OUT-OF-PLANE BENDING OF MASONRY WALLS WITH NEAR-SURFACE-MOUNTED AND EXTERNALLY-BONDED CORROSION-RESISTANT REINFORCEMENT

by

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ABSTRACT

Masonry walls subjected to out-of-plane loading, such as in a seismic event, require reinforcement to improve the ductility of the system. In current masonry construction practice, reinforcement is placed internally and fully grouted. For new construction this can make the wall unjustifiably heavy by not taking advantage of its light, hollow structure. For existing construction, it is difficult to retrofit using this technique. Additionally, the reinforcement is located close to the neutral axis which reduces its effectiveness.

Fibre-Reinforced Polymer (FRP) bars, strips and sheets are becoming increasingly popular in construction applications due to their noncorrosive nature and ease of installation. Also, stainless steel bars are used where the structure is exposed to a corrosive environment but have not found wider application for masonry structures.

This study is an experimental investigation of the structural performance of masonry walls reinforced with Near-Surface-Mounted (NSM) FRP and stainless steel reinforcement under out-of-plane bending. Additionally, walls with Externally Bonded (EB) FRP sheets were tested. The study simulates retrofitting applications and also proposes the NSM technique for new wall construction, using pre-grooved blocks, in lieu of the conventional method of internal reinforcing and grouting. To accommodate the NSM reinforcement, the grooves in the masonry blocks were aligned with ducts used to anchor the NSM reinforcement in the concrete footing. Seven wall specimens were tested, including walls reinforced with conventional and stainless steel bars, glass-fibre
reinforced polymer (GFRP), and carbon-FRP (CFRP) reinforcement. The study demonstrated the feasibility and effectiveness of the NSM technique for new construction. Walls with NSM reinforcement showed a superior performance to those with EB reinforcement. It was shown that increasing the FRP reinforcement ratio may result in a change of failure mode, and as such, the increase in strength may not be proportional to the increase in reinforcement ratio. NSM steel-reinforced walls showed a superior performance in terms of strength, stiffness and the ductility associated with the formation of a plastic hinge at the base.
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NOTATION

\( A_f \) Reinforcement Area

\( A_{CFRP} \) Area of Carbon Fiber Reinforced Polymer reinforcement

\( A_{GFRP} \) Area of Glass Fiber Reinforced Polymer reinforcement

\( A_{ss} \) Reinforcement area of stainless steel

\( f_m' \) Compressive strength of masonry normal to the bed joint at 28 days, MPa

\( f_y \) Yield strength of steel

\( f_{yss} \) Yield strength of stainless steel

\( f_u \) Ultimate strength of steel

\( f_{uCFRP} \) Ultimate strength of CFRP

\( f_{uGFRP} \) Ultimate strength of GFRP

\( CMU \) Concrete Unit Masonry

\( URM \) Unreinforced Masonry
CHAPTER 1: INTRODUCTION

1.1 General

This study investigates the flexural behavior of masonry walls reinforced using Near-Surface-Mounted (NSM) and externally bonded reinforcement, under out-of-plane bending. The walls were constructed using standard hollow concrete blocks. For the walls with NSM reinforcement, the blocks had grooves pre-cut in their sides, such that the grooves were aligned to accommodate reinforcement anchored in concrete footings. This simulates an envisioned new method of construction of reinforced masonry, in which external reinforcement is proposed in lieu of internally reinforced and grouted conventional walls. The method could also be used to retrofit existing walls.

In this study, conventional steel bars, stainless steel bars, glass-fiber reinforced polymer (GFRP) bars, carbon-fiber reinforced polymer (CFRP) flat strips, GFRP sheets and CFRP sheets of different reinforcement ratios were used and compared. Fiber reinforced polymers (FRPs) are a relatively new construction material. They are a promising reinforcement material in situations where structures are exposed to harsh environments. Traditional steel reinforced concrete is susceptible to corrosion in these environments.

Past investigations on the application of FRP in masonry construction indicate that FRP can significantly increase ductility of the walls (Baratta and Corbi, 2006). Masonry is commonly used for shear walls and load bearing walls in small- or medium-sized construction. Reinforcement is required to enhance the performance under service loads
and reduce deflection of masonry when subjected to out-of-plane loading. Introducing NSM or externally bonded reinforcement can enhance the strength, stiffness and ductility of the wall. Several studies have addressed the in-plane loading of these walls. This study is focused on out-of-plane bending and moment connections.

1.2 Objectives

The principle objective of this research program is to investigate the flexural performance of masonry walls reinforced with NSM and externally bonded corrosion-resistant reinforcement. The specific objectives addressed in this study are:

1. To compare the flexural performances of various types of corrosion-resistant reinforcements in terms of the ultimate load capacity and ability to sustain large deflections.

2. To evaluate the effects of these specific parameters on the flexural response of the masonry walls subjected to monotonic loading:
   a. Reinforcement material (GFRP, CFRP, Stainless Steel, Carbon Steel)
   b. Reinforcing material type (bars, sheets, strips)
   c. Reinforcement ratio
   d. Influence of the bond method (NSM versus external bonding)

3. To evaluate the feasibility of this system for the needs of the masonry industry.

4. To investigate potential changes in standard masonry blocks to accommodate the NSM reinforcement.
1.3 Scope

This study is an experimental investigation of the structural performance of reinforced masonry walls subjected to out-of-plane bending. In a seismic event, a masonry wall may be oriented such that its strong axis is parallel to the direction of ground acceleration and is subjected to out-of-plane loading, as seen in Figure 1.1.

As such, reinforcement is required to improve the strength, stiffness and ductility of the system. Currently walls are reinforced by installing 10M or 15M bars in a cell and grouting it in place. The spacing of the grouted cells varies from every 5th cell to every single cell. Figure 1.2 depicts the reinforcement layout.
The innovation proposed in this thesis is to use corrosion-resistance reinforcing materials installed by NSM or EB to resist out-of-plane loading. The flexural response is examined by subjecting seven medium scale masonry walls reinforced with NSM or externally bonded reinforcement to out-of-plane loading until failure. The experimental investigation’s main goal is to quantify the improvement in flexural performance of the masonry walls. Figure 1.3 shows examples from the literature of FRP being used to reinforce masonry walls.
Various types of corrosion-resistant reinforcement are used and compared, namely stainless steel, GFRP and CFRP rebars and strips as NSM reinforcement as well as externally bonded GFRP and CFRP sheets. Due to the relocation of the reinforcement away from the center of the wall and the use of various types of reinforcement, the design of the wall for the experiments is an important part of the research.
1.4 Outline of Thesis

The contents of this thesis are as follows:

**Chapter 2**: a review of previous research on FRP applications in masonry construction, behavior of masonry in out-of-plane loading situations, and potential retrofit of damaged unreinforced masonry walls using FRP reinforcement. As the literature on NSM reinforcement in masonry is very limited, corresponding reinforced concrete research literature is referenced where appropriate.

**Chapter 3**: presents a detailed description of the experimental program. The items described in this chapter include fabrication of the test specimens and coupon prisms, instrumentation, test setup and testing procedures. Ancillary tests conducted to obtain material properties are also described.

**Chapter 4**: presents the results of the experimental investigation, discusses failure modes and compares in detail the performance of the specimens in the study.

**Chapter 5**: presents the conclusions drawn from this research study and provides recommendations for further research and implementation.

**References**

**Appendix A**: a validation of one of the test results due to unexpected failure that occurred during the main phase of testing.

**Appendix B**: additional testing data
2.1 Introduction

Fiber reinforced polymer (FRP) reinforcement provides engineers with an innovative and effective reinforcing system. Its diversity and versatility provides an opportunity to explore new designs.

The ultimate goal of this study is to investigate FRP and stainless steel as viable reinforcement options for masonry construction. The goal is to improve the structural performance of the wall system. However, there are other potential benefits to using these reinforcement materials. The NSM reinforcement technique precludes the need to grouting cells, which will reduce the weight of the wall system. It could be argued that weight savings could be achieved by using typical carbon steel as an NSM reinforcement rather that the conventional interior placement and grouting. However, carbon steel requires a minimum cover of concrete or mortar to inhibit corrosion and this may limit its use as a NSM reinforcement. This is not an issue for corrosion resistant reinforcement.

As will be discussed in this chapter, previous research has examined FRP as NSM reinforcement. This work has shown that stiffness can be substantially improved and the performance of masonry walls when subjected to forces typical of earthquakes can be drastically improved. Reinforcing the wall from both sides effectively results in a sandwich panel composite wall with a higher moment of inertia compared to a wall reinforced conventionally.
2.2 Out-of-Plane Behavior of Masonry

Hamed and Rabinovitch (2006) studied the out-of-plane behavior of masonry walls externally strengthened with GFRP strips. The effects of four combinations of supporting conditions with different restrictions of the longitudinal elongation and rotation were examined; models simulating various support conditions were investigated. Equilibrium equations, and compatibility requirements between all of the structural components are constituents of the model used. The testing phase is summarized in Figure 2.1.

![Figure 2.1 Geometry, material properties, and loads](image)

(a) Geometry and loads; (b) Cracked mortar joint and debonded regions; (c) Cross section; (d) Mechanical properties; (e) Supporting conditions. (Hamed and Rabinovitch, 2006)

Out-of-plane deflections, bending moments and the axial forces registered during the testing, divided by different supporting conditions are summarized in Figure 2.2. They found that by restraining longitudinal elongation, out-of-plane deflection can be significantly decreased. This was based on a comparison between two simply supported cases.
Figure 2.2 Response of the strengthened wall with different supporting conditions: (a) Out-of-plane deflections; (b) Axial forces in the masonry; (c) Moments in the masonry; (d) Axial forces in the FRP, (Hamed and Rabinovitch, 2006)

Arching forces under such restraint are credited with increasing the cracking load and providing the wall with the ability to resist bending moments by means of eccentric thrust forces. The authors also reported in their findings that the variation of the flexural rigidity of the masonry wall between the masonry unit section and the cracked joint section influences the distributions of the bending moments and the axial forces through the height of the wall. Samples with support conditions not inducing the arching action in the masonry were predicted to fail by rupture of the FRP. Laboratory tests confirmed their hypothesis.
Shear and out-of-plane stresses within the adhesive layer are typical local effects for externally bonded systems. Figure 2.3 depicts the stress distribution through the height of the wall. Distribution of the (peeling) stresses near midspan for (I) and (II) loading conditions reveal the development of the stress concentrations near the cracked mortar joints. The authors found that in free support conditions, the absence of arching action emphasizes the contribution of the FRP strengthening system.

![Figure 2.3 Stresses in the adhesive under supporting conditions (I) and (II): (a) Shear stresses; (b) Peeling stresses at the adhesive-FRP interface; (c) Peeling stresses at the adhesive-masonry interface, (Hamed and Rabinovitch, 2006)](image)

The conclusion of the study was that the potential restriction of the longitudinal deformation plays a major role in the detailing of the strengthening system. It directly influences the expected failure mode of the strengthened wall.

### 2.3 Retrofit of Damaged Structures

Willis, Yang, Seracino, Xia and Griffith (2006) conducted an investigation on the out-of-plane behavior of severely damaged unreinforced masonry walls retrofitted with vertical external CFRP and GFRP strips. Two main failure modes were reported: FRP debonding and masonry collapse. Figure 2.4 depicts the static load-displacement behavior of an
unreinforced wall in comparison to its behavior after reinforcing it with FRP. The envelope of the unreinforced wall cyclic behavior after damage and the static behavior of the retrofitted wall are presented as well. It was reported that the retrofitted wall has a lateral load carrying capacity approximately 2.5 times that of the damaged wall.

![Figure 2.4 Typical Load-Displacement Behavior, (Willis, Yang, Seracino, Xia and Griffith, 2006)](image)

**Figure 2.4 Typical Load-Displacement Behavior, (Willis, Yang, Seracino, Xia and Griffith, 2006)**

### 2.4 Structural Ductility Improvement

Baratta and Corbi (2006) examined the CFRP reinforcement of masonry portal arches made of tuff brick with lime mortar. Both reinforced and unreinforced arches were subjected to self-weight and lateral forces. Figures 2.5 and 2.6 respectively depict the two kinds of test specimens considered and the comparison of model predictions to experimental data.
Chapter 2  Literature Review

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Figure 2.6 Absolute displacement $u$ (mm) of the right abutment versus the horizontal force $F$ (N)
 a) unreinforced and b) reinforced model.
Derakhshan and Ingham (2007) conducted a study on the seismic behavior of unreinforced masonry walls under out-of-plane loading. Testing performed included out-of-plane uniform static loading on a full-scale unreinforced masonry wall with a slenderness ratio (h/t) of 16. Several different levels of pre-compression were applied using an air bag before testing to failure. The authors found that differing levels of overburden significantly changed the shape of the tri-linear hysteresis behaviour. The hysteresis can be seen in Figure 2.7.

![Force-Displacement Curve](image)

Figure 2.7 Force-Displacement Curve

(Derakhshan and Ingrham 2007)

### 2.5 Bond Related Studies

Bond characteristics are not in the scope of this research. Research in this area is ongoing and looks into the bond between FRP and masonry or concrete.
2.5.1 Externally Bonded FRP

Ceroni and Pecce (2006) examined bond behavior under cyclic loading. The effect of temperature on the specimen was examined as well. Various types of anchorage options for FRP applications on both concrete and masonry were investigated.

Concrete masonry blocks were subjected to tensile loading. CFRP sheets with a width of 100m were used as the reinforcement and the bonded length was varied. Figure 2.8 lists the failure modes and corresponding ultimate loads based on the geometrical properties of the specimen. Physical tests results were compared with various theoretical debonding loads reported in terms of maximum tensile load according to CNR DT200/2004 and other sources and were successfully compared.

<table>
<thead>
<tr>
<th>Specimen</th>
<th>Test</th>
<th>$B$ [mm]</th>
<th>$H$ [mm]</th>
<th>$L$ [mm]</th>
<th>$B_o$ [mm]</th>
<th>$L_o$ [mm]</th>
<th>$F_{mai}$ [kN]</th>
<th>Failure</th>
<th>Fib bulletin 14 [kN]</th>
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<td>$PCR$</td>
<td>12.5</td>
<td>15.8</td>
<td>13.5</td>
</tr>
</tbody>
</table>

DB = superficial debonding, $P$ = debonding with concrete detachment, $CR$ = concrete failure, $SH$ = shear failure of block

Figure 2.8 Experimental tests on concrete specimens strengthened with CFRP (Ceroni and Pecce 2006)
Camli and Binici (2006) researched the strength of CFRPs bonded to low and normal compressive strength concrete and masonry units finished with or without plaster. The ultimate strength of CFRPs bonded to masonry walls not finished with plaster was significantly higher than those with plaster. This research is relevant to the current thesis as it looked into dowel connections for FRP, proving that development length and connection research is a potential topic for a research. "It was found that ultimate strength of surface bonded CFRPs with the use of embedment at the free end and FRP dowels that act as shear connectors can result in strength enhancements as high as three times of those without any special anchorage." (Camli and Binici (2006))

The research was conducted using the setup shown in Figure 2.9 and utilized three different types of CFRP configurations. CFRP sheets for strip anchors, embedded anchors and for hollow clay tile are shown in Figure 2.10.
In total 51 double shear pull out tests were performed. Parameters studied included the strength of the concrete prisms or HCTs, presence of plaster, CFRP width, CFRP bond length, embedment depth and anchor type (strip, and embedded anchors). Figure 2.11 presents the results of the experimental program in the form of normalized strength versus bond length. The results showed that up to a bond length of 100 mm the load carrying capacity increased for the strip anchor specimen. Increasing the bond length beyond 100 mm did not result in strength enhancement. The failure mode for all specimens in this group was debonding of the CFRP from the concrete surface. Specimens with a plaster finish proved that even a thin layer adversely affected the ultimate strength. Specimens with embedded anchors showed that embedment enables the strength of CFRPs to develop up to about 65% of their ultimate strength. However, full utilization of the strength of CFRPs was not possible due to stress concentrations occurring around smoothened corners. Compressive strength of CMUs was not a factor when it came down to ultimate shear capacity of anchorages. Also the authors reported
that for specimens with embedded anchors presence of plaster reduced the strength by about 30%. The strength of CFRPs bonded to HCTs equipped with CFRP dowels were comparable to the strength of CFRPs bonded to concrete using embedded anchors.

![Figure 2.11 Test Results for Strip and Embedded Type CFRP Anchors (Camli and Binici 2006)](image)

**2.5.2 FRP to Concrete Bond Studies**

Diab and Wu (2006) focused on developing a new nonlinear viscoelastic model as a means to understand the long term behavior of an FRP-concrete interface. According to their tests the performance over time of the adhesive layer is directly affected by time
dependent degradation of the bond strength. The theoretical model was verified by a series of sheer and creep laboratory tests. The calibration was conducted using double lap shear test which setup is presented in Figure 2.12. Comparison of their new model to previously developed ones is depicted in Figure 2.13.

Figure 2.12 Detail of FRP sheet-concrete bonding joints (Double-lap shear test) (Diab and Wu, 2006)

![Figure 2.12 Detail of FRP sheet-concrete bonding joints (Double-lap shear test) (Diab and Wu, 2006)](image)

Figure 2.13 A nonlinear viscoelastic model, Linear Model and time-dependent behavior of maximum shear stress, (Diab and Wu, 2006)

![Figure 2.13 A nonlinear viscoelastic model, Linear Model and time-dependent behavior of maximum shear stress, (Diab and Wu, 2006)](image)

2.5.3 NSM-FRP to Concrete Bond Studies

The research of Galati and Lorenzis (2006) is important as reinforcing concrete with NSM (NSM) fiber-reinforced polymer (FRP) reinforcement is the closest parallel to the NSM of FRPs in masonry construction. Their paper deals with bond characteristics obtained from tests conducted on short NSM-bar anchorages. Among the variables were the groove depth and width to depth ratio and the mechanical properties of the groove-
filling epoxy with a constant bond length of 37.5 mm, corresponding to 5 times the nominal diameter of the bar. A total of 24 NSM joints were tested. Figure 2.14 shows the average bond stress-slip curves and Figure 2.15 shows the transverse strain-load curves.

Figure 2.14 Average bond stress-slip curves, (Galati and De Lorenzis, 2006)
Figure 2.15 Transverse strain-load curves, (Galati and De Lorenzis, 2006)

Figure 2.16 lists the failure modes. Research showed that the type of epoxy used can induce specific failure mode. Increasing groove dimensions has little effect on local bond strength, but it does significantly improve fracture energy of the joint.
<table>
<thead>
<tr>
<th>Specimen</th>
<th>w/d</th>
<th>$k = \frac{d}{d_{cr}}$</th>
<th>Measured w/d</th>
<th>Measured k</th>
<th>$P_{max}$ (kN)</th>
<th>$\tau_{wp}$ (MPa)</th>
<th>$s_{eff}$ (mm)</th>
<th>$s_{ip}$ (mm)</th>
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Note: BE-C = failure at the bar-epoxy interface - cohesive shear failure in the epoxy; LC = longitudinal cracking of the epoxy; SP = splitting failure by fracture in the concrete along inclined planes; N/A = not available; *average of available data.

Figure 2.16 Test program and results, (Galati and De Lorenzis, 2006)
2.6 Retrofit of Masonry Structures

In recent years, amplified by the tragic events of September 11, well deserved attention has been paid to the subject of the resistance of structures to explosives. Military research facilities and their personnel have been looking into the resistance of new structures to blast effects and into possibilities of retrofitting existing, potentially damaged structures with FRP composites.

2.6.1 CMU Wall Retrofit Analysis

Dennis, Baylot and Woodson (2002) conducted a series of experiments on one-way 1/4-scale CMU walls to determine their response mechanism to the detonation of explosive charges. Results of the laboratory research were verified with finite-element analysis. Each CMU was modeled with eight-node continuum elements. Adjacent CMUs were tied together using a slide surface that models a rigid connection until a failure criterion is met. The results of five blast-load experiments confirmed their model predictions hence proved the models to be effective tool for response mechanism prediction.

Slawson, Johnson and Davis (2004) researched spray-on elastomers as a reinforcement on masonry structures subjected to a blast wave. In this technique a thin layer of a commercially available material is sprayed directly on the masonry. Methods of analysis considered include the behavior of the composite and CMU wall in determining ultimate flexural capacity.

They compared a series of 1/4 scale tests to the predicted resistance. Figure 2.17 depicts their comparison and confirmation of the model. Among their conclusions the ultimate
resistance of the unretrofitted hollow CMU wall was half of the retrofitted wall, which indicates significant composite action that cannot be ignored in design unless overly conservative predictions are acceptable.

Figure 2.17 (a) Comparison of the predicted resistance function for a hollow CMU wall with data, (b) Comparison of elastomer retrofit resistance functions and (c) Comparison of calculated retrofit performance, (Slawson, Johnson and Davis 2004)
2.6.2 Explosion Resistance

Ward (2004) examined methods to protect existing, damaged and undamaged masonry. FRP composites used to reinforce the wall for a blast wave were applied in sheets. His conclusions on ease of application were: “Requires extensive wall preparation and complete reprovision of internal building services. Special provision is required for load-bearing walls and around windows”. The structural ductility increase that Ward reported was considerable over other materials.

2.7 Summary

The current research proposes to investigate the flexural performance of masonry wall reinforced with a wide range of reinforcement materials. The literature review outlined above indicates this has not been done previously. The current research will also investigate different reinforcement configurations and locations (i.e. NSM and EB). Past researchers have not conducted a comparison like this previously.

In addition, previous NSM research used a technique that involved the cutting of the groove into the already assembled structure. Another innovation of the current research is the use of pre-grooved blocks.
CHAPTER 3: EXPERIMENTAL INVESTIGATION

3.1 General

The experimental program focused on investigating reinforced masonry walls with NSM and externally bonded FRP, stainless steel and carbon steel reinforcement subjected to out-of-plane flexural loading. Seven medium scale masonry walls were constructed as part of the investigation. The connection between the walls and reinforced concrete footings was a key part of the study.

Three types of corrosion-resistant reinforcement were used and compared to conventional steel reinforcement: 316LN stainless steel rebar, glass fiber reinforced polymer (GFRP) rebar (Aslan 100); and, carbon fiber reinforced polymer (CFRP) tape (Aslan 500). These materials have several advantages that make them attractive for use as reinforcement in masonry. All the materials are corrosion-resistant. The FRPs have high strength- and stiffness-to-weight ratios in comparison to conventional steel. The parameters studied were the NSM longitudinal reinforcement ratio, in terms of rebar size; type of reinforcing material (GFRP, CFRP and Stainless Steel) and location of the material (NSM and EB) and the effect of the monotonic loading normal to the weak axis of the wall, on the strength, stiffness, and ductility of the reinforced masonry wall.

This chapter describes in detail the test specimens, parameters, fabrication process, instrumentation, test setup, and testing procedures.
3.2 Materials used for Fabrication of Test Specimens

The materials that were used in the experimental program are discussed in this chapter in order of usage for the construction of the specimens and are as follows:

1. Concrete masonry blocks
2. Concrete for the footings
3. Mortar
4. Carbon steel rebar
5. Stainless steel rebar
6. GFRP rebar
7. GFRP sheets
8. CFRP strip
9. CFRP sheets
10. Epoxy resin used to bond NSM reinforcement to the walls
11. Epoxy resin used to bond FRP sheets to the walls.

Concrete properties were determined from standard concrete cylinder tests; the strength of the masonry was established from a combination of mortar coupon tests and standard masonry prism tests. The mechanical properties of the reinforcement were taken as per specifications from manufacturers. This was done to avoid performing coupon tests for the large number of reinforcement materials used in the experiments. Properties of PVC corrugated pipes used to embed the various kinds of reinforcement into the concrete footing will not be discussed in this chapter. Their purpose, location and brief characteristics will be described in Section 3.5.2. Even though their usage was crucial for proper reinforcement placement and embedment they were not a structural component of the wall system.
3.2.1 Concrete Masonry Blocks

The concrete masonry blocks that were used for this research were standard 20 MPa concrete masonry units (CMUs), grade I of a nominal size 200mm x 200mm x 400mm. The dimensions of the blocks are shown in Figure 3.1.

Figure 3.1 Standard concrete masonry block sizes used in the research

Some of the blocks had groves cut into the outer surface, as part of this study, to accommodate external NSM reinforcement (shown, for example in Fig. 3.2).

Figure 3.2 NSM specimens details and dimensions
### 3.2.2 Concrete

All the footings for the masonry walls used in this study were cast and poured on the same day using one delivery of ready mix concrete. Concrete of minimum 70MPa compressive strength at 28 days was obtained from a local concrete supplier. The purpose of the high strength was to ensure that no failure occurred in the footing. Cylinders were cast to confirm the compressive strength of concrete.

### 3.2.3 Mortar

The mortar used for the masonry joints was mixed in-place during construction of the walls. Materials used in the mix were rigorously screened. The lime used was typical masonry lime; the cement used was ordinary portland cement, according to CAN/CSA-A179-04. The design mix of the mortar can be found in Table 3.1.

<table>
<thead>
<tr>
<th>Materials</th>
<th>Parts by weight</th>
</tr>
</thead>
<tbody>
<tr>
<td>Cement</td>
<td>3</td>
</tr>
<tr>
<td>Lime</td>
<td>1</td>
</tr>
<tr>
<td>Sand</td>
<td>12</td>
</tr>
<tr>
<td>Water</td>
<td>1.5</td>
</tr>
</tbody>
</table>

Sand used in the mortar mix was subjected to a sieving analysis and was properly dried to maintain uniform properties of all batches of mortar.
3.2.4 Carbon steel rebar

The steel used to reinforce the control specimen was standard 10M (11.3mm diameter) conventional ‘black’ steel reinforcing bar (in accordance with CAN/CSA G30.18-09), two bars per side. The modulus of elasticity is 200 GPa.

3.2.5 Stainless steel rebar

The stainless steel rebar used was 316 LN alloy grade. Two 9.6mm diameter reinforcing bars were used per side. The modulus of elasticity is 200 GPa.

3.2.6 GFRP rebar

Aslan 100 GFRP reinforcement bars were used. Walls were reinforced with two 9.53mm nominal diameter GFRP bars on each side of the wall. Figure 3.3 shows the typical tensile properties as provided by the manufacturer.

![Typical Stress / Strain Curve for GFRP Rebar](image1)
![Typical Load/Slide Curve for GFRP Rebar](image2)

Figure 3.3 Aslan 100 GRFP properties, (www.hughesbros.com)
The modulus of elasticity is 40.8 GPa. Typical installation layout is presented in Figure 3.4.

![Figure 3.4 External specimens details and dimensions](image)

### 3.2.7 GFRP sheets

Tyfo SEH-51A-SW-1 glass fiber sheets were used. The tensile strength and Young’s modulus of the cured laminate as reported by the manufacturer were 517 MPa, and 23.5 GPa, respectively. The nominal thickness of a cured laminate is 1.3 mm. Detailed properties of the Tyfo SEH-51A-SW-1 system are listed in Table 3.2.

<table>
<thead>
<tr>
<th>Fiber Properties</th>
<th>Design Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Primary fiber direction</td>
<td>0° Unidirectional</td>
</tr>
<tr>
<td>Tensile strength</td>
<td>3.24 GPa</td>
</tr>
<tr>
<td>Tensile E-modulus</td>
<td>72.4 GPa</td>
</tr>
<tr>
<td>Ultimate Elongation</td>
<td>4.5%</td>
</tr>
<tr>
<td>Density</td>
<td>2.55 g/cm³</td>
</tr>
<tr>
<td>Area weight</td>
<td>915 g/m²</td>
</tr>
<tr>
<td>Cured Laminate Properties</td>
<td></td>
</tr>
<tr>
<td>with Tyfo SW-1 at 21 days</td>
<td></td>
</tr>
<tr>
<td>Epoxy Resin</td>
<td></td>
</tr>
<tr>
<td>Tensile strength</td>
<td>517 MPa</td>
</tr>
<tr>
<td>Tensile modulus</td>
<td>23.5 GPa</td>
</tr>
<tr>
<td>Elongation at Break</td>
<td>2.0%</td>
</tr>
<tr>
<td>Laminate thickness</td>
<td>1.3 mm</td>
</tr>
</tbody>
</table>
3.2.8 CFRP strip

Aslan 500 No.2 CFRP strip was used. The strip has nominal cross-sectional dimensions of 16x2 mm, with a 31.2 mm$^2$ cross-sectional area. The tensile strength and Young’s modulus reported by the manufacturer were 2068 MPa and 124 GPa, respectively.

3.2.9 CFRP sheets

Tyfo SCH-41S-SW-1 carbon fiber sheets were used. The tensile strength and Young’s modulus of the cured laminate as reported by the manufacturer were 786 MPa, and 55.3 GPa, respectively. The nominal thickness of a cured laminate is 1.0 mm. Detailed properties of Tyfo SCH-41A-SW-1 system are listed in Table 3.3.

<table>
<thead>
<tr>
<th>Fiber Properties</th>
<th>Design Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Primary fiber direction</td>
<td>0° Unidirectional</td>
</tr>
<tr>
<td>Tensile strength</td>
<td>3.79 GPa</td>
</tr>
<tr>
<td>Tensile E-modulus</td>
<td>230 GPa</td>
</tr>
<tr>
<td>Ultimate Elongation</td>
<td>1.7%</td>
</tr>
<tr>
<td>Density</td>
<td>1.74 g/cm$^3$</td>
</tr>
<tr>
<td>Area weight</td>
<td>644 g/m$^2$</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Cured Laminate Properties with Tyfo SW-1 at 21 days Epoxy Resin</th>
</tr>
</thead>
<tbody>
<tr>
<td>Tensile strength</td>
</tr>
<tr>
<td>Tensile modulus</td>
</tr>
<tr>
<td>Elongation at Break</td>
</tr>
<tr>
<td>Laminate thickness</td>
</tr>
</tbody>
</table>

3.2.10 Epoxy for Embedment of Reinforcement

Sikadur 330 epoxy resin of a thick consistency was used as a bonding material for all the NSM reinforcement.
3.2.11 Epoxy Resin for External GFRP and CFRP Sheets

Tyfo S epoxy resin with a thin consistency was used to bond the EB sheets using the wet-lay up process. To facilitate bonding the sheets to the masonry surface a layer of primer was first applied to the wall. It consisted of the same Tyfo S epoxy resin mixed with silica fume powder to provide a thicker consistency.

3.3 Test Specimens and Parameters

The full list of specimens, their properties, reinforcement type per specimen and reinforcement ratio, as well as the designations are provided in Table. 3.4. The walls were reinforced in the following way (amount given is per side):

1. NSM double carbon steel rebar (DBSR)
2. NSM double GFRP rebar (DGFRPR)
3. NSM single GFRP rebar (SGFRPR)
4. NSM double Stainless Steel rebar (DSSR)
5. NSM single CFRP strip (CFRPS)
6. External GFRP sheet (EGFRPS)
7. External CFRP sheet (ECFRPS)
<table>
<thead>
<tr>
<th>Spec. ID</th>
<th>Reinforcement Type</th>
<th>Reinf. Quant.</th>
<th>Unit $A_r$/side [mm$^2$]</th>
<th>Total $A_r$/side [mm$^2$]</th>
<th>Young’s Modulus $E$ [GPa]</th>
<th>Yield (*) and Ultimate (**) Strength $f_y$ - $f_u$[MPa]</th>
<th>Tension force Ty-Tu[kN]</th>
</tr>
</thead>
<tbody>
<tr>
<td>DBSR</td>
<td>Black steel</td>
<td>2 No. 10</td>
<td>100</td>
<td>200</td>
<td>200.0</td>
<td>425(*) – 680(**)</td>
<td>85 - 136</td>
</tr>
<tr>
<td>DSSR</td>
<td>Stainless steel</td>
<td>2 No. 10</td>
<td>71</td>
<td>142</td>
<td>200.0</td>
<td>550(*) – 820(**)</td>
<td>78 – 117</td>
</tr>
<tr>
<td>SGFRPR</td>
<td>GFRP bars</td>
<td>1 No. 9</td>
<td>84</td>
<td>84</td>
<td>40.8</td>
<td>760(**)</td>
<td>64</td>
</tr>
<tr>
<td>DGFRPR</td>
<td>GFRP bars</td>
<td>2 No. 9</td>
<td>84</td>
<td>168</td>
<td>40.8</td>
<td>760(**)</td>
<td>128</td>
</tr>
<tr>
<td>CFRPS</td>
<td>CFRP strips</td>
<td>1 No. 2</td>
<td>31</td>
<td>31</td>
<td>124.0</td>
<td>2068(**)</td>
<td>64</td>
</tr>
<tr>
<td>ECFRPS</td>
<td>Tyfo SCH-41A-SW-1</td>
<td>1 ply 160mm wide</td>
<td>1.0mm</td>
<td>160</td>
<td>230</td>
<td>786(**)</td>
<td>126</td>
</tr>
<tr>
<td>EGFRPS</td>
<td>Tyfo SEH-51A-SW-1</td>
<td>1 ply 200mm wide</td>
<td>1.3mm</td>
<td>260</td>
<td>72.4</td>
<td>517(**)</td>
<td>134</td>
</tr>
</tbody>
</table>
3.4 Specimen Comparison Method

Due to the variety of reinforcement types used, it was decided to design the walls such that they have similar tensile reinforcement capacity. This is based on an assumption that at the ultimate capacity of each wall, yield or rupture of the tensile reinforcement would occur.

For the purpose of this study the “control” specimen is DBSR (Table 3.4). In practice, if a masonry wall is reinforced, the steel rebar is placed inside the block near the centre of the masonry block and then the block is filled with grout. In the case of DBSR, the NSM carbon steel rebar was placed in the external grooves of the masonry blocks. This is the proposed method in this study, to enhance effectiveness, even for new construction.

The area of reinforcement of the other six walls was selected so that the expected tensile force at the failure of the wall was equal to that of DBSR.

For example, given the area of carbon steel, $A_{BS}$ on the tension side of wall DBSR, and assuming the wall fails when the rebar reaches its yield strength $f_{yBS}$ then a wall reinforced with CFRP would be designed so that:

$$A_{BS}f_{yBS} = A_{CFRP}f_{uCFRP}$$  \hspace{1cm} (1)

where $A_{CFRP}$ is the cross-sectional area of the CFRP on the tension side of the wall and $f_{uCFRP}$ is the tensile strength of the CFRP. Therefore, the area of the CFRP reinforcement is selected so that:
In the mentioned above design consideration compression reinforcement is not mentioned for ease of comparison explanation. It does not mean that the compression side reinforcement will not add to capacity of the wall but it is assumed that its capacity will not be as well utilized as the tension side reinforcement. In a seismic event the wall would not be subjected to monotonic loading. Ground acceleration would cause the structure to move back and forth hence tension and compression developed in the reinforcement would be constantly switching.

It was not clear at the beginning of the study if the wall would fail when the steel reinforcement reached its yield or its ultimate strength, so in general the area of the other reinforcement types was selected such that its ultimate tensile force would fall in the range between yield and ultimate tensile forces.

As the forces for the DBSR wall ranges from 85kN at yield to 136kN at ultimate, all other specimens were designed to relate to this range. Variety of reinforcement types equals variety of mechanical properties which poses a design problem. As reinforcement bars come in standard sizes its quantity can be only designed in increments equal to one bar. The only reinforcements that are fully adjustable in the design stage are CFRP and GFRP sheets. As they can be cut to desired width of the sheet design tension forces developed in the tension side reinforcement can be precisely adjusted to directly compare to the control sample.
Specimen DGFRPR has reinforcement with a tensile force capacity of 128kN, allowing it to be compared directly to DBSR. Both specimens SGFRPR and CFRPS were designed to provide 50 percent of the reinforcement ratio, and hence tensile strength, of specimen DGFRPR. Since they have similar tensile capacities, SGFRPR and CFRPS can be compared to quantify the performance of two different FRP reinforcement types, namely CFRP strips and GFRP bars. The tensile force in specimen DSSR with stainless steel rebar ranged from 78 kN at yield to 117 kN at ultimate, a range that overlaps with specimen DBSR and allows for a direct comparison. Specimen ECFRPS will be compared to CFRPS to quantify the performance of CFRP sheets against NSM strips. Specimen EGFRPS will be compared to SGFRPR to quantify the performance of GFRP sheets against NSM bars.

3.5 Fabrication of Test Specimens

3.5.1 Block Preparation

The cutting of the blocks was done in the laboratory with a table stone saw.

The blocks for the NSM specimens had to be grooved prior to the construction of the walls. The grooving of the blocks was done with the hand held masonry repointer; this device is usually used in the retrofitting of masonry structures to remove mortar beds.

Details of all sizes of the grooves, as per the reinforcement kind that they had to accommodate, are depicted in Figure 3.5.
All grooves had a constant depth of 16mm. Blocks for specimens DBSR, DSSR and DGFRPR were prepared with a 32mm wide groove, blocks for specimens SGFRPR had 16mm wide grooves, and blocks for specimens CFRPS had a 6mm wide groove. After accomplishing proper depth and width of the grooves, the surface of the blocks inside the grooves was roughened with a chisel and a hammer to provide a better bonding surface for the epoxy.

Figure 3.5 Modifying and Grooving blocks
3.5.2 Footing Construction

To simplify construction and analysis, the reinforcement was fabricated to be continuous throughout the whole length of the wall.

Even though the footings on which the walls were constructed were not the focus of the study themselves, proper embedment of the reinforcement into the footing ensures premature failure does not occur. Due to differences in required development length for the different reinforcement types, the footing was designed for the largest of the development lengths. Steel reinforcement was provided in the footing to avoid their flexural failure.

To accommodate the reinforcement, corrugated polyvinyl chloride (PVC) ducts were cast inside the footings. The reinforcement was inserted after the walls were constructed. The setup of all the PVC ducts and reinforcement of the footings is depicted on Figure 3.6. As the epoxy that was used to bond the reinforcement to the walls was very viscous, the duct was chosen to be larger than the reinforcement diameter to ensure no air voids were trapped in the ducts.
Figure 3.6 Footing for the masonry walls
(a) Location of ducts in the cast (b) ducts in place 
(c) batch cast (d) reinforcement (e) curing process
3.5.3 Wall Construction

All the walls were constructed with the following dimensions:

- Height of the wall without the footing: 1.2m (6 rows of blocks including the standard 10mm mortar joints between the rows)
- Length of the wall: 1.0m (2.5 standard two-cell stretcher blocks per row in a half running bond pattern)
- All footings have dimensions of height 0.6m, thickness 0.3m, and length 1.2m

The walls were constructed by a professional mason using conventional equipment and techniques. Construction time for the walls with grooves was not significantly longer than that for walls without grooves. Quality standards for the mortar were maintained according to CAN/CSA-A179. Figure 3.7 depicts different stages of the wall construction.
Figure 3.7 Wall construction,
Sequence of construction: (a) verification of alignment (b) bottom row (c) running bond pattern (d) finished sample for external reinforcement (e) and for NSM (f) finished batch
3.5.4 Reinforcement Preparation

During the curing period of the walls, the reinforcement was prepared.

3.5.4.1 NSM Reinforcement Preparation

The reinforcement was precut to a length of 1.7m. GFRP rebar and CFRP strips were cleaned with alcohol to remove all possible oily substances from the surface. Stainless steel was also cleaned with alcohol. Carbon steel rebars were cleaned of rust with a wire brush and stored in a very low humidity area awaiting the installation.

3.5.4.2 External Reinforcement Preparation

Glass fiber sheets were precut to the following dimensions: four strips 1.7m long (with the direction of the fibers) and 200mm wide were cut for the vertical reinforcement, two strips 1.0m long and 200mm wide for bottom row protection as it was undesirable for the specimen to fail prematurely due to debonding of the reinforcement induced by the geometry of the bottom of the wall.

Carbon fiber sheets were precut to the following dimensions: four strips 1.7m long and 160mm wide for the vertical reinforcement and two strips 1.0m long and 200mm wide for the bottom row protection for the same reasons as for EGFRPS specimen.

3.5.5 NSM Reinforcement Installation

After curing of the masonry, the reinforcement installation commenced. The NSM reinforcement installation is depicted in Figure 3.8.
Figure 3.8 Application of NSM reinforcement, Ducts Embedded Into Footings, (a) Embedment, (b) After Wall Construction, (c) With Placed Stainless Steel Rebars Ready for Epoxing., (d) to (i) depict phases of application of epoxy.
First, the PVC ducts embedded in the footings were carefully filled with epoxy to assure no air voids would be present. A trial insertion of the reinforcement into the ducts was carried out to ensure that a sufficient amount of epoxy was injected.

As soon as overflow of the epoxy was observed the reinforcement was removed and a preliminary layer of epoxy was placed in the wall grooves as shown in Figure 3.8 (g) to provide the rebars with a proper bed of epoxy for proper bond to the masonry and to eliminate any air voids.

Then the reinforcement was carefully placed first into the footing and then pressed into the groove partially filled with epoxy. Subsequently, the reinforcement was covered with another layer of epoxy. Clamps were used on top of the wall to ensure that the reinforcement remained inside the grooves. The surface was smoothened manually using a spatula. The epoxy for the first 2 hours after the application was in a semi workable condition that allowed for any necessary surface smoothing. The temporary clamping of the rebars was removed after seven days of curing.

### 3.5.6 External Sheet Installation

The CFRP and GFRP external sheet installation is depicted in Figure 3.9 and Figure 3.10 respectively.
Figure 3.9 CFRP sheet installation (a) impregnating, (b) treating surface (c) application of fly ash strengthen epoxy (d) Sheet application (e) removing air bubbles (f) final construction
The procedure started with epoxy resin preparation according to the manufacturer’s specifications. First the sheets were impregnated with the resin. The air was pressed out of the sheets with a handheld roller and the epoxy resin was applied directly to the wall. A layer of a mix of epoxy resin with fly ash was then applied on the prepared wall. After the surface of the epoxy resin on the walls seemed to be uniform the sheets were placed on the wall and rolled to remove the excess resin and the air voids from between the wall and the sheet.

To prevent the FRP sheets from delaminating at the bottom of the walls, where the wall surface is offset from the footing surface, a special detail was designed. Masonry mortar was used to build up a transition region at the junction between the wall and footing with a slope of 45 degrees. The external sheets were then anchored with a horizontal GFRP or CFRP sheet, respectively, as per the kind of reinforcement used on the specimen. Figure 3.11 depicts that in detail.
The instrumentation of NSM specimens and externally reinforced ones differed. In addition, the instrumentation of tension and compression reinforcement was different.

3.6.1 Instrumentation of the NSM Reinforcement

Instrumentation of the NSM specimens included: instrumentation of the reinforcement prior to mounting them on the walls, and application of other instrumentation on the surface of the hardened epoxy.

Carbon steel bars, stainless steel bars and GFRP bars were instrumented in the same way, both for the tension and compression side of the walls. The CFRP strip was instrumented at the same locations as the bars but due to its geometry and position in the groove, the
strain gauges were positioned sideways. Location and details of the strain gauge arrangement on NSM reinforcement is shown in Figure 3.12.

![Figure 3.12 Instrumentation of NSM reinforcement](image)

Location of the strain gauges along the length of the reinforcement above the footing was determined in the following way: (a) at the bottom mortar bed of the lowest row of the masonry to monitor the region of the highest moment in the wall, (b) at mid height of the first row of the blocks, and (c) at the top mortar bed of the same row of masonry. Only one strain gauge on the compression reinforced was determined to be sufficient to monitor the strain in the reinforcement at the highest moment location. The location of the strain gauges measured from the top of the wall was respectively: (a) 1200mm (both tension and compression), (b) 1100mm and (c) 1000mm. Electrical resistance strain gauges with a 5mm gauge length and a 120 ohms resistance, manufactured by Sokki Kenkyujo Co. Ltd., Japan of type FLA-6-11-5L were attached using M-Bond 200
adhesive system to measure the strains in the longitudinal direction. As there would be no chance for potential repairs to the reinforcement’s strain gauges, three thick coats of protective isolator were applied to protect the gauge. For additional protection, the strain gauges were encapsulated in silicon. Two displacement-type strain transducers (PI-gauges) with 100mm gauge length were mounted on the bottom of the epoxy filled groove to measure the average longitudinal strains in the surface of the epoxy.

### 3.6.2 Instrumentation of the External Reinforcement

Location of the strain gauges along the length of the reinforcement was determined in a very similar way to the NSM specimens. The target points were the same but due to the horizontal anchor strip at the bottom row of the masonry the strain gauge locations had to be adjusted to accommodate it and to assure that the readings were taken at the most extreme fiber of the main vertical strip. Location and details of the strain gauge arrangement on EB reinforcement are shown in Figure 3.13.

![Figure 3.13 Instrumentation of External Reinforcement](image)

(a) externally applied compression strain gauges, (b) tension side
3.7 Ancillary Tests

In order to determine the mechanical properties of the materials, ancillary tests were carried out as described in the following sections.

3.7.1 Water Retentivity Test

Figure 3.14. depicts the test setup and apparatus details in accordance with ASTM C91-05.
These tests were conducted to determine the usability of the mortar for construction. The test was performed once per every batch of mortar prepared. The goal of this test is to assure that the mortar will have a proper water to solids ratio. Mortar sampled for the test was discarded disregarding its outcome in accordance with the standard.

3.7.2 Flow Table Test

This test’s goal is to measure the workability of the mortar. The determination is made on the increase of the diameter of a 100mm diameter cone of mortar after 25 drops on standard flow table. Flow measurements are a very good indicator of physical characteristics of fresh mortar from the mason’s perspective. Satisfying properties like consistency, spreading ability is crucial for providing a proper bed of mortar for the successive rows of masonry blocks. Figure 3.15. depicts test setup and apparatus as per ASTM C91-05.

![Figure 3.15 Flow test](image)

(a) test cone, (b) after 20 standard drops.
3.7.3 Mortar Compressive Strength

During the construction of the walls 48, 50mm x 50mm x 50mm, mortar cubes were prepared for compression tests of the mortar. Nine standard size cubes of mortar were taken per batch.

3.7.4 Masonry Prism Compressive Strength Tests

Masonry prisms were tested according to CSA 179. Figure 3.16 depicts the setup of these tests.

![Figure 3.16 Masonry prism test setup](image)

3.8 Test Setup and Loading Scheme of Test Walls

The test setup was designed to partially simulate the boundary conditions of a typical masonry wall, which is fixed both at the top and the bottom. However, the fixed-roller condition at the top of the wall is difficult to simulate in laboratory conditions.
As such, it was determined to take advantage of symmetry as shown in Figure 3.17 and to test a ‘cantilever’ wall with a free top rotation, since it corresponds to the zero-moment inflection point in the actual wall. Test set-up details are shown schematically in Figure 3.18.

![Figure 3.17 Loading diagram schematics](image)

![Figure 3.18 Masonry wall test setup schematics](image)
Figure 3.19 depicts an overall view of the test set-up. The specimens were loaded using a 1m wide steel spreader beam attached to a 150 kN capacity hydraulic actuator. The load was recorded using a 111kN load cell with an error of 0.7% full scale or 150N. Displacement or rotation was monitored with longitudinal transducers with an error of 0.01mm. The actuator piston was mounted to a ball joint to accommodate the deflection of the walls during the test. The spreader beam was placed on the walls prior to the test using a thin bed of mortar to ensure equal spread of the load across the width of the wall. The load from the actuator was transferred to the spreader beam through a swivel joint that accommodates the rotation of the wall. Schematics of that connection can be found in Figure 3.20. The center point of the loading beam was located 100mm from the top of the wall so the actual cantilever arm was 1100mm. The concrete footings of the specimens were clamped at the bottom location to the steel reaction frame using heavy duty HSS beams and 40mm threaded steel rods.
The actuator was placed horizontally on a steel support mounted on a reaction frame and clamped to the frame to ensure it did not deviate from its position during the test. A spacer bracket was placed between the wall footing and the reaction frame to accommodate the actuator during clamping.

The load was applied with hand-operated hydraulic pump. The specimens were tested to failure in monotonic loading. The rate of the load application was specimen specific and will be described in the discussion of the test results in Chapter 4.
CHAPTER 4: TEST RESULTS

4.1 General

This chapter presents the test results of the seven, out-of-plane bending tests conducted on medium-scale masonry walls. A detailed discussion of the flexural behavior as well as various failure modes of all the specimens will be undertaken.

The flexural behavior of the CFRP-, GFRP-, stainless steel- and traditional black steel-reinforced specimens are evaluated based on their load-deflection and load-strain responses. The effect of reinforcement material type, reinforcement type (bars, strips and sheets), reinforcement ratio and location with respect to the center of the walls (NSM versus external) is discussed in detail. The chapter begins with a discussion of the performance of the test setup, and the corrections to the deflection measurements that were necessary. The load versus deflection response of each wall is then presented. The observed failure modes are discussed, and the strains measured during the tests are evaluated.

Ancillary test results used to establish mechanical properties of the components of masonry walls are presented in Appendix B

4.2 Performance of the Test Setup

To be able to properly analyze data recorded from each test it was essential to be able to confirm the exact deflection of the wall against the reaction frame for a specific load. The
reaction frame is not perfectly rigid and the footing, even when clamped to the reaction frame and torqued, was still susceptible to rotation. The arising deflections were measured with two LP’s placed at both top corners of the footing. The footing deflection for specimen DBSR is depicted in Figure 4.1. The results show that the footing deflected horizontally under the applied load, and furthermore the footing rotated (the deflection on the right-hand side differs from the deflection on the left-hand side).

![Figure 4.1 DBSR footing deflection behavior](image)

The deflection recorded at the top of the wall is presented in Figure 4.2. Again, a small rotation is evident as the deflection on the right-hand and left-hand sides differed very slightly.
Chapter 4

Test Results

Figure 4.2 DBSR wall deflection,

The left-hand side and right-hand side deflection measurements were averaged to give the estimated deflection along the vertical axis of each wall.

The measured deflections for the top of the wall were corrected by subtracting the average footing deflection, which is magnified at the top due to footing rotation, as illustrated in Figure 4.3. For the subsequent discussion in this chapter all the values of corrected and averaged top of the wall displacement will be used and referred to as “wall deflection”. Figure 4.4 depicts the corrected values of the wall deflection for specimen DBSR.
a – untrue displacement
b – actual deflection from the plane
x – measured deflection
y – measured footing displacement
l – length of correction

Figure 4.3 Geometry of the Test Setup

Figure 4.4 Actual deflection of DBSR wall corrected for footing rotation

FAVG – Footing Average
WAVG – Wall Average
TAVG – Total Average
4.3 Load-Deflection Response

The assumption in the design of each wall was that at the ultimate limit state, yield or rupture of the tensile reinforcement would occur. The “control” specimen was DBSR with NSM black steel reinforcement. The reinforcement was expected to fail between 85 kN and 136 kN.

Compared directly, specimen DGFRPR has reinforcement with a tensile force capacity of 128kN. Specimens SGFRPR and CFRPS were designed to provide 50 percent of the DGFRPR reinforcement and develop half the tensile strength of specimen DGFRPR. Similar theoretical tensile capacities of SGFRPR and CFRPS allow comparison of the performance of two different FRP reinforcement types. The tensile force in specimen DSSR is 78 kN at yield to 117 kN at ultimate, a range overlapping with the control specimen DBSR hence permitting a direct comparison. Specimens ECFRPS and EGFRPS will be compared to CFRPS and SGFRPR, respectively, to quantify the performance of sheets against NSM reinforcement.

Figure 4.5 shows a comparison of the load-lateral deflection behavior of all seven specimens and Figure 4.6 depicts in detail the behavior of all specimens for the first 2400 N of loading.
Figure 4.5 The Comparison of Load vs Deflection of all Wall Specimens

Figure 4.6 Initial part of Load vs Deflection Responses of All Walls
Specimen DBSR exhibited linear behavior up to 2000 N, followed by preliminary debonding of masonry from the mortar at the first and second course and a second linear response, at a reduced stiffness, up to 16000 N, which corresponded to about 10mm of deflection. At this point, yielding of the reinforcement led to a flat plateau until the load began to drop at deflection of about 65 mm. This specimen demonstrated the most ductile behavior after a 2000N deflection of +/- 5mm. All other specimens exhibited larger deflection rates (i.e. were lower in stiffness) as the difference of the stiffness with masonry contribution versus without was greater.

Specimen DSSR exhibited a somewhat similar behavior to specimen DBSR, up to an ultimate load of 18000 N. At this point, the load quickly dropped, and there was no ductile yield plateau observed. This was attributed to the use of a relatively flexible steel plate oriented transversely to apply the load to the wall in this particular test, which was the first test to be carried out. In subsequent tests, this plate was replaced by a stiff beam. It is believed that the flexibility of the plate resulted in non-uniform loading of the wall. The evidence for this non-uniformity was the different failure mode for DSSR compared to DBSR. This will be discussed further in the Failure Modes section.

Specimen DGFRPR also had a linear response after cracking, up to 2000 N, but at a significantly (five times less) reduced stiffness compared to DBSR and DSSR, due to the significantly lower Young’s modulus of GFRP (40.8 GPa) compared to steel (200GPa). The wall reached a comparable but slightly lower load (13000 N) than specimen DBSR. The deflection at ultimate was about 60mm. The behaviour between the cracking point
and ultimate is fairly linear, as expected for FRP reinforcement. At ultimate, the GFRP bars were not ruptured as will be discussed later.

Walls SGFRPR and CFRPS, reinforced with a single GFRP bar and a single CFRP strip at each side, respectively, exhibited remarkably similar responses to each other. Following cracking at 2000 N, these walls had a much lower cracked stiffness than the previous three walls. They also reached much lower ultimate loads than the other specimens (both just over 10000 N), due to the lower reinforcement ratio. Both walls failed by rupture of the tensile reinforcement at maximum deflection (for SGFRPR it was 80mm and for CFRPS was 90mm). It was the researchers' intent to construct samples SGFRPR and CFRPS similarly for direct comparison. The large difference in Young's modulus between CFRP (124.0 GPa) and GFRP (40.8 GPa) was offset by providing the GFRP wall with a higher reinforcement ratio so that the axial stiffness (EA) of the walls was within 12% of each other. This successfully resulted in a similar response for both walls. Also, specimen CFRPS exhibited a similar flat plateau at the ultimate load.

The GFRP sheet was mounted on a triangular mortar buildup along the bottom of the wall of the footing, as was mentioned earlier. This appears to be the reason that specimen EGFRPS exhibited an abrupt 10mm deflection when subjected to loads up to 100N. As the tension plane of the GFRP sheet was moved away from the face of the wall, it pivoted around the bottom of the back of the wall until enough tension was built up in the tension side of the reinforcement to participate in the load resistance. Debonding of the main vertical sheet from the sloped corner of the wall occurred with the initial application of load.
Preliminary debonding of the sheet at the bottom of the wall location occurred at 2500 N. The response was then linear up to the failure at the ultimate load. The sample failed at almost 10000N by tension reinforcement rupture.

Specimen ECFRPS was expected to exhibit the highest ultimate load due to the high strength of the reinforcement material. Preliminary cracking in the masonry occurred at 1000N. As soon as the bond between the rows of masonry was broken (at 2000N) the tension side reinforcement separated from the mortar buildup at the bottom of the first row of the wall and it went into pure tension. The sheet stretched between the top edge of the footing and the bottom of the horizontal bonding strip. At that point the sample exhibited rapid deflection (from 8mm to 40mm) under a small increase in load (increase from 2000N to 3700N) at which load the sample failed by rupture of the tension reinforcement. The premature failure of specimen ECFRPS is discussed in detail in Appendix A.

4.4 Load-Strain Response

For the clarity of the review of the test results, strains in reinforcement on the tension and compression sides of the test specimen were divided into two separate figures. Figure 4.7 presents strains occurring in tension reinforcement and Figure 4.8 gives compression side strains.
Chapter 4  Test Results

Figure 4.7 Load-Strain Comparison for all the Walls in Tension

Figure 4.8 Load-Strain Comparison for all the Walls in Compression
Figures 4.9 and 4.10 present in detail the responses within the first 2400 N of loading of Figures 4.7 and 4.8 respectively.

Figure 4.9 First 2400N of Strain Comparison for all the samples in Tension

Figure 4.10 First 2400N of Strain Comparison for all the samples in Compression
The traditional carbon steel that was used for reinforcement bars (with a nominal yield strength of 425MPa) in the control specimen DBSR has three characteristic stages of behaviour: linear elastic, yield plateau and strain hardening. Up to 2100 microstrain a linear behaviour of the steel is noted after which a yield plateau was developed until 14000 microstrains. At this point failure occurred. As can be noticed, the compression strains corresponding to the failure load of the wall are much lower (maximum of 1000) which indicates that the compression reinforcement did contribute but not as significantly to the capacity of the wall as the steel on the tension side.

The stainless steel 316LN in specimen DSSR showed different behaviour than carbon steel. After a linear elastic stage, the specimen failed at a strain of 7022 microstrain without experiencing a hardening plateau. The compression side reinforcement reached approximately 20% of the tension strain, indicating that it did not contribute to load the resistance greatly.

Unidirectional (FRP) reinforcement has a linear stress-strain response, and so the failure of the FRP is more brittle.

According to manufacturer’s specifications, the GFRP ultimate strain is 15000 microstrain. Specimen SGFRPR failed at 14800 microstrains in tension at the location of the maximum bending. At the ultimate load, the strain in the tension reinforcement of specimen DGRFPR was 15200 microstrain; however, the FRP did not rupture. The compression reinforcement in SGFRPR behaved in a more desirable way than the steel reinforcement in the other walls, reaching 50% of its strain capacity. It is probable that if the reinforcement were embedded into concrete on top, the specimen would perform even
better. Due to the high reinforcement ratio in specimen DGFRPR, the compression reinforcement did not reach 10% of the maximum strain.

The specimen reinforced with CFRP tape failed due to rupture of the FRP after reaching its ultimate strain of 17000 microstrain. The compression side reinforcement reached only 2000 microstrain at failure, so again the compression reinforcement did not contribute significantly to the capacity of the wall.

Specimens with external reinforcement have to be examined in a slightly different manner. Figure 4.7 show that specimens EGFRPS and ECFRPS exhibited small negative (compression) strains on the tension side of the wall. The tension side reinforcement was protected from premature delaminating by a horizontal strip of FRP. It is believed that the protective strip induced local compression in the tension side reinforcement. It was observed that the specimens failed by rupture of the tension side reinforcement. The reinforcement on the compression side of the wall did not reach its ultimate strain.

4.5 Failure Modes

Crushing of the masonry at the compression side of the wall was observed in specimens DGFRPR and DBSR at ultimate, depicted in Figure 4.11 (a). This occurred after yielding of the steel bars in tension in specimen DBSR. In wall DGFRPR, a splitting failure within the masonry blocks occurred parallel to the GFRP bars on the tension side. Note that failure took place within the masonry and not at the interface between the epoxy resin and masonry. This is shown in Figure 4.12, which shows the GFRP bars still encapsulated within a thick layer of masonry. It should also be noted that the GFRP bars did not reach rupture as a result of this masonry failure.
Figure 4.11 Crushing of the Masonry at Ultimate, (a) Specimen DGFRPR, (b) Specimen DBSR

Figure 4.12 DGFRPR Failure Mode – (a) Front View, (b) Side View, (c) Top View

For specimens SGFRPR and CFRPS, the GFRP bar and the CFRP strip, respectively, fractured in tension, as shown in Figure 4.13 (a), (b) and (c). The GFRP rebar failed 200mm above the base. This is at the mortar joint between the first and second rows of masonry. The failure could have occurred higher on the wall (not at the very bottom where the maximum moment occurs) if a larger quantity of epoxy resin accumulated at the bottom making that section overly stiff.
The CFRP strip ruptured at the joint with the footing. Failure of the SGFRPR specimen on the tension side is presented as mentioned above in Figure 4.13. Comparing the two specimens reinforced with GFRP bars, SGFRPR and DGFRPR, it can be noted that the one with a single GFRP bar exhibited a failure typical of an under-reinforced wall, whereas the specimen with two GFRP bars exhibited a failure typical of an over reinforced wall, with the masonry surrounding the tension bars shearing and masonry crushing on the compression side. This provides insight into why doubling the reinforcement ratio led to only a 50% increase in moment capacity.

Specimen DSSR failed prematurely by fracture of the masonry blocks through diagonal planes as a result of the relatively flexible loading plate used to apply the single point load at the mid-width of the wall. It is believed that transverse bending has occurred as a result of this flexibility, which triggered the premature failure of the masonry.
Figure 4.14 depicts masonry debonding of different specimens at failure. Debonding of masonry blocks from the cementious mortar on both vertical and horizontal joints was common as a secondary failure mode.

![Figures 4.14](image)

Figure 4.14 Debonding of Mortar from Masonry, (a) Horizontal Joint (HJ) in DBSR, (b) HJ and Vertical Joint in DGFRPR, (c) HJ in SGFRPR, (d) HJ in DSSR.

First debonding on horizontal joints appeared as early as 3 kN for specimen DSSR, and 5 kN for specimens DBSR, DGFRPR, SGFRPR and CFRPS. More extensive debonding on the horizontal joint between rows of masonry and between the mortar and the footing was observed up to failure for all the specimens. Debonding on vertical joints, Figure 4.15, appeared at higher loads.
Figure 4.15 Shear Failure of the Masonry Around Reinforcement and Side. (a) Comp. Side (CS) DSSR, (b) Ten. Side (TS) DSSR, (c) CS DGFRPR, (d) TS DBSR, (e) Side DSSR, (f) TS DGFRPR.

Specimen EGFRPS experienced complete debonding of the bottom row of masonry from the footing within the first 1000 N and ultimately failed by rupture of the tension reinforcement. The condition of the tension and compression reinforcement after the test is depicted in Figure 4.16.
Specimen ECFRPS failed by rupture of the tension side reinforcement at an unexpectedly low load of 3400N. Both the tension and compression reinforcement are depicted in Figure 4.17. The failure of the tension reinforcement was a predicted failure mode for this specimen; however, there was neither prior cracking in masonry nor debonding of the reinforcement.
Two potential reasons for the premature failure of the ECFRPS specimen were identified. First, defective reinforcement material was a potential reason, but that was very unlikely. That option included assumptions that either the CFRP fibers were defective prior to the application on the wall or that fibers were damaged during the application process. The second option, which was most likely, was the possibility that the failure mode and the low failure load was due to the fact that CFRP sheets were bent around the transition zone between the wall and the footing at about 45 degrees angle, through the triangular mortar filling at the corner. To address this difficulty, another test was performed on the wall as a slab with flat tension reinforcement and no directional change in the fibers. This second test is discussed in detail in Appendix A. The failure load of the second test performed on the specimen proved that the FRP quality (i.e. the first hypothesis) was not the cause of the premature failure during the original test as the specimen failed by complete masonry crushing in the center of the span and no rupture of the tension reinforcement was observed. The second hypothesis was confirmed, which is related to the bending of the FRP sheet at the junction between wall and footing caused the failure.
4.6 Discussion

Table 4.1 summarizes the observed failure modes, measured failure loads, maximum deflection, and reinforcement failure strain for each specimen. The first objective of this research was to compare the flexural performance of various types of corrosion-resistant reinforcing materials for masonry walls. This criterion can be discussed from two different points of view: maximum load or maximum deflection at failure. Specimen DBSR was the control sample and had a failure load of 17kN and maximum deflection of 25mm. Specimen CFRPS had a failure load of 7.9kN (the lowest observed) and a maximum deflection of 97 mm.

<table>
<thead>
<tr>
<th>Specimen ID</th>
<th>Failure Load [kN]</th>
<th>Deflection at Failure [mm]</th>
<th>Strain at Failure</th>
<th>Failure mode</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td>Tension [ms]</td>
<td>Compression [ms]</td>
</tr>
<tr>
<td>DBSR</td>
<td>17.1</td>
<td>25.1</td>
<td>14478</td>
<td>-15325</td>
</tr>
<tr>
<td>DSSR</td>
<td>18.6</td>
<td>74.9</td>
<td>7167</td>
<td>-913</td>
</tr>
<tr>
<td>SGFRPR</td>
<td>10.0</td>
<td>87.4</td>
<td>15281</td>
<td>-5915</td>
</tr>
<tr>
<td>DGFRPR</td>
<td>14.3</td>
<td>64.5</td>
<td>16007</td>
<td>-1345</td>
</tr>
<tr>
<td>CFRPS</td>
<td>7.9</td>
<td>97.4</td>
<td>16063</td>
<td>-1013</td>
</tr>
<tr>
<td>EGFRPS</td>
<td>11.1</td>
<td>92.0</td>
<td>-7502</td>
<td>-3215</td>
</tr>
<tr>
<td>ECFRPS</td>
<td>3.4</td>
<td>41.3</td>
<td>-4274</td>
<td>-2402</td>
</tr>
</tbody>
</table>
Another very important consideration is the efficiency of the reinforcement which can be reviewed in terms of the specimen failure mode. Ideally, the reinforcement will reach its material strength at the ultimate limit state of the wall as improvement in load capacity is one of the major considerations for this research. All the specimens, but one, failed ultimately by rupture or yielding of the tension reinforcement. Specimen DGFRPR failed due to masonry failure in a shear along the interface of the epoxy resin and masonry but the weak link was the masonry as is evident by masonry residue that remained attached to the epoxy resin. Specimens SGFRPR and DGFRPR had failure loads and deflections of 10kN and 87mm versus 14.3kN and 64mm respectively. This indicates that doubling the area of GFRP reinforcement does not lead to a doubling of the capacity of the wall. The GFRP wall strengths can also be compared to the control values of 17kN and 25mm deflection. Note that neither GFRP-reinforced wall reached the control wall strength, although deflection at ultimate was increased. Note also that GFRP reinforcement is more expensive than black steel, and justifying its usage as masonry reinforcement may be justifiable from the standpoint of ease of installation even over traditional steel due to its smaller weight and durability, but could be difficult from an economic perspective.

The second objective of the research was to evaluate the effects of specific reinforcement parameters on the flexural response of the masonry walls subjected to monotonic loading. From the comparison of types of reinforcement (GFRP, CFRP, Stainless Steel, Carbon Steel), two definitely deserve recommendation. CFRP strips provided the specimen with exceptional deflection capacity, four times the DBSR value. Reinforcing material type (bars, sheets, strips) was another way of comparison. CFRP flat strips were excellent from a bond capacity standpoint, due to the high surface area relative to cross sectional
area. CFRP also behaved better in the form of bars rather than sheets. CFRPS failed at 7.9kN under 97mm deflection while ECFRPS exhibited merely 30% of that load (3.4kN) and half of the deflection at failure (41mm). GFRP performed reasonably well as both bars and sheet. It is worth noting that it is believed GFRP sheets were not vulnerable to the weakening effect arising from stress concentrations due to bending at the junction between wall and footing. Unlike CFRP sheets, the lower modulus of the GFRP sheet alleviated this negative effect.

Carbon fibers are more vulnerable than glass fibers to damage induced by bending or kinking because of their higher Young’s Modulus. In this case they could potentially fail at the lower strengths.

The comparison between reinforcement ratios was examined through specimens SGFRPR and DGFRPR. Increasing the reinforcement ratio was not proportionally beneficial to the structural behavior of the wall. SGFRPR failed by rupture of the tension reinforcement in the maximum moment location, after showing signs of preliminary masonry cracking and utilizing up to 45% of capacity of the compression reinforcement. DGFRPR failed under a proportionally comparable load by tension reinforcement being pulled out of the wall. What the specimen exhibited was a bond failure governed by strength of masonry not adhesive. The masonry remains on the epoxy resin. It is possible that the bond would be better if the reinforcement was embedded on the top as it would act most likely in more direct bending and not developing shear near interface of the adhesive (in the masonry). Potentially it was happening due to the inherent weakness of the masonry block, close to the bond line.
Specimens may also be compared from the standpoint of influence of location of the reinforcement (internal versus NSM versus external), as per one of the objectives. Although internal reinforcement (i.e., conventional construction using grouted rebar inside the blocks) was not experimentally tested in this study, it is quite logical to assume that specimen DBSR with NSM black steel reinforcement would have behaved significantly better than one with similar but internal reinforcement. This is because the DBSR wall developed its full potential strength without debonding and at the same time the rebar is located further away from the neutral axis of the wall, hence maximizing its effectiveness. Also, it appears that NSM technology is more effective for masonry application than external bonding, particularly if externally bonded CFRP sheets were used. NSM CFRP strips on the other hand showed superior behavior with excellent bonding, allowing them to reach rupture. In an NSM system, particularly for flat strips, the potential resistance surface is maximized.

Investigating potential changes in standard masonry blocks to accommodate the NSM reinforcement for new construction (not retrofitting applications) was a very important objective of this study. It is envisioned that the masonry industry can very easily develop new grooved blocks through the molding process of the block. This will make construction even easier as grooves will not need to be cut into individual blocks before construction as was done in this study.

To determine which type of corrosion-resistant reinforcement is the most efficient for this particular application was the final objective of this research. As the specimens were compared to DBSR which ductility proven to be overall superior to other specimen, three
specimens exhibited superior results in partial areas of focus of this research. CFRPS specimen with NSM single CFRP flat strip failed under the largest deflection of 97mm at 7.9kN ultimate load in flexure and not by debonding. Specimen SGFRPR with NSM single GFRP bar failed at 10kN and exhibited a deflection of 87mm, also failing in flexure and not due to debonding. The third specimen was DSSR with double NSM stainless steel bars, which reached a load slightly larger than DBSR (by about 10%), but at a deflection of only about 23 mm at the failure load of 18.6kN. Had this specimen been loaded with a stiff spreader beam like other specimens, it is believed that it would have reached a higher ductility.

Full conclusions drawn from this research program and recommendations for further steps are presented in Chapter 5 of this thesis.
5.1 Summary

The flexural performance of masonry walls with corrosion-resistant NSM or externally bonded reinforcement has been studied through experimental investigations. A total of seven medium-scale specimens were tested in cantilever bending. Five specimens were reinforced with GFRP rebar, CFRP strip, stainless steel bars and conventional ‘black’ steel in grooves at the face of the walls. The remaining two specimens were reinforced with externally applied CFRP and GFRP sheets. For the control specimen, a specimen with NSM conventional carbon steel was selected.

The tests were designed to simulate out-of-plane behavior of a masonry wall subjected to lateral forces acting on the structure, such as during an earthquake or wind load. In a seismic event, walls oriented such that their strong axis are normal to the direction of ground acceleration are subjected to out-of-plane loading, in terms of a lateral drift at floor levels. The test setup used for this research took advantage of the fact that a zero moment exists at half the distance between floors. As such, a cantilever setup was used. The effects of reinforcement type, amount, and method of installation on structural performance were studied.
5.2 Conclusions

The following conclusions can be drawn from this experimental investigation:

1. Construction of new masonry walls with NSM (NSM) reinforcement is feasible and practical. It has the advantage of spreading the reinforcement away from the neutral axis, and hence increases its effectiveness, as well as avoiding grouting that is necessary with internal reinforcement. It requires a new masonry block with a pre-molded groove which can easily be developed by the industry.

2. NSM steel reinforcement demonstrated a superior performance to all other types of reinforcement, with excellent ductility, strength and stiffness. A plastic hinge was formed at the joint with the footing. Given the fact that steel bars were completely encapsulated by epoxy resin, hence protected from corrosion, and are relatively lower in cost, it could be argued that in this particular application, steel bars were superior to other types of non corrosive reinforcement.

3. Increasing the reinforcement ratio of NSM FRP bars does not necessarily lead to a proportional increase in the moment capacity of the wall, due to a change in failure mode. In walls that are heavily reinforced by FRP bars, longitudinal splitting of the masonry may occur in the vicinity of the tension reinforcement, contributing to a bond failure.

4. The walls reinforced by NSM CFRP single strip and GFRP single bar reached their full potential strength by rupture of the reinforcement. Flat CFRP strips appear to have exceptional bond strengths based on the fact that their surface area is maximized.
5. Externally bonded CFRP sheets had a premature failure at a reduced load at the junction between the wall and footing, due to the change in direction of the sheet, leading to bending of the sheet at about 45 degrees. This effect on strength was not observed in GFRP sheets with similar conditions, but the wall stiffness with the GFRP sheets was lower than those with NSM GFRP reinforcement.

6. NSM reinforcement can be embedded into vertical ducts pre-placed in the footing, and aligned with the grooves in the blocks. The ducts are then filled with epoxy resin (or grout), a technique that was successfully used in this study. This enabled excellent anchorage of the reinforcement into the footing. Alternatively, another technique could have been used but was not tested in this study. Short dowels embedded in the footing during casting and sticking out could be accommodated in the masonry groves and lap spliced with other NSM reinforcement. This technique was considered beyond the scope of this study.

5.3 Recommendations for Future Work

5.3.1 Potential Applications

1. Seismic activity zones masonry walls oriented such that their strong axis is normal to the direction of ground acceleration can be reinforced with NSM or external FRP reinforcement. This applies to both new and retrofit or upgrade of existing structures.

2. Low rise industrial buildings often have load bearing masonry walls. The average height is 6m to 8m. Those walls can be erected hollow and reinforced later using
either of the technologies discussed here. This will not only speed up the process but could reduce the cost.

3. Walls give support to Open Web Steel Joists (O.W.S.J.) at the top and are connected to the concrete footings. Dead loading from the steel structure in combination with wind loading can cause the occurrence of a very similar loading conditions as was applied the specimen’s in this research.

4. Masonry walls are conventionally reinforced by internal steel reinforcing bars, placed approximately every 1.2m and grouted. There is a great opportunity for further research in this specific area of masonry construction as the possibility of mixing vertical and horizontal reinforcement using both the NSM and EB reinforcing technique. This could lead to potentially larger spans.

5. The steel reinforcement is heavy and requires proper equipment (crane) to elevate it to its location. That increases the cost of construction. The potential savings on the cost of construction is considerable with FRP reinforcement.

5.3.2 Innovation to Practice:

1. This study investigated the structural performance of reinforced masonry walls with corrosion-resistant reinforcing materials applied near the surface or externally on the walls, rather than internally. Additionally, these techniques are proposed for new construction, not just retrofitting applications. All of those components are opening the door for masonry research to explore further site
specific applications in potentially corrosive environments or in applications
where even the slightest decrease of the weight of the structure counts.

2. The promising performance of Carbon Fibre-Reinforced Polymer (CFRP) strips,
   Glass Fibre-Reinforced Polymer (GFRP) rods and Stainless Steel reinforcing bars
   as NSM reinforcement for masonry walls and the performance of GFRP sheets as
   external reinforcement for masonry walls was proven to be successful.
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Appendix A. SLAB TEST

A.1 Introduction

The ultimate load for the ECFRPS test was much lower than expected. This unexpected premature failure had to be further researched. The cause of it had to be determined to be able to establish the usefulness of the test result for the comparison study. The sample was subjected to additional testing. This appendix describes this testing in detail the investigation in the following order: procedure of establishing the investigative sequence, determination of testing setup, instrumentation and testing procedure. Finally test results are presented, analyzed and the conclusions for this separate investigation are drawn. The additional study is concluded with an explanation of the relevance of the findings from the slab test to the original testing specimen group and their results.

A.2 Original Test Summary

The behavior and the failure load of ECFRPS specimen is described at the end of Chapter 4 and is depicted in Figure 4.17, Figure A.1 and A.2 respectively. Additional data can be found as well in Appendix B where all the data from the testing phase is recorded. The results for the EGFRPS specimens are depicted in Figure 4.16.
Figure A. 1 Forces acting on CFRP in original test

Figure A. 2 Failure Mode of the ECFRPS Specimen During Original Testing
A.3 Investigation of Premature Failure

Two potential reasons for the premature failure of the ECFRPS specimen were identified. Defective reinforcement material was a potential option. That option included assumptions that either the CFRP fibers were defective prior to the application on the wall or that fibers were damaged during the application process. The second option was the possibility that the failure mode and the low failure load was due to the fact that the CFRP sheets were subjected to a load not parallel with the fiber orientation of the FRP but on an angle that resulted in a lower failure load due to its high Young’s Modulus. The second assumption summary is depicted in Figure A.1. The analysis of the EGFRPS specimen was the only source of comparison used to determine the cause.

Possibility number one for obvious reasons would cause the results from both tests to be unusable for research purposes. The second possibility, if it was found to be the correct one, would validate the original test’s results and would give a very strong argument in the discussion against the use of CFRP sheets in determining most promising corrosion-resistant reinforcement for the out-of-plane applications for masonry walls.

A.4 Testing Procedure Considerations

Before determining the testing procedure all aspects of retesting the previously tested sample were considered. It was determined that the retesting procedure would have to take into consideration:

1. The unreinforced masonry part of the specimen is very fragile and can be easily damaged during any attempt to move it,
2. As the specimen underwent the original testing procedure during which it was subjected to out-of-plane load which at approximately 2kN to 3kN for other specimens, resulted in early stages of debonding between the rows of masonry and the mortar, hence the quality of the mortar beds will be visually investigated and monitored throughout the moving and testing procedures,

3. As this research’s main goal was to determine the most promising corrosion-resistant reinforcement type none of the samples were multiplied, each was unique so no comparison testing could have been conducted,

4. As the failure mode for the original testing was the rupture of the reinforcement at the bottom location, subjecting the sample to the same testing scheme was impossible as the wall was detached from the footing,

5. The new test setup will have to give an opportunity to confirm or exclude option one and two at the same time,

6. All the samples are being tested to failure in one continuous stroke; therefore the original testing scheme will have to be maintained for reference.

A.5 Slab Test Setup

Taking into consideration all points mentioned in the previous paragraph it was determined that the ECFRPS specimen would be retested as a slab. The details of the test set-up are shown schematically in Figure A.3.
Figure A. 3 Test setup schematics
Appendix A

The view of the test setup and testing area is presented in Figure A.4. The specimen were tested in four-point bending using a closed-loop testing machine with an actuator capacity of 150 kN. The specimen was loaded using two concentrated loads, 200 mm apart. The specimen were simply supported on one end with a small I beam and the roller condition was simulated with a steel round HSS hollow section inside a C channel to maintain its position.

Figure A. 4 Testing setup and testing area
Load was applied through a steel spreader beam that consisted of a 30mm thick steel plate (300 x 300mm) with two steel rollers welded to its bottom at exactly 200mm apart. The spacing of the roller assured the load application was as close as possible to the center of the slab but at the same time the very center of the specimen was a mortar bed joint that could not be loaded directly as it is the weakest location on the slab. The rollers spanned 1m (the width of the slab) to assure equal load distribution. The 150mm stroke actuator was jacked against the frame above the specimen and rested on the load cell that was placed directly on the steel plate.

Load was applied with a hand-held pneumatic pump. The rate of the load application was determined by the slab’s behavior. The specimen was tested to failure in one monotonic loading scheme.

A.6 Test Instrumentation

Instrumentation of the slab, detailed locations of all strain gauges and displacement transducers is presented in Figure A.5. All of the strain gauges were allocated in the area of the highest bending moment on the specimen. As the original test ended prematurely previously installed strain gauges along the length of the bottom of the reinforcement were tested for conductivity to see if they survived the original test. Although the strain gauges were still able to perform correctly due to a changed testing setup their location was not desired and they were not used.
For this test no strain gauge on the compression side was installed. The location of the strain gauges was respectively from the center of the slab out: at the very centre 100mm and 200mm each way away along the fiber orientation of the FRP. Electrical resistance strain gauges with a 5 mm gauge length and a 120 ohms resistance, manufactured by Sokki Kenkyujo Co. Ltd., Japan of type FLA-6-11-5L were attached using M-Bond 200 adhesive system to measure the strains in the axial direction. Surface preparation of the CFRP was the same as in Section 3.6.2.
Longitudinal transducers of 150mm stroke monitored the response of the wall to the loading in six different locations. Two pairs were monitoring the potential rotation and negative deflection on four corners of the slab and the two remaining ones were monitoring the deflection in the center of the wall on both sides of it.

A.7 Test Results

In this section, the results recorded during the retesting procedure are presented. The flexural behavior and then failure mode is described and analyzed.

A.8 Flexural Behaviour

As this test had a test setup different to all the other tests it is difficult to discuss it in reference to the other samples. This study’s intent was to closely analyze differences and similarities in behavior between samples ECFRPS and EGFRPS. The ECFRPS sample underwent original test with no signs of arching on the length of the wall. The specimen was subjected to very detailed visual inspection of the mortar bed quality that would be the best indicator if the specimen sustained damage beyond usefulness. The sample reached the deflection in the center span of 4mm at early stages of the test and maintained it until sudden failure at the end. The flexural response of the slab is presented on a comparative graph in Figure A.6 and the strain recorded in the most extreme fiber of the tension reinforcement is depicted in Figure A.7.
Potential slippage of the LPs

Figure A. 6 Load vs. Deflection response

Figure A. 7 Load vs. Strain characteristics
A.9 Failure Mode

In a sequence as the test progressed the sample failure occurred in the following way: the sample demonstrated signs of partial debonding of the FRP at around 15 to 20kN at the deflections ranging between 3 to 5mm in the center span. That occurred only at the location of the mortar beds and can be accredited to crushing of the mortar. Ultimately the sample failed by crushing of the masonry blocks at the ultimate load of 54KN in various locations along the span of the slab. The CFRP did not debond from the surface of the blocks on the tension or the compression sides. Failure mode and details of the failure mode are presented in Figure A.8 and Figure A.9

A.10 Conclusions and Recommendations for Future Work

The results of the retesting of the ECFRPS specimen led to the following conclusions:

1. The failure load and the failure mode indicated that the FRP quality most likely was not the cause of the premature failure during the original test,

2. The suspected option two was correct and confirmed that application of the load on the angle to the fibers can result in a very brittle behavior of the sample,

3. It was possible as well that the fibers got damaged during installation by being bent.
Figure A. 8 Failure mode, Sample failure near the fixed end support.

Figure A. 9 Detailed Failure
Appendix B.

ADDITIONAL RESEARCH DATA

B.1 Scope

The first part of this appendix deals with the series of ancillary tests described in Chapter 3 that were conducted in preparation for and during the mainstream testing phase. As the results of these tests were not directly linked to the discussion of conclusions drawn from the research they are presented in this appendix to give the reader valuable insight regarding the discussion found in the previous chapters.

In Chapter 4 results of all the seven specimen tests were presented in a comparative discussion. Details of processing of the test results were presented based on the DBSR control specimen. The second part of this appendix presents specimen specific data for remaining 6 walls not described in Chapter 4.

B.2 Ancillary Test Results

In this section the results of ancillary tests are presented in detail. Supplementary tests were conducted to verify if the material characteristics of the components of the masonry walls meet standard requirements for masonry construction and to determine the material characteristics of the system. Due to the importance of the sequence of the conducted ancillary test, they will be discussed in two groups: tests conducted prior to the
construction of the main specimen, tests conducted during the construction of the walls and post construction tests.

**Pre-construction Test**

**Sieving Test** - Before the sand could have been subjected to sieving analysis it had to be properly dried. Aggregate was specified in terms of minimum and maximum aggregate size and its volume in the pile as well. The structure of the apparatus for the sieving analysis is presented in Figure B.1. To maintain constant quality every batch of sand was sampled three times to minimize the error.

Each batch of the sand that was dried, with the purpose of using it for the mortar mix, was subjected to a sieving test to determine if the particular batch met standard specifications for aggregate sizing. Results are recorded in Table B.1

![Figure B. 1 Sieving Apparatus](image-url)
Table B. 1 Sample Sieving recordings

<table>
<thead>
<tr>
<th>DATE</th>
<th>20/03/2007</th>
</tr>
</thead>
<tbody>
<tr>
<td>MATERIAL</td>
<td>SAND FOR MORTAR BATCH 1</td>
</tr>
<tr>
<td>SIEVE ON SIEVE</td>
<td>RETAINED</td>
</tr>
<tr>
<td>25mm 0.00</td>
<td>0.00</td>
</tr>
<tr>
<td>20mm 0.00</td>
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</tr>
<tr>
<td>5mm 0.00</td>
<td>0.00</td>
</tr>
<tr>
<td>2.0mm 3.00</td>
<td>0.20</td>
</tr>
<tr>
<td>1.168mm 27.00</td>
<td>1.77</td>
</tr>
<tr>
<td>600mu 174.00</td>
<td>11.41</td>
</tr>
<tr>
<td>295mu 622.00</td>
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<tr>
<td>150mu 492.00</td>
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<tr>
<td>75mu 156.00</td>
<td>10.23</td>
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<td>Pan 51.00</td>
<td>3.34</td>
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<tr>
<td>1525 100</td>
<td>100.00</td>
</tr>
<tr>
<td>CSA A179-04</td>
<td>PERCENT PASSING</td>
</tr>
</tbody>
</table>

Compressive Strength of Concrete - The behavior of the reinforced concrete foundation of the walls was not in the scope of the research but nevertheless its even minor failure could have resulted in invalidation of the data recorded from that specific test and consequential loss of a valuable specimen.

Due to high clamping force applied at the top of the reinforced concrete base it was critical to confirm the compressive strength of the foundation concrete. Prior to
construction of the specimen, concrete cylinder samples were tested for each of the foundation blocks. Recorded strengths from those tests can be found in Table B.2.

<table>
<thead>
<tr>
<th>Sample #</th>
<th>Footing Strength in MPa @ 28 days</th>
<th>@ test day</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>61</td>
<td>84</td>
</tr>
<tr>
<td>2</td>
<td>64</td>
<td>80</td>
</tr>
<tr>
<td>3</td>
<td>68</td>
<td>86</td>
</tr>
<tr>
<td>4</td>
<td>65</td>
<td>88</td>
</tr>
<tr>
<td>5</td>
<td>67</td>
<td>83</td>
</tr>
<tr>
<td>6</td>
<td>65</td>
<td>85</td>
</tr>
</tbody>
</table>

**Construction Phase Tests**

**Water Retentivity and Flow Table Test** - One of the standard tests; conducted to assure proper mortar mix properties that directly correlate to providing proper bond strength between masonry blocks. Due to the nature of the test and its frequency during the construction results were just to determine the usability and there was no need to record them.

**Post Construction Tests**

These tests were conducted on the days of the specimen testing.

**Mortar Compressive Strength** - Although the samples were collected at the time of the construction and per batch of the mortar that represented the specific wall and specific small scale prism the tests were conducted later. From each batch 6 standard cubes of
mortar were collected: 3 to be tested at the 28 day strength and 3 to be tested at the day of the corresponding wall / prism test. Results are recorded in Table B.3.

<table>
<thead>
<tr>
<th>Sample</th>
<th>Mortar Avg. Strength in MPa @ days</th>
<th>@ test day</th>
</tr>
</thead>
<tbody>
<tr>
<td>Batch 1 Specimen</td>
<td>6.0</td>
<td>12.0</td>
</tr>
<tr>
<td>Batch 2 Specimen</td>
<td>6.5</td>
<td>11.8</td>
</tr>
<tr>
<td>Batch 3 Specimen</td>
<td>6.4</td>
<td>12.2</td>
</tr>
<tr>
<td>Batch 4 Specimen</td>
<td>6.1</td>
<td>11.7</td>
</tr>
<tr>
<td>Batch 5 Specimen</td>
<td>5.9</td>
<td>11.5</td>
</tr>
<tr>
<td>Batch 6 Specimen</td>
<td>6.6</td>
<td>12.8</td>
</tr>
<tr>
<td>Batch 7 Specimen</td>
<td>6.4</td>
<td>12.6</td>
</tr>
</tbody>
</table>

**Masonry Prism Tests** - Tests to obtain $f_m$ of the masonry walls. Table B.4 presents values of force at failure and resultant values of $f_m$.

**B.3 Detailed Masonry Walls Test Results**

The load deflection curves were all corrected for footing deflection as described in Chapter 4 at the beginning of the discussion of the flexural behavior of all the samples. Here presented are details of the specimen specific behavior in the following order:

1. DSSR
2. SGFRP
3. DGFRP
4. CFRPS
5. EGFRPS
6. ECFRPS
B.4 DSSR

Figure B. 2 DSSR Specimen Footing Deflection

Figure B. 3 DSSR Specimen Recorded Wall Deflection
Figure B. 4 Actual DSSR Out-of Plane Deflection

B.5 SGFRPR

Figure B. 5 Footing Deflection SGFRPR
Figure B. 6 Wall Deflection SGFRP

Figure B. 7 Actual deflection of the wall without error
B.6 DGFRPR

Figure B. 8 Footing Deflection DGFRP

Figure B. 9 Wall Deflection DGFRP
Figure B. 10 Actual deflection of the wall without error

B.7 CFRPS

Figure B. 11 Footing Deflection CFRPS
Figure B. 12 Wall Deflection CFRPS

Figure B. 13 Actual deflection of the wall without error CFRPS
B.8 ECFRPS

Figure B. 14 Footing Deflection ECFRPS

Figure B. 15 Wall Deflection ECFRPS
B.9 EGFRPS

Figure B. 16 Actual deflection of the wall without error ECFRPS

Figure B. 17 Footing Deflection ECFRPS
Figure B. 18 Wall deflection EGFRPS

Figure B. 19 Actual deflection of the wall without error EGFRPS
Figure B. 20 Strains in DBSR

Figure B. 21 Strains in DSSB
Figure B. 22 Strains in DGFRP

Figure B. 23 Strains in SGFRP
Figure B. 24 Strains in CFRPS

Figure B. 25 Strains in EGFRPS
Figure B. 26 Strains in ECFRP