROGRAMME

The two-storey one-bay frame was aimed to represent a typical low-rise RC frame in a low-to-medium seismic region (such as Singapore). The frame was designed according to British Standard BS8110 (1997), without considering specific seismic provisions. The test frame was constructed at half-scale. Figure 1 shows the dimensions and reinforcement arrangement. The model materials have the following characteristic properties: concrete compressive strength 30 MPa, flexural reinforcement yield strength 460 MPa.

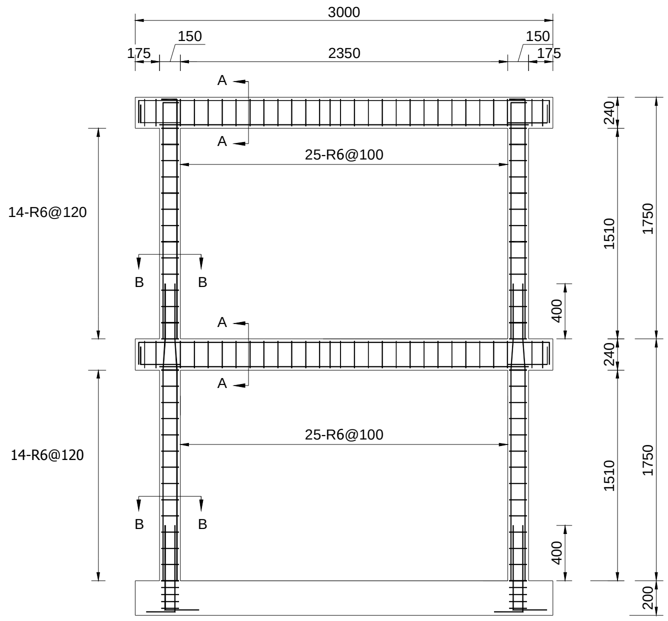


Figure 1 Geometry of test frame

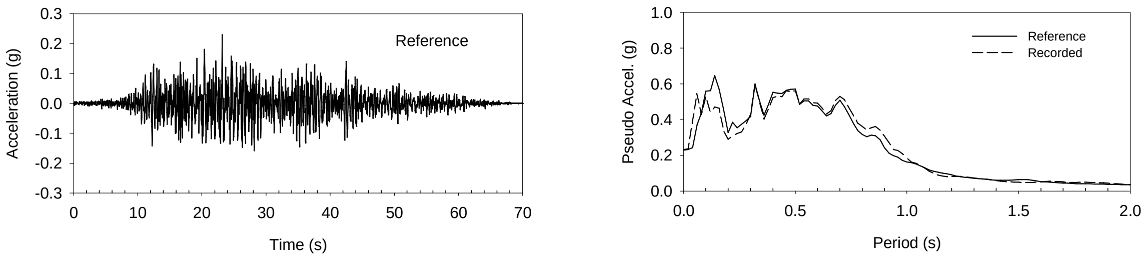


Figure 2 Accelerogram and response spectra of the simulated ground motion

According to the similitude laws (e.g. Harris and Sabnis 1999), additional mass is required in the reduced scale model structure dynamic test. Considering the limit capacity of the shake table and the fact that two pieces of frames were tested simultaneously, an additional mass of 700 kg per storey per frame was applied. The additional mass was made from concrete blocks and they were fixed on the floor flanges with bolts. It should be mentioned the two-frame test setting was devised to result in a self-supported system in the out-of-plane direction. But the two frames were uncoupled in the main loading direction, by means of specially designed sliding couplers, so the response was practically independent from each other. The results presented in this paper concern one test frame.

For the shake table test, the ground motion was modeled after the 1985 Chile earthquake, recorded at Vina del Mar site. The earthquake had a magnitude (Ms) of 7.8 and it occurred off the coast of central Chile. The selected record had a long effective period of about 40 seconds. For the reduced scale models with a length scale 1:2, the input excitation was obtained by compressing the time scale of the original ground motion by a factor of $1/\sqrt{2}$ according to the similitude laws. Figure 2 illustrate the time history of the simulated ground motion and the corresponding response spectrum.

The original frame was tested with a series of simulated earthquake ground motions of increasing intensity, with the peak ground acceleration (PGA) varying through 0.12g, 0.23g and 0.46g. The damaged frame after the 0.46g test was then repaired. The repaired frame was subsequently tested following the same sequence of the ground motions. An additional test with PGA equal to 0.69g was performed at the final stage to bring the repaired frame to failure. A low amplitude random base excitation test was conducted on the original and each damaged state of the frame to measure the natural frequencies of the frame at different stages.

3. BRIEF DESCRIPTION OF TEST RESULTS FROM ORIGINAL FRAME

The original frame at its initial state was measured to have a fundamental natural frequency equal to 4.6Hz. No visible damage was observed after the 0.12g test. Minor cracks appear after the frame was subjected to the 0.23g test, but the frame essentially remained intact. Such a good performance is worth noting considering that the frame was not designed for this level of seismic actions. One of the reasons could be attributed to the inherent lateral load resistance of the monolithically cast concrete frame. Besides, the axial load level in the columns of the two storey frame was quite low, and this rendered the columns to behave in an appreciably ductile manner.

The frame exhibited significant damage after the 0.46g test, with wide spread cracks throughout the whole frame, especially at the bottom ends of the columns and around the beam-column joints. Figure 3 shows some typical crack patterns. The maximum inter-storey drift was measured to be about 2.5% (first storey).

Figure 3 Typical crack patterns of original frame after 0.46g test

4. REPAIRING SCHEME

The primary interest here is to investigate a commonly-used repairing scheme for moment-resisting frames for seismic retrofitting. Thus, the uniaxial fiber texture was selected among the available fibers such as biaxial and quadriaxial fibers. The TYFO FIBRWRAP SYSTEM with uniaxial Glass Fabric Reinforced Polymer (GFRP), provided by a local supplier, was used for this repair study. The GFRP material is orientated in the 0-degree direction with additional yellow glass cross fibers at 90-degree direction. Before the FRP wrapping was applied, some preparation work on the damaged frames was performed. The cracks were epoxy injected and the concrete surface at member ends was cleaned by grinding off all loose materials. A layer of epoxy was applied to the finished surface of the test frames. The glass fabric was saturated with epoxy and then applied on the surface areas for repair.

The arrangement of the glass fabric was made according to the damage patterns so that the fiber orientation is aligned perpendicular to the crack lines where possible. Since the damage was mainly concentrated at the member end regions, the FRP wrapping was only applied to these critical damage regions. The repair scheme was completed in four steps, 1) installation of L-shaped FRP laminates around the interface corners, 2) installation of vertical laminates, 3) installation of horizontal laminates, and 4) installation of wrapping laminates at member ends. The repaired frame after completion of the FRP wrapping is shown in Figure 4.

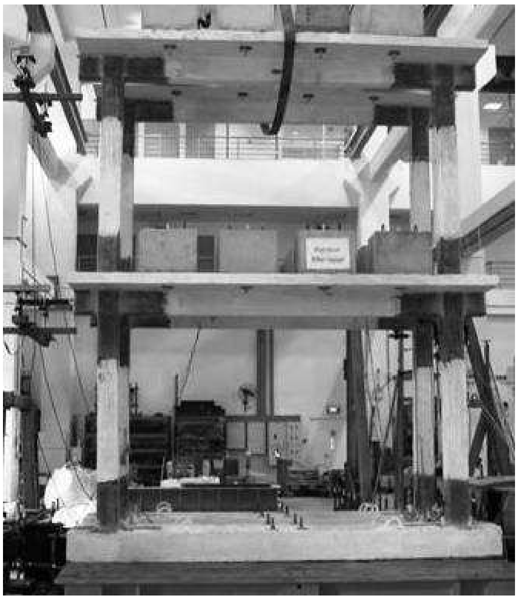


Figure 4 FRP repaired frame

5. TEST RESULTS OF REPAIRED FRAME AND DICSUSSION

5.1. General test results

The fundamental natural frequency of the repaired frame was found to be 4.82 Hz. Comparing to the frequency of the damaged frame of 2.05 Hz before repairing, this indicates a considerable increase of the overall stiffness of the frame by several folds. In fact, comparing to the frequency of the original undamaged frame which was 4.6 Hz, it can be seen that the repairing scheme was able to completely recover the initial stiffness of the original frame.

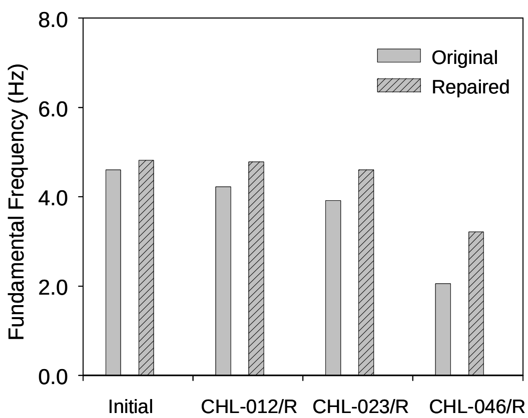
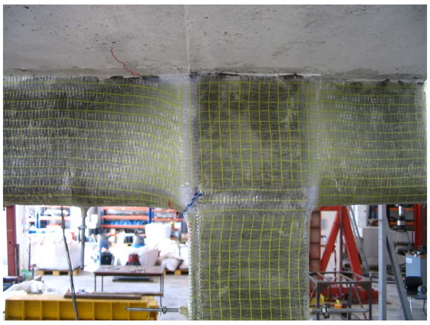


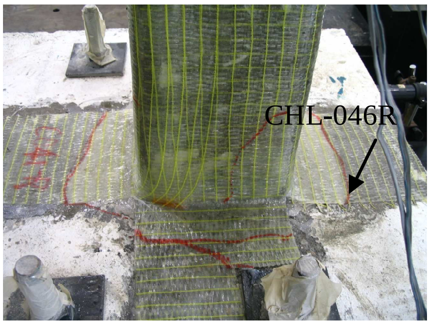
Figure 5 Fundamental frequencies of test frames (original and repaired)

The progress of the degradation of the repaired frame during the course of the tests can be observed from the reduction of the fundamental frequency after each consecutive earthquake excitation, as shown in Figure 5. Comparing to the original frame, it can be seen that the repaired frame exhibited a similar rate of stiffness degradation up to the maximum intensity that the original frame was subjected to. This is interesting in view of the fact that the repaired frame actually started to show noticeable debonding at the critical bottom ends of the columns following the 0.46g tests. The development of debonding did not effect to cause an abrupt deterioration of the global stiffness of the frame.

Visual inspection of the condition of the FRP wrapped regions revealed that minor degree of debonding initiated during the test of 0.23g. Significant debonding actually occurred at the column-foundation interface during test 0.46g, and the debonding region extended outward from the column edge for a length about half of the column depth, as shown in Figure 6. On the other hand, no debonding was observed at the beam-column joint regions. This may be partly attributed to the fact that the geometry around the joint region in the present 2D test frame allowed for a continuation of the FRP wrapping across the joint region, thus alleviated the concentration of the bonding stress during the response. It should be noted here that this observation may not be simply extended to 3D joints, as in a 3D setting the interface condition between FRP and the frame at the joint regions would be similar to the bottom end of the columns. This implies that in a 3D setting the repaired beam-column joints could experience a similar debonding problem as what is observed here at the bottom ends of the columns.



(a) No debonding in the joint



(b) Debonding at the base interface

Figure 6 Close-up of observed damage of repaired frame after 0.46g test

During the final test with PGA reaching 0.69 g, the FRP layer was observed to tear off from the foundation surface across a substantial length, marking a complete loss of the retrofitted connection effect at the column bottom end. The column connection at the base level became almost like a hinge support. This is shown in Fugure 7b). At the same time, failure of the first-storey column also occurred in the upper portion near the edge of the repaired region, as can be observed from Figure 7a).

(a) Column failure

(b) Debonding at the base interface

Figure 7 Close-up of observed damage of repaired frames after CHL-069R

To further examine the behaviour of the repaired frame, the envelope relationship between the base shear and the roof displacement was extracted from the measured hysteresis loops. Figure 8 illustrates the comparison of the base shear-roof displacement skeleton curves between the original and the repaired frame. It can be seen that the repaired frame had similar initial stiffness as the original frame, this echoes the observation mentioned earlier from the measured natural frequencies. The repaired frame exhibited a slightly reduced base shear resistance, but an appreciable increase in the ductility.

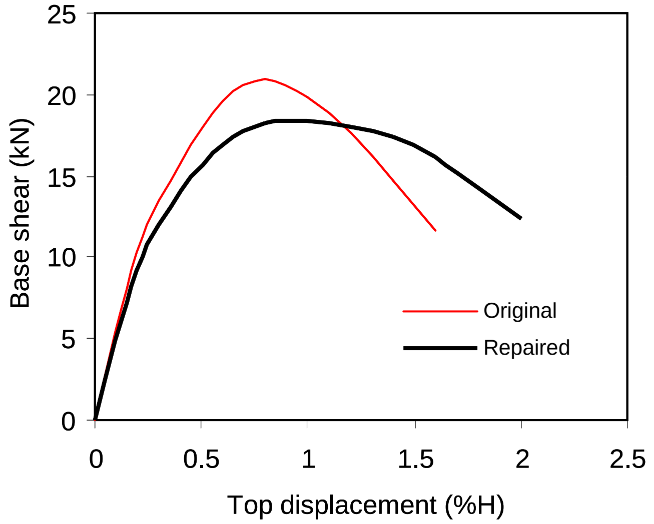


Figure 8 Base shear vs. top displacement envelopes of test frame

5.2. Failure mechanism and discussion

Failure of the repaired frame occurred during the final test of 0.69g. The failure pattern of the repaired frame can be characterized by major cracks across the wrapped edge in the upper portion of the first storey columns and plastic hinges. The FRP wrapped joint regions with two overlaid plies of fibers appeared to have effectively strengthened the flexural/shear strength of the joint regions, such that the whole wrapped region around a joint appeared to behave like an expanded rigid zone. Consequently, the column section immediately below the wrapped zone became a critical location. As a result, a plastic hinge occurred at this location rather than at the usual column upper ends.

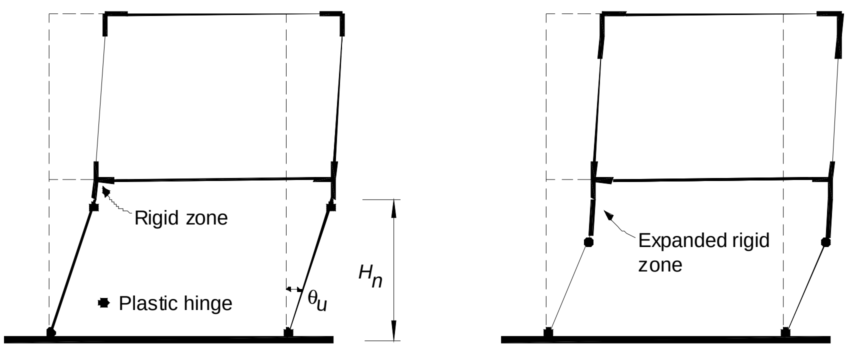


Figure 9 Schematic of final failure modes of test frame

Figure 9 illustrates a schematic comparison of the final failure modes between the original and the repaired frame. Despite a similar storey mechanism in the first storey upon failure, the expanded rigid zones around beam-column joints of the repaired frame tend to result in a shorter clear height of the first-storey columns. Since the curvature capacity at a particular section of the upper part column can be regarded as pre-determined, this reduction of the column clear height would potentially result in a reduction of the inter-storey drift capacity from a general point of view. The reason that in the current tested frame, the drift and ductility capacities in the repaired frame were actually higher than the original frame can be attributed to the low ductility capacity of the original frame due to its non-seismically designed features.

The above observations bring up a significant aspect that one should take into account in the design and evaluation of FRP repaired frames. The creation of extended "rigid" zones could alter the configuration of the moment and shear force distributions, and hence shift the critical regions to locations which were not anticipated and designed as critical regions in the original design.

6. CONCLUSIONS

In this paper, the seismic performance of a RC frame repaired with FRP wrapping is investigated by means of shake table tests. The test results are discussed in comparison with the results from the original frame. Based on this study, the following conclusions may be drawn:

1) The application of FRP laminates proved to work very well in restoring the initial strength and stiffness of a seismically damaged RC frame.

2) Comparing to the behaviour of the original frame which was designed without seismic considerations (thus had a relatively low ductility capacity), the FRP repaired frame exhibited increased drift capacity as well as the overall ductility. The ductility was increased from about 2.0 to about 3.0.

3) The FRP wrapping was most effective on the body of individual columns, followed by the beam-column joint regions. The least effective region appeared to be at interface between the column bottom and the foundation top face. Consequently, debonding started to occur at a relatively low drift level, and became significant when the interstorey reached about 2%.

4) FRP repaired frame exhibited a distinctive failure mode, which may be characterized by a storey mechanism involving hinges at the column bottom ends, partly attributable to debonding, and shifted hinges in the upper portion of the columns at the lower edge of this repaired region. This mechanism results in a general reduction of the column clear height, therefore to a certain extent would affect negatively the drift capability of the columns.

Despite the generally encouraging results as observed from this study regarding FRP retrofitting for seismic resistance, it becomes clear that care should be exercised in dealing with the debonding problems, especially in regions where effective wrapping or continuation of the FRP strips can not be guaranteed. Effective methods for enhanced bonding and anchorage in such regions should be further investigated.

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