OUT-OF PLANE STRENGTHENING OF UNREINFORCED MASONRY WALLS USING NEAR SURFACE MOUNTED FIBRE REINFORCED POLYMER STRIPS

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ABSTRACT

The development of cost effective minimally-invasive seismic retrofit techniques is required for clay brick unreinforced masonry (URM) buildings because of their recognised poor seismic performance. A laboratory-based experimental study with well defined but artificial boundary conditions, which utilises constituent construction materials that replicate the material properties of masonry found in historic URM buildings, is currently addressing this need. The purpose of this study is to investigate the performance of near-surface mounted (NSM) carbon fibre reinforced polymer (CFRP) strips as a seismic retrofit solution for out-of-plane lateral loading of the walls in URM buildings. In addition, five retrofitted URM walls located in four different buildings were tested in-situ by applying out-of-plane loading, to complement the laboratory-based study. Testing confirmed that the NSM CFRP retrofit technique is an excellent minimally-invasive and cost effective option for seismic strengthening of URM buildings. Provisional details of the design methodology for the NSM CFRP retrofit technique, and laboratory and in-situ test results are reported. Two recent projects that implemented the NSM CFRP technique are also briefly presented.

1.0 INTRODUCTION

One of the most critical deficiencies of clay brick unreinforced masonry (URM) buildings is their out-of-plane seismic response (Griffith et al. 2003), and to mitigate the risk that these structures pose, various retrofit solutions have been developed. Using fibre reinforced polymer (FRP) material to retrofit URM walls is an established technique for strengthening and increasing the ductility capacity of URM walls subjected to in-plane and out-of-plane earthquake loading. Externally bonded (EB) FRP sheets or plates and, more recently, near-surface mounted (NSM) FRP bars or strips, are the two application techniques that are commonly used (Korany and Drysdale 2006; Mosallam 2007). Using the NSM technique provides several advantages over EB reinforcement which include: significantly higher axial strain at debonding, protection from fire and the environment, minimal impact upon the aesthetics of the structure (Goodwin et al. 2009), and reduced construction time (Seracino et al. 2007; Petersen et al. 2009), thus providing a cost effective and minimally-invasive option for seismically strengthening URM buildings.

A sound understanding of the bond behaviour between concrete and CFRP has been achieved following extensive research (Stone et al. 2002; Hassan and Rizkalla 2003), and reliable analytical models have been established (Seracino et al., 2007). Initial experimental validation of NSM CFRP as a technique for seismic improvement of URM buildings involved laboratory testing that focused on strengthening single leaf modern masonry walls, and based on these experimental results

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an analytical model that predicts the masonry to CFRP bond behaviour was developed by Willis et al. (2009b).

In-order to validate the suitability of the analytical model developed by Willis et al. (2009b) for application to multi-leaf New Zealand historic masonry, a companion test program to that reported by Willis et al. (2009a) was established to conduct full scale laboratory-based tests on as-built walls, and walls strengthened using the NSM CFRP technique.

In-situ testing of real buildings is required to further validate the efficacy of the NSM CFRP technique for improving the seismic performance of historic URM buildings. A number of opportunities emerged during the laboratory experimental study to also conduct in-situ tests on URM buildings and building components for subject buildings scheduled to undergo major alteration, seismic strengthening or partial demolition. In total five walls from four different URM buildings were tested in-situ, after being strengthened with NSM FRP. In addition, a number of tests were performed on site and in the laboratory on extracted wall samples to establish accurate material properties of each building.

The NSM technique that was used involved insertion of a CFRP strip having a Modulus of Elasticity of 165 GPa and a mean tensile strength of 3100 MPa (properties provided by the manufacturer). A 15 mm wide × 1.2 mm thick strip was positioned in vertical groove that was cut in the wall so that it extended over the entire height of the wall. A two-part epoxy was used to bond the CFRP strip into the masonry substrate. To ensure maximum bond area the groove was entirely filled with epoxy prior to insertion of the CFRP strip (installation procedure is illustrated in Figure 1).

2.0 PROVISIONAL DESIGN METHODOLOGY

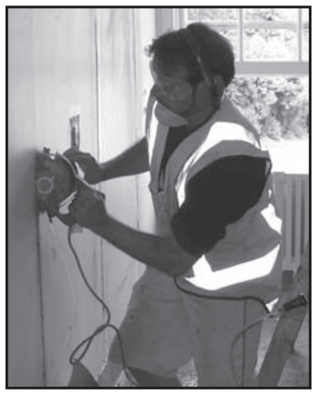
2.1 Prediction of IC Debonding Resistance

Predictions of the flexural strength of NSM CFRP retrofitted URM walls tested in this experimental study were based on an existing generic analytical model that was initially developed by Seracino et al. (2007) for concrete and later modified by Willis et al. (2009b) for use with modern clay brick masonry. The axial force in the CFRP strip required to cause the onset of intermediate crack (IC) debonding in clay masonry, P_{IC}, is given by Equation (1) (Willis et al. 2009b). The accuracy of this model has been verified against 29 pull tests (Yang 2006) and full scale laboratory-built wall tests (Willis et al. 2009a). In Equation (1), L_{per} is the perimeter of the debonding failure plane (units of mm, from experimental observations t_d and t_p are taken as 1 mm), φ_f is the failure plane aspect ratio, $(EA)_p$ is the axial stiffness of the CFRP strip, b_p and t_p are the width and thickness of the CFRP strip, respectively and f_{ut} is the brick rupture strength. The retrofit parameters are shown in Figure 2.

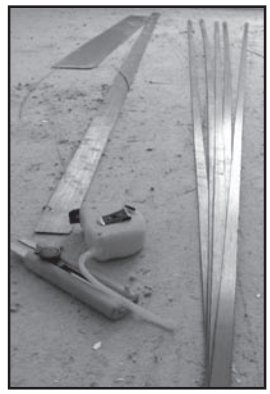
Table 1 indicates the key material properties used for design of the vertical FRP reinforcement in the unreinforced masonry walls when subjected to out-of-plane bending, for the most commonly used CFRP.

Table 1. Typical values of material properties for CFRP retrofitted URM

Parameter	Carbon FRP
Modulus of elasticity of CFRP, E_p	165 x 10^3 MPa
Tensile strength of CFRP, f_{up}	2700 MPa
Rupture stress of the brick unit, f_{ut}	3.5 MPa
Modulus of elasticity of masonry, E_m	3500 MPa
Specific weight of masonry, γ	18 kN/m^3



(a) Groove cutting



(b) 15 mm wide CFRP strips

(c) Epoxy application

(d) Groove pointing

Figure 1. NSM CFRP retrofit application

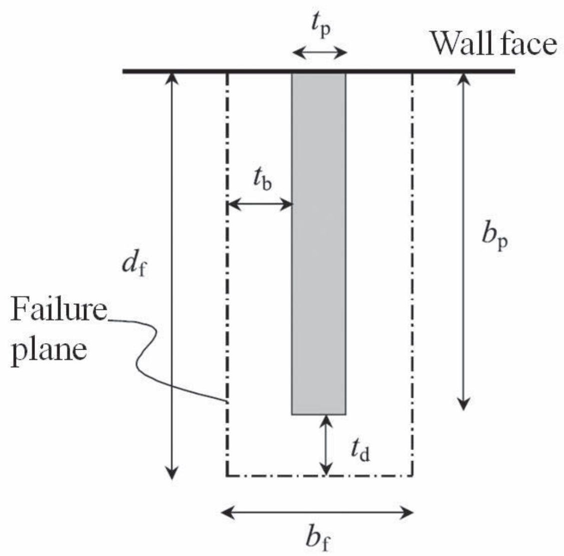


Figure 2. Retrofit parameters ($t_b \approx t_d \approx 1$ mm)

2.2 Design for Vertical Bending

The design process for an unreinforced masonry wall strengthened with vertical CFRP strips is based on the aim that the preferred wall behaviour will be governed by IC debonding of the CFRP strip, rather than the development of alternative more brittle failure modes, such as CFRP rupture, horizontal bending failure of masonry between vertical CFRP strips, or masonry crushing. The methodology presented is generic for either FRP material (Carbon CFRP or Glass GFRP) and therefore final selection of the FRP material is at the discretion of the design engineer. In this experimental study only CFRP material was used due to its high rupture strength and stiffness, and thus GFRP materials are not referred to. The critical design parameters for the CFRP retrofitting scheme include: (i) material (i.e. the modulus of elasticity of the CFRP, E_p); (ii) cross-sectional dimensions (i.e. the width, b_p, and thickness, t_p, giving a cross-sectional area, A_p); and, (iii) horizontal spacing between vertical CFRP strips (i.e. wall design strip width, *s*). The CFRP cross-section and the spacing between strips determine the reinforcement ratio.

The usual flexural theory design assumptions for the cross-sectional analysis of a CFRP reinforced section (as illustrated in Figure 3) are:

(1) plane sections remains plane after bending;

(2) full composite action exists between the CFRP strip and the masonry interface (i.e. no slip or opening-up at the interface); and,

(3) the tensile resistance of the masonry is neglected (i.e. the CFRP does not contribute to strength until the masonry has cracked).

The design process for earthquake (or wind) loading depends on the horizontal acceleration (or wind pressure) that the wall is subject to in the out-of-plane direction. For a given uniformly distributed load, w_d, the required flexural strength (i.e. vertical moment demand, M_d) for a wall spanning vertically between simple supports at its top and bottom edges is given by

$$M_{\mathrm{d}} = \frac{w_{\mathrm{d}} \cdot h^2}{8} \tag{2}$$

which can be written in terms of the design inertial acceleration (demand), a_d, as

$$M_{\mathrm{d}} = \frac{a_d \cdot t_{\mathrm{m}} \cdot s \cdot \gamma \cdot h^2}{8} \tag{3}$$

where

a_d= demand acceleration (units of 'g', acceleration normalised by the acceleration of gravity;

h = clear span of the URM wall in the vertical direction (i.e. height of the URM wall);

s = horizontal spacing between the vertical CFRP strips;

t_m = thickness of the URM wall; and

γ = specific weight of the masonry.

$$P_{\mathrm{IC}} = 0.988\varphi_{\mathrm{f}}^{0.263}\left(\frac{f_{\mathrm{ut}}}{0.53}\right)^{0.6}\sqrt{L_{\mathrm{per}}(EA)_{\mathrm{p}}} = 1.45\varphi_{\mathrm{f}}^{0.263} f_{\mathrm{ut}}^{0.6}\sqrt{L_{\mathrm{per}}(EA)_{\mathrm{p}}} \tag{1}$$

Where $L_{\mathrm{per}} = 2d_f + b_f \cong 4 + 2b_p + t_p$ and $\varphi_{\mathrm{f}} = \dfrac{d_f}{b_f} \cong \dfrac{1+b_p}{2+t_p}$

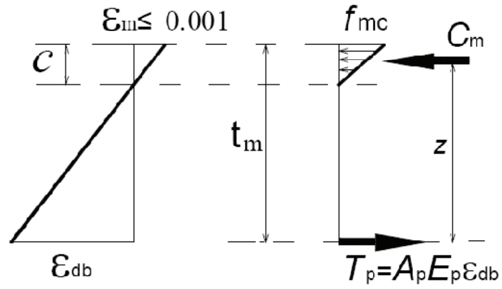


Figure 3. Stress and strain profile (Willis et al. 2007)

The steps in the NSM CFRP retrofit technique are then as detailed below:

Step 1: Select the horizontal spacing, *s*, of the vertical CFRP reinforcement so that the strengthened URM wall does not fail in horizontal bending between the strips due to the inertial load caused by a_d. To do this, calculate the horizontal bending capacity of the URM wall, M_{ch}, (e.g. using AS 3700 to determine M_{ch} for a unit length of wall in the vertical direction in units of kNm/m) and then choose a strip spacing, s, such that

$$\frac{8M_{\text{ch}}}{s^2} > w_{\text{d}} = a_d \cdot t_{\text{m}} \cdot \gamma \tag{4}$$

Equation (4) can be rearranged so that the spacing *'s'* is given by:

$$s < \sqrt{\frac{8M_{\text{ch}}}{a_d \cdot t_{\text{m}} \cdot \gamma}} \tag{5}$$

Step 2: Assume a strip cross-section for the CFRP (i.e. choose b_{p} and t_{p}) and then calculate the strip IC debonding capacity, P_{IC}, given by Equation (1). Check that P_{IC} is less than the tensile rupture capacity of the CFRP strip, P_{rupture} ($f_{\text{up}}\ A_{\text{p}}$). If not, adjust the CFRP cross-section, usually by increasing the thickness, t_{p} and start over at Step 2.

Step 3: Solve for the neutral axis location, *c,* using Equation (6) to satisfy axial force equilibrium (where C_{m} is the masonry compressive force, T_{p} is the CFRP tensile force and ε_{m} is the masonry strain) and Equation (7) corresponding to plane sections remaining plane (where ε_{db} is the CFRP debonding strain).

$$P_{\text{IC}} = T_{\text{p}} = C_{\text{m}} \quad \text{gives} \quad c = \frac{P_{\text{IC}}}{\frac{1}{2} \cdot \varepsilon_{\text{m}} \cdot E_{\text{m}} \cdot s} \tag{6}$$

$$\varepsilon_{\text{m}} = \frac{c}{\left(z - \frac{2}{3}c\right)} \varepsilon_{\text{db}} \tag{7}$$

where the lever arm, $z = t_{\text{m}} - \frac{b_p}{2} - \frac{c}{3}$

For simplicity, it is assumed for the NSM strip that the level of embedment is small compared to t_p. Therefore the neutral axis location, c, may be approximated by substituting Equation (7) into (6) to give:

$$c = \frac{-\alpha + \sqrt{\alpha^2 + 4 \cdot \alpha \cdot t_{\text{m}}}}{2} \tag{8}$$

$$\left(\text{use } z \cong t_{\text{m}} - \frac{c}{3} \text{ to give } \varepsilon_{\text{m}} = \frac{c}{t_{\text{m}} - c} \varepsilon_{\text{db}}\right)$$

$$\text{where } \alpha = \frac{2 \cdot P_{\text{IC}}}{\varepsilon_{\text{db}} \cdot E_{\text{m}} \cdot s} \text{ and } \varepsilon_{\text{db}} = \frac{P_{\text{IC}}}{(EA)_{\text{p}}}$$

Step 4: Check that the masonry compressive stress is less than the masonry strength capacity, f'_{mc}, using Equation (7), ($f'_{\text{mc}} = \varepsilon_{\text{m}} \cdot E_{\text{m}}$). If not, return to Step 2 and decrease the CFRP cross-section. Otherwise, continue to Step 5.

Step 5: Calculate the vertical bending capacity of the CFRP reinforced section, M_{cv}, using Equation (9) to check if the strip capacity is greater than the demand. If not, then return to Step 2 and increase the CFRP cross-section, and/or decrease the CFRP spacing and restart the procedure. If OK, then the design is complete.

$$M_{\text{cv}} = P_{\text{IC}} \cdot z > M_{\text{d}} \tag{9}$$

2.3 Other Considerations

Vertically oriented NSM CFRP strips can either be placed through the brick units or the perpend joints. Due to aesthetics and ease of placement it is likely that in most applications the strip would be run through the perpend joints. However, it should be noted that positioning vertical NSM FRP strips through the perpend joints can cause a reduction in bond strength in the order of 10% (Yang et al. 2006). However, this level of strength reduction may be deemed insignificant given the beneficial effects of the relative ease of placement (i.e. cutting through half the amount of brick units) and the reduced aesthetic impact.

3.0 EXPERIMENTAL VALIDATION

3.1 Laboratory-based Studies

As an extension to the experimental studies conducted by Willis et al (2007; 2009a) and Yang (2006) that investigated out-of-plane loading of single leaf full scale walls constructed from modern masonry, an experimental study is currently being executed (4 out of 6 walls already tested) using historic New Zealand clay brick masonry and considering multi-leaf walls. The currently available results are presented in this section. The setup consisted of an assemblage of a steel framing and plywood backing frame, and utilized airbag(s) to apply uniform face pressure. Lateral load was typically measured using four 10 kN load cells.

The typical test setup that was used is illustrated in Figure 4. The wall specimens were typically tested in the as-built condition and then retrofitted using the NSM CFRP retrofit technique.

The upper portion of Table 2 conveys the wall geometries and a summary of tests that were performed (walls R1 to R4). All walls were tested using pseudo-static loading. Walls R1 and R2 were tested in one direction only, whereas walls R3 and R4 were tested using reversed cyclic loading. Wall R4 had an overburden of 0.1 MPa acting on the top of the wall panel, which is equivalent to the weight of a second storey. All walls were retrofitted using a 15 mm × 1.2 mm CFRP strip inserted into a 25 mm × 8 (typically) mm groove filled with epoxy.

3.2 In-situ Testing

Two out-of-plane wall tests were conducted in the Allen's Trade Complex, one being in the as-built condition and the second in the seismically retrofitted condition. The building is located in Gisborne and was originally constructed in 1911. Full building and test details are reported in Dizhur et al. (2010c).

Three out-of-plane wall tests were conducted in the 1917 Wintec Block F (located in Hamilton), two tests being in the as-built condition and one test being in the retrofitted condition. A non-load bearing, 2-leaf clay brick masonry partition wall was selected for testing. Following completion of the two as-built tests the wall segment was retrofitted using the NSM technique with vertical CFRP strips embedded into the plaster layer only, and the test was repeated (Dizhur et al. 2010a).

(a) Two vertical CFRP strip retrofit (wall R1)

(b) Single CFRP strip retrofit (wall R2)

(c) Typical test setup for cyclic out-of-plane testing (wall R3)

(d) Failure at maximum loading (wall R4)

Figure 4. Experimental setup

Table 2. Test Program

Building	Wall	Boundary conditions		Nominal thickness t_n (mm)	Wall height h (mm)	Wall length l (mm)	Axial Load (kPa)	Plaster thickness t_p(mm)	
		Top	Bottom					Loaded face	Non-loaded face
-	R1	SS	SS	330	4100	1150	0	N/A	N/A
	R2			330	3000	1150			
	R3			230	4100	1150			
	R4			230	4100	1150	100		
Allen's Trade Complex	AT	M	TDM	240	3000	1200	55[c]	10[a]	N/A
Wintec Block F	WB	TD	TDM	250	4000	1250	0	15[a]	15[a]
Avon House	AH1	TD	TDM	150	3300	1170		20[b]	20[b]
	AH2	SG	TDM	270	2700	1170		20[b]	10[a]
William Weir House	WH	Free	CFD	130	2730	3480		15[a]	15[a]

SS: Simply Supported
M: Continuous masonry wall
TDM: Continuous masonry wall and timber diaphragm
TD: Timber diaphragm
SG: Simply supported with 25 mm gap to allow vertical movement
CFD: Concrete floor diaphragm
N/A – data not available
[a]: Cement based plaster
[b]: Lime based plaster
[c]: Axial load estimated as a column of masonry above the tested wall segement

Seven out-of-plane wall tests and three in-plane wall tests were conducted at the 1884 Avon House (located in Wellington) (Derakhshan et al. 2010b; Dizhur et al. 2010b), with two tests being on walls that were retrofitted using the NSM CFRP technique. One wall was a single leaf non-load bearing internal partition wall with a weak 20 mm thick lime-sand plaster layer on both sides. The second wall was scheduled to be partially demolished up to a height of 2700 mm above the floor level, to allow the building to remain watertight. Thus, the gravity load from the above remaining brickwork was temporarily supported by an assemblage of timber lintels, and the wall section was then isolated from the main wall by gradual removal of masonry until a clear gap of 25 mm was formed. In order to provide support to the top of the wall, additional lateral support was assembled.

Out-of-plane testing of two masonry partition walls on the top storey (level 3) of the 1932 William Weir House (located in Wellington) was conducted. One partition wall was tested in the as-built condition only (Derakhshan et al. 2010a), and the second wall was initially tested in the as-built condition (WH-A) and then retrofitted with NSM CFRP strips (WH-R). The retrofitted partition wall measured 2730 mm high × 3480 mm wide and was located on the third level of the building, where the airbag loaded area covered a significantly large portion of the wall surface (70%).

External views of the four buildings that were tested in-situ are shown in Figure 5.

The lower portion of Table 2 conveys the summary of tests that were performed on each building, with a two-part code used to identify the buildings and the walls that were tested. The first part of the code identifies the name of the building where the test was conducted, for example "AH" for Avon House. As the wall specimens were first tested in the as-built condition and then retested in the retrofitted condition, the second part of the code is used to identify the wall condition with reference to the tests. Thus "A" denotes testing the wall in the as-built condition and "R" denotes testing the wall in the retrofitted (repaired) condition.

In order to simplify the tests and to provide direct comparison to laboratory-based companion experiments, test walls AT, WB, AH1 and AH2 were first isolated from the full wall length within the building, to induce one-way flexural bending. Isolation of the test wall segments was achived by cutting vertically through the wall using a concrete cutting chainsaw, with the vertical cuts made on an inward angle to eliminate wedging effects of the walls (typical test layout is illustrated in Figure 6). The locations of the wall segments, where possible, were selected adjacent to door openings to minimise the length of cutting required. Wall WH was not modified and was tested in two-way flexural bending.

(a) Allen's Trade Complex, Gisborne **(b) Wintec Block F, Hamilton**

(c) Avon House, Wellington **(d) William Weir House, Wellington**

Figure 5. Buildings tested in-situ

All walls were first tested in the as-built condition and then seismically retrofitted with NSM CFRP strips and retested. Following the as-built tests the damage in the walls was limited to a horizontal crack that typically occurred at approximately 60% of the wall height. The top and bottom boundary conditions, and geometries for each wall, are summarised in Table 2.

(a) Typical test setup (Wintec Block F)

(b) Typical test setup for one-way bending (Avon House)

(c) Typical instrumentation setup (Allen's Trade Complex)

Figure 6. In-situ test setup

Parameters for NSM retrofit are illustrated in Table 2, with the values for each parameter summarised in Table 3 for each wall test reported here. For all walls, with the exclusion of WH, a single strip was positioned in the middle of the wall segment, whereas for wall WH two strips were used, having a spacing of approximately 1200 mm.

3.3 Material Properties

Material properties for each building test, and for the laboratory-built walls, were determined by conducting in-situ and laboratory tests on extracted samples from URM walls.

Individual bricks extracted from buildings for laboratory testing were subjected to the half brick compression (f'_{bc}) test according to ASTM C 67 – 03a (ASTM 2003b).

Table 3. CFRP details for NSM test walls

Wall	No. of wall faces retrofitted	d_f (mm)	b_f (mm)	b_p (mm)	t_p (mm)
R1	1[a]	20	8	15	1.2
R2	1	20	8		
R3	2	20	8		
R4	2	20	6		
AT	1	25*	6		
AH1	1	35*	8		
AH2	1	25*	6		
WH	1	30*	5		
WB	1[b]	20	6		

*- depth including plaster layer
[a]- CFRP strips did not continie full height of wall
[b]- CFRP strip embedded into cement plaster substrate only

Table 4. Material properties

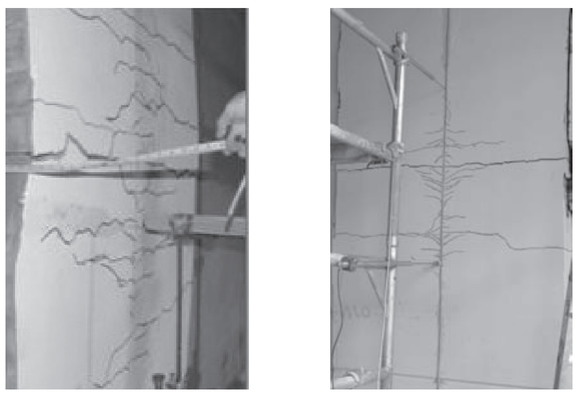
Building/Wall	f'_p (MPa)	f'_{bc} (MPa)	f'_{mc} (MPa)	f'_c (MPa)	f'_{fb} (MPa)	f'_{ut} (MPa)
R1	-	39.5 (6)	2.10 (8)	18.30 (4)	0.15 (5)	3.84 (7)
R2	-	39.5 (6)	8.80 (8)	24.50 (4)	0.17 (5)	3.84 (7)
R3	-	35.8 (7)	3.38 (16)	23.16 (8)	0.15 (7)	3.84 (10)
R4	-	35.8 (7)	3.38 (16)	23.16 (8)	0.15 (7)	3.84 (10)
Allen's Trade Complex	N/A	18.3 (5)	5.7 (9)	9.4 (5)	N/A	1.68 (5)
Wintec Block F	15.1 (7)	34.5 (7)	17.7 (6)	9.7 (5)	N/A	2.06*
Avon House	1.4 (9)	10.1 (7)	3.3 (8)	3.3 (6)	0.04 (6)	1.19 (5)
William Weir House	3.4 (7)	N/A	N/A	13.8 (4)	0.80 (3)	N/A

N/A – data not available
(#) No. of samples tested
* -rupture strength of plaster based on McGregor (1988) transformation to obtain tensile strength from compression strength.

The compressive strength of the mortar was obtained through compression testing of irregular mortar samples in the laboratory. The compression test recommended in ASTM C 109 – 08 (ASTM 2008) was followed for laboratory-built walls, but could not be used for in-situ tested walls as 50 mm cube samples are typically unattainable from actual buildings, where mortar joints are only 15 to 20 mm thick. Instead, irregular samples were cut into reasonable cubic shapes and tested in compression (f'_{mc}). Also, irregular plaster samples were extracted and tested in compression (f'_p). The masonry prism compressive strength (f'_c) was determined following ASTM C 1314-03b (ASTM 2003a), and the flexural bond strength (f'_{fb}) was determined on site following ASTM 1072 – 00a (ASTM 2000). The rupture strength (f'_{ut}) was determined following ASTM C 67 – 03a (ASTM 2003b). Results from the aforementioned tests are summarised in Table 4.

3.2.1 Experimental Results

The retrofitted walls were tested using the same setup and boundary conditions as used for as-built tests. The pressure in the airbag(s) was increased until visible cracking within the vicinity of the CFRP strip occurred, with a typical crack pattern that resulted being shown in Figure 7. All walls were loaded and unloaded several times to examine if repeated loading resulted in stiffness degradation. The total lateral load – mid-height displacement response for laboratory-built and in-situ tested walls are shown in Figure 8. The effective wall face pressure was calculated as the total measured lateral force sensed by the load cells, divided by the wall total area. All walls exhibited ductile behavior with the exception of AH2, where a sudden debonding of the CFRP strip occurred (Dizhur et al. 2010b).



(a) Test AT-R, crack pattern

(b) Moderate cracking during test WB-R

Figure 7. Typical crack pattern

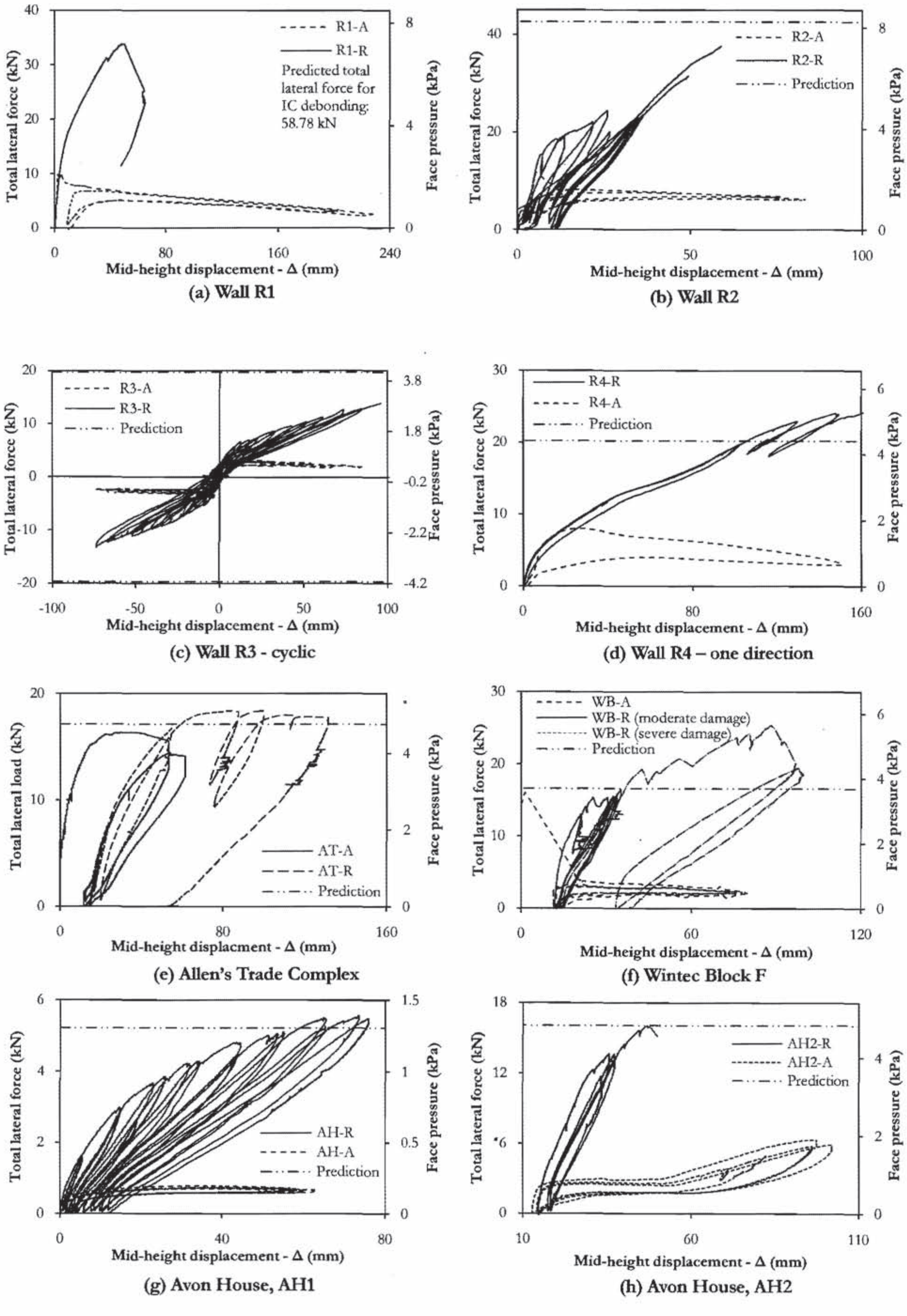
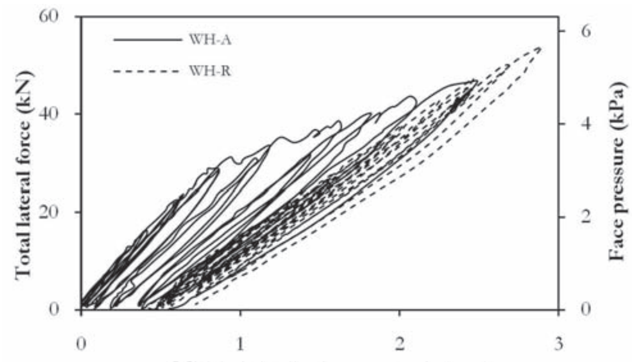


Figure 8. Experimental results (continued over page)



Mid-height displacement - Δ mm

(j) William Weir House

Figure 8. Experimental results

Table 5. Summary of test results

Wall	Post cracking as-built face pressure (kPa)	Retrofitted maximum achieved face pressure (kPa)	Retrofit strength increase (%)
R1	2.10	7.18	240
R2	3.43	17.12	400
R3	0.68	3.03	345
R4	1.74	5.30	205
AT	3.98	5.15	30
WB	0.87	5.51	535
AH1	0.18	1.38	670
AH2	1.08	4.80	345
WH	4.93	5.67*	N/A

*- effectiveness of the retrofit is not evident

3.3 Prediction Using Analytical Model

The prediction of flexural strength of NSM CFRP retrofitted walls tested in this experimental study was based on the existing generic analytical model presented in Section 2.

Using material properties from Table 1 and retrofit details from Table 3, the aforementioned equations were used to predict flexural strength of the walls tested in the laboratory and in-situ, and the results are summarised in Table 6. The values of M_{Exp}/M_{IC} ranged between 0.73 and 1.50 (Table 5), with a mean value of 1.10, indicating that the predictions of moment capacity were typically conservative. This was also observed by Willis et al. (2009a), where a mean value of 1.32 was obtained.

3.4 Discussion of Results

A single CFRP NSM strip substantially increased the post-cracking out-of-plane flexural strength of walls WB, AH1 and AH2 by 535%, 670% and 345%, respectively, as shown in Figure 8 (a to j). Due to arching action and additional axial load on the wall segment the flexural strength increase for wall AT was not substantial, being only 30%. Also, the ductility for walls AT, WB and AH1 remarkably increased following the seismic retrofit. Due to relatively small displacements achieved during test WH-R the effectiveness of the retrofit was not conclusive, although a slight increase in flexural wall stiffness is evident from Figure 8(j).

Only the CFRP strip in wall segment AH1 was instrumented with strain gauge transducers attached directly to the strip. The maximum strain obtained in the CFRP strip during test AH1-R was 7500 με (analogous to a tensile stress of 1240 MPa), compared to the CFRP manufacturer's suggested maximum design tensile strength of 3100 MPa. Thus, debonding of the CFRP strip is evident and the stresses that developed in the strip were well below that required for strip rupture.

The presented design methodology is provisional and further research is scheduled to finalise the design guidelines for this type of retrofit. However, both the laboratory-based and in-situ components of the experimental study have illustrated that the NSM CFRP retrofit technique provides a simple and cost effective alternative to seismically strengthen URM buildings and their components. Furthermore, the technique has minimal impact on the aesthetics of the retrofitted building.

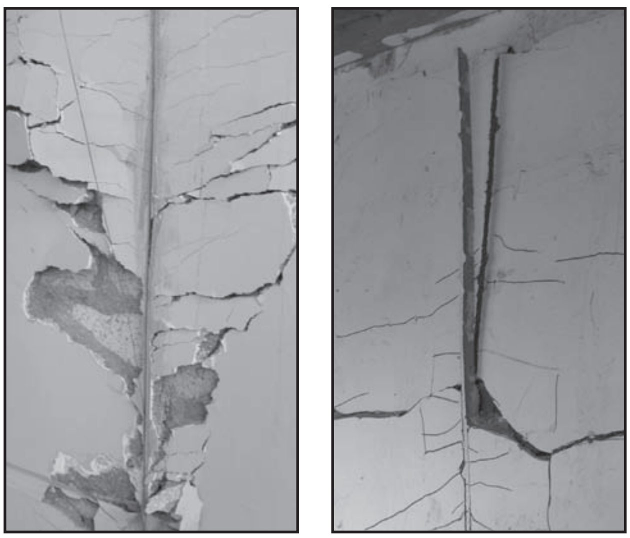
3.4.1 Failure Modes

The test walls exhibited three different modes of failure: gradual debonding of the CFRP strip from the substrate, a sudden CFRP strip pull-out, and shear failure of masonry. Wall segments R2, R3, R4, AT, WB and AH1 exhibited CFRP debonding failure (see Figure 9(a)), whereas wall segment AH2 failed when the top portion of the CFRP strip suddenly pulled-out (see Figure 9(b)). Wall R1 failed when the bottom portion of the wall segment failed in shear. Due to relatively small displacement of wall WH the failure mechanism was not observed.

Table 6. Analytical predictions

Wall	Experimental moment M_{Exp} (kNm)	P_{IC} (kN)	Predicted total lateral force (kN)	Predicted moment M_{IC} (kNm)	$\frac{M_{Exp}}{M_{IC}}$
R1	17.36*	101.23	58.78	30.12	0.58*
R2	14.13**	50.61	40.37	15.14	0.93
R3	7.32	50.61	19.66	10.07	0.73
R4	12.13	50.61	20.20	10.07	1.20
AT	7.13	30.82	17.15	6.43	1.11
WB	12.5	34.83	16.62	8.31	1.50
AH1	2.29	25.02	5.22	2.17	1.05
AH2	5.23	25.02	16.07	5.42	0.97
				Average:	1.10

Note: prediction for wall WH was not calculated due to absence of material data
*- wall failed in shear near the base thus. This result is excluded from calculation of average M_{Exp}/M_{IC} ratio
** - Due to a limited capacity of the setup, the failure of the wall was not achieved



(a) Debonding (b) Sudden pull-out

Figure 9. Wall failure mechanisms

4.0 RECENT IMPLEMENTATIONS

4.1 Rob Roy Hotel (Birdcage)

The 1886 Birdcage Tavern (formerly known as the Rob Roy Hotel) is one of Auckland's iconic buildings. As shown in Figure 10(a), the Birdcage Tavern is a 2 storey registered heritage masonry building which is one of a few New Zealand hotels built in the 1880s that still exists today. Located at 133 Franklin Road at the south edge of Victoria Park, the heritage hotel is in the way of the Victoria Park Tunnel (VPT) project. Consequently, the tavern is scheduled for relocation as part of the New Zealand Transport Agency's (NZTA) Victoria Park Tunnel motorway extension project, which is intended to relieve one of New Zealand's worst motorway links by constructing a multi-lane northbound tunnel under Victoria Park (New Zealand Herald 2007 and Matthews & Matthews Architects 2003).

The building is being relocated 30 meters up the road to allow the new VPT to be constructed. Following completion of the VPT project, the building is planned to be shifted back to its original location. The relocation required the building to be assessed and strengthened before the relocation project commenced. To achieve effective seismic/relocation strengthening, and to provide a cost effective and minimally intrusive solution, the building's four URM chimneys were strengthened using vertically oriented NSM CFRP strips. The retrofit involved cutting vertical grooves from two sides of the chimneys and inserting 1.4 mm × 40 mm CFRP strips into a groove filled with epoxy (see Figures 10(b) and (c)). The building is placed on reinforced concrete runway beams, just below the ground level, and will be pushed at a slow rate with hydraulic arms along the beams up Franklin Road. The the same method will be used to return the hotel to its new permanent location when the Victoria Park Tunnel is complete in 2012.

(a) Birdcage Tavern - Front view

(b) Rear chimneys retrofitted using NSM CFP

(c) Chimneys retrofitted using NSM CFP

Figure 10. Birdcage Tavern (Rob Roy Hotel)-NSM CFRP retrofit

4.2 Campbell Free Kindergarten

The former Campbell Free Kindergarten (CFK) is a registered heritage building located on the southern side of Victoria Park in central Auckland. The building has significant heritage value as it is the earliest purpose built free kindergarten in New Zealand, and is the first institution of the Auckland Kindergarten Association, which remains active to this day. The original building consists of URM, and is double storeyed with an adjacent single storey block that was constructed in 1910 (Jones 2004; Goodwin 2008). Currently the building is undergoing considerable refurbishment and seismic strengthening following over two decades of disuse, which left the building in a seriously deteriorated condition and in danger of collapse (see Figure 11).

In order to provide structural integrity and out-of-plane seismic resistance, parts of the URM walls of the Campbell Free Kindergarten will be strengthened with horizontally aligned NSM CFRP strips. Due to implementation of the unobtrusive NSM retrofit technique, the strengthened walls will retain their heritage characteristics.

Figure 11. 1910 Campbell Free Kindergarten

Parapets are usually the first component to fail in an earthquake, as their positioning at the top of the building and their lack of a securing mechanism makes them susceptible to failing out-of-plane. Thus, the parapets of the former CFK are scheduled to be strengthened using vertically positioned NSM CFRP strips to ensure that the parapets are suitably secured.

5.0 CONCLUSIONS

In-situ testing is of great assistance to understand the seismic response of heritage URM buildings, and the data acquired from such tests is essential to validate laboratory-based studies.

A single CFRP NSM strip was shown to remarkably increase both the post-cracking wall flexural strength (between 30% and 670% improvement) and the ductility of the URM walls.

The test results identified two basic modes of failure: debonding of the reinforcing CFRP material from the substrate was the most common type of failure, with only one failure by CFRP strip pull-out. None of the tests resulted in rupture of the CFRP strip.

The maximum strain that developed in the CFRP strip during test AH1-R was 7500 $\mu\varepsilon$ (analogous to a tensile stress of 1240 MPa), which was well below the manufacturer's recommended limit.

Predictions of flexural strength of NSM CFRP retrofitted wall sections using a currently available laboratory-based analytical generic IC debonding model are in good agreement and suitably conservative when compared to the measured flexural strength of the walls tested in-situ in real buildings.

Although, the presented design methodology is provisional and further research is planned to finalise the design guidelines for this type of retrofit, the laboratory-based and in-situ experimental study has illustrated that the NSM CFRP retrofit technique provides a simple and cost effective alternative to seismically strengthen URM buildings and their components.

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