

**BLAST RETROFIT OF UNREINFORCED MASONRY WALLS USING FABRIC
REINFORCED CEMENTITIOUS MATRIX (FRCM) COMPOSITES**

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ABSTRACT

Unreinforced masonry (URM) walls are commonly found in existing and heritage buildings in Canada, either as infill or load-bearing walls. Such walls are vulnerable to sudden and brittle failure under blast loads due to their insufficient out-of-plane strength. The failure of such walls under blast pressures can also result in fragmentation and wall debris which can injure building occupants.

Over the years, researchers have conducted experimental tests to evaluate the structural behaviour of unreinforced masonry walls under out-of-plane loading. Various strengthening methods have been proposed, including the use of concrete overlays, polyurea coatings and advanced fiber-reinforced polymer (FRP) composites. Fabric-reinforced cementitious matrix (FRCM) is an emerging material which can also be used to strengthen and remove the deficiencies in unreinforced masonry walls. This composite material consists of a sequence of one or multiple layers of cement-based mortar reinforced with an open mesh of dry fibers (fabric). This thesis presents an experimental and analytical study which investigates the effectiveness of using FRCM composites to improve the out-of-plane resistance of URM walls when subjected to blast loading.

As part of the experimental program, two large-scale URM masonry walls were constructed and strengthened with the 3-ply of unidirectional carbon FRCM retrofit. The specimens included one infill concrete masonry (CMU) wall, and one load-bearing stone wall. The University of Ottawa Shock Tube was used to test the walls under gradually increasing blast pressures until failure, and the results were compared to those of control (un-retrofitted) walls tested in previous research. Overall, the FRCM strengthening method was found to be a promising retrofit technique to increase the blast resistance of unreinforced masonry walls. In particular, the retrofit was effective in increasing the out-of-plane strength, stiffness and ultimate blast capacity of the walls, while delaying brittle failure and reducing fragmentation.

As part of the analytical research, Single Degree of Freedom (SDOF) analysis was performed to predict the blast behaviour of the stone load-bearing retrofit wall. This was done by computing wall flexural strength using Plane Section Analysis, and developing an idealized resistance curve for use in the SDOF analysis. Overall, the dynamic analysis results were found to be in reasonable agreement with the experimental maximum displacements.

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NOTATIONS

Symbol	Definition
A_e	Effective cross-sectional area
A_f	Area of fabrics by unit width of FRCM
A_n	Net area of masonry
A_{fm}	Area of FRCM composite
A_n	Net area of a masonry wall
C'	Compressive force due to the axial load
E_f	Tensile modulus of elasticity of the cracked FRCM
E_m	Modulus of elasticity of a masonry wall
$F(t)$	Applied load as a function of time
H	Height of a wall
I_{cr}	Cracked moment of inertia
I_o	Uncracked moment of inertia
I_r	Reflected Impulse
K_{LM}	Load mass factor
L	Length of a wall
L_d	Driver length
M_{cr}	Cracking flexural strength
M_f	Moment contribution of FRCM
M_m	Moment contributed by reinforced masonry
M_n	Nominal flexural strength
M_u	Ultimate flexural strength
P_0	Nominal concentric axial capacity
P_d	Axial load applied to the wall
R	Standoff distance
$R(t)$	Restoring force as a function of time
$P_{ri}(t)$	Idealized triangular pressure as a function of time
S	sectional modulus of the uncracked wall
W	Charge weight
Z	Scaled distance
d_{max}	Maximum displacement
d_{res}	Residual displacement
c	Depth of the neutral axis measured from the extreme compression fiber
f_{cs}	Axial compressive stress due to an axial load
f_{fe}	Effective tensile stress of FRCM
f_{fu}	Ultimate tensile stress of FRCM
f'_{fm}	Compressive strength of FRCM mortar
f'_m	Compressive strength of masonry prisms
$f'_{m.factor}$	Factored compressive strength of masonry prisms
f_r	Modulus of rupture
f_{rfm}	Modulus of rupture of FRCM mortar
f_{rm}	Modulus of rupture of masonry prisms
f_{fu}	Ultimate tensile strength of FRCM
k_1	Uncracked stiffness of masonry

k_2	Post cracking stiffness of masonry
p_d	Driver pressure
p_r	Reflected pressure
p_{cr}	Uniformly distributed pressure at just before cracking
p_u	Ultimate uniformly distributed pressure
t_f	Thickness of FRCM
t_d	Time of duration
t_m	Thickness of the masonry wall
$u(t)$	Displacement of masonry as a function of time
$\dot{u}(t)$	Velocity of masonry at mid-height as a function of time
$\ddot{u}(t)$	Acceleration of masonry at mid-height as a function of time
w	Uniformly distributed load
w_{cr}	Uniformly distributed load at just before cracking
w_u	Ultimate uniformly distributed load
δ_{anls}	Analytical displacement at mid-height
δ_{exp}	Experimental displacement at mid-height
Δ_{cr}	Deflection at just before cracking
Δ_u	Ultimate deflection
ϵ_f	Tensile strain of FRCM
ϵ_{fd}	Design tensile strain of FRCM
ϵ_{fe}	Effective tensile strain of FRCM
ϵ_{fu}	Ultimate compressive strain of FRCM
ϵ_{mu}	Ultimate compressive strain of masonry prisms
ρ	Ratio between area of FRCM reinforcement and net area of URM wall
Φ_m	Strength reduction factor for flexure
$\Phi_{v,f}$	Strength reduction factor for shear
ω_f	Calibrated reinforcement ratio

ACRONYMS

Acronym	Definition
ACI	American Concrete Institute
ASTM	American Society for Testing and Materials
CFCM	Carbon Fiber Cement Matrix
CFRP	Carbon Fibre Reinforced Polymer
CMG	Cementitious Matrix-Grid system
CMU	Concrete Masonry Unit
DAS	Data Acquisition System
DIF	Dynamic Increase Factor
ECC	Engineered Cementitious Composite
FE	Finite Element
FRCM	Fabric-Reinforced Cementitious Matrix
FRP	Fibre Reinforced Polymer
GFRP	Glass Fibre Reinforced Polymer
IMG	Inorganic Matrix Grid system
LVDT	Linear Variable Displacement Transducers
MBC	Mineral Based Composite
NSM	Near Surface Mounted
RC	Reinforced Concrete
RM	Reinforced Masonry
SRG	Steel Reinforced Grout
SDOF	Single Degree of Freedom
TNT	Trinitrotoluene
TRC	Textile Reinforced Concrete
TRM	Textile Reinforced Mortar
UDL	Uniformly Distributed Load
URM	Unreinforced Masonry Wall

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CHAPTER 1. INTRODUCTION

1.1. GENERAL

Masonry has been used as a construction material for many centuries due to its ease of construction, moderate strength, low cost and good architectural appearance. Indeed, masonry has been used throughout history to construct walls, buildings and bridges. According to Babatunde et al. (2017), masonry buildings continue to account for more than 70% of all buildings worldwide. In Canada and North America, masonry can be found in various structural and non-structural components in office buildings, hospitals, schools, houses, churches and heritage structures, including as infill or load-bearing walls. However, masonry walls in many existing buildings lack internal steel reinforcement and were constructed without consideration of rational design methods, making them vulnerable to the effects of out-of-plane loads. Similarly, stone walls found in many heritage buildings were mostly constructed using conservative “rule-of-thumb” designs without consideration of lateral loads.

Because they are constructed by stacking units using thin layers of mortar, such unreinforced masonry (URM) walls are especially vulnerable to brittle and sudden collapse under the effects of out-of-plane bending (Babaeidarabad, 2013). Figure 1-1 shows examples of out-of-plane flexural failures of URM walls during the 1997 Umbria Earthquake in Italy (Babaeidarabad, 2013).



Figure 1-1. Out-of-plane failures of load-bearing walls, Umbria, Italy, 1997 (Babaeidarabad, 2013)

The insufficient out-of-plane resistance of URM walls also makes them vulnerable to collapse under the effects of shock wave blast loading. Shock wave blast loading can result from either accidental explosions or deliberate bombing attacks. Events such as the Lac-Mégantic rail disaster, Toronto Sunrise Plant Explosion and Oklahoma City Bombing demonstrate the need to protect and strengthen critical buildings and infrastructure against the effects of blast loads.

Over the years many researchers have conducted experimental tests to evaluate the structural behaviour of unreinforced masonry (URM) elements such as beams, columns and walls under varying loading conditions, including under seismic and blast loads. Researchers have also

proposed various retrofits, including the use of concrete overlays, polyurea coatings and advanced fiber-reinforced polymer (FRP) composites to enhance the structural response of URM elements made of CMU, clay brick and stone masonry.

Fabric-reinforced cementitious matrix (FRCM) is a novel and emerging material which can also be used to strengthen and remove the deficiencies in masonry structures. This composite material consists of one or multiple layers of cement-based mortar which are reinforced with an open mesh of dry fibers. The high tension capacity of FRCM, and its compatibility with masonry substrates, make it an ideal retrofit material to increase the out-of-plane flexural resistance of unreinforced masonry walls, including under blast loads.

This thesis investigates the effectiveness of using FRCM composites to improve the out-of-plane resistance of URM walls under the effects of extreme blast loading.

1.2. OBJECTIVES AND SCOPE

The main objective of this research is to investigate the effectiveness of using novel FRCM composites to increase the out-of-plane blast resistance of unreinforced masonry walls. To achieve this objective, this thesis includes the following scope:

1. An experimental program which includes two URM walls (one concrete masonry infill wall and one stone load-bearing wall) which are retrofitted with FRCM composites and tested under air blast loading using a Shock Tube. The effectiveness of the retrofits will be evaluated by comparing the results in terms of ultimate blast resistance, displacement control and failure mode to those of identical control (un-retrofitted) walls tested by Ciornei (2012) and by Gandia (2019).
2. An analytical study which aims at predicting the flexural strength of the FRCM-retrofitted URM walls, and dynamic analysis using a single-degree-of-freedom analysis approach to predict the blast performance of the walls in terms of maximum mid-height displacement.

1.3. THESIS BREAKDOWN

Chapter 1 - Introduction:

- Overall introduction, purpose, and scope of this research.

Chapter 2 - Literature review:

- Introduction of FRCM systems, properties and existing design guidelines;
- Review of previous research on the use of FRCM to strengthen concrete and masonry structures under static and dynamic loads.

Chapter 3 - Experimental program:

- Description of the experimental program, test matrix, specimen designs, methods of construction, testing protocols, instrumentation and material properties used in this research.

Chapter 4 – Experimental Results:

- Detailed results from the experimental blast tests which includes; pressure-time history, displacement-time history, damage progression and basic observations during the tests.

Chapter 5 – Discussion:

- More in-depth discussion of the experimental results and causes of failure in the walls;
- Comparison of the retrofit wall performance against the response of the control URM walls and alternative retrofit methods.

Chapter 6 – Analysis:

- Development of a procedure to predict the flexural capacity and resistance of load-bearing stone masonry walls retrofitted with FRCM composites;
- Single degree of freedom analysis of the tested walls using the analytically derived resistance function and comparison of the analytical and experimental displacement results.

Chapter 7 – Conclusion:

- Summary of the overall thesis with conclusions from the experimental and analytical research on the blast retrofit of URM walls with FRCM composites;
- Recommendations for further research.

CHAPTER 2. LITERATURE REVIEW

2.1. CHAPTER OVERVIEW

This chapter presents a literature review presenting background information and previous research related to Fabric-Reinforced Cementitious Matrix (FRCM) and its use as a retrofit material to improve the structural performance of unreinforced masonry (URM) elements. In particular, the following sections are presented:

- Section 2.2: Introduces FRCM, its constituents and tensile behaviour;
- Section 2.3: Introduces existing FRCM design guidelines;
- Section 2.4: Reviews previous research on use of FRCM to strengthen concrete beams;
- Section 2.5: Reviews previous research on use of FRCM to strengthen URM walls;
- Section 2.6: Reviews previous research on use of FRCM to strengthen structures tested under extreme dynamic loading.

2.2. INTRODUCTION TO FRCM

2.2.1. FRCM definition

Fabric-Reinforced Cementitious Matrix (FRCM) can be defined as: “*a composite material consisting of a sequence of one or more layers of cement-based matrix reinforced with dry fibers in the form of open single or multiple meshes that, when adhered to concrete or masonry structural members, forms a FRCM system*” (ACI 549.4R-13, 2013). Many other names are also used to represent FRCM systems in the literature (Jos, 2016), including: TRM (Textile Reinforced Mortar), TRC (Textile Reinforced Concrete), MBC (Mineral Based Composites), CMG (Cementitious Matrix-Grid system), IMG (Inorganic Matrix Grid system) or CFCM (Carbon Fiber Cement Matrix).

FRCM systems can be applied to concrete and masonry structures to strengthen various elements (columns, walls, beams, slabs, etc.) as shown in Figure 2-1. Various reinforcement configurations of FRCM are also suggested in the literature for masonry walls to improve out-of-plane flexural strength and in-plane shear/seismic resistance as shown in Figure 2-1 (b).

The mortar in FRCM is typically sprayed, and can be applied using the procedures used to apply wet shotcrete repair mortars. Alternatively, the mortar can be applied (by hand) using troweling. The fiber grids (carbon or other) can be unidirectional or bidirectional and the system can be applied in multiple grid layers (1-4 layers) to achieve the desired strengthening. Each grid layer is typically $\frac{1}{2}$ ”-1” (12.7-25.4 mm) thick.

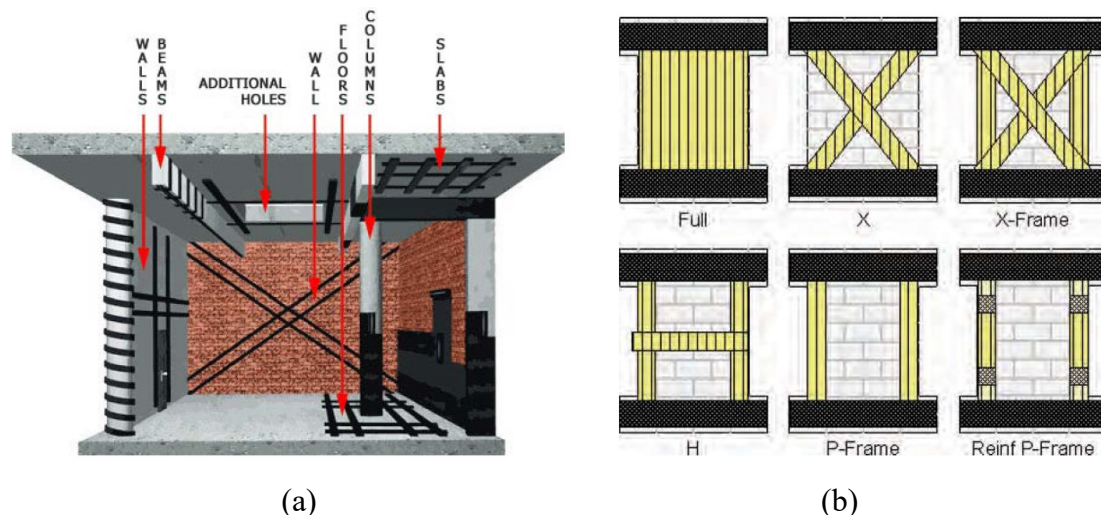


Figure 2-1. (a) Typical structural elements of a building showing repairs (in black) (Arboleda, 2014), (b) Reinforcement configuration for wall (ACI 549.4R-13, 2013)

2.2.2. FRCM advantages

FRCM is a viable alternative to other traditional concrete/masonry repair and strengthening methods including the use of reinforced concrete overlays, steel plates and externally-bonded fiber-reinforced polymer (FRP) composite systems (ACI 549.4R-13, 2013).

Many studies demonstrate the effectiveness of FRP to improve the structural response of unreinforced masonry (URM) walls. For example, Gustavo et al. (2003) has shown that FRP can increase the in-plane seismic strength of URM walls. Likewise, Galati et al. (2006) has demonstrated that FRP can increase the out-of-plane flexural response of unreinforced and reinforced masonry walls. Important research by Carey et al. (2013) has also shown the ability of FRP to increase the blast resistance of masonry construction.

The advantages of FRP include high strength-to-weight ratio, ease of application, minimal effect on existing structural geometry and corrosion resistance (Sulaiman, 2016). On the other hand, FRP systems, also show some deficits which include: the use of an organic binder (epoxy resin), susceptibility to be weakened at high temperatures, incompatibility to concrete or masonry substrates compared to mortar, inability to be applied on wet surfaces, irreversibility of the reinforced system and harmfulness to workers (Mobasher, 2011).

To overcome these disadvantages, epoxy resin is replaced by an inorganic cementitious matrix in FRCM. Moreover, the manually applied fiber sheets used in FRP are replaced by a structural reinforcing mesh, or fabric (ACI 549.4R-13, 2013). ACI 549.4R-13 (2013) “Guide to Design and Construction of Externally Bonded Fabric-Reinforced Cementitious Matrix (FRCM) Systems for Repair and Strengthening Concrete and Masonry Structures” lists the main advantages of FRCM systems:

- Compatibility with the properties of concrete and masonry substrates;
- Ease of installation (i.e. use of traditional mortar application methods);
- Porous matrix structure (allows air/moisture transport in substrate);

- Good performance at elevated temperatures, including fire and high durability;
- Ease of reversibility (i.e. the ability to fix or undo the FRCM retrofit without damaging the original structure).

2.2.3. Constituents of the FRCM composite

Fabrics

A textile grid, which is also called a fabric, is a structural reinforcing mesh made of dry fibers, yarns, or both. A yarn is a bundle of hundreds or thousands of fibers. Yarns can be either natural or artificial. Codispoti et al. (2015) introduces different kinds of fabrics made of natural fibers such as sisal, jute, hemp and flax as shown in Figure 2-2.

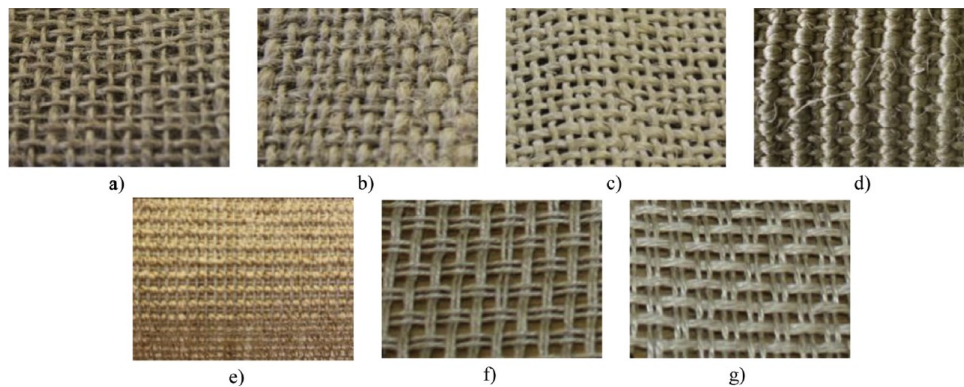


Figure 2-2. Different types of natural fabrics: (a-c) jute, (d, e) sisal, (f) hemp, (g) flax (Codispoti et al., 2015)

Artificial fibers have been developed with the interest to achieve higher strength than natural fibers and include: aramid, carbon, alkali-resistant (AR) glass, basalt or polymers like polypropylene (PP), polyethylene (PE) and polypara-phenylene benzobisoxazole (PBO) as shown in Figure 2-3.

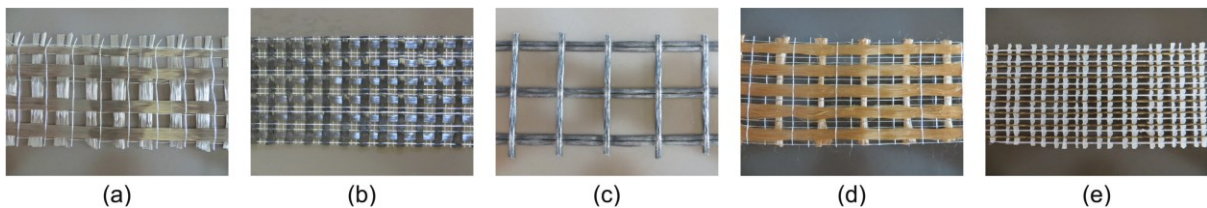


Figure 2-3. Different types of artificial fabrics: (a) basalt, (b) carbon, (c) glass, (d) PBO and (e) steel. (Escrig et al., 2017)

Either unidirectional or bidirectional fabrics can be used depending on the loading condition as shown in Figure 2-4.



Figure 2-4. FRCM carbon grid: Bidirectional (left) and unidirectional (right) (taken from manufacturer's data sheet)

Dry fibers can result in limited bond adhesion at the mortar interface, so the fabrics are normally coated with a flexible epoxy resin by manufacturers to enhance the adhesion capacity as shown in Figure 2-5 (Donnini et al., 2017). According to ACI 549.4R-13 (2013), the total coverage area of the fiber mesh should not be greater than two-thirds of the total area of the fabric, so that at least one third of the total area is opened among strands to secure the adhesion of fabric to the mortar.

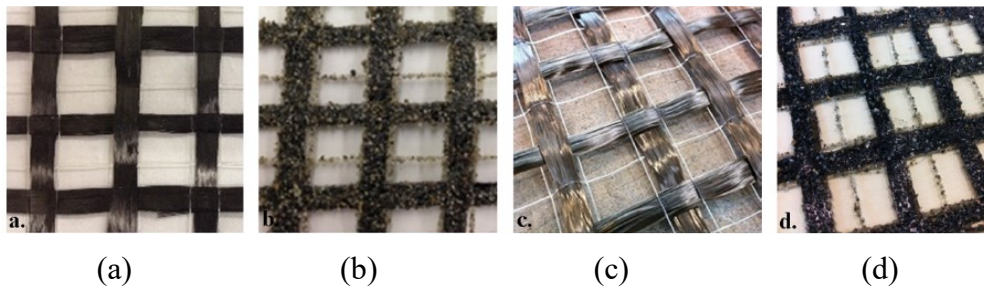


Figure 2-5. Dry or coated fabrics: (a) Dry carbon, (b) Coated carbon, (c) Dry basalt, (d) Coated basalt (Donnini et al., 2017)

Cementitious matrix

The cementitious matrix is the second main component in FRCM. Non-hydraulic mortars, such as lime-based mortars, are sometimes used for masonry construction, especially in heritage buildings, however cement-based mortars are commonly used for repairing/strengthening concrete elements. Donnini et al. (2017) conducted direct tensile tests with FRCM coupons using either lime-based or cement-based mortar, and concluded that the mechanical performance (tensile resistance and stiffness modulus) of the coupons with cement-based mortar were slightly better than those with lime-based mortar for the same fabrics. According to ACI 549.4R-13 (2013), cement based mortars can contain a low dosage (up to 5% by weight of cement) of dry polymers for the purpose of improving its workability or mechanical properties. The mortar can also be reinforced with micro-synthetic fibers. Depending on the manufacturers, mortar can be applied by hand troweling or by spraying (wet shotcrete).

2.2.4. Tensile behaviour of FRCM composites

The tensile behaviour of FRCM composites is considerably affected by the physical and mechanical characteristics of fibers, the structure of the fabrics, and the interaction between the fabrics and matrix. The typical stress-strain relationship of the FRCM composites in tension can be described by 3 phases as shown in Figure 2-6 (Caggegi et al., 2017). OA (Stage I) is defined as an un-cracked stage since the matrix mainly governs the tensile behavior of the coupon without cracking. AB (Stage II) is a crack development stage: in this stage the matrix in the FRCM system is no longer able to endure the tensile load which results in gradual load transfer to the textile. This stage can be ignored if the mortar shows poor response in tension. The intersection point of the two linear lines is defined as a transition point or a bend-over point. BC (Stage III) is the cracked stage in which the textiles govern the behavior of the coupon with the load transferred to the textile. The slope of this stage is very similar to the modulus of the fabric itself. The red line in Figure 2-6 is the idealized stress-strain response of the FRCM system in tension and consists of segments with three slopes; E_1 for Stage I, E_2 for Stage II, and E_3 for Stage III.

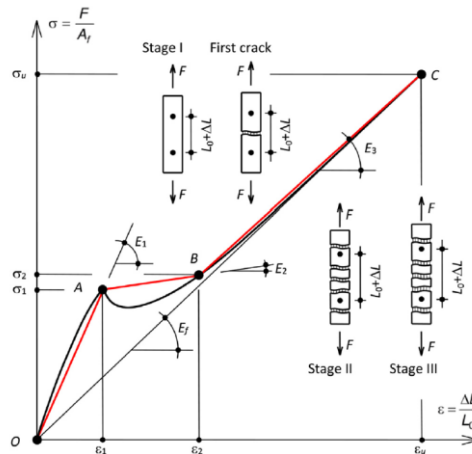


Figure 2-6. Typical tensile stress-strain response of FRCM system. L_0 : gage length; A_f : cross-sectional area of the fibers (Caggegi et al., 2017)

According to Parisi et al. (2019), the tensile behavior of FRCM system is affected by the types of fabrics, numbers of fabric layers, geometry of the fabrics, fabric coating, type of mortar and the way the ends of tension coupons are clamped (gripping conditions). Figure 2-7 compares the representative stress-strain curves of various FRCM systems (Parisi et al., 2019). Parisi et al. (2019) mentioned that coated fabrics can enhance the interfacial adhesion so that the failure mode is governed by fiber rupture rather than fiber slippage. Donnini et al. (2017) explained that the ultimate tensile strength of FRCM can be increased with multiple layers of fabrics, but at the same time, the ductility can be decreased. Caggegi et al. (2017) investigated the failure modes of FRCM coupons and the interaction between the constituents of the system. Figure 2-8 describes various failure modes; mode A is the failure at the clamps; mode B is the failure of fibers following the matrix cracking; mode C is the slippage of fibers with the matrix cracking; mode D is the detachment of the matrix from the fibers.

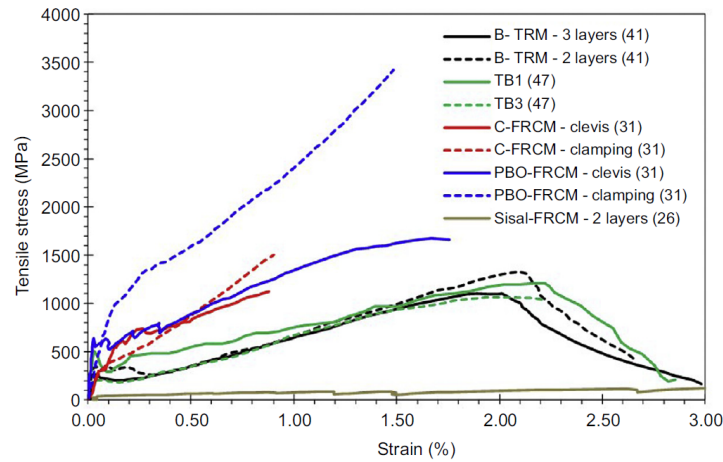


Figure 2-7. Representative average stress-strain curves of different FRCM systems subjected to direct tensile tests. (Parisi et al. 2019)

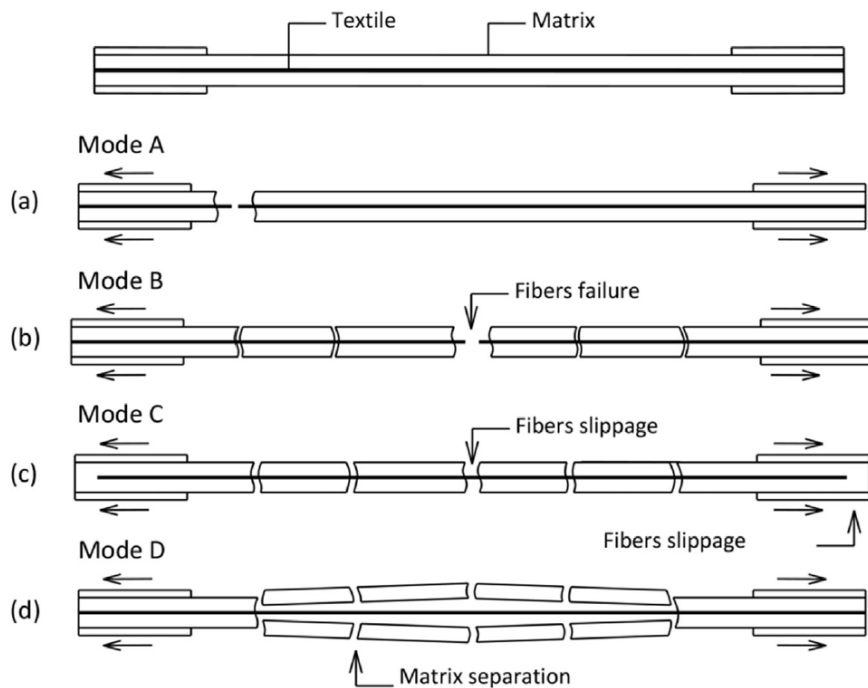


Figure 2-8. Typical failure modes of FRCM materials tested in tension. (Caggegi et al., 2017)

2.2.5. Bond behaviour of FRCM composite

The bond strength between the FRCM cementitious matrix and fabrics is achieved through the mechanical interlocking effect from the penetration of the mortar into the openings of the fabric (structural reinforcing mesh) (Caggegi et al., 2017). According to ACI 549.4R-13 (2013), the coverage area of the textile grid should be less than 2/3 of the total area; i.e. at least 1/3 of the total area in the reinforcing mesh should remain open to ensure proper adhesion between the matrix and fabric.

Figure 2-9 shows the typical (bond) failure modes in FRCM-substrate joint tests. The bond behaviour of FRCM composites to the substrate can be affected by various factors, including: the number of textile grid layers, the type of fabrics, the bond width/length and temperature conditions. (Awani et al., 2016).

According to ACI 549.4R-13 (2013), the bond strength of an FRCM system can be determined using the testing procedures specified in AC434 (2013) “Masonry and Concrete Strengthening Using Fabric-reinforced Cementitious Matrix (FRCM) and Steel Reinforced Grout (SRG) Composite Systems”. Chapter 14 of ACI 549.4R-13 (2013) also provides guidance for detailing externally bonded FRCM to avoid bond-related failures including rounding cross-section corners, providing adequate development length and providing sufficient overlap when splicing fiber meshes.

In terms of development length, ACI 549.4R-13 (2013) states that to develop the effective FRCM stress at a section, the minimum anchorage length should be more than 152 mm (6”) for any type of substrate. Donnini et al. (2017) studied the contact length between FRCM and masonry substrates, and also showed that 150 mm of contact length is sufficient to develop the full capacity of FRCM in double-shear bond tests.

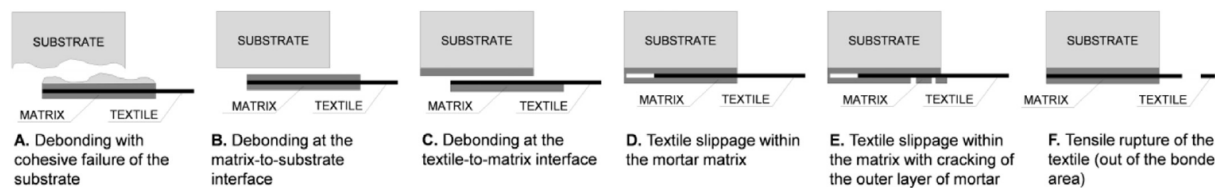


Figure 2-9. Typical failure modes observed for FRCM-substrate joints (Caggegi et al., 2017)

2.3. EXISTING GUIDELINE FOR USING FRCM

2.3.1. AC434 Acceptance Criteria

AC434 (2013) “Masonry and Concrete Strengthening Using Fabric-reinforced Cementitious Matrix (FRCM) and Steel Reinforced Grout (SRG) Composite Systems”, published by ICC-ES, prescribes material, durability and structural tests and minimum acceptable design criteria for the use of FRCM in the strengthening of concrete and masonry structures. AC434 (2013) acceptance criteria addresses strength, serviceability, fire safety, combustibility and durability (Eknel et al. 2018). Figure 2-10 shows the direct tensile test specified in Annex A of AC434 (2013). In this test, a FRCM coupon, cut from a panel, is used. Thin metal tabs are glued at both ends of the coupon. Clevis grips are used to hold the specimen while it is pulled by the testing machine. A clip-on extensometer measures the tensile strains.

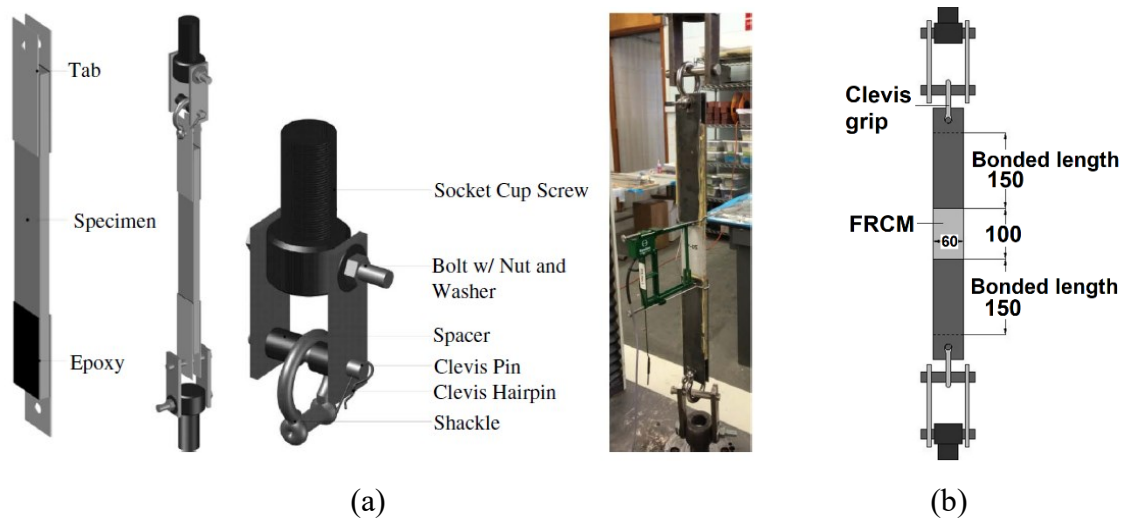


Figure 2-10. (a) FRCM direct tension test setup, and (b) Coupon dimensions (Pino et al., 2017) according to AC434 Annex A (Donnini et al., 2017)

2.3.2. ACI 549.4R Guide to Design and Construction of FRCM

ACI 549.4R-13 (2013) “Guide to Design and Construction of Externally Bonded Fabric-Reinforced Cementitious Matrix (FRCM) Systems for Repair and Strengthening Concrete and Masonry Structures”, published by ACI Committee 549, is another document that provides guidelines for the strengthening of concrete and masonry structures using FRCM systems. This document confirms the ability of FRCM to enhance the shear and flexural capacities of concrete and masonry structures. In addition to providing background information, it also provides design guidelines for the strengthening of concrete and masonry structures. Appendix B of ACI 549.4R-13 (see Table 2-1) provides a summary of design limitations related to different strengthening applications.

Table 2-1. Design limitations for FRCM for different applications (ACI 549.4R-13)

Parameter	Concrete			Masonry	
	Flexure	Shear	Axial	Out-of-plane	In-plane
ϵ_{fe} or ϵ_{fd}	Less than 0.012	Less than 0.004	Less than 0.012 and ϵ_{cw} less than 0.01	Less than 0.012	Less than 0.004
ϕ	0.9 to 0.65 based on ϵ_t	0.75	0.9 to 0.65 based on ϵ_t	0.6 for flexure 0.8 for shear	0.75
f_B/f_{Bt}	0.2 to 0.55 based on fiber	NA	NA	NA	NA
Allowable maximum enhancement*	50 percent	50 percent	20 percent	URM: 6000 lbf/ft (87.6 kN/m); Reinforced masonry: 50 percent	50 percent

*Allowable maximum enhancement is above existing capacity. ACI 562-13 supersedes when limits are lower than as listed in this table.

According to ACI 549.4R-13 (2013), FRCM can be bonded to the surface of masonry walls to increase out-of-plane flexural strength and act as tension reinforcement. For the case of out-of-plane flexural strengthening, which is the case for this research, the primary direction of the fabrics should be parallel to the bending span.

The nominal flexural out-of-plane strength can be determined using sectional analysis based on the following assumptions (ACI 549.4R-13, 2013):

- Strain compatibility between the FRCM composite and FRCM material;

- Plane sections remain plane;
- Maximum compressive strain in the masonry is obtained from field tests or MSJC-11;
- FRCM is assumed to have a bi-linear behaviour and the FRCM contribution before cracking is ignored.

The standard also mentions the different failure modes that are possible when FRCM is used to strengthen the out-of-plane flexural strength of masonry walls (ACI 549.4R-13, 2013):

- Crushing of the masonry in compression;
- FRCM debonding from the substrate;
- Fiber slippage within the FRCM mortar;
- Tensile yielding of the steel (in the case of reinforced masonry walls).

The effective tensile strain (ϵ_{fe}) of the FRCM composite system at failure is limited to the following design limit:

$$\epsilon_{fe} = \epsilon_{fd} \leq 0.012 \quad [2-1]$$

The effective tensile stress of the FRCM composite at failure (f_{fe}) is obtained as:

$$f_{fe} = E_f \epsilon_{fe} \quad \text{where } \epsilon_{fe} \leq \epsilon_{fd} \quad [2-2]$$

where E_f is the tensile modulus elasticity of the cracked FRCM composite.

Then, the design flexural strength can be computed as shown below:

$$\Phi_m M_n = \Phi_m (M_m + M_f) \quad [2-3]$$

where M_n is the nominal flexural strength. M_m and M_f are the contributions of reinforced masonry (RM) and FRCM composite material, respectively. In the case of unreinforced masonry (URM) walls with no axial load, M_m is taken as zero. The strength reduction factor for flexure (Φ_m) for RM and URM is 0.6, and taken as 0.8 for shear (Φ_{vf}). When the FRCM system is applied to both sides of the wall, the one on the compressive side is not considered. In addition, it is assumed that the FRCM does not contribute to enhancing the out-of-plane shear resistance of masonry walls, and therefore the shear resistance can be calculated using standard procedures in masonry design codes such as MSJC-11 (2011) or CSA S304.1-04 (2004).

The ACI 549.4R-13 (2013) document also provides a design procedure for calculating the in-plane shear strength of masonry walls with FRCM acting as shear reinforcement (this procedure is discussed in section 13.2 of ACI 549.4R-13 (2013)).

It is noted that the ACI 549.4R-13 (2013) design guidelines for masonry walls are limited to out-of-plane bending, in-plane shear and axial loading conditions. The design of FRCM systems under extreme dynamic loads such as impact and blast are not specifically addressed in this document.

2.4. PREVIOUS RESEARCH ON FRCM FOR FLEXURAL STRENGTHENING OF CONCRETE/RC BEAMS

This section outlines previous research on the use of FRCM for the flexural strengthening of concrete and reinforced concrete (RC) beams. Table 2-2 summarizes the details of various studies conducted in the literature along with their main conclusions. A selected number of studies are reviewed below.

Kolsch (1998)

Kolsch (1998) tested the flexural behaviour of concrete beams strengthened with carbon fiber cement matrix (CFCM) system. Preliminary tests with unreinforced concrete beams were carried out to attain the optimal combination of CFCM systems. Unreinforced concrete beams were strengthened by various systems, i.e. four glass fabrics with cementitious matrix, four glass fabrics with epoxy resin matrix, and four carbon fabrics with cementitious matrix. The beams were tested under three-point quasi-static loading (Figure 2-11). The results showed that CFCM strengthened beams failed at significantly higher loads with greater deformations when compared to the reference beams. Using a base cementitious matrix was better when compared to using a polymer matrix (epoxy resin) in strengthening beams.

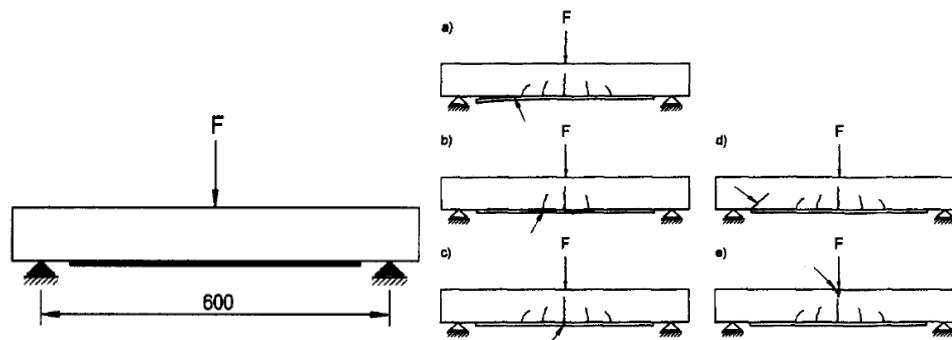


Figure 2-11. Test setup and failure patterns of CFCM strengthened concrete beams (Kolsch, 1998)

Elsanadedy, H. M., Almusallam, T. H., Alsayed, S. H., & Al-Salloum, Y. A. (2013)

Elsanadedy et al. (2013) carried out four-point bending tests on reinforced concrete (RC) beams strengthened with TRM or FRP systems to investigate their ability to increase flexural resistance. Six beams were tested: 2 control beams were built to be under-reinforced without any strengthening, 3 beams were retrofitted with either 5 or 10 layers of U-shaped basalt textiles (TRM), and one beam was strengthened with single layer of carbon polymer sheet (FRP) at the soffit. For TRM systems, cementitious or polymer-modified mortar was used for the matrix. Each layer of mortar was 2 mm thick. For FRP system, epoxy was used for the matrix. Single layer of carbon FRP lamina was chosen to be compared since it had similar flexural resistance to that of 5-layer basalt textiles (TRM). Monotonically increasing load was applied with a rate of 1 mm/min until failure. The TRM strengthened beams with polymer-modified cementitious mortar showed better bonding capacity to the substrate; such beams failed due to the textile rupture at a higher maximum load, whereas the same beam with cementitious mortar failed by

TRM de-bonding at a comparably lower maximum load as shown in Figure 2-12. Also, it showed better flexural capacity and deflection ductility compared to the controls. The TRM systems strengthened the flexural capacity of the beams by up to 91 %. In comparison with FRP, TRM was slightly less effective in strengthening the flexural capacity, but more effective in deflection ductility. According to the Finite Element (FE) analysis, it was possible to preclude or delay the end de-bonding effect by providing the U shaped TRM anchorage at the end of the TRM layers.

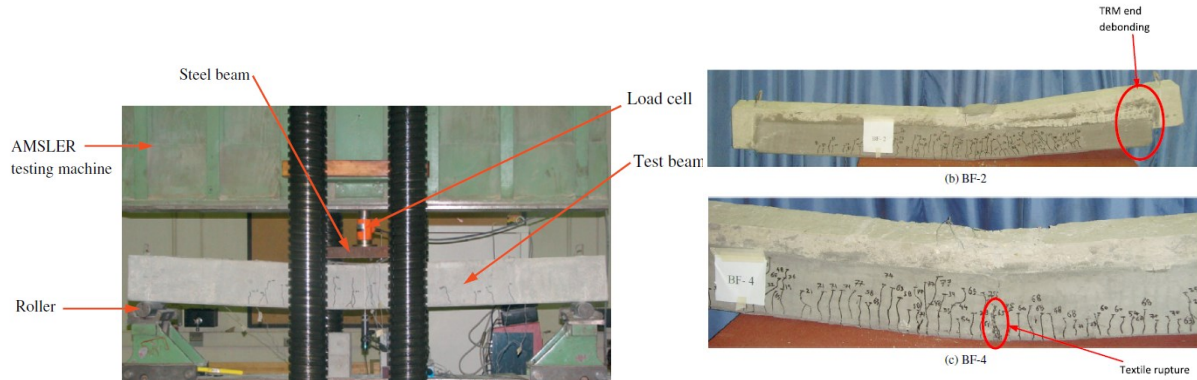


Figure 2-12. Experimental setup and test results (Elsanadedy et al., 2013)

Azam et al. (2018)

Azam et al. (2018) conducted an experimental study on six reinforced concrete (RC) beams, which were critical in shear resistance. Three RC beams were strengthened with one-layer of carbon FRCM composite systems providing different amount of internal transverse reinforcement of none, and 6 mm stirrups at 150 mm or 250 mm on centre (c/c). A side-bonded strengthening scheme was applied instead of U-jacketing techniques. The other three beams had same amount of shear reinforcement without external FRCM retrofit. Longitudinal reinforcements were the same for all specimens (6-25M in tension, 3-25M in compression). The large-scale shear-critical beams (400 mm (H) x 250 mm (W) x 2700 mm (L)) were tested under four-point bending with a loading rate of 0.3 mm/min to investigate their structural performance such as the shear transfer mechanism and load-deflection response including failure modes. The ACI 549.4R-13 design guide was used to predict the shear strength contribution of the retrofit. Carbon FRCM system was effective in strengthening the shear-critical RC beams showing a maximum increase of 87.4 % in ultimate load for the beams without shear reinforcements. Also, external retrofit was found to reduce the shear strength contribution of the shear reinforcement as the strengthened beam showed steeper shear cracks compared to the control beam (i.e. without FRCM strengthening). ACI 549.4R-13 design guide predicted the shear strength of the FRCM retrofitted beams to be in good correlation with the tested values.

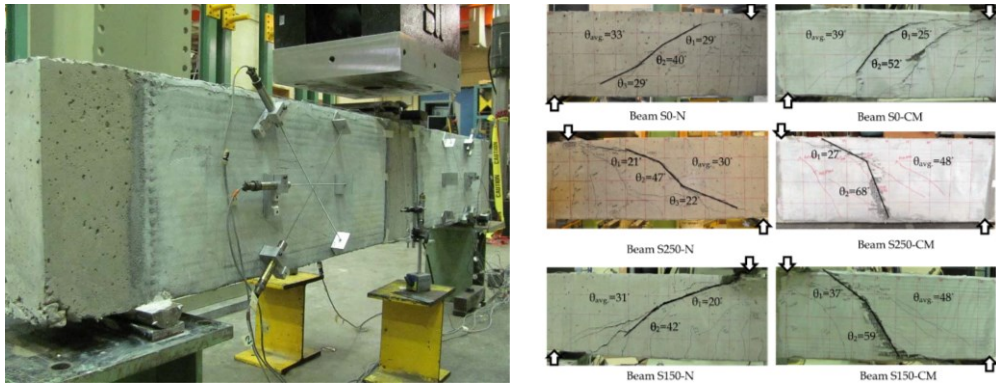


Figure 2-13. Test setup and results (Azam et al., 2018)

Table 2-2. Summary of research on use of FRCM for strengthening concrete beams

Author(s)	Testing Elements			Loading		FRCM (TRM) systems			
	Type	Detail	Dimensions (h x w x l)	Type	Properties	Applied Surface	Fabrics (no. of layers)	Matrix	Thickness
Kolsch (1998)	Concrete beams	Non reinforced	0.1 x 0.15 x 0.7 m	3 point	Quasi-static loading	Bottom	Glass / Carbon (4)	Cementitious mortar Epoxy resin	Variable (one layer of mortar: 2 mm)
Conclusion	Textile strengthened beams were failed at significantly higher loads and deformations compared with the reference beams. Using cementitious matrix was competitive to using a polymer matrix (epoxy resin) in strengthening the beams.								
Elsanadedy et al. (2013)	RC beams	Steel reinforced	0.2 x 0.15 x 2.0 m	4 point	Monotonic load (1 mm/min)	Bottom and both sides (U-Shape) * Bottom only for FRP	Basalt (5, 10) * Carbon (1) for FRP	Cementitious mortar, Polymer modified cementitious mortar * Epoxy for FRP	Variable (one layer of mortar: 2 mm)
Conclusion	Polymer modified cementitious mortar had better bondage between TRM and substrate than cementitious mortar. TRM increased flexural capacity, but was slightly less effective than CFRP. TRM performed better in deflection ductility.								
Escrig et al. (2017)	RC beams	With insufficient reinforcement	0.5 x 0.2 x 4.4 m	4 point loading	Hydraulic actuator (1 mm/min)	Tension face (Bottom)	Basalt / Carbon / Glass / PBO / Steel (1)	Cement based mortar	Not mentioned
Conclusion	TRM strengthening (bottom face, both ends with U-jacket shape TRM) increased the flexural capacity at the first crack and at yielding. Different types of fibers showed different flexural behaviour.								
Jabr et al. (2017)	RC beams	Steel reinforced	0.25 x 0.15 x 2.4 m	2 point loading	Monotonic load (2 mm/min)	Tension face (Bottom) w/ partial U-wrap	Carbon / Glass / PBO (1, 4)	Cement based mortar	Variable (one layer of mortar: 3 mm)
Conclusion	PBO FRCM significantly increased the ultimate capacity of the strengthened beams, whereas carbon and glass FRCM did not show noticeable increase.								
Azam et al. (2018)	RC beams	Steel reinforce	0.4 x 0.25 x 2.7 m	4 point	Monotonic load (0.3 mm/min)	Shear face (Side)	Carbon (1)	Cement based mortar	6-8 mm
Conclusion	FRCM was effective in strengthening shear-critical RC beams showing maximum increase of 87.4 % in the ultimate load without shear reinforcements. FRCM reduced the shear strength contribution of the shear reinforcement as the strengthened beam showed steeper shear cracks.								
Ebead et al. (2019)	RC beams	Steel reinforced	0.26 x 0.15 x 2.5 m	4 point loading	Monotonic load (1 mm/min)	Tension face (Bottom)	Carbon / Glass / PBO (2)	Cement based mortar	10-15 mm
Conclusion	Near-surface embedded (NSE) and externally bonded (EB) FRCM showed 62.5 % and 72.3 % gain of load carrying capacity, respectively. Both methods reduced the ductility of the beams. Glass FRCM strengthened presented the highest FRCM fabric utilization.								

2.5. PREVIOUS RESEARCH ON FRCM FOR FLEXURAL STRENGTHENING OF MASONRY WALLS

This section presents previous studies on the use of FRCM for the flexural (out-of-plane) strengthening of masonry walls. Previous research includes tests on beam-like elements under monotonic or cyclic flexural loading, and tests on larger-scale walls tested under out-of-plane monotonic or cyclic loads using varying setups, including air-bag testing. Additional research has examined the use of FRCM for the shear (in-plane) seismic strengthening of masonry walls. The various test methods and results from previous research are summarized in Table 2-3. The following sections provide a summary of some of this research.

2.5.1. Beam-like elements

Harajli, M., ElKhatib, H., & San-Jose, J. T. (2010)

Harajli et al. (2010) tested various types of URM walls with different combinations of FRCM fabrics and mortar under 4-point bending. A total of 25 URM walls were built using hollow core blocks, sandstones, or bricks. These walls were strengthened by using FRCM systems having various types of textiles (bitumen coated E-glass, basalt, and coated basalt) or steel wire mesh with either lime mortar or a commercial cement-based mortar. One layer of each type of textile, or one or two layers of steel wire mesh were applied on a single (tension) face under static loading, or on both faces under cyclic loading. Monotonic load at a rate of 5 mm/min until failure was applied in 4-point bending for the static tests (9 walls), and cyclic load at a rate between 0.1 mm/s and 1 mm/s was applied up to failure for the cyclic tests (16 walls) (see Figure 2-14). From the static tests, out-of-plane flexural and displacement capacities were increased considerably for all the strengthened walls, and the corresponding improvements were similar regardless of the types of masonry units. From the cyclic tests, load and displacement capacities were significantly increased, and seismic performance was improved compared with URM walls. Wire mesh reinforced walls showed the highest improvement in strength, but failed in a brittle manner. The wall strengthened by a coated basalt textile with a lime mortar showed the highest energy absorption and dissipation capacities, and the least stiffness and strength degradation.

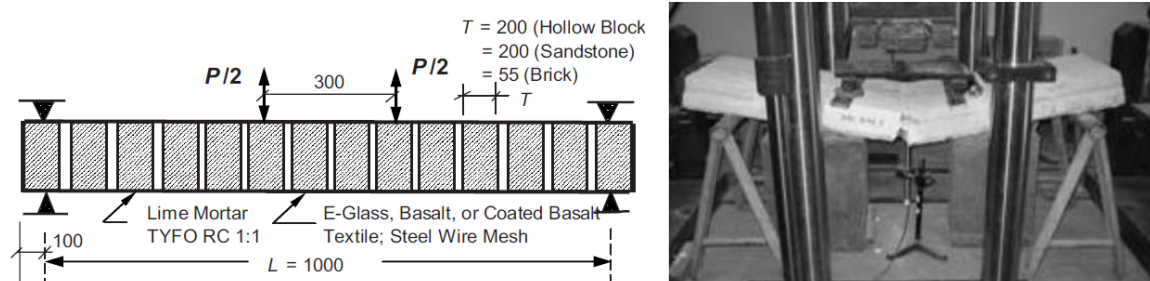


Figure 2-14. Test setup and test results (Harajli et al., 2010)

Kariou, F. A., Triantafyllou, S. P., Bournas, D. A., & Koutas, L. N. (2018)

Kariou et al. (2018) investigated the flexural behaviour of 18 solid clay brick masonry walls (9 single-wythe and 9 double-wythe) under 3-point loading. One wall from each group (single- or double-wythe) was tested as a control with no reinforcement. One layer of carbon fiber textiles with either epoxy resin coated or uncoated, 3 or 7 layers of glass fiber textiles with either epoxy resin coated or uncoated, or 3 or 7 layers of coated basalt fiber textiles were used to strengthen the specimens on the tensile faces. A cement-based mortar with polymers was used as a binder between the textile and the masonry substrate. The quasi-static load was applied using a servo-hydraulic actuator at a rate of 1 mm/min to simulate out-of-plane flexural behaviour. In general, the epoxy-resin coated textiles enhanced the bonding capacity, and the maximum load capacity was improved. The maximum load was increased as the number of TRM layers was increased since the interlocking condition of the TRM system was improved. TRM system on single-wythe walls was more efficient compared with that on double-wythe walls considering the ratio of the maximum load capacities of the strengthened versus the control walls.

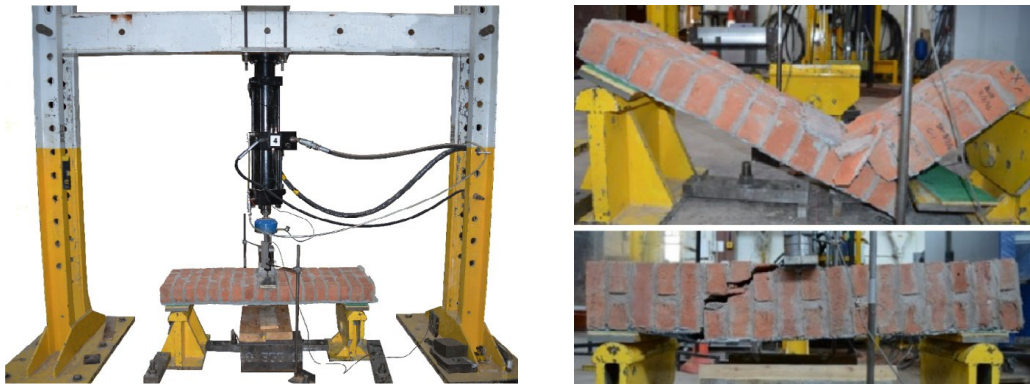


Figure 2-15. Experimental setup and test results of single-wythe (right-up) and double-wythe (right-down) with one layer of coated carbon grid (Kariou et al., 2018)

Al-Jaberi, Z., Myers, J. J., & ElGawady, M. A. (2018)

Al-Jaberi et al. (2018) evaluated pseudo-static cyclic performance of 12 reinforced masonry (RM) walls strengthened in out-of-plane with either FRCM composites or near surface mounted (NSM) reinforcement. The specimens were built using concrete blocks in running or stack patterns with the same amount of steel reinforcement. One or two layers of FRCM composites having carbon or PBO (polyparaphenylene benzobisoxazole) fabrics were used to cover the tension faces of the walls completely or partially. Carbon or glass bars were used for NSM reinforcement on the tension faces. Cycles of loading and unloading at a rate of 1.25 mm/min with the displacement increment of 6.35 mm were applied under 4-point bending as shown in Figure 2-16. The lateral load capacity of RM walls strengthened with 2 layers of PBO fabrics (FRCM) and 2 carbon bars (NSM) systems were increased by 97 % and 75 %, respectively. More layers of textiles resulted in higher load capacity. One specimen using one layer of PBO fabric had the fabric anchored beyond the two supports. This specimen showed an 80% increase in energy-dissipation capacity when compared to the control, with an increase

of 38 % compared to the specimen with 2-layer PBO without anchorage. FRCM systems improved the energy dissipation of the stack-bonded walls considerably, however the ductility of the strengthened walls was decreased. The authors noted it could be increased by using end anchorages in the FRCM system. Most of the walls failed due to de-bonding of FRCM or NSM systems from the masonry substrates, while the anchored specimen (PBO) failed due to the fiber slippage from the cementitious matrix within the FRCM system (Figure 2-16).

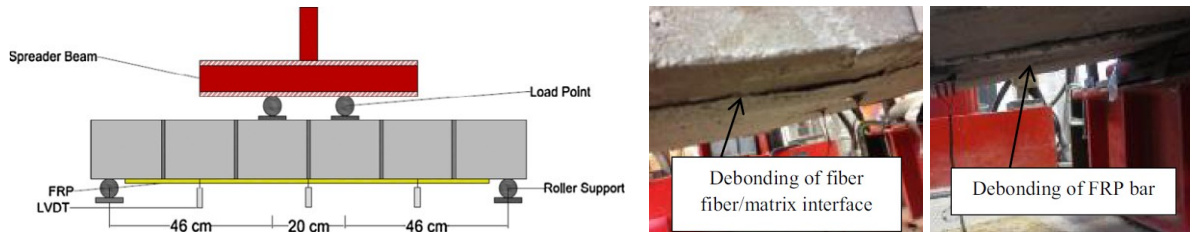


Figure 2-16. Test setup and test results (Al-Jaberi et al., 2018)

Papanicolaou, C. G., Triantafillou, T. C., Papathanasiou, M., & Karlos, K. (2008)

Papanicolaou et al. (2008) tested 12 masonry walls under cyclic out-of-plane loading to evaluate the effectiveness of TRM versus FRP retrofitting systems as shown in Figure 2-17. Specimens were divided into 2 groups along the shapes of the walls: Series A (1300 mm x 400 mm) with the failure plane parallel to the bed joints, Series B (400 mm x 1300 mm) with the failure plane perpendicular to the bed joints. The walls were strengthened on both sides with one or two layers of high strength carbon fiber using either cement mortars (TRM) or epoxy adhesives (FRP). The quasi-static increasing cyclic load was applied at a rate of 0.1 mm/s with the displacement increment of 1 mm. As a result, the flexural capacity was increased significantly with all retrofit methods. TRM strengthened walls mostly outperformed FRP strengthened walls in terms of the maximum load and displacement at failure when the failure mechanisms were related to the masonry as shown in Figure 2-17 (a). Whereas, when it was related to the textile (Figure 2-17 (b)), TRM was slightly less effective. As the number of strengthening layers increased, more improvement was observed.

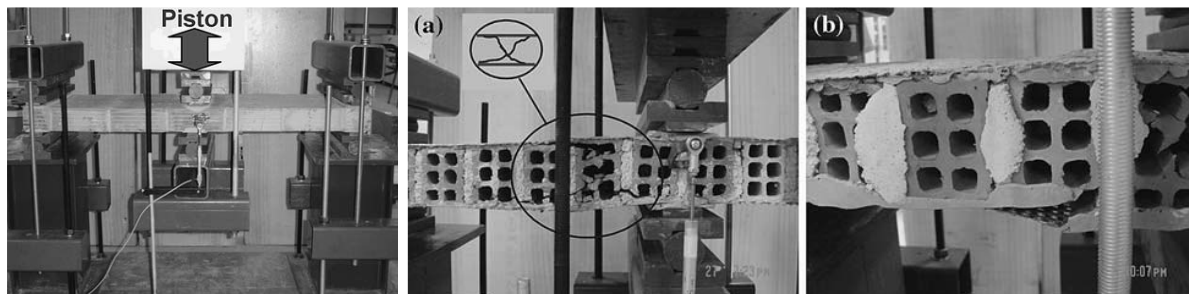


Figure 2-17. Experimental setup and test results; (a) Diagonal cracking of a masonry wall, (b) Debonding of the fiber layer at large displacements (Papanicolaou et al., 2008)

2.5.2. Wall elements

Kolsch (1998)

In this study, the carbon fiber cement matrix (CFCM) system was investigated as a mean of strengthening infill masonry walls under out-of-plane loading after conducting preliminary tests on unreinforced concrete beams. One infill masonry wall was constructed with sand lime bricks and was strengthened by 3 layers of carbon fabric and polymer modified mortar on the tension side of the wall (Figure 2-18). A quasi-static cyclic load was applied by inflating and deflating a pressure airbag on the other side of the wall. It was concluded that the CFCM system significantly increased out-of-plane load and deflection capacity of the wall.

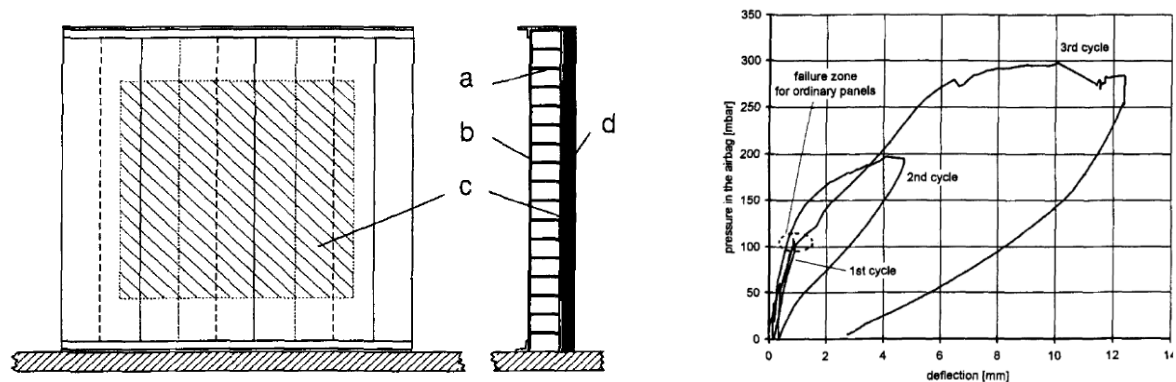


Figure 2-18. Test setup; a) masonry wall (3 m x 3 m), b) overlay, c) pressurized airbag (2 m x 2 m), d) reaction wall, and Load-Deformation Curve of the wall (Kolsch, 1998)

Ismail, N., & Ingham, J. M. (2016)

Ismail et al. (2016) tested URM specimens constructed with solid clay bricks with a low strength cement mortar which were strengthened with different types of fiber textiles (Aramid FRP, and glass FRP) at single faces. In-plane or out-of-plane reversed cyclic loadings were applied by using hydraulic jacks or airbags on 2 pier-spandrel assemblages (Figure 2-19) or 3 slender walls (Figure 2-20), respectively. Out-of-plane tests were performed in two stages with the reversed cyclic loading up to a drift of 4 %, then with one direction cyclic loading until failed. Galvanized steel meshes fixed by 100 mm long steel anchor assemblies were added to one specimen for each of in-plane and out-of-plane specimens to avoid TRM de-bonding. This additional anchorage system slightly increased the strength and ductility of the walls. The in-plane strength of TRM reinforced pier-spandrel assemblages was increased up to 136 % compared with that of non-reinforced one. They failed by either the rupture of textiles or the shear failure of piers with no cracks on the spandrels. The piers were not strengthened by TRM. The out-of-plane strength in tension of TRM reinforced walls was increased up to 7.86 times compared with that of non-reinforced one. The out-of-plane strength in compression was not increased.

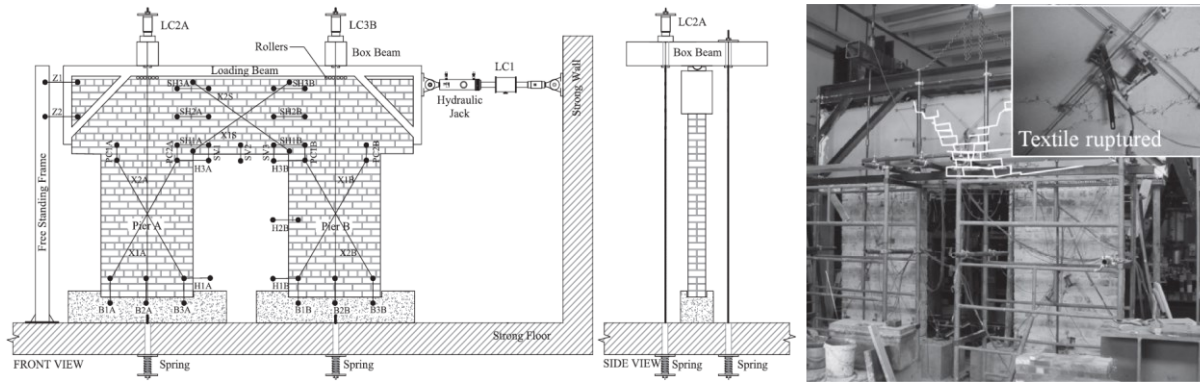


Figure 2-19. Test setup under in-plane reversed cyclic loading, and test result (Ismail et al., 2016)

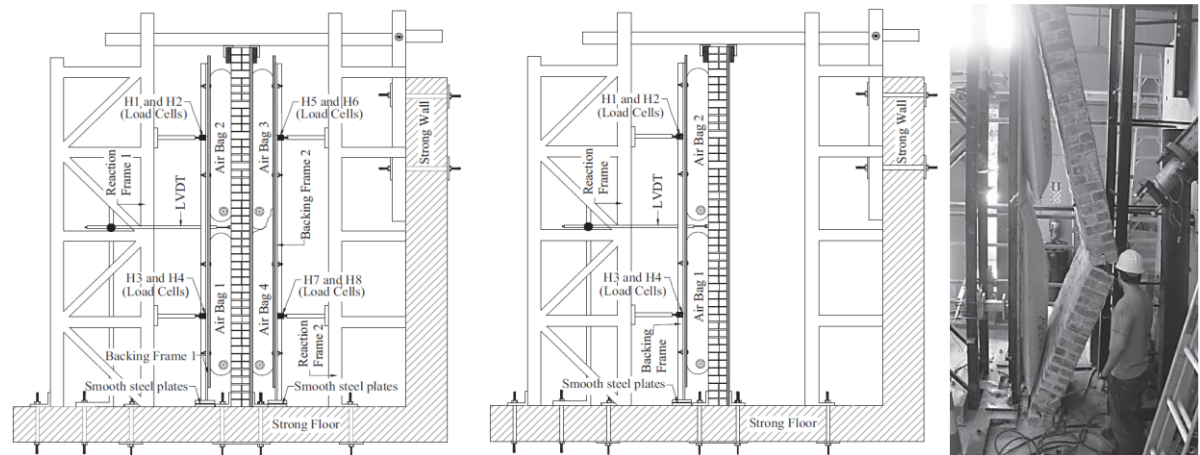


Figure 2-20. Test setup under out-of-plane reversed cyclic loading (left) and one directional cyclic loading (middle), and test result (Ismail et al., 2016)

Santis, S. D., Casadei, P., Canio, G. D., Felice, G. D., Malena, M., Mongelli, M., Roselli, I. (2016)

Santis et al. (2016) tested a full-scale U-shaped masonry structure consist of façade and two walls using a shake table to simulate seismic loading conditions. The façade and two side walls were not interlocked to simulate the weakness in older buildings. Initially, this specimen was shaken without any reinforcement resulting in complete separation between the façade and walls. Then, the damaged specimen was tested after being retrofitted with steel connectors and steel reinforced grout (SRG) strips as shown in Figure 2-21. Two types of reinforcement textiles were used with a natural hydraulic lime mortar. Two horizontal SRG strips using ultra high tensile strength steel textiles were applied onto the façade. Three (3) steel connectors were anchored at each ends to prevent the overturning of the façade and to transfer the seismic load to the side walls efficiently. The same reinforcements were applied to the side walls. One SRG strip using aramid-stainless steel textiles was applied on the façade vertically. SRG strengthening provided higher strength as it failed at an acceleration of 1.49g while the unreinforced one failed at 0.29g (see Figure 2-22). Steel connectors were efficient to prevent

the overturning of the façade as it behaved as one box-type structure rather than a vertical cantilever.

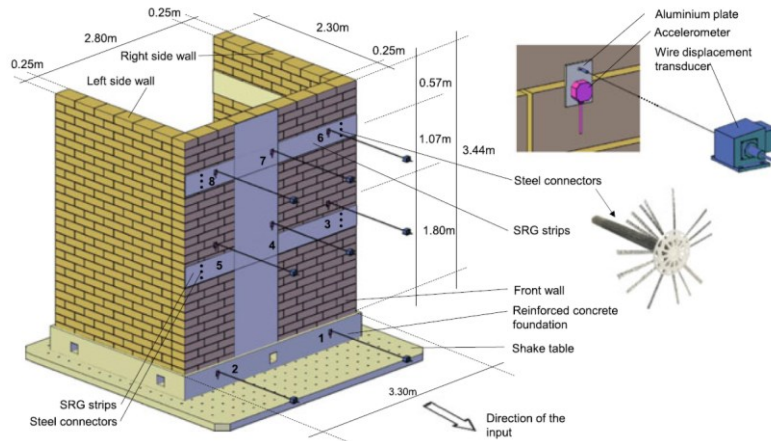


Figure 2-21. Specimen under study and test set up (Santis et al., 2016)

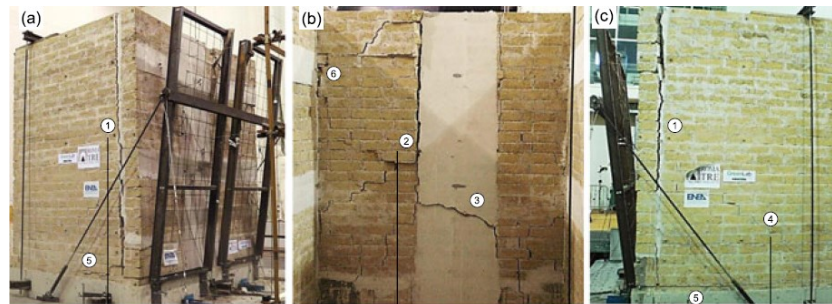


Figure 2-22. Retrofitted specimen after failure: (a) vertical cracks separating the façade from the side walls, (b) damage in the internal side of the façade, (c) damage in the side walls (Santis et al., 2016)

Babaeidarabad, S. and Nanni, A. (2015a)

Babaeidarabad et al. (2015a) tested 18 concrete block and clay brick masonry walls (1422 mm (H) x 1220 mm (L) x 102 mm (W)) to investigate the out-of-plane behaviour of un-strengthened and strengthened masonry walls. Concrete block walls consisted of 3 control walls, 3 reinforced walls with 1-ply of carbon FRCM, and 3 reinforced walls with 4-ply carbon FRCM. The same strengthening matrix was kept for the clay brick walls. An air bag was used to apply uniformly distributed lateral pressure (maximum 0.14 MPa) in three cycles of loading and unloading to the retrofitted specimens. The FRCM retrofit was installed on the tension side to enhance the flexural strength of the walls. The retrofit resulted in increases in the ultimate out-of-plane moment resistance. For the concrete masonry walls, it increased by 2.7 and 7.8 times using one and four plies, respectively compared to the control walls. For brick walls, it was increased by 2.8 and 7.5 times. The failure modes were either by flexure with FRCM slippage (1-ply) or by shear at the support (4-ply). It was found that FRCM retrofit increased the flexural strength,

stiffness, and pseudo-ductility. With 4-ply FRCM strengthening, however, the wall was controlled by the shear resistance showing lower pseudo-ductility compared to that with 1-ply retrofit.



Figure 2- Failure mode of wall specimens:
CMU-Control (a); CMU-1ply (b); CMU-4ply (c); CL-Control (d); CL-1ply (e); and CL-4ply (f)

Figure 2-23. Test setup and results (Babaeidarabad et al., 2015a)

Babaeidarabad, S. and Nanni, A. (2015b)

Babaeidarabad et al. (2015b) tested 9 concrete block masonry walls (1220 mm (H) x 1220 mm (L) x 92 mm (W)), and 9 clay brick masonry walls (1145 mm (H) x 1220 mm (L) x 92 mm (W)) to investigate the in-plane behaviour (i.e. shear stress-strain relationship, crack patterns, and failure modes) of un-strengthened and strengthened masonry walls. Concrete block walls consisted of 3 control walls, 3 reinforced walls with 1-ply carbon FRCM, and 3 reinforced walls with 4-ply carbon FRCM. This test matrix was kept for the clay brick walls. The retrofit was applied to both faces of the walls. Diagonal compression was applied on a top corner by using a double acting hollow plunger cylinders (maximum capacity of 299 kN). The FRCM retrofit resulted in important increases in shear capacity. For the concrete masonry walls, the ultimate load was increase by 2.4 and 4.7 times when using one and four plies, when compared to the control walls. For the clay brick walls, it was increased by 2.0 and 2.4 times.



Figure 2– Failure mode of wall specimens:
CMU-Control (a); CMU-1ply (b); CMU-4ply (c); CL-Control (d); CL-1ply (e); and CL-4ply (f)

Figure 2-24. Test setup and results (Babaeidarabad et al., 2015b)

Table 2-3. Summary of previous research on FRCM strengthening masonry walls

Author(s)	Testing Elements			Loading		FRCM (TRM) systems			
	Type	Detail	Dimensions (h x w x l)	Type	Properties	Applied Surface	Fabrics (no. of layers)	Matrix	Thickness
Papanicolaou et al. (2008)	Beam - like (URM)	6-hole Clay Brick	1.3 x 0.085 x 0.4 m 0.4 x 0.085 x 1.3 m	3 point	Cyclic loading (0.1 mm/s)	Double face	Carbon (1, 2) * Carbon (1, 2) for FRP	Polymer-modified cement mortar * Epoxy for FRP	Variable (one layer of mortar: 2 mm)
Conclusion	TRM outperformed FRP in terms of the maximum load and displacement at failure when the failure mechanisms were related to the masonry. When it was related to the textile, TRM was slightly less effective. As the number of strengthening layers increased, more improvement was observed.								
Harajli et al. (2010)	Beam - like (URM)	Concrete Block, Brick, Sandstone	1.0 x 0.1 x 0.4 m 1.0 x 0.1 x 0.3 m 1.0 x 0.1 x 0.2 m	4 point	Monotonic load (5 mm/min) Cyclic loading (0.1-0.5 mm/s)	Single face (for static) Double face (for cyclic)	Basalt / E-Glass (1), Steel mesh (1, 2)	Lime mortar cement based mortar	Variable (one layer of mortar: 2-3 mm, for base mortar: 5 mm)
Conclusion	TRM showed higher flexural strength (static), increased load and displacement capacity (cyclic). Highest improvement in strength was gained but was brittle with steel meshes. Combination of a basalt textile with a lime mortar showed the highest energy absorption and dissipation capacities, and the least stiffness and strength degradation.								
Kariou et al. (2018)	Beam - like (URM)	Solid clay brick	1.34 x 0.1 x 0.44 m (Single-wythe) 1.34 x 0.2 x 0.44 m (Double-wythe)	3 point	Monotonic load (1 mm/min)	Single face	Carbon (1), Basalt / Glass (3, 7),	Cement based mortar with polymers	Variable (3~13mm)
Conclusion	Epoxy-resin coated textile enhanced the bonding capacity and the max load capacity. The maximum load was increased as the number of TRM layers was increased due to the improved interlocking condition. Single-wythe walls were more efficient considering the ratio of the maximum load capacities of the strengthened vs. the control.								
Al-Jaberi et al. (2018)	Beam - like (RM)	Concrete Block	1.22 x 0.15 x 0.61 m	4 point	Cyclic loading (1.25 mm/min)	Single face	Carbon / PBO (1, 2) * Carbon (1), Glass (1, 2) for NSM	Cement based mortar	Variable (one layer of mortar: 5 mm)
Conclusion	FRCM and NSM systems improved the energy dissipation of the stack-bonded walls considerably. Ductility of strengthened walls was significantly decreased, however it could be increased by using the end anchorages in FRCM system, or strip bars in NSM system.								
Kolsch (1998)	Infill walls (URM)	Sand Lime Brick	3 x 0.24 x 3 m	UDL (Air bag)	Cyclic loading (Out-of-plane)	Double face	Carbon (3)	Polymer modified cementitious mortar	Approx. 4.5 mm
Conclusion	Retrofitting system was effective in strengthening the masonry walls against out-of-plane seismic loading by enhancing the bending resistance, and in prevents partial or complete collapse of walls (out-of-plane).								
Babaeidarabad et al. (2015a)	Infill walls (URM)	Concrete Block, Clay Brick,	1.4 x 0.1 x 1.2 m	UDL (Air bag)	Cyclic loading (max 0.14 MPa) (Out-of-plane)	Single face (tension) (out-of-plane)	Carbon (1, 4)	Cement based mortar	Variable (one layer of mortar: 5 mm)
Conclusion	Moment capacity, stiffness, and pseudoductility, depending on the amount of FRCM, enhanced. Pseudoductility is higher for lower FRCM amounts (4-ply strengthened wall failed by shear).								

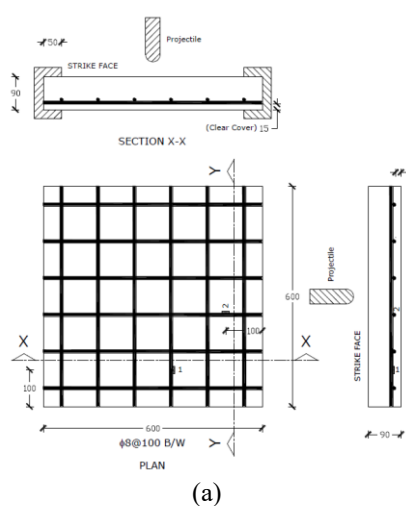
Author(s)	Testing Elements			Loading		FRCM (TRM) systems			
	Type	Detail	Dimensions (h x w x l)	Type	Properties	Applied Surface	Fabrics (no. of layers)	Matrix	Thickness
Ismail et al. (2016)	Infill walls (URM)	Solid Clay Brick	3.76 x 0.22 x 1.2 m	UDL (Air bag)	Reversed Cyclic (up to 4% drift), One Direction Cyclic (up to collapse)	Single face (tension) (out-of-plane)	Aramid FRP / Steel mesh (1)	Commercial adhesive mortar	5-7 mm
Conclusion	The strength in tension of TRM reinforced walls was increased up to 7.86 times compared with that of non-reinforced one.								
Santis et al. (2016)	Infill walls (URM)	Tuff Brick	3.44 x 0.25 x 3.3 m (Façade) 3.44 x 0.25 x 2.3 m (Side walls)	Shake table	Seismic inputs from six natural recordings in Italy	Single face partially (external for façade, internal for side walls) (out-of-plane)	Ultra high tensile strength steel (1) Aramid-stainless steel (1)	Natural hydraulic lime mortar	7-8 mm
Conclusion	SRG strengthening provided higher strength than the unreinforced and the steel-bar reinforced. SRG connectors were efficient to prevent the overturning of the façade.								
Babaeidarabad et al. (2015b)	Infill walls (URM)	Concrete Block, Clay Brick,	1.2 x 0.09 x 1.2 m (Concrete) 1.1 x 0.09 x 1.2 m (Clay)	UDL (Air bag)	Plunger cylinder (max 299 kN) (In-plane)	Both faces (In-plane)	Carbon (1, 4)	Cement based mortar	Variable (one layer of mortar: 5 mm)
Conclusion	FRCM increased the shear capacity of the walls. For concrete masonry walls, the ultimate in-plane load was increase by 2.4 and 4.7 times using one and four plies, respectively compared to the control walls. For clay brick walls, it was increased by 2.0 and 2.4 times								
Ismail et al. (2016)	Pier-spandrel assemblages	Solid Clay Brick	0.94 x 0.22 x 4.42 (Spandrel) 1.71 x 0.22 x 1.19 (2 Piers)	Lateral loading (Hydraulic ram)	Reversed cyclic pseudo-static	Single face on the spandrel (in-plane)	Aramid FRP / Steel mesh (1)	Commercial adhesive mortar	5-7 mm
Conclusion	The in-plane strength of TRM reinforced pier-spandrel assemblages was increased up to 136 % compared with that of non-reinforced one.								

2.6. PREVIOUS RESEARCH ON USE OF FRCM TO STRENGTHEN STRUCTURES TESTED UNDER EXTREME DYNAMIC LOADING

This section reviews previous research on the impact and blast performance of FRCM strengthened concrete and masonry structures. Since FRCM is a comparably new retrofitting method, very limited research exists on this topic, especially in the case of blast research. Moreover, no research exists on the use of FRCM to retrofit masonry structures subjected to extreme dynamic loading (impact or blast). Table 2-4 provides a summary of previous research.

Abbas, H., Almusallam, T., Al-salloum, Y., Siddiqui, N., & Abadel, A. (2016)

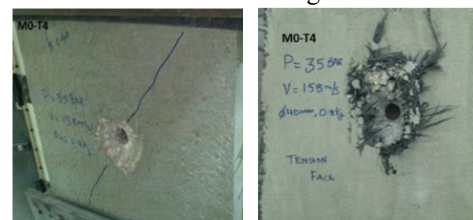
In this study reinforced concrete (RC) slabs (600 mm square, 90 mm thick) strengthened by either TRM or FRP systems were investigated under impact loads. A hemispherical nose steel projectile (Dia. 40 mm) shown in Figure 2-25 (a) was used with tests conducted at different impact velocities from 108 to 158 m/s. Three (3) slabs were strengthened on the rear side using a single layer of carbon FRP with epoxy resin. Another 3 slabs were strengthened on the rear side using 2 layers of carbon fiber textiles with a polymer modified cement mix. A single layer of FRP sheet and 2 layers of textile were selected for comparison purposes since they had the same quantity of carbon fibers. Four (4) RC slabs were tested as controls with no strengthening. A gas gun was used to strike the projectile normal to the specimens. Penetration depth, and crater size on the front and rear faces were investigated for the different projectile velocities. Figure 2-25 (b) and (c) show some of the test results using CFRP and TRM strengthening, respectively. It was concluded that both strengthening systems were efficient to reduce local damage and secondary debris of RC slabs subjected to projectile impacts. The ballistic limit velocity of CFRP and TRM systems was increased by 15 % and 21 %, respectively compared with that of the control specimens. TRM performed equally or better than CFRP reinforcement.



(a)



(b) Front and rear side of CFRP strengthened RC slab (129 m/s)



(c) Front and rear side of TRM strengthened RC slab (158 m/s)

Figure 2-25. (a) Preparation of test specimens, and (b, c) test results (Abbas et al., 2016)

Liu, S., Zhu, D., Ou, Y., Yao, Y., & Shi, C. (2018)

Liu et al. (2018) studied the impact resistance of 10 basalt textile reinforced concrete (BTRC) specimens having dimensions of 60 x 60 x 15 mm using different impact velocities (0.77 m/s, 2.31 m/s, and 4.62 m/s), different temperatures (-25, 0, 25, 50, and 100°C), and different numbers of textile layers (4 and 6) using a drop hammer of 3.75 kg. The effect of different boundary conditions (free and fixed) as shown in Figure 2-26 (a) was also investigated. Increasing velocity resulted in higher maximum impact force and energy absorption. More layers of textiles resulted in higher impact resistance. The fixed boundary condition increased maximum impact force, but decreased energy absorption of the BTRC specimens. Increasing temperatures were found to weaken the BTRC specimens showing lower impact force and energy absorption. However, the failure patterns were not affected considerably with increasing temperatures. Some examples of the test results can be seen in Figure 2-26 (b).

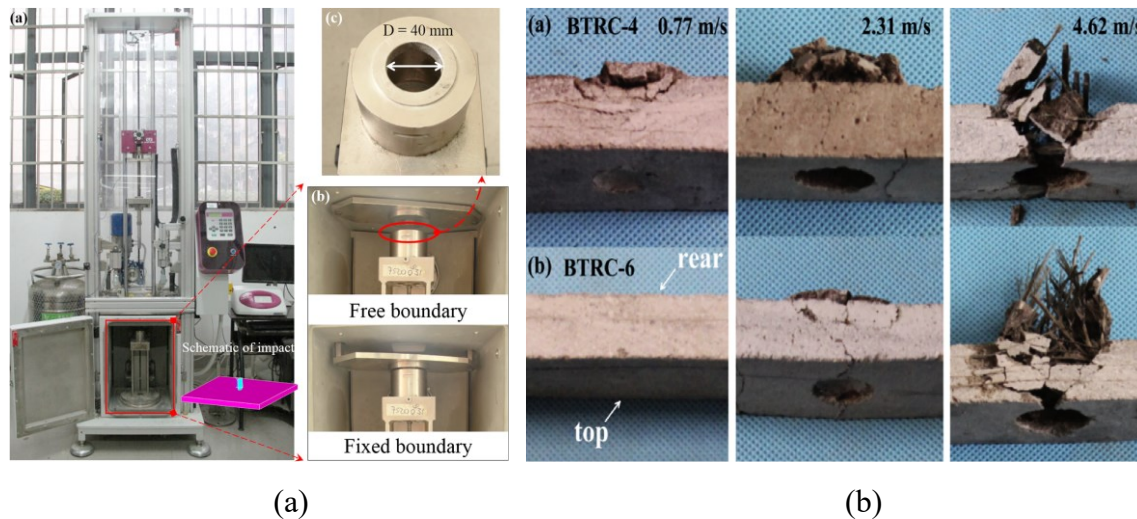


Figure 2-26. (a) Test setup and (b) examples of test result (Liu et al., 2018)

Zhu, D., Gencoglu, M., & Mobasher, B. (2009).

In this study, 18 panels made with Alkali Resistant (AR) glass fabric reinforced cement composites were tested to evaluate their response against dynamic impact loading. 6 or 8 layers of composite laminates, equivalent to 4.0 or 6.0 % of reinforcement content by volume, respectively, were used to build cement based panel specimens (50 x 150 x 7-13 mm). A drop hammer was released onto the specimens at heights of 50, 100, 200, and 250 mm to produce impact loading under 3-point bending as shown in Figure 2-27 (a). The specimens were mounted on their supports either vertically (beam types) or horizontally (plate types). The beam types showed higher load carrying capacity and lower deflection than the plate type specimens. The beam type specimens with 8 layers of textiles resulted in higher maximum flexural stress and better energy absorption capacity than with 6 layers of textiles. The opposite effect, however, was shown from the plate type specimens due to inter-laminar shear failure. For 6-layer beam type specimens, increasing drop heights led to increasing of the total absorbed energy. For 8-layer plate specimens, it increased up to 100 mm height, then decreased. The

beam-type specimens were not failed completely, while the plate specimens failed completely with inter-laminar shear failure mode as shown in Figure 2-27 (b).

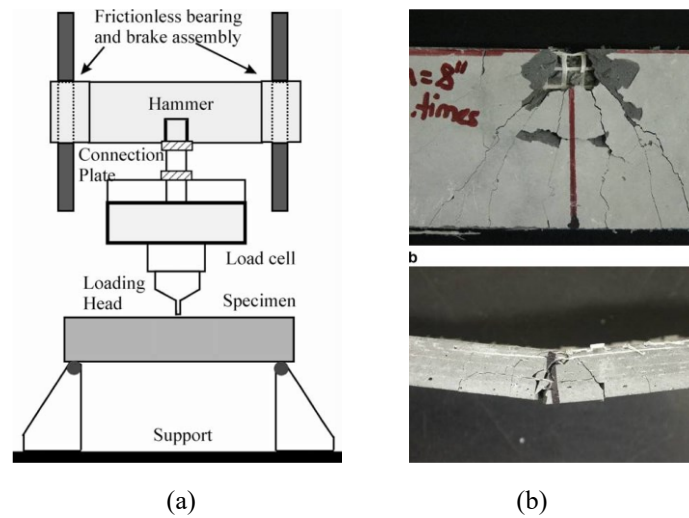


Figure 2-27. (a) Test setup, and (b) test results (beam types (up), plate types (down)) (Zhu et al., 2009)

Silva, F. de A., Butler, M., Mechtcherine, V., Zhu, D., & Mobasher, B. (2011)

A few research studies have also been conducted to examine the high-strain rate response of TRC and FRCM systems. Silva et al. (2011) investigated the effect of strain rates on textile-reinforced concrete (TRC) plates of 300 x 50 x 10 mm. Five plates were tested with low-speed tensile strain rates of 10^{-4} to 10^{-1} using an Instron Universal servo-hydraulic machine (Figure 2-28 (a)). Six specimens were tested at higher strain rates of 5, 20, and 50 s^{-1} using a servo-hydraulic MTS (Figure 2-28 (b)). 3 layers of alkali-resistant (AR) glass textiles were used as reinforcement. Finely grained matrix was used either without or with short AR-glass fibers (dia. $14 \text{ }\mu\text{m}$, 6 mm long) of 0.5 % by volume as secondary reinforcement. The effect of gauge length from 50 to 150 mm was investigated at a rate of 20 s^{-1} . Tensile strength, strain capacity, work-to-fracture, and first crack strength were increased under low-rate tensile loadings for all tested specimens. Tensile strength and work-to-fracture increased up to 50 s^{-1} , but the strain capacity decreased due to the increased stiffness of the composites in the post-crack region. Short fibers in the matrix resulted in higher first-crack and tensile strength. Increased gauge length resulted in decreased tensile strength and strain capacity.

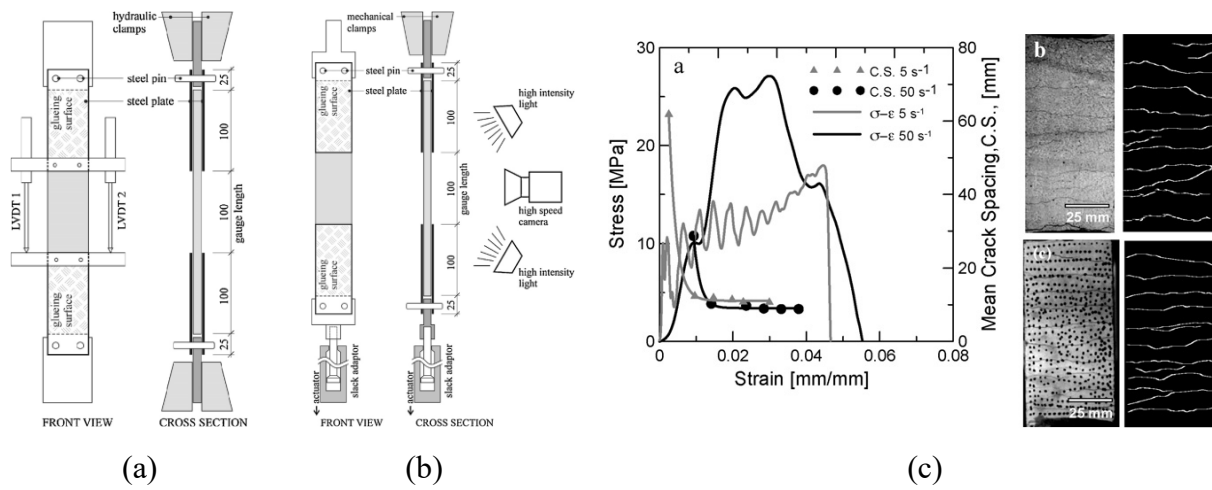


Figure 2-28. (a) Test setup for low strain rate, (b) High strain rate, and (c) results (Silva et al., 2011)

Table 2-4. Summary of impact and blast research on use of FRCM to strength structural elements

Author(s)	Testing Elements			Loading		FRCM systems			
	Type	Detail	Dimensions (h x w x l)	Type	Properties	Applied Surface	Fabrics (no. of layers)	Matrix	Thickness
Abbas et al. (2016)	RC slab	Reinforced with Ø8 x 100 mm	600 x 600 x 90 mm	Strike of hemispherical nosed steel projectile (40mm)	Striking velocity of 108-152 m/s	Rear face	Carbon (2) * 1 layers for FRP	Cement based mortar * Epoxy for FRP	Unknown
Conclusion	CFRP and TRM were effective in reducing the local damage of RC slabs. Production of flying debris were reduced considerably. TRM reinforcement performed equally or better than CFRP reinforcement .								
Liu et al. (2018)	Concrete panel	Reinforced with basalt textiles	60 x 60 x 15 mm	Drop weight	Impact velocity of 0.77, 2.31, and 4.62 m/s * Under different temperature; -25, 0, 25, 100°C	Embedded within the concrete panel	Basalt (4, 6)	Concrete (TRC)	15 mm
Conclusion	The fixed boundary condition increased maximum impact force, but decreased energy absorption. Increasing temperatures weaken the BTRC specimens showing lower impact force and energy absorption. The failure patterns were not affected considerably with increasing temperatures.								
Zhu et al. (2009)	Concrete panel	Reinforced with AR glass textiles	25 x 150 x 7-9 (with 6 layers), 25 x 150 x 11-13 (with 8 layers) mm	Free fall drop hammer on 3 point bending	Drop height of 50, 100, 200, and 250 mm	Embedded within the concrete panel	AR glass (6, 8)	Concrete (TRC)	7-13 mm
Conclusion	The max. flexural stress and absorbed energy increased with the number of fabric layers. The total absorbed energy increased with higher drop height. The beam specimens were not failed completely, while the plate specimens failed completely with inter-laminar shear failure mode.								
Silva et al (2011)	Concrete panel	Reinforced with AR glass textiles	300 x 50 x 10 mm	High tensile strain rate	Low-rate; 10^{-4} to 10^{-1} High-rate; 5, 20, and 50 s^{-1}	Embedded within the concrete panel	AR-glass (3)	Concrete * with or without short AR-glass fibers	- (TRC)
Conclusion	Tensile strength and work-to-fracture increased up to 50 s^{-1} , but the strain capacity decreased due to the increased stiffness of the composites in the post-crack region. Short fibers in the matrix resulted in higher first-crack and tensile strength. Increased gauge length resulted in decreased tensile strength and strain capacity.								

2.7. SUMMARY OF LITERATURE REVIEW

This section summarizes the key findings from the literature review:

- The FRCM system is a viable alternative to FRP with the main advantages of compatibility with the properties of concrete and masonry substrates, ease of installation, porous matrix structure, good performance at elevated temperatures, high durability, and ease of reversibility. The tensile stress-strain relationship of FRCM, which is assumed to have a bi-linear response in design, is affected by the types of fabrics, numbers of fabric layers, geometry of the fabrics, fabric coating, type of mortar, etc. The minimum anchorage length of FRCM systems should be more than 152 mm (6") for any type of substrate to develop the effective stress capacity of FRCM. The ACI 549.4R-13 (2013) design guidelines can be used for the repair and strengthening of concrete and masonry structures with FRCM. The guidelines are limited to out-of-plane bending, in-plane shear and axial loading conditions.
- Previous research confirms that FRCM can improve the flexural and shear capacities of reinforced concrete beams when the retrofit is applied on the tension face (bottom) and shear faces (sides). The enhancement is affected by the number of FRCM plies, types of fabrics (i.e. carbon, glass, basalt, PBO, etc.) and application method (i.e. NSE, EB, U-shape, etc.). In some research, the retrofit reduced the deflection ductility of the beams.
- Previous research confirms that FRCM composites are effective in strengthening masonry walls under both out-of-plane and in-plane loading. In the case of out-of-plane loads the use of FRCM improves flexural strength, stiffness, displacement capacity and ductility of URM walls. The improvement is proportional to the number of strengthening layers. However, the failure mode can be governed by shear when using increased number of FRCM plies. Research also demonstrates the effectiveness of using FRCM to improve the in-plane shear resistance and seismic response of URM walls.
- Very limited studies have investigated the performance of FRCM strengthened elements under extreme dynamic loading, such as impact or blast. Moreover, there exists no research on the use of FRCM to retrofit masonry structures under impact or blast. However, limited research on small-scale RC slabs and concrete panels tested under projectile or drop hammer testing demonstrate the effectiveness of FRCM strengthening in reducing local damage and flying debris under impact loads.

CHAPTER 3. EXPERIMENTAL PROGRAM

3.1. CHAPTER OVERVIEW

In this chapter, the details of the test specimens including the materials used and their properties are described. The application method of the FRCM composites and their mechanical characteristics are also described. The test setup, instrumentation, and shock tube are also detailed.

3.2. DETAILS OF TEST SPECIMENS

3.2.1. Specimen overview

As part of the experimental program, two masonry walls were constructed and strengthened with the FRCM retrofit: a concrete masonry (CMU) infill wall and a stone load-bearing wall. The walls were built by a professional mason and the FRCM installation was performed by professional applicators who had been trained properly by the manufacturer. Hand trowels were used to apply the mortar, and to smooth the final surface with a satisfactory uniform thickness. Further details are provided in the following sub-sections.

3.2.1.1. CMU wall

The first wall in the research program was an infill wall built with concrete masonry units (CMU) and had planar dimensions of 2 m x 2 m. The CMU blocks used in the construction had a nominal size of 100 mm (W) x 200 mm (H) x 400 mm (L) (actual size 90 mm x 190 mm x 390 mm) as shown in Figure 3-1(a). The CMU wall was built in a running bond pattern, with 10 mm thick mortar joints with Type S mortar. The wall was built on a steel HSS (152 mm x 152 mm x 6.4 mm, 2440 mm long) beam for the purpose of transportation and setup. The bottom layer of CMU blocks was placed two inches back from the HSS beam to ensure that the FRCM retrofit would be flush with the HSS. Figure 3-1(b) shows the dimension and the running pattern of the wall. About one third of the wall height was constructed per day to ensure that the mortar joints would set without unnecessary sinking due to the self-weight of additional blocks. To promote arching action, a fast set high strength mortar was applied to fill the top gap of the wall. The wall was cured for more than 28 days and kept safe from overturning by a wood safety frame until the retrofit was applied. Figure 3-2 shows some of the procedures of constructing the wall.

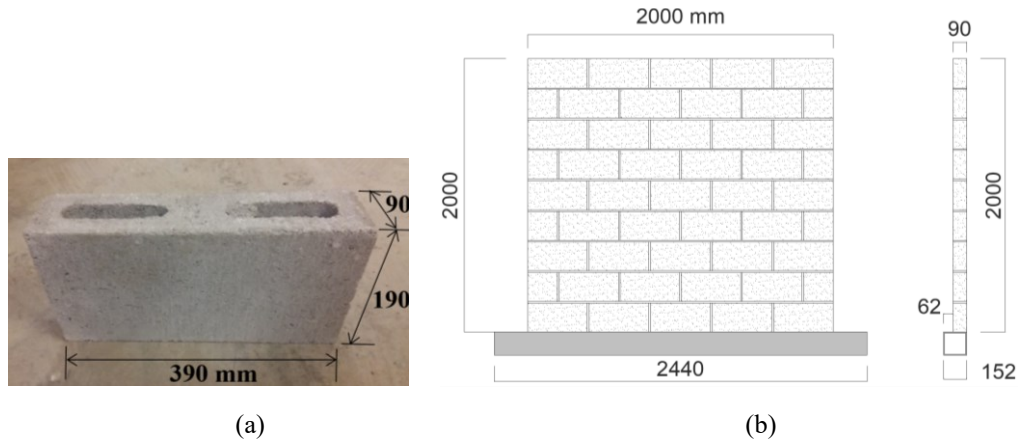


Figure 3-1. (a) Actual size of the CMU block, and (b) Detail of the CMU wall (front and side view)

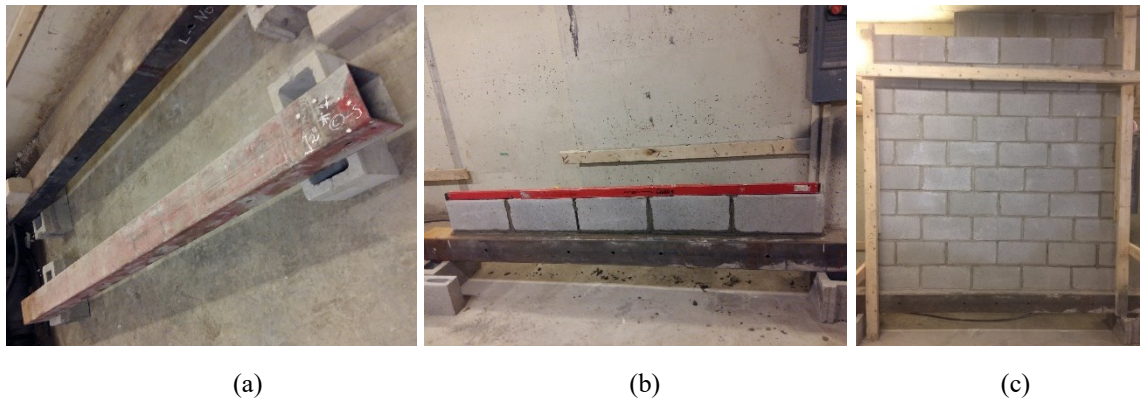


Figure 3-2. Construction of CMU wall: (a) Bed HSS, (b) Horizontal Leveling, (c) Completion with safety wood frame

3.2.1.2. Stone wall

The second wall in this research program was a load-bearing wall built with stone masonry. Similar to the first wall, the stone wall had planar dimensions of 2 m x 2 m. 100 mm (4 in) masonry blocks were used to construct the wall. The stone blocks had an actual width and length of 90 mm and 600 mm, respectively. As shown in Figure 3-3, the height of the blocks varied from 60 mm, 133 mm and 206 mm. Unless the whole length of the unit was used, the block was cut into the required length with a hammer and a chisel. Like the CMU wall, Type S mortar was used for the joints with a thickness of 10 mm, and one third of the wall was constructed per day. The wall was built on the same size HSS and placed back to ensure that the final wall surface was flush with the HSS considering the thickness of the FRCM retrofit. The gap between the top of the wall and the Shock Tube frame was filled with high-strength mortar. The wall cured more than 28 days before the retrofit was applied, and was kept safe from overturning by a wood safety frame. Some of the procedures of constructing the wall can be seen in Figure 3-4.

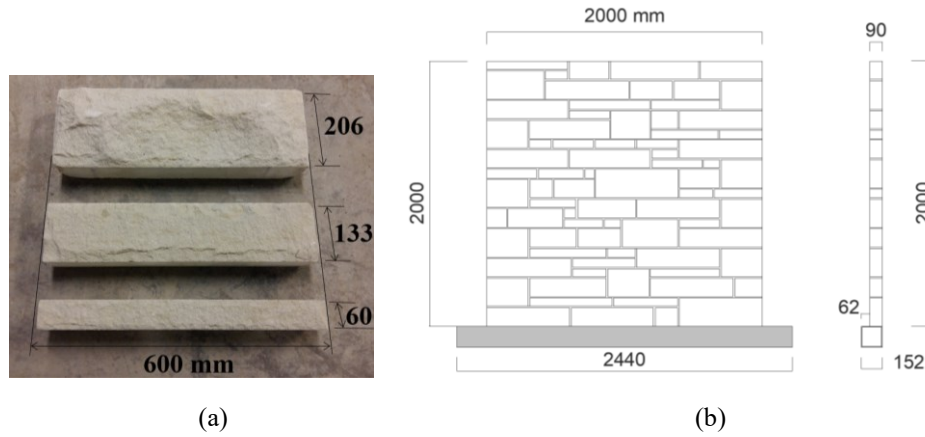


Figure 3-3. (a) Actual size of the stone units, and (b) Detail of the stone wall (front and side view)

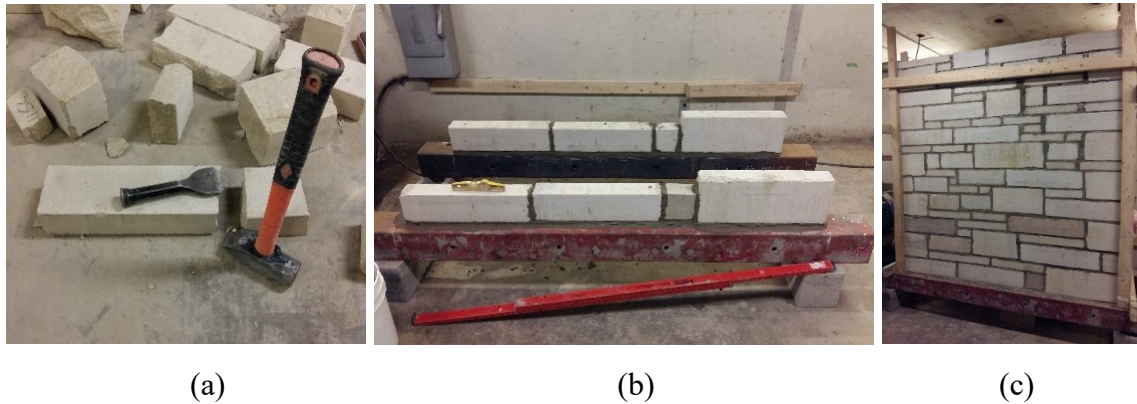


Figure 3-4. Construction of Stone wall: (a) Chisel and hammer, (b) Horizontal Leveling, (c) Completion with a safety wood frame

3.2.1.3. FRCM strengthening

The Fabric-Reinforced Cementitious Matrix (FRCM) strengthening system used in this research consisted of two components: a high-performance cementitious mortar and a carbon-fiber grid. Since the walls were tested in one-way bending, a unidirectional carbon grid was used in this research. The vertical (black) strands in Figure 3-5(a) are the main carbon strands designed to resist the loads in the longitudinal direction, and the horizontal (white) lines are supplementary plastic lines to keep them in position. The average distance between two strands was 17 mm, or 4 inches per 6 strands. The carbon fabrics were shipped as a roll (Figure 3-5(b)). The cement based mortar was reinforced with polypropylene fibers and was used as the binder for the retrofit. Figure 3-6 shows the mortar powder and its bag. The length of a polypropylene fiber was measured 6 mm, or 1/4in as shown in Figure 3-6 (b).

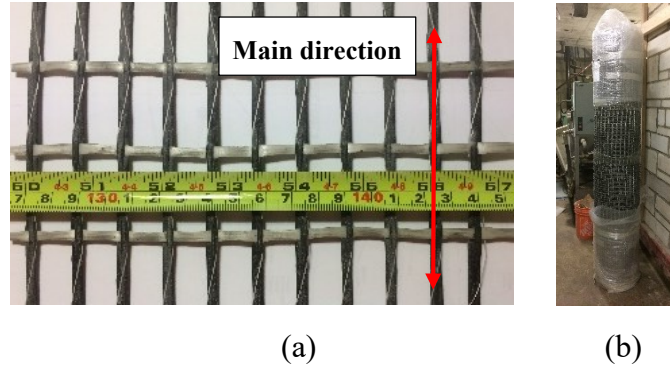


Figure 3-5. Unidirectional carbon fiber fabrics; (a) Grid size, (b) One roll of fabrics (2 m high)

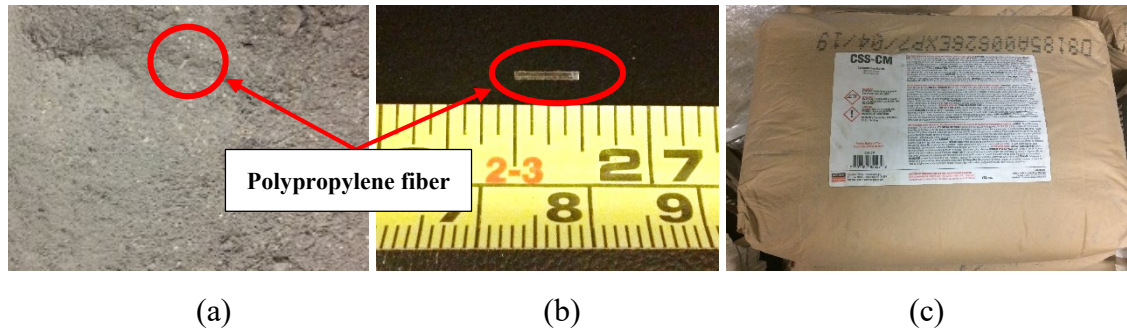


Figure 3-6. Polypropylene-reinforced mortar: (a) Mortar powder, (b) Polypropylene fiber, (c) Mortar bag

The installation of the FRCM composite system followed the steps below:

- 1) Surface preparation: All foreign materials such as dust were removed from the wall surface to secure the bond between the substrate (concrete or stone units) and the mortar. The substrate was watered for a day to have a saturated surface dry condition. (Figure 3-7(a))
- 2) Mortar mixing: An electric mixer was used to mix the mortar with the required amount of water to achieve proper consistency following the manufacturer's instructions (i.e. "2.8-3.2 L of water per 24.9 kg bag"). (Figure 3-7(b))
- 3) Application: A 0.5 in thick mortar layer (base mortar) was first applied onto the wall using a hand trowel. One layer of carbon grid was then placed immediately on top of the base layer. Two 0.25 in thick mortar layers were then applied for the following two layers of carbon grids. A final 0.5 in thick mortar layer (final layer) was then applied. The surface of the final layer was smoothed by trowel to get uniform thickness. (Figure 3-7(c)-(e))
- 4) Curing: After application of the FRCM composites, the wall was cured for 5 days with water (using wet burlap) to prevent premature drying or cracking. (Figure 3-7(f))

The procedure of FRCM installation was the same for both walls, and the main steps of installation are shown in Figure 3-7. The average total thickness of the FRCM strengthening system was 45 mm (1.8 in) for the CMU wall, and 43 mm (1.7 in) for the stone masonry wall. In both cases, 3 layers of FRCM composites were applied onto the walls.



Figure 3-7. FRCM application: (a) Surface Preparation, (b) Mortar mixing, (c) Base mortar (1/2''), (d) Additional mortar (1/4''), (e) Final mortar (1/2''), (f) Water curing with burlap and plastic tarp

3.2.2. Description of test specimens

Two masonry walls were built and retrofitted with FRCM composites as part of this research. The performance of the FRCM retrofit was evaluated using the results of two control walls from previous research at the University of Ottawa. These control walls had no retrofit but had similar properties to the retrofit specimens. Table 3-1 describes the details of the four specimens with an indication of masonry type (CMU, stone), wall type (infill or load-bearing) and strengthening method. The specimens tested for this research are identified as “R-C-N” and “R-S-L”. The first letter “R” indicates that the wall was retrofitted. The second letter describes the masonry type: “C”

for CMU or “S” for Stone, respectively. The last letter designates the axial loading conditions: “N” for non-load bearing (infill) and “L” for load bearing. The walls from previous researchers have the same nomenclatures as those used in their original thesis (URM-1 is an unreinforced concrete masonry infill wall and URM-S-L is an as-built Stone load-bearing wall).

An axial load of 171kN was applied on the stone masonry wall using a hydraulic pump and two hydraulic jacks which reacted against the lab strong-floor. The axial load level corresponds to 4% of the nominal concentric axial capacity (P_0) of the stone wall (without the retrofit). This value was computed using the following equation from the CSA S304.1-04 (2004) standard:

$$P_0 = f'_m A_e \quad [3-1]$$

where, f'_m is the average masonry prism strength, and A_e is the effective wall cross-sectional area.

Table 3-1. Description of Test Specimens

Wall	Type (Axial Load)	Retrofit	Tested by
URM-1	CMU Infill (-)	None (Control)	1
R-C-N		FRCM (1.8” thick, 3-layer carbon grids)	3
URM-S-L	Stone Load Bearing (171kN)	None (Control)	2
R-S-L		FRCM (1.7” thick, 3-layer carbon grids)	3

1. Ciornei (2012), 2. Gandia (2019), 3. Author

3.2.2.1. URM-1

URM-1 is a non-load bearing (infill) unreinforced concrete masonry wall tested by Ciornei (2012). This control wall had dimensions of 2 m x 2 m and was constructed with standard CMU blocks with dimensions of 100 mm x 200 mm x 400 mm and Type S mortar. The gap produced by the height differences between the wall and the Shock Tube frame was reduced to 6 mm using structural steel filler to activate arching action, which can be considered as a retrofit. Two horizontal HSS (152 mm x 152 mm x 6.4 mm) supports were used on the top and bottom of the wall to prevent its out-of-plane translation during the blast tests, resulting in simply supported boundary conditions. The horizontal HSS beams were secured to the shock-tube frame using four vertical HSS (76 mm x 76 mm x 6.4 mm) members which were placed at both ends of each horizontal HSS. The detailed configuration of URM-1 is shown in Figure 3-8.

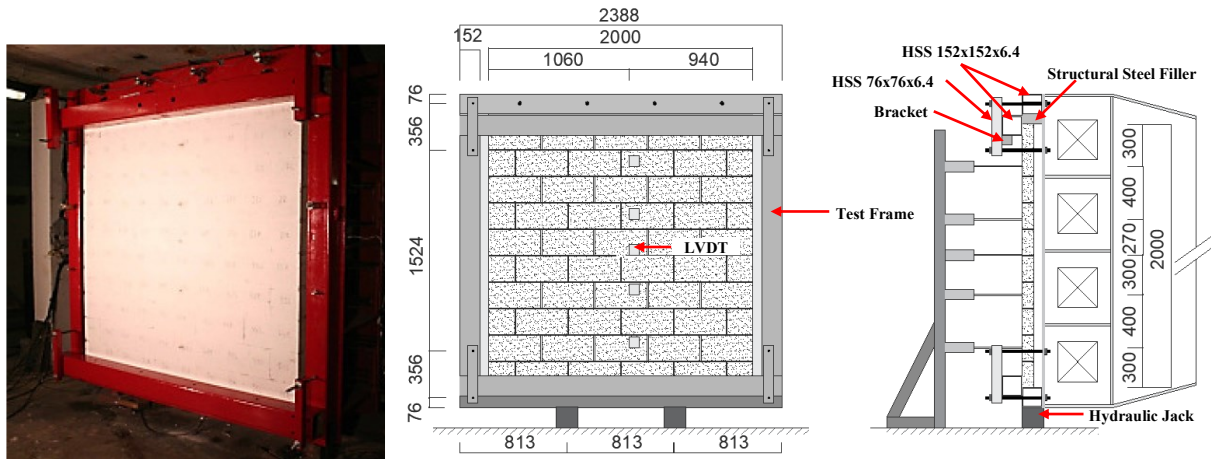


Figure 3-8. Detail of URM-1 (Dimensions in mm) (Ciornei, 2012)

3.2.2.2. *R-C-N*

R-C-N is a CMU infill wall retrofitted with FRCM composites. The wall was built using the same types of CMU blocks and mortar used in URM-1. Three layers of carbon grids with a total FRCM thickness of 46 mm (1.8 in) were installed onto the wall before it was moved to the Shock Tube. Arching action was promoted by filling the gap between the Shock Tube frame and the top of the wall using high strength mortar. The top and bottom supports consisted of 13 mm thick steel plates (2200 mm x 310 mm) which were fixed with 19 mm bolts to the top and bottom of the Shock Tube frame to prevent out-of-plane translation. Stiffeners were used to weld the plate ends to the top and bottom HSS beams. Figure 3-9 explains the details of the setup.

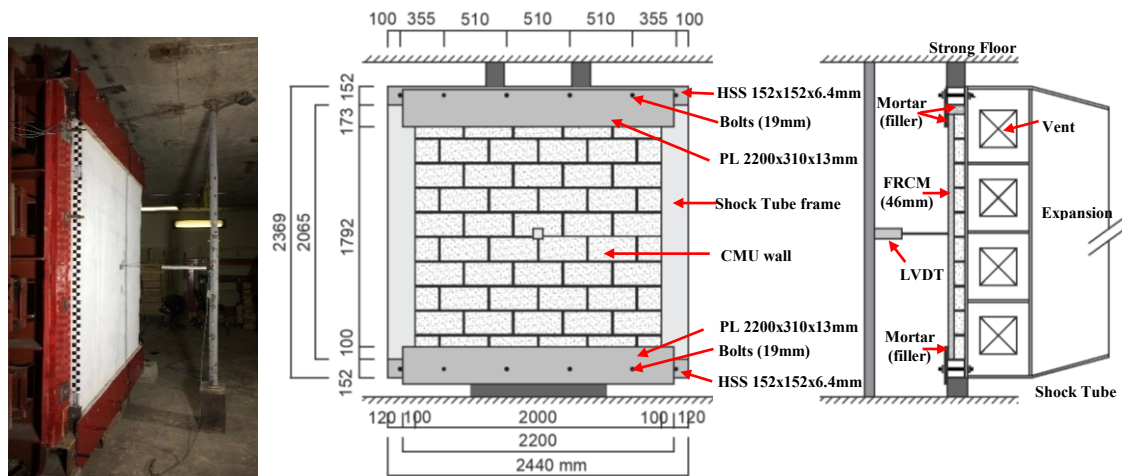


Figure 3-9. Detail of R-C-N (FRCM is not shown on the front view)

3.2.2.3. *URM-S-L*

URM-S-L is a control stone load-bearing wall tested by Gandia (2019). The wall was 2 m x 2 m in plan and was built with 100 mm (4 in) stone blocks. The same stone units used in wall R-S-L and Type S mortar were used. The gap on top of the wall was filled with high strength mortar to

activate arching action. The same steel plates used for R-C-N and R-S-L were used to prevent out-of-plane translation. The vertical gaps between the plates and the wall surface were filled with mortar to ensure that the plates were flush with the wall. An axial load corresponding to 4% of the nominal axial capacity of the wall (171 kN) was applied using a hydraulic pump and two jacks which reacted against the laboratory strong-floor as shown in Figure 3-10.

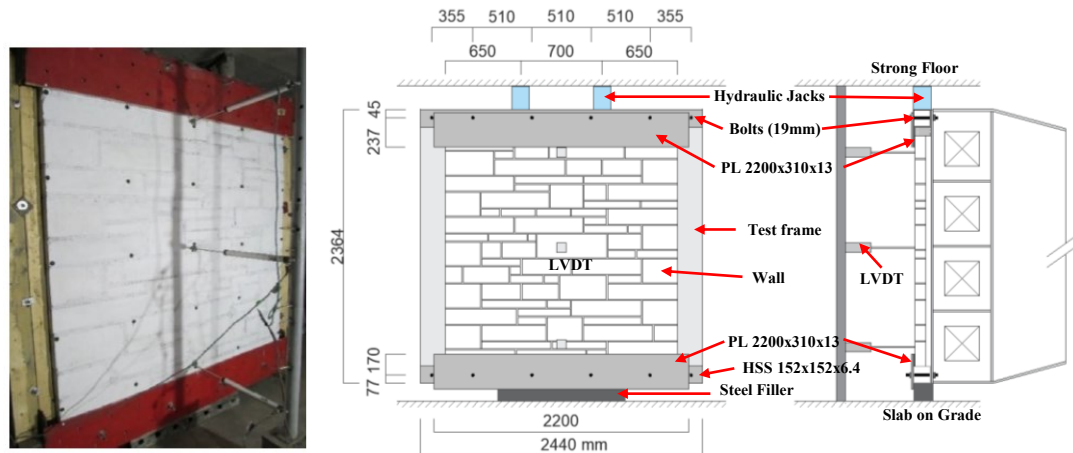


Figure 3-10. Detail of URM-S-L (Gandia, 2019)

3.2.2.4. R-S-L

R-S-L is a stone load-bearing wall retrofitted with FRCM composites. The same types of stone units and mortar used for URM-S-L were used in this wall. Three layers of carbon fiber textile were applied with a total thickness of 43 mm (1.7 in), and the gap on top of the wall was filled with high strength mortar for arching action. The same steel plates and bolts were used to prevent out-of-plane translation at the supports. It should be noted that an additional small HSS (50 mm x 50 mm x 4 mm) was used to prevent the top steel plate from excessive rotation during testing. The gaps between the wall and steel plates were filled with high strength mortar. The same axial load of 171 kN was applied as URM-S-L. Figure 3-11 shows the detail of R-S-L.

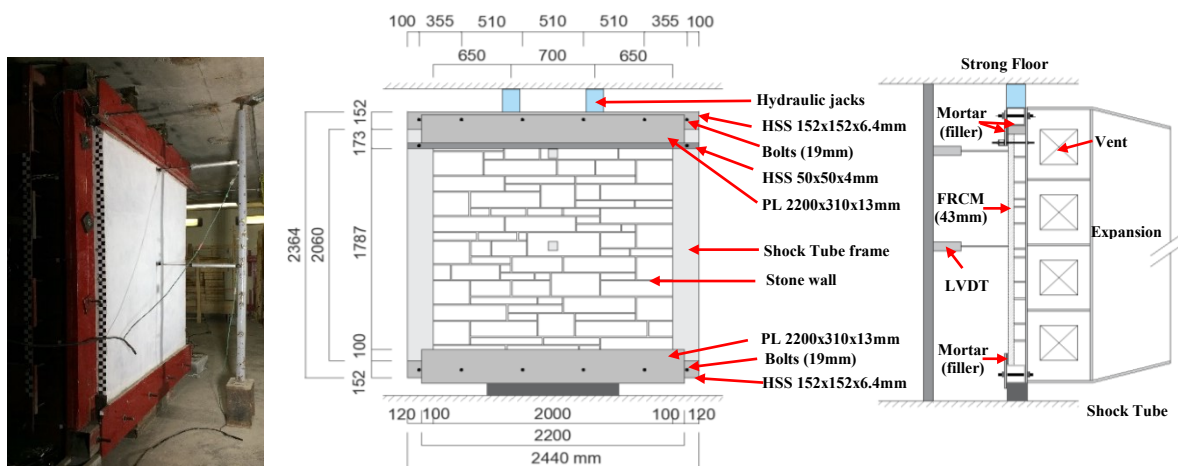


Figure 3-11. Detail of R-S-L (FRCM is not shown on the front view)

3.3. TEST SETUP AND DATA ACQUISITION

3.3.1. Test setup

3.3.1.1. Shock tube

The University of Ottawa Shock Tube is a blast simulator that can be used to safely test various types of structural elements such as walls, slabs, columns and beams using compressed air. It is comprised of four sections which include: a driver section, a diaphragm section, an expansion section and end-frame as shown in Figure 3-12 (a). By increasing or decreasing the driver length, which varies from 305 mm (12 in) to 5185 mm (204 in) with 305 mm (12 in) increments (Figure 3-12 (d)), the reflected pressure and impulse can be adjusted as shown in Figure 3-13. Figure 3-13 also compares the equivalent TNT mass, standoff distance, and scaled distance for different driver length/pressure combinations. Two sets of 32 in x 32 in aluminum foils with capacities which are slightly higher than half of the desired driver pressure are inserted at each side of the diaphragm section (Figure 3-12 (c)). The nominal strength of aluminum foils is 5 psi [34 kPa], 13 psi [90 kPa], or 25 psi [172 kPa] with corresponding thickness of 0.005 in, 0.008 in, or 0.014 in, respectively. The air is filled into the driver section by using the firing station control box until the desired pressure is achieved (Figure 3-12 (e)). Half of this pressure is filled automatically into the diaphragm section. Then, the diaphragm pressure is released when the driver pressure reaches its desired value. By doing so, the increasing differential of pressure between the diaphragm section and the expansion section which has atmospheric pressure ruptures the foils in the diaphragm section and the driver section sequentially. The compressed air then travels through the expansion section until it interacts with the test specimen at the opening of the Shock Tube. The expansion section is 6096 mm (240 in) long and has a 2032 mm x 2032 mm square (80 in x 80 in) opening at the end (Figure 3-12 (b)). Twelve sliding pressure relief vents are placed on the sides and top of the opening. The vents are closed before testing and opened due to the shock wave to generate a blast wave pressure-time history with a negative. Wooden frames were installed along the sides of the Shock Tube end-frame to cover any gaps since the wall size of 2 m x 2 m was slightly smaller than the opening. According to Lloyd et al. (2009), the shock tube can simulate the shock waves caused by the far field detonation of high explosives.

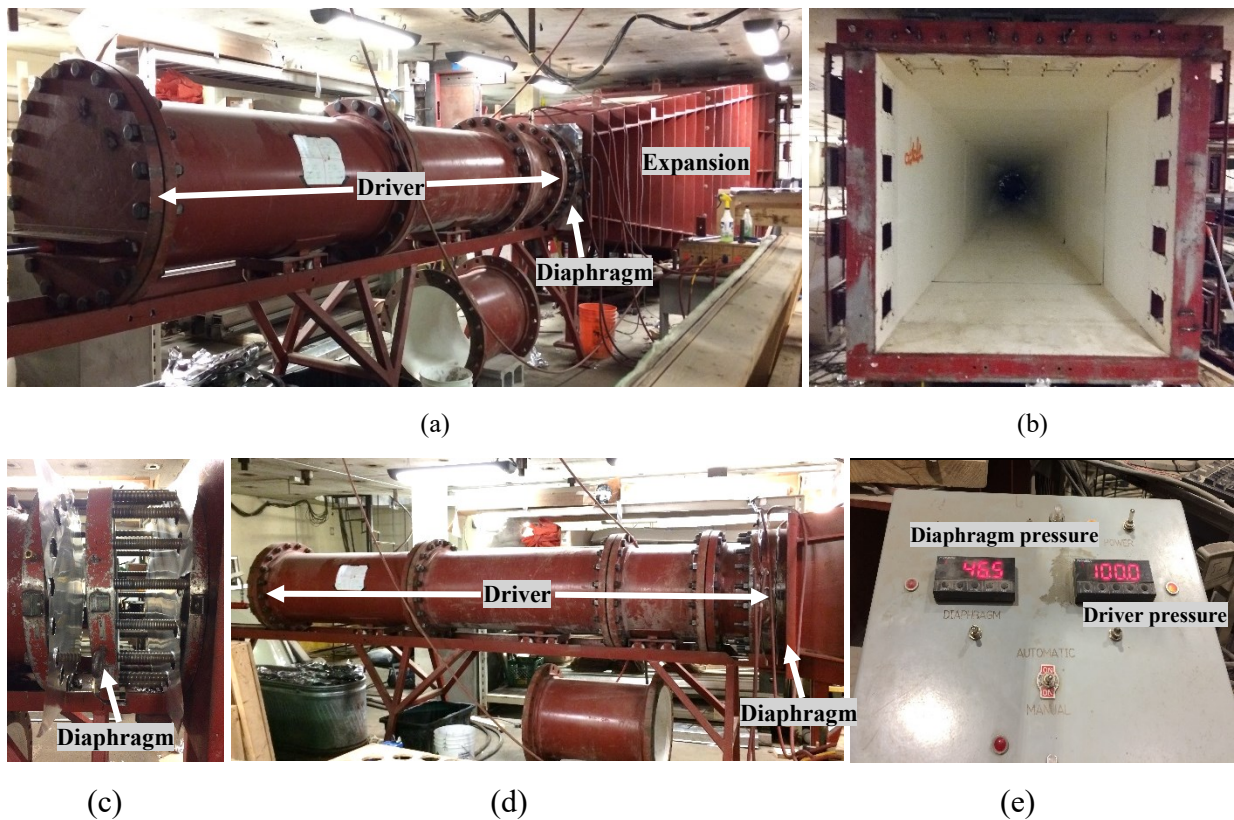


Figure 3-12. Detail of Shock tube: (a) Shock Tube, (b) Opening/Vents, (c) Opened diaphragm with foils, (d) Driver section (13ft long), (e) Firing control box

Driver Length (mm)	Reflected Pressure (kPa)	Reflected Impulse (kPa-ms)	Approximate Equivalent TNT (Hemispherical-Reflected)		
			Mass (kg)	Standoff (m)	Scaled Distance (m/kg ^{1/3})
305	78	217	8	12	6.0
915	92	410	42	18	5.2
1830	100	840	290	33	5.0
3355	103	1760	2500	67	4.9
4880	104	2690	10000	106	4.9

Figure 3-13. Maximum Shock Tube range (Lloyd, 2009)

3.3.1.2. Boundary conditions

Boundary conditions:

The same boundary conditions were used for walls R-C-N and R-S-L. The supports were intended to prevent out-of-plane wall translation and result in simply supported boundary conditions to promote one-way bending behavior. Each supports consisted of a steel plate (2200 mm x 310 mm x 13 mm) which was attached to the top/bottom HSS using four bolts (19 mm). The plates resulted in overlap lengths of about 100 mm and 173 mm at the bottom and top (including filling mortar) of the walls, respectively. Any vertical gaps between the plates and the wall surface were filled with high strength mortar to ensure contact between the plates and wall specimens. Excessive rotation of the steel plates was prevented by welding small steel stiffeners to each corner of the plates and the HSS beams at the top and bottom of the wall. This method allowed some rotation of the plates so that the pin-pin condition was reproduced according to Gandia (2019). It should be mentioned that excessive rotation of the top steel plate happened during the testing of wall R-C-N due to the high stiffness of the FRCM retrofit. Therefore, an additional small HSS (50 mm x 50 mm x 4 mm) was attached to the top plate for the 9th shot of R-C-N, and for all shots during the testing of R-S-L to prevent the excessive rotation of the top steel plate. The additional HSS was bolted to the shock tube frame using two (19 mm) bolts. Steel fillers or hydraulic jacks were placed between the top HSS and underside of the lab strong floor, and under the bottom HSS and lab slab on grade to prevent the vertical movement of the wall during testing. Figure 3-14 and Figure 3-15 shows the boundary conditions for walls R-C-N and R-S-L, respectively.

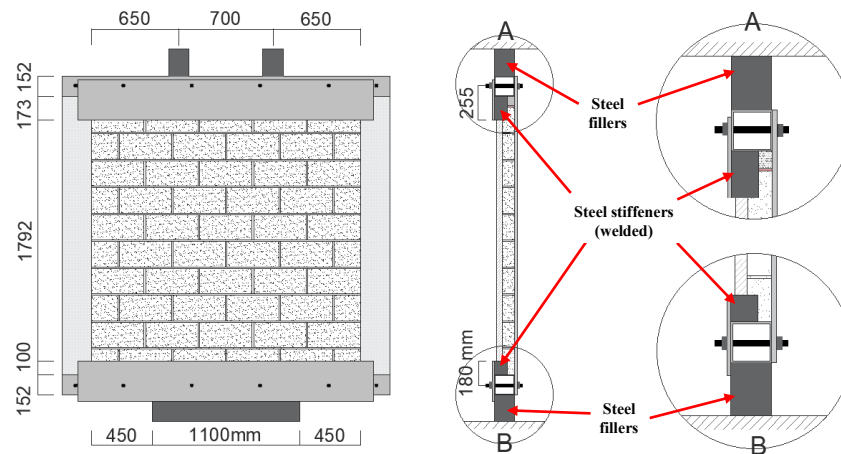


Figure 3-14. Boundary condition for R-C-N (FRCM is not shown on the front view)

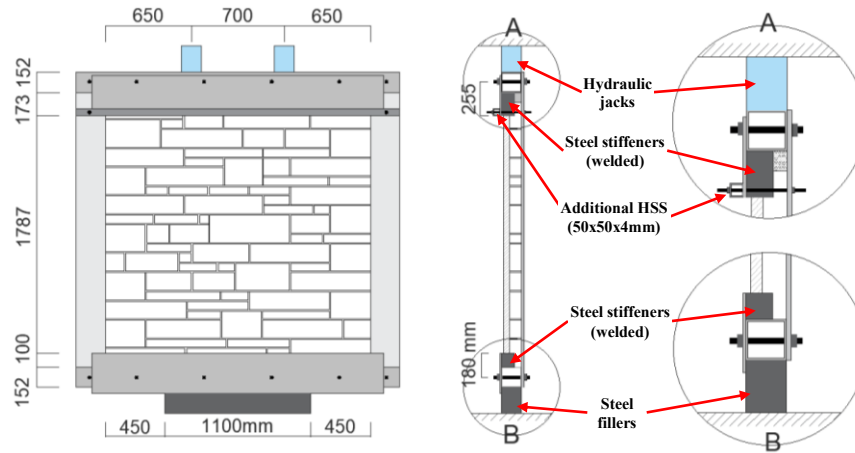


Figure 3-15. Boundary condition for R-S-L (FRCM is not shown on the front view)

Axial load application:

R-S-L is a retrofit load-bearing stone wall. The axial load was applied to the wall using a hydraulic pump and two hydraulic jacks placed on the top HSS as shown in Figure 3-15. The jacks reacted against the underside of the structures lab strong floor. The axial load was 171 kN which corresponds to 4 % of the nominal axial strength of the wall, which was the same as URM-S-L. The two jacks were located symmetrically with respect to the centerline of the top HSS.

Arching action:

To promote arching action, the gap between the top of the wall and the top HSS beam was filled with high strength mortar. For wall R-C-N, a plastic tarp was placed on top of the wall before casting the mortar to prevent the ingress of mortar into the voids of the concrete masonry units. The top HSS had slotted bolt holes so that the contact between the wall and HSS could be maximized. For R-S-L, the contact was expected to be better than in R-C-N due to the application of axial load.

3.3.1.3. Data acquisition

Instrumentation set up:

Instrumentation included a Data Acquisition System (DAS), pressure sensors, Linear Variable Displacement Transducers (LVDTs), two high speed cameras, and camera tracking software as shown in Figure 3-16. When the blast wave reaches the specimen, the pressure sensors trigger the DAS, which is set to be ready to receive the signal from the sensors. Pressure sensors are installed near the opening of the shock tube, one on the side and the other on the bottom. As soon as the DAS is triggered, it sends a signal to trigger the cameras. The video data captured by cameras is sent to the laptop to be stored and replayed. Reflected pressure measured by pressure sensors, and displacement measured by LVDTs are sent to the DAS to be stored and analyzed.

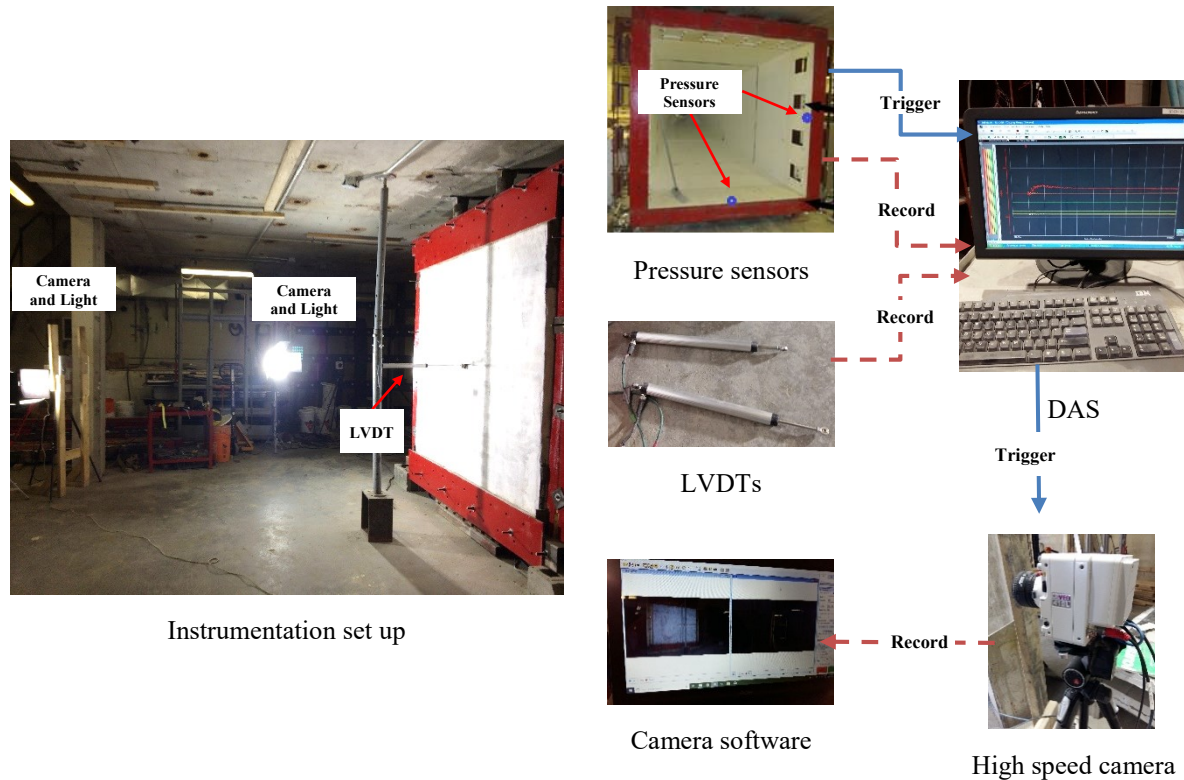


Figure 3-16. Instrumentation of blast tests

Data Acquisition System (DAS):

The DAS had sixteen channels to receive various data, including: reflected pressures (from the pressure sensors on the Shock tube), displacement (from the LVDTs), strains (from the strain gauges). For the blast tests of this project, two reflected pressure sensors and two LVDTs (one for R-C-N) were used. The measured pressures and displacements were recorded at a rate of 1000 kilo-samples per second (kS/s).

Pressure sensors:

Two pressure sensors are installed at the side and bottom of the shock tube frame as shown in Figure 3-16. The sensors trigger the DAS and measure the reflected pressures acting on the specimens. In general, measured pressure values by the bottom and side sensors were not the same, so the larger one, which was the bottom pressure, was used in the remaining chapters of this thesis. Due to the flying aluminum foils, especially for the bottom sensor, some unexpectedly high pressures could be measured, but they were ignored.

Linear Variable Displacement Transducers (LVDTs):

An LVDT is a device used to track displacement. One LVDT was installed at the middle of R-C-N, and two LVDTs were used for R-S-L at the middle and the top of the wall (Figure 3-17). LVDTs were fixed to a post by bolts and nuts to keep them in position. Initially, only the middle displacement was measured for R-C-N, then one more LVDT was added later on top for R-S-L

because R-C-N showed high movement on top. The measured displacement was utilized to draw a displacement-time history graph. An LVDT was detached from the 5th shot ($P_d = 70$ psi [483 kPa]) for R-C-N, since the failure pressure was expected. For R-S-L, LVDTs were kept until the 5th shot ($P_d = 100$ psi [689 kPa]) considering the previous results from R-C-N. When the LVDTs were removed, the wall displacement was tracked by the high speed cameras.

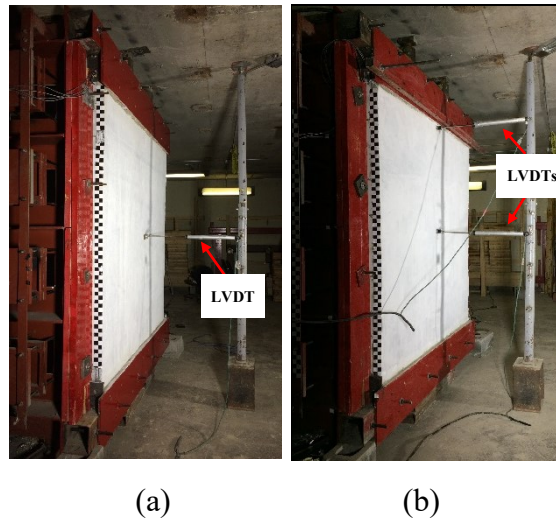


Figure 3-17. LVDTs installation: (a) R-C-N, (b) R-S-L

High speed video cameras:

Two cameras were used to record the motion of the walls. One was placed at the side of the wall to capture the bending behavior and displacements, while the other was placed at about 45 degrees from the wall to capture the response of the front face of the wall (Figure 3-16). White and black squared tracking stickers were glued to the side of the wall (Figure 3-17) to facilitate tracking. By doing so, the camera software could track the displacements along the wall height. The cameras captured the images of the wall response at a rate of 2000 frames per second (fps).

3.3.1.4. Test sequence

The test sequence for this project is summarized in Table 3-2. Blast IDs are numbered based on the driver pressure (in psi) used in each blast test so that the results from the different walls can be compared. The initial driver length for all tests was 6 ft (1829 mm). The blast sequence for R-C-N followed the same sequence as URM-1 (control) up to Blast ID #3 (24 psi [165 kPa]), which was the failure shot of URM-1, then the driver pressure jumped to 60 psi [414 kPa] which was the computed ultimate capacity of the wall according to ACI 549.4R-13 (2013). Since it was regarded to be close to failure, a 70 psi [483 kPa] blast shot was performed. The next shot jumped up to 100 psi [100 kPa] with a 6 ft driver length, then the driver was extended to 11 ft (3352 mm) with 80 psi [552 kPa] and 100 psi [689 kPa] blasts to produce higher impulses. The final blast used a driver pressure of 85 psi [586 kPa] with a 13 ft (3962 mm) driver. For R-S-L, first two shots were the same as the control, URM-S-L, which were considered in the elastic range of the wall. A 36 psi [248 kPa] shot was then chosen to compare the behaviour to URM-S-L, then the pressure jumped

to 60 psi [414 kPa] and 100 psi [689 kPa] to compare the behaviour to wall R-C-N. The final shot tested the wall using a 100 psi [689 kPa] driver pressure with a 13 ft driver and caused failure of the wall.

Table 3-2. Summary of test sequence with Blast ID and Driver pressure (Driver length is indicated in the brackets in feet for other than the 6 ft driver)

Blast ID	URM-1 *		R-C-N		URM-S-L **		R-S-L	
	psi	kPa	psi	kPa	psi	kPa	psi	kPa
#0	6	41						
#1	11	76	12	83	12	83	12	83
#2	18	124	18	124	19	131	18	124
#3	24	165	24	165	25	172		
#4					31	214		
#5					36	248	36	248
#6					42	290		
#7					48	331		
#8					54	372		
#9			60	414			60	414
#10			70	483				
#11			100	689			100	689
#12			80 (11)	552				
#13			100 (11)	689				
#14			85 (13)	586				
#15							100 (13)	689
Total	4 shots		9 shots		8 shots		6 shots	

* Tested by Ciornei (2012), ** Tested by Gandia (2019)

Table 3-3 shows the properties of the shock waves in terms of reflected pressure and impulse for each test shot. The table also compares the results with the approximate values of equivalent TNT. Driver pressure was controlled manually by using the firing station shown in Figure 3-12 (c). Reflected pressures were measured by two sensors (Figure 3-16), and reflected impulse was the area under the reflected pressure-time history diagram for the positive phase duration. It should be noted that the reflected pressure measured by the bottom sensor was used when presenting the experimental results, and when conducting the analysis for this thesis. The reflected pressure data from both the bottom and the side pressures can be found in the Appendix. Approximate equivalent values of TNT were computed according to CSA S850-12 (2012) and Lloyd (2010). The CSA S850-12 (2012) suggests to estimate blast loading parameters using the following equation:

$$x = c_0 + c_1z + c_2z^2 + c_3z^3 + c_4z^4 \text{ where, } x = \ln(x), z = \ln(Z) \quad [3-2]$$

In this equation, “x” is a scaled wave front parameter as specified “normally reflected pressure” with the scaling factor 1 and the unit kPa. The coefficients “c” obtained for the spherical TNT

surface burst are suggested as 9.7457, -4.7276, 1.1734, and -0.1337 for c_0 , c_1 , c_2 , and c_3 , respectively in the case of “normally reflected pressure $1.95 < Z < 39.67$ ”. By choosing the standoff distance from Figure 3-13 suggested by Lloyd (2010), scaled distance, Z , and charge weight, W , could be computed by using the above equation.

Table 3-3. Blast properties from Shock tube and Equivalent values to TNT

Shock Tube (University of Ottawa)								Approximate Equivalent to TNT (CSA-S850-12, Lloyd (2010))		
Blast ID	Driver Pressure: P_d		Driver Length: L_d		Avg. Reflected Pressure: P_r	Avg. Reflected Impulse: I_r	Avg. Positive Phase Duration: t_d	Scaled Distance: Z	Charge Weight: W	Standoff Distance: R
	(psi)	(kPa)	(mm)	(ft)	(kPa)	(kPa-ms)	(ms)	(m/kg ^{1/3})	(kg)	(m)
#1	12	83	1829	6	22.5	147.0	13.1	12.7	17.5	33
#2	18	124	1829	6	34.5	214.6	12.5	9.4	43.3	33
#3	24	165	1829	6	38.1	295.0	15.5	8.8	53.0	33
#4	31	214	1829	6	39.5	287.4	16.0	8.6	56.8	33
#5	36	248	1829	6	53.4	347.3	13.0	7.1	99.9	33
#6	42	290	1829	6	46.5	371.9	16.3	7.7	77.5	33
#7	48	331	1829	6	50.2	411.1	16.7	7.4	89.3	33
#8	54	372	1829	6	57.1	447.4	18.0	6.8	112.6	33
#9	60	414	1829	6	73.1	581.4	16.0	5.9	171.7	33
#10	70	483	1829	6	92.3	650.7	14.1	5.2	249.1	33
#11	100	689	1829	6	112.8	869.8	15.6	4.7	337.2	33
#12	80	552	3353	11	101.0	1580.2	31.3	5.0	2384	67
#13	100	689	3353	11	103.3	1715.3	33.2	5.0	2475	67
#14	85	586	3962	13	101.0	1862.6	36.9	5.5	5080	86
#15	100	689	3962	13	114.3	2002.7	35.0	4.7	6056	86

3.4. MATERIAL PROPERTIES

3.4.1. Masonry and mortar properties

3.4.1.1. CMU units and prisms

The compressive strength of the CMU units and prisms was tested according to ASTM C140 and CSA S304.1-04 (2004). Hard capping with high strength mortar was used to cover the loaded area of the CMU to meet the requirements of the standard. A leveling bar and a flat board were used to ensure the top surface was level before testing (Figure 3-18). The final thickness of the hard capping did not exceed 10 mm. Five CMU units and five prisms were tested under a displacement control rate of 0.1 mm/min until failure by the Universal Floor Standard Testing Machine at the University of Ottawa. The actual dimensions of a CMU block used for the tests were 90 mm (W) x 190 mm (H) x 390 mm (L), and the average dimensions of the prisms with one joint were 90 mm (W) x 386 mm (H) x 390 mm (L). The prisms were constructed in the same manner as the CMU wall with about 10 mm thick joints of Type S mortar. Since the prisms were composed of hollow concrete blocks, a correction factor of 1.0 was applied to the tested strength according to Table D.1 of CSA S304.1-04 (2004). LVDTs were placed at both sides of the prisms and the units to measure the strain as shown in Figure 3-19. This figure also shows the typical failure modes of CMU units and prisms with vertical cracks. Table 3-4 summarizes the results from compressive tests. Figure 3-20 shows the stress strain relationships during the compressive tests. The averaged compressive strength was found to be 14.2 MPa and 11.3 MPa for the CMU units and prisms, respectively. The averaged strain capacities were 0.00304 for the CMU units, and 0.00379 for the prisms. The stress-strain response of the CMU units was expected to have a parabolic behavior with an ultimate strain of ~ 0.003 and ultimate strength of ~ 15 MPa which is similar to the measured response. The stress-strain curves from the CMU prisms show an “S” shape due to the LVDTs being installed to the steel plates on the top and bottom of the prisms, and not on the specimens themselves as shown in Figure 3-19. At the start of the tests, there could have been small gaps between the plates and specimens that were closed even though the surfaces were smoothed to limit this potential error. Therefore, the curves are shifted by removing the S-shape as shown in Figure 3-20 (c); with this correction, the average ultimate strain of the prisms becomes 0.00317.



Figure 3-18. Hard capping of the specimens using high strength mortar



Figure 3-19. Compressive test set up, and typical failures: (a) CMU units, (b) CMU prisms

Table 3-4. Compressive strength and ultimate strain of CMU blocks and prisms

CMU		1	2	3	4	5	Average
Blocks	f'_m (MPa)	13.7	15.4	15.3	14.5	13.5	14.5
	ϵ_{mu} (mm/mm)	0.0029	N/G	0.00291	0.00296	0.00338	0.00304
Prisms	f'_m (MPa)	12.1	11.9	11.6	9.8	11.3	11.3
	ϵ_{mu} - actual	0.0035	0.00390	0.00403	0.00339	0.00410	0.00379
	ϵ_{mu} - shifted	0.0029	0.00310	0.00353	0.00272	0.00361	0.00317

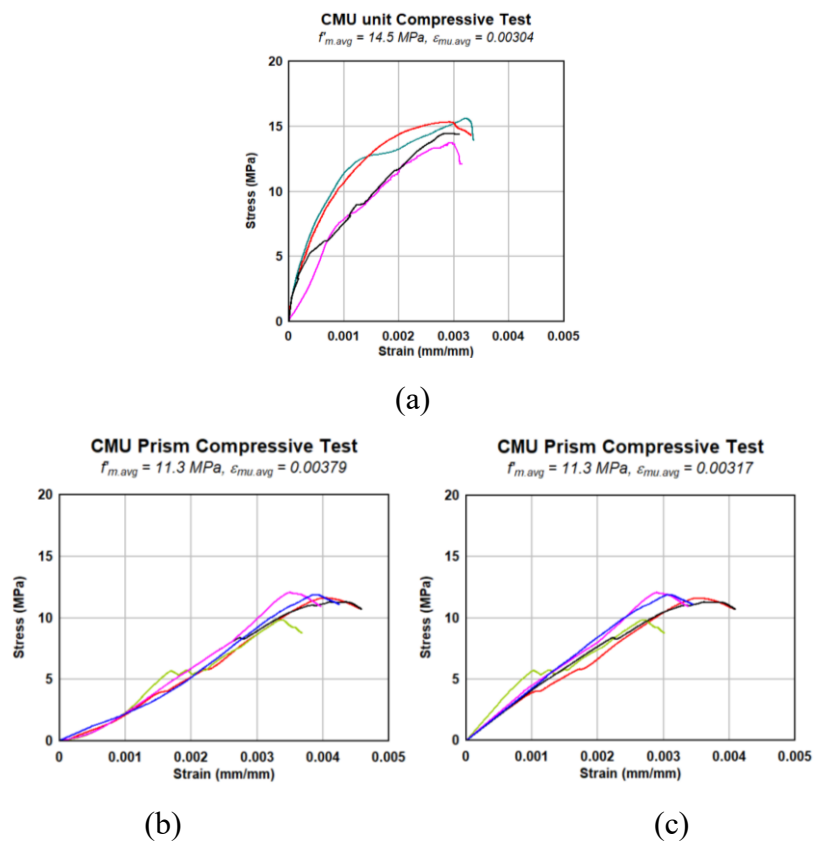


Figure 3-20. Compressive stress-strain curves: (a) CMU units, (b) CMU prisms - actual, (c) CMU prisms - shifted

3.4.1.2. Stone masonry units and prisms

The compressive strength of the stone masonry units and prisms were tested in a similar manner to that of CMU units and prisms according to ASTM C140 and CSA S304.1-04 (2004). A soft capping method with thin rubber pads was used considering the smooth and plane surfaces of the stone blocks as shown in Figure 3-21 (a). Five stone units of each size were tested, and the results are summarized in Table 3-5. The averaged compressive strength was 22.7 MPa, 19.2 MPa, and 12.6 MPa for the small, medium, and large size specimens, respectively. The failure of the stone masonry units are shown in Figure 3-21 (b), (c), and (d) for small, medium, and large size of stone units, respectively. A total of five stone masonry prisms were constructed using small size units with the average dimensions of 98.8 mm (W) x 199.3 mm (L) x 268.8 mm (H). A correction factor of 0.87 was determined using the height to thickness (h/t) ratio of 2.72, and applied to the measured strength according to Table D.1 of CSA S304.1-04 (2004). LVDTs were placed at both sides of the prisms to measure the strain as shown in Figure 3-22. Figure 3-22 also shows the typical failure mode of the stone prisms. The averaged compressive strength was found to be 9.7MPa, and the averaged ultimate strain was 0.00728. Table 3-6 summarizes the results from compressive tests. Figure 3-23 shows the compressive stress-strain relationships. The first segments of the curves show an ‘S’ shape (Figure 3-23 (a)) which could be due to the settlement of the test setup at the start of testing (i.e. possible gap between the top steel section and the stone unit). Therefore, the curves were shifted by removing these segments as shown in Figure 3-23 (b). In this case, the average ultimate strain becomes 0.0058.

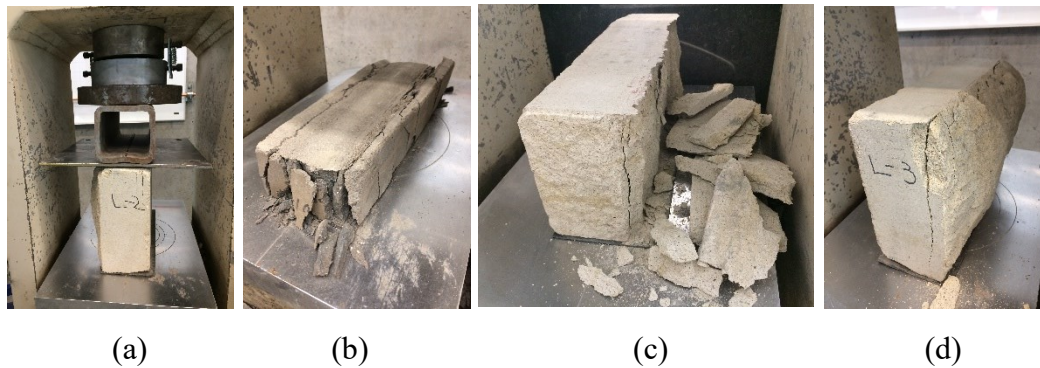


Figure 3-21. (a) Compressive test setup for stone units, and failure modes: (b) Small, (c) Medium, and (d) Large

Table 3-5. Compressive strength of stone units

Stone units (average size, W x L x H)	Compressive Strength (MPa)					
	1	2	3	4	5	Average
Small (98.3x300.7x60.4 mm)	25.1	21.1	21.8	22.2	23.2	22.7
Medium (94.6x301.4x134 mm)	19.1	20.1	18.0	19.7	19.0	19.2
Large (89.1x299.6x209.5 mm)	11.9	12.8	13.4	12.2	12.7	12.6

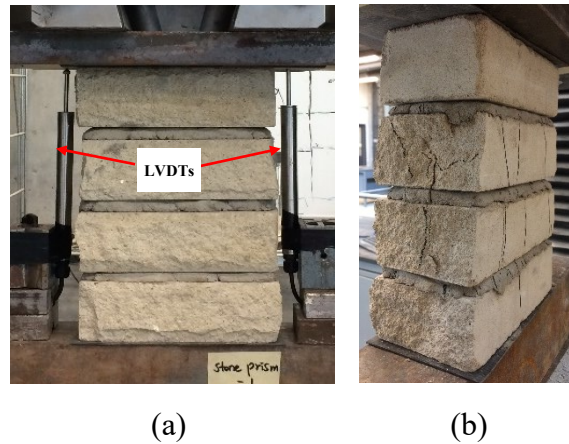


Figure 3-22. (a) Compressive test setup for Stone prisms, and (b) Typical failure mode

Table 3-6. Compressive strength and ultimate strain of stone prisms

Stone prisms	1	2	3	4	5	Average
f'_m (MPa)	10.4	9.4	12.6	13.0	10.4	11.1
$f'_{m, \text{factored}}$ (MPa)	9	8.2	10.9	11.3	9.1	9.7
ϵ_{mu} (mm/mm) - actual	0.00789	0.00734	0.00625	0.00746	0.00747	0.00728
ϵ_{mu} (mm/mm) - shifted	0.00599	0.00684	0.00511	0.00561	0.00547	0.00580

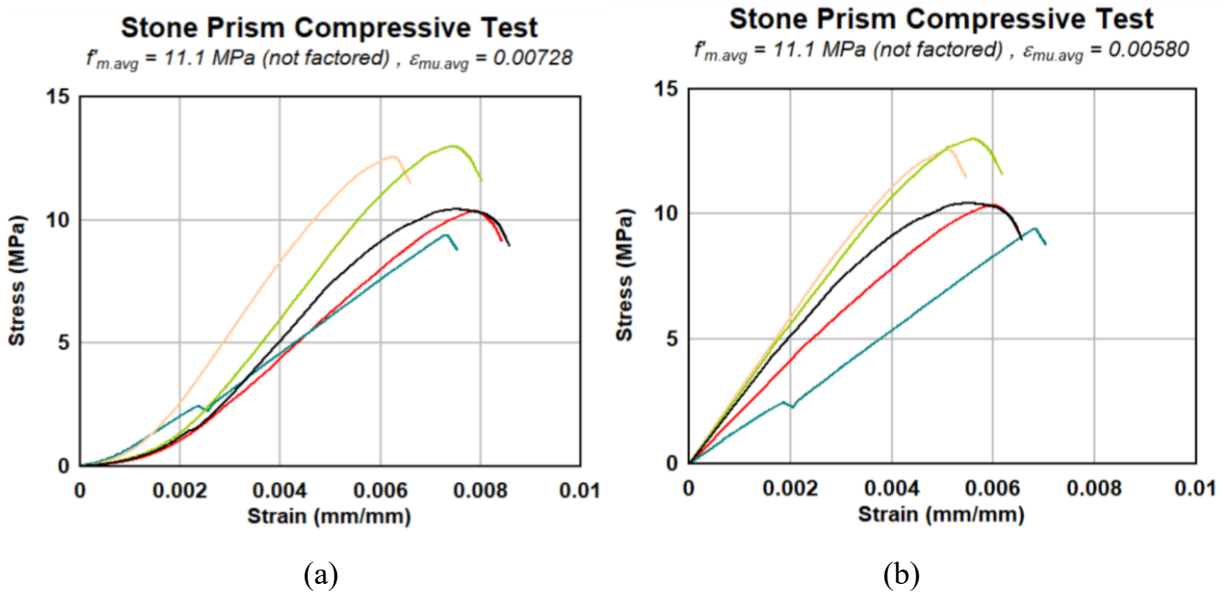


Figure 3-23. Compressive stress-strain curves of Stone prisms: (a) Actual, (b) Shifted

3.4.1.3. Mortar

The Type-S mortar was tested to determine its compressive strength according to ASTM C109. Five standard specimens (50 mm cubes) were cast and cured in the lab for more than 28 days. The results are reported in Table 3-7, and Figure 3-24 shows the test setup and a typical failure mode. The averaged compressive strength was 13.3 MPa.

Table 3-7. Compressive strength of mortar cubes for joints

Mortar (Joints)	1	2	3	4	5	Average
Compressive Strength (MPa)	13.6	14.4	12.5	13.0	12.9	13.3

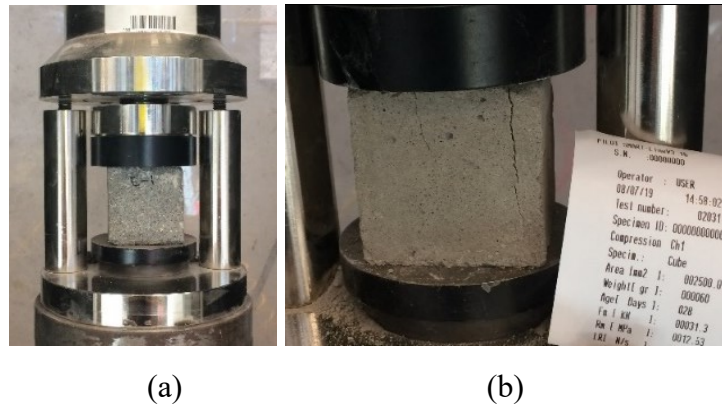


Figure 3-24. (a) Compressive test setup for mortar cubes, and (b) Typical failure mode

3.4.2. FRCM composite properties

3.4.2.1. Cementitious matrix (Mortar)

The mortar used as a binder in the FRCM composite was tested to measure its compressive strength according to ASTM C39. Three standard cylinders with dimensions of 100 mm (diameter) x 200 mm (height) were cast and cured in the lab for more than 28 days. The mortar is enriched with micro fibers (Figure 3-6), which have the properties shown in Table 3-9. The test results are summarized in Table 3-10. The stress-strain curves are shown in Figure 3-25, and Figure 3-26 shows the test setup and failure modes. The average compressive strength was 45 MPa, and the average ultimate strain was 0.0028. It should be noted that the first segment of the stress-strain curves shows an 'S' shape (Figure 3-25 (a)) which may have resulted from the settlement of the test frame. Therefore, the curves were shifted as shown in Figure 3-25 (b). In this case, the average ultimate strain becomes 0.0025. By applying the conversion factor of 0.85 from Viso et al. (1998) to the cylinder strength, it is possible to estimate the compressive strength of a standard 50mm cube. Using this procedure the cube strength was estimated as 53 MPa, which is similar to the value provided by the manufacturer (Table 3-8). As shown in Figure 3-26, the fracture patterns were either 'well-formed cone on one end, vertical cracks running through caps, no well-defined

cone on other end (Type 2)', or 'Columnar vertical cracking through both ends, no well-formed cones (Type 3)' as described in ASTM C39.

Table 3-8. Material properties of FRCM mortar *

Unit weight	2,240 kg/m ³	Flexural strength	6.9 MPa
Tensile modulus	26.8 GPa	Compressive strength	21 MPa (1 day)
Direct tensile bond strength	2.7 MPa		43 MPa (7 day)
			52 MPa (28 day)

* Provided by the manufacturer

Table 3-9. Physical characteristics of polypropylene fibers in mortar *

Fiber type	Polypropylene film	Specific gravity	0.91 g/cm ³
Width	250-400 μ m	Tensile strength	250-525 N/mm ²
Thickness	35-40 μ m	Elongation at break	6-13 %
Length	6 mm	Modulus	10500-20840 N/mm ²

* Provided by the manufacturer

Table 3-10. Compressive strength of FRCM mortar cylinders

Cylinders (Ø100 x H 200mm)	1	2	3	Average
Compressive Strength (MPa)	48.5	42.6	44.0	45.0
Strain (mm/mm) - actual	0.00322	0.00269	0.00249	0.00280
Strain (mm/mm) - shifted	0.00287	0.00237	0.00217	0.00247

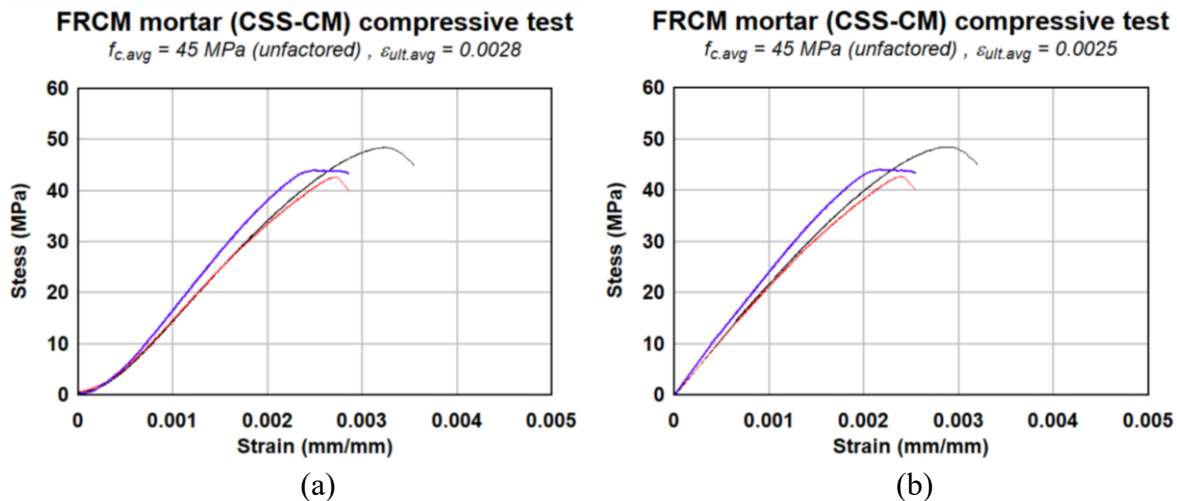


Figure 3-25. Compressive stress-strain curves of FRCM mortar: (a) actual, (b) shifted

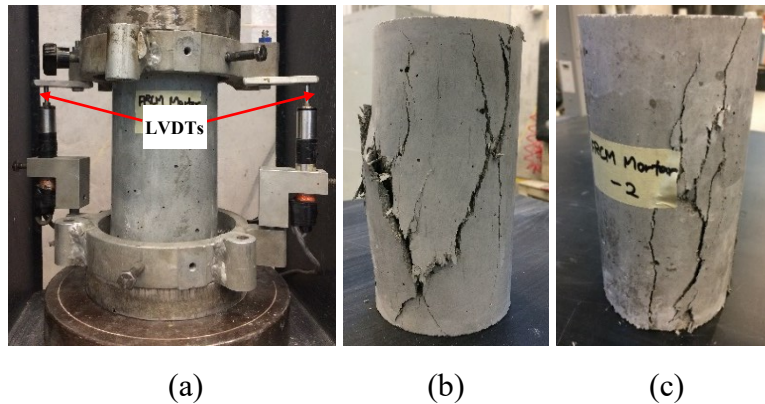


Figure 3-26. (a) Compressive test setup, and failure modes: (b) Type 2, (c) Type 3

3.4.2.2. FRCM coupons

Coupons were made and tested to measure the tensile properties of the FRCM composite according to Annex A of AC434 (2013): Acceptance Criteria for “Masonry and Concrete Strengthening Using Fabric-reinforced Cementitious Matrix (FRCM) and Steel Reinforced Grout (SRG) Composite Systems”. The following steps are required to build the specimens according to this standard:

- At least 5 coupons for each test condition;
- A larger panel which can be cut into coupons;
- Rectangular shapes with the width at least 4 times the thickness, and with the minimum length including gripping distance, twice the width, and gauge length;
- Suitable width with at least 3 strands;
- Metallic tabs with the minimum thickness of 2 mm glued at least 75 mm each end and the clevis-type grips for grip systems, and;
- Moisture curing for more than 7 days, and at least 14 days to be tested.

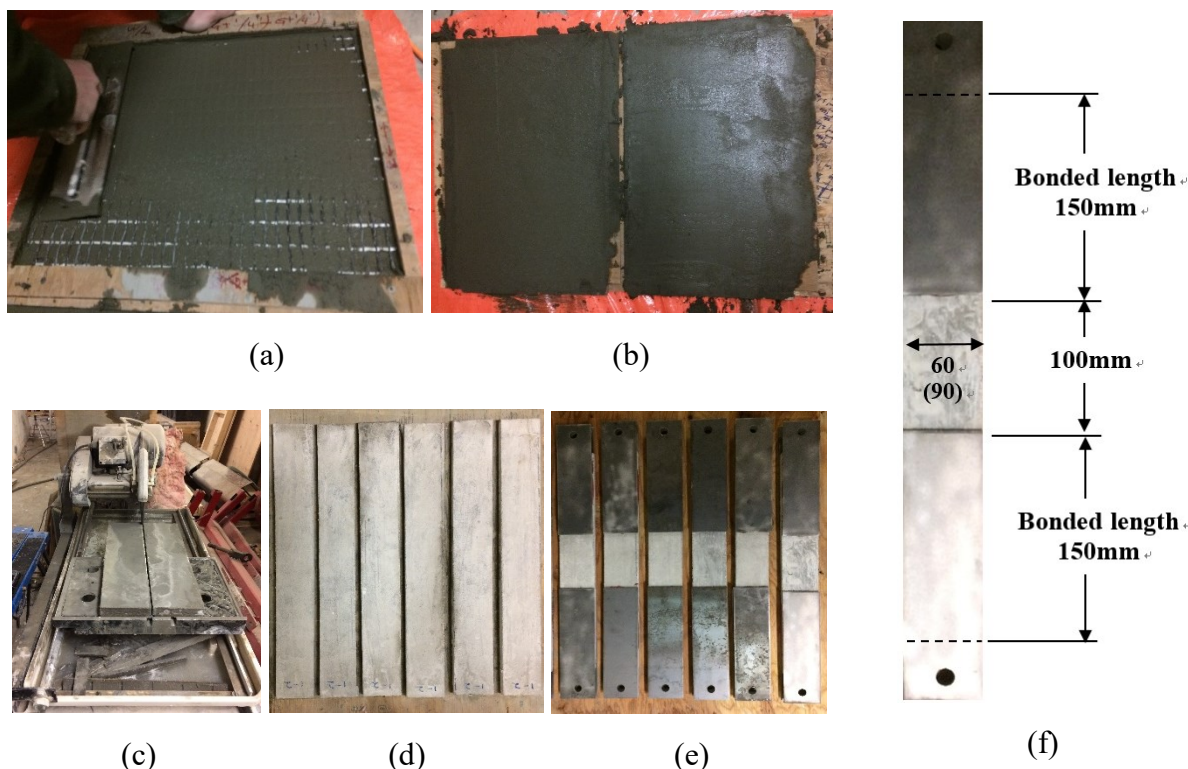


Figure 3-27. FRCM coupons: (a) FRCM application, (b) After application, (c) Concrete wet saw, (d) After cutting, (e) Completion, (f) Intended dimensions (Brackets for 2 layers)

Following these instructions, a total of 11 coupons were cast using a wooden mold of 430 mm x 560 mm; this allowed for 6 coupons for 1 layer of FRCM and 5 coupons for 2 layers of FRCM. The widths were 60 mm and 90 mm for 1 layer and 2 layers, respectively, which corresponded to 4 times the coupon thickness, and ensured that each coupon had at least 3 strands. The thickness depended on the number of layers which was about 10 mm for 1 layer, and 20 mm for 2 layers. The same length of 400 mm was used for all coupons, considering the gripping distance of 150 mm for each metal tab. The gauge length was 100 mm. The averaged dimensions from three points along the length of the coupons are listed in Table 3-11. Three (3) mm thick metal tabs with the same width as the coupons were glued onto the sample ends with high strength epoxy. Although AC434 suggests a minimum of 75 mm for the gripping distance, 150mm was chosen to improve the bond according to the suggestion of Donnini et al. (2017). This study concluded that 150 mm of contact length between a metal tab and specimen was sufficient to get a satisfactory bond strength. After 3 weeks of curing, including 7 days of moist curing in the lab, the panels were cut into coupon size by a concrete wet saw. The preparation and the intended dimensions of coupons are shown in Figure 3-27.

Annex A of AC434 (2013) recommends to use clevis-type grips during testing, which allows movement in the perpendicular plane to minimize bending stresses without clamping forces on the specimen (Figure 3-28 (a)). An extensometer with at least 50 mm long gage length, which can include more than one transverse crack, can be used. If the bearing points of the extensometer are

disturbed by cracking, the specimen can be retested by unloading and relocating the extensometer. Disturbed data can be removed after the test is complete. It is suggested to load the coupons at a rate of 0.2 mm/min. For this research, a clip-on extensometer with a 50 mm gage length was used to measure axial deformation of coupons. Instead of the socket cup screws, rectangular shaped holders were used as shown in Figure 3-28 (b) due to a technical issue with the loading machine. Monotonically increasing uniaxial tensile load was applied under the displacement control of 0.2 mm/min by the Universal Floor Standard Testing Machine at the University of Ottawa. Figure 3-28 (c) shows a typical cracking example from the tests.

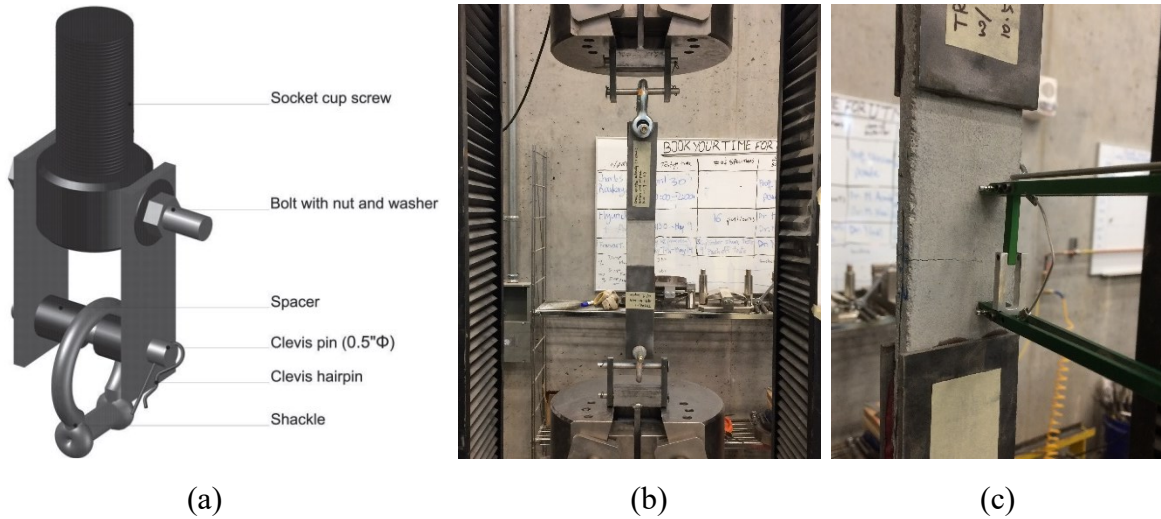


Figure 3-28. Tensile test: (a) Clevis-type gripping mechanism (AC434, 2013), (b) Test setup for this study, (c) Example of tests with cracking

According to ACI 549.4R-13 (2013), the tensile capacity of a coupon, T (kN), can be calculated as:

$$T = n \times A_f \times E_f \times \epsilon_{fe} \quad [3-3]$$

where n is the number of FRCM plies, A_f is the area of mesh reinforcement by unit width (mm^2/mm), E_f is the tensile modulus of elasticity of the cracked specimen (MPa), and ϵ_{fe} is the FRCM effective tensile strain, which is computed as the minimum of the ultimate tensile strain (ϵ_{fd}) or 0.012. Table 3-11 compares the tested and computed tensile capacities of the FRCM coupons (computed values were determined using $A_f = 0.157 \text{ mm}^2/\text{mm}$, $E_f = 49000 \text{ MPa}$, and $\epsilon_{fd} = 0.011 \text{ mm/mm}$, as specified in the manufacturer datasheet). It should be noted that the numbers of strands in each coupon varied even when the coupons had the same width. For example, either 3 or 4 strands were found in the 1-layer coupon samples. Therefore, the number of strands inside a coupon was multiplied by the area of a strand to calculate the total area of fabric reinforcement instead of using A_f . Due to technical issues with instrumentations or the coupon itself, the data from some coupons was rejected. Also, the results could have some variability due to the sensitivity of coupon preparation; such as the alignment and positioning of strands, mortar

thickness, etc. The tensile capacity values with 1 layer of FRCM textile are 5 to 13 % lower than the calculated ones with one outlier (coupon 1-4). However, with 2-layer coupons, tested capacities are 34 to 40 % higher than the calculated ones.

Table 3-11. Average dimensions of tested coupons and tensile capacities; calculated vs. tested

Coupons		Avg. Dimension (mm)			Tensile capacity (N)		
		Length	Thickness	Width	Calculated	Tested	% of calc.
1 Layer	1-1	400	12.0	60.1	4302.9	4102	95.3
	1-2	400	12.4	60.9	5737.2	5363	93.4
	1-3	400	11.0	58.7	5737.2	5012	87.4
	1-4	400	11.2	59.0	4302.9	5358	124.6
	1-5	400	11.7	60.8	Reading was not good; extensometer was disturbed a lot due to a crack at the bearing point.		
	1-6	400	11.3	56.6			
2 Layers	2-1	400	22.3	94.3	14342.9	19269.0	134.3
	2-2	400	22.6	91.6	14342.9	20087.3	140.1
	2-3	400	23.7	92.7	Reading was not good; extensometer was disturbed a lot due to a crack at the bearing point. Or, rods were bent.		
	2-4	400	23.6	92.2			
	2-5	400	23.5	87.4			

From the tensile tests conducted as shown above, the following mechanical properties can be obtained in accordance with AC434 (2013):

- The tensile modulus of elasticity of the uncracked specimen, E_f^* which is equal to $\Delta f / \Delta \epsilon$, where Δf and $\Delta \epsilon$ are the differences in tensile stress and strain, respectively between any two points before cracking;
- The tensile modulus of elasticity of the cracked specimen, E_f which is the slope of two points at stresses equal to $0.90 f_{fu}$ and $0.60 f_{fu}$, which can be calculated as $E_f = (0.90f_{fu} - 0.60f_{fu}) / (\epsilon_{f@0.90ffu} - \epsilon_{f@0.60ffu})$, and;
- The ultimate tensile strain, ϵ_{fu} , which can be calculated as $\epsilon_{fu} = (f_{fu} - y \text{ intercept}) / E_f$ where $y \text{ intercept} = 0.60f_{fu} - E_f - \epsilon_{f@0.60ffu}$.

The results from FRCM tensile tests are reported in Table 3-12. The reported data includes the tensile modulus of elasticity of the uncracked or the cracked specimen, the ultimate tensile strength and the ultimate tensile strain. The results confirmed that the use of two layers of carbon grids is better to improve the mechanical properties of the FRCM retrofit compared to those with one layer. However, even if ACI 549.4R (2013) allows the use multiple grids, the effect of using many piles should be studied further to generalize the mechanical behaviour due to the lack of research (Donnini et al., 2017).

Table 3-12. Mechanical properties of FRCM coupons

Property	Symbol	Unit	One layer		Two layers	
			(1)	(2)	(1)	(2)
Tensile modulus of elasticity of the uncracked specimen (divided by A_f)	E_f^*	GPa	1009	1070.9	5974.6	6485.9
Tensile modulus of elasticity of the uncracked specimen (divided by gross area of coupons)	E_f^*	GPa	13.5	13.5	41.6	41.6
Tensile modulus of elasticity of the cracked specimen	E_f	GPa	22.6	22.5	54.1	59.4
Ultimate tensile strength	f_{fu}	MPa	529.5	540	1348.9	1470.5
Ultimate tensile strain	ϵ_{fu}	mm/mm	0.0209	0.0209	0.0236	0.0236

(1) Fiber area by unit width, A_f (0.157 mm²/mm) was used for computing the properties.

(2) Fiber area by the number of strands in each tested coupon was used for computing the properties.

Figure 3-29 shows the behavior of the tested FRCM coupons with one or two layers of fiber grids under the direct tensile tests. Generally, the trends of the diagrams are very similar to those described in Section 2.2.5 with three stages; the un-cracked stage (Stage I), the crack development stage (Stage II), and the cracked stage (Stage III). A sudden drop of stress with the first crack in a coupon occurred after Stage I since most of the applied load to the coupon transferred to the strands. It should be noted that the stress plotted in Stage I was computed by dividing the applied load by the area of grid reinforcement (A_f) in accordance with AC434 (2013), not by the overall cross-sectional area of specimen. The values up to the first crack on the diagrams, or E_f^* in Table 3-12 seemed too high, so E_f^* calculated with the gross area of a coupon is presented in the same table, assuming a perfect mortar-to-fiber bond. The stresses on the second stage fluctuated but increased gradually as more cracks formed. In this stage, the behaviour was associated with the development of multiple cracks. The mortar between cracks was still able to carry load. Stage III resulted in an almost linear branch after completion of crack formation. At this stage, only the strands could carry the load applied to the specimen with one specific crack opening widely. Due to minor cracks occurring outside of the bearing length of the extensometer, the measured strain was not always representative of the behaviour of the entire specimen. However, the same issue was raised in the tests conducted by Antino et al. (2017), and the author insisted that the results with outside cracks showed consistency with no outside cracks. It can therefore be assumed that the outside cracks were minor enough not to affect the elongation of the specimens. It should be noted that A_f used to calculate the stress according to AC434 (2013) is the area of reinforcement by unit width which is 0.157 mm²/mm. In a 60 mm wide coupon, the strands embedded in the coupon was either three or four. Therefore, the columns numbered as (1) in Table 3-12 are the values computed with A_f in accordance with AC434 (2013), and the column (2) are based on the fiber area according to the number of strands. Figure 3-29 is plotted based on the stress calculated by the AC434 (2013) method.

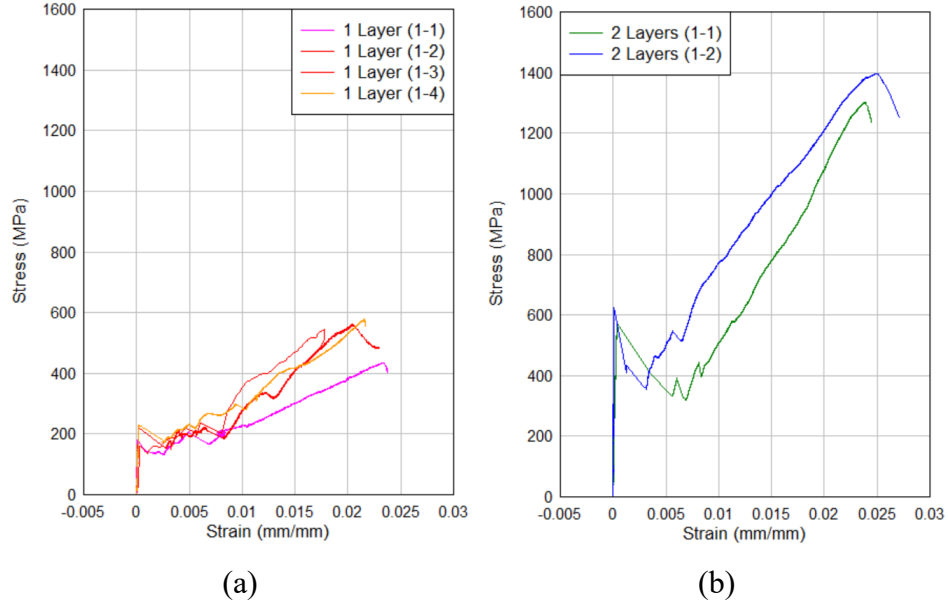


Figure 3-29. Stress-strain curves from FRCM coupon direct tensile tests: (a) 1 layer, (b) 2 layers

Figure 3-30 shows the failure modes of the tested coupons. In all cases, coupons with one or two layers failed due to slippage of fibers from the embedding mortar. Three cracks formed for all of 1-layer coupons; one wide critical crack, and two minor cracks. For the 2-layer coupons, either two or three cracks were created, always with one major crack. The slippage of the fiber strands without rupture during the tests indicates that the tensile strength of the fibers was not fully engaged in the failure mechanism due to the loss of bond between grids and mortar. The gripping length of the coupons is suggested to be longer for future tests.

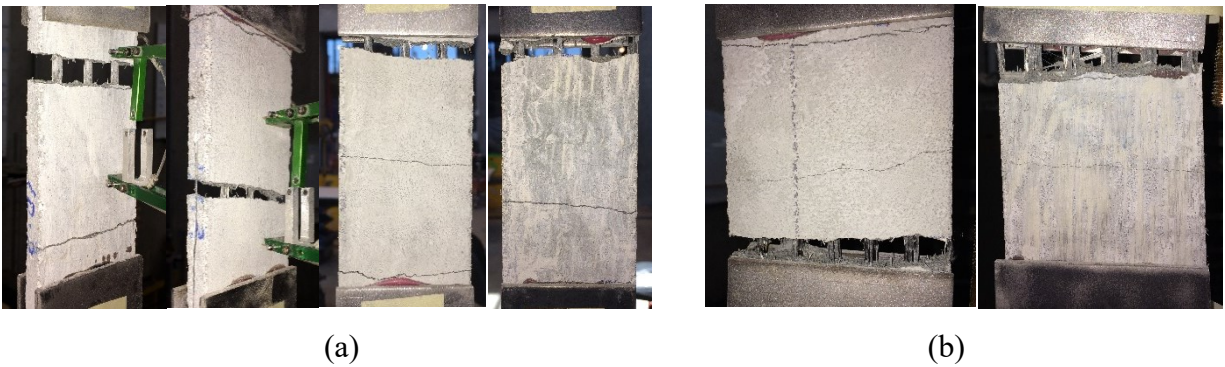


Figure 3-30. Failure modes of tested coupons with the fibers slippage: (a) 1 layer, (b) 2 layers

A total of five FRCM coupons with three layers of carbon grids were also prepared for testing. Their average dimensions were 400 mm long, 111 mm wide, and 32.4 mm thick. Unfortunately, they were too strong to be held by half inch-thick rods, and the rods were bent as shown in Figure 3-31. The tab holes should have been drilled larger to have thicker and stronger rods, however since insufficient space was left on the tab ends, testing was discontinued.



Figure 3-31. The rod bent during the 3-layer FRCM coupon test

3.4.2.3. *Small beams strengthened with FRCM composites*

Eight small masonry beams were tested under two-point monotonically increasing loads as per ASTM C1390. Each specimen was composed of 4 courses of CMU blocks with the dimensions of 90 mm x 390 mm x 790 mm. One specimen was a control without retrofit, and six specimens were retrofitted by FRCM systems on the tension faces with one, two, or three layers of carbon fabrics. Type S mortar was used for the joints with 10 mm thickness. The application method of FRCM composites was the same as that for the walls described in Section 3.2.1.3. Concrete surface strain gauges (60 mm, I-274-PL-60-11, 120 OHM) were installed according to manufacturer's instruction at the center of the tension faces. A wire displacement transducer was connected at the middle of the beam to measure the central beam displacement. The main objective of this test was to determine the effect of the number of retrofit layers on load-displacement response and failure mode.

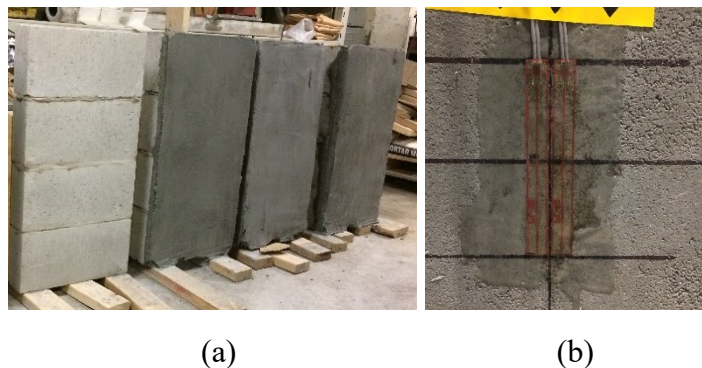


Figure 3-32. Specimen preparation: (a) Construction, (b) Installation of strain gauges

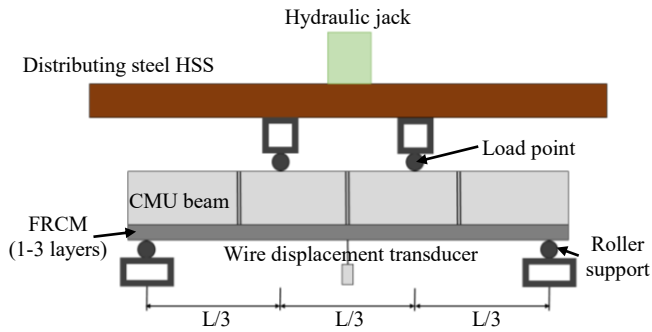


Figure 3-33. The third-point loading test setup for small beams

The control beam failed while it was transported which gives an indication of the very weak out-of plane resistance of unreinforced masonry walls. The failure of the beam with one layer of FRCM fabric was governed by flexural failure at the middle (Figure 3-35). The peak load applied was 48.2 kN, and the middle displacement was measured as 5.3 mm. Figure 3-34 shows the relationship between the applied load and the mid-point displacement as measured by the wire transducer. Up to the load of 14 kN, the beam was not affected by the load. Then, it deformed with flexural cracks until failure. It should be noted that the branch between 14 kN and 48.2 kN was smoothed due to significant noise in the data record. The specimen with 1 layer did not show any signs of delamination.

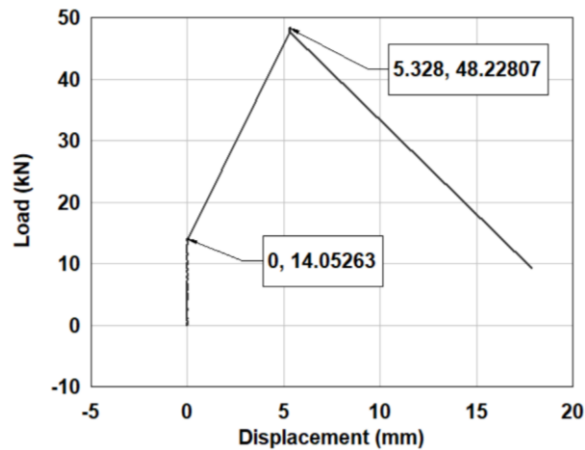


Figure 3-34. Load-Displacement Diagram of a beam with 1-layer of FRCM retrofit

Two sets of beams with two or three layers of carbon fabrics were built and tested. The beams with two or three layers experienced either shear failure (Set 1) or delamination (Set 2) as shown in Figure 3-35. The beams in Set 1 were retrofitted at the same time as the walls by professional applicators. Unfortunately, the data from these tests was not saved due to an error in the data acquisition system. Therefore a second set of specimens with 2 or 3 layers were subsequently built by the author (Set 2). The beams in Set 2 showed delamination due to the imperfect bond between CMU blocks and FRCM mortar. After delamination, the strength decreased significantly and the

peak loads were 15.1 kN and 18.5 kN for 2 and 3 layers, respectively. The middle displacements at the peak loads were 6.1 mm and 18 mm, respectively. The results from Set 2 are plotted in Figure 3-36. The way of mixing and quality of the batch could be the reason of the weaker strength and delamination observed in the second set of beams.

No. of layers	Failure modes	
	Set 1 (By shear or flexural)	Set 2 (By delamination)
1	 $P_{max} = 48.2 \text{ kN}$, $d_{mid@P.max} = 5.3 \text{ mm}$	(only one beam tested with 1 layer)
2	 $P_{max} = \text{NG}$, $d_{mid@P.max} = \text{NG}$	 $P_{max} = 15.1 \text{ kN}$, $d_{mid@P.max} = 6.1 \text{ mm}$
3	 $P_{max} = \text{NG}$, $d_{mid@P.max} = \text{NG}$	 $P_{max} = 18.5 \text{ kN}$, $d_{mid@P.max} = 18.0 \text{ mm}$

Figure 3-35. Typical failure modes of small beams with different numbers of fabrics

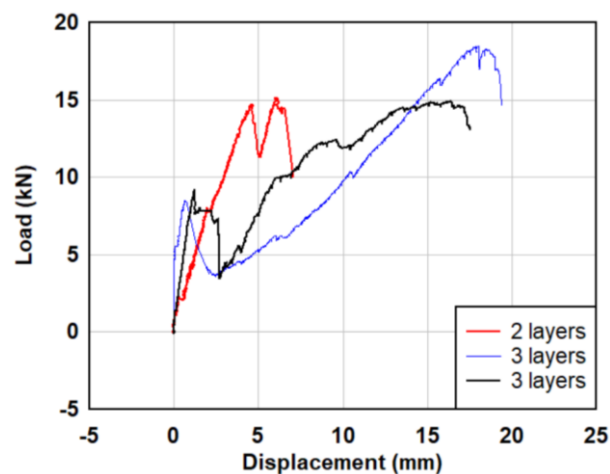


Figure 3-36. Load-Displacement Diagram of beams with multiple layers of FRCM retrofit

CHAPTER 4. EXPERIMENTAL RESULTS

4.1. SUMMARY OF RESULTS

The chapter presents the detailed experimental results for walls R-C-N and R-S-L. The results from the tests in terms of Shock Tube parameters, shock wave properties and specimen response are summarized in Table 4-1. Walls R-C-N and R-S-L were subjected to 9 and 6 shots of blast loading, respectively. Each shot is numbered sequentially and matched to the corresponding blast ID in Table 3-2 of Chapter 3. The following data is reported in the table:

- **Shock Tube parameters:** P_d is the driver pressure in kPa; L_d is the driver length which started at 6 ft (1829 mm), and was extended to 11 ft (3353 mm) and 13 ft (3962 mm) during later stages of testing to produce stronger impulse.
- **Shock wave parameters:** P_r is the maximum reflected pressure in kPa which acted on the wall and was obtained from the bottom pressure sensor (this sensor recorded larger peak values than the side pressure sensor); I_r is the positive impulse in kPa-ms which was computed as the area under the reflected pressure-time history curve up to the end of the positive phase; t_d is the positive phase time duration in milliseconds.
- **Specimen response:** The maximum and residual displacements at the mid-height of the wall are designated as d_{max} and d_{res} , respectively. To account for the accumulation of displacements from previous shots, the cumulative residual displacement after each shot ($\sum d_{res}$) is also reported. The time at the maximum displacement is t_{dmax} . For wall R-C-N, the displacements at the top of the wall were found to be larger than those at mid-height due to the rotation of the top steel plate; therefore, the displacements at the top of the wall are reported in the brackets. This behaviour will be discussed in detail later in this chapter.

Table 4-1. Summary of testing

Wall ID	Shot	Blast ID	P _d (kPa)	L _d (mm)	P _r (kPa)	I _r (kPa-ms)	t _d (ms)	d _{max} (mm)	d _{res} (mm)	Σd _{res} (mm)	t _{dmax} (ms)
R-C-N	1st	#1	82.7	1829	23.1	155.2	13.5	4.15	0	0	19.0
								(3.96)*	(0)*	(0)*	
	2nd	#2	124.1	1829	32.6	225.5	13.8	5.9	0	0	19.6
								(11.3)	(0)	(0)	
	3rd	#3	165.5	1829	38.1	295.0	15.5	6.9	0.3	0.3	19.8
								(9.9)	(0)	(0)	
	4th	#9	413.7	1829	69.8	646.3	18.5	15.9	1.5	1.8	21.6
								(20.6)	(0.56)	(0.56)	
	5th	#10	482.6	1829	92.3	650.7	14.1	17.2	0.6	2.4	23.5
								(24.6)	(0)	(0.56)	
	6th	#11	689.5	1829	102.6	883.5	17.2	27.6	3.7	6.1	27.0
								(31.2)	(0.56)	(1.12)	
	7th	#12	551.6	3353	101.0	1580.2	31.3	42.4	5.6	11.7	32.5
								(46.4)	(2.8)	(3.92)	
	8th	#13	689.5	3353	103.3	1715.3	33.2	57.6	3.1	14.8	46.0
								(69.4)	(7.0)	(10.92)	
	9th	#14	586.1	3962	101.0	1862.6	36.9	71.1	19.2	34.0	51.5
								(no data due to failure)			

Wall ID	Shot	Blast ID	P _d (kPa)	L _d (mm)	P _r (kPa)	I _r (kPa-ms)	t _d (ms)	d _{max} (mm)	d _{res} (mm)	Σd _{res} (mm)	t _{dmax} (ms)
R-S-L	1st	#1	82.7	1829	21.9	138.8	12.7	2.55	0.38	0.38	14.6
	2nd	#2	124.1	1829	36.3	203.7	11.2	3.05	1.33	1.71	15.2
	3rd	#5	248.2	1829	53.4	347.3	13.0	5.54	2.02	3.73	22.7
	4th	#9	413.7	1829	76.4	516.6	13.5	12.0	4.95	8.68	22.5
	5th	#11	689.5	1829	123.1	856.2	13.9	23.2	7.02	15.7	26.1
	6th	#15	689.5	3962	114.3	2002.7	35.0	(no data due to complete failure)			

* The values in the brackets represent the displacements at the top of the wall R-C-N.

4.2. R-C-N

R-C-N was a CMU infill wall which was strengthened with FRCM composites on its tension face. The average thickness of the FRCM retrofit was 46 mm with 3 layers of carbon grids. A total of nine shots were applied onto the wall. The test results are summarized in Table 4-2 along with descriptions of the main events for each shot. The displacements on the middle of the wall were measured by a LVDT up to the fourth shot, then by a high speed camera. Displacements on the top of the wall, which were measured by a camera from the beginning, were also recorded and are reported in the brackets so that those at the top and middle can be compared.

Figure 4-1 shows the displacement profiles of the wall at the instant of maximum deformation for each shot. To get the profile, several points from the high speed video were selected and tracked using the camera software. Video stills from each shot are also shown in this figure with the same scales as the displacement profile plots for comparison.

Figure 4-5 shows the pressure and impulse time histories, and the pressure and displacement time histories for each shot. The impulse was computed as the area under the pressure curve within the first positive duration. As the impulse increases, the duration of the positive phase of the blast (t_d) increases which is striking the wall longer.

Table 4-2. Summary of testing results with main events (R-C-N)

Shot	Blast ID	P _d (kPa)	L _d (mm)	P _r (kPa)	I _r (kPa-ms)	t _d (ms)	d _{max} (mm)	d _{res} (mm)	Σd _{res} (mm)	t _{dmax} (ms)
1st	#1	82.7	1829	23.1	155.2	13.5	4.15	0	0	19.0
							(3.96)	(0)	(0)	
							No damage appeared			
2nd	#2	124.1	1829	32.6	225.5	13.8	5.9	0	0	19.6
							(11.3)	(0)	(0)	
							No damage appeared			
3rd	#3	165.5	1829	38.1	295.0	15.5	6.9	0.3	0.3	19.8
							(9.9)	(0)	(0)	
							Shear crack (diagonal) on CMU block of the 1 st course (top left side of the wall)			
4th	#9	413.7	1829	69.8	646.3	18.5	15.9	1.5	1.8	21.6
							(20.6)	(0.56)	(0.56)	
							Horizontal hairline cracks along the joints (front): Figure 4-2 (a) Shear crack on CMU block of the 1 st course (top right side of the wall): Figure 4-3 (a)			

Shot	Blast ID	P _d (kPa)	L _d (mm)	P _r (kPa)	I _r (kPa-ms)	t _d (ms)	d _{max} (mm)	d _{res} (mm)	Σd _{res} (mm)	t _{dmax} (ms)
5th	#10	482.6	1829	92.3	650.7	14.1	17.2	0.6	2.4	23.5
							(24.6)	(0)	(0.56)	
		Diagonal hairline cracks at top right (front): Figure 4-2 (b) Shear cracks widened (both sides at the top of the wall)								
6th	#11	689.5	1829	102.6	883.5	17.2	27.6	3.7	6.1	27.0
							(31.2)	(0.56)	(1.12)	
		More cracks in the middle with maximum 0.1 mm wide (front): Figure 4-2 (c) Shear cracks widened (both sides at the top of the wall) Possible top CMU crush (back)								
7th	#12	551.6	3353	101.0	1580.2	31.3	42.4	5.6	11.7	32.5
							(46.4)	(2.8)	(3.92)	
		More cracks with maximum 0.2 mm wide (front): Figure 4-2 (d) Top CMU blocks crushed due to shear cracks (back): Figure 4-3 (b) Horizontal cracks occurred (sides): Figure 4-3 (b)								
8th	#13	689.5	3353	103.3	1715.3	33.2	57.6	3.1	14.8	46.0
							(69.4)	(7)	(10.92)	
		More cracks with maximum 0.3 mm wide (front): Figure 4-2 (e) Top CMU blocks crushed more (back): Figure 4-3 (c) Horizontal cracks developed more (sides)								
9th	#14	586.1	3962	101.0	1862.6	36.9	71.1	19.2	34.0	51.5
							(67.0)	(34.7)	(45.62)	
		Failed with more cracks especially at top (front): Figure 4-2 (f) Failed with CMU blocks crushed (back): Figure 4-3 (d)								

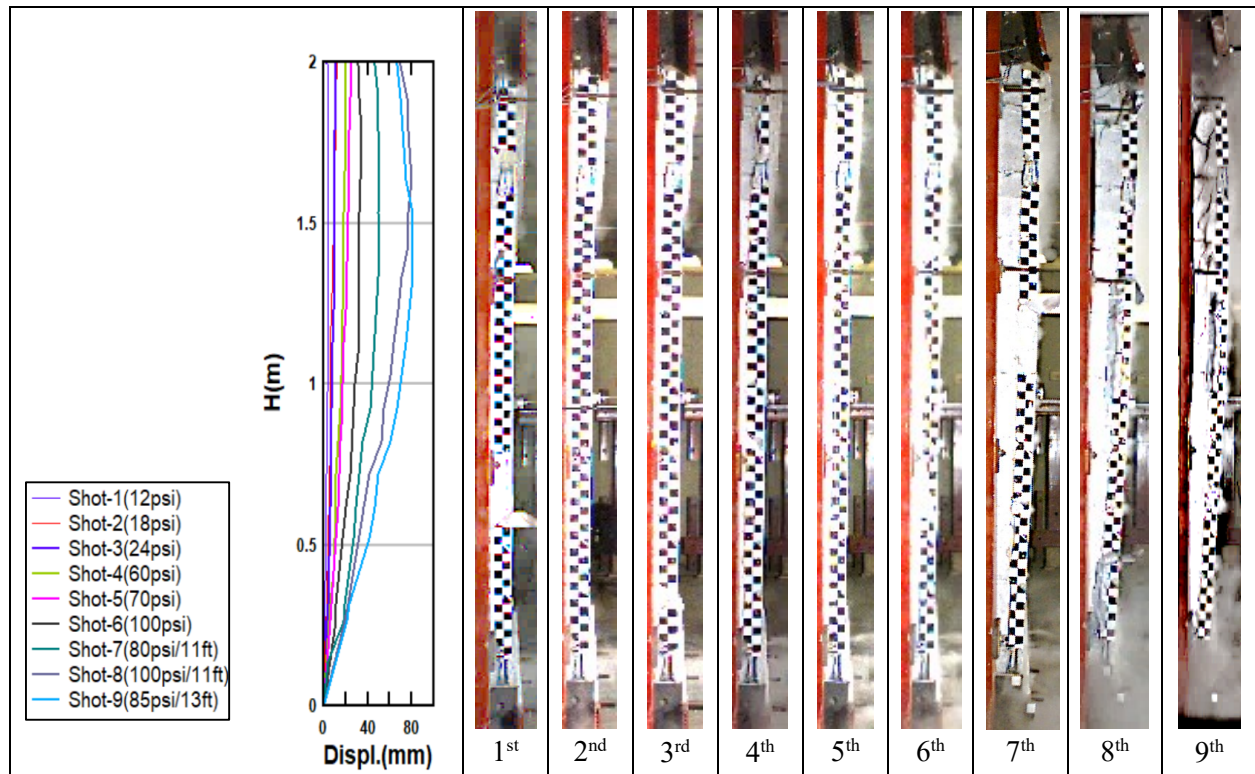


Figure 4-1. Displacement profile with video stills for each shot at maximum displacement (R-C-N)

1st shot (Blast ID #1)

A driver pressure of 82.7 kPa (12 psi) with a length of 1829 mm (6 ft) was used for the first shot on wall R-C-N. The peak reflected pressure and reflected impulse were 23.1 kPa and 155.2 kPa-ms, respectively. The positive phase duration was 13.5 ms. The top and middle displacements were almost the same at the first shot; mid-height displacement was 4.15 mm, and top displacement was 3.96 mm. The maximum deflection occurred at a time of 19.0 ms. The pressure-impulse time history and the pressure-displacement time history are shown in Figure 4-5 (a).

The first shot resulted in a small displacement at the mid-height of the wall without any residual deformations or damage and therefore the wall remained in the elastic range.

2nd shot (Blast ID #2)

The second shot used a driver pressure of 124.1 kPa (18 psi), with the same driver length of 1829 mm. The peak reflected pressure and reflected impulse were 32.6 kPa and 225.5 kPa-ms, respectively. The positive phase duration was 13.8 ms. Maximum mid-height displacement and top displacement were 5.9 mm and 11.3 mm, respectively. Maximum deflection occurred at a time of 19.6 ms. The pressure-impulse time history and the pressure-displacement time history are shown in Figure 4-5 (b).

The top displacement was nearly double the one at mid-height. This can be seen from the displacement profile in Figure 4-1, where the wall deformation profile is nearly linear with a large top translation.

During this blast, the weld connecting the top steel stiffeners and plate broke. The stiffeners were welded again before the third shot. This is why the top displacement of the third shot is lesser than the second shot.

No visible damage was observed during this test, without any residual deformations.

3rd shot (Blast ID #3)

The same driver length was used in the third shot; however, the driver pressure was increased to 165.5 kPa (24 psi). The peak reflected pressure and reflected impulse were 38.1 kPa and 295 kPa-ms, respectively, with a positive phase duration of 15.5 ms. The maximum displacements at mid-height and top were 6.9 mm and 9.9 mm, respectively. The displacement on the top was not doubled up, but still bigger than that at mid-height due to the rotation of the top steel plate. This can be seen in Figure 4-1. Maximum deflection occurred at a time of 19.8 ms. Residual displacement was 0.3 mm at the mid-height of the wall. No displacement remained on the top. The pressure-impulse and the pressure-displacement time histories are shown in Figure 4-5 (c).

A diagonal shear crack was observed to have occurred in the masonry blocks near the top support, on the left side of the wall as shown in Figure 4-3 (a).

Up to the third shot, no flexural cracks appeared on the front face of the wall.

4th shot (Blast ID #9)

The driver pressure was increased to 413.7 kPa (60 psi) with no change in the driver length for the fourth shot. The peak reflected pressure and reflected impulse were 69.8 kPa and 646.3 kPa-ms, respectively, with a positive phase duration of 18.5 ms. The mid-height and top maximum displacements were 15.9 mm and 20.6 mm, respectively. Maximum deflection occurred at a time of 21.6 ms. Residual displacements were 1.5 mm and 0.56 mm at the mid-height and top of the wall, respectively. The pressure-impulse and the pressure-displacement time histories are shown in Figure 4-5 (d).

Another diagonal shear crack on the right top side of the wall occurred at the fourth shot.

Horizontal flexural hairline cracks were also created on the front face at this shot. They were spaced about 20 cm apart mostly at the same locations as the CMU mortar joints. No flexural cracks appeared on the very top and bottom of the wall. The crack progression on the front of the wall from the fourth shot to the last shot is demonstrated in Figure 4-2 (a).

5th shot (Blast ID #10)

The driver pressure was increased to 482.6 kPa (70 psi) with no change in driver length for the fifth shot. The peak reflected pressure and reflected impulse were 92.3 kPa and 650.7 kPa-ms, respectively, with a positive phase duration of 14.1 ms. The maximum mid-height displacement and top displacement were 17.2 mm and 24.6 mm, respectively. Maximum deflection occurred at a time of 23.5 ms. Residual displacement was 0.6 mm at the mid-height of the wall. No displacement remained on the top. The pressure-impulse time history and the pressure-displacement time history are shown in Figure 4-5 (e).

Shear cracks on the top sides of the wall were widened during this blast.

More flexural cracks were generated mostly on the middle of the wall, but existing cracks were not widened much, remaining about 0.1 mm wide. Some inclined cracks began to appear on the top right front of the wall (Figure 4-2 (b)).

6th shot (Blast ID #11)

The driver pressure was increased to 689.5 kPa (100 psi) with no change in driver length for the sixth shot. The peak reflected pressure of 102.6 kPa and reflected impulse of 883.5 kPa-ms were recorded, with a positive phase duration of 17.2 ms. Maximum mid-height displacement and top displacement were 27.6 mm and 31.2 mm, respectively. Maximum deflection occurred at a time of 27.0 ms. Residual displacements were 3.7 mm at the mid-height, and 0.56 mm at the top of the wall. The pressure-impulse and pressure-displacement time histories are shown in Figure 4-5 (f).

Shear cracks on the top sides of the wall were widened more.

More flexural cracks were generated on the tension face of the wall. Cracks still remained less than 0.1 mm wide (Figure 4-2 (c)).

7th shot (Blast ID #12)

A driver pressure of 551.6 kPa (80 psi) was used for this blast. At this shot, the driver length was extended to 3353 mm (11 ft) to produce higher impulse with a longer positive duration. This blast resulted in a peak reflected pressure of 101 kPa, reflected impulse of 1580.2 kPa-ms and a positive phase duration of 31.3 ms. Maximum mid-height displacement and top displacement were 42.4 mm and 46.4 mm, respectively. Maximum deflection occurred at a time of 32.5 ms. Residual displacements were 5.6 mm at the mid-height, and 2.8 mm at the top of the wall. The pressure-impulse time history and the pressure-displacement time history are shown in Figure 4-5 (g).

More cracks occurred on the front with the maximum crack width reaching 0.2 mm (Figure 4-2 (d)). Upon closer inspection the top CMU blocks on the back of the wall showed signs of crushing as shown in Figure 4-3 (b).

8th shot (Blast ID #13)

A driver pressure of 689.5 kPa (100 psi) was used with the same driver length of 3353 mm. The peak reflected pressure was 103.3 kPa, and the reflected impulse was 1715.3 kPa-ms, with a positive phase duration of 33.2 ms. Maximum mid-height displacement and top displacement were 57.6 mm and 69.4 mm, respectively. Maximum deflection was shown at a time of 46.0 ms. Residual displacements were 3.1 mm at the mid-height, and 7.0 mm at the top of the wall. The pressure-impulse and the pressure-displacement time histories are shown in Figure 4-5 (h).

Flexural cracks were opened up to 0.3 mm with creation of additional cracks (Figure 4-2 (e)). Many diagonal/inclined cracks occurred in the top region of the front face of the wall as the top CMU blocks experienced further crushing at the back of the wall (Figure 4-3 (c)).

The still for the 8th shot in Figure 4-1 shows that the top steel plate was rotating significantly with warping of the HSS. Therefore an additional small HSS (50mm x 50mm x 4mm) was installed on top of the steel plate for the last shot to prevent excessive rotation of the plate as shown in Figure 4-6.

9th shot (Blast ID #14)

A driver pressure of 586.1 kPa was used for the final blast. The driver length was extended to 3962 mm (13 ft) to produce higher impulse. The peak reflected pressure was 101 kPa, and the reflected impulse was 1862.6 kPa-ms with a positive phase duration of 36.9 ms. Maximum mid-height displacement was 71.1 mm. Top displacement was not measured due to the dust generated during the failure (crushing) of the CMU blocks at the top. Maximum deflection was shown at a time of 51.5 ms. Residual displacements at the mid-height was 19.2 mm. The pressure-impulse and the pressure-displacement time histories are shown in Figure 4-5 (i).

The additional HSS gave more restraint to the top steel plate, so the displacement at the 9th shot was lesser than that of the 8th shot even with higher blast load as shown on the displaced wall profile in Figure 4-1.

At the last shot, the wall was considered to be failed as most of CMU blocks were demolished on the back (compression) face of the wall (Figure 4-3 (d)). Nevertheless, the retrofit was still standing, though it showed significant cracking at the top and middle as shown in Figure 4-2 (f). The top HSS was warped significantly at this shot. Figure 4-4 shows video stills of the failure mechanism during this shot.

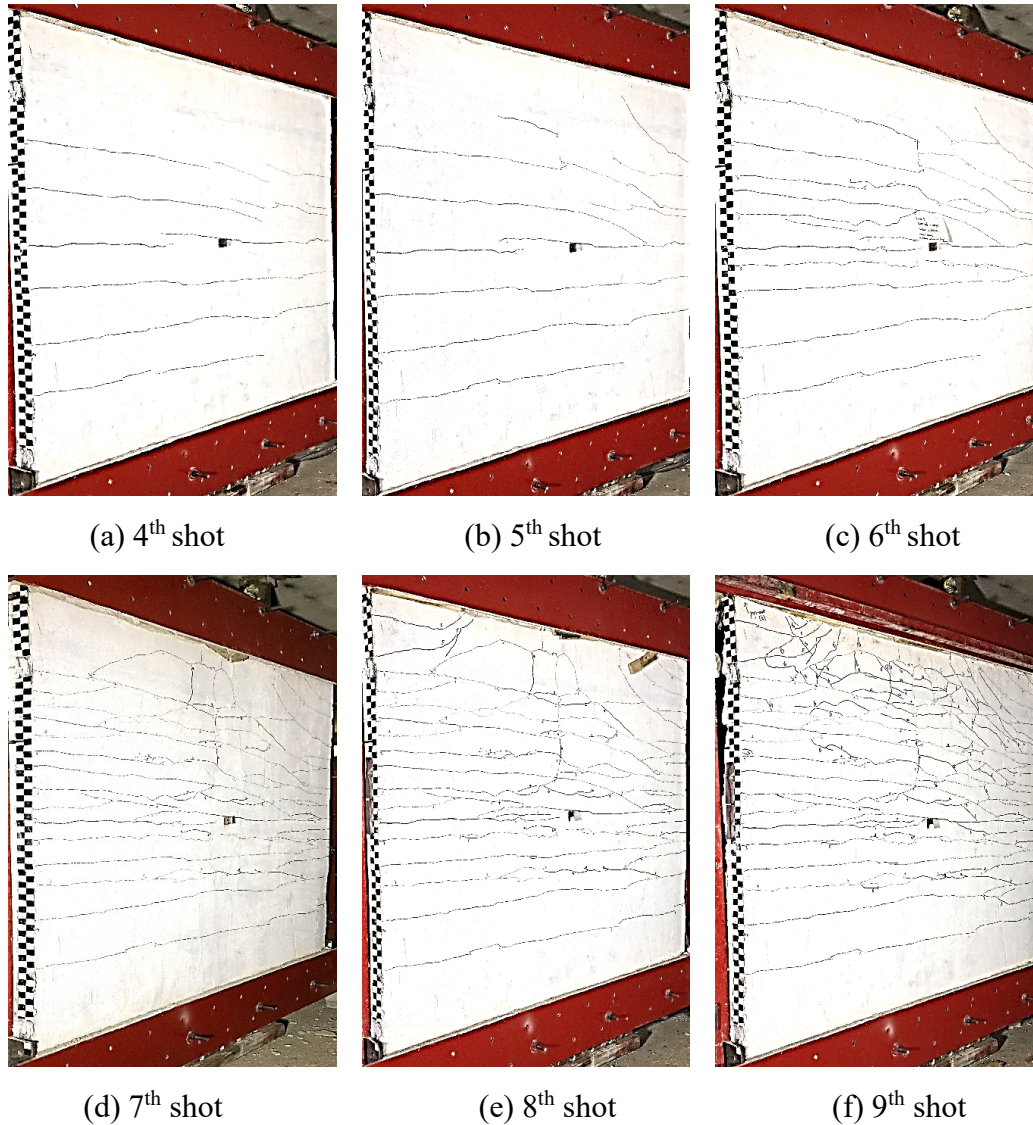


Figure 4-2. Damage Progression (from 4th to 9th shots, no flexural cracks up to 3rd shot)



(a) from 3rd shot (b) 7th shot (back and side) (c) 8th shot (d) 9th shot (back)

Figure 4-3. Major shear cracks and CMU crushing (R-C-N)

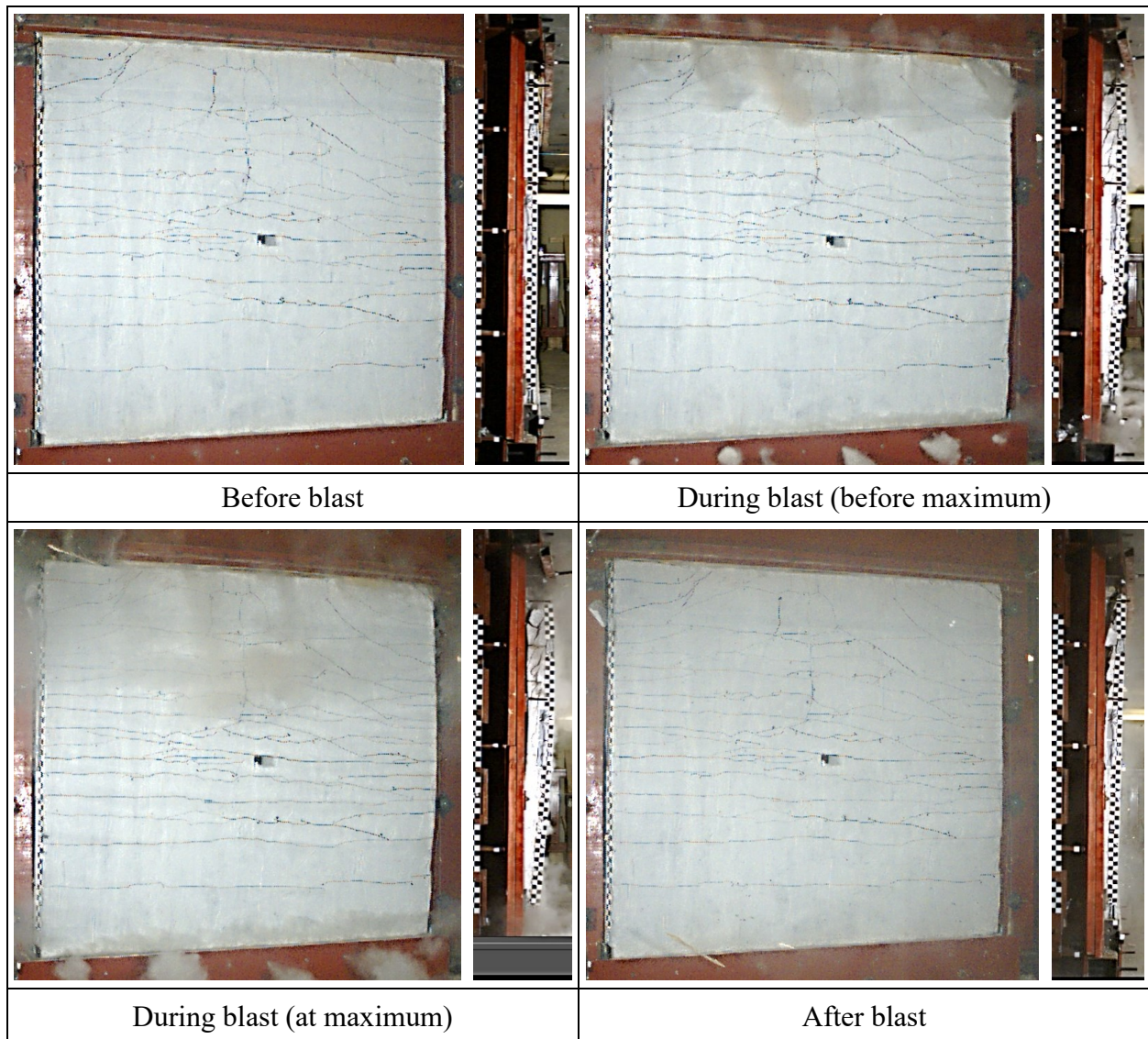
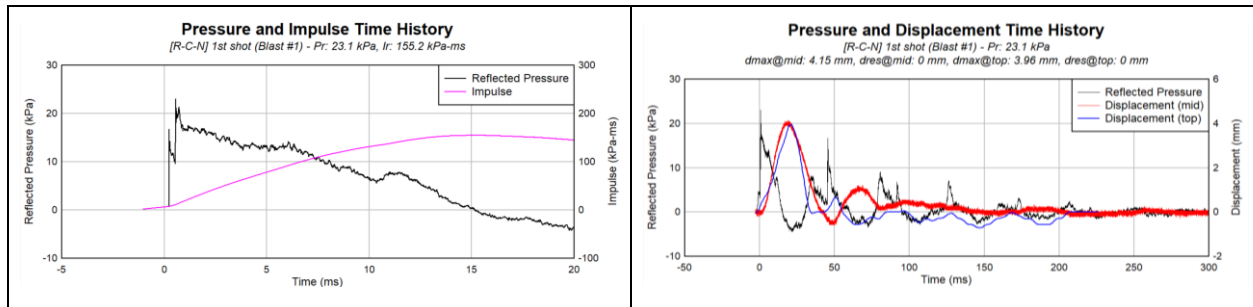
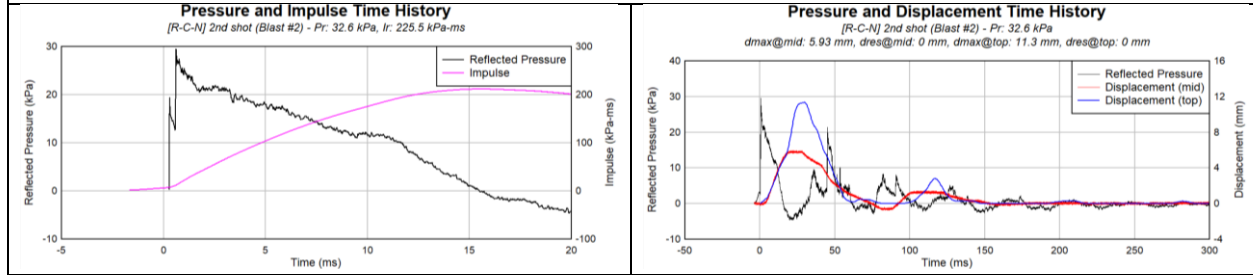


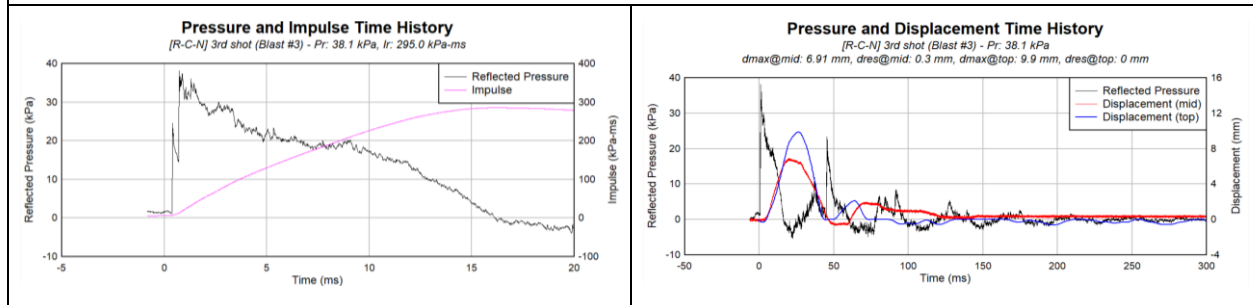
Figure 4-4. Video stills of failure mechanisms of R-C-N at the 9th shot



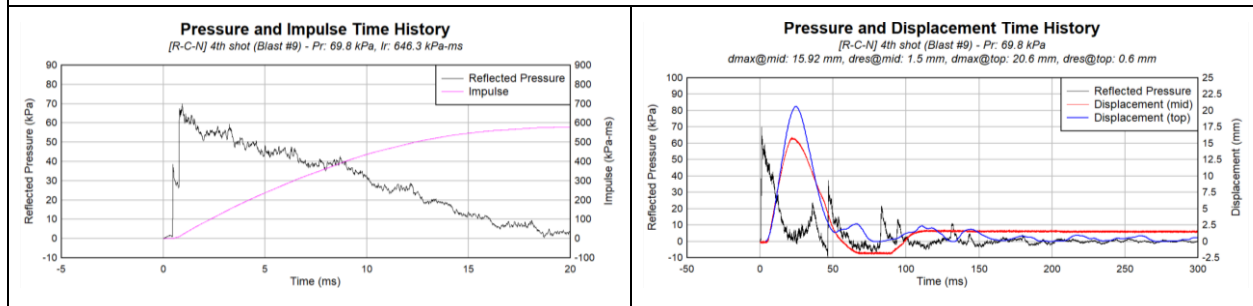
(a) 1st shot (Blast ID #1)



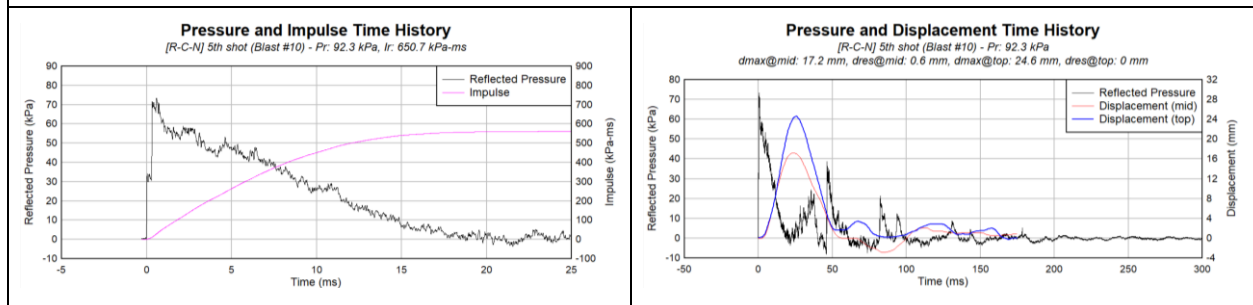
(b) 2nd shot (Blast ID #2)



(c) 3rd shot (Blast ID #3)



(d) 4th shot (Blast ID #9)



(e) 5th shot (Blast ID #10)

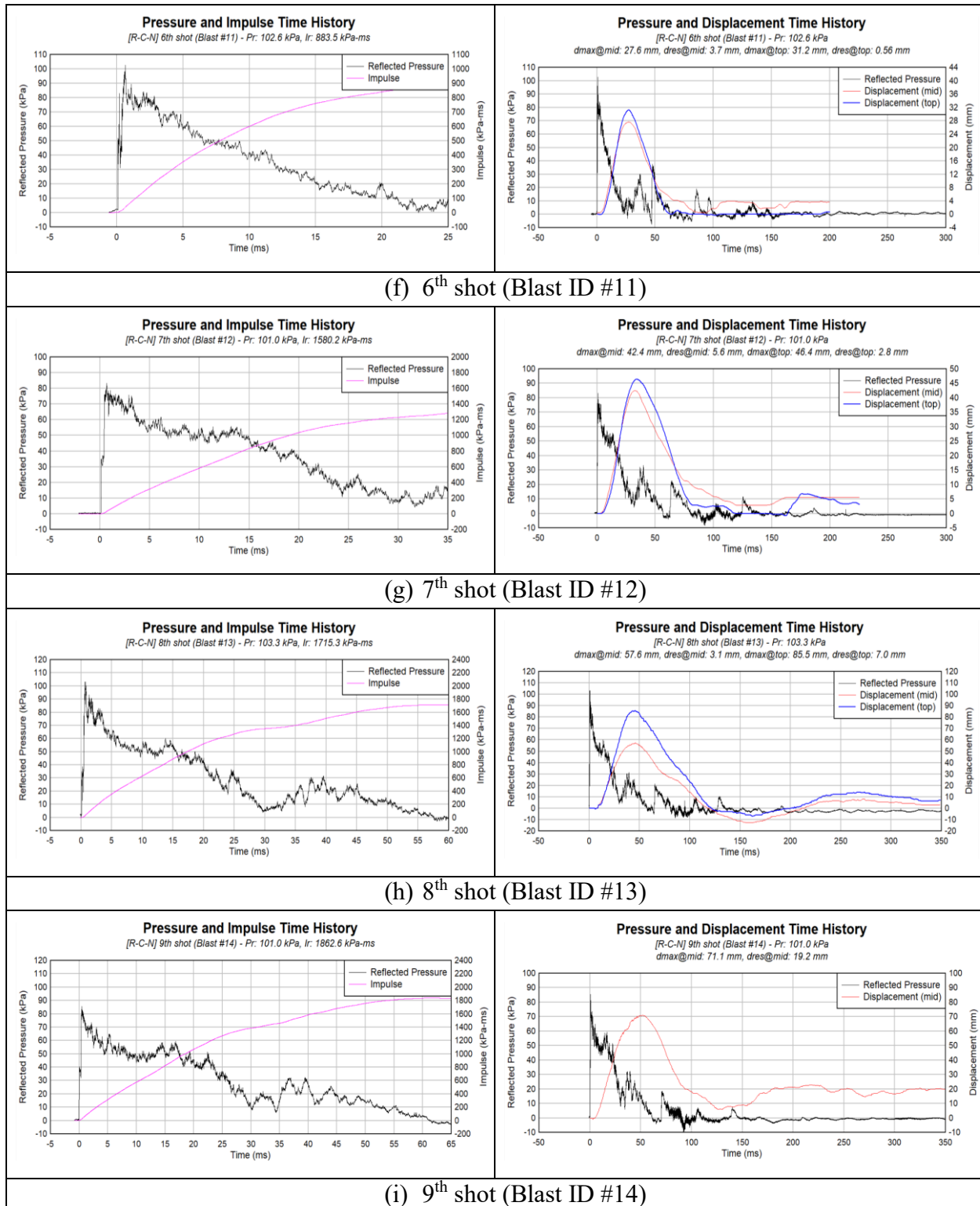


Figure 4-5. Pressure, Impulse, and Displacement (top, middle) Time history (R-C-N)

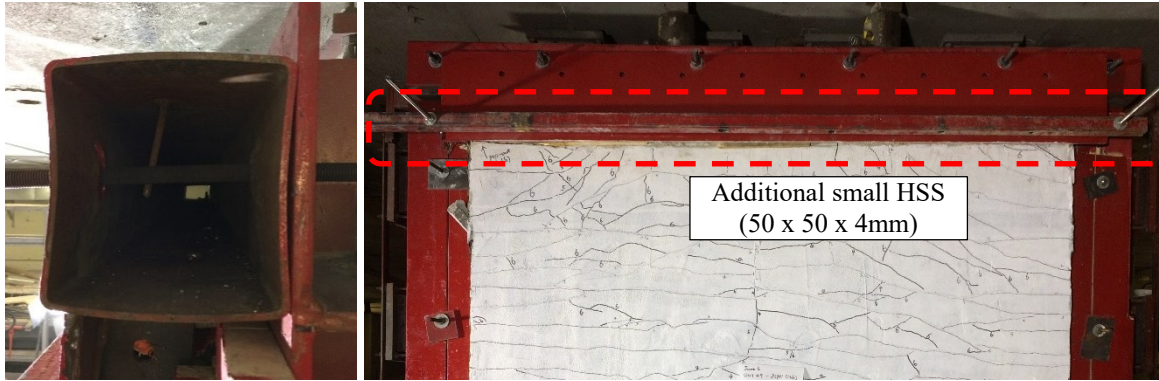


Figure 4-6. Warped HSS on 8th shot (left), and additional HSS installed for 9th shot (right)

4.3. R-S-L

R-S-L was a stone load-bearing wall strengthened with FRCM composites on its tension face. The average thickness of the FRCM retrofit was 43 mm with 3 layers of carbon grids. An axial load of 171 KN, representing 4 % of the wall nominal axial capacity, was applied to the wall during the tests. The small HSS (50 mm x 50 mm x 4 mm) used for R-C-N was used for R-S-L from the first shot. A total of six shots were applied on the wall before failure. The testing results are summarized in Table 4-3 with descriptions of main events for each shot. The displacements on the middle and top of the wall were measured by two LVDTs up to the fifth shot, then by a high speed camera.

Figure 4-7 describes the displacement profiles of the wall at maximum for each blast up to the fifth shot, just before the failure. Several points from the wall were chosen and tracked to get the displacement profiles. Video stills are also shown in the figure with the same scales to compare their behaviours at the point of maximum deformation.

Figure 4-11 shows the pressure and impulse time histories, and the pressure and displacement time histories for each shot. The impulse was computed in the same manner as for R-C-N. The displacements on the middle of the wall were measured by a LVDT up to the fifth shot. For the last shot, the middle displacement was measured from the high speed video by tracking a point at the middle of the wall.

Table 4-3. Summary of testing results with main events (R-S-L)

Wall ID	Shot	Blast ID	P_d (kPa)	L_d (mm)	P_r (kPa)	I_r (kPa-ms)	t_d (ms)	d_{max} (mm)	d_{res} (mm)	$\sum d_{res}$ (mm)	t_{dmax} (ms)
R-S-L	1st	#1	82.7	1829	21.9	138.8	12.7	2.55	0.38	0.38	14.6
			No damage								
	2nd	#2	124.1	1829	36.3	203.7	11.2	3.05	1.33	1.71	15.2
			One short hairline crack (~ 0.1 mm) at the right middle of the wall: Figure 4-8 (a)								
	3rd	#5	248.2	1829	53.4	347.3	13.0	5.54	2.02	3.73	22.7
			More hairline cracks (~ 0.1 mm) at the upper middle: Figure 4-8 (b)								

Wall ID	Shot	Blast ID	P _d (kPa)	L _d (mm)	P _r (kPa)	I _r (kPa-ms)	t _d (ms)	d _{max} (mm)	d _{res} (mm)	Σd _{res} (mm)	t _{dmax} (ms)
	4th	#9	413.7	1829	76.4	516.6	13.5	12.0	4.95	8.68	22.5
			More hairline cracks (~ 0.1 mm) at the upper and lower middle: Figure 4-8 (c)								
	5th	#11	689.5	1829	123.1	856.2	13.9	23.2	7.02	15.7	26.1
			More cracks (~0.2 mm) over the front, diagonal cracks at top corners: Figure 4-8 (d)								
			Horizontal cracks, debonding (vertical) cracks at the side: Figure 4-8 (e)								
	6th	#15	No damage on the back of the wall, No shear cracks								
689.5			3962	114.3	2002.7	35.0	(no data due to complete failure)				
			Totally demolished due to debonding between stone units and FRCM mortar, and fiber ruptures at the main crack as opened widely: Figure 4-8 (f)								

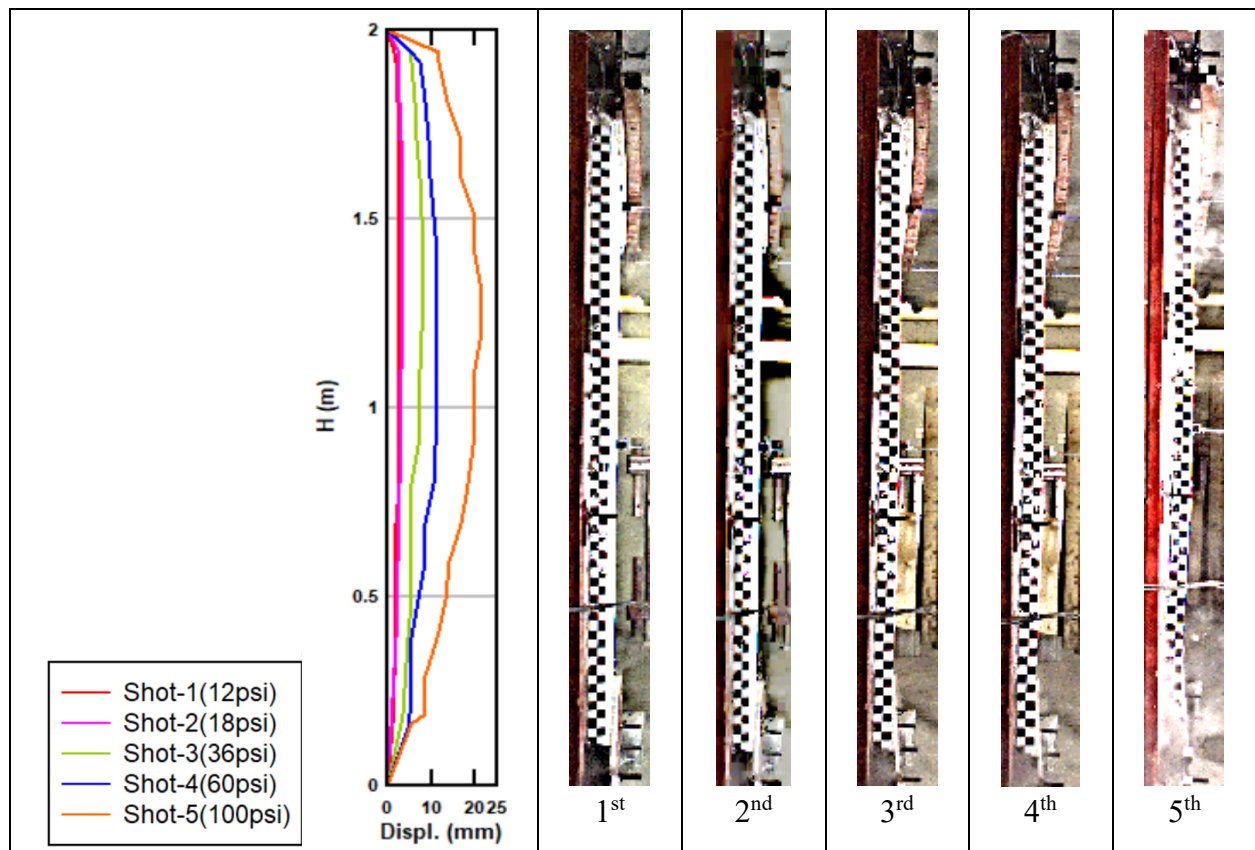


Figure 4-7. Displacement profile with video stills for each shot at maximum displacement (R-S-L)

1st shot (Blast ID #1)

A driver pressure of 82.7 kPa (12 psi), and a driver length of 1829 mm (6 ft) were used for the first shot on wall R-S-L. The peak reflected pressure and reflected impulse were 21.9 kPa and 138.8 kPa-ms, respectively. The positive phase duration was 12.7 ms. Maximum mid-height displacement was 2.55 mm with residual displacement of 0.38 mm. Maximum deflection occurred

at a time of 14.6 ms. The pressure-impulse and the pressure-displacement time histories are shown in Figure 4-11 (a).

No visible cracking or damage was observed.

2nd shot (Blast ID #2)

A driver pressure of 124.1 kPa (18 psi), and the same driver length were used for the second shot. The peak reflected pressure and reflected impulse were 36.3 kPa and 203.7 kPa-ms, respectively. The positive phase duration was 11.2 ms. Maximum mid-height displacement was 3.05 mm with residual displacement of 1.33 mm. Maximum deflection occurred at a time of 15.2 ms. The pressure-impulse and pressure-displacement time histories are shown in Figure 4-11 (b).

One small horizontal hairline crack appeared near the wall mid-height as shown in Figure 4-8 (a).

3rd shot (Blast ID #5)

A driver pressure of 248.2 kPa (36 psi), and the same driver length were used for the third shot. The peak reflected pressure and reflected impulse were 53.4 kPa and 347.3 kPa-ms, respectively. The positive phase duration was 13.0 ms. Maximum mid-height displacement was 5.54 mm with residual displacement of 2.02 mm. Maximum deflection occurred at a time of 22.7 ms. The pressure-impulse and pressure-displacement time histories are shown in Figure 4-11 (c).

More flexural hairline cracks were created at the third shot, mostly above the mid-height of the wall. The cracks did not extent across the width of the wall (Figure 4-8 (b)).

4th shot (Blast ID #9)

An increased driver pressure of 413.7 kPa (60 psi), and the same driver length were used for the fourth shot. The peak reflected pressure and reflected impulse were 76.4 kPa and 516.6 kPa-ms, respectively, with a positive phase duration was 13.5 ms. Maximum mid-height displacement was 12.0 mm with residual displacement of 4.95 mm. Maximum deflection occurred at a time of 22.5 ms. The pressure-impulse and pressure-displacement time histories are shown in Figure 4-11 (d).

More flexural cracks occurred above and below the mid-height. Some cracks now extended across the wall width, but the main cracks at the middle were still less than 0.1 mm wide. No cracks appeared on the side of the wall (Figure 4-8 (c)).

5th shot (Blast ID #11)

The driver pressure was increased to 689.5 kPa (100 psi) with no change in driver length. A peak reflected pressure of 123.1 kPa and reflected impulse of 856.2 kPa-ms were recorded. The positive phase duration was 13.9 ms. Maximum mid-height displacement was 23.2 mm. Maximum deflection occurred at a time of 26.1 ms. Residual displacement was 7.02 mm at mid-height. The pressure-impulse and pressure-displacement time histories are shown in Figure 4-11 (e).

The cracks developed more severely on the front face as well as on the side of the wall. Flexural cracks on the front of the wall, which were still hairlines less than 0.1 mm wide, were extended to

the top with some diagonal/inclined patterns as shown in Figure 4-8 (d) . It should be noted that the cracking patterns generally followed the mortar joints of the stone units as shown in Figure 4-9.

On the side of the wall, vertical cracks at the FRCM/masonry interface were created as shown in Figure 4-8 (e) indicating some initial delamination of the FRCM mortar from the stone substrate.

No diagonal shear cracks appeared on top side of the wall.

6th shot (Blast ID #15)

The driver length was extended to 3962 mm (13 ft) with the driver pressure increased to 689.5 kPa (100 psi), which approached the limiting Shock Tube testing capacity. A peak reflected pressure of 114.3 kPa and reflected impulse of 2002.7 kPa-ms were recorded. The positive phase duration was 35.0 ms. The displacement was not recorded due to the complete failure. The pressure-impulse time history is shown in Figure 4-11 (f).

After this shot, the wall was demolished completely and fell to the ground as shown in Figure 4-8 (f). The failure mechanism was a combination of debonding between the stone units and FRCM mortar, and rupture of the carbon fiber grid at the main crack which opened widely.

Figure 4-10 shows video stills of the failure mechanism of wall R-S-L during the sixth shot. The main crack located above the mid-height opened widely during the test, and the wall was detached from the frame resulting in carbon fiber rupture at the main crack. Debonding of the stone units from the substrate can also be seen.

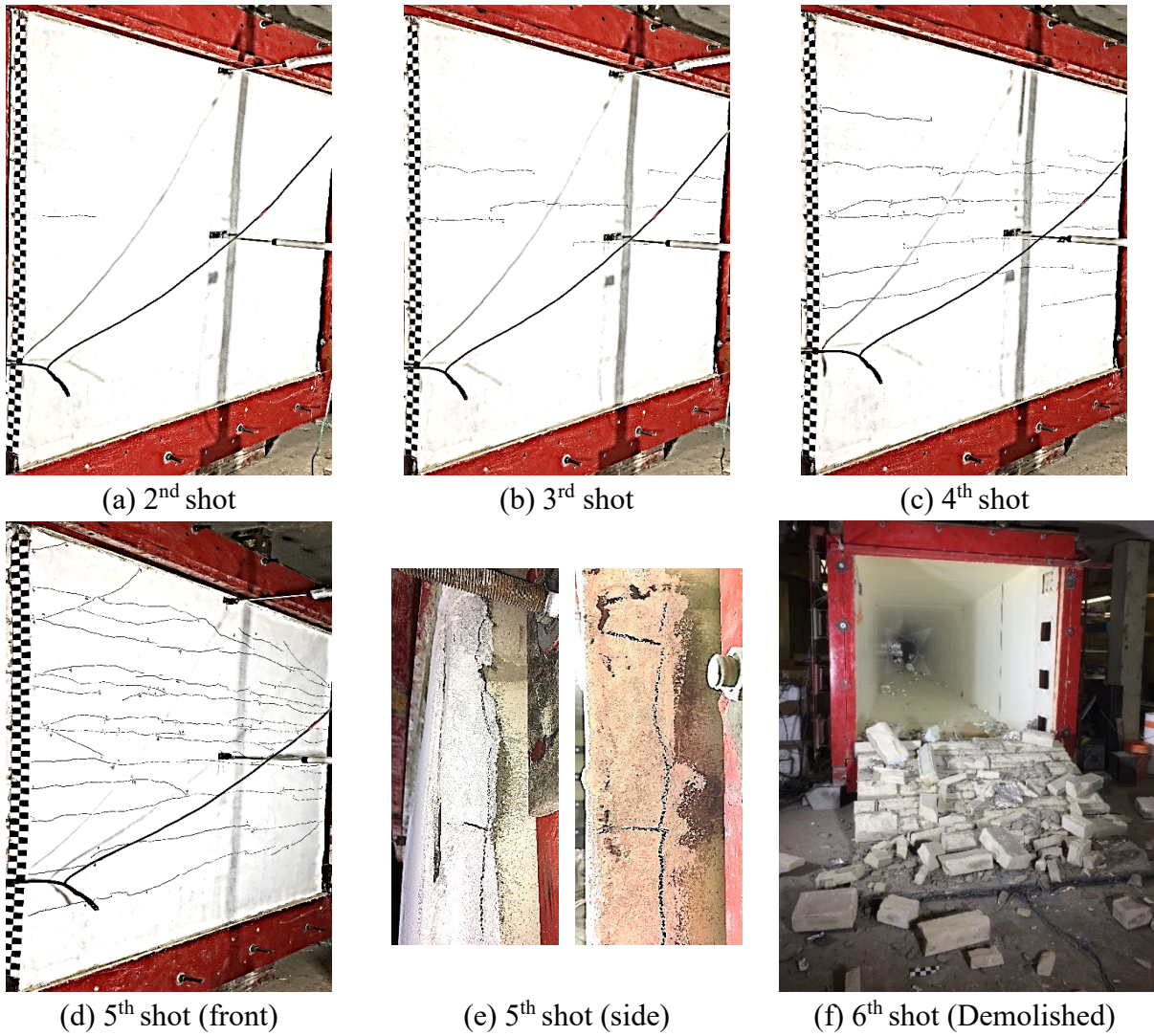


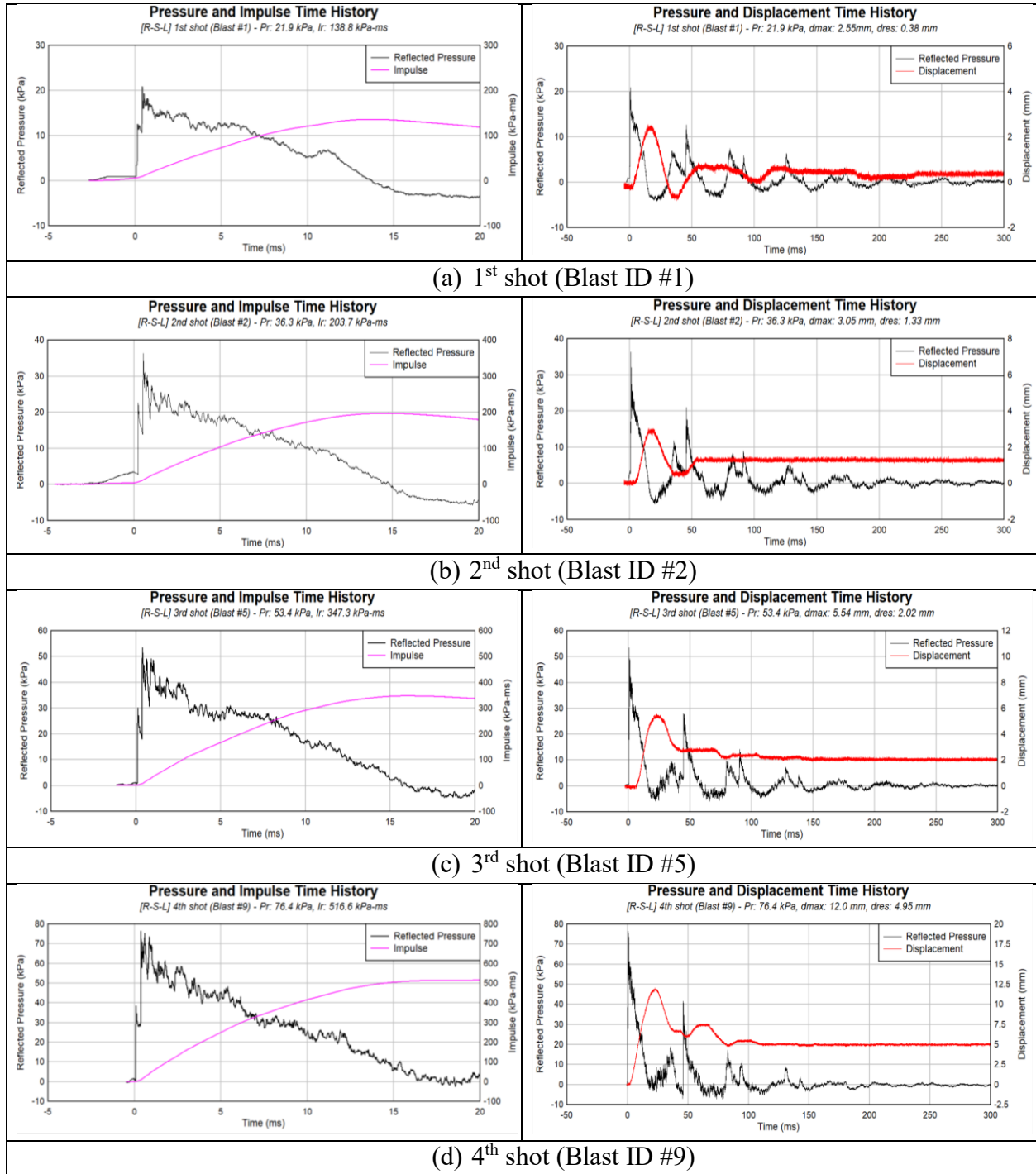
Figure 4-8. Damage Progression (from 2nd to 6th shots, no flexural cracks on 1st shot)



Figure 4-9. Comparison between the patterns of flexural cracks and joints of stone units



Figure 4-10. Video stills of failure mechanisms of R-S-L at the 6th shot



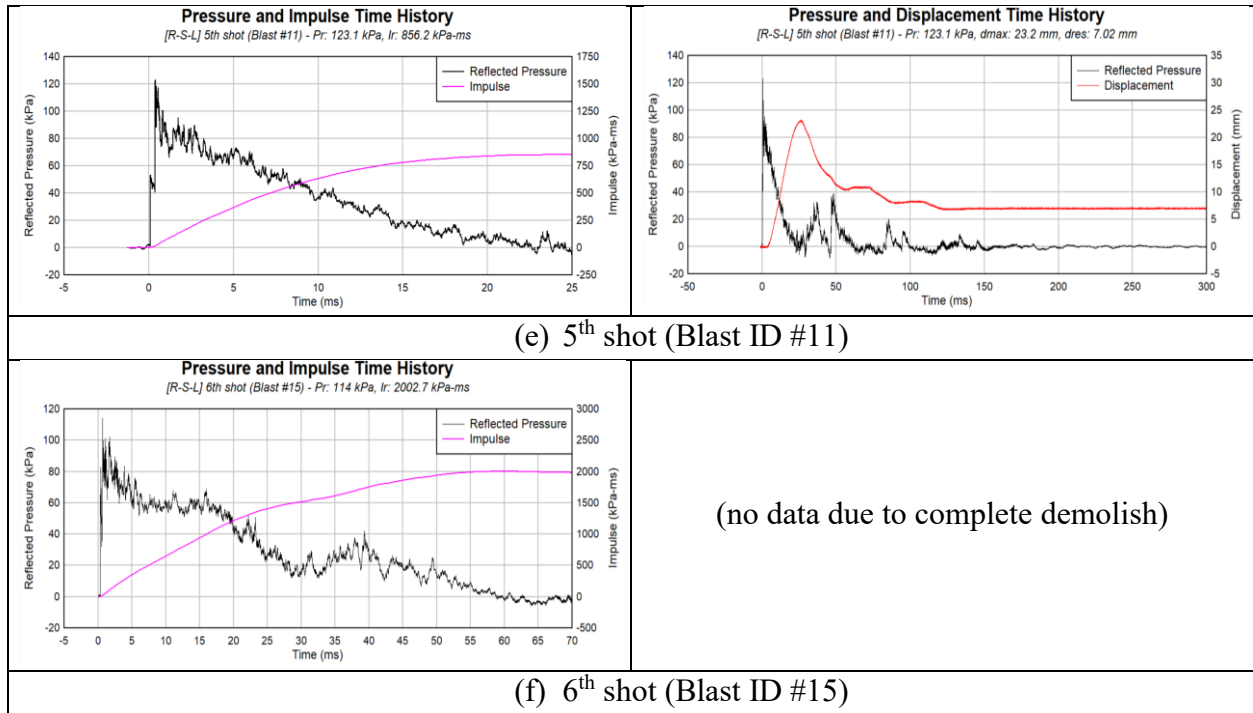


Figure 4-11. Pressure, Impulse, and Displacement Time history (R-S-L)

CHAPTER 5. DISCUSSION

5.1. OVERVIEW

The performance and test results of walls R-C-N and R-S-L are discussed in detail in this chapter. The results are compared with those obtained from the control walls tested by Gandia (2019) and Ciornei (2012). They are also compared with test results obtained by previous researchers on other retrofit methodologies.

5.2. OBSERVATIONS ON R-C-N

Stages in response: The behaviour of wall R-C-N was different from what was expected both in terms of its displacement response and failure mode, as discussed in Chapter 4. There was no visible damage and residual displacement after the first two shots. This stage can be considered to be an elastic stage (Stage I). A diagonal shear crack formed near the top support at the end of the third shot (Stage II). The fourth shot resulted in horizontal flexural cracks on the front (protected) face of the wall with additional shear cracks forming at the top. The behaviour of the wall essentially remained unchanged at the end of the fifth shot. The cracking pattern at this stage was a combination of flexural and shear cracks (Stage III). The flexural cracks widened and the CMU blocks began to crush on the back (compression) face (threat side) of the wall during the sixth shot. This type of damage continued to be observed after the eighth shot. Finally, at the ninth shot, the CMU blocks on the back face were almost destroyed due to the crushing of CMU units, with significant cracking on almost the entire front face. Therefore, these shots can be classified as the crushing stage (Stage IV).

Displacement response at wall top vs. mid-height: Figure 5-1 shows the aforementioned stages of loading and the recorded displacements at the top and mid-height points of the wall. The residual displacements at the top were generally lower than those recorded at mid-height, except for the last shot. However, it is clear that the maximum displacements were higher at the top when compared with mid-height, except for the last shot. The difference was negligibly small during the first shot (about 0.2 mm). However, the top displacement was on average 38 % higher at Shots 2 to 8. During the last shot, the top maximum displacement dropped significantly as compared to that of the previous shot. This can be explained by the additional restraint (additional horizontal HSS) which was installed prior to the last shot. The above results indicate that the wall did not have the originally intended ideal pin-pin support conditions during testing. This can also be confirmed when examining the wall displacement profile in Figure 4-1.

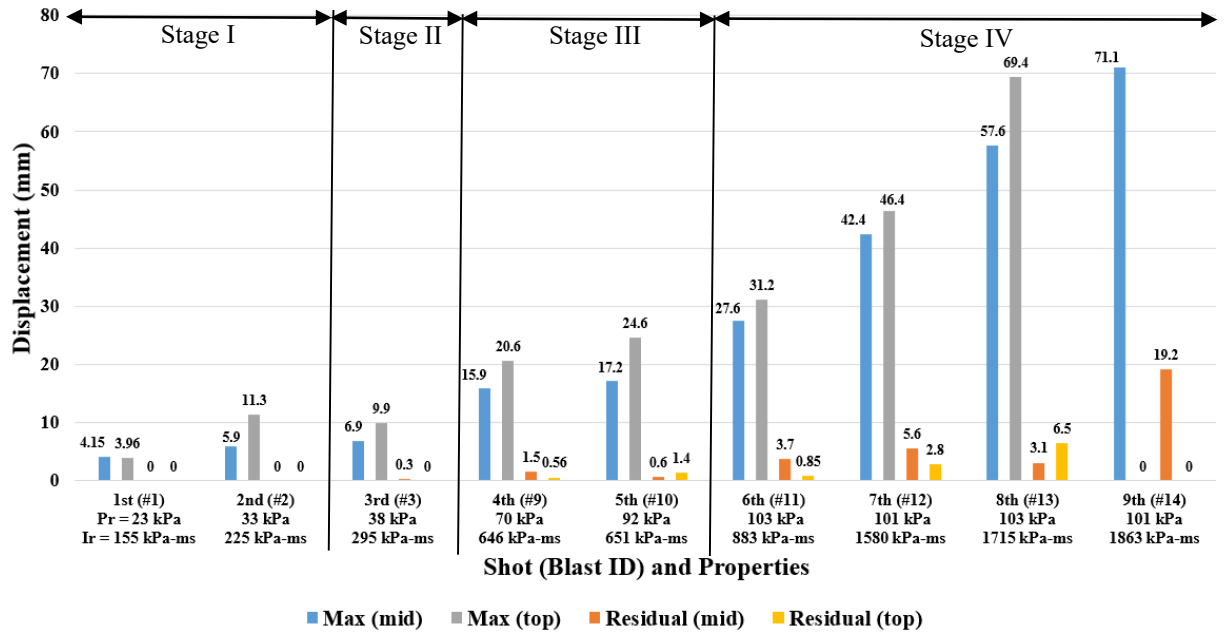


Figure 5-1. Classification of R-C-N behaviour with displacements on top and middle

Rigid body rotation: During Stage I (Shots 1 and 2), the wall displacement profile along the height was nearly linear with a maximum value at the top support. Moreover, the wall did not show any flexural cracking. Close inspection of the displacement profile shows that the wall showed a rigid body motion/rotation over the bottom support during this early stage of testing (several factors may have contributed to this motion: failure of top support stiffeners, slotted holes in the top HSS, plastic tarp between the masonry wall and grout filler). Figure 5-2 (a) shows a schematic of this rigid body motion with an exaggerated angle of rotation ($\angle \alpha$) measured from the bottom support. Using Figure 5-2 (b), it can be determined that this rigid body rotation may have contributed to crushing at the corner of the top units of CMU blocks, which may have in turn contributed to the shear cracking observed at Shot 3 and the loss of support restraint in the later stages of testing.

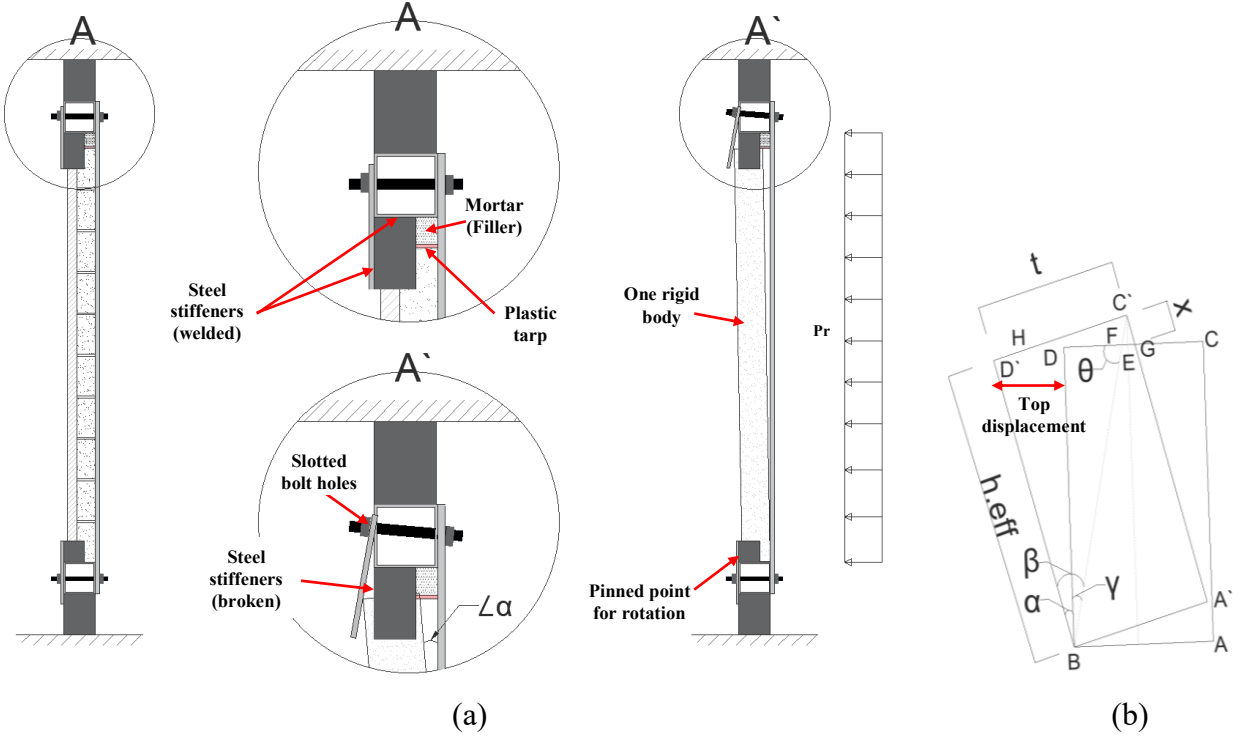


Figure 5-2. Detail of the wall movement at the 2nd shot: (a) Details before (A) and after (A') the second shot, (b) Simplified geometry

Shear cracking: As noted above a diagonal shear crack was generated during shot 3 (Stage II). To study this effect, the flexural and shear capacities of the retrofitted wall can be compared. The out-of-plane shear capacity of the wall can be computed according to CSA S304.1 (2004) as follows:

$$V = 0.16 \sqrt{f'_m} A_n + 0.25 P_d \leq 0.4 \sqrt{f'_m} A_n \quad [5-1]$$

where V , f'_m , A_n , and P_d are the out-of-plane shear resistance (N), compressive strength of masonry (MPa), net sectional area of the wall (mm^2), and axial load applied to the wall (N), respectively. For wall R-C-N, A_n was $131,400 \text{ mm}^2$, f'_m was assumed to be 11.3 MPa (average of prism tests), and the axial load, P_d was equal to zero. To account for dynamic effects a dynamic increase factor of 1.1 (CSA S850-12, 2012) was applied to f'_m . Based on these assumptions, the shear strength of the wall (V_n) is found to be 70.7 kN . Based on the assumption in Figure 5-3, the corresponding uniform pressure, $p_{u,v}$ is computed to be 35.3 kPa (i.e. $p_{u,v} = \frac{2V_n}{H \times L}$, where H : height of the wall, L : length of the wall). This pressure is a slightly lower than the peak reflected blast pressure at the third shot (38.1 kPa). Indeed, the high speed video shows a shear crack forming at the third shot, suggesting that the shear capacity of the wall was exceeded during this blast. In addition to resulting in very large shear demands, it can further be demonstrated that the use of 3 layers of carbon fabrics resulted in “over-reinforced” conditions; the flexural capacity (M_n) of R-

C-N is found to be 44 kN-m, and the corresponding pressure $p_{u,f}$ was computed as 44 kPa (i.e. $p_{u,f} = \frac{8M_n}{H^2}$) by following the procedures described in Chapter 6. Table 5-1 compares the shear and flexural capacities of the tested wall (3 plies) as well as the results for 1-2 plies of FRCM. It can be observed that the failure of the wall R-C-N, which had 3 layers of fabrics, is controlled by shear rather than flexure. It can further be demonstrated that the failure mode is governed by the crushing of the masonry, rather than the rupture of the FRCM in tension.

Table 5-1. The flexural and out-of-plane shear capacities of R-C-N**

No. of plies	Out-of-plane shear		Flexural capacity			Governing capacity
	V_n (kN)	$p_{u,v}$ (kPa)	M_n (kN-m)	$p_{u,f}$ (kPa)	Failure*	
1	70.7	35.3	21.9	21.9	F	Flexural
2			37.7	37.7	C	Shear
3			44.0	44.0	C	Shear

* F: FRCM rupture in tension, C: crushing of the masonry

** Note: for wall R-S-L: $p_{u,v}$ =66.2 kPa (shear) vs. $p_{u,f}$ =34.2 kPa, 44.2 kPa, 50.7 kPa for 1, 2, 3 plies (flexural) - Flexural failure governs for all the cases

Babaeidarabad et al. (2013) observed the same behaviour when using 4-ply of FRCM to strengthen clay masonry walls tested under out-of-plane loads, noting that “*Shear failure preceded flexural failure indicating that the level of strengthening was excessive*”. Conversely, the authors found that the use of 1-ply of FRCM resulted in flexural failure with “*formation of a horizontal crack followed by fabric slippage within the matrix until the fabric ruptured*”. The authors further noted that “*FRCM is effective in increasing stiffness and pseudo-ductility; however, pseudo-ductility is higher for lower FRCM amounts (i.e., pseudo-ductility of 4-ply strengthened wall is limited by shear failure)*”. The authors of this study further noted: “*For ω_f values larger than 0.6, shear controls failure. Consequently, increments of FRCM beyond this value are unproductive*”, where ω_f is a calibrated reinforcement ratio defined as: $\omega_f = \frac{\rho E_f}{f'_m (h_{eff}/t_m)}$, and where ρ is the ratio between the area of FRCM reinforcement and the net area of URM wall, E_f is the modulus of elasticity of FRCM, f'_m is the compressive strength of masonry, and (h_{eff}/t_m) is wall slenderness ratio.

Based on the above definition, the calibrated reinforcement ratio for wall R-C-N is determined to be $\omega_f = 1.4$ which is larger than 0.6. Therefore, it may have been more appropriate to use a lesser amount of FRCM layers to retrofit this CMU wall. Table 5-2 lists possible failure modes of the wall for 1-2-3 plies based on the calibrated reinforcement ratios, and the results are the same as the governing failure modes indicated in Table 5-1. Indeed, flexural failure is predicted to govern the response when using 1 ply of FRCM ($p_{u,shear} > p_{u,flexure}$, $\omega_f < 0.6$), but not 2 or 3 plies.

Table 5-2. Expected failure modes of R-C-N along the calibrated reinforcement ratios

No. of plies	ρ	h_{eff}/t_m	E_f	f'_m	ω_f	Failure*
	(mm ² /mm ²)	(mm/mm)	(GPa)	(MPa)		
3	0.00717	22.2	49	11.3	1.40	S
2	0.00478	22.2	49	11.3	0.93	S
1	0.00239	22.2	49	11.3	0.47	F

* S: shear failure, F: flexural failure

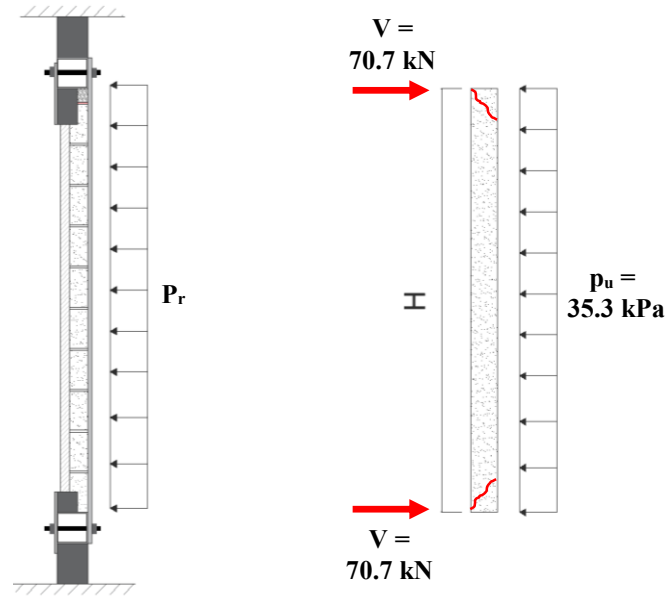


Figure 5-3. Out-of-plane shear strength capacity of R-C-N

Stage III of wall response: Despite the shear crack at Shot 3, the wall continued to resist blast pressures during Stage III (Shot 4 and 5). The wall now showed clear signs of flexural cracking indicating that the behaviour was not dominated by rigid body rotation any more. Indeed, the displacement profile was no longer linear as shown in Figure 5-4. The sketch was obtained by connecting multiple points tracked by the high speed camera software. The boundary support conditions at this stage could be assumed as consisting of a pin at the bottom and a spring at the top since the wall was able to partially translate horizontally, with some restraint due to the steel plate, which was bolted to the HSS. Despite the shear cracking at Shot 3 it is clear that the wall continued to resist significant blast pressures without collapse.

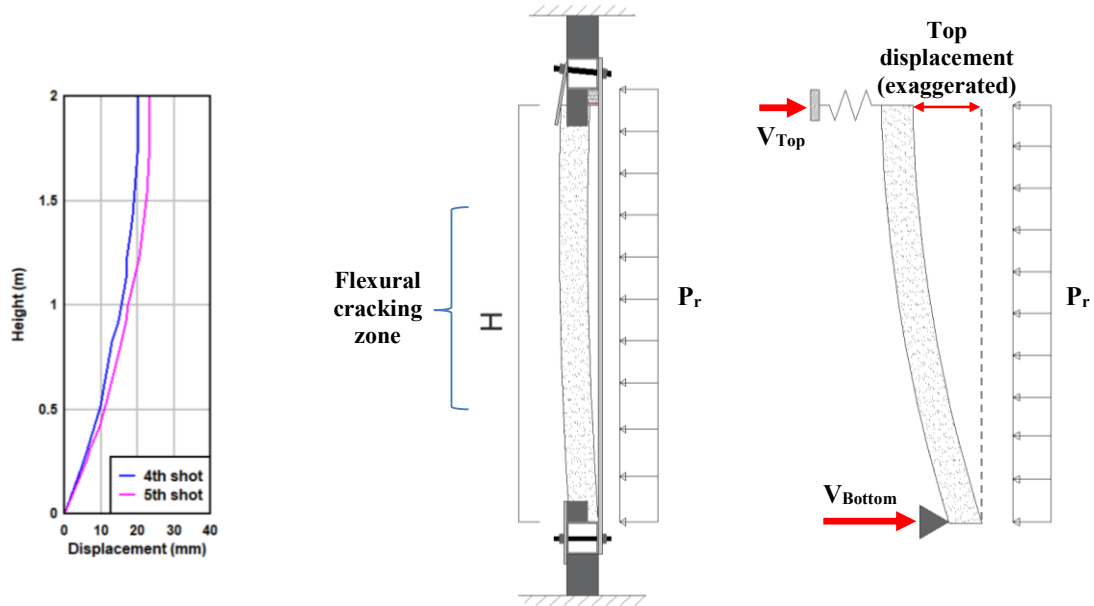


Figure 5-4. Profiles at the maximum displacement of the 4th and 5th shots

Stage IV of wall response: During Stage IV, from the sixth shot to the last shot, the wall experienced crushing of the top CMU blocks on the back face of the wall. This is illustrated schematically in Figure 5-5. Unlike the profile during Stage III, the maximum displacement occurred at 0.7-0.8 H of the wall height, with a more “curved” displacement profile. Not many flexural cracks occurred within the existing flexural cracking zone. However, many irregularly scattered cracks formed on the upper part of the wall due to the crushing of the CMU blocks. After the 8th shot, an additional horizontal HSS beam was installed to reduce the rotation of the top plate. As a result, the location of the maximum displacement at the 9th shot was lower than those of previous shots. The wall was still standing safely without any flying debris even though the back of the wall was completely destroyed. The demolished back side of the wall could be seen clearly after taking the wall down as shown in Figure 5-6 (a). The bottom four courses of CMU blocks were still intact. Figure 5-6 (b) shows the interface between CMU blocks and FRCM mortar, which was proven to have a near perfect bond.

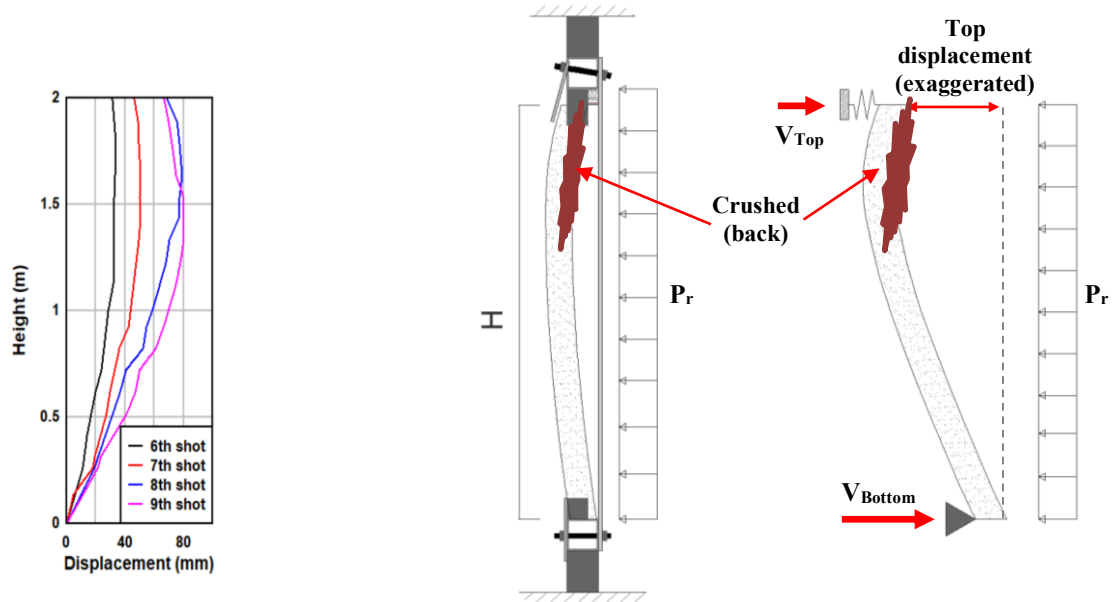


Figure 5-5. Profiles at the maximum displacement from the 6th to 9th shots



Figure 5-6. After taking the wall down: (a) Back side of the wall, (b) Interface between a CMU block and FRCM mortar

5.3. COMPARISON OF R-C-N

5.3.1. Control (URM-1) vs. Retrofit (R-C-N)

Despite the differences in boundary conditions, it is useful to compare the behavior of R-C-N with that of the control wall URM-1, which was tested by Ciornei (2012), since both are infill walls. The end supports in wall URM-1 consisted of two horizontal HSS (152 x 152 x 6.4 mm) at the top and bottom which prevented out-of-plane translation. These supports were connected to the Shock Tube frame using four vertical HSS's (76 x 76 x 6.4 mm). Figure 5-7 compares the differences in the boundary conditions in walls URM-1 and R-C-N.

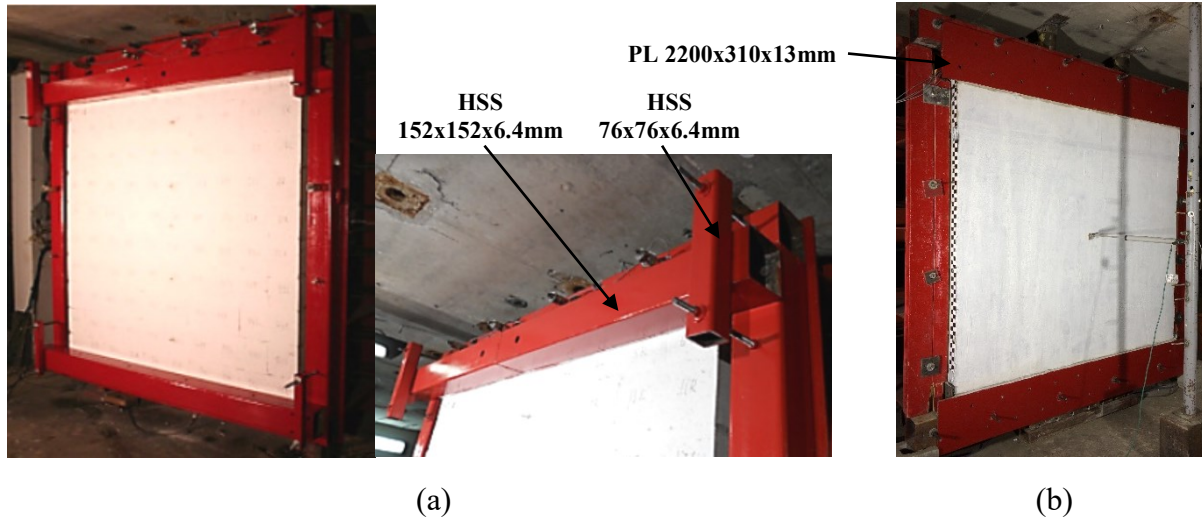


Figure 5-7. Support condition: (a) URM-1 (Ciornei, 2012), (b) R-C-N

Table 5-3. Testing results of URM-1 (Ciornei, 2012)

Wall	Blast ID (Shot)	P_d (kPa)	L_d (mm)	P_r (kPa)	I_r (kPa-ms)	d_{max} (mm)	d_{res} (mm)	Comments
URM-1	#0 (1 st)	40	1829	6.1	53.1	9.1	6.2	Cracks along the joints
	#1 (2 nd)	75.8	1829	13.7	150.9	23.8	12	More cracks and widened
	#2 (3 rd)	125.5	1829	23.4	179.3	56.5	13.9	More cracks and widened
	#3 (4 th)	165.5	1829	30.7	265.1	n/a	n/a	Completely failed
	    (1) (2) (3) (4) [Damage progression of URM-1]							

URM-1 was tested with four blast shots until failure. The test results are listed in Table 5-3. Since URM-1 did not deflect at the top, the wall displacements at mid-height in both walls are compared in Figure 5-8. URM-1 survived up to Blast #2 (P_r : 23.4 kPa, I_r : 179.3 kPa-ms) with a maximum displacement of 56.5 mm at mid-height. The wall failed at Blast #3 (30.7 kPa, 265.1 kPa-ms) in a brittle manner with flying debris (Figure 5-10). In comparison, R-C-N survived up to Blast #13 (103 kPa, 1715 kPa-ms) with a maximum displacement of 57.6 mm at mid-height. Failure in the retrofitted wall occurred at Blast #14 (101 kPa, 1863 kPa-ms). As a result, the reflected pressure and reflected impulse were increased by 229 % and 603 % at failure when compared to the control

wall (see Table 5-5). The retrofitted wall did not produce any flying projectiles at failure, despite the crushing of the masonry blocks at the back of the wall (Figure 5-10).

Table 5-4 compares the results of URM-1 and R-C-N at Blasts #0-3. Figure 5-9 compares the displacement time histories for the walls at Blast #1 and Blast #2. The maximum midspan displacements (d_{max}) of wall URM-1 at Blast #1 and Blast #2 were 23.8 mm and 56.5 mm, respectively. In comparison, wall R-C-N showed displacements of 4.15 mm and 5.9 mm. Hence, the FRCM retrofit improved the control of displacements at equivalent blasts, with reductions of 83 % and 90 % in d_{max} at Blast #1 and Blast #2 (see Table 5-5).

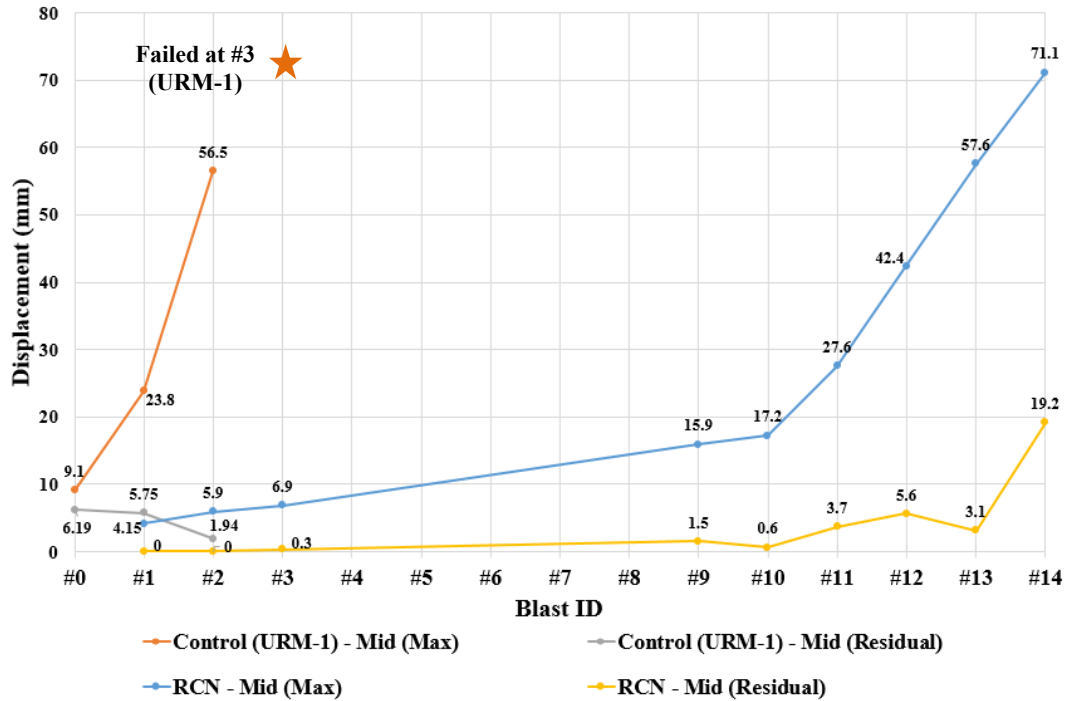


Figure 5-8. Comparison of displacement

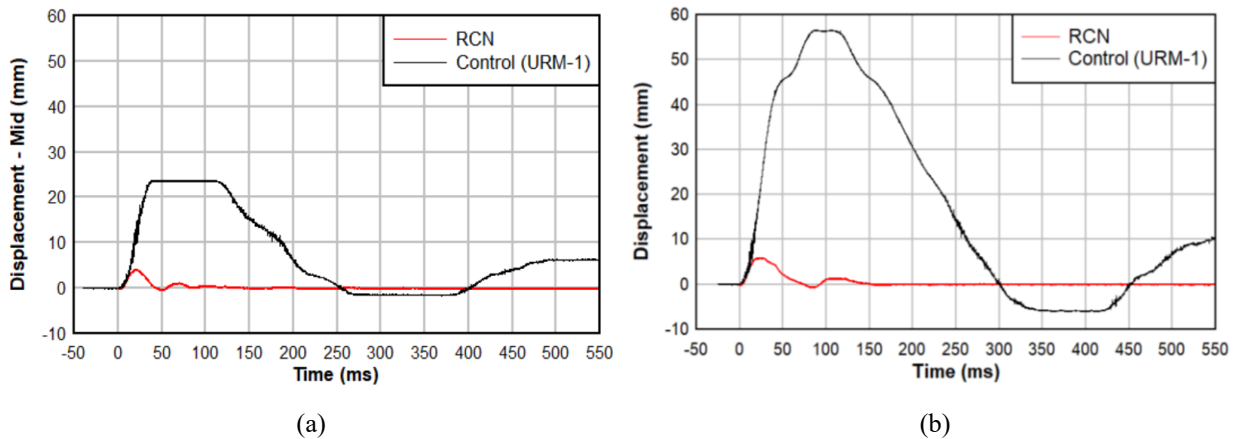


Figure 5-9. Comparison of mid and residual displacement: (a) Blast #1, (b) Blast #2

Table 5-4. Comparison of test results for R-C-N vs. URM-1 at equivalent blasts

Blast ID	Wall	P_d (kPa)	L_d (mm)	P_r (kPa)	I_r (kPa-ms)	t_d (ms)	d_{max} (mm)	d_{res} (mm)	t_{dmax} (ms)
#0	URM-1	40	1829	6.1	53.1	10	9.1	6.2	40
	R-C-N	Not tested							
#1	URM-1	75.8	1829	13.7	150.9	11.4	23.8	5.8	40
	R-C-N	82.7	1829	23.1	155.2	13.5	4.15	0	19.0
#2	URM-1	125.5	1829	23.4	179.3	13.5	56.5	1.9	92
	R-C-N	124.1	1829	32.6	225.5	13.8	5.9	0	19.6
#3	URM-1	165.5	1829	30.7	265.1	13.9	Failed		
	R-C-N	165.5	1829	38.1	295.0	15.5	6.9	0.3	19.8

Table 5-5. Comparison of performance

Wall	Blast resistance capacity at failure Blast #3 (URM-1), Blast #14 (R-C-N)		Displacement control capacity ($d_{max@mid}$, mm)	
	P_r (kPa)	I_r (kPa-ms)	Blast #1	Blast #2
URM-1	30.7	265.1	23.8	56.5
R-C-N	101.0	1862.6	4.15	5.9
Increased (or reduced) by	229 %	603 %	83 %	90 %

In summary, the retrofitted wall showed improved strength and ductility when compared to the control CMU wall, which failed in a brittle manner as shown in Figure 5-10. The improvements are due to the ability of the FRCM retrofit to increase flexural strength and stiffness, and its contribution to the increase in wall mass. The use of synthetic micro-fibers in the mortar also contributed to the increase in flexural strength and controlled the formation of cracks, while also preventing spalling. Once the main flexural cracks were formed at higher blast loads, the carbon fiber grids in the FRCM system were activated, and resisted loads until the end of testing. Even with significant cracks forming over the tension face of the wall, no spalling of the cementitious matrix occurred during the tests. The FRCM system also maintained near perfect bond to the CMU blocks with no delamination even after the intense blast testing.

However, the wall did experience a shear crack near the top support at Blast #3 (P_r : 38.1 kPa, I_r : 295 kPa-ms). Therefore, the shear resistance should be considered when strengthening unreinforced concrete masonry walls with FRCM composites.

Finally, it may be speculated that the wall could have developed additional resistance due to the arching mechanism, if the shear cracking and crushing in the top CMU blocks did not occur. It is possible that the prevention of shear damage in the support region can improve the wall behavior.

Blast ID	URM-1	R-C-N
Blast #2		
	Many flexural cracks ($d_{\max} = 56.5\text{mm}$)	No flexural cracks ($d_{\max@mid} = 5.9\text{mm}$)
At failure		
	Brittle collapse with flying debris (Blast #3)	Back crushed without flying debris (Blast #14)

Figure 5-10. Comparison of damage at the same blast and failure

5.4. OBSERVATIONS ON R-S-L

Wall R-S-L was a stone load-bearing masonry wall retrofitted with FRCM. To prevent the excessive rotation of the top support observed in the previous test an additional HSS beam (50 mm x 50 mm x 4 mm) was attached to the top plate support at the beginning of the test. As a result, the wall deformed in a typical bending pattern, and showed the behaviour of a wall with pin-pin supports, as shown in Figure 5-11. This time no shear cracks were observed at the supports. This was attributed to higher shear capacities of solid stone masonry units. An axial load of 4% (171 kN) of wall concentric capacity was applied on the wall. Therefore, the arching mechanism was expected to be activated, providing additional resistance to the wall. Figure 5-12 shows the progression of maximum and residual displacements at the mid-height of the wall.

The wall survived up to Blast #11 (P_r : 123.1 kPa, I_r : 856.2 kPa-ms), but some delamination was observed on the side of the wall at this later stage of testing (see Figure 5-11 which shows loss of bond between stone units and FRCM mortar). Stone units had relatively smoother surfaces when compared to the CMU blocks; as a result, the bond between the stone units and the FRCM mortar was not as strong as that between the CMU blocks and FRCM. Sandblasting stone surfaces may result in increased bond, potentially improving the performance of the retrofit method.

The wall was demolished completely at Blast #15 (P_r : 114 kPa, I_r : 2002.7 kPa-ms). The reflected pressure at Blast #11 was higher than that at Blast #15. However, the reflected impulse at Blast #15 was much higher than Blast #11 due to the increase in the driver length (positive phase duration was larger). The increased impulse and positive phase duration result in increased blast energy and a longer duration of load application on a wall. Furthermore, multiple shots applied to the wall may have some effect on its behavior, as opposed to a single larger shot. It is recommended that future research include walls tested under single blast shots to better understand the ultimate capacity of the FRCM retrofit methodology.

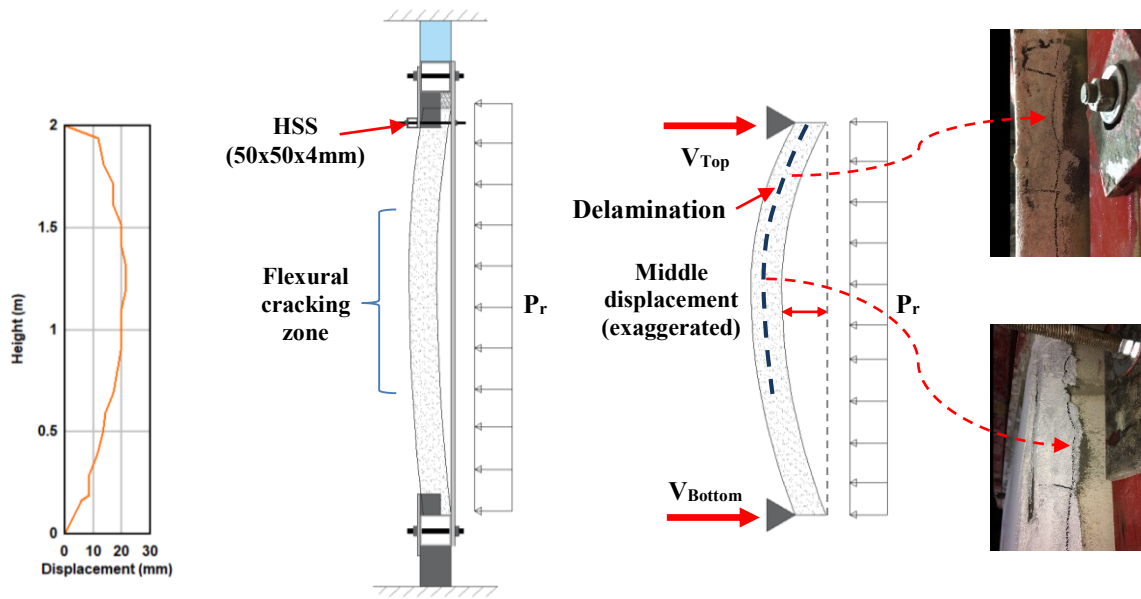


Figure 5-11. Profile at the maximum displacement of the 5th shot

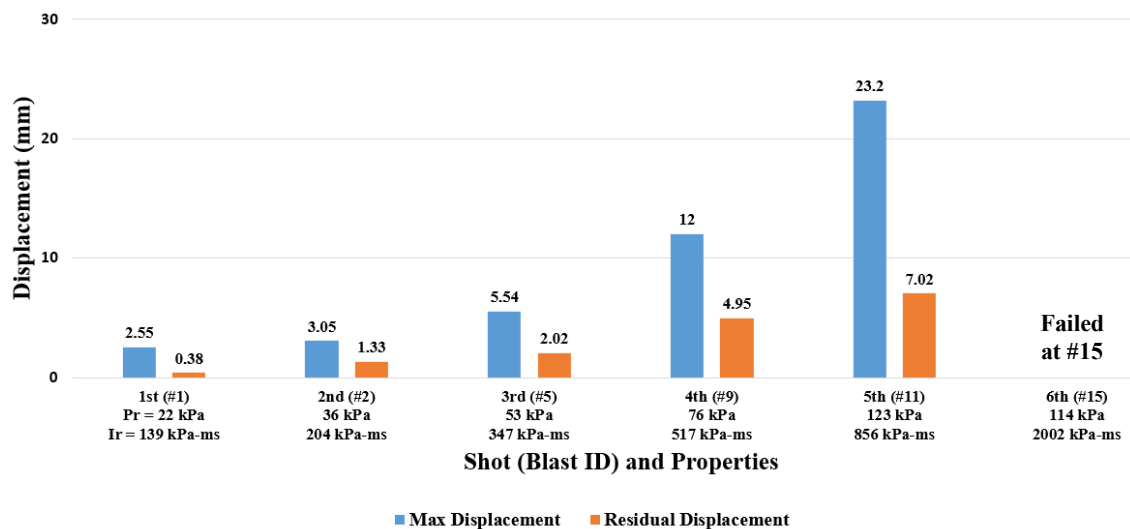


Figure 5-12. Maximum and residual displacements at the middle of R-S-L

5.5. COMPARISON OF R-S-L

5.5.1. Control (URM-S-L) vs. Retrofit (R-S-L)

URM-S-L was a stone load bearing wall without strengthening, tested by Gandia (2019). The boundary support conditions of both URM-S-L and R-S-L were the same except for the additional small HSS applied to the top support in wall R-S-L. The same types of stone units and joint mortar were used. The top edges of the walls were filled with the same type of high strength mortar. Two hydraulic jacks were used to apply equal axial loads. Instead of using three LVDTs, two LVDTs were used for R-S-L. Figure 5-13 compares the boundary conditions in the companion walls.

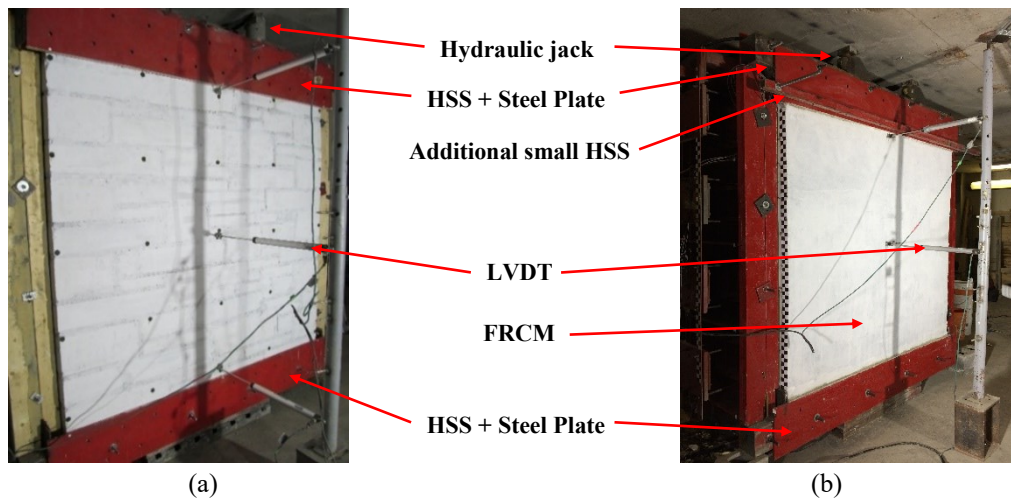
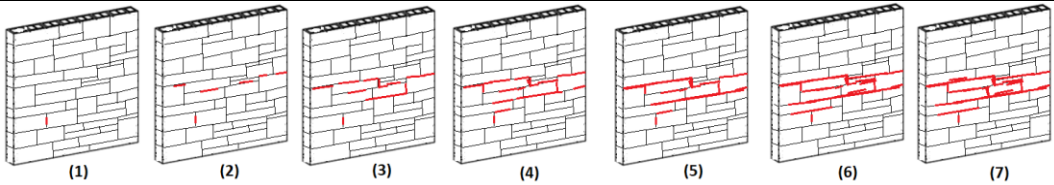


Figure 5-13. Test setup: (a) URM-S-L (Gandia, 2019), (b) R-S-L

URM-S-L was tested with eight blast shots until failure. The testing results from those eight shots are listed in Table 5-6. Figure 5-14 compares the maximum and residual displacements at mid-height in walls URM-S-L and R-S-L throughout testing. The control wall survived up to Blast # 7 (P_r : 50.2 kPa, I_r : 411.1 kPa-ms) showing horizontal and vertical cracks along the mortar joints at mid-height, and failed at Blast #8 (57.1 kPa, 447.4 kPa-ms) in a brittle manner producing many debris as shown in Figure 5-16. Despite the failure, this wall showed high blast capacity due to the increase in out-of-plane resistance from arching action.

In comparison, R-S-L survived up to Blast #11 (123.1 kPa, 856.2 kPa-ms) with many flexural cracks, and failed at Blast #15 (114 kPa, 2002.7 kPa-ms) with debonding of the FRCM, followed by rupture of the carbon fiber grids as shown in Figure 5-16. The maximum displacements at mid-height right before the failure were 38.4 mm and 23.2 mm for URM-S-L and R-S-L, respectively.

Table 5-6. Testing results of URM-S-L (Gandia, 2019)

Wall	Blast ID (Shot)	P_d (kPa)	L_d (mm)	P_r (kPa)	I_r (kPa-ms)	d_{max} (mm)	d_{res} (mm)	Comments
URM-S-L	#1 (1 st)	79.3	1829	13.5	122.2	5.8	0	Small vertical crack at midpoint
	#2 (2 nd)	130.3	1829	26.6	193.3	11.7	0.2	Horizontal hairline cracks
	#3 (3 rd)	173.7	1829	33.9	236.1	15.5	0.2	Main crack widened to 0.1mm
	#4 (4 th)	213.7	1829	39.5	287.4	18.3	0.5	Crack widened to 0.4mm
	#5 (5 th)	248.2	1829	43.7	321.2	21.7	0.3	Cracks elongated and widened to 0.7mm
	#6 (6 th)	288.2	1829	46.5	371.9	28.2	0.6	Some mortar joint at mid-height fell out
	#7 (7 th)	300.2	1829	50.2	411.1	38.4	2.3	Cracks elongated and widened
	#8 (8 th)	370.2	1829	57.1	447.4	n/a	n/a	Complete failure of wall
 [Damage progression of URM-S-L]								

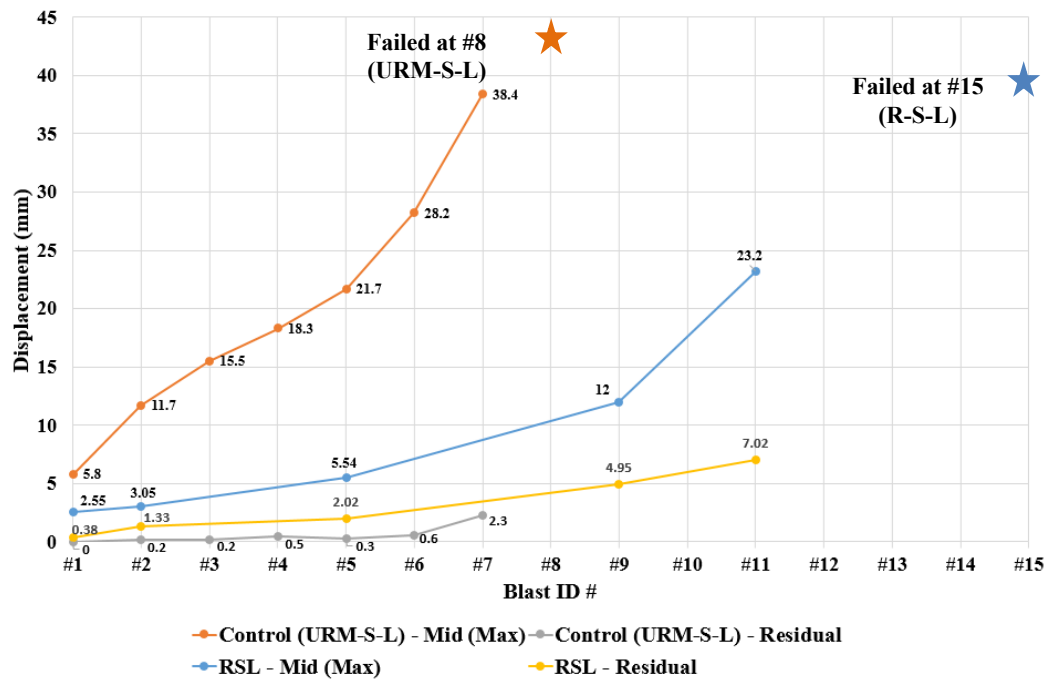


Figure 5-14. Comparison of mid and residual displacement

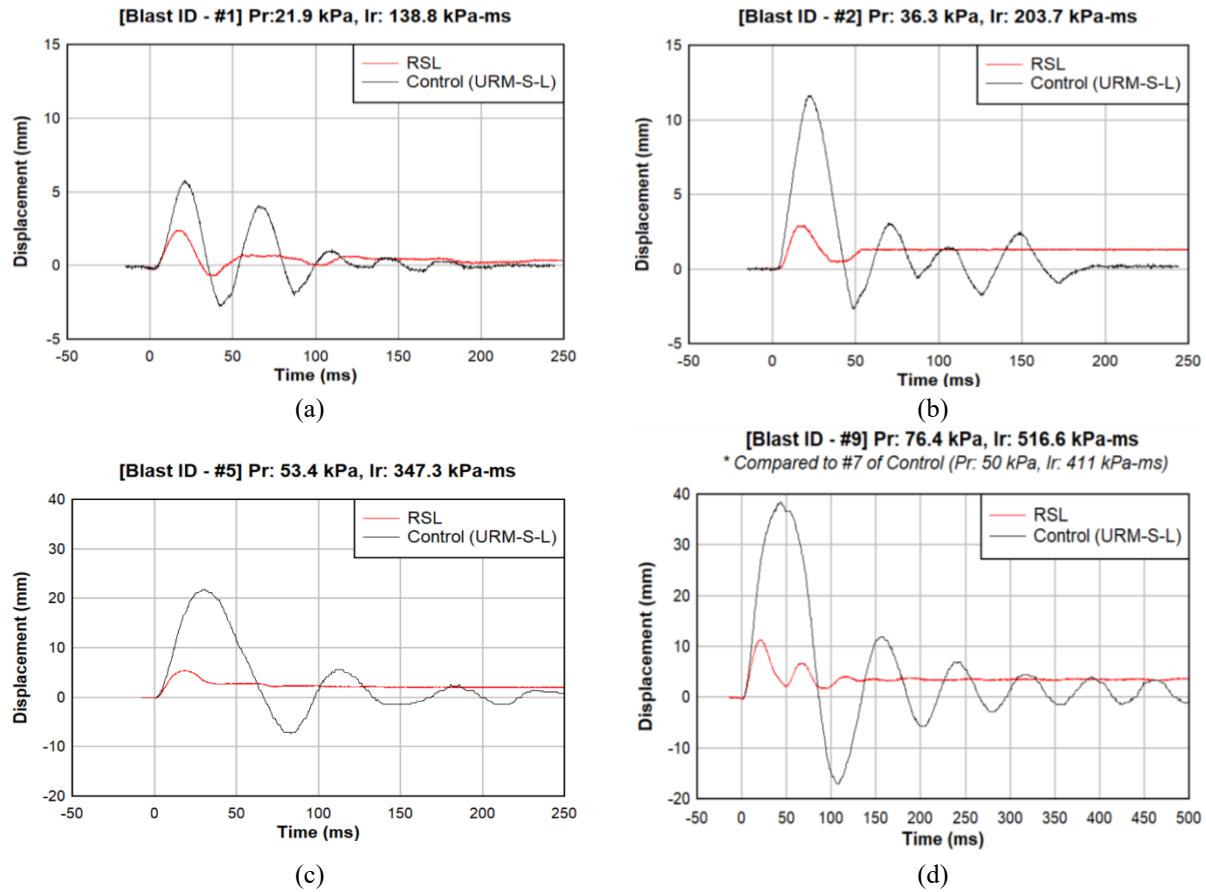


Figure 5-15. Comparison of mid-displacement: Blast ID (a) #1, (b) #2, (c) #5, (d) #9

Table 5-7 compares the displacement results for the walls at similar blasts (same blast IDs). Figure 5-15 shows the comparison of the mid-height displacement time histories at these equivalent blasts. Since Blast #7 was not conducted on R-S-L, Blast #9 of R-S-L is compared to Blast #7 of URM-S-L, which was the last shot before failure. The maximum displacements of the control wall at Blast #1, #2, and #4 were 5.8 mm, 11.7 mm, and 21.7 mm, respectively, whereas those of the retrofit were 2.55 mm, 3.05 mm, and 5.54 mm. Even though the maximum displacements of the control were always larger than those of R-S-L, the residual displacements of the control were smaller than those of R-S-L as shown in Figure 5-14 and Figure 5-15.

Table 5-7. Comparison of test results for R-S-L vs. URM-S-L at equivalent blasts

Blast ID	Wall	P_d (kPa)	L_d (mm)	P_r (kPa)	I_r (kPa-ms)	t_d (ms)	d_{max} (mm)	d_{res} (mm)	t_{dmax} (ms)
#1	URM-S-L	79.3	1829	13.5	122.2	18.1	5.8	0	21.4
	R-S-L	82.7	1829	21.9	138.8	12.7	2.55	0.38	14.6
#2	URM-S-L	130.3	1829	26.6	193.3	14.5	11.7	0.2	22.5
	R-S-L	124.1	1829	36.3	203.7	11.2	3.05	1.33	15.2

Blast ID	Wall	P_d (kPa)	L_d (mm)	P_r (kPa)	I_r (kPa-ms)	t_d (ms)	d_{max} (mm)	d_{res} (mm)	t_{dmax} (ms)
#5	URM-S-L	248.2	1829	43.7	321.2	14.7	21.7	1.2	30.7
	R-S-L	248.2	1829	53.4	347.3	13.0	5.54	2.02	22.7
(#7) #9	URM-S-L	330.2	1829	50.2	411.1	16.4	38.4	4.1	42.5
	R-S-L	413.7	1829	76.4	516.6	13.5	12	4.95	22.5

Table 5-8 summarizes the performance of the two walls. At failure, the reflected pressure and reflected impulse were greatly increased by 100 % and 348 % for the retrofit wall; the control failed at Blast #8 (57.1 kPa, 447.4 kPa-ms), and the retrofit failed at Blast #15 (114 kPa, 2002.7 kPa-ms). The maximum mid-span displacements were also reduced by 56 %, 74 %, and 74 % at Blast #1, #2, and #5, respectively. Thus, the FRCM retrofit was proven to be very effective in strengthening the stone masonry wall by increasing its blast resistance and improving its displacement control at equivalent blasts.

Table 5-8. Comparison of performance

Wall	Blast resistance capacity at failure Blast #8 (URM-S-L), Blast #15 (R-S-L)		Displacement control capacity ($d_{max@mid}$, mm)		
	P_r (kPa)	I_r (kPa-ms)	Blast #1	Blast #2	Blast #5
URM-S-L	57.1	447.4	5.8	11.7	21.7
R-S-L	114	2002.7	2.55	3.05	5.54
Increased (or reduced) by	100 %	348 %	56 %	74 %	74 %

According to the comparison of the crack patterns shown in Figure 5-16, R-S-L controlled the cracks better than URM-S-L by distributing cracks all over the tension face owing to the polypropylene fibers embedded in the FRCM mortar and the carbon fabrics in the FRCM system. The failure modes in both walls were brittle. Wall URM-S-L failed due to complete collapse of the stone blocks. Wall R-S-L experienced local detachment of the blocks from the FRCM retrofit prior to the failure shot, and this loss of bond was the main issue leading to the failure of the wall as shown in the failure shot of Figure 5-16. The blast resistance of the retrofit could have possibly been increased by improving the bond/anchorage between the stone blocks and FRCM mortar.

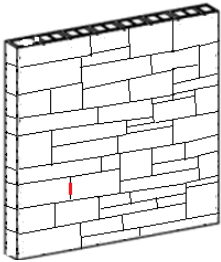
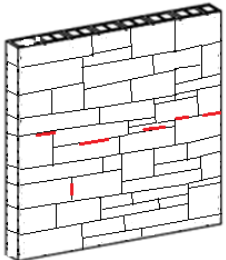
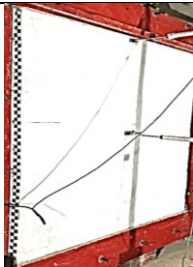
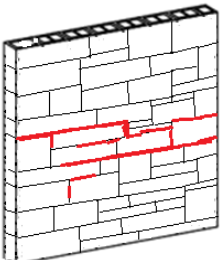
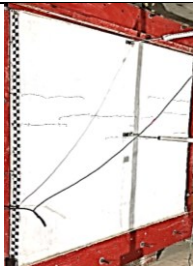
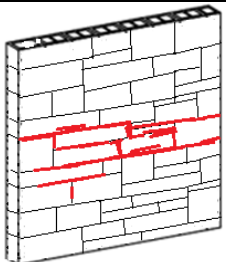
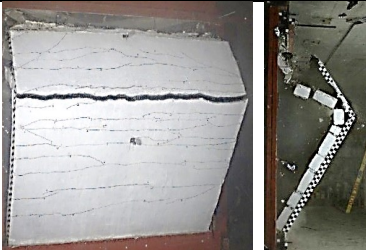
Blast ID	URM-S-L	R-S-L
Blast #1		
	Small vertical crack ($d_{\max} = 5.8\text{mm}$)	No flexural cracks ($d_{\max} = 2.55\text{mm}$)
Blast #2		
	Hairline cracks ($d_{\max} = 11.7\text{mm}$)	A short hairline crack ($d_{\max} = 5.9\text{mm}$)
Blast #5		
	Elongated and widened to 0.7mm ($d_{\max} = 14.7\text{mm}$)	More hairline cracks ($d_{\max} = 5.54\text{mm}$)
Before failure		
	Blast #7	Blast #11
At failure		
	Failure at Blast #8	Failure with partial debonding at Blast #15

Figure 5-16. Comparison of damage at the same blasts and failure

5.6. COMPARISON OF RETROFITS (FRCM vs. Shotcrete)

5.6.1. RMW-S-L (Gandia, 2019) vs. R-S-L

RMW-S-L was a stone load bearing wall strengthened with an engineered cementitious composite (ECC) shotcrete retrofit system tested by Gandia (2019). The ECC shotcrete retrofit system consisted of a 51 mm thick layer of shotcrete and embedded steel mesh (Gage 2, 152 mm x 152 mm spacing with 6.54 mm dia.) which was welded to the top and bottom HSS. Table 5-9 shows the properties of ECC and steel mesh used for RMW-S-L, and compares them to those of the FRCM system used for R-S-L. The same types of stone units and joint mortar were used. The gaps on top of the wall were filled with high strength grout. The support conditions were the same as those of R-S-L other than the additional small HSS as shown in Figure 5-17. An axial load corresponding to 4 % of the axial capacity of the wall (171 kN) was applied on both walls.

Table 5-9. Comparison of retrofits

Item	RMW-S-L (Gandia, 2019)	R-S-L
Overlay	ECC: 51 mm	FRCM mortar: 43 mm
Fiber enriched	Micro-synthetic PVA	Micro polypropylene
Compressive strength	57.2 MPa (28 days)	52 MPa (28 days)
Flexural strength	8.2 MPa (28 days)	6.9 MPa (28 days)
Reinforcement	Steel mesh (Gage 2, 6.54 mm dia.)	Unidirectional carbon grids
Yielding stress / strain	921.7 MPa / 0.005	n/a
Ultimate stress / strain	969.7 MPa / 0.011	450 kN/m / 0.015

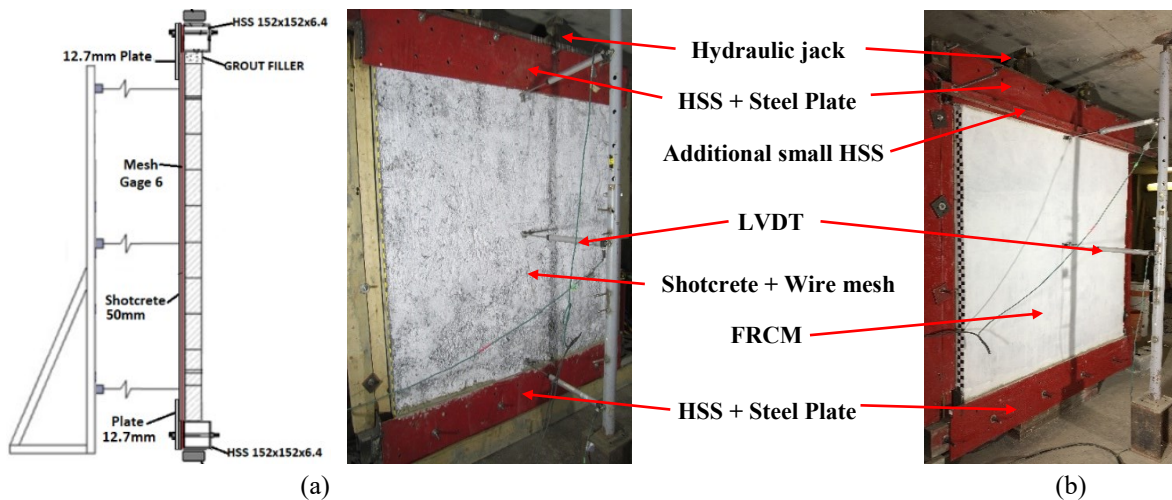


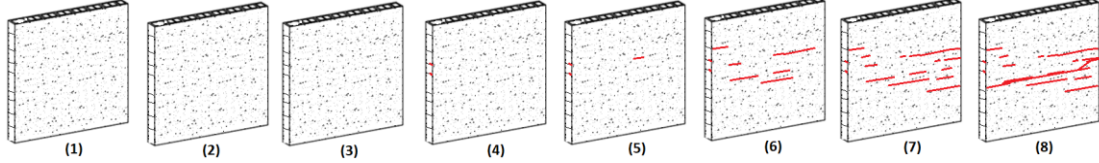
Figure 5-17. Test setup: (a) RMW-S-L (Gandia, 2019), (b) R-S-L

RMW-S-L was tested with eight shots until failure. Table 5-10 details the damage progression in this comparison wall. The first three shots did not create any cracks on the wall shotcrete layer. From the fourth shot, horizontal cracks and compression cracks were formed and lengthened until

the wall failed with a 7 mm wide main crack at mid-height of the shotcrete. The wall did not fully break apart since the steel mesh inside the retrofit did not rupture.

The progression of maximum and residual displacements at the mid-height of both walls is shown in Figure 5-18. Since both walls were tested with a different number of shots (8 shots for RMW-S-L, 6 shots for R-S-L), the results cannot be directly compared. However, the trends of the curves are very similar, indicating that both retrofits had similar blast performance. Figure 5-19 compares the mid-height displacement-time histories at similar blasts (data is also compared in Table 5-11).

Table 5-10. Testing results of RMW-S-L (Gandia, 2019)

Wall	Shot	P_d (kPa)	L_d (mm)	P_r (kPa)	I_r (kPa-ms)	d_{max} (mm)	d_{res} (mm)	Comments
RMW-S-L	1 st	129.6	1829	34.1	203.9	4.9	1.1	No damage
	2 nd	248.2	1829	56.4	339.5	8.1	2.6	No damage
	3 rd	330.9	1829	61.9	429.7	9.5	2.4	No damage
	4 th	410.2	1829	75.6	503.1	11.4	3.4	Minor cracks on the side
	5 th	555.7	1829	86	632.1	13.9	2.8	Hairline crack above mid-height
	6 th	696.4	1829	100.7	811.4	19.8	5.5	More cracks along face of wall. Compression cracks in stone blocks
	7 th	772.2	1829	107.1	865	20.6	5.1	Cracks lengthened on face of wall
	8 th	806.7	2743	109.9	1291.7	139.4	n/a	Failure at main crack (7mm width)
 [Damage progression of RMW-S-L]								

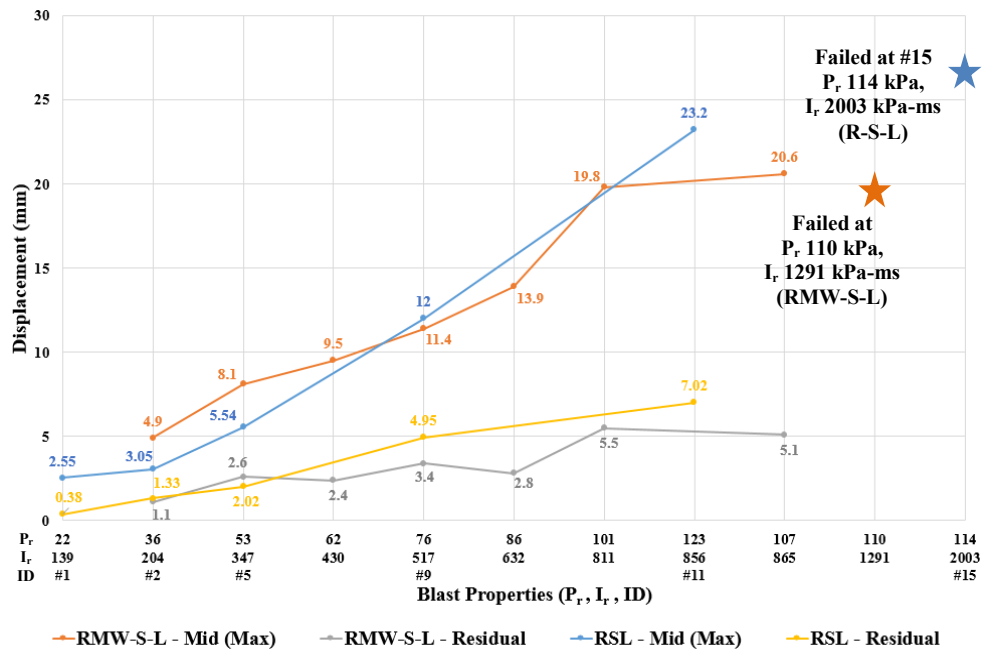


Figure 5-18. Comparison of mid and residual displacement

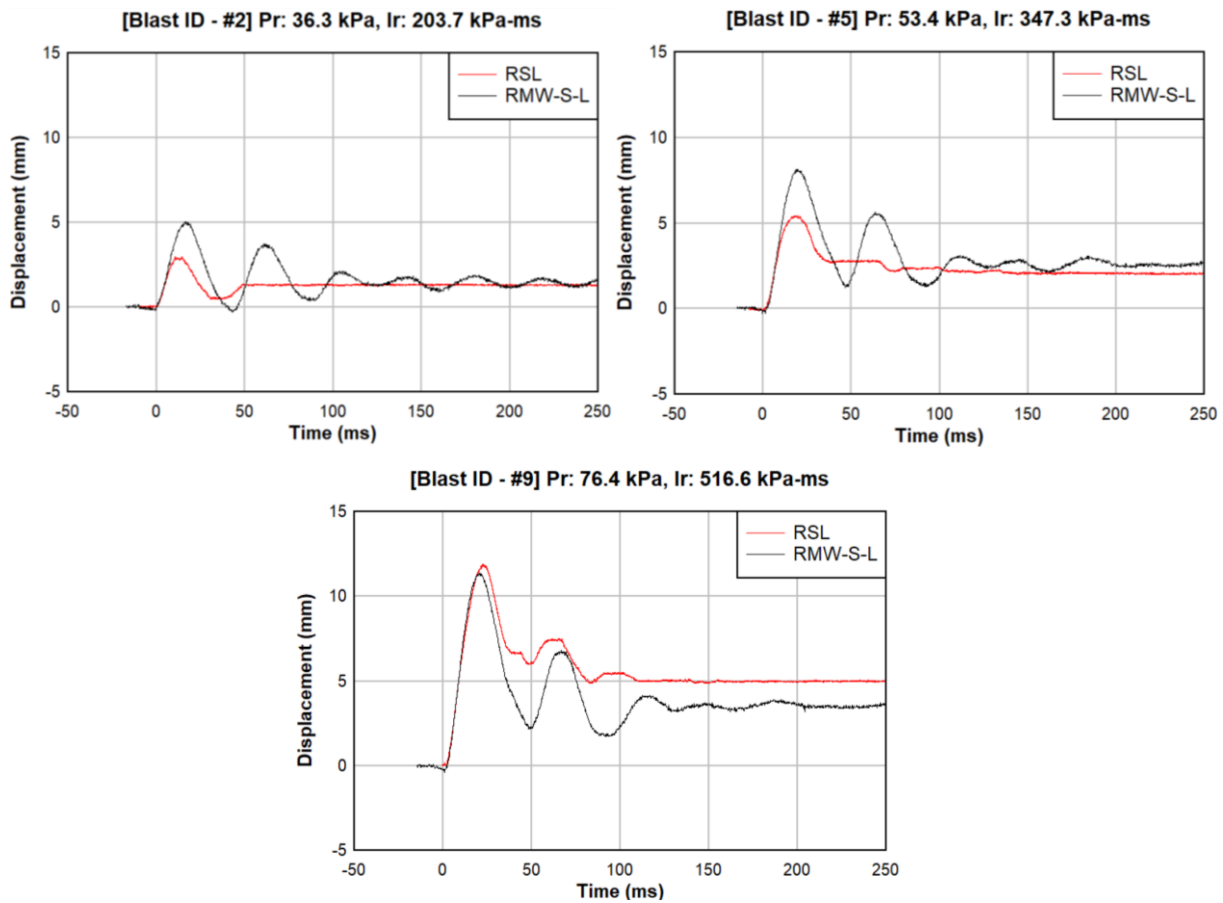


Figure 5-19. Comparison of mid-displacement at the same blasts

Table 5-11. Comparison of test results at the same blasts

Blast ID	Wall	P_d (kPa)	L_d (mm)	P_r (kPa)	I_r (kPa-ms)	t_d (ms)	d_{max} (mm)	d_{res} (mm)	t_{dmax} (ms)
#2	RMW-S-L	129.6	1829	34.1	203.9	12.0	4.9	1.1	18.4
	R-S-L	124.1	1829	36.3	203.7	11.2	5.9	0	19.6
#5	RMW-S-L	248.2	1829	56.4	339.5	12.0	8.1	2.6	19
	R-S-L	248	1829	53.4	347.3	13.0	5.54	3.73	22.7
#9	RMW-S-L	410.2	1829	75.6	503.1	13.3	11.4	3.4	21.1
	R-S-L	414	1829	76.4	516.6	13.5	12	4.95	22.5

Table 5-12 further compares the performance of the walls in terms of blast resistance and displacement control. At Blast #2 (P_r : 36 kPa, I_r : 204 kPa-ms), which was similar to the first shot (34 kPa, 204 kPa-ms) of RMW-S-L, wall R-S-L showed a 38 % reduction in maximum

displacement when compared to RMW-S-L. The same trend was found for Blast #5 (53 kPa, 347 kPa-ms), which was similar to the second shot (56 kPa, 340 kPa-ms) of RMW-S-L, with R-S-L displaying a 37 % improvement in displacement control. At Blast #9 (76 kPa, 517 kPa-ms), or the fourth shot (76 kPa, 503 kPa-ms) of RMW-S-L, the maximum displacements were similar for both walls, with a 5 % difference in results.

In terms of blast resistance capacity, RMW-S-L failed at $P_r = 109.9$ kPa, $I_r = 1291.7$ kPa-ms, whereas R-S-L failed at $P_r = 114$ kPa, $I_r = 2002.7$ kPa-ms. Even though the reflected pressures were almost the same, their reflected impulse were different due to the difference in driver lengths: 9 ft and 13 ft long drivers were used for RMW-S-L and R-S-L, respectively. Thus wall R-S-L was tested under a higher impulse (blast energy) and subjected to a shock wave with a longer positive phase duration. Based on the above, the failure blast for R-S-L had a 4 % increase in pressure and 55 % increase in impulse when compared to its companion. Given that the wall failed in a brittle manner no conclusions can be drawn regarding the increase in blast resistance.

Table 5-12. Comparison of performance

Wall	Blast resistance capacity at failure		Displacement control capacity ($d_{\max@mid}$, mm)		
	P_r (kPa)	I_r (kPa-ms)	Blast #2	Blast #5	Blast #9
RMW-S-L	109.9	1291.7	4.9	8.1	11.4
R-S-L	114	2002.7	3.05	5.54	12.0
Increased (or reduced) by	+ 4 %	+ 55 %	- 38 %	- 37 %	+ 5 %

Figure 5-20 shows the comparison of damage at the same blasts (Blast # 2, #5, #9) before failure, and the failure modes in both wall specimens. RMW-S-L did not experience many cracks until it failed, whereas R-S-L had many scattered horizontal and diagonal cracks all over the tension face. RMW-S-L did not collapse at failure as the steel mesh did not rupture, but R-S-L was destroyed and fell down entirely as the portions of stone units were detached from the FRCM system. In conclusion both retrofit systems can be considered to be effective in increasing the blast resistance of stone load-bearing masonry walls.

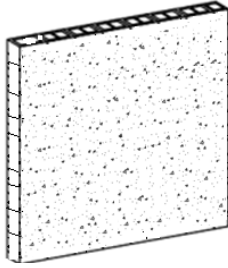
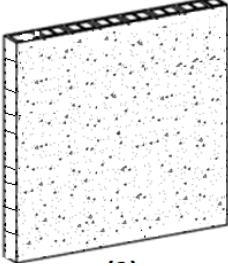
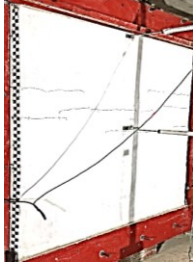
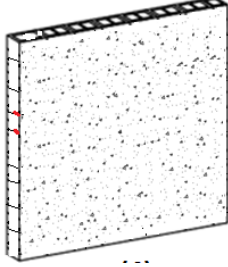
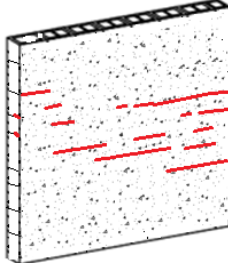
Blast ID	RMW-S-L	R-S-L
Blast #2		
	No damage ($d_{\max} = 4.9\text{mm}$, 1 st shot)	A short hairline crack ($d_{\max} = 3.05\text{mm}$, 2 nd shot)
Blast #5		
	No damage ($d_{\max} = 8.1\text{mm}$, 2 nd shot)	More hairline cracks ($d_{\max} = 5.54\text{mm}$, 3 rd shot)
Blast #9		
	Minor cracks on the side ($d_{\max} = 11.4\text{mm}$, 4 th shot)	More hairline cracks ($d_{\max} = 12.0\text{mm}$, 4 th shot)
Before failure		
	P_r 107 kPa, I_r 865 kPa-ms ($d_{\max} = 8.1\text{mm}$, 7 th shot)	P_r 123 kPa, I_r 856 kPa-ms ($d_{\max} = 23.2\text{mm}$, 5 th shot)
At failure		
	P_r 110 kPa, I_r 1291 kPa-ms (8 th shot)	P_r 114 kPa, I_r 2003 kPa-ms (6 th shot)

Figure 5-20. Comparison of damage at the same blasts and failure

CHAPTER 6. ANALYSIS

6.1. OVERVIEW

Single Degree of Freedom (SDOF) analysis was performed to predict the blast behaviour of wall R-S-L in terms of maximum mid-height displacement. The first step in the analysis was the computation of idealized resistance curve for the wall. This was done by computing wall flexural strength using a sectional analysis (Plane Section Analysis). The experimental resistance function was also obtained from the recorded test data as wall's restoring force–deformation relationship. The idealized and recorded resistance functions were compared. The SDOF analysis was conducted utilizing the resistance functions. The analytical and experimental displacement time histories were then compared to assess the behaviour of the wall and the effectiveness of the retrofit strategy implemented.

Wall R-C-N was not analyzed by this method since it failed prematurely in shear as discussed in Chapter 5. The resistance function for this wall could not be established easily, and the SDOF analysis based on a flexural resistance function would not result in a meaningful response history.

6.2. FLEXURAL STRENGTH OF R-S-L

The flexural strength of the FRCM retrofitted masonry wall can be calculated according to ACI 549.4R (2013) and CSA S304.1 (2004). The sectional analysis can be conducted based on the following assumptions: i) strain compatibility between the masonry and FRCM; ii) plane sections before bending remain plane after bending; iii) the contribution of FRCM prior to cracking can be neglected. The governing failure mode can be the smaller of compressive crushing of masonry (i.e. when the limiting compressive strain capacity of masonry is exceeded), FRCM fabric rupturing (i.e. the tensile strain of the fabric exceeds its ultimate tensile capacity), or FRCM debonding from the masonry substrate. ACI 549.4R (2013) states that the minimum length of the FRCM system applied to any substrate should be more than 152 mm (6") to secure effective stress development without fabric pull-out from the FRCM system which was the case in the current study.

ACI 549.4R (2013) limits the effective tensile strain (ϵ_{fe}) of a FRCM system to 0.012, which is the limiting design tensile strain (ϵ_{fd});

$$\epsilon_{fe} = \epsilon_{fd} \leq 0.012 \quad [6-1]$$

The effective tensile stress (f_{fe}) can be obtained from Equation [6-2], where E_f is the tensile modulus of elasticity of the cracked FRCM system:

$$f_{fe} = E_f \epsilon_{fe} \quad [6-2]$$

The nominal flexural strength (M_n) is calculated with Equation [6-3]. M_m is the moment contribution of reinforced masonry, and can be neglected in URM walls which are not subjected to axial load. M_f is the moment contribution of the FRCM system.

$$M_n = M_m + M_f \quad [6-3]$$

The moments can be calculated using sectional analysis as shown in Figure 6-1. The parabolic distribution of compressive stress of the masonry is simplified using an equivalent rectangular stress block defined by γ and β_1 factors. ACI 549.4R (2013) suggests using 0.7 for both γ and β_1 for simplicity. The tension force (T) provided by FRCM, and the compression force (C) provided by the masonry can be calculated with Equations [6-4] and [6-5]. For the tensile strain (ϵ_f) of 3 layers of fabrics, a single layer of fabric with the same cross-sectional area is assumed to be placed at the centreline of the FRCM composite (Figure 6-1). For the load-bearing wall R-S-L, the axial load can be considered as an additional compressive force (C') described with Equation [6-6].

$$T = (A_f L) f_f = (A_f L) E_f \epsilon_f \quad [6-4]$$

$$C = \gamma f'_m (\beta_1 c) L \quad [6-5]$$

$$C' = f_{cs} (t_m + t_f) L \quad [6-6]$$

where A_f , L , f'_m , c , t_m , t_f , and f_{cs} are the total area of fabrics, length of the wall, compressive strength of masonry prisms, depth of the neutral axis measured from the extreme compression fiber, thickness of a masonry wall, thickness of FRCM retrofit, and axial compressive stress due to an axial load, respectively. Then, the moment taken about the mid-depth of the section can be calculated with Equation [6-7].

$$M_n = C \left(\frac{t_m + t_f}{2} - \frac{\beta_1 c}{2} \right) + T \left(\frac{t_m + t_f}{2} - \frac{t_f}{2} \right) \quad [6-7]$$

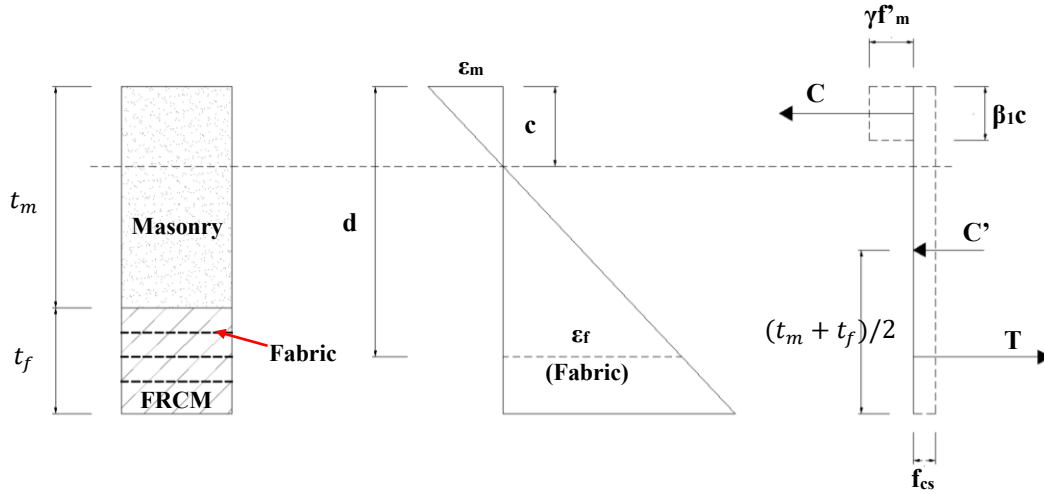


Figure 6-1. Sectional analysis of the FRCM strengthened wall

According to ACI 549.4R (2013), the arching mechanism can be ignored if a wall acts like a simply supported element. It can be activated when a wall has stiff supports with a height-to-thickness ratio of less than 8. Also, the out-of-plane shear strength of a masonry wall is not affected by the FRCM composite.

The cracking flexural strength (M_{cr}) can be calculated in accordance with Equation [6-8], where S and f_r are the sectional modulus of the uncracked wall, and modulus of rupture, respectively. For the modulus of rupture of the FRCM retrofitted masonry wall, a weighted average strength can be used based on relative areas, since the elastic behaviour of one material is assumed for the purpose of section analysis. Modulus of rupture of masonry prisms (f_{rm}) is taken as 0.4 MPa for Type S mortar as per CSA S304.1-04 (2004), and that of FRCM mortar (f_{rfm}) is taken as 4.8 MPa from the manufacturer data sheet.

$$M_{cr} = (f_r + f_{cs})S \quad [6-8]$$

$$f_r = \frac{f_{rm}A_n + f_{rfm}A_{fm}}{A_n + A_{fm}} \quad [6-9]$$

where, A_n is the net sectional area of masonry, and A_{fm} is the area of FRCM composite.

According to CSA S304.1-04 (2004), the mid-height deflection of a simply supported wall can be obtained in accordance with Equation [6-10] at cracking (Δ_{cr}), and Equation [6-11] at ultimate (Δ_u), without considering the arching mechanism.

$$\Delta_{cr} = \frac{5M_{cr}H^2}{48E_mI_o} \quad [6-10]$$

where H , E_m , I_o are the height of a wall, modulus of elasticity of a masonry wall, and uncracked moment of inertia of a wall, respectively. The modulus of elasticity of a masonry wall can be calculated with Equation [6-12] as per CSA S304.1-04 (2004).

$$\Delta_u = \frac{5M_{cr}h^2}{48E_mI_o} + \frac{5(M_n - M_{cr})H^2}{48E_mI_{cr}} \quad [6-11]$$

$$E_m = 850f'_m \leq 20000 \text{ MPa} \quad [6-12]$$

where I_{cr} is the cracked moment of inertia of the wall, which can be computed according to Equation [6-13]. The transformed section of the retrofitted wall is used.

$$I_{cr} = \frac{Lc^3}{12} + Lc\left(\frac{c}{2}\right)^2 + \frac{E_f}{E_m}A_f\left(t_m + \frac{t_f}{2} - c\right)^2 \quad [6-13]$$

Once the flexural strength of the wall is known, the moment can be converted to the corresponding uniformly distributed load (w) and the pressure (p) in accordance with Equation [6-14] and [6-15], respectively, with the assumption of simply supported condition.

$$w = \frac{8M}{H^2} \quad [6-14]$$

$$p = \frac{w}{L} \quad [6-15]$$

Calculation of Flexural strength of R-S-L

The flexural strength of the stone load-bearing wall (R-S-L) can be calculated following the procedure in Section 6.2. The material properties for the wall were obtained from the material tests (for the masonry) and the manufacturer data sheet (for the FRCM composite). The material properties for wall R-S-N are listed in Table 6-1. The properties of FRCM composite were taken from the data sheet since no coupon tests were conducted for the case of three layers of carbon fabric and because the tests conducted by the author (using 1 and 2 layers) were found to fail due to fabric slippage rather than rupturing, whereas the wall R-S-L experienced fabric rupturing.

Table 6-1. Material Properties for R-S-L

Masonry (Stone prism)	Compressive strength (f'_m)	9.7 MPa
	Ultimate compressive strain (ϵ_{mu})	0.0058
FRCM composite *	Ultimate tensile strain (ϵ_{fu})	0.011
	Ultimate tensile strength (f_{fu})	885 MPa
	Cracked tensile modulus of elasticity(E_f)	49000 MPa
	Area of a fabric by unit width (A_f)	157 mm ² /m
FRCM mortar *	Modulus of rupture (f_{rfm})	4.8 MPa
	Compressive strength (f'_{fm})	52 MPa

* from the data sheet provided by the manufacturer

According to CSA S850-12 (2012), material strengths should be increased appropriately due to the effect of high strain rates. Hence, a Dynamic Increase Factor (DIF) of 1.2 with the flexural failure mode of masonry is applied to both compressive and tensile strengths under far field blast, which is what was simulated by the shock tube in this experimental research. For the FRCM composite, however, there is no reliable DIF values due to lack of research. DIF for FRCM system should be studied further to confirm its strength change under high strain rate loading. In this study, a DIF of 1.2 is assumed for the FRCM system because of its similarity with mortar and concrete.

The ultimate capacity, and the capacity at cracking of wall R-S-L, obtained using the above procedure, are summarized in Table 6-2. The analysis demonstrated that the tensile strain in the FRCM fabric reached the ultimate rupturing strain prior to the crushing of masonry in compression. Therefore, the failure mode is governed by the rupturing of FRCM fabric.

Table 6-2. Analytical flexural strength of R-S-L

at cracking				ultimate capacity			
M_{cr} (kN-m)	Δ_{cr} (mm)	w_{cr} (kN/m)	p_{cr} (kPa)	M_u (kN-m)	Δ_u (mm)	w_u (kN/m)	p_u (kPa)
28.9	1.5	57.8	28.9	50.7	15.1	101.3	50.7

6.3. RESISTANCE CURVES OF R-S-L

The internal restoring force-displacement relationship of a masonry wall under blast loading can be experimentally derived, with the assumption that the behaviour of the wall can be simplified as a single degree of freedom (SDOF) system. Equation [6-16] shows the generalized equation of motion of a SDOF system:

$$m\ddot{u}(t) + c\dot{u}(t) + ku(t) = F(t) \quad [6-16]$$

where, m is the mass; $\ddot{u}(t)$, $\dot{u}(t)$ and $u(t)$ are the acceleration, velocity and displacement of the wall at mid-height; c = damping coefficient; k = stiffness; and $F(t)$ = applied load as a function of time. Ignoring damping, Equation [6-17] is obtained:

$$K_{LM}m\ddot{u}(t) + R(t) = F(t) \quad [6-17]$$

where, $R(t)$ is the wall restoring force, m is the wall mass, $\ddot{u}(t)$ is the acceleration of the wall at the mid-height, and K_{LM} is load mass factor which converts the real system into an equivalent SDOF system. This factor depends on the wall shape function (UFC 3-340-02, 2008) and can be taken as 0.78 for a wall subjected to uniformly distributed load in one-way bending. $F(t)$ is the forcing function which describes loading over time, which can be calculated as $P_r(t) \times A$ (reflected pressure measured in the experiments as a function of time $\times$ the loaded area of the wall).

The acceleration $\ddot{u}(t)$ can be obtained numerically by differentiating the displacement function measured by the LVDTs/camera during the tests twice. In order to smooth out the measured displacement, an eighth-degree polynomial was used to fit the displacement data. Since the mass (m) and acceleration ($\ddot{u}(t)$) are known quantities, and the forcing function is measured for each test, the restoring force, $R(t)$ can then be found by rearranging Equation [6.17], i.e. $R(t) = F(t) - K_{LM}m\ddot{u}(t)$. This force was then plotted against the wall mid-height displacements to determine the restoring force-displacement relationships for the wall, for each blast test.

Experimentally-derived restoring force function of R-S-L

The restoring force functions for wall R-S-L were obtained using the procedure explained in Section 6.3. The first step was to describe the displacement function using a polynomial equation over the first positive cycle of response. Figure 6-2 shows one example of this step for Blast ID #11, which was the shot just before the failure of R-S-L. The next step was to compute the velocity and acceleration of the wall by differentiating the polynomial equation obtained from the first step. Figure 6-3 shows an example of this process for the same shot. Then, the inertia force can be computed as a function of time by multiplying the wall mass and the load mass factor with the acceleration values at each point of time, as depicted in Equation [6-17]. The applied load and the restoring force are shown in Figure 6-4 for the same shot.

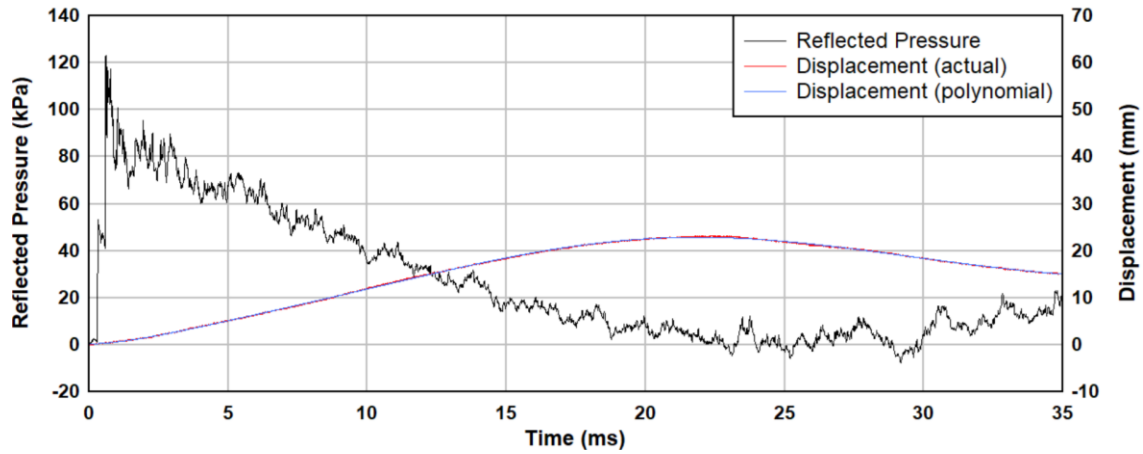


Figure 6-2. A polynomial matching to the actual displacement time history

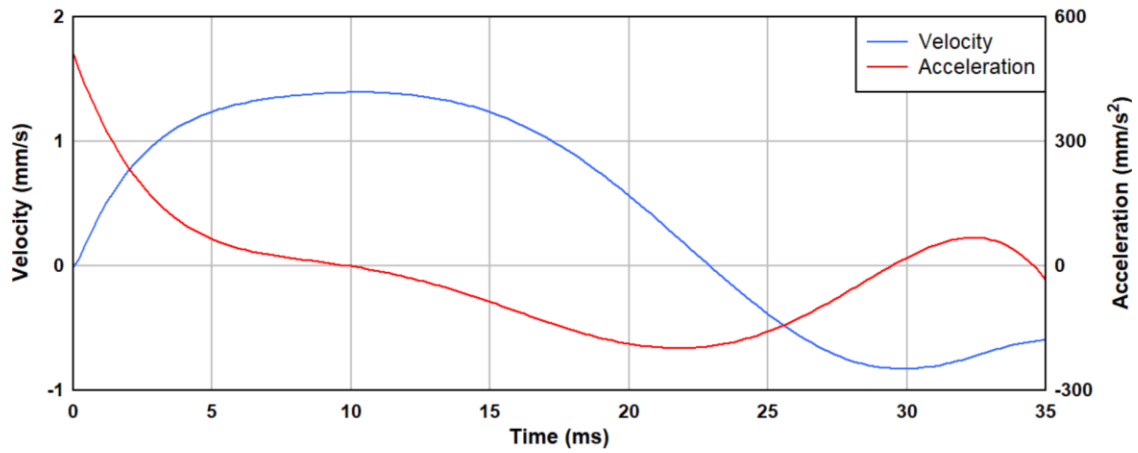


Figure 6-3. Velocity and acceleration curves

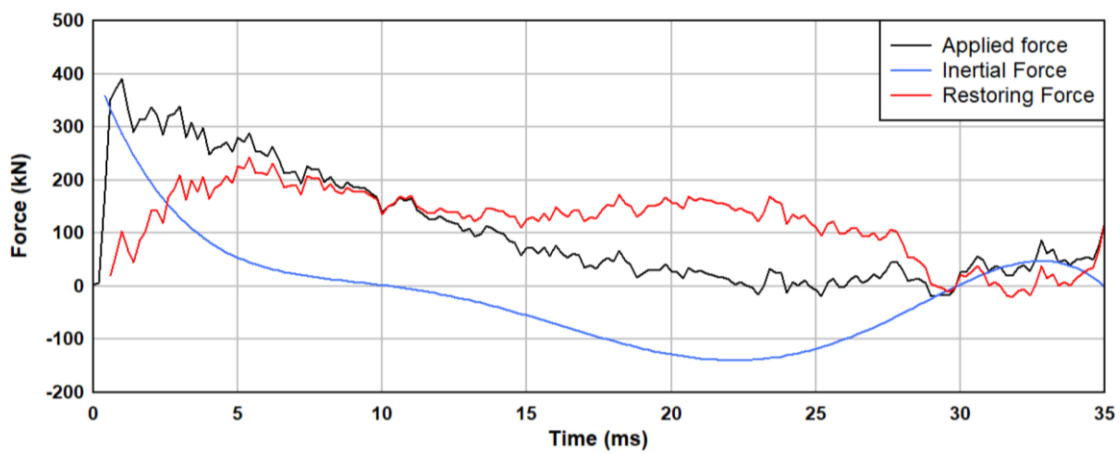


Figure 6-4. Inertial, applied, and restoring forces

This restoring force time history can be replotted to get a relationship between the restoring force and mid-height deflection at the same time points as shown in Figure 6-5.

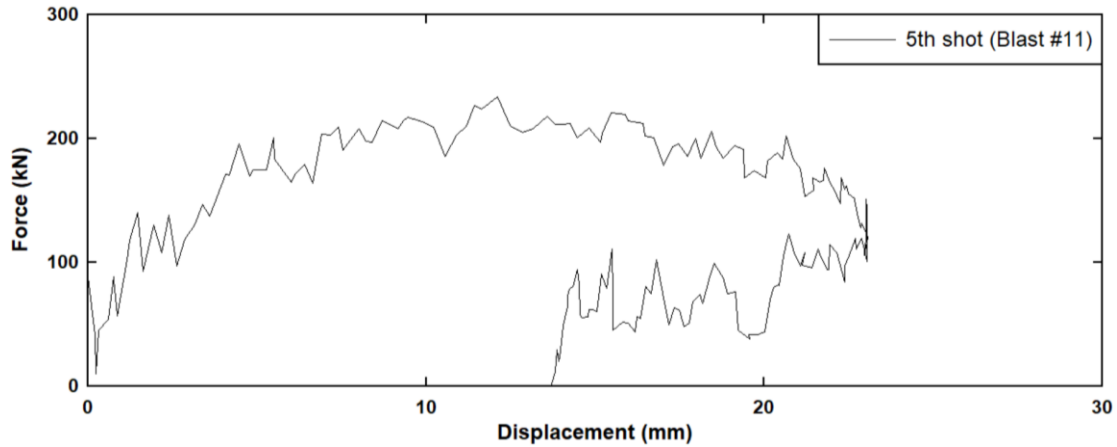


Figure 6-5. Resistance curve

The resistance curves for the other test shots of R-S-L were obtained in the same manner. They are shown in Figure 6-6. It should be noted that the resistance curve of the 5th shot, which was just prior to failure, does not fully represent the actual ultimate wall capacity due to the accumulated damage during testing.

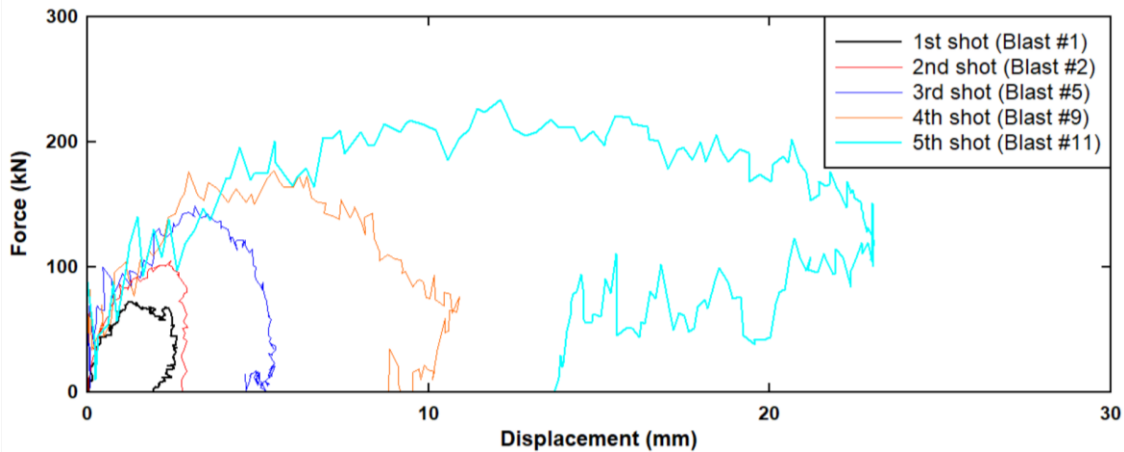


Figure 6-6. Resistance curves of R-S-L over testing

Idealized resistance curve of R-S-L

The idealized resistance curve of a stone wall retrofitted with FRCM is assumed to have a tri-linear load deflection relationship with 3 linear stages of behaviour: i) elastic behaviour up to the cracking moment, ii) post-cracking response up to the ultimate resistance, and iii) strength decay region. The cracking and ultimate capacities of R-S-L have been computed previously in Section 6.2, and the resistance force (F) can be obtained with Equation [6-18].

$$F = wL \quad [6-18]$$

where, w is the uniformly distributed load computed with Equation [6-14], and L is the length of the wall. The slope of the elastic region (first segment) can be obtained using Equation [6-19], and represents the uncracked stiffness of the wall:

$$k_1 = \frac{384E_m I_o}{5H^3} \quad [6-19]$$

In the same manner, the slope of the post cracking region (the second segment) can be obtained using Equation [6-20]:

$$k_2 = \frac{384E_m I_{cr}}{5H^3} \quad [6-20]$$

Lastly, the slope of the strength decay region (the third segment) should be defined, however in this study the idealized curve was assumed to have a plateau after the ultimate point up to the maximum displacement of 23 mm for the purpose of dynamic analysis (dashed line in Figure 6-7). The various points which define the resistance curve are summarized in Table 6-3, and the idealized resistance function of wall R-S-L is depicted as Figure 6-7.

Table 6-3. Computed values for each segment of the idealized resistance curve

1 st segment			2 nd segment			3 rd segment*
F_{cr} (kN)	Δ_{cr} (mm)	k_1 (kN/mm)	F_u	Δ_u	k_2	k_3
116	1.54	75	203	15.1	6.4	-

* Assumed to be a plateau up to $\Delta = 23$ mm for dynamic analysis

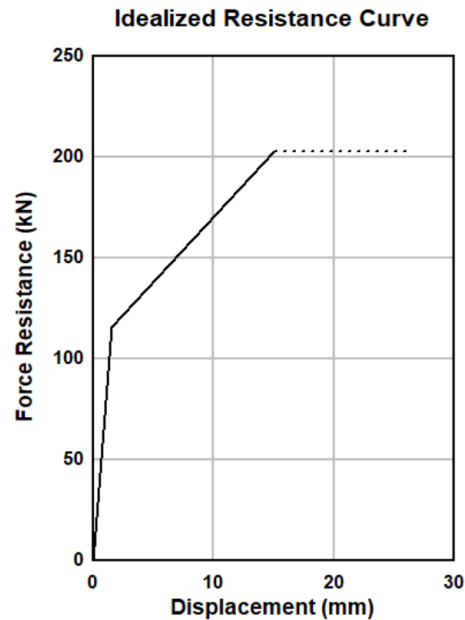


Figure 6-7. Idealized resistance curve of R-S-L

The idealized resistance curve is compared to the experimentally-derived resistance (restoring) functions in Figure 6-8. It can be observed that the analytical and experimentally-obtained curves have similar initial stiffness for all the shots. This was expected at the first shot since no flexural cracking was visible on the wall. Interestingly, the remaining restoring functions still showed similar initial slopes up to the fifth shot. The cracked stiffness, which is the second segment of response, can be observed from the fourth shot, and the maximum load continues to increase up to the 5th shot with a trend which closely matches the second branch in the idealized resistance curve. It should be noted that the fifth shot was not the final blast applied on the wall. The response at Shot 5 also includes the effects of accumulated damage. However, it can be observed that the idealized curve matches the behaviour observed in the experimental restoring functions with reasonable accuracy.

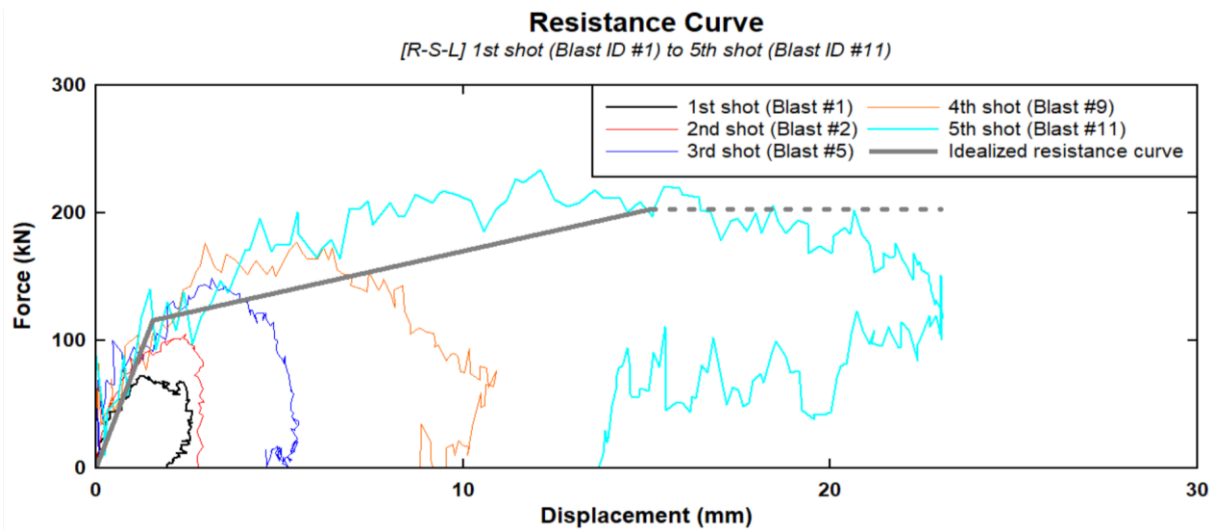


Figure 6-8. Actual and idealized resistance curves of R-S-L

6.4. SDOF ANALYSIS

SDOF analysis was performed on wall R-S-L by solving the SDOF equation of motion shown below:

$$K_{LM}m\ddot{u}(t) + R_i(t) = A \cdot P_{ri}(t) \quad [6-21]$$

The analysis was conducted using the idealized resistance function $R_i(t)$ which was derived using the procedure presented in the previous section. In the analysis, the mass of the wall m was taken as being equal to 1310 kg, which accounts for the weight of the masonry wall and FRCM retrofit overlay. The forcing function was defined using an idealized triangular pressure time history $P_{ri}(t)$ having the same peak reflected pressure (P_r) and impulse (I_r) observed in the experiments, but with an idealized time $t_d = 2I_r/P_r$, while the loaded area (A) was taken as being equal to the surface area of the wall (4 m²). The load mass factor (K_{LM}) was assumed to be 0.78 for the uncracked and 0.66 for the post-cracking response regions, respectively (UFC 3-340-02, 2008).

With these inputs, the equation of motion was solved in software RCblast (Jacques, 2014) to predict the mid-height displacements of wall R-S-L. This software solves the equation of motion using the average acceleration method. Figure 6-9 shows a sample of the inputs and graphical interface used in the analysis for wall R-S-L during the first shot, along with a graphical representation of the computed SDOF displacement curve.

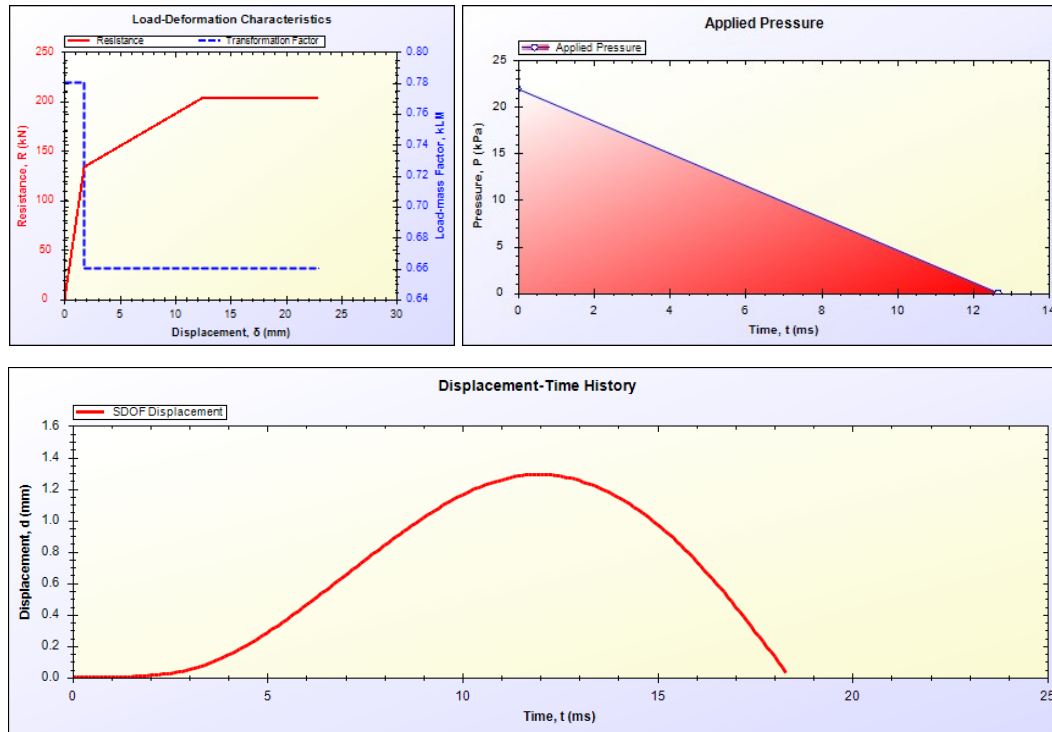


Figure 6-9. Example of the first shot of R-S-L using the software RCblast

Table 6-4 compares the results for the experimental (δ_{exp}) and analytical (δ_{anls}) maximum displacements for wall R-S-L, along with the prediction errors. The results are also presented graphically in Figure 6-10. Figure 6-11 shows the displacement-time histories from the SDOF analysis and the experiments for each shot up to maximum response.

The percentage errors between the analytical and experimental displacement results varied between 6.6 % to 44.9 % with a mean value of 19 %. The standard deviation was 14.6 %. Except for the fifth shot, the analytical maximum mid-height displacements were consistently lower than those recorded during the tests (i.e. analytical resistance was over estimated). This can partly be explained by the effects of accumulated damage from repeated testing which were not considered in the analysis. Another possible source of error includes the assumption regarding the DIF factor for the FRCM which was taken as 1.2. The use of $DIF = 1$ would result in lower resistance and larger analytical displacements (Note: use of $DIF = 1$ is commonly used for FRP). Other sources of error include assumptions regarding the material properties (Masonry and FRCM), the

assumption of idealized boundary conditions, idealized pressure time histories and the choice of load mass factors.

As a final note: the error during the fifth shot was 11 %, however, it should be noted that the experimental deflection exceeded the ultimate displacement in the idealized resistance curve (the result was only obtained by assuming a plateau in the resistance curve beyond Δ_u , which may not be appropriate). It is recommended that further experimental research be directed to studying the resistance (load-deflection) response of FRCM retrofitted masonry walls under out-of-plane loading (i.e. by conducting simple beam tests or by using air-bag testing on larger scale walls).

Table 6-4. SDOF analytical results of R-S-L

Wall	Blast ID	Blast properties			Maximum mid-height displacements			
					Test	Analysis	Error (%)*	$\delta_{anls}/\delta_{exp}$
		P_r (kPa)	I_r (kPa-ms)	t_d (ms)	δ_{exp} (mm)	δ_{anls} (mm)		
R-S-L	#1	21.9	138.8	12.7	2.6	1.5	44.9 (-)	0.55
	#2	36.3	203.7	11.2	3.0	2.3	20.0 (-)	0.80
	#5	53.4	347.3	13.0	5.5	5.1	6.6 (-)	0.93
	#9	76.4	516.6	13.5	12.0	10.5	12.4 (-)	0.88
	#11	123.1	856.2	13.9	23.2	25.6	11.0 (+)	1.11
	#15	114.3	2000.7	35.0	Failed	Failed	n/a	n/a
Statistical data of SDOF Analysis over experiments ($\delta_{anls}/\delta_{exp}$)					Mean Error (%)		19.0	0.85
					Standard Deviation (%)		14.6	0.15
					Maximum Error (%)		44.9	0.55
					Minimum Error (%)		6.6	0.93

* Error (%) = $(\delta_{anls} - \delta_{exp})/\delta_{exp} \times 100$ (%)

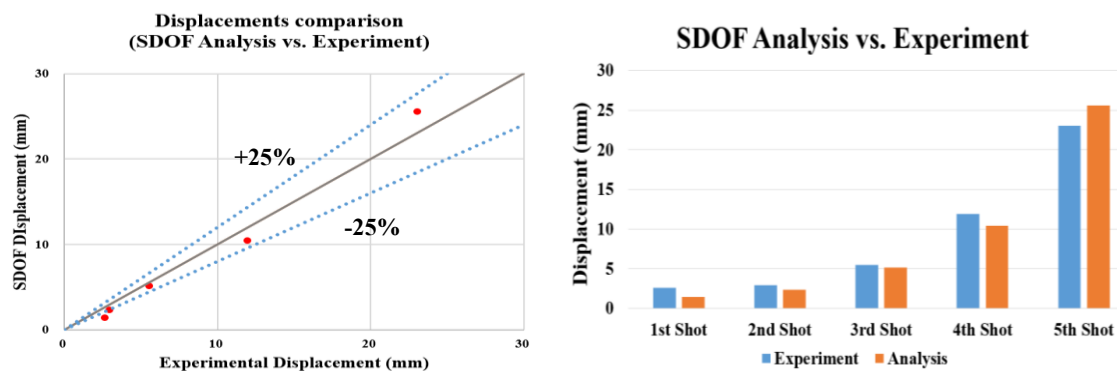


Figure 6-10. Displacement comparison (SDOF Analysis vs. Experiment)

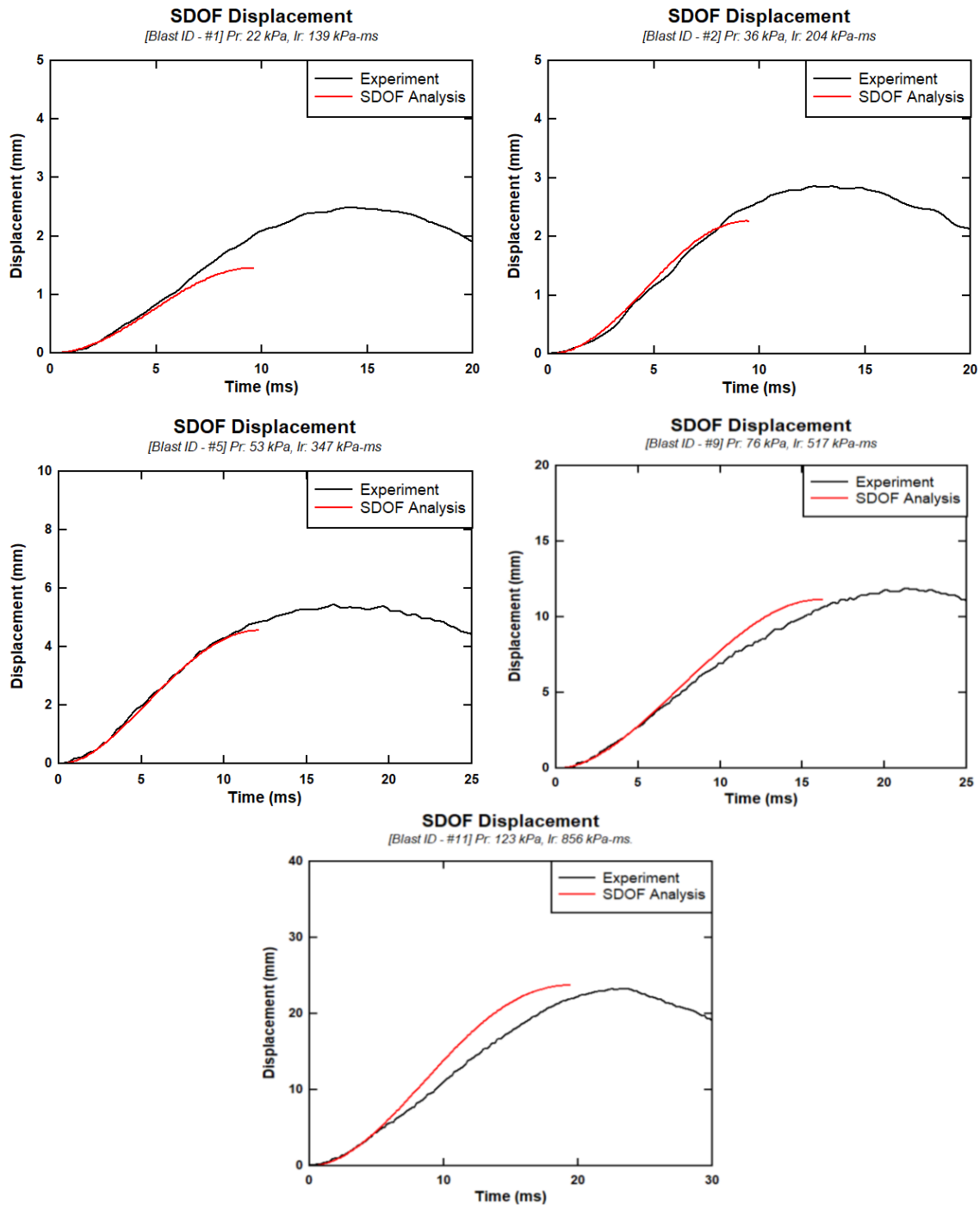


Figure 6-11. Displacement Time History of R-S-L (SDOF Analysis vs. Experiment)

6.5. SENSITIVITY ANALYSIS

A sensitivity analysis is conducted to verify the influence of the modelling assumptions on the SDOF predictions. The following aspects were considered in the sensitivity analysis:

- Cracking moment of the wall (M_{cr}) with or without considering the FRCM composite system;
- Modulus of rupture of the FRCM composite system (f_{rfm}) which affects the cracking moment of the wall (M_{cr});
- Dynamic increase factor (DIF) for the FRCM composite system.

The following sections examine the influence of these aspects on the SDOF results in order to obtain a better match with the experimental data.

6.5.1. Cracking moment of the wall with/without considering the FRCM composite system

ACI 549.4R (2013) indicates that the contribution of FRCM system prior to cracking can be neglected in the flexural design of FRCM retrofitted masonry walls. This is a conservative design assumption. Therefore, the combinations listed in Table 6.5 were examined to confirm if the FRCM composite contributes to the cracking strength of the wall in this study. ACI 549.4R (2013) suggests to calculate the cracking moment with the equation of $M_{cr} = (f_r + f_{cs})S$, where f_r is modulus of rupture, f_{cs} is axial compressive stress due to an axial load, and S is sectional modulus of the uncracked wall. In this equation, the modulus of rupture (f_r) can be considered as either that of just the masonry (i.e. CASE-2) or that of the masonry plus FRCM (i.e. CASE-1). The equations for each case are shown in Table 6.5 with illustration of the section. Other variables are fixed to $f_{rfm} = 4.8$ MPa and DIF = 1.2 as assumed previously in Section 6.2.

Table 6.5. Sensitivity analysis - Cracking moment w/ or w/o FRCM

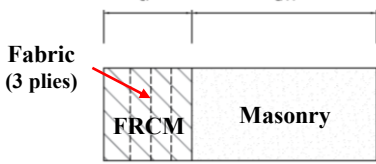
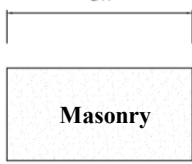
Sensitivity analysis	CASE-1	CASE-2
Assumed section for cracking moment (M_{cr})	w/ FRCM 	w/o FRCM 
$M_{cr} = (f_r + f_{cs})S$	$f_r = \frac{f_{rm}A_n + f_{rfm}A_{fm}}{A_n + A_{fm}}$	$f_r = f_{rm}$

Table 6.6 shows the cracking and ultimate moment capacities calculated based on CASE-1 and CASE-2 by following the procedure described in Section 6.2. For CASE-1, the cracking strength is higher due to the contribution of the FRCM system.

The idealized resistance curves for both cases are compared to the actual resistance curves at each shot in Figure 6-12. The idealized resistance curve from CASE-1 clearly shows a better match to the actual resistance curves. Using the CASE-1 and CASE-2 idealized resistance curves, the maximum mid-height displacements under blast loading were predicted the software RC Blast, and the results are listed in Table 6.7. Figure 6-13 compares those predicted displacements (δ_{anls}) from CASE-1 and CASE-2 to the experimental displacement (δ_{test}) results. Figure 6-14 compares the ratio of the analytical and the experimental displacements for these two cases. The average $\delta_{anls}/\delta_{test}$ ratio from CASE-1 is close to 1.0, whereas that of CASE-2 is over 2.0. Therefore, it can be concluded that accuracy of the predictions improves significantly when the contribution of the FRCM system is taken into account. Hence, for the purpose of blast analysis, the FRCM system should not be ignored when calculating the cracking strength of the wall.

Table 6.6. Capacities at cracking and ultimate; CASE-1 vs. CASE-2

Sensitivity analysis	at cracking				at ultimate			
	M_{cr} (kN-m)	Δ_{cr} (mm)	w_{cr} (kN/m)	p_{cr} (kPa)	M_u (kN-m)	Δ_u (mm)	w_u (kN/m)	p_u (kPa)
CASE-1	28.9	1.5	57.8	28.9	50.7	15.1	101.3	50.7
CASE-2	3.9	1.3	7.7	3.9	50.7	30.5	101.3	50.7

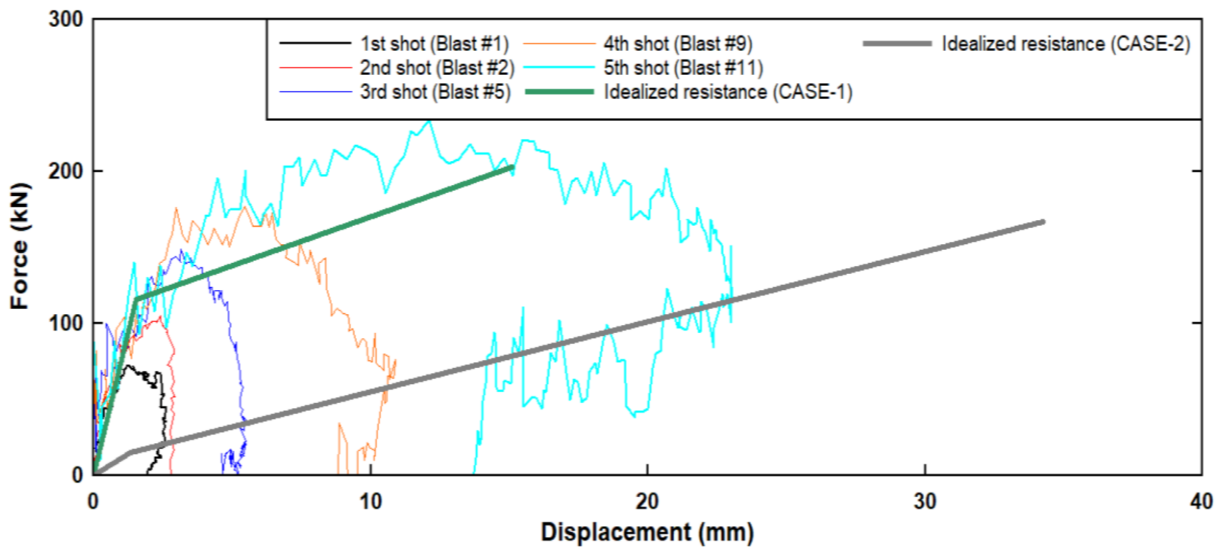


Figure 6-12. Idealized resistance curves: CASE-1 vs. CASE-2

Table 6.7. Maximum mid-height displacements; Analytical vs. Experimental

Shot #	Maximum mid-height displacement (mm)				
	δ_{test}	CASE-1		CASE-2	
		δ_{anls}	$\delta_{\text{anls}}/\delta_{\text{test}}$	δ_{anls}	$\delta_{\text{anls}}/\delta_{\text{test}}$
Shot 1	2.64	1.45	0.55	5.38	2.04
Shot 2	2.93	2.35	0.80	8.42	2.87
Shot 3	5.51	5.15	0.93	15.30	2.78
Shot 4	11.94	10.46	0.88	25.66	2.15
Shot 5	23.06	25.59	1.11	41.96	1.82
Mean		-	0.85	-	2.33
Std. Dev.		-	0.18	-	0.42
Max.		-	1.11	-	2.87
Min.		-	0.55	-	1.82

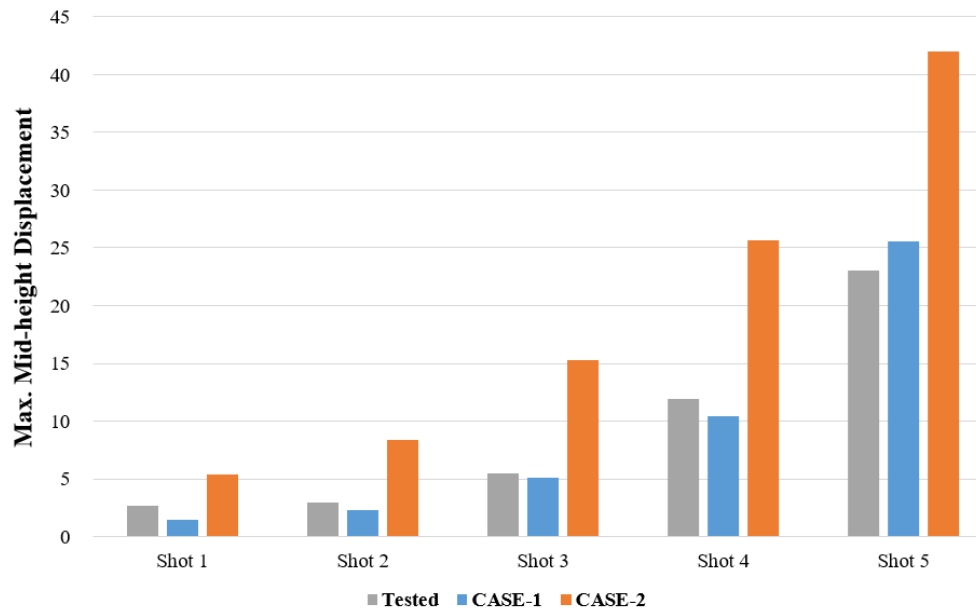


Figure 6-13. Analytical / Experimental Displacement: Tested vs. CASE-1 vs. CASE-2

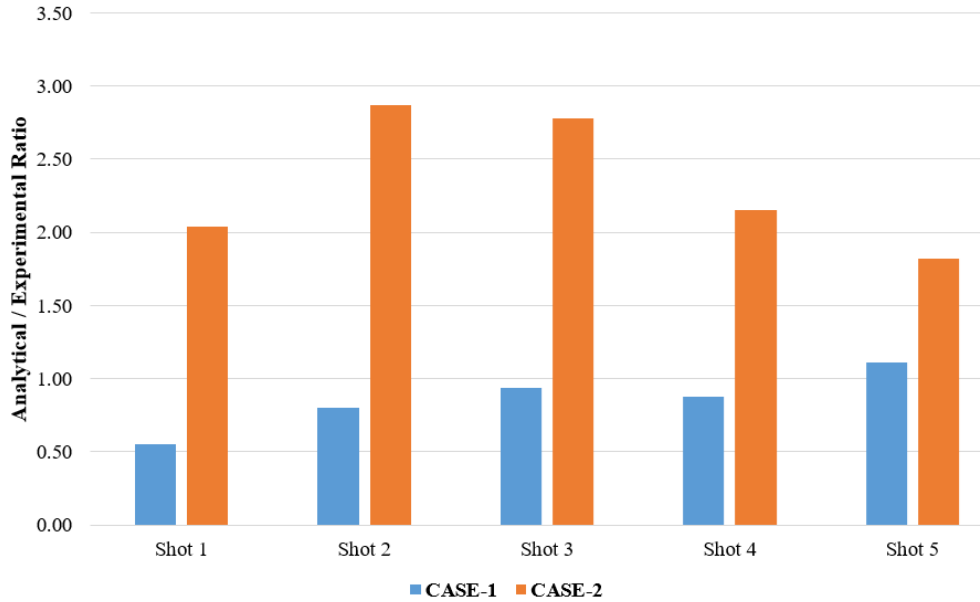


Figure 66-14. Analytical / Experimental Displacement Ratio: CASE-1 vs. CASE-2

6.5.2. Modulus of rupture and DIF of FRCM composite system

According to the manufacturer's data sheet the flexural strength (ASTM C348) and splitting tensile strength (ASTM C496) of the FRCM mortar are 6.9 MPa and 4.8 MPa, respectively. To examine the influence of this parameter either 6.9 MPa or 4.8 MPa are considered for f_{rfm} . Moreover, dynamic increase factors of either 1.0 or 1.2 are considered for the FRCM system. The former is selected based on previous research on the dynamic strain-rate sensitivity of FRP (Mutalib et al., 2011), while the latter is selected considering the effects of the mortar which is similar to concrete (CSA S850-12, 2012). Based on these variables, the six cases listed in Table 66.8 can be examined to find the appropriate combination of the modulus of rupture and DIF. DIF of '1.2/1.0' in the table means that a DIF = 1.2 is applied to FRCM system up to the point of first cracking (i.e. to f_{rfm} of the FRCM mortar in calculating the cracking strength), and a DIF = 1.0 is applied to FRCM system after the first crack (i.e. to the tensile strength of the carbon textiles in calculating the ultimate strength). The contribution of the FRCM system on cracking strength is applied to all cases reflecting the results of the sensitivity analysis presented in section 6.5.1.

Table 66.8. Sensitivity analysis - Modulus of rupture and DIF

Sensitivity analysis	CASE-1	CASE-2	CASE-3	CASE-4	CASE-5	CASE-6
Modulus of rupture of FRCM (f_{rfm})	4.8 MPa	4.8 MPa	4.8 MPa	6.9 MPa	6.9 MPa	6.9 MPa
DIF for FRCM system	1.2	1.0	1.2/1.0	1.2	1.0	1.2/1.0

Table 66.9 shows the cracking and ultimate moment capacities calculated based on the six cases described in Table 66.8. As f_{rfm} increases from 4.8 MPa to 6.9 MPa (CASE-1 vs. CASE-4, or

CASE-2 vs. CASE-5, or CASE-3 vs. CASE-6), the cracking strength and displacement increase, whereas the ultimate strengths and displacements remain the same, and decrease, respectively. The influence of the choice of DIF factor can be found from CASE-1 vs. CASE-2, or CASE-4 vs. CASE-5. As the DIF increases from 1.0 to 1.2, the cracking strength the ultimate strengths increase, whereas the cracking and ultimate displacements increase and decrease slightly, respectively. When the combined case DIF of 1.2/1.0 (before/after cracking) is used, the cracking moment and the ultimate displacement are also affected.

Table 66.9. Cracking and ultimate capacities; CASE-1 vs. CASE-2 vs. CASE-3 vs. CASE-4

Sensitivity analysis	at cracking				at ultimate			
	M_{cr} (kN-m)	Δ_{cr} (mm)	w_{cr} (kN/m)	p_{cr} (kPa)	M_u (kN-m)	Δ_u (mm)	w_u (kN/m)	p_u (kPa)
CASE-1	28.9	1.5	57.8	28.9	50.7	15.1	101.3	50.7
CASE-2	24.2	1.4	48.5	24.2	41.8	16.6	83.5	41.8
CASE-3	27.7	1.5	55.5	27.7	41.8	13.8	83.5	41.8
CASE-4	38.5	2.1	77.1	38.5	50.7	9.6	101.3	50.7
CASE-5	31.9	1.8	63.7	31.9	41.8	10.4	83.5	41.8
CASE-6	36.9	2.1	73.7	36.9	41.8	6.3	83.5	41.8

The idealized resistance curves for each case are compared to the actual resistance curves at each shot in Figure 6-15. Based on this comparison, it is found that the idealized resistance curve of either CASE-1 or CASE-4 have greater agreement with the actual resistance curves when compared to those of the remaining four cases.



Figure 6-15. Idealized resistance curves: CASE-1 to CASE-6

By using these idealized resistance curves, the maximum mid-height displacements from CASE-1 to CASE-6 are calculated using the dynamic analysis procedure in software RCblast, and the results at each shot are listed in Table 6.10. Considering the average ratios and statistical variation

for $\delta_{\text{anls}}/\delta_{\text{test}}$, the results from CASE-1 to CASE-3 (i.e. with f_{rfm} of 4.8 MPa) agree better with the experimental displacements when compared to the remaining cases (CASE-1: Mean 0.85/Std. Dev. 0.18, CASE-2: 1.01/0.27, CASE-3: 0.94/0.24, CASE-4: 0.74/0.13, CASE-5: 0.87/0.21, CASE-6: 0.81/0.19). The comparisons in Figure 6-16 and Figure 6-17 also confirm this trend.

It should be considered that; i) the displacement predictions for the first shot are relatively small, and ii) those for the last shot are based on the assumption of a load plateau (see Section 6.3). Therefore, the prediction of the maximum mid-height displacement using CASE-1 (f_{rfm} of 4.8 MPa, and DIF of 1.2) can be considered to agree reasonably well with the experimental data. Hence, for the purpose of this study, it is recommended that f_{rfm} of 4.8 MPa, and DIF of 1.2 be used in calculating the capacities of the wall at cracking and ultimate. However, further experimental studies are needed to better study the influence of these modeling parameters.

Table 6.10. Maximum mdi-height displacements; Analytical vs. Experimental

Shot #	Maximum mid-height displacement (mm)												
	δ_{test}	CASE-1		CASE-2		CASE-3		CASE-4		CASE-5		CASE-6	
		δ_{anls}	$\delta_{\text{anls}}/\delta_{\text{test}}$	δ_{anls}	$\delta_{\text{anls}}/\delta_{\text{test}}$	δ_{anls}	$\delta_{\text{anls}}/\delta_{\text{test}}$	δ_{anls}	$\delta_{\text{anls}}/\delta_{\text{test}}$	δ_{anls}	$\delta_{\text{anls}}/\delta_{\text{test}}$	δ_{anls}	$\delta_{\text{anls}}/\delta_{\text{test}}$
Shot 1	2.64	1.45	0.55	1.50	0.57	1.52	0.57	1.45	0.55	1.52	0.57	1.52	0.57
Shot 2	2.93	2.35	0.80	2.64	0.90	2.47	0.84	2.24	0.76	2.37	0.81	2.33	0.79
Shot 3	5.51	5.15	0.93	6.27	1.14	5.56	1.01	4.12	0.75	4.93	0.89	4.41	0.80
Shot 4	11.9	10.5	0.88	12.9	1.08	11.5	0.97	8.19	0.69	10.2	0.85	9.03	0.76
Shot 5	23.1	25.6	1.11	32.0	1.39	30.1	1.30	22.1	0.96	28.2	1.22	26.5	1.15
Mean	-	-	0.85	-	1.01	-	0.94	-	0.74	-	0.87	-	0.81
Std. Dev.	-	-	0.18	-	0.27	-	0.24	-	0.13	-	0.21	-	0.19
Max.	-	-	1.11	-	1.39	-	1.30	-	0.96	-	1.22	-	1.15
Min.	-	-	0.55	-	0.57	-	0.57	-	0.55	-	0.57	-	0.57

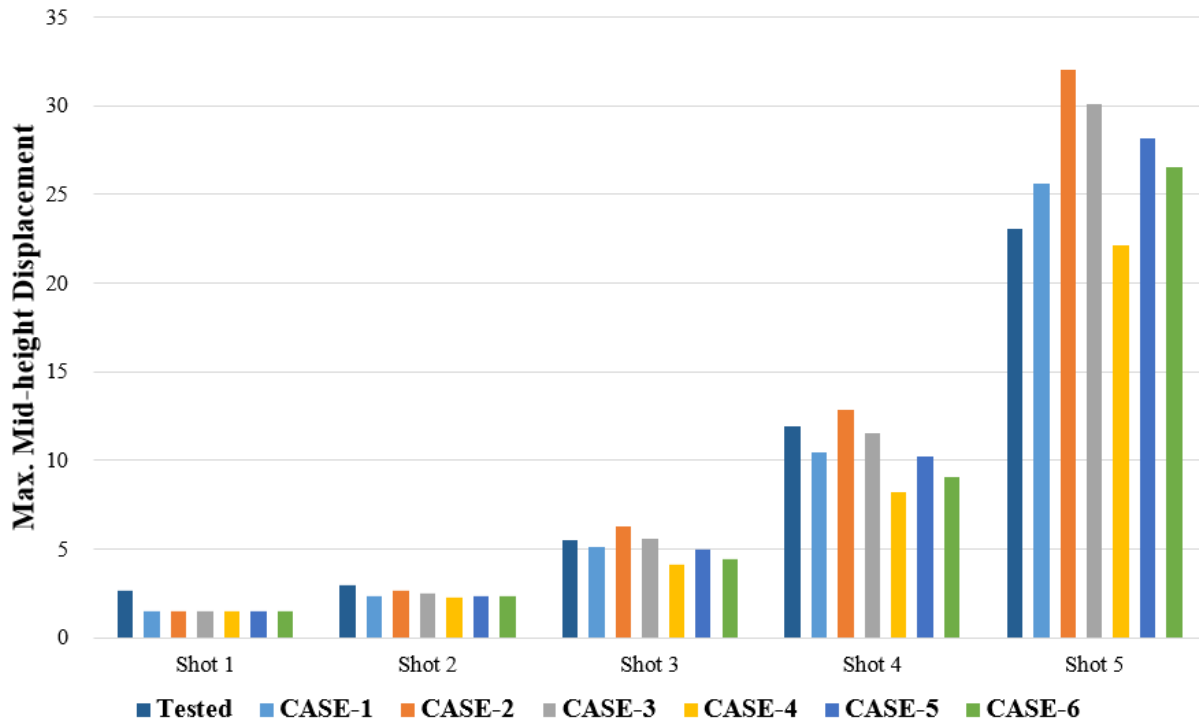


Figure 6-16. Analytical / Experimental Displacement: Tested vs. CASE-1 to CASE-6

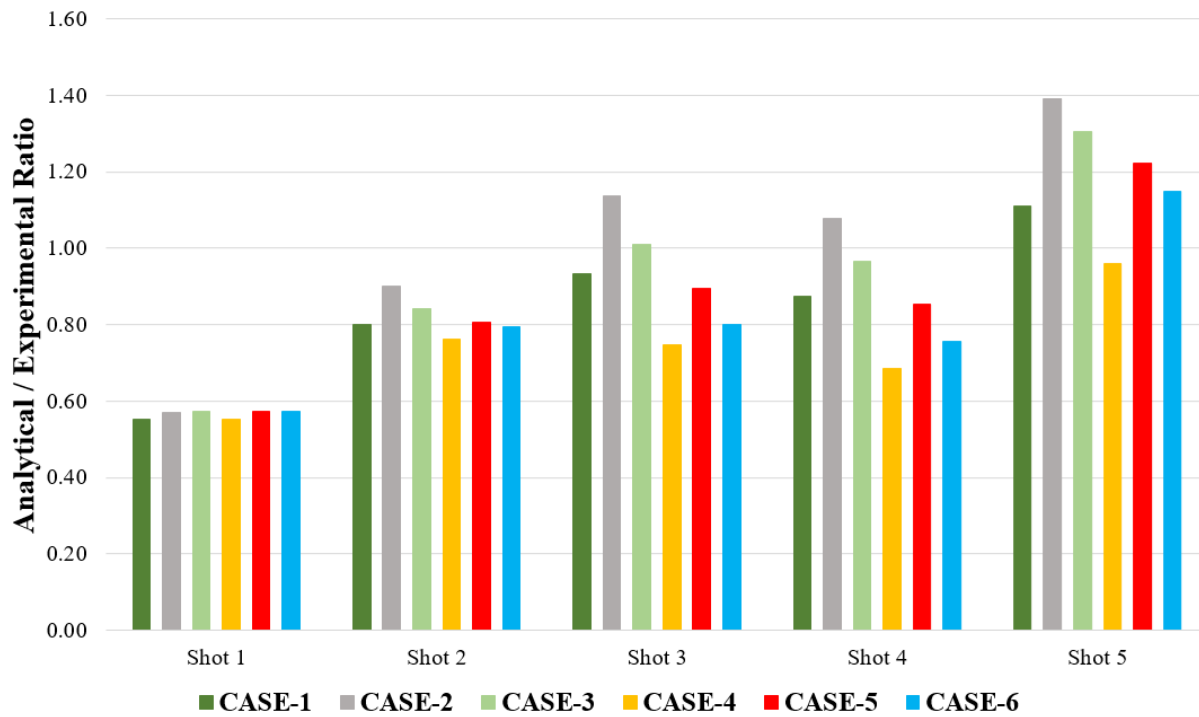


Figure 6-17. Analytical / Experimental Displacement Ratio: CASE-1 to CASE-6

CHAPTER 7. CONCLUSION

7.1. CONCLUSION

In this thesis, an experimental research program was conducted to investigate the effectiveness of using Fabric-Reinforced Cementitious Matrix (FRCM) composites to improve the blast performance of unreinforced masonry (URM) walls.

As part of the study, two masonry walls, one infill CMU wall and one load-bearing stone wall, were retrofitted with FRCM composites and tested under air blast loading using the University of Ottawa's Shock tube. The test results were compared to two control (un-retrofitted) walls with identical properties tested in previous research.

The following conclusions are drawn from this research study:

1. Overall, FRCM strengthening proved to be an effective means of increasing the strength and energy-absorption capacity of unreinforced masonry walls subjected to blast loading.
2. The FRCM retrofit system was effective in improving the blast performance of the CMU infill wall. Wall R-C-N showed an ability to resist greater blasts without collapse when compared to the control un-retrofitted CMU infill wall (URM-1). The wall showed an ability to dissipate significant blast energy with failure governed by crushing of the masonry blocks at the back of the wall. In comparison, the control wall (URM-1) failed in a brittle manner, with significant wall-debris, at relatively low blast pressures. However, the use of three plies of FRCM resulted in over-reinforced conditions and diagonal shear cracking during testing.
3. The FRCM retrofit system was also effective in improving the blast performance of the stone load-bearing wall. Wall R-S-L showed an ability to sustain greater blast impulse and improved control of displacements when compared to the control un-retrofitted stone load-bearing wall (URM-S-L). The retrofit also showed high damage tolerance, limited to scattered cracks across its height, until very late stages of testing. However, during the final blast, brittle failure occurred due to debonding of the FRCM from the stone units and rupture of the carbon fiber grids.
4. The performance of wall R-S-L was also compared to a companion wall retrofitted with fiber-reinforced shotcrete tested in previous research. Both retrofits showed comparable blast performance in terms of control of displacements and blast capacity.
5. Single Degree of Freedom (SDOF) analysis was performed to predict the blast behaviour of wall R-S-L. This was done by computing wall flexural strength using Plane Section Analysis, and developing an idealized resistance curve for use in the SDOF analysis. When comparing the analytical and experimental maximum displacements, the analysis produced errors which varied between 6.6 % to 44.9 %, with a mean value of 19 %. In general, the analytical resistance was over estimated resulting in lower analytical displacement predictions when compared to the experiments.

7.2. RECOMMENDATIONS FOR FUTURE RESEARCH

To achieve better understanding and wider application of FRCM retrofit systems in blast applications, the following studies are recommended for future research:

- The tensile behaviour of FRCM coupons with multiple layers of fabrics;
- The tensile behaviour of FRCM under dynamic load (i.e. at high-strain rates);
- Tests on FRCM-retrofitted masonry walls subjected to single blast loads;
- The effect of the number of FRCM plies, and different types of fabrics on failure mode in FRCM retrofitted masonry walls;
- Further blast research on the use of FRCM to retrofit load-bearing concrete masonry URM walls, as well as reinforced masonry (RM) walls;
- Experimental research to better understand the resistance (load-deflection) response of FRCM retrofitted masonry walls under out-of-plane loading (i.e. using beam tests or air-bag testing);
- Research on the performance of FRCM retrofitted masonry walls subjected to close-in (near field) blast loads;
- Finite Element Analysis of FRCM retrofitted masonry walls under blast loads.

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APPENDIX

The table below lists the reflected pressures measured by the bottom and side pressure sensors of the Shock Tube during the tests. The bottom sensor was used when presenting the experimental results in Chapter 3, 4 and 5, and when conducting the analysis in Chapter 6.

Recorded reflected pressures from the bottom and side pressure sensors during the tests

Wall ID	Shot	Blast ID	P _d (kPa)	L _d (mm)	Bottom sensor			Side sensor		
					P _r (kPa)	I _r (kPa-ms)	t _d (ms)	P _r (kPa)	I _r (kPa-ms)	t _d (ms)
R-C-N	1st	#1	82.7	1829	23.1	155.2	13.5	19.0	138.3	14.6
	2nd	#2	124.1	1829	32.6	225.5	13.8	26.5	198.1	15.0
	3rd	#3	165.5	1829	38.1	295.0	15.5	32.0	283.1	17.7
	4th	#9	413.7	1829	69.8	646.3	18.5	66.9	689.6	20.6
	5th	#10	482.6	1829	92.3	650.7	14.1	73.4	562.0	15.3
	6th	#11	689.5	1829	102.6	883.5	17.2	85.0	827.7	19.5
	7th	#12	551.6	3353	101.0	1580.2	31.3	83.2	1521.6	36.6
	8th	#13	689.5	3353	103.3	1715.3	33.2	93.2	1370.1	29.4
	9th	#14	586.1	3962	101.0	1862.6	36.9	85.5	1941.9	45.4
R-S-L	1st	#1	82.7	1829	21.9	138.8	12.7	19.8	133.2	13.5
	2nd	#2	124.1	1829	36.3	203.7	11.2	30.3	191.0	12.6
	3rd	#5	248.2	1829	53.4	347.3	13.0	51.3	340.4	13.3
	4th	#9	413.7	1829	76.4	516.6	13.5	76.0	599.7	15.8
	5th	#11	689.5	1829	123.1	856.2	13.9	102.5	903.6	17.6
	6th	#15	689.5	3962	114.3	2002.7	35.0	108.4	1146.5	21.2