**Repair and Strengthening of Reinforced Concrete Shear Walls
for Earthquake Resistance Using Externally Bonded Carbon
Fibre Sheets and a Novel Anchor System**

By

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A thesis submitted to the Faculty of Graduate Studies and Research
in partial fulfillment of the requirements for the degree of

Master of Applied Science

Department of Civil and Environmental Engineering

Carleton University

Ottawa, Ontario, Canada

May 2004

The Master of Applied Science in Civil Engineering is a joint program with the
University of Ottawa, administered by the Ottawa-Carleton Institute for Civil
Engineering

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Abstract

Four 2/3-scale reinforced concrete walls were constructed and tested to evaluate the effectiveness of the method of using externally epoxy bonded Carbon Fiber Reinforced Polymer Sheets (CFRP) for seismic retrofit and repair of reinforced concrete shear walls. The present experimental study is the second phase continuation of an earlier study at Carleton University on the feasibility of the strengthening and repair method of reinforced concrete shear walls by CFRP tow sheets. In the first phase of the research, out of plane bending behaviour of the test walls was observed, which had some effects on the ultimate lateral load capacities of the test specimens. In the present study, the out of plane bending behaviour of the test specimens is greatly reduced or eliminated by improvements to the experimental setup. Another major contribution of the present study is the development of a new anchoring method of the FRP sheet for the transfer of the tensile load carried by the FRP sheet bonded to the strengthened or repaired wall to the supporting structural foundation. The new anchor system eliminates the eccentricity problem observed with the L-shape angle anchor used in the first phase of the research program.

The experiments carried out in the present study include the cyclic load testing of a control wall, a repaired wall and three strengthened walls. The wall specimens were loaded until ultimate failure, defined as the softening behaviour of reducing by 80% of the maximum lateral load resistance capacity after reaching the peak load.

The results of the experimental study show that the application of CFRP sheets can effectively recover the initial elastic stiffness and strength of damaged walls, and can also significantly increase the yield and ultimate loads of both repaired and retrofitted walls. The experimental results demonstrated that the new anchor system performs very well. It provides effective and reliable anchor to the CFRP sheets for load transfer. Because of its inherent concentric characteristic in its behaviour, the new anchor system allows full utilization of the high ultimate tensile load capacity of CFRP sheets without premature failure in resisting the imposed seismic lateral load, thus permitting a more efficient use of the high cost FRP reinforcing materials.

Acknowledgements

I would like to express my gratitude to my supervisor Prof. D. T. Lau for his guidance and support throughout this study. I would also like to acknowledge the donation of the CFRP reinforcing materials and technical assistance to the research program provided by Dr. Andrew Lam and Mr. Yap Chan Tiong of Brighten Engineering Co. Ltd (Taiwan). Many thanks to my fellow students and especially Nicolas Londoño for their invaluable contribution, and physical and moral support. They made it possible for me to complete this work.

The contribution of Public Works and Government Services Canada, in phase one of this study is gratefully acknowledged.

Last but not least I want to thank everyone in the Engineering Lab for their unwavering patience, assistance, and support.

Special thanks go to Ekaterina for her assistance, support, patience, and encouragement throughout my studies.

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Nomenclature

A_c	Effective area
α_f	Angle between shear friction reinforcement and shear plane
A_g	Gross area of the wall cross section
A_h	Cross sectional area of one horizontal reinforcing bar
A_s	Area of flexural reinforcing bars
A_{vf}	Area of shear friction reinforcement
A_{cv}	Area of concrete section resisting shear transfer.
α_1	Stress block parameter
α_c	Aspect ratio factor
b	Thickness of wall
β_1	Stress block parameter
c	Depth of neutral axis
C_c	Compressive force in concrete
C_s	Compressive force in steel reinforcement
d	Effective shear depth = 0.8 L
δ_f	Flexural component of total deflections
δ_{sh}	Shear component of total deflections
δ_{Total}	Total top horizontal deflection
δ_{sld}	Sliding top horizontal deflection
E_c	Elastic tensile modulus
E_{CFRP}	Elastic tensile modulus of the carbon fibre sheets

E_s	Steel reinforcement's elastic modulus
ϵ_c	Strain in concrete
ϵ_{cu}	Ultimate compressive strain
ϵ_{Fmax}	Maximum strain in carbon fibre sheets
ϵ_o	Strain in concrete at peak compressive stress
ϵ_s	Strain in steel reinforcement
ϕ	Rotation angle at the top of the wall
f_c	Compressive stress in concrete
f'_c	Specified compressive strength of concrete
f''_c	Effective compressive strength = $0.9 f'_c$
f_F	Average tensile stress in carbon fibre sheets
f_r	Modulus of rupture of concrete
h	Height of wall
λ	Factor to account for low density concrete
L	Length\width of wall
L_e	Length of equilibrium zone
M	Bending moment
M_r	Ultimate flexural capacity
μ	Coefficient of friction
N_{uxy}	Poisson's ratio (Anchor finite elements analysis)
n	Number of horizontal reinforcing bars per layer
n_F	Number of horizontal FRP sheets
P	Applied load

ρ_v	Ratio of shear friction reinforcement
ρ_{FRP}	Ratio of FRP reinforcement
σ	Effective normal stress
s	Spacing of transverse reinforcement
T_{FRP}	Tensile forces in CFRP Sheets
T_s	Tensile force in steel reinforcement
t_{FRP}	Thickness of FRP sheets per side
θ	Angle of the critical inclined flexural shear cracks to the member's longitudinal axis
V_c	Contribution of the concrete to shear capacity
V_F	Contribution of the horizontal carbon fibre sheets to the shear capacity
V_r	Ultimate shear strength
V_s	Contribution of the steel reinforcement to shear capacity
V_{sl}	Sliding shear strength
V_v	Shear force transferred along the vertical flange of the structural steel angles

Chapter 1

Introduction

1.1. General

In the last 50 years, shear wall type construction has been widely used in the construction of new reinforced concrete structures despite the fact that there are considerably more experimental and analytical data on the behaviour of moment resistant frame structures compared to the lack of sufficient shear walls (Fintel, 1995).

Many analytical studies during the 1950s and 1960s led to general observations on the seismic response of reinforced concrete structures to higher seismic forces in more rigid structures, such as those with shear wall lateral load resistant systems, and to observations of brittle response of shear wall structural members subjected to in-plane seismic excitations. The findings of early research on seismic behaviour have led to the belief that shear wall buildings will generally suffer more severe damage during a major earthquake (Fintel, 1995).

More recent research on seismic behaviour and performance has provided more information and data, which show that reinforced concrete structures constructed with shear walls can be designed and detailed to provide reliable and adequate lateral load resistance during severe earthquakes.

An early report on the response and behaviour of shear wall buildings was the earthquake damage study from the Chilean earthquake in May 1960. The report states that “the

Chilean experience confirms the efficiency of concrete shear walls in controlling structural and non structural damage in severe earthquakes. There were instances of cracking of shear walls but this did not affect the overall performance of the building.” The observation was reported that even after damage had occurred the walls continued to function (Fintel 1995).

During the 1963 Scopje (Macedonia) earthquake, it was observed that buildings with a shear wall structural system swayed considerably, but did not suffer any serious or significant damage. It should be noted that during this earthquake 1100 people lost their lives from the collapse of buildings (Fintel 1995).

Observations from the 1967 Caracas earthquake in Venezuela, the 1971 San Fernando earthquake in California, the 1972 Managua earthquake in Nicaragua, the 1977 Bucharest earthquake in Romania, the Mexico City earthquake in 1985, the Chile earthquake in 1985, and the earthquake in Armenia in 1988, show that properly designed shear wall buildings performed in a satisfactory manner by withstanding the earthquake load, suffering no structural damage that would cause collapse and protecting the lives of the occupants in the buildings (Fintel, 1995).

Due to the extensive earthquake activities in Chile, buildings in Chile have frequently experienced strong ground motions of significant intensity. During the 1985 Llo-LLeo earthquake, building structures with structural shear walls suffered only limited damage (Boroschek 2000).

In seismic strengthening or retrofitting, shear walls are often added to existing frame structures as a means to control drift or displacement resulting from earthquake excitations, and to increase the lateral load resistance capacity of the structure. The

method of addition of shear walls has been applied to retrofit a number of historical buildings that sustained damage during the 1989 Loma Prieta earthquake. These buildings suffered significant damage, including in-plane shear fracture of the masonry in fills, damage due to large displacements and pounding of adjacent buildings, severe cracking of interior clay tile partitions, and other damage to the contents if the buildings (Jokerst, 1996). These historical buildings were retrofitted by the addition of cast in place shear walls to reduce seismic response displacements and increase the lateral resistance of the buildings. All this information demonstrates the extensive use of the shear walls in the construction of reinforced concrete buildings and that shear wall buildings have performed satisfactorily during seismic loading.

1.2. Problem Definition

Although properly designed modern shear wall structures have been proven to perform well during earthquakes, there is a large inventory of existing shear wall structures which need remedial work. This can be due to inadequate design, aging of the structure, change in the use of the structure or the increase in the seismic design load requirements because of new seismological data available after the design and construction of the structure.

Observations from earthquake damage studies show that common deficiencies of old shear wall structures include insufficient flexural capacity and ductility due to insufficient flexural reinforcement, and inadequate shear strength, especially in short or squat structural walls, attributed again to the lack of sufficient shear reinforcement (Priestley, 1995). In order to reduce the danger of catastrophic collapse of these old deficient shear

wall buildings during earthquakes, different methods have been developed for the seismic strengthening or retrofit of these structures. In seismic engineering, repair implies the restoration to the original performance level or close to the original level of the damaged structure by design and/or design techniques such as adding reinforcement or lateral restraint elements or systems. On the other hand retrofit or strengthening of existing structures generally refer the enhancement or increase in the performance of undamaged existing structures.

One retrofit method is the addition of steel X-Bracing on a shear wall in an attempt to increase the in-plane stiffness of the whole structure (Galano, 1998). The application of properly designed steel jackets on shear walls is another method developed to increase the lateral resistance of older reinforced concrete structures. Other methods that have been developed to enhance lateral in-plane resistance of existing reinforced concrete structures are the application of cast in place reinforced concrete overlays or shotcrete on existing shear walls.

Since the mid 1980s, the application of FRP plates instead of steel plates as a method for repair of reinforced concrete structural members is considered a reliable alternative to the conventional repair methods. The use of FRP materials as reinforcement in reinforced concrete structures has many advantages compared to the methods that use concrete or steel as the reinforcing material. Fibre reinforced polymer materials are lightweight which do not add to the dead load of the structure. The application of the FRP materials is less disruptive to the operation of the buildings and demands less labour. From an architectural perspective, the application of FRP materials for the retrofit of existing

structural members does not significantly alter the shape or architectural appearance of the structure or structural member.

Extensive research has been carried out in Germany, Switzerland, Saudi Arabia, United States, Canada, and Japan to study the behaviour of reinforced concrete structures with externally bonded FRP reinforcement. Studies in Japan have focused more on research on the application of FRP sheets and plates for retrofit and repair of reinforced concrete structures.

Nanni (1995) presented a paper with information on the state of the art in the application of externally bonded flexible fibre reinforced plastic (FRP) sheets in Japan on actual structures. The following are the applications mentioned in the paper.

- Hata Bridge (1994). In order to accommodate the installation of a larger wind break wall the cantilevered wing slab had to be reinforced. Fibre reinforced plastic sheets were applied on the soffit of the slab so as to provide adequate load carrying capacity.
- Hiyoshikura Bridge Deck (1994). The load carrying capacity of the deck was increased with the application of 2 ply carbon fiber sheets placed in parallel and perpendicular to the road way. A total of 164 m^2 were covered.
- Wakayama Oil Refinery (1994). The steel reinforcement of beams of the waterfront pier had shown extensive signs of corrosion. These beams were repaired with the application of single ply carbon fibre sheets.

- Yoshimo Route Tunnels (1994) 500 m² of the thin concrete lining of the tunnels was covered with double ply carbon fibre sheets so as to repair cracking caused by unexpected fluctuation of the underground water pressure.
- Fujimi Bridge (1993) Parallel and perpendicular direction carbon fibre sheets were applied on the soffit of the culvert to increase the load carrying capacity of the bridge.

1.3. Literature Review

This section presents a literature survey on some of the experimental studies of the application of FRP materials as a method of retrofit and repair of existing structures.

1.3.1. Application of Fibre Reinforced Polymers to Reinforced Concrete Beams

Thanasis Triantafilou (Triantafilou 1998) conducted a series of experiments to study the effectiveness of using carbon fibre reinforced polymer (CFRP) sheets for the shear capacity enhancement of beams. The objectives of the study were to expand the experimental database on CFRP reinforced beams and to develop an analytical model for the application of the CFRP sheets to enhance the shear capacity of beams in relation to the traditional stirrup shear reinforcement.

In this research program, 11 identically reinforced 1000 mm long concrete beams were tested. The beams had only longitudinal steel reinforcement and no shear steel reinforcement. Nine of these beams were reinforced with externally epoxy bonded

carbon fibre laminates or fabrics with different combinations of fibre configuration. The two remaining beams were used as controls without any additional reinforcement.

An effective FRP strain was defined as a function of the product $\rho_{frp}E_{frp}$ to determine the contribution of the FRP reinforcement in the shear load resistance of the specimen. It was observed that the FRP reinforcement provided a positive contribution which linearly increased the shear load carrying capacity of the beam in proportion to the value of $\rho_{frp}E_{frp}$, up to approximately 0.4 GPa. For value higher than this, the contribution of the FRP reinforcement ceased to be positive. This observation is very useful for optimum utilization of the FRP materials.

The results demonstrate the effectiveness of the application of CFRP sheets and laminates as a method of strengthening for reinforced concrete beams. Regarding the orientation of the fibres, it was observed that the effectiveness of the reinforcement increased as the orientation of the fibres tends to be perpendicular to the diagonal shear cracks plane.

Sharif et al., (1994) studied the effectiveness of the application of FRP plates as a method for retrofit and repair of reinforced concrete preloaded beams, and the effect of the FRP plate thickness on the premature failure due to peeling and debonding of the reinforcement plate. Ten reinforced concrete beams (150 x 150 x 1250 mm) were tested. The beams had a steel reinforcement ratio of 0.0098 to ensure under reinforced behaviour under loading. The beams were first loaded with 85% of the ultimate flexural load and then reinforced with 2 or 3 mm plates in different schemes of reinforcement and anchor. Bolts were used to anchor the FRP plate at the two ends, or I jackets were attached at the two ends of the plates, to prevent premature failure of the plates due to debonding.

The experiments demonstrated that the stresses at the plates curtailment increases as the thickness of the plate increases, resulting in a more brittle failure mode due to debonding and concrete spalling and tearing off. The most effective anchoring scheme to prevent debonding was the application of the jackets at the ends of the plates.

It was also demonstrated that the application of the FRP plates as method of retrofit and repair was successful in increasing the load carrying capacity of the beams by up to 55%, while maintaining a ductile failure mode despite the brittleness of the FRP plates.

Alex Li et al. (2000) studied the effect of the shear strengthening of reinforced concrete beams with externally bonded CFRP sheets on the stress distribution, crack occurrence and propagation, and ultimate strength of the beams. The experimental study included the testing of beams 1350 mm long, 200 mm deep and 130 mm wide reinforced, in five different schemes. One beam was strengthened in flexure only i.e. the CFRP plate was bonded only along the flange. The remaining four beams were strengthened in flexure and shear. The specimens were loaded in flexure until failure. The tests demonstrated that the stiffness was increased to a degree proportional to the area of the CFRP sheets applied. The increase in the stiffness was, however, limited. Another observation from this test was that the application of the CFRP sheets along the entire lateral face of the beam was not necessary and that the material could be used more efficiently while still provide the same strengthening results. The experimental results demonstrated a slight advantage for those beams strengthened in both flexure and shear, as compared to those strengthened in flexure alone. This gain was in the range of 11%. Furthermore, it was observed that the application of CFRP sheets on the lateral faces of the beams did not

significantly contribute to the increase of the flexural strength. The crack pattern of the beams was a function of the dimension and the location of the reinforcement sheet. The sheets that were placed on the lateral faces of the beams had a significant contribution in controlling the crack propagation. It was also observed that in order to be effective, the strengthening scheme of the reinforced concrete beam in shear requires the application of flexural reinforcement. This is because the flexural capacity of a non-reinforced beam would be reached before the shear capacity is reached; therefore, the beam would fail in flexure and the shear CFRP reinforcement would not be utilized.

White et al., (2001) worked on an experimental program where reinforced concrete beams were reinforced with CFRP laminates and tested to determine the effectiveness of the application as a strengthening method generally, as well as its effect on the behaviour of the beams under loading at a high rate in particular. Nine beams of 3 metres long were tested. One beam was used as a control beam. The remaining eight beams were strengthened with two different kinds of CFRP laminates. The beams were subjected to four different loading patterns. The range of stroke rates was between 0.0165mm per second to 36mm per second. The beams were subjected to either one or twelve cycles of loading before the application of the high rate loading until failure. This experimental program led to the following conclusions:

- The application of the CFRP laminates on reinforced concrete beams does increase the flexural capacity and stiffness of the beam, but reduces the energy absorption and ductility.

- The number of layers of CFRP reinforcement changes the flexural capacity and stiffness, in combination with the amount and location of steel reinforcement and the anticipated failure mode.
- The comparison between the results from the beams that were preloaded with only one cycle as compared to those loaded with twelve cycles showed that preloading does not adversely affect the flexural capacity of the beams.
- The beams designed with similar failure modes and loaded at high rates demonstrated a 5% increase in the flexural capacity, energy absorption, and stiffness, as compared to similar beams loaded with low rates.

Toutanji and Gomez (1997) conducted a durability study of concrete beams strengthened with externally bonded FRP sheets. The objective of this study was to observe the effect of harsh environmental conditions such as wet and dry cycles using salt water, on the strength of the FRP strengthened concrete beams. The beams were reinforced with four different kinds of FRP sheets (two kinds of carbon fibre, and two kinds of glass fibre) bonded using three different epoxies. Epoxy type 1 was a modified amine-epoxy resin blend; type 2 was polyoxypropylenediamin hardener/epoxy resin; and type 3 was amine saturant/solvent-free epoxy. A total of twenty-four beams were conditioned in two different environments: twelve at room temperature (20 degrees C); and twelve in 300 wet/dry cycles. At the end of the conditioning, the beams were tested to their maximum capacity. The unstrengthened beams that were exposed to wet and dry conditions demonstrated an increase in flexural capacity of approximately 23% as compared to the ones kept at room temperature. This can be attributed to the ideal curing conditions of

the wet and dry exposure. The FRP strengthened specimens demonstrated an increase in their flexural strength as compared to the unstrengthened ones. Comparing the results from the beams that were kept at room temperature with those exposed to wet/dry conditions, the later demonstrated a reduction in the load carrying capacity varying between 2 to 32.5%. Beams where the bonding agent between concrete and FRP sheets was epoxy type 2 demonstrated the highest load carrying capacity both under wet/dry and room temperature, as compared to beams where bonding agent type 1 or 3 was used. Therefore, although the effectiveness of the application of FRP sheets as a strengthening method has been demonstrated, it is important to consider also where this strengthening method is applied. Also, the type of epoxy or bonding agent used plays a significant role and ought to be carefully chosen. During this study, the effect of the environmental conditions on the FRP sheets could not be evaluated since the failure of the beams occurred before the rupture of FRP sheets.

Deniaud and Cheng (2003) tested the interaction of concrete, steel stirrups, and externally bonded FRP sheets and their impact on the shear load carrying capacity of reinforced concrete beams. Four 3-metre long T-beams were fabricated and tested. Three different types of FRPs were used for the strengthening of the web of the T-beams, namely uniaxial carbon fibre, uniaxial glass fibre, and triaxial glass fibre. The load was applied in four-point loading. As a result of the application of the strengthening material, an increase in the maximum shear strength was measured ranging from 15.4% to 42.2%, as compared to the unstrengthened beams.

It was observed that the effectiveness of the FRP strengthening method depends on the amount of the internal shear reinforcement having a lesser benefit for the strengthened structure when the structure is heavily reinforced with internal shear reinforcement. The FRP sheets prolong the plane section behaviour of the beam. It was further observed that the beams strengthened with triaxial fibre reinforcement demonstrate more ductile behaviour as compared to the other strengthened specimens. The use of FRP reinforcement delays the carrying of shear forces by arching action.

Quantrill et al., (1996) performed a series of tests on one metre long reinforced concrete beam specimens with a cross sectional area of 100x100mm. The test beams were reinforced externally with epoxy bonded FRP plates on the soffit. The objective of this research program was to observe the response of the test beams. The response was a function of different parameters, namely the plate area, plate-aspect ratio, plate material, and the method of anchoring the plate ends. The plate material used was glass fibre plates and carbon fibre plates. The plate size ranged from 30x1.2x860 mm to 80x1.2x1000 mm. The method for anchoring the plate ends varied. For example, some plate ends were anchored with steel clamps by placing the plate ends under the support and by applying GFRP angles 100x70x40 mm. The results of this experimental program demonstrated the following:

- The aspect ratio of the plate had little effect on the overall response of the beams;
- Application of CFRP plates as compared to GFRP plates increased the serviceability, yield, and ultimate load of the beam, increasing at the same time the cracking and yielding stiffness;

- GFRP reinforced beams demonstrated more ductile behaviour than their CFRP reinforced counterparts;
- All anchor methods used had a positive contribution to the increase of the ultimate load carrying capacity of the beams that were applied;
- The most effective plate support system was the one with the plate ends placed under the beam supports.

As a general note, failure of the specimens occurred because of compression failure of the beam at the extreme compressive fibre, and therefore, the GFRP and CFRP plates were not able to reach their maximum strain. The CFRP plate reached a greater percentage of its failure strain than did the GFRP plate.

A study, including the testing of four full scale reinforced concrete beams, was conducted by Kachlakev and McCury (2000). The specimens were replicas of concrete beams from an existing bridge. The capacity of these beams was significantly lower than the capacity required based on the projected increase of traffic loads. One beam was used as a control beam. The remaining three were strengthened with CFRP and GFRP plates; CFRP unidirectional sheets were used as additional flexural reinforcement and GFRP unidirectional sheets were used as additional shear reinforcement. One beam was reinforced for flexure only, a second for shear only, and the third for flexure and shear. The beams were tested to failure and the failure modes were diagonal tension cracking (shear failure) for the control beam, diagonal tension cracking (shear failure) for the beam reinforced in flexure only, yielding of tension steel followed by crushing of compression concrete for the shear reinforced beam, and the same as the latter for the shear and

flexural reinforced specimen. The results of these tests demonstrated an increase of approximately 150% in the static load carrying capacity as compared to the control beam. The application of the GFRP sheet alone managed to offset the lack of steel shear reinforcement in the beam (stirrups) and changed the failure mode from shear failure to flexural failure. The application of CFRP reinforcement only, although it again resulted in shear failure, that happened in a 31% greater load than the control beam ultimate load.

Lee et al., (1997) performed a series of tests on reinforced concrete beams with corroded tension rebars strengthened with CFRP sheets externally bonded with epoxy resin. Five different beams were used, and the variables were the tension rebar ratio, the number of CFRP sheets, and the presence or absence of anchors. The beams were subjected to static loading. The testing of the beams resulted in the following observations:

- The CFRP sheets increased the load carrying capacity of the beams and the failure of the specimen occurred when the sheets debonded or ruptured. At that point, the load carrying capacity of the beam reduced rapidly, and the beam would fail, demonstrating strength similar to the one achieved by the un-strengthened specimen.
- The increase of the number of the applied CFRP sheets resulted in an increase of the flexural strength of the beams, and reduction of the stresses sustained by the rebar.
- When anchor was applied on the CFRP sheets, the flexural strength was increased further, also increasing the ductility at maximum load.

- The strain of the CFRP sheets begun to increase sharply at 0.002 strain, demonstrating agreement with the yield load of the specimens, indicating that after yielding of the steel reinforcement occurs most of the additional load is carried by the CFRP sheets.
- The corroded specimens demonstrated a lower increase in the strength after the application of the CFRP sheets because of spalling of concrete, loss of bond, and the deterioration of the mechanical properties of the rebars which induced cracking. The method was proven to be effective in increasing the flexural strength of corroded RC beams beyond the level of the sound beams.
- Reinforced concrete beams reinforced with CFRP sheets exhibit behaviour consistent with the assumption that plane sections remain plane.
- Continued research and study of the bond strength between CFRP sheets and concrete surfaces is required to take full advantage of the composite material.

Tadatsugu et al., (1997) conducted a study on the effectiveness of CFRP sheets applied to reinforced concrete beams that had been damaged by corrosion of the traditional stirrup shear reinforcement. A total of nineteen beams were subjected to an anti-symmetric monotonically increasing static load. The variables that distinguished the different beams were the level of corrosion of the steel stirrup reinforcement, the number of stirrups, the number of CFRP sheets applied and their orientation, and the anchor or not of the CFRP sheets. The testing of the beams resulted in the following conclusions:

- CFPR sheets applied on RC beams can carry shear stresses just as stirrups do.

- The shear strengthening effect increases by increasing the number of CFRP sheets applied.
- Anchor of CFRP sheets increases that capacity of the beam to deform.
- The application of CFRP sheet in cross orientation increases the strengthening effect of the method.
- The performance of the beams with the corroded stirrups is controlled by the failure of the concrete cover due to cracking caused as a result of the corrosion of the steel reinforcement.
- Where epoxy patching was used on the specimens damaged by corrosion to increase the bond strength between CFRP sheets and concrete, the deformability and shear strength were also increased.

Grace et al., (1999) conducted an experimental study to determine the effectiveness of using FRP materials for strengthening against sagging of continuous beams. Five two-span T-section reinforced concrete beams were constructed and tested. One specimen was used as a control specimen and the remaining four, different strengthening systems were applied. Two beams were strengthened with two different types of CFRP sheets, one beam was strengthened with GFRP sheets, and the fourth with CFRP plates. The beams were loaded with a cyclic load, and at least one full cycle was completed before failure. A maximum of 66% increase in the load carrying capacity, as compared to the control specimen, was reached for the specimen that was strengthened with the glass fibre sheets. This increase was caused by the application of two layers of GFRP sheets in the positive moment area, and four layers at the negative moment area. The general

conclusions of this experimental program were that the use of FRP laminates as a strengthening method of continuous beams is effective in the reduction of deflections and the increase of the load carrying capacity. It was observed that the cracks were distributed evenly and were smaller in size as compared to those on the control specimen. The CFRP plates however, were observed to lead to premature concrete cover spalling. When used for shear strengthening, the CFRP plates were not fully utilized due to the lack of sufficient development length.

Khalifa et al., (1999) presented the results of the experimental investigation conducted to evaluate the response of continuous reinforced concrete beams with shear deficiencies, strengthened with externally bonded carbon fibre sheets. Nine full scale two-span rectangular cross section continuous beams were constructed for these tests. The specimens were split in three groups of three beams per group. One beam per group was left unstrengthened and used as a control specimen. The remaining beams were strengthened with different schemes, varying in the amount of steel shear reinforcement, the amount of CFRP applied, and orientation of CFRP fibres and wrapping schemes. The specimens were tested as continuous beams under concentrated load which was applied to the midpoint of each span. The load was a progressive cyclic load. An increase in the shear load carrying capacity ranging from 22-135% was observed. Therefore, it was concluded that the method is effective in increasing the strength of two-span continuous RC beams. The tests also showed that the strengthening effect is more evident with specimens without stirrups as compared to those carrying adequate steel shear reinforcement.

1.3.2. Application of Fibre Reinforced Polymers to Reinforced Concrete Beams and Columns Joints

An experimental investigation program has been carried out by Antonopoulos and Triantafillou (2003) on the effect of FRP strengthened reinforced concrete beam-column joints. A series of tests were carried out involving the testing of eighteen reinforced concrete joint specimens, two-third scaled. All specimens were poorly reinforced so as to represent a poorly designed exterior T-joint of a reinforced concrete frame. Two specimens were constructed with only one column stirrup in the joint; the remaining sixteen had no stirrups in the joint. Three of the sixteen specimens were constructed with a transverse beam in one side to account for the effect of confinement of transverse elements. All specimens were designed so that failure would occur due to shear in the joint. Two specimens were used as control specimens. The remaining sixteen were reinforced through varying reinforcement schemes. The reinforcement included the external epoxy bonding of different combinations of carbon or glass fibre sheets on the column and the beam. The results of the tests performed in this study demonstrated the effectiveness of the application of externally bonded FRP reinforcement as a strengthening method for reinforced concrete joints, especially where such joints are poorly designed in shear. The strength of the specimens compared to the control specimens was increased by up to approximately 70-80%. The stiffness of the specimens was increased in the order of 100%. The conclusions from this study include the following: flexible sheets are more effective than non flexible thin plates for the same reinforcement area; the dissipated energy increased considerably by 70-80%; mechanical anchors increased the effectiveness of both FRP strips and sheets; the effectiveness of the

FRP reinforcement increases as the transverse steel reinforcement in the joint decreases; and debonding of the external reinforcement is one of the main reasons for failure.

1.3.3. Application of Fibre Reinforced Polymers to Reinforced Concrete Columns

Miyauchi et al., (1997) conducted an experimental study on the effectiveness of the application of carbon fibre reinforced polymer sheets on concrete columns as a reinforcing method. Two different kinds of concrete were used, one with f'_c of 30 MPa and another with 50 MPa. The number of layers applied ranged from zero to three and the columns' dimensions were Φ 100x200 mm and Φ 150x300 mm. A set of specimens were manufactured for each and every combination set of the aforementioned parameters. The specimens were loaded with uniaxial compressive load. According to the test results, both the compressive strength and the maximum strain increased in proportion to the number of layers of carbon fibre sheets applied. It was generally concluded that:

- The compressive strength of the columns strengthened with the carbon fibre sheet is increased in proportion to the increase of the ultimate confining stress and it is not influenced by the dimensions of the column or the concrete type used.
- The axial strain at the maximum concrete stress for the specimens strengthened with the carbon fibre sheets increased exponentially, accompanied with the increase of the ultimate confining stress, and is influenced by the concrete's f'_c .
- The application of the carbon fibre sheets as a strengthening method of concrete columns has been proven to be successful and could be applied on existing reinforced concrete bridge piers.

Hamakada (1997) conducted a series of tests to assess the effect of the application of carbon fibre reinforced plastic (CFRP) sheets on single columns. The objective of the strengthening method was to increase the ductility capacity of the specimens and the bending capacity at the base of the column. The specimens tested had both rectangular and round cross sections. The models used were 1/5-scale models. Three round and three rectangular cross section specimens were constructed. One of the rectangular specimens was tested up to a point where it would sustain significant damage, but before reaching the maximum load carrying capacity, it was repaired and again tested so as to assess the effectiveness of the application of CFRP sheets as a repair method. The reinforcement was applied by CFRP in a circumferential direction. For the rectangular section columns the corners were rounded with a radius of 50 mm so as to prevent the damage of the CFRP sheet by the sharp concrete corners. The columns were reinforced with two or four layers of CFRP sheets at the base and for length equal to half the depth of the section and the rest of the columns were reinforced by one layer. The testing of both rectangular and round section columns lead to the following conclusions:

- RC columns retrofitted with the application of externally bonded CFRP sheets demonstrate considerable structural improvement for both round and rectangular cross section columns.
- The effect of the application of CFRP sheets on round columns is more considerable when it comes to ductility as compared to the corresponding effect on rectangular cross section columns.

- Rectangular section RC columns retrofitted with CFRP sheets demonstrated an improved failure mode from bending shear to simple bending.
- Significant moment capacity increase can be achieved when the CFRP sheet is anchored with a bolt through the column cross section.

1.3.4. Application of Fibre Reinforced Polymers to Reinforced Concrete Columns with Wing Walls

A study was conducted by Iso et al., (1997) to determine the effect of the application of CFRP sheets on reinforced concrete columns with wing walls. The experimental program included the testing of fourteen specimens. Two of the specimens were tested without applying any reinforcement and subsequently repaired with the application of externally bonded CFRP sheets. The primary variables for the different specimens were: (1) the configuration of the wing walls – column connection; (2) the application method of the CFRP sheet (sticking or wrapping); and (3) the amount of shear reinforcement. The specimens were axially loaded by two actuators with a capacity of 3,000 kN. The axial load was kept constant during the test. A lateral load was applied to the specimens by a horizontal actuator with a capacity of 1,500 kN. The results of the tests were successful in proving the effectiveness of the application of CFRP sheets as a method for retrofitting and repair. The following conclusions were reached:

- The application of CFRP sheets has the capacity to enhance the shear capacity of reinforced concrete columns with wing walls and their ductile behaviours.
- Damaged reinforced concrete columns with wing walls can be successfully repaired using epoxy resin injections and resin mortars followed by the

application of CFRP sheets. This repair method can recover the undamaged stiffness of the specimens.

An experimental study was conducted by Sonobe et al., (1999) to determine the effect of the application of FRP sheets on reinforced concrete columns with wing walls as shear reinforcement. Sixteen specimens were constructed and tested. The FRP sheets were externally bonded with epoxy resin. The variables distinguishing the specimens were the width of the wing wall, the type of FRP sheets, and the amount used as shear reinforcement. It was observed that all but one specimens failed in shear. The shear capacity of the specimens increased in proportion to the number of FRP layers applied. The following conclusions were reached:

- The application using FRP sheets (carbon or aramid fibre) can contribute to a better seismic behaviour of columns with wing walls when it comes to shear strength.
- The retrofitting effect of the carbon fibre sheet was confirmed for all the widths of wing walls used in the experiment.
- The shear strengthening effect of the FRP sheets on the specimens can be limited by the ultimate shear strength.

1.3.5. Application of Fibre Reinforced Polymers in Masonry Structures

Saadatmanesh (1997) presented the results of a study on the application of fibre reinforced plastics on concrete masonry structures as a method for strengthening and retrofit. The concrete specimens tested consisted of ten 4.88 metres long girders and ten

columns with varying steel reinforcement ratios. The girders were reinforced with the application of composite plates epoxy bonded to the soffit and the columns were wrapped with FRP straps which were used as an external confinement of the concrete columns. These specimens were loaded with a simulated earthquake excitation or load. The results from the testing of these specimens revealed that in the case of the concrete beams, significant gain in strength can be achieved. The positive effect of the FRP reinforcement was more obvious in the specimens where the steel reinforcement ratio was lower. The success of this reinforcement technique depends on the careful preparation of the concrete surface and the selection of the epoxy to be used as the bonding agent. It was noted that before this method is to be applied in the field, studies must be conducted on fatigue and long term behaviour of the bond between the FRP and concrete surface. The results obtained from the testing of the columns compared to the un-reinforced control specimens revealed an increase in the ductility level capacity up to four times. The straps used for the retrofitting of the columns were E-glass unidirectional 127mm wide straps. The concrete columns had insufficient lateral reinforcement and inadequate starter bars lap length. The loading of the specimens was based on load control until yielding, and displacement control after yielding of the steel reinforcement occurred. In order to examine the effectiveness of FRP materials as a repair method, the control specimen was loaded until complete failure and repaired by recasting loose and spalled concrete and wrapping the column with the FRP straps. The results of the repaired specimen showed that the method managed to recover and increase the strength of the column and to increase the energy absorption and dissipation capacity of the specimen.

In order to evaluate the effectiveness of the application of FRP composites on masonry structures, a number of masonry beams reinforced with composite fibre were tested in bending to obtain their out of plane load carrying capacity. The construction of the specimens differed in the mortar that was used, the type of the epoxy, and the roughness of the wall surface where the reinforcement fabric was applied on. The beams were subjected to a four point bending load. Significant observations resulted from this test, including the fact that the performance of the specimen is a function of the epoxy used, the mortar and the brick type also have an impact on the specimen's performance.

The US Army Construction Engineering Research Laboratory (Marshall et al., 1999) in partnership with the Composites Institute of the Society of Plastics Industry Inc. conducted an experimental study in an attempt to investigate the feasibility and effectiveness of the application of FRP materials as a strengthening method for unreinforced masonry walls. This program also had as an objective the development of design criteria for the application of the method. The test specimens were 4ft x 4ft double brick wall panels with the FRP composite reinforcing material applied to one face of the wall. The experiment also included the testing of brick mortar joints with FRP composites applied across. A total of forty specimens were constructed and four different FRP composite systems were used, namely glass epoxy system, carbon epoxy system, glass vinylester system, an adhesively bonded dash epoxy grid system. The reinforcing fibre orientation was 0 and 90 degrees to the mortar joints. The specimens were subjected to lateral load. The results demonstrated that the use of FRP composites for seismic retrofit of masonry walls has great potential. The FRP composite provided

additional support and helped the wall to maintain its structural integrity even when the specimen had failed. The strengthening effect of the application of the FRP material to the mortar joint was also confirmed. The strengthening effect of the FRP composite is a function of the width of the applied material. It was also concluded that additional testing is required so as to determine the optimum use architecture of the FRP composites.

Kuzik (2003) investigated the out-of-plane flexural behaviour of masonry walls reinforced with externally bonded glass fibre reinforced polymer sheets. The experimental program consisted of the testing of eight wall specimens. During the tests the parameters studied were the level of compressive axial load, the amount of internal steel reinforcement, and the amount of glass reinforced polymer sheets applied. The specimens were full scale and were subjected to cyclic load. The loading program was a combination of load controlled cycles for the first three load steps and thereafter, displacement controlled. The application of the load continued until reaching the maximum load carrying capacity of the specimen. This study resulted in the following observations:

- All specimens maintain their structural integrity throughout the test with only minor cracking.
- The only parameter that significantly affected the walls' response was the number of glass FRP sheets applied. The remaining two parameters studied, did not have the same significant effect on the behaviour of the wall.
- The application of the glass FRP sheets increased both the wall stiffness and the ultimate strength.

Overall, the behaviour of the wall specimens was very predictable, making the application of the glass FRP sheets a very suitable reinforcement method for masonry structures that require reinforcement due to insufficient original design in order to be able to withstand earthquake loading.

1.3.6. Application of Fibre Reinforced Polymers to Reinforced Concrete Slabs

An experimental program was conducted by Erki and Heffernan (1995) to determine the effectiveness of the application of externally bonded FRP materials on reinforced concrete slabs. The reinforced concrete slab specimens were both two-way and one-way slabs. Six specimens were reinforced with two-way reinforced slabs being supported on all four edges, and two with one-way reinforced slabs that were supported only on two edges in the direction of the rebars. One one-way and two two-way slabs were used as control specimens. The remaining slabs were reinforced with externally bonded FRP sheets which were applied on the tension face of the slab. Both glass and carbon FRP were used for that application. The slabs were subjected to a punch load that was expected to induce two-way shear or punching shear failure. As compared to the control specimens, the strengthened slabs demonstrated an increase in the strength and stiffness. The one-way slab specimen reinforced with glass FRP sheets demonstrated a 18% increase in strength compared to the control specimen. The two-way specimen reinforced with glass FRP exhibited a 19% increase in strength compared to the corresponding control specimen. The specimens that were strengthened with CFRP

sheets exhibited a much larger increase in their strength which ranged from 72% to 138% depending on the orientation of the fibres and the number of sheets.

As a result of this study it was generally concluded that the application of FRP sheets externally bonded on the tension face of thin flexural members such as slabs, increases the ultimate strength and stiffness of the members. The effect of the application of the FRP sheets largely depends on the orientation of the applied fibres and the angle between the fibres and the internal steel reinforcement.

1.4. Objectives and Scope

As presented in the previous section there is extensive literature on the utilization of composite materials for the strengthening / retrofit and repair of structural members of buildings and structures in all. The areas covered in the existing literature are mainly dealing with application of composite materials on beams, wing walls, and columns. There is until today very little research on the effect of the application of composite materials and more specifically the effect of carbon fibre reinforcement polymers on reinforced concrete shear walls (structural walls). The main objective of the present study is to assess the effectiveness of the application of externally epoxy bonded carbon fibre sheet on shear walls as a method for retrofit and repair along with the introduction and testing of a new innovative load transfer anchoring system. It constitutes the second phase of a broader research program adding to phase one's experimental data and observations and attempting to solve problems encountered during the phase one experimental tests series.

The phase one experimental test series revealed that the load transfer mechanism (anchor) that was used for those tests did not perform as expected.

The current study focuses in part on the design and testing of a new load transfer mechanism/anchor in an attempt to utilize the strengthening material (Carbon Fibre Sheets) to its maximum capacity.

The experimental set up was designed to assure that only in-plane deformations were allowed so as to be able to assess the in-plane response of the specimen only.

The instrumentation was to be designed to give sufficient data so as to illustrate the stress profile at the extreme strain fibre, and to provide information on the strain distribution between shear and flexural deformation.

Five walls, 2/3 scale, were built and tested during phase two of this program. The specimens were built following the exact same design as the walls of phase one, in order to be able to compare and validate the experimental data.

The walls were to be loaded with an increasing static load up to maximum load capacity and then continued until its resistance dropped to 80% of its peak resistance.

The data acquired from the experiments were to be analyzed, presented and compared with the phase one results.

Chapter 2

Overview of Shear Wall Response

2.1. General

The performance of reinforced concrete buildings that include shear walls in their structural system against earthquake excitations has been proven to be very efficient (Mo, 1998). These responses as well as the failure modes of shear walls are dependant on a number of factors, such as wall dimensions and reinforcement details, etc. Deformations of shear wall can be separated into individual components due to different deformation mechanisms as shown in Figure 2.1.

The different failure modes of shear walls and factors affecting shear wall behaviour are outlined and briefly discussed in the following section so as to give a better understanding of the behaviour of shear walls subjected to lateral in-plane loading.

2.2. Failure Modes of Shear Walls.

The failure mode of a shear wall is influenced by its height to length aspect ratio. A slender shear wall with an aspect ratio greater than 2 demonstrates different failure modes than a squat wall with an aspect ratio less than 2 (Priestley, 1992). In the following sections typical failure modes of slender and squat shear walls are discussed. However, it should be noted that the failure modes of slender and squat walls can be significantly influenced by other structural design parameters or factors, such as amount of flexural

and shear reinforcements, reinforcement details and configuration. It is possible that a squat wall may fail in a flexural mode which is typically expected of a slender wall and vice versa. In the present experimental study all the wall specimens tested are designed to fail in even though they are squat walls (aspect ratio 1.2) which are more commonly expected to fail in shear. The reinforcement details of the installation stirrups around the four extreme pairs of flexural reinforcement on both ends of the wall are the main factor responsible for the flexural failure mode of these walls.

2.2.1. Failure Modes of Slender Shear Walls

Typical failure modes of slender shear walls when subjected to lateral in-plane loading are the following:

- *Flexural tensile reinforcement failure.* This is the preferred failure mode, because the wall fails in a ductile manner and the yielding of the vertical steel reinforcement in the plastic hinge regions is the principal energy dissipation source for a slender shear wall.
- *Shear failure.* This failure mode can occur when large shear cracks with an inclination of about 45° occur on the wall. At that point the horizontal reinforcement may fail and concrete crushing at the root of the critical path may occur.
- *Concrete crushing failure.* This failure mode is the most common one, and occurs usually following extensive yielding of the wall's flexural reinforcement.
- *Sliding Shear Failure:* Failure of the shear wall due to sliding failure at its base. Though possible, it is not very likely to occur during the loading of a slender

wall. A slender wall is likely to reach its moment carrying capacity at a level where the shear load that generates the moment is at a level lower than its sliding shear capacity.

2.2.2. Squat Shear Walls Failure Modes

A shear wall with an aspect ratio less than 2 is generally considered a squat wall. A squat shear wall is expected to fail in shear with the occurrence of large inclined shear cracks. These cracks occur between the concrete areas which, following the occurrence of the initial shear cracks and the change in the mechanism of the shear resistance resulting from these cracks, resist the internal compression forces. These concrete areas are called compression struts. In the shear failure mechanism, vertical reinforcement is critical in maintaining equilibrium against the vertical component of the compressive force carried by the compression strut (Ferguson, 1988).

Squat walls can also fail by sliding shear. They are more susceptible to this failure mode compared to the slender walls, since they have a larger shear load carrying capacity allowing the walls to reach the sliding shear capacity level.

2.2.3. Failure Modes to be Avoided

Failure modes that should be avoided in properly designed shear walls are the shear failure mode, and the sliding shear failure mode. The structures should be designed to have a ductile failure mode in order to be able to dissipate more energy and avoid sudden, catastrophic failure.

In order to achieve this, the following detailings should be implemented in the design of a shear wall.

- The concrete in the compression zone of the wall should be confined. This way even when the concrete has crushed, the wall will still be able to carry the compression load generated by the in-plane lateral load, avoiding the occurrence of buckling of the vertical reinforcement and a brittle failure mode (Paulay, 1975).
- In a slender wall, it is beneficial to concentrate the vertical reinforcement towards the edges of the wall so as to take advantage of the tensile capacity of the reinforcement as well as the compressive one. In the case of a squat wall, such scheme does not provide equivalent benefit since the internal lever arm in such a wall is quite large and the compression zone is significantly larger.
- The vertical flexural reinforcement should be distributed evenly along the full length of the wall with a nominal concentration towards the edges. This even distribution also improves the conditions for shear friction and dowel action, features that are significant for the sliding shear capacity of the wall (Priestley, 1992).

2.3. Factors that Affect Shear Wall Responses

The behaviour of shear walls depends on certain factors relating to geometry, design, material, construction methods, and loading. These factors affecting the response and general behaviour of a shear wall are outlined below:

- Aspect ratio (height to length ratio).
- Steel reinforcement configuration and steel reinforcement ratio.

- Connection fixity between the wall and the top and bottom slabs
- Load application
- Concrete strength

A brief discussion of these factors and the way they affect shear wall responses are presented as follows.

2.3.1. Wall Dimensions and Height to Length Ratio.

As discussed in the previous section the failure modes of a shear wall depend on whether the wall behaves as a squat or slender wall, with a height to length aspect ratio less than, equal to or greater than 3, respectively.

Slender wall can be designed using the beam theory with good approximation. The behaviour of a squat wall is better analyzed by the deep beam theory. The behaviour and load carrying capacity of squat shear walls are governed by shear behaviour. This makes them not the best medium to dissipate earthquake energy since shear failure is brittle as it occurs before the maximum flexural capacity is reached.

2.3.2. Steel Reinforcement Configuration and Steel Reinforcement Ratio

The configuration of steel reinforcement and the location and number of rebars in a shear wall, control the behaviour of the wall to a large extent. As mentioned before in slender walls, it is beneficial to concentrate the vertical rebars towards the edges of the wall (Flanged Wall). This way both tension and compression zones are reinforced. Since the wall is not expected to fail in sliding shear, this reinforcement detail allows the efficient

use of the reinforcement in resisting the applied lateral load. The capacity of the wall is therefore enhanced along with its curvature ductility. In the case of a squat wall, the effective way to lay out the vertical reinforcement is to have it evenly distributed in order to have the desirable behaviour in the previous section. It should also be noted that experimental studies on the behaviour of squat walls have shown that the shear strength of squat walls is not significantly effected by differences in the amount of flexural reinforcement (Barba, 1977) (McCormac, 2001).

2.3.3. Connection Fixity Between the Wall and the Top and Bottom Slab

The presence of top and bottom slab has a significant effect on the behaviour of a shear wall structure. The slabs that interconnect the shear walls have an important effect on the increased load carrying capacity of the structure, provided the slabs are designed to provide sufficient interconnecting stiffness (MacLeod, 1971).

The effect of the presence of top and bottom slab in the shear capacity of a shear wall is even more significant in the case of squat shear wall with a height to length aspect ratio less than 0.5 (Barba, 1971).

The slabs are responsible for the equal distribution of the load along the wall's cross-section and are controlling and minimizing the occurrence of diagonal flexural cracks in the wall. The top and bottom slabs also control torsional deformation that may tend to occur, acting as horizontal diaphragms. A slip at the construction joint between the wall and the slab can slightly reduce the structures strength but can increase the energy absorption capacity.

2.3.4. Load Application

The shear walls are mainly subjected to lateral in-plane cyclic loading and vertical axial compression load. The vertical compression load affects the load carrying capacity of the wall as outlined below.

- Increases the shear strength
- Increases the sliding shear strength
- Increases the flexural capacity of the wall
- Decreases the horizontal and vertical displacement.

The in-plane cyclic loading results in a decrease of the load carrying capacity of the wall, due to deterioration of the compressive and shear capacity. Experiments have shown that this decrease can be in the range of 7% (Cardenas, 1980) to 10% (Barba, 1977). The multidirectional cracks resulting from the cyclic loading pattern decrease the compression strength of concrete and affect the load transfer mechanism from the wall to the footing through the diagonal compression struts.

Deformation Modes

Total Deformation Flexural Deformation Shear Deformation Sliding Shear Deformation

Figure 2.1 Shear wall Response Components

Chapter 3

Carbon Fiber Reinforcement Polymer System

3.1. Introduction

Fibre Reinforced Polymers (FRP) were traditionally used in applications related to the aerospace, automotive and marine industries (Farmer, 2000). Their use has expanded during the last 20 years to many other applications including, the construction of railway carriers, the manufacturing of athletic and recreational equipment, and infrastructure, medical, industrial and electronic applications (Herakovich, 1998).

The common types of FRP materials widely used in industry are made off aramid, carbon or glass fibres embedded in a polymer thermosetting or thermoplastic matrix. In the last two decades, FRP materials have been applied to the civil engineering field, especially as reinforcement materials for retrofit and repair of existing structures. Carbon FRP (CFRP) materials are one common type of FRP materials for such applications (Belardi, 2001). Table 3.1 presents the comparison of material properties of traditional construction and engineering materials with those of fibres and polymer matrices. A description of the different kinds of fibres and matrices is presented as follows.

3.1.1. Glass Fibers

Glass fibres are generally divided in two groups. The first group includes the Glass Fibre types E, A, C, and E-CR. These fibres have a typical modulus of elasticity of 70 GPa,

and strength that can reach 1000 to 2000 MPa. The second group of glass fibres includes the fibre types, R, S and AR. These fibres have a modulus of elasticity of around 85 GPa, and a demonstrated strength that can reach 2000 to 3000 MPa. Table 3.1 presents the material properties of representative fibre types from group A and B.

Group A glass fibres are primarily used for reinforcement in low alkali content environments. Group B Glass fibres are more alkali resistance and are used in higher performance applications and the reinforcement of cement (Hollaway & Leeming 1999). Glass fibre reinforced polymers are sensitive to moisture and can suffer degradation of their mechanical and material properties when exposed to it, especially when the fibres are under strain. The condition known as stress-corrosion cracking occurs when due to corrosion a reduction in the fibre/matrix adhesion occurs (Hollaway & Leeming 1999).

3.1.2. Aramid Fibres

The modulus of elasticity of aramid fibres ranges between 60 to 120 GPa. Aramid fibres have a strength that can reach 3000 MPa. The density of aramid fibres is considered low and has a value of around 1400 Kg/m³. The low density makes these fibres very attractive due to its high strength and high stiffness to weight ratio compared to glass and carbon fibres. Aramid fibres have the characteristics of good fire resistance and electrical and heat insulation. Aramid fibres are also resistant to common chemical attacks due to fuels, lubricants and solvents (Hollaway & Leeming 1999).

Aramid fibres are divided into two categories. The first category includes fibres with a modulus of elasticity between 60 to 70 GPa. The most typical fibre type in this category is Kevlar 29. The second category includes fibres with a modulus of elasticity value

around 120 GPa. Kevlar 49 is a common representative of this group. The most common aramid fibres are the ones in the second category. Aramid fibres of the first category are used in applications involving high strain rates.

3.1.3. Carbon Fibres

Carbon fibres are the common choice of fibres to be used in applications where the objective is to achieve high stiffness and strength. Carbon fibres are fibres that contain at least 92% in weight of carbon (Chang, 1994). The density of carbon fibres has a value of about 1900 kg/m^3 . The modulus of elasticity of carbon fibres ranges between 230 to 300 GPa. Its ultimate strength can vary between 3000 to 5000 MPa.

Carbon fibres are made by controlled chemical decomposition by heat (pyrolysis) of precursor material in fibre form such as Polyacrylonitrile (PAN), rayon or pitch at temperature that range between 1000° to 3000° C (Herakovich, 1998). The properties of the fibres, such as modulus of elasticity, conductivity and density, depend on temperature under which the pyrolysis takes place. The fibres produced by pyrolysis of PAN are considered of higher performance compared to the ones produced using pitch or rayon.

Individual Carbon filaments produced by this method have a diameter of 4 to $10 \text{ }\mu\text{m}$.

The term carbon fibre is understood as the material consisting of 3,000 to 30,000 filaments. For carbon fibres from a pitch precursor, the fibres are made by spinning a petroleum-based product so as to form a pitch precursor (Herakovich, 1998).

Although carbon fibres have very high tensile stiffness, they do not have the capacity to carry compressive or bending load. The structural properties of carbon fibre composites make them ideal for structural engineering applications. Specifically, carbon fibre

composites are typically ten times stronger than steel and eight times stronger than aluminum, and 5 and 1.5 times lighter than steel and aluminum, respectively (Belardi, 2001). When used with the right resin it becomes a very corrosion resistant composite. The forms that carbon fibre reinforcement polymers are available include sheets and plates and laminates. For the experimental program of the present study, carbon fibre sheets are used as external reinforcement. A description of the carbon fibre sheet and the application method are presented in Section 3.2. Some carbon fibres are the product of heating, oxidizing, and carbonizing polyacrylonitrile polymer fibres.

3.1.4. Matrix Materials

As previously mentioned the matrix in the composite material is the material responsible for the transfer of load from the structure strengthened or repaired with the bonded FRP reinforcing sheet to the fibres of the FRP materials. The matrix material of the FRP composite has a lower elastic modulus than that of the fibre. There are two kinds of polymer matrices used in FRP composite materials, thermoset and thermoplastic ones. The most commonly used thermoplastic matrices include polypropylene, polyvinyl chloride, nylon, polyurethane, poly-ether-ether-ketone (PEEK), polyphenylene sulphide (PPS) and polysulfone. The thermoplastic materials soften if reheated and can be reshaped after they have been applied and cured.

Thermoset materials cannot be reshaped after curing is complete. They include materials like polyester, epoxy, and polyamides. Polyesters are resistant to environmental exposure and are mainly used as matrix for glass fibre composites. Epoxies have better moisture resistance and shrink less during curing. Their maximum use temperature is around

175 °C. Polyamides have a higher operational temperature of around 300 °C, but their fabrication process is more complicated.

For the experimental study here epoxy matrix is used. Table 3.1 presents typical values of the properties of an epoxy material.

3.2. Carbon Fibre Sheets

3.2.1. General

The Carbon Fibre Reinforcement Sheets used for this experimental study are 500 mm wide rolled sheets. The thickness of the sheet is 0.11 mm. The mechanical properties of the sheets used for this experimental study, are shown in Table 4.2. It is a woven carbon fibre sheet where the fibres are kept together with a fine thread matrix. These sheets are very flexible before the application of the epoxy resin and are easy to cut with a pair of scissors or a utility knife.

3.2.2. Application of CFRP Sheets

The areas on which CFRP sheets were to be applied to and bonded with epoxy required preparation in order to achieve the optimum adhesion and bond between the concrete surface and the CFRP sheet. The concrete surface was wire brushed and cleaned of all loose material. It was then washed three times with muriatic acid to ensure that the surface was completely free of any grease, dust, or other foreign particles. The surface was then left to dry for twenty-four hours.

Following the washing process, an epoxy primer resin was applied to the concrete surface of the wall using a brush. The epoxy primer resin was then left to cure for forty-eight hours.

With the epoxy primer cured, the application of the CFRP sheets on the concrete surface followed the process described below.

First, a coating of epoxy resin was applied with a roller onto the primed concrete surface as a bonding material. Second, a CFRP sheet was placed on to the epoxy-covered surface, and pressure was applied through the use of a hard roller to ensure against air pockets between the wall surface and the CFRP sheet. Third, a second application of epoxy resin followed, so as to saturate the CFRP sheet fibres, thus ensuring optimum bonding between concrete and the CFRP sheet.

To continue this reinforcement process with more than one-ply of CFRP sheet, the same procedure is repeated following the application of the first ply, with the epoxy resin allowed to dry for twenty four hours before the application of the second ply, following the same process as with the first ply, excluding the primer application.

3.3. Proposed Anchor System

3.3.1. General

During Phase 1 of this experimental project, a number of observations were made related to the performance of the anchor system used as a load transfer mechanism of the stresses developed in the CFRP sheet to the specimen footing.

The load transfer path was from the CFRP sheets to the flange of the steel angle anchor through epoxy bonding and then to the footing through the anchoring bolts (see Figure 3-1).

A critical design parameter of this system is the development length and the bond between the CFRP sheet and the vertical part of the steel angle anchor. That length needs to be sufficient to transfer the load from the CFRP sheet to the anchor and subsequently to the footing of the specimen. The design objective of the L-shaped angle anchor system was to prevent bonding failure which leads to failure of the whole reinforcing system. During the first phase of the investigation, the testing of one of the specimens ended prematurely due to pull out of the mounting (anchoring) bolts of the anchor system. This same specimen anchor was repaired and again tested to failure so as to observe once again the behaviour of the anchor system in extreme stress conditions.

The behaviour of the anchor was the same as it has been reported by Josh Lombard (Lombard, 1999). The prying action was again the main cause for the failure of the CFRP sheets. The failure mechanism occurs in two stages. Due to the prying action caused by the moment arm between the line of action of the force on the CFRP sheet and the anchoring bolt, the vertical part of the steel angle tends to pull the CFRP sheet away from the wall surface which eventually detaches from the CFRP sheet, as shown in Figures 3.2 to 3.6.

In case bond is maintained between the CFRP sheet and the anchor, failure of the reinforcing system can occur in two stages. With the flange of the steel anchor pulled away from the wall surface, the CFRP sheet still bonded with the flange is also pulled

away from the wall surface. This leads to the complete debonding of the CFRP sheet from the wall surface and failure of the system. When the bond between the CFRP sheet and the flange was maintained and the bond between wall and CFRP at level higher than the level of the anchor was also maintained, a hinging was observed at the level of the top of the vertical part of the steel angle and the very brittle in bending CFRP sheet, cracked and failed.

It was observed that in the case when the CFRP sheet was debonded from the flange of the anchor, but still remained bonded to the wall surface, the CFRP sheet continued to carry the tensile load resulting from the flexural behaviour of the wall. The load transfer from the CFRP to the footing was still maintained through the bond between a strip of the CFRP sheet and the horizontal part of the steel angle.

Based on the lessons learned from the first Phase of the experimental study, a new anchor system was designed for the second Phase. The main objective of this new anchor is to eliminate the prying action effect of the L-shaped angle anchor (i.e. elimination of the moment arm between the lines of action of CFRP resistance force and the reaction load). In order to achieve this, a new anchor is designed based on the pulley behaviour using a cylindrical hollow section (CHS). By wrapping the CFPR sheet around the cylindrical hollow anchor, the load carried by the vertical CFRP sheet is transferred to the horizontal surface of the wall footing. A development length of 350 mm is provided to transfer the load from the horizontal sheets to the footing. The CHS acts as a pulley in the load transfer mechanism. The tension stress carried by the vertical CFPR sheet is equal to the tension at the interface between the horizontal part of the CFRP sheet which is epoxy bonded to the concrete surface of the footing.

The CHS was anchored on the wall footing by threaded anchor rods (TAR) that were epoxy bonded into the concrete footing of depth sufficient to allow that load transfer from the CFRP sheets. The required depth was obtained by following the anchor bolt manufacturer design guideline. Anchor rods holes of 1 ½ " diameter were drilled in the wall, as shown in Figures 3.8, 3.9. The TARs were installed at a 45-degree angle in the direction of the resultant of the two CFRP forces on the wall and on the footing (see Figure 3-7). In Figures 3.7 two alternative ways for the installation of the CHS are also presented. In both cases, the pulley principle is effective. The choice of the preferred installation method depends on the structure layout. For this series of test, the arrangement illustrated in Figure 3-10 was used.

3.3.2. Details of Anchor's Structural Design

The Anchor was designed with the assistance of the structural analysis program Cosmos. The design load was the triangular load of the CFRP sheet acting on the CHS anchor due to lateral load of 500kN applied at the top of the wall specimen. The dimensions of the CHS were selected so as the Von Misses stresses under the max load are less than yield stress of the material (400 MPa). This triangular design load had a maximum of 877 kN at the end of the anchor and had value zero at 1161 mm from the maximum load point. The magnitude of the load was determined using beam theory on the wall's base section. The maximum load carrying capacity of a specimen reinforced with two plies of CFRP sheet on each side of the wall was calculated and the load carries by the CFRP sheets was obtained. This load was only one part of the actual load applied to the anchor. The second part was the load applied from the CFRP sheet extending horizontally on the

footing surface. Due to the pulley effect as discussed earlier, this is equal to the load of the vertical CFRP wall sheet. The resultant of these two components led to the design load mentioned before. This triangular line load was then applied as pressure load on the surface of the anchor tube that was in contact with the CFRP sheets.

In the design analysis, the anchor was modeled as a hollow tube using 3D solid 8 node elements in the analysis model. The material properties of the system were:

E_x : 200×10^9 Pa and $N_{uxy} = 0.29$.

Following this design analysis, the numerical results are presented in Appendix B.

A 3-inch (76.2 mm) external diameter mechanical pipe was chosen with a 0.5 inch (12.7 mm) wall thickness for the fabrication of the anchors.

3.3.3. Fabrication of the Proposed Anchor System

The anchor system was fabricated in the University's Workshop with the assistance of the Civil and Environmental Engineering Lab Technologists. The mechanical pipe was delivered to the Lab pre-cut in the required length of 1500 mm. The fabrication of the designed anchor was completed in three stages.

The first stage included the fabrication of the washers, five per anchor. The washers were fabricated from a 2.5 inch x 1.0 inch (63.5mm x 25.4mm) steel plate cut in sections of 3.5 inch (88.9mm) long. After the 3.5 inch x 2.5 inch x 1.0 inch sections were cut, the one 2.5 inch x 3.5 inch side of all plates was machined to create a concave surface with a radius matching the exterior radius of the mechanical pipe of 3.0 inch (76.2 mm) (see Figure 3-11). Stage two included the welding of the washers on the mechanical pipe on predetermined positions along the same side of the pipe (Figure 3-12). Stage three

involved the drilling of openings at the locations where the washers were welded, with a diameter of 1 ½ inch (38.1 mm) for the housing of the anchoring rods (Figure 3.13). The anchoring system was completed with five 1 ¼ inch (31.75 mm) diameter, 20 inch (508 mm) long threaded rods with flat washers and nuts.

Material	Density ρ g/cm ³	Modulus of Elasticity E_L GPa	Poisson's Ratio ν_L	Strength σ^u_L MPa	Specific Stiffness (E/ρ)/(E/ρ) _{Al}	Specific Strength (σ^u/ρ)/(σ^u/ρ) _{Al}	Thermal Expansion Coefficient
<u>Metals</u>							
Steel	7.8	200	0.2	1724	1.0	1.2	12.8
Aluminium	2.7	69	0.33	483	1.0	1.0	23.4
Titanium	4.5	91	0.36	758	0.95	1.2	8.8
<u>Fibres</u>							
AS4 carbon	1.8	235	0.20	3599	5.1	11.1	-0.8
P100s Carbon	2.15	724	0.20	2199	13.2	5.5	-1.4
IM8 Carbon	1.8	310	0.20	5171	6.7	16.1	-
Kevlar 49	1.44	124	0.34	3620	3.6	13.9	-2.0
S-2 Glass	2.46	86.8	0.23	4585	1.4	10.4	1.6
E-Glass	2.58	69	0.22	3450	1.05	7.5	5.4
<u>Matrix Materials</u>							
Epoxy	1.38	4.6	0.36	58.6	0.08	0.4	63
Polyimide	1.46	3.5	0.35	103	0.03	0.4	36
Copper	8.9	117	0.33	400	0.5	0.3	17
Silicone Carbide	3.2	400	0.25	310	4.9	0.5	4.8

Table 3.1 Properties of engineering Materials, Fibres and Matrices (Herakovich, 1999)

Figure 3.1 Illustration of the Anchor used in previous test

Figure 3.2 Prying Action

Figure 3.3 Debonding between CFRP and Steel Angle due to Prying Action

Figure 3.4 Debonding due to Prying Action

Figure 3.5 Debonding due to Prying Action

Figure 3.6 CFRP Sheet Failure due to Prying Action

(a)

(b)

Figure 3.7 Anchor Systems

Figure 3.8 45- degree Core Drilling for Anchoring Rods

Figure 3.9 Openings for the Anchoring Treaded Rods

Figure 3.10 Anchor Installed

Figure 3.11 Fabrication of Anchor System - Washers

Figure 3.12 Fabrication of Anchor – Welding of Washers

Figure 3.13 Fabrication of Anchor – Anchoring Rods Openings

Chapter 4

Design of Test Specimens

4.1. Test Specimens

In the present study, the second phase of a comprehensive research program on the retrofit and repair of reinforced concrete shear wall using advanced composite materials, four reinforced concrete shear wall specimens were fabricated. These specimens were identical to the specimens used in the first phase of the experimental project. The general dimensions of the specimens are shown in Figure 4.1. The general layout of the steel reinforcement installed in the specimens is shown in Figures 4.2 and 4.3. Maintaining the same specimen dimensions and design details facilitated the comparison of results from the two phases of the study, and expanded the database of test data. The test specimens included a control wall, a repaired wall, a reinforced wall with one vertical layer of CFRP sheets, a reinforced wall with two vertical layers of CFRP sheets, and a reinforced wall with three vertical layers of CFRP sheets and one horizontal.

The shear wall specimens were constructed in three sections. The first section was the footing of the specimen. It was a heavily reinforced concrete block, measuring 1730mm x 900mm x 435mm, with twelve No. 20 deformed bars in the longitudinal direction and twelve stirrups distributed along the full length (1730mm) using No.10 deformed bars. Six PVC pipes of 75mm external diameter were placed in each footing formwork, aligned with the hold down anchor points on the laboratory strong floor for the anchor of the specimens for testing. The footing reinforcement cage as well as the wall section

reinforcement are shown in Figure 4.4. A view of the specimens after the completion of the first stage of the fabrication of the specimens is shown in Figure 4.5.

The shear wall was the second section of the wall specimen with dimensions of 100mm thick x 1500mm wide and 1795mm high. The vertical reinforcement of the shear wall was composed of six pairs of No.10 deformed bars uniformly distributed along the wall and spaced at 280mm, corresponding to a steel reinforcement ratio of 0.8%. The horizontal reinforcement of the shear wall consisted of five pairs No.10 deformed bars uniformly distributed and spaced at 400mm, with a corresponding steel reinforcement ratio of 0.5%. Stirrups spaced at 80mm along the total height of the wall were also installed around the two extreme vertical reinforcement pairs so as to prevent premature buckling under compression. The stirrups were made with No.10 deformed reinforcement bars (see Figures 4.4 and 4.5). The reinforcement of the control wall is that of a well-designed wall that would not normally require retrofit or strengthening with additional reinforcement. However for the purposed of the present study in evaluating the behaviour and performance and effectiveness of the new anchor system, the testing of well designed wall specimens provides results and information of the anchor system under the most critical level, and allows the CFRP sheets to reach their full capacity and thus subjecting the anchor system to the maximum possible loads.

The third section of the wall specimen was the head. The head consisted of a heavily reinforced concrete block measuring 1730mm x 530mm x 410mm, with eight No20 deformed rebars in the longitudinal direction and eleven sets of stirrups uniformly spaced along the full length of the block (Figure 4.6). In order to create the required holes to attach the actuator head on the wall specimen, four 50mm PVC pipes were installed in

the formwork. The complete formwork is shown in Figure 4.7. A complete shear wall specimen after the completion of the fabrication process is shown in Figure 4.8.

In terms of testing procedure, one of the specimens was tested as a control wall without the addition of any external CFRP reinforcement. This wall was then repaired and was tested again as a repaired wall specimen to assess the effectiveness of the method for retrofit of structural members that have suffered structural damage. Details on the design, fabrication, and materials used on these specimens are presented here. Each of the remaining specimens was reinforced differently, and tested to failure. The walls were identified, labelled, and reinforced as follows:

Reinforced wall # 1 was a test specimen reinforced with one layer of CFRP per side in the vertical direction. Reinforced wall # 2 was reinforced with two layers of CFRP sheets externally epoxy bonded in the vertical direction.

Reinforced wall # 3 was reinforced with three vertical layers and one horizontal layer of CFRP sheets on each side. The design methodology of the test specimens is presented in the following sections.

4.1.1. Design Assumptions and Approach

The design of the reinforced concrete shear wall specimen was based on the assumption that plain sections remain plain. Furthermore, the effect of strain hardening was ignored in the design. The methods used for the design of the specimens were similar to the approach followed by Lombard (1999). Specifically, in order to determine the in-plane flexural strength of the shear wall test specimens, a modified plane section analysis procedure was used. The shear capacity of the specimens was determined using a

modified Wiradinata's approach (Wiradinata, 1985) as proposed by Lombard (1999).

The sliding shear capacity at the joint between the wall and the footing was determined in accordance with the procedures stipulated in clause 11.6.1 of the CSA standard A23.3-94 Design of Concrete Structures (1994). The details of the design are illustrated in Appendix A.

4.1.2. Fabrication of Specimens

As discussed in Section 4.1, the test specimens comprised three sections. Each of these sections was fabricated in separate stage. Stage one included the fabrication of the reinforcement cage of the footing and the wall specimen, the fabrication of the formwork for the footing, and the concrete pouring of the footing. The installation of the six 75mm PVC pipe sections in a vertical direction in the formwork was an important part of fabrication. The PVC pipe sections contributed to the alignment of the formwork with the laboratory's strong floor holes. This alignment step was rather critical in ensuring that the mounting bolts went through both the footing and the strong floor, thus fixing the wall to the strong slab. The reinforcement cage was equipped with four carrying loops, allowing for the transportation of the wall specimen (Figure 4.8).

Stage two involved the installation of the strain gauges on the vertical reinforcement of the shear wall, the fabrication of the formwork, and the pouring of the concrete for this part of the specimen. Superplasticizer was used as an additive to the concrete mix at this stage, ensuring that the concrete would take up all available space in the formwork. The layout and installation of the strain gauges are discussed in greater detail in Section 5.3.

Stage 3 involved the fabrication of the reinforcement cage for the head, the fabrication of the formwork for the head, and the pouring of the concrete. Four 50mm PVC pipes are placed in the head for the attachment of the head of the wall to the actuator plate by four dywidag bars. Bolts were imbedded at the top of the concrete head as part of a new lateral restraint mechanism that is discussed in detail in Section 5.1.

4.1.3. Material Properties

4.1.3.1. Reinforced Steel

The reinforcing steel used for the fabrication of the wall specimens was grade 400. Number 10 deformed reinforcing bars were used for the vertical and horizontal reinforcement of the shear wall specimens and for the stirrups. Grade 400 No. 20 deformed reinforcing bars were used for the longitudinal reinforcement of the footing and the head of the specimen. Two specimens of No. 10 reinforcing bars were tested in the lab to obtain the monotonic stress-strain relationship of the rebar used for the vertical and horizontal reinforcement of the shear wall specimens. Two 5mm electrical strain gauges were used per specimen to measure the strain during this test. The stress-strain relationship obtained from these tests is shown in Figure 4-9.

4.1.3.2. Carbon Fibre Sheets

The Carbon Fibre Reinforcement Sheets that were used for this experimental study were 500 mm wide rolled sheets. Its main attribute is that it is a unidirectional woven carbon

fibre sheet with the fibres kept together by a fine thread matrix (Figure 4.10). The thickness of the sheet was 0.11 mm and each roll was 100m long. The material properties of the sheets used for this experimental study are shown in Table 4.1. The tensile modulus of the carbon fibres is 230 GPA, and the maximum tensile strain is 0.017. The bonding agent between concrete and the CFRP sheets is a two-part epoxy resin, applied on the concrete surface after the surface has been thoroughly cleaned of all foreign matter and primed with a two-part epoxy primer.

4.1.3.3. Concrete

The concrete used for the fabrication of the shear wall specimens had a specific nominal 28 day compressive strength of 30MPa, 100mm slump, no air entrainment, and 10mm maximum aggregate size. Superplasticizer was used to avoid air-pocket problems during pouring of the concrete for the wall section of the specimens. Throughout the fabrication stage, control cylinders were taken and tested at the 28 day mark and on the day of testing of the wall specimens to obtain the material properties of the concrete mix delivered and used for the specimens. Table 4.2 summarizes the results from the compression test of the concrete cylinders. Figure 4.11 presents the stress-strain relationship obtained from such cylinder testing.

4.1.3.4. Steel for Anchor

The mechanical pipe used for the anchoring system was anchored on the wall with five epoxy threaded rods bonded with epoxy adhesive. The threaded rods were 24 inches long, 1 ¼ inch diameter, made of high strength stainless steel. The yield stress is 204 MPa and the ultimate stress is 515 MPa.

Specimen	Age at test (Days)	28 days f'_c (MPa)	28 days f_r (MPa)	f'_c at test (MPa)	f_r at test (MPa)
Control Wall	175	35.9	3.0	36.2	3.1
Repaired Wall	254	35.9	3.0	38.7	3.5
Strengthened Wall # 1	203	35.9	3.0	36.4	3.3
Strengthened Wall # 2	213	35.9	3.0	39.0	3.3
Strengthened Wall # 3	220	35.9	3.0	38.6	3.4

Table 4.1 Concrete Cylinder Test Results

FRP Sheet

	MRL -T7 – 200W(RENO) Phase 2	CF150 (Master Builders) Phase 1
Weight per unit area	200 g/m ²	200 g/m ²
Tensile Strength	4800 MPa	3480 MPa
Tensile Modulus	230.5 GPa	230 GPa
Thickness	0.11 mm	0.11 mm
Ultimate Strain	1.7%	1.5%
Width of Sheet	50 cm	50 cm

Table 4.2 Comparison Table between Properties of CFRP sheets used during the Phase 1 and Phase 2 of the experimental program.

Epoxy Resin

	RENO	Master Builders
Pot life@25°C	60-120 min.	40 min.
Compressive Strength (MPa)	49	24
Tensile Strength (MPa)	17.65	12

Table 4.3 Comparison Table between Epoxy resins used during the Phase 1 and Phase 2 of the experimental program

	RENO	Master Builders
Pot life@25°C	60-120 min.	20 min.
Viscosity @25°C (cps)	600-1300	400

Table 4.4 Comparison Table between Epoxy Primer used during the Phase 1 and Phase 2 of the experimental program

Property	Range
Shear Strength (MPa)	15-35
Tensile Strength(MPa)	20-40
Tensile Modulus(GPa)	1-10
Tensile Strain to Failure (%)	1-4
Fracture Energy (kJ/m ²)	0.2 – 1.0
Coefficient of thermal expansion (10 ⁻⁶ °C ⁻¹)	30

Table 4.5 Typical Properties of Epoxy Adhesives

Side view of the reinforced concrete shear wall specimen

End view of the reinforced concrete shear wall specimen

Figure 4.1 Front and Side View of the Wall specimen

Figure 4.2 Side View of the Specimens' Steel Reinforcement

Figure 4.3 Front View of the Specimens' Steel Reinforcement

Figure 4.4 Footing and Wall steel Reinforcement Cage

Figure 4.5 Wall footing and Wall Vertical Steel Reinforcement

Figure 4.6 Specimen's Head Steel Reinforcement

Figure 4.7 Complete Formwork

Figure 4.8 Control Wall Specimen

Figure 4.9 Stress Strain Curve for No.10 steel rebars (experimental)

Figure 4.10 CFRP Sheets

Figure 4.11 Concrete Cylinder Test - Stress Strain Curve

Chapter 5

Experimental Set-up

The components of the experimental set up for this phase of testing include an actuator frame, a hydraulic actuator, and a lateral restraint frame which effectively prevents any significant out-of-plane deformations of the wall specimen during tests.

5.1. Lateral Restraint Frame

Observations from the first phase of this experimental program showed that the wall specimens experienced significant out-of-plane bending deformations due to lack of sufficient restraint of the specimens in the out-of-plane direction. This behaviour resulted in the reduction of the resistance and load carrying capacity of the shear wall in the in-plane direction, since it was now subjected to a combination of in-plane and out of plane loads. The resulting out-of-plane bending deformations have been found to have some significant effect on the overall behaviour of the shear walls. To avoid or minimize this torsional behaviour during Phase I of the experimental research program, a lateral restraint frame system was designed. The frame was analyzed and designed by using the Finite Element computer program Cosmos. The design load was a moment of 50kN-m resulting from a 500kN lateral load applied at the top loading block of the shear wall at the level of the actuator. The results of this analysis are presented in Appendix C. Figure C-1 illustrates the model frame of the analysis. The frame members were designed based

on the criterion of a maximum transverse displacement of 1mm when the frame was subjected to the maximum test loads. The lateral restraint mechanism consists of three parts. Parts one and two are frames each consisting of two columns of size W310x118 and a beam of size W250x89 and 2700 mm long. The frame is 3135 mm high and fixed to the strong floor by a mounting plate. Part three consisted of two vertical adjustable cantilever W beams were fastened at approximately the mid-span of the beam, with two rollers at the end (Figures 5.1 and 5.2). These beams were used to restrain the third part of the system which was a double angle L152 x 152 x 2, welded flange to flange, creating a T-section fixed at the top of the head of the wall specimen (Figure 5.3). The T-section was designed to transfer the torsional load from the wall specimen to the frames through the rollers, thus operating as a track. The restrained frame system prevents any torsional out-of-plane movements of the wall specimens during tests. Figures 5.3 and 5.4 illustrate this system.

5.1.1. Potentiometer Instrumentation

A total of eighteen potentiometers were utilized to monitor the displacement behaviour of the shear wall during test. The potentiometers were placed in various locations to measure in-plane and out-of-plane displacements (see Figure 5.5 for location of the potentiometers). At the contact point of every potentiometer a steel plate was attached to the concrete surface to minimize friction between the head of the potentiometer and the wall during testing, thus ensuring the accuracy of the readings. Mechanical oil was also applied on the plate for further reduction of friction (Figure 5.6).

5.1.2. In-plane Displacement

Three potentiometers were placed at the two ends of the wall in the in-plane direction, with one pair at the top of the wall, another pair at mid-height, and a third pair at the bottom, for a total of six potentiometers. The potentiometers at the bottom of the wall were installed as close to the footing as possible in order to obtain information of the base slip of the wall. Although the measurements of these potentiometers also include flexural and shear deflection displacements especially after the formation of plastic hinge at the base of the wall, observations of the wall deformation pattern indicated by the grid painted on the wall surface, show that the contribution of these displacement measurements due to flexure and shear deformations of the wall at such a small distance from the base is small. Consequently the base slip potentiometer measurement is a good indicator of the base slip behaviour of the test specimen. For the purpose of this study the displacement measured at the bottom of the walls is taken as base slip. The readings of the potentiometers at each level were averaged to obtain the displacement at each level. A 2mm thick steel angle 100mm wide, 250mm long vertical leg, adhered to the wall surface, and 150mm long horizontal leg, were glued to the two ends of the wall at the bottom part with construction adhesive, to maintain contact of the potentiometers with the concrete wall as the toe is expected to be crushed during tests as a result of high compressive stress in the concrete. Two potentiometers were placed at each end of the wall at the bottom in a vertical direction to measure the uplift of the wall. The end of the potentiometer was placed on the horizontal plate of the steel angle that was bonded to the end of the wall, ensuring that after the toe of the wall was crushed, a reading could still be obtained (see Figure 5.12).

5.1.3. Out of Plane Displacement

A total of eight potentiometers were utilized to measure the out-of-plane displacement of the specimen during testing. Potentiometers were placed on both longitudinal vertical sides of the head of the specimen. One potentiometer was placed at one end and the other at the opposite end, for a total of four potentiometers. The potentiometers were placed in a direction to measure any displacement of the head in the horizontal direction. Because of the positioning of these four potentiometers, the rotation of the head of the wall, and hence the top of the shear wall, could also be measured. The distance between the potentiometers on each side was 1500mm. Four potentiometers were also placed on the underside of the head, a pair on each end of the wall to measure the vertical displacement at the top, rotation of the head in the in-plane direction, and twist about the longitudinal axis of the head. The distance between the potentiometers at the two ends was 1500mm in the longitudinal direction. The distance of the potentiometers in the transverse direction was 300mm.

5.1.4. Shear Deformation

Two diagonal potentiometers were installed along the two diagonals of the shear wall (Figure 5.7, 5.8). In order to attach these potentiometers, an aluminum pipe was used as an extension at both ends to match the length of the diagonal. At the two ends of these pipes mounting plates were installed attached to a swivel-capable joint to allow the free movement of the potentiometers. These plates were fixed on the footing and the head of the wall specimen with tapcon screws (anchor-free), at locations corresponding to the two

diagonal corners of the shear wall, so as to have a length equal to the diagonal of the wall and therefore follow the deformations of the actual wall diagonal. The data obtained from these diagonal potentiometers have not been analysed. The focus of this study was the load transfer mechanism between the stressed FRP sheets and the footing of the wall. The data from the diagonal potentiometers and from the rosettes are available for future studies to determine the relative contributions of the FRP reinforcement system in the shear and flexural resistance of the shear wall.

5.2. Strain Gauges

The strain gauges used during this test were Showa 5mm. One of these strain gauges is shown in Figure 5.8. They were installed in the wall specimen to measure the strain on the flexural reinforcement of the shear wall at different levels. Rosette strain gauges were also installed on the surface of the wall on the top of the CFRP reinforcement sheet (Figure 5.9). The objective was to measure the shear strain of the shear wall specimen during the testing.

5.2.1. Reinforcement Strain Gages

One strain gauge was installed on each flexural rebar at the height 50mm from the footing for a total of twelve strain gauges. At the 250mm height-level, six gauges were installed along the full length of just one side of the wall. At the 975mm height level, three strain gauges were installed, with one on each of the two end rebars, and another one on the third rebar. At the 1750mm height-level, three strain gauges were installed,

following the exact same configuration as the configuration of the gauges at the 975mm height level. Figure 5.10 presents the installation layout of the strain gauges on the vertical reinforcement. In order to achieve the optimum bonding between the gauges and the steel reinforcement, the location where the gauges were to be installed on the rebar was ground to ensure a smooth and clean surface. Following the bonding of the strain gauges, they were sealed and covered with water-resistant latex material and further protected with a silicone overlay, and finally brushed with a cement-based mix to ensure that they were protected during the concrete casting process. All strain gauges were tested following installation, confirming that they were functioning properly. The strain gauges were also tested after the casting of the concrete of the wall section of the test specimen and found to be functioning with the exception of strain gauges No. 23 and 24 of strengthened wall No.2 test specimen, which probably got damaged either during pouring of the concrete or as a result of the concrete vibration process.

The strain gauges installed on the steel reinforcement provided data sufficient to obtain the load vs strain profiles at the base of the wall. These strain profiles are briefly discussed in section 6 for each wall. This information can be useful for future studies on the behaviour of FRP reinforced shear walls and the development of design equations for the application of the FRP reinforcing system.

5.2.2. Rosettes

Four rosette strain gauges were installed at the centre of the wall in a square configuration of a distance of 350mm. All rosettes were installed on the surface of the shear walls on top of the CFRP sheet. They were bonded on the cured epoxy resin

overlay of the CFRP sheet. Furthermore, the rosettes were tested following their installation with a potentiometer to confirm that the installation was successful and that they functioned properly. The data collected from the rosettes have not been analyzed in this study.

5.3. Loading Program

The reinforced concrete shear wall specimens were subjected to a slowly increasing static cyclic load pattern. The specimens were loaded by load control steps up to the calculated yield load. The load steps after yield were based on displacement control of increasing ductility level. The load steps were from ductility two to three and four, and then ductility six, eight, ten and onwards until complete failure and upon reaching 80% resistance reduction as compared to the maximum resistance of the wall specimen. Each load step consisted of two complete load cycles, namely the loading cycle, and the softening cycle. During the loading cycle but before yielding, the load step was load-controlled and the softening cycle was displacement-controlled. After yield, both load and softening cycles of each load step were displacement-controlled. Yield displacement for this experimental study was defined as the horizontal displacement reached at the top of the wall specimen when yielding of the wall specimen occurred. Displacement ductility level is defined as the horizontal displacement reached at the top of the wall over the yield displacement. Yield displacement for the purposes of this study is defined as the displacement measured at the top of the wall when yielding of the extreme flexural reinforcement occurs. Figure 5.11 shows a sample of the loading sequence for the wall specimens.

Figure 5.1 Lateral Restrain Mechanism (Front View)

Figure 5.2 Lateral Restrain Mechanism (Top Beam Detail)

Figure 5.3 Lateral Restrain Mechanism (Front View)

Figure 5.4 Lateral Restrain Mechanism (Side View)

Figure 5.6 Potentiometer with Sliding Plate

Figure 5.7 Diagonal Potentiometers

Figure 5.8 Strain Gauge Installed on a Vertical Rebar at 50mm Height

Figure 5.9 Location of Rosettes Installation

Figure 5.10 Vertical Reinforcement Strain Gauges Configuration

Figure 5.11 Typical Load Pattern

Figure 5.12 Potentiometers and Steel Angle Configuration at the Wall Base

Chapter 6

Experimental Results

This experimental study produced a large amount of test data about the behaviour of the reinforced concrete shear walls strengthened or repaired with externally epoxy bonded CFRP sheets when subjected to an increasing static cyclic loading until failure. These results are presented and discussed in the following sections of this chapter. Specifically, comments are made on the in-plane response of the wall specimens and the load vs flexural rebars strain relationship with data obtained from the strain gauges installed on the rebars.

6.1. Control Wall and Repaired Wall

6.1.1. General

The control wall used in this experimental study was first tested so as to obtain the control yield load and displacement, and the control maximum load. It was subsequently repaired to assess the effectiveness of the application of the externally bonded CFRP sheets as a repair method.

The control wall was loaded until the ultimate resistance was reached and continued the testing until its peak capacity dropped by 15%.

The repair of the control wall involved the removal of all crushed and loose material, and filling all gaps and cracks with an epoxy concrete repair mix. The surface of the wall was then washed with muriatic acid three times, and left to dry. It was then primed with epoxy resin primer. Once the epoxy primer cured, the CFRP sheets were applied on the wall by first applying a coat of epoxy resin on the primed wall surface, followed by the application of the CFRP sheets and the second coat of the epoxy resin to saturate the fibres of the reinforcing sheets. The repaired wall was tested to failure i.e. until reaching a 70 to 80% drop in resistance as compared to its maximum load carrying capacity. The test results of both the control and the repaired wall are presented in the next section.

6.1.2. Experimental Results

6.1.2.1. Control Wall

6.1.2.1.1. Load/Top Displacement-Observed Behaviour

The load vs top displacement hysteretic diagram with the displacement taken as the average of the readings from the potentiometers installed at the two ends is shown in Figure 6.1. All critical observed loads and displacement are presented in Table 6.1. The wall was subjected to a cyclic load increasing in a controlled manner based on load control until the yielding load, and then the load was controlled with the increase of the top displacement of the head of the actuator and based on the readings obtained by the top potentiometers, No.5 and 6. Two load cycles, loading and softening cycle, per load

step were applied to the wall specimen. The loading pattern was intended to apply to the specimen a load approximately 50% of the yield load which was calculated to be 107.25 kN for the first load step. The second load step was taken at 75% of the calculated yield load and the third at 100%. After yielding the applied displacement at the top part of the wall was increased in ductility level intervals based on the yield displacement observed during the test. As mentioned in Chapter 4, the control wall specimen is a well-designed shear wall. This is reflected in the load versus top displacement behaviour of the control wall (Figure 6.1) where at approximately 1.7 % drift the wall does not show significant strength degradation.

During the first load step, the applied load in the push direction was 58.78 kN corresponding to a displacement of 0.40 mm. The first cycle was completed with the application of a -57.63 kN load in the pull direction with a displacement of 0.40 mm. During the softening cycle of the first load step, a 50.4 kN resistance was reached for the applied displacement of 0.41 mm in the push direction, and -59.76 kN resistance was reached corresponding to a top displacement of 0.44 mm. No cracking was observed during this first load-step.

The specimen was subjected to an 85.57 kN load in the push direction during the first cycle of the second load step corresponding to a top displacement of 1.52 mm. It was then loaded with -85.57 kN in the pull direction being displaced by -1.21 mm. During this load step, flexural cracking occurred for the first time at a load obtained by the load

deflection graph of 67.66 kN and top displacement of 0.63 mm in the push direction.

Two flexural cracks were observed at the height of 430 and 610 mm. Three flexural cracks also occurred during the pull loading of this cycle, the first at a load of -59.31 kN and at a height of 300, 390 and 470 mm. The measured average stiffness of the specimen was calculated to be 152.21 kN/mm with an average cracking load of 63.48 kN. During the softening cycle of this load step two shear cracks of approximately 600 mm length occurred in both directions (push and pull) Figure 6.1a.

At the third load step the wall specimen was subjected to the calculated yield load. Specifically, a load of 113.5 kN was applied on the specimen in the push direction, corresponding to 2.44 mm of top displacement, and a load of -110.95 kN in the pull direction corresponding to a top displacement of -3.34 mm. Contrary to expectations, yielding of the extreme flexural reinforcement did not occur during this load step. During the softening cycle, two more flexural cracks occurred during the pull step of the cycle at 1000 and 1200 mm height.

In ensuring that the yielding of the extreme flexural reinforcement occurred in a way that the specimen's behaviour was controlled, the specimen was again loaded at 126.5 kN being displaced at the top by 3.45 mm. Yielding of the extreme flexural reinforcement did occur during this cycle of the fourth load step and at a load of 121.06 kN and a displacement of 2.92 mm. Yielding in the pull direction also occurred during this cycle 123.58 kN load and 3.09 top displacement. The average measured yield load was 122.32

kN and displacement 3.01(Ductility level 1) mm. The average secant stiffness at yield was 40.70 kN/mm.

During this load step two more flexural cracks occurred during the softening push cycle at 300 and 500 mm height, as well as a number of new shear cracks.

Load step # 5 was the last load step where the load controlled load was applied. A load of 137.41 kN was applied in the push direction corresponding to a 4.3 mm top displacement. The load in the pull direction was -135.52 kN with a displacement of -4.1 mm. The strain gauges on the extreme reinforcement indicated the following values at the pick load: No.1(0.003782), No.7(Failed), No13(0.002740), No.5(0.001935), No.6(0.001975), No11(0.002784), No.12(0.002681). Any strain gauge reading with values over 0.0024 indicates occurrence of tensile rebar yielding.

Starting at load step # 6, the loading was controlled by displacement control so as to avoid a catastrophic damage to the wall and to be able to observe the behaviour of the wall gradually. The target displacement was 6mm corresponding to ductility level 2. The resistance of the wall corresponding to the applied displacement of 6.20 mm was 158.96 kN. During this load step strain gauges No. 6 & 7 failed and the reading from strain gauges No. 11 & 12 was 0.004605 and 0.004762 respectively. A new flexural crack occurred at 1500 mm height, and a large shear crack opened at the center of the wall approximately 600mm long and 0.5 mm wide. In the pull direction the applied top displacement was 6.00 mm and the corresponding load was -155.82 kN. Strain gauge

No. 1 failed and strain gauge No.13 read 0.002715. During the softening cycle of this load step the same top displacement was applied in the pull and push direction and the resistance was decreased 8% in the push and by 3.4% in the pull direction.

The seventh load step consisted of the application of a +/- 9 mm (Ductility Level 3) top displacement. During the load cycle of this load step the applied displacement in the push direction was 9.13 mm. The specimen's resistance was 180.03 kN. The applied displacement in the pull direction was -10.47 mm corresponding to a -180.43 kN load during this load cycle. A large crack along the construction joint between wall and footing (pull out crack) was observed in both push and pull directions along the construction joint between the wall and the footing. Flexural cracks had reached a width of 1-1.5 mm. The softening cycle of this load step demonstrated a drop in the resistance of the specimen of an average of 12.65 %.

Load step # 8 involved the application of a top displacement of +/- 12 mm (Ductility level 4). The applied displacement in the push direction was 12.13 mm. The specimen's resistance was measured at 196.09 kN. In the pull direction the specimen's resistance was -175.28 kN and the applied top displacement was -12.10 mm during the softening cycle an average decrease of 8.1% was observed in the load carrying capacity of the specimen. Furthermore, the existing flexural cracks reached a width of approximately 2 mm.

During load step # 9 the specimen was subjected to a displacement of ± 15 mm (Ductility level 5). In the push direction a 15.22 mm top displacement was reached. The specimen demonstrated a resistance of 212.66 kN which was also the peak load in the push direction. In the pull direction, a -15.20 mm displacement was applied corresponding to a -178.44 kN load. During the second cycle of this load step in the resisting capacity of the specimen there was a drop of 7.3% on average. Concrete spalling was observed during this load step between two flexural cracks and a shear crack.

The displacement for load step # 10 was raised to a ductility level 6, corresponding to a ± 18 mm top displacement. The applied displacement in the push direction was 18.23 mm loading the specimen with 205.90 kN. Furthermore, concrete crushing occurred during this load cycle at a load of 184.47 kN and top displacement of 15.24 mm. The resistance of the wall in the pull direction was -177.56 kN for a 18.05 mm displacement which was the peak load in this direction. Toe crushing also occurred during this part of the load cycle at a load of -117.58 kN and -7.46 mm displacement. Concrete spalling and exposure of the stirrups of the vertical wall reinforcement on the compression zone occurred. The flexural cracks reached a width of 3 mm (see through). The uplift of the wall from the footing at the construction joint reached a 5 mm width. The decrease in the resistance of the wall during the softening cycle of this load step was 11.4 % for the push, and 5.4% for the pull direction. The extent of the damage of the specimen at this stage of the testing is illustrated in Figure 6.1b.

The loading continued so as to reach a reduction in the load carrying capacity of the specimen of 10 to 15% compared to the maximum load. At load step # 11 the wall was subjected to a displacement of +/- 21 mm. The resistance reached in the push direction was 201.13 kN for a displacement of 21.23 mm. A -21.00 mm displacement was applied in the pull direction, generating a resistance load of -175.74 kN. During the softening cycle the resistance of the specimen decreased by an average of 7.5%. More cracking and compression crushing of the toe uncovering the extreme vertical reinforcement rebars occurred during this load step.

Load cycle 12 involved the application of +/- 24 mm top displacement. The resistance reached during this cycle was 199.2 kN at 24.9 mm and -172.32 at -24.2 mm in the push and pull direction respectively. During this load cycle crushing of the concrete at the base of the wall was observed. The load carrying capacity was reduced at the softening cycle by an average of 8.3%.

For the last load step of the control wall testing the displacement was set to the ductility level of 9, corresponding to a displacement of 27mm. In the push direction the resistance for 26.98 mm displacement was 185.92 kN, corresponding to a 12.5 % drop of the load carrying capacity as compared to the maximum load. The resistance of the specimen at the displacement of 15.22 mm where peak load was reached was 103.93 kN, corresponding to 51% of the peak load. The testing of the control wall specimen stopped

at this stage due to the extensive damage that the wall had sustained and the fact that as the loading continued the wall could not be repaired with the available lab resources. The secant stiffness of the specimen at this last stage was 6.87 kN/mm. Figure 6.1c and illustrates the extent of the damage that the wall specimen sustained during this test.

6.1.2.1.2. Load vs. Mid-level Displacement Relationship.

The mid level response of the specimen is illustrated in Figure 6.2 in the hysteretic diagram of load versus mid displacement. Comparing this diagram to the load versus top displacement hysteretic diagram it is observed that the mid level was similar to the top level response with a magnitude that averaged 44.76% of the top displacement. The location of the potentiometer measuring the mid level response in the in-plane direction was at a 975mm height from the footing of the specimen. This corresponded to 53.31% of the total wall height. This potentiometer was located 788 mm from the one measuring the top wall deflection.

6.1.2.1.3. Load vs. Base slip Relationship

The load versus base slip relationship is obtained by the average of two measurement readings from the two potentiometers, one at each end of the wall in the in-plane direction positioned at a height of 100mm. This was done so as to ensure a reading from the potentiometers even following the crushing of the concrete toe due to compression.

The slip of the base of the specimen is presented in Figure 6.3. The base slip measured before the yielding of the vertical reinforcement occurred was 8.46% of the corresponding measured deflection at the top of the wall. The average base slip following yielding of the vertical reinforcement was 5.13 % of the corresponding deflection at the top of the wall. The horizontal deflection was generally averaging at 5.33 % of the deflection as measured at the top of the wall.

6.1.2.1.4. Load vs. Rotation Relationship

The total rotation of the wall was measured at the top using four potentiometers. These potentiometers were installed in pairs (Potentiometers 7-8, 11-12) at the top end of the wall. The average value of the readings of each pair was used in the calculations for the top vertical displacement and the total wall rotation. The ratio between bottom rotation and total rotation was 87.9 % and 50.25% before the wall cracking and yielding point, respectively. The percentage of top versus bottom rotation after the wall had yielded was calculated at 40.69%.

6.1.2.1.5. Load vs. Strain Relationship

Figure 6.4 presents the strain profile of the wall at the peak displacement and for the first seven load steps. Most strain gauges failed after this point and the data obtained could not be used.

The data obtained by the strain gauges installed on the flexural reinforcement provided the strain profile presented in Figure 6.4. It was observed that the behaviour of the wall was almost elastic until yielding of the extreme flexural reinforcement. The location of the neutral axis far from the centroid at the middle of the wall section, clearly indicates that the plain section remain plain assumption of the simple beam theory does not apply to walls. Figures 6.5 and 6.6 present the load versus. strain curve for strain gauges # 6 and 1 which were installed on the extreme flexural reinforcement at a 50 mm distance from the footing of the wall.

6.1.2.2. Repaired Wall

The control wall was repaired after it had been damaged to evaluate the effectiveness again of the CFRP sheet reinforcement method for stiffness recovery of the shear wall subjected to lateral load. The repair of the damaged control wall consisted of the removal of the crushed and loose concrete parts and the filling of the empty pockets by an epoxy concrete repair mix. A more detailed or extensive method for repairing the damaged control wall was not applied given that the objective was the determination of the effect of the application of the CFRP sheets on the wall behaviour, rather than the effect of other aspects of the repair method (e.g. epoxy injection at the shear and flexural cracks). The response of the repaired wall is described in the following sections.

6.1.2.2.1. Load / Top displacement - Observed Behaviour

The load versus top displacement hysteretic diagram is presented in Figure 6.7.

The loading of the repaired wall started with a 70kN load, which corresponded to the 50% of the calculated extreme flexural reinforcement yield load.

During this first load step, the load applied in the push direction was 70.5 kN corresponding to a displacement at the top of the wall of 1.15 mm. In the pull direction a -70.3 kN load was applied causing a -1.091 mm top displacement. Furthermore, three pre-existing flexural cracks at 340, 390 and 480 mm height re-opened. During the softening cycle of the first load step the resistance of the wall was reduced by 9.78%. Load step 2 consisted of the loading of the wall specimen with 75% of the calculated yield load. A load of 104.6 kN was applied in the push direction, displacing the top of the specimen by 3.31 mm. A pre-existing flexural crack at 365mm height re-opened during this load cycle. A -2.03 mm displacement was measured in the pull direction as a result of the -105.2 kN load that was applied during this load step. A drop of 9.2 % and 2.5% in the push and pull direction respectively, was measured during the second load step's softening cycle.

The next load step involved the application of a load equal to the calculated yield load. A load of 140.1 kN was applied in the push direction deflecting the wall at the top by 5.83 mm. The wall was then loaded in the opposite direction (pull) up to a load of -140. kN, which corresponded to a top displacement of -3.27 mm. Compression debonding was observed during this load step at the location of pre-existing flexural cracks, and at a 450

mm height, starting from the extreme compression fibre and extending to 500mm inside the wall depth on both sides of the wall (front and back). The softening cycle of this load step followed and a reduction in the resistance of the strength of 8.6% in the push and 5.5 % in the pull direction was measured. During the softening cycle the compression debonding phenomenon occurred to a greater extent at the locations of pre-existing flexural cracks, and at heights of 100mm and 500 mm on both sides of the wall with a length of 200 and 500 mm respectively.

During load step 4 the specimen was loaded with 125 % of the calculated yield load and therefore 175 kN and -175 kN were applied in the push and pull direction respectively. The top displacement measured at this load was 9.12 mm in the push, and -4.75 mm in the pull direction. An increase in the range of 10% was observed in areas where compression debonding occurred. During the softening cycle the resistance of the wall specimen for the same maximum displacement decreased by an average of 1.6%. From the Load versus Top displacement diagram it was determined that the yield displacement was 4.93 mm in the push direction and -3.81 mm for the pull direction. These are the displacements used to determine the ductility level of this specimen during testing.

Starting from load step 5, the loading of the test specimen was based on displacement control. A displacement of 12 mm in the push and -7 in the pull direction was applied, corresponding to resistance of 188 kN and -140.1 kN respectively. During this load step,

two pre-existing cracks re-opened at heights of 620 and 710 mm. A softening cycle completed this load step with the resistance of the wall dropping by an average of 6.6%.

Load step 6 had target applied displacements of 15 mm and -8.75 mm, corresponding to ductility 3.04 in the push and 2.29 in the pull direction. The measured resistance of the wall specimen was 209.2 kN and -226.4 kN respectively. The compression debonding observed at this stage of the testing was spread more and localized around pre-existing cracks as shown in Figure 6.7a. During the softening cycle the same displacement as the first cycle was applied on both directions and the resistance of the specimen was reduced by 11.9 % in the push and 4.3 % in the pull direction.

Load step 7 consisted of the application of a 18 mm peak displacement at the top of the specimen in the push direction. The measured resistance at this displacement was 202.2 kN which was lower than the resistance that was measured during the previous load step by 7 kN. This decrease can be attributed to the crushing of the wall toe during this load step cycle, and thus the shifting of the neutral axes and lowering of the resistance at this stage. A top displacement of -10.7 mm was applied in the pull direction resulting in a -208.44 kN load. During this loading stage and at displacement -9.25 mm, one extreme flexural rebar failed causing a drop in the resistance from 222.3 kN to 199.6 kN, for applied displacement of -10.22 mm. The resistance of the specimen increased again from that point on, reaching -208.44 kN. The softening cycle followed and a reduction in the resistance of the specimen for the same applied displacement, of 5.3% was measured in

the push direction. In the pull direction the measured resistance for displacement of -10.55 mm was -223.9 kN corresponding to an increase in the specimen's resistance of 6.9%. This increase can be attributed to the shifting of the neutral axes at the base section of the specimen due to the rupture of the flexural rebar occurring during the first load step, and the engagement of a larger area of CFRP.

At load step 8 the target was to apply displacement 24 mm in the push and 14 mm in the pull direction, corresponding to ductility 4.86 in the push and 3.67 in the pull direction. The resistance measured 260.1 kN in the push and -269.9 kN in the pull direction. During the first push cycle of this load step, a strip of CFRP that was 20 mm wide at the edge of the extreme tensile fibre ruptured at displacement of 22.9 mm and load 247.88 kN. The resistance of the specimen dropped to 240.05 kN instantly and then increased again to reach the peak load of 260.1 kN. During the softening cycle that followed, the specimen demonstrated a reduction of 13.7 % in the resistance in the push and 6.6 % in the pull direction.

During Load step 9 the maximum resistance in the push direction was reached. The target top displacement was 30 mm at a ductility 6.08, and the load reached was 279.3 kN at displacement 29.79 mm when rupture of a 55mm wide CFRP strip ruptured. With this rupture, the resistance instantly decreased to 262.01 kN. The load was then increased to reach 271.86 kN at 30.07 mm displacement. In the pull direction the target displacement was -17.5 mm at ductility 4.59. At displacement -16.68 mm and corresponding load of -

272.22 kN, a 10mm wide strip of CFRP ruptured causing a reduction in the resistance of 2.7%. The resistance of the wall continued to increase reaching -283.87 kN at displacement -17.4 mm. At this point a 70 mm wide CFRP strip 150 mm from the extreme tensile fibre failed in tension. This resulted in a drop in the load carrying capacity of the specimen of 7.17% from -283.87 kN to -262.65 kN. The loading continued until reaching displacement of 18.4 mm, which corresponded to a load of -281.15 mm. During the softening cycle, the resistance of the specimen was reduced by 6.8% in the push and 7.9% in the pull direction.

During load step 10, the peak displacement was 36 mm in the push and -21 mm in the pull direction. The load corresponding to the maximum displacement was 261.6 kN in the push and -281.3 kN in the pull direction which was also the ultimate load of the specimen in this direction. Toe crushing was observed during this load cycle in the pull direction as shown in Figure 6.7b. During the softening cycle, the resistance of the specimen was reduced by 18.39% and 10.32 % in the push and pull direction respectively.

Load step 11 involved the application of displacement corresponding to ductility 8.54 in the push and 6.46 in the pull direction. The maximum displacement applied in the push direction was 42.1 mm resulting in a resistance of 183.8 kN. Two 20 mm wide CFRP strips, one at each side of the wall, failed during the application of the displacement in the push direction. A maximum displacement of -24.6 mm was applied in the pull direction

corresponding to a measured load of -262.2 kN. The softening cycle that followed resulted in a reduction of the load carrying capacity of the wall of 38.7 % in the push direction and 12.6% in the pull direction.

During load step 12 a displacement corresponding to ductility 9.74 in the push and 7.32 in the pull direction was applied. The specimen was subjected to a top displacement of 48 mm in the push direction and the measured resistance at this displacement was 113.6 kN. In the pull direction the applied -27.9 mm displacement resulted in a -228.76 kN resistance. During this load step failure of CFRP strips occurred due to the additional strain that was applied on the fibres from concrete rubble produced as a result of the compression failure and spalling of the wall. The softening cycle demonstrated a reduction in the resistance of the specimen of 8.8% and 18.2 % in the push and pull direction respectively.

The maximum displacement applied on the specimen during load step 13 was 54 mm and -32 mm in the push and pull direction respectively. The maximum load reached at this load step was 128.5 kN in the push and -212.32 kN in the pull direction. The load carrying capacity of the wall during the softening cycle was lower by 32.8 % in the push and 26.4% in the pull direction.

During load step 14 a ductility of 12.17 in the push and 9.31 in the pull direction was reached. The wall was subjected to a displacement of 60 mm in the push direction

corresponding to a 110.65 kN load. A displacement of -35.5 mm was applied on the wall in the pull direction resulting in a -172.2 kN resistance load. The softening cycle resulted in a reduction in the resistance of the load carrying capacity of the wall of 34.5% in the push and 23.1% in the pull direction.

During the final load step the wall was subjected to a displacement corresponding to a ductility level of 12 in the push and 10.41 in the pull direction. The applied displacement of 59.2 mm in the push direction resulted in a resistance of 71.84 kN. A displacement of -40 mm was applied in the pull direction and the corresponding resistance was -124.24 kN. The softening cycle that followed demonstrated that the resistance of the specimen was not reduced. This phenomenon was mainly attributed to the dowel action of the vertical steel reinforcement. It was therefore decided that the testing of the specimen should stop.

The load versus top displacement diagram of the repaired wall (Figures 6.7) shows a delay in the contribution of FRP in the push direction compared to the pull direction. This is attributed to larger plastic deformations in the push direction sustained by the wall during the control test probably due to the slight shift of the reinforcement cage during the concrete placement of the specimens.

6.1.2.2.2. Load vs. Mid-level Displacement Relationship

The mid level response of the specimen to the cyclic loading that was applied during the test is presented in Figure 6.8. The general behaviour of the wall at mid level is very

similar to the behaviour observed and recorded at the top of the wall. By comparing the data of mid level and top level response of the wall, it was calculated that on average, the mid level displacement corresponded to 59.07% of the top displacement response at the top of the wall.

6.1.2.2.3. Load vs. Base Slip Relationship

Figure 6.9 illustrates the behaviour of the wall at the base. The displacement of the wall at the bottom was 14.08% on average compared to the displacement of the wall at the top. During load step 5 yielding of the extreme vertical reinforcement occurred. Until and including load step 5, the base slip values were averaging 11.5% of the top displacement. After that point this percentage increased to 15.8%. The hysteretic loop was larger at this stage showing that a larger amount of energy was dissipated at the base of the wall.

6.1.2.2.4. Load vs. Rotation Relationship

Figure 6.10 shows the total top rotation vs. load hysteretic curve. The rotation at the bottom of the wall was calculated to be 40.06% of the total top rotation just before cracking. When yielding of the vertical steel reinforcement occurred the rotation at the bottom of the wall was found to be 50.32% of the total top rotation. The ratio of bottom versus total top reinforcement was then reduced to 56.87%.

6.1.2.2.5. Load vs. Strain Relationship

Due to the fact that this specimen was tested first as a control wall, it follows that the strain gauges on the reinforcement were damaged and their reading can not therefore be considered reliable. Therefore a strain profile diagram cannot be produced for this test.

6.2. Strengthened Wall # 1

The reinforced wall # 1 had a single ply of carbon fibre sheet on each side of the wall. The calculated maximum load carrying capacity of the specimen was 340.6 kN. The maximum load that was reached during the test was 313.24 kN in the push direction, and -274.88 kN in the pull direction. These results correspond to an average increase of the load carrying capacity of the wall as compared to the control specimen of 49.8%. The hysteretic diagram illustrating the response of the specimen at the top is shown in Figure 6.11. The behaviour and response of the specimen during testing is described in the following sections.

6.2.1. Load vs. Top Displacement Relationship

The objective for the load pattern was to subject the specimen to predetermined load steps with load corresponding to 50%, 75%, 100%, and 125% of the extreme tensile steel reinforcement calculated yield load. After the yield point was derived from the Load

versus. top displacement plot, the loading of the specimen was based on displacement control in multiples of the yield displacement.

The first load step consisted of the application of a 113.1 kN load in the push direction, which corresponded to a top displacement of 1.15 mm. In the pull direction a -120.9 kN load was applied resulting in a displacement of -1.39 mm. Three flexural cracks were observed at this stage at heights of 320 mm 400 mm and 560 mm from the footing. During the loading cycle that followed, a load of 141.13 kN was applied in the push direction resulting in a 2.12 mm displacement. A -141.03 kN was applied in the pull direction, corresponding to a displacement of -2.02 mm. A softening cycle followed demonstrating a reduction that averaged less than 1% in the resistance of the specimen. Although the calculated yield load was reached, yielding of the specimen had yet to occur.

The second load step consisted of loading of the specimen until reaching a resistance of 155.11 kN in the push and -150.3 kN in the pull direction, with the corresponding displacements being 2.47mm and -2.30 mm, respectively. During the application of the load in the pull direction the readings of the strain gauges went beyond the yield strain for the two extreme tensile rebars. The softening cycle followed during which the specimen showed a reduction in the load carrying capacity of 3.22 % in the push and 6.9 % in the pull direction.

During load step #3 the specimen was loaded with 155.11 kN in the push and -150.3 kN in the pull direction. The wall was displaced at the top by 2.47 mm and – 2.38 mm respectively. The specimen during the softening cycle of this load step demonstrated a reduction in the load carrying capacity of 4.2 % in the push and 4.6% in the pull direction. During the pull softening cycle, the reading of the strain gauges on both extreme tensile rebars exceeded the yield strain.

Load step 4 consisted of the application of 171.2 kN load in the pull and 167.9 kN load in the push direction, resulting in a displacement of 3.06 mm and -3.46 mm respectively. The load carrying capacity of the strengthened wall was reduced by 7.9% in the push and 4.56% in the pull direction, when the same displacement resulting from the load mentioned above was applied on the wall. New flexural cracks were observed when the wall was subjected to load in both the pull and push direction. The CFRP sheets were clearly under stress; as load was applied on the specimen the sheets made crackling sounds.

In load step # 5 the wall was subjected to a displacement load of 5.2mm in the push and - 5.1 in the pull direction. The measured resistance of the strengthened wall was 216.63 kN and -177.12 kN in the push and pull direction respectively. More flexural cracks opened during this load step. Debonding between CFRP sheets and concrete wall surface occurred in places where space existed between the sheet and the anchoring pipe (the anchoring pipe was not in full contact with the wall). The resistance of the specimen

reduced by 9.83% and 7.68% in the push and pull direction respectively when the wall was subjected to the same displacement level for a second time.

Load step # 6 had as target the application of displacement ± 7.4 mm top displacement. This displacement was applied in the push direction resulting in a load of 235.42 kN. A displacement of -7.23 mm was applied in the pull direction and the wall displayed a resistance of -188.91 kN. New flexural cracks occurred during this load cycle in the pull and push direction and at a vertical distance of 130 mm from the wall's footing. During the softening cycle of this load step, the specimen's load carrying capacity decreased by 12.29% in the push and 7.48% in pull direction. Furthermore, compression debonding of the CFRP sheets from the wall concrete surface was observed, in areas where flexural cracks had already occurred.

During load step 7 the specimen was subjected to a displacement of 9.71mm top displacement in the push and -9.3mm in the pull direction. The measured resistance of the wall was 247.74 kN and -195.64 kN respectively. A large pull out crack occurred at the construction joint between wall and footing during the application of the load in the pull direction. The softening cycle followed and the wall demonstrated a resistance decrease of 8.99% in the push and 6.37% in the pull direction. The debonding of the CFRP sheet from the surface of the concrete wall continued. Debonding started from the base of the wall and progressed towards the top. The debonding strips continued to carry load as long as some adhesion to the concrete wall remained.

For the next load step the wall was subjected to a displacement load of 13.7 mm in the push and -13.94 mm in the pull direction. The resistance of the wall was measured at 295.71 kN in the push direction. In the pull direction a resistance of -232.18 kN was reached for applied displacement of -13.62 mm when debonding failure of a 40 mm wide CFRP strip occurred, resulting in an instant decrease in the resistance reaching 196.69 kN. Figure 6.19 illustrates this CFRP debonding failure. The resistance of the wall increased as the displacement increased, reaching 223.23 kN for displacement of 13.94 mm. During this load cycle the first signs of concrete compression failure were observed on both sides of the wall with a few aggregates from the concrete being detached. The load carrying capacity of the wall was further reduced during the softening cycle of this load step by an average of 16.27%.

The maximum load carrying capacity in the push direction was reached during the first load cycle of load step # 9. The wall was subjected to a displacement load of 18.69 mm. At that displacement the resistance of the wall was 313.24 kN which corresponded to the maximum load carrying capacity of the wall. At this displacement level tension failure of a 60mm wide CFRP strip occurred at the front and 220 cm wide CFRP strip at the back. The resistance of the wall dropped instantly to 289.33 kN. In the pull direction the wall was displaced by -18.69 mm and the resistance was measured at -234.93 kN. A 30 mm wide CFRP strip failed 180 mm from the edge of the wall at the peak displacement, resulting in an instant drop in the resistance of the wall to -218.68 kN. The wall was then

subjected to a softening load cycle and a decrease was observed of 14.87% and 11.48% in the push and pull direction respectively.

In load step # 10 the wall was first subjected to a displacement in the push direction of 23.35 mm, corresponding to a load of 302.6 kN. A 170mm wide CFRP strip at the front side of the wall as well as a 100mm wide one at the back failed in tension. A displacement of -22.95 mm corresponding to a resistance of -240.34 kN was applied to the specimen in the pull direction. Compression failure of the concrete toe occurred during this load cycle. In the pull direction a displacement of -23.35 mm was applied on the wall. A 70 mm wide CFRP strip failed in debonding from the concrete surface. The wall's peak resistance was measured at -246.04 kN. The softening cycle resulted in a drop in the load carrying capacity of the wall by 10.03% in the push and 17.58% in the pull direction.

Load step # 11 was the loadstep where the maximum resistance of the wall in the pull direction was reached. The wall was first subjected to a displacement in the push direction of 27.03 mm. The resistance was measured at 298.32 kN. The CFRP strips suffered additional debonding failure at the front side of the wall. In the pull direction the wall was subjected to a displacement of -27.38 mm resulting in the maximum load in this direction measuring -255.26 kN. A 90 mm wide strip of CFRP sheet failed in debonding from the wall's surface 180 mm from the edge of the wall. The load carrying capacity of the wall was reduced by 18.64% in the push direction during the softening

cycle that followed. In the pull direction a tensile rebar failed at -26.99 mm displacement and load -220.66, resulting in drop of the wall's resistance to -188.07 kN at -28.27 m displacement.

During load cycle # 12 the failure of the CFRP sheets continued, resulting in sudden loss of load carrying capacity. At 29.04 mm of displacement in the push direction and 257.15 kN load, a 70 mm wide CFRP strip located at the back of the wall 200 mm from the edge of the wall failed in tension. This resulted in a drop of the load carrying capacity of the wall to 251.86 kN at 29.79 mm of displacement. The wall was further pushed until reaching a top displacement of 32.66 mm which corresponded to a resistance of 273.54 kN. The specimen was subsequently pulled to a displacement of -26.08 mm with a resulting load of -183.03 kN. Debonding failure of a CFRP strip occurred at this point, dropping the resistance of the wall to 17.24 kN at 27.16 mm of displacement. The displacement loading of the specimen in the pull direction continued and the resistance reached was 202.51 kN at 32.17 mm displacement. The softening load cycle followed, resulting in a drop of the wall's load carrying capacity in the push direction of 16.45%. In the pull direction, at displacement of -30.17 mm and load 193.00 kN, a tensile rebar failed resulting in a decrease in the resistance of the wall to 149.58 kN at 31.30 mm top displacement. The specimen was further pulled reaching a resistance load of -162.53 kN at -32.72 mm of displacement. Compression failure and crushing of the concrete toe also occurred at this stage of the test in the pull direction.

Load step #13 involved the application at the top of the wall of a +/- 36.8 mm displacement. The wall was first loaded in the push direction and a tensile rebar failure occurred at 32.8 mm displacement and 211.03 kN load. The resistance of the wall dropped to 198.84 kN at 33.95 mm. The loading of the wall continued and the peak resistance of 220.06 kN was reached at 36.82 mm of top displacement. The wall was then loaded in the pull direction and at displacement of -36.6 mm and -219.77 kN load further CFPR debonding occurred, resulting in a instant drop of the load carrying capacity to 189.87 kN at 36.87 mm. The only CFRP reinforcement remaining active at the front side of the wall was a 450 mm strip in the centre of the wall. The loading continued with the softening cycle, where the peak resistance reached was 164.98 kN at 37.06 mm and -149.36 kN at -36.8 mm in the push and pull direction, respectively.

During the next load step the wall was subjected to a 41.94 mm displacement in the push and -42.06 mm in the pull direction. These displacements resulted in 170.54 kN and -179.48 kN resistance loads respectively. The softening load cycle demonstrated a drop in the load carrying capacity of the wall of 17.39% in the push direction and 12.20% in the pull direction. The CFRP strip remaining on the front side of the wall continued debonding.

Load step # 15 targeted the application of +/- 46 mm top displacement on the wall. While the wall was subjected to the push load, a tensile rebar failed at 40.23 mm displacement and 114.99 kN load. This resulted in a sudden drop in the resistance

measured at to 101.14 kN at 41.18 mm. The loading of the wall continued and the peak resistance was measured at 125.88 kN corresponding to a 46.00 mm top displacement. A -46.05 mm displacement was applied in the pull direction and the wall's resistance was measured at -168.9 kN. For the same displacement load, the wall's resistance was reduced during the softening cycle by 14.88% and 21.21% in the push and pull direction respectively.

During load step #16 the wall was subjected to a displacement load of +/- 50.6 mm. In the push direction the resistance reached was 142.33 kN at 50.18 mm top displacement. While loading in this direction a 40 mm wide CFRP strip ruptured, 640 mm from the tensile edge at the back of the wall. Three strips failed in debonding from the concrete surface, each being 20 mm wide and at 840 mm, 880 mm, and 900 mm from the tensile edge at the front side of the wall. This resulted in a significant weakening of the wall, displaying a drop in the resistance during the softening cycle in the push direction of 32.63%. In the pull direction a displacement of 50.66 mm was originally applied resulting in a resistance of -146. The softening cycle in the pull direction displayed a drop only in the load carrying capacity of the wall by 16.02%. The wall had now sustained significant concrete damage, namely, concrete spalling in the compression areas where the tensile rebars were raped with stirrups; extensive compression failure at the toe of the wall; and extensive shear cracks that could now be observed since the CFRP sheet on the front side of the wall had failed in debonding and the concrete surface became uncovered.

For the next load step # 17, the wall was loaded in the pull direction with a target displacement of 55.2 mm. While the wall was loaded at 52.83 mm displacement and 97.55 kN load, the last remaining CFRP strips failed in tension and debonding, instantly reducing the wall's load carrying capacity in this direction to 84.15 kN at 53.55 mm. The wall was then pulled to a target displacement of -55.2 mm and the resulting resistance was -119.56 kN at -55.67 mm displacement. During the softening cycle that followed there was an average reduction in the wall's load resistance of the wall of 42.54%.

The last load step#18 involved the application of a 59.8 mm displacement load in the push and -59.8 mm in the pull direction. The resistance of the wall was measured at 60.17 kN in the push and -90.3 kN in the pull direction. With the softening cycle the resistance of the wall was reduced by 11.7 % on average.

During this last load step, it was observed that the displacement of the wall specimen at the top and mid level had identical values. Therefore the resistance of the wall was due only to the dowel action of the remaining tensile rebars and the aggregate interlock. It was decided therefore to cease testing at this stage.

The resistance of the wall was reduced by 75.85% on average, as compared to the average maximum load carrying capacity of the wall.

Figures 6.20 and 6.21 illustrate the extent of the damage that the strengthened wall # 1 sustained during this test.

The failure mode was in conformity with the analytical model, namely with the CFRP tensile failure initiating the wall failure, followed by compression failure of the concrete toe and failure of the tensile reinforcement.

6.2.2. Load vs. Middle Displacement Relationship

The mid level response of strengthened wall # 1 is presented in Figure 6.12 with the Load versus Mid Level deflection curve. Comparing the load versus top deflection and mid level deflection curves it is observed that the response at the top of the specimen and at mid level follows the same pattern. The deflection measured at mid level is on average 46.72% of the deflection measured at the top.

6.2.3. Load vs. Base Slip Relationship

Figure 6.13 presents the response of the wall at the level of the base as a function of the applied load. Comparing the response at the top of the wall with that at the bottom, it was calculated that the displacement at the bottom corresponded to 9.73% of the top displacement at the cracking point. This percentage was calculated at 8.97% when yielding of the flexural reinforcement occurred. During the following cycle, the base displacement corresponded to 9.24% of the total top displacement.

6.2.4. Load vs. Rotation Relationship

The load versus top rotation and load versus bottom rotation relationship of strengthened wall #1 is presented in Figures 6.14 and 6.15 respectively. At the cracking load level, the bottom rotation corresponded to 52.19% of the total rotation. When yielding of the flexural reinforcement occurred, the rotation at the bottom of the wall corresponded to 45.77% of the total. This percentage was also calculated during the cycle following the one during which yielding occurred and was calculated at 51.48%. In general, the rotation of the wall at the top and bottom was calculated by finding the vertical displacement differential between the two ends of the wall and dividing this number with the length of the wall (1500mm). In the case of the top rotation where two potentiometers were installed one at each end of the wall, the average of the readings of the potentiometers on each side was used for the calculation.

6.2.5. Load vs. Strain Relationship

Figure 6.16 shows the strain profile of strengthened wall # 1 for the first 6 load step. After that point the strain exceeded the capacity of the strain gauges and their readings could not be relied on. In agreement with what observed in the control wall load versus strain graph, the section of this wall corresponding to a 50 mm height from the footing of the wall seems to have similar behaviour, where the plane sections for the first 4 load steps remain relatively plane. Following the fourth cycle, the strain distribution readings for the wall section become more complex. This change in the strain distribution

indicates that the simple beam theory is not applicable. The load versus strain curves of strain gauge # 1 and 6 installed on the two opposite extreme tensile rebars are presented in Figures 6.17 and 6.18 respectively.

6.3. Strengthened Wall # 2

Strengthened wall # 2 was reinforced with two vertical layers of CFRP sheets at the front and backside of the wall. The application process for these sheets is discussed in section 3.2. The calculated maximum load was 424.8 kN. The experimental max load was 426.39 kN and -412.43 kN in the push and pull direction respectively. The specimen was tested to failure and until reaching an 80% decrease in the load carrying capacity as compared to the maximum. The load pattern was for the first six load steps based on load control, and continued with displacement control until the end of the test. The response of the wall specimen is described in detail in the five following sections. This wall failed prematurely due to delamination of the CFRP sheets. The premature failure can be attributed either to problems in the installation of the CFRP sheets, such as insufficient epoxy application, or crushing of concrete at the diagonal corners of the wall resulting from diagonal compression due to shear. Observations of the wall tests indicate the first reason is more likely the cause of the premature failure.

6.3.1. Load vs. Top Displacement Relationship

The response of strengthened wall # 2 at the top is shown in Figure 6.22, Load versus. top deflection relationship curve.

The loading of the specimen started with the application of 95.5kN in the push and 95.6kN in the pull direction, corresponding to 50% of the calculated yield load. The response of the specimen was 0.3 mm and -0.56 mm respectively. No flexural cracking occurred during this first load cycle. The load step was continued with a target of +0.3 and - 0.55 mm of displacement. While loading, it was observed that at displacement of 0.17 mm the load was already 96.2 kN and the loading was stopped at that point in the push direction for that load cycle. The wall was then pulled to a displacement of -0.55 mm and the wall's resistance was measured at 86.26 kN. In order to neutralize the behaviour of the wall, a third load cycle was included in this load step. The wall was subjected to a 0.27 mm displacement resulting in a resistance of 113.95 kN. At this stage the first flexural crack occurred at 510 mm height. The resistance of the specimen instantly dropped to 106.6 kN. The loading of the specimen continued in the pull direction with the application of a 99.15 kN load resulting in a 0.65 mm displacement.

Load step 2 started with the application of a 152.13 kN load in the push direction. The corresponding displacement was 0.95 mm. Two new flexural cracks occurred at a 600 mm and 750 mm height. The wall was then loaded in the pull direction with a -160.7 kN load being displaced by -1.9 mm. Four new flexural cracks occurred at heights of 500

mm, 660 mm, 730 mm, and 810 mm. The loading of the wall continued in the push direction and the specimen was subjected with a 152.6 kN load corresponding to a 1.07 mm displacement. No new flexural cracks were observed. In the pull direction a -1.96 mm displacement was applied resulting in a resistance of -148.44 kN. Two new flexural cracks occurred during this load cycle at height of 420 mm and 490 mm. A second softening cycle followed and the resistance of the wall decreased by 6.2% and 3.1 % in the push and pull direction respectively. During the push loading of this last load cycle, three new flexural cracks occurred at 330 mm, 430 mm, and 660 mm distance from the footing.

The next load step involved the application of a 169.3 kN load in the push and -171.4 kN load in the pull direction. The corresponding displacement was 1.45 mm and -2.42 mm respectively. During the push loading, one new flexural crack occurred at 820 mm from the footing of the wall. Three new flexural cracks occurred while the wall was loaded in the pull direction at heights of 1050 mm, 1120 mm, and 1280 mm. A softening cycle followed and the resistance of the wall was reduced by 7.14% in the push and 5.23% in the pull direction. While the wall was loaded in the push direction, one new flexural crack occurred at 1070 mm height.

Load Step 4 subjected the wall to a 191.2 kN load in the push and -188.8 kN load in pull direction. The resulting displacements were 2.02 mm and -2.95 mm in the push and pull direction respectively. Although new flexural cracks did not occur during this load cycle,

the width and length of existing cracks increased. The wall was loaded again, applying the same level of displacement and the resistance demonstrated a decrease of 5.9% in the push and 6.4% in the pull direction. During the push loading of this softening cycle, however, two new flexural cracks occurred at 1220 mm and 1310 mm height.

During load step 5 the wall was loaded with 206.76 kN in the push and -210.7 kN in the pull direction. The specimen's response was 2.63 mm of top displacement in the push and -3.62 mm in the pull direction. Three new flexural cracks occurred while the wall was loaded in the push and one in the pull direction at heights of 940 mm, 440 mm, 1150 mm and 1360 mm respectively. The strain gauges' readings showed that the extreme flexural reinforcement was very close to the yielding point. At the softening cycle that followed, the wall resistance measurement was reduced by 4.47% in the push and 5.61% in the pull direction. More flexural cracks occurred during the softening cycle. Figure 6.23 shows the number of flexural cracks that had occurred up to the end of the fifth load step.

The sixth load step consisted of the application of 227.28 kN load in the push direction and -237.15 kN in the pull direction, resulting in the displacement at the top of the wall by 3.36 mm and -4.7 mm respectively. During this load cycle, yielding of the extreme flexural reinforcement occurred and the yield displacement was set at 2.9 mm for the push and -4.2 mm for the pull direction. New flexural cracks occurred at 370 mm in the push and 360 mm height in the pull direction. During the softening cycle that followed,

the wall displayed a decrease in the load carrying capacity by 0.5% and 13.55 % in the push and pull direction respectively. A new flexural crack also occurred during the loading in both directions.

Starting from load step 7 the loading of the wall was based on displacement control. The wall was subjected to top displacement load of 5.7 mm resulting in a resistance of 312.52 kN. Three new flexural cracks occurred at this stage at 1470 mm, 1530 mm, 1600 mm height. The wall was then pulled to a top displacement of -8.0 mm. The resistance of the wall was measured at -303.99 kN. At this stage there was no sign of debonding between the CFRP sheet and the concrete wall surface. The wall was loaded again in the push and pull direction with the same displacement, the resistance of the wall dropped by 3.91% and 7.78% respectively.

The next load step consisted of the application of a 9 mm top displacement in the push and -11.48 mm in the pull direction. The corresponding load was 388.78 kN and -360.69 kN respectively. New flexural cracks occurred and existing ones were now expanding in width. During the pull direction of this first load cycle compression debonding of the CFPR sheet occurred at the area around the toe of the wall. Figure 6.24 illustrates this occurrence. During the softening load cycle that followed, the resistance of the wall dropped by 8.9% in the push and 7.2 % in the pull direction. Two new flexural cracks occurred, one on each side of the wall at height 30 mm in the push and 740 mm in the pull direction.

Following the 8th load step the wall was loaded with a displacement of 11.66 mm in the push direction and -15.99 mm in the pull direction. The resistance of the wall was measured at 415.5 kN and -399.69 kN respectively. Two new flexural cracks occurred in the pull direction, at 1010 mm and 1090 mm, and the compression debonding previously observed continued to progress. During the softening cycle, compression debonding also occurred in the push direction. Signs of onset of concrete crushing in both directions were observed. The resistance of the wall was decreased by 8.39% and 4.84% in the push and pull directions respectively.

The ultimate load carrying capacity of the wall in both directions was reached during load step 10. The wall was subjected to a maximum displacement of 15.2 mm in the push and -20.01 mm in the pull direction. While the wall was loaded in the push direction and at 12.07 mm displacement crushing of the concrete toe occurred and the resistance of the wall remained almost at the same level although the displacement was increasing, until the displacement level of 13.5 mm. At that point the resistance started to again increase reaching the maximum load at of 426.39 kN at 15.15 mm displacement. At this point a 50 mm wide CFRP strip failed in tension at the edge of the extreme tensile fibre, resulting in an instant drop of the resistance to 395.78 kN at 15.44 mm. Once the wall was then loaded in the pull direction and at displacement -16.4 mm and -376.24 kN load, the toe of the wall crushed, resulting in a reduction of the wall's resistance to -370.26 kN at -17.29 mm of displacement. The displacement of the wall in the pull direction

continued and the maximum load -412.43 kN was reached at -19.83 kN of displacement. It was then that a 70 mm wide CFRP strip, 130 mm failed in tension (Figure 6.25) from the tensile edge of the wall and decreased the resistance of the wall to -398.08 kN at -20.01 mm of displacement. The loading continued with the softening cycle and the resistance of the wall for the same level of displacement was measured at 6.73% and 6.55% lower than the one during the first load cycle of this load step.

Load step 11 consisted of the application of a 18.38 mm displacement in the push and 23.91 mm in the pull direction. The load carrying capacity of the wall reached 411.12 kN in the push direction. During the pull loading of the wall and at displacement -20.55 mm and load -387.38 kN, a 100 mm wide CFRP strip failed in tension 20 mm from the tensile edge of the wall. This resulted in an instant drop in the wall's resistance to -367.62 kN at -22.17 kN of displacement. The loading continued until a displacement of -23.8 mm and load of -374.11 kN. A softening cycle followed targeting the same displacement level as during the first load cycle. While loading in the push direction at 18.38 mm displacement and load 382.57 kN a 130 mm wide CFRP strip failed in tension located at the edge extreme tensile fibre and resulted in an instant drop in resistance of 5.13% to 362.92 kN. The wall was then loaded in the pull direction. Upon reaching a -23.96 mm displacement and -349.17 kN load, a 120 mm wide CFRP strip failed in debonding from the concrete surface 400 mm from the tensile edge. This caused a sudden drop in the load carrying capacity of the wall to -323.44 kN at -24.19 mm of displacement.

Load step 12 involved the application of a displacement load of 24.84 mm in the push and 32.24 mm in the pull direction. While the wall was loaded in the push direction at displacement 20.58 mm and resistance measured at 392.47 kN, two CFRP strips debonded from the wall surface, a 250 mm and a 300 mm wide one approximately 200 mm from the tensile edge of the wall at the front and the back of the wall respectively, resulting in a drop in the wall's resistance to 337.07 kN at 24.84 mm of displacement. The wall was then loaded in the pull direction where at -22.67 mm of displacement and load of -282.44 kN all the CFRP reinforcement sheet gradually debonded from the wall surface except for an 80 mm strip 160 mm from the tensile edge of the wall. The resistance of the wall fell to -188.12 kN at -32.24 mm of displacement. A second load cycle followed and the wall responded in the application of a 24.86 mm displacement with a 242.25 kN resistance. During this loading phase all remaining CFRP strips gradually debonded completely. The toe of the wall had now been completely crushed in both directions. In the pull direction the resistance of the wall in the applied -32.14 mm displacement was -235.43 kN, which was 25.14 % higher than the wall's resistance for the same displacement during the previous load cycle. This could be attributed to shifting of the neutral axis of the wall section and aggregate interlock.

During load step 13 the wall was subjected to a 29.2 mm displacement in the push and -40.8 mm displacement in the pull direction. The response of the wall was a resistance of 193.30 kN and -241.12 kN respectively. The wall was then subjected to a softening cycle and the resistance was measured at 182.34 kN at 30.14 mm of displacement in the push

direction and -139.29 kN at -41.92 mm of displacement in the pull direction. The damage to the wall was now quite severe with extensive compression damage along the construction between the wall and the footing.

Load step 14 the wall was loaded with 36 mm of displacement in the push and -49.2 mm in the pull direction. The corresponding resistance was measured at 196.41 kN and -115.86 kN. During the softening cycle that followed, the wall's resistance was reduced by 43.93 in the push and 40.39 in the pull direction. It was observed at this stage that the top and mid level displacement had almost the same value throughout the load cycle. Therefore the resistance of the wall was attributed to the dowel action of the tensile reinforcement rebars.

Load step 15 was the last load step. The wall was subjected to displacement level 39.7 mm in the push and -57.8 mm in the pull direction. The wall's resistance was 48.7 kN and -57.8 kN respectively. The wall was loaded again for the second and last load cycle and the resistance for the same level of displacement as with the first load cycle was 32.38 kN in the push and -34.90 kN in the pull direction, corresponding to almost 10 % of the maximum load carrying capacity demonstrated during this test.

The extent of the damage sustained by the strengthened wall # 2 specimen during this test is illustrated in Figure 6.26 and 6.32

6.3.2. Load vs. Mid Level Displacement Relationship

Figure 6.27 presents the response of strengthened wall #2 at mid level versus applied load. Comparing this hysteretic diagram with the one corresponding to the response at the top of the wall, it is observed that the behaviour is very similar. When cracking occurred, the mid level displacement corresponded to 39.62% of the total top displacement. This percentage was calculated at 42.24% of the total displacement at the point of yielding of the vertical extreme tensile reinforcement. At the load cycle following yielding, the mid level response corresponded to 43.60% of the top displacement. The displacement at all stages was measured with two strain gauges at the same height one on each side of the wall where the average of both readings was used for the plotting of the response graphs.

6.3.3. Load vs. Base Slip Relationship

The load versus base slip relationship curve of strengthened wall #2 is presented at Figure 6.28. The general shape of the hysteretic diagram as compared to mid and top level response was again similar, especially until the point where severe compression failure at the base of the wall occurred. The mid level response when tensile cracking occurred was calculated at 1.6% of the total response. At the level of yielding of tensile reinforcement, the base slip corresponded to 6.02% of the total displacement at the top level. During the cycle following the yielding cycle, the bottom displacement was calculated to be 6.73%.

6.3.4. Load vs. Rotation Relationship

Figure 6.29 presents the total top rotation of the wall versus the applied yield load. The rotation at the base of the wall was measured to correspond to 55.95% of the total rotation at the top of the wall when flexural cracking occurred. This percentage was calculated to be 42.97% when yielding of the extreme flexural reinforcement occurred. The base slip relationship versus top displacement percentage was calculated to be 42.34% for the load cycle following the yield cycle.

6.3.5. Load vs. Strain Relationship

During the removal of the form work of this specimen, the wiring of the majority of the strain gauges was accidentally damaged. Although the wiring was then reconnected, it was not possible to allocate the wire ends with the specific strain gauges. Consequently, a stress profile diagram could not be generated for this specimen. Individual readings from the strain gauges were taken as presented in Figure 6.30 and 6.31 where strain gauges # 7 and 6 versus load relationships are illustrated.

6.4. Strengthened Wall # 3

Strengthened wall # 3 was reinforced with three plies of CFRP sheets on each side of the wall in the vertical direction for a total of six and one ply in the horizontal direction per side, for a total of two. The calculated maximum load of the specimen was 456.69 kN.

The specimen during the test demonstrated a 541.09 kN load carrying capacity in the push and -486.94 kN in the pull direction. The response of the wall is discussed in detail in the following section.

6.4.1. Load vs. Top Displacement Relationship

The response of the wall at the top during this test is presented in Figure 6.33. (Load versus. top deflection curve). The wall was subjected to two load reversals per load step and was load controlled for the first 5 load steps. The load applied at these load steps corresponded to 50%, 75%, 100%, 115%, and 125% of the calculated yield load. For all remaining load steps until failure, the loading was based on displacement control.

The first load step involved the application of a 97.3 kN load in the pull and -97.56 kN in the pull direction. The corresponding top displacement was 0.383 mm and -0.343 mm respectively. A softening cycle followed and the resistance of the wall demonstrated no losses for the same amount of displacement. No cracking was observed during this first load step.

The first flexural cracks in both directions, occurred during the second load step where the wall was subjected to a +/- 146.4 kN load. Five cracks occurred during the push loading at 390 mm, 460 mm, 570 mm, 660 mm, and 800 mm vertical distance from the footing of the specimen. From the load deflection curve it was determined that the first crack occurred at a load of 119.17 kN and 0.528 mm top deflection. The peak load in the

push direction was 146.4 kN, corresponding to 0.915 mm of displacement. The first crack in the pull direction occurred at -123.95 kN load and 0.581 mm displacement. A total of four flexural cracks occurred during this load cycle at 380 mm, 460 mm, 640 mm, and 780 mm height. The maximum load reached in this direction was -146.6 kN. A second load cycle was applied on the specimen with loads of 147.4 kN and -146.3 kN in the push and pull direction, corresponding to 0.973 mm and -1.01 mm displacements. These loads reflect a decrease in the resistance of the wall by 1.4 % in the push and 7.78% in the pull direction.

During load step 3, a +/- 195 kN load was applied. This load corresponded to the 100% of the calculated yield load. As observed from the readings of the strain gauges installed on the flexural rebars, the strain at the extreme tensile reinforcement had yet to reach the yield value. The wall was loaded with 195.7 kN in the push and -194.8 kN in the pull direction. The top deflection was measured at 1.69 mm and -2.02 mm respectively. New flexural cracks occurred at 720 mm, and 1040 mm in the push and at 290 mm, 300 mm, 1040 mm, 1100 mm, 1700 mm, and 1360 mm in the pull direction. During the softening cycle that followed, the resistance of the wall demonstrated a drop of 7.01% at 180.84 kN (1.65 mm) in the push and 6.22% at 182.23 kN (-2.01 mm) in the pull direction. No new flexural cracks occurred at this stage. However, existing cracks extended in width and length.

During load step 4 the wall was subjected to a load of 223.8 kN in the push and -224.6 kN in the pull direction (115% of the calculated yield load). The peak deflection at the top of the wall was 2.43 mm and -2.65 mm respectively. During this load step, the reading of the strain gauges was closer to the yielding point as compared to the load step three reading, but did not exceed it. Two new flexural cracks occurred at the edge of the wall in the push direction at 810 mm and 820 mm, and in the pull direction at 1510 mm vertical distance from the footing. The second cycle of the load step followed and the load applied on the wall was 224.11 kN in the push and -220.76 kN in the pull direction corresponding to displacement of 2.61 mm and -2.84 mm respectively. This response reflects a decrease in the load carrying capacity of the specimen by 6.8% in the push and 6.47% in the pull direction. Two new flexural cracks occurred during this second cycle, one on each edge of the wall.

The target load for load step 5 was +/- 253.5 kN. The wall was loaded with 253.1 kN in the push direction and the displacement was measured at 3.24 mm. From the load versus displacement curve and the strain gauges' readings it was determined that the extreme flexural reinforcement yielded during this load step at load of 247 kN and displacement of 3.19 mm. The wall was loaded in the pull direction with 253.5 kN and -3.9 mm of displacement. While loaded in the pull direction the wall yielded at displacement of -3.31 mm and load of -234.7 kN. A softening cycle followed and the wall was subjected to a 253.5 kN load at 3.49 mm of displacement in the push and -253.5 kN at -4.1 mm in the pull direction. These numbers correspond to a decrease in the resistance of the wall

by 7.16% and 4.87% respectively. During this load step, a flexural crack occurred at the construction joint level in the push direction. A number of additional flexural cracks occurred on both edges of the wall and all existing cracks extended in both width and depth.

Load step 6 started with the wall being loaded by controlling the displacement level applied. The specimen was subjected to a displacement level of ± 6.4 mm. The corresponding wall resistance was 358.9 kN and -323.46 kN for 6.46 mm and -6.45 mm of displacement in the push and pull direction. Two new flexural cracks occurred at the edge of the wall on each side. A second load cycle was applied to the specimen with loads of 347.30 kN in the push and -301.15 kN in the pull direction at ± 6.4 mm of displacement. These results reflect a decrease in the wall's load carrying capacity during this softening cycle of 3.23% in the push and 6.89% in the pull direction.

The next load step 7 involved the application of a displacement level of ± 9.6 mm. The wall was pushed to a 9.64 mm displacement level resulting in a 456.22 kN resistance load. No new flexural cracks occurred during this push load cycle. However, existing cracks continued to extend and open further. The wall demonstrated a resistance of -375.83 kN in the push direction for the applied displacement of -9.64 mm. A pull out flexural crack occurred at the construction joint level between the wall and the footing, as well as a flexural crack occurring at the edge of the wall at 590 mm vertical distance from the footing. During the softening cycle that followed, the wall was subjected to

displacement level of 9.6 mm and -9.7 mm in the push and pull direction respectively, with corresponding resistance load of 436.69 kN and -365.26 kN. These loads reflect a drop in the resistance of the wall by 4.2% in the push and 4.5% in the pull direction.

Load step 8 targeted the application of a displacement level of ± 12.8 mm. The wall was subjected to a 12.9 mm displacement in the push direction, resulting to a resistance load of 478.36 kN and to -12.9 mm displacement in the pull direction corresponding to a load of -415.02 kN. One additional flexural crack was observed during this load cycle in the pull direction at 590 mm height. During the second cycle, the wall was loaded at ± 13.3 mm displacement level, resulting in resistance of 414.15 kN and -391.54 kN in the push and pull direction respectively. The load carrying capacity of the wall was reduced by 13.42% and 5.66% respectively. At this stage, signs of debonding between the CFRP sheets and the concrete surface on the footing of the wall were observed. This is an indication that the postulated pulley principal does apply in practice. Figure 6.34 illustrates this observation.

During load step 9, the wall reached that maximum load carrying capacity in the push direction. The wall was first loaded in the push direction with a displacement of 19.8 mm resulting in a resistance of 519.47 kN. The debonding of the CFRP sheet at the footing progressed further and the pull out flexural crack at the level of the construction joint reached a width of 4 mm. The wall was subsequently loaded in the pull direction with a displacement of -19.2 mm and a corresponding load of -487.3 kN. While pulling at load

186 kN and displacement of 7.4 mm, compression debonding of the CFRP sheet from the wall surface occurred at the extreme compression zone. This debonding is shown in Figure 6.35. A second load cycle was applied to the specimen, subjecting the wall to a 19.2 mm displacement in the push and -19.5 mm displacement in the pull direction. The load carrying capacity of the wall was measured at 434.25 kN and -425.98 kN, respectively. During this second load step, compression debonding occurred at the extreme compression zone in the push direction. At the pull direction the onset of the toe compression failure was observed. The response of the wall at this second load cycle reflects a reduction in the specimen's load carrying capacity of 15.47% in the push and 14.71% in the pull direction.

During load step 10, the wall was subjected to a displacement of 25.68 mm in the push and -25.70 mm in the pull direction. The resistance of the wall was measured at 533.84 kN and -462.89 kN respectively. During the pull direction, loading the toe of the wall failed in compression and crushed at load -265.41 kN and 11.41 mm of displacement. Compression debonding continued to expand and the pull out crack reached a 6 mm width. During the softening cycle that followed, the resistance of the wall in the push direction was measured at 483.23 kN for a displacement of 25.54 mm. This corresponds to a reduction in the wall's load carrying capacity of 9.48% in this direction. The wall was displaced at the top in the pull direction by -25.7 mm, with a corresponding load of -413.22 kN. At that maximum load, a 300 mm wide CFRP strip failed in tension, resulting in a drop of the wall's resistance to -303.07 kN at -28.7 mm.

The maximum load carrying capacity of the wall in the push direction was reached during load step 11. The wall was loaded with a displacement of 30.9 mm in the push direction which resulted in a resistance of 541.09 kN. At this point, a 20cm CFRP strip failed in tension and the resistance of the wall instantly dropped to 412.42 kN at 3.24 mm, corresponding to 23.8% loss of load carrying capacity. The wall was then loaded in the pull direction and at displacement -28.42 mm and at a load of 348.02 kN a tensile rebar failed. This resulted in a decrease in the load carrying capacity of the wall to -302.37 kN at 30.34 mm. The loading continued and the load at the peak displacement of -32.22 mm was 341.85 kN. During the second load cycle the wall was displaced at 31.78 mm in the push and -32.1 mm in the pull direction. The wall's resistance was measured at 401.62 kN and -312.10 kN respectively. The concrete crushing was more extensive during the course of this last load cycle, and the compression debonding of the CFRP sheets from the wall surface extended further.

Load cycle 12 involved the application of a 38.71 mm displacement in the push and -39.0 mm in the pull direction. The load carrying capacity of the wall reached 404.88 kN and -328.88 kN respectively. While the wall was loaded in the push direction at displacement of 35.45 mm and 437.19 kN load, the concrete toe failed in compression. This resulted in a gradual decrease in the resistance of the wall to 354.65 kN at 37.78 mm. The loading continued and the resistance rose to the 404.88 kN level for the peak displacement. In the pull direction the debonding of the CFRP sheets from the concrete

surface of the footing extended further. The wall was subjected to the second load cycle with the application of 38.37 mm displacement in the push and -40.37 mm in the pull direction. In the push direction the resistance was measured at 367.61 kN. The compression debonding of the CFRP sheet was now more evident. The extent of this debonding is shown in Figure 6.36 and 6.37. While the wall was loaded in the pull direction at displacement of -37.89 mm and load of -341.13 kN a tensile rebar failed resulting in a sudden drop in the resistance of the wall to -233.31 kN at the -40.37 mm peak displacement.

Load step 13 started with the loading of the specimen in the push direction by applying a displacement of 44.8 mm. This resulted in a resistance of 408.49 kN. In the pull direction the target was the application of a -44.8 mm displacement. While pulling and at displacement of -29.89 mm and -247.66 kN load, a 40 mm wide CFRP strip failed in tension. This resulted in a drop in the wall's resistance to 242.07 kN at -31.22 mm. As the loading continued, and at 35.91 mm of displacement of -35.91 mm and -288.57 kN load, a 70 mm wide CFRP strip failed in tension, resulting in a loss in the wall's load carrying capacity to -263.48 kN at -37.05 mm. The pulling of the wall continued and at load of -323.21 kN and -43.23 kN of displacement a tensile rebar failed. This resulted in a significant reduction in the wall's resistance of 22.39% reaching -250.85 kN at -43.92 mm of displacement. The resistance at the peak displacement was -300.07 kN at top displacement of -45.33 mm. During the push loading of the second load cycle, CFRP tensile failure occurred at displacements of 36.61 mm and 40.64 mm, corresponding to

loads of 229.92 kN and 249.29 kN. The peak load of 267.39 was reached at the peak displacement of 44.64 mm. The pull loading followed and the CFRP debonding between the wall surface and the reinforcing sheets continued.

The peak load was reached at the peak displacement measuring -195.83 kN at 45.23 mm.

The next load step 14 involved the application of a displacement load of +/- 51.2 mm.

The resistance of the wall in the push direction was measured at 330.13 kN at 51.2 mm of displacement and in the pull direction 308.96 kN at displacement of 56.5 mm. During the softening cycle the wall demonstrated a decrease in the load carrying capacity of 36.04% in the pull and 28.07% in the push direction. The concrete compression failure was now extensive; it included complete loss of the concrete toe at both sides of the wall and at this stage only a small strip of CFRP sheet remained bonded to the concrete wall surface approximately 350 mm on each side of the wall.

During load step 15, a displacement load of 57.89 mm in the push and -57.6 mm in the pull direction was applied. While the wall was loaded in the push direction at displacement 56.8 mm and load 317.57, complete debonding of the CFRP sheet from the concrete wall surface occurred off of one side of the wall. This resulted in a sudden drop in the wall's resistance to 235.48 kN at 57.26 mm of displacement. The wall's resistance corresponding to the peak displacement was 308.17 mm. in the pull direction the resistance reached -195.22 kN at 57.89 mm of displacement. The second load cycle was next, and the wall was displaced in the push and pull direction by 58.29 mm and -58.07

mm respectively. The load carrying capacity of the wall was measured at 241 kN and - 217.29 kN respectively. These values correspond to a decrease in the load carrying capacity of the wall of 23.56% in the push direction and an increase of 5.77% in the pull direction. This increase can be attributed to a shifting of the neutral axis due to permanent deformation of the wall during the previous load cycle.

The last load step targeted a displacement level of 64 mm in the push and pull direction. The results recorded during this last load step cannot be considered reliable as the instrumentation frame had reached the maximum displacement capacity and therefore the readings of the potentiometers could not be considered accurate. It was nevertheless observed that during this last load step the only remaining part of the CFRP sheet that had remained somewhat bonded to the concrete wall surface up until that point, got completely detached during the push direction loading of the first cycle of this step, as shown in Figure 6.38.

It can thus be observed that if the CFRP reinforcing sheets were less stiff, the compression debonding could have been avoided to a large extent, and the performance of the wall could have been more ductile. The multiple layers of epoxy used for the application of the four plies of CFRP sheets ensured the stiffness of the sheets and to a large extent provoked the compression debonding failure. A detail from the extent of the damage sustained by the strengthened wall # 3 during this test is shown in Figure 6.39.

6.4.2. Load vs. Mid Level Displacement Relationship

Figure 6.40 presents the Load versus Mid level Displacement Curve for the Strengthened Wall # 3. Comparing the top displacement versus load to the mid level displacement versus load curve, it is observed that the pattern of the mid level response is very similar to the top level response. When cracking occurred, the mid level response of the wall corresponded to 42.67% of the total displacement at the top of the wall. This percentage was calculated at 43.35% when yielding of the flexural reinforcement occurred and 45.06% during the load cycle following the one during which yielding occurred. On an average, the mid displacement corresponded to 43.35% of the total displacement.

6.4.3. Load vs. Base Slip Relationship

Figure 6.41 shows the Load versus. Base slip Curve of Strengthened wall # 3. The general pattern of the bottom displacement versus load curve as compared to the corresponding curves for mid and top displacement follows a similar pattern, especially at the point where, as mentioned before, extensive compression failure of the concrete at the base of the wall occurred. The bottom displacement at the point where flexural cracking occurred corresponded to 8.5% of the total displacement. When yielding occurred, this percentage was calculated at 7.73% and for the load cycle following this cycle, at 9.06%.

6.4.4. Load vs. Rotation Relationship

The load versus top displacement relationship is presented in Figure 6.42. The bottom rotation corresponded to 45.29% of the total top rotation and at the point where flexural cracking occurred. When the flexural reinforcement yielded, the bottom rotation corresponded to 38.76% index of the total top rotation. The bottom rotation was also compared to the top rotation during the cycle following the yielding cycle, and was calculated at 44.66% of the total top rotation.

6.4.5. Load vs. Strain Relationship

The Load versus strain profile of Strengthened wall # 3 is presented in Figure 6.43. It can be observed that until the occurrence of the yielding of the extreme tensile reinforcement, the behaviour of the wall was almost completely linear elastic.

Comparing the Load versus Strain profile Figure for Strengthened wall # 3 to the ones corresponding to the Control wall and Strengthened wall # 1, it can be observed that the strain distribution is more linear in the case of strengthened wall # 3. This can be attributed to the application of the three plies of CFRP sheet on strengthened wall # 3.

This linear distribution of strains changes to a more complex after load step 5 which is when the extreme flexural reinforcement in both push and pull directions yielded.

Figures 6.44 and 6.45 present the Load versus. strain curves for stain gauges # 6 and 7 respectively.

	Control Wall		Repaired Wall		Strengthened Wall # 1		Strengthened Wall # 2		Strengthened Wall # 3	
Cracking Load	67.66	63.49	71.48	67.815	119.92	120.07	112.31	98.33	147.98	146.72
	-59.31		-64.15		-120.22		-84.35		-145.46	
Cracking Displacement	0.63	0.53	1.25	1.19	1.06	1.2	0.26	0.395	0.98	0.98
	-0.43		-1.13		-1.34		0.53		-0.98	
Cracking Secant Stiffness	107.4	122.7	57.184	56.977	113.13	101.4242	431.96	295.56	151	149.71
	137.9		56.77		89.71		-159.15		148.43	
Stiffness compared to Control Wall (Ratio)	1.00	1.00	0.53	0.47202	1.05	0.851925	4.02	2.588	1.41	1.2411
	1.00		0.41		0.65		-1.15		1.08	
Yield Load	121.10	122.3	124.83	140.82	154.87	151.65	193.09	204.91	247.10	240.9
	-123.6		-156.81		-148.43		-216.73		-234.70	
Yield Load Displacement	2.99	3.04	4.93	4.37	3.06	2.68	2.9	3.55	3.19	3.25
	-3.09		-3.81		-2.3		-4.2		-3.31	
Ratio of Spec. Yield Load vs Control Wall	1.00	1.00	1.03	1.15002	1.28	1.240184	1.59	1.6744	2.04	1.97
	1.00		1.27		1.20		1.75		1.89	
Yield Secant Stiffness	40.49	40.24	25.32	33.24	50.61	57.57295	66.58	59.093	77.46	74.18
	39.99		41.15		64.53		51.60		70.91	
Ratio of Spec. Yield Stiffness vs Control Wall	1.00	1.00	0.63	0.82	1.25	1.431825	1.64	1.4674	1.91	1.84
	1.00		1.03		1.61		1.29		1.77	
Ultimate Load Carrying Capacity	212.7	196.5	279.3	280.3	313.24	284.25	426.39	419.41	541.09	514.20
	-180.4		-281.3		-255.26		-412.43		487.30	
Displacement at ultimate load	15.22	12.78	29.79	25.395	18.69	23.035	15.15	17.49	30.90	25.05
	-10.34		-21		-27.38		-19.83		-19.20	
Ratio of Increase in the Load Carrying Capacity	1.00	1.00	1.31	1.43625	1.47	1.443886	2.01	2.1455	2.54	2.62
	1.00		1.56		1.41		2.28		-2.71	

Table 6.1 Consolidation of Experimental Results

**Figure 6.1 Average measured lateral load vs top horizontal deflection curve of
Control wall**

Figure 6.1a Cracking pattern at the end of second load cycle (Control Wall)

Figure 6.1b Damage Sustained by the Control Wall

Figure 6.1C Damage Sustained by the Control Wall

**Figure 6.2 Average measured lateral load vs mid-level horizontal deflection curve
of Control wall**

Figure 6.3 Average load vs bottom vertical displacement curve of Control wall

Figure 6.4 Stress Profile at Base Wall Section of Control Wall

Figure 6.5 Load vs. Strain Reading from Strain Gauge # 1 (Control wall)

Figure 6.6 Load vs. Strain Reading from Strain Gauge # 6 (Control wall)

Figure 6.7 Average measured lateral load vs top horizontal deflection curve of repaired wall

**Figure 6.7a CFRP debonding failure during the end of the first load cycle of
loadstep # 7(Repaired Wall)**

**Figure 6.7b Compression failure (toe crushing) during the end of loadstep # 10
(Repaired Wall)**

Figure 6.8 Average measured lateral load vs mid-level horizontal deflection curve
of Repaired wall

Figure 6.9 Average load vs bottom vertical displacement curve of Repaired wall

Figure 6.10 Average load vs. top vertical rotation curve of Repaired wall

Figure 6.11 Average measured lateral load vs top horizontal deflection curve of strengthened wall # 1

**Figure 6.12 Average measured lateral load vs mid-level horizontal deflection curve
of strengthened wall # 1**

Figure 6.13 Average load vs base slip deflection curve of strengthened wall # 1

Figure 6.14 Average Load vs. top Vertical Rotation curve of Strengthened Wall # 1

Figure 6.15 Average load vs bottom vertical displacement curve of strengthened wall # 1

Figure 6.16 Stress Profile at Base Wall Section of Strengthened Wall # 1

Figure 6.17 Load vs. Strain Gauge # 1 Curve / Strengthened Wall # 1

Figure 6.18 Load vs. Strain Gauge # 6 Curve / Strengthened Wall # 1

Figure 6.19 Wall 1V, Cycle 8bR, CFRP Tensile Failure.

Figure 6.20 Wall 1V, Cycle 18bR, Illustration of Damage sustained by the specimen.

Figure 6.21 Wall 1V, Cycle 18bR, Illustration of Damage sustained by the specimen (Detail).

Figure 6.22 Average measured lateral load vs top horizontal deflection curve of strengthened wall # 2

Figure 6.23 Wall 2V, Cycle 5bRExtent of Flexural Cracks

Figure 6.24 Load Step 8, Compression Debonding, Strengthened Wall # 2

Figure 6.25 Load Step 10, CFRP Tensile Failure, Strengthened Wall # 2

Figure 6.26 Load Step 15, Sustained Damage, Strengthened Wall # 2

**Figure 6.27 Average measured lateral load vs mid-level horizontal deflection
curve of strengthened wall # 2**

Figure 6.28 Average load vs base slip deflection curve of strengthened wall # 2

Figure 6.29 Average Load vs. top Vertical Rotation Curve, Strengthened Wall # 2

Figure 6.30 Average Load vs. Strain Gauge # 6, Strengthened Wall # 2

Figure 6.31 Average Load vs. Strain Gauge # 7, Strengthened Wall # 2

Figure 6.32 Load Step 15, Sustained Damage Detail, Strengthened Wall # 2

Figure 6.33 Average measured lateral load vs top horizontal deflection curve of strengthened wall # 3

**Figure 6.34 Wall 3V1H, Cycle 8bR, Debonding of the CFRP sheet from the footing
concrete surface.**

Figure 6.35 Wall 3V1H, Cycle 8bR, Compression Debonding of the CFRP sheet from the wall concrete surface.

**Figure 6.36 Wall 3V1H, Cycle 12bE, Compression Debonding of the CFRP sheet
from the wall concrete surface**

**Figure 6.37 Wall 3V1H, Cycle 12bE, Compression Debonding of the CFRP sheet
from the wall concrete surface**

Figure 6.38 Complete Debonding of CFRP Sheets

Figure 6.39 Wall 3V1H, Cycle 16bE, Damage sustained by the specimen

**Figure 6.40 Average measured lateral load vs mid-level horizontal deflection curve
of Strengthened Wall # 3**

Figure 6.41 Average load vs base slip deflection curve of strengthened wall # 3

Figure 6.42 Average Load vs. Top Vertical Rotation Curve, Strengthened Wall # 3

Figure 6.43 Stress Profile at Base Wall Section of Strengthened Wall # 3

Figure 6.44. Average Load vs. Strain gauge # 6, Strengthened Wall # 3

Figure 6.45. Average Load vs. Strain gauge # 7, Strengthened Wall # 3

Chapter 7

Discussion / Comparison with Previous Results

In this chapter a comparison of the results obtained during the two different phases of this experimental program is presented. The results obtained during the first phase of the program are presented in table 7.1. The summary of the results obtained during the current phase of the program are presented in table 6.1. A comparison bar chart between the phase I and phase II results is presented in Figure 7.1. Figure 7.2 presents the comparison between the percent increase of the cracking, yield, and ultimate load of the reinforced repaired specimens as compared to the control specimens used in phase I and II of this experimental program.

7.1. Control wall vs. Repaired Wall

In both phases of this experimental program the specimens tested had identical dimensions. The same amount of reinforcement both vertical and horizontal was installed in the specimens for both phases and in the exact same locations. This was intentionally done so that the comparison that follows be more meaningful.

The percentage increase of the ultimate load carrying capacity for the repaired wall compared to the control wall performance was 81.00 % for phase I and 43.62% for phase II corresponding to a 76.4 % difference in the strengthening result between the two phases. This difference can be attributed to the fact that the resistance of the second phase control wall was much higher compared to the phase one control wall, and the wall

was damaged significantly more than the phase one control wall during the test.

Therefore the resistance of the repaired wall in phase two was lower compared to the phase one repaired wall resistance and the percent increase of the strength was calculated based on a high strength control wall for phase two compared to phase one.

The cracking load of the repaired wall compared to the control wall result was increased by 28.94 % during the phase I tests and 6.82% during phase II corresponding in a 76.4 % difference between the two phases. The cracking stiffness recovered during the phase I tests corresponded to 49.6% of the control wall cracking stiffness. During phase II, 52.28% of the control wall's cracking stiffness was recovered with the application of the CFRP sheets.

The yield load of the repaired specimens was increase by 29.61% and 15% during Phase I and phase II respectively corresponding to a difference of 49.34% between the two phases. These differences between the strengthening effect of the method during the two phases can be attributed to the extensive damage that the control wall sustained during the second phase of the tests.

7.2. Control Wall vs. Strengthened Wall # 1

Strengthened wall # 1 was strengthened with the application of the one vertical layer of CFRP sheets on the each side of the wall during both phases.

The cracking load was increased by 83.4% and 89.1% during phase I and II respectively. These numbers correspond to a difference in the strengthening effect of the method at this stage of 6.8%.

The load when yielding of the vertical reinforcement occurred reflected an increase in the strength at that stage of 24.25% and 24.01% for phase I and phase II tests respectively compared to the control walls used for each phase.

The ultimate load carrying of the specimens was greater by 46.04% and 44.38% for phase I and phase II tests respectively.

7.3. Control Wall vs. Reinforced Wall #2

Reinforced wall #2 specimens were retrofitted by applying two vertical layers of CFRP sheets and one horizontal on each side of the wall for phase I, and by applying only two vertical layers on each side of the wall for phase II.

The measured secant cracking stiffness was increased by 65.5% and 158.8% for phase I and II respectively.

The cracking load was increased by 85.1% during phase I, and 54.88% during phase II, corresponding to a 35.45% difference between the two phases, as compared to the corresponding per phase control wall.

The yield load of the specimens was increased by 64.56% and 67.43% during phase I and II respectively.

The ultimate load carrying capacity of the specimens was greater by 132.13% during phase I, and 114.54% during phase II.

Phase 1	Control Wall	Aver.	Repaired Wall	Aver.	Strengthened Wall # 1	Aver.	Strengthened Wall # 2	Aver.
Cracking Load	49.6	55.1	72.5	71.05	97.10	101.05	96.00	101.95
	-60.6		-69.6		-105.00		-107.90	
Cracking Secant Stiffness	88.6	96.55	52.9	47.90	137.70	158.65	177.80	159.90
	-104.5		-42.9		-179.60		-142.00	
Stiffness compared to Control Wall	1	1	0.59	0.49	1.55	1.64	2.01	1.68
	1		0.41		1.72		1.36	
Yield Load	122.7	122.4	175.80	158.35	139.10	152.11	213.50	201.15
	-122.1		-140.90		-165.12		-188.80	
Increase in Yield Load (ratio)	1.014	8E-04	1.45	1.29	1.15	1.24	1.76	1.64
	0.988		1.14		1.34		1.53	
Yield Secant Stiffness	36.5	32.75	30.60	29.40	92.00	95.50	90.00	82.75
	-29		-28.20		-99.00		-75.50	
Increase in Yield Secant Stiffness (ratio)	1	1	0.84	0.91	2.52	2.97	2.46	2.53
	1		0.97		3.41		2.60	
Ultimate Load Carrying Capacity	187.1	177.6	323.70	320.70	260.90	258.75	450.60	413.10
	-168.1		-317.70		-256.60		-375.60	
Increase in the Load Carrying Capacity (ratio)	1	1	1.73	1.81	1.39	1.46	2.41	2.32
	1		1.89		1.53		2.23	

Table 7.1 Results Obtained during the 1st phase of the Tests.

Figure 7.1 Results Comparison Bar chart

Figure 7.2 Comparison of percentage of strength increase between the specimens, compared to the control wall of each phase.

Chapter 8

Summary Conclusions Potential for additional Research

8.1. Summary

As a part of the research program on the use of CFRP tow sheet, a second phase of an overall experimental program has been completed. Reinforcing materials as a strengthening and repair method for earthquake resistance of reinforced concrete walls, completed another focus of the present study. Apart from more detailed observations of the behaviour of reinforced concrete shear walls resulting from application of externally bonded CFRP sheets and evaluation of the effectiveness of the CFRP strengthening method in improving the seismic behaviour and strength of the tested specimens, a significant part of this study was the evaluation of the performance of a new anchor system for the CFRP sheets in relation to the behaviour of the load transfer mechanism between the CFRP sheets, the wall, and the supporting member.

The observations from Phase 1 of this experimental program were carefully studied and taken under consideration in the development and design of the new anchor system implemented in the present study. During the first phase of the research program, it was observed that the anchoring system used did not allow the complete utilization of the load carrying capacity of the CFRP sheets, and was in many cases the instigator or the cause of failure of the CFRP sheets before the material had reached its ultimate strength. The

main reason for the premature failure of the CFRP sheets before they reached their full tensile capacity was due to the eccentricity that existed between the line of action of the tensile force of the CFRP sheets and the line of action of the resistance force of the anchoring bolt.

In the second phase tests of the present study, a new anchor system was designed and implemented on the wall specimens in the attempt to improve the performance of the overall system by eliminating the prying action problems of the conventional L-shaped angle anchor observed during the first series of tests of the research program. The new anchor system was designed to work based on the pulley mechanism which eliminates the eccentricity problem as the cause of the deficient performance of the original conventional L-shaped angle anchoring system. The performance of the new anchor system was found to be excellent.

Taking into consideration the observations made during the previous first series of tests, particularly regarding out-of-plane torsional behaviour of the wall specimens an improved lateral restrain frame was designed and incorporated in the test set up of the present test program to minimize the potential of out-of-plane displacement of the specimen during loading.

Four identical shear wall specimens were fabricated. The control specimen was loaded and after it reached its peak strength, the loading was stopped and the specimen was repaired by applying externally bonded CFRP sheets in the vertical direction of the damaged wall. The effectiveness of the application of CFRP sheets as a method for retrofit and repair and the capability to recover the original in-plane stiffness and increase

the flexural strength of the wall was investigated through testing of the repaired wall and strengthened wall specimens without any prior damage before testing.

8.2. Conclusions

This series of tests can be generally characterized as successful. The results of these tests lead to the conclusions that are outlined below:

- The application of externally epoxy bonded CFRP sheets on shear walls can increase the in-plane load carrying capacity and stiffness. It can be therefore considered an adequate method for retrofit and repair of reinforced concrete structures.
- The application of externally epoxy bonded CFRP sheets on shear wall that had sustained damage was shown to recover and further increase the ultimate load carrying capacity.
- The behaviour of shear walls repaired with CFRP sheets was ductile, reaching ductility levels of 10 at ultimate load carrying capacity, demonstrating a large capacity for absorption of earthquake energy.
- The method for repair of the damaged wall involved the removal of loose material, the filling of cracks, and patching. The epoxy injection of the cracks of the damaged wall may improve the performance of the wall.

- When externally epoxy bonded CFRP sheets are applied on a shear wall that has already sustained damage due to lateral load excitation, the yield load of the wall can be increased.
- The application of externally bonded CFRP on undamaged shear walls resulted in an increase in the cracking load.
- Undamaged walls that have been reinforced with externally bonded CFRP sheets demonstrated an increase in the load when yielding of the tensile reinforcement occurs.
- The reinforcing method involving the application of externally bonded CFRP sheets on undamaged walls was shown to increase the ultimate load carrying capacity by up to 160% for the wall strengthened with three vertical layers of CFRP sheets and one horizontal, as compared to the control wall without any addition of FRP reinforcement.
- The elastic in-plane stiffness of the wall specimens was increased by the application of externally bonded CFRP sheets.
- The behaviour of the walls retrofitted with the application of externally bonded CFRP sheets was ductile, reaching ductility levels of 20 at a resistance that had dropped to 20% of the ultimate strength.
- The multiple layers of CFRP combined with multiple layers of cured epoxy resin, result in a very rigid composite material. This composite material is not easily deformable. Consequently, when the sections are in compression, compression debonding of the CFRP sheet from the concrete surface occurs and propagates rapidly, ultimately leading to delamination failure of the CFRP sheets.

- The proposed anchoring system performed as expected, transferring the load from the CFRP sheets to the footing, and eliminating all the deficiencies associated with the conventional L-shaped angle anchoring system used during Phase I of this experimental program.

8.3. Recommendations for Future Research

The current phase of this research program only focused on a small area of the shear wall behaviour reinforced with externally bonded CFRP sheets. The following include potential topics/initiatives for future research:

- Testing of shear walls with externally bonded diagonal CFRP sheets.
- Optimization of the design of the proposed anchor. Consideration of field installation and aesthetics and / or architectural requirements.
- Application of the proposed new anchor to curved walls and columns.
- Testing of CFRP reinforced shear walls with steel plates bolted on both sides of the wall at the extreme compression zones with the intention to provide confinement, which may potentially increase the compressive strength of the walls.
- Testing of CFRP reinforced shear walls with splices in the steel flexural reinforcement to simulate the conditions encountered in actual structures.
- Research on the relative contribution of the CFPR sheets on the deformation mechanisms related to shear, flexure, and sliding shear.

- Testing of shear walls externally bonded with CFRP sheets loaded with a combined in-plane cyclic lateral load, and vertical static load simulating the gravity load of the structure supported by the shear wall.
- Research the potential effects of the out-of-plane bending response of shear walls in earthquake excitation on the in-plane stiffness of the wall.

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Appendix A

Analytical Design of Wall Specimens

Methods used to calculate the strength of the specimens

Shear strength:

Modified method proposed by Wiradinata (Behaviour of squat shear walls subjected to load reversal, M.A.Sc. Thesis, University of Toronto, ON)

$$V_r = V_c + V_s + V_F \quad \text{where:}$$

$$V_c = \alpha_c A_e (f'_c)^{1/2}$$

$$\alpha_c = 0.5 - (h/6L), A_e = b d, d = 0.9L$$

$$V_F = 2f_F t_f d \cot \theta$$

$$V_s = \frac{n A_h f_y}{s} \cot \theta$$

$$\text{And also } V_r < 0.25 A_e f_c$$

Sliding Shear Strength:

From clause 11.6.1 of the CSA standard A23.3-94, where;

$$V_{sl} = [\lambda(c + \mu\sigma) + \rho_v f_y \cos \alpha_f] A_e, \text{ where}$$

$$V_{sl} < 0.25 A_e f'_c, \text{ and}$$

$$V_{sl} < 7.0 A_e$$

also: from clause 11.6.2(b) $\rightarrow c = 0.5$ and $\mu = 1.0$

from clause 11.6.4 $\rightarrow \sigma = \rho_v f_y \sin \alpha_f + (N/A_g)$

where $\rho_v = A_{vf}/A_{cv}$

Flexural Strength

The flexural strength of the Shear wall specimens is determined using the strain compatibility condition, by assuming that plane sections remain plain and by not considering the effect of strain hardening of the steel reinforcement. The following are the result spreadsheets that were used to determine the ultimate flexural capacity of the shear wall specimens.

Maximum Flexural Load Carrying Capacity / Control Wall

Maximum Flexural Capacity Worksheet

Concrete Cylinder Test Results

Area:		Stresses	
Compression			
Cy/#1:	614.2 kN		17662.5 mm ²
Cy/#2:	668.3 kN		34.774239 Mpa
Cy/#3:	622.5 kN		37.837226 Mpa
			35.244161 Mpa
Indirect Tension			
Cy/#4:	192.3 kN		2.7218684 Mpa
Cy/#5:	233.6 kN		3.3064402 Mpa

f_c= 35.9518754 Mpa

f_t= 3.01415428 Mpa

$$\epsilon_c = \frac{0.002 \times c}{(h - \text{Cover}) - c}$$

$$C_c = \alpha_1 \times f'_c \times b \times c$$

$$C_{S1} = E_s \times \epsilon_{S1} \times A_s$$

$$T_{S2} = E_s \times \epsilon_{S2} \times A_s$$

$$T_{S3} = \epsilon_{S3} \times E_s \times A_s$$

$$T_{S4} = \epsilon_{S4} \times E_s \times A_s$$

$$T_{S5} = \epsilon_{S5} \times E_s \times A_s$$

$$T_{S6} = \epsilon_{S6} \times E_s \times A_s$$

$$\alpha_1 = (\epsilon_0 / \epsilon_c) [\ln(1 + (\epsilon_0 / \epsilon_c)^2 - \ln(\epsilon_0 / \epsilon_c)^2)]$$

$$\beta_1 = 1 - \frac{[2 - 2(\epsilon_0 / \epsilon_c) \tan^{-1}(\epsilon_c / \epsilon_0)]}{[\ln(1 + (\epsilon_0 / \epsilon_c)^2) - \ln(\epsilon_0 / \epsilon_c)^2]}$$

Horizontal cross-section of the reinforced concrete shear wall specimen

Data of Structure

ϵ_{ch}	0.0035
ϵ_0	0.0024

$f_c = 35.95187544$ Mpa
 $E_c = 20000$ Mpa
 $E_s = 200000$ MPa
 $\epsilon_{sy} = 0.002$
 $b = 100$ mm
 $h = 1500$ mm
 $Cover = 50$ mm
 $A_s = 200$ mm²

Stress Block Parameters α/β :

$\alpha = 0.781707212$
 $\beta = 0.412192262$
 $h_{cover} = 1450$

Assume $c = 126.515$ mm

<u>Strains</u>	<u>Forces</u>	<u>Equilibrium</u>
$\epsilon_c = 0.0035$	$C_c = 320000.2$ N	$\square C = 400000.162$ N
$\epsilon_{s1} = 0.002116765$	$C_{s1} = 80000$ N	$\square T = 400000$ N
$\epsilon_{s2} = 0.005629352$	$T_{s2} = 80000$ N	
$\epsilon_{s3} = 0.013375469$	$T_{s3} = 80000$ N	
$\epsilon_{s4} = 0.021121586$	$T_{s4} = 80000$ N	
$\epsilon_{s5} = 0.028867703$	$T_{s5} = 80000$ N	
$\epsilon_{s6} = 0.03661382$	$T_{s6} = 80000$ N	

Satisfying Force Equilibrium the maximum moment is:

<u>Moment arms:</u>	<u>Individual Moments</u>	<u>Sum of Moment Resistance</u>
$C_c = 74.3665$ mm	$M_{C_c} = 23797291$	
$C_{s1} = 76.515$ mm	$M_{C_{s1}} = 6121200$	$M_{tot} = 3.35E+08$ N-mm
$T_{s1} = 203.485$ mm	$M_{T_{s1}} = 16278800$	

T _{s3} =	483.485	mm	M Ts3=	38678800	or:	335.3125	kN-m
T _{s4} =	763.485	mm	M Ts4=	61078800			
T _{s5} =	1043.485	mm	M Ts5=	83478800			
T _{s6} =	1323.485	mm	M Ts6=	1.06E+08	<u>Maximum</u> <u>Load:</u>	167.6562	kN

Maximum Flexural Capacity of Strengthened Wall # 1 and Repaired Wall

$$\epsilon_c = \frac{0.002 \times c}{(h - \text{Cover}) - c}$$

$$C_c = \alpha_1 \times f'_c \times b \times c$$

$$C_{S1} = E_s \times \epsilon_{S1} \times A_{S1}$$

$$C_{S2} = E_s \times \epsilon_{S2} \times A_{S2}$$

$$T_{S3} = \epsilon_{S3} \times E_s \times A_s$$

$$T_{S4} = \epsilon_{S4} \times E_s \times A_s$$

$$T_{S5} = \epsilon_{S5} \times E_s \times A_s$$

$$T_{S6} = \epsilon_{S6} \times E_s \times A_s$$

$$T_{cftp} = 0.5 \times \epsilon_{cftp} \times E_{cftp} \times t_{cftp} \times (h - c)$$

Horizontal cross-section of the reinforced concrete shear wall specimen

Data of Structure

ϵ_{cu}	0.0035
ϵ_0	0.002
f'_c	35.95187544 Mpa

Maximum Flexural Capacity Worksheet

Concrete Cylinder Test Results

Compression	Area:	17662.5 mm ²	Stresses
Cyl#1:	614.2 kN		34.77423921 Mpa
Cyl#2:	668.3 kN		37.83722576 Mpa
Cyl#3:	622.5 kN		35.24416136 Mpa
Indirect Tension			
Cyl#4:	192.3 kN		2.721868365 Mpa
Cyl#5:	233.6 kN		3.306440198 Mpa
	f'_c	35.95187544 Mpa	

$$f_t = 3.014154282 \text{ Mpa}$$

$$\alpha_1 = (\epsilon_0 / \epsilon_c) [\ln(1 + (\epsilon_0 / \epsilon_c)^2) - \ln(\epsilon_0 / \epsilon_c)]$$

$$\beta_1 = 1 - \frac{[2 - 2(\epsilon_0 / \epsilon_c) \tan^{-1}(\epsilon_c / \epsilon_0)]}{[\ln(1 + (\epsilon_0 / \epsilon_c)^2) - \ln(\epsilon_0 / \epsilon_c)]}$$

$E_c=$	20000	Mpa
$E_s=$	200000	MPa
$\epsilon_{sy}=$	0.002	
$b=$	100	mm
$h=$	1500	mm
Cover=	50	mm
$A_s=$	200	mm ²
$t_{frp}=$	0.11	mm
$E_{cfpr}=$	230500	MPa
Num.Plies=	2	

Stress Block Parameters α_1/β_1 :

$\alpha_1=$	0.801027742
$\beta_1=$	0.430649972
$h-cover=$	1430

Assume c=	282	mm
-----------	-----	----

Strains

$\epsilon_c=$	0.0035
$\epsilon_{Sc1}=$	0.002879433
$\epsilon_{St2}=$	0.000595745
$\epsilon_{St3}=$	0.004070922
$\epsilon_{St4}=$	0.007546099
$\epsilon_{St5}=$	0.011021277
$\epsilon_{St6}=$	0.014496454
$\epsilon_{fmax}=$	0.015117021

Forces

$C_c=$	730904.7	N
$C_{s1}=$	80000	N
$T_{s2}=$	23829.79	N
$T_{s3}=$	80000	N
$T_{s4}=$	80000	N
$T_{s5}=$	80000	N
$T_{s6}=$	80000	N
$T_{frp}=$	466849.7	N

Equilibrium

$\square C=$	810904.6506	N
$\square T=$	810679.5339	N

Satisfying Force Equilibrium the Maximum moment is:

<u>Moment arms:</u>		<u>Individual Moments</u>	<u>Sum of Moment Resistance</u>
$C_c =$	160.5567 mm	$M_{Cc} = 1.17E+08$	
$C_{s1} =$	232 mm	$M_{Cs1} = 18560000$	$M_{tot.} = 7.55E+08$ N-mm
$T_{s2} =$	48 mm	$M_{Ts2} = 1143830$	
$T_{s3} =$	328 mm	$M_{Ts3} = 26240000$	or: 755.4975 kN-m
$T_{s4} =$	608 mm	$M_{Ts4} = 48640000$	
$T_{s5} =$	888 mm	$M_{Ts5} = 71040000$	
$T_{s6} =$	1168 mm	$M_{Ts6} = 93440000$	<u>Maximum</u>
$T_{fp} =$	812 mm	$M_{Tcfp} = 3.79E+08$	<u>Load:</u> 377.7487 kN

Maximum Flexural Capacity of Strengthened Wall # 2

Horizontal cross-section of the reinforced concrete shear wall specimen

Maximum Flexural Capacity Worksheet

Concrete Cylinder Test Results

		Area:	17662.5	mm ²
		Stresses		
Compression				
Cyl#1:	614.2	kN	34.77423921	Mpa
Cyl#2:	668.3	kN	37.83722576	Mpa
Cyl#3:	622.5	kN	35.24416136	Mpa
Indirect Tension				
Cyl#4:	192.3	kN	2.721868365	Mpa
Cyl#5:	233.6	kN	3.306440198	Mpa
		f _c =		
		35.95187544		Mpa
		f _t =		
		3.014154282		Mpa

$$\epsilon_c = \frac{0.002 \times c}{(h - \text{Cover}) - c}$$

$$C_c = \alpha_1 \times f'_c \times b \times c$$

$$C_{s1} = E_s \times \epsilon_{sd} \times A_s$$

$$C_{s2} = E_s \times \epsilon_{s2} \times A_s$$

$$T_{s3} = \epsilon_{st3} \times E_s \times A_s$$

$$T_{s4} = \epsilon_{st4} \times E_s \times A_s$$

$$T_{s5} = \epsilon_{st5} \times E_s \times A_s$$

$$T_{s6} = \epsilon_{st6} \times E_s \times A_s$$

$$T_{cftp} = 0.5 \times \epsilon_{cftp} \times E_{cftp} \times t_{cftp} \times (h - c)$$

$$\alpha_1 = (\epsilon_0 / \epsilon_c) [\ln(1 + (\epsilon_0 / \epsilon_c)^2) - \ln(\epsilon_0 / \epsilon_c)]$$

Data of Structure

$$\epsilon_{cu} = 0.0035$$

$$\epsilon_0 = 0.002$$

$$\beta_1 = 1 - \frac{[2 - 2(\epsilon_0 / \epsilon_c) \tan^{-1}(\epsilon_c / \epsilon_0)]}{[\ln(1 + (\epsilon_0 / \epsilon_c)^2) - \ln(\epsilon_0 / \epsilon_c)]}$$

$f_c = 35.95187544$ Mpa
 $E_c = 26981.94726$ Mpa
 $E_s = 200000$ MPa
 $\epsilon_{sy} = 0.002$
 $b = 100$ mm
 $h = 1500$ mm
 $Cover = 50$ mm
 $A_s = 200$ mm²
 $t_{fp} = 0.11$ mm
 $E_{cftp} = 230500$ MPa
 $Num.Plies = 4$

Stress Block Parameters α_1/β_1 :

$\alpha_1 = 0.801027742$
 $\beta_1 = 0.430649972$
 $h-cover = 50$

Assume $c = 349.25$ mm

<u>Strains</u>		<u>Forces</u>		<u>Equilibrium</u>	
$\epsilon_c =$	0.0035	$C_c =$	905207.3 N	$\square C =$	985207.3 N
$\epsilon_{sc1} =$	0.002999	$C_{s1} =$	80000 N	$\square T =$	985240.3 N
$\epsilon_{st2} =$	-0.00019	$T_{s2} =$	-7716.54 N		
$\epsilon_{st3} =$	0.002613	$T_{s3} =$	80000 N		
$\epsilon_{st4} =$	0.005419	$T_{s4} =$	80000 N		
$\epsilon_{st5} =$	0.008225	$T_{s5} =$	80000 N		
$\epsilon_{st6} =$	0.011031	$T_{s6} =$	80000 N		

$\epsilon_{fmax}=$
0.011532

$T_{frp}=$
672956.8
N

Satisfying Force Equilibrium the Ultimate moment is:

<u>Moment arms:</u>		<u>Individual Moments</u>	<u>Sum of Moment Resistance</u>
$C_c=$	198.8455 mm	$M_{Cc}= 1.8E+08$	$M_{tot.}= 8.09E+08$ N-mm
$C_{sl}=$	299.25 mm	$M_{Csl}= 23940000$	
$T_{s2}=$	-19.25 mm	$M_{Ts2}= 148543.3$	
$T_{s3}=$	260.75 mm	$M_{Ts3}= 20860000$	
$T_{s4}=$	540.75 mm	$M_{Ts4}= 43260000$	
$T_{s5}=$	820.75 mm	$M_{Ts5}= 65660000$	or: 809.1275 kN-m
$T_{s6}=$	1100.75 mm	$M_{Ts6}= 88060000$	
$T_{frp}=$	575.375 mm	$M_{Tcfrp}= 3.87E+08$	
			<u>Maximum Load:</u> 404.5637 kN

Maximum Flexural Capacity of Strengthened Wall # 3

Maximum Flexural Capacity Worksheet

Concrete Cylinder Test Results

Area:	17662.5	mm ²
Stresses		
Cyl#1:	614.2	kN
Cyl#2:	668.3	kN
Cyl#3:	622.5	kN
Indirect Tension		
Cyl#4:	192.3	kN
Cyl#5:	233.6	kN
fc=	35.95187544	Mpa
ft=	3.014154282	Mpa

$$\epsilon_c = \frac{0.002 \times c}{(h - \text{Cover}) - c}$$

$$C_c = \alpha_1 \times f'_c \times b \times c$$

$$C_{S1} = E_s \times \epsilon_{S1} \times A_s$$

$$C_{S2} = E_s \times \epsilon_{S2} \times A_s$$

$$T_{S3} = \epsilon_{S3} \times E_s \times A_s$$

$$T_{S4} = \epsilon_{S4} \times E_s \times A_s$$

$$T_{S5} = \epsilon_{S5} \times E_s \times A_s$$

$$T_{S6} = \epsilon_{S6} \times E_s \times A_s$$

$$T_{cftp} = 0.5 \times \epsilon_{cftp} \times E_{cftp} \times t_{cftp} \times (h - c)$$

$$\alpha_1 = (\epsilon_0 / \epsilon_c) [\ln(1 + (\epsilon_0 / \epsilon_c)^2) - \ln(\epsilon_0 / \epsilon_c)^2]$$

$$\beta_1 = 1 - \frac{[2 - 2(\epsilon_0 / \epsilon_c) \tan^{-1}(\epsilon_c / \epsilon_0)]}{[\ln(1 + (\epsilon_0 / \epsilon_c)^2) - \ln(\epsilon_0 / \epsilon_c)^2]}$$

Data of Structure

$\epsilon_{cu} =$	0.0035
$\epsilon_0 =$	0.002

$f_c=$	35.95187544	Mpa
$E_c=$	26981.94726	Mpa
$E_s=$	200000	MPa
$\epsilon_{sy}=$	0.002	
$b=$	100	mm
$h=$	1500	mm
Cover=	50	mm
$A_s=$	200	mm ²
$t_{fp}=$	0.11	mm
$E_{cfp}=$	230500	MPa
Num.Plies=	6	

Stress Block Parameters α_1/β_1 :

$\alpha_1=$	0.801027742
$\beta_1=$	0.430649972
h-cover=	1450

Assume c=	397	mm
-----------	-----	----

Strains

$\epsilon_c=$	0.0035
$\epsilon_{s1}=$	0.003059
$\epsilon_{s2}=$	0.000591
$\epsilon_{s3}=$	0.001878
$\epsilon_{s4}=$	0.004346
$\epsilon_{s5}=$	0.006815
$\epsilon_{s6}=$	0.009283

Forces

$C_c=$	1028969	N
$C_{s1}=$	80000	N
$C_{s2}=$	23627.2	N
$T_{s3}=$	75113.35	N
$T_{s4}=$	80000	N
$T_{s5}=$	80000	N
$T_{s6}=$	80000	N

Equilibrium

$\square C=$	1132595.808	N
$\square T=$	1130969.2	N

$$\epsilon_{fmax}= 0.009724$$

$$T_{frp}= 815855.9 \text{ N}$$

Satisfying Force Equilibrium the maximum moment is:

Moment arms:

$C_c=$	226.032	mm
$C_{s1}=$	347	mm
$C_{s2}=$	67	mm
$T_{s3}=$	213	mm
$T_{s4}=$	493	mm
$T_{s5}=$	773	mm
$T_{s6}=$	1053	mm
$T_{frp}=$	551.5	mm

Individual Moments

$M_{Cc}=$	2.33E+08
$M_{Cs1}=$	27760000
$M_{Cs2}=$	1583023
$M_{Ts3}=$	15999144
$M_{Ts4}=$	39440000
$M_{Ts5}=$	61840000
$M_{Ts6}=$	84240000
$M_{Tcfrp}=$	4.5E+08

Sum of Moment Resistance

$M_{tot.}=$	913386459.2	N-mm
or:	913.3864592	kN-m
<u>Maximum Load:</u>	456.6932296	kN

Appendix B

Design of Anchor

Due to the fact that the finite element model that was developed composed of 3792 elements and had 6296 nodes, it is not practical to include in this paper the complete set of the results of the analysis. Only the maximum values for the displacement and the stresses and loads are presented below.

MINIMUM/MAXIMUM DISPLACEMENTS (meters)

NODE	2258	2382	5321	1
MIN.	-7.85E-06	-9.72E-06	-8.00E-05	0
NODE	205	185	5474	6246
MAX.	4.01E-04	4.01E-04	2.81E-04	0

MAXIMUM RESULTANT DISPLACEMENT

NODE	195
MAX.	5.66E-04

MINIMUM/MAXIMUM AVG. NODAL STRESSES (Pa)

	SIGMA-X1	SIGMA-X2	SIGMA-X3	TAU-X12	TAU-X13	TAU-X23
NODE	4237	4077	4237	388	4257	4097
MIN.	-1.66E+8	-1.66E+8	-4.84E+8	3.75E+08	4.44E+08	4.39E+08
NODE	332	388	388	4237	4249	4089
MAX.	1.16E+08	1.16E+08	9.40E+08	2.98E+08	3.89E+08	3.94E+08

MAXIMUM	NODAL	VON	MISES	STRESS
NODE	6246			
MAX.	4.56E+8			

The deformed shape of the anchor for the applied load and with the displacements multiplied by 150 is shown in Figure B 3.

Figure B1 Anchor – Finite Elements Model

Figure B2 Anchor – Finite Elements Model – Side View Section

Figure B-3 Deformed Shape (Scale X 150)

Appendix C

Design of Lateral Restrain Mechanism

Figure C1 presents the finite element model that was created for the analysis of the lateral restrain frame. The load applied was a moment of 50kN-m that corresponds to a 500kN in-plane load applied with an eccentricity of 0.1m. The results of this analysis are presented below.

The maximum displacement that this lateral restrain frame allows is approximately 1.5 mm.

```
CONTROL INFORMATION
NUMBER OF LOAD CASES ..... (NLCASE) =    1
SOLUTION MODE ..... (MODEX) =    0
  EQ. 0, STATIC ANALYSIS
  EQ. 1, BUCKLING ANALYSIS
  EQ. 2, DYNAMIC ANALYSIS
THERMAL LOADING FLAG ..... (ITHERM) =    0
  EQ. 0, NO THERMAL EFFECTS CONSIDERED
  EQ. 1, ADD TEMPERATURE EFFECT
GRAVITY LOADING FLAG ..... (IGRAV) =    0
  EQ. 0, NO GRAVITY LOADING CONSIDERED
  EQ. 1, ADD GRAVITY LOADING EFFECT
CENTRIFUGAL LOADING FLAG ..... (ICNTRF) =    0
  EQ. 0, NO CENTRIFUGAL LOADING CONSIDERED
  EQ. 1, ADD CENTRIFUGAL LOADING EFFECT
IN-PLANE STIFFENING FLAG ..... (INPLN) =    0
  EQ. 0, NO IN-PLANE EFFECTS CONSIDERED
  EQ. 1, IN-PLANE EFFECTS CONSIDERED
SOFT SPRING ADDITION FLAG ..... (ISOFT) =    0
  EQ. 0, NO SOFT SPRING OPTION
  EQ. 1, SOFT SPRING ADDED

SAVE DECOMPOSED STIFFNESS MATRIX FLAG ... (ISAVK) =    0
  EQ. 0, DO NOT SAVE DECOMPOSED K
  EQ. 1, SAVE DECOMPOSED K
FORM STIFFNESS MATRIX FLAG ..... (IFORMK) =    0
  EQ. 0, FORM STIFFNESS MATRIX
  EQ. 1, USE EXISTING DECOMPOSED STIFFNESS MATRIX
```

```
TOTAL SYSTEM DATA
NUMBER OF EQUATIONS ..... (NEQ) =   114
NUMBER OF MATRIX ELEMENTS ..... (NWK) =  1227
MAXIMUM HALF BANDWIDTH ..... (MK) =    24
MEAN HALF BANDWIDTH ..... (MM) =    10
NUMBER OF ELEMENTS ..... (NUME) =    27
NUMBER OF NODAL POINTS ..... (NUMNP) =    24
ADJUSTED BLOCK SIZE ..... (MTBLK) =  1228
NUMBER OF STIFFNESS BLOCKS ..... (NBLK) =    1
```

```
MAXIMUM DIAGONAL STIFFNESS MATRIX VALUE = .223181E+14 ( 60)
MINIMUM DIAGONAL STIFFNESS MATRIX VALUE = .606385E+06 ( 12)
```

```
*****
*   RESPONSE PRINTOUT (LOAD CASE 1)   *
```

STATUS OF THERMAL EFFECTS FOR THIS LOAD CASE:

- Thermal flag is off (no thermal effect)

FORCES IN THE CLOSED BOUNDARY/GAP ELEMENT(S), NCE = 2
 ELEMENT FX FY FZ GAP DISTANCE NORMAL FORCE
 18 .0000E+00 .0000E+00 .3208E+05 .0000E+00 .3208E+05
 21 .0000E+00 .0000E+00 -.3208E+05 .0000E+00 .3208E+05
 TOTAL .0000E+00 .0000E+00 -.3019E-03

DISPLACEMENTS

NODE	X-DISPL.	Y-DISPL.	Z-DISPL.	XX-ROT.	YY-ROT.	ZZ-ROT.
1	0	0	0	0	0	0
2	0	-1.19E-05	1.43E-03	3.42E-04	0	0
3	0	-2.47E-05	1.44E-03	-8.66E-05	0	0
4	0	1.19E-05	1.43E-03	3.68E-04	0	0
5	0	0	0	0	0	0
6	0	0	0	0	0	0
7	0	1.19E-05	-1.43E-03	-3.68E-04	0	0
8	0	-4.99E-05	-1.44E-03	6.01E-05	0	0
9	0	-1.19E-05	-1.43E-03	-3.42E-04	0	0
10	0	0	0	0	0	0
11	0	5.37E-05	1.44E-03	9.85E-05	0	0
12	0	-2.71E-05	1.52E-03	-2.01E-04	0	0
13	0	-4.99E-05	1.44E-03	-6.01E-05	0	0
14	0	-6.92E-05	1.43E-03	1.49E-04	0	0
15	0	-6.92E-05	-1.43E-03	-1.49E-04	0	0
16	0	-4.98E-05	-1.44E-03	4.63E-05	0	0
17	0	-2.47E-05	-1.44E-03	8.66E-05	0	0
18	0	5.37E-05	-1.44E-03	-9.85E-05	0	0
19	0	0	1.52E-03	1.57E-10	2.23E-03	0
20	0	0	1.88E-10	1.57E-10	2.23E-03	0
21	0	0	0	0	0	0
22	0	0	-1.52E-03	1.57E-10	2.23E-03	0
23	0	-4.98E-05	1.44E-03	-4.63E-05	0	0
24	0	-2.71E-05	-1.52E-03	2.01E-04	0	0

MINIMUM/MAXIMUM DISPLACEMENTS

NODE	1	15	22	7	1	1
MIN.	0	-6.92E-05	-1.52E-03	-3.68E-04	0	0
NODE	24	11	19	4	20	24
MAX.	0	5.37E-05	1.52E-03	3.68E-04	2.23E-03	0

MAXIMUM RESULTANT DISPLACEMENT

NODE 12
 MAX. 1.51667E-03

TOTAL STRAIN ENERGY.....= .556872E+02

REACTION FORCE FOR LOAD CASE NO. 1

NODE	CSYS	Fx	Fy	Fz	Mx	My	Mz
1	0	0	1.14E+04	-1.65E+04	-3.13E+04	0	0
5	0	0	-1.14E+04	-1.56E+04	-3.05E+04	0	0
6	0	0	-1.14E+04	1.56E+04	3.05E+04	0	0
10	0	0	1.14E+04	1.65E+04	3.13E+04	0	0
21	0	0	0	-3.04E-04	-5.45E-04	-6324	0

FOR REQUESTED (Global Cartesian Coord. System)
NODES FX FY FZ MX MY MZ
Total React. .0000E+00 -.2201E-09 -.1669E-05 -.5121E-02 -.6324E+04 .0000E+00

SOLUTION TIME LOG IN SEC
FOR PROBLEM
TIME FOR INPUT PHASE= 1
TIME FOR CALCULATION OF STRUCTURE STIFFNESS MATRIX= 0
TRIANGULARIZATION OF STIFFNESS MATRIX= 0
TIME FOR LOAD CASE SOLUTIONS= 0
TOTAL SOLUTION TIME= 1

1

STRESS EVALUATION FOR STATIC ANALYSIS

* STRESS PRINTOUT (LOAD CASE 1) *

STATUS OF THERMAL EFFECTS FOR THIS LOAD CASE:

- Thermal flag is off (no thermal effect)

STRESS OUTPUT FOR BEAM ELEMENT GROUP 1 CASE NO. 1

ELEMENT	FORCES	MOMENTS	STRESSES			
NUMBER	NODE1	NODE2	NODE1	NODE2	NODE1	NODE2
1	[1 , 2]					
	Fr=	.1142E+05	-.1142E+05	Tr=	.0000E+00	.0000E+00
					(P/A)=	-.7732E+06
	Vs=	.1645E+05	-.1645E+05	Ms=	.0000E+00	.0000E+00
					(Ms/Ss)=	.0000E+00
	Vt=	.0000E+00	.0000E+00	Mt=	.3133E+05	.1922E+05
					(Mt/St)=	-.1811E+08
					(Tr*CTOR/Jp)=	.0000E+00
					Smax =	.1733E+08
					Smin =	-.1888E+08
2	[4 , 5]					
	Fr=	-.1142E+05	.1142E+05	Tr=	.0000E+00	.0000E+00
					(P/A)=	.7732E+06
	Vs=	.1562E+05	-.1562E+05	Ms=	.0000E+00	.0000E+00
					(Ms/Ss)=	.0000E+00
	Vt=	.0000E+00	.0000E+00	Mt=	.1749E+05	.3050E+05
					(Mt/St)=	-.1011E+08
					(Tr*CTOR/Jp)=	.0000E+00
					Smax =	.1088E+08
					Smin =	-.9337E+07

3 [6 , 7]
Fr=.1142E+05 .1142E+05 Tr=.0000E+00 .0000E+00 (P/A)=.7732E+06 .7732E+06
Vs=-.1562E+05 .1562E+05 Ms=.0000E+00 .0000E+00 (Ms/Ss)=.0000E+00 .0000E+00
Vt=.0000E+00 .0000E+00 Mt=-.3050E+05 -.1749E+05 (Mt/St)=.1763E+08 -.1011E+08
(Tr*CTOR/Jp)=.0000E+00 .0000E+00 Smax =.1840E+08 .1088E+08
Smin =-.1686E+08 -.9337E+07

4 [9 , 10]
Fr=.1142E+05 -.1142E+05 Tr=.0000E+00 .0000E+00 (P/A)=-.7732E+06 -.7732E+06
Vs=-.1645E+05 .1645E+05 Ms=.0000E+00 .0000E+00 (Ms/Ss)=.0000E+00 .0000E+00
Vt=.0000E+00 .0000E+00 Mt=-.1922E+05 -.3133E+05 (Mt/St)=.1111E+08 -.1811E+08
(Tr*CTOR/Jp)=.0000E+00 .0000E+00 Smax =.1034E+08 .1733E+08
Smin =-.1188E+08 -.1888E+08

5 [2 , 11]
Fr=.1645E+05 -.1645E+05 Tr=.0000E+00 .0000E+00 (P/A)=-.1460E+07 -.1460E+07
Vs=-.1142E+05 .1142E+05 Ms=.0000E+00 .0000E+00 (Ms/Ss)=.0000E+00 .0000E+00
Vt=.0000E+00 .0000E+00 Mt=-.1922E+05 .1458E+05 (Mt/St)=.1774E+08 .1345E+08
(Tr*CTOR/Jp)=.0000E+00 .0000E+00 Smax =.1628E+08 .1199E+08
Smin =-.1920E+08 -.1491E+08

6 [11 , 3]
Fr=-.4064E+04 .4064E+04 Tr=.0000E+00 .0000E+00 (P/A)=.3606E+06 .3606E+06
Vs=-.1940E+05 .1940E+05 Ms=.0000E+00 .0000E+00 (Ms/Ss)=.0000E+00 .0000E+00
Vt=.0000E+00 .0000E+00 Mt=-.1424E+05 .7630E+03 (Mt/St)=.1315E+08 .7042E+06
(Tr*CTOR/Jp)=.0000E+00 .0000E+00 Smax =.1351E+08 .1065E+07
Smin =-.1278E+08 -.3435E+06

7 [3 , 13]
Fr=-.1562E+05 .1562E+05 Tr=.0000E+00 .0000E+00 (P/A)=.1386E+07 .1386E+07
Vs=-.1142E+05 .1142E+05 Ms=.0000E+00 .0000E+00 (Ms/Ss)=.0000E+00 .0000E+00
Vt=.0000E+00 .0000E+00 Mt=.2659E+04 -.4908E+04 (Mt/St)=-.2454E+07 -.4529E+07
(Tr*CTOR/Jp)=.0000E+00 .0000E+00 Smax =.3841E+07 .5916E+07
Smin =-.1068E+07 -.3143E+07

8 [13 , 14]
Fr=-.1346E+05 .1346E+05 Tr=.0000E+00 .0000E+00 (P/A)=.1194E+07 .1194E+07
Vs=-.1179E+05 .1179E+05 Ms=.0000E+00 .0000E+00 (Ms/Ss)=.0000E+00 .0000E+00
Vt=.0000E+00 .0000E+00 Mt=.4383E+04 -.1258E+05 (Mt/St)=-.4045E+07 -.1161E+08
(Tr*CTOR/Jp)=.0000E+00 .0000E+00 Smax =.5239E+07 .1280E+08
Smin =-.2851E+07 -.1041E+08

9 [14 , 4]
Fr=-.1562E+05 .1562E+05 Tr=.0000E+00 .0000E+00 (P/A)=.1386E+07 .1386E+07
Vs=-.1142E+05 .1142E+05 Ms=.0000E+00 .0000E+00 (Ms/Ss)=.0000E+00 .0000E+00
Vt=.0000E+00 .0000E+00 Mt=.1285E+05 -.1749E+05 (Mt/St)=-.1186E+08 -.1614E+08
(Tr*CTOR/Jp)=.0000E+00 .0000E+00 Smax =.1324E+08 .1753E+08
Smin =-.1047E+08 -.1476E+08

10 [7 , 15]
Fr=-.1562E+05 .1562E+05 Tr=.0000E+00 .0000E+00 (P/A)=.1386E+07 .1386E+07
Vs=-.1142E+05 -.1142E+05 Ms=.0000E+00 .0000E+00 (Ms/Ss)=.0000E+00 .0000E+00
Vt=.0000E+00 .0000E+00 Mt=.1749E+05 -.1285E+05 (Mt/St)=.1614E+08 -.1186E+08
(Tr*CTOR/Jp)=.0000E+00 .0000E+00 Smax =.1753E+08 .1324E+08
Smin =-.1476E+08 -.1047E+08

11 [15 , 8]
Fr=-.1346E+05 .1346E+05 Tr=.0000E+00 .0000E+00 (P/A)=.1194E+07 .1194E+07
Vs=-.1179E+05 -.1179E+05 Ms=.0000E+00 .0000E+00 (Ms/Ss)=.0000E+00 .0000E+00
Vt=.0000E+00 .0000E+00 Mt=.1258E+05 -.4383E+04 (Mt/St)=-.1161E+08 -.4045E+07
(Tr*CTOR/Jp)=.0000E+00 .0000E+00 Smax =.1280E+08 .5239E+07
Smin =-.1041E+08 -.2851E+07

12 [8 , 17]
Fr=-.1562E+05 .1562E+05 Tr=.0000E+00 .0000E+00 (P/A)=.1386E+07 .1386E+07
Vs=-.1142E+05 -.1142E+05 Ms=.0000E+00 .0000E+00 (Ms/Ss)=.0000E+00 .0000E+00
Vt=.0000E+00 .0000E+00 Mt=.4908E+04 -.2659E+04 (Mt/St)=-.4529E+07 -.2454E+07
(Tr*CTOR/Jp)=.0000E+00 .0000E+00 Smax =.5916E+07 .3840E+07
Smin =-.3143E+07 -.1068E+07

13 [17 , 18]
Fr=-.4064E+04 .4064E+04 Tr=.0000E+00 .0000E+00 (P/A)=.3606E+06 .3606E+06
Vs=-.1940E+05 -.1940E+05 Ms=.0000E+00 .0000E+00 (Ms/Ss)=.0000E+00 .0000E+00
Vt=.0000E+00 .0000E+00 Mt=-.7632E+03 .1424E+05 (Mt/St)=.7043E+06 .1315E+08
(Tr*CTOR/Jp)=.0000E+00 .0000E+00 Smax =.1065E+07 .1351E+08
Smin =-.3437E+06 -.1278E+08

14 [18 , 9]
Fr=.1645E+05 -.1645E+05 Tr=.0000E+00 .0000E+00 (P/A)=-.1460E+07 -.1460E+07
Vs=-.1142E+05 -.1142E+05 Ms=.0000E+00 .0000E+00 (Ms/Ss)=.0000E+00 .0000E+00
Vt=.0000E+00 .0000E+00 Mt=-.1458E+05 .1922E+05 (Mt/St)=.1345E+08 .1774E+08

(Tr*CTOR/Jp)=.0000E+00 .0000E+00 Smax =.1199E+08 .1628E+08
Smin =-.1491E+08 -.1920E+08

17 [20 , 21]
Fr=.0000E+00 .0000E+00 Tr= -.6324E+04 .6324E+04 (P/A)=.0000E+00 .0000E+00
Vs=.0000E+00 .0000E+00 Ms= -.2978E-13 -.5450E-03 (Ms/Ss)=.1191E-10 -.2180E+00
Vt=.3036E-03 -.3036E-03 Mt=.0000E+00 .0000E+00 (Mt/St)=.0000E+00 .0000E+00
(Tr*CTOR/Jp)=-.1315E+07 .1315E+07 Smax =.1191E-10 .2180E+00
Smin =-.1191E-10 -.2180E+00

22 [3 , 12]
Fr= -.7973E+04 .7973E+04 Tr=.0000E+00 .0000E+00 (P/A)=.1686E+07 .1686E+07
Vs=.1156E+05 -.1156E+05 Ms=.0000E+00 .0000E+00 (Ms/Ss)=.0000E+00 .0000E+00
Vt=.0000E+00 .0000E+00 Mt=.3422E+04 -.7264E+02 (Mt/St)=-.1249E+08 -.2650E+06
(Tr*CTOR/Jp)=.0000E+00 .0000E+00 Smax =.1417E+08 .1951E+07
Smin =-.1080E+08 .1421E+07

23 [13 , 23]
Fr=.3646E+03 -.3646E+03 Tr=.0000E+00 .0000E+00 (P/A)=-.7709E+05 -.7709E+05
Vs=.2164E+04 -.2164E+04 Ms=.0000E+00 .0000E+00 (Ms/Ss)=.0000E+00 .0000E+00
Vt=.0000E+00 .0000E+00 Mt=.5248E+03 .1022E+03 (Mt/St)=-.1915E+07 .3728E+06
(Tr*CTOR/Jp)=.0000E+00 .0000E+00 Smax =.1838E+07 .2957E+06
Smin =-.1992E+07 -.4499E+06

24 [8 , 16]
Fr=.3646E+03 -.3646E+03 Tr=.0000E+00 .0000E+00 (P/A)=-.7709E+05 -.7709E+05
Vs=.2164E+04 -.2164E+04 Ms=.0000E+00 .0000E+00 (Ms/Ss)=.0000E+00 .0000E+00
Vt=.0000E+00 .0000E+00 Mt=.5248E+03 .1022E+03 (Mt/St)=-.1915E+07 .3728E+06
(Tr*CTOR/Jp)=.0000E+00 .0000E+00 Smax =.1838E+07 .2957E+06
Smin =-.1992E+07 -.4499E+06

25 [17 , 24]
Fr= -.7973E+04 .7973E+04 Tr=.0000E+00 .0000E+00 (P/A)=.1686E+07 .1686E+07
Vs=.1156E+05 -.1156E+05 Ms=.0000E+00 .0000E+00 (Ms/Ss)=.0000E+00 .0000E+00
Vt=.0000E+00 .0000E+00 Mt=.3422E+04 -.7264E+02 (Mt/St)=-.1249E+08 -.2650E+06
(Tr*CTOR/Jp)=.0000E+00 .0000E+00 Smax =.1417E+08 .1951E+07
Smin =-.1080E+08 .1421E+07

28 [11 , 12]
Fr=.2201E+05 -.2201E+05 Tr=.0000E+00 .0000E+00 (P/A)=-.1100E+08 -.1100E+08
Vs=.5369E+03 -.5369E+03 Ms=.0000E+00 .0000E+00 (Ms/Ss)=.0000E+00 .0000E+00
Vt=.0000E+00 .0000E+00 Mt=.3317E+03 .7264E+02 (Mt/St)=-.1068E+08 .2339E+07
(Tr*CTOR/Jp)=.0000E+00 .0000E+00 Smax =-.3208E+06 -.8664E+07
Smin =-.2169E+08 -.1334E+08

29 [23 , 14]
Fr= -.2137E+04 .2137E+04 Tr=.0000E+00 .0000E+00 (P/A)=.1069E+07 .1069E+07
Vs=.4960E+03 -.4960E+03 Ms=.0000E+00 .0000E+00 (Ms/Ss)=.0000E+00 .0000E+00
Vt=.0000E+00 .0000E+00 Mt=.1022E+03 .2714E+03 (Mt/St)=-.3291E+07 .8740E+07
(Tr*CTOR/Jp)=.0000E+00 .0000E+00 Smax =.4360E+07 .9809E+07
Smin =-.2222E+07 -.7672E+07

30 [15 , 16]
Fr= -.2137E+04 .2137E+04 Tr=.0000E+00 .0000E+00 (P/A)=.1069E+07 .1069E+07
Vs=.4960E+03 .4960E+03 Ms=.0000E+00 .0000E+00 (Ms/Ss)=.0000E+00 .0000E+00
Vt=.0000E+00 .0000E+00 Mt= -.2714E+03 -.1022E+03 (Mt/St)=.8740E+07 -.3291E+07
(Tr*CTOR/Jp)=.0000E+00 .0000E+00 Smax =.9809E+07 .4360E+07
Smin =-.7672E+07 -.2222E+07

31 [18 , 24]
Fr=.2201E+05 -.2201E+05 Tr=.0000E+00 .0000E+00 (P/A)=-.1100E+08 -.1100E+08
Vs=.5369E+03 -.5369E+03 Ms=.0000E+00 .0000E+00 (Ms/Ss)=.0000E+00 .0000E+00
Vt=.0000E+00 .0000E+00 Mt=.3317E+03 .7264E+02 (Mt/St)=-.1068E+08 .2340E+07
(Tr*CTOR/Jp)=.0000E+00 .0000E+00 Smax =-.3207E+06 -.8664E+07
Smin =-.2169E+08 -.1334E+08

SOLUTION TIME LOG IN SEC FOR STRESS CALCULATIONS

READING GENERAL INFORMATION AND ELEMENT DATA... = 0
STRESS CALCULATION AND PRINTOUT = 0
UPDATING DATABASE..... = 0
TOTAL SOLUTION TIME..... = 0

Figure C-1 Undeformed and Deformed (scaled at + 200 times the actual finite element model).

Appendix D

Formwork Design Drawings

Base (footing) Frame-work Parts

Dimensions in meters

Shear Wall Frame-work Parts

Fixed End
Part 2