Context-Sensitive Seismic Strengthening and Repair of Substandard Confined Masonry

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CONTEXT-SENSITIVE SEISMIC STRENGTHENING AND REPAIR OF SUBSTANDARD CONFINED MASONRY

by

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DEDICATION

This dissertation is dedicated to my parents for their love, endless support and encouragement.
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First and foremost, I would like express my sincere gratitude to my advisor, Dr. Fabio Matta, for his guidance, patience and support throughout this study. His support and guidance were invaluable and essential to my success.

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ABSTRACT

Confined masonry (CM) is a construction system consisting of load-bearing masonry panels that are confined with cast-in-place reinforced concrete tie columns and beams. Due to satisfactory seismic performance, CM has become the predominant low-rise residential construction system in several areas around the world. However, in developing regions, the use of substandard materials, details and construction practices may result in inadequate performance as highlighted in the 2010 earthquakes in Haiti and Chile. In the aftermath of an earthquake, households are often reluctant to reoccupy their dwellings due to concerns about safety of structures. If feasible, preventive (pre-hazard) strengthening or structural repair (post-hazard), to complement to temporary sheltering, are realistic options to respond to the pressing need for shelter on a large scale, since reconstruction poses greater barriers of cost and time. However, there is little knowledge on whether strengthening and repair can realistically improve the seismic behavior of a CM dwelling structure, especially using context-sensitive techniques with locally available (and often relatively low-quality) materials. Addressing this knowledge gap is important to inform pre- as well as post-hazard planning and decision making for hazard mitigation and disaster recovery.

The goal of this research is to contribute to filling this gap by investigating whether it is feasible to strengthen (pre-hazard) or repair (post-hazard)
substandard CM walls using context-sensitive materials and practices, and make
them safe, that is, offering a performance comparable to that of an undamaged
counterpart built with acceptable-quality materials and seismic details.
Supporting experimental evidence is based on in-plane cyclic tests on six full-
scale CM wall specimens (including control, strengthened and repaired
specimens) built with substandard materials (e.g., concrete with cylinder
compressive strength in the range of 9 - 14 MPa) and seismic details (e.g., open
stirrups with relatively large on-center spacing).

The in-plane load-displacement envelopes of the strengthened and
repaired specimens are compared with the theoretical envelope of a benchmark
CM wall built with acceptable-quality materials (e.g., concrete with cylinder
compressive strength of 26 MPa) and seismic details (e.g., closed stirrups with
suitable on-center spacing). It is shown that the strengthened and repaired
specimens exhibited comparable shear strength and ductility to those of
“standard” walls. It is also shown that the shear strength of all walls tested can be
conservatively predicted. Finally, the seismic performance of the wall specimens
is assessed in accordance with the Mexico City Building Code (MCBC)
Requirements for Design and Construction of Masonry Structures (NTCM 2004).
This code was selected as an authoritative reference since masonry construction
in Mexico is code-regulated since 1976 and the seismic provisions for masonry
structures were developed from results of a comprehensive research program of
over 20 years, and were updated after the 1985 Mexico City earthquake. It is
shown that the strengthened and repaired specimens do satisfy all criteria to
qualify as earthquake-resistant structures. It is concluded that it is feasible to strengthen or repair a substandard CM wall using context-sensitive materials and practices, such that both strength and ductility are comparable to or better than those of a CM wall built with acceptable-quality materials and details.

In addition, this study offers a novel contribution for large-scale structural testing by demonstrating a three-dimensional digital image correlation method for the non-contacting full-field measurement and visualization of deformations, offering comparable accuracy to that of traditional contact-based and point-wise sensors. As presented in chapter 2, these results enabled an in-depth description of the load-resistance mechanisms and damage evolution in CM wall specimens. The measurement setup and procedure demonstrated herein can be applied to large-scale specimens to obtain radically more detailed information compared to traditional measurement methods for large-scale laboratory testing of civil engineering structures.
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LIST OF SYMBOLS

\( A_r \) Cross sectional area of horizontal reinforcement.
\( A_p \) Cross sectional area of plaster equal to \( 2 \times t_p \times l^3 \).
\( A_T \) Wall transverse cross sectional area.
\( B \) Width of specimen.
\( d \) Drift (displacement at beam middle section).
\( d_{cr} \) Drift at cracking limit state.
\( d_p \) Drift at peak load limit state.
\( d_u \) Drift at ultimate limit state.
\( D \) Diameter of specimen.
\( E_p \) Modulus of elasticity of mortar plaster equal to 11,500 MPa.
\( f_m \) Compressive strength of masonry.
\( f_p \) Compressive strength of plaster.
\( f_r \) Stress in horizontal reinforcement.
\( f_{yr} \) Yield strength of horizontal reinforcement.
\( f_{yvc} \) Yield strength of column longitudinal reinforcement.
\( f_c' \) Compressive strength of concrete.
\( G_p \) Shear modulus of plaster equal to \( 0.4E_p \).
\( h \) Height of CM walls equal to 2,490 mm.
\( H \) Shear strength.
\( H_1 \) Height of specimen.
$H_{cr}$ Shear strength at cracking limit state.

$H_p$ Shear strength at peak load limit state.

$H_u$ Shear strength at ultimate limit state.

$I_p$ Moment of inertia of plaster’s cross section equal to $2 \times t_p \times l^3/12$.

$K$ Secant stiffness.

$K_e$ Effective stiffness.

$l$ Length of CM walls equal to 2,430 mm.

$L$ Length of specimen.

$L_1$ Span length.

$P$ Maximum applied load.

$P_s$ Weight of specimen.

$R$ Estimated shear strength.

$R_1$ Modulus of rupture.

$R_2$ Shear force reached at $\theta = 0.004$.

$R_{max}$ Maximum shear strength reached at $\theta \leq 0.004$.

$R_{1\ max}$ Peak shear strength.

$S$ Section modulus.

$tp$ Thickness of plaster.

$T$ Splitting tensile strength.

$T_1$ Thickness of specimen.

$V_p$ Shear strength of plaster.

$V_r$ Shear strength of horizontal reinforcement.

$V_{tot}$ Estimated shear strength of strengthened and repaired specimens.
$V_{CM}$ Peak shear strength of CM wall estimated by analytical model, equal to 124±30 kN.

$V_{CMA}$ Average peak shear strength of CM wall estimated by analytical model, equal to 124 kN.

$\kappa$ Shear coefficient for rectangular cross section equal to 1.2.

$\lambda$ Overstrength factor.

$\eta$ Efficiency factor for horizontal reinforcement.

$u_m$ Masonry shear strength from diagonal compression test.

$u_{cr}$ Cracking shear stress.

$u_p$ Peak load shear stress.

$u_u$ Ultimate shear stress.

$\delta_{cr}$ Drift ratio ($d/h$) at cracking limit state.

$\delta_p$ Drift ratio ($d/h$) at peak load limit state.

$\delta_u$ Drift ratio ($d/h$) at ultimate limit state.

$\rho_s$ Mortar plaster reinforcement ratio.

$\rho_{vc}$ Tie column longitudinal reinforcement ratio.

$\rho_{hr}$ Horizontal reinforcement ratio.

$\sigma_v$ Axial stress.

$\theta$ Drift ratio ($d/h$).

$\Delta$ Difference (in percent) between DIC and PWS measurements.
LIST OF ABBREVIATIONS

3D-DIC ............................................. Three-Dimensional Digital Image Correlation
CM .......................................................... Confined Masonry
COD .......................................................... Crack Opening Displacement
CV ............................................................ Coefficient of Variation
EEDR .......................................................... Equivalent Energy Dissipation Ratio
FE ............................................................. Finite Element
LED .......................................................... Light-Emitting Diode
LVDT .......................................................... Linear Variable Differential Transformer
OPC .......................................................... Ordinary Portland Cement
PWS .......................................................... Point-Wise Sensor
RC ............................................................ Reinforced Concrete
ROI ............................................................ Region of Interest
SA ............................................................ Substandard materials, Adequate details
SA-S ...................................................... Substandard materials, Adequate details, Strengthened
SA-P ...................................................... Substandard materials, Adequate details, Plastered
SS ............................................................ Substandard materials, Substandard details
SS-S ....................................................... Substandard materials, Substandard details, Strengthened
SS-P ....................................................... Substandard materials, Substandard details, Plastered
SD ............................................................. Standard Deviation
SWWM ........................................................ Steel Welded Wire Mesh
CHAPTER 1
INTRODUCTION

1.1 BACKGROUND

Confined masonry (CM) is a construction system consisting of unreinforced masonry panels built first, followed by cast-in-place small-section reinforced concrete (RC) tie columns, tie beams, and floor or roof slabs. CM was first used in the reconstruction of some cities destroyed by earthquakes, such as Messina in Italy, after 1908 (Meli and Alcocer 2004). Over the years, due to satisfactory seismic performance, CM has become the predominant low-rise residential construction system in several areas around the world ranging from Central and South America to Mediterranean Europe and Middle East (Meli and Alcocer 2004; Meli et al. 2011). However, in developing regions, inadequate quality of materials together with construction and detailing deficiencies may negatively affect the seismic performance of CM buildings, as it was observed in the 2003 Colima earthquake in Mexico (Alcocer and Klingner 2006), the 2010 Haiti earthquake (Eberhard et al. 2010), and the 2010 Chile earthquake (Brzev et al. 2010).

In the aftermath of the January 12, 2010 Haiti earthquake, residential structures exhibited a broad range of damage state, from minor damage to total or partial collapse (Lang and Marshall 2011). The number of people displaced and relocated in temporary camps was estimated up to 1.6 million (UNOPS
Households often abandoned their dwellings, even when minimal damage had occurred, due to a strong perception of the structures’ poor performance in the event of an aftershock or another earthquake (Beunza and Eresta 2011). As of July 2010, still 1.5 million people were still living in camps (Phillips 2011) in part due to a reluctance to return home because of different concerns, among which was the safety of structures (USAID 2011). If feasible, preventive (pre-hazard) strengthening or rapid structural repair (post-hazard), where the latter should complement to temporary sheltering (Beunza and Eresta 2011), are realistic options to respond to the pressing need for shelter on a large scale, since reconstruction poses greater barriers of cost and time. In fact, strengthening, and in particular repair as permanent housing solutions, need to be integrated into recovering strategies together with temporary shelter solutions. In fact:

- The widely supported strategy of delivering temporary shelters was unrealistic in timing and cost-wise, while other approaches such as repair became more relevant and cost-efficient. In fact, the average price of temporary shelter solutions became 77% of the average cost for house repair (Beunza and Eresta 2011).

- 26% of the houses required minor repairs to be safe (Beunza and Eresta 2011), while 10% may have required major repairs (Marshall et al. 2011); these data highlight a significant potential for partial housing stock recovery.
The will of a majority of affected people was to return to their original place and 48% of house owners needed help to repair their houses instead of temporary sheltering alone (Beunza and Eresta 2011).

While repair can be considered as a realistic and effective post-hazard option, strengthening becomes realistic as a cost-effective pre-hazard option provided that highly substandard structures can be transformed into structures with acceptable seismic performance.

However, there is little knowledge and experience on whether strengthening and repair can realistically improve the seismic behavior of a CM dwelling structure, especially using context-sensitive techniques that entail the use of locally available (and often relatively low-quality) materials (Beunza and Eresta 2011) and construction practices that are familiar to local practitioners. As a result, as of January 12, 2011, enough houses were repaired to relocate only 1,875 out of 300,000 households (USAID 2011).

1.2 OBJECTIVE

The overall goal of this research is to contribute to filling this gap by understanding whether it is feasible to retrofit (strengthen or repair) a substandard CM wall, using context-sensitive materials and practices, and make it safe, that is, offering a performance comparable to that of an undamaged counterpart built with acceptable-quality materials and seismic details.

While it is unrealistic to use more advanced retrofitting materials and methods [e.g., FRP bars embedded in a polymeric adhesive (Li et al. 2005; Tumialan et al. 2001)] that are often enlisted in “developed” areas, a retrofitting
practice suitable for developing regions requires using context-sensitive materials and techniques (Kijewski-Correa and Taflanidis 2012; Mix et al. 2011).

Context-sensitive materials are locally available, accessible and commonly used, albeit sometimes substandard. They may also be recycled from collapsed buildings for strengthening and repair purposes. For example, low-strength CMUs and concrete may be used for reconstruction (DesRoches et al. 2011), including repairs, together with steel rebars recycled from collapsed building. Context-sensitive retrofitting practices entail construction and installation operations that can be performed following practices familiar to local workers, without the need for additional training.

1.3 STATE OF THE ART

An efficient means to retrofit a structure including structural masonry walls is to incorporate reinforcement in the masonry walls bed joints or apply reinforced overlays on one or both faces of the walls. Bed joint reinforcement, typically in the form of stainless steel or fiber-reinforced polymer (FRP) bars, can be embedded in mortar along bed joints during the construction of masonry walls (Aguilar et al. 1996; Alcocer and Zepeda 1999) or inserted into grooves cut along mortar joints which are later filled with polymeric adhesives or traditional mortar [Structural Repointing (SR) technique] (Li et al. 2005; Loayza Seminario 2008; Tumialan et al. 2001). Reinforcement in reinforced overlays can be in the form of: FRP layers bonded to the wall surface (Koutromanos et al. 2012; Santa-Maria and Alcaíno 2011); reinforcing bars sprayed with mortar or concrete (Kahn 1984); multiple layers of mesh embedded in cement mortar (Amanat et al. 2007); or
steel welded wire mesh covered with cement mortar (Alcocer et al. 1996; Ashraf et al. 2012).

A CM structure may be considered substandard due to attributes related to: materials, such as quality and thus strength of masonry units, mortar and concrete; and detailing, such as masonry-RC frame interface and reinforcement layout and detailing in the RC frame. In developing regions (e.g., Haiti), the mixture design of concrete masonry units (CMUs), mortar and concrete regularly lacks sufficient amount of cement (to reduce cost) and may have a relatively high water/cement ratio (to improve workability) (EERI 2010). As a result, compressive strength values have been reported in the range of:

- 3 - 10 MPa, and 1.4 - 8 MPa for CMUs (on the net area) in Chile (Moroni et al. 2004) and Venezuela (Lafuente et al. 1998), respectively;
- 7 MPa, and 7.8 MPa for mortar in Venezuela (Lafuente et al. 1998) and Mexico (Tena-Colunga et al. 2009), respectively;

The longitudinal reinforcement of the RC frame typically consists of four Ø10 mm or Ø13 mm deformed or smooth rebars. The stirrups are usually made of Ø6 mm smooth rebars and spaced apart between 150 and 300 mm, with no decrease in the spacing at the column and beam ends, and bends not over 90 degrees, resulting in limited confinement for the longitudinal reinforcement and core concrete (Astroza et al. 2012; Lang and Marshall 2011; Marshall et al. 2011). The RC beam-column connections may also lack seismic details, for example due to absence of or excessive spacing between ties, and insufficient
rebar overlap (Astroza et al. 2012; Eberhardt et al. 2010). The lack of seismic details may also be due to the fact that seismic effects were not accounted for during construction (Mix et al. 2011). For example, toothing at the masonry-tie column interface, which is created by staggering of masonry units in a saw-tooth manner, enhances the interface strength between the masonry panel and the tie columns and offsets separation (Astroza et al. 2012). In Haiti, a flush (i.e., non-toothed) interface was frequently used as reported by Lang and Marshall (2011) and Mix et al. (2011). In addition, sometimes the masonry panels did not extend to the full story height, resulting in non-load-bearing masonry panels (Marshall et al. 2011).

1.4 METHODOLOGY

Feasibility of retrofitting is assessed based on two criteria, henceforth referred to as ‘feasibility criteria’, drawing from evidence gained through in-plane cyclic tests on representative full-scale CM walls:

F1. Obtaining comparable or better strength and deformability than those of a CM wall with acceptable-quality materials and details.

F2. Satisfying the earthquake-resistance criteria specified in the Mexico City Building Code (MCBC) Requirements for Design and Construction of Masonry Structures (NTCM 2004). In Mexico, 70 % of buildings include structural masonry walls and CM is the most popular masonry construction system (Alcocer et al. 2003). In addition, masonry construction in Mexico is code-regulated since 1976 and the seismic provisions for masonry structures were developed from results of a comprehensive research program of over 20 years...
and were updated after the 1985 Mexico City earthquake (Alcocer and Meli 1995).

Two types of CM wall specimens were designed. The first type, herein referred to as ‘SS’, is a representative example of a CM wall built with substandard (‘S’) materials (e.g., masonry with compressive strength of 5.8 MPa, and concrete with cylinder compressive strength in the range of 9 - 14 MPa) and seismic details (e.g., open stirrups with relatively large on-center spacing, and non-toothed masonry-tie column interface). The second type, herein referred to as ‘SA’, is representative of CM walls with substandard materials and adequate (‘A’) seismic details (e.g., closed stirrups with suitable on-center spacing, and toothed masonry-tie column interface), and was used to investigate the relative influence of substandard detailing vis-à-vis substandard materials. Two SS and two SA wall specimens were constructed. One SS specimen and one SA specimen were tested as control subjecting them to cyclic in-plane loading up to failure. The remaining SS and SA specimens, herein referred to as ‘SS-S’ and ‘SA-S’, respectively, were strengthened using aluminum strips that were embedded in mortar within selected bed joints and then load-tested. The failed SS and SA specimens were re-tested after repair by means of reinforced plaster made of low-strength mortar and inexpensive steel welded wire mesh, and are herein referred to as ‘SS-P’ and ‘SA-P’, respectively.

1.5 RESEARCH NOVELTY AND SIGNIFICANCE

The main novel contribution of this study is offered by:
• Using context-sensitive materials and practices for the seismic strengthening and repair of highly substandard CM walls, which are often encountered in developing regions.

• Demonstrating proof of concept that it is feasible to either strengthen or repair substandard CM walls using locally accessible materials and simple construction methods, transforming strength and ductility to the point where they are comparable to those of undamaged CM walls built with acceptable-quality materials and details.

The retrofitting techniques consider the economic and technological limitations found in developing regions, where highly-substandard buildings are common. More advanced retrofitting techniques require expensive materials (e.g., FRP bars embedded in a polymeric adhesive) and aim at ensuring optimal performance in terms of structural response and durability under the assumption of sound structural elements. The retrofitting techniques in this study enlist context-sensitive materials for highly substandard CM walls, as a strategy to improve performance and facilitate acceptance of the technology. To the best of the writer's knowledge, no systematic knowledge and technology base has been developed on the retrofitting of substandard CM structures that combine poor materials, design and construction detailing, which are often encountered in developing areas. The use of context-sensitive materials and methods is important and necessary for a retrofitting practice that has the potential to be accepted and enable local practitioners to independently retrofit their own structures and train future generations.
Another novel contribution of this study is offered by the demonstration of a three-dimensional digital image correlation (3D-DIC) method for the non-contacting full-field measurement and visualization of deformations on large-size concrete and masonry specimens in a laboratory environment. 3D-DIC uses a stereovision system that modifies subset-based DIC to measure full-field 3D motions of curved or planar surfaces (Sutton 2013; Sutton et al. 2009). This technique has been developed since the early 1990s (Helm et al. 1996; Luo et al. 1993) and is based on a comparative analysis of digital images of specimen surfaces at different load levels. 3D-DIC has been used in a limited number of studies to monitor displacement and detect cracks on large-scale civil engineering structures (Destrebecq et al. 2011; Smith et al. 2010; Tung et al. 2008; Yoneyama et al. 2007). The 3D-DIC measurements are aimed at visualizing full-field strain and crack maps, thus providing new information to describe the load-resisting mechanism and the progression of damage until failure, through a combination of size of the measurement area and spatial resolution that is unattainable with point-wise sensors. A correct implementation of this method enables measurements with comparable accuracy to traditional point-wise, contact-based sensors, providing new information to describe load-resisting mechanisms and the progression of damage until failure.

1.6 OUTLINE OF DISSERTATION

This dissertation reports on an investigation that aims at understanding whether it is feasible to retrofit a substandard CM wall, using context-sensitive materials and practices, and make it safe, comparable to a CM wall built with
acceptable-quality materials and seismic details. In Chapter 2, the feasibility of using 3D-DIC to accurately measure surface deformations on large masonry walls, and produce faithful strain and crack maps is evaluated. These results enable an in-depth description of the load-resistance mechanisms and damage evolution in the CM wall specimens discussed in chapters 3 and 4. Chapter 3 focuses on evaluating the ‘feasibility criteria’ for seismic strengthening (pre-hazard) of substandard CM walls using metallic strips embedded in mortar joints. The focus of Chapter 4 is on evaluating the ‘feasibility criteria’ for seismic repair (post-failure) of highly-damaged CM walls by means of reinforced plaster made with common steel welded wire mesh and mortar. Chapter 5 summarizes the conclusions drawn from the findings reported in Chapters 2 through 4, and offers recommendations for future research.

1.7 REFERENCES


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CHAPTER 2
FULL-FIELD DEFORMATION MEASUREMENT AND CRACK MAPPING ON
CONFINED MASONRY WALLS USING DIGITAL IMAGE CORRELATION

ABSTRACT. The understanding of the load-resistance mechanisms and failure
modes of large-scale concrete and masonry structures relies on accurate
measurements of surface motions and deformations, and faithful crack maps.
Measurements are typically taken using surface-mounted point-wise sensors
(PWSs), and crack maps are hand-drawn based on visual inspection. It is
impractical to obtain detailed displacement and deformation maps that describe
the complex response of large structures based on PWS measurements. In
addition, manual crack drawing is difficult, time-consuming, and prone to human
errors, which makes it challenging to consistently produce faithful crack maps.

This chapter reports on a pilot study to test the use of three-dimensional
digital image correlation (3D-DIC) as a non-contacting method to measure
surface deformation fields on full-scale masonry walls, and produce detailed
crack maps. Three confined masonry walls were tested under horizontal in-plane
reverse-cycle loads. The specimens were designed to attain different levels of
strength and deformability through different load-resistance mechanisms.
Representative 3D-DIC measurements of drift, diagonal deformations, and
interface slip between the reinforced concrete tie columns and the masonry infill
were evaluated vis-à-vis benchmark PWS measurements, showing a comparable
accuracy. Strain maps based on 3D-DIC measurements were enlisted to visualize the development of the fundamental strut-and-tie resisting mechanism in confined masonry walls subjected to horizontal in-plane loads, and illustrate practical structural analysis and design implications. More detailed crack maps compared with traditional hand-drawn maps were obtained based on 3D-DIC maximum principal strain contours.

2.1 INTRODUCTION

The measurement of surface displacements, deformations and crack opening displacements (CODs), and the mapping of surface cracks, are key to gain qualitative and quantitative information to understand the load-resistance mechanisms and failure modes of concrete and masonry structures. As the spatial resolution of these information increases, so does their value in informing the development of analysis and design algorithms as well as the verification and calibration of numerical models. In laboratory tests, displacements and deformations are traditionally measured using point-wise sensors (PWSs) such as linear potentiometers, crack opening gauges, and strain gauges (Shedid et al. 2009; Tena-Colunga et al. 2009; Voon and Ingham 2006). However, PWSs provide local surface measurements with a set resolution, which can also be affected by the integrity of the PWS connections to the surface and random cracks forming near these connections or along the PWS gauge length. In addition, crack maps are typically hand-drawn at different loading (or displacement) steps or after failure. This practice is time-consuming and poses safety concerns especially when testing large-scale structures, and is highly
dependent on the personnel's skills in recognizing the presence and length of cracks, including elusive but relevant hairline cracks. Therefore, it is impractical to rely on PWSs to obtain detailed displacement and deformation maps, while manual crack drawing makes it challenging to produce faithful crack maps.

Three-dimensional (3D) digital image correlation (DIC) is poised to overcome these limitations. 3D-DIC combines subset-based DIC with stereo-vision to measure essentially full-field 3D surface motions (Sutton 2013; Sutton et al. 2009). This measurement technique is based on a comparative analysis of digital images of patterned (i.e., having a random distribution of gray levels) surfaces. The images are acquired during the loading process using a calibrated stereo-vision system, which consists of a pair of rigidly-mounted digital cameras that are oriented to focus on the target surface. At each loading step, the 3D motion and deformation is extracted using a 3D-DIC software by selecting reference subsets in the undeformed state in one camera, extracting the matching image positions by comparing the reference subsets to those in the deformed image pairs, and performing triangulation between the matching subset centers in both cameras to locate the spatial position of the object point (Sutton et al. 2009).

Subset-based 3D-DIC is attractive to complement measurements taken from inside concrete and masonry specimens (e.g., strain gauges mounted on reinforcing bars) with accurate surface deformation and crack maps. The goal is to gain new information to describe the load-resistance mechanisms and the progression of damage until failure, through a combination of size of the
measurement area and spatial resolution that is unattainable with PWSs. This chapter presents a pilot study on the use of 3D-DIC as a non-contacting method to accurately measure surface deformations on large masonry walls loaded in their plane, and produce faithful strain and crack maps to better describe the load-resistance mechanisms and damage evolution. Three full-scale confined masonry (CM) wall specimens were designed for different performance in terms of in-plane strength and deformability, and tested under reverse-cycle loads. The 3D-DIC test setup deployed is assessed by comparing relevant full-field displacement measurements with benchmark measurements performed with traditional contact-based displacement transducers. Then, the use of strain maps is demonstrated to visualize load-resistance mechanisms and crack maps, and discuss practical implications for structural analysis and design.

2.2 CHALLENGES IN LARGE-SCALE TESTING

There is little experience in performing DIC measurements on large-scale concrete and masonry structures (Sutton 2013), which present specific challenges. The first challenge is the design and application of high-contrast speckle patterns that ensure a suitable balance between measurement accuracy and spatial resolution. To maximize accuracy, the subset size used in the correlation analysis must be tailored to provide a distinctive intensity pattern to distinguish one subset from the others (Lecompte et al. 2006). This is typically accomplished by using a speckle size between 2 and 5 pixels (Zhou and Goodson 2001), and tailoring the subset size such that each subset contains at least 3×3 speckles (Sutton et al. 2009). However, the use of spray paint or toner
powders may yield speckle patterns with a high-frequency content that cannot be captured with a standard-resolution (~2-6 megapixels) camera, resulting in aliasing problems. Conversely, the use of relatively large speckles, for example easily applied through manual painting, would result in reduced spatial resolution (Bornert et al. 2009). For the case of large masonry walls, suitable speckle patterns can be obtained on large (over 6 m²), smoothed and whitewashed surfaces by spraying dark paint through flexible polymer stencils (Ghorbani et al. 2014). In the writers’ experience, this approach is the most suitable as it consistently results in high-quality patterns, whereas the use of spackling brushes requires more practice and remains prone to inconsistent results, and direct painting of individual dots is viable but extremely time-consuming.

The second challenge is the setup of the stereo-vision system, with an emphasis on the selection of appropriate lenses and stereo angle (Sutton et al. 2009). To image a large surface, cameras with short focal length (wide-angle) need to be positioned at a relatively large distance from the specimen. Therefore, large-scale 3D-DIC tests call for sizable (e.g., 30 m² or more), unobstructed and uniformly-illuminated setup areas (Ghorbani et al. 2014) whose suitability must be verified experimentally. The use of wide-angle lenses also results in an increase in the variability in 3D positions measured by image matching for image points that are located off the camera axis. This shortcoming can be offset by increasing the stereo angle (and thus the spacing between the cameras) to a maximum of 60°, whereas larger angles pose issues related to image foreshortening and loss of contrast (Ke et al. 2011; Sutton 2013).
The third challenge is the effective calibration of the stereo-vision system, with an emphasis on lens distortion corrections. The process entails having both cameras observe a planar grid pattern placed on the target surface in different positions and orientations. Ideally, the grids should be sufficiently large to encompass a significant portion (e.g., one fourth) of the field of view, and sufficiently light to be manually moved so that cameras parameters, including distortion correction parameters, can be accurately determined. If smaller and easily movable grids are used on large surfaces, such as in this study, then assessing the measurement accuracy becomes especially important. In particular, more calibration images may be taken at the surface boundaries to quantify the effect of radial lens distortion, for which the associated measurement errors can be compensated for by using modern radial distortion models (Bräuer-Burchardt 2004; Pan et al. 2013; Yoneyama et al. 2006).

2.3 PREVIOUS WORK

DIC techniques are becoming main-stream in experimental mechanics research using relatively small concrete and masonry specimens. In these instances, full-field deformation measurements proved especially useful to gain insight into complex mechanisms such as the delamination of externally bonded fiber-reinforced composite laminates (Carloni and Subramaniam 2010; Fedele et al. 2014; Ghiassi et al. 2013). Conversely, very few case studies on large-scale concrete and masonry structures have been reported in the literature, in part due to the challenges introduced in the previous section. These studies included DIC measurements on relatively small regions of interest (ROIs), and presented some
quantitative comparisons between DIC and benchmark PWS measurements.

Lecompte et al. (2006) enlisted a 3D-DIC system to monitor the surface deformations of a small ROI on the maximum tensile stress area of a prestressed concrete beam subjected to vertical cyclic loads. The spatial resolution allowed to recognize the position and extent of surface cracks based on maximum principal strain maps. Küntz et al. (2006) used a 2D-DIC system to monitor a shear crack in a 245×190 mm ROI on a reinforced concrete (RC) beam of a bridge subjected to a diagnostic load test. The resulting displacement fields had a resolution of less than 10 μm, and offered comparable crack opening measurements to those of a benchmark linear potentiometer. However, the experiment highlighted the importance of applying a high-contrast speckle pattern (e.g., by painting or roughening the surface) as the insufficient contrast due to the lack of surface preparation allowed to analyzed only a 110×130 mm portion of the ROI. Destrebecq et al. (2011) used a 2D-DIC system to monitor the surface deformations of a 718×102 mm ROI including the maximum tensile stress area of a large reinforced concrete (RC) beam subjected to vertical cyclic loads. Similar midspan deflections were measured compared with a benchmark linear variable differential transformer (LVDT) sensor, and it was shown that the technique holds potential to determine the location and width of tensile cracks based on horizontal displacement measurements. Tung et al. (2008) used a 2D-DIC system with a 3,072×2,048 pixel camera to monitor the damage progression on the 400×300 mm surface of a 87 mm thick masonry wall with 45°-oriented mortar joints subjected to uniaxial compression, and on the 1,500×1,200 mm surface of
a steel-framed masonry wall subjected to in-plane cyclic loading. A dark speckle pattern was spray-painted on a light background. It was shown that using Von Mises strain maps allowed to effectively recognize cracks based on a comparison with the result of visual inspections, and a better definition of crack positions and sizes was attained with smaller subsets (16×16 pixels instead of 32×32 pixels). Smith et al. (2010) enlisted a 3D-DIC system to monitor the surface deformations of an approximately 1.3×1.0 m ROI at the base joint of a 0.4-scaled hybrid precast concrete wall subjected to in-plane reverse-cycle loading. It was shown that DIC axial strain maps accurately depicted the damage progression at the joint, and DIC displacement measurements were comparable with those of PWSs for gap opening and shear distortion at the wall base, respectively. More recently, Guerrero et al. (2014) demonstrated the use of DIC strain measurements to gain insight into the load-resistance mechanism (specifically, the angle of inclination of compressive struts) of full-scale masonry-infilled RC frames loaded horizontally in their plane. No assessment of the accuracy of DIC measurements was reported based on benchmark PWS measurements.

2.4 EXPERIMENTAL PROGRAM

2.4.1 Specimens and materials

The test matrix included three full-scale CM wall specimens (SS, SA and SA-S) whose geometry, reinforcement layout and details are illustrated in Figure 2.1 and Figure 2.2. Each wall had width, height, and thickness of 2,430, 2,490, and 203 mm, respectively, and consisted of a RC frame (including two tie
columns and a tie beam) that was cast on a 2,024×2,235×203 mm masonry infill, and connected to a RC footing having width, height, and thickness of 3,556, 381, and 406 mm, respectively [Figure 2.1(a)]. The masonry infills were constructed with hollow concrete masonry units (CMUs) having nominal dimensions of 203×203×406 mm with a 51 % net cross-sectional area, and Type N mortar consisting of three volume parts of sand, one part of Type N masonry cement, and water to ensure a slump of 150 mm. The CMUs were laid in a running bond pattern with bed and head joints having a thickness of approximately 10 mm. The RC columns and beams were constructed with four Ø13 mm Grade 60 deformed steel rebars as longitudinal reinforcement, Ø6 mm Grade 40 smooth steel stirrups as transverse reinforcement, and Type I ordinary Portland cement (OPC) concrete. The salient strength properties of masonry and concrete materials are summarized in Table 2.1 (see Appendix A).

Each specimen was designed to attain a different level of in-plane strength and deformability at failure through different load-resistance mechanisms.

Specimen SS is representative of a wall built with substandard (‘S’) details [Figure 2.1(b)], including: column-beam connections with longitudinal bars terminating with short 180° hooked ends, which make the joints prone to premature failure due to the lack of tensile reinforcement resisting joint opening due to combined bending and shear; open stirrups with relatively large (203 mm) on-center spacing at both the column-beam and column-footing connections, reducing the effectiveness of the tie columns to resist shear forces and undergo large deformations without collapsing; and a smooth interface between RC tie
columns and masonry infill, whose separation produced by in-plane loads may negatively affect the strength and deformability of the CM wall.

Specimen SA is representative of a wall built with adequate (‘A’) details [Figure 2.1(c)], including: column-beam connections tailored to resist the opening of the corner joint through longitudinal bars terminating with 90° bent ends having a length of 50 times the bar diameter to ensure an effective anchorage, and one well-anchored Ø13 mm diagonal steel bar; closed steel stirrups with a reduced (102 mm) on-center spacing at both the column-beam and column-footing connections; and a toothed masonry-RC interface to enhance the mechanical interlocking between masonry infill and tie columns.

Specimen SA-S consisted of a SA wall that was strengthened with aluminum strips embedded in the masonry bed joints to enhance in-plane strength and deformability, as illustrated in Figure 2.2. The horizontal reinforcement included ten 6061-T6 aluminum strips having a cross section of 3.2×12.7 mm and strength properties presented in Table 2.1. Each strip was inserted in a saw-cut groove along a bed joint [Figure 2.2(a)] and its 90° bent ends were anchored into pre-drilled slots in the RC tie columns [Figure 2.2(b)], alternating from one face to the other face of the wall [Figure 2.2(c)]. The reinforcement was then embedded in a conventional OPC mortar [Figure 2.2(d)].

2.4.2 Test setup and instrumentation

2.4.2.1 Loading apparatus and point-wise sensors

The load test setup and PWS layout used for each specimen is shown in Figure 2.3. The RC footing was tied to the structural floor using pre-tensioned...
steel threaded rods. The horizontal in-plane load was imparted using a hydraulic actuator with capacity of 500 kN and stroke of ±76 mm. The actuator had a swivel end bolted to a steel spreader beam that was rigidly connected to the top of the specimen RC tie beam by means of steel anchors. A constant vertical compression load of 88.3 kN was applied on top of the wall using a hydraulic jack and two steel spreader beams. The resulting uniformly distributed pressure of 0.2 MPa was intended to simulate the dead load of a second story.

The following PWSs were mounted on one face of each specimen (Figure 2.3): (a) one linear potentiometer with stroke of ±76 mm and accuracy of ±0.08 %, labeled ‘H1’, which was connected to the top of the RC tie beam at its midspan and to an exterior fixed support, to measure the maximum horizontal displacement (story drift); (b) two linear potentiometers with stroke of ±51 mm and accuracy of ±0.1 %, labeled ‘D1’ and ‘D2’, which were connected at the tie column-beam corners and at the base of the RC tie columns using 3 m aluminum extension rods, to measure diagonal deformations; and (c) two linear displacement transducers with stroke of ±25 mm and accuracy of ±0.35 %, labeled ‘S1’ and ‘S2’, which were connected to the tie columns at their midspan, to measure the differential displacement (slip) at the masonry-RC interface. Close-up photographs of the setup for sensors H1, D1 and S1 are shown in Figure 2.4.

2.4.2.2 Loading protocol

The in-plane reverse-cycle load, $H$, was imparted in displacement control mode following the sensor H1 displacement history in Figure 2.5(a). For each
displacement amplitude, three cycles were repeated at a frequency of 0.004 Hz. The third cycle included two 30-s constant-displacement plateaus that served to capture images (30 per plateau) for 3D-DIC measurements [Figure 2.5(b)]. This loading protocol aimed at accurately estimating the in-plane load-story drift ($H-d$) coordinates for three states that describe the mechanism of shear resistance of CM walls subjected to seismic loads, as illustrated in Figure 2.6 (Meli et al. 2011): (1) ‘cracking’ limit state, when diagonal cracking occurs in the masonry; (2) ‘peak load’ limit state, when the maximum load given by the combined shear strength of masonry infill and RC tie columns is attained, and the diagonal cracks propagate into the columns; and (3) ‘ultimate’ limit state, corresponding to about 20 % reduction in the peak load.

2.4.2.3 3D-DIC setup

The 3D-DIC setup presented in Figure 2.7 was devised to meet the challenges illustrated earlier for measurements on large surfaces. The images were acquired with two CCD digital cameras having a 2,448×2,048 pixel resolution (Grasshopper GRAS-50S5M-C, Point Grey) and equipped with lenses with F-number of 1.4 and focal length of 17.6 mm (Xenoplan 1.4/17, Schneider). The cameras were mounted on a rigid crossbar and spaced 1.1 m apart. The crossbar was secured to a tripod, and the stereo-vision system was positioned at 6.7 m from the wall surface [Figure 2.7(a)]. The relatively small stereo angle of 9.5° was considered acceptable as the main goal was to analyze in-plane rather than out-of-plane motions. Two banks of light-emitting diode (LED) lamps were used to illuminate the measurement surface. A desktop PC was used to store the
images and analyze them through a 3D-DIC software (Vic-3D, version 7, Correlated Solutions) [Figure 2.7(b)]. The stereo-vision system was calibrated by taking 60 pairs of images of a calibration grid with different positions and orientations. The grid included 12×9 dots with nominal diameter of 20 mm and on-center spacing of 50 mm. The calibration results indicated that no lens distortion corrections were necessary, thus supporting the selected setup with respect to field of view, depth of field, and stereo-vision system components. During each load test, for each 30-s constant-displacement plateau in Figure 2.5(b), 30 images were acquired to calculate average displacements and strains, thus minimizing measurement uncertainty.

2.4.2.4 Speckle pattern

The 2,430×2,490 mm surface of each CM wall was whitewashed [Figure 2.8(a)], smoothed with sandpaper, and cleaned with a blow gun to create a light background. A dark speckle pattern was then spray-painted using a flexible polymer stencil placed against the wall surface, as shown in Figure 2.8(b). The resulting speckle pattern is presented in Figure 2.8(c). The diameter of the speckles was approximately 3.2 mm, resulting in a speckle-to-surface area ratio of 33 %. Based on the field of view (3,330×2,790 mm) and camera resolution, each speckle was approximately 2.3 pixels in diameter. This approach was devised to address related challenges for large measurement surfaces by producing a high-quality pattern that offers good contrast and consistency throughout the region of interest and for different specimens, while being simple to apply. Figure 2.9 shows the histogram of the gray levels in the
speckle pattern for a representative portion of each specimen, indicating a bell-shaped distribution that is suitable for DIC analysis.

2.4.2.5 Subset size for DIC analysis

The selection of the subset size for the DIC analysis is key to maximize the measurement accuracy while ensuring a sufficient spatial resolution. Ideally, the same subset size should be used for displacement and deformation measurements as well as crack mapping, thus requiring a single analysis of the images. Figure 2.10 compares subsets sizes ranging from 9×9 to 31×31 pixels using grids marked on the patterned wall surface. It is noted that a 9×9 pixel subset may not provide enough variation in the gray level for accurate measurements, whereas using a 31×31 pixel subset may reduce the spatial resolution of measurements. A 15×15 pixel subset seems suitable as it contains approximately 3×3 speckles (Sutton et al. 2009).

The influence of subset size on the accuracy of horizontal and vertical displacement measurements was assessed by analyzing 31 images acquired from each unloaded specimen. One image was chosen as the reference, and the mean and standard deviation of the horizontal and vertical displacement components (\(u\) and \(v\), respectively) for all data points were calculated to evaluate bias and standard deviation errors. A negligible change in bias was noted. Figure 2.11 presents the standard deviation error (‘SD’) for \(u\) and \(v\) for two representative specimens as a function of subset size, ranging from 9×9 to 61×61 pixels. The results show that the error decreases abruptly as the subset size increases. Since the tradeoff for enhanced accuracy is a reduced spatial
resolution, a compromise was sought by selecting a 15×15 pixel subset size. This choice is also supported by the fact that each subset contains about 3×3 speckles (Sutton et al. 2009) as shown in Figure 2.10(b).

2.5 RESULTS AND DISCUSSION

For each specimen, the in-plane load-displacement response is presented in Figure 2.12. The positive (pull) and negative (push) load-displacement values (H-d in Figure 2.6) at the cracking, peak load and ultimate state (~75-85 % of the peak load) are summarized in Table 2.2.

For specimen SS, failure was triggered by the opening of the column-beam joints after first cracking of the masonry infill due to the substandard joint reinforcement [Figure 2.1(b)], resulting in little energy dissipation [Figure 2.12(a)]. The improved details allowed specimen SA to attain a maximum increase in peak load and ultimate displacement of 21 and 79 %, respectively, compared with specimen SS, with a major enhancement in energy dissipation [Figure 2.12(b)]. In particular, the use of diagonal Ø13 mm steel bars at the column-beam joints [Figure 2.1(c)] offset the opening of the corner joints at increasing drifts. The main effect was to enable the development of the typical 'strut-and-tie' resisting mechanism of CM walls as the masonry infill acted as a diagonal strut resisting compression forces, and the RC tie columns resisted primarily axial forces (tension or compression, depending on the direction of the horizontal load) (Meli et al. 2011) until diagonal cracking failure of the CM wall occurred. For specimen SA-S, the additional reinforcement embedded in the bed joints and anchored in the RC tie columns (Figure 2.2) contributed by offsetting the opening of diagonal
cracks and their propagation into the columns, resulting in a maximum increase in peak load and ultimate displacement of 24 and 27 %, respectively, compared with specimen SA, and further enhancing energy dissipation [Figure 2.12(c)]. These results show that three different performance levels were attained, consistent with the objectives set forth in the design of the specimens. In the following sections, the results of 3D-DIC analysis are discussed based on displacement measurements, full-field strain maps, crack maps, and practical implications for structural analysis and design.

2.5.1 Displacement measurements

To compare the 3D-DIC measurements of in-plane horizontal displacement, diagonal deformation, and interface slip, with those from PWSs, the motions of the points on the patterned surface [Figure 2.7(b)] corresponding to those monitored with PWSs on the opposite surface (Figure 2.3) were considered.

2.5.1.1 Horizontal displacement

The drift measurements from sensor H1 [Figure 2.3 and Figure 2.4(a)] are compared with their 3D-DIC counterparts in the load-drift envelopes in Figure 2.13, which are derived from the hysteretic curves in Figure 2.12. The maximum standard deviation for the 30-image sets analyzed was ±15 μm (~ ±0.01 pixels), confirming the validity of the 3D-DIC setup deployed. It is noted that for all specimens the DIC measurements consistently mark similar envelopes to those of sensor H1, irrespective of the load direction and displacement level. To enable a quantitative comparison, the percent difference,
\( \Delta \), between DIC and PWS measurements, \( \delta_{\text{DIC}} \) and \( \delta_{\text{PWS}} \), was calculated via Equation 2.1 for any given constant-displacement plateau point in Figure 2.5(b):

\[
\Delta = \left| \frac{\delta_{\text{DIC}} - \delta_{\text{PWS}}}{\delta_{\text{PWS}}} \right| \times 100\%
\] (2.1)

\( \Delta \) is plotted in Figure 2.14 as a function of the positive and negative drifts measured through sensor H1. The vertical dashed lines mark the displacement levels associated with first crack, peak load, and ultimate state (Table 2.2). For all specimens, the measurement difference rapidly reduces to less than 4 % after the formation of the first shear crack, that is, as the load-resistance mechanisms of interest to assess the strength and deformability of a seismic-resistant CM wall develop. The fact that 3D-DIC measurements attained a comparable accuracy to PWS measurements indicates that the DIC setup and analysis approach were effective in meeting the challenges posed by the large measurement surfaces. This conclusion is reinforced by the results of diagonal deformation and interface slip measurements discussed below. In addition, this evidence supports the hypothesis that the full hysteretic response can be traced using 3D-DIC measurements without using surface-mounted PWSs, provided that images are acquired continuously, and possibly without the need of constant-displacement plateaus [Figure 2.5(b)].

2.5.1.2 Diagonal deformation

The measurements from sensor D1 [Figure 2.3 and Figure 2.4(b)] are compared with their 3D-DIC counterparts in the load-diagonal displacement envelopes in Figure 2.15. The maximum standard deviation for the 30-image sets analyzed was ±45 \( \mu \)m (~ ±0.03 pixels). This value is higher than for the horizontal
displacements, reflecting the fact that diagonal deformation measurements are more susceptible due to the effect of random cracks developing along and across the gauge length. This issue also applies to PWS measurements, in addition to the effect of vibrations of relatively long extension rods (in this case, 3 m). For all specimens, the DIC measurements consistently mark similar envelopes to those of sensor D1 in both elongation and contraction. The percent difference between DIC and PWS measurements was calculated per Equation 2.1 and, for each specimen, is plotted in Figure 2.16 as a function of the positive and negative displacements measured through sensor D1. Such difference, $\Delta$, rapidly reduces to less than 10 % (and typically below 6 %) after first cracking. Again, 3D-DIC measurements offered a comparable accuracy to those of the counterpart PWS.

2.5.1.3 Interface slip

The slip between masonry infill and RC tie columns is an important indicator of the integrity of CM walls subjected to in-plane shear forces, especially when non-toothed interfaces are used as in the case of specimen SS [Figure 2.1(b)]. The time-history of sensor S1 [Figure 2.3 and Figure 2.4(c)] measurements for specimen SS is plotted in Figure 2.17(a) vis-à-vis the horizontal in-plane load. A negligible slip is noted at any given load level. Figure 2.17(b) presents the 3D-DIC vertical displacement profile along an 80-mm long reference line (including 200 data points), which lies perpendicular to the masonry-RC interface at the patterned location opposite to that of sensor S1. There are negligible discontinuities (i.e., slip) along the displacement profiles irrespective of the load level, in agreement with the measurements from sensor
S1. This evidence suggests that 3D-DIC measurements can be used in lieu of PWS ones, whose accuracy is especially sensitive to the presence of random cracks in the tie columns near the sensor connection. In addition, 3D-DIC slip measurements can be made anywhere along the masonry-RC interfaces, offering a far more versatile assessment tool that capitalizes on the full-field nature of DIC measurements. From a practical standpoint, the absence of interface slip indicates that the masonry infill acted monolithically with the RC frame, which is a fundamentally different behavior from RC frames with masonry infills where the masonry contributes to the load-resistance mechanism only under relatively large drifts due to the presence of interface gaps. In particular, for the case of specimen SS, the lack of interface slip highlights the negligible benefit to strength and deformability of using a toothed instead of a non-toothed interface, contrary to popular belief, and emphasizes the predominant importance of a suitable reinforcement details in the RC column-beam joints.

2.5.2 Full-field strain maps

Full-field strain maps were derived from the measured 3D-DIC in-plane motions to gain an insight into the hypothetical strut-and-tie load-resistance mechanism (Meli et al. 2011) governing shear strength and deformability. Figure 2.18 shows the positive in-plane load-drift envelope for each specimen, where the markers indicate representative points including: (A) uncracked state; (B) cracking state; (C) increasing load past the first crack state and before reaching the peak load; (D) peak load state; and (E) ultimate state. For each of these points, Figure 2.19, Figure 2.20, and Figure 2.21 present the 3D-DIC map
of the strain component parallel to the hypothetical compression strut in the masonry infill, $\varepsilon_X$, for specimens SS, SA, and SA-S, respectively.

In specimen SS, the $\varepsilon_X$ contours indicate the progressive development of a compression strut along the $X$-direction once the first diagonal crack formed close to the wall base in point (B), at a load $H=+107$ kN and drift $d=1.7$ mm, together with flexural cracks along the left RC tie column (Figure 2.19). The strut is visualized through the $\sim -500 \ \mu\text{m/m}$ negative strain contours. The cracks are visualized as narrow discontinuity regions with positive (tensile) strain peaks of the order of $10^3 \ \mu\text{m/m}$. The compression strut degrades after reaching the peak load of $+152$ kN in point (D) until failure occurs due to the opening of the left column-beam joint. The 3D-DIC $\varepsilon_X$ map provides compelling visual evidence that the strength of the diagonal compression strut in the masonry, which is expected to form in well-functioning CM walls (Meli et al. 2011), was not exploited. In fact, the limited diagonal cracking indicates that this mechanism did not fully develop due to the premature opening of the column-beam joint, resulting in a significantly smaller strength, deformability, and energy dissipation compared with specimens SA and SA-S. This observation is confirmed in Figure 2.22, which shows the 3D-DIC map of the strain along the diagonals, $\varepsilon_X$ at the positive (pull) peak load, and $\varepsilon_Y$ at the negative (push) peak load state for specimens SS, SA, and SA-S. The maps clearly show a less developed compression strut in specimen SS.

The 3D-DIC $\varepsilon_X$ and $\varepsilon_Y$ strain contours for specimen SA (Figure 2.20 and Figure 2.22) explain the enhanced hysteretic response in Figure 2.12(b). In fact,
the formation of well-defined diagonal compression struts was enabled by the resistance of column-beam joints against opening. The strain maps offer comprehensive visual evidence up to the ultimate state ($H<+133$ kN, $d<18.3$ mm), where the DIC analysis cannot be performed on the patterned areas where extensive spalling of the masonry infill occurred (Figure 2.20).

The effectiveness of 3D-DIC measurements is further demonstrated in the case of specimen SA-S. First, the $\varepsilon_X$ and $\varepsilon_Y$ strain contours in Figure 2.21 and Figure 2.22 visualize the development of the compression strut past the first crack state ($H=+137$ kN, $d=2.0$ mm). Then, the crack-bridging contribution of the bed-joint reinforcement (Figure 2.2) is rendered in the widening of the compression struts ($\varepsilon_X$ and $\varepsilon_Y\sim500 \, \mu$m/m) involving nearly the entire diagonal length of the masonry infill at the peak load state and under the large drifts attained at the ultimate state [Figure 2.12(c)].

To the best of the writers’ knowledge, this study is the first to systematically validate 3D-DIC measurements on full-scale masonry structures vis-à-vis PWS measurements, and demonstrate the visualization of a strut-and-tie and shear-resistance mechanism based on full-field deformation measurements. In particular, the strut-and-tie mechanism was originally hypothesized by Polyakov (1956) and Holmes (1961) for masonry infills, and became the theoretical foundation for the in-plane strength analysis of masonry-infilled RC frames as well as CM walls (Meli et al. 2011; Paulay and Priestley 1992; Tomažević 2006). In perspective, DIC measurements can be enlisted to validate and calibrate existing strength analysis algorithms, for example by
accurately defining the inclination [e.g., (Guerrero et al. 2014)] and width of compression struts in the failure region.

2.5.3 Full-field crack maps

3D-DIC strain maps were also evaluated to understand the feasibility of producing faithful crack maps at different loading stages. The spatial resolution of DIC measurements is influenced primarily by the subset size used in the correlation analysis (Bornert et al. 2009). The selection of a 15×15 pixel subset size as a reasonable compromise between accuracy and resolution (Figure 2.11) is further supported in Figure 2.23. The maximum principal strain, $\varepsilon_1$, maps based on subset sizes of 15×15, 21×21, and 31×31 pixels, are used to illustrate the influence of the subset size on the crack mapping resolution for specimen SA-S at the peak load of +213 kN. In all cases, the discontinuities indicating open cracks are marked by $\varepsilon_1$ with peak values of the order of $10^3 \mu$m/m, thus well above those associated with concrete and masonry cracking ($~10^2 \mu$m/m). However, a better level of detail is attained using 15×15 pixel subsets, which were previously shown to provide accuracy comparable to that of PWSs for the purpose of displacement measurements (Figure 2.14 and Figure 2.16). The 3D-DIC crack maps based on $\varepsilon_1$ values at the first crack, peak load, and ultimate state under positive horizontal in-plane loads are presented in Figure 2.24(a), Figure 2.25(a), and Figure 2.26(a) for specimens SS, SA and SA-S, respectively.

For all specimens, the 3D-DIC $\varepsilon_1$ maps visualize the flexural cracks that formed horizontally on the left RC tie column and propagated into the masonry infill at the first crack state. For specimen SS, the limited energy dissipation
[Figure 2.12(a)] is described by the rapid opening of the left column-beam joint once the peak load was attained ($H=+152$ kN, $d=8.3$ mm), resulting in a limited development of diagonal cracks between the peak load and ultimate state, until the joint failed [Figure 2.24(a)]. For specimen SA, the enhanced joint reinforcement [Figure 2.1(c)] resulted in significantly higher strength and deformability than specimen SS [Figure 2.12(b)] with comparable diagonal cracks at the peak load and ultimate state [Figure 2.25(a)]. For specimen SA-S, the denser $\varepsilon_1$-based crack maps at the peak load and ultimate state highlight the contribution of the additional bed-joint reinforcement (Figure 2.2) in further enhancing strength and deformability [Figure 2.12(c)]. In fact, multiple diagonal cracks formed due to the crack-bridging action exerted by the reinforcement, and the entire upper half of the masonry infill was involved in the load-resistance mechanism [Figure 2.26(a)]. From a practical standpoint, the evidence provided through 3D-DIC strain maps can be used to define the amount and location of bed-joint reinforcement, and verify the effectiveness of these design choices, irrespective of the specific type of masonry structure (e.g., confined or infilled). For example, in the case of specimen SA-S, the limited damage developing in the lower third of the masonry infill suggests that reinforcement is needed primarily in the top two thirds, whereas reinforcement used elsewhere may not significantly contribute to strength and deformability. This consideration is especially important when designing seismic-resistant strengthening or repair systems because prescribing redundant reinforcement entails more time-
consuming and labor-intensive construction operations, with a negative impact on typically stringent time and budget constraints.

Figure 2.24(b), Figure 2.25(b), and Figure 2.26(b) show the final hand-drawn crack maps from the opposite face of each specimen, which were mirrored (left-right) to facilitate the comparison with their DIC counterparts. Comparing 3D-DIC with hand-drawn crack maps in Figure 2.24, Figure 2.25, and Figure 2.26 shows that more detailed maps are obtained through non-contacting DIC measurements. In particular, for all specimens the hand-drawn maps do not indicate most of the damage highlighted by the DIC maps along the RC tie beam, including the column-beam joints. Human error is inevitably a factor. However, it is noted that for safety purposes hand-drawn maps were made on unloaded specimens after failure, when most of the cracks in the concrete were closed with the exception of the failed left corner in specimen SS (Figure 2.24), and thus were difficult to recognize. This is not a concern for DIC maps as they are based on $\varepsilon_1$ values derived from displacements measured on loaded specimens, when the cracks were open. The evidence presented indicates that faithful 3D-DIC crack maps can be obtained, with the following advantages over hand-drawn maps: (a) better level of detail, especially for closing cracks; (b) minimized influence of human errors; (c) ability to map cracks at any loading stage, thereby enabling one to monitor damage formation and development, which is impractical otherwise; and (d) safety, as non-contacting measurements are made without the need to closely inspect brittle specimens approaching collapse. In particular, the ability to map damage progression in a full-field fashion and at different loading
stages makes 3D-DIC measurements an attractive means to obtain valid experimental evidence to underpin the verification and calibration of numerical (e.g., finite element) models.

2.5.4 Potential for COD calculation

The potential to estimate CODs through the analysis of 3D-DIC displacement measurement is illustrated in Figure 2.27. Figure 2.27(a) shows the DIC vertical displacement map for specimen SA-S at the positive peak load state \((H=+213 \text{ kN}, d=12.2 \text{ mm})\), marking line GH that intersects multiple flexural (horizontal) cracks along the left RC tie column. In Figure 2.27(b), the vertical displacement profiles along line GH are plotted for different positive load levels vis-à-vis the interested portion of the \(\varepsilon_1\)-based crack map at \(H=+213 \text{ kN}\). It is noted that the discontinuities in the vertical displacement profiles indicate the open cracks along line GH. These evidence suggests that the amplitude of the discontinuities provides a measure of COD as it increases at increasing drifts up to the ultimate state \((H=+179 \text{ kN}, d=22.5 \text{ mm})\), thus capturing the progressive opening of the tensile cracks along the tie column. This outcome is similar to that presented by Destrebecq et al. (2011) for the case of tensile cracks in the constant moment region of a RC beam. However, experiments where progressive crack openings are locally measured with benchmark PWSs (e.g., crack opening gauges) are needed to test if and how the amplitude of the discontinuities in a given displacement profile can be used to accurately estimate CODs.
2.6 CONCLUSIONS

Based on the evidence presented from load tests and 3D-DIC measurements on full-scale confined masonry walls, the following conclusions are drawn:

1. Suitable high-contrast speckle patterns can be applied on large masonry and concrete surfaces by spraying dark paint on flexible stencils. Whitewash can be used to provide a light background.

2. Wide-angle camera lenses are typically needed to capture images of a full-scale masonry wall specimen. Relatively small stereo angles (e.g., 10°) are sufficient when minimal out-of-plane motions are expected, which is typically the case for structural walls loaded horizontally in their plane.

3. It is feasible to define a subset size for 3D-DIC analysis that yields accurate displacement measurements as well as high-resolution crack maps.

4. 3D-DIC measurements of story drift and diagonal deformation offer comparable accuracy to surface-mounted PWSs. To the best of the writers’ knowledge, this study is the first to systematically assess 3D-DIC measurements on large masonry structures vis-à-vis benchmark PWS measurements.

5. 3D-DIC measurements of interface slip can be used instead of those from PWSs, which are sensitive to random flexural cracks in the vicinity of the PWS connections to the specimen surface. 3D-DIC displacement fields offer a far more versatile analysis tool as the slip can be assessed virtually anywhere along the masonry-RC interface.

6. Specific strain components can be rendered in 3D-DIC maps to visualize load-resistance mechanisms and failure modes. By using diagonal
strain maps, experimental evidence of the development of diagonal struts in CM walls was presented for three specimens with different in-plane strength and deformability. To the best of the writers’ knowledge, this study is the first to present the experimental full-field visualization of strut-and-tie mechanisms in masonry infills.

7. Faithful crack maps can be obtained based on 3D-DIC maximum principal strain maps. This method offers significant advantages over hand-drawn maps, including improved level of detail, minimized influence of human errors, ability to map cracks at any loading stage, and safety.

8. Cracks can be accurately located and their progressive opening can be monitored based on 3D-DIC displacement measurements. Further research is necessary to test the hypothesis that the amplitude of the discontinuities in full-field displacement maps can be used to determine crack opening displacements (CODs).

9. The 3D-DIC analyses presented herein were performed using a standard desktop PC. Therefore, non-contacting 3D-DIC measurements stand as a powerful and accessible tool to advance the understanding of the behavior of concrete and masonry structures, inform their analysis and design, and underpin the verification and calibration of numerical models. In perspective, there is value and potential in exploring the integration of high-speed stereo-vision systems into novel hybrid testing platforms for structures under dynamic (e.g., seismic, wind) loads. For the specimens discussed in this paper, full-field strain maps provided quantitative and visual evidence on the importance of different design details to
enable the development of an effective strut-and-tie and shear resisting mechanism (e.g., corner joint reinforcement vis-à-vis toothed masonry-RC interfaces), and the contribution to shear strength and crack-control of bed-joint reinforcement in a representative strengthened wall (to be used to optimize reinforcement amount and location).

2.7 REFERENCES


monitoring and mechanical modeling." *Cement and Concrete Composites*, 45(0), 243-254.


digital image correlation." Optics and Lasers in Engineering, 44(11), 1132-1145.


Figure 2.1 Confined masonry specimens: (a) schematic; (b) details of specimen SS; and (c) details of specimens SA and SA-S for enhanced strength and deformability (note enhanced connections reinforcement and toothed interface between masonry panel and RC tie columns.) Dimensions in mm.
Figure 2.2 Strengthening in specimen SA-S: (a) cutting of grooves along bed joints; (b) installation of aluminum strips; (c) close-up of reinforcement anchorage into RC tie column; and (d) embedding reinforcement in mortar.

Figure 2.3 Loading frame and point-wise sensor layout.
Figure 2.4 Close-up photographs of point-wise sensors: (a) H1; (b) D1; and (c) S1 at interface between RC tie column and masonry panel in specimen SS.
Figure 2.5 Loading protocol: (a) time-horizontal displacement (sensor H1) function; and (b) close-up showing sample constant displacement plateaus for image acquisition.

Figure 2.6 Load resisting mechanism of CM wall under combined in-plane horizontal load, $H$ (e.g., seismic load) and vertical load (e.g., weight of top floors and roof system) (Meli et al. 2011): (1) diagonal cracking of masonry panel (‘cracking’ state); (2) propagation of diagonal cracks into RC tie columns at maximum load (‘peak load’ state); and (3) shear failure of masonry panel and RC tie columns (‘ultimate’ state).
Figure 2.7 3D-DIC setup: (a) schematic of plan view; and (b) photograph. Dimensions in mm.
Figure 2.8 Speckle pattern: (a) whitewashing wall; (b) spray-painting of speckle pattern on whitewashed background using stencil; and (c) close-up of finished speckle pattern.
Figure 2.9 Histogram of gray levels in speckle pattern: (a) specimen SS; (b) specimen SA; and (c) specimen SA-S.
Figure 2.10 Subset size compared with speckle pattern: (a) 9×9 pixels; (b) 15×15 pixels; (c) 21×21 pixels; and (d) 31×31 pixels.

Figure 2.11 Standard deviation error as function of subset size at zero load: (a) horizontal displacement, $u$; and (b) vertical displacement, $v$. 
Figure 2.12 Hysteretic load-displacement response based on sensor H1 measurements: (a) specimen SS; (b) specimen SA; and (c) specimen SA-S.
Figure 2.13 Comparison between displacement measurements through sensor H1 and 3D-DIC based on load-displacement envelopes: (a) specimen SS; (b) specimen SA; and (c) specimen SA-S.
Figure 2.14 Measurement difference between sensor H1 and 3D-DIC within range of positive and negative displacement at ultimate state: (a) specimen SS; (b) specimen SA; and (c) specimen SA-S.
Figure 2.15 Comparison between displacement measurements through sensor D1 and 3D-DIC based on load-displacement envelopes: (a) specimen SS; (b) specimen SA; and (c) specimen SA-S.
Figure 2.16 Measurement difference between sensor D1 and 3D-DIC within range of positive and negative displacement at ultimate state: (a) specimen SS; (b) specimen SA; and (c) specimen SA-S.
Figure 2.17 Interface slip between masonry panel and RC tie column in specimen SS: (a) slip measured through sensor S1 vis-à-vis loading history; and (b) 3D-DIC vertical displacement profile along virtual line EF across masonry-RC interface at different load-drift (H-d) levels.
Figure 2.18 Load-drift envelopes with markers for 3D-DIC strain maps in Figures 2.18-2.20 and 2.22-2.24: (a) specimen SS; (b) specimen SA; and (c) specimen SA-S. Markers (B), (D) and (E) indicate cracking, peak load and ultimate state, respectively.
Figure 2.19 DIC $\varepsilon_X$ maps at increasing positive drift (sensor H1) in specimen SS ($\varepsilon_X < 0$ indicates compression).

Figure 2.20 DIC $\varepsilon_X$ maps at increasing positive drift (sensor H1) in specimen SA ($\varepsilon_X < 0$ indicates compression).

Figure 2.21 DIC $\varepsilon_X$ maps at increasing positive drift (sensor H1) in specimen SA-S ($\varepsilon_X < 0$ indicates compression).
Figure 2.22 (a) DIC $\varepsilon_X$ maps at positive peak load; and (b) DIC $\varepsilon_Y$ maps at negative peak load, in specimens SS, SA and SA-S ($\varepsilon_X$ and $\varepsilon_Y < 0$ indicates compression).
Figure 2.23 Influence of subset size on crack mapping resolution based on DIC maximum principal strain maps from specimen SA-S at peak load of +213 kN: (a) 15×15 pixels; (b) 21×21 pixels; and (c) 31×31 pixels.
Figure 2.24 Crack mapping on specimen SS: (a) DIC maximum principal strain map at first crack (B), peak load (D), and ultimate (E) state using 15×15 pixel subsets. Displacement values indicate drift per sensor H1; and (b) hand-drawn maps based on visual inspection. Red corresponds to +3000 μm/m.

Figure 2.25 Crack mapping on specimen SA: (a) DIC maximum principal strain map at first crack (B), peak load (D), and ultimate (E) state using 15×15 pixel subsets. Displacement values indicate drift per sensor H1; and (b) hand-drawn maps based on visual inspection. Red corresponds to +3000 μm/m.

Figure 2.26 Crack mapping on specimen SA-S: (a) DIC maximum principal strain map at first crack (B), peak load (D), and ultimate (E) state using 15×15 pixel subsets. Displacement values indicate drift per sensor H1; and (b) hand-drawn maps based on visual inspection. Red corresponds to +3000 μm/m.
Figure 2.27 Potential to estimate CODs in specimen SA-S: (a) vertical displacement map at peak load; and (b) maximum principal strain map at peak load vis-à-vis vertical displacement profile along line GH at different positive loads, $H$, showing progressive crack opening in RC tie column.
Table 2.1 Material strength properties.

<table>
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Table 2.2 Summary of cyclic load test results.

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<th>‘Ultimate’ state</th>
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<td>+132 -135</td>
<td>+11.6 -13.7</td>
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<tr>
<td>SA-S</td>
<td>+2.0 -1.5</td>
<td>+137 -125</td>
<td>+12.2 -14.3</td>
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CHAPTER 3
CONTEXT-SENSITIVE SEISMIC STRENGTHENING OF SUBSTANDARD
CONFINED MASONRY

ABSTRACT. The most common failure mode of CM is in-plane shear cracking, which is characterized by diagonal (X-shaped) cracks, and in multi-story buildings may result in collapse at the ground-floor level. An efficient method to prevent the brittle behavior associated with the in-plane shear cracking is to strengthen the walls using horizontal reinforcement, thereby enhancing shear strength and deformability. While using strengthening materials and methods typical in developed areas is unlikely, a context-sensitive strengthening practice suitable for developing regions requires using simple techniques and locally available and inexpensive materials which are familiar to local practitioners. Considering these limitations, this chapter addresses the question of whether it is feasible to strengthen a substandard CM wall, using context-sensitive materials and practices.

Proof of concept is demonstrated based on experimental evidence from in-plane cyclic testing of two full-scale CM wall specimens built with substandard materials (e.g., masonry with compressive strength of 5.8 MPa and concrete with cylinder compressive strength in the range of 9 - 14 MPa) and details (e.g., open stirrups with relatively large on-center spacing). One specimen was tested as-is to serve as the control specimen. The second specimen was strengthened using
metallic strips that were embedded in mortar within selected bed joints and then load-tested. Three-dimensional digital image correlation was enlisted to gain insight into the load-resistance mechanisms and damage evolution.

The strength and deformability of the strengthened specimen were compared with those of the control specimen, and with theoretical estimates for a benchmark CM wall built with acceptable materials (e.g., masonry with compressive strength of 10 MPa and concrete with cylinder compressive strength of 26 MPa) and details (e.g., closed stirrups with suitable on-center spacing). It is shown that the strengthened CM wall exhibited comparable shear strength and deformability to those of the “standard” wall, different from the unstrengthened counterpart, and that the strength can be conservatively predicted. The seismic resistance of the unstrengthened and strengthened CM walls was assessed based on the acceptance criteria set forth in Mexico City Building Code (MCBC) Requirements for Masonry Structures (NTCM 2004). It is shown that only the strengthened specimen qualifies as an earthquake-resistant structure. These results demonstrate proof of concept that it is feasible to strengthen a substandard CM wall, using context-sensitive materials and practices, and make it safe, comparable to a CM wall built with acceptable-quality materials and details.

3.1 INTRODUCTION

Confined masonry (CM) consists of load-bearing masonry panels confined with cast-in-place reinforced concrete (RC) tie columns and beams. The distinguishing feature of CM is that the masonry panel is constructed before
casting the tie columns and beams, resulting in monolithic behavior of the masonry panel and the RC frame when subjected to lateral loads. Due to satisfactory seismic performance, CM has become the predominant low-rise construction system in several areas around the world (Meli et al. 2011). However, in developing regions, substandard materials, and construction and detailing deficiencies may negatively affect the seismic performance of CM buildings, as it was observed in the 2003 Colima earthquake in Mexico (Alcocer and Klingner 2006), the 2010 Haiti earthquake (Eberhard et al. 2010), and the 2010 Chile earthquake (Brzev et al. 2010).

On 12, January 2010, a Mw 0.7 earthquake struck the southern region of Haiti. Over 220,000 Haitian were killed and thousands more were injured; up to 1.6 million were displaced and settled in camps; 20 % of structures were severely damaged or destroyed and another 27 % required repairs (UNOPS 2011). If feasible, a preventive (pre-hazard) strengthening is a realistic and relatively low-cost option that may result in saving of lives and reducing cost. However, there is little knowledge and experience on whether strengthening can realistically improve the seismic behavior of a CM dwelling structure, especially using context-sensitive techniques with locally available (and often relatively low-quality) materials (Beunza and Eresta 2011). The overall goal of this chapter is to contribute to filling this gap by understanding whether it is feasible to strengthen a substandard CM wall, using context-sensitive materials and practices, and make it safe, comparable to a counterpart built with acceptable-quality materials and seismic details.
The key factors resulting in substandard CM construction in developing regions can be categorized as those related to:

1. Materials, such as quality and thus strength of masonry units, mortar and concrete. The mixture design of concrete masonry units (CMUs), mortar and concrete typically lacks a sufficient amount of cement (to reduce cost) and may have a relatively high water/cement ratio (to improve workability) (EERI 2010). As a result, compressive strength values have been reported in the range of:
   - 3 - 10 MPa and 1.4 - 8 MPa for CMUs (on the net area) in Chile (Moroni et al. 2004) and Venezuela (Lafuente et al. 1998), respectively;
   - 7 MPa and 7.8 MPa for mortar in Venezuela (Lafuente et al. 1998) and Mexico (Tena-Colunga et al. 2009), respectively;

2. Detailing, such as:
   - RC tie column-tie beam connection – The connections may lack seismic detailing, for example due to absence of or excessive spacing between ties, and discontinuity in longitudinal reinforcement through the connection (Astroza et al. 2012; Eberhard et al. 2010).
   - Reinforcement detailing in tie columns and beams – The longitudinal reinforcement of the RC frame typically consists of four Ø10 mm or Ø13 mm deformed or smooth rebars. The stirrups are usually made of Ø6 mm smooth rebars and spaced apart between 150 and 300 mm, with no decrease in the spacing at the column and beam ends, and bends not over 90 degrees, resulting

- Masonry-RC frame interface – Construction of masonry panel prior to RC frame in CM structures, results in a modest bond connection between the RC frame and the masonry panel. As a result, the masonry panel acts as the main load-bearing element when a CM wall is subjected to in-plane loads. Tothing at the masonry-tie column interface, which is created by staggering of masonry units in a saw-tooth manner (Figure 3.1), enhances the interaction between the masonry panel and the tie columns and offsets separation (Astroza et al. 2012). In Haiti, a flush (i.e., non-toothed) interface was frequently used as reported by Lang and Marshall (2011) and Mix et al. (2011). In addition, the masonry panels may not be assembled to the full story height. The gap between the masonry panel and the bottom of tie beam or slab is filled with rock or masonry debris resulting in masonry walls that are not load-bearing (Marshall et al. 2011).

The most common failure mode of CM is in-plane shear cracking, which is characterized by diagonal (X-shaped) cracks and in multi-story buildings may result in collapses at the ground-floor level, i.e., “soft-story” effect (Alcocer et al. 2004; Alcocer and Klingner 2006; Astroza et al. 2012; Flores and Alcocer 1996; Tomaževič and Gams 2012). An efficient way to avoid the fragile behavior associated with in-plane shear cracking is to provide horizontal reinforcement in masonry walls, thereby enhancing shear strength and deformability (Schultz 1994). This may be accomplished through the following two methods.
1. Horizontal reinforcement can be incorporated in the wall during construction. For example, Aguilar et al. (1996) showed that CM specimens horizontally reinforced with deformed cold-drawn steel wires had a considerably higher peak strength (up to 70 %) and ultimate displacement (up to 140 %) under cyclic in-plane loads compared to unreinforced counterparts, with a more uniform distribution of inclined cracks. Alcocer and Zepeda (1999) investigated the behavior of four large-scale CM walls with different horizontal reinforcement ratios and column details tested under cyclic lateral load. The reinforced walls exhibited substantial increase in strength (up to 76 %) and ultimate displacement (up to 150 %) compared to unreinforced walls. It was concluded that multi-perforated clay bricks can be used to ensure earthquake resistance if a minimum amount of horizontal reinforcement and proper column detailing are provided.

2. Horizontal reinforcement in the form of stainless steel or fiber-reinforced polymer (FRP) bars can be inserted into grooves cut into the masonry [Near-Surface Mounting (NSM) technique (De Lorenzis and Teng 2007)] or cut along mortar joints [Structural Repointing (SR) technique (Tumialan et al. 2001)], and the grooves are filled with polymeric adhesives or mortar. There are a few studies where full-scale masonry walls were strengthened or repaired using the NSM or SR technique. Li et al. (2005) reported on strengthening of unreinforced masonry walls using different strengthening configurations and various materials. The strengthened walls, tested under diagonal compression, exhibited significantly higher in-plane strength and deformability compared to unreinforced walls. The maximum increment in shear strength was 80 % in walls strengthened
with glass FRP (GFRP) bars placed at every bed joint. Tumialan et al. (2003) reported the test results of three infill masonry walls with RC frames, two strengthened by FRP SR technique. The results showed that while the shear strength of the unstrengthened specimen started to decrease at a 0.5 % drift under in-plane cyclic loading, the FRP strengthened specimens reached lateral drift of 0.7 % without losing lateral carrying capacity. Loayza Seminario (2008) conducted experiments to investigate the effectiveness of GFRP SR technique in repairing a CM wall that previously failed in shear under cyclic loading. The results showed that the initial stiffness of the repaired wall was 5.8 times the final stiffness of the original wall and the maximum shear strength of the CM wall was fully recovered.

This chapter addresses the question of whether it is feasible to strengthen a substandard CM wall, using context-sensitive materials and practices, and make it safe, comparable to a wall built with acceptable-quality materials and seismic details. The feasibility is evaluated based on the two ‘feasibility criteria’ (i.e., F1, F2) defined in Chapter 1. Aluminum strips were used to mimic low-stiffness reinforcement (e.g., a smaller amount of steel) to be embedded with cement mortar into grooves cut along bed joints. The use of accessible materials and simple methods is important and necessary for a context-sensitive strengthening practice that has the potential to be accepted and enable communities to strengthen their own CM structures and train future generations (Kijewski-Correa and Taflanidis 2012; Mix et al. 2011). It is emphasized that
while aluminum strips were used as the horizontal reinforcement to test the concept, other more accessible options such as steel may be used instead.

Proof of concept is demonstrated based on evidence from in-plane cyclic tests on two full-scale CM wall specimens. In addition, the strength and deformability of the strengthened specimen were compared with those of a CM wall with acceptable materials and details, which were estimated using an existing well-validated semi-empirical model (Riahi et al. 2009). The earthquake resistance of the specimens was then verified based on the criteria set forth in the Mexico City Building Code (MCBC) Requirements for Design and Construction of Masonry Structures (NTCM 2004), which are representative of the ASCE 7 criteria for Seismic Design Category D (ASCE 2010).

3.2 EXPERIMENTAL PROGRAM

3.2.1 Specimens and materials

The test matrix included two full-scale CM wall specimens (SS and SS-S). A comprehensive description of specimen SS can be found in Chapter 2. As illustrated in Figure 3.2, specimen SS-S consisted of a SS wall that was strengthened with aluminum strips in six out of 10 bed joints (labeled AS-1 to AS-6). Aluminum strips were instrumented with strain gauges along the wall diagonals, as marked with “X” in Figure 3.2. The same strengthening technique was followed as for specimen SA-S: 10 mm wide and 15 mm deep grooves were cut along the bed joints in the mid-height portion of the wall using a grinder [Figure 3.3(a)], alternating from one face of the wall to the other. The grooves were then cleaned with an air gun and pre-moistened before repointing. 6061-T6
aluminum strips with cross section of 3.2×12.7 mm were inserted in the bed joints and covered with cement mortar [Figure 3.3(b)]. Six bed joints were selected for repointing to increase the efficiency of strengthening based on the results of the test performed on specimen SA-S, in which the damage in the masonry panel was mainly observed in the mid-height portion of the wall. The reinforcement ratio, $\rho_{hr} = 0.05\%$, was calculated as the ratio of cross sectional area of the aluminum strips to the wall vertical cross section area. Since specimen SS had substandard beam-column joints, it was necessary to strengthen the joints to prevent premature opening during the test. Ø13 mm steel rebars with 90° end bends were inserted in two grooves (one per side of the wall) that were cut diagonally in each corner of the wall [Figure 3.3(c)]. The grooves were filled with Type I OPC mortar [Figure 3.3(d)]. The mix design of CMUs, concrete and mortar was tailored to reach a compressive strength representative of substandard CM construction. Salient properties of the materials used are summarized in Table 2.1(see Appendix A).

3.2.2 Test setup and measurement systems
3.2.2.1 Loading apparatus and protocol

The load test setup is illustrated in Figure 3.4 and Figure 3.5. The horizontal in-plane cyclic load, $H$, was imparted by a hydraulic actuator with maximum capacity of 500 kN and stroke of ±76 mm. The actuator swivel base was secured to a reaction frame and the swivel head was bolted to a steel spreader beam that was rigidly connected to the RC beam by means of steel anchors. A constant vertical compressive load of 88.3 kN was applied using a
hydraulic jack and two steel spreader beams on top of each wall. The resulting uniform pressure of 0.2 MPa aimed at simulating the dead load acting on the first story of a typical two-story dwelling.

The load test was carried out by applying horizontal reverse-cyclic displacements with increasing amplitude. For specimens SS-S, the same displacement history was used as for specimen SS (Figure 2.5). The displacement history aimed at accurately determining the three limit states that can be used to describe the mechanism of shear resistance of CM walls under seismic loads, as illustrated in Figure 2.6.

3.2.2.2 Measurement systems

Aside from the point-wise sensors (PWSs) mounted on one face of each specimen and strain gauges, a three-dimensional digital image correlation (3D-DIC) system was used on the other face of the specimens. As shown in Figure 3.6, the PWSs included several linear potentiometers, linear displacement transducers, strain gauges, and one pressure transducer. The linear potentiometers were used to measure the in-plane horizontal displacement (at midspan of the tie beam) and diagonal and vertical deformations. The linear displacement transducers were used to monitor sliding along different interfaces such as footing-strong floor, wall-footing and masonry panel-tie columns. Strain gauges were mounted on both vertical and horizontal steel rebars at beam-column joints and columns bases, diagonal steel rebars used for strengthening the beam-column joints, and aluminum strips used as horizontal reinforcement.
The pressure transducer was mounted on a hydraulic jack to measure the vertical load applied on the specimens.

The 3D-DIC setup is illustrated in Figure 2.7. 3D-DIC was enlisted aimed at visualizing full-field strain and crack maps, thus providing new information to describe the load-resisting mechanisms and damage evolution of specimens. In Chapter 2, it was shown that 3D-DIC displacement measurements have comparable accuracy to PWSs, and high-resolution DIC-based crack maps gave improved level of details compared to conventional hand-drawn maps.

3.3 RESULTS AND DISCUSSION

For each specimen, the in-plane load-displacement response is presented in Figure 3.7. The positive and negative load-displacement values at the cracking, peak load and ultimate limit state are summarized in Table 3.1.

First, the strength and failure modes of the specimens are described. Second, the strength and deformability of the strengthened specimen are assessed by comparing them to those of a benchmark CM wall with acceptable materials and details. Third, it is shown that shear strength of the strengthened specimen can be theoretically estimated. Fourth, the seismic performance of the specimens is assessed according to NTCM (2004).

3.3.1 Strength and failure mode:
3.3.1.1 Control specimen (SS)

Specimen SS peak load was 152 kN, attained at an in-plane displacement of 8.3 mm. The ultimate displacement after a 24 % drop in the peak load was 11.0 mm. Figure 3.8(a) and Figure 3.8(b) illustrate the DIC-based crack maps for
specimen SS at the ultimate limit state in the positive and negative load directions, respectively. The open cracks are marked by principal tensile strains, $\varepsilon_1$, with peak values in the order of $10^3$ µm/m, well above those associated with concrete and masonry cracking. The strength degraded rapidly and the test was stopped after failure of a beam-column joint as shown in Figure 3.8(a). The crack pattern of specimen SS was characterized by a limited number of inclined cracks with no masonry spalling. In fact, as was shown in Chapter 2, failure of the beam-column joint hindered the full development of the strut-and-tie load-resistance mechanism that is expected in well-functioning CM walls (Meli et al. 2011). The results from strain gauges mounted on steel rebars at the beam-column joints and at the columns bases (e.g., Figure 3.9) showed that yielding was not attained, confirming the premature opening of the joint and limited contribution of the steel reinforcement in dissipating energy.

3.3.1.2 Strengthened specimen (SS-S)

Specimen SS-S reached a peak load of 200 kN at an in-plane displacement of 15.0 mm. The ultimate displacement upon 25 % drop in the peak strength was 21.0 mm. Compared to specimen SS, strengthening resulted in a maximum increase in peak strength and ultimate displacement of 35 and 106 %, respectively. The DIC-based crack maps for specimen SS-S at the ultimate limit state in the positive and negative directions are illustrated in Figure 3.10(a) and Figure 3.10(b), respectively. The crack pattern of the strengthened specimen was characterized by well distributed inclined cracks in the mid-height region of the specimen and diagonal cracks propagating into the columns below the joint
diagonal rebar, as illustrated in Figure 3.11. Propagation of cracks below the joint diagonal rebars shows that the rebars were effective in delaying the opening of the joints, thus preventing premature failure as it was observed for specimen SS (Figure 3.8). A comparison between the crack map of specimen SS-S and that of specimen SS highlights the role of the horizontal reinforcement in resisting tension forces and distributing the cracks over a relatively large area, thereby exploiting the load bearing capacity of the masonry panel and producing a significant increase in shear strength and deformability (Table 3.1). The strain measurement from the strain gauges mounted on the aluminum strips is presented in Figure 3.12. The results show that the strain in five out of six aluminum strips reached a value of 4000 μm/m, which is associated with the onset of plastic deformation (Figure A.30). In fact, the high level of strain in the horizontal reinforcement confirms their significant contribution to the shear strength and ductility of specimen SS-S. In addition, the strain measurements for selected steel rebars in the beam-column joints and columns bases for specimen SS-S (e.g., Figure 3.9) confirms yielding of the reinforcement, which contributed to energy dissipation.

3.3.2 Strength and deformability assessment – Criterion F1

The strength and deformability of the strengthened specimen were assessed by comparing them to those of a benchmark CM wall built with acceptable materials and details. For the latter, theoretical in-plane load-displacement curves were defined using an existing and well-validated semi-empirical model proposed by Riahi et al. (2009). The model was specifically
developed to predict the shear strength and deformability of CM at the three limit states by which the behavior of CM can be characterized (Figure 2.6). The model is based on a database of 102 CM walls encompassing a wide range of material properties, also including substandard materials (e.g., concrete and masonry with compressive strength in the range of 10 to 35 MPa and 2.5 to 25 MPa, respectively). The typical CM for which the model has been developed has the following characteristics:

- Two tie columns;
- Multiple longitudinal rebars per confining element;
- No bed joint reinforcement;
- No openings within the confined masonry panel;
- Height-to-length ratio in the range of 0.7-1.2;
- Governed by shear failure mode.

For each limit state, the equations predict mean model parameters associated with shear strength and horizontal displacement, while the variability in the model is represented by coefficients of variation (CV). Table 3.2 summarizes the statistical characteristics of the equations, including the mean, CV and the coefficient of determination ($R^2$). The equations are presented as follows.

3.3.2.1 Cracking shear stress ($\upsilon_{cr}$) and associated drift ratio ($\delta_{cr}$)

$$\upsilon_{cr} = 0.424 \upsilon_m + 0.374 \sigma_u \leq \upsilon_m$$  \hspace{1cm} (3.1)

$$\delta_{cr} = 0.72 \frac{\upsilon_{cr}}{f_m}$$  \hspace{1cm} (3.2)
where $u_m$ is the masonry shear strength from diagonal compression tests, $\sigma_u$ is the axial stress (e.g., from upper floors), and $f_m$ is the masonry compressive strength.

3.3.2.2 Peak load shear stress ($u_p$) and associated drift ratio ($\delta_p$)

$$u_p = 0.21u_m + 0.363\sigma_u + 0.0141\sqrt{\rho_{vc}f_{yvc}f'_c} \geq u_{cr}$$

(3.3)

$$\delta_p = 0.65\delta_u$$

(3.4)

where $\rho_{vc}$ is the tie column longitudinal reinforcement ratio, $f_{yvc}$ is the yield strength of column longitudinal reinforcement, $f'_c$ is the concrete compressive strength, and $\delta_u$ is the ultimate drift calculated from Equation 3.6.

3.3.2.3 Ultimate shear stress ($u_u$) and associated drift ratio ($\delta_u$)

$$u_u = 0.8u_p$$

(3.5)

$$\delta_u = 0.72\mu \frac{u_p}{\sqrt{f_m}} , \mu = \frac{0.5}{u_p^2} + 1.3 \leq 6.0$$

(3.6)

The model predictions for the case of substandard materials are first compared with the results for specimens SS as summarized in Table 3.3. In Figure 3.13(a), the model average estimate of load and displacement for each limit state and the associated variability (in the form of error bars indicating lower and upper bounds) are plotted and compared with the load-displacement envelopes for specimen SS. The results show that the peak and ultimate load values for specimen SS are in agreement with the upper limit of the model. However, the associated displacements did not reach the expected values due to premature opening of the joint. It is noted that the theoretical average peak load
equal to 124 kN (Table 3.3) offers a conservative estimate that may be suitable for design purposes.

The theoretical predictions assuming acceptable material properties were then compared with the experimental results from the strengthened specimen. Table 3.4 summarizes the load and displacement estimates based on the model (Riahi et al. 2009) assuming acceptable materials. Figure 3.13(b) shows the load-displacement envelopes of specimens SS-S and those based on the analytical model assuming acceptable materials. The error bars indicate the variability of strength and displacement values based from model calibration (Riahi et al. 2009). For the model predictions assuming acceptable materials, a compressive strength of masonry (on the net area) and concrete of 10 and 26 MPa, respectively, were used, consistent with minimum requirements from applicable standards (ACI 2011; ASTM C90; MSJC 2011). The masonry shear (diagonal tension) strength was calculated as 1.36 times the tensile strength of masonry (Yokel and Fattal 1976), and the tensile strength was assumed as 10 % of the compressive strength (Tomaževič 2006). Figure 3.13(b) shows that the strength and deformability of specimen SS-S at the peak and ultimate limit states are in agreement with the average estimates based on the model (Riahi et al. 2009) assuming acceptable materials. These results support the hypothesis that strengthening enables the transformation of a substandard CM wall into a wall with comparable strength and deformability to a standard CM wall built with acceptable materials and details. Therefore, Criterion F1 of the ‘feasibility criteria’ set forth in Chapter 1 is met for the strengthened specimen.
3.3.3 Shear strength estimation

The shear strength of a composite masonry wall system is typically estimated by adding the contribution of different elements (e.g., masonry, concrete layers and horizontal reinforcement) (Aguilar et al. 1996; Ghiassi et al. 2011). The shear strength of the strengthened specimen was predicted using the following equation:

\[ V_{tot} = V_{CM} + V_r \]  \hspace{1cm} (3.7)

in which, \( V_{CM} \) equal to 124±30 kN (Table 3.3) is the peak shear strength of the CM wall, which was estimated using the semi-empirical model by Riahi et al. (2009), and \( V_r \) equal to 45 kN is the contribution of horizontal reinforcement, which was calculated using the following equation:

\[ V_r = \eta f_{yr} A_r \]  \hspace{1cm} (3.8)

where: \( f_{yr} \) equal to 265 MPa is the yield strength of the horizontal reinforcement material; \( A_r \) equal to 242 mm\(^2\) is the cross sectional area of the horizontal reinforcement; and \( \eta \) is an “efficiency factor” equal to the ratio of the load resisted by the horizontal reinforcement to that associated with yielding of the reinforcement as the peak load is attained by the wall (Aguilar et al. 1996; Alcocer and Zepeda 1999). An efficiency factor of 0.7 was recommended (Alcocer 1996) based on the results of tests performed on CM walls horizontally reinforced with high-strength steel wires. The recommended \( \eta \) considers the non-uniform strain distribution in the reinforcement over the wall height, where the largest strains were recorded near the wall center and the strains at lower and upper part of the wall were typically negligible. By adding the contribution of
horizontal reinforcement (45 kN) to the peak strength of benchmark CM wall (124±30), the peak strength of specimen SS-S is estimated to be 169±30 kN as marked in Figure 3.14. The results show that the average estimate (169 kN) is in good agreement with the experimental peak strength, while the lower bound (139 kN) is a conservative estimate that may be suitable for design purposes.

In order to experimentally verify the efficiency factor, the measurements from strain gauges mounted on the aluminum strips were used to estimate \( \eta \). The efficiency factor, \( \eta \), is marked in Figure 3.15, where the ratio of the load resisted by the horizontal reinforcement to that associated with yielding of the reinforcement is plotted as a function of the ratio of horizontal displacement to the displacement associated with the peak load (Table 3.1), \( \frac{d}{d_p} \). At the peak load \( (d/d_p = 1) \), the average efficiency factor is equal to 68 %, which is similar to the efficiency factor of 70 % recommended by Alcocer (1996). The relatively high value obtained for \( \eta \) suggests that the reinforcement layout for strengthening is adequate, that is, adding more reinforcement along the bed joints closer to the RC tie beam and footing may not provide significant contributions to the shear strength.

3.3.4 Earthquake-resistance assessment – Criterion F2

In order to show if the strengthened CM specimen qualifies as an earthquake-resistant structure, the seismic design criteria specified in the Mexico City Building Code (MCBC) Requirements for Design and Construction of Masonry Structures (NTCM 2004) were assessed. In Mexico, 70 % of buildings include structural masonry walls and CM is the most popular masonry
construction system (Alcocer et al. 2003). In addition, masonry construction in Mexico is code-regulated since 1976 and the seismic provisions for masonry structures were developed from results of a comprehensive research program of over 20 years and were updated after the 1985 Mexico City earthquake (Alcocer and Meli 1995). Appendix A of NTCM (2004) sets forth criteria for earthquake-resistant qualification of masonry structures built in regions of high seismic hazard as defined in the Manual of Civil Structures MOC-93 (1993) (Alcocer et al. 2003). This seismic zone is equivalent to a Seismic Design Category (SDC) of D, in the ASCE/SEI 7-10 (2010) standard (Alcocer 1996; Tena-Colunga et al. 2009a). A large database of experimental results from CM walls were used to establish these criteria (Alcocer et al. 2003). There are three main criteria that should be evaluated for walls made with hollow units, using the information obtained from cyclic load testing and material characterization (Tena-Colunga et al. 2009b):

A1. The maximum shear strength from the in-plane cyclic test, obtained at a lateral drift ratio $\theta \leq 0.004$, $R_{\text{max}}$, should be equal or greater than the theoretical shear resistance $R$. Shear strength of specimens SS was estimated equal to $124\pm30$ kN. Shear strength of specimens SS-S was estimated equal to $169\pm30$ kN.

A2. $R_{\text{max}} \leq \lambda R$, in which $\lambda$ is an overstrength factor that takes into account the wall connecting details, such as intersecting walls, foundations, floor or roof systems, etc. The minimum value of the overstrength factor $\lambda$, is 1.3. Based on this criterion, wall specimens should provide sufficient strength under relatively
large drift ratios without being overdesigned, that is, enlisting the contribution of structural connections when incorporated into a well-designed structural system.

A3. The characteristics of the cycle at a drift ratio $\theta = 0.004$ must satisfy the following criteria:

a. The shear force reached, $R_2$, must be at least $0.8 R_{1,max}$, in which $R_{1,max}$ is the peak shear strength obtained in the same loading direction.

b. The peak-to-peak secant stiffness should not be less than 0.05 times the stiffness of the cycle at formation of the first crack ($K / K_e \geq 0.05$).

c. The equivalent energy dissipation ratio (EEDR) at that cycle must be greater than 0.15. As illustrated in Figure 3.16, the EEDR is calculated as the area contained by the hysteresis curve for that cycle divided by the area circumscribed by the parallelograms defined by the stiffness of the first cycle after cracking and the maximum load of the cycle for which the equivalent dissipated energy is calculated.

Criterion A3 is used to ensure that the wall maintains a significant reserve of shear strength and stiffness at a drift ratio, $\theta$, equal to 0.004, while providing a sufficient contribution to energy dissipation.

These criteria (i.e., A1, A2 and A3) were verified based on the cyclic tests on the control and strengthened specimens, as summarized in Table 3.5 and Table 3.6. Calculation of the peak-to-peak secant stiffness, $K / K_e$, for each specimen is illustrated in Figure 3.17. According to these results, specimen SS did not satisfy the first requirement of Criterion A3 due to premature opening of the joint, which resulted in a rapid loss of strength. Conversely, the strengthened
specimen satisfied all the requirements and qualified as an earthquake-resistant structure according to NTCM (2004). Based on these results, Criterion F2 of the ‘feasibility criteria’ is met for the strengthened specimen.

3.4 CONCLUSIONS

This paper discusses the feasibility of seismic strengthening substandard CM walls using context-sensitive materials and construction practices. Context-sensitive materials are locally available and commonly used, and thus relatively affordable (albeit sometimes substandard). They may also be recycled from collapsed buildings to be reused for strengthening purposes. Context-sensitive practices entail construction and installation operations that can be performed following practices familiar to local workers, without the need for additional training.

The following conclusions are drawn:

1. An efficient reinforcement layout was used to strengthen a substandard CM wall. Metallic strips were embedded in six (out of 10) bed joints located around the mid-height portion of the wall. The efficiency factor for the reinforcement contribution to shear strength was 68 %, which is similar to that recommended by Alcocer (1996) for CM walls. Adding reinforcement in the bed joints closer to the RC tie beam and the footing may not significantly contribute to the shear strength.

2. The diagonal steel rebars used for strengthening the beam-column joints were effective in offsetting the opening and failure of the joints, thus enabling the yielding of the horizontal reinforcement.
3. Strengthening resulted in a maximum increase in peak strength and ultimate displacement of 35 and 106 %, respectively, compared with those of the unstrengthened specimen.

5. The DIC-based crack map for the strengthened specimen was characterized by well-distributed inclined cracks, whereas in the control specimen, a limited number of cracks formed. This highlights the role of the horizontal reinforcement in resisting shear strength and enhancing ductility by bridging the cracks and distributing them over larger portions of the masonry panel, thus exploiting the load-bearing capacity of the masonry.

6. The strength and deformability of the strengthened specimen were assessed by comparing them to those of a benchmark CM wall built with acceptable materials and details, and estimated by a well-validated semi-empirical model (Riahi et al. 2009). It was shown that strengthening transformed a substandard CM wall into a wall with comparable strength and deformability to a standard wall built with acceptable materials and details. Based on these results, Criterion F1 of the ‘feasibility criteria’ was met for the strengthened specimen.

7. The shear strength of the strengthened specimen can be estimated by adding the contribution of horizontal reinforcement to the peak shear strength of the CM wall. It was shown that the average estimate is in good agreement with the experimental peak strength, and the lower-bound estimate is reasonably conservative and may be considered for design purposes.
8. Earthquake-resistance performance of the strengthened specimen was assessed according to the criteria set forth in Mexico City Building Code (MCBC) Requirements for Masonry Structures (NTCM 2004). It was shown that the strengthening technique transformed the substandard CM wall into a masonry wall that qualifies as earthquake-resistant. Based on these results, Criterion F2 of the ‘feasibility criteria’ was met for the strengthened specimen.

9. The two ‘feasibility criteria’ were evaluated for the strengthened specimen. Based on the criteria, It is shown that it is feasible to strengthen a substandard CM wall using readily accessible (and sometimes substandard) materials in conjunction with familiar construction practices, and make it safe, comparable to a CM wall built with acceptable materials and details.

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Figure 3.1 Tooothing at masonry-tie column interface made by staggering of CMUs and quarter blocks.

Figure 3.2 Schematic of strengthening technique. Positions of strain gauges mounted on metallic strips are marked with “X”. Dimensions in mm.
Figure 3.3 Practice of strengthening technique: (a) cutting grooves along bed joints; (b) inserting aluminum strips into the grooves and filling the grooves with cement mortar; (c) cutting diagonal grooves at corners and inserting Ø13 mm steel rebar; and (d) filling the grooves with cement mortar.
Figure 3.4 Schematic of load test setup. Dimensions in mm.
Figure 3.5 Photograph of load test setup.

Figure 3.6 Point-wise sensor layout.
Figure 3.7 Load-displacement response of specimens: (a) SS; and (b) SS-S.
Figure 3.8 DIC-based crack map of specimen SS at ultimate limit state: (a) positive load direction; and (b) negative load direction.
Figure 3.9 Strain in vertical rebar at column base.
Figure 3.10 DIC-based crack map of specimen SS-S at ultimate limit state: (a) positive load direction; and (b) negative load direction.
Figure 3.11 Diagonal crack in tie column below joint diagonal rebar in specimen SS-S.

Figure 3.12 Strain in horizontal bed-joint reinforcement in specimen SS-S (note that 5 out of 6 aluminum strips reached yield strain of 4000 μm/m). Positions of strain gauges are marked in Figure 3.2.
Figure 3.13 (a) Comparison between load-displacement envelopes of specimen SS and analytical model assuming substandard materials; and (b) comparison between load-displacement envelopes of specimen SS-S and analytical model assuming acceptable materials.
Figure 3.14 Shear strength estimation: comparison between load-displacement envelopes of specimen SS-S and analytical estimate of peak load.

Figure 3.15 Experimental verification of efficiency factor, $\eta$: $A_r$, $f_{ri}$, $\rho_{hr}$, and $f_{yr}$ are respectively, cross sectional area, stress, reinforcement ratio, and yield strength of horizontal reinforcement; and $A_T$ is the wall transverse cross sectional area.
Figure 3.16 Definition of equivalent energy dissipation ratio per NTCM (2004)
Figure 3.17 Normalized secant stiffness as function of drift ratio [used to evaluate Criterion A3 (b) (Table 3.6)].
Table 3.1 Summary of cyclic load test results.

<table>
<thead>
<tr>
<th>Specimen</th>
<th>‘Cracking’ state</th>
<th>‘Peak load’ state</th>
<th>‘Ultimate’ state</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>$d$ [mm]</td>
<td>$H$ [kN]</td>
<td>$d$ [mm]</td>
</tr>
<tr>
<td>SS</td>
<td>+1.7</td>
<td>+107</td>
<td>+8.3</td>
</tr>
<tr>
<td></td>
<td>-1.5</td>
<td>-118</td>
<td>-7.4</td>
</tr>
<tr>
<td>SS-S</td>
<td>+1.4</td>
<td>+125</td>
<td>+9.6</td>
</tr>
<tr>
<td></td>
<td>-1.7</td>
<td>-145</td>
<td>-15.0</td>
</tr>
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</table>

Table 3.2 Statistical characteristics of semi-empirical model by Riahi et al. (2009).

<table>
<thead>
<tr>
<th>Model parameters</th>
<th>Mean (experiment/model)</th>
<th>CV (experiment/model)</th>
<th>$R^2$</th>
<th>Equation</th>
</tr>
</thead>
<tbody>
<tr>
<td>$u_{cr}$</td>
<td>1.046</td>
<td>0.245</td>
<td>0.958</td>
<td>(3.1)</td>
</tr>
<tr>
<td>$u_p$</td>
<td>1.001</td>
<td>0.223</td>
<td>0.960</td>
<td>(3.3)</td>
</tr>
<tr>
<td>$\delta_{cr}$</td>
<td>1.065</td>
<td>0.303</td>
<td>0.887</td>
<td>(3.2)</td>
</tr>
<tr>
<td>$\delta_{p}$</td>
<td>1.015</td>
<td>0.237</td>
<td>0.701</td>
<td>(3.4)</td>
</tr>
<tr>
<td>$\delta_u$</td>
<td>0.937</td>
<td>0.189</td>
<td>0.801</td>
<td>(3.6)</td>
</tr>
</tbody>
</table>

Table 3.3 Summary of analytical load-displacement estimates assuming substandard material properties.

<table>
<thead>
<tr>
<th></th>
<th>‘Cracking’ state</th>
<th>‘Peak load’ state</th>
<th>‘Ultimate’ state</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>$d$ [mm]</td>
<td>$H$ [kN]</td>
<td>$d$ [mm]</td>
</tr>
<tr>
<td>Lower bound</td>
<td>1.6</td>
<td>75.0</td>
<td>8.3</td>
</tr>
<tr>
<td>Average</td>
<td>2.2</td>
<td>99.0</td>
<td>10.7</td>
</tr>
<tr>
<td>Upper bound</td>
<td>3.0</td>
<td>134.0</td>
<td>13.4</td>
</tr>
</tbody>
</table>
Table 3.4 Summary of analytical load-displacement estimates assuming acceptable material properties.

<table>
<thead>
<tr>
<th></th>
<th>'Cracking' state</th>
<th>'Peak load' state</th>
<th>'Ultimate' state</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>$d$ [mm]</td>
<td>$H$ [kN]</td>
<td>$d$ [mm]</td>
</tr>
<tr>
<td>Lower bound</td>
<td>2.1</td>
<td>135.0</td>
<td>7.1</td>
</tr>
<tr>
<td>Average</td>
<td>2.9</td>
<td>171.0</td>
<td>9.2</td>
</tr>
<tr>
<td>Upper bound</td>
<td>4.0</td>
<td>223.0</td>
<td>11.5</td>
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</table>

Table 3.5 Verification of Criteria A1 and A2 of NTCM (2004) for qualification of masonry walls as earthquake-resistant.

<table>
<thead>
<tr>
<th>Specimen</th>
<th>$R_{max}$ (kN)</th>
<th>$R$ (kN)</th>
<th>$\lambda R$ (kN)</th>
<th>Criterion A1(^{(1)})</th>
<th>Criterion A2(^{(1)})</th>
</tr>
</thead>
<tbody>
<tr>
<td>SS</td>
<td>152</td>
<td>124±30</td>
<td>161±39</td>
<td>S</td>
<td>S</td>
</tr>
<tr>
<td>SS-S</td>
<td>183</td>
<td>169±30</td>
<td>220±39</td>
<td>S</td>
<td>S</td>
</tr>
</tbody>
</table>

(1) S = Satisfied; N = Not Satisfied.

Table 3.6 Verification of Criterion A3 of NTCM (2004) for qualification of masonry walls as earthquake-resistant.

<table>
<thead>
<tr>
<th>Specimen</th>
<th>$R_1^{max}$ (kN)</th>
<th>$R_2$ (kN)</th>
<th>$0.8 R_1^{max}$ (kN)</th>
<th>$K/K_e^{(2)}$</th>
<th>EEDR</th>
<th>Criterion A3(^{(1)})</th>
</tr>
</thead>
<tbody>
<tr>
<td>SS</td>
<td>148</td>
<td>116</td>
<td>118</td>
<td>0.14</td>
<td>0.20</td>
<td>NS</td>
</tr>
<tr>
<td>SS-S</td>
<td>173</td>
<td>162</td>
<td>138</td>
<td>0.17</td>
<td>0.18</td>
<td>S</td>
</tr>
</tbody>
</table>

(1) S = Satisfied; NS = Not Satisfied.
(2) Refer to Figure 3.17.
ABSTRACT. In the aftermath of the 2010 Haiti earthquake, households were often reluctant to reoccupy their dwellings due to concerns about the safety of damaged structures. If feasible, structural repair (to complement to temporary sheltering) becomes a critical and realistic option for rapid reoccupancy as reconstruction poses greater barriers of cost and time. Considering the economic and technological limitations found in developing regions, this chapter addresses the question of whether it is feasible to repair a severely damaged substandard CM wall in a context-sensitive fashion, that is, using: (a) materials that are locally available and commonly used and often substandard (e.g., low-strength concrete and mortar); (b) materials that may be recycled from collapsed buildings to be reused for repair purposes; and (c) construction and installation practices that are familiar to local workers, without the need for additional training.

Two full-scale CM wall specimens were subjected to cyclic quasi-static in-plane load test till failure, and re-tested after repair by means of reinforced plaster made of low-strength mortar and inexpensive steel welded wire mesh. Both specimens were built with substandard materials (e.g., concrete with cylinder compressive strength in the range of 9 - 14 MPa). In addition, one of them was built with a substandard reinforcement layout (e.g., open stirrups with
relatively large on-center spacing). Three-dimensional digital image correlation was used to provide full-field strain maps to describe the load-resistance mechanisms and damage evolution.

The hysteretic in-plane load-displacement envelope of the repaired specimens was compared with that theoretically estimated for a benchmark CM wall built with acceptable materials (e.g., concrete with cylinder compressive strength of 26 MPa) and details (e.g., closed stirrups with a suitable on-center spacing). It is shown that the repaired CM walls exhibited substantially higher shear strength and deformability compared with those of the “standard” wall, and that the shear strength can be theoretically estimated. The seismic resistance of the repaired walls was verified based the acceptance criteria set forth in Mexico City Building Code Requirements for Masonry Structures (NTCM 2004). It is shown that only the repaired specimens qualify as earthquake-resistant structures. It is concluded that it is feasible to repair a highly-damaged substandard CM wall in a context-sensitive fashion and make it safe, comparable to a CM wall built with acceptable materials and details.

4.1 INTRODUCTION

In the aftermath of an earthquake, CM structures usually exhibit a broad range of damage state, from minor damage to total or partial collapse. The Haiti post-earthquake damage assessment survey reported by Lang and Marshall (2011) shows that almost 40% of structures with masonry walls experienced “moderate” to “very heavy” damage corresponding to Grade 2 to Grade 4 damage according to the European Macroseismic Scale 1998 (EMS-98)
(Grünthal 1998). Households were often reluctant to return home, due to different concerns, among which was safety of structures (USAID 2011). If feasible, structural repair (to complement to temporary sheltering) becomes a critical and realistic option for rapid reoccupancy as reconstruction poses greater barriers of cost and time. However, there is little knowledge and experience on whether repair can realistically improve the seismic behavior of a CM dwelling structure, especially using context-sensitive techniques with locally available (and often relatively low-quality) materials (Beunza and Eresta 2011). The overall goal of this chapter is to contribute to filling this gap by understanding whether it is feasible to repair a severely damaged substandard CM wall, using context-sensitive materials and practices, and make it safe, that is, offering a performance comparable to an undamaged counterpart built with acceptable-quality materials and seismic details.

An efficient means to repair damaged CM buildings is providing reinforced overlays on one or both faces of structural (shear) walls. Reinforcement can be in the form of: FRP layers bonded to the wall surface (ElGawady et al. 2007; Koutromanos et al. 2012; Santa-Maria and Alcaino 2011); reinforcing bars sprayed with mortar or concrete, i.e., shotcrete (Kahn 1984); multiple layers of mesh or fine rods embedded in cement mortar, i.e., ferrocement (Amanat et al. 2007); and steel welded wire mesh embedded in cement mortar, i.e., reinforced plaster (Alcocer et al. 1996; Ashraf et al. 2012). Alcocer et al. (1996) reported on the repair of a damaged two-story full-scale CM building specimen with steel welded wire mesh embedded in a 25-mm thick cement mortar overlay on the
exterior face of the specimen. The specimen was re-tested under cyclic in-plane loading up to failure. The maximum shear strength and the ultimate drift ratio in the repaired specimen were 1.3 and 2.6 times that for the original specimen, respectively. A more uniform distribution of inclined cracking was observed in the repaired specimen, highlighting the role of steel wire mesh in evenly distributing the cracks in the specimen. Ashraf et al. (2012) reported on the repair of a damaged CM wall with a 19 mm reinforced plaster and the re-test under in-plane cyclic lateral loads. The initial stiffness of the repaired wall was restored to its pre-damage condition and the maximum lateral strength was increased by 17%.

This chapter discusses the feasibility of repairing a severely damaged [i.e., Grade 4 per EMS-98 (Grünthal 1998)] substandard CM wall in a context-sensitive fashion. The repair aims at making the failed wall as safe as or safer than an undamaged counterpart built with adequate materials and details. The feasibility is evaluated based on the two ‘feasibility criteria’ (i.e., F1, F2) as defined in Chapter 1. Reinforced plaster consisting of steel welded wire mesh embedded in low-strength cement mortar was applied on both faces of failed walls. This repair technique enlists accessible and familiar materials without requiring highly-trained workers. In fact, this strategy responds to the financial and technological limitations that are often encountered in developing regions, which is necessary for a truly context-sensitive practice that aims to empower local practitioners to become self-reliant and train future generations (Kijewski-Correa and Taflanidis 2012; Mix et al. 2011).
The proof of concept was demonstrated based on evidence from cyclic in-plane tests on four full-scale CM walls. Full-field strain and crack maps based on three-dimensional digital image correlation (3D-DIC) measurements were used to visualize and describe the load-resistance mechanisms and failure modes. The strength and deformability of the repaired specimens were compared with those of a benchmark CM wall built with acceptable materials and details, which were estimated using an existing and well-validated semi-empirical model (Riahi et al. 2009). Finally, the earthquake resistance of the control and repaired CM walls were then verified based on the criteria set forth in the Mexican guidelines for masonry structures (NTCM 2004).

4.2 EXPERIMENTAL PROGRAM

4.2.1 Specimens and materials

The test matrix included four full-scale CM wall specimens (SS, SA, SS-P and SA-P). A comprehensive description of specimens SS and SA can be found in Chapter 2. Salient properties of the materials used are summarized in Table 4.1 (see Appendix A). The mix design of CMUs, concrete and mortar was adjusted to reach a relatively low compressive strength, similar to that occurs in substandard CM construction.

Specimens SS and SA were first tested up to failure, then repaired and re-tested as specimens SS-P and SA-P (where ‘P’ stands for ‘plastered’), respectively. As illustrated in Figure 4.1, the repair technique consists of bonding a layer of Type N mortar reinforced with steel welded wire mesh (SWWM) on either side of a given control specimen after failure. For the mortar overlays to
work compositely with the existing structural walls, it is essential to facilitate the transferring of shear forces to the overlays by means of fasteners or shear keys (Alcocer et al. 2006).

Therefore, the control specimens (SS and SA) were repaired in two steps:

1. The crushed and spalled parts of the tie columns and beam were removed and replaced with concrete with similar strength. The spalled part of the masonry wall was also repaired with new blocks [Figure 4.2(a)], and the largest cracks in the wall were filled with block debris and mortar [Figure 4.2(b)]. In the case of specimen SS, it was necessary to strengthen the substandard beam-column joints to avoid premature opening of the joint. Ø10 mm aluminum bars with 90° bends at the ends were inserted in four grooves (two per side of wall) that were cut diagonally in each corner, and filled with Type I OPC mortar [Figure 4.2(c)]. It is noted that while aluminum bars were used for the sole purpose of testing the concept using a low-stiffness material, a smaller amount of steel bars may be used instead.

2. The specimen surfaces were bush-hammered to facilitate bonding with the reinforced mortar plasters, cleaned, and moistened. Finally, the overlays were applied. The SWWMs consisted of galvanized steel wires with nominal diameter of 2.0 mm and spaced at 50 and 100 mm (\(\rho_s = 0.12 \%\) and 0.24 \%) in the horizontal and vertical directions, respectively. The SWWMs were connected together and secured by metallic anchors placed through the wall in holes perforated with hand drills [Figure 4.1 and Figure 4.3(a)]. The anchor density was 8/m². The reinforcing meshes were kept at 12.5 mm from the face of the wall.
using steel spacers. Mortar plasters, with similar mix design as the Type N mortar used for construction of the masonry panels, were on average 25 mm thick and were applied on both wall faces [Figure 4.3(b)].

The thickness of the mortar plaster was calculated to restore the lateral stiffness of the walls. The lateral stiffness of structural walls is an important parameter since the period of vibration of the structure depends on the stiffness of the wall and the seismic shear forces are distributed among the walls according to their lateral stiffness. In order to preserve the position of center of rigidity and avoid soft story effects in future earthquakes, an ideal repair technique should restore the wall stiffness. However, for practical reasons, the effective stiffness of the wall, $K_e$, which defines the slope of the first branch of the idealized in-plane load-horizontal displacement envelope, is used (Tomaževič and Klemenc 1997). It should be noted that the effective stiffness of the repaired walls is independent of amount of reinforcement (Aguilar et al. 1996; Alcocer et al. 1996). If the residual stiffness of the failed specimens SS and SA, which is about 10 % of the effective stiffness, is neglected, then $K_e$ for the repaired walls depends on the dimensions and mechanical properties of the mortar plaster and boundary conditions. Based on the theory of elasticity, $K_e$ is calculated as:

$$K_e = \left( \frac{h^3}{3E_p I_p} + \frac{kh}{G_p A_p} \right)^{-1} \quad (4.1)$$

which takes into account both the flexural and shear stiffness contributions (Tomaževič 2006). In this equation, $h$ is the height of the wall, $\kappa$ is the shear coefficient for rectangular cross section equal to 1.2, $E_p$ and $G_p$ are the modulus
of elasticity and shear modulus of the plaster, respectively, and \( I_p \) and \( A_p \) are the moment of inertia and area of the plaster's horizontal cross section, respectively. Based on the effective stiffness of the control specimens, a target effective stiffness equal to 80 kN/mm was chosen for the repaired walls and the thickness of the plaster was calculated equal to 25 mm.

4.2.2 Test setup and measurement systems

4.2.2.1 Loading apparatus and protocol

The in-plane load test setup and protocol were similar to those used for load-testing of the control (SS and SA) and strengthened specimens (SS-S and SA-S) as illustrated in Figure 3.5 and Figure 2.5, respectively.

4.2.2.2 Measurement systems

Similar to the tests on the control and strengthened specimens, two measurement systems were used to monitor the response of the repaired specimens:

1. A conventional system including PWSs mounted on one face of each specimen and strain gauges, as illustrated in Figure 3.6; and

2. A 3D-DIC system as described in section 2.4.2.3 and illustrated in Figure 2.7, on the other face of the specimens.

4.3 RESULTS AND DISCUSSION

The in-plane load-displacement response for the control and repaired specimens are presented in Figure 4.4 and Figure 4.5, respectively. The positive and negative load-displacement values at cracking, peak load and ultimate limit states are summarized in Table 4.2.
First, the strength and failure modes of the specimens are described. Second, the strength and deformability of the repaired specimens are assessed by comparing them to those of a CM wall with acceptable materials and details, and estimated using a well-validated model. Third, it is shown that shear strength of the repaired specimens can be theoretically estimated. Fourth, the seismic performance of the specimens are assessed according to NTCM (2004).

4.3.1 Strength and failure modes

4.3.1.1 Control specimens (SS and SA)

The strength and failure mode of specimen SS are discussed in Section 3.3.1.1. The peak load for specimen SA was 179 kN and was attained at an in-plane displacement of 11.6 mm. The ultimate displacement was 17.7 mm. Figure 4.6(a) and Figure 4.6(b) illustrate the DIC-based crack maps for specimen SA at the ultimate limit state, in the positive and negative load directions, respectively. The adequately detailed beam-column joints did not open, however cracks propagating into the joints were identified in DIC-based crack maps. In fact as was shown in Chapter 2, the adequate detailing of the joints enabled the development of a compression strut in the masonry which resulted in increased strength, deformability and energy dissipation compared with specimen SS. Spalling of the masonry occurred mainly near the center of the masonry panel. The specimen showed gradual stiffness degradation after reaching the peak load. Yielding of the horizontal and vertical reinforcement in the beam-column joints [e.g., Figure 4.7(a)] and column bases provided an important contribution to energy dissipation. Compared with specimen SS, the adequate joint detailing
resulted in a maximum increase in peak strength and ultimate displacement of 21 and 79 %, respectively (Table 4.2).

4.3.1.2 Repaired specimens (SS-P and SA-P)

Specimen SS-P attained a peak load of 271 kN at an in-plane displacement of 14.9 mm. The ultimate displacement was 30.2 mm. Compared with specimen SS, the repair resulted in a maximum increase in peak strength and ultimate displacement of 83 and 196 %, respectively. The effective stiffness of SS-P specimen was evaluated from the envelope curves (Figure 4.8) to be 82 kN/mm and 75 kN/mm in the positive and negative directions, respectively. In fact, the repair technique restored the effective stiffness of the damaged wall to the design value, 80 kN/mm. The DIC measurements showed that the slip at the wall-footing interface started to increase from the beginning of the test and reached 13 and 36 mm in the positive and negative directions, respectively. In fact, since the stirrups were not sufficiently close to each other and bent over 90° at the base, where there was the maximum shear force on the columns, the vertical rebars yielded. In order to observe the damage in the CM wall, some parts of the plaster were removed. The tie columns and masonry at the base were crushed as shown in Figure 4.9. The diagonal bars used to strengthen the beam-column joints remained in the elastic range highlighting the effectiveness of the joint strengthening technique [Figure 4.7(b)].

Specimen SA-P reached a peak load of 257 kN at an in-plane displacement of 12.7 mm. The ultimate displacement was 24.0 mm. Compared with specimen SA, the repair technique resulted in a maximum increase in peak
strength and ultimate displacement of 49 and 36% respectively. The effective stiffness of specimen SA-P was evaluated 80 kN/mm and 75 kN/mm in the positive and negative directions (Figure 4.8), respectively, comparable with that of specimen SS-P and the design value (80 kN/mm). Unlike specimen SS-P, the well-confined tie column-footing connections did not fail till the end of the test, highlighting the role of closely spaced stirrups with 135° bends in increasing the shear strength of tie columns.

Figure 4.10 and Figure 4.11 present the DIC-based crack maps for specimens SS-P and SA-P, respectively, at the ultimate limit state (Figure 2.6). The crack pattern of the repaired specimens was characterized by well-distributed inclined cracks. Unlike specimens SS and SA, no clear critical first crack was observed. The stiffness degradation of the specimens was gradual since the original CM walls were already cracked and the SWWMs helped distribute the cracks gradually over the specimens. Figure 4.12 shows similar crack pattern developed in the masonry panel and the reinforced plaster, indicating that good connection was ensured. The connection was created by the anchors placed through the wall and the bond between the roughened surface of the masonry panel and reinforced plaster. This mechanism formed a composite reinforced plaster-masonry section with higher tensile and compressive strength in the specimens. The contribution of the well-connected SWWM in resisting in tension (in both the horizontal and vertical direction) was key to increase the shear resistance of the wall by offsetting the growth and widening of the shear cracks, and distributing the cracks over a significantly larger portion of masonry
panel compared to the control specimens. In addition, in both the repaired specimens, the two layers of well-connected reinforced plaster formed a robust cover around the damaged wall and prevented falling of masonry, highlighting that the repair eliminates the risk of post-hazard collapse and falling of debris.

4.3.2 Strength and deformability assessment – Criterion F1

The strength and deformability of the repaired specimens were assessed by comparing their load-displacement envelopes with that of the benchmark CM wall built with acceptable materials and details, and estimated using the model by Riahi et al. (2009) presented in Chapter 3.

The results from the model are first compared with those from specimens SS and SA. Figure 4.13(a) shows the envelopes of load-displacement curves for specimens SS and SA and the analytical model in which substandard material properties were incorporated. The model average estimate of load and displacement for each limit state and their variability (in the form of error bars) are plotted. The results showed that the peak and ultimate load and the associated in-plane displacements of specimen SA are in good agreement with the upper limit of the analytical model. This is likely due to the development of an effective confining mechanism. While the peak and ultimate load of specimen SS were in agreement with the upper limit of the model, the associated displacements did not reach the predicted values due to premature opening of the joint, which impaired the ability of the wall to undergo plastic deformations without collapsing.
The results from the model in which acceptable material properties were incorporated were then compared with the load-displacement envelopes of specimens SS-P and SA-P, as illustrated in Figure 4.13(b). The results show that the strength and deformability of the repaired specimens at the peak and ultimate limit state are well above the upper limit of the analytical model assuming acceptable materials. These results support the hypothesis that, through a suitable context-sensitive repair, it is possible to transform a failed substandard CM wall into a wall with higher strength and deformability than those of a standard CM wall built with quality materials. Based on these results, Criterion F1 of the ‘feasibility criteria’ is met for the repaired specimens.

4.3.3 Shear strength estimation

The total shear strength of the repaired specimens was calculated by the following equation:

\[ V_{tot} = V_{CMA} + V_p + V_r \]  \hspace{1cm} (4.2)

where: \( V_{CMA} \) is the average peak load of the analytical model with substandard materials, equal to 124 kN (Table 3.3); \( V_p \) is the shear strength of the mortar layer calculated using conventional ACI 318-11 (ACI 2011) relation:

\[ V_p = 0.17 \sqrt{f_p} \cdot 2t_p \cdot l \]  \hspace{1cm} (4.3)

equal to 60 kN; where \( f_p \) and \( t_p \) are the compressive strength and thickness of mortar plaster, respectively, and \( l \) is the length of the wall. \( V_r \) is the SWWM contribution calculated as

\[ V_r = \eta f_{yr} A_r \]  \hspace{1cm} (4.4)
equal to 50 kN; where \( f_y \) and \( A_r \) are the yield strength and cross sectional area of the horizontal (SWWM) reinforcement, respectively, and \( \eta \) is an efficiency factor equal to the ratio of the force resisted by the SWWM to that associated with yielding of the horizontal SWWM reinforcement. In the absence of data on the force resisted by the SWWM, an efficiency factor of 0.6 was used based on the results of tests performed on retrofitted CM walls using SWWM embedded in cement mortar (Alcocer et al. 1996). It is noted that since the failed specimens were partially repaired before plastering, the average peak load of the analytical model, instead of the upper limit of peak load, was used as a reasonable estimate for the strength contribution of the CM wall.

The estimate for the shear strength of specimens SS-P and SA-P is marked (with a solid line) in Figure 4.14 and compared with the load-displacement envelopes of the specimens. In order to determine a conservative lower-limit estimate suitable for design purposes, the lower limit for the peak load of the analytical model, equal to 94 kN (Table 3.3), was added to the contribution of the reinforced plaster \( (V_p + V_r) \) and marked with a dashed line in Figure 4.14. It is shown that: (1) the strength estimate \( (V_{tot}) \) is in good agreement with the experimental peak strength; and (2) the lower limit strength is a conservative estimate that may be considered for design purposes.

4.3.4 Earthquake-resistance assessment – Criterion F2

In order to show if the repaired specimens are qualified as earthquake-resistant structures, seismic design criteria given by Mexico City Building Code (MCBC) Requirements for Design and Construction of Masonry Structures
(NTCM 2004), were used. The details of the criteria can be found in Section 3.3.4. Calculation of the peak-to-peak secant stiffness, $K / K_e$, for each specimen is illustrated in Figure 4.15.

The criteria were verified based on the cyclic tests on the control and repaired specimens, as summarized in Table 4.3. According to the criteria, specimen SS did not satisfy the first requirement of Criterion A3 due to premature opening of the joint, which resulted in a sudden drop in the shear strength. Specimen SA also did not qualify as earthquake-resistant because it did not satisfy the requirement related to energy dissipation [Criterion A3 (c)]. Finally, specimens SS-P and SA-P satisfied all the criteria and qualified as earthquake-resistant structures according to NTCM (2004). Based on these results, Criterion F2 of the ‘feasibility criteria’ is met for the repaired specimens.

4.4 CONCLUSIONS

This chapter discusses the feasibility of repairing highly-damaged substandard CM walls in a context-sensitive manner (i.e., with locally available and accessible – and often substandard – materials, and using construction and installation practices that are familiar to local workers), thereby providing strength and ductility comparable to those of CM walls built with acceptable materials and details. The following conclusions are drawn:

1. Compared with the evidence from the control specimens, the repair technique resulted in a maximum increase in peak strength and ultimate displacement of 83 and 196 %, in specimen SS and 49 and 36 % in specimen SA, respectively.
2. It was shown that premature opening of a beam-column joint in specimen SS hindered the development of the necessary confining mechanism for the formation of effective compression strut in the masonry. Conversely, improving the beam-column joint detailing in specimen SA resulted in the formation of an effective compression strut in the masonry, with an increase in shear strength and ductility.

3. The anchors in the repaired specimens were key to form a composite reinforced plaster-masonry section with higher tensile and compressive strength, and the welded wire mesh helped distribute the cracks over the specimens.

4. The repair technique dramatically reduces the risk of post-hazard collapse and falling of debris. The two layers of reinforced plaster formed a robust cover around the damaged walls and prevented falling of masonry.

5. The strength and ductility of the repaired specimens were assessed by comparing them to those of a CM wall built with acceptable materials and details, and estimated by a well-validated semi-empirical model (Riahi et al. 2009). It is shown that the seismic repair transformed a failed substandard CM wall into a wall with higher strength and deformability than a standard wall built with quality materials. Based on these results, Criterion F1 of the ‘feasibility criteria’ was met for the repaired specimens.

6. The shear strength of the repaired specimens was estimated analytically by adding the contribution of reinforced plaster to the theoretical estimate of average peak load. Adding the shear strength contribution of the
reinforced plaster to the lower limit of the peak load estimate provides a conservative value that may be considered for design purposes.

7. Earthquake-resistance performance of the control and repaired specimens was assessed according to the criteria set forth in Mexico City Building Code (MCBC) Requirements for Masonry Structures (NTCM 2004). The results showed that while specimens SS and SA did not qualify as earthquake-resistant, the repaired specimens qualified as earthquake-resistant structures. Based on these results, Criterion F2 of the ‘feasibility criteria’ was met for the repaired specimens.

8. The two ‘feasibility criteria’ were evaluated for the repaired specimens. Based on the criteria, it is shown that it is feasible to repair a failed substandard CM wall in a context-sensitive manner, and make it safer than an undamaged CM wall built with acceptable materials and details. While the repair technique is shown to be effective for highly damaged [i.e., Grade 4 per EMS-98 (Grünthal 1998)] masonry walls, it is reasonably hypothesized that it can be effective also for less damaged walls (Grade 2 and 3).

4.5 REFERENCES

ACI (2011). "Building code requirements for structural concrete (ACI 318-11) and commentary." American Concrete Institute, Farmington Hills, MI.


Proc., 11th World Conference on Earthquake Engineering, Acapulco, Mexico, Paper No. 1380.


Figure 4.1 Schematic of repair technique using SWWM embedded in mortar cement.
Figure 4.2 Practice of repair technique (step 1): (a) replacing damaged parts of masonry; (b) filling cracks with mortar; and (c) strengthening of beam-column joint using aluminum bars.
Figure 4.3 Practice of repair technique (step 2): (a) connecting SWWMs with anchors placed through wall; and (b) covering wall and SWWM with cement mortar.
Figure 4.4 Load-displacement response of control specimens: (a) SS; and (b) SA.
Figure 4.5 Load-displacement response of repaired specimens: (a) SS-P; and (b) SA-P.
Figure 4.6 DIC-based crack map of specimen SA at ultimate limit state: (a) positive load direction; and (b) negative load direction.
Figure 4.7 Strain in reinforcement at beam-column joint: (a) specimen SA, vertical rebar; and (b) specimen SS-P, diagonal bar.
Figure 4.8 Load-displacement envelopes.

Figure 4.9 Damage at column base and masonry in specimen SS-P.
Figure 4.10 DIC-based crack map of specimen SS-P at ultimate limit state: (a) positive load direction; and (b) negative load direction.
Figure 4.11 DIC-based crack map of specimen SA-P at ultimate limit state: (a) positive load direction; and (b) negative load direction.
Figure 4.12 similar crack patterns in reinforced plaster and masonry.
Figure 4.13 (a) Comparison between load-displacement envelopes of specimens SS and SA and analytical model assuming substandard materials; and (b) comparison between load-displacement envelopes of specimens SS-P and SA-P and analytical model assuming acceptable materials.
Figure 4.14 Shear strength estimation: comparison between load-displacement envelopes of specimens SS-P and SA-P and analytical estimate of peak load.
Figure 4.15 Normalized secant stiffness as function of drift ratio [used to evaluate Criterion A3 (b) (Table 4.3)].
Table 4.1 Material strength properties.

<table>
<thead>
<tr>
<th>Property</th>
<th>Test standard</th>
<th>Number of specimens</th>
<th>Average [MPa]</th>
<th>Standard deviation [MPa]</th>
</tr>
</thead>
<tbody>
<tr>
<td>Concrete masonry units (CMU)</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Compressive strength</td>
<td>ASTM C140</td>
<td>6</td>
<td>7.2</td>
<td>0.30</td>
</tr>
<tr>
<td>Type N mortar</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Compressive strength</td>
<td>ASTM C780</td>
<td>8</td>
<td>8.3</td>
<td>0.88</td>
</tr>
<tr>
<td>Masonry (CMU and mortar assemblies)</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Compressive strength</td>
<td>ASTM C1314</td>
<td>3</td>
<td>5.8</td>
<td>0.43</td>
</tr>
<tr>
<td>Shear strength</td>
<td>ASTM E519</td>
<td>3</td>
<td>0.3</td>
<td>0.02</td>
</tr>
<tr>
<td>Ordinary Portland cement concrete</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Compressive strength</td>
<td>ASTM C39</td>
<td>15</td>
<td>12.1</td>
<td>1.66</td>
</tr>
<tr>
<td>Splitting tensile strength</td>
<td>ASTM C496</td>
<td>8</td>
<td>1.2</td>
<td>0.13</td>
</tr>
<tr>
<td>Steel welded wire mesh (SWWM)</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Tensile strength</td>
<td>ASTM A185</td>
<td>3</td>
<td>525</td>
<td>1.30</td>
</tr>
</tbody>
</table>

Table 4.2 Summary of cyclic load test results.

<table>
<thead>
<tr>
<th>Specimen</th>
<th>‘Cracking’ state</th>
<th>‘Peak load’ state</th>
<th>‘Ultimate’ state</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>(d) [mm]</td>
<td>(H) [kN]</td>
<td>(d) [mm]</td>
</tr>
<tr>
<td>SS</td>
<td>+1.7 -1.5</td>
<td>+107 -118</td>
<td>+8.3 -7.4</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>+11.0 -10.2</td>
</tr>
<tr>
<td>SA</td>
<td>+1.4 -1.8</td>
<td>+132 -135</td>
<td>+11.6 -13.7</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>+18.3 -17.7</td>
</tr>
<tr>
<td>SS-P</td>
<td>_ (1)</td>
<td>+12.2 -14.9</td>
<td>+258 -271</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>+30.2 -23.8</td>
</tr>
<tr>
<td>SA-P</td>
<td>_ (1)</td>
<td>+12.7 -14.0</td>
<td>+257 -255</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>+21.2 -24.0</td>
</tr>
</tbody>
</table>

(1) Due to gradual propagation of cracks, no clear first crack was observed.
Table 4.3 Verification of NTCM (2004) criteria for qualification of masonry walls as earthquake-resistant.

<table>
<thead>
<tr>
<th>Specimen</th>
<th>$R_{max}$ (kN)</th>
<th>$R$ (kN)</th>
<th>$\Delta R$ (kN)</th>
<th>Criterion A1&lt;sup&gt;(1)&lt;/sup&gt;</th>
<th>Criterion A2&lt;sup&gt;(1)&lt;/sup&gt;</th>
<th>$R'_{max}$ (kN)</th>
<th>$R_2$ (kN)</th>
<th>$0.8 R'_{max}$ (kN)</th>
<th>$K/K_e$&lt;sup&gt;(2)&lt;/sup&gt;</th>
<th>EEDR</th>
<th>Criterion A3&lt;sup&gt;(1)&lt;/sup&gt;</th>
</tr>
</thead>
<tbody>
<tr>
<td>SS</td>
<td>152</td>
<td>124±30</td>
<td>161±39</td>
<td>S</td>
<td>S</td>
<td>148</td>
<td>116</td>
<td>118</td>
<td>0.14</td>
<td>0.20</td>
<td>NS</td>
</tr>
<tr>
<td>SA</td>
<td>173</td>
<td>124±30</td>
<td>161±39</td>
<td>S</td>
<td>S</td>
<td>172</td>
<td>163</td>
<td>138</td>
<td>0.18</td>
<td>0.13</td>
<td>S S NS</td>
</tr>
<tr>
<td>SS-P</td>
<td>258</td>
<td>234</td>
<td>304</td>
<td>S</td>
<td>S</td>
<td>271</td>
<td>258</td>
<td>217</td>
<td>0.34</td>
<td>0.17</td>
<td>S S S</td>
</tr>
<tr>
<td>SA-P</td>
<td>249</td>
<td>234</td>
<td>304</td>
<td>S</td>
<td>S</td>
<td>255</td>
<td>249</td>
<td>204</td>
<td>0.34</td>
<td>0.20</td>
<td>S S S</td>
</tr>
</tbody>
</table>

(1) S=Satisfied; NS = Not Satisfied.
(2) Refer to Figure 4.15.
CHAPTER 5
CONCLUSIONS AND RECOMMENDATIONS FOR FUTURE WORK

The main novel contribution of this study is offered by:

- Using context-sensitive materials and practices for the seismic strengthening and repair of highly substandard CM walls, which are often encountered in developing regions.

- Demonstrating proof of concept that it is feasible to either strengthen or repair substandard CM walls using locally accessible materials and simple construction methods, transforming strength and ductility to the point where they are comparable to those of undamaged CM walls built with acceptable-quality materials and details. In fact, strengthening resulted in a maximum increase in peak strength and deformability of 35 and 106%, respectively; and repair resulted in a maximum increase in peak strength and deformability up to 83 and 196%, respectively.

Context-sensitive materials are locally available and commonly used, and thus relatively affordable (albeit sometimes substandard). They may also be recycled from collapsed buildings to be reused for strengthening and repair purposes. For example: low-strength CMUs and concrete may be used for repair purposes (DesRoches et al. 2011), together with steel rebars recycled from collapsed building. Context-sensitive retrofitting practices entail construction and installation operations that can be performed following practices familiar to local
workers (e.g., embedding metallic strips in mortar along bed joints or applying mortar plaster on wall faces), without the need for additional training.

Another novel contribution of this study is offered by the demonstration of a three-dimensional digital image correlation (3D-DIC) method for the non-contacting full-field measurement and visualization of deformations on large-size concrete and masonry specimens in a laboratory environment. A correct implementation of this method enables measurements with comparable accuracy to traditional point-wise, contact-based sensors, providing new information to describe load-resisting mechanisms and the progression of damage until failure.

Feasibility is assessed based on two criteria drawing from evidence gained through in-plane cyclic tests on representative full-scale CM walls:

F1. Obtaining comparable or better strength and deformability than those of a CM wall with acceptable-quality materials and details.

F2. Satisfying the earthquake-resistance criteria specified in the Mexico City Building Code (MCBC) Requirements for Design and Construction of Masonry Structures (NTCM 2004). In Mexico, 70 % of buildings include structural masonry walls and CM is the most popular masonry construction system (Alcocer et al. 2003). In addition, masonry construction in Mexico is code-regulated since 1976 and the seismic provisions for masonry structures were developed from results of a comprehensive research program of over 20 years and were updated after the 1985 Mexico City earthquake (Alcocer and Meli 1995).
Supporting experimental evidence is based on in-plane cyclic tests on six full-scale CM wall specimens, including control, strengthened and repaired specimens.

The main conclusions drawn from the studies in chapters 2 through 4, and related recommendations for future work, are summarized as follows:

1. 3D-DIC measurements of story drift and diagonal deformation offer comparable accuracy to that of surface-mounted PWSs. To the best of the writers’ knowledge, this study is the first to systematically assess 3D-DIC measurements on large-scale civil engineering structures vis-à-vis benchmark PWS measurements.

Further research is necessary to test the hypothesis that hysteretic load-drift responses can be traced based on 3D-DIC measurements with comparable accuracy to PWSs, and without the need to introduce constant-displacement plateaus in the displacement-control loading history. To increase the accuracy of drift measurements, larger speckles (up to 10 pixels) can be applied by manual painting on the specimen surface where the drift is needed to be measured (e.g., top of the tie beam at midspan).

The application of 3D-DIC to measure deformations on large-scale structures under dynamic (e.g., seismic, wind) loads may also be explored. In dynamics, acquiring images of a moving target causes a motion effect (i.e., blurring). This is due to the fact that during the exposure time (effective duration that a camera’s shutter is open), the target slightly displaces. This factor is an
important source of uncertainty that needs to be quantified (Mazzoleni et al. 2015; Zappa et al. 2014).

2. Specific strain components can be rendered in 3D-DIC maps to visualize load-resistance mechanisms and failure modes. By using diagonal strain maps, experimental evidence of the development of diagonal struts in CM walls was presented for the control, strengthened and repaired specimens with different in-plane strength and deformability. To the best of the writers’ knowledge, this study is the first to present the experimental full-field visualization of strut-and-tie mechanisms in masonry infills.

3. Faithful crack maps can be obtained based on 3D-DIC maximum principal strain maps. In addition, cracks can be accurately located and their progressive opening can be monitored based on 3D-DIC displacement measurements.

Further research is necessary to test the hypothesis that the amplitude of discontinuities in full-field displacement maps can be used to determine crack opening displacements (CODs) on large-scale structures. In previous studies, discontinuities in full-field strain maps have been used to measure CODs in relatively small specimens (Alam et al. 2010; Lin and Labuz 2013; Mekky and Nicholson 2006; Nunes and Reis 2012), or on relatively small regions of interest (ROIs) in large-scale specimens (Destrebecq et al. 2011). However, no benchmark PWSs (e.g., crack opening gauges) were used to verify the DIC measurements. Experiments where progressive crack openings are locally measured with benchmark PWSs are needed to test if and how the amplitude of
the discontinuities in a given displacement profile can be used to accurately estimate CODs in large-scale specimens.

4. The fact that 3D-DIC measurements attained a comparable accuracy to PWS measurements and faithful 3D-DIC-based crack maps were obtained indicates that the DIC setup and analysis approach were effective in meeting the challenges posed by the large measurement surfaces.

5. An efficient reinforcement layout was used to strengthen a substandard CM wall. Metallic strips were embedded in six (out of 10) bed joints in the vicinity of the mid-height section of the wall. The efficiency factor for strength contribution of the strips in shear strength was 68%, which is similar to that recommended by Alcocer (1996) for CM walls. Adding reinforcement in the bed joints closer to the RC tie beam and the footing may not significantly contribute to the shear strength.

More experiments and research are necessary to find an optimized reinforcement ratio \( \rho_{hr} \) and layout for the particular type of reinforcement used. It is noted that the efficiency factor, \( \eta \), for contribution of horizontal reinforcement in shear strength is inversely proportional to \( \rho_{hr} f_{yr} \), in which \( f_{yr} \) is the yield strength of horizontal reinforcement (Aguilar et al. 1996; Alcocer and Zepeda 1999). The higher \( \rho_{hr} f_{yr} \), the higher are the loads and also the deformations needed to engage the horizontal reinforcement. However, the attainment of large \( \eta \) values at high load levels may not occur due to crushing of the masonry, that is, with a different failure mode that does not entail yielding of the horizontal reinforcement. When using an optimized \( \rho_{hr} \), further increasing the reinforcement does not help
to increase the shear strength, irrespective of the failure mode.

6. The strength and deformability of the retrofitted specimens were assessed by comparing them with those of a CM wall built with acceptable materials and details, which were estimated using a well-validated semi-empirical analytical model (Riahi et al. 2009). It is shown that strengthening and repair enable the transformation of a substandard CM wall into a masonry wall with comparable strength and deformability to a wall built with materials having acceptable-quality materials and details. Based on these results, Criterion F1 of the ‘feasibility criteria’ was met for the retrofitted specimens.

7. The shear strength of the strengthened and repaired specimens was successfully estimated by adding, respectively, the contribution of horizontal reinforcement and reinforced plaster, to the shear strength estimate of the CM wall. It is shown that the average peak load estimate is in good agreement with the experimental peak strength, while the lower-bound estimate yields a conservative value that may be used for design purposes.

8. Earthquake-resistance performance of the control, strengthened and repaired specimens was assessed according to the criteria set forth in NTCM (2004). It is shown that only the strengthened and repaired specimens qualify as earthquake-resistant structures. Based on these results, Criterion F2 of the ‘feasibility criteria’ was met for the retrofitted specimens.

9. The two ‘feasibility criteria’ were evaluated for the retrofitted specimens. Based on the results, it is shown that it is feasible to strengthen or repair a substandard CM wall, using context-sensitive materials and construction
practices, and make it safe, that is, comparable to a CM wall built with acceptable-quality materials and details, and compliant with earthquake-resistance criteria per NTCM (2004).

It is noted that the seismic performance of the specimens and the feasibility of retrofit were assessed based on in-plane cyclic quasi-static tests on isolated single CM wall specimens. The seismic performance of a CM building consisting of several walls and slabs, depends also on the performance of slabs in distributing the seismic loads among structural walls, and connections (e.g., slab to tie-beam and tie beam connections at wall intersections) that transfer seismic loads to the walls and foundation. In addition, dynamic loads (e.g., shake-table tests) are more representative for assessing the seismic performance of CM buildings than quasi-static loads. More research is necessary to assess the seismic performance of CM structures, based on dynamic (i.e., shake-table) tests on full-scale partial or complete one or two-story CM building structures. Performing nonlinear dynamic analyses using robust finite element (FE) models is recommended for the preliminary assessment of seismic performance of CM structures, and to inform the design of test matrices for large-scale static and dynamic experiments to gain hard evidence.

There are two major approaches for the FE analysis of masonry structures: micro-modeling and macro-modeling. In the micro-modeling approach, masonry constituents (masonry units and mortar joints) are modeled separately. This approach can be subdivided into detailed micro-modeling in which units and mortar joints are represented by continuum elements and
contact surfaces between units and mortar by interface elements, and simplified micro-modeling, in which expanded units are represented by continuum elements and nonlinear behavior of mortar joints and contact surfaces is collapsed into surface elements (Lotfi and Shing 1994; Lourenço 1996). In the macro-modeling approach, the whole structure is schematized as a homogenized continuum without any distinction between masonry constituents and mechanical properties of the masonry may be derived from experimental data. Micro-modeling strategy is more detailed and facilitates understanding the local behavior of masonry. Conversely, it is not suitable for simulating the global behavior of buildings because of its computational cost. The macro-modeling approach is less detailed and is suitable for large structures, thus becoming more attractive for practice-oriented analyses.

Considering the similarity between mechanical properties of masonry units and mortar in substandard CM and assuming a homogenized material property for masonry, a macro-modeling approach appears reasonable for FE modeling of the CM buildings. The use of macro-models requires coarser mesh and hence leads to less computationally-expensive numerical solutions compared with more detailed micro-models, making it more attractive for modeling complete structures. Verification and calibration of the FE macro-models can be based on the results from material characterization tests and cyclic tests on CM walls. Nonlinear cyclic quasi-static analyses can be performed to calibrate the hysteresis parameters related to ductility, strength degradation and stiffness degradation. In order to support the calibration of macro-models, nonlinear cyclic
analyses on a micro-model of a single CM wall can be used. By performing nonlinear dynamic analyses on the developed FE macro-models, seismic performance of the control and retrofitted specimens can be investigated. In addition, the macro-models can be used to investigate more in depth the effect of different design parameters and details related to CM, such as tie column reinforcement ratio, toothing at tie column-masonry interface and beam-column joint details.

REFERENCES


APPENDIX A – MATERIAL CHARACTERIZATION TESTS

This appendix covers the material characterization tests on concrete, mortar, concrete masonry units (CMUs), masonry, aluminum strips, steel rebars and steel wires. The thickness of bed and head joints for masonry specimens was 10 mm approximately.

A.1 CONCRETE

A.1.1 Compression test on concrete cylinders

Fifteen concrete cylinders (102×203 mm) were sampled from wall specimens and instrumented with four 5 mm linear displacement transducers and one 100 kN load cell to obtain the compressive strength (ASTM C39) and modulus of elasticity (ASTM C469) of the concrete used for tie columns and beams (Figure A.1). Table A.1 summarizes the compression test results on concrete cylinders. The average compressive strength, standard deviation (SD), and coefficient of variation (CV) were 12.1 MPa, 1.7 MPa and 13.7 %, respectively. The average displacement measurement from four transducers and the measured load were used to obtain the strain-stress curve for specimens [Figure A.2(a)]. As illustrated in Figure A.2(b), the secant stiffness between 10 % and 40 % of the peak stress was used to calculate the modulus of elasticity. The average modulus of elasticity and standard deviation were 12.9 GPa and 1.7 GPa, respectively.
Figure A.1 Test setup for compression test on concrete cylinders.
Table A.1 Summary of compression test results on concrete cylinders.

<table>
<thead>
<tr>
<th>Specimen</th>
<th>Compressive strength [MPa]</th>
<th>Modulus of elasticity [GPa]</th>
</tr>
</thead>
<tbody>
<tr>
<td>Specimen SS</td>
<td></td>
<td></td>
</tr>
<tr>
<td>SS-1</td>
<td>9.13</td>
<td>12.53</td>
</tr>
<tr>
<td>SS-2</td>
<td>11.78</td>
<td>11.14</td>
</tr>
<tr>
<td>SS-3</td>
<td>9.38</td>
<td>11.18</td>
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<tr>
<td>SS-4</td>
<td>11.85</td>
<td>11.67</td>
</tr>
<tr>
<td>SS-5</td>
<td>12.34</td>
<td>9.43</td>
</tr>
<tr>
<td>SS-6</td>
<td>9.10</td>
<td>12.49</td>
</tr>
<tr>
<td>Specimen SA</td>
<td></td>
<td></td>
</tr>
<tr>
<td>SA-1</td>
<td>12.43</td>
<td>14.57</td>
</tr>
<tr>
<td>SA-2</td>
<td>12.50</td>
<td>12.66</td>
</tr>
<tr>
<td>SA-3</td>
<td>13.40</td>
<td>14.38</td>
</tr>
<tr>
<td>Specimen SA-S</td>
<td></td>
<td></td>
</tr>
<tr>
<td>SA-S-1</td>
<td>13.25</td>
<td>14.51</td>
</tr>
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<td>SA-S-2</td>
<td>14.33</td>
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<td>SA-S-5</td>
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<td>14.83</td>
</tr>
<tr>
<td>SA-S-6</td>
<td>13.95</td>
<td>11.49</td>
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<tr>
<td><strong>Average</strong></td>
<td><strong>12.08</strong></td>
<td><strong>12.89</strong></td>
</tr>
<tr>
<td><strong>SD</strong></td>
<td><strong>1.66</strong></td>
<td><strong>1.73</strong></td>
</tr>
<tr>
<td><strong>CV [%]</strong></td>
<td><strong>13.70</strong></td>
<td><strong>13.39</strong></td>
</tr>
</tbody>
</table>
Figure A.2 (a) Stress-strain curve for specimens SS-1, SS-2, SA-1, SA-2, SA-S-1 and SA-S-2; and (b) calculation of modulus of elasticity for specimen SS-2.
A.1.2 Splitting test on concrete cylinders

Eight concrete cylinders (102×203 mm) were sampled from wall specimens and instrumented with one 100 kN load cell to obtain the splitting tensile strength of the concrete (ASTM C496) (Figure A.3). The splitting tensile strength is calculated according to:

\[ T = \frac{2P}{\pi LD} \]  

(A.1)
in which \( P \) is the maximum applied load, \( L \) is the length and \( D \) is the diameter of each specimen. Table A.2 summarizes the splitting test results on concrete cylinders. The average splitting tensile strength and standard deviation were 1.2 MPa and 0.1 MPa, respectively.

Figure A.3 Test setup for splitting test on concrete cylinders.
Table A.2 Summary of splitting test results on concrete cylinders.

<table>
<thead>
<tr>
<th>Specimen</th>
<th>Maximum applied load [kN]</th>
<th>Splitting tensile strength [MPa]</th>
</tr>
</thead>
<tbody>
<tr>
<td>Specimen SA</td>
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<td></td>
</tr>
<tr>
<td>SA-1</td>
<td>41.57</td>
<td>1.28</td>
</tr>
<tr>
<td>SA-2</td>
<td>32.75</td>
<td>1.01</td>
</tr>
<tr>
<td>SA-3</td>
<td>38.50</td>
<td>1.19</td>
</tr>
<tr>
<td>Specimen SA-S</td>
<td></td>
<td></td>
</tr>
<tr>
<td>SA-S-1</td>
<td>40.80</td>
<td>1.26</td>
</tr>
<tr>
<td>SA-S-2</td>
<td>32.48</td>
<td>1.00</td>
</tr>
<tr>
<td>SA-S-3</td>
<td>39.01</td>
<td>1.20</td>
</tr>
<tr>
<td>SA-S-4</td>
<td>35.35</td>
<td>1.09</td>
</tr>
<tr>
<td>SA-S-5</td>
<td>44.31</td>
<td>1.37</td>
</tr>
<tr>
<td><strong>Average</strong></td>
<td><strong>38.10</strong></td>
<td><strong>1.17</strong></td>
</tr>
<tr>
<td><strong>SD</strong></td>
<td><strong>4.25</strong></td>
<td><strong>0.13</strong></td>
</tr>
<tr>
<td><strong>CV [%]</strong></td>
<td><strong>11.16</strong></td>
<td><strong>11.16</strong></td>
</tr>
</tbody>
</table>
A.2 MORTAR

A.2.1 Compression test on mortar cylinders

Eight Type N mortar cylinders (76×152 mm) were sampled from wall specimens and instrumented with three 5 mm linear displacement transducers and one 100 kN load cell to obtain the compressive strength and modulus of elasticity of the Type N mortar used for building the masonry panels and the reinforced plaster (ASTM C780) (Figure A.4). Table A.3 summarizes the compression test results on mortar cylinders (MC-1 to MC-8). The average compressive strength and standard deviation were 8.3 MPa and 0.9 MPa, respectively. The average displacement measurement from the three transducers and the measured load were used to obtain the stress-strain curve for each specimen [Figure A.5(a)]. As illustrated in Figure A.5(b), the secant stiffness between 10 % and 40 % of the peak stress was used to calculate the modulus of elasticity. The average modulus of elasticity and standard deviation were 11.5 GPa and 0.9 GPa, respectively.
Figure A.4 Test setup for compression test on mortar cylinders.

Table A.3 Summary of compression test results on mortar cylinders.

<table>
<thead>
<tr>
<th>Specimen</th>
<th>Compressive strength [MPa]</th>
<th>Modulus of elasticity [GPa]</th>
</tr>
</thead>
<tbody>
<tr>
<td>MC-1</td>
<td>8.51</td>
<td>11.05</td>
</tr>
<tr>
<td>MC-2</td>
<td>9.58</td>
<td>12.08</td>
</tr>
<tr>
<td>MC-3</td>
<td>8.08</td>
<td>12.25</td>
</tr>
<tr>
<td>MC-4</td>
<td>7.46</td>
<td>10.69</td>
</tr>
<tr>
<td>MC-5</td>
<td>7.60</td>
<td>12.75</td>
</tr>
<tr>
<td>MC-6</td>
<td>7.54</td>
<td>10.44</td>
</tr>
<tr>
<td>MC-7</td>
<td>9.69</td>
<td>-</td>
</tr>
<tr>
<td>MC-8</td>
<td>8.29</td>
<td>-</td>
</tr>
<tr>
<td><strong>Average</strong></td>
<td><strong>8.34</strong></td>
<td><strong>11.54</strong></td>
</tr>
<tr>
<td><strong>SD</strong></td>
<td><strong>0.88</strong></td>
<td><strong>0.94</strong></td>
</tr>
<tr>
<td><strong>CV [%]</strong></td>
<td><strong>10.55</strong></td>
<td><strong>8.14</strong></td>
</tr>
</tbody>
</table>
Figure A.5 (a) Stress-strain curve for MC-1 to MC-6; and (b) calculation of modulus of elasticity for MC-2.
A.2.2 Flexure test on mortar prisms

Three point bending tests (ASTM C348) were conducted on eight Type N mortar prisms (Figure A.6) to obtain the tensile strength. The mortar coupons were cast using 40×40×160 mm molds. The flexural strength is calculated according to:

\[ S_I = \frac{3PL_1}{2BH_1^2} \]  \hspace{1cm} (A.2)

in which \( P \) is the maximum applied load, \( L_1 \) is the span length, \( B \) is the width and \( H_1 \) is the height of each specimen. Table A.4 summarizes the flexure test results for mortar prisms. The average flexural strength and standard deviation of the mortar prisms were 1.8 MPa and 0.2 MPa, respectively.

Figure A.6 Test setup for flexural test on mortar prisms.
Table A.4 Summary of flexure test results on mortar prisms.

<table>
<thead>
<tr>
<th>Specimen</th>
<th>Maximum applied load [kN]</th>
<th>Flexural strength [MPa]</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Specimen SS</strong></td>
<td></td>
<td></td>
</tr>
<tr>
<td>SS-1</td>
<td>829.50</td>
<td>1.89</td>
</tr>
<tr>
<td>SS-2</td>
<td>752.55</td>
<td>1.65</td>
</tr>
<tr>
<td>SS-3</td>
<td>712.54</td>
<td>1.63</td>
</tr>
<tr>
<td><strong>Specimen SA</strong></td>
<td></td>
<td></td>
</tr>
<tr>
<td>SA-1</td>
<td>957.89</td>
<td>2.18</td>
</tr>
<tr>
<td>SA-2</td>
<td>898.59</td>
<td>1.97</td>
</tr>
<tr>
<td><strong>Specimen SS-S</strong></td>
<td></td>
<td></td>
</tr>
<tr>
<td>SS-S-1</td>
<td>744.22</td>
<td>1.70</td>
</tr>
<tr>
<td>SS-S-2</td>
<td>827.53</td>
<td>1.81</td>
</tr>
<tr>
<td>SS-S-3</td>
<td>776.48</td>
<td>1.72</td>
</tr>
<tr>
<td><strong>Average</strong></td>
<td><strong>812.41</strong></td>
<td><strong>1.82</strong></td>
</tr>
<tr>
<td><strong>SD</strong></td>
<td><strong>83.33</strong></td>
<td><strong>0.19</strong></td>
</tr>
<tr>
<td><strong>CV [%]</strong></td>
<td><strong>10.26</strong></td>
<td><strong>10.26</strong></td>
</tr>
</tbody>
</table>

A.3 CONCRETE MASONRY UNITS (CMUs)

A.3.1 Compression test on CMUs

Six tests (ASTM C140) were carried out on hard-capped single blocks (Figure A.7) instrumented with four 5 mm linear displacement transducers and one 500 kN load cell to obtain the compressive strength and modulus of elasticity (Figure A.8). Table A.5 summarizes the compression test results on CMUs. The average compressive strength (on the net area) and standard deviation were 7.2 MPa and 0.3 MPa, respectively. For each specimen, the average displacement measurement from the four transducers and the measured load were used to obtain the stress-strain curve [Figure A.9 (a)]. As illustrated in Figure A.9(b), the secant stiffness between 5 % and 33 % of the peak stress was used to calculate the modulus of elasticity (MSJC 2011). The average modulus of elasticity and
standard deviation were 5.8 GPa and 1.1 GPa, respectively. As illustrated in Figure A.10, the typical failure mode for the specimens was conical shear failure due to end restraint effect.

Figure A.7 Concrete masonry unit with nominal size of 203×203×406 mm, net cross-sectional area of 391 cm² and solid percentage of 51 %.

Figure A.8 Test setup for compression test on single blocks.
Figure A.9 (a) Stress-strain curve for CMU-1 to CMU-6; and (b) calculation of modulus of elasticity for CMU-3.
Table A.5 Summary of compression test results on CMUs.

<table>
<thead>
<tr>
<th>Specimen</th>
<th>Compressive strength [MPa]</th>
<th>Modulus of elasticity [GPa]</th>
</tr>
</thead>
<tbody>
<tr>
<td>CMU-1</td>
<td>7.37</td>
<td>4.67</td>
</tr>
<tr>
<td>CMU-2</td>
<td>6.97</td>
<td>4.39</td>
</tr>
<tr>
<td>CMU-3</td>
<td>6.82</td>
<td>6.87</td>
</tr>
<tr>
<td>CMU-4</td>
<td>7.38</td>
<td>5.49</td>
</tr>
<tr>
<td>CMU-5</td>
<td>7.59</td>
<td>6.13</td>
</tr>
<tr>
<td>CMU-6</td>
<td>7.01</td>
<td>7.08</td>
</tr>
<tr>
<td><strong>Average</strong></td>
<td><strong>7.19</strong></td>
<td><strong>5.77</strong></td>
</tr>
<tr>
<td><strong>SD</strong></td>
<td>0.30</td>
<td>1.12</td>
</tr>
<tr>
<td><strong>CV [%]</strong></td>
<td><strong>4.16</strong></td>
<td><strong>19.36</strong></td>
</tr>
</tbody>
</table>

Figure A.10 Conical shear failure mode for hollow CMUs.
A.3.2 Flexure test on CMU prisms

Three point bending tests (ASTM C348) were conducted on six concrete block prisms (Figure A.11) to obtain the tensile strength. The blocks were cut into prisms with average dimension of 30×50×200 mm. The flexural strength is calculated according to Equation A.2. Table A.6 summarizes the flexure test results on CMU prisms. The average flexural strength and standard deviation of the CMU prisms were 1.7 MPa and 0.1 MPa, respectively.

Figure A.11 Test setup for flexural test on CMU prisms.
Table A.6 Summary of flexure test results on CMU prisms.

<table>
<thead>
<tr>
<th>Specimen</th>
<th>Maximum applied load [kN]</th>
<th>Flexural strength [MPa]</th>
</tr>
</thead>
<tbody>
<tr>
<td>Prism-1</td>
<td>599.98</td>
<td>1.63</td>
</tr>
<tr>
<td>Prism-2</td>
<td>693.61</td>
<td>1.83</td>
</tr>
<tr>
<td>Prism-3</td>
<td>566.42</td>
<td>1.61</td>
</tr>
<tr>
<td>Prism-4</td>
<td>579.19</td>
<td>1.53</td>
</tr>
<tr>
<td>Prism-5</td>
<td>845.62</td>
<td>1.81</td>
</tr>
<tr>
<td>Prism-6</td>
<td>831.41</td>
<td>1.82</td>
</tr>
<tr>
<td>Average</td>
<td>686.04</td>
<td>1.70</td>
</tr>
<tr>
<td>SD</td>
<td>126.32</td>
<td>0.13</td>
</tr>
<tr>
<td>CV [%]</td>
<td>18.41</td>
<td>7.63</td>
</tr>
</tbody>
</table>

A.4 Masonry

A.4.1 Compression test on two-blocks prisms

Three tests were carried out on hard-capped two-blocks prisms (ASTM C1314) instrumented with four vertical 50 mm and two horizontal 5 mm linear displacement transducers, and one 500 kN load cell (Figure A.12). Table A.7 summarizes the compression test results on two-blocks prisms. The average compressive strength (on the net area) and standard deviation were 5.8 MPa and 0.4 MPa, respectively. The strain-stress curve for each specimen was obtained by using the average displacement measurement and the measured load [Figure A.13(a)]. As shown in Figure A.13(b), the secant stiffness between 5 % and 33 % of the peak stress (MSJC 2011) was used to calculate the modulus of elasticity. The average modulus of elasticity and standard deviation were 11.3 GPa and 1.8 GPa, respectively. The typical conical shear failure mode of two-blocks prisms under compression is shown in Figure A.14.
Figure A.12 Test setup for compression test on two-blocks prisms.

Table A.7 Summary of compression test results on two-blocks prisms.

<table>
<thead>
<tr>
<th>Specimen</th>
<th>Compressive strength on net area [MPa]</th>
<th>Modulus of elasticity [GPa]</th>
</tr>
</thead>
<tbody>
<tr>
<td>Prism-1</td>
<td>5.61</td>
<td>10.99</td>
</tr>
<tr>
<td>Prism-2</td>
<td>6.25</td>
<td>13.24</td>
</tr>
<tr>
<td>Prism-3</td>
<td>5.43</td>
<td>9.67</td>
</tr>
<tr>
<td><strong>Average</strong></td>
<td><strong>5.76</strong></td>
<td><strong>11.30</strong></td>
</tr>
<tr>
<td><strong>SD</strong></td>
<td><strong>0.43</strong></td>
<td><strong>1.81</strong></td>
</tr>
<tr>
<td><strong>CV [%]</strong></td>
<td><strong>7.50</strong></td>
<td><strong>15.98</strong></td>
</tr>
</tbody>
</table>
Figure A.13 (a) Stress-strain curve for Prism-1 to Prism-3; and (b) calculation of modulus of elasticity for Prism-3.
A.4.2. Compression test on small masonry walls

Three compression tests were performed on small masonry walls (203×803×1006 mm) (CEN 1999). Each wall was built with ten CMUs and instrumented with two 100 mm linear potentiometers mounted vertically, one 50 mm linear displacement transducer mounted horizontally, one 500 kN load cell and one pressure transducer to measure the maximum vertical load (Figure A.15). Table A.8 summarizes the compression test results on scaled walls. The tests on three specimens resulted in an average compressive strength (on the net area) and standard deviation of 4.3 MPa and 0.5 MPa, respectively. The typical failure mode of specimens is illustrated in Figure A.16.

Figure A.14 Conical shear failure mode for two-blocks prism under compression.
Figure A.15 Compression test on scaled masonry walls: (a) test setup; and (b) schematic. Dimensions in mm.
Figure A.16 Failure mode of scaled masonry wall under compression: (a) Wall-1; (b) Wall-2; and (c) Wall-3.
Table A.8 Summary of compression test results on scaled masonry walls.

<table>
<thead>
<tr>
<th>Specimen</th>
<th>Compressive strength on net area [MPa]</th>
</tr>
</thead>
<tbody>
<tr>
<td>Wall-1</td>
<td>4.52</td>
</tr>
<tr>
<td>Wall-2</td>
<td>3.69</td>
</tr>
<tr>
<td>Wall-3</td>
<td>4.71</td>
</tr>
<tr>
<td>Average</td>
<td>4.31</td>
</tr>
<tr>
<td>SD</td>
<td>0.54</td>
</tr>
<tr>
<td>CV [%]</td>
<td>12.57</td>
</tr>
</tbody>
</table>

A.4.3 Diagonal tension test on masonry assemblages

Diagonal tension tests on masonry assemblages (ASTM E519) were conducted to determine the diagonal tensile (shear) strength of masonry. Each specimen was built with eighteen CMUs, having dimensions of 203×1220×1220 mm, and instrumented with one 5 mm linear displacement transducer mounted horizontally and one 5 mm transducer mounted vertically and one pressure transducer to measure the maximum vertical load (Figure A.17). The tensile strength on the basis of gross area is calculated according to:

\[ S_s = 0.707 \frac{P}{BT_1} \]  

(A.3)

in which \(P\) is the maximum applied load, \(B\) is the width, and \(T\) is the thickness of the specimen. Figure A.18 shows the load-deformation curves for the three specimens. Table A.9 summarizes the diagonal tension test results. The tests on three specimens resulted in average tensile strength and standard deviation of 0.32 MPa and 0.02 MPa, respectively. Figure A.19 shows the typical failure mode of the specimens under diagonal tension.
Figure A.17 Test setup for diagonal tension test on masonry assemblages. Dimensions in mm.

Figure A.18 Load-deformation curve for diagonal tension test on specimen-1, specimen-2 and specimen-3.
Table A.9 Summary of diagonal tension test results on masonry assemblages.

<table>
<thead>
<tr>
<th>Specimen</th>
<th>Maximum applied load [kN]</th>
<th>Tensile strength [MPa]</th>
</tr>
</thead>
<tbody>
<tr>
<td>Specimen-1</td>
<td>115.62</td>
<td>0.35</td>
</tr>
<tr>
<td>Specimen-2</td>
<td>104.92</td>
<td>0.31</td>
</tr>
<tr>
<td>Specimen-3</td>
<td>102.40</td>
<td>0.31</td>
</tr>
<tr>
<td><strong>Average</strong></td>
<td><strong>107.65</strong></td>
<td><strong>0.32</strong></td>
</tr>
<tr>
<td><strong>SD</strong></td>
<td><strong>7.02</strong></td>
<td><strong>0.02</strong></td>
</tr>
<tr>
<td><strong>CV [%]</strong></td>
<td><strong>6.52</strong></td>
<td><strong>6.52</strong></td>
</tr>
</tbody>
</table>

Figure A.19 Failure mode of masonry assemblage under diagonal tension: (a) Specimen-1; (b) Specimen-2; and (c) Specimen-3.
A.4.4 Flexure test on masonry

Five seven-block specimens were built and tested in four-point bending to obtain the modulus of rupture of masonry (ASTM E518) (Figure A.20). These specimens were instrumented with two 5 mm linear displacement transducers on each side to measure the midspan deflection and a 50 kN load cell to measure the applied load. The modulus of rupture is calculated according to:

$$R_1 = \frac{(0.167P + 0.125P_s)L_1}{S}$$  \hspace{1cm} (A.4)

in which $P$ is the maximum applied load, $P_s$ is the weight of specimen (equal to 1132 N), $L_1$ is the span (equal to 1220 mm), and $S$ is the section modulus of actual net bedded area (equal to 1765048 mm$^3$). Table A.10 summarizes the flexure test results performed on specimens. The average modulus of rupture and the standard deviation were 0.18 MPa and 0.03 MPa, respectively.

Figure A.20 Test setup for flexure test on masonry.
Table A.10 Summary of flexure test results on masonry assemblages.

<table>
<thead>
<tr>
<th>Specimen</th>
<th>Maximum applied load [N]</th>
<th>Modulus of rupture [MPa]</th>
</tr>
</thead>
<tbody>
<tr>
<td>Specimen-1</td>
<td>541.84</td>
<td>0.16</td>
</tr>
<tr>
<td>Specimen-2</td>
<td>526.49</td>
<td>0.16</td>
</tr>
<tr>
<td>Specimen-3</td>
<td>981.81</td>
<td>0.21</td>
</tr>
<tr>
<td>Specimen-4</td>
<td>797.63</td>
<td>0.19</td>
</tr>
<tr>
<td>Specimen-5</td>
<td>_ (1)</td>
<td>0.10</td>
</tr>
<tr>
<td><strong>Average</strong></td>
<td><strong>711.94</strong></td>
<td><strong>0.18</strong></td>
</tr>
<tr>
<td><strong>SD</strong></td>
<td><strong>218.71</strong></td>
<td><strong>0.03</strong></td>
</tr>
<tr>
<td><strong>CV [%]</strong></td>
<td><strong>30.72</strong></td>
<td><strong>14.03</strong></td>
</tr>
</tbody>
</table>

(1) The specimen broke under self-weight.
A.4.5 Shear test on mortar joints

Shear tests were performed on CMU triplets (CEN 2003). Three different pre-compression levels of 0.1, 0.2 and 0.3 MPa were used to investigate the cohesion and the friction coefficient of the mortar joints. Three specimens per pre-compression level were tested. The specimens were instrumented with four 50 mm linear displacement transducers mounted vertically to measure the slip of the mortar joints, four 5 mm transducers mounted horizontally the dilatancy, one 100 kN load cell to measure the shear load, and one 500 kN load cell to measure the level of pre-compression (Figure A.21). Figure A.22 shows the shear load versus the slip of the mortar joints for different pre-compression levels (0.1, 0.2 and 0.3 MPa). Table A.11 summarizes the shear test results on mortar joints. The three levels of pre-compression resulted in an average shear stress of 0.17, 0.29 and 0.38 MPa, respectively. Figure A.23 shows the shear stresses versus the pre-compression levels. By comparing the obtained equation ($\tau = 0.07 + 1.05 \sigma$) with the Mohr-Coulomb criterion given by:

$$\tau = c + \sigma \tan \varphi$$

(A.5)

the cohesion and internal friction angle were found to be 0.07 MPa and 46°, respectively.
Figure A.21 Shear test on mortar joints: (a) test setup; and (b) close-up of instruments on one side of specimen.
Table A.11 Summary of shear test on mortar joints.

<table>
<thead>
<tr>
<th>Pre-compression level(^{(1)}) [MPa]</th>
<th>0.1</th>
<th>0.2</th>
<th>0.3</th>
</tr>
</thead>
<tbody>
<tr>
<td>Shear stress(^{(1)}) [MPa]</td>
<td>0.15</td>
<td>0.28</td>
<td>0.37</td>
</tr>
<tr>
<td></td>
<td>0.17</td>
<td>0.31</td>
<td>0.38</td>
</tr>
<tr>
<td></td>
<td>0.19</td>
<td>0.30</td>
<td>0.39</td>
</tr>
<tr>
<td>Average</td>
<td>0.17</td>
<td>0.29</td>
<td>0.38</td>
</tr>
</tbody>
</table>

\(^{(1)}\) on the gross area
A.4.6 Pullout test on aluminum strips

Two types of mortars were used for the pullout testing of the aluminum strips (ASTM E754). The first was a conventional latex-modified mortar and the second was a Portland cement mortar with fine sand and no special additives. The embedment length of aluminum strips in the mortar joint was 200 mm (Figure A.24). Six specimens were tested for each mortar type. The specimens were instrumented with two 50 mm linear displacement transducers to measure the slippage at the loaded and unloaded ends of each aluminum strip (Figure A.25). Figure A.26 shows the test setup and close-ups of the loaded and unloaded ends. The load-displacement curves for the loaded and unloaded ends of an aluminum strip and the comparison with the ultimate tensile strength is illustrated in Figure A.27. Table A.12 and Table A.13 summarize the pullout test results for the specimens with latex-modified (ST-EM-1 to ST-EM-6) and Portland
cement mortar (ST-CM-1 to ST-CM-6), respectively. The results showed an average pullout force of 10.4 kN, an average bond stress (on the perimeter of aluminum strips) of 1.6 MPa, and an average tensile stress (on the cross section) of 258 MPa for the latex-modified mortar specimens. For the case of the Portland cement mortar specimens, the average pullout force was 10.7 kN, the average bond stress was 1.7 MPa, and the average tensile stress was 265 MPa, thus showing negligible differences with respect to the latex-modified mortar. Figure A.28 shows the close-up of failure in a Portland cement mortar specimen.
Figure A.24 Embedment of aluminum strip in mortar joint. Dimension in mm.

Figure A.25 Instrumentation of specimen with two 50 mm transducers.
Figure A.26 Pullout test on aluminum strips: (a) test setup; (b) close-up of loaded end; and (c) close-up of unloaded end.
Figure A.27 Load-displacement curves for loaded and unloaded ends of aluminum strip (specimen ST-CM-2).

Table A.12 Summary of pullout test results for latex-modified mortar specimens.

<table>
<thead>
<tr>
<th>Specimen</th>
<th>Pullout load [kN]</th>
<th>Bond stress [MPa]</th>
<th>Tensile stress [MPa]</th>
</tr>
</thead>
<tbody>
<tr>
<td>ST-EM-1</td>
<td>11.27</td>
<td>1.75</td>
<td>279.46</td>
</tr>
<tr>
<td>ST-EM-2</td>
<td>9.62</td>
<td>1.49</td>
<td>238.69</td>
</tr>
<tr>
<td>ST-EM-3</td>
<td>10.87</td>
<td>1.68</td>
<td>269.57</td>
</tr>
<tr>
<td>ST-EM-4</td>
<td>11.13</td>
<td>1.73</td>
<td>276.12</td>
</tr>
<tr>
<td>ST-EM-5</td>
<td>11.04</td>
<td>1.71</td>
<td>273.88</td>
</tr>
<tr>
<td>ST-EM-6</td>
<td>8.57</td>
<td>1.33</td>
<td>212.66</td>
</tr>
<tr>
<td><strong>Average</strong></td>
<td><strong>10.42</strong></td>
<td><strong>1.61</strong></td>
<td><strong>258.40</strong></td>
</tr>
<tr>
<td><strong>SD</strong></td>
<td>1.08</td>
<td>0.17</td>
<td>26.84</td>
</tr>
<tr>
<td><strong>CV [%]</strong></td>
<td><strong>10.39</strong></td>
<td><strong>10.39</strong></td>
<td><strong>10.39</strong></td>
</tr>
</tbody>
</table>
Table A.13 Summary of pullout test results for Portland cement mortar specimens.

<table>
<thead>
<tr>
<th>Specimen</th>
<th>Pullout load [kN]</th>
<th>Bond stress [MPa]</th>
<th>Tensile stress [MPa]</th>
</tr>
</thead>
<tbody>
<tr>
<td>ST-CM-1</td>
<td>10.11</td>
<td>1.57</td>
<td>250.62</td>
</tr>
<tr>
<td>ST-CM-2</td>
<td>11.27</td>
<td>1.75</td>
<td>279.40</td>
</tr>
<tr>
<td>ST-CM-3</td>
<td>10.93</td>
<td>1.69</td>
<td>271.15</td>
</tr>
<tr>
<td>ST-CM-4</td>
<td>11.24</td>
<td>1.74</td>
<td>278.67</td>
</tr>
<tr>
<td>ST-CM-5</td>
<td>8.89</td>
<td>1.38</td>
<td>220.37</td>
</tr>
<tr>
<td>ST-CM-6</td>
<td>11.71</td>
<td>1.81</td>
<td>290.32</td>
</tr>
<tr>
<td><strong>Average</strong></td>
<td><strong>10.69</strong></td>
<td><strong>1.66</strong></td>
<td><strong>265.09</strong></td>
</tr>
<tr>
<td><strong>SD</strong></td>
<td><strong>1.03</strong></td>
<td><strong>0.16</strong></td>
<td><strong>25.58</strong></td>
</tr>
<tr>
<td><strong>CV [%]</strong></td>
<td><strong>9.65</strong></td>
<td><strong>9.65</strong></td>
<td><strong>9.65</strong></td>
</tr>
</tbody>
</table>

Figure A.28 Close-up of failed anchorage.
A.5 ALUMINUM

A.5.1 Tension test on aluminum strips

Tension tests on three 6061-T6 aluminum strips having cross section of 3.2×12.7 mm and instrumented with stain gauges were performed (ASTM E8) (Figure A.29) to obtain the tensile and yield strength and modulus of elasticity. Figure A.30 shows the stress-strain curve for aluminum strip and the offset method to calculate the yield strength and strain. The summary of tension test results is shown in Table A.14. The average tensile and yield strength and modulus of elasticity were 295 MPa, 265 MPa and 65 GPa, respectively.

Figure A.29 Test setup for tension test on aluminum strips.
Figure A.30 Stress-strain curve for aluminum strip.

Table A.14 Summary of tension test results on aluminum strips.

<table>
<thead>
<tr>
<th>Specimen</th>
<th>Tensile strength [MPa]</th>
<th>Yield strength [MPa]</th>
<th>Yield strain [μm/m]</th>
<th>Modulus of elasticity [GPa]</th>
</tr>
</thead>
<tbody>
<tr>
<td>Al-1</td>
<td>306.28</td>
<td>278.80</td>
<td>6301.80</td>
<td>64.80</td>
</tr>
<tr>
<td>Al-2</td>
<td>284.92</td>
<td>254.60</td>
<td>6028.40</td>
<td>63.20</td>
</tr>
<tr>
<td>Al-3</td>
<td>292.38</td>
<td>261.80</td>
<td>6004.00</td>
<td>65.40</td>
</tr>
<tr>
<td>Average</td>
<td>294.53</td>
<td>265.07</td>
<td>6111.40</td>
<td>64.47</td>
</tr>
<tr>
<td>SD</td>
<td>10.84</td>
<td>12.43</td>
<td>165.34</td>
<td>1.14</td>
</tr>
<tr>
<td>CV [%]</td>
<td>3.68</td>
<td>4.69</td>
<td>2.71</td>
<td>1.76</td>
</tr>
</tbody>
</table>
A.6 Steel

A.6.1 Tension test on steel rebars

Tension tests on three Ø13 mm steel rebars (ASTM A370), instrumented with strain gauges, were performed to obtain the tensile and yield strength and modulus of elasticity (Figure A.31). Table A.15 summarizes the tension test results on Ø13 mm steel rebars. The average tensile and yield strength and modulus of elasticity were 755 MPa, 440 MPa and 189 GPa, respectively.

Figure A.31 Test setup for tension test on steel rebars.
Table A.15 Summary of tension test results on steel rebars.

<table>
<thead>
<tr>
<th>Specimen</th>
<th>Tensile strength [MPa]</th>
<th>Yield strength [MPa]</th>
<th>Yield strain [μm/m]</th>
<th>Modulus of elasticity [GPa]</th>
</tr>
</thead>
<tbody>
<tr>
<td>Rebar-1</td>
<td>756.75</td>
<td>436.50</td>
<td>2150.00</td>
<td>212.90</td>
</tr>
<tr>
<td>Rebar-2</td>
<td>753.53</td>
<td>451.70</td>
<td>2450.00</td>
<td>192.20</td>
</tr>
<tr>
<td>Rebar-3</td>
<td>754.74</td>
<td>432.20</td>
<td>2460.00</td>
<td>163.10</td>
</tr>
<tr>
<td>Average</td>
<td>755.01</td>
<td>440.13</td>
<td>2353.33</td>
<td>189.40</td>
</tr>
<tr>
<td>SD</td>
<td>1.63</td>
<td>10.25</td>
<td>176.16</td>
<td>25.02</td>
</tr>
<tr>
<td>CV [%]</td>
<td>0.22</td>
<td>2.33</td>
<td>7.49</td>
<td>13.21</td>
</tr>
</tbody>
</table>
A.6.2 Tension test on steel wires

Three tension tests were performed on steel wires (ASTM A185) to determine the tensile strength of wires used in the reinforced plaster (Figure A.32). The average tensile strength and standard deviation were 525 MPa and 1.3 MPa, respectively.

Figure A.32 Test setup for tension test on steel wires.
APPENDIX B – IN-PLANE CYCLIC TEST RESULTS

This appendix provides the data from PWS measurements performed during the in-plane cyclic load tests on full-scale confined masonry specimens.

B.1 SPECIMEN SS

Figure B.1 and Figure B.2 illustrate the layout of displacement transducers and strain gauges in specimen SS, respectively. The data from transducers and strain gauges are shown in Figure B.3 and Figure B.4, respectively.

Figure B.1 Layout of displacement transducers mounted on specimen SS.
Figure B.2 Layout of strain gauges mounted on steel reinforcement (SG1-SG9) in specimen SS.
Figure B.3 Data from displacement transducers mounted on specimen SS.
Figure B.4 Data from strain gauges mounted on steel reinforcement (SG1-SG9) in specimen SS.
B.2 SPECIMEN SA

The layout of displacement transducers in specimen SA is similar to that of specimen SS (Figure B.1). Figure B.5 illustrates the layout of strain gauges mounted on steel reinforcement in specimen SA. The data from displacement transducers and strain gauges are shown in Figure B.6 and Figure B.7, respectively.

Figure B.5 Layout of strain gauges mounted on steel reinforcement (SG1-SG8) in specimen SA.
Figure B.6 Data from displacement transducers mounted on specimen SA.
Figure B.7 Data from strain gauges mounted on steel reinforcement (SG1-SG8) in specimen SA.
B.3 SPECIMEN SS-S

Figure B.8 and Figure B.9 show the layout of displacement transducers and strain gauges in specimen SS-S, respectively. The data from displacement transducers is shown in Figure B.10. The data from strain gauges mounted on steel reinforcement and metallic strips embedded in mortar joints are shown in Figure B.11 and Figure B.12, respectively.

Figure B.8 Layout of displacement transducers mounted on specimen SS-S.
Figure B.9 Layout of strain gauges mounted on steel reinforcement (SG1-SG10) and metallic strips (SGH1-SGH6) in specimen SS-S.
Figure B.10 Data from displacement transducers mounted on specimen SS-S.
Figure B.11 Data from strain gauges mounted on steel reinforcement (SG1-SG10) in specimen SS-S.
Figure B.12 Data from strain gauges mounted on horizontal reinforcement (SGH1-SGH6) in specimen SS-S.
B.4 SPECIMEN SA-S

The layout of displacement transducers in specimen SA-S is similar to that of specimen SS and SA (Figure B.1). Figure B.13 illustrates the layout of strain gauges mounted on steel reinforcement and metallic strips embedded in mortar joints in specimen SA-S. The data from displacement transducers is shown in Figure B.14. The data from strain gauges mounted on steel reinforcement and metallic strips are shown in Figure B.15 and Figure B.16, respectively.

Figure B.13 Layout of strain gauges mounted on steel reinforcement (SG1-SG8) and metallic strips (SGH1-SGH6) in specimen SA-S.
Figure B.14 Data from displacement transducers mounted on specimen SA-S.
Figure B.15 Data from strain gauges mounted on steel reinforcement (SG1-SG8) in specimen SA-S.
Figure B.16 Data from strain gauges mounted on horizontal reinforcement (SGH1-SGH6) in specimen SA-S.
B.5 SPECIMEN SS-P

The layout of displacement transducers and strain gauges in specimen SS-P are illustrated in Figure B.17 and Figure B.18, respectively. The data from displacement transducers and strain gauges are shown in Figure B.19 and Figure B.20, respectively.

Figure B.17 Layout of displacement transducers mounted on specimen SS-P.
Figure B.18 Layout of strain gauges mounted on steel reinforcement and diagonal aluminum bars (SG1-SG10) in specimen SS-P.
Figure B.19 Data from displacement transducers mounted on specimen SS-P.
Figure B.20 Data from strain gauges mounted on steel reinforcement and diagonal aluminum bars (SG1-SG10) in specimen SS-P.

B.6 SPECIMEN SA-P

The layout of displacement transducers and strain gauges in specimen SA-P are similar to that of specimen SS-P and specimen SA, respectively. The data from displacement transducers and strain gauges are shown in Figure B.21 and Figure B.22, respectively.
Figure B.21 Data from displacement transducers mounted on specimen SA-P.
Figure B.22 Data from strain gauges mounted on steel reinforcement (SG1-SG8) in specimen SA-P.
Appendix C – 3D-DIC (Calibration, Speckle Pattern)

This appendix complements Sections 2.4.2.3 and 2.4.2.4 and provides more details on the calibration of the stereo-vision system and the speckle pattern.

C.1 Calibration of Stereo-vision System

Figure C.1 illustrates the calibration procedure for the stereo-vision system. During calibration, a calibration grid is rotated and tilted into different orientations while images are acquired. The stereo-vision system was calibrated by taking 60 pairs of images of the calibration grid while it was held in different positions against the wall. As illustrated in Figure C.2, the calibration grid included 12×9 dots with nominal diameter of 20 mm and on-center spacing of 50 mm.

C.2 Speckle Pattern

Figure C.3 shows the stereo-vision system setup and the close-up of the speckle pattern. The diameter of each speckle pattern was approximately 3.2 mm. Based on the field of view (3,330×2,790 mm) and resolution of cameras (2,448×2,048 pixel), the size of each pixel was 3,330 / 2,448 = 1.36 mm. Hence, the diameter of each speckle was equal to 3.2 / 1.36 ≈ 2.3 pixels.
Figure C.1 Typical images obtained during calibration process from left camera. 60 pairs of images acquired with grid in different orientations and positions.

Figure C.2 Calibration grid. Dimensions in mm.
Figure C.3 Stereo-vision system setup (inset shows close-up of speckle pattern).