Earthquake Protection of Masonry Shear Walls Using Fibre Reinforced Polymer Strengthening

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A thesis submitted in partial fulfilment of the requirements for the degree of
Doctor of Philosophy

School of Engineering
The University of Newcastle
Australia
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I hereby certify that the work embodied in this thesis contains published papers of which I am a joint author. Those publications are listed under the List of Publications. I am solely responsible for the research presented in this joint publication, under the supervision of Dr Mark Masia and E/Prof Adrian Page.'

(Signed):______________________________
K M C Konthesingha

(Signed):______________________________
Mark Masia (Supervisor)
“Many are ready to do what’s easy. Only the one with outstanding courage dares the hard work. If you need to stand out, you must work hard”

-My father's message whenever I was down
Acknowledgements

It is with pleasure that I acknowledge the support and contributions of the following wonderful people.

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Abstract

Unreinforced masonry (URM) buildings are highly vulnerable to damage during earthquakes, due to their high mass, limited ductility and low tensile strength. However, being economical, durable, easy to procure and good for thermal and sound insulation ensures that URM is widely used both for low-rise structural walls and for infill to framed structures. In addition to that, many of the existing historically and culturally important buildings have been identified as URM constructions. Therefore, strengthening of URM buildings to resist earthquake damage has a remarkable importance.

URM shear wall strengthening with near surface mounted (NSM) fibre reinforced polymer (FRP) strips is a relatively new and effective seismic retrofitting technique to improve their earthquake resistance. This technique involves inserting thin FRP strips into grooves cut into the surface of the wall. The aesthetic impact to the structure is minimal due to this strengthening technique compared with attaching FRP reinforcement to the surface of the wall (External Bonding (EB) technique). The other advantages of NSM FRP are the ability to develop higher strains in the FRP before debonding compared to EB techniques, and protection from vandalism, to some extent from fire and other environmental influences.

In this research study an extensive experimental study along with numerical analyses were carried out to investigate the cyclic in-plane shear behaviour of unreinforced masonry (URM) walls retrofitted/strengthened with near surface mounted (NSM) fibre reinforced polymer (FRP) strips. Carbon FRP (CFRP) strips were used in this technique and were designed to enhance the performance of URM walls which fail by diagonal cracking or bed joint sliding within the height of the wall.

The bond-slip behaviour between NSM FRP strips and clay brick masonry was investigated using six experimental pull tests under cyclic loading. The results including bond strengths, critical bond length and the local bond-slip behaviour were determined and were compared with a similar monotonically loaded pull
test results. The bond-slip curves for monotonic and cyclic loading cases were approximately similar.

Two major experimental investigations were carried out in this project to investigate the effectiveness of retrofitting/strengthening of URM walls panels with NSM CFRP strips using previously damaged and newly constructed undamaged wall panels.

The effectiveness of NSM CFRP strip retrofitting applied to damaged URM walls was investigated using sixteen previously damaged wall panels with two different damage levels (lightly and highly) subjected to vertical pre-compression combined with increasing reversing cycles of in-plane lateral displacement. The damaged walls were partially repaired, retrofitted with NSM FRP strips and retested. The study assessed the effect on strength, displacement capacity, energy dissipation and ductility achieved by FRP retrofitting compared to the undamaged URM panels under different pre-compression levels. The retrofitted walls displayed higher displacement capacities compared with URM walls. The ultimate loads were not enhanced due to retrofitting under higher pre-compression levels. However the presence of the reinforcement restored the ultimate loads to those observed for the original undamaged URM state. The improvements in the behaviour of the URM walls due to retrofitting were generally similar irrespective of the amount of pre-existing damage in the URM walls.

A new test setup representing realistic boundary conditions to simulate the earthquake behaviour of shear walls in actual buildings was designed and built for the series of experiments with newly constructed wall panels. It was designed to impose zero in-plane rotation (fixed-fixed) boundary conditions at the top and bottom of the masonry wall specimens. A representative finite element (FE) model was used to obtain the actual dimensions of the test setup. The design parameters for the experimental series, including test specimen dimensions and pre-compression loads to achieve diagonal cracking failure modes, were obtained using the same FE model. A total of twenty three wall
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panels constructed with two wall aspect ratios (height : length = 1 and 0.5) were tested. They were strengthened with NSM CFRP strips in six different reinforcement arrangements including vertical, horizontal and a combination of both. Four panels were tested under monotonically increasing in-plane lateral displacement and the others under increasing reversing cycles of in-plane lateral displacement combined with a vertical pre-compression. The expected zero in-plane rotation (fixed-fixed) boundary conditions were achieved from the new setup with classic diagonal failure occurring through the test walls. The displacement capacity, energy dissipation and ductility of the wall panels were enhanced due to the NSM FRP strengthening. The maximum load of the strengthened walls was increased compared to URM when the strengthening contained vertical FRP strips. The reinforcing scheme which used a combination of vertical and horizontal FRP strips performed the best.

A finite element model was developed to validate the experimental results. The micro-modelling approach was used in this masonry model. The FRP strips were attached to the masonry model using the bond-slip relationship established from the experimental pull tests. The key behaviours of the experimental test results could be reproduced by the developed FE model.
List of Publications


C Konthesingha, M Masia, R Petersen, and A W Page. Cyclic in-plane shear behaviour of unreinforced masonry wall panels strengthened with NSM FRP strips. *15th International Brick and Block Masonry Conference*, Florianópolis, Brazil, June 2012.


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<thead>
<tr>
<th>Symbol</th>
<th>Description</th>
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<tbody>
<tr>
<td>$A$</td>
<td>Bedded area of masonry prism</td>
</tr>
<tr>
<td>$A_d$</td>
<td>Cross sectional area of the bond wrench test specimen</td>
</tr>
<tr>
<td>$A_f$</td>
<td>Cross sectional area of the FRP bar</td>
</tr>
<tr>
<td>$A_{fw}$</td>
<td>Area of FRP shear strengthening</td>
</tr>
<tr>
<td>$A_{FRP}$, $A_i$</td>
<td>Cross sectional area of the FRP reinforcement</td>
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<tr>
<td>$A_{w}$, $A_n$</td>
<td>Cross sectional area of the masonry wall</td>
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<td>$b$</td>
<td>Height of masonry specimen</td>
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<td>$b$</td>
<td>Shear stress distribution factor</td>
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<td>$b_f$</td>
<td>Total width of the fabric crossing the diagonal cracks</td>
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<td>$b_0$</td>
<td>Width of CFRP</td>
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<td>$b_p$</td>
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</tr>
<tr>
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<td>Cohesion</td>
</tr>
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<td>Experimentally determined coefficient</td>
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<td>$C_S$</td>
<td>Shear traction control factor</td>
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<td>$d$</td>
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<td>$d$</td>
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<td>$d$</td>
<td>Length of masonry unit</td>
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<tr>
<td>$d_1$</td>
<td>Distance from the inside edge of the tension gripping block to the centre of gravity</td>
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<td>$d_2$</td>
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<td>$d_b$</td>
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<td>$d_y$</td>
<td>Actual depth of masonry in the direction of shear considered</td>
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<td>$d_{max}$</td>
<td>Maximum horizontal displacement</td>
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<td>$E$</td>
<td>Modulus of elasticity of CFRP</td>
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<td>$E_d$</td>
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<td>Stiffness of FRP in direction of the tensile diagonal</td>
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<td>$E_{st}$</td>
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<td>$E_{FRP}$, $E_f$</td>
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<td>$E_{mor}$</td>
<td>Elastic modulus of mortar</td>
</tr>
<tr>
<td>$E_{mas}$</td>
<td>Elastic modulus of masonry</td>
</tr>
<tr>
<td>$E_{unit}$</td>
<td>Elastic modulus of brick unit</td>
</tr>
<tr>
<td>$F_r$</td>
<td>Lateral resistance of retrofitted specimens</td>
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### List of Symbols

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<thead>
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<th>Symbol</th>
<th>Description</th>
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<tr>
<td>$F_{sp}$</td>
<td>Total compressive force on the bedded area of the tested joint</td>
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<td>Tensile strength of masonry</td>
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<td>$f_u$</td>
<td>Maximum tensile strength of NSM FRP bar</td>
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<td>$f_{d1}$</td>
<td>Design strength of FRP reinforcement</td>
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<td>$f_{te}$</td>
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<td>$f_{sa}$</td>
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<td>$f_{mtm}$</td>
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<td>$f_{vk}\ell$</td>
<td>Limiting value of $f_{vk}$ and depends on the type of masonry units and mortar strength</td>
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<td>$f_m'$</td>
<td>Compressive strength of masonry</td>
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<td>Tensile strength of the FRP</td>
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<td>$G$</td>
<td>Elastic shear modulus</td>
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<td>$G_C$</td>
<td>Compressive fracture energy</td>
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<td>$G_{unit}$</td>
<td>Shear moduli of brick unit</td>
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<td>$G_{mor}$</td>
<td>Shear moduli of mortar</td>
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<td>Tensile fracture energy</td>
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<td>$G_f^H$</td>
<td>Shear fracture energy</td>
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<tr>
<td>$H$</td>
<td>Depth of the wall parallel to the direction of applied shear force</td>
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<td>$H_{max}$</td>
<td>Maximum horizontal loads</td>
</tr>
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<td>$h_{mor}$</td>
<td>Mortar joint thickness</td>
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<tr>
<td>$l_{beam}$</td>
<td>Second moment of area of steel beam</td>
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<tr>
<td>$K_{el}$</td>
<td>Initial stiffness</td>
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<td>$K_{FRP}$</td>
<td>Constant for mechanism of failure (0.5)</td>
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<tr>
<td>$k_s$</td>
<td>Shear elastic stiffness</td>
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<tr>
<td>$L$</td>
<td>Length of the wall</td>
</tr>
<tr>
<td>$L_e$</td>
<td>Effective length of FRP bar</td>
</tr>
<tr>
<td>Symbol</td>
<td>Description</td>
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<td>--------</td>
<td>-------------</td>
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<tr>
<td>$L_i$</td>
<td>Effective bond length of the $i$-th bar intersecting the diagonal crack</td>
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<td>$L_0$</td>
<td>Length of CFRP</td>
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<td>$L_t$</td>
<td>Sum of bond lengths of all the rods crossed by the crack, calculated in the most unfavourable crack position</td>
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<td>$L_{mas}$</td>
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<td>$L_{mor}$</td>
<td>Length of the mortar joint</td>
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<td>$L_{per}$</td>
<td>Bonded perimeter of FRP</td>
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<td>Length of the brick unit</td>
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<td>Unbonded length</td>
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<td>Length of the wall</td>
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<td>Bending moment about the centroid of the bedded area of the test joint at failure</td>
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<td>$m_1$</td>
<td>Mass of the wrench</td>
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<tr>
<td>$m_2$</td>
<td>Mass applied to the end of the wrench lever</td>
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<tr>
<td>$m_3$</td>
<td>Mass of the unit</td>
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<td>$N_{Rd}$</td>
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<td>$N_{FRP}$</td>
<td>Additional compression force</td>
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<tr>
<td>$n$</td>
<td>Number of FRP sheets</td>
</tr>
<tr>
<td>$n$</td>
<td>Number of plies of FRP laminates</td>
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<tr>
<td>$n$</td>
<td>Number of strengthened sides of the wall (1 or 2)</td>
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<tr>
<td>$n$</td>
<td>Total number of bars intersected by the diagonal crack</td>
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<td>$P, P_c$</td>
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<td>Bond strength of the specimen</td>
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<td>Reinforcement efficiency factor</td>
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<td>Number of rods in the rupture controlled region</td>
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<td>Spacing of reinforcement</td>
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<td>$t_{F}$</td>
<td>Thickness of the FRP sheet</td>
</tr>
<tr>
<td>$t_f$</td>
<td>Total thickness of the fabric crossing the diagonal cracks</td>
</tr>
<tr>
<td>$t_m$</td>
<td>Thickness of the mortar joints</td>
</tr>
<tr>
<td>$t_{FRP}$</td>
<td>Thickness of FRP</td>
</tr>
<tr>
<td>$t_u$</td>
<td>Width of the masonry unit</td>
</tr>
<tr>
<td>$u_p$</td>
<td>Plastic component of the normal displacement</td>
</tr>
<tr>
<td>$V$</td>
<td>Shear capacity</td>
</tr>
<tr>
<td>$V_b$</td>
<td>Shear strength in bond controlled region</td>
</tr>
<tr>
<td>$V_t$</td>
<td>Shear strength in rupture controlled region</td>
</tr>
<tr>
<td>$V_m$</td>
<td>Shear strength contributed by masonry</td>
</tr>
</tbody>
</table>
List of Symbols

\( V_{m,1} \) Shear capacity due to sliding shear failure along the bed joint
\( V_{m,2} \) Shear capacity due to stepped shear sliding failure
\( V_{m,3} \) Shear capacity due to stepped diagonal tensile failure
\( V_{m,4} \) Compression failure
\( V_{\text{FRP}} \) Shear strength contributed by FRP
\( \nu_p \) Plastic component of the shear displacement
\( w \) Effective width of the FRP sheet carrying tension
\( w_f \) Width of the FRP reinforcement
\( Z_d \) Section modulus of the cross-sectional area of the member
\( \alpha \) Angle between the fibres of fabric and lines of brick head joint
\( \alpha_{\text{cf}} \) Shear force coefficient
\( Y_M \) Partial safety factor for masonry
\( \gamma_{td} \) Partial factor of the model
\( Y_{\text{FRP}} \) Partial safety factor for FRP in uni-axial tension
\( \Delta \varepsilon \) Change in strain over length \( \Delta L \)
\( \Delta L \) Incremental length along FRP
\( \delta \) Degradation coefficient
\( \delta_1 \) Horizontal deflection in URM
\( \delta_t \) Slip corresponding to maximum shear stress
\( \delta_{\text{max}} \) Final slip
\( \delta_{\text{mas}} \) Displacement of masonry
\( \delta_{\text{mor}} \) Displacement of mortar
\( \delta_{\text{unit}} \) Displacement of brick unit
\( \varepsilon_1 \) Strain in the wrap in the CFRP sheet
\( \varepsilon_2 \) Strain in the weft in the CFRP sheet
\( \varepsilon_{\text{fdd}} \) Maximum strain of FRP reinforcement before debonding
\( \varepsilon_{\text{FRP,u}} \) Ultimate strain of FRP
\( \varepsilon_{\text{FRP,e}} \) Effective FRP strain
\( \theta \) Angle of the fibre direction to the member axis
\( \theta \) Angle between the wrap of sheet and horizontal base line
\( \kappa_p \) Equivalent plastic relative displacement
\( \kappa_v \) bond-dependent coefficient for shear
\( \mu \) Coefficient of internal friction
\( \mu \) Ductility factor
\( \nu \) Poisson's ratio
\( \rho \) Reinforcement ratio
\( \rho_h \) FRP area fraction in the horizontal direction
\( \sigma \) Normal stress
\( \sigma_0 \) Average compressive stress in the wall due to vertical load
<table>
<thead>
<tr>
<th>Symbol</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>$\sigma_n$</td>
<td>Normal compressive stress on the wall</td>
</tr>
<tr>
<td>$\sigma_n$</td>
<td>Tensile stress normal to the joint</td>
</tr>
<tr>
<td>$\sigma_t$</td>
<td>Tangential tensile stress</td>
</tr>
<tr>
<td>$\sigma_U$</td>
<td>Confining normal stress</td>
</tr>
<tr>
<td>$\tau$</td>
<td>Shear stress</td>
</tr>
<tr>
<td>$\tau_0$</td>
<td>Shear bond strength of mortar joint</td>
</tr>
<tr>
<td>$\tau_b$</td>
<td>Average bond strength between paste and masonry</td>
</tr>
<tr>
<td>$\tau_a$</td>
<td>Assumed bond stress</td>
</tr>
<tr>
<td>$\tau_r$</td>
<td>Residual shear stress</td>
</tr>
<tr>
<td>$\tau_u$</td>
<td>Ultimate shear stress</td>
</tr>
<tr>
<td>$\tau_{avg}$</td>
<td>Average shear stress transferred from the FRP to the masonry through the epoxy over the length $\Delta L$</td>
</tr>
<tr>
<td>$\tau_{max}$</td>
<td>Maximum shear stress</td>
</tr>
<tr>
<td>$\Phi$</td>
<td>Internal friction coefficient</td>
</tr>
<tr>
<td>$\Phi_i$</td>
<td>Initial friction coefficient (Initial tangent of friction angle)</td>
</tr>
<tr>
<td>$\Phi_r$</td>
<td>Residual friction coefficient</td>
</tr>
<tr>
<td>$\psi_0$</td>
<td>Initial tangent of dilatancy angle</td>
</tr>
<tr>
<td>$\omega_f$</td>
<td>FRP reinforcement index</td>
</tr>
<tr>
<td>$\Gamma_{FK}$</td>
<td>Specific fracture energy of the FRP strengthened masonry</td>
</tr>
</tbody>
</table>
Introduction

Unreinforced masonry (URM) buildings are highly vulnerable to damage during earthquakes due to their high mass, low tensile strength and limited ductility. However, URM is still the most popular method of construction in the world due to its many advantages including its remarkable resistance to fire, its better performance in heat and sound insulation compared to many other construction materials, its durability and the fact that it remains a very economical building material (Edmund and Key, 2006). Further, a greater percentage of historical buildings are masonry constructions. They should be preserved for their historical and/or cultural significance. Most of the URM buildings in the world were built before proper earthquake resistive guidelines were developed. Therefore improving earthquake load resistance of URM buildings by strengthening the newly constructed buildings and retrofitting the existing (damaged or undamaged) buildings has a remarkable importance. The main failures of unreinforced masonry walls in an earthquake can be in-plane, out-of-plane or combinations of both. In the research reported in this thesis, the effectiveness of a technique for strengthening/retrofitting URM shear walls under in-plane loading was investigated.

Bonding fibre reinforced polymer (FRP) materials to URM walls is a relatively new strengthening technique. The advantages of strengthening/retrofitting with FRP over the conventional retrofitting techniques include adding minimal weight to the structure and high durability. Strengthening with FRP materials can be performed either by externally bonding (EB) the FRP reinforcement to the surface of the wall or by inserting the reinforcement into grooves cut into the surface of the wall using a technique called near surface mounting (NSM).
FRP laminates and thin FRP strips can be used in the EB strengthening technique. FRP strips and FRP bars can be used in the NSM strengthening technique.

The NSM FRP strengthening technique has several advantages over the EB FRP strengthening technique such as: development of higher strain in the FRP before debonding, protection from vandalism, and to some extent from fire and other environmental influences, and a minimal impact on the aesthetics of the structure. Few research studies have been conducted to investigate the behaviour of NSM FRP strengthened masonry shear walls as the technique is relatively new. The objective of this research program was to investigate the in-plane shear behaviour of NSM FRP strengthened/retrofitted wall panels under cyclic loading and thereby to assess the effectiveness of the strengthening technique. The effectiveness of different reinforcement arrangements using NSM FRP strengthening was investigated. Both pre-damaged URM wall panels and newly constructed URM wall panels were used in the study. Following are the main aims for this research.

1. Characterise experimentally the bond slip behaviour between NSM FRP strips and clay brick masonry subjected to cyclic loading. Thereby compare those results with monotonic loading pull test results. A single idealized bilinear bond slip model was developed under cyclic loading, for the purpose of using in finite element modelling.

2. Investigate and build a test setup for the experiments which can represent realistic boundary conditions to simulate the earthquake behaviour of shear walls in real buildings. A test setup was developed to impose zero in-plane rotation (fixed-fixed) boundary conditions at the top and bottom edges of the wall specimens. The test setup arrangement was modelled using a representative finite element (FE) model to obtain the dimensions of the apparatus components and other experimentally related parameters such as test specimen sizes and vertical pre-compression levels.
3. Investigate the in-plane shear behaviour of NSM FRP strengthened masonry wall panels via a laboratory experimental program using damaged and newly built URM wall panels.

4. Investigate the in-plane shear behaviour of NSM FRP strengthened masonry wall panels using a representative finite element (FE) model. The FE model was verified using the experimental results obtained from NSM FRP strengthened wall panel tests.

**Thesis outline**

Chapter 2 provides a literature review related to the topic including descriptions of the different strengthening techniques used in the past, different load types used in testing (i.e. cyclic and monotonic) and different types of test setups as well as describing the methods for analytically and numerically analysing FRP strengthened masonry construction.

Chapter 3 details the experimental program used to characterise the bond slip behaviour between NSM FRP strips and clay brick masonry when subjected to cyclic loading. Moreover a comparison of the cyclic test results with monotonically loaded pull test results is included.

Chapter 4 describes an experimental program on retrofitted wall panels with NSM FRP strips. Damaged URM wall panels obtained from a previous study (Mojsilović et al., 2009) were used. The experimental results discuss the effectiveness of the NSM FRP strengthening technique using three different reinforcement arrangements.

Chapter 5 details the development of the test setup designed to impose zero in-plane rotation (fixed-fixed) boundary conditions at the top and bottom edges of the wall specimens. FE models were used to determine appropriate dimensions for the test setup. The dimensions of the experimental wall panels (described in
Chapter 6) and the level of vertical pre-compression to be applied during shear loading were also determined using same FE model.

Chapter 6 describes the experimental program on strengthening wall panels with NSM FRP strips. The test wall panels were constructed with two sizes representing aspect ratios 1 and 0.5. Six different reinforcement arrangements were studied. The test setup described in Chapter 5 was used for testing. Both cyclic and monotonic load cases were used.

Chapter 7 reports the development of the finite element (FE) model and the verification using the experimental results given in Chapter 6.

Chapter 8 provides the conclusions of this research study and recommendations for future works.
Literature review

2.1 Earthquakes

Trembling or shaking movement of the earth’s surface is called an earthquake. Earthquakes are one of the major forms of natural disaster, claiming many lives and causing huge damage to properties and economy. The shaking of the earth’s surface is not usually life threatening, but the consequential collapse of structures can lead to deaths and injuries and cause huge economic losses (Figure 2.1). Tsunami, landslides, liquefaction of soil and seiches are some of the other natural hazards associated with earthquakes (Edmund, 1994).

Figure 2.1: Damages due to earthquake in Christchurch, Feb 2011

2.1.1 URM buildings and Earthquakes

Earthquakes are unpredictable, occur without warning, and the ground motion of an earthquake lasting not more than few minutes can cause severe damage to structures. One of the recent examples is the Wenchuan earthquake with a magnitude of 7.9M which occurred in China in May 2008. Over 69,000 people
died, around 374,171 were injured, 18,340 people were reported missing and five million people were estimated to have been left without housing (Wibowoo et al., 2008). Many similar examples can be found from different parts of the world.

Unreinforced masonry (URM) buildings are more vulnerable to earthquake damage than other types of construction used in the building industry. URM buildings typically experience comparatively more damage than other building types, and URM buildings are overrepresented amongst those damaged, during earthquake events. For example, it was reported that in 1976 Thanshan earthquake in China, the majority of the 240,000 deaths were reported to have been caused by URM brick building collapses (EERI, 1986). As reported by De Lorenzis et al. (2000), due to effects involving creep phenomena, under prolonged seismic actions, sudden failure of the structural function of masonry assemblage can result, even if the actions are not close to the ultimate damage levels.

Some of the reasons for poor performance of masonry structures during earthquakes can be listed as:

1. The material itself is brittle, and strength degradation due to load repetition is severe
2. Heavy weight
3. Large stiffness, which leads to large response to earthquake waves of short natural period
4. Large variability in strength depending on the quality of construction (Wakabayashi, 1986)

However, being economical, durable, easy to procure, and good for insulation ensures that URM is widely used both for low-rise structural walls and for infill to framed structures. In addition to that, many of the existing historically and culturally important buildings have been identified as URM constructions.
Due to its vulnerability to earthquake loads, US earthquake codes have banned the use of unreinforced masonry altogether in earthquake countries (Edmund and Key, 2006). European codes however, allow low-rise URM housing provided certain stringent conditions are met (e.g. Eurocode 8 (2004) specifies that horizontal concrete beams or steel ties should be placed around the building perimeter at every floor level with a minimum steel area of 200 mm$^2$ (Edmund and Key, 2006)).

Many existing URM buildings in the world were designed and constructed before the development of rational earthquake design procedures (Zhuge et al., 1996; Page, 1996; Li et al., 2005a). For example, large numbers of North America’s older buildings were constructed without the earthquake guidelines (Bruneau, 1994). URM buildings in Australia too were not designed to withstand earthquake loads until after 1989 (Page, 1996). The Newcastle earthquake in 1989 is one of the major earthquakes in Australia which highlighted the need for earthquake actions to be specifically considered in design as it was evident that most of the seriously damaged buildings were URM constructions (Shrive, 2006). Furthermore, most of the historical buildings in the world which many nations including Australians love and adore as their national heritage are URM constructions. Such historical constructions typically require greater attention in order to secure their performance under earthquake loading (Chuang and Zhuge, 2005). Therefore, strengthening of URM buildings to resist the earthquake damages has a remarkable importance.

### 2.1.2 Effect of earthquakes on URM shear walls

Earthquake loading is cyclic in nature. It produces deflections and lateral reactions in the structure which change direction rapidly. The earthquake action very much depends on the dynamic characteristics of structure. The failures of URM buildings during earthquakes can be put into the following categories (Bruneau, 1994):
2. Literature review

- Lack of anchorage
- Anchor failure
- In-plane failure
- Out-of-plane failure
- Combine in-plane and out-of-plane effects
- Diaphragm related failure

As the research reported in this thesis focusses on in-plane shear performance of URM and strengthened/retrofitted URM walls, only that aspect of the overall structural behaviour is considered further in this literature review. Lateral loads imposed on loadbearing masonry buildings are resisted primarily by the shear walls in the building. In moderate to large earthquakes, URM shear walls are displaced / loaded past their elastic limit. Shear walls which have not been designed and detailed properly to withstand inelastic deformations and to dissipate energy can suffer heavy damage and even result in collapse of the building. Zhuge et al., (1996) divided the failure modes of URM shear walls into two main groups: flexural bending and shear. Zhuge et al., (1996) further divided these two groups into three failure modes in flexural bending and two failure modes in shear failure as listed below.

Failure modes in flexural bending:
- (a) flexural cracking at heel,
- (b) rocking and
- (c) toe crushing

Failure modes in shear failure:
- (a) shear sliding and
- (b) diagonal cracking

According to the results of an earthquake damage analysis and subsequent experiments, Tomazevic (1999) highlighted three types of mechanism/failure mode, defining the seismic behaviour of structural masonry walls when
subjected to in-plane seismic loads. Those three types contain two shear failure modes as in Zhuge et al. (1996) and the flexural failure mode as shown in Figure 2.2.

![Figure 2.2: Failure modes of masonry shear walls: (a) Sliding shear (b) Diagonal cracking (c) Rocking (Tomazevic, 1999)](image)

The failure mechanisms observed in practice are influenced by the wall aspect ratio (height/length ratio), thickness of the wall, compressive strength of masonry and quality of the materials, vertical pre-compression stress and also by the boundary restraints and the way the load is applied to the wall (Lee et al., 2008; Tomazevic, 1999; Magenes and Calvi, 1997; ElGawady et al., 2007). Among the parameters investigated in the experiments reported in the literature, the magnitude of vertical loads was found to be the most significant factor affecting the behaviour of URM walls subjected to earthquake loads (Zhuge et al., 1996). This can be demonstrated by the following observations.

(a) When the vertical load was very low, the wall failed by shear sliding at the base of the wall. These walls possessed nearly zero shear strength and failed in a brittle manner.

(b) For low to moderate vertical load, the mortar joint cracks dominated and walls failed by rocking.

(c) For moderate to high vertical load, walls failed by diagonal shear and in a brittle manner.
(d) With very high vertical load, failure occurred in the brick units, either by tensile cracking or crushing of the unit.

The relationship between vertical pre-compression stress and aspect ratio for the URM wall failure mode is shown in the table below (Table 2.1) (Lee et al., 2008).

<table>
<thead>
<tr>
<th>Lower pre-compression</th>
<th>Higher pre-compression</th>
</tr>
</thead>
<tbody>
<tr>
<td>Lower aspect ratio</td>
<td>Sliding failure mode</td>
</tr>
<tr>
<td></td>
<td>Diagonal tension failure mode</td>
</tr>
<tr>
<td>Higher aspect ratio</td>
<td>Rocking failure mode</td>
</tr>
<tr>
<td></td>
<td>Toe crushing failure mode</td>
</tr>
</tbody>
</table>

### 2.2 Simulation of seismic behaviour on masonry walls

The seismic behaviour of masonry shear walls can be investigated in the laboratory using different experimental test methods. Shake table test is the main test method of modelling earthquakes in a laboratory. It can simulate ground motions including reproduction of recorded earthquake time histories to shake the test specimens accordingly. Different test specimen types including full scale building models can be tested using shake tables (Figure 2.3).

Apart from the shake table test, two main test types can be identified in the literature based on the external load application to the test specimen (Vliet, 2004). In the first type, the loading is applied by means of only a compression force. The different tests setups used by researchers to investigate the shear behaviour according to this principle are shown in Figure 2.4.

In the second type, the masonry wall panel with bed joints aligned in the horizontal direction is supported at the lower edge, and loaded in-plane by a
horizontal load applied at the upper edge (Vliet, 2004). This type of testing allows different boundary conditions to be imposed at the upper edge of the walls to attempt to replicate the types of restraint imposed on walls in real buildings. Also, different pre-compression stress levels can be applied to these specimens to simulate the gravity loads likely to be imposed on the walls in service. Examples of test setups of this type are shown in Figure 2.5.

Figure 2.3: Different specimen types used in shake table tests (a) ElGawady et al. (2006b) (b) Al-Chaar and Hasan (2002) (c) McLean et al. (2011)
Vliet (2004) concluded that the realistic shear tests can be carried out using the test setups in which the upper edge of the test specimen is forced to remain parallel to the lower edge as shown in Figure 2.6. This will result in a crack pattern with diagonal shear cracks. Bosiljkov et al. (2008) also identified some test methods similar to Vliet (2004). They concluded that the necessary behavioural parameters for the seismic assessment and full performance analysis of masonry shear walls can be obtained only with the test arrangements shown in Figure 2.7 under cyclic loading. Tomazevic (1999) has mentioned when simulating earthquake, a typical pier needs to be fixed at the top and bottom to the structural system preventing the rotation.
While identifying the test setup and method used to evaluate seismic behaviour of masonry shear walls, the other important parameter to consider is the
loading pattern to be used in the experimental study. Tomazevic et al. (1996) examined the influence of four different lateral displacement patterns on masonry walls:

a. Monotonically increased displacements (Figure 2.8a)

b. Cyclic lateral displacements with amplitudes increasing in three different blocks and repeated three times at each amplitude peak (Cyclic type B) (Figure 2.8b)

c. Cyclic lateral displacements with uniformly increasing displacement amplitudes, repeated three times at each amplitude peak, with decreasing amplitudes between two consecutive blocks (Cyclic type C) (Figure 2.8c)

d. Simulated displacement response of masonry building to an earthquake (Figure 2.8d)

Significant differences in the ultimate loads and displacement capacities for the different loading patterns were obtained in this study. Higher resistance and larger ultimate displacements were observed in the case of monotonic loading than for cyclic loading procedures of any type (Figure 2.9). The study concluded that, to obtain data regarding strength and stiffness degradation, and energy dissipation capacity of masonry walls, the cyclic character of seismic loads should be simulated. This conclusion was also reached by Bosiljkov et al. (2008). Further, they concluded that both (b) and (c) loading patterns in Figure 2.9 were adequate to simulate earthquake effects on masonry walls.

The vertical pre-compression is also a main parameter to consider when simulating earthquake effects on wall panels under laboratory conditions. Although not normally simulated in laboratory tests, for walls in real buildings, the vertical pre-compression changes during an earthquake due to stresses developed as a result of the boundary conditions that prevent rotation of the wall at large lateral displacements (Tomazevic, 1999).
2. Literature review

Figure 2.8: Lateral displacement time histories (a) Monotonic (b) Cyclic type B (c) Cyclic type C (d) Simulated earthquake response (Tomazevic et al., 1996)

Figure 2.9: Hysteresis envelopes for different displacement patterns (Tomazevic et al., 1996)
2. Literature review

2.3 Existing retrofitting/strengthening techniques for URM shear walls

The basic concept of seismic retrofitting/strengthening is to upgrade the structural strength and improve the inelastic deformation capacity or ductility of the structure (Chuang and Zhuge, 2005). In Australia, although URM is one of the most popular types of construction, research into seismic retrofitting of masonry structures are rare. AS 3826-1998 (Standards Australia, 1998) provides only broad guidance on the extent of strengthening required for existing buildings (Chuang and Zhuge, 2005). Chuang and Zhuge (2005) classified seismic retrofitting techniques for masonry structures into two main categories as follows.

1. Reduce the earthquake force
2. Upgrade the existing building to resist earthquake load, further divided into:
   - Change the structural system
   - Upgrade the element strength

Installation of base isolation and installation of damper devices can be identified as the techniques used to strengthen structures which aim to reduce the earthquake force.

To improve the strength and/or displacement capacity of individual structural elements (walls), a number of conventional strengthening techniques have been reviewed (Chuang and Zhuge, 2005; ElGawady et al., 2004; Corradi et al., 2008). These techniques include surface treatment with ferrocement, reinforced plaster or shotcrete, grout and epoxy injection, FRP and Polypropylene jacketing, confining the URM using reinforced concrete tie columns, post-tensioning the wall using steel tendons, and centre core technique. The advantages and disadvantages of some of these techniques are listed in Table
2. Literature review

2.2 (ElGawady et al., 2004). The main disadvantages of conventional retrofitting techniques highlighted by most researchers (El-Dakhakhni et al., 2004; ElGawady et al., 2004; El-Dakhakhni et al., 2006; Corradi et al., 2008) are that they may; add considerable mass, reduce space, require skilled labour, interrupt the normal function of the building, be costly and restrict the use of certain types of buildings. Also most of the conventional techniques adversely affect the aesthetic appearance of the building.

2.4 Retrofitting/strengthening of URM using fibre reinforced polymers (FRPs)

Fibre reinforced polymer (FRP) strengthening/retrofitting is emerging as an effective seismic retrofitting technique for URM buildings. FRP's consist of high strength fibres embedded in a resin matrix. Epoxy is generally used as the adhesive to bond the FRP to the walls. The FRP composite is very strong in the direction of the fibres and generally weak in lateral direction. There are three common types of FRPs depending on the material used for the fibres, namely Glass (GFRP), Aramid (AFRP) and Carbon (CFRP). Ranges of stress-strain diagrams for each FRP type are shown in Figure 2.10. The stress-strain behaviour of mild steel is also shown in the same figure for comparison. Typically the fibres show no ductility. They behave completely linear elastic until failure (Shrive, 2006). However the properties of FRP are continuously improving and some research studies show that combinations of fibres can provide some ductility to the strengthened member (Grace et al, 2002; Wu, 2004).

FRP is manufactured in different forms such as rods, sheets, tendons, laminates and strips. All of them have been successfully used in strengthening/retrofitting of URM structures, yet FRP rods and laminates are the most common (Zhuge, 2008a). FRP tendons are used for pre-tensioning or for post-tensioning. When the fibres are aligned in the longitudinal direction of the sheet, strip, tendon or
2. Literature review

Table 2.2: Advantages and disadvantages of retrofitting techniques (ElGawady et al., 2004)

<table>
<thead>
<tr>
<th>Retrofitting Technique</th>
<th>Advantages</th>
<th>Disadvantages</th>
</tr>
</thead>
<tbody>
<tr>
<td>Ferrocement</td>
<td>• Low cost&lt;br&gt;• Low technology&lt;br&gt;• Limited added mass</td>
<td>• Space reduction&lt;br&gt;• Architectural impact&lt;br&gt;• Requires architectural finishing&lt;br&gt;• Limited efficiency&lt;br&gt;• Limited energy dissipation</td>
</tr>
<tr>
<td>Reinforced Plaster</td>
<td>• Low technology&lt;br&gt;• Limited added mass</td>
<td>• Space reduction&lt;br&gt;• Architectural impact&lt;br&gt;• Requires architectural finishing</td>
</tr>
<tr>
<td>Shotcrete</td>
<td>• High increment in $F_{ur}$&lt;br&gt;• Very significant improvement in energy dissipation</td>
<td>• Space reduction&lt;br&gt;• Heavy mass&lt;br&gt;• Disturbance to occupancy&lt;br&gt;• Architectural impact&lt;br&gt;• Required architectural finishing</td>
</tr>
<tr>
<td>Injection</td>
<td>• No added mass&lt;br&gt;• No effect on building function&lt;br&gt;• No space reduction&lt;br&gt;• No architectural impact</td>
<td>• Epoxy creates zones with varying stiffness and strength&lt;br&gt;• High cost of epoxy&lt;br&gt;• No significant increment in $F_r$ using cement-based grout</td>
</tr>
<tr>
<td>External steel reinforcement</td>
<td>• High increment in $F_{ur}$&lt;br&gt;• Prevent disintegration&lt;br&gt;• Improves ductility and energy dissipation</td>
<td>• Corrosion&lt;br&gt;• Heavy mass&lt;br&gt;• Violation of performance level&lt;br&gt;• Requires architectural finishing&lt;br&gt;• Disturbance to occupancy</td>
</tr>
<tr>
<td>Confinement</td>
<td>• Prevent disintegration&lt;br&gt;• Improve ductility and energy dissipation</td>
<td>• Not easy to introduce&lt;br&gt;• Limited effect on $F_{ur}$&lt;br&gt;• Requires architectural finishing&lt;br&gt;• Disturbance to occupancy</td>
</tr>
<tr>
<td>Post tension</td>
<td>• No added mass&lt;br&gt;• No effect on building function</td>
<td>• High losses&lt;br&gt;• Requires complicated anchorage system&lt;br&gt;• Corrosion potential</td>
</tr>
<tr>
<td>Centre Core</td>
<td>• No space reduction&lt;br&gt;• No architectural impact&lt;br&gt;• No effect on building function</td>
<td>• Creation of zones with varying stiffness and strength</td>
</tr>
</tbody>
</table>

$F_{ur}$ - lateral resistance of unretrofitted specimens, $F_r$ - lateral resistance of retrofitted specimens
bar, the FRP element exhibits highly anisotropic behaviour which results in very high strength and high stiffness in that direction. FRP sheets are also available with multidirectional fibres which provide more orthotropic properties (Shrive, 2006).

FRP reinforcements are introduced in URM wall systems to enhance the tensile capacity of the wall. The FRP in the strengthened/retrofitted wall will absorb the tensile stresses and increase overall stiffness, ductility and bearing capacity. It will also make the wall more seismic resistive by reducing the rapid degradation of stiffness, strength and energy dissipation capacity observed in the URM wall under seismic action.

The FRP strengthening method has several advantages compared with conventional retrofitting techniques. Major advantages are very high strength, light weight and high durability. The light weight of the FRP adds less additional weight to the structure which may not highly affect the overall properties of the structure. Also this may result in easy handling which reduces the labour cost. High durability is mainly due to the corrosion insensitivity of the FRP materials.
Minimal loss of usable space due to the strengthening application and relatively easy installation process are some other benefits over conventional retrofitting techniques. FRP strengthening methods may result in less interruption to the existing services and building occupants (Korany and Drysdale, 2004; Zhao et al., 2004; Li et al., 2005a; El-Dakhakhni et al., 2006; ElGawady et al., 2006c; Silva et al., 2006; Shrive, 2006). In addition to the above advantages, FRP reinforcement will also resist crack propagation (ElGawady et al., 2006c).

FRP retrofitting/strengthening schemes may fail by FRP rupture or, more commonly, by debonding either at the FRP masonry interface (epoxy failure) or within the masonry itself. These failure modes are brittle in nature. This is one drawback of FRP retrofitting techniques. However strengthening of URM walls using FRP will increase the strength and ductility of masonry compared with the brittleness of masonry of URM alone. Susceptibility of the resin in FRP to ultraviolet light is another disadvantage of FRP. The resins slowly become brittle, and often become plastic as they “weather” over the years when exposed to sunlight. Thus, FRP must be protected from exposure to direct sunlight (Shrive, 2006). Lack of fire resistance is another disadvantage of FRP. The near surface mounted (NSM) FRP strengthening technique (detailed in section 2.5.2) is an effective technique to overcome some of the above disadvantages (Petersen et al., 2009).

### 2.5 Past experimental research on shear strengthening of URM using FRP

Three different FRP reinforcement techniques have been used by researchers to strengthen/retrofit masonry walls. The distinction between the two techniques is based on the way the FRP is bonded to the masonry wall;

1. Externally bonded (EB) technique - FRP sheets, laminates or strips are externally bonded to the surface of a wall in this technique
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2. Structural repointing (SR) technique - FRP rods or strips are inserted and bonded into grooves cut into the surface of the mortar joint in this technique

3. Near surface mounting (NSM) technique - FRP strips or rods are inserted and bonded into grooves cut into the surface of a wall in this technique

More details about each technique and the past research studies conducted using each technique are discussed in the sections 2.5.1 and 2.5.2 below.

2.5.1 Externally bonded (EB) FRP technique

This technique requires that the FRP reinforcement is adhered directly to the external surface of the masonry. Therefore, the first step in the process is the preparation of the wall surface. The wall surface is ground with a sanding machine and any imperfection of the wall is filled by a putty or epoxy to make the surface levelled (Stratford et al, 2004; Alcaino and Santa-Maria, 2008; Mahmood and Ingham, 2011). The FRP reinforcement is then adhered to the wall externally, usually using epoxy adhesive.

Some of the advantages of EB FRP techniques in strengthening/retrofitting are: with FRP sheets, it provides dowel resistance across the sliding joints; it is simple to apply and it is capable of providing confinement to masonry in compression if sheets are applied to both sides of the wall. Also, when FRP strips are used there is no restriction in selecting the widths of the reinforcement strips unlike the NSM technique in which the strip dimensions are limited by the wall thickness. EB FRP techniques also have some disadvantages namely, adverse impact on the aesthetics of the building, highly susceptible to debonding failure and it exposes the FRP to vandalism, fire and other environmental influences. Furthermore, it may buckle from the surface of the wall if subjected to compression.
Past research studies performed using EB FRP techniques to strengthen/retrofit masonry walls under both monotonic and cyclic loading cases are described in the following sections.

2.5.1.1 Monotonic loading case

CFRP and GFRP are the most common types of FRP used by researchers to retrofit masonry using the EB technique under monotonic loading (Corradi et al., 2008; Valluzzi et al., 2002; Stratford et al., 2004; Hamid et al., 2005; Marcari et al., 2007; Luccioni and Rougier, 2011; Mahmood and Ingham, 2011). In these studies, FRP strips or FRP sheets were tested in different orientations to identify the effectiveness due to its orientation. In some studies, the whole surface of the wall was covered by gluing FRP sheets to the masonry wall (Stratford et al., 2004; Hamid et al., 2005; Luccioni and Rougier, 2011; Mahmood and Ingham, 2011).

Valluzzi et al. (2002) used polyvinyl-alcohol unidirectional fibres (PVAFRP) in addition to carbon (CFRP) and glass FRP (GFRP) to investigate the in-plane shear performance of masonry. Coupon-size masonry panels were constructed for the study. Both squared grid strengthening configuration and diagonal strengthening configuration were used to retrofit the panels as shown in Figure 2.11. The results highlighted that the effectiveness of asymmetrical applications of FRP (single-side reinforcement) on masonry panels is limited. For the three FRP types used (CFRP, GFRP and PVAFRP), single side strengthened walls with squared grid configuration had not shown an increase in the failure load compared with URM. Only the diagonal grid strengthening configuration showed a load increase compared with URM (11% and 15.5% for the two FRP types; CFRP and GFRP respectively). The squared grid strengthening configuration of double side strengthened wall showed 3.4%, 14.2% and 47.3% increase for the three different FRP types respectively. The failure load for the double side diagonal grid strengthening configuration showed 25% and 56% more than the single side strengthened walls for the two FRP types used; CFRP.
and GFRP respectively. These observations highlight the effectiveness of double side strengthening compared with single side.

![Figure 2.11: FRP configurations (a) square grid strengthening (b) diagonal strengthening (Valluzzi et al., 2002)](image)

Significant out of plane deformations were observed by Valluzzi et al. (2002) in the single side strengthened walls. Similar behaviour in single side strengthened walls was observed in some other research studies (eg: Mahmood and Ingham, 2011). The main failure mode in single side reinforced panels was diagonal splitting and more damage was observed on the unreinforced side. Double side strengthening was found to result in brittle failure by either de-lamination (peeling) or rupture of FRP strips.

Valluzzi et al. (2002) have also found that the less stiff FRP material appeared to be more effective than the higher stiffness FRPs in terms of ultimate strength and stiffness in EB technique. They have concluded that the diagonal reinforcement configuration is more efficient in terms of shear capacity than the grid configuration. A test series similar to Valluzzi et al. (2002) was performed by Luccioni and Rougier (2011) with similar size test specimens. In addition, they also investigated the effect of FRP strengthening of pre-damaged walls. They concluded retrofitting the walls with CFRP bands was more effective than covering the complete wall with FRP reinforcement.
Corradi et al. (2008) investigated the effectiveness of traditional and innovative strengthening techniques on URM walls. EB GFRP, grout injection, joint replpointing, surface treatment (with ferrocement) and polypropylene jacketing are the strengthening techniques used in these studies. EB GFRP wet lay-up composite applied to masonry panels resulted in a stronger system compared to the unreinforced walls. It resulted in a 186% increase in lateral resistance compared to unreinforced panels. Also, Corradi et al. (2008) found that the polypropylene jacketing did not enhance the lateral strength of masonry walls.

GFRP sheets were used by Stratford et al. (2004) and Hamid et al. (2005) to retrofit URM shear wall panels. In this technique, the FRP sheets were bonded to the whole surface of the wall. This retrofitting method of bonding FRP sheets to the entire surface of the wall is easy, compared to the other FRP arrangements. This is the main advantage of this technique (Stratford et al., 2004). Figure 2.12 shows the construction sequence of the GFRP-strengthened masonry wall panel used in the research study by Stratford et al. (2004). In this study, the GFRP sheets debonded while the cracks were developing in the masonry wall. Debonding failure occurred in the walls either at the epoxy–brick interface, between the adhesive and filler layers or beneath the surface of the brick. Here the strengthening increased the load capacity by 65% compared with URM.

![Figure 2.12: Schematic arrangement of in-plane shear strengthening using sheet glass-fibre reinforced polymer (Stratford et al., 2004)](image-url)
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Figures 2.13 (a) and (b) shows the FRP reinforcement arrangements used to strengthen URM wall panels with GFRP and CFRP by Marcari et al. (2007). While proving that the FRP strengthening techniques were an effective technique to strengthen URM walls, the experiment showed that the large effective axial stiffness of the FRP strips changed the original shear failure mode of the panel to shear/flexural mode. They observed buckling of FRP sheets due to compression (Figure 2.13c). The latter is an adverse effect of EB retrofitting techniques when subjected to compression along the longitudinal axis. In the EB FRP strengthening technique, the lateral restraint against buckling of FRP is based on the FRP to masonry bond. Buckling and debonding occurs when this bond strength is reached. Finally the researchers found that the GFRP showed more compatibility with masonry than CFRP in terms of stiffness and thus resulted in the larger strength increase.

Mahmood and Ingham (2011) investigated the effectiveness of FRP strengthening under in-plane shear loading by testing seventeen double wythe (225 mm thick) wall panels. Different reinforcement arrangements were investigated as shown in Figure 2.14. The walls WTC6, WTC7, WTC8 and WTC9 were strengthened with NSM CFRP rectangular bars (see section 2.5.2.1.1) but all the other specimens were retrofitted using the EB technique. A modified diagonal tension test setup was used (similar to the test setup in Figure 2.4b). The shear strength was increased by up to 325% due to strengthening with FRP. It was found that the single face strengthening was effective. However large out-of-plane displacements were observed in the single face strengthened walls that failed by diagonal shear cracking. Mahmood and Ingham (2011) also found that the vertical FRP reinforcement arrangements were effective in shear strengthening.

2.5.1.2 Cyclic loading case

The following researchers used EB FRP techniques to improve the shear performance of masonry walls. The tests were conducted under cyclic loading to
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Figure 2.13: (a) and (b) FRP reinforcement arrangements (c) Bucking and debonding failure of FRP (Marcari et al., 2007)

Figure 2.14: Different FRP arrangements (Mahmood and Ingham, 2011)

Al-Chaar and Hasan (2002) performed shaking table tests to model the dynamic effect on the masonry wall. They found that retrofiting of one side of an URM wall can increase both in-plane and out-of-plane strengths and can enhance the
seismic resistance of masonry walls. One important point highlighted in this research is that the connections of wall to roof and foundation have significant influence on the enhancement of the seismic resistance.

Maria et al. (2006) tested two unreinforced wall panels and four reinforced panels with EB CFRP under in-plane cyclic loading. The test setup and the loading history used in this study are shown in Figure 2.15. The test specimens were fixed at the bottom and the top was left free to laterally displace and rotate. A pre-compression load was applied to the specimens and it was taken as the first floor load of a three storey building with concrete slabs in the upper two floors and a light roof. Two reinforcement configurations for strengthening were used in this study namely diagonal strip configuration and horizontal strip configuration (Figure 2.16). The reinforcement ratio \( \rho \) was calculated using the following equation.

\[
\rho = \left( \frac{b_f t_f}{h} \right) \frac{\cos \alpha}{h b} \quad (2.1)
\]

Where \( b_f \) and \( t_f \) are total width and thickness of the fabric that is crossing the diagonal cracks, \( \alpha \) is the angle between the fibres of fabric and courses of bricks, and \( h \) and \( b \) are the height and the thickness of the masonry wall.

Figure 2.15: (a) Test setup (b) Loading history (Maria et al., 2006)
Maria et al. (2006) found that the diagonal reinforced walls had a brittle failure with sudden loss of strength whereas horizontal reinforced wall panels showed a less brittle failure. They observed 60% to 80% strength increase due to CFRP retrofitting. Increases in energy dissipation, compared to URM, were observed in diagonal reinforced panels but not in horizontal reinforced panels in this study.

Capozucca (2011) showed that the increase in resistance under cyclic shear load was higher in walls retrofitted with diagonal CFRP strips than vertical and horizontal CFRP strips. The reinforcement strip arrangements are shown in Figure 2.17. The reason for this was that delamination was avoided in the diagonal FRP arrangement. Here the “I” shape historic unreinforced masonry walls were used and the walls were first tested until damaged. Each damaged wall was then retrofitted at one side. Capozucca (2011) used a steel nail anchorage system to improve the adhesion of FRP strips to the masonry. He concluded the historic masonry required an adequate anchorage system as well as gluing using epoxy resins to improve the FRP bond.

Figure 2.16: FRP reinforcement configurations (a) diagonal strip configuration (b) horizontal strip configuration (Maria et al., 2006)
Almusallam and Al-Salloum (2007) conducted a test series to evaluate the behaviour of infill walls retrofitted with GFRP sheets under quasi static cyclic loading. The test setup is shown in Figure 2.18(a). Two infill wall panels were used in three different tests in this study. One infill panel was first tested (control test) until diagonal cracking in the masonry infill. This damaged specimen was then repaired using GFRP sheets and tested as the second test (repaired test). The other infill wall panel was strengthened with GFRP sheets and tested as the third test (strengthened test). Same FRP strengthening arrangement was used for both repaired and strengthened wall panels (Figure 2.18b). The loading histories for this experiment were developed based on the conventional guidelines of quasi static type testing by referring to the several researchers in testing RC structures. For the above three types of infill wall panels, three displacement histories were used as shown in Figure 2.18. Figure 2.19 shows the load versus displacement envelops for the three types of test specimens obtained in this study. Almusallam and Al-Salloum (2007) found that the displacement capacity for both repaired and strengthened wall panels were enhanced by 47% and 98% than the control specimen respectively. However the load carrying capacity of the strengthened specimen was decreased compared with the control wall. But the repaired specimen did increase the maximum load by 6% than control wall.
Figure 2.18: (a) Test setup (b) Loading history for control specimen (c) Loading history for repaired specimen (d) Loading history for strengthened specimen (e) FRP strengthening scheme (Almusallam and Al-Salloum 2007)
Zhao et al. (2003, 2004) used a similar test setup to Almusallam and Al-Salloum (2007) to apply quasi static cyclic shear loading to FRP retrofitted masonry specimens. Carbon fibre sheets (CFRP) in two different configurations were used in this study as shown in Figure 2.20. A constant compressive stress of 1.2 MPa was maintained during the test. For the URM specimen, diagonal cracking failure was observed. For the X FRP arrangement the main diagonal crack was shifted with diagonal cracks outside the strengthened area with some secondary cracks near the main crack. In the other FRP arrangement (\(\triangle\) arrangement), mainly diagonal cracks occurred while toe crushing was also observed. Zhao et al. (2003, 2004) concluded that the strengthening with FRP improved cracking load, ultimate load, lateral deformation and energy dissipation capacity of the URM wall. Numerical models were developed for both the above arrangements (more details are in section 2.7.1.1).

Figure 2.20: FRP strengthening configuration (a) \(\triangle\) arrangement (Zhao et al., 2004) (b) X arrangement (Zhao et al., 2003)
Schwegler (1995) conducted a testing series to investigate the shear strength of masonry under cyclic loading. CFRP sheets were used to retrofit the masonry panels. The FRP configurations used were single side, both sides and with and without anchoring to the concrete slab. Constant pre-compression load was applied during the tests and was calculated based on the loading due to the weight of three stories above the wall. He observed that the diagonally arranged CFRP sheets debonded from the masonry through a thin layer of brick underneath the FRP. Greater ductility was observed in strengthened walls compared with URM. He found that, when the same reinforcement ratio was used in single sided strengthening and double sided strengthening, the difference in strength increase was negligibly small. The experiments highlighted that the anchoring of CFRP sheets to the concrete slabs was the decisive factor for the increase of the bearing resistance.

ElGawady et al. (2007) used both newly strengthened URM wall panels and damaged retrofitted URM wall panels similar to Almusallam and Al-Salloum (2007). In this research study, the anchorage failure was prevented by clamping the FRP ends to the specimen’s footing and head beam using steel plates and bolts. A constant gravity load was applied to the specimens using post-tensioning bars. The total pre-compressing stress was 0.35MPa which included the self weight and instrument weights additionally to the post-tensioning bars. GFRP sheets were used to retrofit in this experiment. The quasi static cyclic load was applied using two hydraulic jacks. The loading arrangement and the loading history are shown in Figure 2.21.

ElGawady et al. (2007) concluded that the mode of failure of the wall was strongly dependent on the FRP axial rigidity. The higher amount of FRP axial rigidity led to very brittle failures. Unlike the results shown in Almusallam and Al-Salloum (2007), both the directly retrofitted and repaired specimens showed an increase in the load carrying capacity compared with unretrofitted specimens. The minimum coefficient of friction in URM was measured to be 0.75
in this study. The effectiveness of the EB FRP technique has also been investigated by ElGawady et al. (2005, 2006c) with different aspect ratios (slender wall panels and squat wall panels) and different loading cases (dynamic and quasi-static cyclic loading). Dynamic loading was applied using a shake table.

![Figure 2.21: (a) Test setup (b) loading history (ElGawady et al., 2007)](image)

In the experimental study performed by Mosallam and Banerjee (2011) axial load rods were used (Figure 2.22) to apply the pre-compression load to the test walls similar to the pre-compression applied by ElGawady et al. (2007). This test allowed the upper loading beam to rotate in the plane of the wall during the in-plane cyclic loading. Six walls specimens were tested, strengthened with externally bonded CFRP sheets, CFRP strips and GFRP sheets. First one URM wall was loaded until it was damaged. It was then retrofitted with CFRP laminates. The study showed that the application of FRP resulted in a 20%
strength increase in the pre-damaged wall and a 35% strength increase, on average, in the other walls. Further, it showed that the diagonal cracking of URM walls had changed to toe crushing failure due to retrofitting.

Figure 2.22: Test setup (Mosallam and Banerjee, 2011)

Alcaino and Santa-Maria (2008) used the test setup shown in Figure 2.15(a) to investigate two different EB CFRP strip configurations applied to hollow clay brick walls, additionally with steel shear reinforcement (SRM), and without steel shear reinforcement (NSRM). They found that the maximum shear strength and the displacement capacity were larger in diagonally reinforced walls than horizontally reinforced walls. Also they found that the FRP reinforcement has a greater influence on the NSRM walls than with SRM walls. The same researchers (Santa-Maria and Alcaino, 2011) conducted a similar study on damaged walls using a similar test setup and similar cyclic displacement history as used in their previous research (Alcaino and Santa-Maria (2008)) to compare the effectiveness of the retrofitting on walls with and without initial damage. Two types of walls, as used in their 2008 study (Alcaino and Santa-Maria (2008)) were used in this study (SRM and NSRM). The study concluded that the increase in shear strength and maximum displacement at failure of a repaired wall is similar to that of a wall with no initial damage and with the same amount of CFRP overlays. Also it showed that the undamaged
walls initial lateral stiffness cannot be recovered by retrofitting the damaged walls. The increase in the maximum strength was larger in retrofitted NSRM walls compared to retrofitted SRM walls.

Unlike the steel shear reinforcement used in the Alcaino and Santa-Maria (2008) study, Holberg and Hamilton (2002) used a ductile steel connection to connect the walls pier or shear walls to the rest of the structure against rocking in their hybrid strengthening system. Here the researchers used GFRP sheets to resist the shear and flexural stresses within the pier/wall while ductile connections were used to restrain the bottom of the wall against rocking and sliding. Figure 2.23 illustrates the concept of the hybrid strengthening system. The ductile steel connection was designed to yield before the FRP composites failed either by rupture or by debonding providing increased lateral capacity and ductility.

Figure 2.23: Ductile connections with shear strengthening (Holberg and Hamilton, 2002)

Four unreinforced concrete masonry specimens strengthened with GFRP sheets were used to investigate this hybrid strengthening system. Ductile connections were made using traditional structural steel and reinforcing steel. Vertical and diagonal FRP strips were used to improve the flexural capacity and shear capacity within the wall. In general the system was effective. The structural steel and reinforcing steel connections yielded and the in-plane rocking behaviour of the wall was improved, in terms of both strength and ductility. They recommended further investigations on this technique.
Marshall and Sweeney (2002) conducted an experimental series to investigate the in-plane shear behaviour of EB reinforced masonry walls under cyclic loading. Here the testing was performed such that the top of the walls were prevented from rotating and only sliding was allowed. Different FRP configurations were used including full surface cover with FRP sheets, diagonal FRP strips, vertical and horizontal FRP arrangement. Marshall and Sweeney (2002) found that the stiffer materials (such as carbon FRPs) and thicker materials were more susceptible to debonding and allowed the masonry to fail catastrophically.

2.5.2 Structural repointing

The technique of inserting FRP bars or thin FRP strips into a groove cut into the surface of the mortar joints referred as structural repointing (SR). This is a form of near surface mounted (NSM) technique and is particularly attractive from an aesthetic point of view because it can be completely concealed from view. The FRP reinforcement installation procedure for the structural repointing technique consists of:

1. cutting out part of mortar in the mortar joint using a grinder
2. filling the joints with an epoxy-based paste
3. embedding the rod in the joint
4. retooling (Tumialan et al., 2001)

Typically the technique involves placing the FRP reinforcements horizontally in the mortar bed joints. But it is also used in placing the reinforcement vertically in the mortar head joints in the case of stack bonded masonry. A typical cross section through a masonry thickness with structurally repointed FRP bar is shown in Figure 2.24. A possible disadvantage of the SR technique is the diameter size of the FRP rods gets limited by the thickness of the mortar joint. Also if deep grooves required to be cut into the wall surface through the mortar joints, it may cause cracking through the thickness of the wall in this technique.
Several researchers used structural repointing to strengthen the masonry walls including; Tinazzi and Nanni (2000); Tumialan et al. (2001); Tumialan and Nanni (2002); Li et al. (2005a and b); Silva et al. (2006); Turco et al. (2006); and Corte et al. (2008). These studies were performed under both monotonic and cyclic loading cases and are described in the following sections.

Figure 2.24: Structural repointing with FRP bar (Li et al., 2005)

2.5.2.1 Monotonic loading case

The following researchers investigated the shear behaviour of the structurally repointed masonry walls under monotonic loading: Tumialan and Nanni (2002); Li et al. (2005a); Silva et al. (2006) and Turco et al. (2006). Concrete masonry unit (CMU) walls were used in all these studies. Tumialan and Nanni (2002), Li et al. (2005a) and Turco et al. (2006) tested their wall panels by applying diagonal compression loads in closed loop systems. The typical test setup used in these studies is shown in Figure 2.25. Silva et al. (2006) applied monotonic horizontal load using a similar test setup to that used by Almusallam and Al-Salloum (2007) as discussed in section 2.5.1.2.

Tumialan and Nanni (2002) tested three masonry walls; one unreinforced, one strengthened with GFRP bars by SR and one strengthened with EB GFRP laminates to compare the results with the SR results. Equal amounts of GFRP were used in the two strengthened walls. Tumialan and Nanni (2002) found that both the strengthened (SR and EB) walls had similar shear capacity. However, pseudo-ductility (ratio of ultimate shear strain and shear strain, corresponding
to the point where the in-plane load vs. shear strain curve tends to be flat) was less than the wall strengthened with EB FRP strips. The shear capacity of the GFRP strengthened masonry wall showed about 80% increase than the URM in this study. Li et al. (2005a) reported that if the SR GFRP bars are not inserted into every bed joint in the wall, failure may occur in the unstrengthened joint. By placing GFRP bars in every bed joint, Li et al. (2005a) achieved 80% increase in shear capacity compared to URM. They also showed that the SR strengthened walls have exhibited high pseudo-ductility and more stability than the URM after failure.

Six stack bonded CMU wall panels with the aspect ratio (height to length ratio of the wall) 1.83 were used by Silva et al. (2006) in their study. Structural repointing technique was used to insert the GFRP rods to the mortar joints. They found that the shear capacity of wall panels with different reinforcement schemes were increased from 15% to 56% due to strengthening compared with URM. Silva et al. (2006) reported that the wall aspect ratio has a significant effect on the failure of both the unstrengthened and strengthened URM walls. They concluded that for walls with aspect ratios lower than one, it is more advantageous to place GFRP rods in the vertical direction in order to increase the shear capacity and for walls with aspect ratios higher than one, placing GFRP rods in the horizontal direction is more advantageous.

Turco et al. (2006) used GFRP circular bars and GFRP and CFRP rectangular bars with the NSM technique. It was found that the strength and pseudo-ductility can be substantially increased by NSM FRP technique. But the GFRP, in spite of its low elastic modulus, proved to be a better material to strengthen masonry than CFRP. Also the circular FRP bars (low bond) were found to be appropriate for shear strengthening, while rectangular FRP bars have good performance in the case of flexural strengthening. Finally the low bond systems (sand coated FRP bars + cementitious paste, smooth FRP bars + epoxy paste)
were recommended for shear strengthening. They allow some sliding and a better redistribution of the stresses in the system.

### 2.5.2.2 Cyclic loading

Tinazzi and Nanni (2000), Tumialan et al. (2001), Li et al. (2005b) and Corte et al. (2008) used SR technique to investigate the shear behaviour of FRP strengthened masonry wall tested under cyclic loading. Tinazzi and Nanni (2000) tested their walls using the diagonal tension test (ASTM E519-93, 1993). Tumialan et al. (2001) tested their wall by applying diagonal compression load in a closed loop system using a similar test setup as shown in Figure 2.25. In both these studies (Tinazzi and Nanni, 2000 and Tumialan et al., 2001), wall panels were tested by applying loading and unloading cycles diagonally.

![Test setup (Li et al., 2005a)'](image)

Tumialan et al. (2001) tested stack bonded wall panels strengthened with glass FRP rods using the structural repointing technique. Wall panels were built using concrete masonry units (CMU). Three reinforcement arrangements were used to strengthen the wall panels; (i) every horizontal joint reinforced in one side of the wall, (ii) every horizontal and vertical joint reinforced in one side of the wall, and (iii) every horizontal joint in one side and every vertical joint in the other
side of the wall reinforced. It was found that the walls strengthened with same amount of reinforcement, distributed in one or in both faces, behaved similarly, despite a tilt towards the reinforced face of the wall being observed in single side reinforced panels. A 100% wall capacity increase was achieved compared with unreinforced walls.

Li et al. (2005b) conducted an experimental program using six masonry walls specimens, with rectangular cross section and rectangular openings occupying 16% of the wall area. The main retrofitted method Li et al. (2005b) proposed in this study was structural repointing (SR). Five different strengthening configurations; consisted of horizontal and/or vertical GFRP bars and one with EB GFRP laminates on one side of the wall for comparison were used. The horizontal GFRP bars were inserted in the bed joints using structural repointing. The horizontal GFRP bars were inserted into the pre-cut grooves using structural repointing. The FRP reinforcement was installed symmetrically on both faces to prevent tilting or twisting as a result of eccentric stiffness and strength distribution for the strengthened walls. Two cyclic loading histories were used for the study. The test setup was similar to Figure 2.22 used by Mosallam and Banerjee (2011). Li et al. (2005b) found that the load carrying capacity of the URM with openings did not improve by strengthening the spandrels with horizontal GFRP bars. They also found that the vertical reinforcement in the piers significantly increased the stiffness, maximum lateral load and energy dissipation capacity of URM walls.

Two full scale tests were performed by Corte et al. (2008) using a real reinforced concrete building with masonry infill walls. Cyclic push pull tests were carried out. The building was first tested in its original condition (unstrengthened). This resulted in damaging the in-plane masonry infill walls with diagonal tension cracking. Reinforced concrete columns and staircase were also extensively damaged during the first test. Then the damaged building was partially repaired. The damaged external in-plane masonry walls were rebuilt
and strengthened with CFRP strips (1.5 mm thick and 5 mm wide) by structural repointing. The CFRP strips were structurally repointed into every bed joint. The results showed that the CFRP strips were effective in changing the failure mode of masonry panels from dominated diagonal tension cracking to shear sliding mode. However the maximum lateral strength of the partially repaired building was 60% and 50% lower than unstrengthened test (first test) for the pushing and pulling cycles respectively.

2.5.3 Near surface mounted (NSM) FRP technique

Near surface mounted FRP techniques are emerging as an efficient and effective technique to strengthen URM walls against seismic loading. The technique involves attaching thin FRP strips or FRP bars into grooves cut into the surface of a masonry wall. The FRP reinforcement is bonded into the groove using two-part epoxy adhesive. Comparing the EB and NSM techniques, the NSM technique provides several advantages: higher strain development in the FRP before debonding, protection from vandalism and protection to some extent from fire and other environmental influences, and minimal impact on the aesthetics of the structure. FRP bars or FRP strips (rectangular cross section in which the larger dimension is orientated normal to the surface of the wall) can be used to strengthen the shear walls to withstand the earthquake loading. When the FRP bonded into the brick units, the aesthetic impact can be reduced by selecting an epoxy with a colour close to the colour of the bricks. A NSM reinforcement buried a little deeper than the wall surface and a filler material with a similar colour to brick pasted over it may conceal the embedded reinforcement. A possible disadvantage of the NSM technique is the quantity of reinforcement is limited by the thickness of the wall available to cut grooves and by the depth of the groove which can cut into the wall, unlike the EB technique. The deep grooves required to be cut into the wall surface in this technique may cause cracking through the thickness of the wall thus be another disadvantage.
Being relatively a new technique, very few research studies were found in the literature regarding NSM FRP strip strengthening on URM walls. To the author's knowledge, Mahmood and Ingham (2011), Petersen (2009) and Marshall and Sweeney (2002) are the only researchers to investigate the effectiveness of FRP strips in NSM technique on shear strength of masonry walls. Out of these research studies, Petersen (2009) and Mahmood and Ingham (2011) performed studies of URM wall panels retrofitted using NSM CFRP strips and tested under monotonic loading. Marshall and Sweeney (2002) conducted the experimental program under cyclic loading, but this research was more focused on EB technique (Section 2.5.3.2).

2.5.3.1 Monotonic loading

Petersen (2009) tested four URM walls and seven NSM CFRP strip strengthened walls in diagonal tension/shear (ASTM E519–93, 1993). Square wall panels (1.2 m width x 1.2 m length) were constructed from solid clay bricks using the mortar mix ratio of 1:1:6 (cement: lime: sand by volume) in this study. Due to the mortar being retempered (adding water) to improve its workability while constructing the walls, this mortar was categorised as “weak” and “strong” based on its bond strength. The test setup is shown in Figure 2.4(a).

Wall panels were strengthened with CFRP strips in four different arrangements; reinforced with two vertical strips on one side of the wall, reinforced with two vertical strips on each side of the wall, reinforced with two horizontal strips on each side of the wall, reinforced with two vertical strips on one side and two horizontal strips on the other side of the wall as shown in Figure 2.26. As observed in this study, URM walls failed in a brittle manner either by sliding along bed joints (walls with weak mortar) or by diagonal cracking through brick units and mortar joints (walls with strong mortar). The CFRP reinforcement was found to prevent the URM failure modes, increase the ultimate load and ductility of the walls, and also increase the amount of cracks that developed in the walls. It was observed that in some cases vertical reinforcement provided
some dowel action across the sliding joint. Moreover, it was found that the horizontal reinforcement restrained the opening of diagonal cracks and the vertical reinforcement prevented URM sliding failure by restraining the opening of the sliding cracks. Further, the non-symmetrical reinforcement schemes were found to cause out-of-plane deformation. Petersen (2009) concluded that because most URM constructions require strengthening against earthquake loads, future research is needed to be extended to include cyclic loading. Petersen (2009) also developed a FE model to simulate the URM and NSM strengthened walls. The results from this computer model were verified against his experiment results (see section 2.7.2.2).

Figure 2.26: Wall Panel Reinforcement Schemes (Petersen, 2009)

Mahmood and Ingham (2011) conducted an experimental study to investigate the effectiveness of a range of FRP systems for shear strengthening of URM walls. Out of the total of seventeen walls, four were retrofitted with NSM CFRP...
strips and the rest were retrofitted using EB techniques. The wall panels were 225 mm thick. Vintage clay bricks extracted from pre-1964 New Zealand URM buildings were used to construct the panels. The CFRP strips were 1.2 mm thick and 15 mm wide. Vertical and horizontal strip arrangements were investigated with single side and both side retrofitting (walls WTC6, WTC7, WTC8 and WTC9 in Figure 2.14). A modified diagonal tension test setup was used (similar to Figure 2.4b) in this study. The highest strength increase was observed in the walls retrofitted with vertical strips in both sides. Mahmood and Ingham (2011) also concluded that the single faced retrofitting was comparatively ineffective, thus indicating better performance when the NSM application was installed on both faces of the wall panels. The walls strengthened with vertical FRP showed a significant increase in the shear strength highlighting the effectiveness of vertical FRP arrangement. Similar behaviour was observed by Petersen (2009).

2.5.3.2 Cyclic loading

The research study by Marshall and Sweeney (2002) was found to be the only study that has used NSM FRP strips to strengthen masonry walls under cyclic loading. This research study was conducted to identify the in-plane shear performance of masonry walls with CFRP and GFRP strengthening with EB and NSM techniques. Although this experimental program included specimens with NSM strips and rods for strengthening, it was mainly focussed on the effectiveness of different orientations of EB FRP reinforcement. Marshall and Sweeney (2002) had expected the NSM rods and strips to be not effective against the main failure modes such as shear sliding and diagonal cracking. But they had expected the NSM technique to be effective in resisting rocking. Therefore the NSM FRP reinforcement used in their study was vertically aligned near the edges of the wall panels.

Fifty three “I” shaped wall panels with centre pier dimensions of 1.2 m high and 1.2 m wide were tested in this investigation. Out of 53 panels, only 4 panels were tested with NSM technique; two with GFRP rods and two with CFRP strips
using both clay brick and concrete masonry units (CMU) in each case. In this study, glass rods of 6.4 mm diameter and CFRP strips of 2.3 mm thickness, 15.2 mm width were used for NSM strengthening. Two cycles at increasing displacement were applied to the specimens by a horizontal actuator, beginning at 0.5 mm and continuing until the specimen failed. Strengths were reported to have increased approximately 20 kN for clay bricks and 40 kN for CMU with NSM FRP strips. Strengths with Glass rods were observed to increase 20 kN with clay bricks and decrease by 10 kN in CMU. The percentage increase in these values could not be obtained as the authors did not report unreinforced masonry panel strengths.

2.6 Bond behaviour of FRP and masonry

The effectiveness of FRP reinforcement in strengthening URM walls is directly dependent on the integrity of the bond between the reinforcement and the masonry substrate. Therefore, investigation of the bond behaviour of the FRP to masonry interface is vital in assessing the strengthening technique and knowledge of material properties such as the bond strength and debonding strain are essential for the development of analytical and numerical models and to develop design guidelines. Chen and Teng (2001) identified six failure modes related to the bond between FRP and concrete that can be considered also for the masonry to FRP bond. Those are; (1) concrete failure (2) FRP rupture failure (3) Adhesive failure (4) FRP delamination for FRP to concrete joints (5) Concrete to adhesive interface failure (6) FRP to adhesive interface failure. The actual failure can be a mixture of these failure modes. The rupture failure of FRP reinforcement is an adverse failure mode for a strengthening technique due to its very brittle nature and so design aims to avoid FRP rupture. More ductile behaviour is observed when the FRP reinforcement debonds from the masonry in some way. Therefore, for practical design, the tensile behaviour of FRP reinforcement governed by the debonding behaviour of the joint is of primary interest.
Researchers have used different types of tests to investigate the bond behaviour of FRP to masonry such as pull tests and tests on different small assemblages (i.e. couplets, triplets and other different wall assemblages). These are discussed in the following sections.

### 2.6.1 Pull tests

Pull tests are generally used to characterise the bond behaviour of FRP reinforcement to masonry and to concrete. In experimental pull tests the FRP reinforcement (such as EB laminates, NSM strips or bars) is subjected to a direct tensile force (Figure 2.27). The upper face of the test specimen (closest to the FRP loading end) is restrained by a steel plate and tensile force is applied to the exposed end of the FRP reinforcement.

The pull tests are used to determine the interface properties including the bond strength, critical bond length and the local bond slip relationship for the debonding of the reinforcement interface. The bond strength and the critical bond length are generally used in simple analytical models whereas the bond slip relationship is used in finite element models to predict the strength of the masonry walls reinforced with FRP. The bond slip curves can be obtained by three main methods (De Lorenzis and Teng, 2007):

- Approximate it with the curve relating the average bond stress to the loaded-end slip (or the free-end slip, or the average of the two) from specimens with a short bond length.
- Obtain bond stresses and slips from free-end slip and strain measurements at discrete points along the bond length. This method is usually adopted when long bond length specimens are used, as strain gages on a short bond length specimen are likely to significantly affect the bond performance.
- Calibrate the unknown parameters in the local bond–slip equation, whose form needs to be known or assumed in advance, from loaded-end slip and free-end slip measurements.
Willis et al. (2009) and Petersen et al. (2009) conducted pull test series to investigate the bond behaviour of NSM FRP to masonry joints. Willis et al. (2009) also conducted pull tests to investigate the bond behaviour of EB FRP to masonry joints which is discussed later in this section. The pull test specimens with FRP strips aligned in vertical direction (i.e. perpendicular to the mortar bed joint) were used in both investigations (Figure 2.27b). Petersen et al. (2009) additionally used specimens with FRP strips aligned in horizontal direction (parallel to bed joint) (Figure 2.28). Monotonic load was applied to FRP strips in both studies. The experimental parameters investigated by Willis et al. (2009) included; geometric properties (bonded length, thickness and width of FRP strip); location of FRP (relative to perpend joints and cores).

Figure 2.27: Pull test schematics (a) EB technique (Oliveira et al., 2011) (b) NSM technique (Petersen et al., 2009)
The significant differences between the pull test studies to investigate the bond behaviour of NSM FRP to masonry joints by Petersen et al. (2009) and Willis et al. (2009) (NSM technique) are, firstly the tests on cored bricks (Willis et al. 2009) were replaced with solid clay brick units (Petersen et al. 2009) to remove the presence of coring as a variable when assessing the influence of mortar head joints on the FRP-to-masonry joint properties. Secondly a new pull test specimen was developed by Petersen et al. (2009) where the FRP was aligned parallel to the mortar bed joints. This specimen type was used to study bond behaviour of horizontally aligned NSM FRP reinforcement under different pre-compression loads applied perpendicular to the bed joints. Petersen et al. (2009) and Willis et al. (2009) recommended that the vertically aligned NSM FRP reinforcement should not be inserted through head joints in strengthening/retrofitting URM walls. Willis et al. (2009) concluded that the narrow NSM strips are an effective and desirable retrofitting technique due to reduced surface preparation, negligible aesthetic impact and increased bond strength and ductility. Petersen et al. (2009) found that the FRP strips parallel to mortar joint reduced the bond strength. The largest reduction in bond strength they found was 31% compared with vertically aligned FRP strip specimens. Comparing the two studies, Petersen et al. (2009) conclude that the presence of cores in the bricks may have decreased the bond strength.
The following researchers conducted pull tests to investigate the bond behaviour of EB FRP to masonry joints; Willis et al. (2009), Aiello and Sciolti (2006), Camli and Binici (2007) and Oliveira et al. (2011). The test variables investigated in these studies include; surface preparation (needle gun, sander and grinder), bonding agent of bed joints (mortar and quick drying paste), bonded length of FRP, use of anchor schemes for FRP, shape of the substrata (concave and convex surfaces of the brick masonry specimen to represent the interior and exterior surfaces of masonry arches and vaults). All the above tests were conducted under monotonic loading. Considering the NSM and EB technique pull test results, a main conclusions made by Willis et al. (2009) was the mean peak shear stress at failure for NSM is more than double that for EB.

The experimental studies to investigate the bond behaviour of FRP to masonry are comparatively fewer compared with the number of studies on FRP to concrete behaviour. A large database of pull test results exists for FRP-to-concrete joints. Variables that affect the bond behaviour of a FRP-to-concrete joint include (particularly in NSM FRP bond): concrete strength; bonded length; FRP reinforcement cross-section dimensions; different FRP types; distance between the FRP reinforcement and concrete edge; and spacing between multiple parallel FRP reinforcement (Seracino et al. 2007a,b; Oehlers et al. 2008; Rashid et al. 2008). Other than the pull tests, researchers have also used double-shear tests and beam (or bending) tests to investigate the bond behaviour of FRP to concrete joints (Yao et al., 2005, Cruz et al., 2006, Chen and Teng, 2001).

Apart from the above variables, Cruz et al. (2006) and Ko and Sato (2007) investigated the cyclic bond behaviour on FRP to concrete joints. Cruz et al. (2006) used beam pull tests to investigate the bond behaviour between NSM CFRP and concrete under cyclic and monotonic loading. They found that the peak pull out force was not influenced by the cyclic loading. Ko and Sato (2007)
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derived local bond stress-slip relationships for FRP bonded to concrete by testing 54 pull test specimens under monotonic and cyclic loading conditions.

Only one research study investigating the cyclic bond behaviour of FRP to masonry joints was found in the literature, conducted by Hosking et al. (2008). Cored clay bricks were used in this study and NSM FRP technique was used with CFRP strips.

The numerical models developed to investigate the bond behaviour in FRP to concrete (Seracino et al., 2007b; Ali et al., 2008; Cruz and Barros, 2004; Faella et al., 2009) and FRP to masonry (Kashyap et al., 2011) can be found in the literature for both NSM and EB strengthening techniques.

2.6.2 Masonry assemblage tests

Different masonry assemblages have been used by researchers to investigate the composite behaviour of FRP to masonry. Roca and Araiza (2010) used 141 brick shear couplets to investigate the externally bonded FRP to masonry behaviour for strengthening applied across the mortar joint being subjected to combined shear and axial loading. The main variables investigated in this study were (i) type of material used as strengthening (i.e. AFRP, GFRP and plywood) (ii) orientation of the fiber in the case of AFRP and GFRP laminates (iii) number of sides with reinforcement applied (one or two) (iv) the type of loading process (monotonic or cyclic) (v) level of normal stress applied. A specific device was designed which allowed simpler preparation and execution of the tests (Figure 2.29). FRP delamination failure was observed in the majority of the tests and rupture of the fibre was also observed. The study highlighted that the reinforcing laminates applied perpendicular to mortar joints were effective in improving the shear stress in masonry. However the reinforcement parallel to mortar joints can be more effective in brick cracking failure.

Investigations of the shear behaviour of externally bonded FRP to masonry using triplet tests were reported in the literature by Ehsani et al. (1997); Ehsani
and Saadatmanesh (1996); and Luccioni and Rougier (2010) (Figure 2.30). Ehsani et al. (1997) conducted 37 tests and the test variables investigated included the strength of the fabric, the orientation of the fibers, and the anchorage length of the fibers. Two modes of failure were observed; shear failure along the bed joint (with weak FRP laminates) and delamination of fabric at the middle brick region or fabric edges (with strong FRP laminates). The study showed that the strength and the stiffness of the specimens were highly influenced by the fibre orientation. When the fibres were aligned at 45 degrees and 135 degrees a stiffer response was observed with a higher load. When the fibres were aligned at 0 degrees and 90 degrees a more ductile response was observed. That was because for the 45/135 orientation of the fabric, the fibres are subjected to direct tension, where the stiffness of the fibres is largest.

![Schematic view and picture of the equipment](image)

Figure 2.29: Testing apparatus used by (Roca and Araiza, 2010): (a) and (b) actuators, (c) masonry specimen (couplet or triplet), (d), (e) and (f) stiff elements, (g) fixed connection, and (h) free sliding surfaces

Luccioni and Rougier (2010) used triplet specimens to investigate the shear behaviour of brick mortar interfaces retrofitted with externally bonded CFRP laminates. New and damaged triplets were retrofitted and tested/retested in this study. Monotonic and cyclic load cases were used. A numerical model was developed and the different variables such as, CFRP width and length,
orientation of the fibre, pre-compression pressure were investigated using the model. The test specimens failed by debonding of the CFRP laminates (Figure 2.30b) showing improvement in shear capacity and ductility. The researchers found that in the case of initially damaged specimens, the CFRP bands not only recovered the URM strength but also increased their shear capacities.

Hamid et al. (2005) and El-Dakhakhni et al. (2004) investigated the in-plane behaviour of hollow concrete masonry unit URM assemblages strengthened with FRP laminates. Hamid et al. (2005) used 42 unretrofitted and FRP retrofitted specimens consisting of seven different types of masonry assemblage (Figure 2.31). The assemblages were strengthened on both sides with externally bonded GFRP sheets. It was found that the FRP strengthening significantly improved the behaviour for the masonry assemblages which failed by sliding along the mortar joints (assemblages tested in joint shear, diagonal tension, and 30° and 45° off-axis compression). The Assemblages which typically failed in compression mode (0 and 90° assemblages) were the least improved using the FRP laminates in terms of strength. However, instead of the assemblages failing suddenly and totally disintegrating as in unreinforced state, the FRP laminates
in the strengthened assemblages provided stability to the shells of the masonry units after the webs had split.

![Figure 2.31: Different masonry assemblages as parts of a wall (Hamid et al., 2005)](image)

El-Dakhakhni et al. (2004) used a total of 57 test specimens with three different types of assemblages namely: prisms loaded by compression, normal to the bed joints, prisms loaded by compression, parallel to the bed joints and direct shear specimens. The study concluded that the GFRP laminates can provide the required shear strength to the mortar joints to enhance the shear capacity.

### 2.7 Modelling of FRP strengthened masonry

#### 2.7.1 Analytical models for FRP strengthening

Several analytical models have been proposed by researchers for the design of shear walls strengthened with FRP reinforcement for different arrangements. The analysis and design of reinforced masonry in shear is typically based on the assumption that the total contribution to shear capacity ($V$) is given as the sum of two terms, similarly to reinforced concrete. The first term is for the contribution of uncracked masonry ($V_m$) and the second term is for the effect of reinforcement ($V_{FRP}$) (Triantafillou, 1998) (Equation 2.2).
\[ V = V_m + V_{FRP} \]  
(2.2)

\( V_m \) is the in-plane shear strength of unreinforced masonry and the major difference between most of the analytical models is in the FRP contribution \( V_{FRP} \). The strength \( V_{FRP} \) is affected by many factors such as: the strength of FRP, the orientation of FRP, the anchorage length of FRP and more importantly the strain distribution of FRP (Zhuge, 2008a).

### 2.7.1.1 Models for EB FRP to masonry

The following models have been proposed for masonry retrofitted with EB FRP, with different reinforcement arrangements; Triantafillou (1998), Nanni et al. (2003), ElGawady et al. (2006a), ACI 125 (2007), Stratford et al. (2004).

#### Triantafillou model

Triantafillou (1998) developed a model based on truss analogy for externally bonded FRP strips. Considering the Equation 2.2, the shear capacity of FRP reinforced masonry was taken as follows (as per Eurocode 6, 1994):

\[ V = \frac{f_{vk}td}{\gamma_M} + V_{FRP} \leq \frac{0.3f_{vk}td}{\gamma_M} \]  
(2.3)

Where \( V_m = \frac{f_{vk}td}{\gamma_M} \) (Eurocode 6, 1994), \( d \) is the effective depth of the wall = 0.8 x the wall length (\( l \)), \( f_k \) is the characteristic compressive strength of masonry, \( t \) is the thickness of the wall, \( \gamma_M \) is a partial safety factor for masonry and \( f_{vk} \) is the characteristic shear strength of masonry, given as:

\[ f_{vk} = \min\{f_{v0k} + 0.4 \frac{N_{Rd}}{lt}, 0.7 f_{v0k}, 0.7 \max(0.065f_b, f_{v0k})\} \]  
(2.4)

where \( f_{v0k} \) the characteristic shear strength of masonry under zero compressive stress, is between 0.1 and 0.3 MPa in the absence of experimental data, \( f_b \) is the normalized compressive strength of masonry units, \( N_{Rd} \) is the vertical axial
force, and \( f_{lvk,\text{lim}} \) is the limiting value of \( f_{lvk} \) and depends on the type of masonry units and mortar strength.

Due to the difficulty of quantifying the contribution of FRP reinforcement to shear capacity, it was assumed that the shear capacity due to vertical FRP is negligible. Therefore the only shear resistance mechanism left is associated with horizontal laminates and it was modeled in analogy to the action of stirrups in reinforced concrete beams. Equation 2.5 shows the contribution of horizontal FRP to shear capacity in this model.

\[
V_{\text{FRP}} = \rho_h E_{\text{FRP}} \left( \frac{\varepsilon_{\text{FRP},u}}{\gamma_{\text{FRP}}} \right) 0.9d
\]  

(2.5)

Where \( r \) is the reinforcement efficiency factor, depending on the exact FRP failure mechanism (debonding or tensile fracture), \( \rho_h \) is the FRP area fraction in the horizontal direction, \( \varepsilon_{\text{FRP},u} \) is the ultimate tensile strain of FRP and \( \gamma_{\text{FRP}} \) is the partial safety factor for FRP in uni-axial tension (1.15, 1.2 and 1.25 for CFRP, AFRP and GFRP respectively). Thus, Equation 2.5 can be expressed as,

\[
V_{\text{FRP}} = \frac{0.7}{\gamma_{\text{FRP}}} \rho_h E_{\text{FRP}} \varepsilon_{\text{FRP},e} l_t
\]

(2.6)

where \( \varepsilon_{\text{FRP},e} \) is the effective FRP strain, which depends on the area of the FRP-masonry bonded interface. Equation 2.7 shows the adopted \( \varepsilon_{\text{FRP},e} \) expression for strengthened masonry with FRP in shear by Triantafillou (1998).

\[
\varepsilon_{\text{FRP},e} = 0.0119 - 0.0205(\rho_h E_{\text{FRP}}) + 0.0104(\rho_h E_{\text{FRP}})^2
\]

(2.7)

**Triantafillou and Antonopoulos Model**

Triantafillou and Antonopoulos (2000) proposed improved expressions for the effective FRP strain in addition Equation 2.7 to demonstrate different failure modes (FRP debonding or rupture) and types of FRP material (CFRP and AFRP) with reference to concrete.
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Fully wrapped CFRP:

\[ \varepsilon_{FRP,c} = 0.17 \left( \frac{f_{c}^{2/3}}{E_{FRP} \rho_h} \right)^{0.3} \varepsilon_{FRP,u} \] \hspace{1cm} (2.8)

Side or U-shaped CFRP jacket

\[ \varepsilon_{FRP,c} = \min \left[ 0.65 \left( \frac{f_{c}^{2/3}}{E_{FRP} \rho_h} \right)^{0.56} \times 10^{-3}, 0.17 \left( \frac{f_{c}^{2/3}}{E_{FRP} \rho_h} \right)^{0.3} \varepsilon_{FRP,u} \right] \] \hspace{1cm} (2.9)

Fully wrapped AFRP

\[ \varepsilon_{FRP,c} = 0.048 \left( \frac{f_{c}^{2/3}}{E_{FRP} \rho_h} \right)^{0.47} \varepsilon_{FRP,u} \] \hspace{1cm} (2.10)

Where \( f_c \) is the compressive strength of masonry and \( \varepsilon_{FRP,u} \) is the ultimate FRP strain.

Nanni et al. model

Nanni et al. (2003) developed an equation to model the URM wall strengthened with FRP reinforcement based on the truss analogy considering the following assumptions:

- Inclination angle of shear cracks is constant and equal to 45°
- Effective strength is reached in all reinforcement intersected by the diagonal crack
- Compression - shear transfer decreases due to load reversal. It is considered that FRP reinforcement carries all the shear demand

According to the Equation 2.2, \( V_m \) is calculating by existing design codes. The proposed equation for shear strength contribution due to FRP reinforcement is:

\[ V_{FRP} = K_{FRP} \left( \frac{A_{FRP}}{s} \right) f_{y'u} d_v \] \hspace{1cm} (2.11)
Where $A_{FRP}$ is the cross-sectional area of FRP shear reinforcement, $s$ is the spacing of reinforcement, $d_v$ is the actual depth of masonry in the direction of shear considered and $f_u^*$ is the tensile strength of the FRP. The factor $K_{FRP} = 0.5$ is to account for the observed mechanism of failure by assuming the limiting effective stress in FRP reinforcement is equal to half of the ultimate strength.

**AC 125 model**

Following are the equations in the AC 125 (2007) model for rectangular wall sections.

For FRP bonded on both sides:

$$V_{FRP} = 2t_{FRP} f_j H \sin^2 \theta$$  \hspace{1cm} (2.12)

where $f_j = 0.004E_j \leq 0.075 f_{uj}$

For FRP bonded on only one side at an angle ($\theta$) $\geq 75^0$ to the member axis:

$$V_{FRP} = 0.75t_{FRP} f_j H \sin^2 \theta$$  \hspace{1cm} (2.13)

where $f_j = 0.0015E_j \leq 0.075 f_{uj}$, $t_{FRP}$ is the thickness of FRP, $H$ is the depth of the wall parallel to the direction of applied shear force, $\theta$ is the angle of the fibre direction to the member axis and $f_{uj}$ is the ultimate tensile strength of the composite material (MPa).

**Zhao et al. model**

Zhao et al. (2003, 2004) developed models for two different strengthening arrangements of EB FRP with URM based on truss analogy. Here the shear strength ($V_m$) contributed by masonry was computed by the following equation given in Tomazevic (1999).

$$V_m = 0.9 A_w \left( \frac{f_t}{b} \right) \sqrt{\frac{\sigma_0}{f_t}} + 1$$  \hspace{1cm} (2.14)
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Where, \( f_t \) is the tensile strength of masonry, \( b \) is a shear stress distribution factor, 1.2, \( A_w \) is the cross sectional area of the wall and \( \sigma_0 \) is the average compressive stress in the wall due to vertical load. The shear capacity of CFRP sheets in \( \Lambda \)-shaped strengthening is given by (Zhao et al., 2004):

\[
V_{FRP} = \frac{nE_t b_0 \sin \theta + \frac{3}{8} \varepsilon_2 I_0^2}{H} \tag{2.15}
\]

The shear capacity of CFRP sheets in \( X \)-shaped strengthening is given by (Zhao et al., 2003):

\[
V_{FRP} = \frac{n \alpha_{cfs} E_t b_0 \sin \theta + \frac{1}{2} \varepsilon_2 I_0^2}{H} \tag{2.16}
\]

where \( n \) is the number of FRP sheets, \( E \), \( t \) and \( b_0 \) and \( L_0 \) are the modulus of elasticity, thickness, width and the length of CFRP, \( L \) and \( H \) are the length and height of the wall, \( \varepsilon_1 \) and \( \varepsilon_2 \) are the strains in the wrap and weft in the CFRP sheet, \( \theta \) is the angle between the wrap of sheet and horizontal base line, and \( \alpha_{cfs} \) is the shear force coefficient taken as one in this study. Zhao et al. (2003, 2004) had verified both equations above by experimental results.

**Stratford et al. model**

Stratford et al. (2004) proposed an analytical model for masonry shear walls with strengthened FRP sheets covering the entire area of wall. This model was also based on truss analogy and according to the Equation 2.2, the authors derived equations for \( V_{FRP} \). The horizontal stiffness (\( V_{FRP} / \delta \)) of the bonded FRP mechanism was modelled by a truss mechanism shown in Figure 2.32. Also it was assumed that the debonded FRP is linear elastic in this model.

Stratford et al. (2004) proposed that the load carrying by FRP (\( V_{FRP} \)) given by the following equation assumes that the FRP is restrained by fixed anchors.
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\[ V_{FRP} = \frac{w t_F E_{FD}}{l} \delta_1 \]  

(2.17)

where \( w \) is the effective width of the FRP sheet carrying tension, \( t_F \) is the thickness of the FRP sheet, \( E_{FD} \) is the stiffness of FRP in direction of the tensile diagonal, \( \delta_1 \) is the horizontal deflection in URM and \( l \) is the unbonded length.

The FRP also increases the load carried through the masonry, due to confinement by the vertical compression as shown in Figure 2.32. This additional compression is:

\[ N_{FRP} = V_{FRP} \tan \theta \]  

(2.18)

Figure 2.32: Truss mechanism for carrying load through debonded strengthening (Stratford et al., 2004)

By reviewing the analytical models developed for EB FRP strengthening techniques on masonry shear walls, Zhuge (2008a) concluded that the research on FRP strengthening of masonry is limited and most research has concentrated on the assessment of effectiveness of the retrofitting scheme through experimental testing. Further to this, experimental testing has proved that retrofit schemes would make a significant effect on the ultimate strength of the wall. However, the above models do not reflect such difference clearly.
CNR DT 200/2004
Following are the equations proposed by the National Research Council (2004) to calculate the shear capacity of the masonry wall panels strengthened with FRP.

The contribution of the masonry \( V_m \) and the FRP reinforcement \( V_{FRP} \) for the total shear capacity are calculated as follows:

\[
V_m = \frac{1}{\gamma_{Rd}} d f_{vd}
\]

\[ (2.19) \]

\[
V_{FRP} = \frac{1}{\gamma_{Rd}} \frac{0.6 d A_{fw} f_{fd}}{s}
\]

\[ (2.20) \]

where, \( \gamma_{Rd} \) is the partial factor (equal to 1.2 for shear), \( d \) is the distance from extreme compression fiber to centroid of tension reinforcement, \( t \) is the masonry panel thickness, \( f_{vd} \) is the design shear strength of the masonry, \( A_{fw} \) is the area of FRP shear strengthening in the direction parallel to the shear force, \( s \) is the center-to-center spacing of FRP reinforcement measured orthogonally to the direction of the shear force and \( f_{fd} \) is the design strength of FRP reinforcement, defined as the lesser between FRP tensile failure strength and debonding strength.

When debonding involves the first masonry layer and the bond length is equal or longer to the optimal bond length, the design bond strength, \( f_{fdd} \) shall be expressed as follows:

\[
f_{fdd} = \sqrt{\frac{2E_{FRP} \Gamma_F K}{I_{FRP}}}
\]

\[ (2.21) \]

And the corresponding bond strain is:

\[
\varepsilon_{fdd} = \frac{f_{fdd}}{E_{FRP}}
\]

\[ (2.22) \]
where $E_{FRP}$ is the elastic modulus of FRP reinforcement, $t_{FRP}$ is the thickness of FRP, $\Gamma_{FK}$ is the specific fracture energy of the FRP strengthened masonry and is given as:

$$\Gamma_{FK} = c_1 \sqrt{f_k f_{mtm}}$$

(2.23)

where $c_1$ is the experimentally determined coefficient. Unless specific data is available, the value of $c_1$ may be assumed equal to 0.015, $f_k$ is the characteristic compression strength of masonry and $f_{mtm}$ is the masonry average tensile strength and unless specific data is available, it may be assumed equal to $0.10 f_k$.

This model is suitable only if the FRP strengthening is placed parallel to the mortar joints. Therefore the vertical FRP reinforcement arrangement (perpendicular to mortar joints) could not be calculated according to this model.

**ACI4 440.7R-10**

ACI 440.7R-10 (2010) provides the following equations to determine the shear strength of the masonry walls strengthened with FRP. Shear strengths of both the externally bonded (EB) and the near surface mounted (NSM) systems can be determined according to this guideline.

The FRP contribution to the shear strength ($V_{FRP}$) for externally bonded systems:

$$V_{FRP} = p_f W f \frac{d_v}{s}$$

(2.24)

The FRP contribution to the shear strength ($V_{FRP}$) for near surface mounted (NSM) systems:

$$V_{FRP} = p_f d \frac{d}{s}$$

(2.25)
where \( w_f \) and \( s \) respectively represent the width of the FRP reinforcement and the center to center spacing between each strip and \( d_v \) is the length of masonry wall in the direction of shear force. \( p_{fv} \) is the shear force per unit width of FRP laminates which can be evaluated as:

\[
P_{fv} = \begin{cases} 
nt_{FRP} f_{fe} \leq 260 \, N/\text{mm} & \text{for EB FRP systems} \\
A_f f_{fe} \leq 44500 \, N/\text{bar} & \text{for NSM FRP systems} 
\end{cases}
\]  

(2.26)

where \( n \) is the number of plies of FRP laminates with thickness \( t_{FRP} \), \( f_{fe} \) is the effective stress of FRP and \( A_f \) is the cross sectional area of FRP bar.

The effective stress of the FRP is determined as follows:

\[
f_{fe} = \kappa_v E_{FRP} \epsilon_{FRP,u}
\]

(2.27)

Where, \( E_{FRP} \) and \( \epsilon_{FRP,u} \) are the modulus of elasticity of FRP and ultimate shear strain respectively. \( \kappa_v \) is the bond-dependent coefficient for shear-controlled failure modes depends on FRP reinforcement index \( \omega_f \) as follows.

\[
\omega_f = \frac{1}{85} \frac{A_{FRP} E_{FRP}}{A_n \sqrt{f_c}}
\]

(2.28)

\[
\kappa_v = \begin{cases} 
0.4 & \text{for } \omega_f \leq 0.2 \\
0.64 - 1.2 \omega_f & \text{for } 0.2 < \omega_f \leq 0.2 \\
0.6 & \text{for } \omega_f > 0.45 
\end{cases}
\]

(2.29)

where \( A_{FRP} \) and \( A_n \) are the cross-sectional areas of FRP and masonry wall, respectively and \( f_c \) is the masonry compressive strength. The above equations are proposed for masonry walls strengthened with FRP reinforcement placed with a spacing \( s_f \) in the direction of applied in-plane shear. Therefore the total shear capacity of masonry walls strengthened with FRP reinforcement perpendicular to the applied in-plane shear load cannot be determine from this guideline.
2.7.1.2 Models for strengthened with SR technique

Li et al. model

Li et al. (2005a) developed a model based on truss analogy. As explained in above sections, it was assumed that the shear strength of a wall is the sum of the contribution of the unreinforced masonry wall strength and the FRP reinforcement (Equation 2.2). The paper presents the shear capacities due to different failure modes of masonry to account for the shear capacity of masonry wall ($V_m$) in Equation 2.2. Li et al. refers to research by Mann and Muller (1982) and Crisafulli et al. (1995) to obtain these equations. Following are the equations for different failure modes in masonry shear walls.

- Shear capacity due to sliding shear failure along the bed joint ($V_{m,1}$):
  \[
  V_{m,1} = (\tau_0 + \mu \sigma_n) A_n
  \]
  (2.30)

- Shear capacity due to stepped shear sliding failure ($V_{m,2}$):
  \[
  V_{m,2} = (\tau_0^* + \mu^* \sigma_n) A_n
  \]
  (2.31)

Where $\tau_0 = \frac{\tau_0}{1 + 1.5 \mu b / d}$ and $\mu^* = \frac{\mu}{1 + 1.5 \mu b / d}$

- Shear capacity due to stepped diagonal tensile failure ($V_{m,3}$):
  \[
  V_{m,3} = \frac{f_t}{2.3} \sqrt{\frac{\sigma_n}{f_t} + 1} A_n
  \]
  (2.32)

- Compression failure ($V_{m,4}$):
  \[
  V_{m,4} = (f_{m'} + \sigma_n) \frac{2d}{3b} A_n
  \]
  (2.33)

Where $\tau_0$ is the shear bond strength of the mortar joint, $\mu$ is the coefficient of internal friction, $\sigma_n$ is the normal compressive stress on the wall, $A_n$ is the cross sectional area of the masonry wall, $b$ and $d$ are the height and the length of masonry unit, $f_t$ is the tensile strength of masonry, and $f_{m'}$ is the compressive strength of masonry.
Li et al. (2005a) considered the horizontal FRP bars for the shear contribution (Equation 2.35) in this model. From the experimental investigation, they observed that the FRP did not fail either by debonding within the masonry or rupture of the FRP. Therefore they considered that the NSM FRP shear reinforcement is always limited by bond failure between paste and masonry. It was assumed that the bond stress between the paste and masonry is uniform along the effective length of FRP bar (Figure 2.33). The effective length of FRP bar is stated as follows.

\[
L_e = \frac{f_u A_f}{(2D + t_m)\tau_b}
\]  

(2.34)

Where \(f_u\) is the maximum tensile stress of NSM FRP bar, \(\tau_b\) is the average bond strength between paste and masonry, \(A_f\) is the cross sectional area of the FRP bar, \(D\) is the depth of the groove and \(t_m\) is the thickness of the mortar joints.

By considering the diagonal crack propagation, the shear force carried by FRP reinforcement is given as follows by Li et al. (2005a).

\[
V_{FRM} = \tau_b (2D + t_m) \sum_{i=1}^{n} L_i, \ L_i \leq L_e
\]  

(2.35)

Where \(n\) is the total number of bars intersected by the diagonal crack and \(L_i\) is the effective bond length of the \(i\)-th bar intersecting the diagonal crack.
Tumialan et al. model

Based on the truss analogy, Tumialan et al. (2001) developed an analytical model to evaluate the shear strength of URM walls strengthened with SR FRP rods considering the following assumptions.

- Inclination angle of the shear cracks are constant and equal to 45°.
- Constant distribution of bond stresses along the FRP rods at ultimate.
- The ultimate bond strength is reached in all the rods intersected by the crack at ultimate.
- The spacing between rods is the masonry unit layer height.

Two key areas which control the shear contribution due to FRP bars were identified namely, bond controlling region and rupture controlling region (Figure 2.34). Therefore the model was developed by taking the effect of shear capacity in each region as follows:

**Shear in bond controlled region** ($V_b$):

$$V_b = n \pi d_b \tau_b L_t$$  \hspace{1cm} (2.36)

Where $n$ is the number of strengthened sides of the wall (1 or 2), $\tau_b$ is the assumed bond stress, $L_t$ is the sum of bond lengths of all the rods crossed by the crack, calculated in the most unfavourable crack position (minimum total length) and $d_b$ is the diameter of the FRP bar.

**Shear in rupture controlled region** ($V_t$):

$$V_t = nr_t A_i f_{tu}$$  \hspace{1cm} (2.37)

Where $r_t$ is the number of rods in the rupture controlled region, $A_i$ is the cross sectional area of the FRP bars, and $f_{tu}$ is the tensile strength of FRP bar. Finally the shear force resisted by FRP rods was estimated as in Equation 2.38.

$$V_{FRP} = V_b + V_t$$  \hspace{1cm} (2.38)
2. Literature review

2.7.2 Finite element models for FRP strengthening

Researchers for the last decades have investigated the development of sophisticated numerical tools to evaluate the behaviour of masonry buildings against the tradition of rules-of-thumbs or empirical formulae. The finite element formulation is the most common advanced strategy currently used for structural behaviour simulation (Grande et al., 2008). Numerical simulations are necessary and helpful in; assessing existing masonry structures, investigating the effectiveness of strengthening techniques, validating the design of complex masonry structures under complex loading conditions, understanding experimental testing programmes and assisting in the development of design rules (Lourenço, 1996a, 2008).

Modelling masonry is complex due to its composite material behaviour. Masonry consists of units (e.g. clay bricks or concrete blocks) and mortar joints with varied material properties. The unit/mortar interface is identified to have much lower strength than the intact unit or mortar. Therefore the failures initiate and propagate through these weak planes. The overall behaviour of the masonry composite is affected by the properties of the intact materials (units or mortar) and the strength and orientation of the unit/mortar interface (Sutcliffe et al., 2001).

Two different approaches are used to model masonry, depending on the level of simplicity and accuracy desired. They are micro-modelling and macro-modelling (Lourenço, 1996a).
2. Literature review

Micro-modelling

Micro-modelling approaches can be divided into two categories based on the accuracy and the simplicity expected from the model (Lourenço, 1996a). Figure 2.35 demonstrates the individual modelling approaches using sketches. Figure 2.35(a) shows the masonry sample.

- Detailed micro-modelling: here the units and mortar in the joints are represented by continuum elements. The unit-mortar interface is represented by discontinuous elements (Figure 2.35b)
- Simplified micro-modelling: here the behaviour of the mortar joints and unit-mortar interface is lumped into discontinuous elements and the expanded units are represented by the continuum elements (Figure 2.35c)

The Young’s modulus, Poisson’s ratio and optionally, inelastic properties of unit and mortar are used in the detailed micro-modelling approach. The simplified micro-modelling approach does not include the Poisson’s effect of the mortar due to the mortar and the two unit-mortar interfaces being lumped into an average interface while the units are expanded to keep the geometry unchanged. This simplification will reduce the accuracy of the model (Lourenço, 1996a).

The material properties required for the unit and mortar joints in the micro-modelling approach are obtained through experimental tests on the single material components (compression test, tension test, bending test, etc.). More details on characterisation tests and recommendations for material properties can be found in Rots (1997), and Lourenço (1996b).

Five basic failure modes of masonry were identified by Lourenço and Rots (1997) which need to be included in an accurate micro-model for URM. They are (1) direct tensile splitting of the joints, (2) sliding along the bed or head joints under low values of normal stress, (3) direct tensile splitting of the units, (4)
diagonal tensile splitting of the units at values of normal stress sufficient to develop friction in the joints, and (5) compression failure, characterised by splitting of the units in tension as a result of mortar dilatancy at high compression values (Lourenço and Rots, 1997).

**Macro-modelling**

In this approach, a distinction is not made between units and joints but treats masonry as a homogeneous anisotropic continuum. The units, mortar and the unit-mortar interface are smeared out in this continuum (Figure 2.35d).

![Figure 2.35: Modelling strategies for masonry structures (Lourenço, 1996a): (a) masonry sample (b) detailed micro-modelling (c) simplified micro-modelling; (d) macro-modelling](image)

One modelling strategy cannot be preferred over the other because different application fields exist for micro and macro-models. Micro-modelling studies are necessary to give a better understanding about the local behaviour of masonry structures where macro-models can be used when the structure is composed of solid walls with sufficiently large dimensions so that the stresses across or along a macro-length will be essentially uniform. Macro-modelling is more practice oriented due to the reduced time and memory requirements as
well as a user-friendly mesh generation, but it has to compromise between accuracy and efficiency (Lourenço, 1996a).

2.7.2.1 FE models for externally bonded FRP technique

Numerous finite element (FE) models used to analyse the in-plane behaviour of masonry walls strengthened with EB FRP can be found in the literature; Luccioni and Rougier (2011), Grande et al. (2008), Gabor et al. (2005, 2006), Ascione et al. (2005), Van Zijl and Vries (2005), Haroun and Ghoneam (1997), Engebretson et al. (1996).

Luccioni and Rougier (2011) developed a 2D non-linear finite element program to study the behaviour of retrofitted and repaired walls. The results of the FE model were compared with the experimental results. Simulations used triangular plane stress elements with three nodes but, to model the debonding of EB CFRP laminates, some of the simulations were carried out with three dimensional models using tetrahedral elements with four nodes. The modelling showed that the initial stiffness of the model did not change with different retrofitting schemes.

Zhuge (2008b) developed a model with micro-modelling approach based on Distinct Element Method (DEM) to simulate the behaviour of URM walls before and after retrofitting with EB CFRP strips. The material properties of the bond-slip model were obtained from a large experimental database of FRP to concrete test results due to no such data for masonry (Zhuge, 2008b). Here the experimental results and the model results agreed well.

Ascione et al. (2005) simulated a hypothetical model and it was not verified by experimental program. EB FRP strips were used for strengthening in this model and the model accounted for the main phenomena which characterise the limit behaviour of such structures (crushing and fracturing of masonry, interface debonding, stabilising effects by masonry self-weight). An increase in load
carrying capacity due to the FRP strengthening was observed using these models.

Gabor et al. (2005, 2006) used a micro-modelling approach to investigate the behaviour of masonry walls strengthened with EB FRP. The brick units were considered fully elastic and the mortar joints were considered as elasto-plastic material to represent the non-linear behaviour of the brick/mortar interface in shear. They did not consider the debonding failure mechanism in the model. The FE model results and the experimental results were compared with the load displacement behaviour for both unreinforced and strengthened masonry walls. The majority of the wall failure modes in the experiments were able to be reproduced in the model.

Grande et al. (2008) used two models based on macro-modelling approach and homogenised limit analysis approach to investigate the behaviour of unreinforced and FRP strengthened masonry wall panels. Five masonry panels with and without CFRP strip reinforcement (Figure 2.36a) were examined in this study. The wall panels were supported on beams and were loaded externally via steel plates as shown in Figure 2.36(a). In the limit analysis approach perfectly plastic material response was assumed for the masonry, FRP and FRP-masonry interface, in which the softening effects were not considered. They found that the model results and the experimental results compared well for some cases (Figure 2.36b). In the macro-model approach, the damage and the softening were modelled. Truss elements were used for the FRP and they were considered as directly connected to the nodes of the mesh of the URM panels without using interface elements. Without using the interface elements they simplified the model and concluded that the simplified model was in satisfactory agreement with experimental results (Model MRB in Figure 2.36b).

Van Zijl and Vries (2005) FE model was verified by an experimental program with EB CFRP strengthened wall panels. Debonding failure was not observed in both the experimental program and the FE model. Reasonable agreement
between numerically predicted results and experimental results were observed. The FE models developed by Horoun and Ghoneam (1997) and Engebretson et al. (1996) did not consider the debonding failure mechanism in the models. None of the above researchers used cyclic loading for their model simulations.

Figure 2.36: (a) Walls panels with FRP strip arrangement (b) results of PAN-A1 (Grande et al. 2008)
2.7.2.2 FE models for NSM FRP technique

To the author’s knowledge, Petersen (2009) is the only researcher to have developed a FE model to analyse NSM FRP strengthening behaviour of masonry under in-plane loading. All of the wall panel experimental tests were simulated using displacement finite element method and to model the masonry, a simplified micro-modelling approach was used. The commercial FE analysis package DIANA was used for the analyses. The relationship between shear traction and shear relative displacement for the interface element used to model the FRP to masonry bond was defined using local bond slip behaviour of masonry from experimental pull test results. The debonding behaviour of FRP was modelled using this. The modelling strategy used to attach the FRP to the masonry is illustrated in Figure 2.37. Petersen (2009) modelled the diagonal shear test under monotonic loading. The modelling results compared well with experimental results until the failure occurred.

![Figure 2.37: FRP attachment in FE model (Petersen, 2009) (a) NSM FRP reinforcement (b) FE model representation (c) Connection across interface](image)

2.8 Summary

Earthquakes represent a major natural hazard which can cause damage to people and property and result in significant economic loss. Unreinforced masonry (URM) buildings are highly vulnerable to damage during earthquakes, more so than most other types of construction, due to URM’s high mass, limited
ductility and low tensile strength. Despite this, URM is still the most popular method of construction in the world due to many advantages such as remarkable resistance to fire, durability and economical construction. Most of the URM buildings in the world were built before the development of, or without attention to, proper earthquake design guidelines. Consequently, many existing URM buildings do not satisfy modern seismic design guidelines and are in need of strengthening / retrofitting in some way. Also falling into this category is the preservation of valuable historical buildings of which most are masonry constructions.

The key structural elements in a load-bearing masonry structure which resist the lateral loads, such as those induced by earthquakes, are shear walls. Three main failure modes identified in masonry shear walls under earthquake loading are: shear sliding, diagonal cracking and rocking. The main aims of strengthening/retrofitting are to control or prevent these failure modes to improve the strength and/or displacement capacity and hence improve the energy dissipation potential of the shear walls. Researchers have introduced several strengthening techniques, each with distinct advantages and disadvantages. Near surface mounted (NSM) fibre reinforced polymer (FRP) strengthening technique using FRP strips is found to be a relatively new, effective and efficient modern technique to strengthen masonry shear walls. This technique has several distinct advantages compared to other FRP strengthening techniques (for example, externally bonded (EB) technique, structural repointing (SR) using FRP bars). Some advantages are: ability to develop higher strains in the FRP before debonding, protection from vandalism, to some extent from fire and other environmental influences, and minimal impact on the aesthetics of the structure.

In assessing the effectiveness of seismic strengthening measures, it is very important to consider a representative loading pattern to simulate the earthquake effects on masonry walls. In this respect, cyclic loading patterns
were found to be more representative than monotonic loading patterns. Both monotonic and cyclic loading cases have been used by researchers to investigate the behaviour of EB FRP strengthened masonry walls with different FRP arrangements. The boundary conditions imposed on the walls during testing and the pre-compression load while testing were also found to be governing factors affecting the failure modes of shear walls.

The author acknowledges three past research studies on NSM FRP strengthening technique with FRP strips. Petersen (2009) performed a detailed investigation on the behaviour of masonry walls strengthened with NSM FRP strips. The walls here were tested in diagonal tension/shear (ASTM E519–93, 1993) under monotonic loading. Mahmood and Ingham (2011) investigated the effectiveness of the NSM FRP strengthening technique as part of a large experimental investigation on FRP strengthening of URM under shear loading. All the tests were done under monotonic loading using a modified diagonal tension test setup. The results of both studies highlighted that the NSM FRP strengthening technique is an effective strengthening/retrofitting technique to enhance the shear performance of URM walls. Marshall and Sweeney (2002) used cyclic loading to investigate the NSM FRP strengthening. But the latter research mainly focused on EB FRP technique and few details on the NSM technique were reported. Furthermore, the NSM FRP reinforcement arrangements tested by Marshall and Sweeney (2002) were not informative regarding the effectiveness in resisting shear failure modes. This is because the authors had expected that the NSM rods and strips would have no effect in resisting the main failure modes such as shear sliding and diagonal cracking, but rather they expected the NSM technique to be effective in resisting rocking and so the FRP arrangements chosen were limited to resisting this flexural mode.

Most of the available analytical models were developed mainly based on truss analogy and those models were verified using EB FRP strengthened wall panels. No comprehensive rationally based design equations/guidelines were found to
model NSM FRP technique. Several FE models were found in the literature and mainly modelled EB FRP technique. Only one FE model was found to model the behaviour of NSM FRP strengthened walls (Petersen, 2009).

2.9 Research gaps and proposed works

2.9.1 Research gaps

Considering the literature, the use of near surface mounted (NSM) fibre reinforced polymer (FRP) strips to strengthen masonry shear walls was found to be relatively newer, more effective and efficient, and displaying several advantages compared to many competing FRP strengthening techniques. Very few research studies have been conducted to investigate this technique. Because this technique is relatively new, some research gaps were identified which need consideration. Filling these gaps will make the technique more effective and more accessible to practitioners. Following are some of the facts which have triggered the need for future investigation.

Experimental tests on strengthened/retrofitted walls

Experimental studies investigating the behaviour of URM shear walls strengthened with NSM FRP are very few. Petersen (2009) and Mahmood and Ingham (2011) conducted investigations of NSM technique using CFRP strips under monotonic loading. Further investigation under cyclic loading on this technique was recommended by Petersen (2009). Also, the studies by Petersen (2009) and Mahmood and Ingham (2011) were essentially pilot studies, considering only a limited range of reinforcement arrangements and only one wall aspect ratio and only one precompression to shear force ratio (both were diagonal tension tests). Only a single study by Marshall and Sweeney (2002) investigated the cyclic shear behaviour of masonry walls in order to understand the behaviour of shear walls in earthquake conditions. They used cyclic loading to investigate the NSM FRP strengthening but this research mainly focused on EB FRP technique and few details on NSM technique were reported.
Furthermore, they did not use this technique to prevent sliding and diagonal cracking failure of URM walls.

To further develop the NSM FRP strip strengthening technique and to understand its effectiveness and limitations, experimental programs which consider the following aspects are required:

1. Experimental tests on shear walls which simulate appropriate (more realistic) boundary conditions and apply cyclic loading to the test specimens, in order to examine the earthquake behaviour of masonry walls.

2. Comprehensive experimental investigations assessing the effectiveness of a range of different reinforcement schemes, failure modes, and the bond behaviour between masonry and FRP within the wall.

3. Experimental studies on retrofitting of pre-damaged masonry walls are also an important aspect for investigation. These research studies are useful in identifying the effectiveness of the retrofitting technique against earthquake loading for existing damaged buildings. No research study was found in this regard using NSM CFRP strengthening technique.

**Characterisation tests**

Few investigations on the bond behaviour of FRP to masonry can be found in the literature. Petersen et al. (2009) and Willis et al. (2009) conducted pull tests with two different brick types under monotonic loading. But investigation of the cyclic behaviour of FRP to masonry bond was considered by only one research study (Hosking et al., 2008). They used hollow brick units for their study. Further investigations are required to find the cyclic behaviour using different brick types.

**Finite element models**

The existing FE models are described in section 2.7.2. Only one study was done by Petersen (2009) to model the NSM CFRP strengthening technique. The wall
tests done by diagonal tension test setup were simulated in his model. The development of a model which can include more realistic boundary conditions on walls which simulates cyclic loading is required.

**Analytical models**

The existing analytical models are described in section 2.7.1. Those models were developed to simulate strengthening schemes particular to the experimental program on which they were based. Therefore the models cannot be used as a general model for all FRP strengthening schemes. No analytical model for NSM FRP strengthening technique was found in the literature.

**2.9.2 Proposed work**

Considering the above research gaps, this research study was proposed to investigate the in-plane shear behaviour of NSM FRP strengthened/retrofitted URM walls under cyclic loading. Thin rectangular CFRP strips were used in this study. FRP strips were designed to prevent/control sliding along the mortar bed joints within the wall and to prevent/control diagonal cracking through the mortar joints and brick units. FRP strips were not anchored to the footing beams in this study considering the practical application issues.

The targeted objectives in this project were achieved by following the five major phases as given below. More consideration was given to the detailed experimental study.

1. **Investigate the cyclic bond slip behaviour between the NSM CFRP strips and masonry using the pull tests**

Six experimental pull tests were carried out under cyclic loading. The bond strength, critical bond length and the cyclic bond slip behaviour were obtained from these experiments. Furthermore, the relationship between cyclic and monotonic bond slip behaviour was investigated using the experimental pull tests results of Petersen (2009).
2. Experimental program on FRP retrofitted damaged wall panels under in-plane cyclic loading

An experimental program was conducted to investigate the in-plane shear behaviour of damaged URM walls retrofitted with CFRP strips. Sixteen damaged walls were retrofitted and retested with three different reinforcement schemes. Damaged walls were obtained from a previous research study conducted to investigate the effects of a damp proof course (DPC) layer incorporated near the base of the walls under cyclic in-plane shear loading.

3. Development of test apparatus for in-plane shear tests using fixed-fixed boundary conditions

A test setup was developed to impose zero in-plane rotation (fixed-fixed) boundary conditions at the top and bottom edges of the wall specimens. FE modelling was used to investigate the behaviour of this test apparatus and to determine the appropriate dimensions of the actual test apparatus. The dimensions of the wall panels to be tested in the next series of experiments using this test apparatus are also determined using the same FE model. The new test setup was built in the laboratory using the dimensions from the FE model.

4. Experimental program on FRP strengthened undamaged wall panels under in-plane cyclic loading

An experimental investigation was conducted to investigate the in-plane shear behaviour of URM walls strengthened with NSM CFRP strips. Twenty three wall panels with two different aspect ratios were tested in this study. The effectiveness of six different FRP arrangements was investigated. The new test setup built (as mentioned above in point 3) was used for these tests to model the fixed fixed boundary conditions. A FE model was used to develop the new test setup.

5. Develop finite element models to predict the behaviour of NSM FRP retrofitted masonry walls subjected to in-plane cyclic loading
A FE model was used to simulate the cyclic behaviour of NSM FRP strengthened masonry wall panels with fixed fixed boundary conditions. The masonry was modelled using the micro-modelling approach. The FRP was attached to the masonry model using the bond-slip relationships determined from the pull tests. The behaviours of masonry wall panels with two different aspect ratios and different FRP arrangements were modelled.

The development of an analytical model for NSM FRP reinforcing applied to URM shear walls was considered to be outside the scope of this study. However, the proposed work under the current study can provide the understanding of the strengthening system, thereby assisting the future development of such a model.
3.1 Introduction

The investigation of the bond behaviour of NSM CFRP strip to solid clay brick masonry is presented in this chapter. The experimental pull tests were used to characterise the bond behaviour (Figure 3.1).

As described in the literature review in Chapter 2 (Section 2.6.1), two loading patterns; namely monotonic loading and cyclic loading, have been used by researchers for testing pull test specimens (Hosking et al., 2008; Petersen et al., 2009; Willis et al., 2009; Oliveira et al., 2011). Out of all the experimental pull test investigations found in the literature, only one research study has used cyclic loading pattern to investigate the bond behaviour of FRP to masonry joints using cored clay bricks (Hosking et al., 2008; Section 2.6.1 in Chapter 2). In
the current study, cyclic loading was used to investigate the cyclic bond behaviour of FRP to solid clay brick masonry joints.

Six pull test specimens were tested using two different load cases under quasi static cyclic loading. The results presented here include the bond strength, critical bond length and cyclic bond slip behaviour. Furthermore, the degradation due to cyclic loading compared to monotonic loading is also discussed using the results of Petersen et al. (2009).

3.2 Experimental program

3.2.1 Specimen construction and preparation

Six identical pull test specimens, each consisting a four brick high stack bonded prism, were built for this experimental study. A schematic diagram of the pull test specimens is shown in Figure 3.2. All the specimens were built using solid clay bricks with the nominal dimensions of 230 mm x 110 mm x 76 mm (length x width x height). The flexural tensile strength of the bricks was determined from lateral modulus of rupture tests (AS/NZS 4456.15, 2003). More information about this test is given in Section 7.3.1.3 in Chapter 7. The mortar used to construct the specimens was mixed in a ratio of 1:1:6 (cement: lime: sand by volume) and the mortar joint thickness was 10 mm for all specimens. The flexural tensile strength of the mortar joint was determined using the bond wrench test (AS3700-2001). See Section 7.3.1.1 in Chapter 7 for more details about the bond wrench test.

Unidirectional pultruded CFRP strips were used in this study. The cross section of the strips was 2.8 mm x 15 mm (thickness x width). A 50 mm wide and 1.4 mm thick CFRP strip was cut into 15 mm wide strips to make the required width. Two 15 mm wide strips were glued together with ‘Super Strength’ Araldite™ adhesive to make 2.8 mm thick strip. The purpose of gluing two strips together was to achieve a ratio of FRP cross sectional area to bonded
perimeter which ensures debonding failure instead of FRP strip rupture. Petersen et al. (2009) conducted a preliminary test and found that the FRP strip ruptured when one CFRP strip (cross section: 1.4 mm thick by 15 mm wide) was bonded with a similar brick masonry used as in this study. Moreover Petersen et al. (2009) showed that a lower strength FRP (such as glass FRP) would not be suitable in the NSM application because of the likelihood of FRP rupture. The dimensions selected for the CFRP strips were similar to Petersen et al. (2009). He had selected the width of the CFRP to ensure the full embedment in typical Australian modern masonry walls and to limit the possibility of cracking through the thickness of the brick.

![Figure 3.2: Schematic of a pull test specimen](image)

The vertical slots were cut into the brick units using a brick cutting saw. The FRP strips were then glued into the slots with a two-part epoxy adhesive. The cross section of the vertical slots was 20 mm deep and 6 mm wide. For four of the specimens, strain gauges were sandwiched between the two FRP strips to record the axial strain distribution along the bonded length of the FRP strip. Eight strain gauges per strip were attached which were spaced at 42 mm as
shown in Figure 3.3. The strain gauges were sandwiched between two FRP strips to avoid the effect they may have on the epoxy-FRP bonded interface.

The elastic modulus of the FRP strip was determined for each specimen during the pull tests. This was measured using the two strain gauges placed on either side of the unbonded portion of the strip (strain gauges 9 and 10 in Figure 3.3). The specimens were painted in white to allow easier visual detection of cracking during the tests. The material properties are shown in Table 3.1.

### 3.2.2 Test setup and Procedure

Figure 3.4 shows the test setup. The steel base plate of the apparatus was first attached to the bottom of the Instron Universal Testing Machine. This machine was used to apply the quasi static cyclic displacements to the FRP strip. The specimen was then positioned on top of the base plate. A 5 mm thick plywood sheet and a 12 mm thick steel specimen plate were placed on top of the test specimen. Finally, the top steel restraining plate was tied down and horizontally levelled using bolts. Both the specimen plate and the plywood sheet had a small slot cut into the edge to allow the FRP strip to pass through. The plywood sheet was used to ensure full contact between the top of the masonry specimen and the 12 mm steel plate.

To determine the displacement history to be used for the pull tests, consideration was given to the conditions that are likely to be present in a strengthened/retrofitted masonry wall subjected to cyclic in-plane shearing. As a shear wall is displaced laterally in-plane, FRP reinforcement strips in the tension region will be subjected to tensile strain. Depending on the magnitude of the applied displacement, the wall may crack and the FRP may partially debond. When the displacement direction reverses, the FRP strip starts to unload. As the wall returns to its starting position, any cracks will be closed and the FRP will be completely unloaded. If FRP debonding has occurred then the FRP may be subjected to some compression while returning to its starting
3. Bond behaviour of FRP to brick masonry – Pull tests

Figure 3.3: Strain gauge locations for (a) Specimens 1A, 1B, 2A and 2B (b) Specimens 1C and 2C

Table 3.1: Material properties

<table>
<thead>
<tr>
<th>Material</th>
<th>Property</th>
<th>Mean</th>
<th>Standard deviation</th>
<th>Source</th>
</tr>
</thead>
<tbody>
<tr>
<td>Masonry units</td>
<td>Lateral modulus of rupture (MPa)</td>
<td>3.57</td>
<td>0.75</td>
<td>Petersen et al. (2009) (average of 4 specimens)</td>
</tr>
<tr>
<td>Mortar batch</td>
<td>Flexural tensile strength (MPa)</td>
<td>0.52</td>
<td>0.21</td>
<td>AS3700 – 2001 (average of 10 joints)</td>
</tr>
<tr>
<td>CFRP</td>
<td>Elastic modulus (MPa)</td>
<td>207050</td>
<td>7643.62</td>
<td>Current pull test (average of 6 specimens)</td>
</tr>
<tr>
<td>CFRP</td>
<td>Rupture Strain (µε)</td>
<td>12000</td>
<td>-</td>
<td>Manufacturers data</td>
</tr>
<tr>
<td>Epoxy</td>
<td>Flexural strength (MPa)</td>
<td>&gt;30</td>
<td>-</td>
<td>Manufacturers data</td>
</tr>
</tbody>
</table>
position. As the wall travels past its starting position and displaces in the other
direction, the FRP reinforcement will be then located in the compression region
of the wall. In this instance the compressive loads will be shared by the masonry
and FRP in proportion to their relative areas and stiffnesses. Due to
comparatively low area of the FRP, the shear stress transferred between the
masonry and FRP will be negligible. This behaviour will repeat with each full
cycle of displacement until failure occurs. The current pull tests were designed
to simulate such behaviour. Therefore, a displacement history in which the FRP
is subjected to cycles of tensile displacement which returns to zero after each
cycle was considered to be the most representative of the behaviour in a
retrofitted/strengthened wall.

It was expected that returning the FRP displacement to zero after each cycle
would induce some compressive force into the FRP strips. But it was not known
whether the compression would be sufficient to cause buckling of the FRP over
its free length. Therefore, an alternative displacement/load history was also
trialed. The latter used the same cyclic displacement history as the former but
after each cycle the tensile load (rather than tensile displacement) was returned
to zero. In this way, the risk of buckling of the FRP was removed, offering a more
robust test approach for future use if the two approaches were observed to
yield similar overall results.

The adopted quasi static cyclic displacement histories are shown in Figure 3.5.
They were developed in accordance with the recommendations of Park (1989).
The vertical axis in Figure 3.5 represents the displacement of the universal
testing machine grips attached to the free end of the FRP strip. Each complete
cycle was repeated three times at the same amplitude. The displacement
increment in each history was 1.5 mm and that was selected such that each
increment represented approximately 10-15% of the displacement required to
reach the maximum load in a monotonic pull test (Petersen et al., 2009). The
specimens were subjected to increasing cycles of displacement until failure
occurred. After each cycle, the load was returned to zero for Load case 1 (Figure 3.5a) and the displacement was returned to zero for Load case 2 (Figure 3.5b).

Figure 3.4: Experimental test setup
3. Bond behaviour of FRP to brick masonry – Pull tests

(a) Load case 1

(b) Load case 2

Figure 3.5: Time history
The quasi static cyclic displacements were applied with a rate of 0.6 mm/min from the Instron Universal Testing Machine. Three specimens were tested for each load case. The testing machine was computer controlled. The load, machine displacement and the strain distributions were continuously logged using a computer. The nature and extent of the cracking was continuously observed during the tests.

### 3.3 Experimental results

The Experimental results are summarised in Table 3.2. In Table 3.2, the first crack load is the load related to the visual observation of the first crack in each test specimen; Bond strength is the maximum load resisted in the test. Average bond strengths for the two load cases are also given in the table. For Load case 2, the corresponding coefficient of variation (COV, shown in brackets) is also given. Due to specimens 1B and 1C failing by sliding along the epoxy-FRP interface, the bond strengths of these two specimens were not considered for the calculation of average bond strength.

Some specimens failed suddenly when they reached their maximum loads thus the maximum load and the load immediately before failure are equal for those specimens. Other specimens reached their maximum loads and then the load started to drop prior to failure. Failure loads of the latter specimens are lower than their corresponding maximum loads. The majority of the debonded specimens fall into this latter category. Failure load and the failure mode for each test specimen are also given in the Table 3.2. Furthermore, for each specimen, the displacement cycles related to the first crack and the final failure are given in the table.

#### 3.3.1 Failure modes

Four out of six specimens failed by cracking within the masonry close to the masonry/epoxy adhesive interface (denoted as debonding in Table 3.2) as
shown in Figure 3.6a. Of these four specimens, one was of Load case 1 (specimen 1A) and the other three were of Load case 2 (specimens 2A, 2B and 2C). The failure mode for these four specimens was independent from the type of loading.

For the debonding mode of failure minor diagonal cracks first appeared in the masonry adjacent to the FRP strip close to the loaded end (Figure 3.6(b) highlighted with black lines). These minor cracks then extended further down the specimen with increasing displacement until the FRP strips completely debonded. After removing these specimens from the apparatus, cracks were observed extending through the thickness of the specimen, in line with the FRP. These extended cracks were visible on the top (Figure 3.7a) and at the back (Figure 3.7b) of the specimen. This type of cracking most likely results from lateral tensile stresses induced in the prisms by shear induced dilation upon FRP debonding. A similar failure was observed with similar pull test specimens in the monotonic tests by Petersen et al. (2009).

The remaining two specimens (specimens 1B and 1C) failed by sliding between the FRP strip and the epoxy adhesive (denoted as pullout failure in Table 3.2) (Figure 3.8). Some minor cracks were observed in these specimens at the 6mm displacement cycles close to the loading end. However, these cracks did not propagate further prior to the specimens failing suddenly along the epoxy-FRP interface.

### 3.3.2 Elastic modulus of FRP

The elastic modulus of the CFRP ($E_{FRP}$) used in the current study was experimentally determined using normal stress vs. strain graphs plotted for each specimen. Two strain gauges attached to the FRP strip within its unbonded length (strain gauges SG9 and SG10 in Figure 3.3) were used to obtain the strains. These two strain measurements were recorded at the same height, but on opposite sides of the FRP strip and averaged to cancel any bending effect of
Table 3.2: Summary of experimental results

<table>
<thead>
<tr>
<th>Loading pattern</th>
<th>Specimen ID</th>
<th>Load on first crack (kN)</th>
<th>Bond strength (kN)</th>
<th>Load at Failure (kN)</th>
<th>Average Bond strength (kN)</th>
<th>Failre mode</th>
<th>Displacement cycle observation</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>1A</td>
<td>48</td>
<td>61.5</td>
<td>50</td>
<td>61.5</td>
<td>Debonding (Masonry failure)</td>
<td>1st cycle of 6 mm displacement</td>
</tr>
<tr>
<td>Load case 1</td>
<td>1B</td>
<td>35</td>
<td>56.8</td>
<td>56.8</td>
<td>61.5</td>
<td>Pull out (Adhesive failure)</td>
<td>1st cycle of 6 mm displacement</td>
</tr>
<tr>
<td></td>
<td>1C</td>
<td>46</td>
<td>58</td>
<td>58</td>
<td>61.5</td>
<td>Pull out (Adhesive failure)</td>
<td>1st cycle of 6 mm displacement</td>
</tr>
<tr>
<td></td>
<td>2A</td>
<td>30.5</td>
<td>64</td>
<td>63</td>
<td>64.5 (COV 2.8%)</td>
<td>Debonding (Masonry failure)</td>
<td>1st cycle of 4.5 mm displacement</td>
</tr>
<tr>
<td>Load case 2</td>
<td>2B</td>
<td>33</td>
<td>63</td>
<td>58</td>
<td>64.5 (COV 2.8%)</td>
<td>Debonding (Masonry failure)</td>
<td>3rd cycle of 7.5 mm displacement</td>
</tr>
<tr>
<td></td>
<td>2C</td>
<td>45</td>
<td>66.5</td>
<td>66.5</td>
<td>64.5 (COV 2.8%)</td>
<td>Debonding (Masonry failure)</td>
<td>1st cycle of 6 mm displacement</td>
</tr>
</tbody>
</table>
3. Bond behaviour of FRP to brick masonry – Pull tests

Figure 3.6: (a) Debonding failure (b) Initial cracks

Figure 3.7: (a) Cracks at the top of specimen (b) Cracks in the back face of the specimen

Figure 3.8: Pull out failure
the FRP strip in the calculation. Figure 3.9 shows the stress-strain distributions of the two strain gauges for Specimen 2C. The gradient of the average stress vs strain plot gives the $E_{\text{FRP}}$ for the test. Table 3.3 shows the $E_{\text{FRP}}$ values calculated for all specimens. According to the manufacturer’s data, the elastic modulus of CFRP is given as 210000 MPa. Considering the average $E_{\text{FRP}}$ (207050 MPa) obtained from the current tests, the elastic modulus of CFRP obtained from the experiment closely agrees with the manufacturer’s data.

![Figure 3.9: Normal stress-strain distribution for Specimen 2C](image)

**Table 3.3: Elastic modulus of CFRP**

<table>
<thead>
<tr>
<th>Sample ID</th>
<th>$E_{\text{FRP}}$ (MPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1A</td>
<td>216150</td>
</tr>
<tr>
<td>1B</td>
<td>214950</td>
</tr>
<tr>
<td>1C</td>
<td>195450</td>
</tr>
<tr>
<td>2A</td>
<td>206250</td>
</tr>
<tr>
<td>2B</td>
<td>204000</td>
</tr>
<tr>
<td>2C</td>
<td>205500</td>
</tr>
<tr>
<td><strong>Average (E_{FRP}) for CFRP</strong></td>
<td><strong>207050</strong></td>
</tr>
</tbody>
</table>


3. Bond behaviour of FRP to brick masonry – Pull tests

3.3.3 FRP to masonry bond behaviour

The average bond strengths for the two load cases are shown in Table 3.2. The bond strength of Load case 2 showed a 5% increase than Load case 1. For specimens 1A, 1B, 2A and 2B with strain gauges attached along the FRP bonded length, the local slip of FRP relative to masonry was calculated by numerically integrating the strain distributions at increasing load increments up to the failure load. Two assumptions were made in calculating the slip; the axial strain in the masonry is negligible and the slip at the unloaded end is zero. Load versus slip (i.e. relative slip between the FRP and masonry at the loaded end of the specimen) diagrams for all four specimens are shown in Figure 3.10. The diagrams show that, with increasing load the stiffness reduced in all specimens after their first crack. The load-slip behaviour of specimen 1B in Figure 3.10 (b) is different to the other specimens due to its pull out failure through the epoxy-FRP interface. Figure 3.11 shows the comparison of the load-slip envelop curves for the specimens under the two loading cases (except specimen 1B). The load-slip behaviour was reasonably similar for all specimens irrespective of the load case. The load-slip curve for the monotonic test obtained by Petersen et al. (2009) is also plotted in the same graph and the detailed comparison will be discussed under Section 3.3.4.

For strain gauged specimens, the shear stress transferred from the FRP to the masonry through the epoxy was determined from the strain distributions using the following equation (Eq 3.1).

\[
\tau_{avg} = \frac{(\Delta \varepsilon)E_{FRP}b_pt_p}{(\Delta L)(2b_p + t_p)}
\]

Where \(\tau_{avg}\) = average shear stress transferred from the FRP to the masonry through the epoxy over the length \(\Delta L\); \(\Delta \varepsilon\) = change in strain over length \(\Delta L\); \(E_{FRP}\) = Elastic modulus of FRP strip; \(b_p\) = width of strip; \(t_p\) = thickness of strip; and \(\Delta L\) = incremental length along FRP (equal to the strain gauge spacing).
3. Bond behaviour of FRP to brick masonry – Pull tests

(a) 1A

(b) 1B
Figure 3.10: Load-slip curves
Figure 3.11: Comparison of load-slip envelope curves

Figure 3.12 shows the shear stress distribution through the epoxy for increasing increments of load for Specimen 2B. At low levels of load the shear stress is high close to the loaded end but rapidly trails off with distance from the loaded end. As the load is increased more of the bonded length is engaged with significant shear stresses being recorded at greater distances from the loaded end. Eventually (during the 6 mm displacement cycles) the shear stress at the loaded end reaches the shear bond strength. During the next displacement increment (7.5 mm) cracking was observed near the loaded end. The shear stress cannot increase further and the only way that the increasing load can be accommodated is for the shear stresses to transfer (redistribute) further along the bonded length. By the time the maximum load (bond strength) is reached during the 9.0 mm displacement cycles, the shear stress near the bonded end has reduced due to the damage which has occurred to the FRP-masonry bond within that region. At this point in time, a fully developed bell shaped shear stress distribution can be seen and the peak shear stress is observed near the middle of the bonded length. Eventually as the shear stress (and damage) is
redistributed further along the bonded length, the end of the bonded length is reached. No further stress can be transferred between the FRP and masonry. The load which can be resisted drops and the FRP strip debonds from the masonry.

The critical bond length \( L_e \) is the bonded length required to develop full bond strength. This was estimated from the shear stress distribution (at the maximum load) as the distance between the two points: i) where interface cracks are fully developed and the shear stress is approximately equal to zero; and ii) in the uncracked masonry where the shear stress is approximately equal to zero. When the bonded lengths were greater than the critical bond length the bell-shaped shear stress distribution was able to fully develop and propagate away from the loaded end as debonding occurred. For specimen 2B in Figure 3.12, the above two points to calculate \( L_e \) were determined from the curve plotted at 63.5 kN as: 50 mm (i) and 250 mm (ii). Therefore the critical bond length for the specimen 2B is 200 mm. The \( L_e \) values for 1A and 2A are 235 mm. The critical bond length for the specimen 1B was not calculated due to its premature pull out sliding failure.

![Figure 3.12: Shear stress versus distance from loaded end for Specimen 2B](image-url)
For all strain gauged specimens, the bond (shear stress) versus slip (shear displacement) relationship was determined by combining the shear stress and the relative slips along the bonded length for all imposed displacements. The bond-slip curves for all the strain gauged specimens are shown in Figures 3.13 to 3.16.

The bond-slip curves derived for specimen 1B (Figure 3.14) show that the failure of the specimen is due to pull out / sliding. As seen in the figure, it has initially reached a higher shear stress and then dropped to a lower stress (an average of 3.5 MPa). The maximum shear stresses at the positions along the bonded length, where the middle of the strain gauges were attached, reached an average of 9.1 MPa at a very low slip value (less than 0.11 mm slip) and then suddenly dropped. The average maximum shear stresses calculated for Specimens 1A, 2A and 2B are 13.02 MPa, 13.05 MPa and 12.99 MPa respectively. Therefore the reduction in bond strength for the Specimen 1B confirms that the failure had occurred in the epoxy-FRP interface.

![Figure 3.13: Specimen 1A](image-url)
3. Bond behaviour of FRP to brick masonry – Pull tests

**Figure 3.14: Specimen 1B**

**Figure 3.15: Specimen 2A**
As shown in Figures 3.13 to 3.16, negative shear stresses were observed during some parts of the applied displacement history. This was observed even in Load Case 1 in which the load applied to the FRP was at all times tensile. Negative shear stress implies that the axial strain in the FRP is increasing (rather than decreasing) with distance from the loaded end of the specimen (Equation 3.1). This is believed to have resulted from the discrete locations of some strain gauges coinciding with a debonding crack in the specimen and therefore recording a higher strain than an adjacent gauge, closer to the loaded end, at which the FRP is sharing the tensile load with well bonded surrounding masonry. This phenomenon disappears with distance from the loaded end and for higher applied displacements/loads.

For use in finite element modelling and for use in analytical modelling, single idealized bilinear bond slip models were derived for each specimen in this experimental study. The maximum shear stress ($\tau_{\text{max}}$) and corresponding slip ($\delta_1$) were averaged from the experimental bond slip curves. The bond slip curve 10.5 mm from the loaded end was ignored when calculating the maximum shear...
stresses because it was affected by the restraint conditions at the loaded end. A back calculation of Equation 3.2 using the experimentally determined bond strength for $P_{IC}$ was used to determine the final slip ($\delta_{\text{max}}$) instead of estimating the final slip from the bond-slip curves. Equation 3.2 represents the theoretical strength model for a strip bonded to concrete (or masonry). The equation is derived by considering equilibrium and compatibility of a strip-to-concrete joint with a bilinear relationship to model the interface (Seracino et al., 2007b). The bilinear bond-slip parameters for all specimens are shown in Table 3.4 including the monotonic parameters derived by Petersen et al. (2009). It was found that, for each specimen the bilinear bond-slip models developed here reasonably fit with the experimental bond slip data. The bilinear models for each strain gauged specimen are shown in Figure 3.13, 3.15 and 3.16.

$$P_{IC} = \sqrt{\tau_{\text{max}} \delta_{\text{max}}} \sqrt{\frac{L_{\text{per}}}{(EA)_{\text{FRP}}}}$$  (3.2)

Where $P_{IC}$ = the bond strength of the specimen, $L_{\text{per}}$ = bonded perimeter of FRP which was 32.8 mm, $(EA)_{\text{FRP}}$ = axial stiffness of the strip.

<table>
<thead>
<tr>
<th>Load case</th>
<th>Specimen</th>
<th>$\delta_1$ (mm)</th>
<th>$\tau_{\text{max}}$ (MPa)</th>
<th>$\delta_{\text{max}}$ (mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Load case 1</td>
<td>1A</td>
<td>0.19</td>
<td>13.02</td>
<td>1.57</td>
</tr>
<tr>
<td>Load case 2</td>
<td>2A</td>
<td>0.13</td>
<td>13.05</td>
<td>1.82</td>
</tr>
<tr>
<td></td>
<td>2B</td>
<td>0.16</td>
<td>12.99</td>
<td>1.92</td>
</tr>
<tr>
<td>Monotonic</td>
<td>S1-A-SG</td>
<td>0.34</td>
<td>12.2</td>
<td>1.71</td>
</tr>
</tbody>
</table>

The bilinear bond-slip curves for specimens 1A, 2A and 2B and are shown in Figure 3.17. Due to the sliding failure of Specimen 1B, a bilinear bond slip curve was not determined from the test data. The bilinear bond-slip curve for the monotonic test is also plotted in Figure 3.17 for comparison purposes (more details are in Section 3.3.4). It is clear from Figure 3.17 that the bilinear bond-slip curves show very similar relationships independent of the loading pattern. The bilinear bond-slip curve for the quasi-static cyclic loading calculated using...
the average of the results of three specimens (specimens 1A, 2A and 2B) is also shown in Figure 3.17.

![Figure 3.17: Bilinear Bond behaviour under cyclic and monotonic loading](image)

### 3.3.4 Monotonic vs. cyclic bond behaviour

As mentioned previously in this chapter, Petersen et al. (2009) conducted an experimental program on pull tests using similar materials and the same test apparatus as the current study, except under monotonic loading. Petersen et al. (2009) did three similar pull tests to investigate the bond behaviour of FRP to masonry. The average bond strength comparison for monotonic and cyclic loading is shown in Table 3.5. It can be seen that generally there is approximately 20% reduction in the bond strength for cyclic loading compared with monotonic. The failure pattern of a debonded pull test specimen through the FRP-masonry interface for a monotonic test is shown in Figure 3.18 (Petersen et al., 2009). By comparing the cracks observed in the current cyclic test (Figure 3.6a) with Figure 3.18, the masonry specimen was more widely
damaged due to the monotonic test than due to the cyclic test. This can be mainly be attributed to the higher failure load in monotonic tests than the cyclic tests.

Figure 3.18: Debonding cracks for monotonic loading (Petersen, 2009)

The initial stiffness of the cyclic load-slip is higher than the monotonic load-slip as shown in Figure 3.11. The displacement rate applied for the monotonic tests was 0.3 mm/min which is half the displacement rate used in current study. This can be a reason for the low initial stiffness in monotonic test. Another reason is the number of test specimens used for the comparison. Only one monotonic test specimen was used with strain gauges to calculate the slip.

Figure 3.17 shows that the monotonic and cyclic bilinear bond-slip behaviours were very similar irrespective of the loading pattern. However further experiments are needed to confirm this observation. If this observation can be confirmed then a considerable efficiency can be obtained in numerical modelling of the cyclic behaviour of strengthened/retrofitted walls based on monotonic, rather than cyclic pull tests. The former are considerably easier to conduct.
3.4 Summary and conclusions

Six pull tests were conducted to identify the bond behaviour of FRP to clay brick masonry under cyclic loading. The specimens were strengthened with NSM CFRP strips. Two cyclic load cases were investigated in this study namely; the load returned to zero (load case 1) and the displacement returned to zero (load case 2) at the end of each displacement cycle in each displacement history. The bond strengths, critical bond length and the local bond-slip behaviour were determined.

Four of the six pull test specimens failed by cracking in the masonry adjacent to the FRP to masonry interface (debonding) as expected. The other two specimens failed through the epoxy to FRP interface (pull out). For all the specimens failed by debonding, the end failure was similar irrespective of the load case. The damage observed due to cyclic loading was less extensive than for similar monotonically loaded specimens which also debonded through the masonry.

Cracks through the specimen in-line with the FRP strip were observed in the debonded specimens. In the NSM FRP strengthening technique, deep grooves need to be cut into the specimens. The above failure is initiated from the dilation effect of the FRP strips glued to these grooves. This type of cracking will potentially create a plane of weakness in a masonry wall which could adversely affect the overall behaviour of the URM wall. Therefore minimizing the groove depth in NSM CFRP strengthening technique would be a key factor to enhance...
the overall behaviour of the strengthened wall. However, it is believed that such 'through specimen' cracking may not occur in a full size NSM FRP strengthened wall due to the extra confinement afforded by the surrounding masonry in the wall compared to the single brick wide pull test specimens.

The load-slip behaviour of the two load cases is almost identical as shown in figures 3.11 and 3.17. Therefore, returning the displacements to zero at the end of each displacement cycle (Load case 2) is not necessary. Returning the load to zero at the end of each displacement cycle (Load case 1) can be used instead. The advantage of doing this is to avoid buckling the FRP strip should compressive force be induced into the strip while returning to zero displacement.

A single idealized bilinear bond slip model was developed under cyclic loading, for the purpose of using in finite element modelling and in analytical modelling. The values of the bilinear bond-slip parameters developed, the maximum shear stress $\tau_{\text{max}}$, corresponding slip $\delta_1$ and the final slip, $\delta_{\text{max}}$ are 13.02 MPa, 0.16 mm and 1.76 mm respectively.

The bond strength was reduced by about 20% in cyclic pull tests compared with monotonic tests. The bond-slip curves for monotonic and cyclic loading cases are approximately similar. If this observation can be verified with further testing then it may be possible to numerically model cyclic debonding behaviour using bond slip curves established from monotonic tests.
Experimental study on damaged masonry walls retrofitted with FRP

4.1 Introduction

This chapter presents the experimental results of unreinforced masonry (URM) shear wall panels retrofitted with near surface mounted (NSM) carbon fibre reinforced polymer (CFRP) strips. All wall panels were previously tested (prior to retrofitting) under vertical pre-compression combined with increasing reversing cycles of in-plane lateral displacements by Mojsilović et al. (2010) (Figure 4.1).

Figure 4.1: Previously tested wall panels (Mojsilović et al., 2009)

These pre-tested, damaged walls were repaired, retrofitted with NSM FRP strips and retested in this study. The main objective is to assess the effectiveness under cyclic in-plane shear loading of the NSM CFRP strip strengthening
technique on previously tested wall panels with different levels of pre-existing damage.

Twenty one wall panels were previously tested in shear by Mojsilović et al. (2010) to investigate the effects of a damp proof course (DPC) layer incorporated near the base of the walls. Three pre-compression stress levels; 2.8 MPa, 1.4 MPa and 0.7 MPa were used by Mojsilović et al. (2010) for these cyclic shear tests. The wall panels showed various levels of damages during the tests.

A total of sixteen wall panels which could be reused were repaired for the current study. Repaired wall panels were retrofitted with thin CFRP strips using three different retrofitting schemes. Greater emphasis was given to the use of horizontally aligned FRP strips (compared to vertically aligned strips) due to the reduced aesthetic impact of horizontal strips and their ability to better restrain the opening of vertical compression cracks which were dominant in the damaged walls following the previous study. Therefore all three schemes consisted of horizontal CFRP strips for retrofitting. In one of the retrofitting schemes (Scheme 1) vertical FRP strips were also used because these had been shown in a previous study by Petersen et al. (2010) to provide greater increases in strength and ductility than horizontal reinforcement for retrofitted wallets subjected to in-plane shear loading using the diagonal tension test (ASTM E 519-93, 1993). The walls in the current study were tested with cyclic in-plane shear loading similar to the previous study (Mojsilović et al., 2010). Three different pre-compression levels; 2.8 MPa, 2 MPa and 1.4 MPa were used for the tests.

In this chapter, the results of tests on the retrofitted walls are presented including the load-displacement behaviour, crack patterns, ductility behaviour and the energy dissipation. Comparisons are made between the retrofitted wall results and URM results to evaluate the effectiveness of the retrofitting
techniques. Furthermore, the effect of the three different pre-compression levels and three different reinforcing schemes used in this study are discussed.

4.2 Experimental study by Mojsilović et al. (2009, 2010)

Mojsilović et al. (2010) tested 21 URM wall panels under vertical pre-compression combined with cyclic in-plane shear to investigate the effects of a damp proof course (DPC) layer incorporated near the base of the walls. Three different pre-compression levels were used to attempt to cover all possible failure modes such as sliding, diagonal shear and compression (toe crushing) failure. However, only sliding and toe crushing were observed. Two different DPC layer positions, as well as no DPC, were investigated. The DPC was placed either between the first two courses of masonry (Series A) or between the concrete footing beam and first masonry course (Series B). In addition, three control specimens with the same dimensions and without a DPC were tested (Series C). The specimens were first subjected to the vertical pre-compression load which was kept constant during the test and then subjected to increasing reversing cycles of in-plane shear displacement applied via a steel beam at the top of the wall. The apparatus used was the same as that used for the current study and described in Section 4.3.3 below, except that sliding along the base of the wall was not prevented in the Mojsilović et al. (2010) tests. Specimen designation for the Mojsilović et al. (2010) study is summarised in Table 4.1.

Table 4.1: Specimen designation and test program (Mojsilović et al., 2010)

<table>
<thead>
<tr>
<th>Series</th>
<th>Pre-compression stress (MPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>0.7</td>
</tr>
<tr>
<td>A</td>
<td>A3</td>
</tr>
<tr>
<td></td>
<td>(3 specimens)</td>
</tr>
<tr>
<td>B</td>
<td>B3</td>
</tr>
<tr>
<td></td>
<td>(3 specimens)</td>
</tr>
<tr>
<td>C</td>
<td>C3</td>
</tr>
<tr>
<td></td>
<td></td>
</tr>
</tbody>
</table>
Of the 21 panels, 16 panels remained in a state which allowed them to be reused for the current research, albeit having suffered various degrees of damage. The damaged walls were divided into two categories namely highly damaged (HD) and lightly damaged (LD). This distinction was relatively easy to make as it corresponded roughly to the level of pre-compression stress used in the Mojsilović et al. (2010) study. Walls tested at the highest level of pre-compression (2.8 MPa) failed primarily in a compression mode resulting in predominantly vertical cracks distributed throughout the walls in addition to crushing of the masonry at the corners of the walls. These walls were identified as highly damaged (Figure 4.2a). The sliding recorded at the DPC level was negligible for these panels (between -1.5 mm and 0.6 mm on average (Mojsilović et al., 2009). Wall B1-2 (tested under 1.4 MPa pre-compression) also displayed a predominantly compression failure mode with negligible sliding along the bottom (-1.9 mm minimum and 0.7 mm maximum (Mojsilović et al., 2009) (Figure 4.2b) and was also considered to be in the highly damaged category. The remaining walls were classified as lightly damaged. These walls included three walls tested at a pre-compression of 1.4 MPa and six walls tested at 0.7 MPa. The lightly damaged walls (Figure 4.2c) failed predominantly by sliding at the base with some minor corner crushing and so did not display cracking through the centre of the wall typical of the highly damaged specimens.

4.3 Experimental program

In the current study, the sixteen wall panels from the study of Mojsilović et al. (2010) were grouped as shown in Table 4.2. Walls previously tested under pre-compression 2.8 MPa (highly damaged category) were repaired, retrofitted and retested under the same pre-compression stress to assess the effectiveness of the various retrofitting techniques in restoring and/or improving the performance of the walls compared to the previous URM tests. The walls previously tested under pre-compression 0.7 MPa (lightly damaged category) were repaired, retrofitted and retested under 2.8 MPa pre-compression to allow
Figure 4.2: Typical damage following Mojsilović et al. (2009) study (a) Walls tested under 2.8 MPa pre-compression (highly damaged) (b) Panel B1-2 (highly damaged) (c) Walls tested under 0.7 MPa pre-compression (lightly damaged)

comparison with the highly damaged walls with the view to assessing the influence that the degree of damage has on the effectiveness of retrofitting. There was little point to retesting any walls at 0.7 MPa pre-compression as this level of compressive stress was observed by Mojsilović et al. (2010) to result in
sliding and/or rocking failures for URM walls. Therefore, retrofitted walls tested at this compressive stress would also be expected to rock (for all tests in the current study, the specimens were prevented from sliding at the base of the wall as described in section 4.3.3 below) leading to a trivial outcome in terms of the effectiveness of retrofitting.

Wall B1-2 was retested under 1.4 MPa pre-compression after repairing and retrofitting with Scheme 3 (see section 4.3.2 below). Due to its compression failure with negligible sliding in the previous study (Mojsilović et al., 2009), the previous test result for wall B1-2 was used as the URM result for comparison with the retrofitted result at this pre-compression level. The remaining three specimens previously tested under 1.4 MPa pre-compression were repaired and retested under 2.0 MPa pre-compression without retrofitting. These results were used as the URM test results under 2.0 MPa pre-compression. Of these three walls, Wall B1-1 was then retrofitted (Scheme 3) and retested under the same pre-compression of 2.0 MPa.

4.3.1 Test specimens

The nominal dimensions of each panel were 1200 mm x 1200 mm x 110 mm (length x height x thickness) as shown in Figure 4.3. The test specimens were built on reinforced concrete beams with the dimensions of 200 mm x 200 mm x 1600 mm (height x width x length). Extruded clay bricks with nominal dimensions of 230 mm (length) x 110 mm (width) x 76 mm (height) and void area of 25% (vertical coring) were used to build the specimens. The mortar used to construct the specimens was mixed in a ratio of 1:1:6 (cement: lime: sand by volume) and the wall panels were built in running bond. Both the bed and the head joints were 10 mm thick and fully filled. Material properties as obtained by Mojsilović et al. (2009) are shown in Table 4.3.
4.3.2 Specimen repairing and retrofitting

Prior to retrofitting the panels with FRP all the crushed corners were repaired. Any cracks through the wall panels, remote from the corners, were not repaired (Figure 4.4). Most of the crushed corners were repaired using concrete. Heavily damaged sections were replaced with new bricks with 1:1:6 (cement: lime: sand by volume) mortar. The corners with minor cracks were repaired with dental plaster. The average compressive strengths of the concrete, mortar and dental plaster were 16.6 MPa, 7 MPa and 55.2 MPa respectively.

Unidirectional pultruded CFRP strips were used to retrofit the specimens. Three different retrofitting schemes were used (Figure 4.5). The retrofitting scheme used for each wall panel is shown in Table 4.2. These schemes were chosen for the current study based on the following considerations: (i) horizontal FRP strips inserted into the mortar bed joints (used in all three schemes) can be completely concealed and therefore preferred from an aesthetic viewpoint, (ii) schemes with reinforcement on both faces of the wall (Schemes 1 and 2) are structurally more efficient than single side reinforcement (Scheme 3) because eccentricity is avoided. However, Scheme 3 was still investigated because in practice it is often not possible to access both sides of an existing wall, (iii) Scheme 1 trialled the use of vertical FRP strips because these had been shown in a previous study (Petersen et al., 2010) to provide greater increases in strength and ductility than horizontal reinforcement for retrofitted wallets subjected to in-plane shear loading using the diagonal tension test (ASTM E 519-93, 1993).

In all cases, slots were cut into the surface of the panels using a brick cutting saw and the FRP strips were glued into the slots using a two-part epoxy adhesive (Figure 4.6). Strip dimensions were 1200 mm x 10 mm x 1.4 mm for horizontal strips and 1100 mm x 10 mm x 1.4 mm for vertical strips. The 10 mm cross section direction was orientated normal to the surface of the wall. The elastic modulus and the rupture strain of the CFRP were 207050 MPa (Section 3.3.2 in Chapter 3) and 12000 µε (manufacturers’ data) respectively. The
flexural strength of the epoxy adhesive is greater than 30 MPa according to the manufacturers’ data.

Table 4.2: Test specimen matrix for the current study

<table>
<thead>
<tr>
<th>Specimen ID</th>
<th>Pre-compression in URM test (MPa)</th>
<th>Damage level</th>
<th>Pre-compression in Retrofitted test (MPa)</th>
<th>Retrofitting scheme</th>
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4. Experimental study on damaged masonry walls retrofitted with FRP

Figure 4.3: Schematic of the test specimen

Table 4.3: Material properties (Mojsilović et al., 2009)

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<th>Property</th>
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<th>Test method</th>
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</table>
Figure 4.4: Partially repaired wall panel

(a) Scheme 1 - 3 horizontal strips on one side and 3 vertical strips on the other side

(b) Scheme 2 – 6 horizontal strips, 3 on each side

(c) Scheme 3 – 3 horizontal strips on one side only

Figure 4.5: CFRP retrofitting schemes
Experimental study on damaged masonry walls retrofitted with FRP

Figure 4.6: (a) Cutting the slots (b) FRP strips attached to the wall

The horizontal slots were cut into the mortar bed joints with a cross-section of 10 mm deep and 10 mm wide (thickness of mortar bed joint). The vertical slots were cut into the brick units mid-way between the masonry head joints. The cross section of the vertical slots was 10 mm deep and 8 mm wide. For eight of the wall panels, the FRP strips were fitted with strain gauges to record the axial strain distribution along the bonded length of the FRP strips. Six strain gauges were attached per strip. The spacing of the gauges was 171.4 mm in the horizontal strips and 172.1 mm (starting from the top) in the vertical strips. The wall panel specimens were painted white to allow easier visual detection of cracking during the tests.

4.3.3 Test setup and loading arrangement

Figure 4.7 shows the test set-up. The wall panel specimens were first subjected to a vertical compressive stress which was kept constant during the test. The pre-compression load was applied by means of the hydraulic jack (2) placed between the support frame (1) and the upper spreader beam (3). The test specimen (5) together with concrete beam (7) was placed between two spreader beams (3) and (9). The concrete beam (7) was fixed to the lower
spreader beam (9) which in turn lay directly on the laboratory’s strong floor (8). A neoprene plate (6) was placed between the specimen (5) and upper spreader beam (3) to ensure a uniform load contact over the specimen. After applying the vertical pre-compression, the panel specimens were subjected to cyclic shear displacements by means of the hydraulic actuator (4) which was fixed to the reaction wall (10). Steel channel sections (12) were attached to the lower spreader beam (9) to prevent sliding at the bottom of the panel. The spaces between the steel channel sections (12) and the wall panel (5) were packed with heavy steel plates (11).

Ten potentiometers were used in this experimental program in different locations as shown in Figure 4.8. Vertical deformations of the masonry were measured by two potentiometers, POT5 and POT6, which both had gauge lengths of 1115 mm. Diagonal deformations were measured by potentiometers POT3 and POT4, which had gauge lengths of 1453 mm. Slip of the specimen across the second bed joint from the bottom of the panel was measured by POT8 and POT10. This is the first bed joint above the two brick courses which were prevented from sliding. The uplifts of the panels were measured from the fourth brick course from the bottom of the wall by POT7 and POT9. All measuring devices were connected to a computer, which logged the data continuously throughout the tests. The data recording was started after the vertical pre-compression was applied. During the tests, the nature and extent of the cracking was continuously observed.

The application of the shear displacement was computer controlled and was applied in reversing cycles of increasing amplitude. The displacement history was the same as that used by Mojsilović et al. (2010). Each complete cycle was repeated three times at the same amplitude in the form of a sinusoidal wave (Figure 4.9). The displacement rate was set to 1 mm/min for small displacement amplitudes and was gradually increased up to 6 mm/min for higher displacement amplitudes (Table 4.4). The testing was stopped when failure of
the specimen occurred under one direction of shear displacement even if the maximum load had not been reached in the other direction. Failure was defined when either the maximum load showed a rapid rate of decrease or the post peak load dropped by 20% of the maximum load.

### 4.4 Results and Discussion

The experimental results are summarised in Table 4.5. In Table 4.5, the Category ID is as follows: URM = unreinforced masonry specimen, RHD = reinforced, highly damaged specimen, RLD = reinforced, lightly damaged specimen, RD = reinforced damaged specimen and the number indicates the average vertical pre-compression stress in MPa. The results for the categories URM2.8 and URM1.4 were taken from the previous study (Mojsilović et al., 2010). In Table 4.5, maximum horizontal loads \((H_{\text{max}})\) and maximum horizontal actuator displacements \((d_{\text{max}})\) for panel displacements in the East and West directions (Figure 4.7) are given. The maximum displacements were obtained from the load-displacement envelopes developed using each load versus displacement history.

The initial stiffness \((K_{\text{el}})\) values shown in Table 4.5 were calculated by dividing the maximum load \((H_{\text{max}})\) by the yield displacement \((d_y)\) - defined in Section 4.4.2). The stiffness values were consistent for all wall panels in each category (Table 4.5) highlighting the similar damage level in walls within the category. The initial stiffness values for damaged wall panels were less than the URM stiffnesses and the walls categorised as highly damaged (RHD2.8 and RHD1.4) showed the lowest stiffness values. The lower stiffness values compared to URM walls are thought to result from the different damage levels which occurred in the specimens during their previous test. The calculation of ductility factors \((\mu)\) and energy dissipation \((E_d)\) in Table 4.5 are described in the Sections 4.4.2 and 4.4.3 respectively.
4. Experimental study on damaged masonry walls retrofitted with FRP

Figure 4.7: Test setup

Figure 4.8: Potentiometer positions
Figure 4.9: Lateral displacement history used for the Mojsilović et al. (2010) and the current study

Table 4.4: In-plane cyclic displacement history

<table>
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<th>Displacement amplitude (mm)</th>
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### Table 4.5: Summary of experimental results

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<th>Specimen ID</th>
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### Experimental study on damaged masonry walls retrofitted with FRP

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* walls with strain gauged FRP strips  ‡results from Mojsilović et al. (2010)
The horizontal in-plane displacements of the specimens were measured at three locations; the horizontal actuator movement, the movement of the upper spreader beam (POT1) and the movement of the wall at POT2 (Figure 4.7). In all specimens, an average of 4.4 mm displacement drop from the actuator displacement to POT1 and 3 mm displacement drop from POT1 to POT2 were observed when the actuator moved in the East direction. Displacement drops of 2.8 mm from actuator to POT1 and 3.6 mm from POT1 to POT2 were observed in the West direction. The reduction in displacement from actuator to POT1 resulted from elastic strain in the steel members connecting the actuator and upper spreader beam and movements in the connections between the actuator and spreader beam. The displacement drop from POT1 to POT2 can be due to the vertical height difference between the locations of POT1 and POT2 as well as any local deformation in the masonry over this height (Figure 4.7). The displacement results for the current study are presented based on the actuator displacement to allow direct comparison with the results of the previous study reported by Mojsilović et al. (2009).

4.4.1 Load-displacement behaviour

The behaviour of the retrofitted walls compared with unreinforced walls under different pre-compression levels and different pre-existing damage conditions will be discussed in the following sections.

4.4.1.1 Categories URM1.4 and RHD1.4 (Pre-compression 1.4 MPa)

The retrofitted wall B1-2 (category RHD1.4) failed primarily by compression and shear cracking. The existing cracks formed during the URM test (category URM1.4) extended and widened and new cracks developed. At a horizontal actuator displacement of 5 mm the wall started to uplift (rocking failure mode) one brick course up from the base (Figure 4.10a). It was observed that the maximum uplift displacements at both sides of the wall were twice the average uplift displacements recorded during the URM test. Figure 4.10(b) shows the
load displacement curves of the URM and retrofitted walls. The ultimate load of the retrofitted wall was improved by 24% in the East direction and by 6% in the West direction compared to the URM wall. The displacement capacity improved by 165% on average compared to the URM wall.

![Uplifting of retrofitted wall](image1)

Figure 4.10: Specimen B1-2 (a) Uplifting of retrofitted wall (b) load displacement diagrams

The measured strains in the FRP strips showed that each strip reached around 680 $\mu$ε on average and the maximum strain reached was 1000 $\mu$ε (only around
8% of the rupture strain). Despite the relatively low strain, and hence stress, in the FRP, the contribution of the FRP strips in improving the behaviour of the retrofitted wall is clear.

4.4.1.2 Categories URM2 and RD2 (Pre-compression 2 MPa)

Three walls originally tested under 1.4 MPa pre-compression and only lightly damaged in the previous study (Mojsilović et al., 2009) were tested under this category. These walls were first repaired and retested under 2 MPa pre-compression to obtain URM results (category URM2). Only one top corner of each of these three walls had been crushed during the Mojsilović et al. (2009) study. Upon applying the 2.0 MPa pre-compression for the current study, a major compression crack near the middle of each wall was observed. It is believed that the uneven top surface created by the repair of the crushed corner may have resulted in a stress concentration which precipitated the compression crack in each specimen. Upon subsequent application of the cyclic shearing displacement, the URM walls failed primarily by diagonal cracking along with widening of the initial compression crack (Figure 4.11a). One panel (B1-1) was then retrofitted using Scheme 3 and retested under the same pre-compression of 2 MPa. New diagonal and vertical cracks formed and the existing cracks widened during the retrofitted wall test on B1-1 (Figure 4.11b). Figure 4.11(c) shows the load displacement curves of the URM and retrofitted tests for the wall B1-1.

Both the URM and retrofitted tests displayed rocking with uplift in the second course from the bottom of the wall. The retrofitted wall showed 30% to 50% increase in uplift on both sides compared to the URM test. The results show that the ultimate strength of the URM wall was not improved due to retrofitting for this specimen. However, the displacement capacity was improved by 50% in the retrofitted wall B1-1 compared to the URM test. The FRP strain distributions show that the strains were higher closer to the cracks (Figure 4.12). The maximum strains in the three strips were 1525 με, 1644 με and 2551 με.
4. Experimental study on damaged masonry walls retrofitted with FRP

Figure 4.11: Specimen B1-1(a) crack pattern: solid lines show cracking after the URM (2.0 MPa) test, (b) dash lines show additional cracking after the retrofitted test, (c) load displacement diagrams
4.4.1.3 Categories URM2.8, RHD2.8 and RLD2.8 (Pre-compression 2.8 MPa)

All of the retrofitted specimens tested under 2.8 MPa pre-compression failed in a compression mode with predominantly vertical cracking and crushing of the panels at the corners, similar to the URM tests at the same pre-compression level (category URM2.8) (Mojsilović et al., 2009). New vertical and diagonal cracks were also observed, mainly propagating from pre-existing cracks in the case of the six highly damaged panels (category RHD2.8). The lightly damaged walls in category RLD2.8 also displayed compression cracks similar to the highly damaged walls. The typical failure patterns observed for the categories URM2.8, RHD2.8 and RLD2.8 are shown in figures 4.2(a), 4.13(a) and 4.13(b) respectively. For the walls retrofitted using Scheme 3 in both categories, the cracks appeared to be more diagonal on the unreinforced side. For Schemes 1 and 2, all specimens displayed predominantly vertical cracking. In both categories RHD2.8 and RLD2.8, more cracks were observed on the unreinforced (Scheme 3) and vertically reinforced (Scheme 1) sides than on the horizontally
reinforced side of the walls, presumably due to the restraint provided to vertical cracking by the horizontally aligned reinforcement. The FRP strips did not debond or rupture during any of the tests. The FRP strain distributions show that the maximum strains recorded for each category RHD2.8 and RLD2.8 were $3772 \mu \varepsilon$ (specimen C2) and $3606 \mu \varepsilon$ (specimen A3-3) respectively. The FRP strains were observed to be higher at strain gauges located closer to the cracks which were similar to the observation made for wall panel B1-1 as shown in Figure 4.12.

![Figure 4.13: Typical crack patterns (a) Wall specimen B2-1 in Category RHD2.8 (b) Wall specimen C3 in Category RLD2.8](image)

4.4.1.3.1 RHD2.8 versus URM2.8

The walls tested in the Mojsilović et al. (2010) study under a pre-compression stress of 2.8 MPa (category URM2.8 in the current study) were retrofitted and retested to make a direct comparison between URM and retrofitted states for the same specimens under the same pre-compression. The results show that the maximum loads were increased in one direction in the walls C2 and B2-1 of Scheme 1. These two walls recorded maximum loads greater than 160 kN when displaced in their East direction (Table 4.5) which is above the average
maximum URM load of 145.4 kN (excluding A2-3). The maximum loads were not improved due to the retrofitting in the other four walls in Schemes 2 and 3 compared with URM results.

On the other hand, the displacement capacities of all the retrofitted panels were increased in all three reinforcement schemes (except panel A2-3) compared with the URM tests. Retrofitting Scheme 1 showed higher displacement capacities, on average, than the other two schemes. Petersen et al. (2010) also observed better performance using vertical NSM FRP reinforcement than horizontal reinforcement for diagonal tension (in-plane shear) tests conducted in accordance with ASTM E519-93 (1993). However, this was not expected from the current tests, which are dominated by vertical compression cracking and intuitively would be better restrained using a higher percentage of horizontal reinforcement, such as for Scheme 2.

Figure 4.14 shows the load displacement histories for all URM and retrofitted walls in categories URM2.8 and RHD2.8. It can be identified that the stiffnesses of the retrofitted specimens were generally less than the original URM specimens, due to the pre-existing damage. Retrofitting Scheme 1 (a combination of horizontal and vertical reinforcement) showed better results in ultimate load capacity and in displacement capacity than the other two retrofitting schemes. The displacement capacity in Scheme 1 was increased by 33.3% and 100% for the two panels C2 and B2-1 respectively compared with the URM tests and the ultimate shear strengths in the east side of these two panels were increased 12% on average compared with the average maximum load in the URM tests. The better results in Scheme 1 highlight the importance of vertical and horizontal strip arrangements to hold the cracked areas together in damaged masonry walls even under a higher pre-compression stress condition. The CFRP strips did not rupture or debond during any of the RHD2.8 tests.
4. Experimental study on damaged masonry walls retrofitted with FRP

(a) C2

(b) B2-1
4. Experimental study on damaged masonry walls retrofitted with FRP

(c) A2-3

(d) B2-3
4. Experimental study on damaged masonry walls retrofitted with FRP

Figure 4.14: Load displacement curves
4.4.1.3.2 RLD2.8 versus URM2.8

Figure 4.15 compares the category RLD2.8 results with the average of the URM2.8 results. The comparisons show the variations in maximum load and maximum displacement for each RLD2.8 specimen expressed as percentages of the average URM2.8 values (average maximum load = 145.4 kN, average maximum displacement = 13.5 mm). The comparisons exclude the unusually low strength results for Specimens A2-3 (URM2.8) and A3-1 (RLD2.8).

The ultimate loads increased, on average, by 9.3% for the panels retrofitted with Scheme 1 compared with the URM average strength (Figure 4.15a). The specimens tested with retrofitting Schemes 2 and 3 were unable to achieve the average URM ultimate strength. This indicates that retrofitting with FRP strips in a grid pattern (as in Scheme 1) can increase the shear strength of URM walls. This is consistent with the results of Scheme 1 in RHD2.8. Retrofitting only in the horizontal direction seems not effective in terms of strength increase under the large pre-compression stresses tested (2.8 MPa). This is also consistent with the observations in category RHD2.8. The displacement capacities increased for retrofitted panels in Scheme 1 by 67% and Scheme 3 by 48% on average compared with the URM tests (Figure 4.15b). The displacement capacity for wall A3-2 (retrofitted with Scheme 2) was increased by 11%.

Figure 4.16 shows the load displacement curves for retrofitted walls under category RLD2.8. A typical URM load displacement curve (Specimen B2-3) is also plotted in the same diagram for comparison purposes. The results for each retrofitting scheme are compared with average URM results (category URM2.8) in Figure 4.17 using the load displacement envelopes for each specimen. Scheme 1 shows the best performance of the three retrofitting schemes compared with URM results, similar to category RHD2.8. Comparing the wall panels retrofitted with a higher (Scheme 2) and lower (Scheme 3) number of horizontal FRP strips, better performance was recorded for the walls retrofitted with the lower number of strips, which is counter-intuitive. The reason for this result is not
known but was undoubtedly influenced by the unusually low strength panel A3-1 (Scheme 2) leaving only one specimen within this category upon which to assess the performance of Scheme 2.

Figure 4.15: RLD2.8 compared to average URM2.8: Percentage variation in (a) maximum load (b) maximum displacement
4. Experimental study on damaged masonry walls retrofitted with FRP

(a) C3

(b) B3-2
4. Experimental study on damaged masonry walls retrofitted with FRP

(c) A3-1

(d) A3-2
Figure 4.16: Load displacement curves for the wall specimens in category RLD2.8
4. Experimental study on damaged masonry walls retrofitted with FRP

4.4.1.3.3 RHD2.8 versus RLD2.8

Figure 4.18 shows a comparison of the results of retrofitted walls in the two different damage conditions tested in this study. It shows that the behaviour of the walls after retrofitting, with respect to maximum load and displacement capacity, is similar irrespective of the initial level of damage.

4.4.2 Ductility behaviour

The calculated ductility factors ($\mu$) for each test specimen are shown in Table 4.5. Ductility factors were calculated from the load-displacement envelopes developed using each load versus displacement history and were calculated for the displacement direction in which failure, as defined in Section 4.3.3, occurred. Park (1989) defined ductility as the ability of a structure to undergo large amplitude cyclic deformations in the inelastic range without a substantial reduction in strength. Equation 4.1 was used to calculate the ductility factor for each specimen (Park, 1989).
4. Experimental study on damaged masonry walls retrofitted with FRP

(a)

(b)
4. Experimental study on damaged masonry walls retrofitted with FRP

Figure 4.18: Comparison of results for highly damaged and lightly damaged walls (a) Scheme 1 (b) Scheme 2 (c) Scheme 3

\[ \mu = \frac{d_{\text{max}}}{d_{y}} \]  \hspace{1cm} (4.1)

Where \( \mu \) is the available ductility factor, \( d_{y} \) is the yield deformation and \( d_{\text{max}} \) is the maximum available (ultimate) deformation. The yield displacement \( d_{y} \) was determined from the envelope of the horizontal load versus actuator displacement response using the approach illustrated in Figure 4.19(a) (Park, 1989). The maximum available (ultimate) deformation \( d_{\text{max}} \) (Figure 4.19b) was found as the post-peak displacement when the load carrying capacity has undergone a small reduction (Park, 1989). The post-peak displacement here was taken when either, the post peak portion of the load displacement envelope on the failure side dropped by 20% of the maximum load (if the load dropped more than 20%) or the maximum displacement recorded on the failure side if the maximum load dropped less than 20%.
The ductility factors show a scattered distribution for all the walls. For the walls tested under pre-compression load of 1.4 MPa, the URM wall shows higher ductility than the retrofitted wall yet the ultimate displacement capacity was much larger in the retrofitted wall. The calculated ductility factors increased due to the retrofitting in walls B2-1, A2-3, A2-1, but decreased in other walls in category RHD2.8 compared with URM2.8. Schemes 1 and 3 in category RLD2.8 walls showed higher ductility, while Scheme 2 showed lower ductility compared with the average URM2.8 result (average ductility factor=1.8). The ductility factor increased by 35% in the retrofitted wall in category RD2 compared with URM2.

Considering the load displacement curves, the initial stiffness of the retrofitted walls was lower than the original URM specimens due to the pre-existing damage in the retrofitted specimens. Therefore, using the method shown in Figure 4.19(a), $d_y$ would be larger in the retrofitted walls (compared to the URM walls) potentially resulting in smaller ductility factors of the retrofitted walls when applying Equation 4.1, despite the larger ultimate displacement capacities for the retrofitted specimens. The ductility factors calculated in this way can be misleading and must be treated with caution.
4.4.3 Energy dissipation

The energy dissipation for each test is shown in Table 4.5. It was calculated as the area enclosed by the hysteresis envelopes for the load displacement curves for each specimen as shown in Figure 4.20. All of the retrofitted walls (except A2-3) displayed greater energy dissipation (increases ranging from 25% to 370%) compared with unreinforced walls. The greatest increase in energy dissipation was recorded in the retrofitted wall B1-2 (category RHD1.4); the increase (compared with URM) was 370% and resulted from initial rocking failure as well as an increase in ultimate load and displacement capacity compared with the URM test. The energy dissipation for retrofitted wall B1-1 (category RD2) was increased by 70% compared with its URM test.

![Hysteresis envelope for walls B2-1](image)

Figure 4.20: Hysteresis envelope for walls B2-1

It is clear from the results in Table 4.5 that the energy dissipation of the categories RHD2.8 and RLD2.8 were higher than URM2.8 although all the URM and retrofitted walls failed by compression. This highlights the effectiveness of the retrofitting technique even under a higher pre-compression load. The lightly
damaged walls (RLD2.8) dissipated slightly more energy than highly damaged walls (RHD2.8) mainly due to their reduced initial damage level. Schemes 1 and 3 in both categories RLD2.8 and RHD2.8 showed similar energy dissipation, both being higher than Scheme 2, despite Scheme 2 being more heavily reinforced than Scheme 3. This is consistent with the poor performances observed in load-displacement behaviour of Scheme 2 walls described in Section 4.1. Figure 4.21 shows the comparison of percentage increase in the energy dissipation of walls in categories RHD2.8 and RLD2.8 compared with URM results (except walls A2-3 and A3-1). The retrofitting Schemes 1 and 3 show higher percentage increase in energy dissipation than Scheme 2 irrespective of the initial damage level.

Figure 4.21: Percentage increase in energy dissipation of RHD2.8 and RLD2.8 compared with URM
4.5 Conclusions

An experimental study was conducted to identify the effect on shear strength, displacement capacity, ductility and energy dissipation of retrofitting URM shear wall panels with NSM CFRP strips. The walls were tested under vertical pre-compression combined with increasing cycles of lateral shear displacement. Sixteen previously tested URM panels were used in this study. As a result of the previous testing, the walls displayed different damage levels. They were classified into two categories namely, highly damaged and lightly damaged. The wall panels were repaired, retrofitted using three different reinforcing schemes and retested with three different pre-compression stresses namely 1.4 MPa, 2 MPa and 2.8 MPa.

The test results highlighted that the displacement capacities of the walls were increased due to the retrofitting. Also the retrofitted walls displayed greater energy dissipation ability compared with URM walls. The improvements in these two parameters highlighted the effectiveness of the retrofitting techniques. The wall tested under pre-compression load 1.4 MPa showed improvement in the ultimate shear load compared with the URM wall, but in all other tests, generally the ultimate load was not significantly improved, if at all, after retrofitting. The behaviour of highly damaged walls retrofitted with FRP strips were not clearly represented by the ductility factors calculated based on Park (1989) due to the sensitivity of the latter parameter to the estimated yield displacements. Ductility factors computed in this way must be interpreted with caution for damaged walls. It is the view of the author that the important parameters to observe in retrofitted URM performance are the post peak load resistance, displacement capacities and the energy dissipated by the system whilst being cyclically displaced.

All the specimens tested under 2.8 MPa pre-compression (previously tested URM and current retrofitted highly and lightly damaged) failed primarily in
compression modes displaying vertical cracking throughout the panels and corner crushing. Therefore the effectiveness of FRP strengthening, which is used to enhance the tensile strength of masonry, was not clearly represented under this pre-compression level irrespective of the previous damage conditions of the original URM walls. Despite this, the displacement capacities in all panels increased as a result of the retrofitting. The displacement capacities of category RHD2.8 increased from 20% to 100% in all three reinforcement schemes (except panel A2-3). In category RLD2.8 the displacement capacities increased by 67% and by 48% in Schemes 1 and 3 respectively while no clear improvement was observed for Scheme 2. The retrofitting Scheme 1 (combination of horizontal and vertical reinforcement) showed the best results of the three schemes in terms of ultimate load capacity, displacement capacity and energy dissipation for both categories RHD2.8 and RLD2.8. This highlights the importance of vertical and horizontal strip arrangements to hold the cracked areas together in damaged masonry walls even under relatively high pre-compression stresses.

The walls retrofitted using Scheme 2 (three horizontal strips each side) and Scheme 3 (three horizontal strips one side only) in category RHD2.8 showed similar behaviour while in category RLD2.8 the result was counter-intuitive, with the single sided, lightly reinforced (Scheme 3) panels showing better performance than the double sided, more heavily reinforced (Scheme 2) arrangement. The reason for this result is not known but was undoubtedly influenced by the unusually low strength panel A3-1 (Scheme 2) leaving only one specimen within this category upon which to assess the performance of Scheme 2.

The results for the categories RHD2.8 and RLD2.8 highlighted that the retrofitted wall behaviour was not significantly changed due to the existing damage condition of the original URM walls.
The results in this experimental study highlighted the effectiveness of the NSM CFRP technique in improving the performance of URM construction under cyclic in-plane shear loading. However further experiments with reasonable (lower) pre-compression loads need to be conducted for a clear conclusion. The following chapters present the results of experiments using boundary conditions and pre-compression stress levels more representative of shear walls in service.
Finite element modelling and design of test apparatus for in-plane shear using fixed-fixed boundary conditions

5.1 Introduction

In this chapter, the development of the experimental test setup and the test parameters to be used in the next phase of the experimental program (described in Chapter 6) are discussed. These investigations were carried out using finite element (FE) modelling. As described in the literature (Chapter 2), different types of experimental test setups have been used by researchers to investigate the in-plane shear behaviour of masonry walls.

Figure 5.1: Test setup developed to simulate in-plane shear loading under fixed-fixed boundary conditions
Experimental test setups that can maintain fixed-fixed boundary conditions were found to be more representative (compared to cantilever type boundary conditions) for simulating the earthquake behaviour of shear walls in real buildings. Therefore, for the current research a test setup was developed to impose zero in-plane rotation (fixed-fixed) boundary conditions at the top and bottom edges of the wall specimens (Figure 5.1). This setup is based on one of several setups discussed by Bosiljkov et al. (2008).

In this test setup, pin-ended steel members (truss members) are included between the reaction frame and the steel loading beam attached to the top of the masonry wall specimen (Figure 5.1). The truss members allow the beam to move freely in the horizontal and vertical directions, but at all times geometrically constrain the loading beam to remain horizontal. This imposes a zero in-plane rotation boundary condition to the top edge of the wall. As the bottom edge of the wall sits on a reinforced concrete footing beam which also cannot rotate in the plane of the wall, then the desired “fixed-fixed” boundary conditions are achieved. As the loading beam is displaced horizontally, axial forces are induced in the truss members. The forces in the truss members at each end of the loading beam are at all times equal in magnitude and opposite in sign (one in compression and one in tension). Therefore, there are no net changes to the total vertical pre-compression force or horizontal shear load acting on the wall but a couple is imposed on the loading beam which enforces the zero rotation constraint.

FE modelling of this setup was also conducted with varying pre-compression loads applied externally to the masonry wall. This was done to determine the level of vertical pre-compression required to induce diagonal cracking failure in the experimental test specimens. The FRP reinforcing methods studied in this research are designed to enhance the performance of URM walls which fail by diagonal cracking or bed joint sliding within the height of the wall and so an experimental program designed to assess the effectiveness of the FRP
strengthening schemes is most informative if such “through wall” failure modes are observed. The next phase of the experimental investigation will use two wall aspect (height to length) ratios; aspect ratio 0.5 and 1. Therefore, the FE modelling reported herein was also used to determine suitable dimensions for the wall panel specimens for each aspect ratio such that the specimens were as representative as possible of full size walls, whilst still having in-plane shear capacities within the capacities of the available laboratory equipment and facilities.

5.2 FE modelling

5.2.1 Modelling the URM walls and test setup

The FE modelling for this study was completed in two phases. The first phase was the development of the test setup to simulate fixed-fixed boundary conditions (details in section 5.3). The initial FE model for this study was developed based on trial dimensions for the masonry wall and for the test setup. The wall dimensions were selected to achieve an aspect ratio 0.5 (AR05). Based on these results further models were developed until appropriate dimensions for the test apparatus and for the AR05 test walls were determined. The second phase was the determination of the optimum pre-compression level to induce the diagonal cracking failure mode in wall panels (details in section 5.4). Walls of aspect ratio 1 (AR1) and 0.5 (AR05), with and without FRP strengthening, were modelled under this second phase. The commercially available software package DIANA 9.2 (de Witte, 2007) was used for all FE analyses.

The simplified micro-modelling approach was adopted for the above masonry models. The brick units in the masonry wall were modelled with continuum elements that were expanded to maintain the actual dimensions of the wall panel. Brick units were modelled using eight-node quadratic, rectangular plane stress elements with a thickness of 110 mm. Each half brick was divided into two elements across the length and two elements over the height as shown in
the Figure 5.2(b). Zero thickness interface elements were used at the mid-length of the brick unit to model the potential crack through the middle of the brick. The brick unit elements were modelled elastically in the FE model and the non-linear behaviour was modelled in the interface elements. The mortar joints and the mortar/brick interfaces were lumped into a zero thickness interface elements. These interface elements are discontinuous elements that relate the interface stresses to interface relative displacements. Figure 5.2 shows the masonry assemblage and its representative FE model with the different element types. Note that the interface elements here are shown with a thickness for illustrative purposes. The interface elements for the mortar joints and the potential brick crack were modelled with six-node quadratic rectangular interface elements. These elements were modelled with 110 mm thickness.

![](image)

(a) Masonry assemblage  
(b) FE model representation

Figure 5.2: Masonry assemblage and FE model with micro-modelling approach

The steel loading beam in the test setup was modelled using eight-node quadratic, rectangular plane stress elements. The element thickness was calculated to match the second moment of area (I value) in the FE model with that of the steel “I” section used as the loading beam in the experimental program. The width of the plane stress elements representing the steel loading beam were equal to the brick unit element size in order to match with the FE mesh of the masonry (Figure 5.3). The finite elements representing the steel loading beam were connected to each other by six-node quadratic rectangular
interface elements of the same thickness. The brick units and the steel beam were connected with six-node quadratic rectangular interface elements. The thickness of these brick-steel beam interface elements was equal to the steel beam element thickness. The steel beam elements were modelled elastically in the FE model. The pin ended steel members ("steel truss members") used to impose zero rotation (fixed-fixed) conditions to the steel loading beam were modelled using two node truss elements and were modelled elastically.

Figure 5.4 shows the finite element model developed for the masonry wall panel with AR05 along with that part of the test setup which is designed to move during testing. Note that this model assumes that the reaction frame shown in Figure 5.1 imposes perfectly rigid restraint to the modelled portion of the apparatus. The nodes at the bottom most mortar bed joint were restrained against movements in both x and y directions to simulate the fixed boundary condition at the bottom of the wall. The top most ends of the truss elements were restrained against movement in the x and y directions.

![Figure 5.3: FE model for test setup loading beam](image)

The self-weights of the steel beam and the masonry wall were modelled using in-plane pressure loads applied to the surfaces of the beam and masonry wall respectively. The pre-compression load was applied to the middle of the loading beam as shown in Figure 5.4. The horizontal load was applied at the middle of the beam (Figure 5.4) by displacing the beam in increments. To solve the nonlinear problem, an incremental-iterative solution procedure was used. The
5. Finite element modelling and design of test apparatus for in-plane shear using fixed-fixed boundary conditions.

Displacement was increased by 0.01 mm in each increment and a linear iteration scheme was used to solve for equilibrium at each increment.

5.2.2 Masonry interface element behaviour

The Combined Cracking-Shearing-Crushing interface material model in DIANA 9.2 (de Witte, 2007) was used to model the non-linear behaviour of the mortar joints in masonry. The Combined Cracking-Shearing-Crushing interface material model was developed by Lourenço and Rots (1997) and Van Zijl (2004). This model is based on multi-surface plasticity, including a Coulomb friction model, a tension cut-off model and an elliptical compression cap model (Figure 5.5). Figure 5.6 shows the main failure mechanisms that are required to be included in an accurate masonry model (Lourenço and Rots, 1997) namely; (a) joint cracking (b) joint slip at lower normal compression stresses (c) unit diagonal tension cracking due to the sufficient normal compression stress values to develop friction in the joint (d) masonry crushing due to high level of normal compression stresses (e) masonry unit cracking due to the direct tension. All these failure mechanisms except (e) were included in this mortar joint interface material model.

Figure 5.4: FE model with the test setup details
5. Finite element modelling and design of test apparatus for in-plane shear using fixed-fixed boundary conditions.

The Coulomb friction criterion (DIANA user’s manual, de Witte (2007)):

\[ f = \tau + c \sigma - \phi \] (5.1)

where \( \tau \) is the shear stress, \( c \) is the cohesion, \( \phi \) is the internal friction coefficient and \( \sigma \) is the normal stress. The sliding of the joint is captured by the model and the sliding occurs when the failure criterion is reached \((f \geq 0)\). Softening acts for both the cohesion \((c)\) (adhesion softening) and the internal friction coefficient \((\phi)\) (frictional softening). The rate of adhesion softening and frictional softening are dependent on the shear fracture energy \((G_f^{II})\), which is the area under the shear stress-shear displacement curve. The cohesion softens from the initial value \((c_0)\) to zero and internal friction coefficient softens from the initial value \((\phi_i)\) to a residual value \((\phi_r)\).

The tension cut-off model (DIANA user’s manual, de Witte (2007)):

\[ f = \sigma_n - \sigma_t \] (5.2)

where \( \sigma_t \) is the tensile strength (brick-mortar bond strength) and \( \sigma_n \) is the tensile stress applied normal to the joint. The joint cracking due to the direct tensile stresses occurs when the failure criterion is reached \((f \geq 0)\). The tensile strength softens exponentially from an initial value \((f_t)\) to zero and the softening depends on the tensile fracture energy \((G_f^{I})\).

The compression cap model (DIANA user’s manual, de Witte (2007)):

\[ f = \sigma_n^2 + C_s \tau^2 - \sigma_c^2 \] (5.3)

where \( \sigma_c \) is the compressive strength and the \( C_s \) is a parameter controlling the shear stress contribution to failure. The parameter \( C_s \) is taken equal to 9 (Lourenço, 1996b). This model captures diagonal cracking through the brick and failure occurs when the failure criterion is reached \((f \geq 0)\). The compressive strength is described by a parabolic hardening rule up to its peak strength \((f_c)\) at
the plastic strain $k_p$ and then described by parabolic/exponential softening (Figure 5.7). The parabolic/exponential softening depends on the fracture energy $G_c$ as shown in Figure 5.7.

The normal uplift due to shear-slipping (dilatancy) is included in the Combined Cracking-Shearing-Crushing interface material model and the dilatancy is modelled by the following equations.

$$U_p = \begin{cases} 
0 & \text{if } \sigma_n > \sigma_u \\
\frac{\psi_0}{\delta} \left(1 - \frac{\sigma_n}{\sigma_u}\right)e^{-\delta N_p} & \text{if } \sigma_u \leq \sigma_n < 0 \\
\frac{\psi_0}{\delta} e^{-\delta N_p} & \text{if } \sigma_u \geq 0 
\end{cases} \quad \text{(5.4)}$$

Where $\psi_0$ is the dilatancy gradient when the normal confining stress $\sigma_n$ is zero, $\sigma_u$ is the compressive stress at which the dilatancy becomes zero, $\delta$ is a dilatancy shear-slip degradation coefficient, $U_p$ is the plastic component of the normal displacement and $\nu_p$ is the plastic component of the shear displacement.

The potential brick crack interface elements were modelled using a linear tension softening model. Tensile cracking failure criterion was assigned to the interface elements but no failure criteria in shear and compression were given. The interface elements were assumed to have zero shear resistance after tensile cracking occurred. The interface elements were modelled with high normal and shear stiffness values ($k_n$ and $k_s$) ($10^6$) to ensure the continuity of brick displacement across the interface elements. This avoids frictional sliding along the potential crack line.

Masonry characterisation tests were used to determine the properties for the interface element non-linear models for the masonry used in the experimental program described in Chapter 6. These characterisation tests are described in section 7.3.1 in Chapter 7.
5. Finite element modelling and design of test apparatus for in-plane shear using fixed-fixed boundary conditions.

Figure 5.5: Interface cap model used in the Combined Cracking-Shearing-Crushing interface material model (Lourenço and Rots, 1997)

Figure 5.6: Failure mechanisms (Lourenço and Rots, 1997)

Figure 5.7: Compression behaviour of the Combined Cracking-Shearing-Crushing interface material model (DIANA user's manual, de Witte, 2005 and Lourenço, 1996b)
5.2.3 Modelling the FRP strengthened walls

FE models for the FRP strengthened walls with aspect ratios 0.5 and 1 were developed by attaching elements representing four CFRP strips to the masonry wall in order to evaluate the required pre-compression load for use in the experimental program. Figure 5.8 shows the FE model representation of the NSM FRP strengthening technique in brick masonry walls. The FRP strips were modelled using two-noded linear truss elements. FRP strips with cross sectional area of 21 mm$^2$ and cross sectional dimensions of 15 mm x 1.4 mm (width x thickness) were used in this study. The elastic modulus of FRP was 207050 MPa (Chapter 3). The Poisson’s ratio was assumed as 0.3 (Hussain et al., 2008).

The FRP truss elements were attached to the brick units via zero-thickness six-noded quadratic interface elements similar to the FRP strengthened FE model developed by Petersen (2009) (Figure 5.8b). The bonded area of the FRP strip in the model is equal to the length times the plane stress thickness of the interface element. In the actual experiments, this value is equal to the length of the strip times the bonded perimeter. The bonded perimeter value of 31.4 mm ($15 \times 2 + 1.4$) was used as the plane stress thickness of the interface element. The plane stress thickness direction of this element is shown in Figure 5.8b. The relationship between the shear stress and the shear relative displacement (or slip) of the interface element in the longitudinal/tensile direction of the FRP reinforcement was defined by the local bond-slip relationship. The debonding failure was introduced to the model by providing the bond-slip behaviour of the FRP to masonry interface. The bond-slip relationship used in this model was determined using the equations developed by Seracino et al. (2007b) for the FRP strips with the cross section of 15 mm x 1.4 mm (width x thickness) (Figure 5.9).

The FRP truss elements across each mortar bed joint were connected with zero-thickness node interface elements as shown in Figure 5.8c. A high stiffness was given to these node interface elements in the longitudinal direction to make the
FRP truss elements continuous across the joints. The dowel relationship in the transverse direction as recommended by Petersen (2009) was adopted in the model.

Figure 5.8: FRP strengthening in FE model (Petersen, 2009)

Figure 5.9: Bond-slip model
5. Finite element modelling and design of test apparatus for in-plane shear using fixed-fixed boundary conditions.

5.2.4 Material properties

The material properties of the masonry used by Petersen (2009) in his FE model were used in the current study. Similar type of brick units and similar mortar type used by Petersen (2009) were planned to be used in the main experimental program described in Chapter 6. The material properties for each material used in the FE model are shown in Table 5.1 and Table 5.2.

Table 5.1: Material properties of brick unit elements

<table>
<thead>
<tr>
<th>Property</th>
<th>Unit</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Elastic modulus ($E_{\text{unit}}$)</td>
<td>MPa</td>
<td>32580.7</td>
</tr>
<tr>
<td>Poisson’s ratio ($\nu$)</td>
<td>-</td>
<td>0.2</td>
</tr>
</tbody>
</table>

Table 5.2: Material properties mortar-joint interface elements

<table>
<thead>
<tr>
<th>Property</th>
<th>Unit</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Normal elastic stiffness ($k_n$)</td>
<td>N/mm$^3$</td>
<td>1033.48</td>
</tr>
<tr>
<td>Shear elastic stiffness ($k_s$)</td>
<td>N/mm$^3$</td>
<td>430.62</td>
</tr>
<tr>
<td>Direct tensile strength ($f_t$)</td>
<td>MPa</td>
<td>0.77</td>
</tr>
<tr>
<td>Tensile fracture energy ($G_f^t$)</td>
<td>N/mm</td>
<td>0.009</td>
</tr>
<tr>
<td>Cohesion ($c_0$)</td>
<td>MPa</td>
<td>0.664</td>
</tr>
<tr>
<td>Initial friction ($\Phi_i$)</td>
<td>-</td>
<td>0.900</td>
</tr>
<tr>
<td>Initial dilatancy coefficient ($\Psi_0$)</td>
<td>-</td>
<td>0.614</td>
</tr>
<tr>
<td>Residual friction ($\Phi_r$)</td>
<td>-</td>
<td>0.753</td>
</tr>
<tr>
<td>Stress at which the dilatancy is zero ($\sigma_u$)</td>
<td>MPa</td>
<td>-1.347</td>
</tr>
<tr>
<td>Degradation coefficient ($\phi$)</td>
<td>-</td>
<td>2.028</td>
</tr>
<tr>
<td>Compressive stress ($f_c$)</td>
<td>MPa</td>
<td>21.26</td>
</tr>
<tr>
<td>Shear traction control factor ($C_S$)</td>
<td>-</td>
<td>9.00</td>
</tr>
<tr>
<td>Compressive fracture energy ($G_C$)</td>
<td>N/mm</td>
<td>22.51</td>
</tr>
<tr>
<td>Equivalent plastic relative displacement ($\kappa_p$)</td>
<td>mm</td>
<td>0.034</td>
</tr>
<tr>
<td>Shear fracture energy factor a ($G_f^a$)</td>
<td>N/mm</td>
<td>-0.80</td>
</tr>
<tr>
<td>Shear fracture energy factor b ($G_f^b$)</td>
<td>N/mm</td>
<td>0.050</td>
</tr>
</tbody>
</table>
5.3 Development of test setup

5.3.1 Test setup – FE model

The development of the test setup was based on a sketch from Bosiljkov et al. (2008) (Figure 5.10). The assumed initial dimensions of the test setup and the wall panels are shown in Figure 5.11. The steel loading beam used to make the FE model was initially chosen as a 310UB46.2 (model M1). The cross sectional dimensions of the steel truss members in the model were 45 mm x 75 mm (width x height). The lengths of all inclined truss members were 350 mm each (Figure 5.11). This was selected to provide adequate space between the loading beam and the reaction frame (level with the top of the truss members) to accommodate the pre-compression jack in the actual tests.

Different models were executed to identify the precise dimensions for the test setup. The results for each model are summarised in Table 5.3. A constant pre-compression stress of 1 MPa was applied to all the models. For the model assigned with a realistic elastic modulus \( E \) (200000 MPa) for the steel sections in the test apparatus (model M1_1) sliding failure occurred at the base of the masonry wall (Figure 5.12). The main cause for sliding here was low stiffness of the steel loading beam. An elastic beam curvature was observed during the model simulation (deflection/span = 1/4400) (Figure 5.12). Instead of changing the steel section dimensions for each finite element model simulation, the elastic modulus of the steel section was simply increased. The intention was to obtain the optimum \( E \) value of the steel (to cause diagonal cracking failures in the masonry) and then convert that into matching sectional properties for the steel loading beam to be used in the experimental test setup. A constant load of 269 kN was obtained in the models with steel \( E \) value more than 60 times the \( E \) value of Model M1_1 (Table 5.3). The failure mode of these masonry models was diagonal cracking through the walls (Figure 5.13). The load displacement behaviour of each model is shown in Figure 5.14. All the wall test models
behaved similar after the test setup achieved its optimum stiffness. Also the horizontal loading beam remained horizontal throughout these simulations.

Figure 5.10: Shear wall test setup schematic (Bosiljkov et al., 2008)

Figure 5.11: Dimensions of the initially proposed test setup and the wall panel (AR05)

The effect of the initial truss angle was examined using two FE models; M1_100 and M1_100-45. The truss angles in these models were $30^\circ$ and $45^\circ$ (initial angle between the inclined truss members and vertical) respectively. Other properties were kept constant. The load displacement diagram for these two tests is given
in Figure 5.15. The results show that the wall failure mode was not affected by the truss angle.

Table 5.3: Results for model M1

<table>
<thead>
<tr>
<th>Model Name</th>
<th>$E$ of steel (x 200 GPa)</th>
<th>Truss angle</th>
<th>Maximum load</th>
<th>Failure mode</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Beam</td>
<td>Truss</td>
<td></td>
<td></td>
</tr>
<tr>
<td>M1_1</td>
<td>1</td>
<td>1</td>
<td>30</td>
<td>249</td>
</tr>
<tr>
<td>M1_10</td>
<td>10</td>
<td>10</td>
<td>30</td>
<td>276</td>
</tr>
<tr>
<td>M1_50</td>
<td>50</td>
<td>50</td>
<td>30</td>
<td>270</td>
</tr>
<tr>
<td>M1_60</td>
<td>60</td>
<td>60</td>
<td>30</td>
<td>269</td>
</tr>
<tr>
<td>M1_100</td>
<td>100</td>
<td>100</td>
<td>30</td>
<td>269</td>
</tr>
<tr>
<td>M1_100-45</td>
<td>100</td>
<td>100</td>
<td>45</td>
<td>269</td>
</tr>
<tr>
<td>M1_1000</td>
<td>1000</td>
<td>1000</td>
<td>30</td>
<td>269</td>
</tr>
</tbody>
</table>

Figure 5.12: Model M1_1 showing the sliding at the base and loading beam bending
5. Finite element modelling and design of test apparatus for in-plane shear using fixed-fixed boundary conditions.

Figure 5.13: Diagonal failure of the models M1_60 at 10 mm displacement (20 times exaggeration of the actual deformation)

Figure 5.14: Load versus displacement behaviour of the models with different $E$ values of steel
Figure 5.15: Load versus displacement behaviour due to the different truss angle

The increase of the test setup stiffness (by increasing $E$ value) did not affect the results after the steel $E$ value reached 60 times that of its original stiffness (in Model M1_1)(Table 5.3). Therefore the minimum required stiffness for the actual test setup to simulate the fixed-fixed boundary conditions is a 60 times stiffer setup than the Model M1_1 test setup stiffness. The steel cross section properties were calculated to match the new stiffness (i.e. loading beam and truss members with 60 times the actual $EI$ and $EA$ values of the original loading beam and truss members respectively) as follows.

- Calculation for loading beam

\[ EI \text{ (Flexural rigidity)} = \text{constant} \]

Therefore; \((E_{st} \cdot I_{beam})_{\text{actual}} = 60 \cdot (E_{st} \cdot I_{beam})_{\text{model}}\)

\[ 200000 \text{ MPa} \times I_{\text{beam}} = 60 \times 200000 \text{ MPa} \times 10^8 \text{ mm}^4 \]

Therefore; \(I_{\text{beam}}\) required for the experimental test apparatus = \(6 \times 10^9 \text{ mm}^4\)
Calculation for truss members

As above; \((E_{st} \cdot I_{truss})_{\text{actual}} = (60 \cdot E_{st} \cdot I_{truss})_{\text{model}}\)

\[200000 \text{ MPa} \times I_{\text{truss}} = 60 \times 200000 \text{ MPa} \times 1.582 \times 10^6 \text{ mm}^4\]

Therefore; \(I_{\text{truss}}\) required for the experimental test apparatus = \(9.492 \times 10^7 \text{ mm}^4\)

The required second moment of area of the steel beam section for the study was \(6000 \times 10^6 \text{ mm}^4\). Therefore the required beam section to achieve this target is 1000WB296. The required second moment of area of the truss members was \(9.492 \times 10^7 \text{ mm}^4\). Considering the capacity of available laboratory equipment, it was considered unlikely that the predicted ultimate failure load for the URM wall (269 kN in the FE model) and strengthened walls (expected to be higher than the URM) could be achieved during the experimental program. Therefore, a new FE model was developed using reduced wall dimensions in order to reduce the predicted load and steelwork sizes.

The new dimensions for the proposed wall panels with aspect ratio 0.5 (AR05) were 1000 mm x 2000 mm (height x length) and the stiffness of the loading beam in the new model was that of an 800WB192 steel I section (model M2). The cross section of the truss members were increased to 100 mm x 200 mm (width x height). Figure 5.16 shows the new FE model developed.

Table 5.4 shows the results for the new model (model M2). Three different steel stiffnesses (by changing the \(E\) value of steel) and three different loading beam types were investigated to obtain the precise test setup loading beam requirements. Because the truss angle did not affect the test results, a 30\(^\circ\) truss angle was maintained in all FE analyses. A constant pre-compression level of 1 MPa was applied to all the models.

Figure 5.17 shows the load displacement curves for each model. The models executed with the steel loading beam type 800WB192 show that the ultimate load, failure load and the sliding was nearly equal. The horizontal loading beam remained horizontal throughout all these tests meaning that the stiffnesses
selected for the steel truss members were adequate. Therefore the test setup with the loading beam of 800WB192 and truss member dimensions of 200 mm x 100 mm was found to be appropriate to model the fixed-fixed boundary condition. Out of the three different loading beams, 600UB125 "I" beams were not stiff enough for the test setup. The results with 800WB192 and 700WB173 showed similar behaviour with the URM wall panels under shear loading (Table 5.4).

Figure 5.16: Revised FE model for masonry wall specimen and test setup

5.3.2 Final test setup

Considering the results reported in the section 5.3.1, the actual test setup was developed in the laboratory. It was decided to use the welded beam 800WB192 as the loading beam for the actual test setup to be confident that fixed-fixed conditions would be achieved. The length of the loading beam was 3500 mm. Six stiffeners were welded at each side connecting the two flanges of the "I" section for more stiffness as well as to stiffen the middle of the web which is also the
### Table 5.4: Results for model M2

<table>
<thead>
<tr>
<th>Model Name</th>
<th>Pre-compression load (MPa)</th>
<th>Loading beam type</th>
<th>$E$ of steel (x 200 GPa)</th>
<th>Maximum load (kN)</th>
<th>Sliding from the bottom of the wall at 6mm shear displacement (mm)</th>
<th>Failure mode</th>
</tr>
</thead>
<tbody>
<tr>
<td>M2_1</td>
<td>1</td>
<td>800WB192</td>
<td>1</td>
<td>213</td>
<td>0.19</td>
<td>Diagonal cracking through the wall</td>
</tr>
<tr>
<td>M2_100</td>
<td>1</td>
<td>800WB192</td>
<td>100</td>
<td>215</td>
<td>0.18</td>
<td>Diagonal cracking through the wall</td>
</tr>
<tr>
<td>M2_500</td>
<td>1</td>
<td>800WB192</td>
<td>500</td>
<td>216</td>
<td>0.18</td>
<td>Diagonal cracking through the wall</td>
</tr>
<tr>
<td>M2_700WB</td>
<td>1</td>
<td>700WB173</td>
<td>1</td>
<td>211</td>
<td>0.22</td>
<td>Diagonal cracking through the wall</td>
</tr>
<tr>
<td>M2_600UB</td>
<td>1</td>
<td>600UB125</td>
<td>1</td>
<td>200</td>
<td>4.81</td>
<td>Sliding at the bottom</td>
</tr>
</tbody>
</table>
5. Finite element modelling and design of test apparatus for in-plane shear using fixed-fixed boundary conditions.

Figure 5.17: Load displacement curves for different $E$ values and different steel section of the test setup

middle of the beam, where the loading arms are attached (Figure 5.18). The dimensions of the truss section and their connections are shown in Figure 5.19.

The steel truss members A, B and C were with a cross sectional area of 100 mm x 200 mm (width x height). The final test setup built in the laboratory is shown in Figure 5.1 using the above steel sections.

5.4 Investigating effect of pre-compression load

Four different masonry models with and without FRP strengthening were used in this investigation for the two different wall aspect ratios. FE models were developed using the final test setup determined in Section 5.3.2 above. A range of pre-compression loads were applied in these models to examine the behaviour of the failure mode due to the level of pre-compression.
5. Finite element modelling and design of test apparatus for in-plane shear using fixed-fixed boundary conditions.

Figure 5.18: Final test setup

Figure 5.19: Truss dimensions (a) Connection A and C (b) Part A and C (c) Part B
5. Finite element modelling and design of test apparatus for in-plane shear using fixed-fixed boundary conditions.

The objective of this study was to obtain the optimum pre-compression load which caused the masonry models to fail by diagonal cracking. The FRP reinforcing methods studied in this research are designed to enhance the performance of URM walls which fail by diagonal cracking or bed joint sliding within the height of the wall and so an experimental program designed to assess the effectiveness of the FRP strengthening schemes is most informative if such “through wall” failure modes are observed. Furthermore, diagonal cracking failure modes are the main failure mode resulting in large loss of load carrying capacity and collapse in masonry shear walls during earthquakes. The behaviour due to the varied pre-compression loads of the unreinforced and strengthened masonry models is discussed in the following sections, for the two aspect ratios.

5.4.1 Aspect ratio 0.5 walls

5.4.1.1 Unreinforced masonry model – AR05

Results for different pre-compression loads (expressed as the resulting uniform pre-compression stress level) are shown in Table 5.5. The FE model shown in Figure 5.16 (model M2_1) was used in this study. Table 5.5 shows the predicted sliding at the bottom of the wall (between the wall and support, the latter representing the footing beam in the experimental program) at a loading beam displacement of 6 mm, the predicted maximum load and the failure mode for the different pre-compression levels. In all the models, load becomes nearly constant after 6mm displacement. Therefore 6mm beam displacement was selected as a reference for results comparison.

The results show that the wall tested with a pre-compression stress of 0.75 MPa (model M2_1_075) failed by sliding at the bottom with some diagonal cracks within the wall (Figure 5.20a). All the other URM FE models (pre-compression 0.9 MPa and above) failed by diagonal cracking through the wall. Sliding at the bottom was negligible for these models (Table 5.5). Typical diagonal failure
Table 5.5: Results for URM and strengthened wall models (AR05)

<table>
<thead>
<tr>
<th>Model Name</th>
<th>Wall type</th>
<th>Pre-compression stress (MPa)</th>
<th>Maximum load (kN)</th>
<th>Sliding at the bottom of the wall at 6mm loading beam displacement (mm)</th>
<th>Failure mode</th>
</tr>
</thead>
<tbody>
<tr>
<td>M2_1_075</td>
<td>URM</td>
<td>0.75</td>
<td>171</td>
<td>4.94</td>
<td>Sliding failure with some minor diagonal cracking through the wall</td>
</tr>
<tr>
<td>M2_1_09</td>
<td>URM</td>
<td>0.9</td>
<td>197</td>
<td>0.23</td>
<td>Diagonal cracking through the wall</td>
</tr>
<tr>
<td>M2_1</td>
<td>URM</td>
<td>1</td>
<td>213</td>
<td>0.19</td>
<td>Diagonal cracking through the wall</td>
</tr>
<tr>
<td>M2_1_12</td>
<td>URM</td>
<td>1.2</td>
<td>241</td>
<td>0.15</td>
<td>Diagonal cracking through the wall</td>
</tr>
<tr>
<td>M3_1</td>
<td>Strengthened</td>
<td>1</td>
<td>210</td>
<td>5.09</td>
<td>Sliding failure with some minor diagonal cracking through the wall</td>
</tr>
<tr>
<td>M3_1_12</td>
<td>Strengthened</td>
<td>1.2</td>
<td>240</td>
<td>4.88</td>
<td>Sliding failure with some minor diagonal cracking through the wall</td>
</tr>
<tr>
<td>M2_1_14</td>
<td>Strengthened</td>
<td>1.4</td>
<td>271</td>
<td>0.2</td>
<td>Sliding failure with some minor diagonal cracking through the wall</td>
</tr>
<tr>
<td>M3_1_16</td>
<td>Strengthened</td>
<td>1.6</td>
<td>307</td>
<td>0.24</td>
<td>Diagonal cracking through the wall</td>
</tr>
<tr>
<td>M3_1_18</td>
<td>Strengthened</td>
<td>1.8</td>
<td>337</td>
<td>0.09</td>
<td>Diagonal cracking through the wall</td>
</tr>
</tbody>
</table>
mode observed in the latter models is shown in Figure 5.20 (b). The load–displacement curves for all these models are shown in Figure 5.21.

Figure 5.20: Failure patterns at 6 mm loading beam displacement (deformation factor 20) (a) Shear sliding with diagonal cracks (model M2_1_075) (b) Diagonal crack (model M2_1_09)

Figure 5.21: Load versus displacement behaviour due to the different pre-compression loads for URM walls (AR05)
5.4.1.2 FRP strengthened wall model – AR05

The FRP strengthened masonry FE model was developed as discussed in section 5.2.2, with the final test setup properties (Figure 5.22) (model M3). This model was executed with five different pre-compression loads. Results for each model are shown in Table 5.5. The load displacement behaviour of each model is shown in Figure 5.23. Unlike the unreinforced model, the strengthened walls failed by sliding until the pre-compression level reached 1.4 MPa (Table 5.5 and Figure 5.24). This is caused by increasing the resistance to diagonal cracking due to the application of the FRP strips in the model. The diagonal failure mode for the strengthened wall models were obtained when the pre-compression level exceeded 1.4 MPa (Table 5.5). Therefore in order to achieve the diagonal failure mode, a pre-compression level greater than 1.4 MPa needs to be applied in the experimental program for wall panels with aspect ratio 0.5 and dimensions of 1000 mm x 2000 mm (height x length). The failure patterns of the models with pre-compression levels below and above 1.4 MPa are shown in Figure 5.24.

![Figure 5.22: FE model of strengthened masonry wall (AR05) with FRP strip arrangement](image-url)
5. Finite element modelling and design of test apparatus for in-plane shear using fixed-fixed boundary conditions.

Figure 5.23: Load versus displacement behaviour due to the different pre-compression levels for FRP strengthened walls (AR05)

Figure 5.24: Failure patterns at shear displacement of 6 mm (deformation factor 20) (a) Shear sliding with diagonal cracks (model M3_1_12) (b) Diagonal crack (model M3_1_16)
5.4.2 Aspect ratio 1 walls

5.4.2.1 Unreinforced masonry model – AR1

As described in the section 5.2.1, a 2D unreinforced masonry model was developed with the wall dimensions of 1200 mm x 1200 mm (height x length) (model M4). The behaviour of the URM walls under pre-compression stresses 0.5 MPa, 0.75 MPa, 1 MPa and 1.2 MPa were investigated using this FE model. The sliding at the bottom of the wall at 6 mm beam displacement, the maximum load and the failure mode are reported for the different pre-compression loads for each model in Table 5.6.

The model tested with pre-compression 0.5 MPa mainly failed by sliding at the bottom and top of the wall (Figure 5.25a). All the other wall models showed diagonal cracking failure within the wall. The sliding at the bottom of the wall was negligible for these models. All the models show sliding failure initially from the top first course of the wall. This reduced when the pre-compression load was increased. A typical diagonal failure pattern obtained is shown in Figure 5.25(b). The load–displacement curves for these unreinforced masonry wall models with varied pre-compression loads are shown in Figure 5.26.

5.4.2.2 FRP strengthened wall model – AR1

The FE model for strengthened masonry walls with the dimensions of 1200 mm x 1200 mm (height x length) is shown in Figure 5.27 (model M5). Four FRP strips were attached to the masonry wall as described in section 5.2.2. Results for each model are shown in Table 5.6. The load displacement behaviour of each model is shown in Figure 5.28. The models with pre-compression load 1 MPa and 1.2 MPa failed by sliding along the bottom of the wall showing minor diagonal cracks within the wall (Figure 5.29a). All the other masonry models with the pre-compression loads above 1.2 MPa failed by diagonal cracking through the wall. Figure 5.29(b) shows the typical failure pattern of the walls which failed by diagonal cracking.
Table 5.6: Results for URM and strengthened wall models (AR1)

<table>
<thead>
<tr>
<th>Model Name</th>
<th>Wall type</th>
<th>Pre-compression load (MPa)</th>
<th>Maximum load (kN)</th>
<th>Sliding from the bottom of the wall at 6mm shear displacement (mm)</th>
<th>Failure mode</th>
</tr>
</thead>
<tbody>
<tr>
<td>M4_1_05</td>
<td>URM</td>
<td>0.5</td>
<td>80</td>
<td>2</td>
<td>Sliding failure from top and bottom of the wall with some minor diagonal cracking through the wall</td>
</tr>
<tr>
<td>M4_1_075</td>
<td>URM</td>
<td>0.75</td>
<td>99</td>
<td>0.2</td>
<td>Diagonal cracking through the wall</td>
</tr>
<tr>
<td>M4_1_1</td>
<td>URM</td>
<td>1</td>
<td>118</td>
<td>0</td>
<td>Diagonal cracking through the wall</td>
</tr>
<tr>
<td>M4_1_12</td>
<td>URM</td>
<td>1.2</td>
<td>133</td>
<td>0</td>
<td>Diagonal cracking through the wall</td>
</tr>
<tr>
<td>M5_1</td>
<td>Strengthened</td>
<td>1.0</td>
<td>127</td>
<td>2.72</td>
<td>Sliding failure from the bottom of the wall with minor diagonal cracking</td>
</tr>
<tr>
<td>M5_1_12</td>
<td>Strengthened</td>
<td>1.2</td>
<td>145</td>
<td>1.53</td>
<td>Sliding failure from the bottom of the wall with minor diagonal cracking</td>
</tr>
<tr>
<td>M5_1_14</td>
<td>Strengthened</td>
<td>1.4</td>
<td>159</td>
<td>0.2</td>
<td>Diagonal cracking failure with minor sliding from the top of the wall</td>
</tr>
<tr>
<td>M5_1_16</td>
<td>Strengthened</td>
<td>1.6</td>
<td>173</td>
<td>0.06</td>
<td>Diagonal cracking failure with minor sliding from the top of the wall</td>
</tr>
<tr>
<td>M5_1_18</td>
<td>Strengthened</td>
<td>1.8</td>
<td>185</td>
<td>0.01</td>
<td>Diagonal cracking failure with minor sliding from the top of the wall</td>
</tr>
</tbody>
</table>
Finite element modelling and design of test apparatus for in-plane shear using fixed-fixed boundary conditions.

Figure 5.25: Failure patterns at shear displacement of 6 mm (deformation factor 20) (a) Shear sliding with diagonal cracks (model M4_1_05) (b) Diagonal crack (model M4_1_1)

Figure 5.26: Load versus displacement behaviour due to the different pre-compression loads for URM walls (AR1)
5. Finite element modelling and design of test apparatus for in-plane shear using fixed-fixed boundary conditions.

Figure 5.27: FE model of strengthened masonry wall (AR1) with FRP strip arrangement

Figure 5.28: Load displacement behaviour due to the different pre-compression loads for FRP strengthened walls (AR1)
5. Finite element modelling and design of test apparatus for in-plane shear using fixed-fixed boundary conditions.

Figure 5.29: Failure patterns at shear displacement of 6 mm (deformation factor 20) (a) Shear sliding with diagonal cracks (model M5_1_1) (b) Diagonal cracks through the wall (model M5_1_14)

The diagonal failure mode for the URM wall models were obtained at the pre-compression of 1 MPa or above (Table 5.6). In contrast, at the pre-compression load of 1 MPa the strengthened walls were still failing by sliding along the bed joint. Therefore the appropriate pre-compression level to achieve diagonal failure in both URM and strengthened walls was determined to be 1.4 MPa or above for the wall panels with aspect ratio 1 and dimensions of 1200 mm x 1200 mm (height x length).

5.6 Summary and conclusions

FE models of masonry wall panels including the moving parts of the experimental test apparatus were used to design the experimental program reported in Chapter 6. The methodology and results of this modelling were discussed in this chapter. One of the main objectives was to develop an experimental test setup which can model fixed-fixed boundary conditions for masonry shear wall tests. It was found that the maximum failure load and the ultimate failure mode of the masonry wall panels are highly sensitive to the stiffness of the steel sections in the test setup. Furthermore, once an “adequate”
level of stiffness was achieved, any additional stiffness did not cause further changes to the modelling results. Such a pre-investigation on suitability of the test setup with the proposed test specimen parameters is recommended prior to any major experimental program. It was found that the angle of the truss members in the developed test setup has no effect on the model results. The horizontal loading beam remained horizontal throughout the tests indicating that the cross section dimensions selected for the steel truss members were adequate.

Wall panels with two aspect ratios were proposed for the next phase of the experimental program. The wall dimensions for aspect ratio 0.5 were initially 1200 mm x 2400 mm (height to length) and then reduced to 1000 mm x 2000 mm (height x length) to achieve the diagonal failure load using the available laboratory loading equipment. The dimensions of the aspect ratio 1 walls used for the FE model were 1200 mm x 1200 mm (height x length).

The effect of the externally applied pre-compression load was investigated for the two aspect ratio walls. Here the aim was to achieve the optimum pre-compression load to obtain diagonal cracking failure modes. Both URM and FRP strengthened models for the two aspect ratios were used in this study. The strengthening was modelled by attaching four vertical FRP strips in each aspect ratio wall. The FRP strip arrangement used here was an arrangement with the average number of FRP strips proposed to be used in the next phase of the experimental program. Results show that the pre-compression levels of 0.9 MPa and 1.6 MPa were required to be applied for obtaining the diagonal failure modes for the URM and FRP strengthened walls with aspect ratio 0.5 (AR05) respectively. Therefore to get a diagonal failure mode for both URM and strengthened walls, a minimum pre-compression load of 1.6 MPa needs to be applied in the actual experiments for the walls constructed with similar dimensions and similar material properties. The pre-compression loads of 0.75 MPa and 1.4 MPa were required to obtain the diagonal failure mode for the URM
and strengthened walls with aspect ratio 1 (AR1) respectively. Therefore a minimum pre-compression of 1.4 MPa needs to be applied in the experiments with the similar wall dimensions and with similar material properties used for the FE analysis in this study.