Seismic performance of reinforced concrete interior beam-column joints and walls-strengthened with fibre reinforced polymer composites

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SEISMIC PERFORMANCE OF REINFORCED CONCRETE INTERIOR BEAM-COLUMN JOINTS AND WALLS – STRENGTHENED WITH FIBER REINFORCED POLYMER COMPOSITES

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in partial fulfillment of the requirement for the degree of
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Summary

Amongst the many rehabilitation methods, externally bonded fiber reinforced polymer (FRP) composites using the wet lay-up method has been an attractive solution. To date, there is a major consideration on its effectiveness in restoring and strengthening seismically damaged reinforced concrete structures and hence, further investigations and research is needed to demonstrate its effectiveness. In this research program, experimental studies have been carried out to evaluate the effectiveness of FRP composites in strengthening beam-column joints and structural walls with and without openings.

For the beam-column joints, six specimens were tested and their behavior studied. After which, two proposed FRP strengthening schemes were applied in two experimental studies in this research program to restore six damaged interior beam-column joints and to enhance the seismic performance of three other virgin interior beam-column joints. The FRP strengthening technique involved the use of externally bonded GFRP and CFRP strips and sheets. For the structural walls, six specimens were tested and their behavior studied. After which, proposed FRP strengthening schemes were applied to restore the seismic performance of four damaged walls without openings and to restore the seismic performance of two damaged walls with regular and irregular distributed openings using the strut-and-tie model approach. The FRP strengthening technique involved the use of externally bonded GFRP and CFRP strips and sheets.

Experimental investigations above confirmed the feasibility and viability of the proposed repair and strengthening techniques, towards improving the seismic performance of beam-column joints and structural walls with and without openings. The proposed FRP strengthening schemes provided a platform to help develop FRP strengthening strategy for beam-column joints and serve to be potentially efficient for mass upgrading of structures not suitably designed to withstand earthquakes. Strut-and-tie modeling approach is a useful tool in effective FRP wrapping techniques for restoring the strength of damaged walls with and without openings.

Abstract

Recent earthquakes have highlighted the vulnerability of existing structure to seismic loading. In fact, the ability of structures to endure severe ground motion depends primarily on high inelastic deformation capacities of key structural components that allow seismic energy absorption and dissipation without significant strength degradation. Amongst the many rehabilitation methods to enhance and restore these structural components, externally bonded fiber reinforced polymer (FRP) composites sheets and strips has been an attractive solution. To date, there have been insufficient experimental studies to evaluate the effectiveness of FRP composites in strengthening beam-column joints and structural walls with and without openings.

In the first part of the thesis, two experimental studies provided experimental evidence of the effectiveness of FRP composites in: 1) restoring the seismic performance of six damaged interior beam-column joints and 2) enhancing the seismic performance of three virgin interior beam-column joints. The FRP strengthening technique involved the use of externally bonded GFRP and CFRP strips and sheets. All tested specimens were subjected to cyclic lateral loading and column axial load of $0.10 f'_c A_g$ and $0.30 f'_c A_g$.

In the second part of the thesis, two experimental studies provided experimental evidence of the effectiveness of FRP composites in: 1) restoring the seismic performance of four damaged walls without openings and 2) restoring the seismic performance of two damaged walls with regular and irregular distributed openings using the strut-and-tie model approach. The FRP strengthening technique involved the use of externally bonded GFRP and CFRP strips and sheets. The walls without openings were subjected to both cyclic lateral loading and axial load while the walls with openings were only subjected to cyclic loading.

The experimental results showed that the failure of the repaired beam-column joints was effectively delayed, recovered in strength and exhibited more ductile response compared to the as-built specimens. The strengthened virgin specimens showed to have tremendous increase in strength, stiffness and energy dissipation capacity. The repair and strengthening techniques showed that applying FRP round the column

provided confinement and additional shear reinforcement to the joints, ensuring a flexural dominant mode. This useful technique can be applied to joints without transverse reinforcement. Applying carbon fiber reinforced polymer (CFRP) strips as cross bracings on beam and column faces for strong-column weak-beam was effective in flexural strength. The use of fiber anchors in preventing the delamination of FRP sheets was effective in enhancing the capacity of the FRP sheets. These anchorages also ease constructability on site.

For walls without openings, the experimental results for the repaired specimens showed that flexural strength and stiffness was improved. The additional use of CFRP laminate strips was effective in recovering original strength and ductility. Shear cracking was controlled by much reduced shear displacement by using FRP jackets which were further confined by a glass fiber reinforced polymer (GFRP) sheet. For walls with openings, the repaired specimens recovered their strength, stiffness and energy dissipated capacity reasonably well. The fiber anchors successfully arrested the shear cracks, enhancing the shear and sliding capacities of the repaired walls.

The results of the experimental investigations above confirmed the feasibility and viability of the proposed repair and strengthening techniques, towards improving the seismic performance of beam-column joints and structural walls with and without openings. The proposed schemes will help to develop FRP strengthening strategy for beam-column joints and serve to be potentially efficient for mass upgrading of structures not suitably designed to withstand earthquakes. Strut-and-tie modeling approach is a useful tool in effective FRP wrapping techniques for restoring the strength of damaged walls with and without openings; amount of FRP strips are used efficiently and save FRP material when mass upgrading of structures is concerned.

1 Introduction

1.1 Background

Urban earthquakes (Turkey 1999, 2003, Taiwan 1999, Pakistan 2005 and Sichuan 2008) have highlighted the vulnerability of existing buildings in seismic regions. Older hazardous buildings are responsible for the thousands of lives lost and severe damages observed especially in the last decade. A reconnaissance team dispatched by Miyamoto International and Global Risk Miyamoto surveyed the devastated sites in the recent severe earthquake in on May 12, 2008 in Sichuan, China, which had cost many lives and incurred large economic losses. The team reported that the non-ductile detailing of concrete elements and unreinforced masonry was the cause of a collapsed school building constructed in 1996 (Miyamoto 2008). Such construction which has always performed very poorly in strong earthquakes is still practised though it has been outlawed in California, for example, since the mid 1970s.

To withstand severe ground motions, structures depend primarily on the high inelastic deformation capacities of key structural components that allow seismic energy absorption and dissipation without significant strength degradation. Excessive shear deformation and severe strength degradation of reinforced concrete (RC) structural components in non-seismically detailed buildings led to their full collapse. Depending on the global characteristics of the structural system and the imposed local deformation demand, poorly detailed elements may become critical components during seismic excitation as they possess inadequate resistance to reversed cyclic load.

Being exposed to extreme shear and bond demands, beam-column joints are often the weakest links in a structural system. Their susceptibility to extensive damage resulting in a reduction of the strength and ductility of adjacent framing members has drawn much attention recently; one example is the Northridge earthquake where joint failures were observed (see Figure 1-1). Another example of catastrophic failures due to earthquakes is the 1999 Turkey earthquake which showed the vulnerability of RC beam-column joints built without seismic considerations (see Figure 1-2).

Figure 1-1: Joint failures after 1994 Northridge earthquake (Moehle J.P 2000)

Figure 1-2: Corner joint failure in 1999, Izmit/Turkey earthquake. Photo courtesy of National Information Service for Earthquake Engineering, University of California Berkeley. (Engindeniz *et al.* 2005)

Another key structural component which has drawn recent research attention is RC structural walls. According to Fintel (1991), buildings containing structural walls exhibited extremely good earthquake performance. Attributing to their ability to resist lateral load effectively, RC structural walls serve as the main earthquake resistance mechanism in the seismic design of concrete structures. Post- earthquake reconnaissance missions repeatedly note that RC structural walls fail almost

exclusively in shear with a few exceptions. One example is observed in the recent earthquakes in Turkey where shear dominated failure mechanism, which results in low ductility response typically with severe strength degradation and a low level of energy dissipation, was identified in the RC structural walls as shown in Figure 1-3.

Figure 1-3: Shear failure of a RC wall due to insufficient transverse reinforcement; a) Bolu (Turkey, 1999), b) Bingol (Turkey, 2003)

Building collapses due to RC beam-column joints and structural deterioration due to RC structural walls, especially in non-seismically detailed buildings under seismic hazard, have sounded an overwhelming need to mitigate and restore the structural integrity of such critical structural components in the aftermath of an earthquake. It has provided significant impetus in much research effort devoted in repair and strengthening techniques of beam-column joints and structural walls in the recent years. To date, the common techniques include epoxy injection, removal and replacement of concrete, concrete or steel jacketing, increase of transverse reinforcement, adding of steel bracing elements, filling in openings with RC and masonry infills and FRP applications.

Fiber reinforced polymers (FRP), characterized by its high strength-to-weight ratios, corrosion resistance, ease of application and tailorability; fiber orientation in each ply can be adjusted to meet specific strengthening objectives (Engindeniz *et al.* 2005), has been the material of choice in many applications. In particular, the use of externally bonded FRP sheets has been one of the recent promising techniques in increasingly large number of seismic retrofit projects where increased strength and/or inelastic deformation capacity (ductility) must be achieved. Among the structural elements,

this technique receives wide acceptance for the strengthening of beams and columns and increasingly so for beam-column joints in the recent years. However, it is seen as an uncommon practical retrofit measure on RC structural walls, in particular, those with openings.

1.2 Objectives and Scope

1.2.1 Beam-Column Joints

The experimental studies in this research aim to provide experimental evidence of the effectiveness of FRP composites in: 1) enhancing the seismic performance of non-damaged interior beam-column joints and 2) restoring the seismic performance of damaged interior beam-column joints. All specimens are non-seismically detailed and typical joints found in Singapore. Two experimental studies, each employing the same experimental set-up and procedures, are performed.

The first experimental study aims to enhance the seismic performance of three non-damaged RC interior beam-column joints, namely, one eccentric and two concentric joints. Innovative FRP wrapping schemes, using a combination of glass fiber reinforced polymer (GFRP) and carbon fiber reinforced polymer (CFRP) sheets and strips, are proposed to assess their effectiveness. These three specimens are subjected to axial compression load of $0.35 f'_c A_g$ and quasi-statically simulated low to moderate seismic loading.

The second experimental study aims to restore the seismic performance of six damaged RC interior beam-column joints consisting of two eccentric and four concentric joints. In particular, the key parameters are the level of axial load and the type of FRP (GFRP/ CFRP) used. The six specimens are grouped into “A” and “B” joints; each set consists of 1 eccentric and two concentric joints. Group A joints are repaired with GFRP while Group B joints are repaired with CFRP. Group A and B joints are subjected to $0.1 f'_c A_g$ and $0.35 f'_c A_g$ axial compression load respectively and both undergo quasi-statically simulated low to moderate seismic loading.

1.2.2 Structural Walls

Two experimental studies in this research aim to restore the seismic performance of earthquake damaged RC structural walls with and without openings. The first experimental study aims to understand the seismic performance of repaired and strengthened RC structural walls with boundary elements (confined with limited reinforcement) subjected to seismic loadings. Four RC structural walls without openings, consisting of two low-rise and two medium-rise walls, initially subjected to axial compression loading and cyclic lateral loading to failure, were subsequently repaired and strengthened with externally bonded glass fiber reinforced polymer (GFRP) and carbon fiber reinforced polymer (CFRP) sheets with an addition of key parameter CFRP strips for cross bracings at the main face of the walls.

The second experimental study investigates the effectiveness of using a strut-and-tie approach as a tool to FRP strengthening of earthquake damaged RC structural walls with openings. Two medium-rise RC structural walls with irregular and regular distributed openings, initially subjected to cyclic lateral loading to failure, were repaired and subsequently strengthened by externally bonded glass fiber reinforced polymer and carbon fiber reinforced polymer strips which were designed and applied according to the load magnitudes and paths in the proposed strut-and-tie models.

1.3 Thesis Outline

This thesis comprises of five Chapters which provide a review of relevant literature to this research, experimental investigation on the repair and strengthening of beam-column joints and structural walls using FRP composites and finally a summary of the conclusions drawn from this research.

Chapter 2 provides a review of relevant literature to this research project. Included in this review is a summary of the characteristics of FRP, an overview of common wrapping techniques using externally bonded FRP and a review of the experimental studies on the seismic performance of FRP-strengthened beam-column joints and structural walls.

Two experimental investigations on enhancing and restoring the seismic performance of non-damaged and damaged interior beam-column joints using FRP composites are presented in Chapter 3. It also includes descriptions of the as-built specimens and the experimental program. A brief overview on the failure modes of the as-built specimens is presented. The repair and strengthening techniques for the damaged specimens are also presented. Comparisons of the seismic performance of the specimens before and after repair are discussed.

Chapter 4 presents an experimental study on seismically damaged four RC structural walls without openings repaired with FRP composites. The as-built specimens and the experimental program are described. A brief overview of the failure modes of previously damaged specimens are given. The proposed FRP wrapping schemes are presented. Comparisons of the seismic performance of specimens before and after repair are discussed.

It also aims to find out the effectiveness of strut-and-tie approach as a tool for FRP strengthening through an experimental study on the repair of two medium-rise RC structural walls with irregular and regular distributed openings. The as-built specimens and experimental program are described. A brief overview of the failure modes of previously damaged specimens are given. The strengthening technique involving externally bonded GFRP and CFRP strips, which were designed and applied according to the load magnitudes and paths in the proposed strut-and-tie models, is presented. Comparisons of the seismic performance of specimens before and after repair are discussed.

Conclusions drawn from the experimental investigations performed in this thesis are discussed in Chapter 5. Areas that require future research, as highlighted during this research project are also discussed.

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2 Review of Relevant Literature

2.1 Fiber Reinforced Polymers (FRP)

2.1.1 Introduction

For the past few decades, structural repair and strengthening have received considerable emphasis due to ageing infrastructure and the need for upgrading of buildings to meet stringent design requirements. It is more evident in earthquake-prone areas where seismic retrofitting works has become increasingly important. State-of-the-art strengthening and retrofit techniques increasingly utilize externally bonded FRP composites which offer unique properties in terms of strength, lightness, chemical resistance and ease of application. These techniques are most attractive for their low labor costs, fast execution and tailorability where fiber orientation in each ply can be adjusted to meet specific strengthening objectives.

This section reviews the materials, mechanical behaviors and environmental resistances of FRP composites. It also examines the applications of FRP composites to meet strengthening objectives. First, the various types of fibers and resin matrices are introduced. Second, the mechanical behaviors such as strength, orientation, creep and fatigue resistance are reviewed. Finally, FRP wrapping strategies to meet strengthening objectives are presented.

2.1.2 Materials

FRP composites are formed by embedding the continuous fibers in resins. The three common fibers are carbon (C), glass (G) and aramid (A) fibers. Resins such as epoxy, polyester and vinylester resins are used both as matrix for the FRP and as the bonding adhesive between the FRP and the concrete. Weak adhesives can cause interfacial failures. Epoxy resins are generally used when bonding function is of crucial importance.

There are two common methods of forming FRP composites: wet lay-up method and prefabrication of FRP composites. Wet lay-up method involves the *in situ* application

of resin to either a woven fabric or a unidirectional tow sheet. Prefabrication of FRP composites in various forms includes filament winding for shells for column strengthening and pultrusion for plates. Wet lay-up method is more preferred as it is versatile in site application in terms of bonding to curved surfaces and wrapping around corners while prefabrication allows better quality control (Teng *et al.* 2002).

2.1.3 Mechanical Behaviors

2.1.3.1 Strength

All the three FRP materials (GFRP, CFRP and AFRP) have the same stress-strain behavior:

linear elastic up to brittle rupture when subject to tension. Figure 2-1 shows the comparison of strength and stiffness of A36 steel, glass and carbon fibers. From the figure, GFRP and CFRP do not have the ductility that steel possessed. Their brittleness may limit the ductile behavior of RC members strengthened with FRP composites.

Figure 2-1: Typical FRP and mild steel stress-strain curves (Teng *et al.* 2002)

Another implication of the brittleness behavior of FRP is that the redistribution of stresses is restricted owing to the lack of ductility. As a result, the design of structures with FRP composites cannot follow existing methods for RC structures with FRPs

simply treated as equivalent reinforcement (Teng *et al.* 2002). Table 2-1 shows the typical properties of fibers.

Table 2-1: Typical properties of fibers (fib 2001)

Materials	Elastic modulus (GPa)	Tensile strength (MPa)	Ultimate tensile strain (%)
Carbon			
High strength	215- 235	3500- 4800	1.4- 2.0
Ultra high strength	215- 235	3500- 6000	1.5- 2.3
High modulus	350- 500	2500- 3100	0.5- 0.9
Ultra high modulus	500- 700	2100- 2400	0.2- 0.4
Glass			
E	70	1900- 3000	3.0- 4.5
S	85- 90	3500- 4800	4.5- 5.5
Aramid			
Low modulus	70- 80	3500- 4100	4.3- 5.0
High modulus	115- 130	3500- 4000	2.5- 3.5

2.1.3.2 Orientation

The arrangement and interaction of fibers and surrounding matrix dominates the mechanical behavior of the FRP composites. For instance, chopped fibers mixed with resin matrix would cause the laminate to appear isotropic due to the randomness of the fibers. However, when continuous roving is placed in orthogonal directions, directional properties occur. Maximum strength occurs in the primary fiber direction. Significant structural efficiency is reduced when the load and fiber orientation do not align with each other. On the other hand, unidirectional fibers exhibit fracture upon tensile loading in fiber direction. Resin-fiber interfacial debonding may occur when the same laminate is loaded in transverse direction where there are no load resisting fibers. If the laminate is rotated at an angle with the applied load, shear plane failure will occur. Fiber may not fracture, but the matrix is subjected to substantial shearing stress.

2.1.3.3 Creep

Resin dominates creep behavior; creep is more pronounced in thermoplastic resin matrix than in thermosetting resin matrix such as epoxy. Creep is dependent on magnitude of applied stress and temperature. Under continued application of stress,

the molecular bonds in the resin are strained. Upon slow responses of these bonds and state of equilibrium not reached for a long time, the material continues to deform for long periods after load application. As a result, after load removal, unrecoverable deformation occurs.

Temperature affects molecular activity. Additional creep can be observed when temperature is increased. Thermosetting resin is more resistant to molecular movement by inherent cross linking during curing. As such, creep is less pronounced in thermosetting resin matrix.

2.1.3.4 Fatigue resistance

Among the three fibers mentioned, carbon fibers are excellent in fatigue resistance as the stiff fibers are able to bridge the cracks that occur across the matrix and thus, reduce the stress intensity of the crack tip. Glass fibers, on the other hand, is more susceptible to fatigue; the fibers are comparatively less stiff and the stress transfer mechanism from matrix to fibers is less effective, causing it to be more prone to large stresses and strains. This promotes crack development under a fewer load cycles compared to carbon fibers.

2.2 FRP Wrapping Strategies

2.2.1 Flexural and shear strengthening

Strengthening of RC structures with FRP involves flexural and shear strengthening. Flexural strengthening of RC elements can be provided by using resin matrix such as epoxy to bond the materials to portions of elements with fibers parallel to principal stress direction. Shear strengthening, on the other hand, may be provided by having the FRP material bonded to the elements with fibers as parallel as practically possible to the principal tensile stresses (Bakis *et al.* 2002). Depending on the accessibility, strengthening of the elements can be provided by partial or full wrapping as shown in Figure 2-2.

Figure 2-2: Shear strengthening- (a) closed jacket applied to column; (b) open jacket applied to beam; (c) strips with end anchorage (Bakis *et al.* 2002)

To enhance the flexural and shear capacity of beam-column joints, FRP wrapping strategies in the form of externally bonded FRP sheets and strips for beams and columns can be studied and there after be employed on beam-column joints to ascertain their effectiveness. The following recommends and discusses the strengthening schemes for beams and columns.

2.2.1.1 Beams

According to Teng *et al.* 2002, both FRP strips and continuous sheets have been used to increase the shear resistance of beams by bonding FRP to the sides of a beam only, bonding FRP U jackets to both the sides and the tension face, and wrapping FRP around the whole cross-section of a beam. As FRPs are strong only in the directions of fibers, the fibers may be orientated in such directions as to control shear cracks best. Also, when a building is subjected to reversed cyclic loading during an earthquake, the fibers may also be arranged in two different directions to satisfy shear strengthening in both directions. Ductile behavior of flexural plastic hinges at beam or column ends can be achieved through added confinement in the form of FRP jackets, with fibers placed along the beam or column perimeter (*fib* 2003).

2.2.1.2 Columns

According to *fib* (2003), the shear capacity of the columns can be enhanced by providing externally bonded FRPs with fibers in the hoop direction. The flexural strength of RC columns can only be developed when debonding of the reinforcement in lap splices is prevented. Once vertical cracks develop in the concrete cover and progress with increased dilation and cover spalling, debonding occurs. The associated

rapid flexural strength degradation can be prevented or limited increased in lap splices or fibers along column perimeter.

In confining circular columns, the FRP effectively curtails the lateral expansion of the concrete shortly after the unconfined strength is reached. After which, it reverses the direction of volumetric response and the concrete responds through large and stable volume contraction. It is noted that in the case of steel confinement jackets, yielding is associated with unstable volumetric expansion. As a result, the stress-strain response of FRP-confined concrete is characterized by Figure 2-3: Axial stress-strain response of FRP- confined concrete versus plain concrete. It is a distinct bilinear response with a sharp softening at a stress level near the strength of unconfined concrete, f_{co} . After this softening point, the tangent stiffness stabilizes at a nearly constant value. At ultimate state (f_{cc} , ϵ_{cu}), it is characterized by tensile failure of the FRP wrap. At failure, it is noteworthy that the tensile stress in the FRP wrap is generally less than the uniaxial tensile strength of the FRP material. This is due to triaxial stresses and variations in the quality of installations that could lead to unequal load sharing among the fibers, misalignment and damaged fibers at sharp corners and local protrusions.

Figure 2-3: Axial stress-strain response of FRP- confined concrete versus plain concrete (Bakis *et al.* 2002)

Confinement of RC columns is less effective for columns with rectangular cross-section. This is because the confining stress is transmitted to concrete at the four corners of the rectangular column and increases with corner radius.

2.2.2 Failure modes

According to Bakis *et al.* (2002), the possible failure modes of FRP-strengthened RC elements are described follow:

- 1) Steel yielding followed by FRP fracture;
- 2) Steel yielding followed by concrete compressive crushing (while the FRP is intact)
- 3) Concrete compressive crushing
- 4) FRP peel-off at the termination or cutoff point, due to shear failure of the concrete
- 5) FRP peel-off initiating far from the ends, due to inclined shear cracks in the concrete
- 6) FRP peel-off at the termination point or at a flexural crack due to high tensile stresses in the adhesive
- 7) Debonding at the FRP-concrete interface in areas of concrete surface unevenness or due to faulty bonding

Mode (2) is the most desirable. Modes (4) and (5) will be activated when the element's shear strength is approached; as such, they may be prevented by providing shear strengthening. Mode (6) can be suppressed by limiting the tensile strain in the FRP to a value of about 0.008. Lastly, mode (7) may be avoided by proper quality control.

The mode of failure mainly consists of peeling-off or fracture of FRP. Whether peeling-off or fracture will occur first depends on the following factors: bond conditions, available anchorage length, type of attachment at the FRP termination point (full wrapping versus partial wrapping, with or without mechanical anchors), the axial rigidity of FRP and concrete strength.

2.3 Preceding Research on Repaired and Strengthened Reinforced Concrete Beam-Column Joints with FRP

2.3.1 Introduction

A collection of current information on repair and strengthening of non-seismically designed joints was conducted by Engindeniz *et al.* (2005). It reviews that externally bonded FRP composites have proven to eliminate difficulties in construction and increases in member sizes compared to other methods of strengthening such as concrete and steel jacketing. Shear strength of one-way exterior joints has improved by aligning fibers at $\pm 45^\circ$ in the joint region. Two major problems are: 1) The debonding of FRP composite sheets from the concrete surface before beam plastic hinge is formed and 2) Poor anchorage which results in early debonding of FRP composite sheets and not being able to achieve full development of fiber strength within the small area of joint. In addition to the aforesaid, Engindeniz *et al.* (2005) further commented that research in the area of repair and strengthening of non-seismically designed joints is far from complete, and a significant amount of work is necessary to arrive at reliable, cost-effective, and applicable strengthening methods.

2.3.2 Research by Prota, A., Nanni, A., Manfredi, G. and Cosenza, E. (2001)

Prota *et al.* (2001) tested eleven one-way interior joints with three different levels of axial loads in an attempt to shift failure first from the column to the joint, then from the joint to the beam. CFRP rods were used in combination with externally bonded sheets to upgrade the joints. Figures 2-4 and 2-5 illustrated the upgrading schemes used on the joints. The CFRP rods were placed in epoxy-filled grooves which were prepared near the surface of the concrete as shown in Figure 2-6. Results shown that the intended failure modes could not be controlled and ductile beam failure was not achieved. At low column axial load, Type 2 scheme (Figure 2-4 (a)) moved the failure from compression to tension side of the column while at high column axial load, a combined column-joint failure occurred. The effect of CFRP rods as flexural reinforcement along the column as shown in Type 3 scheme (Figure 2-4 (b)) resulted in a joint shear failure. With Type 4 scheme, the column-joint interface failed which was attributed to termination of FRP sheet reinforcement at that location to account for floor system. Table 2-2 summarizes the strength and ductility of tested specimens. The negative values indicated a loss in ductility. Type 5 scheme (Figure 2-5 (b)) with

U- wrapping of the beam and joint resulted in failure mode similar to that of Type 4.

Figure 2-4: Type 2 and 3 schemes by Prota et al. (2001)

Figure 2-5: Type 4 and 5 schemes by Prota et al. (2001)

Figure 2-6: Plan view of specimens strengthened with CFRP rods by Prota et al. (2001)

Table 2-2: Strength and ductility of tested specimens by (Prota *et al.* 2001)

Scheme	Increases in Strength	Maximum story drift	
		Low column axial load	High column axial load
Type 2	7 to 33%	-11%	25%
Type 3	39 and 62%	6%	-14%
Type 4	37 and 83%	73%	51%

2.3.3 Research by Ghobarah, A., and Said, A. (2002)

Ghobarah and Said (2002) tested four one-way exterior joints which were originally designed to fail in joint shear. The specimens were strengthened with or without unidirectional or bidirectional ($\pm 45^\circ$) GFRP sheets as illustrated in Figure 2-7. Specimens T1R and T2R were previously damaged in the joint region and repaired. Mechanical anchorage using steel plates and threaded rods were core-drilled through the joint. Diagonal unidirectional strips, facilitated by triangular steel bars fitted at four corners of the joint panel, were placed in Specimen T9.

Figure 2-7: GFRP strengthened specimens tested by Ghobarah and Said (2002)

Results showed that GFRP sheet anchored through the joint in Specimen T1R was effective until it failed in tension. For Specimen T2R, no debonding or joint shear cracking was observed; the failure was due to beam plastic hinge. Specimen T4 had early delamination due to lack of threaded-rod anchorage. The scheme employed on Specimen T9 could not prevent expansion of joint concrete which led to delamination and simultaneous failure of beam and joint. On a whole, the experimental study highlighted the importance of anchoring composite sheets in the development of full fiber strength in a small joint area.

2.3.4 Research by El-Amoury, T., and Ghobarah, A. (2002)

El-Amoury and Ghobarah (2002) modified the schemes for Specimens T1R and T2R proposed by Ghobarah and Said (2002) as shown in Figure 2-8. The modified schemes resulted in an approximate 100% increase in load-carrying capacity where Specimens T1R and T2R dissipated three and six times the energy dissipated by the reference specimen respectively. Specimen T1R failed by complete debonding of composites from beam and column surfaces and the pullout of beam bottom bars led by the fracture of the weld around the bolt heads. Specimen T2R failed by joint shear. The use of U-shaped steel plates had eliminated the debonding of GFRP and reduced strength degradation.

Figure 2-8: GFRP strengthened specimens tested by El-Amoury and Ghobarah (2002)

2.3.5 Research by Clyde, C., and Pantelides, C.P. (2002)

Clyde and Pantelides (2002) tested on a CFRP strengthened single, one-way exterior joint. The CFRP layout is illustrated in Figure 2-9. The original specimens failed in joint shear. Results shown that the CFRP wrapped joints failed at beam-column interface with minimal damage. In addition, the joint shear strength, maximum drift and energy dissipation capacity were 5, 78 and 200% respectively.

Figure 2-9: CFRP strengthened specimens tested by Clyde and Pantelides (2002)

2.3.6 Research by Antonopoulos, C. P., and Triantafillou, T.C. (2002)

Antonopoulos and Triantafillou (2002) performed reversed-cycle tests on eighteen strengthened 2/3 scale exterior joints. The specimens were strengthened with various configurations of pultruded carbon strips and with flexible carbon or glass fiber sheets. All the eighteen specimens were designed to fail in joint shear both before and after strengthening to evaluate the contribution of FRP to shear capacity of the joint. The variables investigated in this experimental test were area fraction and distribution of FRP, column axial load, internal joint reinforcement, initial damage, carbon versus glass fibers, sheets versus strips, and the effect of transverse stub beams.

Results showed that the failures of specimens were preceded by partial or complete debonding of composites either at the unanchored ends or corners of joint which led to substantial pinching of hysteresis loops. Strength increased from 65 to approximately 85% and energy increased from 50 to 70% when column axial load increased from 4 to 10% of its axial load capacity. For each loading cycle, the increase in stiffness varied and reached around 100%. On a whole, the experimental study highlighted better performance of flexible sheets over strips, the increase in column axial load enhance the shear capacity of FRP strengthened joints, glass fibers resulted in better energy dissipation than carbon fibers, increased in FRP effectiveness due to less internal joint reinforcement, negative effect of transverse stub beams on FRP effectiveness and the need for mechanical anchorage.

2.3.7 Research by Granata, P. J., and Parvin, A. (2001)

Granata and Parvin (2001) tested six scaled down exterior beam-column connections to examine the effect of various thickness and configuration of FRP fabrics applied to these connections. Kevlar, a type of aramid fiber, was used in strips to wrap around the beam and column. The length of the Kevlar strips were in multiples of the beam and column perimeters. Figure 2-10 shows the arrangement of fiber overlay on beam-column connection specimen. Table 2-3 gives an overview of the configuration of FRP reinforcement for the beam-column connections.

Results indicated that the thickness of the wrap affected the maximum rotation of the connection before fiber failure. Specimens B-C 1, B-C 2 and B-C 4 which used eight or eleven layers of column wrap had fiber failure when the joint rotation was approximately 0.08 radians. On the other hand, Specimens B-C 3, B-C 5 and B-C 6 using fourteen and sixteen layers of column wrap, had fiber failure at joint rotation of approximately 0.06 radians. The following were observed through this experimental study: 1) External FRP reinforcement could increase the moment capacity of beam-column connections by up to 60%; 2) column wraps should be at least 35% greater than the overlay thickness; and 3) the addition of FRP overlays to the bottom of the beam-column connections provided the mechanism for shear transfer between beam and column elements.

Figure 2-10: Arrangement of fiber overlay on beam –column connection specimen
Granata and Parvin (2001)

Table 2-3: Configuration of FRP reinforcement for the beam-column connection
specimens (Granata and Parvin 2001)

Specimen	Overlay thickness (mm)	Column wrap thickness (mm)	Beam wrap thickness (mm)	Shear reinforcement
B-C 1	200	200	200	No
B-C 2	200	280	200	No
B-C 3	300	410	300	No
B-C 4	200	280	200	Yes
B-C 5	200	350	200	Yes
B-C 6	300	410	300	Yes

2.3.8 Research by Mukherjee A. and Joshi M. (2004)

Mukherjee and Joshi (2004) investigated two different types of reinforced cement concrete (RCC) beam-column joints. The first set of joints had adequate steel reinforcements with proper detailing of reinforcements at critical sections; it was characterized by a long plastic zone (ductile). The second set of joints had deficient bond lengths at the junctions with columns; it failed in reinforcement pull out and exhibited sudden failure (non-ductile). Both ductile and non-ductile specimens were

wrapped with Type A and Type B strengthening systems as described below in Figures 2-11 and 2-12. Tables 2-4 and 2-5 below show the test matrices for both ductile and non-ductile specimens.

Type A: L-reinforcement with GFRP/CFRP sheets

Figure 2-11: Type A strengthening system (Mukherjee and Joshi 2004)

Type B: Reinforcement with plates

Figure 2-12: Type B strengthening system (Mukherjee and Joshi 2004)

Table 2-4: Test matrix for ductile specimen (Mukherjee and Joshi 2004)

S. no.	Specimen name	Details
1	D-1	–
2	G1L-D	Type A with single L of GFRP at top and bottom
3	G2L-D	Type A with two L of GFRP at top and bottom
4	C1L-D	Type A with single L of CFRP at top and bottom
5	C2L-D	Type A with two L of CFRP at top and bottom
6	CP1-D	Type B with CFRP plate at top and bottom
7	Rehab.	Type B with CFRP plate at top and bottom

Table 2-5: Test matrix for non-ductile specimen (Mukherjee and Joshi 2004)

S. no.	Specimen name	Details
1.	ND-1	–
2	G1L-ND	Type A with single L of GFRP at top and bottom
3	G2L-ND	Type A with two L of GFRP at top and bottom
4	C1L-ND	Type A with single L of CFRP at top and bottom
5	C2L-ND	Type A with two L of CFRP at top and bottom
6	CP1-D	Type B with CFRP plate at top and bottom

All the specimens were subjected to cyclic load and a constant axial load of 100 kN which is 50 % of ultimate load carrying capacity of the column. Results concluded that glass and carbon composite materials could be used efficiently for seismic retrofitting and rehabilitation of reinforced concrete joints. The joints exhibited enhanced strength regardless of reinforcement detailing and damage state. Considerable yield load was achieved. The yield load and initial stiffness depended on numbers of overlays provided in the joint area. Specimens strengthened using CFRP showed stiffer behavior than those strengthened with GFRP. Energy dissipation capacity could be increased with the use of small amount of composites.

2.4 Preceding Research on Repaired and Strengthened Reinforced Concrete Structural Walls with FRP

2.4.1 Introduction

Recent major earthquakes have caused extensive damage to older structural walls which have insufficient in-plane stiffness, flexural and shear strength and/or ductility. With buildings approaching the end of their service life, the deterioration of other structural elements further exacerbates the inadequacy in the lateral resistance of older structural walls. In order to preserve the buildings, repair and strengthening methods developed involve the application of shotcrete, the filling in openings with RC and masonry infills, epoxy injection and the addition of steel bracing elements. Research studies have shown that though the method of concrete jacketing can restore the gravity load carrying capacity of the structural walls, it is not effective in restoring the lateral stiffness of the structure. More recently, the use of FRP for strengthening existing buildings has increased. Application of bonding FRP sheets on RC walls using epoxy has proven to be the most cost effective alternative in repairing buildings in a short time (Ehsani and Saadatmanesh 1997). Although this has become a common practical retrofit measure, it has not attracted much attention by researchers.

2.4.2 Research by Neale et al. (1997) and Chiew et al. (1999)

Neale et al. (1997) and Chiew et al. (1999) carried out experimental studies involving FRP-strengthened RC structural walls. Wall-like columns with externally bonded GFRP and CFRP sheets as well as conventional steel plates under uniaxial compression only were tested. GFRP (a thicker section) offered higher stability against buckling compared to CFRP. FRP C-channels were fabricated and attached to ends of columns with two horizontal FRP layers confining the columns to prevent the channels from buckling. Results showed an increase in axial capacity of 33% when eight GFRP layers were used.

2.4.3 Research by Iso et al. (2000)

Iso et al. (2000) carried out experimental studies on RC columns with “wing” walls (small elongated members extending from the column) retrofitted with continuous fiber sheets as shear reinforcement. The columns of the specimens to be retrofitted were designed by abolished Japanese seismic code before 1981. A total of fourteen specimens were tested. The primary test variables were the width of the wing wall projected from the columns, the kind of the fiber in the sheets and its amount as shear reinforcement. All the specimens failed in shear mode and the ultimate strengths increase linearly in proportion to the amount of the fiber as shear reinforcement.

2.4.4 Research by Lombard et al. (2000)

Lombard et al. (2000) carried out four series of tests on half scale RC structural walls. Of these series, two tests involved strengthened walls; first strengthened wall consisted of one vertical layer of CFRP sheets on both sides of the wall while the second strengthened wall consisted of one horizontal and two vertical layers of CFRP sheets on both sides of the wall. It was found that the load-carrying capacity and ductility improved. The anchorage system between the sheets and the wall footing showed to play a major role in the upgrading system; inadequate anchorage system may not allow proper load transfer from the sheets to the adjacent elements, causing premature failure such as peeling or pulling off of fiber sheets of the strengthened walls.

2.4.5 Research by Kobayashi (2005)

Kobayashi (2005) tested two scaled specimens; one (W03N-R) was tested without strengthening and subsequently repaired with proposed method and the other (W02R) was strengthened with the proposed method before loading. The proposed method consisted of drilling small holes at every grid intersection on the wall as shown in Figure 2-13.

Figure 2-13: Drilling on wall panel (Kobayashi 2005)

FRP bands are installed on every diagonal two-holes as shown in Figure 2-14. If the FRP band is installed crossing a crack as shown in Figure 2-14, FRP can take full tension force because of the shape of a band, and the tension force transfers to the next FRP band through the un-cracked part of concrete.

Figure 2-14: Installation of FRP bands (Kobayashi 2005)

Figure 2-15: A FRP band over the crack (Kobayashi 2005)

The proposed method was proven to be effective both mechanically and economically. The sewing bands performed as the confining reinforcement to the wall panel as well as the tension braces. The shear capacity and the deformability increased in proportion to the number of strands in a sewing band. This strengthening method can be applicable to the infill-brick wall to keep the bricks inside the frame and to expect them to carry the horizontal force. Also this method can prevent the collapse of the

infill-brick wall (out of frame) even when seismic force comes in the orthogonal direction.

2.4.6 Research by Konstantinos et al. (2005)

Konstantinos *et al.* (2005) carried out an experimental study on the behavior of six 1:2.5-scale RC cantilever wall specimens having an aspect ratio of 1.5, tested to failure and subsequently repaired and strengthened using FRP sheets and strips, with a view to increasing flexural as well as shear strength and ductility. Figure 2-16 shows the repair and strengthening procedures applied to the specimens.

Figure 2-16: Repair and strengthening procedures applied to the specimens

In addition to different arrangements of steel and FRP reinforcement in the walls, a key parameter was the way carbon-FRP strips added for flexural strengthening were anchored; steel plates and steel angles were used to this effect. Steel plates were anchored using U-shaped GFRP strips or bonded metal anchors. Test results have shown that by using FRP reinforcement, the flexural and shear strength of the specimens can be increased. From the anchorage systems tested, metal plates combined with FRP strips appear to be quite efficient. The effectiveness of the bonded metal anchors used was generally less than that of the combination of plates and GFRP strips. In all cases, final failure of the FRP anchorage is brittle, but only

occurs after the peak strength is attained and typically follows the fracture of steel reinforcement in critical areas, hence the overall behavior of the strengthened walls is moderately ductile.

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3 Seismic Performance of Reinforced Concrete Interior Beam-Wide-Column and Beam-Wall Joints

3.1 Background

The first part of the chapter briefly presents the seismic behavior of six typically as-built non-seismically RC beam-column joints subjected to constant axial compression load and reversed cyclic loading simulating low to moderate earthquake forces. Based on studying the failure modes of these specimens, different FRP strengthening schemes were proposed. The second part of the chapter examines the strengthening schemes through the experimental studies: a) Six previously damaged specimens were repaired and strengthened with a proposed FRP wrapping scheme consisting of glass (GFRP) and carbon (CFRP) sheets; b) Three virgin specimens having identical detailing as the as-built beam-column joints were strengthened with GFRP and CFRP strips and sheets. Altogether, the nine specimens were tested under similar loading conditions.

The strengthening schemes proposed, aim to restore and enhance the original strength and to provide minimum drift capacity. The effectiveness of the FRP wrapping schemes in restoring the strength, drift capacity, stiffness and energy dissipation capacity was investigated by comparison between the responses of specimens before and after repair and/ or strengthening through hysteretic loops, secant stiffness versus drift ratio relationship, cumulative energy dissipation capacity versus drift ratio relationship and displacement decomposition percentage versus drift ratio relationship.

3.2 Experimental Program

3.2.1 Specimens

Nine full-scale non-seismically detailed RC interior beam-wide column joints and beam-wall joints design based on BS 8110 (1997) were constructed. Figure 3-1 illustrates the schematic dimensions and detailing of the specimens. All specimens had 1960mm high column and 3600mm long beams. The eccentric joints (E1 series) had the beam flushed with the column edge while the concentric joints (C1 and C2

series) had their respective center lines of beams and columns intersecting each other. The E1 and C1 series are beam-wide column joints which are defined herein as beam-column joints with column width larger than effective joint width. The C2 series are beam-wall joints. The E1 and C1 series have a column cross section dimension of 820x280 mm and a beam cross section dimension of 300x230 mm. The C2 series have a column cross section dimension of 1600x300 mm and a beam cross section dimension of 600x230mm. Specimens E1A, E1B, C1A, C1B, C2A and C2B were tested, repaired and re-tested under similar loading conditions. Specimens SE1B, SC1B and SC2B, on the hand, were strengthened first and then tested. Table 3-1 summarizes the geometric properties of the detailing. Table 3-2 shows the concrete compressive strengths of the specimens. Table 3-3 shows the steel reinforcement yield strength.

Figure 3-1: Schematic dimensions and detailing of the specimens

Table 3-1: Geometric properties of all specimens

Specimen	RE1A, RE1B, RC1A, RC1B	RC2A, RC2B		
Beam	Size (mm)	230 x 300	230 x 600	
	Top bar	2 ϕ 10 (ρ'_t =0.23%) mid span		3 ϕ 25 (ρ'_t =1.07%)
		2 ϕ 13 (ρ'_t =0.38%)		
		Lap spliced within joint		
	Bottom bar	2 ϕ 13 (ρ'_t =0.38%)		3 ϕ 20 (ρ'_t =0.68%)
		Mid span and lap spliced within joint		
Column	Stirrups	R10@230		R10@300
	Size	280 x 820		300 x 1600
	Main bar	4 ϕ 22 + 4 ϕ 25 (ρ_c =1.52%)		24 ϕ 28
				(ρ_c =3.08%)
Joints	Hoops	2 R10@125		9 R10@300
	Hoops	None		None

Table 3-2: Concrete compressive strengths of all specimens at testing

As-built/ Repaired Specimens	f'_c (MPa)	Strengthened Specimens	f'_c (MPa)
E1A, RE1A	18.9	SE1B	19.5
E1B, RE1B	18.4	SC1B	19.9
C1A, RC1A	19.2	SC1B	20.4
C1B, RC1B	19.0		
C2A, RC2A	20.0		
C2B, RC2B	20.0		

Table 3-3: Steel reinforcement yield strength of all specimens

Reinforcement Type	Bar Size	f_y (MPa)
Beam longitudinal (RE1A, RE1B, RC1A, RC1B, SE1B, SC1B)	10	510
	13	508
Beam longitudinal (RC2A, RC2B, SC2B)	20	513
	25	517
Column longitudinal (RE1A, RE1B, RC1A, RC1B, SE1B, SC1B)	25	517
Column longitudinal (RC2A, RC2B, SC2B)	28	504
Stirrups	10	354

3.2.2 Test setup

Figure 3-2 shows the schematic of the experimental set-up. Each specimen was subjected to constant axial compression load and reversed cyclic loading that

simulated low to moderate earthquake forces. Specimens in Group A were subjected to constant axial compression load of $0.10f'_c A_g$ while those in Group B and C were subjected to that of $0.35f'_c A_g$. The constant axial compression load was applied using small hydraulic jacks placed between column top end and bottom suffix of the steel transfer beam. Threaded rods were fixed around the test unit to balance the applied column axial load. A reversible horizontal load was applied in a quasi-static fashion at the top of the column through a double-acting 1000kN capacity long-stroke dynamic actuator mounted on the reaction wall. It was pinned at the end to allow rotation during the test. The actuator was manually operated to have a better control on the load increment. The column was pinned to a strong floor and the beam ends were connected to this strong floor by steel links which allow rotation and free horizontal movement of the beams. No vertical movement was allowed.

Figure 3-2: Experimental setup

3.2.3 Test instrumentation and procedures

Before the start of each test, the column axial load was slowly applied to the column and was balanced in steps until the designated level was achieved. During each test, the axial compression load was maintained by manually adjusting the flat jacks after each load step. The lateral load was applied cyclically through the dynamic actuator in

a quasi-static fashion at the top end of the column. The typical loading procedure consisting of displacement controlled steps is illustrated in Figure 3-3.

Figure 3-3: Typical loading procedures

The test results were explained quantitatively by using instrumentations such as dynamic actuator, strain gauge and displacement transducers which were installed in the test set-up. The horizontal displacement at the top column face was measured using a displacement transducer with 300 mm travel. The displacement transducers were installed to observe the behavior of joint core area, beam and column. Local strains in the reinforcing bars were measured using electric resistance wire strain gauges (TML FLA-5-11-5LT) which were installed on the bars (see Figure 3-4) before casting of specimens. Figures 3-5 and 3-6 show the typical locations of displacement transducers on the specimens.

E1 and C1 series

C2 series

Note: All dimensions are in mm

Figure 3-4: Location of strain gauges on the reinforcement

Figure 3-5: Typical locations of displacement transducers (front view)

Figure 3-6: Typical locations of displacement transducers (back view)

3.3 Failure modes of as-built specimens

3.3.1 Specimen E1A

Eccentric Specimen E1A was loaded under column axial load of $0.10 f'_c A_g$. Cracks were first observed at beam top when drift ratio of 0.4% was exceeded. It was noticeable that cracks at beams propagated rapidly at drift ratio of 0.67% where the cracks in joint core area were found. At drift ratio of 1.0%, cracks were found to propagate rapidly at beam top as well as joint core. Even though no crack was found in column, more cracks formed further from the joint core area while new cracks were found at beam bottom after drift ratio of 1.33%. Due to its drastic drop in lateral resisting capacity, the specimen failed at drift ratio of 2.0%. Slight spalling of concrete was observed as shown in Figure 3-7. The column was also found to be intact with no cracks when the test was terminated. It is noteworthy that the joint core cracks were only observed at the flushed surface of the beam-column joint while the protruded joint on the opposite face was found to be intact.

Figure 3-7: Crack patterns of Specimen E1A

3.3.2 Specimen E1B

Specimen E1B was an eccentric interior beam-wide-column joint subjected to cyclic lateral loading and column axial load of $0.35 f'_c A_g$. Initial cracks formed at beam top during drift ratio of 0.4% and cracks propagated rapidly when drift ratio of 1.0% was attained. It is noteworthy that crack in joint core area was first found at drift ratio of 1.0%. Joint core cracks were only observed at the flushed surface of the beam-column joint while no cracks were found on the protruded joint at the opposite face.

More cracks formed further away from the joint core area, while limited new cracks were found at the beam bottom after drift ratio of 2.0%. Specimen E1B deteriorated after drift ratio of 3.0% with crack patterns shown in Figure 3-8. As the drift ratio increased, more cracks were found to propagate rapidly at beam top as well as joint core. Generally, no cracks were found in column and no spalling of concrete was observed throughout the test. It was declared failure and test was halted when a drift ratio of 4.0% was attained.

Figure 3-8: Crack patterns of Specimen E1B

3.3.3 Specimen C1A

Specimen C1A was a concentric interior beam-wide-column joint which was subjected to cyclic lateral loading and column axial load of $0.10 f'_c A_g$. When a drift ratio of 0.67% was exceeded, cracks were found at beam top of Specimen C1A. It was noticeable that cracks at beams propagated rapidly when drift ratio was increased to 1.0% where several surface cracks in joint core area were found. When drift ratio of 1.33% was attained, cracks were found to propagate rapidly at beam top. No new cracks were formed in joint core area while new cracks were found at beam bottom after a drift ratio of 1.33% was reached. Cracks at beam bottom propagated rapidly at

a drift ratio of 3.0% and joined with cracks formed at beam top. The specimen failed at drift ratio of 3.0% when a drop in its strength was noticed. No crack was found in column when test was completed and no new formation of cracks at joint core area was observed. Figure 3-9 shows the crack patterns of Specimen C1A.

Figure 3-9: Crack patterns of Specimen C1A

3.3.4 Specimen C1B

Specimen C1B was a concentric interior beam-wide-column joint which was subjected to cyclic lateral loading and column axial load of $0.35 f'_c A_g$. When a drift ratio of 1.33% was attained, cracks were observed to propagate rapidly at both beam top and bottom. Most of the crack damage was concentrated in the beams near the column. After drift ratio of 2.0%, the specimen sustained additional horizontal load with good energy dissipation. The largest flexural cracks occurred at the interfaces of the beam ends. By the end of the test, these cracks were excessive, and the beam flexural bars were observed to have slipped through the joint due to loss of bond. This attributed to the gradual strength deterioration and low attainment of structural stiffness of the specimen during the drift ratios of 3.0% and 4.0%. The specimen

began to deteriorate with a significant decrease in lateral resisting capacity after a drift ratio of 3.0% was attained. During then, cracks at beam propagated rapidly.

Figure 3-10 shows its crack patterns at drift ratio of 3%.

Figure 3-10: Crack patterns of Specimen C1B

3.3.5 Specimen C2A

Specimen C2A was an interior beam-wall joint subjected to cyclic lateral loading and column axial load of $0.10 f'_c A_g$. When drift ratio of 0.67% was exceeded, diagonal flexural cracks were found at beam bottom of C2A. It was noticeable that flexural cracks at beams propagated rapidly when a drift ratio was increased to 1.0% level. When a drift ratio of 1.33% was attained, several diagonal cracks were found in joint core area and flexural cracks at beams were found to propagate rapidly. Limited new cracks were observed at beam bottom while more cracks formed in joint core area after drift ratio of 1.33% was reached. More new cracks at beam top were found when the drift ratio was increased to 2.0% and 3.0% respectively. As illustrated in Fig. 4.31b, the cracks on the side face of 1600 mm long column propagated aggressively throughout the test. These cracks were initiated at a drift ratio of 0.67% and they developed rapidly in drift ratios of 1.33% and 2.0%. These cracks were caused by the

fixed end moment of beams which rotated about the long column. These cracks, which propagated and extended to the end of column, formed the shear cracks at column front face. Figure 3-11 shows the crack patterns of Specimen C2A.

Figure 3-11: Crack patterns of Specimen C2A

3.3.6 Specimen C2B

Specimen C2B was an interior beam-wall joint subjected to cyclic loading and $0.35 f'_c A_g$. When drift ratio of 0.67% was exceeded, diagonal flexural cracks were found at beam bottom. Limited new cracks were observed at beam bottom whereas more cracks were formed in joint core area after drift ratio of 1.33% was attained. During then, flexural cracks on the beams were also found to propagate rapidly. More new cracks at beam top were found when the drift ratio was increased to 2.0% and 3.0% respectively. Severe punching cracking was observed on the side face of the column. These cracks, which were formed mainly at the lower portion of column, were caused by the fixed end moment of beams which rotated about the long column. The shear cracks at the column front face were believed to be the extension of these cracks formed at column. At a drift ratio of 4.0%, crushing of concrete at the fixed end of the beams was observed. Figure 3-12 shows its crack patterns at drift ratio of 3%.

Figure 3-12: Crack patterns of Specimen C2B

3.4 Repair and Strengthening Techniques

Nine specimens were tested in this chapter: 1) Six previously damaged as-built specimens namely E1A, E1B, C1A, C1B, C2A and C2B after repaired and strengthened were then named RE1A, RE1B, RC1A, RC1B, RC2A and RC2B and 2) Three other virgin specimens were strengthened and named SE1B, SC1B and SC2B.

3.4.1 Repair process

For all six previously damaged as-built specimens namely E1A, E1B, C1A, C1B, C2A and C2B, all spalled/loose concrete was removed with chisel and hammer until sound concrete was reached. These affected areas were reinstated using patch repair which was done in layers not exceeding 20mm using bonding agent and polymer modified cementitious mortar. All cracks greater than 0.3mm wide were sealed by pressure injecting low viscosity epoxy resin in a sequential manner along the crack length: Injection nipples were installed by applying a bead of Sto epoxy crack sealer on the back of the nipple and spaced at approximately 300mm center to center along the cracks. After the sealer was cured, injection of the epoxy began at the lowest nipple to the highest nipple. This ensured that the epoxy resin filled the crack completely. Sto epoxy crack sealer was applied to seal the remaining parts of the cracks to prevent epoxy leakage.

To ensure good adherence between the FRP sheets and concrete, the surface was first cleared of dust and any deleterious substances which acted as bond barriers. Sharp edges and protrusions were removed by mechanical grinding. Corners of the specimens strengthened were rounded to a minimum radius of about 20mm to avoid FRP cracks due to local failure and stress concentration (Columb *et al.* 2007). The contact surfaces to be strengthened were kept dry. The wet lay-up FRP application was employed: FRP sheets were impregnated with specially modified epoxy and applied to the member surfaces. Upon completion of FRP-strengthening, the system was left to cure at ambient temperatures. Finishes were applied on the composite surface after 24 hours of curing.

3.4.2 Strengthening Techniques

3.4.2.1 Damaged as-built specimens

The strengthening schemes proposed for Specimens RE1A, RE1B, RC1A, RC1B, RC2A and RC2B were based on the failure modes of the damaged specimens. The Tyfo® systems were applied. Two unconventional FRP strengthening schemes were introduced: Scheme 1 uses the Tyfo® SEH-51A Composite system. It comprises of Tyfo® SEH-51A reinforcing fabric and Tyfo® S Epoxy. Tyfo® SEH-51A is a custom weave, uni-directional glass fabric; the glass material is orientated in the 0 degree direction with additional yellow glass cross fibers at 90 degrees. The Tyfo® S Epoxy is a two-component epoxy matrix material. Scheme 2 uses both Tyfo® SEH-51A Composite and Tyfo® SCH-41 Composite systems. The latter system comprises of Tyfo® SCH-41 reinforcing fabric and Tyfo® S Epoxy. Tyfo® SCH-41 is a custom, uni-directional carbon fabric with glass cross fiber for added strength and fabric stability during installation. The carbon material is orientated in the 0 degree direction. Table 3-4 shows the properties of Tyfo® systems.

Table 3-4 Tyfo® Fibrwrap Composite system

Parameter	Properties ^a	
	GFRP with epoxy, Tyfo® SEH-51A Composite	CFRP with epoxy, Tyfo® SCH-41 Composite
Type of FRP	Unidirectional GFRP sheet	Unidirectional CFRP sheet
Ultimate tensile strength in primary fiber direction	575 MPa	986 MPa
Elongation at break	2.2%	1.0%
Tensile Modulus	26.1 × 10 ³ MPa	95.8 × 10 ³ MPa
Laminate thickness	1.3 mm	1.0mm

^a Properties value given are based on test value

Scheme 2 is similar to the first scheme; only the continuous GFRP sheet in the direction parallel to the beam axis across the joint core is replaced by CFRP sheet. Specimens RE1A, RC1B and RC2A were strengthened with Scheme 1 while Specimens RE1B, RC1A and RC2B were strengthened with Scheme 2. The strengthening steps for both schemes are described as follows:

- One layer of GFRP L-wrap is applied at each of the four corners of the joint; the GFRP sheet is bent at 90° and thereafter extended 1000 mm along the beam length and 500 mm along the column.
- To confine the column, it was wrapped round with one layer of GFRP sheet at 500mm along the column from beam-column interface. This applies to both top and bottom columns.
- One layer of continuous GFRP sheet (for Specimens RE1A, RC1B and RC2A) / CFRP sheet (for Specimens RE1B, RC1A and RC2B) in the direction parallel to the beam axis was applied at 500mm from the beam-column interface across the joint core at 1280mm on both sides of the joint.
- Finally, one layer of GFRP U-wrap was applied on both beams. It terminated at 75mm from the top of the beams to account for the presence of the floor system.

3.4.2.2 Virgin as-built specimens

The strengthening schemes proposed for virgin Specimens SE1B, SC1B and SC2B were based on the failure modes of the damaged Specimens E1B, C1B and C2B. Figures 3-13, 3-14 and 3-15 illustrate the proposed FRP strengthening schemes for Specimens SE1B, SC1B and SC2B respectively.

Figure 3-13: FRP wrapping scheme for Specimen SE1B

Figure 3-14: FRP wrapping scheme for Specimen SC1B

Figure 3-15: FRP wrapping scheme for Specimen SC2B

The Tyfo® systems were applied. To upgrade the joint in all specimens, one layer of GFRP L-wrap at each of the four corners of the joint was applied; at each corner, the GFRP strip was bent at and thereafter extended 500 mm along the beam length and 400mm, 200mm and 500 mm along the column height for Specimens SE1B, SC1B and SC2B respectively. The severe punching cracks on the side face of the column of Specimen C2B renders relatively longer extension of GFRP strip along the column height of Specimen SC2B compared to Specimen SC1B which do not have column cracks in Specimen C1B. Though Specimen E1B did not have column cracks, the 400mm GFRP strip extension along column height in Specimen SE1B served mainly to upgrade its joint. The fibers were along the axes of the members.

To better bond the GFRP L-wrap to the column and enhance the confinement of the column. A GFRP sheet was wrapped round the column of Specimens SE1B and SC1B. CFRP strips (50mm width each) were wrapped round the column at 50mm intervals for Specimen SC2B. CFRP, having relatively higher ultimate tensile strength than GFRP, is used in strips to prevent excessive strengthening and to assess if such economical use of CFRP can improve the performance of the beam-column joint considerably. The direction of the confining fibers was perpendicular to the axis of the column.

For Specimen SE1B, two layers of CFRP strips (100mm width each) were applied at the flushed surface; first layer was aligned at $+45^\circ$ while the second layer was aligned at -45° . This was similarly applied at the protruded side, at 500mm along the beam face from the beam-column interface. Shear strengthening of the beam and joint is enhanced, eliminating or delaying the possibility of shear failure at the joint, especially at the flushed area. This will create the opportunity for a ductile plastic flexural hinging in the beam to occur (Ghobarah and Said 2002). As most of the crack damage was concentrated at the beam near the column of Specimen C1B, one layer of CRFP U-wrap extending 500mm from beam-column interface was applied to cover the beam bottom of Specimen SC1B to increase shear strength (Report 2002-005-C-01). This also prevents lap splices in the beam to slip. To serve the same purpose, CFRP strips (100mm width each) formed cross bracings at both sides of the beam and column faces were applied on Specimen SC2B. This is relatively more economical than U-wrapping when strengthening a beam-wide-column joint is concerned.

To ensure better bond, one layer of continuous GFRP in the direction parallel to the beam axis was applied starting from 1000mm distance away from the beam-column interface on the following: across the two layers of CFRP strips at the flushed surface of Specimen SE1B, across the CFRP U-wrapping of beam bottom of Specimen SC1B and across CFRP cross-bracings on beam and column faces of Specimen SC2B, This increases the flexural strength. The fibers were along the axis of the beam.

Finally, one layer of GRFP U-wrap with direction of fibers perpendicular to beam axis was applied to cover the bottom of the beams to increase shear strength (Report 2002-005-C-01) and confinement of GFRP and CFRP strips and sheets of Specimens SE1B and SC1B. In all specimens, the laminates terminated at 75mm from the top of the beams to account for the presence of the floor system.

3.4.2.3 Anchorage

Fiber anchors were placed at various locations along the development length of the sheets to allow the fiber to develop its full capacity and prevent premature delamination of FRP sheets. First, holes are drilled on beam top and beam faces before the application of FRP sheets. The drilled holes were cleaned and kept free of dust. The embedded length of the anchor from the surface of the wall had a minimum

length of 50mm. After the application of FRP sheets, the fiber anchors are inserted into the holes. The anchor consisted of a bundle of main fibers properly formed to size, and fully impregnated with epoxy before insertion to the drilled holes in the wall. The remaining fibers left outside the hole are spreaded out in circular shape to act as the base of anchorage. These anchorages were applied at intervals of 300mm. In particular for Specimen SC2B, anchors were placed at the centre and corners of each cross bracing. This is to prevent the delamination of top and bottom ends of the strips.

3.5 Results and Discussion

3.5.1 Failure modes and response under cyclic loading

The cracks of the specimens could only be visibly observed on areas not strengthened by FRP. The failure modes were evaluated by the type of cracks, breaking of epoxy resin, debonding and delamination of FRP sheets. All repaired and strengthened specimens failed commonly with: 1) flexure and shear cracks on the beams due to concrete deterioration and bar slippage when the beams underwent tension and compression, 2) debonding on column faces near beam-column joint interfaces; debonding initiates at flexure-shear cracks and propagate towards end of the beam and 3) delamination of FRP sheets near the beam-column joint interface due to high stress concentrations at beam ends.

3.5.1.1 Repaired damaged as-built specimens

3.5.1.1.1 Specimen RE1A

For repaired Specimen RE1A, flexural cracks were first observed at beam bottom at drift ratio of 0.13% while that of as-built Specimen E1A was observed at beam top at drift ratio of 0.4%. At drift ratio of 0.4%, the cracks became more inclined in Specimen RE1A, indicating shear cracks. Also, small regions of light shades on the GFRP sheet were observed around beam-column interface. Cracks at joint core of Specimen E1A were observed at drift ratio of 0.67%. At drift ratio of 1%, cracking was heard in Specimen RE1A and cracks propagated rapidly from beam top as well as joint core for Specimen E1A. The joint core cracks were only observed on the flushed surface of Specimen E1A. First crack at beam top for Specimen RE1A was observed at drift ratio of 1.33%. At drift ratio of 2%, Specimen E1A failed with a large

decrease in lateral load resistance. Specimen RE1A, on the other hand, had the light shades observed previously enlarged, showing greater debonding (see Figure 3-16a). At drift ratio of 3%, cracking was heard. Breaking of epoxy resin was observed at the corners of the joint (see Figure 3-16b). At the end of the test, GFRP sheet delaminated at joint core at the flushed side with cracks of epoxy resin at the joint corners as shown in Figure 3-16c. GFRP sheet delaminated within 200mm from beam end at both beams top.

Figure 3-16: Specimen RE1A at final stage

3.5.1.1.2 Specimen RE1B

For Specimen RE1B, the first flexural crack was observed at beam bottom at drift ratio of 0.1% while that for Specimen E1B, the cracks were observed later, at drift ratio of 0.4%. As the drift ratio increased, the flexural cracks propagated. At drift ratio of 0.67%, the first cracking sound was heard and flexure-shear cracks were observed on Specimen RE1B. The first crack in joint core area was found at drift ratio of 1% for Specimen E1B. Similar to Specimen E1A, the joint core cracks were only observed at the flushed surface of the beam-column joint while no cracks were found on the protruded joint. First few flexural cracks at beam top occurred at drift ratio of 1.33% for Specimen RE1B. Slight debonding of GFRP sheets shown by small regions of lighter shades near beam-column joint interface was also observed. As the drift ratio increased, these regions enlarged further, accompanied by continuous audible breaking of epoxy resin. Delamination of CFRP sheet near the top beam-column joint interface caused by shear cracks (see Figure 3-17a) occurred at drift ratio of 2% and was severed in subsequent drift ratios. Towards the last few cycles, the CFRP and GFRP sheet failed with a wide crack at right beam bottom and right column face near the beam-column joint interface. The left beam failed with flexural crack near the

termination of the U-wrapping of beam (see Figure 3-17b). Specimen E1B failed at drift ratio of 3%.

Figure 3-17: Specimen RE1B at final stage

3.5.1.1.3 Specimen RC1A

Cracks were first observed at beam top of Specimen C1A at drift ratio of 0.67%. However, for Specimen RC1A, flexural cracks occurred earlier at beam bottom at drift ratio of 0.4%. As drift ratio increased, cracks continued to propagate rapidly and cracking sound was first heard at drift ratio of 0.67% for Specimen RC1A. The first flexural crack was observed on right beam top at drift ratio 1% and lighter shades of GFRP sheet, indicating debonding, were observed near the beam-column joint interfaces (see Figure 3-18). Subsequently, the inclination of the cracks was more pronounced, indicating shear cracks. At this drift ratio, Specimen C1A had several cracks at joint core area. Towards the last few cycles, continuous audible sounds from breaking of epoxy resin were heard for Specimen RC1A. Epoxy resin failure was initiated near the joint region as significant crack marks were observed. At drift ratio of 3.03%, the delamination of CFRP sheets were observed near the top beam-column joint interfaces. The repair at joint core was intact and there was no anchorage failure. Specimen C1A failed at drift ratio of 3%.

Figure 3-18: Specimen RC1A at final stage

3.5.1.1.4 Specimen RC1B

Similar to Specimen C1A, initial cracks were first observed at beam top of Specimen C1B at drift ratio of 0.67%. For Specimen RC1B, the first flexural crack was observed earlier on beam bottom at drift ratio of 0.1%. Cracks started to propagate and became more inclined at drift ratio of 0.67%. Also, first shear crack was observed on right beam top. At drift ratio of 1%, both flexural and shear cracks started to propagate rapidly, accompanied by cracking sound. Small regions of lighter shades of GFRP sheet were observed near the beam-column joint interfaces. At this stage, cracks propagated rapidly and new cracks were observed at beam bottom for Specimen C1B. As drift ratio increased, the propagation of cracks continued and widening of previous cracks was observed for Specimen RC1B; at drift ratio of 2%, slight concrete cover debonding due to shear crack was observed on left beam top. Towards the last few cycles, similar phenomenon was observed at left beam-column joint interface and more audible cracking could be heard. Finally, the beams failed in shear with GFRP delamination (see Figure 3-19a) and GFRP debonding at beam-column joint interfaces, indicated by lighter shades (see Figure 3-19b), due to high interfacial stresses. No cracks were observed in joint core area for Specimen C1B and it failed at drift ratio of 3%.

Figure 3-19: Specimen RC1B at final stage

3.5.1.1.5 Specimen RC2A

Initial flexural cracks occurred at drift ratio of 0.67% at beam bottom for both Specimens C2A and RC2A; in particular, the cracks were accompanied by cracking sound for the repaired Specimen RC2A. As drift ratio increased, the cracks

propagated rapidly and were more inclined, suggesting flexural to shear mode. Several diagonal cracks were found in joint core area when drift ratio of 1.33% was reached for Specimen C2A. At drift ratio of 2%, the first few shear cracks occurred at right beam top of Specimen RC2A. Lighter shades of GFRP sheet near beam-column joint interfaces enlarged in area as drift ratio increased. Continuous audible sounds from breaking of epoxy resin were heard. Cracks of epoxy were observed (see Figure 3-20a) at beam-column joint interfaces. At drift ratio of 3.03%, cracks propagated aggressively on the side face of the column of Specimen C2A. At this stage for Specimen RC2A, bulging of GFRP L-wrap was observed near the beam-column joint interface on the column. The bulging allowed cracks to be formed and widen under fiber due to limited confinement of the rectangular column section. Towards the last few cycles, the failure of the joint was dominated by shear cracks on the beam faces, flexural cracks occurred on the column faces where the floor space was accounted for as shown in Figure 3-20b. GFRP debonding was observed near the beam-column joint interfaces.

Figure 3-20: Specimen RC2A at final stage

3.5.1.1.6 Specimen RC2B

Initial diagonal flexural cracks were observed at top and bottom beam of Specimen C2B at drift ratio of 0.67%. For Specimen RC2B, the first flexural crack occurred earlier on beam bottom at drift ratio of 0.1%. As drift ratio increases to 0.67%, cracks propagated rapidly and had more pronounced inclination, suggesting flexural to shear cracks. Cracks also occurred at top right beam corner near the beam-column joint interface. Small light shades of GFRP sheet which indicated FRP debonding were

observed at the corners of beam-column joint interface. At drift ratio of 1%, cracking sound was heard. At drift ratio of 1.33%, first shear crack occurred at right beam top and delamination of CFRP sheet was observed near right beam-column joint interface. For Specimen C2B, several shear cracks were noticeable in joint core area. As drift ratio increased, shear cracks continue to propagate rapidly and the GFRP debonding, indicated by lighter shades, were observed near the corners of beam-column joint interface for Specimen RC2B. Towards the last few cycles, there were concrete cover debonding and delamination of CFRP sheet near beam-column joint interfaces (see Figure 3-21a) and flexural cracks occurred on the column faces where the floor space was accounted for as shown in Figure 3-21b. For Specimen C2B, the cracks formed at beam-column interfaces propagated and formed mainly at the lower portion of the column. The shear cracks at the column front face were believed to be the extension of these cracks formed at the column.

Figure 3-21: Specimen RC2B at final stage

3.5.1.2 Strengthened virgin specimens

3.5.1.2.1 Specimen SE1B

Specimen SE1B was an eccentric interior beam-wide-column joint subjected to cyclic lateral loading and column axial load of $0.35 f'_c A_g$. There were no cracks on the column for Specimen SE1B. During the first few cycles of loading, hairline cracking distributed along the beam; the first crack initiated at drift ratio of 0.20%. Most cracks developed were approximately constant inclination, due to diagonal tension with few cracks with variable inclination. More flexural and shear cracks on the beams propagated as drift ratio increased. As the imposed displacement increased, widening

of cracks was accompanied by audible sounds from the breaking resin, indicating initiation of the epoxy resin failure near the joint region where crack marks on the epoxy resin were observed. Significant widening of cracks was observed at beam top at drift ratio of 1.33%. In particular, such cracks were apparent near the joint core corners for Specimen SE1B. It was clear evidence that most inelastic behavior occurred here. Also, it indicated advanced deformation of the yielded steel. Towards the last few cycles of loading, rapid continuous audible sounds from the breaking of resin and debonding of FRP sheets were heard. Final failure occurred due to fracture of epoxy resin and debonding of FRP sheets near the joint region (see Figure 22).

Figure 3-22: Specimen SE1B at final stage

3.5.1.3 Specimen SC1B

Strengthened Specimen SC1B was a concentric interior beam-wide-column joint subjected to cyclic loading and column axial load of $0.35 f'_c A_g$. During the first few cycles of loading, hairline cracking distributed along the beam; the first crack initiated at drift ratio of 0.67%. More flexural and shear cracks on the beams propagated as drift ratio increased. As the imposed displacement increased, widening of cracks was accompanied by audible sounds from the breaking resin, indicating initiation of the epoxy resin failure near the joint region where crack marks on the epoxy resin were observed. Significant widening of cracks was observed at beam top at drift ratio of 3.0%. Towards the last few cycles of loading, rapid continuous audible sounds from the breaking of resin and debonding of FRP sheets were heard. Debonding of FRP

sheets was justified with its lighter shade and hollow echo upon knocking. The debonding observed on the column face near the beam-column interface could be due to the weakening of FRP sheets due to the discontinuous FRP sheets terminated at the beam-column interface to account for the presence of floor system (see Figure 23). Slight delamination was observed at beam top near beam-column interface for Specimen SC1C and edge of FRP sheet at the right beam face nearest to the beam pinned end.

Figure 3-23: Specimen SC1B at final stage

3.5.1.4 Specimen SC2B

Strengthened Specimen SC2B was a concentric interior beam-wall joint subjected to cyclic loading and column axial load of $0.35 f'_c A_g$. Delamination of FRP strips on beam near the beam-wall interface was observed. Anchorage failure was occurred near the beam-wall interface at beam top of Specimen SC2B. During the first few cycles of loading, hairline cracking distributed along the beam, developed in all specimens; the first crack initiated at drift ratio of 0.10%. Most cracks developed were approximately constant inclination, due to diagonal tension with few cracks with variable inclination. More flexural and shear cracks on the beams propagated as drift ratio increased. As the imposed displacement increased, widening of cracks was accompanied by audible sounds from the breaking resin, indicating initiation of the epoxy resin failure near the joint region where crack marks on the epoxy resin were

observed. Significant widening of cracks was observed at beam top at drift ratio of 2.0%.

Towards the last few cycles of loading, rapid continuous audible sounds from the breaking of resin and debonding of FRP sheets were heard. Final failure occurred due to fracture of epoxy resin and debonding of FRP sheets near the joint region. Debonding of FRP sheets was justified with its lighter shade and hollow echo upon knocking. The debonding observed on the column face near the beam-column interface could be due to the weakening of FRP sheets due to the discontinuous FRP sheets terminated at the beam-column interface to account for the presence of floor system. Severe FRP delamination was observed at top and bottom beam near beam-column interface for Specimen SC2B (see Figure 24), indicating that FRP U-wrapping was an important technique to strengthen beam which was absent in Specimen SC2B. The initiation of delamination occurred when the fiber lost its compressive load-carrying capacity when the concrete was in compression. Few crack lines were observed at the intervals of CFRP strips of its column. There was a slight delamination of the first confining CFRP strip on the column near the beam bottom. No rupture of FRP sheets and hairline cracks on the epoxy resin at the joint corners of all specimens were indicative of low level of shear in this area.

Figure 3-24: Specimen SC2B at final stage

3.5.2 Lateral load versus displacement relationship

3.5.2.1 Repaired specimens

Towards the last few cycles, all repaired specimens were able to restore strength up to or more than the theoretical peak strength (P_i). Specimen RE1A was able to restore up to theoretical strength at drift ratio 4.0%. Specimen RE1B, RC1A and RC1B, on the other hand, restored its theoretical strength earlier at drift ratio of 2.0%. Specimen RC1B in particular was able to restore strength much more than that of the theoretical strength. Specimens RC2A and RC2B restored their theoretical strength at drift ratio of 3.0% and achieved greater strength at drift ratio of 4.0%.

Figure 3-25: Comparison of lateral force versus horizontal displacement for Specimens RE1A and E1A. Specimen E1A failed at drift ratio of 2 %. As shown in Figure 25, Specimen E1A reached maximum load of 19.8 kN at drift ratio of 1.33 % in the positive cycle while Specimen RE1A reached that of 20.3 kN at drift ratio of 4 % in the negative cycle. During the cycles until drift ratio of 2 %, the maximum strength of Specimen RE1A was restored to about 68 % and 95 % for positive and negative cycle respectively. After drift ratio of 3 %, towards the last few cycles, the hysteresis loops show considerable pinching. This is primarily due to slippage of beam longitudinal bars, shown by severe inclined cracks near the corners of the joint. As debonding of GFRP sheets propagated, the slope of the hysteresis loops started to increase at a slower rate.

Figure 3-25: Comparison of lateral force versus horizontal displacement for Specimens RE1A and E1A

Figure 3-26 shows the comparison of lateral force versus horizontal displacement for Specimens RE1B and E1B. The maximum load obtained by Specimens E1B and RE1B were 15.8 kN at drift ratio of 2 % (positive cycle) and 25 kN at drift ratio of 3 % (negative cycle) respectively. The strength of Specimen RE1B increased considerably and started to decline after drift ratio of 3 %. Specimen RE1B was able to restore the strength at every stage of Specimen E1B. Compared to Specimen E1B, there was no pinching for Specimen RE1B towards the last few cycles.

Figure 3-26: Comparison of lateral force versus horizontal displacement for Specimens RE1B and E1B

Figure 3-27 shows the comparison of lateral force versus horizontal displacement for Specimens RC1A and C1A. Specimen RC1A was able to restore the strength at every stage of Specimen C1A. The maximum load obtained by Specimens C1A and RC1A were 19.1 kN at drift ratio of 2 % and 28.6 kN at drift ratio 4 % respectively. The development of slower strength degradation, which is favorable in rehabilitation objectives, was evident in Specimen RC1A. The strength of Specimen RC1A in the positive cycle sustained relatively the same amount of load from drift ratio of 2 % to 4 %. Towards the last few cycles, pinching was observed as indicated by the slope of the hysteresis loops of Specimen RC1A at zero displacement.

Figure 3-27: Comparison of lateral force versus horizontal displacement for Specimens RC1A and C1A

There was a tremendous increase in strength for Specimen RC1B as shown in Figure 3-28. The maximum load obtained by Specimens C1B and RC1B were 15.5 kN (at drift ratio of 2 %) and 27.8 kN (at drift ratio of 4 %) respectively. The strength started to deteriorate after drift ratio of 3% for Specimen RC1B. It was observed that more energy was dissipated enclosed by the wider loops in the positive cycle compared to those in the negative cycle.

Figure 3-28: Comparison of lateral force versus horizontal displacement for Specimens RC1B and C1B

The increase in strength of Specimen RC2A was gradual shown in Figure 3-29. The strength was only restored at drift ratio of 4 % where the strength obtained by Specimen RC2A was higher than that of Specimen C2A. The maximum load obtained by Specimens C2A and RC2A were 134.3 kN at drift ratio of 3 % and 142.5 kN at drift ratio of 4 % respectively. Slight pinching was observed.

Figure 3-29: Comparison of lateral force versus horizontal displacement for Specimens RC2A and C2A

The hysteresis response of Specimen RC2B was similar to that of Specimen RC2A. However, the slopes of the hysteresis loops were steeper for Specimen RC2B (see Figure 3-30). CFRP sheets being stiffer than GFRP sheets could account for this observation. Specimen RC2B could restore the strength of Specimen C2B at drift ratio of 3 % in the negative cycle and at drift ratio of 4 %. The maximum load obtained by Specimens C2B and RC2B were 139 kN (at drift ratio of 3 %) and 153.8 kN (at drift ratio of 4 %) respectively.

Figure 3-30: Comparison of lateral force versus horizontal displacement for Specimens RC2B and C2B

3.5.2.2 Strengthened virgin specimens

During the first few cycles of loading, the slopes of the hysteresis loops (see Figures 3-31, 3-32 and 3-33) were steep, indicating a tremendous increase in strength. At drift ratio of 3.0%, when the cracks were apparent near the joint core corners for Specimen SE1B, the slope of the hysteresis loops (see Figure 3-31), was nearly horizontal, indicating that the peak strength of the beam-wide-column joint was reached. Similar to Specimen SE1B, Specimen SC1B also indicated nearly horizontal slope of the hysteresis loops when widening of cracks was observed at drift ratio of 3.0% (see Figure 3-32). However, for Specimen SC2C, though there was significant widening of cracks at drift ratio of 2.0%, the slopes of the hysteresis loops were still increasing, though at a slower rate (see Figure 3-33).

Figure 3-31: Comparison of lateral load versus horizontal displacement for Specimens SE1B and E1B

Figure 3-32: Comparison of lateral load versus horizontal displacement for Specimens SC1B and C1B

Figure 3-33: Comparison of lateral force versus horizontal displacement for Specimens SC2B and C2B

Towards the last few cycles of loading, the slope of the hysteresis loops had a negative slope for Specimen SC1C, indicating that strength of specimens was deteriorating. It was not apparent for Specimen SE1C, where the slope of the hysteresis loops was nearly horizontal; suggesting that after drift ratio 4 %, the strength of the specimen might decrease. Also, compared to the unstrengthened Specimen E1C, there were no severe pinching for Specimen SE1C after drift ratio 3%; implying that there was good energy dissipation at the beam ends. This phenomenon was similar for Specimens SC1B and SC2B. The slope of hysteresis loops for Specimen SC2B continued to increase after drift ratio 3 %, which suggested that it was capable of higher strength and the FRP strips and sheets were not fully utilized.

The strength of the strengthened specimens was definitely higher than the corresponded unstrengthened specimens (see Table 3-5), hence, in this sense, the strengthening methods used could be deemed successful.

Table 3-5 Maximum strength of as-built, repaired and strengthened virgin specimens

Specimen	As-built		Repaired		Strengthened	
	Negative loading (kN)	Positive loading (kN)	Negative loading (kN)	Positive loading (kN)	Negative loading (kN)	Positive loading (kN)
E1A	17.3	19.8	-	-	-	-
RE1A	-	-	20.3	19.8	-	-
E1B	-14.9	15.8	-	-	-	-
RE1B	-	-	25.0	21.6	-	-
SE1B	-	-	-	-	-42.8	36.9
C1A	19.1	18.4	-	-	-	-
RC1A	-	-	28.6	21.6	-	-
C1B	-15.5	15.5	-	-	-	-
RC1B	-	-	27.8	25.0	-	-
SC1B	-	-	-	-	-30.6	31.8
C2A	127.0	134.3	-	-	-	-
RC2A	-	-	135.9	142.5	-	-
C2B	-139.0	139.0	-	-	-	-
RC2B	-	-	143.7	153.8	-	-
SC2B	-	-	-	-	-223.0	214.0

3.5.3 Energy dissipation capacity

A structure surviving through an earthquake depends to a large extent on its ability to dissipate the input energy. In this study, one form of energy dissipation: hysteretic damping was estimated by the area enclosed by the lateral load-displacement loops. Comparisons of cumulated energy dissipation capacity of as-built and repaired/strengthened specimens are presented. The cumulative energy dissipated was calculated by summing up the energy dissipated in consecutive loops throughout the test.

3.5.3.1 Repaired specimens

Initially, the energy dissipation capacity of Specimen RE1A was higher than E1A (see Figure 3-34). Subsequently till drift ratio of 2%, the energy dissipation capacity of E1A could not be restored. After which, the energy gradients are relatively steeper due to energy dissipated via breaking of epoxy resin and debonding of GFRP sheets. On the other hand, the trend of energy dissipation capacity at every stage for both Specimens E1B and RE1B was similar (see Figure 3-35). However, the energy gradient between drift ratio of 3% and drift ratio of 4% was steeper for Specimen RE1B.

Figure 3-34: Comparison of cumulated energy dissipation capacity versus drift ratio of Specimens E1A and RE1A

Figure 3-35: Comparison of cumulated energy dissipation capacity versus drift ratio of Specimens E1B and RE1B

The performance of Specimen RC1A in terms of energy dissipation capacity was not as good as that of Specimen C1A (see Figure 3-36). However, at the second and third cycles of each drift ratio, the energy dissipated was similar and higher at drift ratio of 4%. Unlike Specimen RC1A, Specimen RC1B was not able to restore the energy dissipation capacity of Specimen C1B at every stage (see Figure 3-37).

Figure 3-36: Comparison of cumulated energy dissipation capacity versus drift ratio of Specimens C1A and RC1A

Figure 3-37: Comparison of cumulated energy dissipation capacity versus drift ratio of Specimens C1B and RC1B

For Specimen RC2A, the energy dissipation capacity was about 50% restored as shown in Figure 3-38. The energy gradients for both specimens were similar. Unlike Specimen RC2A, Specimen RC2B was also effective in restoring the energy

dissipation capacity of Specimen C2B as shown in Figure 3-39. Towards the last few cycles, the energy gradient was not as steep as that of Specimen C2B.

Figure 3-38: Comparison of cumulated energy dissipation capacity versus drift ratio of Specimens C2A and RC2A

Figure 3-39: Comparison of cumulated energy dissipation capacity versus drift ratio of Specimens C2B and RC2B

3.5.3.2 Strengthened virgin specimens

The energy dissipation capacity of Specimen SE1B was higher than E1B (see Figure 3.40) for every drift ratio. The trend of energy dissipation capacity at every stage for both specimens was similar. After drift ratio of 2.0%, the energy gradients for Specimen SE1B are relatively steeper due to energy dissipation through increasing cracks and FRP debonding. Similar to Specimen SE1B, Specimens SC1B and SC2B also showed higher energy dissipation energy for every drift ratio compared to the as-

built specimens. Towards the last few cycles, the energy gradients are relatively steeper due to the widening of cracks observe on the beams and the debonding of FRP sheets near the beam-column interface. In particular for Specimen SC2B (see Figure 3-42), the energy dissipated at drift ratio of 4% was significantly higher than Specimen C2B. This could be due to the multi-diffuse cracking between the bands of CFRP strips round the column which permits considerable energy dissipation.

Figure 3-40: Comparison of cumulated energy dissipation capacity versus drift ratio of Specimens E1B and SE1B

Figure 3-41: Comparison of cumulated energy dissipation capacity versus drift ratio of Specimens C1B and SC1B

Figure 3-42: Comparison of cumulated energy dissipation capacity versus drift ratio of Specimens C2B and SC2B

3.5.4 Stiffness degradation

The stiffness of all specimens was estimated on the basis of its secant value (lateral force versus horizontal displacement) at each load cycle. The stiffness degradation may be attributed to the following: flexural and shear cracking, concrete deterioration (non-linear deformation and loss of cover), joint distortion, bar slippage and debonding or delamination of GFRP and CFRP sheets. All repaired specimens were able to restore the stiffness of as-built specimens, except Specimen RC2A. All strengthened virgin specimens improved the stiffness of as-built specimens significantly.

3.5.4.1 Repaired specimens

In the initial stage, Specimen RE1A was much stiffer than E1A (see Figure 3-43). After which, there was a rapid decrease in stiffness due to concrete cracking. The FRP debonding after drift ratio of 2% explained the relatively linear curve in the subsequent cycles. Specimen RE1B, though had lower initial stiffness (see Figure 3-44), generally restored the stiffness of Specimen E1B at every stage. Its initial stiffness started to increase until about drift ratio of 0.2% and decrease rapidly until drift ratio of 1%. After drift ratio of 1%, the curve was relatively linear; slower

stiffness degradation compared to Specimens E1B and RE1A. Such slower stiffness degradation is desirable as often during an earthquake, structures collapse due to sudden loss of stiffness of beam-column joints with increasing lateral movement of the structure.

Figure 3-43: Comparison of secant stiffness versus drift ratio of Specimens E1A and RE1A

Figure 3-44: Comparison of secant stiffness versus drift ratio of Specimens E1B and RE1B

Specimen RC1A restored the stiffness of Specimen C1A and showed to have higher stiffness at every drift ratio except at the initial stage (see Figure 3-45). Before drift ratio of 4%, there was a sharp increase in stiffness for Specimen RC1A. Subsequently,

the increase was gradual due to concrete cracking and towards the later cycles, due to epoxy resin failure with significant crack marks observed near joint regions. Similar to Specimen RC1A, Specimen RC1B restored the stiffness of Specimen C1B at every stage (see Figure 3-46). However, in the initial stage, unlike Specimen RC1A, there was a sharp increase in stiffness of Specimen RC1B. Near drift ratio of 1%, the stiffness degradation started to plateau due to the rapid propagation of flexural-shear cracks.

Figure 3-45: Comparison of secant stiffness versus drift ratio of Specimens C1A and RC1A

Figure 3-46: Comparison of secant stiffness versus drift ratio of Specimens C1B and RC1B

Specimen RC2A had poor stiffness restoration as shown in Figure 3-47. This supported the gentle slopes of the hysteresis loops at every stage of Specimen RC2A. Unlike Specimen RC2A, Specimen RC2B was effective in restoring the stiffness of Specimen C2B (see Figure 3-48). In the initial stage, there was a sharp increase in stiffness until drift ratio of 2% for Specimen RC2B.

Figure 3-47: Comparison of secant stiffness versus drift ratio of Specimens C2A and RC2A

Figure 3-48: Comparison of secant stiffness versus drift ratio of Specimens C2B and RC2B

3.5.4.2 Strengthened virgin specimens

With the increase in displacement and the number of cycles, the hysteresis loops tend to be inclined. In all strengthened specimens, it was observed that there was a tremendous increase in stiffness during the first few cycles. After which, there was a gradual decrease in stiffness for the subsequent cycles. This explains the phenomenon of concrete cracking, steel yielding and failure of steel-concrete adherence taking place. The loss of stiffness may be primarily attributed to concrete deterioration in the beam-wide-column and beam-wall joint region. As the concrete degrades, the load on the FRP strips and sheets increases; this explains the relatively linear curve in the subsequent cycles. It is noteworthy that the increase in stiffness at every drift ratio for Specimen SE1B (see Figure 3-49) was significant; suggesting that the two layers of CFRP strips at the joint and beam faces had contributed to the stiffness.

Figure 3-49: Comparison of secant stiffness versus drift ratio of Specimens E1B and SE1B

On the other hand, there was severe stiffness degradation until drift ratio of 1% for Specimens SC1B and SC2B (see Figure 3-50 and Figure 3-51). Towards the last cycle, the stiffness for all specimens is fairly low. This is due to the cracks, which are caused by the yielding of the beam longitudinal bars, remaining unclosed.

Figure 3-50: Comparison of secant stiffness versus drift ratio of Specimens C1B and SC1B

Figure 3-51: Comparison of secant stiffness versus drift ratio of Specimens C2B and SC2B

3.5.5 Displacement decomposition

The total inter-story drift recorded at the top of the column consisted of several components, comprising lateral displacements due to the beam flexure, beam shear, column flexure deformations and joint shear distortion. Comparisons of displacement decomposition versus drift ratio relationship for as-built and repaired/ strengthened

specimens. Comparisons between as-built and repaired specimens showed a major source of the story drift was beam displacement, indicating strong-column weak-beam response for both specimens. However, for Specimen RC2A the contributions from beam flexure and column flexure were comparatively similar. Comparisons between as-built and strengthened specimens showed a decrease in contribution to total drift from beam displacement was observed in all strengthened specimens. The FRP sheets and strips applied on the beam faces could have increase the stiffness of the beams. Furthermore, the FRP sheets and strips were well-anchored near the beam-column joint interface, adding to the stiffness of the beams. To ensure equilibrium, a larger amount of load was imposed on the columns, resulting in an increase in contribution to total drift from column flexure observed in all strengthened specimens as compared to the unstrengthened ones.

3.5.5.1 Repaired specimens

Figure 3-52 shows the displacement decomposition for as-built and repaired E1 series specimens. Compared to Specimen E1A Specimen RE1A had a decrease in contribution to total drift from beam flexure while an increase from column flexure. The contributions to the total drift from beam flexure, beam shear, column flexure and joint shear at drift ratio of 2% were 22.7%, 3.2%, 10.3% and 9.3% respectively for Specimen RE1A. Note that the LVDTs were not functioning at drift ratio of 4% for Specimen RE1A. From Figure 3-52, Specimen RE1B had a decrease in contribution to total drift from beam flexure while an increase from joint shear compared to Specimen E1B. The contribution to total drift from beam flexure and joint shear was similar until drift ratio of 4%. The contributions to the total drift from beam flexure, beam shear, column flexure and joint shear at drift ratio of 4% were 55.4%, 14.5%, 8% and 14.6% respectively for Specimen RE1B.

Figure 3-52: Displacement decomposition for as-built and repaired E1 series specimens

Figure 3-51 shows the displacement decomposition for as-built and C1 series specimens. Compared to Specimen C1A, Specimen RC1A had a decrease in contribution to total drift from beam flexure and beam shear. There was a slight increase in contribution from column flexure. The contributions to the total drift from beam flexure, beam shear, column flexure and joint shear at drift ratio of 4% were 41.5%, 5.7%, 2.7% and 1.1% respectively for Specimen RC1A. Similar to Specimen RC1A, Specimen RC1B had a slight increase in contribution to total drift from column flexure while a decrease from beam flexure. The contributions to the total drift from beam flexure, beam shear, column flexure and joint shear at drift ratio of 4% were 24.5%, 2.7%, 9.3% and 0% respectively for Specimen RC1B.

Figure 3-53: Displacement decomposition for as-built and repaired C1 series specimens

Figure 3-54 shows the displacement decomposition for as-built and C2 series specimens. A strong-column weak-beam response was not quite apparent in Specimen RC2A. Compared to Specimen C2A, Specimen RC2A had an increase in contribution to total drift from column flexure while a slight increase from joint shear. The contributions to the total drift from beam flexure, beam shear, column flexure and joint shear at drift ratio of 4% were 36.4%, 4.3%, 36.9% and 0.6% respectively for Specimen RC2A. Similar to Specimen RC2A, a strong-column weak-beam response was not quite apparent in Specimen RC2B. Compared to Specimen C2B, Specimen RC2B had an increase in contribution to total drift from column flexure while a slight increase from joint shear. The contributions to the total drift from beam flexure, beam shear, column flexure and joint shear at drift ratio of 4% were 31.5%, 5.2%, 32.7% and 0.4% respectively for Specimen RC2B.

Figure 3-54: Displacement decomposition for as-built and repaired C2 series specimens

3.5.5.2 Strengthened virgin specimens

For both Specimens E1B and SE1B, the major source of the story drift was beam displacement (see Figure 3-55), indicating a strong-column weak-beam response. However, the contributions to the total drift from beam flexure had decreased considerably; it varied from 57.7 to 95.6% in Specimen E1B and 30 to 38% in Specimen SE1B. The contributions to the total drift from beam flexure, beam shear, column flexure, and joint shear at drift ratio of 4% for Specimen SE1B were 30%, 22%, 5% and 15% respectively. The contribution to the total drift from column flexure did not change significantly during the testing while that from joint shear distortion increased gradually.

Figure 3-55: Displacement decomposition for as-built and strengthened E1 series specimens

Similar to Specimen SE1B, the same trend was also observed in Figure 3-56. The contribution to the total drift from beam flexure had decreased tremendously for Specimen SC1B; it varied from 51.6 to 79.2% in Specimen C1B and 1.5 to 13% in Specimen SC1B. Column flexure deformation was predominant instead in the strengthened specimen; it varied from 2 to 5% in Specimen C1B and from 27 to 36% in Specimen SC1B. The contributions to the total drift from beam flexure, beam shear, column flexure, and joint shear were 1.5 to 13%, 0.4 to 6%, 27 to 36% and 0 to 1% respectively. Both Specimens C1B and SC1B had insignificant contribution to total drift from joint shear distortion. Similar to Specimen SE1B, the contribution to the total drift from column flexure for SC1B did not change significantly during the testing.

Figure 3-56: Displacement decomposition for as-built and strengthened C1 series specimens

For Specimen SC2B, according to Figure 3-57, the contribution of beam flexure had decreased slightly compared to the unstrengthened specimen; it varied from 23.2 % to 36.1% in Specimen C2B and from 16 % to 28% in Specimen SC2B. The contributions to the total drift from beam flexure and column flexure deformation were predominant in Specimen SC2B. The contribution from column flexure was relatively higher for Specimen SC2B than C2B. The gradual increase in the contribution to total drift from column flexure was supported by the increase in multi-diffuse cracking between bands of CFRP strips confining the column. The contribution to the total drift from beam flexure, beam shear, column flexure, and joint shear at drift ratio of 4% were 28%, 5%, 34% and 1% respectively. Due to the three dimensional nature of the specimen, the transducers placed diagonally in the joint panel especially for the C1 and C2 series, the joint shear deformation could not be captured.

Figure 3-57: Displacement decomposition for as-built and strengthened C2 series specimens

3.5.6 Distribution of strains in longitudinal reinforcement of beam

The strain profiles of the beam top longitudinal reinforcement at the peak lateral load of each drift ratio for the as-built and strengthened specimens are plotted. These strain profiles represent the local strains along the length of beam top. The strains are directly obtained from the readings measured by electrical strain gauges. In all specimens, distributions of strain along the reinforcement vary considerably with the increase in lateral load. When the longitudinal reinforcement was in compression, the

compressive strains measured were initially small since most of the compression was transferred through the concrete. However, as the loading progressed towards the inelastic range, the strains measured are in positive values, indicating that portion was subjected to tension.

Figure 3-58 shows the strain profiles of the beam top longitudinal reinforcement at the peak lateral load of each drift ratio for Specimens E1B and SE1B, the first yield of longitudinal reinforcement occurred on beam-column interface at drift ratio 1/75 and 1/50 respectively. Similarly, the largest tensile strain was detected on the same location at drift ratio 1/50 and 1/33 for Specimens E1B and SE1B respectively; however, as the drift ratio increased, the recorded maximum tensile strain at the same position was much smaller. This could be attributed to severe cracking and shearing of concrete in the plastic hinge region which had caused the loss of bond strength between the reinforcement and the concrete, resulting in bar slippage. Degradation of anchorage resistance in the reinforcement passing through the joint occurred at drift ratio 1/50 for Specimen E1B; Specimen SE1B displayed good anchorage resistance up to drift ratio 1/25. Strains within the beam-column joint region have never exceeded the elastic range, implying that the plastic hinge had been successfully confined near beam end. The application of two layers of CFRP strips aligned on both sides of beam faces had delayed the yielding of longitudinal reinforcement.

Specimen E1B

Specimen SE1B

Figure 3-58: Local strains of beam top longitudinal reinforcement for Specimens E1B and SE1B

Figure 3-59 shows the strain profiles of the beam top longitudinal reinforcement at the peak lateral load of each drift ratio for Specimens C1B and SC1B. The first yield of longitudinal reinforcement for Specimen C1B occurred within the joint region at drift ratio 1/50, whereas that for strengthened Specimen SC1C occurred on the beam-column interface at drift ratio 1/75. The largest tensile strain was similarly detected at the same corresponding locations for Specimens C1B and SC1B at the same drift ratio of 1/33; however, the strengthened specimen had relatively much smaller largest tensile strain value compared to original specimen. It was further noted that all strains in the inelastic range were observed at the beam-column interface in Specimen SC1B; however, that of Specimen C1B were observed within the joint. This implied that the plastic hinges of SC1B were successfully confined to the beam end. Degradation of anchorage resistance in the reinforcement passing through the joint occurred at drift ratios 1/150 and 1/75 for Specimen C1B and SC1B respectively.

Figure 3-59: Local strains of beam top longitudinal reinforcement for Specimens C1B and SC1B

Figure 3-60 shows the strain profiles of the beam top longitudinal reinforcement at the peak lateral load of each drift ratio for Specimens C2B and SC2B. The first yield of longitudinal reinforcement for Specimen C2B occurred within the joint region at drift

ratio 1/50, whereas that for strengthened Specimen SC2B occurred outside the joint area near the beam-column interface at drift ratio 1/33. At drift ratio 1/25, the largest tensile strain was similarly detected at the same location for Specimens SC2B; however, for Specimen C2B, it was located at beam-column interface. It could be concluded that the plastic hinges of C2B and SC2B were successfully confined to the beam end. Note that two strain gauges were not functional at the extreme left of Specimen SC2B. They could have been damaged during casting. Specimens C2B and SC2B still displayed a good anchorage resistance characteristic up to a drift ratio 1/33 as explained in Figure 3-60. This shows that the shallow column depth was not deep enough for beam bars to develop the full anchorage.

Specimen C2B

Specimen SC2B

Figure 3-60: Local strains of beam top longitudinal reinforcement for Specimens C2B and SC2B

Bond deterioration occurred along the beam bars of all specimens at a drift ratio of 4.0%. Bond condition is determined mainly by the ratio of the beam and column bar diameter to the column and beam depths. The ratio of beam bar diameter to the column depth of all specimens does not satisfy the requirement given by NZS 3101 (1982). Hence, it is not surprising that bond deterioration occurred along the beam bars at the drift ratio of 4%.

With reference to these strain profiles in Figure 3-58, Figure 3-59 and Figure 3-60, yielding of the longitudinal bars spread over a distance of approximately $1.4d$ from the column face for all specimens, thus indicating the concentration of plastic hinges in the vicinity of this region. This indicated that the proposed FRP wrapping schemes were effective in confining the plastic hinges in the vicinity of this region. Anchoring the GFRP strip near the beam-column interface at beam top added to the effectiveness.

3.5.7 Distribution of strains in reinforcement along column height

The local strains of the longitudinal reinforcement along the column height for original and strengthened specimens are plotted. For all specimens, the column strains were in the elastic range when the beam reached its flexural strength, indicating a “strong column-weak beam” response. Figure 3-61 shows the local strains of the longitudinal reinforcement along the column height for Specimens E1B and SE1B. Specimen SE1B had relatively smaller strains compared to Specimen E1B. However, there was not much change for Specimen SC1B compared to Specimen C1B as shown in Figure 3-62. Note that the negative region in Figure 3-62, there are no readings as two strain gauges for Specimen SC1B were not functional during the test. They could have been damaged during casting of specimen. The small strains observed implied that there were little flexural cracks, in particular lesser in Specimen SE1B compared to the original specimen, at the column. Confining the columns with GFRP sheet at 400 mm and 200 mm along the column height from the beam top for Specimen SE1B and SC1B respectively, was sufficient and effective.

Specimen SE1B

Figure 3-61: Local strains of the longitudinal reinforcement along the column height for Specimens E1B and SE1B

Specimen C1B

Figure 3-62: Local strains of the longitudinal reinforcement along the column height for Specimens C1B and SC1B

Figure 3-63 shows the local strains of the longitudinal reinforcement along the column height for Specimens C2B and SC2B. For Specimen SC2B, the column bar strains were comparatively similar compared to Specimen C2B; however, it had considerably reduced strains at its bottom column compared to the original specimen. In column longitudinal reinforcement passing through beam-column joints, bond stress is imposed due to change in column moment over joint depth. If no slippage of column longitudinal reinforcement occurs, the column bar strains should change from tension on one side of the joint and compression on the other side. Slippage of column bars occurred at drift ratios 1/33 and 1/50 at column bottom for Specimen SC2B and C2B respectively. However, the slippage was found only within 200mm along the column height from beam bottom for Specimen SC2B. This implied that using CFRP strips for confining the column was effective and economical.

Figure 3-63: Local strains of the longitudinal reinforcement along the column height for Specimens C2B and SC2B

3.5.8 Discussion

Comparing the hysteretic response of as-built Specimens E1A and E1B, the constant axial load of $0.10 f'_c A_g$ enabled the former to reach a higher maximum load while the latter showed good ductile response and failed later at drift ratio of 3.0 %. It was also observed that CFRP sheet applied across the beams and joint core of Specimen RE1B resulted in higher strength obtained at lower drift ratio. However, it could be due to less severe damage on its as-built Specimen E1B compared to Specimen E1A. On the other hand, for as-built Specimen C1A and C1B, the effect of higher constant axial load of $0.35 f'_c A_g$ was not significant. The maximum load reached was quite similar for both specimens. However, after repair, Specimen RC1A which used CFRP sheets across the beams and joint reached maximum load at earlier drift ratio. For as-built Specimens C2A and C2B, though Specimen C2B underwent higher constant axial load, it had similar energy dissipation capacity as Specimen C2A. However, it had a higher maximum load achieved.

The effect of eccentricity was not significant for as-built Specimens E1A, E1B, C1A and C1B when the maximum load reached achieved, stiffness degradation and energy dissipation capacity were similar. Therefore, the effect of eccentricity does not play a role in the repair works.

Both strengthening schemes employed shown good stiffness restoration in all specimens except Specimen RC2A. The lower initial stiffness for Specimens RE1B and RC1A may be due to the initial damage and yielding of steel bars before repair. The rapid stiffness degradation shown for every specimen till drift ratio 2.0 % was due to breaking of epoxy resin and failure of steel-concrete adherence taking place where shear cracks occurred on the beams. CFRP sheets being higher in elastic modulus performed better in improving the stiffness of the as-built specimens. This was evident at every drift ratio for Specimen RE1B. Similarly, Specimen RC1A which used CFRP sheets shown to have better improvement especially during the first few drift ratios. Specimen RC2B was able to restore its stiffness while Specimen RC2A could not restore its stiffness. Its relative flat curve could be due to severe poor steel-concrete adherence and concrete degradation where bulging of GFRP sheet was observed near the beam-column joint interface.

A structure surviving through an earthquake depends to a large extent on its ability to dissipate the input energy. In this study, one form of energy dissipation: hysteretic damping was estimated by the area enclosed by the lateral load-displacement loops. The steeper energy gradient between drift ratio of 3.0 % and 4.0 % observed in Specimen RE1B could be due to the widening of cracks on the beam, the friction along the cracks and the delamination of FRP sheet near the beam-column interface. CFRP sheets were effective in improving energy dissipation capacity; unlike Specimen RC1A, Specimen RC1B was not able to restore the energy dissipation capacity of as-built Specimen C1B at every stage. Severe concrete cover spalling near beam-column joint interface of left beam could explain the less steep energy gradient towards the last few cycles of Specimen RC2B compared to Specimen C2B.

There was no anchorage failure which indicated that the use of fiber anchors in preventing the delamination of FRP sheets was effective in enhancing the capacity of the FRP sheets. As it was commonly found that there was delamination of FRP sheets near the top beam-column joint interface, fiber anchors could be considered to be applied here. One to caution drill carefully the depth of fiber anchor hole such that it would not meet the reinforcement. A depth of 50 to 75 mm was employed in this study.

References:

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4 Seismic Performance of Reinforced Concrete Structural Walls with and without Openings

4.1 Introduction

This chapter reports on experimental studies aim to understand the seismic performance of repaired and strengthened reinforced concrete structural with and without openings. The failure modes of six previously damaged specimens are briefly presented. These virgin specimens were works of others; Specimens LW2, LW3, MW1 and MW2 were works of Li and Xiang (2006). Specimen S-F2 was works of Wu (2004) while Specimen W3 was works of Zhao (2004). Repair involved replacement of damage concrete by high strength mortar and filling of cracks with epoxy, while strengthening involved wrapping the walls with GFRP and CFRP sheets and strips. To assess the effectiveness of the proposed FRP wrapping schemes, comparisons between responses of specimens before and after repair are made.

4.2 Experimental program

4.2.1 Specimens

The experimental program involved six specimens denoted as LW2, LW3, MW1, MW2, S-F2 and W3. All specimens had three subassemblies: the top beam, the web and the foundation beam. After repair and strengthening, the specimens were denoted as RLW2, RLW3, RMW1, RMW2, RS-F2 and RW3. Specimens LW2, LW3, MW1 and MW2 were cantilever type walls without openings. The low-rise walls, LW2 and LW3, had rectangular 120 mm x 1700 mm cross-sections and height 2250 mm (aspect ratio of 1.125). The medium-rise walls, MW1 and MW2, had 300 mm x 1700 mm cross-sections and height 3500 mm (aspect ratio of 1.75). Note that the height of specimens mentioned here excluded the foundation block. For all specimens, the web reinforcement consisted of a double orthogonal grid of 10mm diameter bars spaced at 250 mm. The 300 mm x 150 mm boundary elements were reinforced with eight bars of 10mm diameter. Confinement in the boundary elements was provided by 6 mm diameter closed stirrups spaced at 75 mm. Figure 4-1 shows the geometry and reinforcement details of Specimens LW2, LW3, MW1 and MW2.

The average compressive strength of concrete was 40 MPa. Average values of measured yield strengths of 10 mm diameter bars as web reinforcement were 462 MPa and 382 MPa, for low-rise and medium-rise walls respectively. The 10 mm diameter bars used in the boundary elements had an average measured yield strength of 380 MPa.

Low-rise walls (Li and Xiang 2006)

Medium-rise walls (Li and Xiang 2006)

Figure 4-1: Geometry and reinforcement details of walls without openings

Specimens S-F2 and W3 were walls with openings. Specimen S-F2, which had irregular distributed openings, modeled Yanez’s specimen (Yanez 1993) except with an addition of 120 mm thick by 140 mm deep flanges to assess the effect of flanges. It was 2000 mm wide, 2300 mm high and 120 mm thick (aspect ratio of 1.30). Each of the three irregularly distributed openings was of size 600 mm x 600 mm. The vertical main bars were distributed at the column zones between the openings and the bottom panels, while the horizontal main bars were distributed in the beam zones. High yield steel deformed bars (denoted as T) were applied as the main bars and the mild steel plain bars (denoted as R) were used as secondary reinforcements. Mild steel bars were used in the flanges to limit concrete cracking. The vertical and horizontal reinforcements were 0.5%. shows the geometry and reinforcement details of Specimen S-F2.

Figure 4-2: Geometry and reinforcement details of Specimen S-F2 (Wu 2004)

Specimen W3, which had regular distributed openings, modeled Marti’s low rise RC structural wall (Marti 1985). The web of Specimen W3 was scaled down to 44% of Marti’s wall to fit the existing loading frame. It was 2600 mm wide, 2300 mm high and 120 mm thick, with aspect ratio of 1.0. The vertical and horizontal reinforcements were 0.60 % and 0.51 % respectively. To limit concrete cracking, mild steel bars were added at panel zones. Figure 4-3 shows the geometry and reinforcement details of Specimen W3.

Figure 4-3: Geometry and reinforcement details of Specimen W3 (Zhao 2004)

Average values of compressive strength of concrete were 37 MPa and 39 MPa for Specimens S-F2 and W3 respectively. High yield steel bar (hot rolled) with nominal yield strength of 460 MPa and mild steel bar (hot rolled) with nominal yield strength of 250 MPa were used.

4.2.2 Test setup

4.2.2.1 Walls without openings

Figure 4-4 illustrates the experimental set-up for repaired walls without openings. An in-plane loading system, which included three double-acting hydraulic actuators, was used. The actuators had a capacity of 1000 kN with a stroke of 1000 mm when fully extended. The steel beam at the top of each wall specimen was bolted to a horizontal double-acting hydraulic actuator and two vertical double-acting hydraulic actuators. Twelve high strength steel rods (six on each side) were used to tighten the loading beam to the wall to enable the lateral loading to be introduced as compression on the end faces of the top loading beam. The lateral load was applied cyclically through the

horizontal double-acting hydraulic actuator in a quasi-static fashion at the top end of the wall.

Figure 4-4: Experimental setup for repaired walls without openings

4.2.2.2 Walls with openings

Figure 4-5 shows the experimental setup for walls with openings. The steel beam at the top of each wall specimen was bolted to a horizontal double-acting hydraulic actuator. The lateral load was applied cyclically through the horizontal double-acting hydraulic actuator in a quasi-static fashion at the top end of the wall. The loading cycles were displacement-controlled where the top displacement is calculated based on drift ratio of $1/2000$, $1/1000$, $1/600$, $1/400$, $1/300$, $1/200$, $1/150$, $1/100$, $1/75$ and $1/50$ multiply by the vertical height of 2540mm from the wall base to the point where the load is applied.

Figure 4-5: Experimental setup for repaired walls with openings

4.2.3 Test instrumentation and procedures

4.2.3.1 Walls without openings

Throughout the testing, the specimens were subjected to constant axial compression loading by two vertical hydraulic actuators. The total axial loading on each wall specimen was calculated based on the tested concrete strength of the specimen:

$$P_i = 0.05 f'_c A_g \text{ ----- Equation 1}$$

where $f'_c = 40 \text{ MPa}$; A_g is the cross- sectional area of the wall. Since the total axial loading is exerted by two vertical hydraulic actuators on each side, vertical force exerted by one actuator is $P_i / 2$.

In order to restrain rotation and sliding of the specimens during the applied loading, a foundation block of 900 mm thick, 3600 mm wide and 500 mm high was designed as the base of the specimens. In addition, the specimens were attached to the strong floor by pre-stressed high strength rods. The boundary elements helped to resist the large bending moment sustained by the base of the specimens. The ultimate load capacity and ductility of the wall specimens were increased which limited the crushing of concrete and buckling of vertical reinforcement. Reaction footings are provided to balance the lateral loading.

The test results were explained qualitatively by using instrumentations such as hydraulic actuators and linear variable displacement transducers (LVDTs). One horizontal LVDT was installed at the top of the wall to measure the top deformation. LVDTs were inclined to measure the sliding displacement of the base and shear deformation of the wall while some LVDTs were placed vertically to measure the axial deformations. The typical loading procedure, recommended by Park [1989] consisting of displacement controlled steps is illustrated in Figure 4-6.

Figure 4-6: Typical loading history for repaired walls without openings

4.2.3.2 Walls with openings

Repaired Specimens RS-F2 and RW3 were tested using an in-plane loading system in which the top flange attached to the loading arm by four steel bars. Lateral cyclic loading was applied on the top flange of the wall by the pushing and pulling action of the loading arm. The loading arm was hydraulically operated by the load cell and controlled manually. Linear Variable differential Transducers (LVDT) of 300 mm, 100 mm and 50 mm were used and strain gauges were attached on the fiber surface. Readings from LVDTs and strain gauges were recorded in the data logger. Positions of the LVDTs and fiber strain gauges are shown in the following Figure 4-7 to Figure 4-10. The loading cycles were displacement-controlled where the top displacement is calculated based on drift ratio of 1/2000, 1/1000, 1/600, 1/400, 1/300, 1/200, 1/150,

1/100, 1/75 and 1/50 multiply by the vertical height of 2540mm from the wall base to the point where the load is applied. The typical loading procedures, recommended by Park [1989] consisting of displacement-controlled cycles are illustrated in Figure 4-11

Figure 4-7: Positions of LVDTs on Specimen RS-F2

Figure 4-8: Position of fiber strain gauges on Specimen RS-F2

Figure 4-9: Positions of LVDTs on Specimen RW3

Figure 4-10: Positions of fiber strain gauges on Specimen RW3

Figure 4-11: Typical loading procedures for walls with openings

4.3 Failure modes of as-built specimens

4.3.1 Walls without openings

Figure 4-12 illustrates the crack patterns of the original specimens. The failure mode was similar for all four specimens. The specimens generally behaved in a flexural manner and were capable of developing their flexural yielding prior to failure. Hairline shear and flexural cracks were formed along the height of the specimens, while heavy damage was concentrated at the base. Spalling of the cover concrete was observed along with failure of the concrete under compression inside the confined core at the base, large inelastic elongation, and buckling of steel bars. Despite their low slenderness, the wall specimens failed in a predominantly flexural mode, characterized by concrete crushing and reinforcement buckling at the confined edges. Moderate diagonal cracking of the web and shear sliding at the fixed base were also observed, but did not significantly affect the failure mode. It was noted that the damage of the specimens was mainly located at the lower part of the walls except for Specimen LW3, where the damage zone extended upwards and occurred within the left flange ranging from approximately 500mm to 1000mm (Li and Xiang 2006).

Figure 4-12: Crack patterns of low-rise and medium-rise as-built walls without openings (Li and Xiang 2006)

4.3.2 Walls with openings

Specimens S-F2 and W3 had been tested to failure (see Figure 4-13 and Figure 4-14 respectively), as indicated by a strength reduction exceeding 20%, and had sustained severe damage. According to Wu (2004), Specimen S-F2 developed a sliding failure at the bottom panel but the sliding face was about 200 mm above the wall base. Damaged sustained by Specimen S-F2 included severe concrete crushing and spalling at bottom flanges and right bottom corner of bottom panel; the shear force had to be sustained by the bottom panel as the bottom right column could not sustain the shear force after the concrete there severely cracked and when the sliding face formed, the strength of the wall decreased obviously. Concrete crushing was also observed along the sliding face. Most of the steel bars in the flanges buckled and some were fractured. Specimen W3, on the other hand, had obvious movement observed along the diagonal cracks which appeared on every panel. Its four bottom columns were severely damaged, in particular, very severe concrete crushing was observed for the two columns at bottom edges. Large cover concrete spalled off and buckling of steel bars was observed.

Figure 4-13: Crack patterns of Specimen SF-2 (Wu 2004)

Figure 4-14: Crack patterns for Specimen W3 (Zhao 2004)

4.4 Repair and strengthening techniques

4.4.1 Repair process

All tested specimens underwent the same repair process. The walls are first visually inspected for cracks and loose concrete. All loose/spalled concrete was removed using chisel and hammer until sound concrete was reached. For the areas that had depth of concrete removed not exceeding 50mm, patch repair using bonding agent and polymer modified cementitious mortar, was done in layers not exceeding 20mm to reinstate the affected portion. Plantitop G40SP. It had a rated compressive strength of more than 40 MPa and a flexural strength of more than 7 MPa after 28 days. The shrinkage was less than 0.1 % and had a volume change +2.0 %. For areas where depth of concrete removed exceeded 50mm, pre-packed aggregate grouting was used. This was done by pre-packing 20mm diameter aggregates into grout tight form work and pressurized grouting cement treated with super plasticizer and grout fluidifier to achieve a high strength and flowable grout. The cracks are filled by injecting low viscosity epoxy resin into nipples attached on the wall surface at approximately 300 mm centre to centre along the cracks. This was done in a sequential manner along the crack length such that the epoxy resin filled the crack completely. Sto epoxy crack sealer was used to seal the remaining parts of the cracks during epoxy injection. The specimens were then left to cure for a day as the epoxy resin is specially formulated to harden for 24 hours and this will allow time for the epoxy resin to penetrate into the cracks. The epoxy used for repairing the cracks was known as StoJet IH-BV. It was a two-component solvent-free, non-pigmented epoxy resin compound with low viscosity. It had a tensile strength of 74 MPa and Young's modulus of 3170 MPa. However, after curing, it had a rated compressive strength of 65 MPa, slant shear bond strength of 16.8 MPa, flexural strength of 40 MPa, Young's modulus of 2400 MPa and an ultimate strain of 4.0 %

4.4.2 Strengthening techniques

The approach to strengthening of walls without openings differs from that of walls with openings. The former consists of applying FRP strips and sheets strategically based on failure modes of as-built specimens. The latter applies FRP strips according to the load magnitudes and paths in the strut-and-tie models developed.

4.4.2.1 Walls without openings

4.4.2.1.1 Proposed FRP wrapping scheme

Figure 4-15 and Figure 4-16 illustrate the FRP strengthening scheme for Specimens RLW2, RLW3, RMW1 and RMW2. Subsequent to repair, specimens were strengthened with FRP material. Before the application of FRP sheets and strips, the specimens were carefully prepared by grinding of several areas to achieve a fully smooth surface and rounding of corners (at a radius of about 20mm) mm to avoid FRP cracks due to local failure and stress concentration (Clyde and Pantelides 2002).

The FRP sheets and strips were then applied on the specimens in the following sequence:

- Application of GFRP strips at the base of the flanges were bent at 90° bent and thereafter extended 300 mm in length to be anchored for Specimens RLW2 and RMW1. Specimens RLW3 and RMW2 used CFRP strips for similar application.
- Application of a GFRP layer at each flange with fibers oriented in the vertical direction to increase flexural strength of the walls for Specimens RLW2 and RMW1. Specimens RLW3 and RMW2 used a CFRP layer for similar application.
- Application of two layers of GFRP U-shaped jacket with fibers oriented in a horizontal direction to confine the flanges of all specimens.
- Application of a GFRP sheet on the main face of the wall with fibers oriented in a vertical direction to increase flexural strength for Specimens RLW2 and RMW1. Specimen RMW2 used a CFRP sheet in a similar fashion. However, for Specimen RLW3, it used two CFRP strips to form a cross bracing on the main face of the wall.
- Application of a GFRP layer to the main face to increase the strength and cover the ends of the previous jackets at the flanges.

Figure 4-15: FRP strengthening schemes for low-rise Specimens RLW2 and RLW3

Figure 4-16: FRP strengthening schemes for medium-rise Specimens RMW1 and RMW2

GFRP anchors were placed at various locations along the development length of the sheets. The anchor consisted of a bundle of main fibers properly formed to size, and fully impregnated with epoxy before insertion to the drilled holes in the wall. The

drilled holes were cleaned and kept free of dust. The embedded length of the anchor from the surface of the wall had a minimum length of 50mm.

4.4.2.2 Walls with openings

4.4.2.2.1 Strut-and-tie models

The strut-and-tie approach modeled as-built Specimens S-F2 and W3 as a truss structure. When subjected to the imposed lateral load, the complex stress fields inside the wall specimens were simplified into discrete compressive and tensile force paths, which were represented, respectively, by compression struts and tension ties. Figure 4-17 shows the strut-and-tie models of as-built Specimens S-F2 and W3. For as-built Specimen S-F2, the loads and load paths were worked out in both positive and negative cycles. For as-built Specimen W3, on each side of the wall face, the loads were resolved into vertical and horizontal forces for half of the wall.

As-built Specimen SF-2

As-built Specimen W3

Figure 4-17: Strut-and-tie models of Specimens SF-2 and W3

The compression struts, shown as dotted lines, were diagonal with different inclination angles and designed to transfer the imposed lateral load. The tension ties, shown in full lines, were all vertical and horizontal to represent full lines represent the tension ties while the dotted lines represent the orthogonal reinforcement layout of the specimens. Based on the developed strut-and-tie models, the widths of the FRP strips were calculated according to the load magnitude and were placed in the direction and location of the load paths.

The following assumptions were made in the development of strut-and-tie model of Specimen S-F2: 1) The beam and column zones were subjected to axial tension when the whole zones were under tension in the load paths; 2) all reinforcement in the beam

or column zones was lumped into one tie where its position was located in the centriodal axis of the reinforcement lumped into it and 3) the strut position was determined by keeping the concrete compressive stress lower than the strength limitations suggested by (Schlaich *et al.* 1987); $0.68 f'_c$ for a strut with cracks parallel to it or $0.51 f'_c$ for strut with skew cracks. The strut was idealized into a prismatic or uniformly tapered shape according to the geometry of the nodal zones while the geometries of the struts and nodes were determined based on the divisions of the beam zones and column zones and the position of the openings.

For Specimen W3, the area of each diagonal strut is half of the available area along its transverse direction while each tie had the area of 3T10 bars. The constitutive relationship of concrete was applied to the struts. $0.72 f'_c$ was taken for vertical and horizontal struts and $0.68 f'_c$ was taken for diagonal struts. To assess if resolving the compression struts into vertical and horizontal components assist in the FRP strengthening, approximately half of each side of the wall faces had diagonal compression struts while the other half had resolved vertical and horizontal forces.

4.4.2.2.2 Proposed FRP wrapping scheme

Based on the strut-and-tie models developed for Specimens S-F2 and W3, the FRP strips were calculated according to the load magnitude and were placed in the direction and location of the load paths. More than one layer of FRP strip may be used depending on load magnitude and space constraint. For Specimen S-F2, each wall face was strengthened for both positive and negative cycles. The repaired and strengthened wall specimens are denoted RS-F2 and RW3. Figure 4-18 and Figure 4-19 show the strengthening schemes of Specimens RS-F2 and RW3 respectively. All compressive strengthening used Tyfo®SEH-51A Composite system. All tensile strengthening, on the other hand, used Tyfo®SCH-41 Composite system. The Tyfo® Composite Systems were described in the previous section. To enhance the shear and sliding capacities of the walls and to prevent early debonding of the FRP strips, the FRP strips were anchored using GFRP fiber anchors.

Compressive strengthening: Tyfo®SEH-51A Composite system

Tensile strengthening: Tyfo®SCH-41 Composite system

Figure 4-18: Proposed FRP strengthening schemes for Specimen RS-F2

Compressive strengthening: diagonal alignment

Compressive strengthening: vertical and horizontal alignment

Figure 4-19: Proposed FRP strengthening schemes for Specimen RW3

Before the application of the FRP strips, similar to that of walls without openings, the wall surface were cleaned and sharp edges were removed by mechanical grinding. Any depressions were filled with polymer modified cementitious grout to achieve an even surface. Corners of the wall were rounded by mechanical grinding to a minimum radius of 20 mm. The wall surface was kept dry and the FRP strips impregnated with specially modified epoxy were applied. Upon completing the strengthening schemes, Specimens RS-F2 and RW3 were left to cure at ambient temperatures. Finishes were applied on the composite surface after 24 hours of curing.

4.5 Results and Discussion

4.5.1 Walls without openings

4.5.1.1 Failure modes and cyclic response to lateral loading

Crack patterns and failure modes for all specimens were generally similar. In all specimens, no visible cracks were observed on the wall at drift ratios ranging from 0.05% to 0.17% compared to the observed flexural cracks at the lower part of the original specimens. Up till drift ratio 1%, the main face of the web and flanges extending up from the base remained intact with no visible signs of cracking and debonding of FRP sheets. As drift ratio increased from 0.25%, the anchor strips under the flanges showed debonding at a greater degree. For Specimen RLW2, flange failure was observed at base region. The delamination of GFRP and CFRP sheets initiated at drift ratio 0.25% for Specimens RMW1 and RMW2 respectively. Compared to the as-built specimens, no spalling of concrete cover at wall base of all repaired specimens except shear cracking of concrete was observed in Specimen RMW1. In all specimens, the final failure involved severe cracking along the base of the main face of the wall and destruction of anchor conditions of the FRP strips that were used to increase flexural strength. Clearly, the failure mechanisms were flexural failure under the flanges and sliding failure along the wall base which similarly occurred in the original specimens.

Figure 4-20 shows the lateral load versus top displacement relationship of low-rise as-built and repaired. In all specimens, pinching was observed after drift 0.67% and became significant for subsequent cycles. At drift ratio 0.67%, the envelope of the hysteresis loops was nearly horizontal, indicating that the peak strength of the walls was reached. After this stage, the slope of the hysteresis loops showed to be negative, indicating strength and stiffness degradation of the specimens. Table 4-2 summarizes the maximum strength at corresponding cycle attained by each wall for negative and positive loadings.

Specimens LW2 and RLW2

Specimens LW3 and RLW3

Figure 4-20: Lateral load versus top displacement relationship of low-rise as-built and repaired specimens

Table 4-1: Maximum strength of low-rise as-built and repaired specimens

Specimen	As-built		Repaired	
	Negative loading (kN)	Positive loading (kN)	Negative loading (kN)	Positive loading (kN)
RLW2	567	594	440	660
RLW3	552	524	554	559
RMW1	422	424	474	432
RMW2	403	410	484	455

For the low-rise walls, Specimen RLW2 recovered almost 80% of its original strength in the negative loading cycle direction and was slightly stronger than the original in the positive loading cycle. An attainment of a higher maximum strength in the positive loading than in the negative loading also indicated that the right side of the wall was much stronger than the left side. Specimen RLW3, on the other hand, managed to recover its original strength in both the negative and positive loading cycle directions with an attainment of similar maximum strength in the respective loading cycles. This showed that both sides of Specimen RLW3 were similar in strength in withstanding lateral loading.

Figure 4-21 shows the lateral load versus top displacement relationship of the as-built and repaired specimens For medium-rise walls, Specimens RMW1 and RMW2 also managed to recover almost all of its original strength. However, after reaching their peak respectively, it was observed that both repaired specimens suffered a more rapid drop in strength compared to the original walls.

Specimens MW1 and RMW1

Specimens MW2 and RMW2

Figure 4-21: Lateral load versus top displacement relationship of medium-rise as-built and repaired specimens

4.5.1.2 Stiffness degradation

Figure 4-22 shows the stiffness curves for the wall specimens at the peaks of their loading cycles. For the low-rise walls, one major observation was that Specimens RLW2 and RLW3 were not as stiff as the original walls, especially in the initial cycles. It was noteworthy that at the higher drift ratios, the repaired walls showed less stiffness degradation than the original walls. This implied that the repaired walls were more ductile than the original ones. For the medium-rise walls, the initial stiffness of Specimens RMW1 and RMW2 was much below that of the as-built specimens. It was also observed that the stiffness of Specimen RMW1 increases slightly at drift ratio 0.5. At the later stage, the stiffness of all the walls was about the same.

Figure 4-22: Comparison of secant stiffness of as-built and repaired specimens

4.5.1.3 Components of top deformation

An analysis was carried out to assess the contribution of deformation components. The top deformation of the walls in this testing was mainly caused by flexural, shear and sliding displacement. Figure 4-23 to Figure 4-26 show the ratios of three displacement components to the total deformation with respect to drift ratios were plotted. The relative contribution of three components vary in terms of the drift ratios with the observed behavior that with increasing lateral displacements up to drift ratio 1%, the contributions of flexural deformations increased while the ratios of the other two components rose slightly. The flexural deformation had about a 30% increase in both Specimens RLW2 and RLW3 whereas for Specimen RMW1, it started to decrease after drift ratio 0.67% and there was no significant increase for Specimen RMW2. In general, similar to the original specimens, the repaired specimens had the flexural deformations dominated the response because it accounted for more than 50% on average of the total displacement.

Figure 4-23: Deformation components for Specimens LW2 and RLW2

Figure 4-24: Deformation components for Specimens LW3 and RLW3

Figure 4-25: Deformation components for Specimens MW1 and RMW1

Figure 4-26: Deformation components for Specimens MW2 and RMW2

Following flexural deformation, the next dominating component was the shear deformation for all as-built specimens. It accounted for about 20%. It was otherwise for the repaired specimens. The shear component was only significant for Specimen RMW1; it was more pronounced at drift ratio 0.5% where it peaked and then decreased significantly.

The sliding shear component accounted for 10% or less in all original specimens. Its contribution increased significantly in Specimens RLW2 and RLW3 but showed otherwise in Specimens RMW1 and RMW2 where the sliding shear component was almost negligible.

4.5.1.4 Curvature distribution along wall height

Figure 4-27 to Figure 4-30 show the wall curvature distribution along the wall height for all specimens. In all repaired specimens, the curvature of the region above 500 mm from the wall base did not have much change. For Specimens RLW2 and RLW3, it was observed that the curvature was highly concentrated at the bottom region. In the negative and positive loading directions, the bottom curvatures of both original and repaired specimens were observed to be close with respect to certain drift ratios and increased significantly after attaining drift ratio of approximately 0.33 %. The curvature distribution for Specimens MW2 and RMW2 was an exception. Above 500mm, the curvature for Specimen MW2 was large as compared to a concentrated distribution for Specimen RMW2.

Figure 4-27: Wall curvature distribution along the wall height for Specimens LW2 and RLW2

Figure 4-28: Wall curvature distribution along the wall height for Specimens LW3 and RLW3

Figure 4-29: Wall curvature distribution along the wall height for Specimens MW1 and RMW1

Figure 4-30: Wall curvature distribution along the wall height for Specimens MW2 and RMW2

4.5.2 Walls with openings

4.5.2.1 Failure modes and cyclic response to lateral loading

For Specimen RS-F2, no visible cracks or debonding were observed during the initial cycles of the test. At drift ratio of 0.25 %, hairline cracking distributed along the height of wall; its presence was inferred from the thin marks in the epoxy resin at the exterior surface of the FRP sheets. At drift ratio of 0.33 %, the breaking of epoxy resin was heard and a horizontal flexural crack was formed at the right base of the wall, at the interface of repair mortar and the anchor block. Subsequently, the crack opened substantially. It was clear evidence that most inelastic behavior occurred at this location. During the first cycle at drift ratio of 0.67 %, the GFRP sheet at the right base of the wall delaminated at the edges exposing the concrete underneath. Horizontal flexure cracks also started to form at the center of the wall base near the opening. In the second cycle, the GFRP sheet at the left base of the wall debonded and warped in the direction away from the loading arm. As the drift ratio increased, loud cracking and debonding of FRP sheets were heard. At drift ratio of 0.1 %, the GFRP sheet delaminated completely from the base. The FRP sheet at the right base of the

wall was now delaminated completely with concrete cover beneath. It was obvious that Specimen RS-F2 had a flexural failure which concentrated between the base and the foundation. Figure 4-31 illustrates a portion of the damaged state of Specimen RS-F2.

Figure 4-31: Damaged state of Specimen RS-F2

Figure 4-32 shows the comparison of lateral load versus top displacement relationship of as-built Specimen S-F2 and repaired Specimen RS-F2. At this stage, the hysteretic response at drift ratio of 0.33 % to 0.50 % was nearly horizontal and it corresponded to a relatively small increase in displacement. Specimen RS-F2 performed satisfactorily with strength recovered reasonably until drift ratios of 0.67 % and 1.0 % in the positive and negative cycles respectively, where strength degradation started to be more drastic. The maximum strength was reached at drift ratio of 0.50 %; in the positive and negative direction for the original specimen was 384.9 kN and 394.3 kN respectively, while that of the repaired specimen was 410.7 kN and 301.5 kN. The drop in strength at peak is much more gradual in the original specimen than the repaired specimen which dropped to more than 50 % of the peak value. A substantial level of pinching was also evident as a result of sliding of the wall along the crack that formed at the wall to foundation interface.

Figure 4-32: Comparison of lateral load versus top displacement relationship of Specimens S-F2 and RS-F2

For Specimen RW3, breaking of epoxy resin was heard at drift ratio of 0.25%. At first cycle of drift ratio of 0.50%, visible epoxy resin cracks were observed at the base edge of the wall. Cracks started to appear at wall base at drift ratio of 0.67%. At drift ratio of 1.00%, diagonal cracks on the wall were observed and cracks concentrated at the lower part of the wall surface. As drift ratio increased, FRP strip debonded from wall to the base and a vertical crack propagated from the base edge upwards; it had a crack length of 15cm. When the maximum strength was reached, the whole wall tilted forward in the out-of-place direction while being restrained by the loading arm. Figure 4-33 illustrates a portion of the damaged state of Specimen RW3.

Figure 4-33: Damaged state of Specimen RW3

Figure 4-34 shows the comparison of lateral load versus top displacement relationship of as-built Specimen W3 and repaired Specimen RW3. Specimen RW3 improved in strength significantly at every drift ratio compared to before repair state. The maximum peak value in each positive and negative cycle for the virgin Specimen W3 were 334 kN and -348.3 kN respectively while that of repaired Specimen RW3 was 455.6 kN and -410.71 kN respectively. For both positive and negative cycles, the repaired wall has a significant gain in strength as compare to the virgin wall. Although the repaired wall can reach a higher value, the drop in strength is more drastic as compare to the controlled wall. The maximum peak value of the positive direction is greater than the negative direction. This might imply that it was FRP strips placed directly on the critical diagonal stress path was more effective than on the vertical and horizontal alignments.

Figure 4-34: Comparison of lateral load versus top displacement relationship of Specimens W3 and RW3

4.5.2.2 Stiffness degradation

The stiffness of both virgin and repaired walls was estimated on the basis of its secant value; at each ductility factor or drift ratio, the larger value of lateral load chosen versus corresponding displacement. Figure 4-35 and Figure 4-36 show the comparisons of stiffness degradation for each tested specimen. From Figure 4-35, Specimen RS-F2 could not restore its stiffness at every stage while that of Specimen RW3 (see Figure 4-36), showed reasonable restoration in stiffness. Specimen RS-F2 recovered about 60 % of its stiffness and towards the last few cycles, its stiffness was poor compared to virgin Specimen S-F2. Specimen RW3 behaved in a similar trend as its virgin Specimen W3. For both repaired specimens, the stiffness degradation was relatively more gradual which was desirable in an earthquake.

Figure 4-35: Comparisons of secant stiffness of Specimens S-F2 and RS-F2

Figure 4-36: Comparisons of secant stiffness of Specimens W3 and RW3

4.5.2.3 Energy dissipation capacity

The energy dissipated was based on the cumulative energy dissipated calculated by summing up the energy dissipated in consecutive loops throughout the test. Figure 4-37 and Figure 4-38 show the comparison of cumulated energy dissipated of as-built and repaired specimens. The repaired specimens not only restored the energy dissipation capacity of the virgin specimens, its energy dissipation capacity increased

by more than two folds. For Specimen RS-F2, after drift ratio of 0.4 %, the energy dissipated was more than doubled that of the as-built specimens and its energy gradient was steeper after drift ratio of 0.6 % which occurred earlier at drift ratio of 1 % for the as-built Specimen S-F2. At every drift ratio for Specimen RW3, it had very significant improvement in the energy dissipated compared to Specimen W3. At final stage, Specimen RW3 reached maximum energy dissipated of about 280 kNm while that for Specimen W3 reached 60 kNm. The energy gradient was not consistent for Specimen RW3; it was the steepest towards drift ratio of 4 % and started to become gentler before becoming steeper again. As-built Specimen W3, on the other hand, had consistent energy gradient at every drift ratio.

Figure 4-37: Comparison of cumulated energy dissipated of Specimens S-F2 and RS-F2

Figure 4-38: Comparison of cumulated energy dissipated of Specimens W3 and RW3

4.5.2.4 FRP strains

The readings in the strain gauges attached to the FRP strips are shown in Figure 4-39 and Figure 4-40 for Specimen RS-F2 and RW3 respectively. For Specimen RS-F2, most of the strain gauges attached to the FRP strips recorded considerable low readings indicating that the strips were not fully utilized. However, strain gauges S9 and S10 recorded high readings indicating the strips placed at that location were utilized. The strains measured were as high as 0.00135 mm/mm at maximum loading for strain gauge S9 and 0.06 mm/mm after failure for strain gauge S10 at final drift ratio. This indicated that the fibers at strain gauge S10 location exceeded the allowable value of 1.0 %, implying that CFRP strips placed along the compression strut path might be effective. The strain in strain gauge S19, which was located near the wall base, was unexpected to read low compressive strains as the bottom panel was subjected to large shear force. This indicated that the FRP anchorage near the wall base was effective increasing the shear and sliding capacity of the wall. After the maximum load, the strains in strain gauges S3 and S9 decreased quite drastically compared to other strain gauges. This was because they experienced both compressive and tensile forces considerably at that location. For Specimen RW3, all strain readings fell below 0.00035 which was far from the allowable value of 1.0 %. This indicated

that the GFRP and CFRP strips were not well utilized. The more obvious strain readings were recorded in strain gauges S3, S4, S20 and S25.

Specimen RS-F2: Location of strain gauges

FRP strains: S1 to S9

FRP strains: S11 to S19

FRP strain: S10

Figure 4-39: FRP strains on Specimen RS-F2

Specimen RW3: Location of strain gauges

FRP strains: S1 to S13

Figure 4-40: FRP strains on Specimen RW3

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5 Conclusion and Recommendations

5.1 Beam-Column Joints

5.1.1 General

The behavior of reinforced concrete interior beam-wide column joints and beam-wall joints repaired and strengthened with GFRP and CFRP strips and sheets has been studied to evaluate the effectiveness of FRP composites in: 1) Restoring the seismic performance of damaged interior beam-wide column joints and beam-wall joints and 2) enhancing the seismic performance of non-damaged interior beam-wide column joints and beam-wall joints. All nine specimens are non-seismically detailed and typical joints found in Singapore. The seismic behavior of the specimens before and after repair and strengthening were compared and studied.

5.1.2 Repaired specimens

In the first experimental study, a repair and strengthening technique was proposed and applied. The residual seismic performance of FRP strengthened beam-wide column joints and beam-wall joints after limited seismic damage was investigated. The following observations and conclusions can be drawn:

- All repaired specimens failed commonly with flexure and shear cracks on the beams due to concrete deterioration and bar slippage when the beams underwent tension and compression and debonding on column faces near beam-column joint interfaces; debonding initiates at flexure-shear cracks and propagate towards end of the beam and delamination of FRP sheets near the beam-column joint interface due to high stress concentrations at beam ends.
- The repair technique effectively delayed the failure of all retrofitted repaired specimens at higher drift ratios compared to the as-built specimens; at drift ratio of 4%, there was no significant decrease in the load carrying capacity of all retrofitted repaired specimens. All repair specimens, except Specimens RC2A and RC2B at later drift ratios, were able to recover 100% strength and reached higher strength at every drift ratio.

- Each repaired specimen exhibited more ductile response than the as-built specimens, reaching at least drift ratio of 3% without a reduction in load carrying capacity. Also, all retrofitted repaired specimens, except Specimens RC1B and RC2A, were able to restore the energy dissipation capacity. Specimen RE1A and RC1A, in particular, exhibited more energy dissipation capacity than their respective as-built specimens.
- In terms of secant stiffness, the initial stiffness of Specimens RE1B, RC1A and RC2A were lower than their as-built specimens. This could be due to initial damage and yielding of steel bars. Nonetheless, all repaired specimens restored their stiffness and exhibited similar trend in stiffness degradation as the as-built specimens except Specimen RC2A which had severely yielded steel bars at top beam at the beam-column interface.
- Similar to most previous studies, the research reconfirmed that the repair of beam-column joints with FRP wrapped round the column provided confinement and additional shear reinforcement to the joints ensuring a flexural dominant mode when transverse reinforcement is not present in the joint core. Joint strength and ductility are significantly improved and concrete integrity maintained.
- The effect of higher constant axial column load $0.35 f'_c A_g$ was marginal. A lower load of $0.1 f'_c A_g$ could be sufficient to enhance the performance of the specimens in this study. The effect of eccentricity was not significant as-built Specimens E1A, E1B, C1A and C1B when the maximum load achieved, stiffness degradation and energy dissipation capacity were similar. Therefore, the effect of eccentricity does not play a role in the repair works.
- The use of fiber anchors in preventing the delamination of FRP sheets was effective in enhancing the capacity of the FRP sheets. Fiber anchors could be considered to be applied near the top beam-column joint interface.

In summary, the retrofit rapid repair technique, which was characterized with the use of: 1) epoxy injection and polymer cementitious mortar to seal the cracks and replace

spalled concrete and 2) wet lay-up FRP application using Tyfo® SEH-51A Composite and Tyfo® SCH-41 Composite systems in the proposed FRP configurations presented herein seems to be appropriate for the specimens tested in this investigation. The repaired specimens failed at higher drift ratios compared to as-built specimens and displayed reasonable restoration in terms of strength, drift capacity, energy dissipation capacity and stiffness degradation. However, only Specimen RC2A did not perform well in terms of stiffness restoration. This was because at the start of the experiment, the specimen was given a relatively larger displacement, causing it to have a relatively greater loss of stiffness. The use of FRP retrofit repair technique should be explored for seismic repair of damaged reinforced concrete frames to understand its effect on the global behavior of the frames.

5.1.3 Strengthened specimens

In the second experimental study, an economical FRP strengthening schemes were proposed and applied on virgin beam-wide column joints and beam-wall joints and their enhanced seismic performance was investigated. The following observations and conclusions can be drawn:

- A comparison between the performance of as-built specimens and strengthened ones shows a tremendous increase in strength, stiffness and energy dissipation capacity.
- The use of two layers of CFRP strips at the joint and beam area was effective in strengthening eccentric joint.
- The use of CFRP strips on strong-column weak-beam was effective in flexural strength: CFRP strips round the column and CFRP strips as cross bracings on the beam and column face.
- Good anchorage in the form of fiber anchors was effective in anchoring FRP sheets and strips; they contributed much to the shear strengthening of beam-column joint and beam. Such anchorages also ease constructability on site.

The proposed strengthening schemes were successful in eliminating or delaying the shear mode of failure. Instead, flexural hinging of the beam, a ductile mode of failure, occurred in the form of cracks near the joint core corners for Specimen SE1B and delamination of FRP strips at beam-column interface near joint core region for Specimens SC1B and SC2B. Since the behavior of beam-column joints is complex and still not completely understood. Thus, adopting a direct extension of the FRP strengthening strategies for beams and columns on beam-column joints would be difficult. The proposed strengthening schemes will help to develop FRP strengthening strategy for beam-column joints and are potentially efficient for mass repair or upgrading of structures not suitably designed to withstand earthquakes.

5.1.4 Recommendations

- To develop the strength of the fiber, it is recommended that the anchorages be installed at beam bottom near the beam-column interface and at the edges of FRP strips and sheets near the joint core. On the other hand, it is noted that beam jacketing in the form of FRP U-wrapping, which was absent in Specimen SC2B, is necessary in preventing shear failure in the beam and allowed a flexural hinge to develop.
- More tests are required for verification. Many aspects of the problem remain to be studied includes more effective bond methods of using fiber anchors. The economical FRP wrapping schemes which were applied in the second experimental study, can be applied to previously damaged specimens in the first experimental study to evaluate the effectiveness of the wrapping schemes in restoring the seismic performance of beam-wide column joints and beam-wall joints after seismic damage.
- Analytical studies can be carried out to validate the effectiveness of the FRP wrapping schemes applied on all specimens.

5.2 Structural Walls

5.2.1 Walls without openings

In this experimental study, effective repair and strengthening techniques were developed and applied on RC structural walls with boundary elements confined with limited transverse reinforcement. Compared to the as-built specimens, useful conclusions could be drawn relating to the behavior of repaired and FRP-strengthened RC structural walls subjected to seismic loading:

- Flexural strength and stiffness could be increased by the use of GFRP and CFRP sheets. The use of CFRP was more effective than GFRP in recovering the strength as shown by the better performance of RMW2 compared to RMW1. Additional use of CFRP laminate strips on RLW3 was effective in restoring the original strength and ductility of LW3.
- By using FRP jackets, the shear strength of the repaired walls increased and shear cracking was effectively controlled by much reduced shear displacement. However, sliding shear cracks could not be prevented from propagating at the base.
- The use of a GFRP sheet to confine the ends of the FRP jackets assisted in preventing the debonding of the jackets.
- Hysteretic behavior of RLW2 was poorer than that of LW2, mainly because inelastic deformation concentrates at the critical crack formed at the interface of the repair mortar and the flange base.
- Stiffness of the repaired specimens was on average higher than that of original specimens at higher drift ratios.
- The L-shaped anchor strips were effective as there was no debonding observed on the main body of the wall in all repaired specimens.
- However, debonding of the L-shaped strips occurred at the base of the wall. Anchoring of FRP strips at critical regions (such as the base of walls) was far from easy, and the anchorage zones are susceptible to failure under tension.

5.2.2 Walls with openings

In this experimental study, two medium-rise RC structural walls with irregular and regular distributed openings, initially subjected to cyclic lateral loading to failure, were repaired and subsequently strengthened by externally bonded GFRP and CFRP strips which were designed and applied according to the load magnitudes and paths in the proposed strut-and-tie models. Compared to the as-built specimens, useful conclusions could be drawn relating to the behavior of repaired and FRP-strengthened RC structural walls subjected to seismic loading:

- Repaired walls recovered their strength, stiffness and energy dissipated reasonably well and failed mainly with FRP debonding and epoxy resin failure at wall base. In particular the restoration of Specimen RS-F2, in terms of stiffness and energy dissipated was good.
- Shear and sliding capacities of the repaired walls were enhanced using fiber anchors. The fibers had successfully arrested the widening of shear cracks which reduces debonding and deterioration of the fibers. However, most of the FRP strain readings indicated that the FRP strips were not fully utilized
- The fiber anchors were generally effective in improving the sliding capacity of the walls as few areas on the wall base showed failure in anchorage. The disadvantage of this technique was it could be time consuming in applying the FRP strips along the desired compressive and tensile paths.

The proposed FRP strengthening scheme using the strut-and-tie approach is an effective technique in restoring the strength of the damaged walls with irregular and regular distributed openings. The amount of FRP strips could be used efficiently according to the degree of stress experienced at certain regions of the wall and save FRP material when mass upgrading of structures was concerned.

5.2.3 Recommendations

- More experimental work is required for the same strengthening schemes, in particular, using the strut-and-tie model approach to study and further clarify the effect of FRP strips applied according to the load magnitudes and paths.
- Difficulties were encountered in measuring the FRP strain gauge readings on repaired walls with openings. The positions of the FRP strain gauges can be explored, for example, placing more FRP strain gauges at critical regions to capture the strains.
- Increasing the anchorage length along with plates or angles which would help in preventing early peeling off.