

ISTANBUL TECHNICAL UNIVERSITY ★ GRADUATE SCHOOL OF SCIENCE
ENGINEERING AND TECHNOLOGY

**SEISMIC BEHAVIOUR OF HISTORICAL STONE MASONRY
MULTI-LEAF WALLS**

Ph.D. THESIS

Cem DEMİR

Department of Civil Engineering
Structural Engineering Programme

JUNE 2012

ISTANBUL TECHNICAL UNIVERSITY ★ GRADUATE SCHOOL OF SCIENCE
ENGINEERING AND TECHNOLOGY

**SEISMIC BEHAVIOUR OF HISTORICAL STONE MASONRY
MULTI-LEAF WALLS**

Ph.D. THESIS

**Cem DEMİR
(501032106)**

Department of Civil Engineering

Structural Engineering Programme

Thesis Advisor: Prof. Dr. Alper İLKİ

JUNE 2012

İSTANBUL TEKNİK ÜNİVERSİTESİ ★ FEN BİLİMLERİ ENSTİTÜSÜ

**ÇOK TABAKALI TAŞ YIĞMA DUVARLARIN DEPREM YÜKLERİ
ALTINDA DAVRANIŞI**

DOKTORA TEZİ

**Cem DEMİR
(501032106)**

İnşaat Mühendisliği Anabilim Dalı

Yapı Mühendisliği Programı

Tez Danışmanı: Prof. Dr. Alper İLKİ

HAZİRAN 2012

Cem Demir, a Ph.D. student of ITU Graduate School of Science, Engineering and Technology student ID 501032106, successfully defended the **thesis** entitled “**SEISMIC BEHAVIOUR OF HISTORICAL STONE MASONRY MULTI-LEAF WALLS**”, which he prepared after fulfilling the requirements specified in the associated legislations, before the jury whose signatures are below.

Thesis Advisor : **Prof. Dr. Alper İLKİ**
İstanbul Technical University

Jury Members : **Prof. Dr. Zekai CELEP**
İstanbul Technical University

Prof. Dr. İlknur KOLAY
İstanbul Technical University

Prof. Dr. Oğuz Cem ÇELİK
İstanbul Technical University

Assoc. Prof. Dr. Şevket ÖZDEN
Kocaeli University

Date of Submission : 04 February 2011
Date of Defense : 28 June 2012

Aileme,

FOREWORD

The present work was carried out at the Department of Civil Engineering of the Istanbul Technical University and was supervised by Prof.Dr. Alper İlki, to whom I express my deepest gratitude for having made this work possible by the support and valuable suggestions.

I would like to express my thanks to Prof.Dr. Zekai Celep, Prof.Dr. Fatma İlknur Kolay and Prof.Dr. Nahit Kumbasar for their profitable suggestions.

I am grateful to Prof.Dr. Mehmet Ali Taşdemir and Prof.Dr. Hüseyin Çimenoglu for making the using of Building Materials Lab. of Civil Eng. Fac. and Metallurgy Lab. of Faculty of Chemical and Metallurgical Eng. possible. I would like to express my gratitude to Assist.Prof.Dr. Gülsün Tanyeli for her technical suggestions on the characterization of the reinforcements and to Prof.Dr. Guido Magenes for his comments on the drift ratio limits.

Financially this work is supported by ITU Institute of Science and Technology. Things would be very difficult without the supports of the people and companies, I would like to express my gratitude to Civil Eng. Engin Cüneyt Seyhan –BASF Construction Chemicals Company for supplying hundreds of kilograms of mortar; to Dr. Kaan Yemez – Rozak Demir Company for generously supplying more than 5 tons of steel profiles; to Civil Eng. Ercan Arslan – Erk Construction and Murat Özkiriş – Er-Pa Metal Company for producing the test rig and the reinforcements at a symbolic price; to Barshan Mining Company for supplying the stone units at a symbolic price; to AKKA Architecture and Engineering Company for supplying two experienced masons for a reduced price; to Kesan Sac Company and Özge Çağlar for supplying the special metal sheets utilized for the production of the cramps.

Special thanks to all my friends that accompanied me during this study:

Emre Doğu, Medine İspir, Barış Güneş, Çağlar Göksu, Mustafa Cömert, Hanife Korkmaz, Pınar İnci, Deniz Korhan Dalgıç, Cengiz Şengül, Haldun Zık.

Many thanks to all laboratory staff of the Structural and Earthquake Engineering Laboratory (particularly to Civil. Eng. Hakan Saruhan and Tech. Ahmet Şahin) and Building Materials Laboratory of Civil Engineering Faculty of Istanbul Technical University.

Last, but not least, I would like to thank my wife Özlem for her understanding and love during the past few years. Her patience, support and encouragement was in the end what made this thesis possible. My parents Kadriye and Ali, and my brother, Kerem, receive my deepest gratitude and love for their dedication and the many years of support during this long journey.

June 2012

Cem DEMİR
(Civil Engineer, M.Sc.)

TABLE OF CONTENTS

	<u>Page</u>
FOREWORD.....	ix
TABLE OF CONTENTS.....	xi
ABBREVIATIONS	xv
LIST OF TABLES	xxi
LIST OF FIGURES	xxiii
SUMMARY	xxxvii
ÖZET.....	xli
1. INTRODUCTION.....	1
1.1 Masonry Walls at a Glance	4
1.2 Objectives of the Thesis	7
1.3 Outline of the Thesis	9
2. MULTI-LEAF MASONRY SHEAR WALLS	13
2.1 Multi-Leaf Stone Masonry Walls in Turkey	13
2.2 Observations on Three Existing Structures	20
2.3 Stone Masonry Walls in Engineering Literature.....	24
2.3.1 Studies on compressive behaviour	25
2.3.2 Studies on shear behaviour	38
3. EXPERIMENTAL INVESTIGATION OF MULTI-LEAF STONE	
MASONRY WALLS.....	49
3.1 Design and Construction of Walls.....	49
3.1.1 Design of experiments	49
3.1.2 Construction of the walls	55
3.2 Test Setup	61
3.3 Instrumentation of the Walls	65
3.4 Loading of the Walls	69
4. CHARACTERIZATION OF THE MATERIALS	71
4.1 Mechanical Characterization of Stone Units.....	71
4.1.1 Previous studies on Küfeki stone	72
4.1.2 Uniaxial compression tests on single stone units.....	73
4.1.3 Three-point bending tests on stone units.....	82
4.2 Mechanical Characterization of Mortar	84
4.2.1 Trials for a mortar composition	84
4.2.2 Compression tests on mortar samples.....	85
4.2.3 Three point bending tests on mortar samples.....	86
4.3 Mechanical Characterization of Rubble Masonry.....	87
4.3.1 Compression tests on rubble masonry cylinders.....	88
4.3.2 Splitting tests on rubble masonry cylinders	91
4.4 Mechanical Characterization of Metal Members	93
4.4.1 Investigations of Tanyeli et al. (1990)	93
4.4.2 Tests on original cramp reinforcements.....	95

4.4.3 Selection of an appropriate metal for the experimental study.....	106
4.5 Initial Shear Tests for Dry Joints.....	110
4.5.1 Testing procedure for initial shear tests	111
4.5.2 Initial shear test results for SET I.....	113
4.5.3 Initial shear test results for SET II	114
4.5.4 Comparison of the test results for smooth and rough surfaces	115
4.5.5 Characterization of the friction surfaces	117
4.6 Compression Tests on Masonry Prisms	125
4.6.1 Single-leaf masonry prism compression tests	126
4.6.2 Multi-leaf masonry prism compression tests	129
4.6.3 Results of the masonry prism compression tests.....	132
4.6.4 Prediction of multi-leaf masonry compressive strength.....	139
5. RESULTS OF THE SHEAR COMPRESSION TESTS.....	141
5.1 Behaviour and Failure of Walls.....	141
5.1.1 Effect of axial stress level	141
5.1.2 Effect of existence of cramps and pins.....	144
5.1.3 Effect of existence of rubble masonry infill.....	147
5.2 Comparison of Hysteresis Diagrams	149
5.2.1 Effect of axial stress level	149
5.2.2 Effect of existence of cramps and pins.....	159
5.2.3 Effect of existence of rubble masonry infill.....	167
6. INTERPRETATION OF THE EXPERIMENTAL DATA	171
6.1 Idealization of the Experimental Envelope Curves	171
6.2 Deformation Capacity	176
6.3 Variation of Stiffness.....	181
6.4 Energy Dissipation Capacity	184
6.5 Mechanical Characteristics Obtained From Shear Compression Tests.....	192
6.5.1 Young's modulus	192
6.5.2 Shear strength.....	194
6.5.3 Tensile strength	196
6.5.4 Shear modulus	199
6.6 Failure Mode Identification.....	200
6.6.1 Mann and Müller (1980) model	202
6.6.2 Turnsek and Cacovic (1971) model	208
6.7 Drift Ratios Corresponding to Performance Levels	211
7. INVESTIGATION OF THE BEHAVIOUR THROUGH FINITE ELEMENT ANALYSIS.....	225
7.1 Finite Element Analysis of Masonry Structures.....	225
7.2 Micro-Modelling of Tested Walls.....	228
7.2.1 Micro-modelling of shear compression tests	229
7.2.2 Micro-modelling of uniaxial compression tests	247
7.2.3 Analysis results for micro-modelling of uniaxial compression tests	252
7.2.4 Analysis results for micro-modelling of shear compression tests.....	259
8. CONCLUSIONS.....	273
8.1 Historical Multi-Leaf Stone Masonry Walls in Turkey	273
8.2 Characterization of Materials	274
8.3 Shear Compression Tests	277
8.4 Finite Element Analysis	282
8.5 Recommendations for Further Studies	284
REFERENCES.....	285

APPENDICES	295
CURRICULUM VITAE.....	447

ABBREVIATIONS

a	: Wall height in ASTM E 564-06 (2006)
ASTM	: American Society for Testing Materials
A_{bilinear}	: Area under the idealized bilinear curve
A_{envelope}	: Area under the experimental envelope curve
b	: Cross-section width, shear distribution factor in Turnsek and Cacovic model
b_w	: Width of the wall
c	: Shear strength at zero nominal compression stress (cohesion)
c_{cr}	: Critical damping coefficient
c_d	: Dilatational wave speed for a linear elastic material
CDP	: Concrete Damage Plasticity
c_{eq}	: Equivalent viscous damping coefficient
COV	: Coefficient of variation
CP	: Collapse Prevention
D	: Diameter
d	: Ultimate deformation for primary components in ASCE/SEI 41-06 (2007)
d_c	: Damage parameter for compression
d_{cr}	: Displacement of the wall corresponding to first significant cracking point of the idealized bilinear load-displacement diagram
d_e	: Displacement component of the equivalent yield point of the elastic-perfectly plastic load-displacement diagram
d_i	: Displacement amplitude at i th cycle
DL	: Damage Limitation
DR_{0.8max}	: Drift ratio corresponding to 20% strength loss of the wall
DR_{CP}	: Drift ratio corresponding to Collapse Prevention performance level
DR_{d,max}	: Drift ratio corresponding to ultimate displacement of the wall
DR_{dia,cr}	: Drift ratio corresponding to first diagonal cracking of the wall
DR_e	: Drift ratio component of the equivalent yield point of the elastic-perfectly plastic load-displacement diagram
DR_{f,cr}	: Drift ratio corresponding to first flexural cracking of the wall
DR_{Hmax}	: Drift ratio corresponding to lateral strength of the wall
DR_{IO}	: Drift ratio corresponding to Immediate Occupancy level
DR_{LS}	: Drift ratio corresponding to Life Safety performance level
d_s	: Net displacement during the initial shear tests
d_t	: Damage parameter for tension
d_u	: Ultimate displacement of the wall on the idealized bilinear load-displacement diagram.
e	: Ultimate deformation for secondary components in ASCE/SEI 41-06 (2007)
E	: Young's modulus
E_{dis,i}	: Energy dissipated at cycle i

E_e	: Total elastic energy absorption of equivalent elastic-perfectly plastic model
E_{es}	: Elastic strain energy stored in an equivalent linear elastic system
E_{FD}	: Energy absorbed by frictional dissipation
E_I	: Internal energy
e_{ini}	: Initial eccentricity of the pre-compression load
E_{inp}	: Energy input
E_{KE}	: Kinetic energy
E_N	: Normalized cumulative energy at any cycle
E_{rm}	: Young's modulus of rubble masonry core
E_{total}	: Total energy of a system
E_{unit}	: Young's modulus of masonry unit
E_V	: Energy absorbed by viscous dissipation
E_W	: Work done by external forces
E_{wc}	: Young's modulus of masonry wall
f_{bt}	: Uniaxial tensile strength of the unit
f_{bo}	: Biaxial compressive strength in CDP model
f_c	: Compressive strength
f_{co}	: Standard cylinder compressive strength, uniaxial compressive strength in CDP model
$f_{c,cube}$	: Equivalent cubical compressive strength
$f_{c,unit}$	: Compressive strength of masonry unit
$f_{c,mortar}$	: Compressive strength of mortar
f_{dt}	: Diagonal tensile strength of masonry
f_e	: Compressive strength of the external leaf
FEA	: Finite Element Analysis
f_i	: Compressive strength of the internal leaf
F_{shear}	: Shear force applied during the initial shear test
f_{st}	: Tensile strength obtained from the splitting test
f_t	: Tensile strength
f_{wc}	: Compressive strength of masonry wall
f_{wt}	: Tensile strength of masonry wall
$f_{y,cramp}$	: Yield strength of the cramp
$G_{1/3}$	: Shear modulus at 1/3 of the maximum lateral resistance
G_{30-60}	: Shear modulus between 30 and 60% of maximum lateral resistance
G_f	: Energy required to open a unit area of crack
GFRP	: Glass Fiber Reinforced Polymer
GPIB	: General Purpose Interface Bus
h	: Cross-section height
h_u	: Height of the stone unit
h_w	: Height of the wall
H	: Lateral load
HB	: Brinell hardness
H_{cr}	: Lateral load of the wall corresponding to first significant cracking point of the idealized bilinear load-displacement diagram.
$H_{dia,cr}$	: Lateral load corresponding to first diagonal cracking of the wall
$H_{d,max}$	: Lateral load corresponding to ultimate displacement of the wall
H_{dt}	: Diagonal shear cracking strength of the wall
H_f	: Lateral load at which flexural cracks occur
$H_{f,cr}$	: Lateral load corresponding to first flexural cracking of the wall

H_i	: Lateral load at i^{th} cycle
H_{max}	: Lateral load corresponding to lateral strength of the wall
H_r	: Rocking strength of the wall
H_{sl}	: Sliding shear strength of the wall
H_u	: Lateral load component of the equivalent yield point of the elastic-perfectly plastic load-displacement diagram
H_{0.8max}	: Lateral load corresponding to 20% strength loss of the wall
HV	: Vickers hardness
I	: Internal force
IMG	: Inorganic Matrix-Grid
IO	: Immediate Occupancy
ITU	: Istanbul Technical University
K	: Stiffness, in CDP model parameter that defines the shape of the yield function in deviatoric plane
K_{cramp}	: Axial stiffness of the cramp
K_e	: Equivalent elastic stiffness
k_n	: Normal joint stiffness
l	: Length of the compressed cross-section area
l_c	: Length of the stone unit
l_e	: Characteristic element length
LPG	: Liquid Petroleum Gas
LS	: Life Safety
l_s	: Span length
l_u	: Length of the stone unit
LVDT	: Linear Variable Differential Transformer
l_w	: Length of the wall
l_{0,cramp}	: Initial length of the cramp
M	: Mass matrix
n	: Number of elements in a roughness profile
N	: Number of observations, axial load in Mann and Müller model, increment number
N/A	: Not available
NC	: Near Collapse
P	: Externally applied load
q	: Number of zero crossings of a roughness profile
r	: Roughness profile
R	: Modulus of rupture
R²	: Coefficient of determination
R	: Modulus of rupture
R_a	: Average roughness
R_{ku}	: Kurtosis shape parameter of a roughness profile
RM	: Reinforced Masonry
R_p	: Maximum roughness profile peak height
R_q	: Root mean square roughness
R_{sk}	: Shape parameter for skewness of a roughness profile
RS_m	: Mean peak spacing parameter of a roughness profile
R_t	: Maximum height of the roughness profile
R_{ti}	: Maximum height of the roughness profile within the i^{th} sampling length
R_v	: Maximum roughness profile valley height

R_z	: Average maximum height of the roughness profile
$S(x)$	: Weighting function
SD	: Significant Damage
S_l	: Geometric scale
$S_m(i)$	: Distance between each zero crossing of a roughness profile
Stdev	: Standard deviation
t_{cramp}	: Thickness of the cramp
t_e	: Thickness of the external leaf
t_i	: Thickness of the internal leaf
t_N	: Time at increment N
t_s	: Echo travel time
TS	: Turkish Standard
u	: Nodal displacements
$\dot{u}$	: Nodal velocities
$\ddot{u}$	: Nodal accelerations
u_t^{ck}	: Crack opening
u_{to}	: Cracking displacement
URM	: Unreinforced Masonry
USB	: Universal Serial Bus
v_s	: Velocity of sound
W	: Peak load for flexural tension tests
W_a	: Average waviness
w_{cramp}	: Width of the cramp
W_{ku}	: Kurtosis shape parameter of a waviness profile
W_p	: Maximum waviness profile peak height
W_q	: Root mean square waviness
W_{sk}	: Shape parameter for skewness of a waviness profile
W_t	: Maximum height of the waviness profile
W_z	: Average maximum height of the waviness profile
x	: Position in relation to the centre of the weighting function
α	: Stiffness degradation parameter
β	: Stiffness degradation parameter
γ	: Shear rotation
Δ	: Lateral displacement of the wall
Δt	: Stable time increment
Δt_N	: Time increment for step N
ϵ_c	: Axial strain
$\tilde{\epsilon}_c^{in}$	: Inelastic strain in CDP model (compression)
$\tilde{\epsilon}_c^{pl}$	: Plastic strain in CDP model (compression)
ϵ_{co}	: Compressive strain corresponding to peak stress
ϵ_h	: Transverse strain
$\tilde{\epsilon}_t^{ck}$	: Inelastic strain in CDP model (tension)
$\tilde{\epsilon}_t^{pl}$	: Plastic strain in CDP model (tension)
ϵ_{vol}	: Volumetric strain
ϵ_{wco}	: Compressive strain at compressive strength of wall
$\epsilon_{y,cramp}$	: Yield strain of the cramp
$\epsilon_{0.8f}$	: Compressive strain corresponding to 20% strength loss
θ_e	: Correction factor for the outer leaf
θ_i	: Correction factor for the inner leaf

λ_{co}	: Cut-off wavelength
λ_v	: Shear slenderness factor
μ	: Ductility, friction coefficient in Coulomb friction model
$\bar{\mu}$	: Reduced global friction coefficient of the wall
μ_{av}	: Mean value of the ductilities of the pushing and pulling directions
μ_{cyc}	: Ductility obtained by using the average displacements in the pushing and pulling directions
ξ_{eq}	: Equivalent damping ratio
ρ	: Density
σ	: Normal stress
σ_{co}	: Uniaxial yield stress in CDP model
σ_{cu}	: Uniaxial compressive strength CDP model
σ_d	: Average compressive stress over the compressed cross-section area
σ_{max}	: Maximum principal stress
σ_{min}	: Minimum principal stress
σ_{to}	: Uniaxial tensile strength in CDP model
σ_x	: Stress in x direction
σ_y	: Stress in y direction
σ_o	: Compressive normal stress
ν	: Poisson's ratio
τ	: Shear stress
τ_{max}	: Shear strength
τ_{xy}	: Shear stress in y direction
τ_{yx}	: Shear stress in x direction
τ_{vo}	: Initial shear strength of the wall
$\bar{\tau}_{vo}$	: Reduced initial shear strength of the wall
ϕ	: Friction angle of the joint, principal plane orientation angle
ψ	: Coefficient to define the support conditions
2D	: Two dimensional
3D	: Three dimensional

LIST OF TABLES

	<u>Page</u>
Table 2.1 : Summary of stone masonry compression tests in literature.....	35
Table 2.2 : Summary of stone masonry shear compression tests in literature.	47
Table 3.1 : Scale factors for masonry specimens (Sabnis et al.,1985).....	52
Table 3.2 : Test matrix for shear compression tests.....	54
Table 3.3 : Target drift ratios and displacements for each cycle.....	70
Table 4.1 : Mechanical and physical characteristics of Küfeki stone (from literature).	73
Table 4.2 : Results of single unit compression tests.	79
Table 4.3 : Results of modulus of rupture tests for Küfeki stone.	84
Table 4.4 : Contents and mean compressive strengths of mortar trial mixtures.	85
Table 4.5 : Results of compression tests for mortar.....	86
Table 4.6 : Results of three point bending tests for mortar.....	87
Table 4.7 : Results of the rubble masonry monotonic compression tests.	90
Table 4.8 : Results of the rubble masonry splitting tests.	92
Table 4.9 : Investigations on metal samples reported in Tanyeli et al. (1990).	94
Table 4.10 : Chemical test results of cramps from Edirne II. Bayezid Mosque (in %).	101
Table 4.11 : Metallographic properties and predicted tensile strengths.....	106
Table 4.12 : Characteristic values for Erdemir 6222 as supplied by the manufacturer.	107
Table 4.13 : Mechanical characteristics from tensile tests of new cramp material.	108
Table 4.14 : Mechanical characteristics from tensile tests of new cramp material.	110
Table 4.15 : Shear strength and coefficient of friction values.	116
Table 4.16 : Roughness texture parameters for smooth surfaces (SET I specimens).	123
Table 4.17 : Roughness texture parameters for rough surfaces (SET II specimens).	123
Table 4.18 : Waviness texture parameters for smooth surfaces (SET I specimens).	124
Table 4.19 : Waviness texture parameters for rough surfaces (SET II specimens).	124
Table 4.20 : The masonry prism size requirements of TS EN 1052-1 (2000).	126
Table 4.21 : The dimensions of the masonry prisms.	126
Table 4.22 : Results of the single-leaf masonry compression tests.....	137
Table 4.23 : Results of the multi-leaf masonry compression tests.....	137
Table 4.24 : Young's moduli depending on the multi-leaf masonry effective areas.	138
Table 4.25 : Predicted compressive strength values for tested multi-leaf wallets	140
Table 5.1 : Observed failure modes of M-25-C, M-50-C, M-75-C and M-100-C..	144
Table 5.2 : Load and drift ratio data of M-25-C, M-50-C, M-75-C and M-100-C.	153

Table 5.3 : $H_{f,cr}/H_{max}$ and $H_{dia,cr}/H_{max}$ ratios (M-25-C, M-50-C, M-75-C and M-100-C).	154
Table 5.4 : Load and drift ratio data for critical states of M-50, M-50-C, M-50-CP, S-50-C and S-50-CP.	163
Table 5.5 : $H_{f,cr}/H_{max}$ and $H_{dia,cr}/H_{max}$ ratios for M-50, M-50-C, M-50-CP, S-50-C and S-50-CP walls.	165
Table 5.6 : Load & drift ratio data for critical states of M-50-C, S-50-C, M-50-CP and S-50-CP.	169
Table 6.1 : Characteristic values of bilinear idealisation.	174
Table 6.2 : Ductility factors of tested walls.	179
Table 6.3 : Parameters for stiffness degradation.	184
Table 6.4 : Mechanical characteristics obtained from shear compression tests.	192
Table 6.5 : Principal stresses obtained from shear compression tests.	198
Table 6.6 : Shear modulus values of tested walls.	200
Table 6.7 : Classification of damage based on residual crack widths.	214
Table 6.8 : Drift ratios of tested walls corresponding to performance levels.	216
Table 6.9 : Drift ratios of tested walls corresponding to performance levels.	221
Table 6.10 : Drift ratios given by codes for different performance levels.	224
Table 7.1 : Mechanical parameters used in the micro-modelling.	238
Table 7.2 : Interaction properties of analysis cases for shear compression tests.	241
Table 7.3 : Joint stiffness values calculated from stone prism compression tests.	251
Table 7.4 : Interaction properties of analysis cases for masonry compression tests.	252
Table C.1 : M-25-C specimen summary table.	319
Table C.2 : M-50-C specimen summary table.	335
Table C.3 : M-75-C specimen summary table.	351
Table C.4 : M-100-C specimen summary table.	368
Table C.5 : M-50 specimen summary table.	384
Table C.6 : S-50-C specimen summary table.	397
Table C.7 : M-50-CP specimen summary table.	411
Table C.8 : S-50-CP specimen summary table.	426

LIST OF FIGURES

	<u>Page</u>
Figure 1.1 : Different arrangements for brick masonry (Lourenço, 1998).	5
Figure 1.2 : Mosque damaged by the 1999 Marmara earthquake (Ilki et al., 2006)... 7	7
Figure 1.3 : Grand Bazaar area after the 1894 Istanbul earthquake (Urekli, 1999).... 8	8
Figure 2.1 : Multi-leaf wall of a church in Priene (Kolay, 1999).	14
Figure 2.2 : Wall typologies from Anatolia (Kolay, 1999).....	14
Figure 2.3 : Timber lintels in rubble masonry walls (Tayla, 2007).	15
Figure 2.4 : Finely cut stone ashlar walls with thin mortar joints (Tayla, 2007).....	16
Figure 2.5 : Finely cut stone ashlar with dry joints (drawing by Tayla, 2007).....	17
Figure 2.6 : Metal reinforcements used in the Ottoman architecture (Tanyeli, 1990).	17
Figure 2.7 : Typical wall configuration (modified from Tanyeli, 1990).	20
Figure 2.8 : Edirne Palace Cihannüma Kasrı (15 th century).	20
Figure 2.9 : Havsa Sokullu Mehmet Paşa Külliyesi (16 th century).....	21
Figure 2.10 : Wall sections from Edirne Palace Cihannüma Kasrı (1 st storey).	21
Figure 2.11 : Wall section from Havsa Sokullu Külliyesi.	22
Figure 2.12 : Cramp slots from ruins of Edirne Palace.....	22
Figure 2.13 : Cramp fixed with molten lead (from ruins of Edirne Palace).	23
Figure 2.14 : Cramps in the Edirne II. Bayezid Mosque (Kolay, 2009).....	23
Figure 2.15 : Pin used in the Edirne II. Bayezid Mosque (Kolay, 2009).....	24
Figure 2.16 : Original cramps from Edirne II. Bayezid Mosque.	24
Figure 2.17 : Wallets tested by Vintzileou and Tassios (1995).	26
Figure 2.18 : Wall geometry and stress-strain relationships (Valluzzi et al., 2004). 26	26
Figure 2.19 : Geometry of wallets tested by Binda et al. (2006).	27
Figure 2.20 : Stress-strain curves of wallets tested by Binda et al. (2006).	27
Figure 2.21 : View and stress-strain diagrams of wallets tested by Silva et al. (2008).	28
Figure 2.22 : Wallets tested by Vintzileou and Miltiadou-Fezans (2008).	29
Figure 2.23 : Schematic and testing views of wallets tested by Vintzileou (2008).. 30	30
Figure 2.24 : Vertical stress-vertical strain diagrams (Vintzileou, 2008).....	30
Figure 2.25 : Axial compression test setup of panels tested by Chiostriani (2008). .. 31	31
Figure 2.26 : Typical stress-strain diagram for stone prism (Oliveira et al., 2006).. 32	32
Figure 2.27 : Stress-strain curves of PP1 & PP2 and damage (Almedia et al., 2010).	32
Figure 2.28 : (a) Specimen, (b) typical stress-strain curve (Augenti and Parisi, 2010).	33
Figure 2.29 : Dimensions and typologies of specimens tested by Romano (2008).. 34	34
Figure 2.30 : Typologies of walls tested by Vasconcelos and Lourenço (2009).	38
Figure 2.31 : Typical damage and load-displacement diagram (dry stack walls).....	39
Figure 2.32 : Crack patterns and typical load-displacement diagram (irregular walls).	39

Figure 2.33 : Crack patterns and typical load-displacement diagram (rubble walls).	40
Figure 2.34 : Walls tested by Galasco et al. (2010).	41
Figure 2.35 : Typical hysteresis and envelope curves (Galasco et al., 2010).	41
Figure 2.36 : Typology of the walls tested by Elmanshawi et al. (2010).	42
Figure 2.37 : Force-displacement diagram and cracking pattern (Costa et al., 2012).	43
Figure 2.38 : Test setup of the tests performed by Corradi et al. (2008).	44
Figure 2.39 : Specimen tested by Augenti et al. (2011) and envelope curves.	45
Figure 2.40 : Failure modes of specimens tested by Lourenço et al. (2005).	46
Figure 2.41 : Load-displacement curves of the walls (Lourenço et al., 2005).	46
Figure 3.1 : Schematic layout of the planned shear test configuration.	49
Figure 3.2 : (a) Isometric, (b) longitudinal and (c) plan views of a cramp and pin assumed for a prototype wall.	51
Figure 3.3 : Three-dimensional layouts of the tested walls.	53
Figure 3.4 : Dimensions of the cramps and pins.	54
Figure 3.5 : Stone units of a cramp reinforced wall.	57
Figure 3.6 : Stone units of a cramp and pin reinforced wall.	57
Figure 3.7 : Base profile used in the wall construction (dimensions in mm).	58
Figure 3.8 : Construction of the outer leaves.	59
Figure 3.9 : Construction of the rubble core.	59
Figure 3.10 : Fitting of the instrumented cramps.	60
Figure 3.11 : Installation of the pins.	61
Figure 3.12 : Design and construction of the test rig.	62
Figure 3.13 : Schematic view of the test setup.	62
Figure 3.14 : General layout of the test setup.	63
Figure 3.15 : Stiffened end plates at foundation level.	63
Figure 3.16 : Assembly of the upper boundary of the walls.	64
Figure 3.17 : Assembly for prevention of out-of-plane movement.	65
Figure 3.18 : Instrumentation of the specimens-Northern face.	68
Figure 3.19 : Instrumentation of the specimens-Southern face.	68
Figure 3.20 : Loading pattern.	70
Figure 4.1 : Single unit compression test specimens ready for testing.	74
Figure 4.2 : Test setup of single unit compression tests.	75
Figure 4.3 : Axial stress-strain relationships of all single unit compression tests.	77
Figure 4.4 : Typical distribution of macro-cracks in single unit compression tests.	78
Figure 4.5 : Comparison of strain gauge and LVDT measurements.	80
Figure 4.6 : Variation of (a) volumetric strain and (b) Poisson's ratio.	81
Figure 4.7 : Interaction of Young's modulus with compressive strength.	82
Figure 4.8 : General (a) and schematic (b) views of the testing apparatus according to ASTM C 99-87 (2006).	83
Figure 4.9 : Rubble masonry standard cylinder samples.	88
Figure 4.10 : Test setup for rubble masonry cylinder compression tests.	89
Figure 4.11 : Axial stress-strain relationships of rubble masonry compression tests.	89
Figure 4.12 : The damage conditions of rubble masonry cylinders.	90
Figure 4.13 : The test configuration of the splitting tests.	92
Figure 4.14 : The view of a typical specimen before and after the splitting test.	92
Figure 4.15 : Original cramps from Edirne II. Bayezid Mosque.	96
Figure 4.16 : Mounting of the metal samples to the acrylic filled moulds.	96

Figure 4.17 :	Grinding of the metal samples until a smooth surface was obtained. .	97
Figure 4.18 :	Polishing of the metal sample surfaces.	97
Figure 4.19 :	Polished and etched metal sample surfaces.	98
Figure 4.20 :	Microscope used for the examinations.	98
Figure 4.21 :	Micrographs of Sample 1 for (a) unetched 10X zoom, (b) etched 10X zoom, (c) unetched 20X zoom and (d) etched 20X zoom cases.	99
Figure 4.22 :	Micrographs of Sample 2 for (a) unetched 10X zoom, (b) etched 10X zoom, (c) unetched 20X zoom and (d) etched 20X zoom cases.	99
Figure 4.23 :	Shimadzu HMV micro-hardness testing device used in this study. ...	102
Figure 4.24 :	Indentations left on the surfaces during the micro-hardness tests.	102
Figure 4.25 :	BH vs. tensile strength (Elban et al., 1998), 1 ksi=6.895 MPa.	105
Figure 4.26 :	Machined test piece for tensile tests (units mm).	108
Figure 4.27 :	Stress-strain curves of new cramp material.	109
Figure 4.28 :	Views of model cramp and pin reinforcements after production.	110
Figure 4.29 :	(a) Triplet and (b) couplet initial shear test configurations.	111
Figure 4.30 :	General and schematic views of the initial shear strength test setup.	112
Figure 4.31 :	Shear and normal stress diagrams of specimen BK-025-2.	113
Figure 4.32 :	Typical shear stress-displacement diagrams for SET I specimens.	114
Figure 4.33 :	Surface textures of SET I (a) and SET II (b) stone units.	115
Figure 4.34 :	Typical shear stress-displacement diagrams for SET II specimens. .	115
Figure 4.35 :	Shear strength vs. normal stress for SET I and II specimens.	117
Figure 4.36 :	Stylus type profilometer for quantification of the surface textures. ...	118
Figure 4.37 :	Typical raw profiles obtained for smooth and rough surfaces.	119
Figure 4.38 :	Profile components (Muralikrishnan and Raja, 2009).	119
Figure 4.39 :	Setting apart of texture components for a smooth surface (5-5-A unit).	120
Figure 4.40 :	Setting apart of texture components for a rough surface (5-S3-C unit).	120
Figure 4.41 :	Views of the surfaces after testing SET I specimens (smooth surfaces).	124
Figure 4.42 :	Views of the surfaces after testing SET II specimens (rough surfaces).	125
Figure 4.43 :	Views of the single-leaf masonry compression test specimens.	127
Figure 4.44 :	Single-leaf masonry prism before and after the instrumentation.	127
Figure 4.45 :	Non-destructive ultrasound test on single-leaf prisms.	128
Figure 4.46 :	Typical damage observed in the single-leaf prism compression tests.	129
Figure 4.47 :	Views of the multi-leaf masonry compression test specimens.	130
Figure 4.48 :	Construction steps of the multi-leaf prisms.	130
Figure 4.49 :	LVDT layout of multi-leaf wallets compression tests.	131
Figure 4.50 :	Multi-leaf masonry prism before and after the instrumentation.	131
Figure 4.51 :	Non-destructive ultrasound test on multi-leaf prisms.	131
Figure 4.52 :	Typical damage observed in the multi-leaf masonry compression tests.	132
Figure 4.53 :	The compressive stress-strain diagrams for the single-leaf walls.	134
Figure 4.54 :	The compressive stress-strain diagrams for the multi-leaf walls.	134
Figure 4.55 :	Comparison of axial force-displacement curves of compression prisms.	135
Figure 4.56 :	Compressive stress-transverse deformation of multi-leaf prisms.	136

Figure 4.57 : Comparison of the stress-strain diagrams of inner and external leaves.	138
Figure 5.1 : Cracking patterns of M-25-C, M-50-C, M-75-C and M-100-C walls.	143
Figure 5.2 : Cracking patterns of M-50, M-50-C and M-50-CP walls.	145
Figure 5.3 : Cracking patterns of S-50-C and S-50-CP walls.	147
Figure 5.4 : Cracking patterns of M-50-C and S-50-C walls.	148
Figure 5.5 : Cracking patterns of M-50-CP and S-50-CP walls.	149
Figure 5.6 : Force-displacement diagrams for M-25-C, M-50-C, M-75-C and M-100-C.	150
Figure 5.7 : Envelope curves for M-25-C, M-50-C, M-75-C and M-100-C (pushing).	152
Figure 5.8 : Envelope curves for M-25-C, M-50-C, M-75-C and M-100-C (pulling).	152
Figure 5.9 : Diagonal strain-lateral displacement hysteresis diagrams for M-25-C, M-50-C, M-75-C and M-100-C.	155
Figure 5.10 : Variation of vertical deformations close to eastern side with respect to lateral displacements.	156
Figure 5.11 : Variation of vertical displacements at bond beam tips, M-25-C.	157
Figure 5.12 : Vertical deformations at the Western toes of M-25-C and M-100-C.	158
Figure 5.13 : Variation of transverse displacement between exterior leaves, M-25-C.	158
Figure 5.14 : Strain versus lateral displacement curves for instrumented cramps.	159
Figure 5.15 : Force-displacement diagrams for M-50, M-50-C and M-50-CP.	160
Figure 5.16 : Force-displacement diagrams for S-50-C and S-50-CP.	161
Figure 5.17 : Envelope curves for M-50, M-50-C and M-50-CP (pushing).	162
Figure 5.18 : Envelope curves for M-50, M-50-C and M-50-CP (pulling).	162
Figure 5.19 : Envelope curves for S-50-C and S-50-CP (pushing).	162
Figure 5.20 : Envelope curves for S-50-C and S-50-CP (pulling).	163
Figure 5.21 : Diagonal strain vs. lateral displacement; M-50, M-50-C and M-50-CP.	166
Figure 5.22 : Diagonal strain vs. lateral displacement; S-50-C and S-50-CP.	166
Figure 5.23 : Envelope curves for M-50-C and S-50-C (pushing).	167
Figure 5.24 : Envelope curves for M-50-C and S-50-C (pulling).	167
Figure 5.25 : Envelope curves for M-50-CP and S-50-CP (pushing).	168
Figure 5.26 : Envelope curves for M-50-CP and S-50-CP (pulling).	168
Figure 5.27 : Diagonal strain vs. lateral displacement; M-50-C and S-50-C.	170
Figure 5.28 : Diagonal strain vs. lateral displacement; M-50-CP and S-50-CP.	170
Figure 6.1 : Idealisation of experimental envelope curve with bilinear relationship.	172
Figure 6.2 : Bilinear idealized diagrams of M-25-C, M-50-C, M-75-C and M-100-C.	174
Figure 6.3 : Bilinear idealized diagrams of M-50, M-50-C and M-50-CP.	175
Figure 6.4 : Bilinear idealized diagrams of M-50, M-50-CP, S-50-C and S-50-CP.	175
Figure 6.5 : Ultimate resistance of the idealised diagram vs. axial stress level.	176
Figure 6.6 : Equivalent yield point drift ratio (DR_e) vs. axial stress level.	177
Figure 6.7 : Ultimate drift ratio of the idealised diagram vs. axial stress level.	178
Figure 6.8 : Comparison of average and cyclic ductility factors.	180
Figure 6.9 : Variation of secant stiffness with respect to axial stress level.	182

Figure 6.10 : Variation of secant stiffness with respect to reinforcement utilization.	183
Figure 6.11 : Variation of normalized stiffness with normalized displacements....	184
Figure 6.12 : Variation of dissipated energy with respect to each cycle.	185
Figure 6.13 : Variation of normalized cumulative energy for M-25-C, M-50-C, M-75-C and M-100-C walls.	187
Figure 6.14 : Variation of normalized cumulative energy for M-50-C, M-50, M-50-CP, S-50-C and S-50-CP walls.	188
Figure 6.15 : Hysteretic energy dissipation (modified from Priestley et al. (1996)).	190
Figure 6.16 : Variation of equivalent damping ratio for M-25-C, M-50-C, M-75-C and M-100-C walls.	191
Figure 6.17 : Variation of equivalent damping ratio for M-50-C, M-50, M-50-CP, S-50-C and S-50-CP walls.	191
Figure 6.18 : Typical axial stress-strain curve obtained in the pre-compression stage.	193
Figure 6.19 : Variation of shear strength with axial stress level.	195
Figure 6.20 : Influence of wall typology on the shear strength.	195
Figure 6.21 : Principal stresses at the given coordinate system.	196
Figure 6.22 : Geometric parameters for shear modulus (ASTM E 564-06, 2006).	200
Figure 6.23 : Damage patterns of masonry shear wall.	201
Figure 6.24 : Types of friction failure (Jäger and Schöps, 2008).	203
Figure 6.25 : Failure envelope of the Mann and Müller model (modified from Graubner and Kranzler (2005)).	205
Figure 6.26 : Mann and Müller model for single-leaf specimens.	207
Figure 6.27 : Mann and Müller model for single-leaf specimens (close-up view).	207
Figure 6.28 : Mann and Müller model for multi-leaf specimens.	208
Figure 6.29 : Mann and Müller model for multi-leaf specimens (close-up view).	208
Figure 6.30 : Turnsek and Cacovic model for single-leaf specimens.	210
Figure 6.31 : Turnsek and Cacovic model for single-leaf specimens (close-up view).	210
Figure 6.32 : Turnsek and Cacovic model for multi-leaf specimens.	211
Figure 6.33 : Turnsek and Cacovic model for multi-leaf specimens (close-up view).	211
Figure 6.34 : Performance levels for URM (Bosiljkov et al., 2010).	214
Figure 6.35 : Limit states for M-25-C, M-50-C, M-75-C and M-100-C.	216
Figure 6.36 : Limit states for M-50-C, M-50 and M-50-CP.	216
Figure 6.37 : Limit states for S-50-C and S-50-CP.	217
Figure 6.38 : Variation of drift ratio at limit states with respect to precompression.	218
Figure 6.39 : Variation of drift ratio values at limit states with respect to axial stress.	220
Figure 6.40 : Idealised force-deformation response in ASCE/SEI 41-06 (2007). ..	222
Figure 7.1 : Modelling approaches for masonry structures (modified from Mistler et al., 2006).	226
Figure 7.2 : Failure mechanisms in micro-modelling (Senthivel and Lourenço, 2009).	228
Figure 7.3 : Parts used in the construction of the micro-model.	230
Figure 7.4 : Assembly of the outer leaf in ABAQUS.	230
Figure 7.5 : Spring elements representing the cramps.	231

Figure 7.6 : Boundary conditions of the micro-model	231
Figure 7.7 : Linear and quadratic brick elements in ABAQUS.	232
Figure 7.8 : Bending deformation of a linear element with reduced integration (ABAQUS, 2009).	232
Figure 7.9 : Meshed models of single- and multi-leaf masonry wall specimens.	233
Figure 7.10 : Uniaxial tension response in CDP model (ABAQUS, 2009).	234
Figure 7.11 : Uniaxial compression response in CDP model (ABAQUS, 2009). ..	234
Figure 7.12 : Post failure stress-displacement curve (ABAQUS, 2009).	236
Figure 7.13 : Postfailure stress-fracture energy curve (ABAQUS, 2009).	236
Figure 7.14 : Yield surface in plane stress space (ABAQUS, 2009).	237
Figure 7.15 : Compressive stress-inelastic strain curves for the micro-modelling.	238
Figure 7.16 : Hard contact and softened contact properties (ABAQUS, 2009).	240
Figure 7.17 : Frictional behaviour in the tangential direction of contact interfaces (ABAQUS, 2009).	241
Figure 7.18 : Load and displacement application zones.	242
Figure 7.19 : Scheme for central difference integration (Dunne and Petrinic, 2005).	244
Figure 7.20 : Smooth step amplitude curve (ABAQUS, 2009).	247
Figure 7.21 : Assemblies for modelling of uniaxial compression tests.	248
Figure 7.22 : Meshed geometries of uniaxial compression models.	249
Figure 7.23 : Damage observed in FEA and compression tests of stone prisms. ...	253
Figure 7.24 : Analytical and experimental compressive stress-strain curves for single-leaf masonry.	254
Figure 7.25 : Analytical and experimental compressive load-displacement curves for multi-leaf masonry.	255
Figure 7.26 : Damage observed in FEA and compression tests of multi-leaf prisms (longitudinal direction).	256
Figure 7.27 : Damage observed in FEA and compression tests of multi-leaf prisms (transverse direction).	257
Figure 7.28 : Comparison of the energies for compression loading.	258
Figure 7.29 : Deformed shapes obtained from FEA and experiment (S-50-C).	260
Figure 7.30 : Principal stress and plastic strain distributions at peak load (S-50-C).	260
Figure 7.31 : Deformed shapes obtained from FEA and experiment (M-50).	261
Figure 7.32 : Principal stress and plastic strain distributions at peak load (M-50).	261
Figure 7.33 : Deformed shapes obtained from FEA and experiment (M-25-C).	262
Figure 7.34 : Principal stress and plastic strain distributions at peak load (M-25-C).	263
Figure 7.35 : Deformed shapes obtained from FEA and experiment (M-50-C).	264
Figure 7.36 : Principal stress and plastic strain distributions at peak load (M-50-C).	264
Figure 7.37 : Deformed shapes obtained from FEA and experiment (M-75-C).	265
Figure 7.38 : Principal stress and plastic strain distributions at peak load (M-75-C).	266
Figure 7.39 : Deformed shapes obtained from FEA and experiment (M-100-C)...	266
Figure 7.40 : Principal stress and plastic strain distributions at peak load (M-100-C).	267
Figure 7.41 : Comparison of typical kinetic and internal energies (M-100-C).	268
Figure 7.42 : Raw and filtered load-displacement curves for FEA.	269

Figure 7.43 : Load-displacement response for micro-modelling (S-50-C and M-50).	270
Figure 7.44 : Load-displacement response for micro-modelling (M-25-C, M-50-C, M-75-C and M-100-C).	271
Figure A.1 : Typical layout of the multi-leaf wall.	298
Figure A.2 : Vertical stresses obtained for the self-weight of the structure.	298
Figure A.3 : Vertical stresses obtained for 0.4g lateral loading.	298
Figure A.4 : Tensile stress distribution obtained for lateral loading.	299
Figure A.5 : Vertical stresses for Mehmet Ağa Mosque under self-weight.	299
Figure A.6 : Vertical stresses for Mehmet Ağa Mosque under earthquake loads.	300
Figure A.7 : Vertical stresses for Beyazıt II Mosque, self-weight (stresses in kPa).	301
Figure A.8 : Vertical stresses for Beyazıt II Mosque, lateral loads (unit kPa).	301
Figure A.9 : Vertical stresses for Atik Ali Paşa Mosque, vertical loads (unit kPa).	302
Figure B.1 : Stylus type profilometer developed for the current study.	304
Figure B.2 : Typical profiles obtained for (a) smooth and (b) rough surfaces.	304
Figure B.3 : Procedure of digital filtering (Thomas, 1999).	306
Figure B.4 : Obtaining texture components for a rough texture (5-S3-C stone unit).	307
Figure B.5 : Obtaining texture components for a smooth texture (5-5-A stone unit).	308
Figure C.1 : Axial stress-axial strain behaviour of M-25-C specimen.	309
Figure C.2 : Cycle 1, M-25-C after 0.55 mm displacement (0.05% drift ratio).	310
Figure C.3 : Cycle 2, M-25-C after 1.10 mm displacement (0.10% drift ratio).	310
Figure C.4 : Cycle 3, M-25-C after 2.75 mm displacement (0.25% drift ratio).	311
Figure C.5 : Cycle 4a, M-25-C after 5.50 mm displacement (0.50% drift ratio).	311
Figure C.6 : Cycle 4b, M-25-C after 5.50 mm displacement (0.50% drift ratio).	312
Figure C.7 : Cycle 4c, M-25-C after 5.50 mm displacement (0.50% drift ratio).	312
Figure C.8 : Cycle 5, M-25-C after 8.25 mm displacement (0.75% drift ratio).	313
Figure C.9 : Cycle 5, M-25-C after 8.25 mm displacement (0.75% drift ratio).	313
Figure C.10 : Cycle 6a, M-25-C after 11.0 mm displacement (1.00% drift ratio).	314
Figure C.11 : Cycle 6b, M-25-C after 11.0 mm displacement (1.00% drift ratio).	314
Figure C.12 : Cycle 6c, M-25-C after 11.0 mm displacement (1.0% drift ratio).	315
Figure C.13 : Cycle 7, M-25-C after 13.75 mm displacement (1.25% drift ratio).	315
Figure C.14 : Cycle 8, M-25-C after 16.50 mm displacement (1.50% drift ratio).	316
Figure C.15 : Cycle 9, North face of M-25-C after 2.0% drift ratio.	316
Figure C.16 : Cycle 9, South face of M-25-C after 2.0% drift ratio.	317
Figure C.17 : Cycle 9, (a) East and (b) West faces of M-25-C after 22.0 mm displacement (2.0% drift ratio).	317
Figure C.18 : Cracks passing through the tips of the cramps.	318
Figure C.19 : Lateral load-displacement diagram for specimen M-25-C.	320
Figure C.20 : Ratio of dissipated energy at each cycle to total, M-25-C.	320
Figure C.21 : Envelope curve of lateral load-displacement diagram, M-25-C.	321
Figure C.22 : Vertical displacements at the bond beam end-points, M-25-C.	322
Figure C.23 : Vertical deformations near the Western and Eastern sides, M-25-C.	322
Figure C.24 : Vertical deformations at the toes and heels of the wall M-25-C.	323
Figure C.25 : Strain variation for mid-height cramp (M-25-C wall).	324
Figure C.26 : Average strains of instrumented cramps of wall M-25-C.	324

Figure C.27 : Detachment between the exterior leaves (Eastern side, M-25-C). ...	324
Figure C.28 : Axial stress-axial strain behaviour of M-50-C specimen.	325
Figure C.29 : Cycle 1, M-50-C after 0.55 mm displacement (0.05% drift ratio). ..	326
Figure C.30 : Cycle 2, M-50-C after 1.10 mm displacement (0.10% drift ratio). ..	326
Figure C.31 : Cycle 3, M-50-C after 2.75 mm displacement (0.25% drift ratio). ..	327
Figure C.32 : Cycle 4a, M-50-C after 5.50 mm displacement (0.50% drift ratio). ..	327
Figure C.33 : Cycle 4a, vertical cracks between the inner and outer leaves, M-50-C.	328
Figure C.34 : Cycle 4b, M-50-C after 5.50 mm displacement (0.50% drift ratio). ..	328
Figure C.35 : Cycle 4c, M-50-C after 5.50 mm displacement (0.50% drift ratio). ..	329
Figure C.36 : Cycle 5, M-50-C after 8.25 mm displacement (0.75% drift ratio). ..	329
Figure C.37 : Cycle 6a, M-50-C after 11.0 mm displacement (1.00% drift ratio). ..	330
Figure C.38 : Cycle 6b, M-50-C after 11.0 mm displacement (1.00% drift ratio). ..	330
Figure C.39 : Cycle 6c, M-50-C after 11.0 mm displacement (1.0% drift ratio). ..	331
Figure C.40 : Cycle 7, M-50-C after 13.75 mm displacement (1.25% drift ratio). ..	331
Figure C.41 : Cycle 8, M-50-C after 16.50 mm displacement (1.50% drift ratio). ..	332
Figure C.42 : Cycle 8, North face of M-50-C after 1.50% drift ratio.	332
Figure C.43 : Cycle 8, South face of M-50-C after 1.50% drift ratio.	333
Figure C.44 : Cycle 8, (a) East and (b) West faces of M-50-C after 16.50 mm displacement (1.50% drift ratio).	333
Figure C.45 : Damage in the vicinity of cramp anchors, M-50-C.	334
Figure C.46 : Damage on the outer and inner leaves, M-50-C.	336
Figure C.47 : Lateral load-displacement diagram for specimen M-50-C.	336
Figure C.48 : Envelope curve of lateral load-displacement diagram of M-50-C. ..	337
Figure C.49 : Ratio of the dissipated energy to total dissipated energy, M-50-C... ..	337
Figure C.50 : Variation of vertical displacements at the bond beam, M-50-C.	338
Figure C.51 : Vertical deformations near the Western and Eastern sides, M-50-C.	338
Figure C.52 : Vertical deformations at the toes and heels of the wall M-50-C.	339
Figure C.53 : Strain variation for lower cramp, M-50-C.	339
Figure C.54 : Average strains of instrumented cramps of wall M-50-C.	340
Figure C.55 : Bending of the cramps, M-50-C.	340
Figure C.56 : Detachment between the exterior leaves (Western side, M-50-C). ..	341
Figure C.57 : Axial stress-axial strain behaviour of M-75-C specimen.	342
Figure C.58 : Cycle 1, M-75-C after 0.55 mm displacement (0.05% drift ratio). ..	342
Figure C.59 : Cycle 2, M-75-C after 1.10 mm displacement (0.10% drift ratio). ..	343
Figure C.60 : Cycle 3, M-75-C after 2.75 mm displacement (0.25% drift ratio). ..	343
Figure C.61 : Cycle 4a, M-75-C after 5.50 mm displacement (0.50% drift ratio). ..	344
Figure C.62 : Cycle 4b, M-75-C after 5.50 mm displacement (0.50% drift ratio). ..	344
Figure C.63 : Cycle 4c, M-75-C after 5.50 mm displacement (0.50% drift ratio). ..	344
Figure C.64 : Cycle 5, M-75-C after 8.25 mm displacement (0.75% drift ratio). ..	345
Figure C.65 : Cycle 6a, M-75-C after 11.0 mm displacement (1.00% drift ratio). ..	346
Figure C.66 : Cycle 6b, M-75-C after 11.0 mm displacement (1.00% drift ratio). ..	346
Figure C.67 : Cycle 6c, M-75-C after 11.0 mm displacement (1.00% drift ratio). ..	346
Figure C.68 : Cycle 6c, Eastern side of M-75-C (11.0 mm, 1.00% drift ratio).	347
Figure C.69 : Cycle 6c, crushing of stone units after 1.0% drift ratio (M-75-C). ..	347
Figure C.70 : Cycle 7, M-75-C after 13.75 mm displacement (1.25% drift ratio). ..	348
Figure C.71 : Cycle 7, North face of M-75-C (13.75 mm, 1.25% drift ratio).	348
Figure C.72 : Cycle 7, South face of M-75-C (13.75 mm, 1.25% drift ratio).	349

Figure C.73 : Cycle 7, (a) East and (b) West faces of M-75-C after 1.25% drift ratio.	349
Figure C.74 : Damage in the vicinity of cramp anchors, M-75-C.	350
Figure C.75 : Lateral load-displacement diagram for specimen M-75-C.	352
Figure C.76 : Envelope curve of lateral load-displacement diagram of M-75-C.	352
Figure C.77 : Ratio of the dissipated energy to total dissipated energy, M-75-C.	353
Figure C.78 : Vertical deformations near the Western and Eastern sides, M-75-C.	353
Figure C.79 : Vertical deformations at the toes and heels of the wall M-75-C.	354
Figure C.80 : Strain variation for the cramp at mid-height of the wall, M-75-C.	354
Figure C.81 : Average strains on instrumented cramps of wall M-75-C.	355
Figure C.82 : Detachment between the external leaves of M-75-C.	355
Figure C.83 : Axial stress-axial strain behaviour of M-100-C specimen.	356
Figure C.84 : Observed damage during the axial loading of M-100-C specimen.	357
Figure C.85 : Cycle 1, M-100-C after 0.55 mm displacement (0.05% drift ratio).	358
Figure C.86 : Cycle 2, M-100-C after 1.10 mm displacement (0.10% drift ratio).	358
Figure C.87 : Cycle 2, Southern face of M-100-C after 0.10% drift ratio.	358
Figure C.88 : Cycle 3, M-100-C after 2.75 mm displacement (0.25% drift ratio).	359
Figure C.89 : Cycle 3, Northern face of M-100-C after 0.25% drift ratio.	359
Figure C.90 : Cycle 4a, M-100-C after 5.50 mm displacement (0.50% drift ratio).	360
Figure C.91 : Cycle 4a, Northern face of M-100-C after 0.50% drift ratio.	360
Figure C.92 : Cycle 4b, M-100-C after 5.50 mm displacement (0.50% drift ratio).	361
Figure C.93 : Cycle 4c, M-100-C after 5.50 mm displacement (0.50% drift ratio).	361
Figure C.94 : Cycle 4c, Southern face of M-100-C after 0.50% drift ratio.	362
Figure C.95 : Cycle 5, toe crushing and spalling of M-100-C after 0.75% drift ratio.	362
Figure C.96 : Cycle 5, M-100-C after 8.25 mm displacement (0.75% drift ratio).	363
Figure C.97 : Vertical cracks on the Southern face of M-100-C (0.75% drift ratio).	363
Figure C.98 : Cycle 6a, M-100-C after 11.0 mm displacement (1.00% drift ratio).	364
Figure C.99 : Cycle 6a, Buckling of the vertical column after 1.00% drift ratio.	364
Figure C.100 : Northern face of M-100-C specimen after the test.	365
Figure C.101 : Southern face of M-100-C specimen after the test.	365
Figure C.102 : (a) East and (b) West faces of M-100-C after the test.	366
Figure C.103 : Lateral load-displacement diagram for specimen M-100-C.	367
Figure C.104 : Envelope curve of lateral load-displacement diagram of M-100-C.	367
Figure C.105 : Ratio of the dissipated energy to total dissipated energy, M-100-C.	369
Figure C.106 : Variation of vertical displacements at the bond beam tips, M-100-C.	369
Figure C.107 : Vertical deformations near the Western and Eastern sides, M-100-C.	370
Figure C.108 : Vertical deformations at the toes and heels of the wall M-100-C.	370
Figure C.109 : Strain variation for the cramp at mid-height of the wall, M-100-C.	371

Figure C.110 : Average strains of instrumented cramps of wall M-100-C.	371
Figure C.111 : Detachment between the exterior leaves (Western side, M-100-C).	372
Figure C.112 : Axial stress-strain behaviour of M-50 specimen.	372
Figure C.113 : Cycle 1, M-50 after 0.55 mm displacement (0.05% drift ratio).	373
Figure C.114 : Cycle 2, M-50 after 1.10 mm displacement (0.10% drift ratio).	373
Figure C.115 : Cycle 2, Northern face of M-50 after 0.10% drift ratio.	374
Figure C.116 : Cycle 3, M-50 after 2.75 mm displacement (0.25% drift ratio).	374
Figure C.117 : Cycle 3, Northern and Eastern faces of M-50 after 0.25% drift ratio.	375
Figure C.118 : Cycle 4a, M-50 after 5.50 mm displacement (0.50% drift ratio). ...	375
Figure C.119 : Cycle 4a, Northern face of M-50 after 0.50% drift ratio.	376
Figure C.120 : Cycle 4a, Western side of M-50 after 0.50% drift ratio.	376
Figure C.121 : Cycle 4b, M-50 after 5.50 mm displacement (0.50% drift ratio). ...	377
Figure C.122 : Cycle 4b, Southern face of M-50 after 0.50% drift ratio.	377
Figure C.123 : Cycle 4c, M-50 after 5.50 mm displacement (0.50% drift ratio). ...	378
Figure C.124 : Cycle 5, Northern face of M-50 after 0.75% drift ratio.	378
Figure C.125 : Cycle 5, M-50 after 8.25 mm displacement (0.75% drift ratio).	379
Figure C.126 : Cycle 5, Toe crushing after 0.75% drift ratio (M-50).	379
Figure C.127 : Cycle 6a, M-50 after 11.0 mm displacement (1.00% drift ratio). ...	380
Figure C.128 : Cycle 6a, Northern face of M-50 after 1.00% drift ratio.	380
Figure C.129 : Cycle 6b, M-50 after 11.0 mm displacement (1.00% drift ratio). ...	380
Figure C.130 : Cycle 6b, Extensive spalling of toe after 1.00% drift ratio (M-50). ...	381
Figure C.131 : Northern face of M-50 specimen after the test.	381
Figure C.132 : Southern face of M-50 specimen after the test.	382
Figure C.133 : (a) Eastern and (b) Western faces of M-50 after the test.	382
Figure C.134 : Lateral load-displacement diagram for specimen M-50.	383
Figure C.135 : Envelope curve of lateral load-displacement diagram of M-50.	385
Figure 5.136 : Ratio of the dissipated energy of each cycle to total dissipated, M-50.	385
Figure C.137 : Variation of vertical displacements at the bond beam tips, M-50. ...	386
Figure C.138 : Vertical deformations near the Western and Eastern sides, M-50. ...	386
Figure C.139 : Vertical deformations at the lower corners of the M-50 wall.	387
Figure C.140 : Detachment between the external leaves of the M-50 wall.	387
Figure C.141 : Axial stress-axial strain behaviour of S-50-C specimen.	388
Figure C.142 : Cycle 1, S-50-C after 0.55 mm displacement (0.05% drift ratio)...	389
Figure C.143 : Cycle 2, S-50-C after 1.10 mm displacement (0.10% drift ratio)...	389
Figure C.144 : Cycle 3, S-50-C after 2.75 mm displacement (0.25% drift ratio)...	390
Figure C.145 : Cycle 4a, S-50-C after 5.50 mm displacement (0.50% drift ratio). ...	390
Figure C.146 : Cycle 4b, S-50-C after 5.50 mm displacement (0.50% drift ratio). ...	391
Figure C.147 : Cycle 4c, S-50-C after 5.50 mm displacement (0.50% drift ratio). ...	391
Figure C.148 : Cycle 5, S-50-C after 8.25 mm displacement (0.75% drift ratio)...	392
Figure C.149 : Sliding between the stone courses, S-50-C (0.75% drift ratio).	392
Figure C.150 : Cycle 6a, S-50-C after 11.0 mm displacement (1.00% drift ratio). ...	393
Figure C.151 : Cycle 6b, S-50-C after 11.0 mm displacement (1.00% drift ratio). ...	393
Figure C.152 : Cycle 6c, S-50-C after 11.0 mm displacement (1.0% drift ratio)...	393
Figure C.153 : Cycle 7, S-50-C after 13.75 mm displacement (1.25% drift ratio). ...	394
Figure C.154 : Cycle 8, S-50-C after 16.50 mm displacement (1.50% drift ratio). ...	394
Figure C.155 : Cycle 8, North face of S-50-C at the end of the test.	395
Figure C.156 : Cycle 8, South face of S-50-C at the end of the test.	395

Figure C.157 : Cycle 8, (a) East and (b) West faces of S-50-C at the end of the test.	396
Figure C.158 : Lateral load-displacement diagram for specimen S-50-C.	398
Figure C.159 : Envelope curve of lateral load-displacement diagram of S-50-C.	398
Figure C.160 : Ratio of the dissipated energy to total dissipated energy, S-50-C.	399
Figure C.161 : Variation of vertical displacements at the bond beam tips, S-50-C.	399
Figure C.162 : Vertical deformations at the toes and heels of the wall S-50-C.	400
Figure C.163 : Strain variation for the cramp at lower half of the wall, S-50-C.	400
Figure C.164 : Average strains of instrumented cramps of wall S-50-C.	401
Figure C.165 : Axial stress-strain behaviour of M-50-CP specimen.	401
Figure C.166 : Cycle 1, M-50-CP after 0.55 mm displacement (0.05% drift ratio).	402
Figure C.167 : Cycle 2, M-50-CP after 1.10 mm displacement (0.10% drift ratio).	402
Figure C.168 : Cycle 3, M-50-CP after 2.75 mm displacement (0.25% drift ratio).	403
Figure C.169 : Cycle 3, M-50-CP after 2.75 mm displacement (0.25% drift ratio).	403
Figure C.170 : Cycle 4a, M-50-CP after 5.50 mm displacement (0.50% drift ratio).	404
Figure C.171 : Cycle 4a, (a) cracks near the pins and cramps, (b) vertical cracks between the inner and outer leaves, M-50-CP.	404
Figure C.172 : Cycle 4b, M-50-CP after 5.50 mm displacement (0.50% drift ratio).	405
Figure C.173 : Cycle 4c, M-50-CP after 5.50 mm displacement (0.50% drift ratio).	405
Figure C.174 : Cycle 5, M-50-CP after 8.25 mm displacement (0.75% drift ratio).	406
Figure C.175 : Cycle 6a, M-50-CP after 11.0 mm displacement (1.00% drift ratio).	406
Figure C.176 : Cycle 6b, M-50-CP after 11.0 mm displacement (1.00% drift ratio).	407
Figure C.177 : Cycle 6c, M-50-CP after 11.0 mm displacement (1.00% drift ratio).	407
Figure C.178 : Cycle 7, M-50-CP after 13.75 mm displacement (1.25% drift ratio).	408
Figure C.179 : Cycle 8, M-50-CP after 16.50 mm displacement (1.50% drift ratio).	408
Figure C.180 : Cycle 9a, M-50-CP after 22.0 mm displacement (2.0% drift ratio).	409
Figure C.181 : Northern face of M-50-CP after 2.0% drift ratio.	409
Figure C.182 : Southern face of M-50-CP after 2.0% drift ratio.	409
Figure C.183 : (a) Eastern and (b) Western faces of M-50-CP after 2.0% drift ratio.	410
Figure C.184 : Damage in the vicinity of pins, M-50-CP.	412
Figure C.185 : Damage on the outer and inner leaves, M-50-CP.	412
Figure C.186 : Lateral load-displacement diagram for specimen M-50-CP.	413
Figure C.187 : Envelope curve of lateral load-displacement diagram of M-50-CP.	413

Figure C.188 : Ratio of dissipated energy to total dissipated energy, M-50-CP. ...	414
Figure C.189 : Variation of vertical displacements at the bond beam tips, M-50-CP.	415
Figure C.190 : Vertical deformations near the Western and Eastern sides, M-50-CP.	415
Figure C.191 : Vertical deformations at the lower corners of the wall M-50-CP. .	416
Figure C.192 : Strain variation for the cramp at lower half of the wall, M-50-CP.	416
Figure C.193 : Detachment between the external leaves of the M-50-CP wall.	417
Figure C.194 : Axial stress-axial strain behaviour of S-50-CP specimen.	417
Figure C.195 : Cycle 1, S-50-CP after 0.55 mm displacement (0.05% drift ratio).	418
Figure C.196 : Cycle 2, S-50-CP after 1.10 mm displacement (0.10% drift ratio).	418
Figure C.197 : Cycle 3, S-50-CP after 2.75 mm displacement (0.25% drift ratio).	419
Figure C.198 : Cycle 4a, S-50-CP after 5.50 mm displacement (0.50% drift ratio).	419
Figure C.199 : Cycle 4b, S-50-CP after 5.50 mm displacement (0.50% drift ratio).	420
Figure C.200 : Cycle 4c, S-50-CP after 5.50 mm displacement (0.50% drift ratio).	420
Figure C.201 : Cycle 5, S-50-C after 8.25 mm displacement (0.75% drift ratio)...	421
Figure C.202 : Cycle 6a, S-50-C after 11.0 mm displacement (1.00% drift ratio).	421
Figure C.203 : Cycle 6b, S-50-CP after 11.0 mm displacement (1.00% drift ratio).	422
Figure C.204 : Cycle 6c, S-50-CP after 11.0 mm displacement (1.0% drift ratio).	422
Figure C.205 : Cycle 7, S-50-CP after 13.75 mm displacement (1.25% drift ratio).	423
Figure C.206 : Cycle 8, S-50-CP after 16.50 mm displacement (1.50% drift ratio).	423
Figure C.207 : Cycle 9a, S-50-CP after 22.00 mm displacement (2.00% drift ratio).	424
Figure C.208 : Cycle 9a, North face of S-50-CP after 1.50% drift ratio.	424
Figure C.209 : Cycle 8, South face of S-50-CP after 1.50% drift ratio.	425
Figure C.210 : Cycle 8, (a) East and (b) West faces of S-50-CP at the end of the test.	425
Figure C.211 : Lateral load-displacement diagram for specimen S-50-CP.	427
Figure C.212 : Envelope curve of lateral load-displacement diagram of S-50-CP.	427
Figure C.213 : Ratio of dissipated energy to total dissipated energy, S-50-CP.....	428
Figure C.214 : Variation of vertical displacements at the bond beam tips, S-50-CP.	428
Figure C.215 : Vertical deformations at the toes and heels of the wall S-50-CP. ...	429
Figure C.216 : Strain variation for mid-height cramp, S-50-CP.....	429
Figure C.217 : Average strains of instrumented cramps of wall S-50-CP.....	430
Figure D.1 : Principal stress distributions of S-50-C model.	431
Figure D.2 : Maximum principal plastic strain distributions of S-50-C model.	431
Figure D.3 : Minimum principal plastic strain distributions of S-50-C model.	431
Figure D.4 : Principal stress distributions of M-50 model.	432
Figure D.5 : Maximum principal plastic strain distributions of M-50 model.	433
Figure D.6 : Minimum principal plastic strain distributions of M-50 model.....	434
Figure D.7 : Principal stress distributions of M-25-C model.....	435
Figure D.8 : Maximum principal plastic strain distributions of M-25-C model.	436
Figure D.9 : Minimum principal plastic strain distributions of M-25-C model.....	437

Figure D.10 :	Principal stress distributions of M-50-C model.....	438
Figure D.11 :	Maximum principal plastic strain distributions of M-50-C model...	439
Figure D.12 :	Minimum principal plastic strain distributions of M-50-C model. ..	440
Figure D.13 :	Principal stress distributions of M-75-C model.....	441
Figure D.14 :	Maximum principal plastic strain distributions of M-75-C model...	442
Figure D.15 :	Minimum principal plastic strain distributions of M-75-C model. ..	443
Figure D.16 :	Principal stress distributions of M-100-C model.....	444
Figure D.17 :	Maximum principal plastic strain distributions of M-100-C model.	445
Figure D.18 :	Minimum principal plastic strain distributions of M-100-C model.	446

SEISMIC BEHAVIOUR OF HISTORICAL STONE MASONRY MULTI-LEAF WALLS

SUMMARY

Having a long dating history Turkey has been home to numerous civilizations starting from pre-historic ages. Apart from artefacts and goods, the most distinctive physical remains are the structures such as houses, city walls, castles, state buildings, religious structures and infrastructure systems around which these civilizations had flourished. Most of the historical structures in Turkey are under high seismic risks since the overall seismicity of the region is considerably high. Particularly, in and around the old imperial city of Istanbul, where thousands of monumental structures from the Byzantine, Roman and Ottoman periods exist, this risk is even more solid. Since the knowledge on the traditional masonry walls and masonry constituents of the historical structures in Turkey is scarce, particularly from the structural engineering perspective, new experimental and analytical studies are needed for development of well established assessment and intervention procedures. This study intends to enlarge the experimental and theoretical data concerning the behaviour of the multi-leaf stone masonry walls under cyclic in plane loads that represent the seismic effects.

In order to achieve this purpose, the mechanical characteristics of multi-leaf stone masonry walls of classical era Ottoman Imperial structures are investigated through an experimental and analytical campaign. A prototype wall, representative of the investigated walls is designed and scaled down wall specimens produced for testing are subjected to combined axial and inplane shear forces. Besides the tests of these walls, mechanical characteristics of the components of historical masonry walls are also examined through material tests. Finally, a micro-modelling approach is followed for explicit finite element modelling of the tested specimens.

This study consists of five main parts. In the first part, the wall typologies observed in the classical age Ottoman monumental structures are investigated through literature and site surveys and main characteristics of a prototype wall are set. A survey of the existing limited literature on the main aspects of the historical masonry walls of classical age Ottoman period monumental structures reveal that it is generally typical for Ottoman architecture, particularly for the monumental structures in and around Istanbul, to have thick cut stone block layers made of limestone (Küfeki) covering the cores of the walls that generally consists of rubble stone and mortar. Limestone blocks are generally connected to each other via metal cramps (and in some cases with pins), which are fixed to the stone units with molten lead, in case no mortar is used between the units. The site surveys to three ancient structures, where these details could be partially observed due to existing damage, also back the literature supplied information and supply some numerical values on the geometrical aspects.

The second part aims to investigate the mechanical characteristics of the constituent materials utilized in historical stone masonry walls. Several tests are performed on materials such as single Küfeki stone units, mortar prisms, rubble masonry standard cylinders, original and new iron reinforcements. As a result of these material tests; mechanical characteristics (such as compressive strength, Young's modulus, tensile strength, friction coefficient, stress-strain diagrams) of the constituents are obtained in details. Moreover, uniaxial compression tests performed on single- and multi-leaf stone assemblies are also presented. Accordingly, the mean compressive strength of limestone material is obtained to be around 18 MPa through testing of single units. However, in case the stone units are juxtaposed and single-leaf masonry prisms are tested, the mean strength almost halves. Uniaxial compression tests on multi-leaf walls addresses to an even lower compressive strength value that is about 20% of the single-unit tests (~3.7 MPa). One striking feature observed during the tests is the wide scatter obtained even for the compressive strength of single unit stone specimens. The shear behaviour of the dry joints between the stone units is characterized via a number of initial shear tests. In addition to that, the effect of surface texture on the shear behaviour of dry joints is also investigated. The coefficient of friction value for smooth interfaces is obtained as 0.76, while it is 0.74 for rough interfaces. The lower friction coefficient of the rough surfaces can be attributed to the decreased effective contact area of rough surfaces. Uniaxial compression and splitting tests on rubble masonry standard cylinders indicate that the heterogeneous structure of the composite material, that is formed by a mortar matrix and randomly distributed stone pieces placed without compaction, leads to a significant scatter in the mechanical characteristics. The chemical, microscopic and metallurgical investigations carried out on the original cramps obtained from Edirne II. Bayezid Mosque show that the original metal material used for production of cramps and pins had very low carbon content and exhibited a mainly ferritic character with several slags and oxides in the microstructure. By using the empirical equations available in the literature, an approximate tensile strength in the order of 300-350 MPa can be obtained, so that a representative steel material was chosen for the experimental study.

Third part aims to investigate the effects of three main parameters on the shear behaviour of historical multi-leaf stone masonry walls. These parameters are: Variation of pre-compression stress, existence of cramps and pins and existence of rubble masonry core. The series of experiments consist of quasi-static cyclic shear tests performed on eight 1/3 scaled-down model walls (1.20×1.20×0.30 m in size). During the application of lateral displacement cycles, vertical stress level is kept constant as one of the following values: 0.25, 0.50, 0.75 and 1.00 MPa. The shear compression tests provided extensive data on the failure modes, lateral load capacity, ductility, damage characteristics and energy dissipation capabilities. In general, the walls exhibited a shear-dominated failure mode, which is a mixture of the shear and flexure type modes. The damage evolution generally started with vertical cracks passing through the stone units that can even emerge at small lateral displacement levels. Further increase of the lateral displacement caused the arise of shear damage in the form of diagonal cracks and vertical cracks aligned on a diagonal band. After opening of the widespread diagonal cracks and failure of some of the cramp anchors, a rocking-type mechanism took place about the lower corners of the walls. Utilization of cramps was seen to be influential on the crack distribution. However, since the tensile strength of the stone units are low, cracking of units inhibited the

effective utilization of cramp reinforcement. Despite that, considerable increase in lateral load and displacement capabilities of cramp reinforced walls was observed with respect to the unreinforced masonry wall. The introduction of pins enhanced the load and deformation capacities. Comparison of the shear test results of single-leaf and multi-leaf walls show that the existence of the low strength rubble masonry enhance the integrity of the wall, so that stresses can be better distributed among the stone units. The obtained hysteresis loops showed that the walls exhibited a highly nonlinear response with significant residual displacements after each load reversal.

In the fourth part, the experimental envelopes of tested walls are idealized with bilinear curves. Additionally, a number of parameters; such as strength, ductility, stiffness, energy dissipation and damping, which are the main indicators of seismic performance are evaluated. Accordingly, all walls exhibited considerable deformation (with ductility values that vary between 5.2 and 10.7) and energy dissipation capabilities (equivalent damping ratios varying between 8 and 21%). An increase in the axial stress level led to higher initial secant stiffness values under lateral actions. Unlike the introduction of pins, the introduction of cramps did not bring an increase in the stiffness parameter. An analogy with the Mohr-Coulomb expression given by the Eurocode 6 (1996) and Turkish Seismic Design Code (2007) showed that the shear strength under zero compression strength was about 0.09 MPa while the global friction coefficient turned out to be 0.30. Furthermore, reasonably good predictions are made by using two widely used failure criterion available in the literature. Finally, comparison of drift ratios of performance levels (such as immediate occupancy, life safety and collapse prevention) recommended by a number of major codes and guidelines showed that there exists a strong need for further experimental and analytical studies, and modification of these documents.

The fifth part aims to develop a finite element model that can successfully predict the damage development and load-displacement response of the tested uniaxial compression and shear compression specimens. For this purpose an explicit analysis is carried out by following a micro-modelling approach. For both types of tests, the development of damage, locations of cracking and crushing, load flow paths and load-displacement response are successfully predicted by the developed explicit finite element model.

ÇOK TABAKALI TARİHİ TAŞ YIĞMA DUVARLARIN DEPREM YÜKLERİ ALTINDA DAVRANIŞI

ÖZET

Derin bir tarihe sahip olan ülkemiz, tarih öncesi çağlardan başlayarak pek çok medeniyete ev sahipliği yapmıştır. Bu uygarlıkların etrafında geliştiği ve hüküm sürdüğü çok sayıda yapı halen ayakta. Ancak, Türkiye'nin bulunduğu bölgenin yüksek depremselliği nedeniyle bu yapıların önemli bir bölümü risk altındadır. Özellikle; Roma, Bizans ve Osmanlı İmparatorluklarına başkentlik yapmış İstanbul ve çevresinde bu risk daha da belirgindir. Buna karşın, geleneksel yapı yığma duvarları ve bunların bileşenleri konusunda ülkemizde gerçekleştirilmiş çalışma sayısı oldukça sınırlıdır. Konuya yapı mühendisliği disiplini penceresinden bakan çalışmalar söz konusu olduğunda ise bu sınırlılık daha da belirgin hale gelmektedir. Bu nedenle gerçekleştirilen çalışma kapsamında, tarihi yapılarımızda sıklıkla karşılaşılan çift cidarlı taş yığma duvarların düzlem içi doğrultuda etkiyen tekrarlı yük etkileri altındaki davranışının, deneysel ve teorik olarak incelenmesi hedeflenmiştir.

Belirtilen hedefe ulaşabilmek amacıyla, klasik dönem Osmanlı İmparatorluğu yapılarında sıklıkla kullanılan çift cidarlı taş duvarların mekanik davranışının inceleneceği deneysel bir çalışma planlanmıştır. Gerçek boyutlu yığma yapı duvarları ve donatıları laboratuvar ortamında test edilebilecek şekilde belirli bir oranda küçültülerek numune duvar üretimi yapılmış ve duvar numuneleri deneye tabi tutulmuştur. Ayrıca bu numunelerde kullanılan malzemelerin mekanik özelliklerini belirleyebilmek için yine laboratuvar ortamında çok sayıda deney gerçekleştirilmiştir. Son olarak, deneye tabi tutulan numunelerin davranışları mikro modellemeye yaklaşımı ile oluşturulan sonlu elemanlar modeli kullanılarak sayısal ortamda incelenmiştir.

Tez çalışması beş ana bölümden oluşmaktadır. Birinci bölümde Osmanlı İmparatorluğu anıtsal yapılarında rastlanılan duvar tipolojileri literatür ve saha çalışmaları ile incelenerek deneysel çalışmaya esas oluşturacak prototip duvara ait temel özellikler ortaya konulmuştur. Yazılı kaynaklar ve konunun uzmanlarına başvurularak, özellikle İstanbul ve çevresinde bulunan klasik dönem Osmanlı İmparatorluğu anıtsal yapı duvarlarının önemli bir kısmının çift cidarlı duvarlarla inşa edildiği, dış cidarların genellikle kesme Küfeki taşı, iç cidarların ise moloz dolgu ile teşkil edildiği görülmüştür. Yaygın olarak kullanılan bir kireç taşı tipi olan Küfeki taşı ile oluşturulan taş birimlerin demir kenet (ve bazı durumlarda zıvanalar ile) birbirlerine bağlandığı ve bu donatıların taş birimlere ankrajında eritilmiş kurşun kullanıldığı tespit edilmiştir. Üç adet tarihi yapı üzerinde gerçekleştirilen saha çalışması ile literatürde yer alan bu bilgiler gözlemlerle desteklenmiş ve çeşitli geometrik özellikler yerinde tespit edilmiştir.

İkinci kısımda tarihi taş yığma duvarlarda kullanılan malzeme bileşenlerinin temel mekanik özelliklerinin incelenmesi amaçlanmıştır. Bu malzeme deneyleri sonucunda söz konusu malzemelerin; basınç ve çekme dayanımı, elastisite modülü, gerilme-şekildeğiştirme ilişkisi ve sürtünme katsayısı gibi temel mekanik özellikleri konusunda detaylı bilgiler elde edilmiştir.

İkinci bölümde tarihi taş duvarlarda kullanılan malzeme bileşenlerinin temel mekanik özelliklerinin incelenmesi amaçlanmıştır. Çok sayıda taş, harç, taş ve moloz yığma, tarihi kenet ve yeni üretim çelik sac numunesi üzerinde deneyler yapılmıştır. Bu malzeme deneyleri sonucunda; sözkonusu malzemelerin basınç ve çekme dayanımı, elastisite modülü, sürtünme katsayısı, gerilme-şekildeğiştirme ilişkisi gibi temel mekanik özellikleri konusunda detaylı bilgiler verilmiştir. Ayrıca, tek ve çok tabakalı yığma duvar numuneleri üzerinde yapılan eksenel basınç deneyleri ile elde edilen sonuçlar da sunulmuştur. Buna göre, kireç taşı malzemenin ortalama basınç dayanımı 18.0 MPa olarak elde edilmiştir. Bununla birlikte, taş birimlerin bir araya getirilmesi ile oluşturulan taş yığma duvar parçasının basınç dayanımı bu değer yaklaşık yarısı olarak elde edilmiştir. Moloz çekirdek tabakasını da içeren çok tabakalı duvar parçalarının basınç dayanımı ise taş blokların yaklaşık %20'si mertebesinde dir. Özellikle taş birim basınç deneylerinden elde edilen mekanik özelliklerde gözlenen dağılımın büyüklüğü dikkat çekici boyutlardadır. Taş birimler arasındaki kuru derzlerin kayma davranışı başlangıç kayma deneyleri ile incelenmiştir. Bu deneyler esnasında, kuru derzlerdeki taş yüzeylerin pürüzlülüğünün sürtünme katsayısı üzerindeki etkisi de ele alınmıştır. Sürtünme katsayısı, makine ile kesilen taşların yüzeyleri için 0.76, pürüzlendirme yapılmış olanlar için ise 0.74 olarak elde edilmiştir. Moloz çekirdek tabakasını oluşturan malzemenin basınç dayanımı ve elastisite modülü gibi özellikleri standart silindir basınç deneyleri ile, çekme dayanımı ise yarma deneyleri ile belirlenmiştir. Taş birimlere nazaran çok daha düşük dayanıma sahip bu malzemenin oldukça heterojen bir yapıya sahip olması nedeniyle elde edilen mekanik özellikler, beklendiği gibi, büyük bir dağılım sergilemektedir. Duvar numunelerinin taş bloklarını birbirine bağlayacak olan donatıların (kenet ve zıvanaların) mekanik özelliklerini belirlemek ve uygun bir malzeme seçebilmek amacıyla kapsamlı bir literatür taraması yapılmıştır. Bu çalışmaya ek olarak Edirne II. Bayezid Cami restorasyon sahasından temin edilmiş iki adet orijinal kenet üzerinde fiziksel, metalürjik ve kimyasal deneyler yapılmıştır. Bu deneyler sonucunda, orijinal demir donatıların oldukça düşük karbon muhteva ettikleri ve iç yapılarının ferrit kristalleri ve bu kristaller arasına dağınık ve yaygın olarak serpilmiş inklüzyonlar nedeniyle oldukça heterojen olduğu gözlenmiştir. Yüzey sertliği ölçümlerinden elde edilen değerler ve literatürde bulunan ampirik ifadeler kullanılarak elde edilen çekme dayanımları ise 300-350 MPa civarlarındadır.

Üçüncü bölümde çalışma kapsamında ele alınan üç ana değişkenin çok tabakalı tarihi duvarların kayma davranışına etkisi ele alınmıştır. Bu değişkenler, eksenel gerilme düzeyi, kenet ve zıvana kullanımı ve dış tabakalar arasında iç moloz dolgunun bulunmasıdır. Planlanan deneysel çalışma, yerdeğiştirme kontrollü olarak etkilenen tekrarlı kesme kuvvetlerine maruz bırakılmış 1/3 ölçekli (1.20×1.20×0.30 m boyutlarında) sekiz adet duvar numunesinden meydana gelmektedir. Bu numunelerin iki adedi moloz dolgu içermeyip yalnızca iki dış tabakadan oluşmaktadır (1.20×1.20×0.20 m boyutlarında). Yatay doğrultudaki yerdeğiştirme çevrimlerinin etkilenmesi esnasında duvar üzerine etkiyen eksenel yük sabit tutulmuştur (0.25, 0.50, 0.75 ve 1.00 MPa gerilme düzeyleri). Gerçekleştirilen deneysel çalışma ile incelenen

çok tabakalı taş duvarların göçme modu, yatay yük kapasitesi, süneklik ve enerji yutma özellikleri konularında bilgiler elde edilmiştir. Duvarların davranışında temel olarak kesme etkileri egemen olmuş ancak eksenel gerilme düzeyine bağlı olarak eğilme etkileri de kısmen ortaya çıkmıştır. Hasar gelişimi taşlarda ve komşu tabakaların arayüzlerinde oluşan düşey kılcal çatlaklar ile başlamıştır. Yatay yer değiştirmelerin artmasıyla birlikte çapraz doğrultularda oluşan çatlaklar belirginleşmiştir. Bu çatlakların ileri düzeyde genişlemesi ve bazı noktalarda kenet ankrajlarının devre dışı kalması sonucunda, duvarlar, alt köşe bölgeleri etrafında devrilme davranışı sergilemeye başlamıştır. Kenetlerin kullanılması çatlak dağılımını etkilemekle birlikte, kullanılan Küfeki taşının çekme dayanımının sınırlı olması nedeniyle taşlar çatlamış ve kenetlerin etkinlikleri sınırlanmıştır. Zıvanaların kullanılması hasarın oluşum şeklini pek etkilememekle birlikte dayanım ve yerdeğiştirme yeteneğini belirgin şekilde iyileştirmiştir. Çok tabakalı ve tek cidarlı duvarlar için elde edilen deney sonuçlarının karşılaştırılması sonucunda, oldukça düşük dayanımlı moloz dolgunun duvarın bütünlüğünü sağlamak konusunda önemli katkısı olduğu görülmüştür. Elde edilen çevrim eğrileri incelendiğinde numunelerin doğrusal elastik sınırın oldukça ötesine geçtikleri ve her bir yük çevriminde önemli miktarda kalıcı deformasyona uğradıkları tespit edilmiştir.

Dördüncü bölümde deneysel yük-yerdeğiştirme ilişkileri iki doğrudan oluşacak şekilde idealleştirilmiştir. Histeretik çevrimler ve idealleştirilmiş eğriler kullanılarak dayanım, süneklik, rijitlik ve enerji yutma gibi deprem performansı üzerinde oldukça etkili olan parametreler ele alınmıştır. Buna göre, tüm duvar numunelerinin ihmal edilemeyecek düzeyde deformasyon yapma yeteneğine ve 5.2 ile 10.7 arasında değişen yerdeğiştirme sünekliği değerlerine sahip olduğu görülmüştür. Hasarın sınırlı düzeyde kaldığı çevrimlerde dahi %8 ila 21 arasında değişen eşdeğer viskoz sönüm oranlarının telaffuz edilebileceği tespit edilmiştir. Duvar üzerine etkiyen eksenel gerilme miktarının artmasıyla beraber duvar başlangıç rijitliğinin de arttığı ve zıvana kullanımının aksine kenet kullanımının rijitlikte önemli bir artışa neden olmadığı tespit edilmiştir. Farklı eksenel gerilme düzeylerinde elde edilen dayanımlar gözönünde bulundurulduğunda, Eurocode 6 (1996) ve Türk Deprem Yönetmeliği (2007) tarafından da benzeştirilerek kullanılan Mohr-Coulomb ifadesindeki genelleştirilmiş sürtünme katsayısının 0.30, eksenel yüksüz durumdaki kayma dayanımının ise 0.09 MPa olarak elde edildiği görülmüştür. Literatürde yaygın olarak kullanılan ve donatısız yığma duvarlar için geliştirilmiş iki farklı göçme zarfı modeli ile yapılan dayanım tahminleri oldukça başarılı olmuştur. Son olarak, performansa dayalı değerlendirme yöntemlerine esas teşkil eden performans düzeylerinin (Hemen Kullanım, Can Güvenliği ve Göçmenin Önlenmesi) karşı gelebileceği öteleme oranı değerleri deneysel sonuçlar gözönünde bulundurularak tespit edilmeye çalışmıştır. Elde edilen sonuçlar belli başlı yönetmeliklerle karşılaştırılmış ve bunun sonucunda, özellikle kesme etkilerinin hakim olduğu farklı duvar tipleri için yürütülecek deneysel çalışmaların bu yönetmeliklerin daha da geliştirilmesi açısından gerekli olduğu sonucuna ulaşılmıştır.

Beşinci bölüm, bu çalışmada ele alınan ve basınç ve kayma deneyleri gerçekleştirilen duvarların sonlu elemanlar yöntemi ile sayısal ortamda modellenmesini içermektedir. Bu maksatla, genellikle dinamik olayların analizinde kullanılan açık çözüm yöntemi statik deneyler için uyarlanmıştır. Modelin oluşturulması esnasında mikro-modelleme yaklaşımı takip edilmiştir. Bu yaklaşım için dış tabakaları oluşturan taş birimler, moloz dolgu çekirdek, taş birimleri birbirine bağlayan kenetler ayrı ayrı

geometrik modele dahil edilmiş. Duvarın bu bileşenlerinin birbirleri arasındaki bağlantıların mekanik özellikleri tanımlanmış ve her bir bileşenin kendine ait malzeme mekanik özellikleri malzeme deneylerinden elde edilen sonuçlar kullanılarak sonlu elemanlar programına girilmiştir. Bu çalışma sonucunda, deneylerde gözlenen hasar gelişimi, göçme modları, gerilme ve şekildeğiştirme dağılımları, çatlak ve yük-yerdeğiştirme ilişkileri oldukça başarılı bir şekilde elde edilmiştir.

1. INTRODUCTION

Having a long dating history, Turkey has been home to numerous civilizations starting from pre-historic ages. Through the time line, inhabitants of Anatolia ranged between small communities to super powers such as Hittite, Roman and Ottoman Empires. Though vanished in history, each of these states left their traces both culturally and physically. Apart from artefacts and goods, the most distinctive physical remains are the structures such as houses, city walls, castles, state buildings, religious structures and infrastructure systems around which these civilizations had flourished.

Most of the historical structures, which are the architectural and cultural heritages of the world, were constructed with masonry walls. Being one of the oldest structural materials and structural systems, even today, masonry construction can find wide usage areas. Masonry is a composite structural system that consists of units laid on top of each other in varying arrangements, either with a bonding mortar or without any adhesion. It is obvious that the material of units (blocks), the type of the mortar and the construction techniques have always been subject to changes through the time line, mainly due to factors such as the local culture and wealth, the knowledge of materials and tools, climate, the availability of materials and architectural reasons.

For centuries, masonry structures had been designed and built by architects and masons equipped with talent, traditions and experience. Since the ancient times, masters passed their knowledge on the empirical technology of structural engineering to their apprentices, (Lourenço, 1998). European masons of the medieval used geometrical techniques developed in time, by using the trial and error method and built massive Romanesque and Gothic structures. Geometrical skills were of great importance, so that members of the masons lodge were sworn secrecy, (Lourenço, 1998). Meanwhile in the east, Ottoman Empire was extending its territories towards south and west of Anatolia. The need for construction of roads and bridges for the marching armies, castles for protection of borders and routes, housing units, state buildings, bathhouses, kervansarays and mosques necessary for the inhabitation of

the conquered areas required a well-developed organization. In return, an extensive multi-layered organization, Hassa Mimarlar Teskilati, had been set up, (Taş, 2003).

With the initiation of the Renaissance “the golden age in science and arts”, theoretical explanations were sought. Leonardo da Vinci worked on the force transfer mechanism in arches. Researchers performed experiments with chains to model the thrust lines of arches and domes and graphic solutions were developed. In the meantime, Master Architect Sinan excelled in skills of construction and hundreds of structures had been built throughout the empire, including the masterpieces such as Selimiye and Süleymaniye Mosques. In 16th century, Simon Stevinus published a book on statics, *Mathematicorum Hypomnemata de Statica*, which provided the basics for the solution of structural problems in the 19th century. While these steps were taken in the area of structural analysis, in 1676 R. Hooke established the mathematical theory of materials science with his famous quote, “Ut tension sic vis” (as the extension, so the force). This elasticity law for homogeneous materials has been widely used since then, (Lorenço, 1998).

With the industrial revolution’s introduction of mass production methods for cast iron members and cement, structural masonry came to a halt in most regions of the world. The new materials (steel and concrete) were durable, strong, easily mouldable and inexpensive. The rise of reinforced concrete as a structural material also affected the research done in the era of structural engineering. Despite the leaps in numerical methods developed in the 20th century, being the most widely used building material of history, masonry has not benefitted enough. Hopefully, the recent decades’ uprising concepts such as conservation and restoration of the historical heritage, brought masonry into scientific community’s interest.

The threat to existence of historical masonry structures, not only stems from introduction of new building systems but also from natural actions such as earthquakes and from factors like fatigue and strength degradation, accumulated damage, wind and temperature loads, soil settlement and the lack of proper structural design by the original constructors, (Lourenço, 2005).

In order to assure the future of these heritage structures, especially that are in high seismic risk areas, it is necessary to assess these structures realistically for probable

seismic and non-seismic loads. Following the assessment phase, decision for the intervention might be given following the general principals of structural restoration.

However, this task is quite complicated because of limited resources and uncertainties. Generally, allocated resources (funds and time) are not enough to study the mechanical behaviour of masonry, which includes in-situ and laboratory tests. The difficulties for an adequate structural analysis as listed by Lourenço (2004) are:

- Geometry data is missing.
- Information about the inner core of the structural elements are missing.
- Characterization of the mechanical properties of the materials used is difficult and expensive.
- Mechanical properties may significantly vary due to workmanship and use of natural materials.
- Significant changes in the core and constitution of structural elements, associated with long construction periods.
- Construction sequence is unknown.
- Existing damage in the structure is unknown.
- Regulations and codes for new structures are non-applicable.

To minimize the adverse effects of expected actions (such as earthquakes); investigation of the applied masonry construction techniques and materials via carefully planned in situ and laboratory experimental studies that are backed with appropriate analytical approaches has paramount importance. These studies would not only greatly enhance our engineering capabilities but also help us to regain the long forgotten knowledge on these structures. However, in contrast with the old dating history of masonry structures and masonry construction techniques, it has only recently been subject to engineering community's interest. Moreover, due to the specific local properties and conditions, direct adaptation of the results of the studies performed abroad cannot be sufficient to evaluate the present state of the historical structures in Turkey. Sadly, although Turkey is rich of monumental structures, the number of scientific studies performed in Turkey that aimed to understand the mechanics of masonry, constructed with local techniques and materials, against the seismic effects is very scarce.

1.1 Masonry Walls at a Glance

Masonry walls have several structural and functional roles such as; supplying a shelter from sight, wind, rain and temperature; supporting the weight of floor and roof systems; providing resistance to lateral forces such as wind and earthquake effects.

Most walls in historical construction are load-bearing ones. In some structural systems non-load-bearing walls can exist. Even in these cases, though negligible, they are still structural members providing lateral strength and resistance.

Due to its composite structure, masonry enables various material wise and geometry wise alternatives for the construction of structural members and structural systems. Different masonry types can be defined by using combinations of these materials (such as bricks, ashlar, blocks, adobes and irregular stones) and typology alternatives.

In the past, bricks, which were either sun-dried or kiln-burnt, were generally made of mud or clay. The earliest bricks were adobes, which included sun-dried mud. Adobe was the first prefabricated element. The main material of historical bricks was clay. Clay was mixed with other materials and then this mixture was baked to temperatures of approximately 1000 °C to make bricks more resistant. Although the colour of bricks was usually red, other colours were also present due to over burning or incomplete burning. While in Roman brickwork, the height of bricks was between 30 to 40 mm, in the middle ages it varied between 20 to 55 mm. The mechanical properties of bricks depend on the drying process, its composition, its baking temperature, its history and the decay phenomena. While the compression strength of good quality bricks is between 15-30 MPa, the tensile strength is about 5-8% of the compression strength. The elasticity modulus increasing with the strength varies between 5000 and 10000 MPa. The Poisson ratio is approximately 0.15-0.20, (Croci 2000). A recent study performed on brick units of 18th century row houses located in Istanbul (Turkey) points to much lower strength and elasticity modulus values. According to Ispir et al. (2010), the compressive strength of bricks varies between 1.9 and 9.8 MPa and elasticity modulus varies between 34 and 196 MPa.

Stone units were preferred particularly in the cases where the material was available locally. The origins of most frequently used stone types were igneous or sedimentary

rocks. Igneous rock is formed deep inside the earth by the melting of solid matter. Granite is an example of this category. Wind or sea-borne deposits produce sedimentary rocks, (Croci, 2000). Sandstone, limestone and tufa are examples of sedimentary rocks. Sedimentary rocks can be split along their natural bedding planes using picks, crowbars and wedges and dressed to size by chisels. Sand was used as an abrasive to grind bedding surfaces flat and toothless saws were utilized to cut rock to desired sizes (Drysdale et al., 1994). The mechanical characteristics of stones may vary largely depending on factors like the type of stone and the nature of the quarry. The compression strength is generally between 30-100 MPa for sandstone, 20-200 MPa for limestone, and 3-10 MPa for volcanic tufa, (Croci, 2000).

In historical structures, the brick or stone units were generally juxtaposed by using mortars that were based on binders such as clay, bitumen, chalk, lime or cement. Mortar generally consists of a binder, an aggregate and water. The durability of mortar is determined by factors such as its porosity, materials used and the proportions of materials. The compression strength of historical mortar may be around 2.5 MPa. The tensile strength is about 5% of the compressive strength (Croci, 2000).

Masonry walls of the historical structures are composed of more or less regular alternating layers of units and mortar. Bond typologies such as a) common (or American), b) cross (or English), c) Flemish, d) stack and e) stretcher bond were generally used (Figure 1.1).

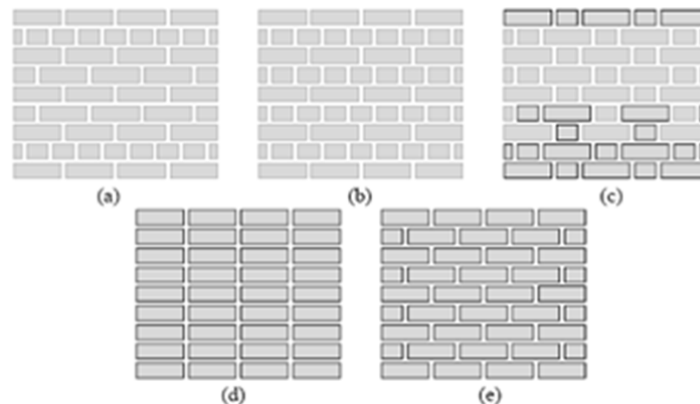


Figure 1.1 : Different arrangements for brick masonry (Lourenço, 1998).

In the case of stone masonry walls, dry joints without any mortar or with very thin layer of mortar, were also preferred. Stone masonry walls might either be

homogenous with regular bond patterns or inhomogeneous (and irregular) depending on the material employed, financial power of the owner, skill of the builders and the technical knowledge of the time (Roberti et al., 2004).

Depending on the size of the structure and the acting loads, masonry walls could be constructed either as single-leaf (single-wythe) or multi-leaf walls. Single-leaf walls had a thickness of the masonry unit whereas multi-leaf walls were made by constructing two outer leafs of stonework or brickwork and infilling the intervening space with rubble (Drysdale et al., 1994).

Though different masonry materials are available, stone masonry has always found wide use in the European and Asian urban centres. The possible reasons for the wide usage of stone might be attributed to the local availability, durability, strength and visual impact of the stone as a construction material. In the Ottoman architecture, due to its visually appealing impact, in many monumental structures, the outer leaves were formed with ashlar masonry that composed of finely cut stones and dry joints. Reasonably, considering the financial and workmanship related issues, the core between the ashlar outer leaves were made with a cheaper and weaker material that consisted of irregular stone or brick pieces and mortar. In some cases, interestingly, the rubble core may constitute the greater part of the thickness.

After site surveys carried out at failed historical structures, such as the collapsed Cathedral of Noto or severely damaged churches Santissimo Crocefisso and Santissima Annunziata, it has been seen that high compressive loading in multiple-leaf pillars and walls play very important roles in the structural behaviour, (Binda et al., 2006; Vintzileou and Tassios, 1995). Most structural problems exhibited by three-leaf walls and pillars result from the insufficiency or absence of connection between the leaves, the weakness of the inner core, or the deterioration of the mortar in the external joints, (Binda et al., 2006 and Valuzzi et. al, 2004).

Despite the complexity of the behaviour of the multi-leaf walls, the road to understanding and interpreting the responses of structures erected with this kind of walls to exerted loads or time-dependent effects, passes from scientific studies mimicking the realistic conditions. In addition to that, retrofitting the vulnerable walls with suitable interventions require proper insight on the structural behaviour and failure mechanisms.

1.2 Objectives of the Thesis

Most of the historical structures in Turkey are under high seismic risks since the overall seismicity of the region is considerably high. Particularly, in and around the old imperial city of Istanbul, where thousands of monumental structures from the Byzantine, Roman and Ottoman periods exist, this risk is even more solid. For instance, recently, the 1999 Marmara earthquakes caused severe damage to the historical structures near the epicentre, as seen in Figure 1.2. The monumental structures in Istanbul could luckily save the day with minor or moderate damages since the epicentre of the earthquake was about 100 km away from the city.



Figure 1.2 : Mosque damaged by the 1999 Marmara earthquake (Ilki et al., 2006).

In history of Turkey, several destructive earthquakes such as the ones occurred in the years 1509, 1719, 1754, 1766 and 1894 AD, caused severe damage to the structures in the Marmara region. Publications and papers such as Ambraseys and Finkel (1995), Mazlum (2001) and Ilki et al. (2006) give an outline of the damages caused by the historical earthquakes on the buildings some of which do not exist anymore (Figure 1.3).

Since the knowledge on the traditional masonry walls and masonry constituents of the historical structures in Turkey is scarce, particularly from the structural engineering perspective, this study intends to enlarge the experimental and theoretical data concerning the behaviour of the multi-leaf stone masonry walls under cyclic in plane loads that represent the seismic effects. In order to accomplish this task, this study aims to:

- Have a glance at the most widespread stone masonry wall typologies available in and around Istanbul city, particularly the multi-leaf stone masonry walls constructed at the Ottoman Empire classical period.
- Perform several material characterization tests on the wall constituents such as Küfeki stone, steel and mortar. These tests aim to determine the physical, mechanical, metallurgical and chemical properties of these materials, where available.
- Experimentally investigate the performance of masonry walls under combined axial stress and cyclic lateral loading. The wall specimens are to be constructed by using the material qualities and techniques comparable to that of Ottoman Empire classical period structures.
- Define seismic performance levels for the investigated type of walls, based on the drift ratios.
- Evaluate the experimental results via analytical expressions available in literature and considering the results of executed nonlinear finite element analyses.



Figure 1.3 : Grand Bazaar area after the 1894 Istanbul earthquake (Urekli, 1999).

1.3 Outline of the Thesis

The thesis includes a total of eight chapters and four annexes.

Chapter 1 addresses to an introduction, a brief history of masonry walls and their components and the objectives, and outline of the thesis.

Chapter 2 reports a review on the main properties of the historical stone masonry walls in Turkey and briefly summarizes the available literature on the mechanical behaviour of stone masonry walls

Chapter 3 presents the main aspects of the experimental study aimed to investigate the in-plane behaviour of multi-leaf stone masonry walls. Determination of the wall characteristics, design of experiments, construction of the specimens and testing procedures are provided in detail. Details of the arrangement of boundary conditions, test setup, the positioning of sensors and data acquisition system are also presented.

Chapter 4 presents the identification of the material characteristics of the main constituents: Küfeki stone, metals and mortar. The material tests' results are presented for the stone and mortar samples tested under flexural tension and compression for assessment of their basic mechanical characteristics. Besides the individual tests on stone and mortar specimens, results of the compression and splitting tension tests, performed on the rubble masonry standard cylinders, are also summarized. These rubble masonry samples represent the core material utilized in the inner leaf of the multi-leaf walls. Since the experimental campaign included usage of steel members, which were also available in the original stone masonry walls with dry joints, characterization of two original cramps via metallurgical tests are also done. Then, the results of the metallurgical tests are compared with the data available in the literature, so that a representative steel material to be used in the wall specimens can be selected. Second part of Chapter 4 aims to investigate the interaction of Küfeki stone units with each other when they are subjected to shear forces and compressive forces along their interfaces. Shear strengths and friction coefficients obtained from the initial shear tests performed on two different surface typologies (smooth and rough surfaces) of the stone units are obtained and compared. Test performed on both typologies lead to interesting results and the causes of these results are sought by identifying the roughness levels utilized on the surfaces of the initial shear test specimens. Finally, the compressive behaviour of single-leaf and

multi-leaf masonry is investigated by carrying out compression tests. With the help of these tests, the compressive stress-strain behaviour and related mechanical characteristics of the tested walls are obtained and compared with the data available in literature.

The experimental study carried out to investigate the behaviour of multi-leaf stone masonry walls provided data on the failure mode, lateral load capacity, ductility and damage characteristics. The data collected from the performed experiments are analysed and appropriate comparisons are presented in Chapter 5. For this purpose, the evolution of damage and failure modes of the tested walls are summarized with references to the investigated test parameters such as pre-compression stress level, configurations of reinforcement and presence of internal rubble masonry core. Finally, the behaviour of the walls, grouped in comparison subsets with respect to investigated tests parameters, is evaluated quantitatively. For this purpose, the hysteresis loops obtained during the tests are presented and compared. Particularly the lateral load-displacement curves are discussed, and critical points on these diagrams are tabulated.

Chapter 6 targets to interpret the experimental data obtained from the multi-leaf wall shear tests. In order to achieve this goal, the load-displacement curves are first transformed into idealized equivalent bilinear curves. So that; the evaluation of stiffness, strength, deformability and other parameters are simplified. Then the lateral displacements in the form of drift ratios that correspond to critical points of the bilinear idealized diagrams are investigated and the ductility levels of the tested walls are discussed. In order to evaluate the variation of stiffness, firstly typical raw stiffness data obtained from the tests are presented. Then the normalized stiffness-displacement curves are given and compared with the empirical equations given in the literature. Finally, the energy dissipation capabilities and damping characteristics of the tested walls are evaluated.

Chapter 7 is devoted to evaluation of the compressive and in-plane shear behaviour of tested walls by using finite element analysis. The specimens are modelled through a micro-modelling approach and the results are compared with the experimental ones. For this purpose, uniaxial compression and shear compression tests are numerically investigated by performing a series of explicit analyses. During the

analyses, nonlinearities caused by the contact conditions and material behaviour are taken into account.

In Chapter 8, the main conclusions derived from the present research are pointed out. Moreover, suggestions for future work are also given.

As well as the main chapters mentioned above, more detailed information on some selected topics and the experimental results are presented in the appendices. The appendices include information on the justification of axial stress levels, the filtering of the raw surface profiles, detailed presentation of the experimental and finite element analysis results.

It should be noted that, in scope of this study, a MSc thesis was also prepared by Emre Doğu (Doğu, 2010) and submitted to Istanbul Technical University.

2. MULTI-LEAF MASONRY SHEAR WALLS

In the first part of this chapter, it is aimed to make a review on the main properties of the historical multi-leaf stone masonry walls in Turkey and briefly summarize the available literature on the characteristics of this type of walls. In addition, observations done on two Ottoman period monumental structures during a field trip are also presented.

The second part consists of a literature survey for compilation of the existing experimental studies carried out on stone masonry walls. The available compression and shear tests are briefly described and main results are summarized in tables.

2.1 Multi-Leaf Stone Masonry Walls in Turkey

It is generally typical for Ottoman architecture, particularly for the monumental structures in and around Istanbul, to have thick cut stone block layers made of limestone (Küfeki) covering the cores of the walls that generally consists of rubble stone and mortar (Yorulmaz and Ahunbay, 1986). Limestone blocks are generally connected to each other via metal cramps (and in some cases with pins), which are fixed to the stone units with molten lead, in case no mortar is used between the units. Cores of the walls are also generally reinforced with members made of timber (hatıl) or iron.

Though different classifications are possible for stone load bearing walls of historical masonry constructions, in this section, the classification based on the construction techniques indicated by Kolay (1999) and Tayla (2007) will be followed.

Multi-leaf walls made of natural stones might be broadly categorized as rubble stone and ashlar walls. No matter how the external leaves are processed, the common point of these two types of walls is the existence of a rubble masonry infill between the external leaves.

Multi-leaf rubble stone walls have been widely used since the late Roman period (i.e. in Milet, Priene and Heraklia) and the Byzantine period (widely used in fortifications

and city walls) as seen in Figure 2.1 (Kolay, 1999). Rubble stone masonry walls were generally constructed by using stones obtained from river beds or stone quarries, or in some cases from debris of older structures.

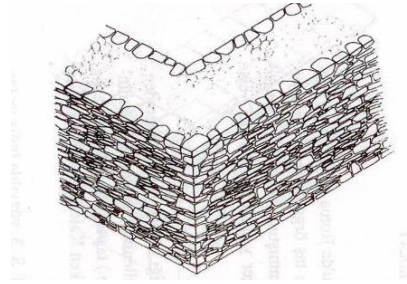


Figure 2.1 : Multi-leaf wall of a church in Priene (Kolay, 1999).

In the later periods, the usage of this type of walls continued in many structures (Kolay, 1999). Some examples of this type of construction are shown in Figure 2.2. In these figures, it is possible to distinguish different usages of multi-leaf rubble masonry walls. In all types, the uncoursed fieldstone rubble exterior leaves confine the rubble masonry infill. However, in Figure 2.2a larger connection cut-stones are used at the corner of the wall; in Figure 2.2b these corner stones do not exist, the wall in Figure 2.2c inhibits corner stones and alternating rows of stone and brick and in Figure 2.2d timber lintels are used for levelling the walls.

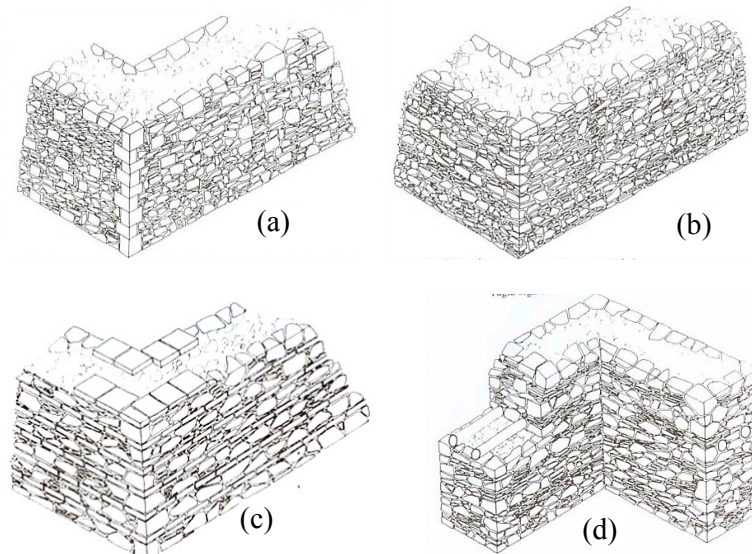


Figure 2.2 : Wall typologies from Anatolia (Kolay, 1999).

These rubble masonry walls were generally levelled at each 1.0-1.5 m height either by using more regular stones, bricks or generally with timber lintels (Tayla, 2007). When the used main blocks were very irregular in shape and small in relation to the

height of the wall, it was even hard to establish stability under self-weight loads. At those cases, the large gaps between the larger blocks were packed with smaller stones or pieces of bricks (UNDP/UNIDO, 1984).

In case of timber lintel usage, lintels with 8-12 cm thickness were placed on both sides of the walls (Tayla, 2007). Pine or cedar trees were preferred for timber lintel production due to their good resistance against environmental conditions. In some cases, these lintels were connected to each other by means of transverse ties at each 3-4 m, that supplied a connection between the walls leafs (particularly at the corner zones of the structures), Figure 2.3.

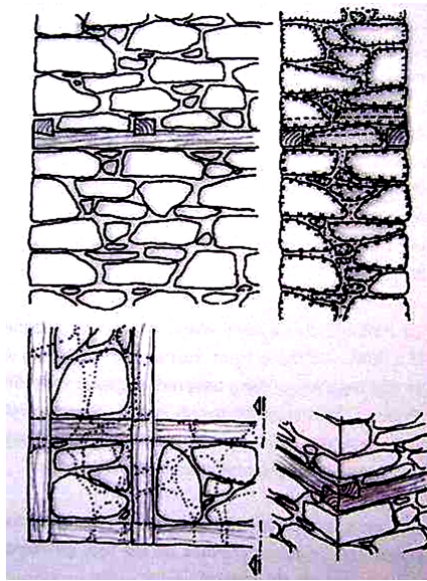


Figure 2.3 : Timber lintels in rubble masonry walls (Tayla, 2007).

Ashlar walls required more workmanship and financial power than the rubble stone walls. However, their stability, durability, strength and aesthetical appearance made them appealing for important structures.

The type of stones used in these walls were strongly dependent on the local availability of the material: Limestone in the Marmara region, trachyte around Afyon, andesites (Ankara stone) and desites in Mid-Anatolia, tuff around Kayseri, basalt around Diyarbakır and limestone around Mardin (Tayla, 2007).

Thanks to shaping of the blocks so that they fitted closely, which significantly improved the stability and reduced the stress concentrations and unwanted bending actions observed in rubble masonry walls, thereby enabled the adoption of slender proportions (UNDP/UNIDO, 1984).

The ashlar multi-leaf walls differ from each other either by the regularity of the stone units of the external leaves or by the type of joints connecting the stone units. The stone units might be finely cut or roughly-hewn. Joints between the stone units might include mortar or they might be dry joints without any mortar. Depending on the quality of workmanship and materials used, it may not be easy to distinguish some rough-hewn stone walls from ashlar walls with finely cut stone units (Tayla, 2007). Typically three different types of ashlar walls exist in the Ottoman architecture: Roughly-hewn stone walls with roughly tooled units, finely cut ashlar walls with thin mortar joints and finely cut stone ashlar walls without mortar joints (dry joints).

In the case of roughly-hewn stone walls, the sides of the stones are rough tooled and dressed with chisels. The mortar joints generally have uniform thicknesses.

Finely cut stone ashlar walls with mortar joints generally have regular stone courses, Figure 2.4. Stones may have length/height ratios from one to four (generally two or three). In general, Khorasan mortar was utilized at the joints of these walls (Tayla, 2007).

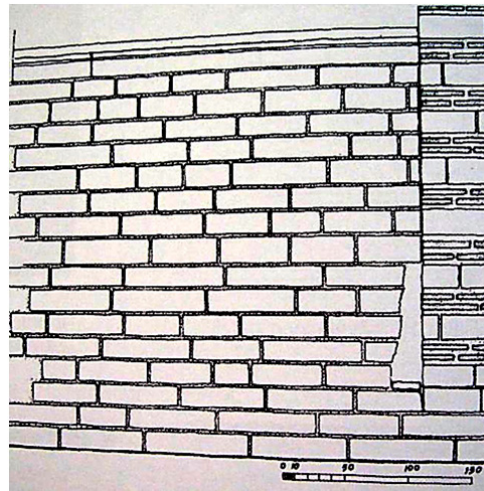


Figure 2.4 : Finely cut stone ashlar walls with thin mortar joints (Tayla, 2007).

Finely cut stone ashlar walls without mortar joints constitute a large group of structures in and around the Ottoman era's capital city Istanbul. Due to time and money consuming process required to construct this type of walls, they were generally used in aesthetically important structures. According to Tayla (2007), external leaves of these walls generally used to have 25-40 cm deep stones with length/height ratios of two or three (ratios such as one, four or five were also used in

some cases). The external stones were cut very finely so that the stones could fit without use of any mortar, Figure 2.5.



Figure 2.5 : Finely cut stone ashlar with dry joints (drawing by Tayla, 2007).

It is a well-known fact that since the antiquity, either metal or timber reinforcing elements had been used in the walls of the masonry structures. Obviously, the evolution of masonry reinforcement has required a long history that was shaped by the improvements in the production capabilities. Beginning from timber and easy to shape metals (such as copper and bronze), various techniques had been developed for the utilization of reinforcing elements in the masonry constructions. With the industrialization, tougher materials (such as iron, steel, titanium and other alloys) have been put into service.

According to Tanyeli (1990), usage of iron in Ottoman Architecture for structural purposes can be classified as: Singular elements such as cramps and pins; support elements such as lintels and connectors (kılıç, simit, etc.); closed ties running around the structures; dome confining elements, Figure 2.6.

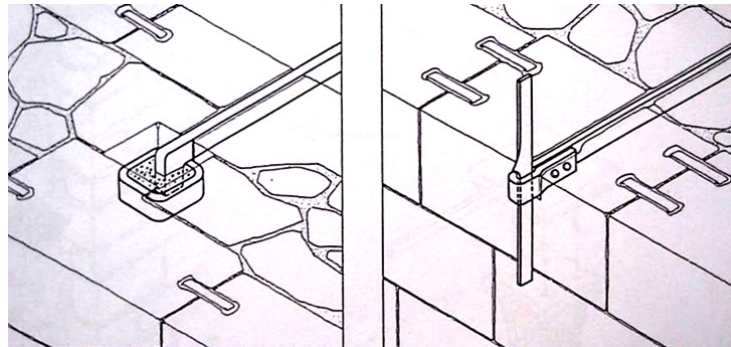


Figure 2.6 : Metal reinforcements used in the Ottoman architecture (Tanyeli, 1990).

It has been possible to observe examples of reinforcing in finely cut stone multi-leaf walls due to damages occurred on the walls. One example to existence of timber members is the Istanbul Mahmud Paşa Mosque (constructed between 1463-1470). In this structure, cavities (16-20 cm×12-22 cm) once used for longitudinal timber lintels were revealed during the restoration works after the 1999 Earthquake. Istanbul Davud Paşa Mosque's 1.9 m thick walls were reinforced with three 20×20 cm timber lintels at each 1.5 m height (Tayla, 2007). Yorulmaz and Ahunbay (1986) mentions usage of timber lintels in Mimar Sinan's structures such as Esekapi İbrahim Paşa Medresesi, Büyükçekmece Sokullu Kervansarayı and İznik Rüstem Paşa Kervansarayı. They also note that the double timber lintels, located inside the wall sections, were tied to each other and generally used at lower and upper levels of openings.

Following the traces in the Ottoman period mosques, it can be seen that usage of iron ties appears in the 15th century (Edirne Üç Şerefeli and II. Bayezid Mosques) and becomes common in the 16th century (Tayla, 2007).

In addition to ties, cramps emerge as elements that reinforce the dry joints of the ashlar walls in the horizontal direction, which solely rely on the friction between the units. However, use of singular reinforcements such as cramps and pins dates back to antiquity. Being perfected through the timeline, the bronze and copper cramps in various geometric shapes transforms into the U shaped iron cramps when they reach the Ottoman period.

Geometry and production process of cramps differ depending on the date of construction. Being widely used in antiquity, the usage of cramps in the main walls vanishes during the mid and late Byzantine, Beyliks and Selcuk periods (Tanyeli, 1990). However, they appear again in the Ottoman architecture particularly at 15th century. Tanyeli (1990) indicates that U shaped cramps in Edirne Arifağa Mosque Minaret (15th century 2nd quarter) were 170 to 200 mm long and 20 mm wide. Cramps of 15th century 3rd quarter Edirne Palace Kitchens were 190 mm long, 18-21 mm wide and both ends were bent for about 35 mm. In Edirne II. Bayezid Mosque, which is situated on the riverbank, every cut stone is connected to each other with approximately 300 mm long clamps that can be observed due to the corrosion-initiated cracks and splitting of the stones. In Istanbul Beyazıt Mosque, the dimensions increase up to 400 mm length and 38 mm width, but since the structure

had gone under repairs in the Mimar Sinan's age, it is hard to define the period for these elements. In 16th century structures, different sizes for cramps can be observed in the same structure. For instance, in the period of Mimar Sinan, the length of the cramps varies between 300 to 500 mm and the width varies between 25 and 50 mm (Tanyeli, 1990).

Yorulmaz and Ahunbay (1986) give the average depth of cramps from façade as approximately 90-100 mm. They also give the lengths of different types of cramps (namely *büzürk*, *büzürk vasat*, *vasat* and *küçük*) used in Süleymaniye İmaret as 250, 390, 440, 600 mm. However, Tanyeli (1990) indicates that lengths corresponding to different types of cramps are still unclear and mentions that according to historical documents, longer cramps (reaching 1800 mm length) might have been used in other places.

Main principles for the application of cramps had almost not changed since the antiquity (Tanyeli, 1990). Cramps were set in lead for protection against corrosion and to achieve close fit (UNDP/UNIDO, 1984). If the corrosion occurs, cramps do far more harm than good, since the resultant swelling of the iron splits the stone in which it is set.

In order to enhance the shear capacity of the walls, pins were introduced to supply additional capacity through dowel action. They were preferred particularly at the joints of slender walls, colonnades, arches, minarets, non-structural members and transition zones between different members. During the utilization of these elements, upper half of the pin was inserted into the hole drilled on the upper stone unit and fixed by using molten lead. Afterwards, the lower half of the pin was fixed to the hole drilled on the upper face of the lower stone unit.

Schematic representation of cramps and pins used in the walls of typical heritage structures of Ottoman Architecture can be seen in and Figure 2.7.

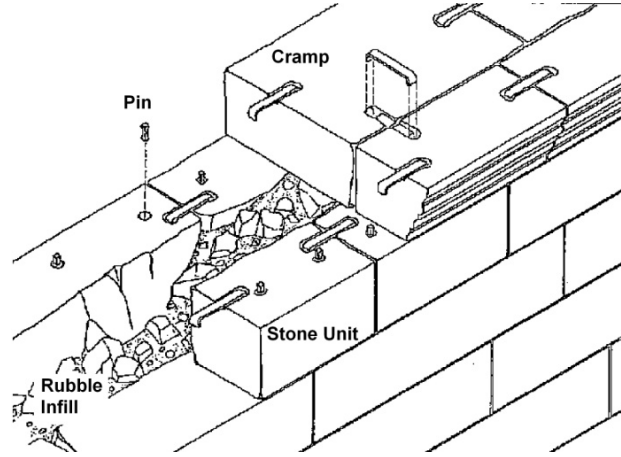


Figure 2.7 : Typical wall configuration (modified from Tanyeli, 1990).

2.2 Observations on Three Existing Structures

In order to gather more data and verify the literature based information delivered in the previous topic, a study on existing monumental structures was necessary. Although most of the existing historical structures erected with multi-leaf stone masonry walls are still standing, it is extremely difficult to observe wall sections in these structures. However, in two structures around Edirne, targeted type of walls could be observed. Namely, these structures were the ruins of Cihannüma Kasrı from Edirne Palace (15th century, Figure 2.8) and Sokullu Mehmet Paşa Külliyesi (16th century, Figure 2.9) at Havsa.



Figure 2.8 : Edirne Palace Cihannüma Kasrı (15th century).



Figure 2.9 : Havsa Sokullu Mehmet Paşa Külliyesi (16th century).

Multi-leaf wall sections of these structures, drawn after a field trip, are schematically presented in Figures 2.10 and 2.11. Total widths of the wall cross-sections were varying between 80 and 110 cm. Outer leaves of the walls consisted of finely cut stone units with dry joints, whereas the internal leaves were made of rubble masonry. Rubble masonry cores consisted of stone and brick pieces surrounded by a mortar matrix. Outer ashlar masonry leaves had smooth finishing at the façade and at the bed and head joints between the stone units; while the inner surfaces were roughly shaped, so that good interaction between the ashlar leaves and the rubble core could be established.

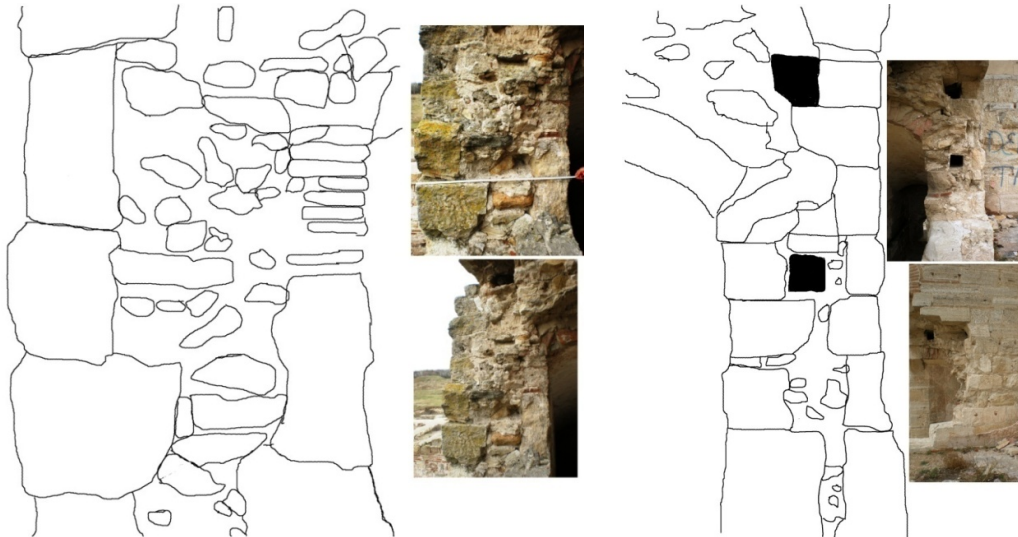


Figure 2.10 : Wall sections from Edirne Palace Cihannüma Kasrı (1st storey).



Figure 2.11 : Wall section from Havsa Sokullu Külliyesi.

Since they were missing, it could not have been possible to identify reinforcements such as cramps and pins in the investigated cross-sections. However, traces left over from the cramps were still visible as seen in Figure 2.12. It is also worthy to notice that molten lead used to fix the reinforcements to the stone units could be still found in some of the slots drilled on the stone units, Figure 2.13.

It should be noted that these structures had been subjected to several interventions in the past. For instance, Edirne Palace was blown up during the Ottoman-Russian 93 War (1877-1878) when the Russian occupation of Edirne was evident. Sadly, in the following years the Palace was unprotected against environmental conditions, plunder and vandalism. Moreover, their current conservation status is far from being adequate. As a result, the drawn wall sections inhibit traces of damages and later interventions.



Figure 2.12 : Cramp slots from ruins of Edirne Palace.



Figure 2.13 : Cramp fixed with molten lead (from ruins of Edirne Palace).

One more structure visited during the field trip was the II.Bayezid Mosque (15th century) in Edirne. Among numerous structures in Edirne, this structure was selected since it was reported by Tayla (2007) that the cramps and pins of the main walls could be observed due to the corrosion induced spalling of the rocks (Figures 2.14 and 2.15). At the date of the field trip, the Mosque was under restoration and the deteriorations on the main walls had already been repaired. However, it could be possible to find two original cramps at the construction site, Figure 2.16. These original cramps were moderately corroded and they had average dimensions as: 350-370 mm length, 30-40 mm width, 9-10 mm thickness and bent tips 42-55 mm long. Later on, these two samples were used for metallurgical analysis so that an idea about the mechanical characteristics of original reinforcement could be provided.



Figure 2.14 : Cramps in the Edirne II. Bayezid Mosque (Kolay, 2009).



Figure 2.15 : Pin used in the Edirne II. Bayezid Mosque (Kolay, 2009).



Figure 2.16 : Original cramps from Edirne II. Bayezid Mosque.

2.3 Stone Masonry Walls in Engineering Literature

Although several experimental and theoretical studies, that aim to investigate the mechanical characteristics of different types of brick masonry elements and structures, exist; with regard to brick masonry, stone masonry has attracted less interest from the structural engineering community.

This topic aims to provide a general overview of the previous research activities carried out on stone masonry walls, particularly experimental studies on the multi-leaf stone masonry walls, which are generally the most encountered stone masonry types in heritage structures. Available literature on stone masonry is classified according to the investigated mechanical characteristics of these walls, such as the ones that define compressive, shear and tensile behaviour. It should be noted that, majority of the existing studies on multi-leaf walls aim to investigate the efficiency of intervention methods such as grout injection and transversal tying. In parallel to

the main aim of this thesis, in this topic, results of the reference specimen tests reported in literature are emphasized rather than the results of repaired or retrofitted specimens.

2.3.1 Studies on compressive behaviour

Axial compressive tests are among the basic test procedures carried out on masonry assemblages and they provide vital information for the mechanical characterization of masonry. Mechanical characteristics such as Young's modulus, compressive strength, strain corresponding to compressive strength, deformation capacity, Poisson's ratio, ductility and failure mechanisms can be investigated through axial compression tests. Moreover, empirical relationships between mechanical characteristics of masonry components (such as units and mortar) and masonry assemblages can be established.

Vintzileou and Tassios (1995) tested eight three-leaf stone masonry wallets under uniaxial compression either before or after grout injection. Limestone squared blocks were utilized for forming the outer leaves of the specimens with average dimensions of 0.60 m length, 1.20 m height and 0.40 m thickness, as seen in Figure 2.17. In two of the specimens, a masonry leaf was provided transversally along the lateral sides of the wallets, Figure 2.17b. The inner leaf was made of lime mortar and limestone coarse aggregates (20-40 mm in diameter). Two types of cement grouts (Type A and B with compressive strengths of 30 and 13 MPa, respectively) were injected at low pressure. Six specimens out of a total of eight were tested before strengthening by injection. All six wallets reached their maximum resistance in compression when vertical cracks appeared on both faces. After formation of the cracks at the interfaces between internal core and external leaves, second order effects caused premature failure. This feature was not observed in case of wallets with transverse leaves. Introduction of cement grout brought drastic improvement in the behaviour due to the homogenization achieved. Compressive strength was increased by 50% to 200% and Young's modulus values were also increased. However, injection of the grout materials caused increased brittleness since the strains corresponding to maximum resistance were reduced. The test results of the reference specimens tested before injection are summarized in Table 2.1.

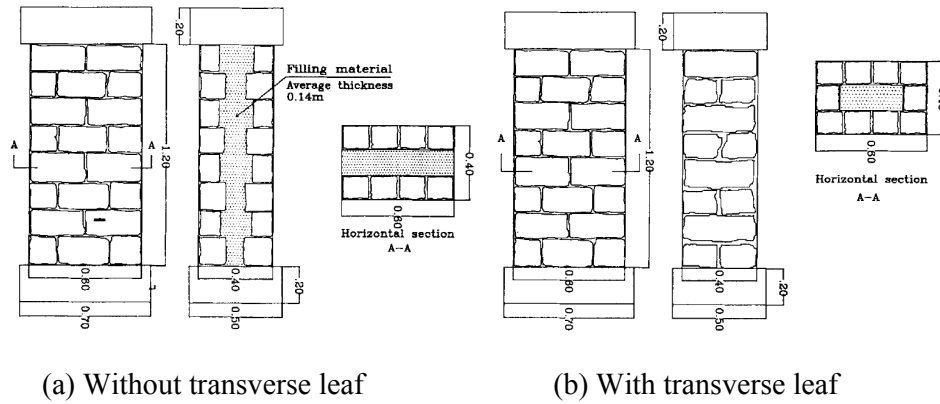


Figure 2.17 : Wallets tested by Vintzileou and Tassios (1995).

Valluzzi et al. (2004) experimentally investigated the effectiveness of three different intervention techniques, such as injection, transverse tying and repointing (renewing of the mortar in the joints), on multi-leaf rubble stone masonry walls. Wall samples were 0.80 m long, 0.50 m thick and 1.40 m high, as seen in Figure 2.18. Outer leaves consisted of 0.18 m thick rough-shaped limestone blocks, bonded in sub-horizontal courses, with mortar joints from 10 to 40 mm thick. Internal core with 0.14 m thickness consisted of mortar and roughly cut limestone pieces. Out of 16 tested wallets, eight specimens were tested as original walls under axial compression prior to the intervention phase. Typical stress-strain relationships obtained before and after injection of grout is shown in Figure 2.18. The original walls had a vertical and sub-vertical crack pattern which were mostly located in the transverse sections, particularly at the interfaces of internal and external leaves. Therefore, the compressive failure occurred due to the out-of-plane detachment of the layers. Characteristic values for the test results of the original walls are summarized in Table 2.1.

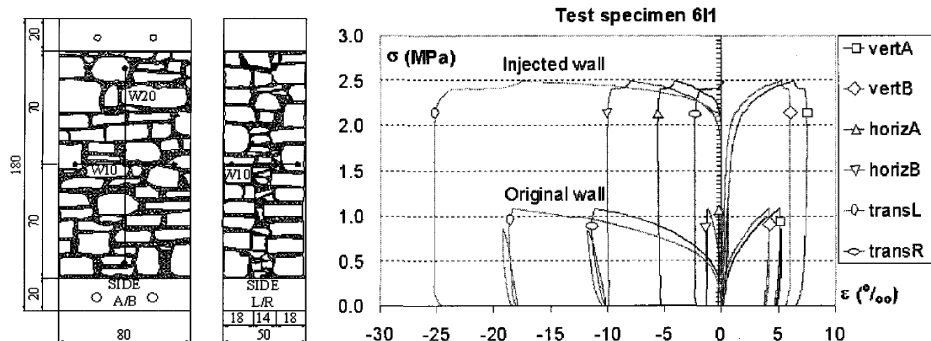


Figure 2.18 : Wall geometry and stress-strain relationships (Valluzzi et al., 2004).

Binda et al. (2006) carried out an extensive experimental campaign in order to investigate the load-transfer mechanisms in multi-leaf walls. For this purpose, 12 three-leaf stone wallets with dimensions of 0.31 m length, 0.51 m thickness and 0.79 m height, were constructed either with or without shear keys between the external and internal leaves. Dimensions and configurations of the tested wallets can be seen in Figure 2.19. Half of the specimens were constructed with higher strength Serena sandstone while the remaining six wallets were constructed with Noto limestone. Three different test procedures were carried out: Shear tests where a monotonic load was applied to the inner leaf while the outer leaves were supported, compression tests on single leaves and compression tests on full wallets. During the tests it was observed that wallets with limestone units and keyed collar joints experienced slightly higher strength and less brittle behavior than the wallets with straight collar joints, Figure 2.20. Characteristic values for the test results of this study are summarized in Table 2.1.

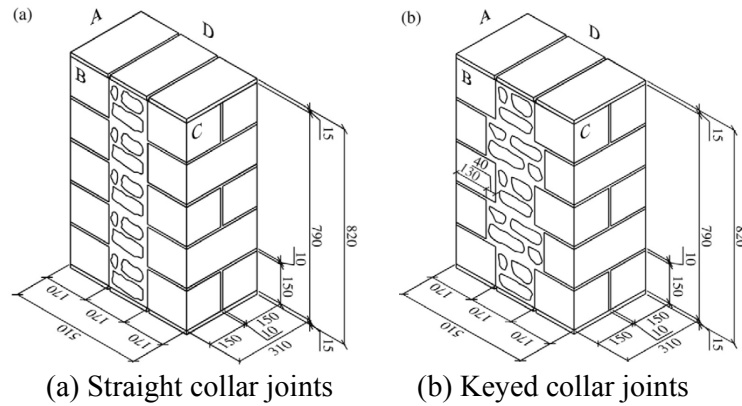


Figure 2.19 : Geometry of wallets tested by Binda et al. (2006).

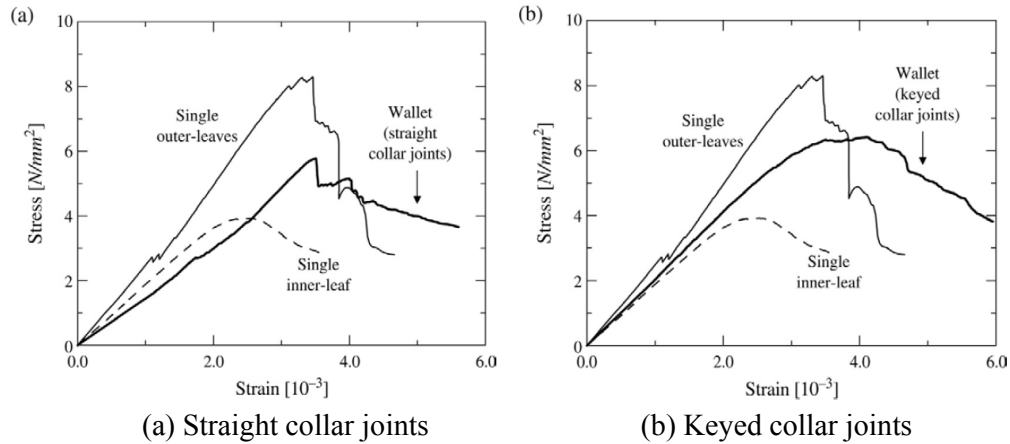


Figure 2.20 : Stress-strain curves of wallets tested by Binda et al. (2006).

Silva et al. (2008) investigated the behaviour of three-leaf stone masonry walls. Out of ten walls tested under uniaxial compression, four were reference walls while the remaining six were retrofitted either by transversal tying with GFRP bars or grout injection or a combination of both. Wallets had global dimensions of 0.60 m length, 0.30 m width and 1.10 m height. Width of each leaf was approximately 0.10 m. External leaves consisted of roughly hewn granite blocks laid by using lime-based mortar while the inner core consisted of granite scrabbings and mortar (Figure 2.21). The failure of the reference walls without any retrofitting intervention was mainly due to the out-of-plane rotation of the external leaves. This failure mode was caused by the development of three hinges along bed joints close to loading plates and the middle height of the wall. Vertical cracks contouring the masonry stone units were also visible. Main results obtained for the reference walls of Silva et al. (2008) are indicated in Table 2.1. Both transversal tying with GFRP rods and grout injection supplied compressive strength enhancement in the order of 55% and 80%, respectively. In the case of combination of both techniques, the enhancement in compressive strength was about 90%. The authors also stressed that scattering is a key issue when dealing with historical constructions as well as with natural and handmade materials.

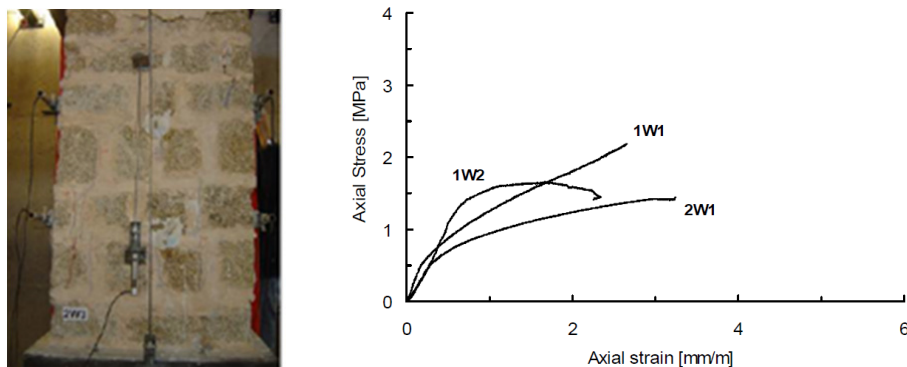


Figure 2.21 : View and stress-strain diagrams of wallets tested by Silva et al. (2008).

Vintzileou and Miltiadou-Fezans (2008) investigated the effect of the injection of ternary grouts (mixes of cement, pozzolan and hydrated lime) and hydraulic lime-based grouts on the compressive strength of masonry. The geometry and typology of the wallets were chosen to simulate the upper and more vulnerable part of the perimeter masonry in the Katholikon of the Dafni Monastery in Greece. The overall dimensions of wallets tested under compression were as follows: Length 1.0 m, height 1.2 m, thickness 0.45 m, average thicknesses of external leaves 192 mm and

135 mm, average thickness of internal leaf (filling material) 123 mm. Geometry and view of the tested wallets are shown in Figure 2.22. As also observed on the actual structure, one of the external leaves was made of larger stones than the other one, whereas solid bricks were arranged along both the horizontal and vertical joints. The inner part of the wallets was a mix of small stones (size between 20 and 50 mm) and mortar. During the tests, vertical cracks occurred on both faces of wallets, crossing mortar joints and stones. Vertical cracks on the transverse sides appeared not only at the interfaces of the external and internal leaves but also cracks passing within the filling material and protruding stones were observed. Main mechanical characteristics obtained from this study are given in Table 2.1.

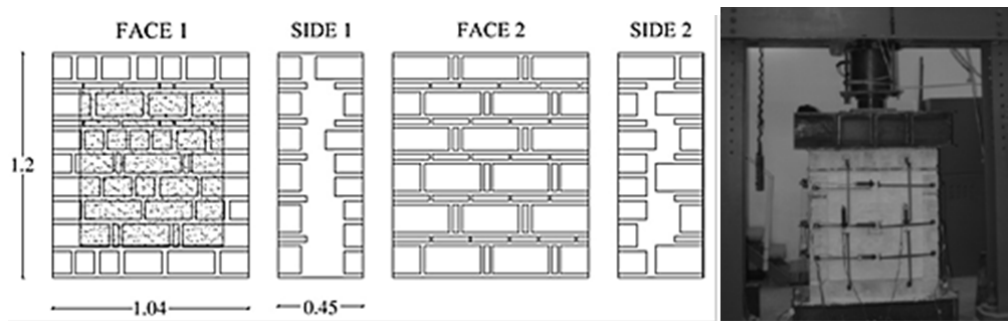


Figure 2.22 : Wallets tested by Vintzileou and Miltiadou-Fezans (2008).

Vintzileou (2008) investigated the effect timber ties on the behaviour of three-leaf stone masonry walls through axial compression and diagonal compression tests. Among the eight wallets constructed with or without timber ties, three specimens were tested under uniaxial compression. Specimens with a length of 0.70 m, height of 0.90 m and thickness of 0.50 m consisted of two limestone outer leaves and a rubble masonry inner core. Wallet 1 was the reference specimen and did not have any timber ties, whereas Wallets 2 and 3 had one timber tie at mid-height, Figure 2.23. In Wallet 2, transverse timber elements were resting on the longitudinal ties and metal nails were used for their connection. In the case of Wallet 3, steel rods were used to connect longitudinal timber elements. Wallet 1 failed after formation of vertical cracks on both longitudinal faces and separation of external and internal leaves. In the cases of walls with timber ties (Wallets 2 and 3), timber ties acted as confining reinforcement, thus, enhanced the compressive strength (in the order of 10 and 20%) and deformation capacity at failure (Figure 2.24). Some major results of this study are summarized in Table 2.1.

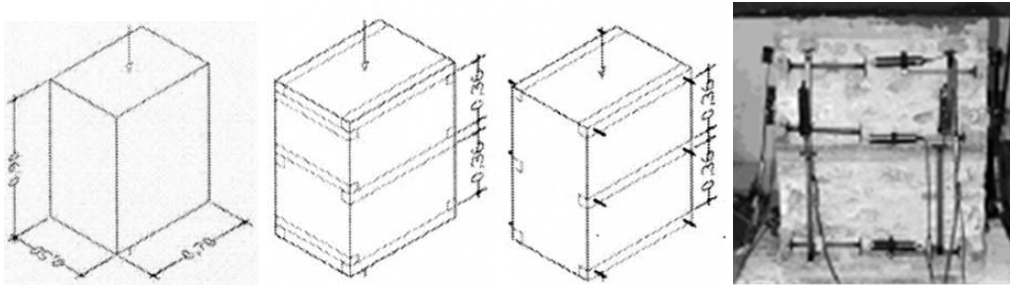


Figure 2.23 : Schematic and testing views of wallets tested by Vintzileou (2008).

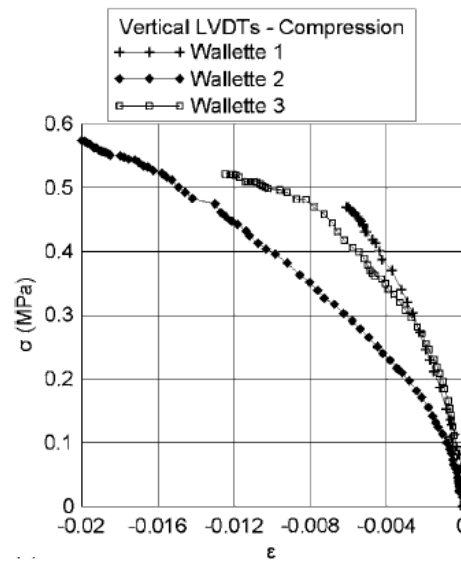


Figure 2.24 : Vertical stress-vertical strain diagrams (Vintzileou, 2008).

Chiostri et al. (2003) realized 22 in-situ tests on nine large panels in six different old masonry buildings in the northern part of Tuscany, Italy. Out of 22 panel tests; which included axial compression, shear-compression and diagonal compression tests carried out either before or after retrofitting with cement grout injection or reinforced concrete jacketing, five were axial compression tests without retrofitting. Three of these reference compression tests were done in the elastic range and the remaining two panels were loaded up to collapse. The CC and DC panels, which were loaded to failure, were located in different buildings and both were double-leaf walls with irregular textures. The former had weak connection between the leaves and had several voids while the latter had better connection. These differences in the typologies of tested panels also show up in the test results as summarized in Table 2.1. Views of the test setup can be seen in Figure 2.25.

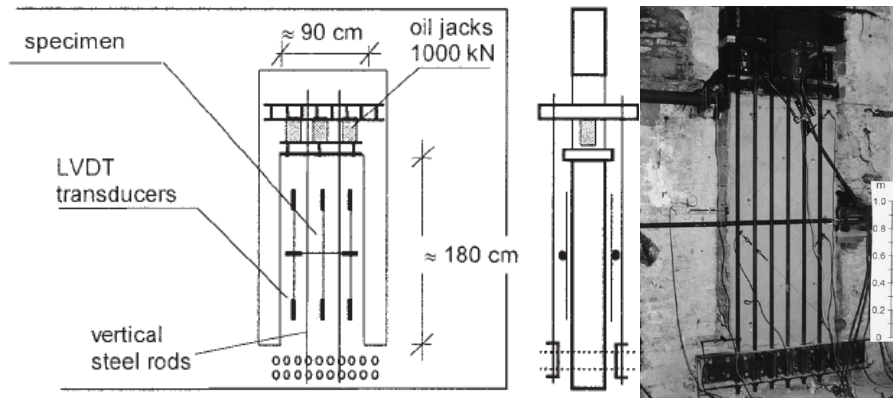


Figure 2.25 : Axial compression test setup of panels tested by Chiostrini (2008).

Corradi et al. (2008) carried out in-situ tests on several walls of buildings that were damaged by the 1997 earthquakes in Umbria, Italy. On site experiments, such as compression, shear-compression and the diagonal compression tests were performed on panels cut from load-bearing walls. In scope of this study, the effectiveness of deep repointing, together with grout injection technique was investigated. Two 1.80 m high, 0.90 m long and 0.48 m thick walls were subjected to axial compression loading, before and after retrofitting. Before retrofitting, panels were not loaded until failure but up to an average axial stress of 0.21 MPa and Young's moduli were obtained as 1289 and 306 MPa. These distant Young's moduli obtained from two panels in the same building once more exhibit the variability of the mechanical characteristics of masonry constructed with natural materials. Some major results of this study are summarized in Table 2.1.

Oliveira et al. (2006) performed an experimental study concerning the uniaxial cyclic compressive behaviour of stone and brick specimens. The sandstone specimens used during the tests had an average compressive strength of 82.7 MPa and an average Young's modulus of 15550 MPa. They exhibited a brittle behaviour particularly at the post-peak domain. The stone prism tests aimed to replicate dry stone masonry, so that the prisms simply consisted of superposition of prismatic stone blocks. Stone units were machine sawn and no special treatment was done before the compression tests. Two prisms were made of three 100×200×100 mm stone pieces and 2 other prisms consisted of four 200×200×100 mm stone pieces, so that height/length ratios of three and two were achieved, respectively. The average compressive strength obtained from prism compression tests (57.1 MPa in average) was obtained almost 30% less than that of single unit tests. However, the Young's moduli obtained from

both test types gave similar results. A typical axial stress-strain curve is shown in Figure 2.26 and results of the prism compression tests are summarized in Table 2.1.

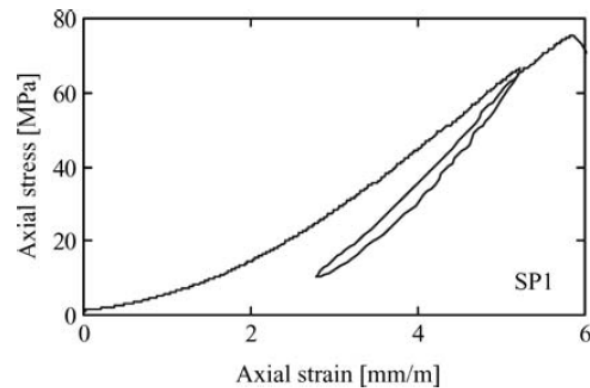


Figure 2.26 : Typical stress-strain diagram for stone prism (Oliveira et al., 2006).

Almeida et al. (2010) carried out compression tests on six wall panels extracted from an existing 20th century building located in Porto, Portugal. Out of these six panels three were tested under uniaxial compression (Panels PP1, PP2 and PP3). The specimens had dimensions of 1.20×2.50×0.40 m for length, height and thickness of the walls, respectively. The single leaf panels consisted of roughly squared granite stones with a widespread distribution of voids. Joints between the stone blocks consisted of mortar and smaller size stone wedges, Figure 2.27. During the implementation of the tests on PP1 and PP2 panels, 8 horizontal ties placed along the height of the wallets were tensioned to 1 kN, so that lateral confinement was applied. Then the walls were loaded monotonically up to a stage very close to failure, and unloaded to zero load level. In the second phase, the lateral confinement was released and axial stress was applied until failure. Panel PP3 aimed to investigate the effectiveness of injection. Some major results of this study for the unconfined cases are summarized in Table 2.1.

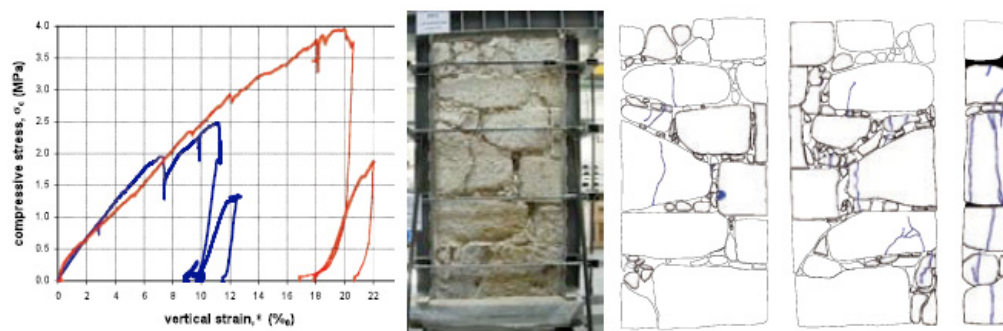


Figure 2.27 : Stress-strain curves of PP1 & PP2 and damage (Almedia et al., 2010).

Augenti and Parisi (2010) studied the monotonic compressive behaviour of single-leaf tuff masonry through laboratory tests, carried out along the directions parallel and orthogonal to the mortar bed joints. By using the experimental data, they proposed different stress-strain models for each loading direction to account for the orthotropy of masonry. The single-leaf masonry prisms tested along the direction orthogonal to the bed joints had dimensions of $0.61 \times 0.65 \times 0.15$ m for length, height and thickness, respectively. View of one of the wallets loaded up to failure and typical stress-strain behaviour obtained through axial compression tests are shown in Figure 2.28. Mean values of main mechanical parameters obtained from eight identical wallets are summarized in Table 2.1.

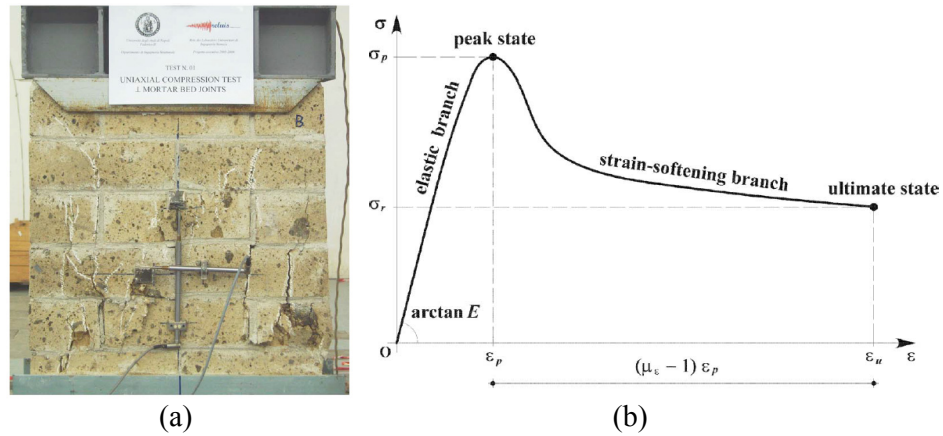


Figure 2.28 : (a) Specimen, (b) typical stress-strain curve (Augenti and Parisi, 2010).

Romano (2008) investigated the compressive behaviour of tuff-brick listed masonry through an experimental campaign, which included all tuff and all brick specimens as well as tuff-brick listed wallets. Typologies and dimensions of the single-leaf wallets are shown in Figure 2.29. Tuff-brick listed specimens were prepared in two different configurations. In the first configuration each row of tuff stone was followed by one row of brick and in the second configuration, each tuff stone row was followed by three rows of brick. In all specimens, regardless of their typologies, vertical cracks passing through the units (either brick or tuff stone) were reported. These cracks were not only visible on the longer side but also on the transverse sides. Mean values of main mechanical parameters obtained from the tested wallets are summarized in Table 2.1. These results indicate that the compressive strength of tuff-brick listed masonry was approximately 30% higher than that of all tuff masonry and the number of bricks did not considerably influence it.

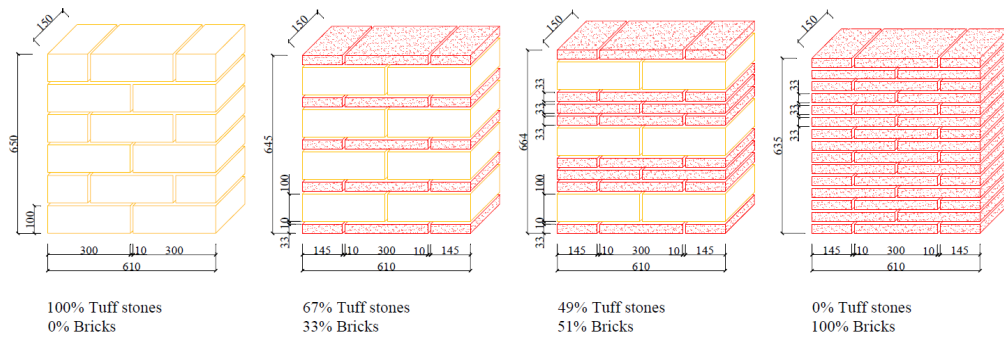


Figure 2.29 : Dimensions and typologies of specimens tested by Romano (2008).

As also mentioned before, main characteristics of the available compression tests in literature are compiled and tabulated in Table 2.1. In this table; $f_{c,unit}$, $f_{c,mortar}$, f_e , f_i and f_{wc} are the compressive strength values of unit, mortar, external leaf, core material and masonry wall, respectively. In addition to these strength parameters, Young's modulus values of the wall specimens (E_{wc}), compressive strain values corresponding to wall compressive strength (ϵ_{wco}), are also indicated. A closer look at the data for these mechanical characteristics (particularly to $f_{wc}/f_{c,unit}$ and E_{wc}/f_{wc} ratios) reveals that significant scatter exists not only between the test results of different studies but also between the specimens of each study.

Table 2.1 : Summary of stone masonry compression tests in literature.

Author	Wallet	Typology	Dimensions l×h×t (m)	h/l-h/t	Leaf thickness ext.-int.-ext. (m)	$f_{c,unit}$ (MPa)	$f_{c,mortar}$ (MPa)	f_e (MPa)	f_i (MPa)	f_{wc} (MPa)	E_{wc} (MPa)	ϵ_{wco} (MPa)	$f_{wc}/f_{c,unit}$ (%)	E_{wc}/f_{wc}
Vintzileou and Tassios (1995)	1	Three-leaf, stretcher bond, squared blocks, transverse leaves	0.60×1.20×0.40	2.0-3.0	0.13-0.14-0.13	100	1.7	–	0.15	2.1	7000	0.001	2.1	3333
Vintzileou and Tassios (1995)	2	Three-leaf, stretcher bond, squared blocks	0.60×1.20×0.40	2.0-3.0	0.13-0.14-0.13	100	1.7	–	0.15	1.3	2706	–	1.3	2081
Vintzileou and Tassios (1995)	3	Three-leaf, stretcher bond, squared blocks, transverse leaves	0.60×1.20×0.40	2.0-3.0	0.13-0.14-0.13	100	1.7	–	0.15	2.4	5000	0.0014	2.4	2083
Vintzileou and Tassios (1995)	4	Three-leaf, stretcher bond, squared blocks	0.60×1.20×0.40	2.0-3.0	0.13-0.14-0.13	100	1.7	–	0.15	1.6	4442	0.0008	1.6	2776
Vintzileou and Tassios (1995)	5	Three-leaf, stretcher bond, squared blocks	0.60×1.20×0.40	2.0-3.0	0.13-0.14-0.13	100	1.7	–	0.15	1.7	5670	0.0028	1.7	3335
Vintzileou and Tassios (1995)	6	Three-leaf, stretcher bond, squared blocks	0.60×1.20×0.40	2.0-3.0	0.13-0.14-0.13	100	1.7	–	0.15	1.35	5625	0.0006	1.35	4167
Valuzzi et al. (2004)	5I1	Three-leaf, rubble stone masonry	0.80×1.40×0.50	1.75-2.80	0.18-0.14-0.18	160	1.57	–	–	1.45	2390	0.0036	0.91	1648
Valuzzi et al. (2004)	6I1	Three-leaf, rubble stone masonry	0.80×1.40×0.50	1.75-2.80	0.18-0.14-0.18	160	1.57	–	–	1.95	2029	0.0046	1.21	1041
Valuzzi et al. (2004)	1I2	Three-leaf, rubble stone masonry	0.80×1.40×0.50	1.75-2.80	0.18-0.14-0.18	160	1.57	–	–	1.97	1450	0.0062	1.23	736
Valuzzi et al. (2004)	8I2	Three-leaf, rubble stone masonry	0.80×1.40×0.50	1.75-2.80	0.18-0.14-0.18	160	1.57	–	–	1.91	1559	0.0062	1.19	816
Valuzzi et al. (2004)	2T	Three-leaf, rubble stone masonry	0.80×1.40×0.50	1.75-2.80	0.18-0.14-0.18	160	1.57	–	–	1.95	1504	0.0066	1.21	771
Valuzzi et al. (2004)	9T	Three-leaf, rubble stone masonry	0.80×1.40×0.50	1.75-2.80	0.18-0.14-0.18	160	1.57	–	–	1.65	2058	0.0041	1.03	1247
Valuzzi et al. (2004)	3R	Three-leaf, rubble stone masonry	0.80×1.40×0.50	1.75-2.80	0.18-0.14-0.18	160	1.57	–	–	0.99	–	0.0032	0.62	–
Valuzzi et al. (2004)	7R	Three-leaf, rubble stone masonry	0.80×1.40×0.50	1.75-2.80	0.18-0.14-0.18	160	1.57	–	–	1.5	1863	0.0044	0.94	1242

Table 2.1 (continued) : Summary of stone masonry compression tests in literature.

Author	Wallet	Typology	Dimensions l×h×t (m)	h/l-h/t	Leaf thickness ext.-int.-ext. (m)	$f_{c,unit}$ (MPa)	$f_{c,mortar}$ (MPa)	$f_{c,ext}$ (MPa)	$f_{c,core}$ (MPa)	f_{wc} (MPa)	E_{wc} (MPa)	ε_{wco} (MPa)	$f_{we}/f_{c,unit}$ (%)	E_{we}/f_{wc}
Binda et al. (2006)	NS3	Three-leaf, squared blocks, straight collar	0.31×0.79×0.51	2.55-1.55	0.17-0.17-0.17	20.6	9.7	8.7	4.1	5.8	1770	0.0035	28.2	305
Binda et al. (2006)	NK3	Three-leaf, squared blocks, keyed collar	0.31×0.79×0.51	2.55-1.55	0.17(0.13)-0.17(0.25)-0.17(0.13)	20.6	9.7	8.7	4.1	6.4	2085	0.0041	31.07	326
Binda et al. (2006)	SS3	Three-leaf, squared blocks, straight collar	0.31×0.79×0.51	2.55-1.55	0.17-0.17-0.17	104.2	9.7	39.8	4	>15.10	2940	>0.005	>14.50	<195
Binda et al. (2006)	SK3	Three-leaf, squared blocks, keyed collar	0.31×0.79×0.51	2.55-1.55	0.17(0.13)-0.17(0.25)-0.17(0.13)	104.2	9.7	39.8	4	>15.10	2725	>0.006	>14.50	<181
Silva et al (2008)	1W1	Three-leaf, roughly squared blocks	0.60×1.10×0.30	1.8-3.7	0.10-0.10-0.10	52.2	2.2	9.2	0.3	2.3	780	0.0068	4.41	340
Silva et al (2008)	1W2	Three-leaf, roughly squared blocks	0.60×1.10×0.30	1.8-3.7	0.10-0.10-0.10	52.2	2.2	9.2	0.3	1.7	1889	0.0029	3.26	1111
Silva et al (2008)	2W1	Three-leaf, roughly squared blocks	0.60×1.10×0.30	1.8-3.7	0.10-0.10-0.10	52.2	2.2	9.2	0.3	1.4	711	0.0094	2.68	340
Silva et al (2008)	3W1	Three-leaf, roughly squared blocks	0.60×1.10×0.30	1.8-3.7	0.10-0.10-0.10	52.2	2.2	9.2	0.3	2.6	1351	0.0039	4.98	520
Vintzileou and Miltiadou-Fezans (2008)	1	Three-leaf, stone masonry with solid bricks in mortar joints	1.00×1.20×0.45	1.20-2.67	0.19-0.12-0.14	25	4.35	—	0.15	1.82	1000	—	7.28	550
Vintzileou and Miltiadou-Fezans (2008)	2	Three-leaf, stone masonry with solid bricks in mortar joints	1.00×1.20×0.45	1.20-2.67	0.19-0.12-0.14	25	4.35	—	0.15	1.74	1440	—	6.96	828
Vintzileou and Miltiadou-Fezans (2008)	3	Three-leaf, stone masonry with solid bricks in mortar joints	1.00×1.20×0.45	1.20-2.67	0.19-0.12-0.14	25	4.35	—	0.15	2.26	1500	—	9.04	664
Vintzileou (2008)	1	Three-leaf, rubble stone masonry	0.70×0.90×0.50	1.29-1.80	—	50	0.8	—	—	0.47	—	0.0062	0.94	—
Vintzileou (2008)	2	Three-leaf, rubble masonry, timber ties in both directions	0.70×0.90×0.50	1.29-1.80	—	50	0.8	—	—	0.58	—	0.0201	1.16	—
Vintzileou (2008)	3	Three-leaf, rubble masonry, ties in longitudinal direction	0.70×0.90×0.50	1.29-1.80	—	50	0.8	—	—	0.52	—	0.0124	1.04	—

Table 2.1 (continued) : Summary of stone masonry compression tests in literature.

Author	Wallet	Typology	Dimensions l×h×t (m)	h/l-h/t	Leaf thickness ext.-int.-ext. (m)	$f_{c,unit}$ (MPa)	$f_{c,mortar}$ (MPa)	$f_{c,ext}$ (MPa)	$f_{c,core}$ (MPa)	f_{wc} (MPa)	E_{wc} (MPa)	ϵ_{wco} (MPa)	$f_{wc}/f_{c,unit}$ (%)	E_{wc}/f_{wc}
Chiostri et al. (2003)	CC	Double leaf, rubble masonry, leaves weakly connected, voids	0.84×1.67×0.44	1.99-3.80	—	—	—	—	—	0.24	1921	—	—	8004
Chiostri et al. (2003)	DC	Double leaf, rubble masonry	0.87×1.80×0.52	2.07-3.46	—	—	—	—	—	1.04	376	0.006	—	362
Oliveira et al. (2006)	SP1	Single-leaf, stacked dry-stone masonry	0.20×0.30×0.10	1.50-3.00	—	82.7	—	—	—	75.2	16210	0.0058	90.9	216
Oliveira et al. (2006)	SP2	Single-leaf, stacked dry-stone masonry	0.20×0.30×0.10	1.50-3.00	—	82.7	—	—	—	49.3	14270	0.0042	59.6	290
Oliveira et al. (2006)	SP3	Single-leaf, stacked dry-stone masonry	0.20×0.40×0.20	2.00-2.00	—	82.7	—	—	—	42.1	13750	0.0038	50.91	327
Oliveira et al. (2006)	SP4	Single-leaf, stacked dry-stone masonry	0.20×0.40×0.20	2.00-2.00	—	82.7	—	—	—	61.9	14960	0.005	74.85	242
Almedia et al. (2010)	PP1	Single leaf, very roughly shaped granite blocks with mortar joints	1.20×2.50×0.40	2.1-6.3	—	61	—	—	—	1.87	500	—	3.07	267
Almedia et al. (2010)	PP2	Single leaf, very roughly shaped granite blocks with mortar joints	1.20×2.50×0.40	2.1-6.3	—	61	—	—	—	1.34	700	—	2.2	522
Augenti and Parisi (2010)	Mean	Single-leaf, tuff stone, stretcher bond, 10 mm thick mortar joints	0.61×0.65×0.15	1.07-4.33	—	4.1	2.1	—	—	3.96	2222	0.0024	95.88	561
Romano (2008)	All tuff, mean	Single-leaf, all units tuff stone, stretcher bond, mortar joints	0.61×0.65×0.15	1.07-4.33	—	4.13	7.14	—	—	1.97	781	—	47.7	397
Romano (2008)	Simple tuff-brick listed, mean	Single-leaf, one row tuff one row brick, mortar joints	0.61×0.65×0.15	1.07-4.33	—	4.13-5.15	7.14	—	—	2.58	930	—	62.47	361
Romano (2008)	Multiple tuff-brick listed, mean	Single-leaf, one row tuff three rows brick, mortar joints	0.61×0.65×0.15	1.07-4.33	—	4.13-5.15	7.14	—	—	2.53	822	—	61.26	325
Romano (2008)	All brick, mean	Single-leaf, all brick, stretcher bond, mortar joints	0.61×0.65×0.15	1.07-4.33	—	25.15	7.14	—	—	4.7	1820	—	18.69	387

2.3.2 Studies on shear behaviour

Studies performed on in-plane shear behaviour of stone masonry walls can be classified with respect to the loading type as diagonal compression and shear compression tests.

In the shear compression tests, the masonry wall is initially loaded to an axial stress level that represents the vertical load acting in an actual structure. Then the lateral displacements are applied either in a monotonic or cyclic fashion and simultaneously the pre-compression applied on the wall specimen is kept constant. The response of the wall to in-plane shear can be obtained through identification of the failure mode together with characteristics such as strength, stiffness, energy dissipation and deformability. In the following, some major experimental studies that include shear compression tests on stone masonry walls are summarized. Some main mechanical parameters; such as f_{wc} wall compressive strength, f_{wt} wall tensile strength, μ global friction coefficient and c shear strength at zero compression stress, obtained from these tests, are also presented in Table 2.2.

Vasconcelos and Lourenço (2009) carried out an experimental campaign on 23 single-leaf stone masonry walls, which were tested under different vertical normal stress levels (0.50, 0.88 and 1.25 MPa) and cyclic in-plane horizontal loads. The investigated bond arrangements included dry stack, irregular and rubble masonry walls as seen in Figure 2.30. Granite stone available in the northern region of Portugal was used in the construction of the walls.

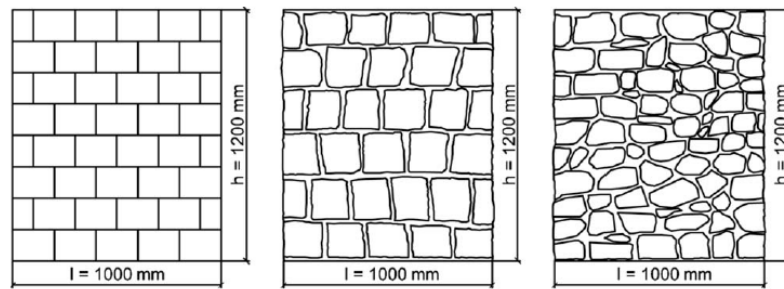


Figure 2.30 : Typologies of walls tested by Vasconcelos and Lourenço (2009).

The observations done during the tests showed that in the case of dry stack walls, for low levels of axial stress, flexural stepped cracks characterized the damage. At higher axial stress levels, the shear resistance of the bed joints was exceeded and a diagonal stepped crack developed along bed and head joints. Then rotating of the

upper part of the wall around the bottom corner (rocking mechanism) and brittle toe crushing was observed. Typical damage distribution and load-displacement hysteresis indicative of rocking behaviour can be seen in Figure 2.31.

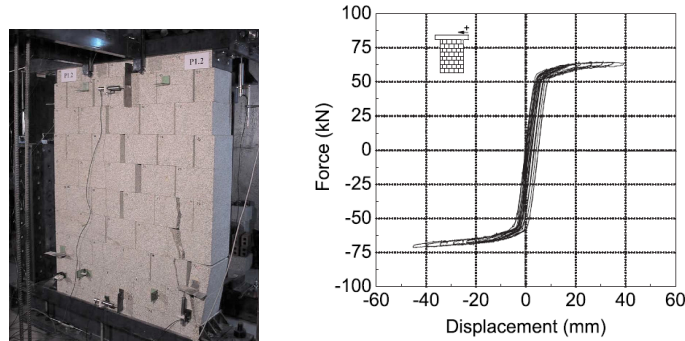


Figure 2.31 : Typical damage and load-displacement diagram (dry stack walls).

The in-plane cyclic behaviour of irregular walls with low to intermediate pre-compression stresses was similar to dry stack walls and was governed by flexural response and rocking mechanism. At higher axial stresses, the failure was due to diagonal stepped cracks and toe crushing (Figure 2.32). In the case of rubble masonry walls, the behaviour is reported to be more dependent on the pre-compression stress level. As the axial stress increases from 0.50 to 1.25 MPa, the dominating behaviour type transforms from flexure to shear with rocking as seen in Figure 2.33.

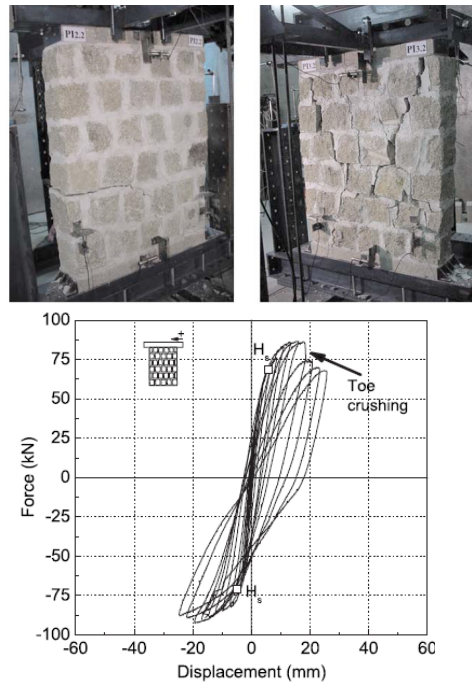


Figure 2.32 : Crack patterns and typical load-displacement diagram (irregular walls).

Apart from the failure modes, mechanical characteristics such as stiffness, ductility and energy dissipation also depend on the masonry texture and pre-compression stress. In all walls, the secant stiffness was dependent on the level of pre-compression and increased as the axial stress level increased. The ductility factor was found to decrease as the irregularity of the masonry bond and the axial load level increased. Rubble and irregular walls were reported as reasonably dissipative walls while dry stack walls dissipated less energy due to the rocking mechanism they exhibited.

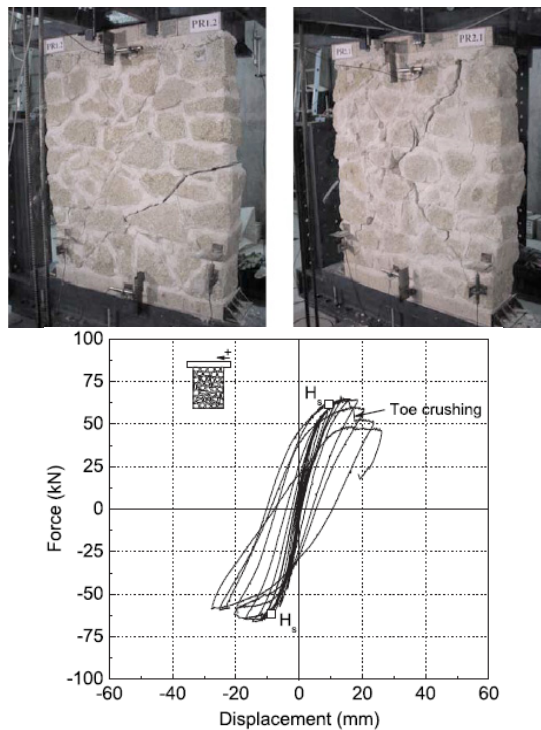


Figure 2.33 : Crack patterns and typical load-displacement diagram (rubble walls).

In a recent study, Galasco et al. (2010) carried out an experimental work on seismic behaviour of undressed double leaf stone masonry walls. Five in-plane cyclic shear tests were performed on wall specimens that had two different height/length ratios and pre-compression stress levels (0.20 and 0.50 MPa) as seen in Figure 2.34. The sole difference of the CS00 specimen with respect to other walls was the utilization of a better quality mortar. According to Galasco et al. (2010), except CS00, that exhibited a flexural failure mechanism with no diagonal cracking, all other specimens collapsed due to shear. The pier named as CS02 (characterised by low compression level) showed an initial phase of flexural damage, before reaching collapse in shear. A typical force-displacement hysteresis and envelope curves of the

tested specimens are shown in Figure 2.35. For panels failed in shear, the ultimate shear resistance was approximately equal to (or slightly higher than) the force at which the first visible diagonal crack appeared at the centre of the pier. The authors pointed out that except the CS00 and CS02 walls, which exhibited flexural type of behaviour, the ultimate drift ratios corresponding to 20% strength loss was between 0.31 and 0.38% while they were 0.67 and 1.92% for CS00 and CS02, respectively. They also underlined that the ultimate drift ratios achieved by the walls with shear failure mode were lower than the 0.40% limit given in the Italian NTC08 (2008) code.

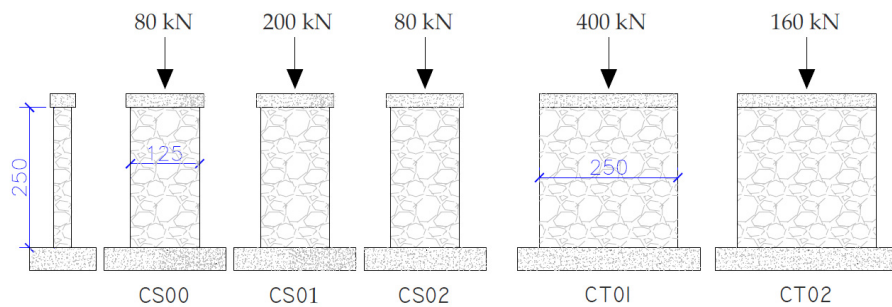


Figure 2.34 : Walls tested by Galasco et al. (2010).

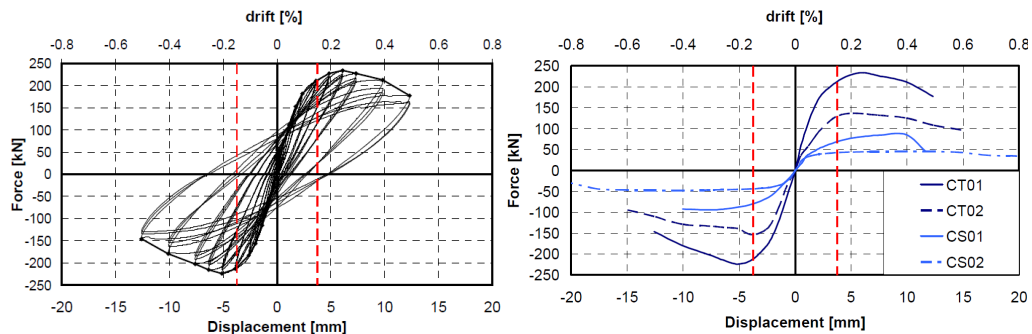


Figure 2.35 : Typical hysteresis and envelope curves (Glasco et al., 2010).

Elmanshawi et al. (2010) tested eight three-leaf masonry walls which were representative of the members present in the West Block on Parliament Hill in Ottawa, Canada. One exterior leaf of the walls was of sandstone in a sneck pattern while the other was of limestone in a running bond pattern (Figure 2.36). The inner rubble core consisted of stone pieces and mortar. Out of eight walls, three of them were tested as reference specimens without any retrofitting while the others were retrofitted by means of transverse ties connecting the external leaves. The transverse ties were formed either by using metal anchors or by using overlapping stone units

extending across the wall width. An extensive loading procedure was applied during the experimental study: Initially the specimens were loaded under eccentric and concentric compression forces that targeted an average axial stress value of 0.6 MPa. After the compression tests, an in-plane free vibration test was performed to determine the natural frequency of vibration of the wall, together with the damping ratio. A cyclic loading pattern was applied until a drift ratio of 0.18% (5 mm top displacement) was achieved, then the walls were subjected to high frequency in-plane lateral loading with a frequency of 5 Hz to a target load of 10 kN. After this test, the walls were tested again in compression, static shear, and free vibration to evaluate its properties after cracking, as a measure of the deterioration the wall has suffered. After the in-plane testing phase, the walls were moved to a one degree of freedom shake table for out of plane testing under two earthquake scenarios. Finally the surviving walls were subjected to a final compression test to determine if they can still carry an average stress of 0.6 MPa and thereby avoid collapse of the building. The test exhibited that no cracking occurred until the 0.18% drift ratio in the in-plane loading direction. Continued racking at this magnitude caused diagonal stepped cracks which followed different paths on the exterior leaves, due to the differing bond patterns, stone sizes and shapes. None of the rehabilitation schemes targeting the tying of the external leaves in the transverse direction could enhance the seismic behaviour of the walls. The equivalent ductility ratio and effective viscous damping ratio were obtained as 1.7 and 9%, respectively.



Figure 2.36 : Typology of the walls tested by Elmanshawi et al. (2010).

Costa et al. (2012) carried out a single in-plane shear test on a pier of an existing building, which was partially damaged during the 1998 Azores earthquake. The double-leaf wall was constructed by using highly irregular basalt blocks (rubble masonry) with surface mortar layers. The lateral force-displacement curve and the final cracking pattern of the tested pier can be seen in Figure 2.37. The global wall behaviour was characterized by diagonal cracking and shear sliding. Costa et al. (2012) reported a displacement ductility of 5.5 and equivalent hysteretic damping values ranging between 12 and 25%.

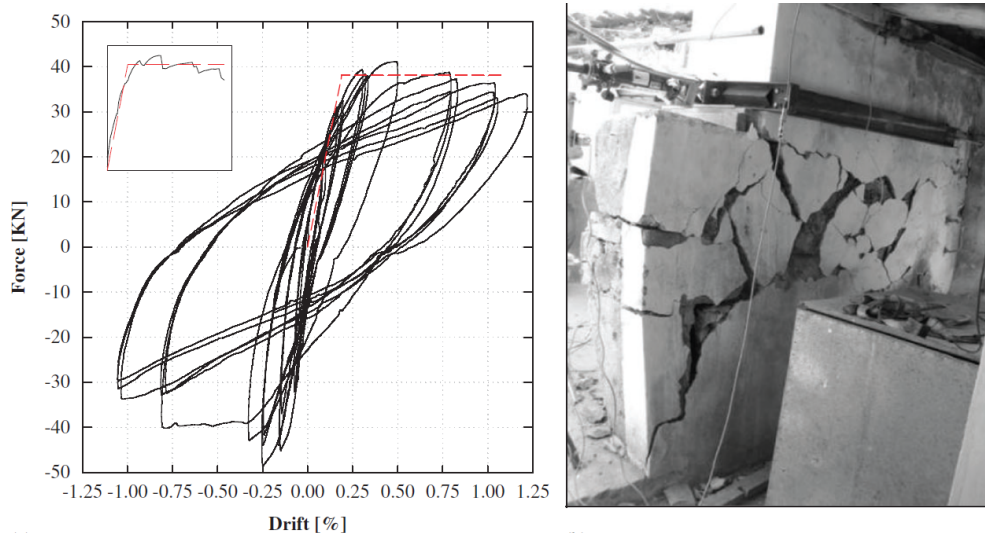


Figure 2.37 : Force-displacement diagram and cracking pattern (Costa et al., 2012).

Corradi et al. (2008) carried out several in situ and laboratory tests on double-leaf masonry panels, in order to investigate the shear behaviour and the effectiveness of conventional and innovative techniques in retrofitting. Herein, only the main results of the unretrofitted reference specimens are summarized. During the test, the masonry panels were subjected to a constant vertical stress (between 0.14 and 0.20 MPa) and simultaneously to a horizontal shear load in the centre of the panel by using the test setup shown in Figure 2.38. The masonry panels consisted of two weakly connected leaves of roughly cut stones and lime-based mortar. A typical shear failure was observed on both sides of the panels, with a characteristic shear strength and shear modulus of 0.09 MPa and 110 MPa, respectively.

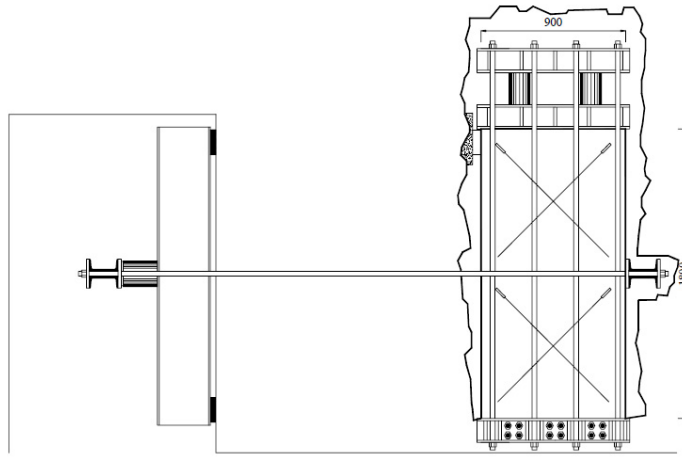


Figure 2.38 : Test setup of the tests performed by Corradi et al. (2008).

Augenti et al. (2011) tested a full-scale unreinforced masonry (URM) wall with an opening, under in-plane lateral loading (Figure 2.39). The wall was first subjected to monotonically-increasing displacements until a moderate damage level was reached. The damaged specimen was then cyclically tested up to almost the same maximum drift attained during the monotonic test in order to investigate the effects of previous damage on its nonlinear response. Finally, the masonry wall was repaired with inorganic matrix-grid (IMG) composites and subjected to a cyclic displacement-controlled test, up to a near collapse state. Most of the observed damage was developed in the spandrel panel, affecting both lateral resistance and strength degradation. Once the piers cracked at their base, vertical cracks due to flexure occurred at the end sections and in the middle of the spandrel panel. As the lateral displacement increased, piers' rocking was more evident and diagonal shear cracking took place in the spandrel panel. Rocking of piers governed lateral stiffness and hysteretic response, which was characterized by low residual displacements and re-centering behaviour. The comparison between the experimental force-displacement curves demonstrated that the IMG strengthening system was able to provide energy dissipation capacity to the spandrel panel, restoring load-bearing capacity of the as built wall, and delaying strength degradation that was observed at larger displacements (Figure 2.39).

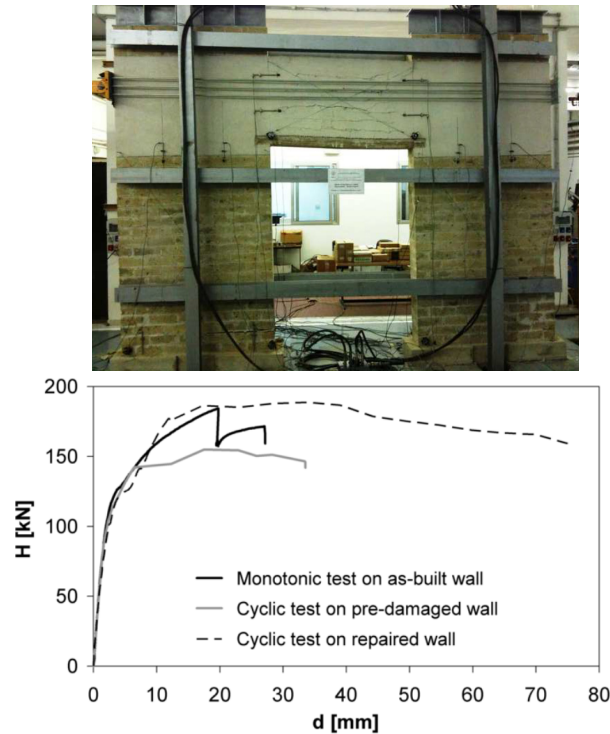


Figure 2.39 : Specimen tested by Augenti et al. (2011) and envelope curves.

The shear compression tests carried out by Lourenço et al. (2005) consisted of a set of seven walls subjected to vertical precompression levels, ranging between 0.15 and 1.25 MPa. The walls were constructed by using stone units juxtaposed with dry joints. Uniaxial compressive strength of the utilized stone material was 82.7 MPa. Before lateral loading, each specimen was axially loaded to the desired axial stress level. During this phase, it was observed that the Young's moduli of the walls were significantly lower than that of stone units (about 3.5-7.5% of stone unit). The authors indicated that, this was mainly due to presence of geometric tolerances between the stone units. The stone units were partly in contact with each other, creating an imperfect joint with localized contacts. They also reported that Young's modulus of the walls increased with the vertical load since the contact area between stone units also increased at higher axial stress levels. After completion of the axial loading, a horizontal in-plane load was acted and increased monotonically until reaching the failure of the wall. For lower stress levels, failure occurred by rotation of the upper part of the wall and sliding along the bed joints, leading to a stepped diagonal crack without visible cracking in the stone units. For higher vertical loads, cracking in the stone units started to become noticeable along a diagonal cracking band developed along the joints and through stone units (Figure 2.40).

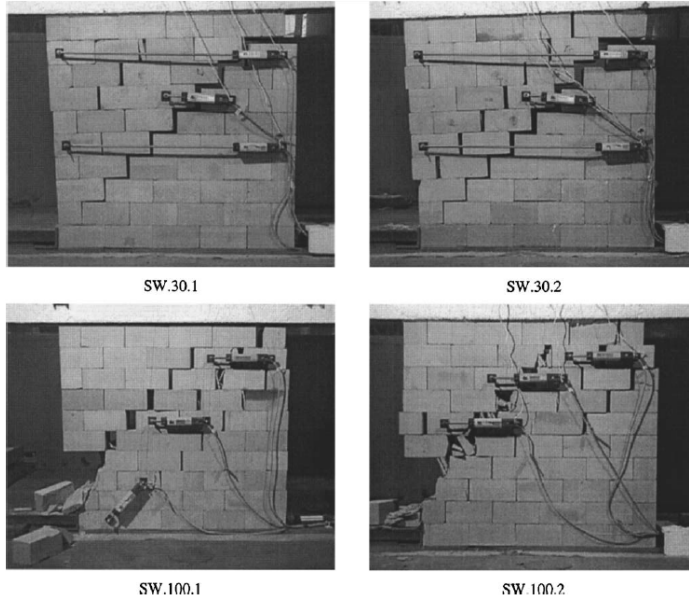


Figure 2.40 : Failure modes of specimens tested by Lourenço et al. (2005).

The horizontal load–displacement diagrams, obtained for walls subjected to combined vertical and horizontal loading, were characterized by linear behaviour preserved up to almost 30% of the peak load. Then, continuous stiffness degradation took place under increasing deformation followed by oscillations of the horizontal load due to sudden movement of the stone units (Figure 2.41).

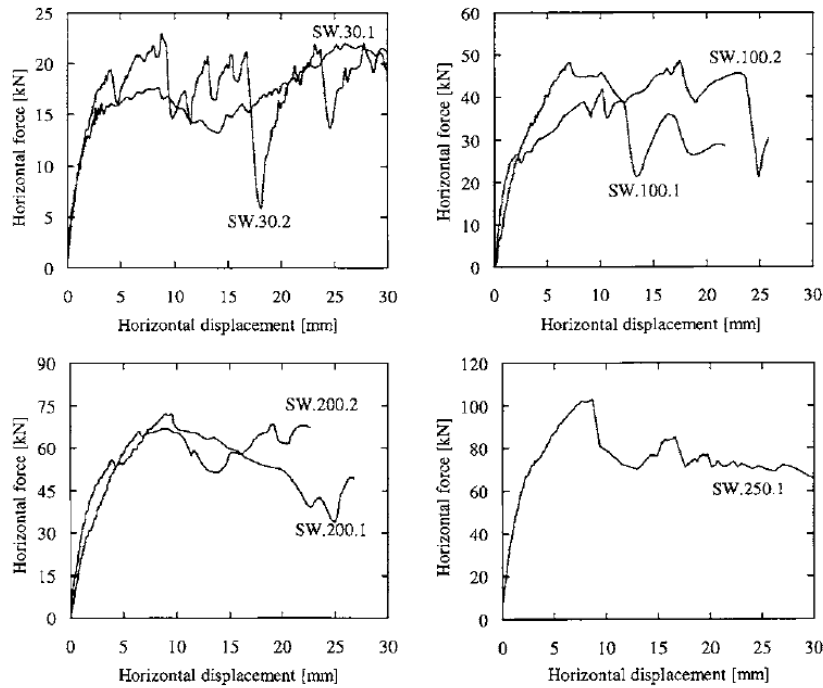


Figure 2.41 : Load-displacement curves of the walls (Lourenço et al., 2005).

Table 2.2 : Summary of stone masonry shear compression tests in literature.

Author	Typology	Dimensions l×h×t (m)	Pre-compression (MPa)	f_{wc} (MPa)	f_{wt} (MPa)	μ	c (MPa)
Vasconcelos and Lourenço (2009)	Single-leaf, dry stack walls, granite	1.00×1.20×0.20	0.50-0.88-1.25	73	N/A	0.38	N/A
Vasconcelos and Lourenço (2009)	Single-leaf, irregular walls, granite	1.00×1.20×0.20	0.50-0.88-1.25	18.4	0.12	0.30	0.05
Vasconcelos and Lourenço (2009)	Single-leaf, rubble walls, granite	1.00×1.20×0.20	0.50-0.88-1.25	N/A	0.11	0.19	0.11
Galasco et al. (2010)	Double-leaf , calcareous sandstone	1.25×2.50×0.32, 2.50×2.50×0.32	0.20-0.50	3.28	0.14	0.36	0.09
Costa et al. (2012)	Double-leaf with very irregular units, basalt	2.27×1.96×0.66	0	N/A	0.03	N/A	N/A
Corradi et al. (2008)	Double-leaf with very irregular units	0.90×1.80×0.50	0.14-0.20	N/A	0.14	N/A	N/A
Augenti et al. (2011)	Single –leaf tuff stone masonry with regular units	1.70×2.30×0.31	0.38	3.85	N/A	N/A	N/A
Elmanshawi et al. (2010)	Three-leaf stone masonry with rubble infill	2.00×2.75×0.54	0.30-0.60	N/A	N/A	N/A	N/A
Lourenço et al. (2005)	Single-leaf, dry stack walls	1.00×1.00×0.20	0.15-0.50-1.00-1.25	57.1	0.1	0.39	N/A

3. EXPERIMENTAL INVESTIGATION OF MULTI-LEAF STONE MASONRY WALLS

The experimental campaign of this study aims to provide data on the failure mode, lateral shear load capacity, ductility and damage characteristics of stone masonry walls constructed with common characteristics observed in the cultural heritage structures around Istanbul city. This chapter presents the outline of the experimental study aimed to investigate the in-plane behaviour of multi-leaf stone masonry walls. Design of experiments, construction of the specimens, testing procedures is explained in detail. Details of the arrangement of boundary conditions, the positioning of sensors and data acquisition system are also presented.

3.1 Design and Construction of Walls

3.1.1 Design of experiments

In order to investigate the in-plane seismic behaviour of multi-leaf stone masonry walls observed in the classical era Ottoman structures, realization of an experimental programme was planned. The planned series of experiments consisted of quasi-static cyclic shear compression tests performed on scaled-down model walls. During the application of lateral displacement cycles, vertical stress level was kept constant. A schematic layout of the shear test configuration is shown in Figure 3.1.

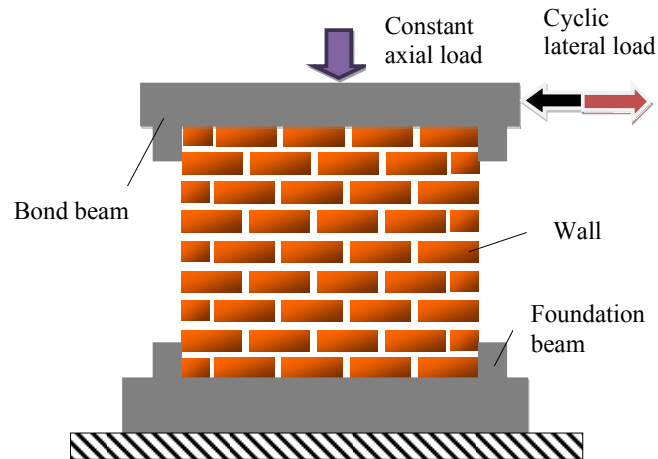


Figure 3.1 : Schematic layout of the planned shear test configuration.

Initially, taking the literature and site surveys into account, the main features of the prototype wall, that can be treated as a reference wall for designing of the scaled down laboratory shear test specimens, were defined as:

- Total wall thickness is 900 mm;
- Each of the three leaves has 300 mm thickness;
- Outer leaves consist of finely cut stones (ashlar) and each stone has dimensions of 900×300×300 mm (length × height × width);
- Cut stones are limestone (Küfeki Stone);
- Stretcher bond is used during the construction of the outer leaves;
- Cut stones of the outer leaves are interconnected with dry joints reinforced by iron cramps and pins;
- Cramps and pins are attached via lead filled slots opened on each stone;
- Internal leaf consists of rubble masonry;
- External leaves are intending into the rubble core so that shear keys between both leaves can be established.

It should be noted that, wall thicknesses reaching more than 2.0 m can also be observed in monumental buildings and much higher values for different structures such as historical city walls can be pronounced. In addition to that, several variations for the dimensions and arrangement of masonry texture and reinforcement can be pronounced. However, considering the limited sources of the experimental study, variety of the applications reported for the applied construction techniques in the heritage structures and lack of knowledge on the inner structure of the existing walls; it has been decided to reflect above-mentioned main characteristics of these walls to the prototype wall.

Though it is not possible to give certain dimensions for the joint reinforcements (cramps and pins) used in the investigated type of walls, average dimensions shown in Figure 3.2 might be extracted from field trip observations and sources such as Tanyeli (1990), Yorulmaz and Ahunbay (1986), Tayla (2007) and through private communication with Prof.Dr. İlknur Kolay (2009). Consequently, these dimensions can be used for designing the reinforcements of a fictitious prototype wall, which

replicates the main characteristics of the multi-leaf stone masonry walls utilized in the heritage structures. Accordingly, the average length of the cramp is about 300 mm, whilst the width is 30 mm and the thickness is 9 mm. Both ends of the cramps are bent for about 50 mm to provide anchorage of the cramps to the stone units. Pins have a square cross-section with dimensions of 21×21 mm and length of the prototype pin is 60 mm.

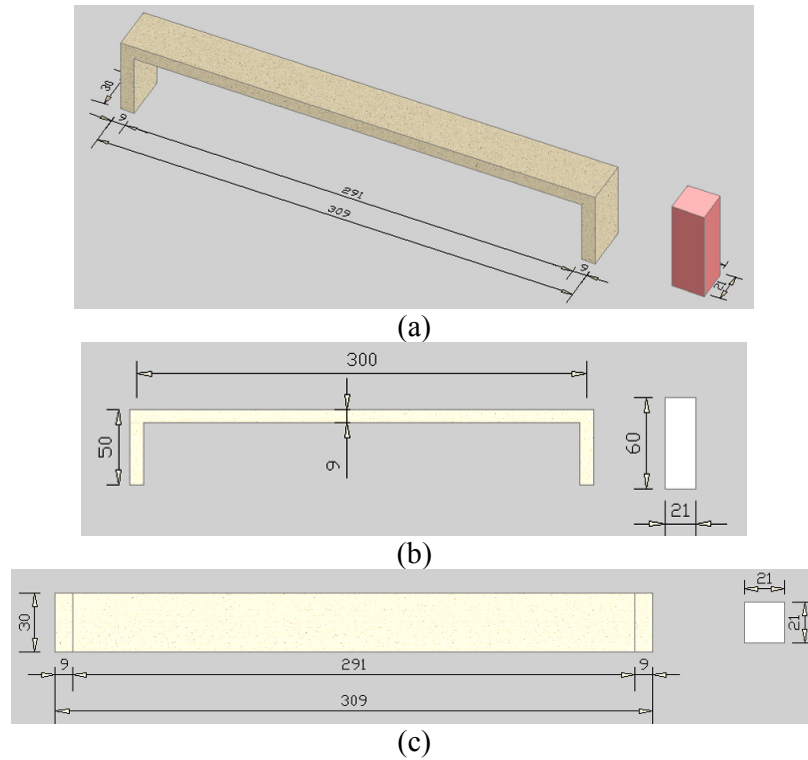


Figure 3.2 : (a) Isometric, (b) longitudinal and (c) plan views of a cramp and pin assumed for a prototype wall.

Scaling down of a prototype to a model requires great attention. Otherwise, unplanned distortions from the prototype to model scaling may result with unexpected results. According to Sabnis et al. (1985), under the assumption that, there are no significant time-dependent effects influencing the structural behaviour (i.e. under static loading), the parameters that enter the modelling process can be listed as in Table 3.1. For a practical true model where the self-weight of the structure is not significant, the scale factors given in this table would be adequate for modelling masonry structures. It should be noted that, for this “practically true” modelling approach, a geometric scale S_l is applicable for most of the geometry related parameters. However, the stress-strain relationships of both model and

prototype masonry must be same. This condition makes the material selection a key parameter for the reliability of the experimental study.

Table 3.1 : Scale factors for masonry specimens (Sabnis et al.,1985).

Group	Quantity	Practical true model
Loading	Concentrated load	S_l^2
	Pressure	1
	Line load	S_l
	Moment	S_l^3
Geometry	Linear dimension	S_l
	Displacement	S_l
	Angular displacement	1
	Area	S_l^2
Material properties	Masonry unit stress	1
	Masonry unit strain	1
	Young's modulus of masonry unit	1
	Masonry unit Poisson's ratio	1
	Specific mass	$1/S_l$
	Mortar stress	1
	Mortar strain	1
	Young's modulus of mortar	1
	Mortar Poisson's ratio	1
	Reinforcement stress	1
	Reinforcement strain	1
	Young's modulus of reinforcement	1

Considering the capacity of the hydraulic actuators and availability of free space for specimen production and storage, along with the available financial and human resources allocated for this study; a geometric scale factor of $S_l=1/3$ has been chosen.

It is a well proven fact that aspect ratio (height/length ratio), together with the axial stress level, is a major parameter which defines the overall behaviour of the wall subjected to in-plane lateral loading (Tomazevic, 2006). Masonry walls tend to exhibit flexure dominated behaviour for high aspect ratios, whereas shear becomes effective for squat walls. In this study, a single aspect ratio was selected and utilized. The selected aspect ratio was 1.0 for all specimens. However, axial stress level was a primary parameter, so that different modes of failure could be observed.

Considering the dimensions of walls and stone units reported in the literature and observed on site; 1/3 scaled down dimensions of the model specimen were obtained as 1.20×1.20×0.30 m in length, height and thickness, respectively. Three-

dimensional general views of the typical specimens with and without rubble infill are shown in Figure 3.3. Since the staggered indentation of the stone units into the rubble masonry was planned, two different widths were chosen for stone units of each stone course. Dimensions of the stone units were $100 \times 100 \times 300$ mm and $100 \times 130 \times 300$ mm (height $\times$ width $\times$ length). Shorter stone units with dimensions of $100 \times 100 \times 150$ mm were also used to form the stretcher bond typology of the walls. During the selection of the dimensions of the walls, another concern was the limitations dictated by the capabilities of the Structural and Earthquake Engineering Laboratory of Istanbul Technical University.

It is important to note that, in this study, the single-leaf term is used as a generic classification, which aims to simplify the naming of the shear compression test specimens constructed with single material (Küfeki stone units) and does not have the rubble core but have two ashlar leaves. As seen in Figure 3.3b, the walls denoted as single-leaf are double leaf walls without rubble core, and consequently, without transverse connection between the ashlar leaves.

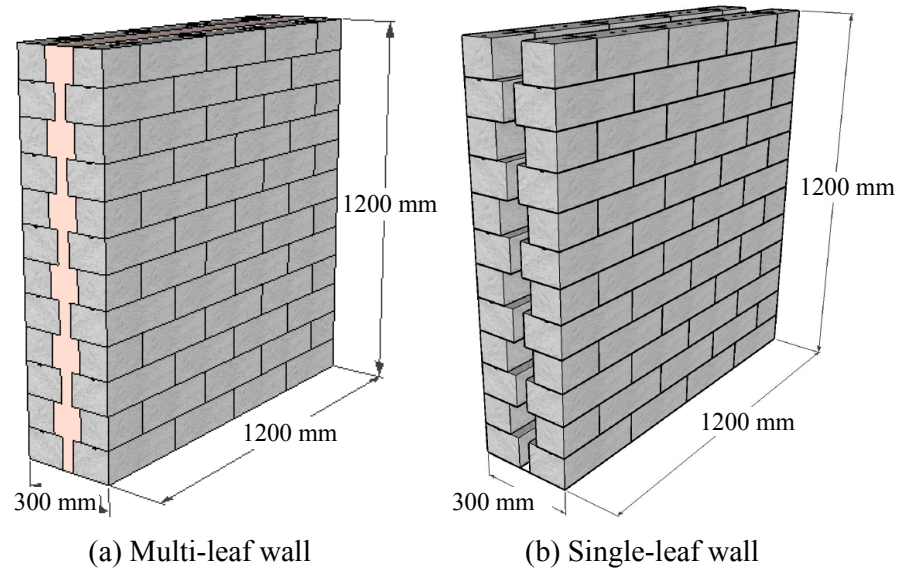


Figure 3.3 : Three-dimensional layouts of the tested walls.

Apart from the wall and stone unit dimensions, dimensions of the reinforcements were also scaled down. However, since it could not have been possible to obtain the identical quality of steel, an additional reduction in the cross section dimensions of the cramps and pins were also done. Details of the material mechanical characteristics and this additional cross-section area reduction shall be presented in

Chapter 4. Consequently, in this topic, only the final dimensions of the cramps and pins are presented as shown in Figure 3.4. Accordingly, the cramps utilized in the specimens had a length of 100 mm, width of 8 mm and thickness of 2.3 mm. Both ends of the cramps were bent (20 mm from each end), so that the bent parts could be inserted into the holes drilled on the stone surfaces. Pins had a circular cross-section with a diameter of 6.4 mm, instead of the square cross-section identified for the prototype wall. Length of the pins was about 20 mm, whose lower half was inserted to the top face of the lower stone course and the upper half was inserted to the bottom face of the upper stone course.

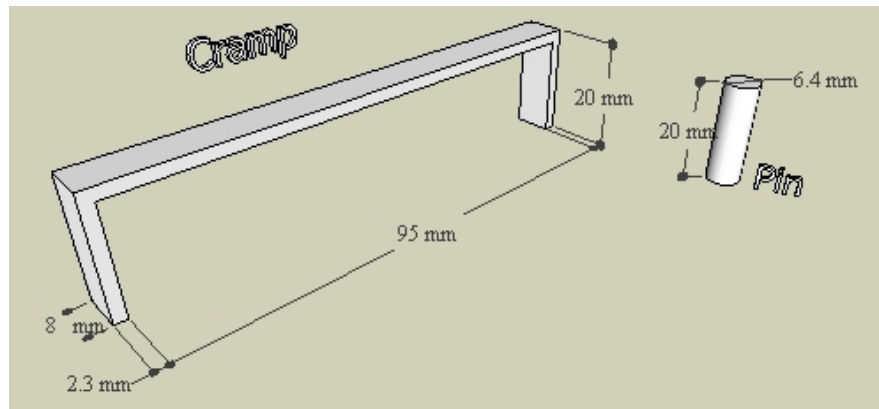


Figure 3.4 : Dimensions of the cramps and pins.

Effects of three main parameters on the shear behaviour of historical stone masonry walls were investigated during the experimental campaign. Namely these parameters were: Variation of pre-compression stress, existence of cramps and pins, existence of rubble masonry core. For this purpose, eight walls were designed with these test parameters, as shown in the test matrix given in Table 3.2.

Table 3.2 : Test matrix for shear compression tests.

Wall	Pre-compression stress	Cramps	Pins	Rubble core
	(MPa)			
M-25-C	0.25	Yes	No	Yes
M-50-C	0.50	Yes	No	Yes
M-75-C	0.75	Yes	No	Yes
M-100-C	1.00	Yes	No	Yes
M-50	0.50	No	No	Yes
M-50-CP	0.50	Yes	Yes	Yes
S-50-C	0.50	Yes	No	No
S-50-CP	0.50	Yes	Yes	No

It is a well-known fact that level of axial stress on the wall has paramount effect on the mechanical behaviour of masonry walls. Four different axial stress levels were applied during the tests: 0.25, 0.50, 0.75 and 1.00 MPa. Consequently, M-25-C, M-50-C, M-75-C and M-100-C walls composed the first comparison group that aimed to investigate the effect of pre-compression stress level. These walls were identical in means of geometry and reinforcement configuration. They had multiple leaves and the sole reinforcement was the cramps interconnecting the stone units. It is apparent that application of appropriate levels of axial loads, which are representative of actual stress levels in actual structures, is necessary for obtaining realistic results. For this purpose, a specific literature survey was carried out and an existing historical structure was analysed to decide the levels of axial stresses to be utilized during the tests. The results of this review study, carried out for obtaining approximate axial stresses in the walls of Classical Period Ottoman structures, are briefly presented in Appendix A.

Presence of cramps and pins was an another parameter so that the influences of these singular joint reinforcements could be evaluated. A comparison group that consists of walls with or without reinforcements were formed. During the design of this comparison group, 0.50 MPa axial stress level was selected as the reference value. Consequently, M-50 unreinforced masonry wall (URM) and M-50-CP wall with pins and cramps could be compared with M-50-C reference wall with cramps. In addition to that, single leaf S-50-C wall reinforced by cramps and S-50-CP wall reinforced by cramps and pins could also be compared with each other to figure out the contribution of pins.

In order to better identify the contribution of each leaf to overall wall behaviour, single-leaf ashlar wall specimens without rubble masonry core were also tested. Single leaf S-50-C wall with cramps and S-50-CP wall with cramps and pins were also subjected to 0.50 MPa pre-compression that was selected as the reference axial stress.

3.1.2 Construction of the walls

Construction of the walls was realized at the Structural and Earthquake Engineering Laboratory of Istanbul Technical University. In order to replicate the main characteristics of the multi-leaf walls in the heritage structures around Istanbul,

special attention was paid to apply similar materials and construction techniques. Two experienced masons, experienced in restoration of historical masonry structures, were hired for the construction of the specimens.

A variant of limestone, which is also locally known as Küfeki stone, was used during the construction of the walls. Küfeki stone was selected due to its wide usage in the past as the main construction material of the investigated wall type. Prior to construction, stone units were quarried and machine sawn to desired dimensions. Cutting of the stone units were done by using a wire saw so that the surfaces of the units were smooth. Dimensions of the stone units were 100×100×300 mm, 100×130×300 mm and 100×100×150 mm (height × width × length). The mortar used in the rubble masonry core was prepared by mixing different proportions of ready-mix mortar (Albaria Struttura from BASF Construction Chemicals Company) with silica sand. Please note that details of the material properties are available in Chapter 4.

Before starting the laying of the stone units, each unit reinforced by cramps or cramps and pins, were prepared for reinforcement attachment. For that purpose, two approximately 30 mm deep holes (with a diameter of 12 mm) were drilled on the top faces of the stone blocks. Two 5 mm deep slots at both ends of the units, where the cramps were to be laid, were excised by using a rotating saw. Stone units belonging to a cramp-reinforced wall is shown in Figure 3.5. In the case of walls reinforced by cramps and pins, in addition to holes drilled for cramps, four additional holes were drilled on the top and bottom faces (two holes on top and two on bottom faces) of the stone units (see Figure 3.6). The drilled holes on the top face had a diameter of 20 mm, while holes on the bottom face had 12 mm diameter. At the upper face, two channels were excised in the transverse direction to enable pouring of molten lead through these channels to the sockets of pins. Corners of the interior side faces of all stone units were chipped by using a small hammer so that a rough surface was obtained as also observed in existing structures, Figures 3.5 and 3.6.

Stretcher bond type (also known as running bond) masonry typology was applied during the construction of the outer leaves of the walls. Multi-leaf walls were 1200×1200×300 mm in length, height and thickness, respectively. Rubble core thickness was variable along the height of the transverse cross-section as seen in Figure 3.3. At stone courses where the width of each stone unit was 130 mm, net

rubble core thickness was reduced to 40 mm; whilst it was 100 mm for courses with 100 mm wide units. Single leaf walls had identical length and height while their net thicknesses were either 200 or 260 mm at successive stone courses.

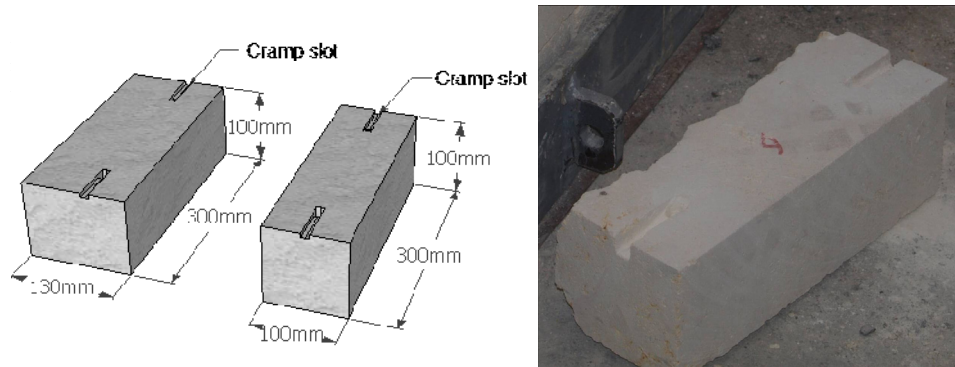


Figure 3.5 : Stone units of a cramp reinforced wall.

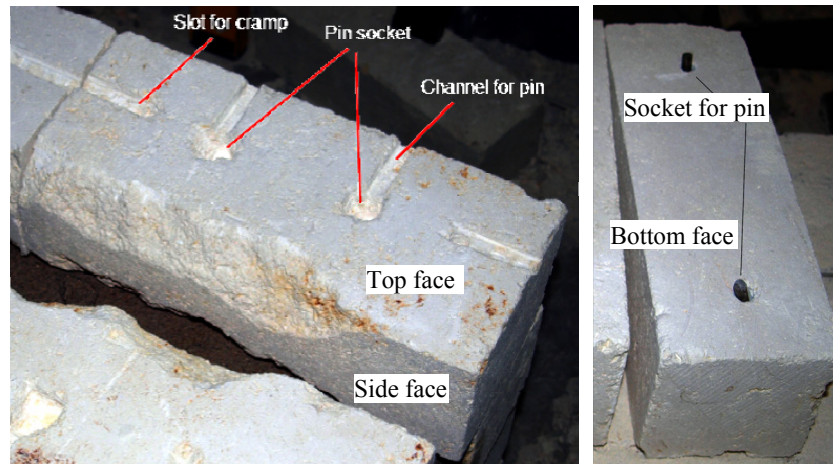


Figure 3.6 : Stone units of a cramp and pin reinforced wall.

Specimen construction was carried out on top of a steel U-profile (U330) as shown in Figure 3.7. Length of the base profile was 20 mm larger than the specimen length to tolerate minor workmanship errors. Due to 105 mm high flanges, first stone courses of the walls were confined by the steel profiles. Therefore, the first stone course was treated as the foundation of the wall. The 300 mm net span length between the flanges of the U-profile functioned as a mould so that wall thicknesses could be easily kept around 300 mm. Apart from acting as a foundation beam; the steel U-profile also eased the transportation of the constructed specimens. Ropes of the 150 kN capacity crane, available at the laboratory, was attached to the four lugs welded to the flanges of the U-profile and specimens could be lifted and positioned inside the test setup without causing any damage to the walls.

Prior to laying of the stone units, the steel U-profile was levelled on the ground and an approximately 15 mm thick layer of high strength repair mortar (Emaco S88C repair mortar of BASF Construction Chemicals) was formed inside it. After laying and levelling the first course, cramps were inserted into their slots (except the M-50 wall that did not have any reinforcement). In the next step, approximately 1.0 kg of lead sheet was melt in a pot that was heated by a LPG flamer. Soon after all lead sheets in the pot were heated and molten, liquefied lead was poured into the cramp slots. All slots in the first course of the wall were filled until excessive lead overflows. Please note that views from these steps are presented in Figure 3.8.

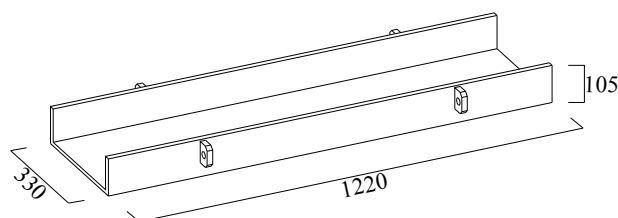


Figure 3.7 : Base profile used in the wall construction (dimensions in mm).

While building the first course of the wall, mortar that will be used in the rubble masonry was mixed aside. Since the mechanical characteristics of the ready-mix Albaria Struttura mortar (from BASF Construction Chemicals) was too good for replicating the mortar quality in existing heritage structures, one proportion of Albaria Struttura was mixed with three proportions of sand and 0.75 proportion of water (1.0:3.0:0.75 by weight for mortar : sand : water, respectively). Then the space left between the two stone courses was filled with this mortar mix until mid-height of the course. Stone pieces with radii between 20 and 50 mm were dropped into the filled mortar from approximately 30 cm height. No additional compaction was applied to the stone pieces so that air voids observed in walls of existing structures could also be formed. Finally, the dropped stone pieces were covered by mortar and laying of next stone course was initiated. Steps described above were repeated for 12 courses of the wall until a height of 1200 mm was achieved (Figure 3.9). At each stone course, the dimensions of the wall and the levelling of the units were checked. Single-leaf walls were also built similarly. The sole difference was the absence of the rubble core.

Three of the cramps at one of the outer leaves were instrumented with strain gauges. These cramps were prepared prior to construction process. Strain gauges that would

enable the monitoring of deformations at the cramps, were bonded to upper and lower sides of the cramps (at mid-length of the cramp) and then coated with an insulation tape. The instrumented cramps were utilized at mid-lengths of the 5th, 7th and 9th stone courses of the walls. As seen in Figure 3.10, small cuts were done with a rotary saw to let out the cables of the strain gauges.



Figure 3.8 : Construction of the outer leaves.



Figure 3.9 : Construction of the rubble core.



Figure 3.10 : Fitting of the instrumented cramps.

In the case of M-50-CP and S-50-CP walls with cramps and pins, two pins were inserted to the bottom faces of each stone unit and fixed by pouring molten lead (as seen in Figure 3.6). While putting the stone units on top of each other, these pins corresponded to holes drilled on the top face of the lower stone course. After finishing laying of the 12 stone courses, molten lead was poured through the transverse channels until the pin socket and the transverse channel was completely filled. Before pouring the lead for transverse channels, the specimens were slightly tilted by using the crane available in the laboratory, so that the flow of lead through the transverse channels was eased. To avoid filling of the lower level channels, pouring process was started from the lower courses of the specimens. A steel sheet was bended and used as a funnel, so that the lead flow could be directed towards the channels.

In Figure 3.11a, stone units with sockets for pin insertion and transverse channels for molten lead pouring are visible. Image obtained by a videoscope that was inserted through a transverse channel is presented in Figure 3.11b. In this image, a pin inserted into the pin socket can be clearly seen. Figure 3.11c and 3.11d shows the pouring of the molten lead through the transverse channel so that the voids around pins could be filled.

Before and during the construction of the walls, several samples were taken from the stone units and mortar mixes. Mortar samples consisted of 40×40×160 mm prisms while the stone samples had different dimension depending on the targeted test type. The rubble masonry samples were built in a similar fashion with the rubble core. All samples were kept in similar conditions with the walls and no curing was applied to

walls and material samples. Please note that the details of the material tests are presented in Chapter 4.



Figure 3.11 : Installation of the pins.

3.2 Test Setup

All walls were tested at a test rig that was particularly designed and constructed for this study. The test rig consisted of steel profiles welded at an atelier and transported to Structural and Earthquake Engineering Laboratory of Istanbul Technical University. During the design process, special attention was paid for ensuring strength and adaptability requirements. Two images from the test rig design and construction processes are shown in Figure 3.12.

Schematic view and general layout of the test setup is shown in Figures 3.13 and 3.14. Test rig and the reaction frame were strictly fixed to the 1.20 m thick strong floor (reaction slab) of the laboratory via post-tensioned rods. These rods were passing through the holes that formed a grid with 1.0 m spacing. Specimens were translated to the test rig by using the 150 kN capacity crane of the laboratory. Cables

of the crane were attached to the lugs welded to the steel U-profiles on which the walls were built.

Two 20 mm thick plates, stiffened with 20 mm thick ribs, were used to restrict the horizontal sliding at the support level (first course) of the walls (Figure 3.15). Specimens were carefully positioned and aligned between these reaction plates that were located at both ends of the walls. Before the tests, any possible gap between the walls and plates were filled with steel sheets and high strength repair mortar.

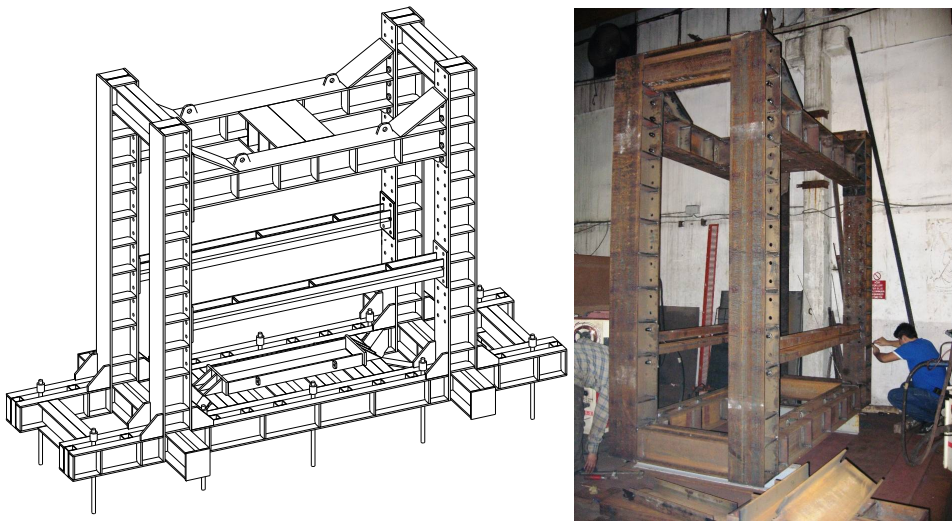


Figure 3.12 : Design and construction of the test rig.

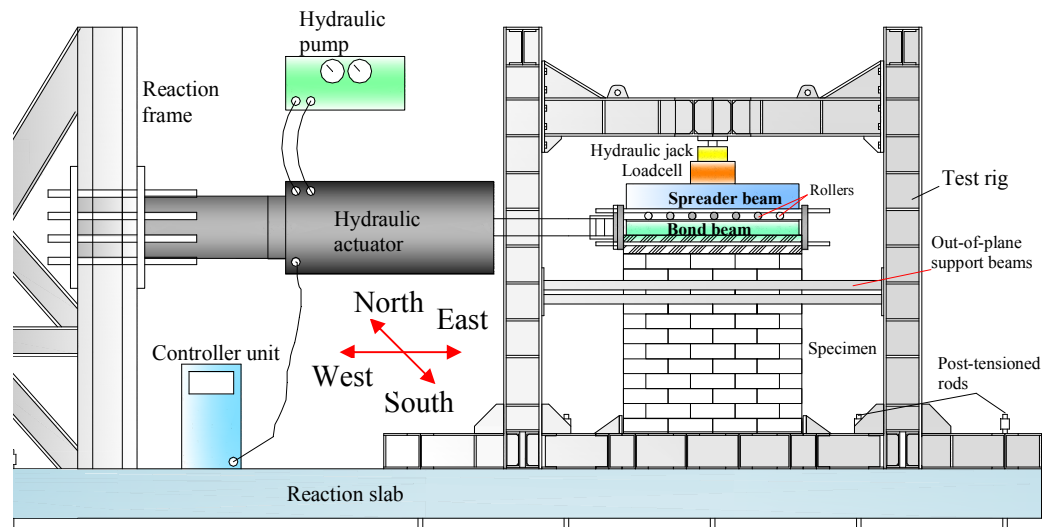


Figure 3.13 : Schematic view of the test setup.

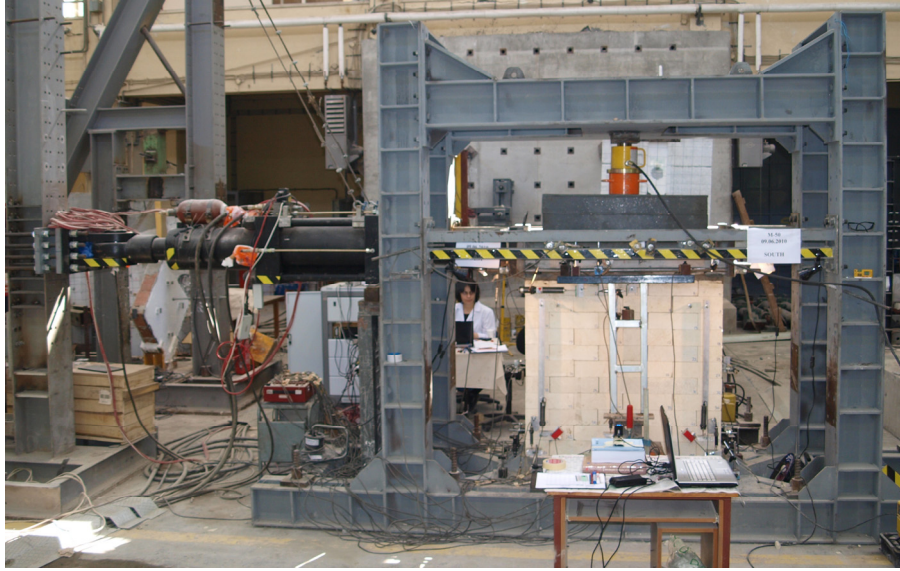


Figure 3.14 : General layout of the test setup.



Figure 3.15 : Stiffened end plates at foundation level.

Assembly that constituted the upper boundary conditions of the walls is shown in Figure 3.16. Following the positioning of the wall into the test rig, top face of the specimen was capped with an approximately 20 mm thick high strength repair mortar layer (Emaco S88 C from BASF Construction Chemicals Company) so that a smooth surface was obtained. Levelling of the mortar cap was checked by using a laser level. Before setting of the repair mortar, a reinforced concrete bond beam, particularly cast for this study, was placed upon the top face of the wall. Again, special attention was paid for alignment and levelling of the bond beam. The bond beam was cast inside a 1200 mm long U330 steel profile, to which one of the actuator end plates was welded. Consequently, the reinforced concrete bond beam, U330 steel profile and the actuator end plate constituted one single part.

Eight 50 mm diameter steel rollers were evenly distributed along the top face of the U330 profile that was lubricated before each test by using grease. These rollers allowed the top of the wall to drift with minimized friction in the longitudinal direction. Above the steel rollers, a very stiff steel spreader beam was placed and carefully aligned, as seen in Figure 3.16. This beam supplied base for the loadcell and hydraulic jack that were utilized to monitor and apply the axial forces of the walls. An Enerpac electric driven hydraulic pump pressurized the 600kN capacity Larzep hydraulic jack used in this study. The force acting on the spreader beam could be monitored by using the 1000 kN capacity TML CLP-100 loadcell and could be kept almost constant by fine tuning the electric pump.

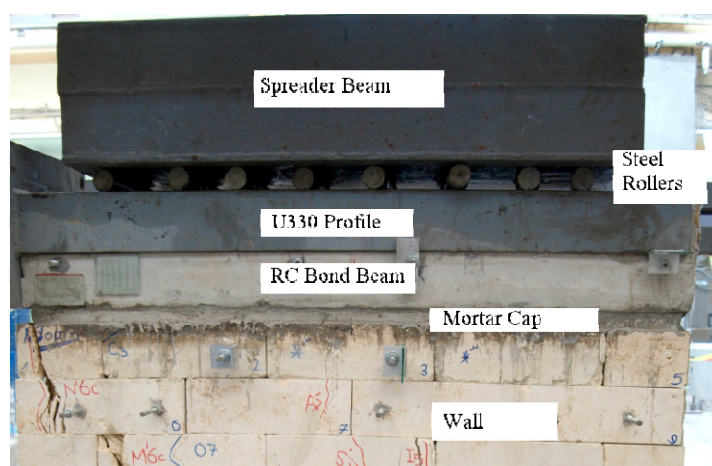


Figure 3.16 : Assembly of the upper boundary of the walls.

Although utmost attention was paid during the alignment of the hydraulic actuator, wall and the axial force assembly, additional measures were also taken to restraint the possible out-of-plane movement of the walls. For this purpose, two steel beams were attached in parallel to both longitudinal sides of the specimens. These beams were bolted to the columns of the test rig at the level of bond beam. Three rollers with polyamide wheels were mounted to these beams at each side of the walls. These wheels were slightly touching the U330 steel profile of the bond beam. A view of this assembly is shown in Figure 3.17.

Prior to fixing of the hydraulic actuator, a small amount of axial force was applied to the walls so that they could have some lateral force capacity, which might be needed in case of a small accidental lateral loading. Lateral force was exerted by a ± 990 kN force and 400 mm displacement (± 190 mm capacity for pushing and pulling directions) capacity TDG actuator that was fixed to the reaction frame. Stroke of the

actuator was mounted to the bond beam by using four high strength steel rods. A Moog controller unit and a computer were utilized to implement the displacement steps during the mounting of the actuator and tests. TDGLAB Structure software supplied the user interface and connection between the Moog controller and the computer.

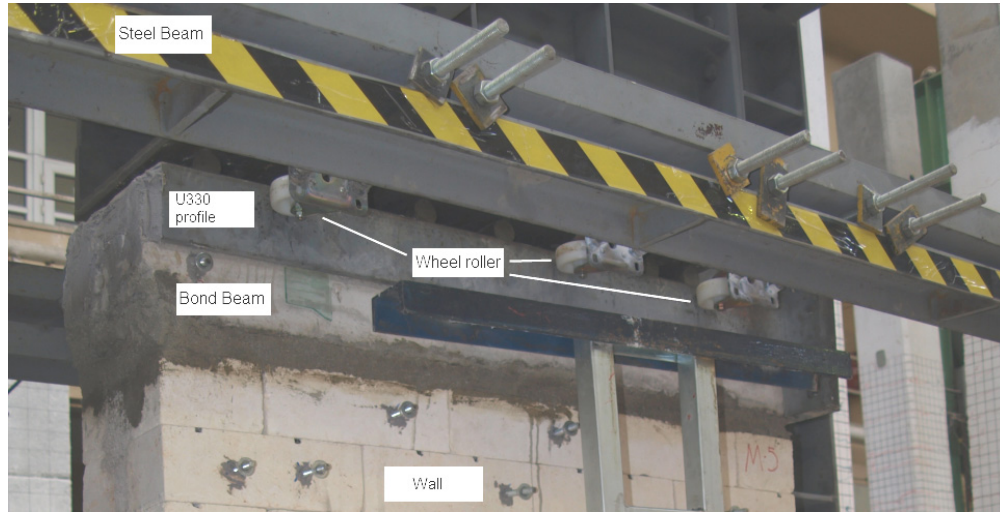


Figure 3.17 : Assembly for prevention of out-of-plane movement.

All sides of the walls were named for easier reference during the recording of the observations and damage. Longitudinal side facing the Moog controller was named as North, the other longitudinal side facing the control room of the laboratory as South face, transverse side near the actuator as West and finally the side facing the 2A gate of the laboratory as East (Figure 3.13).

3.3 Instrumentation of the Walls

The displacements, deformations and loads were measured by using different types of sensors. A total of 43 data collection channels, consisted of 35 linear variable differential transformers (LVDTs), 2 loadcells and 6 strain gauges were installed on each shear compression test specimen. Schematic diagrams of instrumentation are shown in Figures 3.18 and 3.19 for the Northern and Southern faces, respectively. In these figures, the full red dots correspond to gauge points on the wall and red triangles correspond to gauge points on reference measurement frames used at both sides of the wall. Almost all sensors had a counterpart at the opposite side of the wall, so that measurements from both leaves could be compared and averaged.

The LVDTs mounted on the walls had varying stroke length capacities selected according to the targeted measurements. For that purpose; CDP-10, CDP-25, CDP-50, CDP-100 and DP-1000-E type LVDTs from TML company that had 10 mm, 25 mm, 50 mm, 100 mm and 1000 mm stroke lengths, respectively, were installed on the specimens.

Channel 1 was used to record the horizontal displacement of the actuator's stroke. However, this internal displacement transducer of the TDG actuator was not referred during the processing of the experimental data. Instead, Channels 10 and 11 with 25 mm stroke length and Channels 12 and 13 with 100 mm long stroke length were averaged to obtain the horizontal displacement at the top of the wall. LVDTs with shorter stroke length but higher sensitivity were used as the control channels during the initial displacement cycles of the tests. Then Channels 12 and 13 took the duty until the end of the tests. Channels 14 and 15 were compared with Channels 12 and 13 in order to identify any slip between the bond beam and the top of the wall. Channels 16 and 17 read the horizontal displacement of the second stone course from bottom and these were subtracted from top displacement so that the net displacement of the wall could be obtained. Channels 18 and 19 read the horizontal displacement of the bottom U330 profile relative to the test rig. It should be noted that the slip values observed during the tests between the test rig and the bottom U330 profile were very small, thus, negligible.

Channels 33 and 34 measured the vertical displacements at both ends of the bond beam. By using these two LVDTs, it became possible to calculate the rotation of the upper part of the wall. Similarly, Channels 24-27, read vertical displacements at the lower part of the wall.

Channels 28-31 measured the vertical deformations at the toe and heel of the wall. By using these LVDTs, it could be possible to identify the initiation of toe crushing at the compressed zones. Channels 36-41 were used for evaluation of the modulus of elasticity during the initial axial loading of the wall and also used for monitoring the opening of the bending cracks during the lateral loading.

Channels 21-23 generated displacement data for the 1000 mm length along the diagonals of the wall. Channels 44 and 45 measured the transverse opening of the outer leaves of the wall. Channels 42 and 43 were followed to monitor any possible out-of-plane movement or any torsional effects that might take place. It should be

noted that no significant out-of-plane movement or torsion was observed during the tests. Finally, Channel 46 was used to check if any translation of the test rig, relative to laboratory strong floor, occurs. No significant translation was observed during the tests.

Two loadcells, one for measurement of the axial force and one for the horizontal actuator, were utilized during the tests. Channel 0 was the internal loadcell of the TDG actuator. Channel 2 was continuously monitored to ensure application of a constant axial load.

In the case of specimens reinforced by cramps, six strain gauges (Channels 50-55) were bonded to three of the cramps placed at the 5th, 7th and 9th courses at mid-length of the walls. Two strain gauges were used for each instrumented cramp, one at the upper and one at the lower face. The TML YFLA-5 type strain gauges were selected due to their capability to continue functioning even beyond the yielding strain of the steel. Prior to bonding of the strain gauges to the mid-length of the cramps, these zones were rubbed with sand papers of different grades and cleaned with acetone-imbued cotton. A Cyanoacrylate based adhesive was used for the bonding process. Then the strain gauges were coated with an insulation tape to protect them from moist and any possible impact.

Data collected from all sensors were digitized by a TML TDS 302 Data Logger, which was connected to a laptop computer via a GPIB to USB interface converter. Measurements from the internal loadcell and LVDT of the hydraulic actuator were also transferred from the Moog controller unit to the data logger. Since the number of available onboard channels was limited to 10 for the TDS 302 data logger, an ASW 50-C switch box was used for installing the remaining channels.

Apart from the sensors mounted on the specimens and the test rig; development of damage, location and widths of the cracks were recorded on paper at each displacement cycle peak. Additionally, several photo and video shoots were also taken.

Another visual recording was also performed by forming a time-lapse video setup that consisted of a 2 Megapixel camera and a laptop computer. The photo shoots took place at an interval of 30 seconds and the videos were prepared for 25 frames per second. These time-lapse videos significantly eased the after test observation of the

behavior of the walls, since the actual experiments took almost 12 hours for each wall. However, this long duration was compressed to approximately 90 seconds long videos by applying this technique.

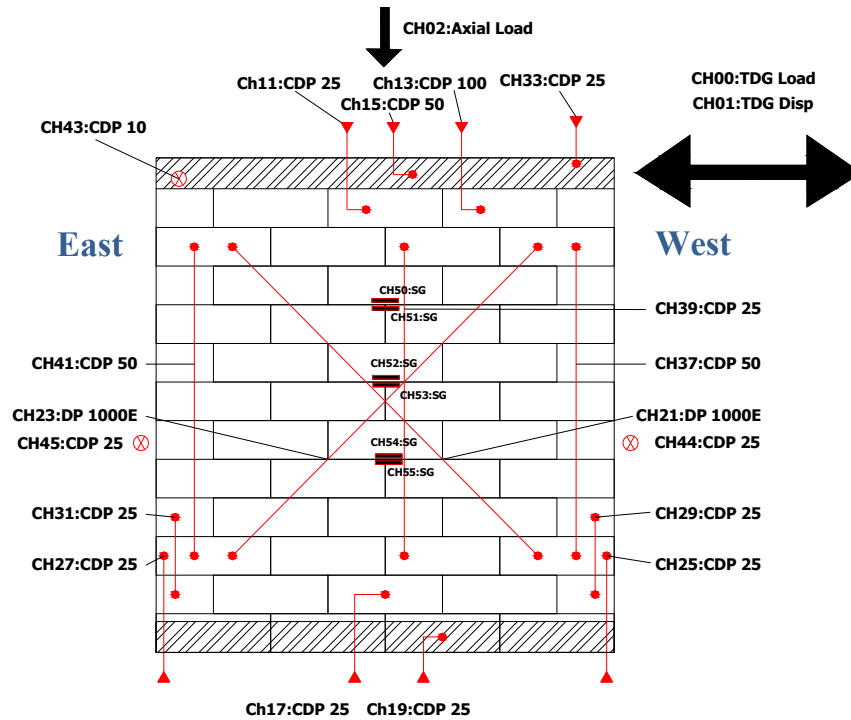


Figure 3.18 : Instrumentation of the specimens-Northern face.

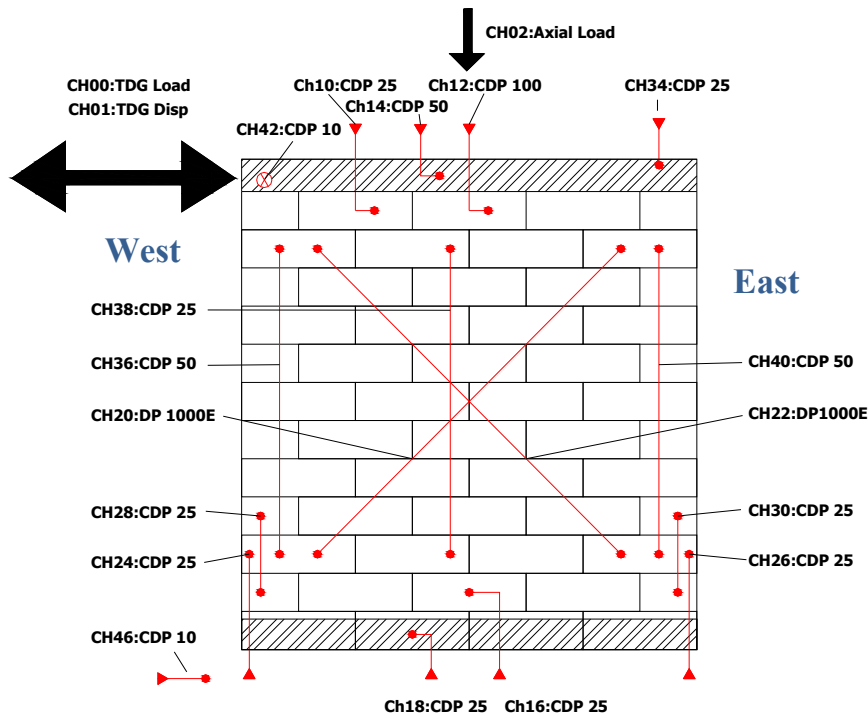


Figure 3.19 : Instrumentation of the specimens-Southern face.

3.4 Loading of the Walls

Loading of the walls were performed at two steps. At the first step, the axial load was slowly applied until the desired pre-compression stress level was achieved and then unloaded to almost zero axial force. These loading and unloading cycles were done for three times. Simultaneously data was collected from all sensors, so that variation of deformations under axial loading and unloading cycles could be recorded.

After completing the first step, all sensor channels were initialized, horizontal hydraulic actuator was turned on and pressure was applied. Then the axial load was exerted for the fourth time but this time kept constant at the targeted load level. During the tests, axial force acting on the walls was carefully monitored and necessary adjustments were done to keep it at the desired value.

In all tests, reversed cyclic loading was applied under displacement control to allow obtaining of the complete load-deformation response including the descending branch in the post-peak region. As indicated earlier, Channels 10 and 11 with 25 mm stroke length and Channels 12 and 13 with 100 mm long stroke length were used as the control channels.

All lateral displacement cycles targeted prescribed drift ratios, which progressively increased in amplitude. Drift ratios and displacements corresponding to each cycle number are tabulated in Table 3.3 and the loading pattern used as the input seismic action is shown in Figure 3.20.

Certain cycles near the peak load and in the post-peak region were applied for three times while the rest of the cycles were applied only for once. The same loading pattern was applied to all eight specimens. For any cycle, the increment for displacement steps was started with small values such as 0.1-0.2 mm/step but then gradually increased to a maximum value of 1.0 mm/step. After each step, a data collection period of 5 seconds was spared for the data logger. Consequently, considering the durations for ramp time, trigger for data collection and data collection; an approximate loading speed of 1.0 mm/10 s can be pronounced.

The loading of the walls continued until peak load dropped in both directions at least by 20% or extensive damage occurred with an indication of loss of stability. Accordingly, depending on the axial stress level and reinforcement configuration,

some of the specimens could survive up to 2.0% drift ratio while some failed at smaller drift ratios.

Table 3.3 : Target drift ratios and displacements for each cycle.

Cycle	Drift ratio	Displacement	Cycle	Drift ratio	Displacement
	(%)	(mm)		(%)	(mm)
1. Push	0.05	0.55	6. Push (a)	1.00	11.00
1. Pull	-0.05	-0.55	6. Pull (a)	-1.00	-11.00
2. Push	0.10	1.10	6. Push (b)	1.00	11.00
2. Pull	-0.10	-1.10	6. Pull (b)	-1.00	-11.00
3. Push	0.25	2.75	6. Push (c)	1.00	11.00
3. Pull	-0.25	-2.75	6. Pull (c)	-1.00	-11.00
4. Push (a)	0.50	5.50	7. Push	1.25	13.75
4. Pull (a)	-0.50	-5.50	7. Pull	-1.25	-13.75
4. Push (b)	0.50	5.50	8. Push	1.50	16.50
4. Pull (b)	-0.50	-5.50	8. Pull	-1.50	-16.50
4. Push (c)	0.50	5.50	9. Push (a)	2.00	22.00
4. Pull (c)	-0.50	-5.50	9. Pull (a)	2.00	22.00
5. Push	0.75	8.25	9. Push (b)	2.00	22.00
5. Pull	-0.75	-8.25	9. Pull (b)	2.00	22.00

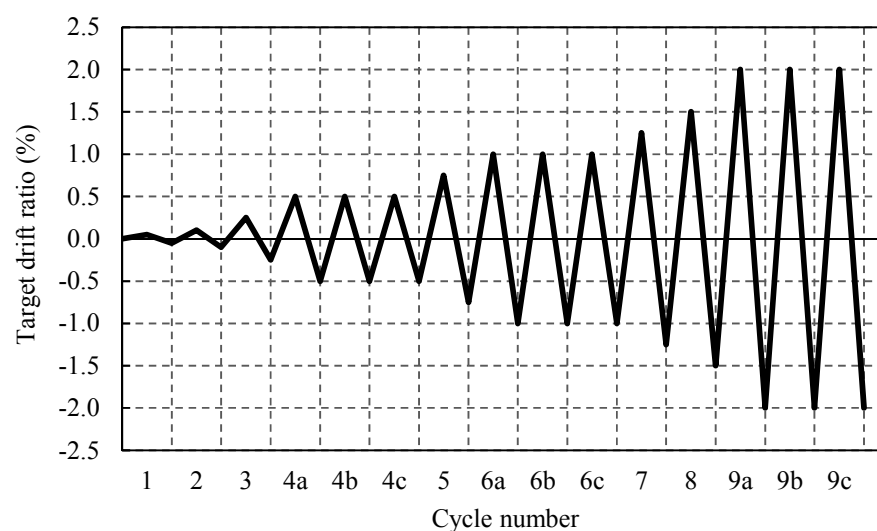


Figure 3.20 : Loading pattern.

4. CHARACTERIZATION OF THE MATERIALS

Three main constituent materials were used during the construction of the walls: Küfeki stone, mortar and steel. Apparently, accurate material selection is a key to success in an experimental study. However, due to the complexity and variety of the materials used in the heritage structures, it is almost impossible to represent them perfectly. Despite this fact, it is still worthwhile to spend efforts to search for realistic materials for experimental campaigns, particularly in terms of mechanical characteristics. In this chapter, main characteristics of each constituent material shall be sought from literature-based data. Then the material tests carried out in this study for original and representative materials shall be described.

4.1 Mechanical Characterization of Stone Units

Küfeki stone was the main construction material of the Ottoman Empire's monumental structures built in the imperial capital Istanbul and her vicinity. This type of stone was widely utilized at the main walls, exterior and interior claddings, arches, portals, slabs, pillars, niches, fences and decorations. In order to accommodate with these different applications; stones were cut, treated and used in various alternative fashions. During the construction of the main load bearing walls, cut-stone, regular coursed rubble or rubble masonry were used (in some cases with stone-brick alternate).

Küfeki stone was preferred due to the availability of the stone quarries in the vicinity of Istanbul and due to the advantages such as being lightweight and easily sawn, having acceptable mechanical and physical properties. In addition to these advantages, as reported by Arıoğlu and Arıoğlu (1999 and 2005), the mechanical and physical properties continue to develop due to carbonation process after the Küfeki stone is quarried and transported to the construction site. Consequently, the stone is rather easily treated soon after the quarry process and continues to develop strength even after the construction is complete.

Küfeki is an Upper Miocene age fossiliferous hollow cryptocrystalline limestone that has been obtained from the quarries in today's Bakırköy, Sefaköy, Halkalı, Hadımköy-Sazlıbosna provinces of Istanbul. However, most of these quarries are not active currently, (Arioğlu and Arioğlu, 2005). Matrix of Küfeki limestone mainly consists of CaCO_3 (93 to 100 %) surrounding the mactra, melanopsis, helix and crinoid fragments. This composite material can be found in nature in massive and intact forms and do not contain any rifts (Arioğlu et al. 1999a and b). The lower layers of the limestone formation are fine grained, very fractured with many fossil shells and the upper level of the formation is clayey and sandy. Especially the lower layers of this type of limestone were used as building materials (Tuğrul and Zarif, 1999).

4.1.1 Previous studies on Küfeki stone

In literature, a small number of studies on the mechanical, chemical and physical characteristics of Küfeki limestone exist. A series of studies performed by Arioğlu and Arioğlu (2005), Arioğlu and Arioğlu (1997), Arioğlu et al. (1999a and b), Arioğlu and Arioğlu (1999) aimed to investigate the mechanical and physical characteristics of Küfeki, with special emphasis to carbonation event that improves the physical and mechanical characteristics by the time. After a series of experiments, they have proposed an empirical expression for the strength development of Küfeki limestone. Another study for characterization of Küfeki limestone has been done by Erdoğan and Mahmutoğlu (2006). In their technical report, results of several physical and mechanical tests for Küfeki limestone quarried from Hadımköy-Sazlıbosna quarry are summarized. Main results of these studies are summarized in Table 4.1.

A close look at the results of the experimental studies performed on Hadımköy-Sazlıbosna Küfeki stone reveals that characteristic values for physical and mechanical properties may even vary in the same quarry. Considering that the existing heritage structures were built by stone units quarried at different locations and dates, it is apparent that no certain material can perfectly represent the stone quality at existing structures.

It should be noted that, the limestone that were used for the specimen construction of this thesis was also obtained from the Hadımköy-Sazlıbosna quarry.

Table 4.1 : Mechanical and physical characteristics of Küfeki stone (from literature).

	Arioğlu and Arioğlu (1999 and 2005)	Arioğlu and Arioğlu (1997)	Erdoğan and Mahmutoglu (2006)
Quarry	Hadımköy-Sazlıbosna	Hadımköy-Sazlıbosna	Hadımköy-Sazlıbosna
Uniaxial compressive strength (MPa)	33.2 (at one day), 45.5 (at 30 days), air cured, 15×15×15 cm	21.1 (at 3 days), 32.9 (at 28 days), air cured 15×15×15 cm	40.3, 40.6, 40.1 MPa (x,y and z directions), D50×100 mm cylinders
Modulus of elasticity (MPa)	24000	N/A	N/A
Tensile strength (MPa)	$f_t=3.6$ (at 30 days)	$f_t=2.6$ (at 28 days)	N/A
Ultrasonic pulse velocity (km/s)	3.88 (at one day), 5.01 (at 30 days)	3.63 (at 3 days), 4.26 (at 28 days)	N/A
Unit weight (kN/m ³)	21.9	23.2	23.1 (dry), 24.1 (water saturated)
Porosity (%)	12.60	N/A	9.91
Water absorption (%)	5.70 (by weight), 11.08 (by volume)	N/A	4.30 (by weight)

4.1.2 Uniaxial compression tests on single stone units

Uniaxial compressive tests, that aimed to define the complete stress-strain behaviour of Küfeki stone, were carried out at the Building Materials Laboratory of Istanbul Technical University, Civil Engineering Faculty. The stone blocks, sawn in required dimensions, were purchased from the Barshan Company, which operates the Küfeki stone quarries in Hadımköy-Sazlıbosna area. Five pallets of stone units, weighing about 10 tons, were transported to the Structural and Earthquake Engineering Laboratory of Istanbul Technical University. In order to represent the general characteristics of the five pallets of stone blocks, similar numbers of units were randomly chosen from each group resulting with a total specimen number of 40.

Pre-peak mechanical characteristics such as Young's modulus and Poisson's ratio; peak characteristics such as compressive strength and corresponding strain were derived from load and deformation measurements done during the tests.

The specimens to be tested under uniaxial compression were prepared in accordance with the ASTM C170-90 (1999) standard. According to this standard, the rectangular prism specimen's lateral dimensions should be at least 50.8 mm and the height to width ratio should be at least one. The height to width ratio of the specimens plays a

crucial role in reducing the influence of the boundary conditions. The confinement effect caused by the friction between the loading platens and the ends of the specimen may significantly increase the compressive strength and alter the stress distribution. An appropriate height to width ratio would minimize the influence of this effect and help to maintain a uniform stress distribution at central zone of the specimen.

Cross-section dimensions of the specimens were either 100×100 mm or 100×130 mm and height to width ratios were between 1.1 and 1.6 (mainly 1.3). All specimens were extracted from larger units by using a rotating saw. Since Küfeki stone does not contain any specific bedding direction or anisotropy, ends of the specimens, which were in better condition after sawing, were selected as loading ends. The upper and lower load bearing ends were carefully ground by using sanding paper to ensure that they are parallel to each other and perpendicular to longitudinal direction of the specimen. Two of the stone unit specimens ready for testing are shown in Figure 4.1.



Figure 4.1 : Single unit compression test specimens ready for testing.

Arioğlu and Arioğlu (2005) reports that the mechanical and physical properties of the Küfeki stone continue to develop due to carbonation process after it is quarried. In order to have similar physical and mechanical characteristics during all tests (material and member tests) that were going to be performed at different times, the fresh units were kept in the laboratory for approximately four months after the quarry process. During this duration, the rate of the carbonation event decreased and the stones lost their excessive water left from the sawing process. To ensure that the stone specimens have reached their natural water contents, the unit weights of the specimens were measured at three successive days until weight change is stabilized.

Unit weights obtained at final measurements are listed in Table 4.2. Since the larger scale masonry wall compression and shear compression tests were done under natural water content, all material tests were also done with natural water contents of the stone units.

All specimens were tested with a 2000 kN capacity universal testing machine. The applied axial force was measured via a TML CLP-100 type 1000 kN capacity loadcell, which was placed beneath the specimens. The axial deformations were monitored by using two linear variable differential transformers (LVDTs), positioned at equal distances to opposite faces of specimens. TML CDP-25 type LVDTs had 25 mm displacement measurement capacity and the magnets holding the LVDTs were fixed to the loadcell, while the tips of their strokes were in contact with the upper platen, Figure 4.2.

In order to measure the Young's modulus and Poisson's ratio, four specimens were instrumented with TML PL-60 type strain gauges. Each specimen had four strain gauges (two for axial and two for transverse strains) and the strain gauges were bonded on mid-heights of two opposite faces, Figure 4.1. Prior to bonding of strain gauges, stone surfaces were carefully prepared firstly by grinding with sand paper and then applying a very thin layer of epoxy.

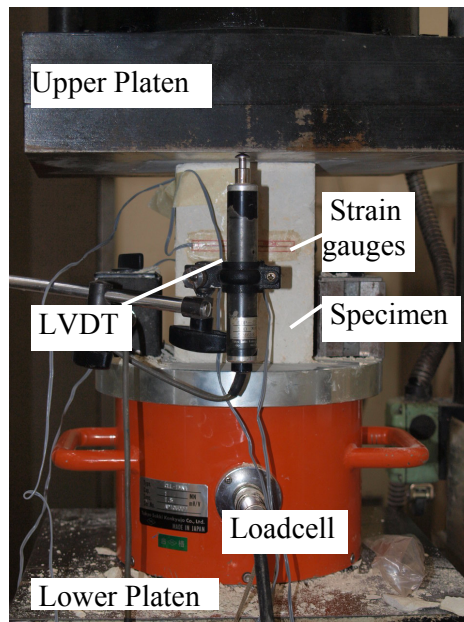


Figure 4.2 : Test setup of single unit compression tests.

Before starting the tests, dimensions of each prism were measured. Then, the specimen was placed onto the loadcell, which was carefully centred on the lower loading platen. Afterwards; upper platen, which had a hinge to prevent unwanted bending effects, was adjusted and a small pre-load of 5.0 kN was applied and unloaded. Prior to loading, all instruments were connected to a computer via a TML TDS 302 type data logger, and their functionality was checked. The connection between the computer and the data logger was established through a GPIB to USB converter.

Loading was supplied with the upwards movement of the lower platen. Therefore, somehow, tests were axial displacement controlled. Rate of loading was 3.0 kN/s and was kept almost constant during the tests.

The average axial strain of each specimen was obtained by dividing the average displacement of LVDTs (change in height) by the initial height of the specimen. The strain gauges on four of the specimens also supplied deformation data that was compared with the LVDTs.

As mentioned before, a total 40 stone unit specimens were loaded under axial compression. However, due to inhomogeneous structure of the natural material and due to possible defects in some of the specimens, a small number of specimens gave incompatible results with the rest of the sample set. In order to identify the outliers, that exhibited significant variance from the mean value of the set, application of an acceptable criterion was required. For this purpose, an old but widely used criterion, the Chauvenet's criterion, was selected for its applicability and simplicity. Chauvenet's criterion is based on the assumption of a normally distributed population from which all values are taken. An observed value is designated as outlier and discarded if the probability of its occurrence is less than or equal to $\frac{1}{2N}$ where N is the number of observations (Akman, 1978).

The Chauvenet's criterion was applied on the compressive strength results of 40 specimens that formed a set. With the application of this outlier detection criterion, three of the test results were rejected. As a result, the final specimen pool consisted of three tests instrumented with strain gauges and LVDTs and 34 tests instrumented only with LVDTs for the measurement of axial deformations.

Axial stress-strain curves of all specimens, obtained by using the LVDTs for axial deformation, are plotted in Figure 4.3. In this figure, each colour coding corresponds to a different pallet used during the packaging of the cut stones before their transportation to the laboratory. Random distribution of colours shows that distribution of mechanical characteristics is also random within the pallets. It is also interesting to note that the compressive strengths vary in a significantly wide range (from 7.5 MPa to 25.5 MPa), which may be expected for natural stones. However, axial strains corresponding to peak stress stay in a narrow band of 3000 and 4000 microstrain.

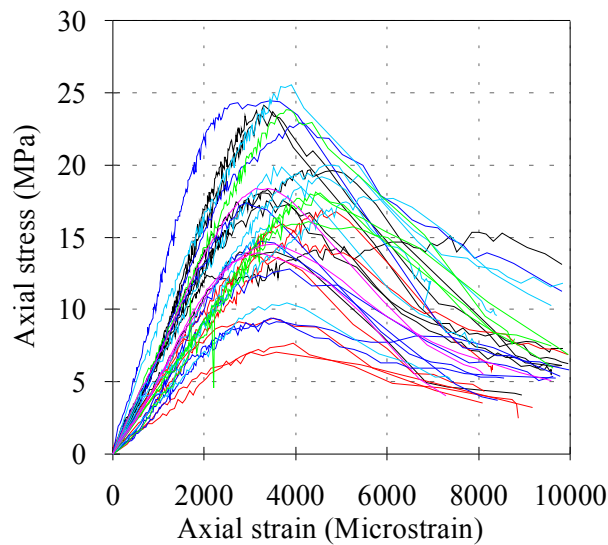


Figure 4.3 : Axial stress-strain relationships of all single unit compression tests.

Almost all specimens exhibited a quasi-brittle failure as in the case of concrete compression tests. Behaviour was almost linear until the vicinity of the peak load and macro-cracks were visible only at stresses near the peak. Unlike most of the brittle stones, like granites and sandstones, the failure did not happen very rapidly, but with a clear descending branch. This quasi-brittle failure might be attributed to the lower strength of the material in comparison with these brittle stone types.

Typical crack distributions of two specimens in the post-peak zone are presented in Figure 4.4. Inclined cracks near the upper and lower ends of the specimens are clear indications of confinement effect, resulting from the friction between the specimen-loading platen interfaces.

As also presented in Table 4.2, mean value of all peak stresses (f_c) is equal to 17.0 MPa while standard deviation and coefficient of variation values are as high as 5.1

MPa and 29.9%. According to ASTM C170-90 (1999) standard, when the ratio of height to lateral dimension differs from unity by 25% or more, compressive strength of an equivalent cubical specimen ($f_{c,cube}$) may be calculated by utilizing Eq. (4.1).

$$f_{c,cube} = f_c / [0.778 + 0.222(b/h)] \quad (4.1)$$

where b and h are the lateral dimension and height of the specimen, respectively. When this relationship is applied, mean value of peak stresses becomes 18.0 MPa, while standard deviation and coefficient of variation becomes 5.3 MPa and 29.6%, respectively.



Figure 4.4 : Typical distribution of macro-cracks in single unit compression tests.

Stress-strain (axial and transverse strains) curves obtained for two of the specimens, instrumented with strain gauges and LVDTs, are presented in Figure 4.5. In this figure, it should be noted that, stress-strain curves obtained from strain gauges and LVDTs differ in the pre-peak and post-peak regions. This difference is mainly due to the differences in the gauge lengths (60 mm for strain gauges and specimen heights for LVDTs) and existence of deformations between the loading platen-specimen interfaces. It is also apparent that Young's modulus values obtained from strain gauge measurements would be slightly higher than that of LVDTs. For instance, the Young's moduli obtained by using strain gauges are 7427 and 7903 MPa for K1 and K2 specimens while they become 6238 and 5820 MPa, respectively, if LVDTs are utilized. It should be noted that, the data supplied by the strain gauges and LVDTs were obtained as average of two sensors for each device type. Comparing the forms of axial stress-axial strain and axial stress-transverse strain curves, it can be observed

that the axial stress-transverse strain curves exhibit a higher nonlinearity while approaching the peak stresses.

Table 4.2 : Results of single unit compression tests.

Specimen	h/b	f_c	$f_{c,cube}$	Young's modulus	Unit weight
		(MPa)	(MPa)	(MPa)	(kN/m ³)
K1	1.3	15.4	16.2	5585	21.2
K2	1.3	19.7	20.7	6798	21.2
K3	1.3	24.1	25.5	9736	20.9
K4	1.3	18.1	19.1	6982	21.1
K5	1.3	14.0	14.8	5388	20.5
K6	1.3	17.5	18.5	6502	21.2
K7	1.3	23.7	24.9	8720	21.3
K8	1.3	16.9	17.8	4493	20.4
K9	1.3	15.9	16.7	3407	20.5
K10	1.3	9.4	9.9	4086	19.7
K11	1.3	13.7	14.5	5187	20.2
K12	1.3	7.1	7.5	3181	19.2
K13	1.3	7.6	8.1	2351	18.9
K14	1.3	9.4	9.9	4054	19.8
K15	1.3	9.2	9.7	4266	19.7
K16	1.3	22.9	24.2	8714	21.2
K17	1.3	24.5	25.7	12191	20.9
K18	1.8	11.9	13.1	N/A	20.1
K19	1.3	17.4	18.3	6805	20.8
K20	1.3	12.8	13.5	5488	19.9
K21	1.3	14.7	15.5	6014	20.1
K22	1.3	17.8	18.7	4123	20.7
K23	1.5	19.3	20.8	2598	22.3
K24	1.3	19.8	21.0	6016	20.5
K25	1.3	20.0	21.1	5341	20.9
K26	1.3	10.4	11.0	3352	19.4
K27	1.3	19.2	20.2	5858	20.2
K28	1.3	25.6	27.1	7747	21.5
K29	1.5	18.0	19.5	4514	20.6
K30	1.5	16.1	17.4	4320	20.4
K31	1.3	23.8	25.2	7110	20.8
K32	1.3	18.1	19.1	4608	20.5
K33	1.3	22.5	23.7	N/A	20.5
K34	1.3	24.0	25.4	6282	21.4
K35	1.5	13.9	15.0	6238	N/A
K36	1.5	14.5	15.6	5820	N/A
K37	1.5	18.4	19.8	6927	N/A
Average		17.0	18.0	5737	20.5
Stdev		5.1	5.3	2059	0.7
COV		29.9%	29.6%	35.9%	3.5%

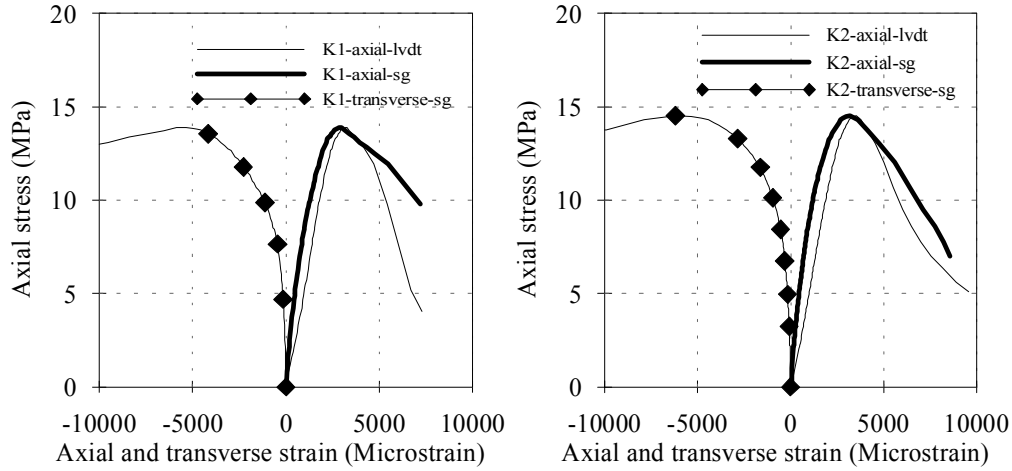


Figure 4.5 : Comparison of strain gauge and LVDT measurements.

Three parameters that can be used to better define the mechanical characteristics of stone units under uniaxial compression are Young's modulus (E), Poisson's ratio (ν) and the volumetric strain (ϵ_{vol}). According to ASTM D 3148-02 (2002) standard, the axial Young's modulus, may be calculated using any of several methods employed in engineering practice; such as tangent, average slope and secant moduli. In this study, average slope approach was applied. In this approach, the average slope can be calculated either by dividing the change in stress by the change in strain or by making a linear least squares fit to the stress-strain data in the straight-line portion of the curve. After a number of trials, this straight-line portion of the curve was observed to stay in a stress range in between 25% and 60% of the compressive strength. A similar range (30% and 60% of f_c) was also designated by Oliveira (2003) for compression tests of sand stone cylinders. Mean value of the Young's modulus values tabulated in Table 4.2 is equal to 5737 MPa with a standard deviation of 2059 MPa and coefficient of variation of 35.9%.

Poisson's ratio (ν) and volumetric strain (ϵ_{vol}) can be defined in terms of transverse strain (ϵ_h) and axial strain (ϵ_c) as follows:

$$\nu = \frac{\epsilon_h}{\epsilon_c} \quad (4.2)$$

$$\epsilon_{vol} = \epsilon_c + 2\epsilon_h \quad (4.3)$$

Variation of volumetric strain and Poisson's ratio with respect to normalized axial stresses are shown in Figure 4.6. It should be noted that, stresses in the vertical axes are normalized with respect to compressive strengths of each specimen.

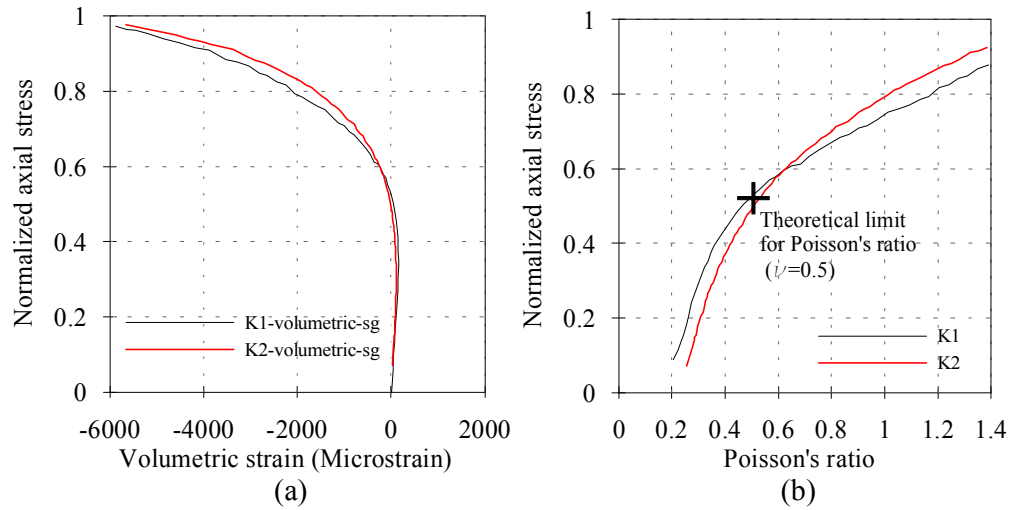


Figure 4.6 : Variation of (a) volumetric strain and (b) Poisson's ratio.

Forming a synthesis of obtained figures (Figures 4.3-4.6) and observations during the tests, the behaviour of Küfeki stone subjected to uniaxial compression can be summarized as:

- At very small stresses: Settling of the loading platen-specimen interfaces.
- Increasing the axial load for small amounts: Closure of the existing micro-cracks and pores. Small decrease in volumetric strain (positive direction in Figure 4.6a) and small Poisson's ratio in the order of 0.20-0.25.
- For further increment in the stresses, the stone specimen exhibits elastic behaviour (almost linear curves in Figure 4.5 for axial and transverse strain changes).
- With the initiation of new micro-cracks in the direction of loading, nonlinearity begins: At about $0.4 f_c$ highly nonlinear axial stress-transverse strain curves in Figure 4.5, volume increases in Figure 4.6a and nonlinearity begins for Poisson's ratio curve in Figure 4.6b.
- At about $0.5 f_c$ the specimen expands to its initial volume (zero volumetric strain) and Poisson's ratio reaches its theoretical limit value of 0.5. Apparently, Poisson's ratio cannot be defined after exceedance of the linear range.

- When the peak stresses are reached the macro-cracks become clearly visible.
- With the increase of damage, axial stresses begin to decrease (post-peak branches in Figures 4.4 and 4.5).

Very similar series of events summarized above are also reported by Oliveira (2003) and Vasconcelos (2005) for sand stone and granite type rocks, respectively.

Another attempt was done for establishing a relationship between the equivalent cube compressive strength and Young's modulus of stone specimens. By fitting a line intersecting the origin, Eq. (4.4) is obtained for the Küfeki stone (Figure 4.7). A close but slightly larger coefficient ($E = 382.7f_c$) was reported by Vasconcelos (2005) for granites.

$$E = 315 f_{c,cube} \quad (4.4)$$

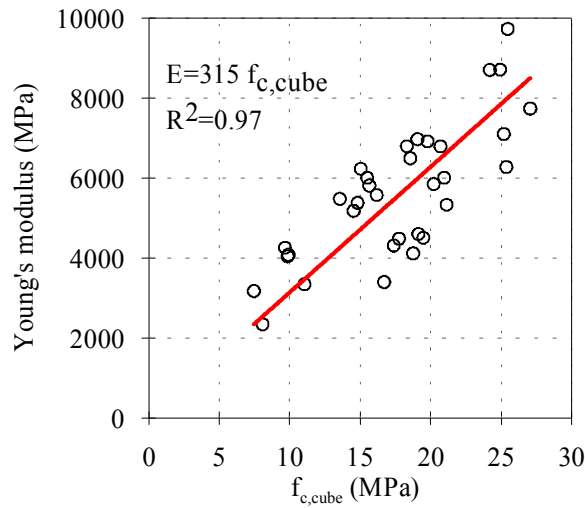


Figure 4.7 : Interaction of Young's modulus with compressive strength.

4.1.3 Three-point bending tests on stone units

In order to have a rough but quantitative idea on the tensile strength of Küfeki stone, ten rectangular prism specimens were tested by utilizing an indirect tensile test method. For this purpose, three-point bending tests were carried out in compliance with the ASTM C 99-87 (2006) standard.

Ten stone units sampled randomly from five transportation palettes were sawn to have approximate dimensions of 130×300×50 mm. Details of specimen dimensions are presented in Table 4.3. These dimensions were not in fully compliance with the

ASTM C 99-87 (2006) regulations which required $101.6 \times 203.2 \times 57.2$ mm in size. However, in order to use an existing testing apparatus available at the Istanbul Technical University Building Materials Laboratory, slightly larger dimensions were employed before testing. Küfeki specimens sawn under water were left for drying (in open air) until their weights were stabilized.

A 100 kN capacity Instron closed-loop testing machine was used for loading under displacement control at a displacement rate of 0.15 mm/min. General and schematic views of the test setup are shown in Figure 4.8. Span length in between the centre lines of the supports (designated as a in Figure 4.8b) was equal to 240 mm. Prior to loading, the surfaces of the specimens that will contact with the support and the loading knife edges were carefully ground to obtain smooth surfaces. Afterwards, the specimen was placed on the supports and a preload of 1.5 kg was applied. Finally, monotonically increasing three-point bending moment was applied until the rupture occurs. Load and displacement data measured by the internal sensors of the Instron device were recorded. By using the peak load data, the modulus of rupture for all specimens were calculated by using Eq. (4.5),

$$R = \frac{3Wl_s}{2bh^2} \quad (4.5)$$

where R is the modulus of rupture, W is the peak load, l_s is the length of span, b is the width and h is the height of the specimen cross-section. Peak loads and modulus of rupture values obtained for ten specimens are tabulated in Table 4.3. Accordingly, mean modulus of rupture value is 3.15 MPa with a standard deviation of 0.42 MPa and a variation coefficient of 13.2%.

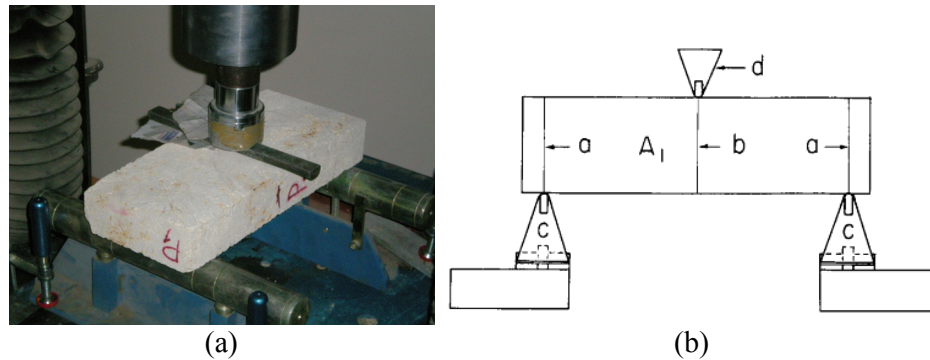


Figure 4.8 : General (a) and schematic (b) views of the testing apparatus according to ASTM C 99-87 (2006).

Table 4.3 : Results of modulus of rupture tests for Küfeki stone.

Specimen	Dimensions			Span	Peak load	Modulus of rupture
	l (mm)	b (mm)	h (mm)	l_s (mm)	W (N)	R (MPa)
P1-1	301	130	47	240	3100	3.89
P1-2	301	131	50	240	2800	3.08
P2-1	301	131	50	240	3260	3.58
P2-2	301	132	47	240	2900	3.58
P3-1	300	126	50	240	2600	2.97
P3-2	300	126	46	240	2200	2.97
P4-1	300	130	45	240	2138	2.92
P4-2	300	130	47	240	2000	2.51
P5-1	300	129	49	240	2436	2.83
P5-2	300	129	48	240	2637	3.19
Average					2604	3.15
Stdev					419	0.42
COV					16.1%	13.2%

4.2 Mechanical Characterization of Mortar

4.2.1 Trials for a mortar composition

Though the outer leaves of the investigated type of wall do not include any mortar between the stone units, the inner leaf is rubble masonry. Thus, inner leaf shall consist of mortar and rubble stone. In this part of the study, a mortar mixture, which is representative of historical mortars (such as Khorasan mortar) in terms of mechanical characteristics, was sought. However, it should be noted that, it is very difficult to pronounce certain mechanical characteristics that for historical mortars. In this study, it was assumed that a mortar that had a compressive strength in the order of 5.0 MPa was acceptable.

For this purpose; a pozzolanic lime based ready-mixed mortar, Albaria Struttura from BASF Company was utilized as the binder. Since the mechanical characteristics of this mortar, such as compressive strength (about 20 MPa) and Young's modulus (16000 MPa as supplied by the manufacturer), were very high in comparison with the original historical mortars, a reduction process was required. This was achieved by adding sand and supplying water that caused a higher water/binder ratio. In order to decide the amount of additional sand and water, four trial mixtures (Mixtures I-IV) were made and three specimens for each mixture were cast into 70×70×70 mm cube moulds. Contents of these trial mixtures are tabulated in Table 4.4.

At 28 days age, specimens were uniaxially loaded under a loading rate of 3.0 kN/s by using a 2000 kN capacity Amsler testing machine. Mean compressive strengths obtained from these tests were 16.5 MPa, 8.7 MPa, 4.5 MPa and 2.2 MPa for Mixture I, II, III and IV, respectively (Table 4.4). Since a mortar with a compressive strength in the order of 5.0 MPa was sought, Mixture III seemed to be feasible for the construction of the wall specimens. However, during the construction of walls, this mixture was slightly modified for better workability and finally Mixture V was obtained.

Table 4.4 : Contents and mean compressive strengths of mortar trial mixtures.

	Mixture I	Mixture II	Mixture III	Mixture IV	Mixture V
Binder (Albaria Strutturata)	1000 g	1000 g	1000 g	1000 g	1000 g
Sand	1000 g	2000 g	3000 g	4000 g	3000 g
Water	400 g	643 g	900 g	1125 g	750 g
Water/Binder	0.40	0.64	0.90	1.125	0.75
Compressive strength	16.5 MPa	8.7 MPa	4.5 MPa	2.2 MPa	4.8 MPa

4.2.2 Compression tests on mortar samples

In order to obtain an average compressive strength for the mortar (at 28 days age), samples were tested under uniaxial compression loads. According to TS EN 1015-11 (2000), the compression tests were performed on the halves of the mortar specimens obtained from the three point bending tests that will be presented in the next section.

A 100 kN capacity Amsler testing machine was used with a special loading assembly that exerts load only to 40×40 mm area of the mortar prism. The measurement system consisted of a 100 kN capacity TML CLP-10B load cell and two TML CDP-25 LVDTs that transmitted the measured values to a data logger which was connected to a computer. The LVDTs measured the total height change of the specimens. The average Young's modulus value obtained compression tests was approximately 250 MPa.

Table 4.5 : Results of compression tests for mortar.

Specimen	Cross-section area	Compressive strength
	mm ²	MPa
MC-28-1A	1600	5.36
MC-28-1B	1600	3.86
MC-28-2A	1600	4.97
MC-28-2B	1600	5.42
MC-28-3A	1600	5.19
MC-28-3B	1600	4.27
Average		4.8
Stdev		0.6
COV		13.6%

4.2.3 Three point bending tests on mortar samples

Mortar test specimens were taken randomly during the construction of masonry walls. Steel moulds having nominal dimensions of 40×40×160 mm were used for the specimens. Before the moulds were filled with mortar; they were cleaned and oiled for preventing the sticking of the samples to the moulds. Mortar mixture was poured into each mould in two layers that were placed on a shaking compactor. Finally, the upper surfaces of the mortar samples in the moulds were smoothed by using a trowel and the samples were left in the same environmental conditions with the walls.

Three point bending test method, which is a widely used indirect test method, was utilized according to TS EN 1015-11 (2000) and ASTM C 3148-02 (2002). Each specimen was placed on top of two steel supports whose distance between the centres was equal to 100 mm. The load applied was transmitted to the midspan of each specimen by means of a steel rod. A 100 kN capacity Instron closed-loop testing machine was used for loading under displacement control at a displacement rate of 0.25 mm/min. Monotonically increasing three-point bending moment was applied until the rupture occurs. By using the peak load data, the modulus of rupture for all specimens were calculated by using Eq. (4.5).

Peak loads and modulus of rupture values obtained for specimens tested at 28 days are tabulated in Table 4.6. According to this table, mean modulus of rupture is obtained as 1.8 MPa.

Table 4.6 : Results of three point bending tests for mortar.

Specimen	Dimensions			Span	Peak load	Modulus of rupture
	l (mm)	b (mm)	h (mm)	l_s (mm)	W (N)	R (MPa)
P1-28	160	40	40	100	814	1.9
P2-28	160	40	40	100	848	2.0
P3-28	160	40	40	100	695	1.6
Average					786	1.8

4.3 Mechanical Characterization of Rubble Masonry

As described earlier, in the walls of heritage structures, the gap between the two external ashlar masonry leaves were filled with rubble masonry, which was an affordable and easy to build solution than constructing the whole mass of the wall with cut stone units.

A similar construction technique was also followed during the preparation of the wall specimens and simultaneously standard cylinder samples were randomly taken from the rubble infill. For this purpose, cylinder moulds with 150 mm diameter and 300 mm height were oiled for easier removal of the samples. Firstly, a mortar layer with an approximate height of 50 to 100 mm was poured into the cylinder mould. Then, stone pieces with radii between 20 and 50 mm were dropped into the filled mortar from approximately 30 cm height. No additional compaction was applied to the stone pieces so that air voids observed in walls of existing structures could also be formed. The dropped stone pieces were covered by mortar and the same steps were followed until the mould was completely filled. The cylinders were kept under the same environmental conditions with the walls after their removal from the moulds (Figure 4.9).

Two material tests were carried out at different ages: Standard cylinder compression test and splitting test. The first test type aimed to investigate the compressive strength, axial stress-strain behaviour and Young's modulus of the rubble masonry material. Splitting test, which is an indirect tension test, was performed to have an idea on the tensile strength of the material and the failure mode of the rubble material under tensile actions.



Figure 4.9 : Rubble masonry standard cylinder samples.

4.3.1 Compression tests on rubble masonry cylinders

Randomly chosen rubble masonry samples were subjected to monotonically increasing compressive stresses by using a 5000 kN capacity Instron Satec 1000RD closed loop testing machine. Specimens were tested at different ages such as 28, 90 and 180 days. Prior to testing, top and bottom surfaces of each cylinder were capped with a mixture of molten sulphur and graphite, so that smooth and parallel loading surfaces could be obtained. Tests were performed under displacement control with a rate of 0.60 mm/min. A TML CLP-100 load cell and two TML CDP-25 LVDTs were used to monitor the load and axial displacement values. The data obtained from the load and displacement sensors were stored to a computer by the help of a TML TDS-303 data logger. In addition to the external measurement devices, internal load cell and displacement transducer of the Instron device were also available. The test setup of the rubble masonry standard cylinder compression tests can be seen in Figure 4.10.

The load values obtained from the uniaxial loading were divided by the cross-section areas to calculate the compressive stresses and the axial displacements in the loading direction were divided by the initial height of the specimens so that the stress-strain curves of each specimen could be plotted. Compressive stress-strain curves obtained at 28, 90 and 180 days are shown in Figure 4.11.

As expected, due to the heterogeneous structure of the composite material, curves were far from being uniform. Curves represented the main characteristics of similar quasi-brittle materials with an elastic region followed by the peak zone and finally the descending branch. These zones were closely related with the damage growth. In most cases, no damage could be observed in the pre-peak elastic region, however, sounds caused by the cracking of the mortar matrix and settlement of the voids could be heard near the peak load. Formation of the vertical macro cracks and crushing of

the material became clearly visible at the post-peak descending branch. The damage conditions of rubble masonry cylinders are shown in Figure 4.12.

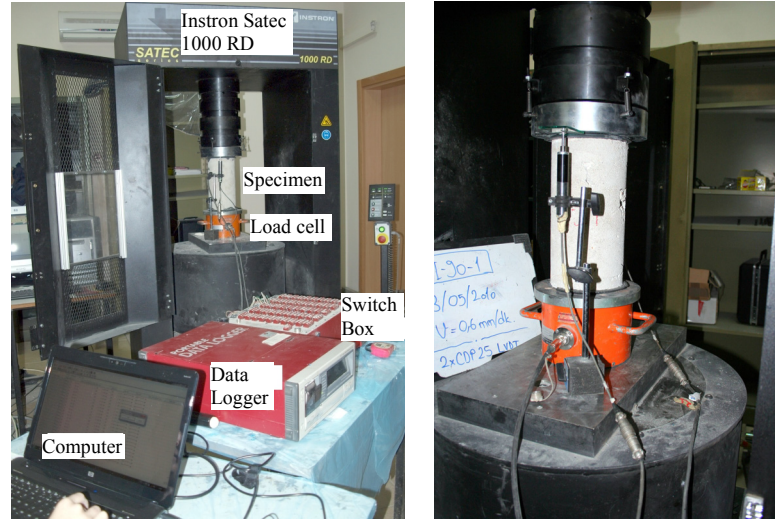


Figure 4.10 : Test setup for rubble masonry cylinder compression tests.

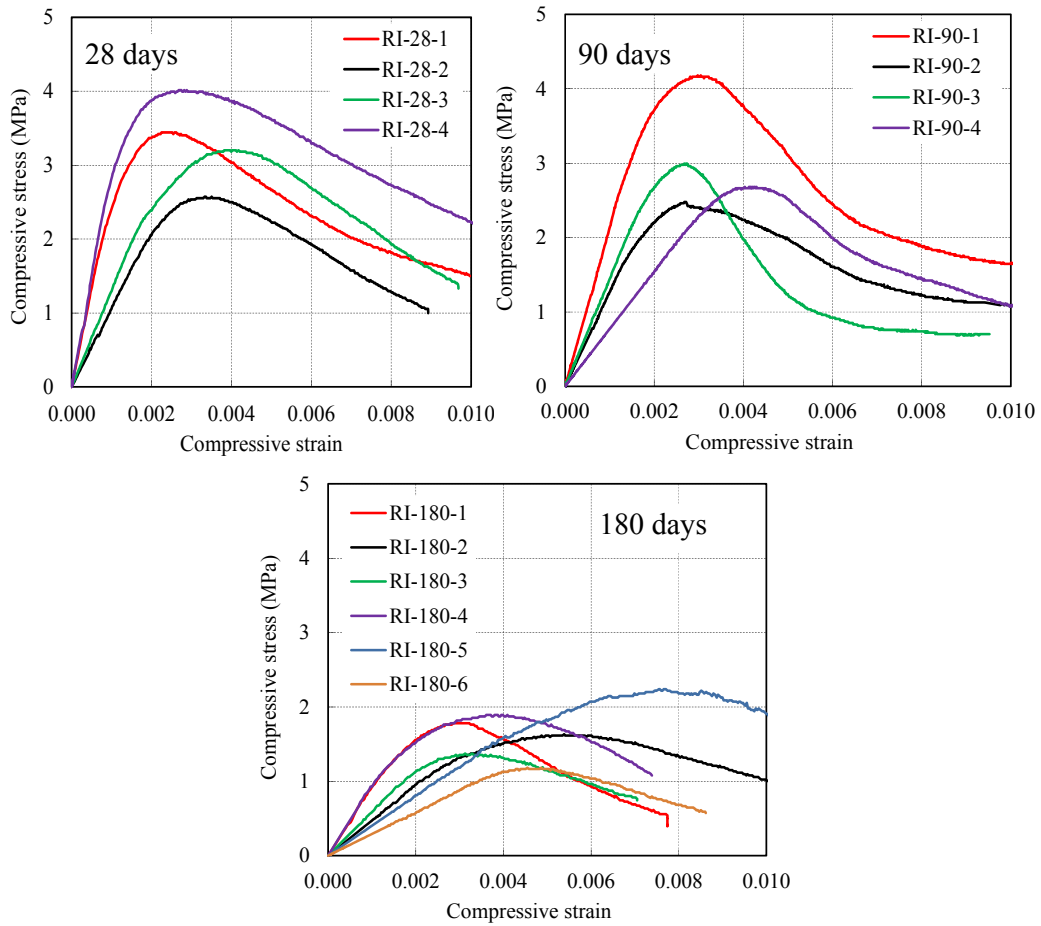


Figure 4.11 : Axial stress-strain relationships of rubble masonry compression tests.



Figure 4.12 : The damage conditions of rubble masonry cylinders.

Mechanical characteristics of the tested rubble masonry cylinders such as; compressive strength, strain corresponding to strength, Young's modulus and strain corresponding to 20% strength loss are summarized in Table 4.7 with respect to age of the specimens. In this table, f_{co} is the standard cylinder compressive strength, ϵ_{co} is the compressive strain corresponding to strength, E_{rm} is the Young's modulus of the rubble masonry specimens (calculated in the range of 20-50% of peak stress) and $\epsilon_{0.8f}$ is the strain corresponding to 20% strength loss.

Table 4.7 : Results of the rubble masonry monotonic compression tests.

Specimen	Age (days)	f_{co} (MPa)	ϵ_{co} (microstrain)	E_{rm} (MPa)	$\epsilon_{0.8f}$ (microstrain)
RI-28-1	28	3.44	2300	2682	4800
RI-28-2	28	2.57	3300	1065	5500
RI-28-3	28	3.20	3800	1311	6300
RI-28-4	28	4.01	2700	2682	6400
RI-90-1	90	4.18	3000	2120	4700
RI-90-2	90	2.48	2700	1262	5000
RI-90-3	90	3.00	2700	1446	3600
RI-90-4	90	2.68	4000	772	5700
RI-180-1	180	1.78	2900	927	4490
RI-180-2	180	1.63	5400	471	8200
RI-180-3	180	1.37	3200	589	5200
RI-180-4	180	1.89	3600	964	6100
RI-180-5	180	2.24	7550	402	10000
RI-180-6	180	1.18	4550	291	6590
Average(COV)	28	3.31 (0.18)	3025 (0.22)	1935 (0.45)	5750 (0.13)
Average(COV)	90	3.09 (0.25)	3100 (0.20)	1400 (0.40)	4750 (0.18)
Average(COV)	180	1.68 (0.23)	4533 (0.38)	607 (0.46)	6763 (0.30)

Test result given in Table 4.7 and the stress-strain curves shown in Figure 4.11 clearly indicates that the heterogeneous structure of the composite material, that was formed by a mortar matrix and randomly distributed stone pieces placed without compaction, lead to an understandable level of variance in the mechanical characteristics. It is also interesting to note that the strength and Young's modulus values tend to decrease significantly with increasing age. This drop can be attributed to the formation of micro-cracks initiated due to shrinkage effects.

4.3.2 Splitting tests on rubble masonry cylinders

Randomly chosen rubble masonry standard cylinders were subjected to splitting test, which is generally used as an indirect test method for determining the tensile strength of concrete. In practice, the concrete standard cylinder specimens are generally partitioned into three pieces with equal height. However, due to very low strength of the rubble masonry cylinders, they were not partitioned and each cylinder was tested as a whole.

The tests were carried out at 28, 90 and 180 days age by using a 1000 kN capacity Amsler testing machine. In order to ensure that the load was applied along a plane passing through the centre of the cross-section, right triangles were drawn on the top surfaces of the cylinders. Each specimen was laid on top of a timber rod when placed on the lower platen of the testing machine and another timber rod was also placed along the upper platen side where the load was applied. The splitting tensile strength was calculated using the following equation:

$$f_{st} = \frac{2P}{\pi l D} \quad (4.6)$$

where f_{st} is the splitting tensile strength, P is the peak load, l is the length and D is the diameter of the cylinder. A schematic view of the test configuration is presented in Figure 4.13.

After the tests, when the split specimens were investigated, it has been observed that the stone pieces in the mortar matrix were scattered randomly. It was interesting to observe that the splitting plane of each specimen was not only passing through the mortar but also through the stone pieces. View of a typical specimen before and after the loading can be seen in Figure 4.14.

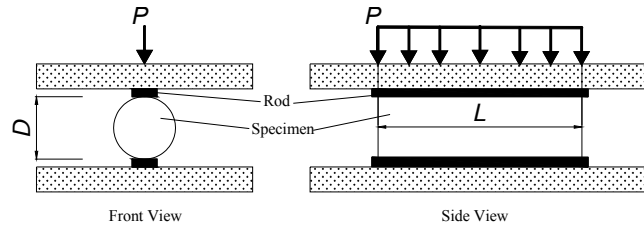


Figure 4.13 : The test configuration of the splitting tests.



Figure 4.14 : The view of a typical specimen before and after the splitting test.

The maximum resisted loads and the splitting tensile strengths of the specimens are given in Table 4.8. It is worthy to note that a coefficient of variation in the order of 0.30 is obtained for all ages. The tensile strength of the rubble masonry tends to decrease as the age of the material increases. Considering the inhomogeneous structure of the material and the variance in the test results, it is apparent that the rubble masonry material should be treated with high safety tolerances.

Table 4.8 : Results of the rubble masonry splitting tests.

Specimen	Age (days)	P (kN)	f_{st} (MPa)
STS-28-1	28	24.0	0.35
STS-28-2	28	19.1	0.27
STS-28-3	28	31.9	0.44
STS-28-4	28	14.7	0.20
STS-90-1	90	31.7	0.44
STS-90-2	90	32.4	0.46
STS-90-3	90	18.6	0.27
STS-90-4	90	20.8	0.30
STS-90-5	90	19.6	0.27
STS-180-1	180	14.2	0.20
STS-180-2	180	18.4	0.26
STS-180-3	180	22.1	0.32
STS-180-4	180	11.0	0.16
STS-180-5	180	12.0	0.17
STS-180-6	180	17.4	0.25
Average (COV)	28	22.4 (0.33)	0.32 (0.32)
Average (COV)	90	24.6 (0.28)	0.35 (0.27)
Average (COV)	180	15.9 (0.27)	0.22 (0.27)

4.4 Mechanical Characterization of Metal Members

It is a well-known fact that, since the antiquity, either metal or timber reinforcing elements were being used in the heritage masonry structures. The evolution of masonry reinforcement has required a long history that was shaped by the improvements in the metal production and processing abilities. Beginning from timber and easy to shape metals such as copper and bronze, various techniques have been developed for the utilization of reinforcing elements in the masonry constructions. With the industrialization and developments in production abilities, tougher materials (iron, steel, titanium and other alloys) have been put into service.

Although metals had been widely used in the Ottoman architecture, very little information is available in the literature. Therefore, in this topic, firstly the usage of metal members in the heritage structures is described briefly. Then the tests carried out on two cramps discovered at the Edirne II. Bayezid Mosque site during the restoration of the structure are presented and the test results are evaluated. Finally, an appropriate material that can approximately represent the ancient metals and can be used for the wall specimen construction is sought and selected.

4.4.1 Investigations of Tanyeli et al. (1990)

Though a number of sources for the functions and physical properties of metal members in Ottoman architecture are available in the literature, studies on the mechanical and metallurgical properties are almost non-existent. To the best knowledge of the author, the only study approaching the issue from metallurgical engineering perspective has been performed by Tanyeli et al. (1990). In this study, revak tie samples from Topkapı Palace Second Hall and Davutpaşa Madrasa; cramps from Edirne Selimiye Mosque and Palace Kitchens; pin from Sultan Ahmet Mosque; retrofitting brace from Edirne Tütünsüz Baba Tomb; and fence ball from the Topkapı Palace Zülüflü Baltacılar Ward were inspected under optical and scanning electron microscopes. In addition, the Vickers micro-hardness values of some of the samples were measured under 1.0 kgf tip load. A summary of the findings obtained in this study are presented in a tabular form in Table 4.9.

Table 4.9 : Investigations on metal samples reported in Tanyeli et al. (1990).

Structure	Function	Specimen properties	Macro-investigation	Micro-structure	Vickers hardness
Palace Kitchens, Edirne (15 th cent. 3 rd quarter)	Cramp	19-22 cm long, 1.8-2.1 cm wide, both tips 3.5 cm bent	Made by folding a thinner and longer wrought iron member	Mainly ferrites, partially pearlite rich regions. Abundant phosphorus and potassium rich inclusions in the structure	Varies between 92-179 due to heterogeneous structure
Davutpaşa Madrasa, Istanbul (1485-?)	Revak tie	4×6.5 cm cross-section, 4.5 m length	Produced by adjoining 2×6.5 cm cross-sectioned, 1.0-1.5 m long members forged under heat	Mainly ferrites with some inclusions	Between 107 and 138
Topkapı Palace Second Hall, Istanbul (Late 15 th -early 16 th cent.)	Revak tie	4.1×6.8 cm cross-section		Mainly ferrites with carbon rich areas. Widmanstatten structure in carbon rich regions (evidence of air cooling from 1000 degree Celsius)	Higher values between 165 and 249, due to existence of carbon rich regions
Selimiye Mosque, Edirne (1569-1575)	Cramp	54 cm long, 3.5-4.0 cm wide, 1.0-1.2 cm thick, tips bent for 6 cm	Wrought iron	Mainly ferrites, partially pearlite rich regions	N/A
Sultan Ahmet Mosque, Istanbul (1609-1617)	Pin	6-12 cm long	Made by folding a thinner and longer iron member by hammering under heat	Mainly ferrites	N/A
Tütünsüz Baba Tomb, Edirne (1488, 18 th cent. repair)	Retrofit brace	8-10 cm wide, 0.6-1.0 cm thick laminates	Wrought iron	Mainly ferrites	N/A
Topkapı Palace Zülüflü Baltacılar Ward	Fence ball	4.5 cm with 2.2 cm hole diameter	Cast iron	N/A	N/A

4.4.2 Tests on original cramp reinforcements

Two original cramps, obtained from the construction site established for the restoration of Edirne II. Bayezid Mosque, were used for the metallurgical tests (Figure 4.15). These original cramps were moderately corroded and they had average dimensions of 350-370 mm length, 30-40 mm width, 9-10 mm thickness and both tips were bent for 42-55 mm. The corrosion-induced linings covering these two metal members were removed through a special chemical process. During this process cramps were kept in an ultrasonic pool filled with a chemical cleaning solution. This chemical cleaning solution was prepared in accordance with the ASTM G 1-03 (2003) Standard Practice for Preparing, Cleaning, and Evaluating Corrosion Test Specimens, with the following content: 1000 ml hydrochloric acid (HCl, sp gr 1.19), 20 g antimony trioxide (Sb_2O_3), 50 g stannous chloride (SnCl_2). The cleaning process removed the corrosion layer covering the surface and furthermore, revealed some major cracks that were propagating into the inner parts of the cross-sections. Since these cracks would negatively affect the mechanical test results, no tensile tests could be done. However, metallographic and chemical analyses were performed which might give an idea about the mechanical characteristics of original steel members. These tests included chemical tests, microscopic investigations and microhardness tests. The sample preparation, metallographic investigations and microhardness tests were performed at Istanbul Technical University, Metallurgical and Materials Engineering Department Laboratories. The chemical tests were realized by a private firm (Silvan Sanayi Ltd.).

Prior to chemical and metallographic tests, a number of samples were extracted from the cramps that were cleaned in the ultrasonic pool. The cramps were cut and partitioned in both longitudinal and transverse directions. A water-cooled rotating saw was used for cutting. Two samples obtained after cutting were submitted to Silvan Sanayi Ltd. for chemical tests. However, more efforts were required for the metallographic investigations.

The extracted samples from longitudinal and transverse directions of the clamps were laid in moulds filled with acrylic resin (Figure 4.16). After the acrylic resin was set, the mounted samples were removed from the moulds.



Figure 4.15 : Original cramps from Edirne II. Bayezid Mosque.

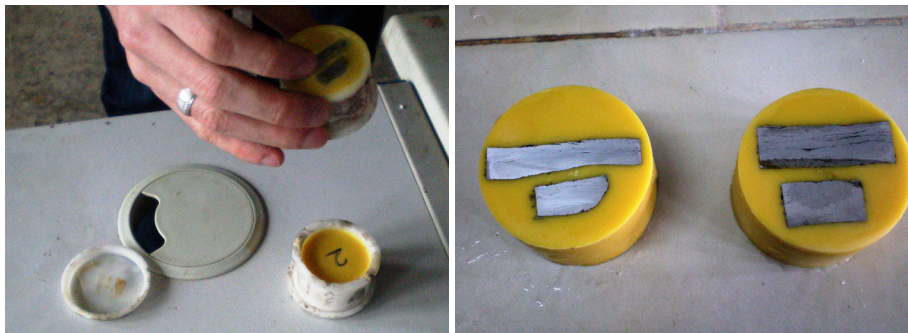


Figure 4.16 : Mounting of the metal samples to the acrylic filled moulds.

After the hardening of the acrylic resin, the rough surfaces with numerous scratches left from the cutting with rotating saw, were rubbed with sand papers. Silicon carbide abrasive papers mounted on a rotating disc were used for this purpose. Ten different types of abrasive papers with varying grades between 100 and 2000 were used at successively finer steps with water as a lubricant. Each sample was ground for more than two hours beginning with the coarser abrasive paper and continuing with a finer one at each step until a smooth surface could be obtained (Figure 4.17). A Presi Mecapol P220S machine was used for the grinding purpose.

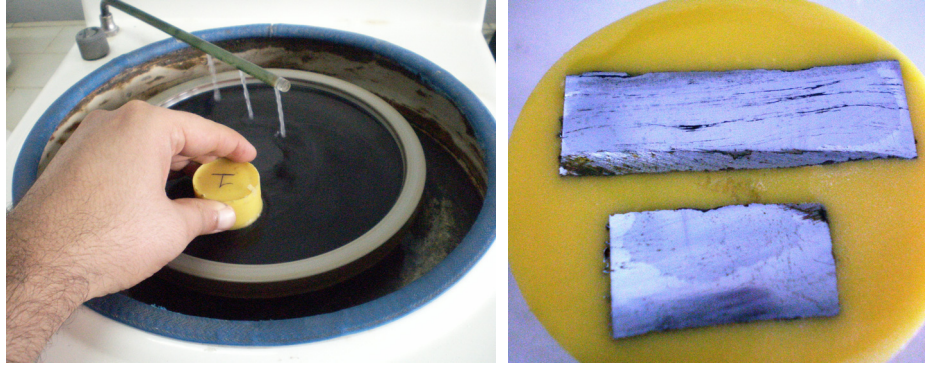


Figure 4.17 : Grinding of the metal samples until a smooth surface was obtained.

The next step in sample preparation was the polishing of the surfaces. This step aimed the removal of the deformations and scratches left from the fine grinding. During the polishing stage, a resilient polishing cloth was fixed to the rotating disc of the Presi Mecapol P230 machine on which a colloidal silica suspension with very small silica grain sizes (OP-U Suspension) and distilled water were poured. The polishing stage was finished when a shiny and scratch-free surface was obtained (Figure 4.18).



Figure 4.18 : Polishing of the metal sample surfaces.

In metallography, the microstructural constituents of the specimen are revealed by using a suitable chemical or electrolytic etchant. An etchant named Nital, consisting of 3% Nitric Acid and 97% Ethanol by volume, was used for the etching process. The surface condition of the sample after etching is presented in Figure 4.19.

Microstructures of the prepared metal samples, as well as the distribution of inclusions, were examined by using an optical microscope available at the Istanbul Technical University, Metallurgical and Materials Engineering Department

Laboratories (Figure 4.20). The observations were done both on unetched and etched specimens and documented by using the camera integrated to the microscope.

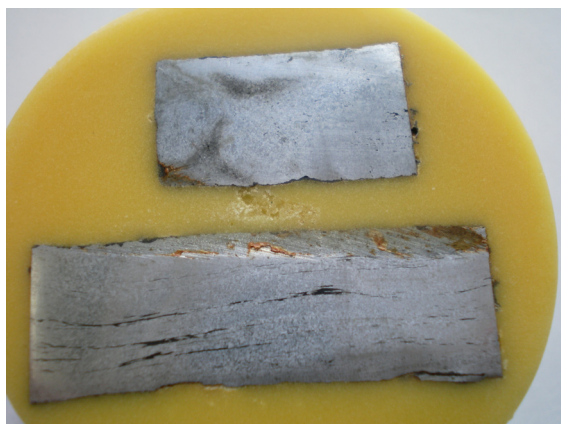


Figure 4.19 : Polished and etched metal sample surfaces.



Figure 4.20 : Microscope used for the examinations.

Images presented in Figures 4.21 and 4.22 for 10X and 20X zoom levels belong to Samples 1 and 2, respectively. Images obtained from optical microscopic examinations (especially obtained from Nital-etched samples) revealed the large polygonal ferrite grains (α -iron matrix) and dispersed tiny globular oxides (mostly FeO and SiO₂). At some localized regions, large inclusions in irregular shapes were also observed. The dispersion of tiny globular oxides was random. However, large and irregular inclusions, especially the slag fibres, were lying vertical to the hammering direction of the cramps. Though most of these inclusions were intergranular slag localizations, inclusions inside the ferrite grains were also visible.

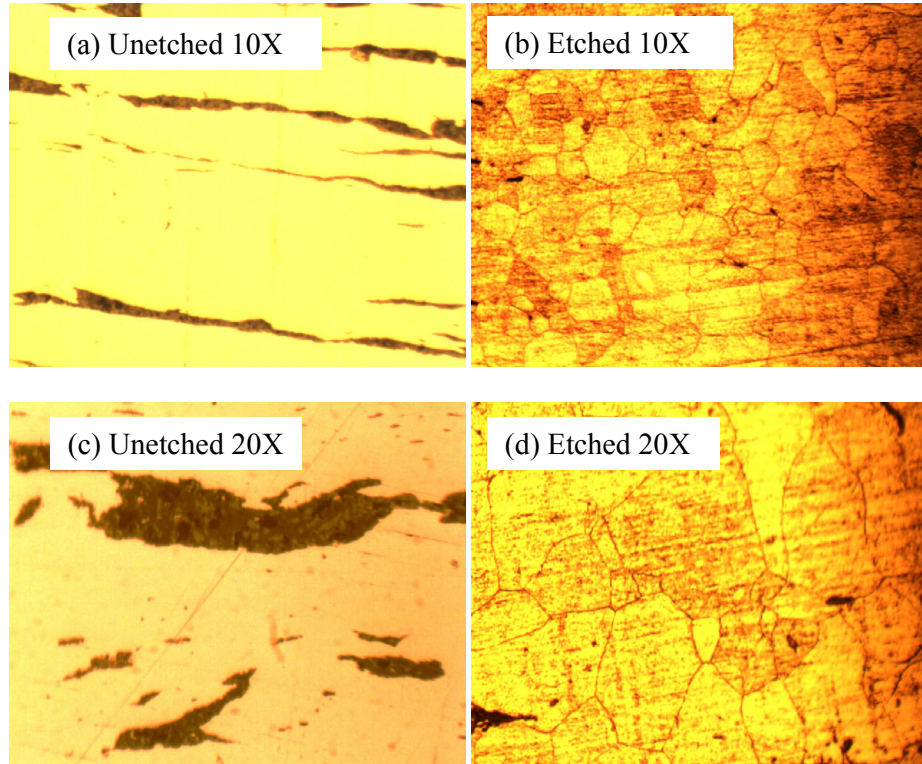


Figure 4.21 : Micrographs of Sample 1 for (a) unetched 10X zoom, (b) etched 10X zoom, (c) unetched 20X zoom and (d) etched 20X zoom cases.

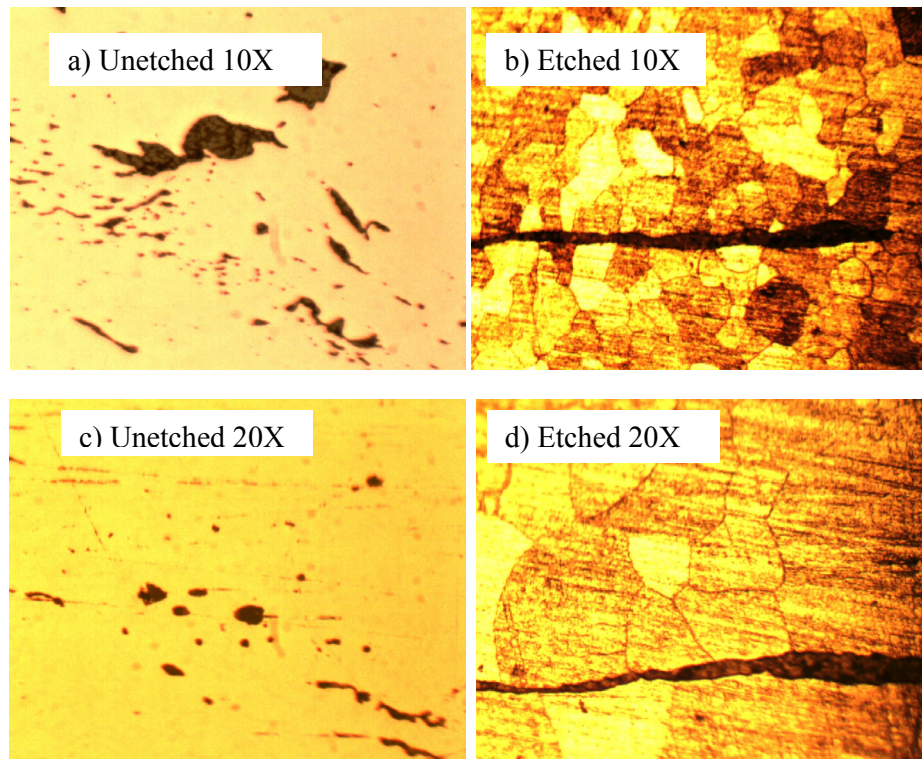


Figure 4.22 : Micrographs of Sample 2 for (a) unetched 10X zoom, (b) etched 10X zoom, (c) unetched 20X zoom and (d) etched 20X zoom cases.

As also noted by Gordon and Knopf (2005), wrought iron is a composite material consisting of a metal matrix containing fibers of non-metallic slag inclusions. The traces of these slag inclusions may be followed back to the production techniques and technology level utilized during the date of production of these ancient wrought iron members. Since it was not possible to attain temperatures close to the melting point of pure iron (1539 °C) until the 19th century, wrought iron must have been produced by direct reduction of iron ore with charcoal at about 1200 °C. The product obtained by this method consisted of iron and incorporated slag. The iron was then separated by hammering in hot conditions (Ray et al., 1997).

Tanyeli et al. (1990) also observed similar microstructures for several metal samples dating from the Ottoman Empire period (Table 4.9) and by Ray et al. (1997) for samples from the pillar at Dhar (11th century) in India. It should be noted that, ductility and strength of iron is heavily dependent on the metalloid content and to a greater degree on the amount and distribution of the slag. The localization of coarse slag pockets are giving signals of inadequate forging deformation (Ray et al., 1997). Such distribution of coarse slag particles, together with the existence of corrosion induced cracks observed during the macro-investigations of the cramps, would certainly negatively affect the ductility and strength characteristics of the metal members. In our case, the laminated structure of the wrought iron cramp cross-sections, formed during the forging process, is visible to human eye even at macroscopic level (Figure 4.19).

As stated earlier, samples extracted from the two original cramps were submitted to Silvan Sanayi Ltd. for chemical analysis. Results obtained from the chemical tests are presented in Table 4.10. According to the results of the chemical analysis, it is interesting to note that the very low Carbon content of one sample was 0.0029% while it was 0.0333% for the other one. Through private communication with Prof.Dr. Hüseyin Çimenoglu (Çimenoglu, 2009), it has been understood that these extremities are normal for materials produced with the production techniques of the time they had been produced. These differences also address the impurities of the contents used in the production phase, as also observed in the microscopic examinations.

Ray et al. (1997) defines typical indications of wrought iron chemistry as; very low concentration of Carbon, virtual absence of Manganese and relatively higher

Phosphorous level. The chemical tests carried out by Ray et al. (1997) for the pillar in Dhar (11th century, India); addresses to a chemical composition of 0.013% Carbon, 0.06% Silicon, 0.072% Phosphorous, 0.003% Sulphur and traces of Manganese. A similar chemical composition can also be easily noticed from Table 4.10 for the wrought iron samples extracted from the original cramps. The very low content of Sulphur indicates the usage of charcoal used for the smelting (Tanyeli et al., 1990). Tanyeli et al. (1990) also did similar observations for the chemical analysis of original cramps from Edirne Palace Kitchens.

Table 4.10 : Chemical test results of cramps from Edirne II. Bayezid Mosque (in %).

	C	Si	S	P	Mn	Ni	Mo	Cu
Sample 1	0.0029	0.0077	0.0040	0.0542	0.0000	0.0007	0.0006	0.0145
Sample 2	0.0333	0.1077	0.0033	0.0590	0.0033	0.0000	0.0009	0.0116
	Ti	As	Sn	Co	Al	Pb	Nb	Fe
Sample 1	0.0027	0.0002	0.0005	0.0227	0.0059	0.0001	0.0002	99.883
Sample 2	0.0218	0.0010	0.0009	0.0079	0.0296	0.0006	0.0009	99.696

During the macroscopic investigations, it was observed that deeply penetrating cracks were formed due to heavy corrosion all over the cramp cross-sections. Thus, performing mechanical tests, such as uniaxial tension test, was likely to raise several doubts concerning the reliability of the tests. Consequently, in order to make an educated guess on the tensile strength of the material, an empirical approach had to be followed.

For this purpose, Vickers micro-hardness test, which consists of indentation of the test material with a diamond indenter, was utilized. In this test, diamond indenter is in the form of a right pyramid with a square base and an angle of 136 degrees between opposite faces. The full load is normally applied for 10 to 15 seconds. The two diagonals of the indentation left in the surface of the material after removal of the load are measured using a microscope and their average is calculated. The area of the sloping surface of the indentation is calculated. The Vickers hardness is the quotient obtained by dividing the kgf load by the mm² area of indentation (Url-1, 2009). For further details for the implementation of this test method, ASTM E 384 (2009) standard can be referred.

The tip load of the Shimadzu HMV micro-hardness testing device, used during the tests of this study, was as low as 0.05 kgf (Figure 4.23). Indentations left by the

diamond tip on the polished metal sample surface can be seen in Figure 4.24. In these images, the hairs named as I and II, which are used to measure the area of the indentation, are also visible. During the application of the Vickers hardness test, it is important to select a representative site before applying the micro-hardness test (Jiménez et al., 2004). Accordingly, the test sites were chosen carefully, avoiding the defects, impurities or slag inclusions on the sample surfaces.



Figure 4.23 : Shimadzu HMV micro-hardness testing device used in this study.

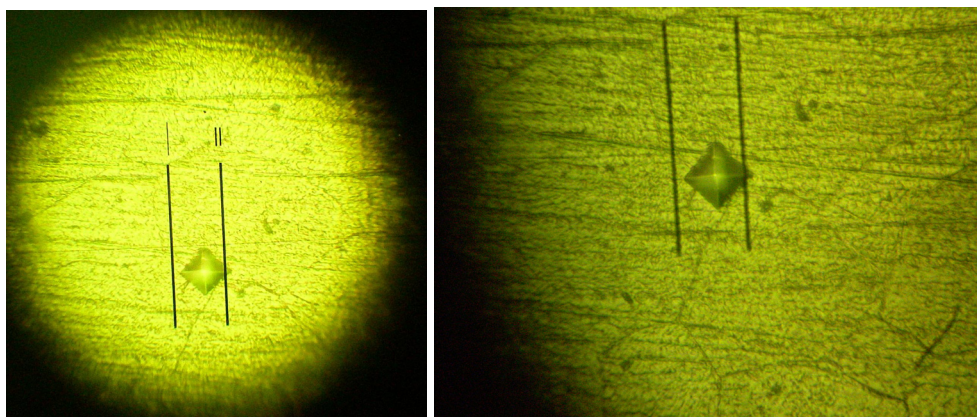


Figure 4.24 : Indentations left on the surfaces during the micro-hardness tests.

The average Vickers micro-hardness values obtained in this study were between 112 and 129. However, dispersed micro-hardness values between 92 and 249 were reported by Tanyeli et al. (1990), mainly due to heterogeneous microstructures of the materials investigated. The exact numbers of micro-hardness tests obtained in this study and previously by Tanyeli et al. (1990) are presented in Table 4.11.

For modern steels, the tensile strength of the metal investigated can be estimated by using the empirical relationships developed for carbon steels. For this purpose, the Vickers hardness (HV) value is first transformed to Brinell hardness (HB) units. However, it should be noted that, hardness conversion between different methods and scales cannot be made mathematically exact for a wide range of materials. Different loads, different shape of indenters, homogeneity of specimen, cold working properties and elastic properties makes the problem very complicated (Url-2, 2009). Such conversions are only limited in scope of the materials they are developed for and should be used with caution, except for special cases where a reliable basis for the conversion has been obtained by comparison tests (ASTM E 384-09, 2009).

The situation becomes even more complicated when it comes to strength estimation of ancient wrought iron materials. As also observed during the microscopic investigations and reported by studies such as Tanyeli et al. (1990), Ray et al. (1997), Elban et al. (1998) and Gordon and Knopf (2005), these materials typically inhibit a very inhomogeneous structure with slag inclusions and cracks. Consequently, any strength estimation for ancient iron materials, based on empirical relationships incorporating hardness values to tensile strengths, needs to be approached carefully.

In this study, a hardness unit conversion chart supplied by Url-3 (2009) was used to convert Vickers hardness values to Brinell hardness values. Then the Brinell hardness values were converted to tensile strength values. The empirical relation between Brinell hardness (HB) and tensile strength of carbon steels (f_t in MPa unit) is reported by Jiménez et al. (2004) as:

$$f_t = 3.45HB \quad (4.7)$$

Similar relationships with approximate Brinell hardness to tensile strength conversion coefficients are also available in the literature. Such as a coefficient of 3.50 reported by Onaran (1993) or coefficients of 3.55 (for Brinell hardness < 175) and 3.38 (for Brinell hardness > 175) reported by Url-3 (2009).

Vickers hardness values obtained by the micro-hardness tests, converted Brinell hardness values and approximate tensile strengths corresponding to these hardness levels, obtained for the iron cramps, are tabulated in Table 4.11. Please note that,

data available for three metal samples from Tanyeli et al. (1990) are also given in the same table.

A similar approach was also followed by Elban et al. (1998) and a wrought iron I-beam element from the Wheeling Custom House (constructed in 1850s) in West Virginia was subjected to hardness measurements and tensile tests. In order to estimate the tensile strength of the I-beam, several Rockwell hardness measurements performed at different locations of the cross-section were used together with a chart, based on the test results of Green (1948) and Flinn and Trojan (1990). Ultimate tensile strengths obtained from destructive tests (331, 331 and 372 MPa from three samples) were found to be comparable with the values 338, 352, 352, 393, 434 and 483 MPa; estimated by using the chart given in Figure 4.25. Yield stresses obtained from destructive tests were 234, 269 and 290 MPa and Young's modulus was in the order of 165000 MPa.

Although the samples belong to different geographies and periods, in our study, the Elban et al. (1998) procedure was also followed for the estimation of the ultimate tensile strengths of the Ottoman period iron members (Table 4.11). When compared with the estimations done by Eq. (4.7), this approach gives both lower and higher strength values depending on the hardness. This situation is expected, since Eq. (4.7) is a line with constant slope while the data given in Figure 4.25 fits to a polynomial.

Ultimate tensile strengths (between 260 and 450 MPa) and yield stresses (between 220 and 345) varying in a similar range was also reported by Gordon and Knopf (2005) for 19th century wrought iron materials.

Buonopane and Kelton (2008) performed hardness measurements and tensile tests on wrought iron members of Penns Creek Bridge (1886). They obtained an average yield stress of 169 MPa (125 MPa minimum and 241 MPa maximum values), average ultimate strength of 270 MPa (180 MPa minimum and 337 MPa maximum values), average ultimate strain of 20% (between 7% and 29%) and average Brinell hardness of 127 (between 111 and 147). For the ultimate strains of other mentioned studies, Gordon and Knopf (2005) reports values between 11% and 30%, while Bright (2007) gives a mean elongation value of 15%.

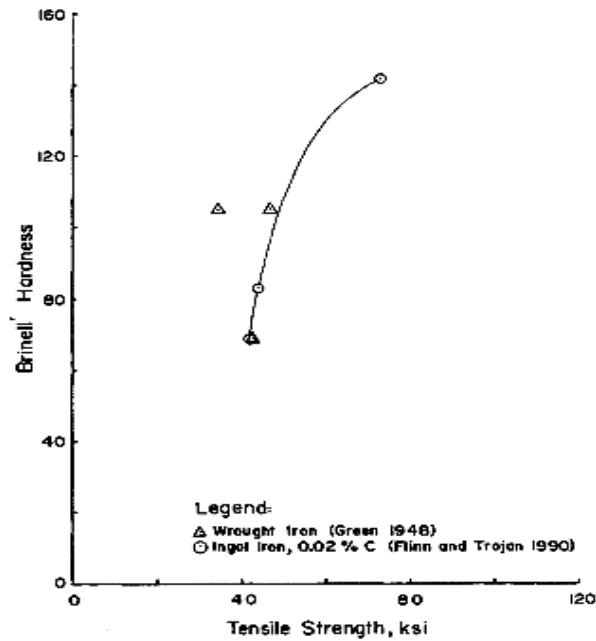


Figure 4.25 : BH vs. tensile strength (Elban et al., 1998), 1 ksi=6.895 MPa.

Though they shed some light on to the subject, the estimation approaches explained above create big question marks when their reliability is questioned. Gordon and Knopf (2005) states that hardness is often well correlated with strength in homogenous metals, such as steel, but does not correlate well with strength or ductility of wrought iron due to the inhomogeneous nature of the fibers and slag. Buonopane and Kelton (2008) also indicate that the plots of tensile strength versus hardness show significant scatter and confirm the lack of a good correlation between strength and hardness in wrought iron. Furthermore, these authors even question the validity of hardness scale conversions between different scales such as Rockwell and Brinell hardness scales. Consequently, the tensile strengths to be obtained with the converted hardness values and empirical relationships should only be used as indicative upper boundary values rather than accepting them as exact mechanical properties.

Table 4.11 : Metallographic properties and predicted tensile strengths.

Structure	Function	Period	Micro-structure	Vickers hardness (HV)	Brinell hardness (HB (500))	Tensile strength Eq. (4.7) (MPa)	Tensile strength Elban et al. (1998) (MPa)
II. Bayezid Mosque, Edirne	Cramp	15 th century	Mainly ferrites with several slag inclusions, laminated structure	Transverse cross section: 110, 114, 118 (average: 114) Longitudinal cross section: 129	Transverse cross section: 101 Longitudinal cross section: 114	Transverse cross section: 348.5 Longitudinal cross section: 393.3	Transverse cross section: 327 Longitudinal cross section: 358
II. Bayezid Mosque, Edirne	Cramp	15 th century	Mainly ferrites with several slag inclusions, laminated structure	Transverse cross section: 124, 121, 115 (average: 122) Longitudinal cross section: 112	Transverse cross section: 108 Longitudinal cross section: 99	Transverse cross section: 372.6 Longitudinal cross section: 341.6	Transverse cross section: 343 Longitudinal cross section: 325
Palace Kitchens, Edirne Tanyeli et al. (1990)	Cramp	15 th century 3 rd quarter	Mainly ferrites, partially pearlite rich regions. Abundant phosphorus and potassium rich inclusions in the structure	Varies between 92-179 due to heterogeneous structure	Between 82 and 152	Between 282.9 and 524.4	Between 299 and 600
Davutpaşa Madrasa, Istanbul Tanyeli et al. (1990)	Revak tie	1485-?	Mainly ferrites with some inclusions	Between 107 and 138	Between 95 and 121	Between 327.8 and 417.5	Between 317 and 377
Topkapı Palace Second Hall, Istanbul Tanyeli et al. (1990)	Revak tie	Late 15 th century-early 16 th century	Mainly ferrites with carbon rich areas. Widmanstätten structure in carbon rich regions	High values between 165 and 249, due to existence of carbon rich regions	Between 142 and 209	Between 489.9 and 524.4	494 for 142 HB

4.4.3 Selection of an appropriate metal for the experimental study

The results obtained from the metallurgical tests performed on original historical reinforcements and the information compiled from the available literature, addresses a metal type with very low carbon content and a material of ferritic character. The macroscopic and microscopic observations reveal the existence of several impurities and inclusions accumulated in the members' microstructures. The average tensile

strength obtained by using the surface hardness values measured in this study and previous studies give an approximate tensile strength in the order of 300-350 MPa. However, due to the extremely heterogeneous microstructure observed during the microscopic investigations, these values should only be taken as upper boundary values.

Keeping the obtained information in mind, efforts were concentrated in seeking for a suitable material that was to imitate the original cramps and pins during the construction of the wall specimens. However, due to improved construction techniques and safety concerns of the modern day's steel manufacturers, in this study, it could not be possible to find a low strength standard steel material available at the market. Eventually, after an extensive market survey, a special type of mild steel sheet, which was being used by the automotive industry, due to its high deformation and energy dissipation capacity, could be obtained for the production of cramps.

The selected material for cramps was manufactured by Erdemir (Ereğli Demir ve Çelik Fabrikaları T.A.Ş.) and specified as Erdemir 6222. Since the required amount was only 60 kg, the material could only be transferred from the stocks of the Kesan Sac Industries. The metal sheet rolls were produced with a thickness of 2.2 mm and width of 1200 mm. The mechanical and chemical characteristics of the material as supplied by the manufacturer are presented in Table 4.12. It should be noted that these values correspond to the actual production lot of the sheet.

Table 4.12 : Characteristic values for Erdemir 6222 as supplied by the manufacturer.

Yield stress (MPa)	262
Ultimate tensile strength (MPa)	387
Ultimate strain (%)	37
Chemical composition (%)	C:0.03; Si:0.01; S:0.016; P:0.006; Mn:0.24; Al:0.045

Prior to production of the cramps, three samples were subjected to uniaxial tension tests in order to verify the manufacturer supplied information. For this purpose, three test pieces were prepared and tested in compliance with the EN 10002-1 (2001) standard. A schematic representation of the test pieces and their dimensions are presented in Figure 4.26. The samples were machine cut by using a precise cutting

laser machine and instrumented by two TML YFLA-5 post-yield type strain gauges. The strain gauges were bonded at mid-length in the longitudinal direction (at both sides) of the test pieces. Tensile tests were carried out by using a 200 kN capacity Amsler tensile testing machine. Data was collected via a TML TDS-302 data logger, which was connected to a laptop computer.

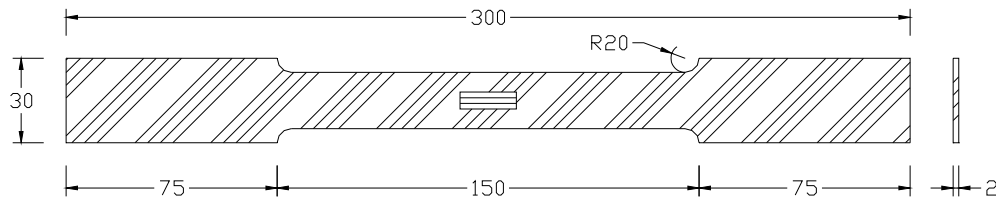


Figure 4.26 : Machined test piece for tensile tests (units mm).

Dimensions of the sample cross-sections and the major mechanical characteristics such as yield stress, yield strain, tensile strength and Young's modulus are summarized in Table 4.13. Accordingly, the material chosen to mimic the relatively low quality original wrought iron cramps during the construction of the wall specimens; reached yielding stress of 249 MPa at about 0.212% strain and the Young's modulus was in the order of 190000 MPa, whilst the average ultimate tensile strength was 337 MPa. The tensile stress-strain relationships obtained for these specimens are presented in Figure 4.27.

Table 4.13 : Mechanical characteristics from tensile tests of new cramp material.

Specimen	Width (mm)	Thickness (mm)	Yielding strain (%)	Yielding stress (MPa)	Tensile strength (MPa)	Young's modulus (MPa)
K1	19.98	2.32	0.218	243	330	191742
K2	19.78	2.34	0.233	261	343	178894
K3	19.92	2.32	0.184	242	338	198665
Average	19.89	2.33	0.212	249	337	189767

The obtained results are only slightly lower than the manufacturer supplied mechanical characteristics. Nevertheless, the material selected for the new scaled-down cramp production turns out to be quite acceptable when compared with the mechanical characteristics addressed for the original iron members. Once more, it should be noted that, the tensile strength estimations based on the empirical relationships between the tensile strength and Brinell hardness should be regarded as

indicative values. Therefore, an additional measure was taken during the design of the imitated cramps. As mentioned in Chapter 3, the cross-section dimensions of the original prototype cramp was assumed as 30×9 mm and for the construction of the model walls a 1/3 scale was preferred which requires a cramp cross-section area of 10×3 mm (area 30.0 mm²). In order to account for the over strength of the selected material, the heterogeneous structure observed in the original cramps and cross-section reduction caused by the corrosion of the original cramps; the imitated model cramps were produced with cross-section dimensions of 8×2.3 mm (area 18.4 mm²). A precise cutting laser machine was utilised to guarantee an exact cross-section width of 8 mm. After cropping the strips of designated dimensions from the steel sheets, tips were bent by using a bending machine. View of a model cramp after the production process is shown in Figure 4.28.

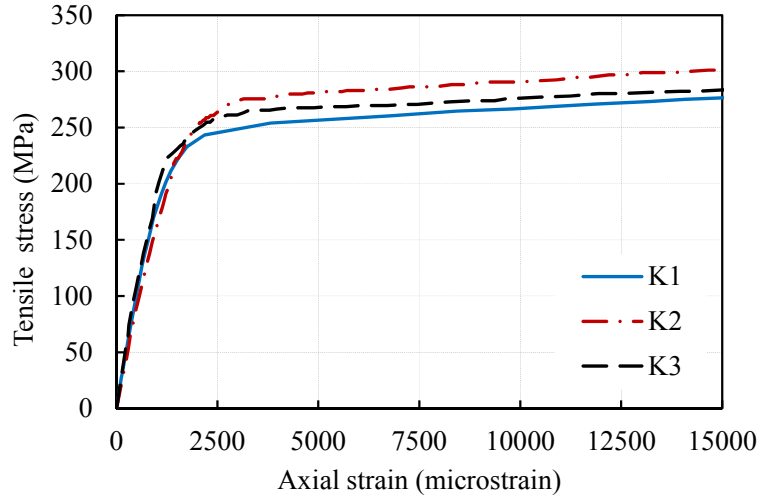


Figure 4.27 : Stress-strain curves of new cramp material.

A similar test process was also followed for the new material used for the pins. As mentioned in Chapter 3, the cross-section dimensions of the original prototype pin was assumed as 21×21 mm and for the construction of the model walls a 1/3 scale was preferred which requires a pin cross-section area of 7×7 mm (area 49.0 mm²). In order to account for the over strength of the selected material, the heterogeneous structure observed in the original wrought iron material and cross-section reduction caused by the corrosion of the original pins; the imitated model pins were produced as short rods with a circular cross-section diameter of 7.1 mm (area 39.5 mm²). The rods used for production of the pins were also tested under uniaxial tension. Two TML YFLA-5 strain gauges were utilized to monitor the strains during the tensile

tests. The test results obtained for the pins are given in Table 4.14. Accordingly, the average values for diameter, yielding strain, yielding stress, ultimate tensile strength and Young's modulus were 7.11 mm, 0.587%, 639 MPa, and 165000 MPa, respectively. Unfortunately, the tests could only be carried out after the production of the walls and despite the claims of the vendor, the utilized material for new pins turned out to be extremely high strength. However, this unexpected state did not actually cause a significant problem during the tests, since the stone units cracked way before the reinforcements reached their nonlinear range. View of a model pin after the production process is shown in Figure 4.28.

Table 4.14 : Mechanical characteristics from tensile tests of new cramp material.

Specimen	Thickness (mm)	Yielding strain (%)	Yielding stress (MPa)	Young's modulus (MPa)
D1	7.12	0.590	638	163500
D2	7.12	0.575	630	167967
D3	7.08	0.596	648	163572
Average	7.11	0.587	639	165013



Figure 4.28 : Views of model cramp and pin reinforcements after production.

4.5 Initial Shear Tests for Dry Joints

Masonry is a continuum, which consists of units that transfer stresses to each other via interfaces. In the case of shear stresses, one of the main contributors to bearing capacity is the friction phenomenon. Particularly, in the case of dry joints, friction is of paramount importance. Although, numerous studies are available for masonry with mortar interfaces, little is known for structural aspects of friction of dry joints. Relatively recent studies, from the perspective of structural engineering, were

performed by Oliveira (2003) and Vasconcelos (2005). To the best knowledge of the author, none of the available studies in literature investigates the shear behaviour of Küfeki stone dry joints. In this topic, results of a series of initial shear tests carried out on Küfeki stone units will be reported, particularly in the form friction coefficients. In addition to that, the effect of surface texture on the shear behaviour of dry joints is also investigated.

The tests performed at the ITU Structural and Earthquake Engineering Laboratory can be grouped into two main sets. The first group (SET I) consisted of stone units with machine-sawn surfaces, which was the actual case during the shear compression tests. However, since in the past, the stone surfaces were prepared by using a hammer and chisel, they were not as smooth as machine-sawn surfaces. Apparently, this difference in surface texture may rise concerns in terms of mechanical characteristics such as shear capacity and internal friction angle. Consequently, a second set of initial shear tests (SET II) were found to be necessary to have a comparison between the smooth and rough Küfeki stone surfaces.

4.5.1 Testing procedure for initial shear tests

The employed test method was in compliance with the initial shear test methodology described in TS EN 1052-3 (2004) standard. Triplet type testing (Figure 4.29a) was preferred to couplet type testing (Figure 4.29b) for its easier application and reduction of overturning moments. A general view of the test apparatus developed for this study is presented in Figure 4.30.

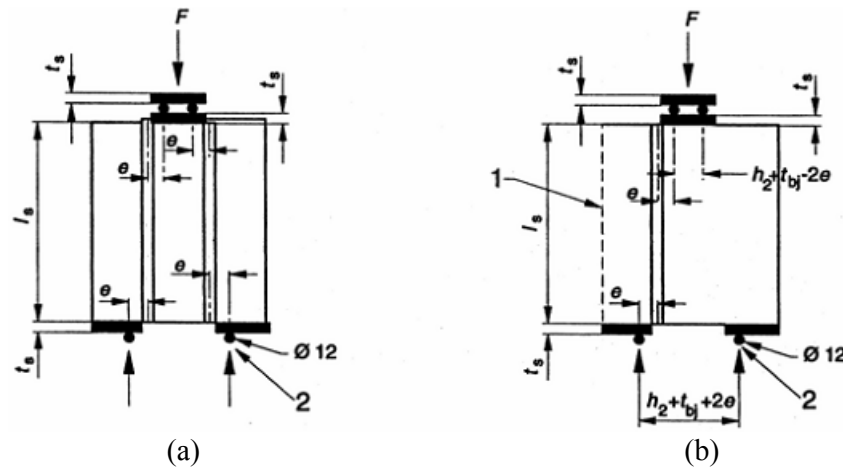


Figure 4.29 : (a) Triplet and (b) couplet initial shear test configurations.

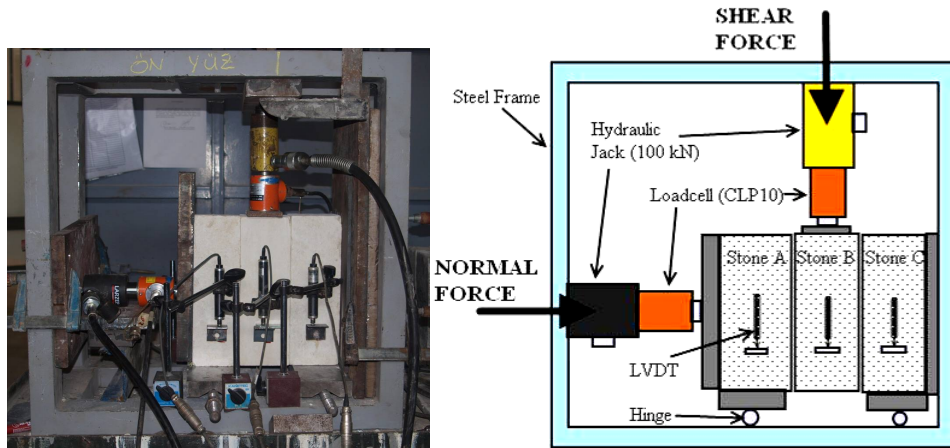


Figure 4.30 : General and schematic views of the initial shear strength test setup.

During the initial shear tests, three stone prisms, $100 \times 100 \times 300$ mm, were placed side by side into a reaction frame. Prior to placing, any potential bulge or irregularity left from rotary saw cutting was corrected. Two of the stones (Stone A and C in Figure 4.30) were supported by two hinged plates. After exerting the normal force (horizontal force in Figure 4.30) by using a 100 kN capacity hydraulic jack, a TML CLP10 load cell and six TML CDP-25 type LVDTs were placed on both sides of the specimen (three LVDTs on each side). Tips of these LVDTs were contacting with the aluminium corners bonded on the stone surfaces and their magnets were fixed to the bottom of the reaction frame. By utilizing these devices, net displacement of Stone B could be obtained. Finally, the shearing force (vertical force in Figure 4.30) was monotonically (and very slowly) increased until a complete yielding was achieved. Special attention was paid to keep the normal stress constant and small adjustments were made throughout the tests. Experimental data was monitored and logged by using a TML TDS-302 type data logger, connected to a computer.

During the tests, three different levels of normal forces were acted to create normal stresses of 0.25, 0.75 and 1.25 MPa. It should be noted that normal stresses recommended by TS EN 1052-3 (2004) standard were 0.2, 0.6 and 1.0 MPa for units with compressive strengths higher than 10 MPa. However, slightly higher values were utilized, since, same normal stress levels were also planned to be used during the wall tests but, later on, could not be achieved for the shear compression test specimen M-100-C planned to be loaded under 1.25 MPa axial stress (which was loaded up to 1.0 MPa).

4.5.2 Initial shear test results for SET I

SET I consisted of six initial shear tests for 0.25, 0.75 and 1.25 MPa normal stress levels. For each stress level, two specimens were prepared by using rotating saw cut stones that had smooth surfaces. In SET I; specimens BK-025-1 and BK-025-2 were subjected to 0.25 MPa normal stress whilst BK-075-1 and BK-075-2 were tested under 0.75 MPa normal stress. Finally, the BK-125-1 and BK-125-2 were subjected to 1.25 MPa normal stress level. Shear stresses were calculated by dividing the shear force at each recorded displacement level by the net shearing area which includes two dry joints between Stones A, B and C:

$$\tau = \frac{F_{shear}}{2b(l - d_s)} \quad (4.8)$$

where τ is the shear stress, F_{shear} is the shear force, d_s is the net displacement, and b and l are the average width and length of the stones, respectively.

Typical shear stress-displacement and normal stress-displacement diagrams of one of the specimens, tested under 0.25 MPa normal stress level, are given in Figure 4.31. From this figure, one can observe that the shear bearing capacity, obtained after initiation of slip between the stone units, is heavily dependent on the variation of normal stress. Even small fluctuations in the normal stress level are effective on the shear capacity.

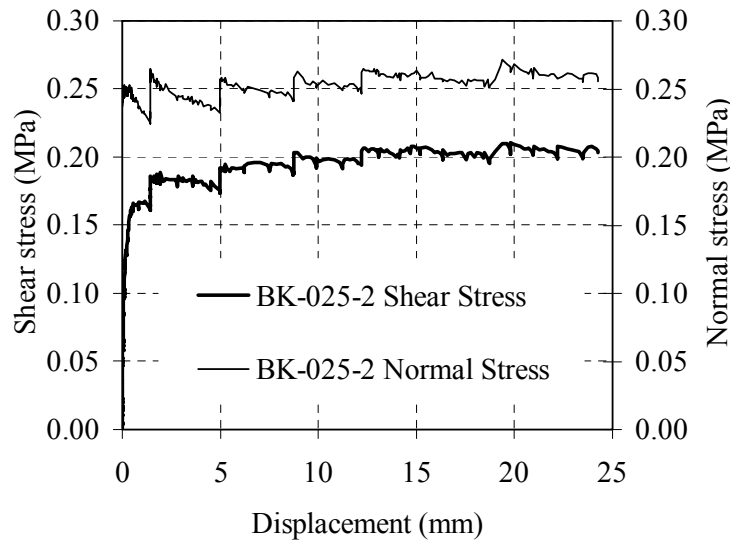


Figure 4.31 : Shear and normal stress diagrams of specimen BK-025-2.

Typical shear stress-displacement plots obtained for 0.25, 0.75 and 1.25 MPa normal stress levels (specimens BK-025-2, BK-075-2 and BK-125-2 respectively) are compared in Figure 4.32. Please note that Figure 4.32a covers the whole displacement range measured during the tests (i.e. 25.0 mm slip), whereas Figure 4.32b presents the displacement range up to 3.0 mm, so that the initial hardening behaviour can be better observed. As expected, maximum shear stresses increase with applied normal stress level. During the tests, at displacements less than 1.0 mm, a nonlinear behaviour can be observed until the peak shear stress, Figure 4.32. At larger displacements, the shear stresses were kept almost constant as long as the normal stress could be kept constant. No damage, except the wearing of the friction surfaces could be observed on the specimens.

It should be noted that, due to a failure in the data logging system, load and displacement data of three specimens could not be obtained. However, peak load data of these tests were manually registered during the tests. By using this data, a mean line for BK-075-2 specimen could be plotted.

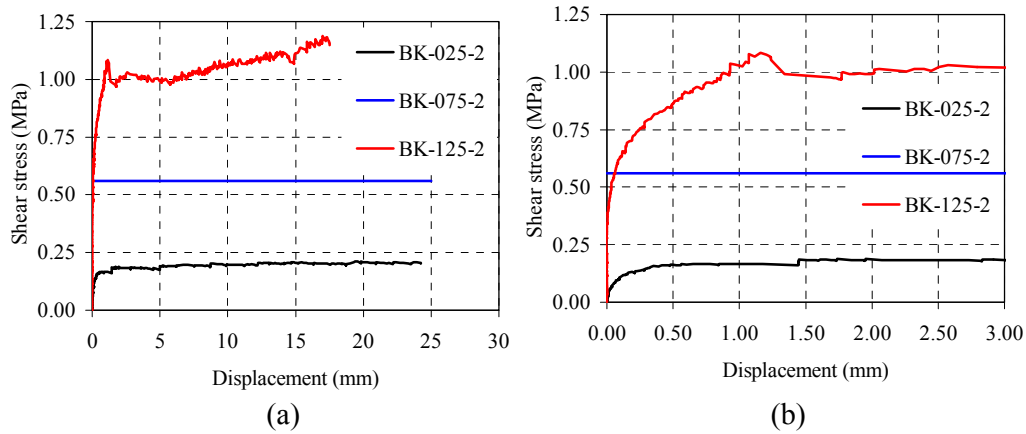


Figure 4.32 : Typical shear stress-displacement diagrams for SET I specimens.

4.5.3 Initial shear test results for SET II

SET II consisted of three initial shear tests for 0.25, 0.75 and 1.25 MPa normal stress levels. For each stress level, one specimen was prepared by using chisel and hammer to roughen the friction surfaces. The roughening process was done carefully to have a similar texture on all stone faces. Close up views for smooth texture of SET I and rough texture of SET II are presented in Figure 4.33.

Typical shear stress-displacement diagrams obtained for 0.25, 0.50 and 0.75 MPa normal stress levels (specimens BK-025-RS, BK-075-RS and BK-125-RS respectively) are compared in Figure 4.34. The observed behaviour was very similar to SET I specimens. Surface wearing was eye catching when the surfaces were inspected after the tests. Observations on the interfaces showed that, the higher the normal stress was, the more surfaces were worn out during the tests.



Figure 4.33 : Surface textures of SET I (a) and SET II (b) stone units.

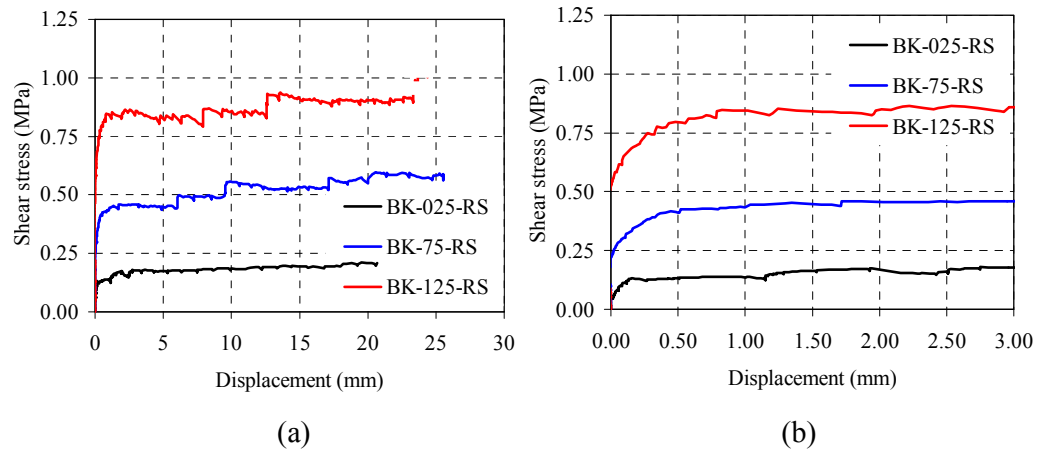


Figure 4.34 : Typical shear stress-displacement diagrams for SET II specimens.

4.5.4 Comparison of the test results for smooth and rough surfaces

One might note that only small differences in shear strength capacities of SET I and SET II, especially for specimens with a normal stress of 1.25 MPa, exist. The similarity in the behaviour can be better understood by comparing the tangent values of internal friction angles (or namely the friction coefficients) that can be obtained by applying the Coulomb friction law:

$$\tau = c + \sigma \tan \phi \quad (4.9)$$

where τ is the shear strength of the joint, c is the bond or cohesion, σ is the normal stress and ϕ is the friction angle of the joint. Since the cohesion in dry joints is non-existent, the c cohesion parameter can be assumed as zero. Shear strength and coefficient of friction ($\tan \phi$) values obtained from SET I and SET II tests are tabulated in Table 4.15. The normal stresses given in this table correspond to actual stress values at peak load levels. In addition, in Figure 4.35, shear strengths are plotted against normal stresses. As also indicated in the TS EN 1052-3 (2004) standard, the friction coefficients for smooth and rough textured specimens (SET I and II, respectively) can be obtained by performing linear least square regression of data points given in this figure. By applying this procedure, the coefficient of friction ($\tan \phi$) value for SET I specimens was obtained as 0.76, while it was 0.74 for SET II specimens. These values are slightly greater than the 0.62 value reported by Oliveira (2003) for sand stone specimens.

Table 4.15 : Shear strength and coefficient of friction values.

Specimen	Surface	Normal stress (MPa)	Shear strength (MPa)	Coefficient of friction $\tan \phi$
BK-025-1	Smooth	0.25	0.18	0.70
BK-025-2	Smooth	0.26	0.20	0.79
BK-075-1	Smooth	0.75	0.55	0.73
BK-075-2	Smooth	0.75	0.57	0.76
BK-125-1	Smooth	1.30	0.96	0.74
BK-125-2	Smooth	1.28	1.06	0.83
BK-025-RS	Rough	0.25	0.18	0.75
BK-075-RS	Rough	0.76	0.54	0.71
BK-125-RS	Rough	1.23	0.93	0.76

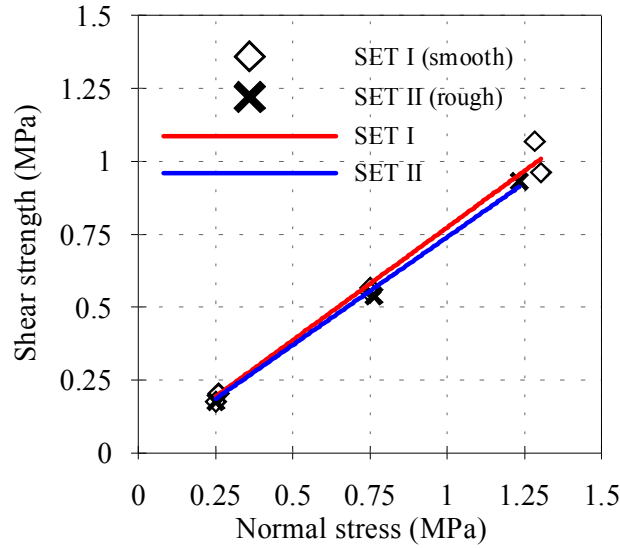


Figure 4.35 : Shear strength vs. normal stress for SET I and II specimens.

In Table 4.15 and Figure 4.35, it is interesting to note that shear capacity and coefficient of friction ($\tan \phi$) values are almost coinciding for both smooth and rough textured specimens. However, this perfect similarity between friction coefficients of two very different surface textures is somehow illusional, simply because the contact areas between adjacent stones (with rough surfaces) used for calculation of normal and shear stresses are taken as gross areas. However, in reality, the net bearing areas in joints are much smaller due to peaks and valleys in the rough surface texture of the SET II specimens. At this point of the study, a perspective from a different point of view may be helpful to materialize this explanation. For this purpose, a study aiming the characterization of the smooth and rough surfaces was carried out.

4.5.5 Characterization of the friction surfaces

Surface textures of contacting materials play a major role in determination of the interface strength. Texture of a surface consists of three main components: Form, waviness and roughness. Quantification of these components can be done either by using sectional or areal measurement devices, (Thomas, 1999). Though several devices based on optical, mechanical, acoustic or laser measurement methods for 3D areal measurement are available, a home-made stylus type sectional device was used in this study. Similar studies were also carried out by Vasconcelos (2005) and Develi (2006) for rock surfaces. Vasconcelos (2005) used an optical 3D topographical system for characterization of fractured surfaces of granites after direct tension tests,

while Develi (2006) developed a computer controlled surface scanner and used fractal geometry for definition of roughness parameters and performed shear tests.

General views of the developed apparatus are shown in Figure 4.36. The profilometer was based on two Mitutoyo LVDTs with digital scopes and 50 mm gauge length. Both LVDTs were fixed on a heavy steel plate that rests on a table. The horizontal LVDT was used for monitoring of the transverse displacement of the stone unit, while a vertical one was used for measurement of the irregularities on the top face of the stone. A needle with a slightly rounded tip was attached to the vertical LVDT, in order to have pinpoint accuracy for sharp valleys and peaks of the texture. During the measurements, after the needle of the vertical LVDT was lifted, the stone unit was moved very slowly in the longitudinal direction. This very small displacement (kept less than 1 mm) was observed from the scope of the horizontal LVDT and registered. Afterwards, the tip of the vertical LVDT was placed upon the surface very slowly, and the position of the tip with respect to initial vertical and horizontal coordinates was registered. The measurements were done for a total length of 50 mm, along a previously drawn line. A total of 9 profiles (3 smooth and 6 rough textures) were extracted from initial shear test specimen stone units before the implementation of the tests. Typical profiles obtained for SET I (smooth texture) and SET II (rough texture) specimens are presented in Figure 4.37.

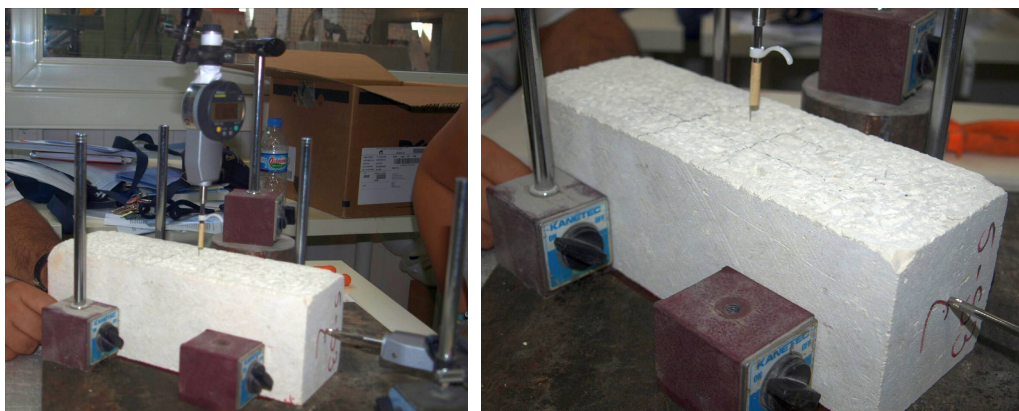


Figure 4.36 : Stylus type profilometer for quantification of the surface textures.

As also indicated before, the surface profile texture is a composition of structures with different ranges of irregularities and irregularity distributions, or a range of frequency and wavelength components (Vasconcelos, 2005). The longest wavelengths are associated with errors of form (such as curvature); shorter wavelengths are associated with errors of form (such as curvature); shorter wavelengths constitute waviness; and the shortest wavelengths are called roughness

(Figure 4.38), (Thomas, 1999). Apparently, the definitions of these components vary depending on the investigated subject. For instance, a range for waviness of an engine cylinder surface might be classified as roughness for a concrete surface.

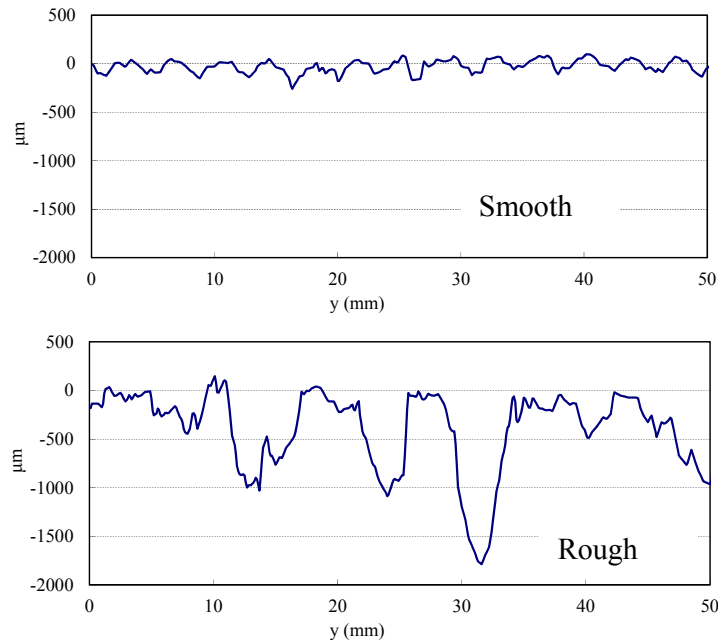


Figure 4.37 : Typical raw profiles obtained for smooth and rough surfaces.

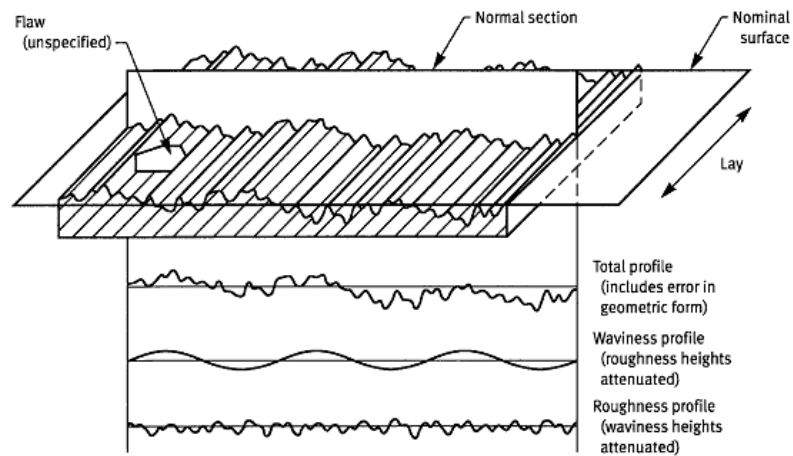


Figure 4.38 : Profile components (Muralikrishnan and Raja, 2009).

In order to set apart form, waviness and roughness from the raw texture profile, the texture profile needs to be processed for appropriate frequency and wavelength components. Figures 4.39 and 4.40 address results of the followed filtering process in the case of smooth and rough stone surfaces, respectively. Please note that the details of the filtering process are presented in Appendix B.

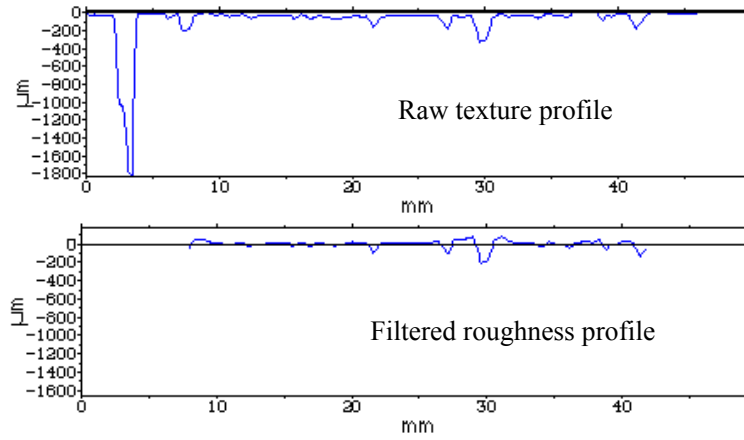


Figure 4.39 : Setting apart of texture components for a smooth surface (5-5-A unit).

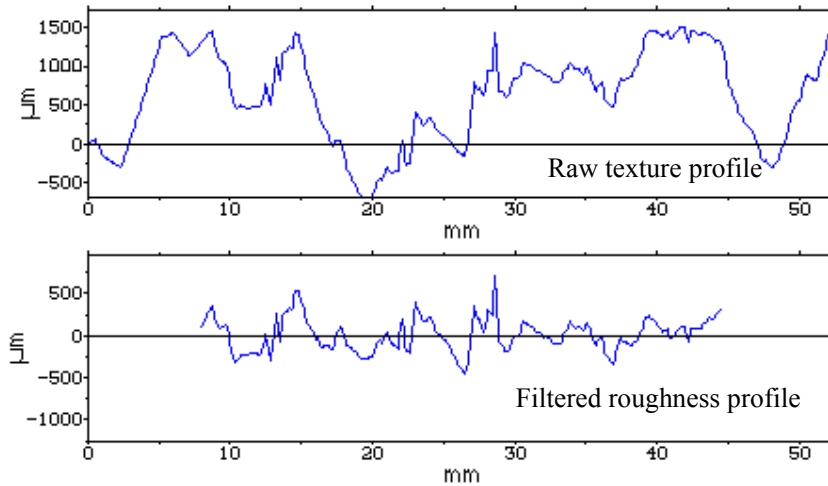


Figure 4.40 : Setting apart of texture components for a rough surface (5-S3-C unit).

After setting apart the components of texture profiles, quantitative values such as amplitude, spacing and shape parameters for waviness and roughness profiles can be calculated. In literature, various parameters defined for investigating certain aspects of the profiles are available, (Thomas, 1999 and Muralikrishnan and Raja, 2009).

Among the available amplitude parameters; average roughness R_a , root mean square roughness R_q , maximum profile peak height R_p , maximum profile valley height R_v , maximum height of the profile R_t and average maximum height of the profile R_z are the most widely utilized ones. Average roughness R_a can be defined as the arithmetic average of the absolute values of the roughness profile measured from the mean line and computed by:

$$R_a = \frac{1}{n} \sum_{i=1}^n |r(i)| \quad (4.10)$$

where r represents a roughness profile containing n elements and $r(i)$ is the i^{th} element of r . R_q parameter is the root mean square average of the profile measured from the mean line and obtained by:

$$R_q = \sqrt{\frac{1}{n} \sum_{i=1}^n r(i)^2} \quad (4.11)$$

Maximum profile peak height R_p and the maximum profile valley height R_v , are the distance to the highest and lowest points of the profile measured from the mean line, respectively. Naturally, maximum height of the profile R_t , that is the distance between the highest and lowest points of the profile, becomes the sum of R_p and R_v . Average maximum height of the profile R_z is the average of the successive values of R_{ti} , calculated over the length of the profile. R_{ti} is defined as the maximum height of the profile within the i^{th} sampling length.

The distribution of the peaks and valleys along the profile can be quantified by the mean peak spacing parameter RS_m , which is the average distance between zero crossings of the profile. If there are q zero crossings with distances of $S_m(i)$, the mean peak spacing parameter can be calculated by:

$$RS_m = \frac{1}{q} \sum_{i=1}^q S_m(i) \quad (4.12)$$

Finally, the shape parameters skewness R_{sk} and Kurtosis R_{ku} define the asymmetry and spikiness of the profile, respectively. Negative skew indicates a predominance of valleys, while positive skew is seen on surfaces with peaks. Kurtosis is a measure of the spikiness of the profile: for spiky surfaces $R_{ku} > 3$, for bumpy surfaces $R_{ku} < 3$, and for perfectly random surfaces $R_{ku} = 3$. These parameters can be calculated by:

$$R_{sk} = \frac{1}{nR_q^3} \sum_{i=1}^n r(i)^3 \quad (4.13)$$

$$R_{ku} = \frac{1}{nR_q^4} \sum_{i=1}^n r(i)^4 \quad (4.14)$$

Similar parameters can also be computed for the waviness profiles such as; average waviness W_a , root mean square waviness W_q , maximum profile peak height W_p , maximum profile valley height W_v , maximum height of the profile W_t , average maximum height of the profile W_z , skewness W_{sk} and Kurtosis W_{ku} .

Surface texture parameters defined above are calculated for the measured profiles of smooth and rough surfaces. Results for roughness parameters of SET I (with smooth surfaces) and SET II (with rough surfaces) type specimens are presented in Tables 4.16 and 4.17, respectively. The results for waviness parameters of the same specimen groups are presented in Tables 4.18 and 4.19. Comparing the values of these parameters obtained for roughness and waviness components of the raw profiles one can observe that:

- Amplitude parameters obtained both for waviness and roughness components of the measured profiles vary significantly for smooth and rough surfaces. This huge difference backs the visual observations for qualification of these surfaces as smooth or rough.
- Waviness has an important role in forming the final geometry of the texture profile, not only for rough textures but also for smooth ones.
- Peak and valley amplitudes are close to each other for both smooth and rough profile types.
- Valleys are more predominant for the roughness component of smooth surfaces, whereas peaks are slightly more predominant for rough surfaces. However, the situation is just the opposite for waviness components.
- Waviness components of both surface types are bumpy. However, roughness components of smooth surfaces are spiky, while they are bumpy for rough surfaces.
- Interestingly, variation coefficients of texture parameters, obtained for the machine sawn surfaces, were significantly higher than that of roughened surfaces.

After-test-views of the surfaces for smooth and rough textures are presented in Figures 4.41 and 4.42, respectively. From these figures, it can be observed that, independent of the surface texture type, the contact between the units was partial and not uniform. However, contact area increases with the increase in normal stresses. As also seen in the quantification of the surface texture parameters, waviness plays a very important role, even for the SET I specimens (smooth texture). Moreover, mechanical constant estimation for friction of such dry joints should be approached with caution and the obtained friction coefficients should be accepted as global parameters rather than exact values. Finally, it can be concluded that, usage of smooth surfaced Küfeki stone units is acceptable when the test results are compared with the results of more realistic rough surfaced ones.

Table 4.16 : Roughness texture parameters for smooth surfaces (SET I specimens).

Profile	R_a	R_q	R_p	R_v	R_t	R_z	RS_m	R_{sk}	R_{ku}
	μm	μm	μm	μm	μm	μm	μm		
P-4	47.5	56.7	94.0	129.5	285.7	223.5	3454.1	-0.46	2.30
4-6-A	104.8	147.1	250.2	344.4	1158.5	610.5	2615.3	-0.60	5.36
5-5-A	27.1	43.0	80.0	206.0	286.0	150.9	2865.2	-2.17	10.05
Average	59.8	82.3	141.4	226.6	576.7	328.3	2978.2	-1.1	5.9
St. Dev.	40.3	56.6	94.5	108.9	503.9	247.1	430.7	0.9	3.9
COV (%)	67.4	68.8	66.8	48.1	87.4	75.3	14.5	-88.1	66.1

Table 4.17 : Roughness texture parameters for rough surfaces (SET II specimens).

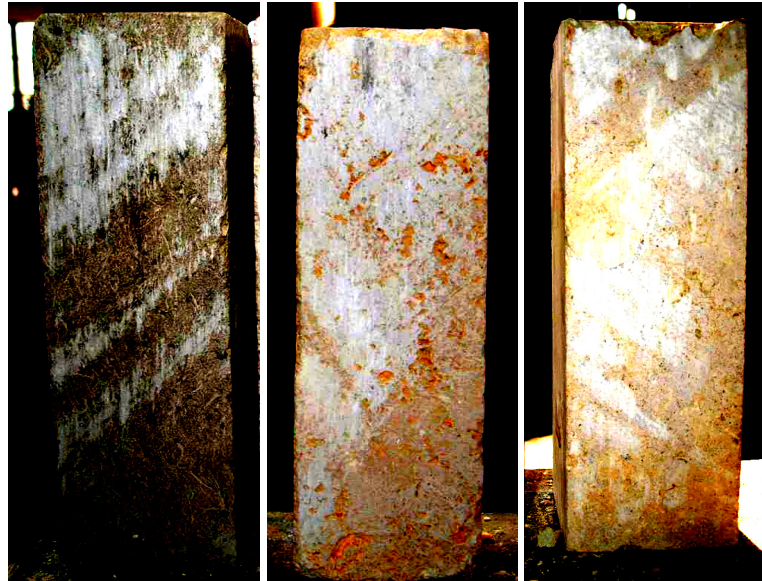
Profile	R_a	R_q	R_p	R_v	R_t	R_z	RS_m	R_{sk}	R_{ku}
	μm	μm	μm	μm	μm	μm	μm		
4-S1-A	236.1	310.8	1224.1	526.5	1750.6	914.4	3747.1	0.94	4.61
4-S1-C	209.1	253.3	461.3	422.0	1334.0	883.3	1958.9	0.10	2.04
4-S2-A	158.9	158.9	299.0	322.0	688.2	624.9	1864.6	-0.14	1.91
4-S2-C	120.7	142.9	321.1	308.8	630.0	508.0	3968.3	-0.13	2.08
5-S3-A	175.6	212.0	394.8	414.5	977.9	811.0	3195.7	-0.08	2.38
5-S3-C	154.0	189.8	703.2	458.8	1162.0	814.2	3245.8	0.39	3.05
Average	175.7	211.3	567.3	408.8	1090.5	759.3	2996.7	0.18	2.68
St. Dev.	41.3	62.5	353.1	82.6	421.1	159.0	890.8	0.42	1.03
COV (%)	23.5	29.6	62.3	20.2	38.6	20.9	29.7	238.7	38.4

Table 4.18 : Waviness texture parameters for smooth surfaces (SET I specimens).

Profile	W_a	W_q	W_p	W_v	W_t	W_z	W_{sk}	W_{ku}
	μm	μm	μm	μm	μm	μm		
P-4	16.5	20.5	15.3	28.4	85.3	40.8	-0.41	2.69
4-6-A	153.0	203.3	144.0	336.1	900.7	485.9	-0.28	2.64
5-5-A	45.4	56.7	110.4	63.0	173.3	53.6	1.28	2.38
Average	71.6	93.5	89.9	142.5	386.5	193.4	0.2	2.6
St. Dev.	71.9	96.8	66.7	168.6	447.5	253.4	0.9	0.2
COV (%)	100.5	103.6	74.2	118.3	115.8	131.0	478.5	6.6

Table 4.19 : Waviness texture parameters for rough surfaces (SET II specimens).

Profile	W_a	W_q	W_p	W_v	W_t	W_z	W_{sk}	W_{ku}
	μm	μm	μm	μm	μm	μm		
4-S1-A	508.0	602.8	576.2	1164.3	1740.5	871.7	-1.34	2.20
4-S1-C	545.1	669.6	636.5	851.0	2566.9	1502.4	0.04	2.51
4-S2-A	188.6	158.9	206.8	371.4	958.9	576.2	-0.29	2.07
4-S2-C	284.8	353.1	697.7	703.5	1401.3	435.9	0.61	2.39
5-S3-A	246.8	294.7	212.2	575.2	1163.7	792.5	-0.94	2.24
5-S3-C	398.9	471.5	709.4	995.5	1704.9	718.5	-0.57	2.30
Average	362.0	425.1	506.5	776.8	1589.4	816.2	-0.42	2.28
St. Dev.	145.3	193.2	234.9	287.7	566.7	370.5	0.70	0.15
COV (%)	40.1	45.4	46.4	37.0	35.7	45.4	-167.6	6.7



(a) 0.25 MPa normal stress (b) 0.75 MPa normal stress (c) 1.25 MPa normal stress

Figure 4.41 : Views of the surfaces after testing SET I specimens (smooth surfaces).



(a) 0.25 MPa normal stress (b) 0.75 MPa normal stress (c) 1.25 MPa normal stress

Figure 4.42 : Views of the surfaces after testing SET II specimens (rough surfaces).

4.6 Compression Tests on Masonry Prisms

In addition to the single stone unit compression tests, masonry prisms were also tested under compressive forces. The masonry compression tests were carried out in two stages. The first stage aimed to investigate the compressive behaviour of ashlar masonry that constituted the outer leaves of the three-leaf wall specimens. The second test stage included three-leaf wallets, which shared similar characteristics with the three-leaf specimens of the combined shear compression tests.

The wallets were constructed with Küfeki stone units, which were randomly chosen among the units quarried and sawn prior to their transportation to the Structural and Earthquake Laboratory of Istanbul Technical University. However, due to the size restrictions of the 5000 kN capacity Amsler compressive testing machine, the stone units that were used for the construction of the masonry prisms, were partitioned into two separate units: i.e. the 100×130×300 mm units were partitioned as 100×130×100 mm and 100×130×200 mm pieces. The geometrical aspects of the specimens were arranged according to the TS EN 1052-1 (2000) requirements and the size restrictions of the testing machine. The requirements of TS EN 1052-1 (2000) regarding the sizes (length l_w , height h_w and thickness b_w) of the specimens to be built are presented in Table 4.20 for length l_u and height h_u of stone units provided that $l_u \leq 300\text{mm}$ and $h_u \leq 150\text{mm}$.

Table 4.20 : The masonry prism size requirements of TS EN 1052-1 (2000).

l_w	b_w	h_w
$\geq 2l_u$	$\geq b_u$	$\geq 5h_u; \geq 3b_u;$ $\leq 15b_w; \geq l_w$

4.6.1 Single-leaf masonry prism compression tests

Three single-leaf ashlar masonry prisms, representative of the external ashlar leaves of the investigated multi-leaf stone masonry walls, were tested under uniaxial compression loading. The compression tests enabled the derivation of axial stress-strain curves and related strength and deformation characteristics. Prior to loading of the specimens, ultrasound measurements were done on each prism, so that indicative velocity values could be obtained for the utilized limestone. General views of the single-leaf specimens are given in Figure 4.43, where the approximate dimensions of a typical specimen are also shown. Considering Figure 4.43 and the average dimensions of the specimens given in Table 4.21, it can be seen that the selected specimen dimensions conform with the size requirements of TS EN 1052-1 (2000).

Since there were not any mortar joints between the stone units, the construction of the prisms were done easily. After construction of the specimens, the fixing rods, used for attaching the vertical and transverse LVDTs, were installed on the prisms. For this purpose, holes were drilled at gauge points, which were defined by the TS EN 1052-1 (2000) standard. The gauge points used in these tests are shown as full red circles in Figure 4.43. The rods fixed at the gauge points of the masonry compression tests and wall shear tests were prepared as explained below: Firstly, anchor holes were drilled to a depth of 50-60 mm. Secondly, the holes were cleaned by applying pressurized air via an air compressor. Then, by using an epoxy-based adhesive, the 8 mm diameter anchors were inserted and bonded into the drilled holes.

Table 4.21 : The dimensions of the masonry prisms.

Prism	l_w (mm)	h_w (mm)	b_w (mm)
MPC-1	395	506	128
MPC-2	394	508	131
MPC-3	395	500	129

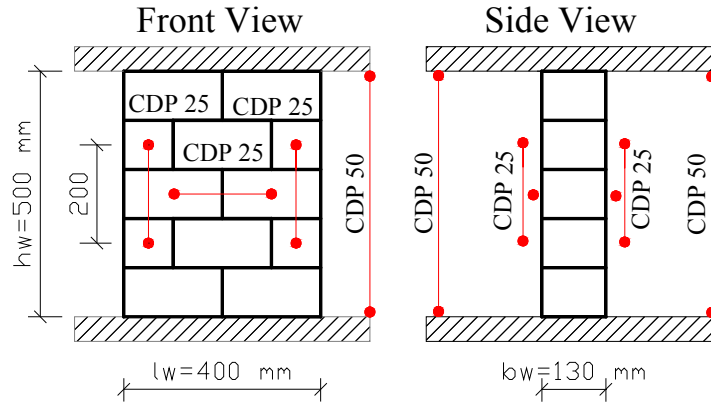


Figure 4.43 : Views of the single-leaf masonry compression test specimens.

Prior to testing of the masonry prisms, all stone units were dismantled for easier transportation and then rebuilt on the lower platen of the Amsler testing machine. Since the positions of the stone units were registered before the fixing of the rods at the gauge points, each stone unit was placed at the same exact location (Figure 4.44). After checking the alignment of the specimen, the displacement transducers were installed. Before each test, a preload of 10 kN was applied.



Figure 4.44 : Single-leaf masonry prism before and after the instrumentation.

After the application of the 10 kN preload, a non-destructive test, ultrasound test, was performed on the stone masonry prisms. The ultrasound device launches a very short pulse-wave from the transducer of the device, which is transmitted by the inspected media to the receiver probe. Then the travelling time is used to calculate the speed of sound in the sample media. In this study, the ultrasonic measurements were done on two points of each wall by using a Pundit ultrasound testing device. The measurement points were located at the middle of the second and fourth stone courses, and directed in the transverse direction. In order to avoid voids between the probes of the device and the investigated object, grease oil was used at the

measurement points. The measured echo travel times t_s and sound velocities v_s are given in Table 4.22. A snapshot from the ultrasound measurements is presented in Figure 4.45.



Figure 4.45 : Non-destructive ultrasound test on single-leaf prisms.

The instrumentation of the single-leaf masonry prisms consisted of four vertical TML CDP-25 LVDTs (with an approximate gauge length of 200 mm), two vertical TML CDP-50 LVDTs (with an approximate gauge length of 500 mm) and two horizontal TML CDP-25 LVDTs (with an approximate gauge length of 200 mm), as shown in Figure 4.43. At each sampling step, a TML TDS-302 data logger collected the displacement data of these displacement transducers while the load data was manually registered. Later on, both manually and electronically registered data were coupled in the computer.

The evolution of damage was identical for all single-leaf stone masonry prisms. Damage growth of the single-leaf wallets began with vertical cracks near the vertical edges of the stone units. When the peak load was reached, simultaneously new vertical cracks appeared around the middle parts of the prisms and signs of crushing started to appear. These cracks and initiation of crushing could be mainly observed on the longitudinal sides of the specimens. The cracks near the top and bottom ends of the walls were slightly inclined due to the friction between the top and bottom faces of the walls and the loading platens of the Amsler testing machine. In some cases, the cracked or crushed stone units near the edges of the specimens spalled. The spalling of the detached stone pieces caused an instant drop in the load and revealed the inclined crack planes near the loading platens (Figure 4.46).



Figure 4.46 : Typical damage observed in the single-leaf prism compression tests.

4.6.2 Multi-leaf masonry prism compression tests

Three-leaf masonry wallets shared similar characteristics with the specimens constructed for the combined shear compression tests. Four prisms with approximate dimensions of 400×500×300 mm (length × height × thickness) were prepared with dry-jointed limestone outer and rubble masonry inner leaves. As also shown in Figure 4.47, thickness of the rubble core was either 40 mm or 100 mm at successive stone courses. Stone units of the outer leaves were connected to each other via cramps as also used in the combined shear compression tests. The wallets were constructed on U330 steel profiles, so that they could be easily transported and positioned in the testing machine. A number of snapshots from the construction process of the multi-leaf compression test specimens are shown in Figure 4.48. Further details of the multi-leaf wallet construction, which are similar to that of shear compression tests, are delivered in Chapter 3.

Due to the limited free height of the Amsler 5000 kN capacity testing machine, height/thickness aspect ratio condition given in Table 4.20 (such as $h_w/b_w \geq 3$) could not be satisfied with an achieved h_w/b_w ratio of 1.67. Therefore, it can be expected to obtain higher strength characteristics than the wallets with code conforming aspect ratios, where second order effects would be larger.

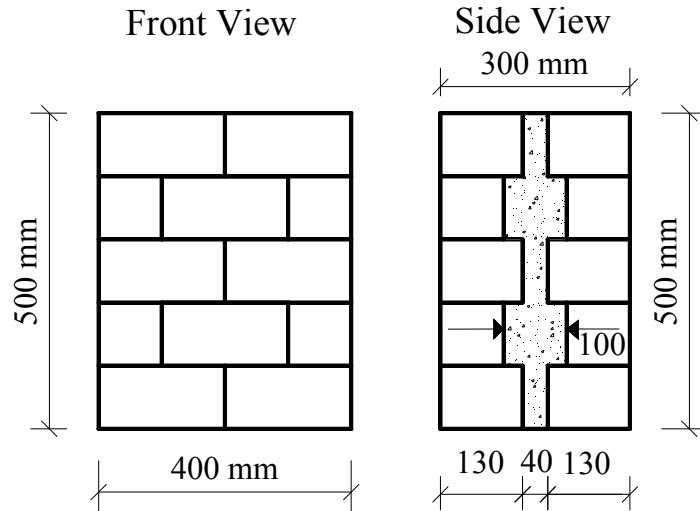


Figure 4.47 : Views of the multi-leaf masonry compression test specimens.



Figure 4.48 : Construction steps of the multi-leaf prisms.

The instrumentation of the multi-leaf masonry prisms consisted of eight vertical TML CDP-25, two vertical TML CDP-50 and eight horizontal TML CDP-25 LVDTs, as shown in Figures 4.49 and 4.50. The LVDTs installed on the specimens were attached to aluminium profiles that were bonded to certain gauge points. Magnets resting on the U330 steel profile supported V9 and V10 displacement transducers (measuring the axial shortening at full specimen height) and all horizontal LVDTs (namely H1-H8).

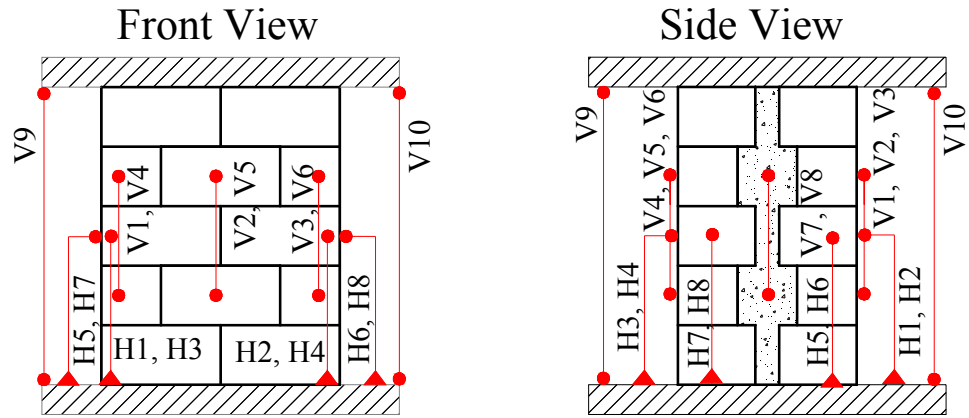


Figure 4.49 : LVDT layout of multi-leaf wallets compression tests.



Figure 4.50 : Multi-leaf masonry prism before and after the instrumentation.

Similar to the single-leaf wallets, prior to axial compression loading, ultrasound measurements were carried out along the transverse direction of the wallets, as seen in Figure 4.51. The axial compression loading was made monotonically after application of a pre-load of 10 kN. During the tests, at each sampling step, a TML TDS-302 data logger collected the displacement data of displacement transducers and the load data was manually registered.



Figure 4.51 : Non-destructive ultrasound test on multi-leaf prisms.

The damage evolution of the multi-leaf masonry prisms began with traces of vertical cracks at the interfaces between the external leaves and the internal core at about 8% of the compressive strength. Sub-vertical cracks started to appear on the stone units of the external leaves at an approximate axial stress level that corresponded to 30% of the compressive strength. Number of vertical cracks, formed on the longitudinal and the transverse sides, increased significantly at about 70% of the peak load value. These cracks were randomly distributed on the longitudinal faces. However on the transverse sides, they were concentrated at three particular zones: At the interfaces between external and internal leaves, on the wider units along the alignment of narrower stone blocks (passing within the infill material and protruding stone units) and at the mid-width of the stone blocks where the cramps were located. While approaching the peak load level, signs of crushing not only at the stone units but also on the internal core surface could be observed (particularly near the loading platens). After reaching the compressive strength, damage evolution gained haste and cracked and crushed stone and mortar pieces began spalling (Figure 4.52). The spalling of the detached stone pieces caused an instant drop in the load.

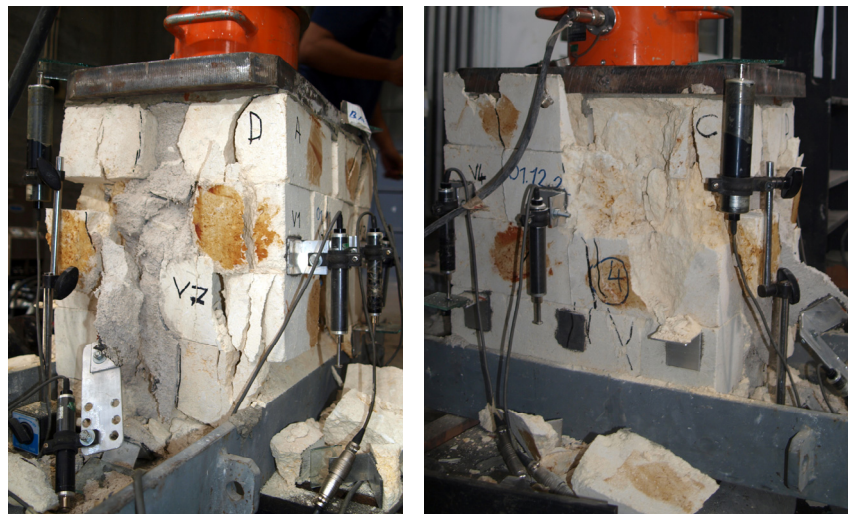


Figure 4.52 : Typical damage observed in the multi-leaf masonry compression tests.

4.6.3 Results of the masonry prism compression tests

The compressive stress-strain relationships obtained from single and multi-leaf masonry prism compression tests are presented in Figures 4.53 and 4.54, respectively. In both type of specimens, the axial stress values are evaluated as the mean vertical stress acting over the gross cross-section of the prisms. Apparently, this is a stretched assumption, particularly in the case of multi-leaf walls, where inner

and external leaves have significantly different deformation characteristics (i.e. Young's modulus values). Moreover, the gross area assumption for the multi-leaf walls may become misleading due to the probable existence of out-of-plane loads acting on the outer leaves that may lead to premature and unsymmetrical damage evolution of the external leaves. However, in this study, the stress-strain curves of the multi-leaf specimens are presented under this assumption, as generally done during the structural analysis of the actual structures.

The strain data of the single-leaf masonry prisms were obtained from the whole length of the specimens, since the LVDTs measuring the deformations at the mid-height gave irrelevant results due to non-uniform closing of the gaps at the dry bed joints. In the case of multi-leaf walls, except the ML-P-3 specimen, the compressive strains in the pre-peak region were obtained from vertical TML CDP-25 type LVDTs, installed over the mid-height of the masonry prisms. It should be noted that, these LVDTs gave abrupt displacement values after the development of extensive damage that corresponded to the peak stress region. Consequently, the post-peak strains of the multi-leaf prisms were obtained from TML CDP-50 type LVDTs that measured the axial shortening of the whole specimen height. In the case of ML-P-3 specimen, the LVDTs located at the mid-height failed to provide accurate measurements both in the pre- and post-peak regions, mainly due to development of damage near the bonding areas of the devices. Consequently, the strain data utilized for this masonry prism solely depended on the LVDTs measuring along the whole specimen height.

The stress-strain curves of single-leaf prisms, shown in Figure 4.53, began with an increasing slope, which is a clear indication of the closing of the gaps caused by the imperfections between the stone units. However, this hardening behaviour was not clearly observed in the case of multi-leaf walls, Figure 4.54. This was probably due to the stability brought by the cramps and rubble infill to the units of the external leaves. The behaviour of both prism types was almost linear until the vicinity of the peak stress. However, right around the peak stress, two of the single-leaf prisms exhibited a plateau like behaviour followed by a quick loss of strength. The multi-leaf prisms did not have a plateau at the peak, but on the other hand, the deformability of the composite wall was significantly higher than that of single-leaf

prisms. Moreover, the post-peak descending branch of the multi-leaf prism stress-strain curves was less steep than the single-leaf ones.

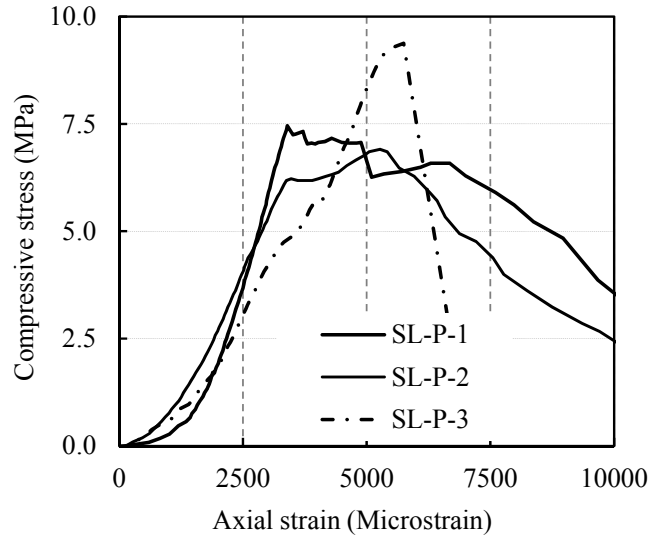


Figure 4.53 : The compressive stress-strain diagrams for the single-leaf walls.

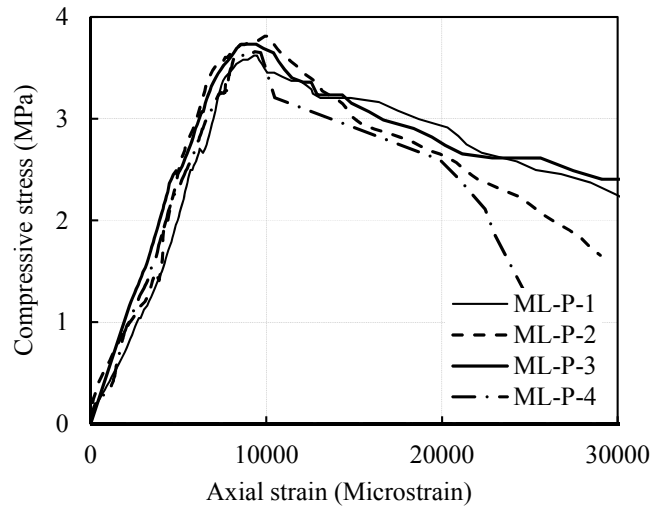


Figure 4.54 : The compressive stress-strain diagrams for the multi-leaf walls.

A summary of the results for single- and multi-leaf masonry wallets obtained from the uniaxial compression tests and ultrasonic measurements are given in Tables 4.22 and 4.23, respectively. In the case of single-leaf tests, the average values for compressive strength (f_{wc}), strain corresponding to compressive strength (ϵ_{wco}), Young's modulus (E_{wc}), echo travel time (t_s) and sound velocity (v_s) were obtained as 7.9 MPa, 0.47%, 2615 MPa, 37 s and 3.51 km/s, respectively. The ratio of Young's modulus, which was assumed as the slope of the curve between 20 to 50%

of the compressive strength, to the prism compressive strength varied between 221 and 453 with an average value of 330. Average single-leaf masonry compressive strength (f_{wc}) turned out to be 44% of the average single unit compressive strength ($f_{c,cube}$). This result is in agreement with the stone prism to unit strength ratios reported by Binda et al. (2006) that varied between 40 and 45%. Mean Young's modulus of the masonry prisms became 45% of the average Young's modulus obtained from single unit compression tests.

In the case of multi-leaf tests, average compressive strength (f_{wc}) values obtained by considering the whole cross section (400×300 mm), only external leaves with wider stone units (400×260 mm) and only external leaves with narrower units (400×200 mm) were 3.7, 4.3 and 5.6 MPa, respectively. Interestingly, regardless of the assumed effective compression area of the composite cross-section, the obtained average compressive strengths were remarkably below the 7.9 MPa strength obtained for the single-leaf masonry prisms. The axial force-displacement curves of both types of specimens can be compared as shown in Figure 4.55, so that the effective cross-section area dilemma of the multi-leaf prisms can be overcome. It should be noted that, in this figure, the axial load values used for single-leaf specimens are calculated for a total thickness of 200 mm.

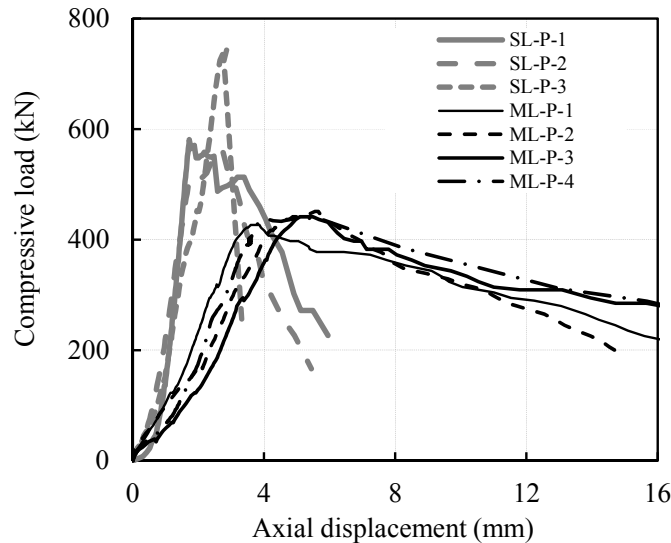


Figure 4.55 : Comparison of axial force-displacement curves of compression prisms.

Similar phenomenon was also observed in the studies of Binda et al. (2006) and Silva et al. (2008) where the drop in compressive strength in the case of three-leaf wallets

was 30 and 78% with respect to strength of external leaves. This huge difference between these two studies mainly stemmed from the failure modes achieved due to the height/thickness ratios of the tested specimens that were 1.55 and 3.70 for Binda et al. (2006) and Silva et al. (2008), respectively. Binda et al. (2006) reported no second order effects due to the detachment of external leaves (for specimens with lower strength Noto stone units) but reported several vertical cracks that diffused into the inner and outer leaves, whereas Silva et al. (2008) mentioned development of three hinges at supports and mid-height of the wallets. Similarity of the height/thickness ratio utilized in this study (height/thickness ratio of 1.67) addresses to a similar failure mode to that of Binda et al. (2006). In our tests, detachment of the leaves was not eye catching due to the low aspect ratio of the multi-leaf masonry prisms tested, particularly in the pre-peak region. However, the detachment was obvious, particularly after reaching the peak stress value, as also measured by H1, H2, H3 and H4 LVDTs that measured the transverse displacement values between the exterior leaves. These measurements addresses to a significant increase in the leaf detachment particularly after the achievement of the peak load as seen in Figure 4.56.

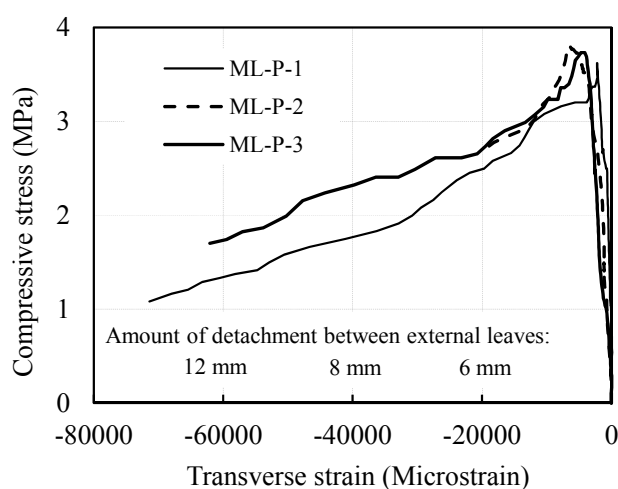


Figure 4.56 : Compressive stress-transverse deformation of multi-leaf prisms.

In summary, it can be concluded that the second order effects caused by the leaf detachment, extensive and unsymmetrical distribution of damage at the outer leaves and wide scatter of the mechanical characteristics of the utilized materials; led to the differences between the stress-strain characteristics of the single- and multi-leaf prisms (as also visualized in Figure 4.55). Considering the results of the compression tests reached in this study and the test results of other researchers with relatively

higher height/thickness ratios (whose data are compiled in Chapter 2), lower strength values can be foreseen for the larger shear compression test specimens.

Other characteristic values of the tested multi-leaf wallets, such as axial strain corresponding to compressive strength (ϵ_{wco}), echo travel time (t_s) and sound velocity (v_s) were obtained as 0.93%, 89 s and 3.36 km/s, respectively. The ϵ_{wco} axial strain corresponds to almost twice of the value obtained from the single-leaf compression tests.

Since the calculation of the Young's modulus (E_{wc}) is highly dependent on the effective cross-section area, the moduli obtained for areas of gross cross-section, only external leaves with narrow units and only external leaves with wider units are presented in Table 4.24. Similar to the compressive strength values and due to previously mentioned reasons, the Young's modulus values ranging between 576 and 867 MPa (depending on the effective cross-section assumption) were significantly less than that of the single-leaf prisms (2615 MPa). In average, the ratio of Young's modulus to compressive strength of multi-leaf prisms was approximately equal to 155, almost half of the 330 value that was obtained for the single-leaf walls.

Table 4.22 : Results of the single-leaf masonry compression tests.

Prism	f_{wc} (MPa)	ϵ_{wco} (%)	E_{wc} (MPa)	t_s (ms)	v_s (km/s)
SL-P-1	7.45	0.34	3400	37	3.53
SL-P-2	6.91	0.51	2366	37	3.52
SL-P-3	9.38	0.55	2079	38	3.47
Average	7.91	0.47	2615	37	3.51

Table 4.23 : Results of the multi-leaf masonry compression tests.

Prism Area	f_{wc} (MPa) (400×300)	f_{wc} (MPa) (400×260)	f_{wc} (MPa) (400×200)	ϵ_{wco} (%)	t_s (ms)	v_s (km/s)
ML-P-1	3.62	4.18	5.44	0.93	92	3.26
ML-P-2	3.81	4.42	5.75	0.99	89	3.36
ML-P-3	3.73	4.33	5.63	0.86	87	3.45
ML-P-4	3.66	4.22	5.49	0.94	N/A	N/A
Average	3.71	4.29	5.58	0.93	89	3.36

Table 4.24 : Young's moduli depending on the multi-leaf masonry effective areas.

Prism	E_{wc} (MPa)	E_{wc} (MPa)	E_{wc} (MPa)
Area	(400×300 mm)	(400×260 mm)	(400×200 mm)
ML-P-1	488	563	732
ML-P-2	656	762	991
ML-P-3	517	599	779
ML-P-4	642	741	964
Average	576	666	867

The curves given in Figure 4.57, which were obtained from LVDTs mounted on the stone units and rubble infill, depicts that the inner and outer leaves of the multi-leaf prisms did not go through similar deformations. In this figure, the compressive stresses in the y-axis are the average values over the gross cross-section.

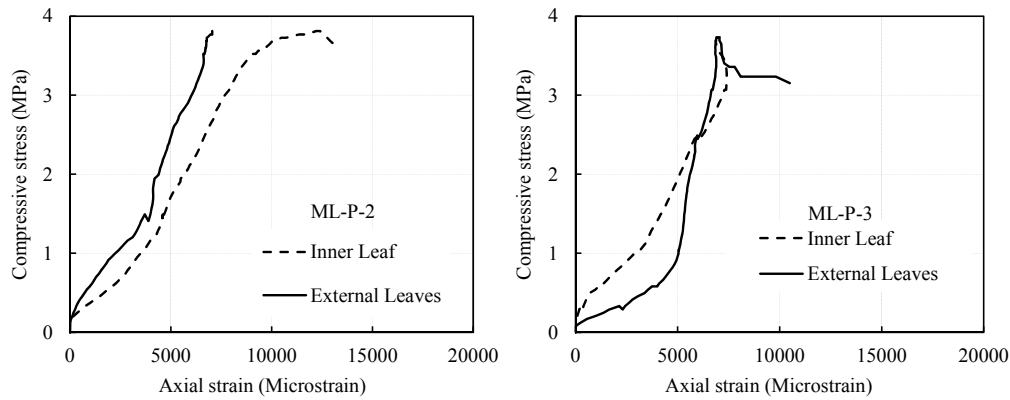


Figure 4.57 : Comparison of the stress-strain diagrams of inner and external leaves.

The results of the ultrasonic tests on external leaves in the transverse direction, which were practically carried on single stone units, were comparable with the results reported by Arıoğlu and Arıoğlu (1997, 1999 and 2005). They did measurements on Küfeki stone samples and obtained average velocities of 3.88, 5.01, 3.63 and 4.26 km/s for single unit compressive strengths of 33.2, 45.5, 21.1 and 32.9 MPa, respectively. It is worth to notice that the average velocity measured in this study was approximately 3.51 km/s for Küfeki stone units with an average compressive strength of 18.0 MPa. In the case of tri-leaf wallets, the average speed was measured as 3.36 km/s. The reduction of sound velocity from 3.51 (for single leaf specimens) to 3.36 km/s (for multi-leaf specimens) mainly stems from the introduction of highly heterogeneous inner core with several voids.

4.6.4 Prediction of multi-leaf masonry compressive strength

Compressive strength of multi-leaf composite walls can be predicted thorough utilization of multi-material approaches under different assumptions (Valluzzi et al. 2004). These approaches mainly depend on the compressive strength and volumetric contribution of the layers. Binda et al. (2006) mentions three equations with different assumptions.

The first equation assumes that the whole load is supported by the stiffer external leaves and neglects the positive contribution of inner core:

$$f_{wc} = \frac{2t_e}{2t_e + t_i} f_e \quad (4.15)$$

The second equation considers the contribution of inner core and the external leaves according to their cross-section area ratios:

$$f_{wc} = \frac{2t_e}{2t_e + t_i} f_e + \frac{t_i}{2t_e + t_i} f_i \quad (4.16)$$

Finally, the third approach introduces corrective factors, which take into account the influence of the mutual interaction between the external layers and the internal core in the global behaviour of the wall (Valluzzi et al., 2004).

$$f_{wc} = \frac{2t_e}{2t_e + t_i} \theta_e f_e + \frac{t_i}{2t_e + t_i} \theta_i f_i \quad (4.17)$$

In the above equations, f_e and f_i are the uniaxial compressive strengths (7.9 and 1.68 MPa) while t_e and t_i are the thicknesses of the external and inner leaves (100 mm for both leaves), respectively. The parameters θ_e and θ_i are the correction factors for the outer and inner leaves, which aim to reflect the stress states in these material layers. According to Egermann and Newald-Burg (1994), the external layers apply lateral confinement to the inner core and the inner core applies lateral thrust to the external leaves. Consequently, the external layers are not only under compression loading but also subjected to out-of-plane bending moments, so that a θ_e parameter smaller than unity (proposed as 0.75) is possible. However, the internal leaf is under a multiaxial state of compressive stresses, leading to higher compressive strength

with a θ_i factor that is larger than unity (proposed as 1.3). Additionally, as also observed during the multi-leaf compression tests of this study, the detachment of the exterior leaves in the transverse direction gains haste in the vicinity of the peak load. Combined with the out-of-plane bending effects, this detachment also justifies the reduction of the compressive strength of the external leaves.

The compressive strength values predicted by these equations are presented in Table 4.25, together with the experimental strength values calculated for different effective cross-section area assumptions. During the calculations, average compressive strengths obtained from single-leaf masonry prism and rubble cylinder (at 150 days) compression tests were utilized. Accordingly, if it is assumed that the whole axial load is supported by the narrower units of the external leaves (a cross-section of 400×200 mm), the strength values predicted by Eq. (4.15) approximates the experimental value (5.27 MPa versus 5.58 MPa). However, under the assumption that the area of the external leaves is constituted by the wider stone units leading to a cross-section of 400×260 mm, it can be seen that Eq. (4.15) overestimate the strength.

The contribution of the inner core considered in Eq. (4.16) results with an overestimation of the strength obtained by using the gross cross-section area (400×300 mm), whereas Eq. (4.17) more successfully predicts the experimental value (4.41 MPa versus 3.71 MPa). It should be noted that, Eqs. (4.16) and (4.17) can further approach the experimental compressive strength, in case a further reduction to the compressive strength of rubble masonry can be considered due to size and shape effects.

Table 4.25 : Predicted compressive strength values for tested multi-leaf wallets

Wallet	f_{wc} (MPa) (400×300mm)	f_{wc} (MPa) (400×260mm)	f_{wc} (MPa) (400×200mm)	Eq. (4.15) predicted f_{wc} (MPa)	Eq. (4.16) predicted f_{wc} (MPa)	Eq. (4.17) predicted f_{wc} (MPa)
ML-P-1	3.62	4.18	5.44			
ML-P-2	3.81	4.42	5.75	5.27	5.83	4.41
ML-P-3	3.73	4.33	5.63			
ML-P-4	3.66	4.22	5.49			
Average	3.71	4.29	5.58	5.27	5.83	4.41

5. RESULTS OF THE SHEAR COMPRESSION TESTS

The experimental study carried out to investigate the behaviour of stone masonry walls provided extensive data on the failure mode, lateral shear load capacity, ductility and damage characteristics. In this chapter, initially, the behaviour and failure modes of the walls tested under combined shear and compression are outlined. Then, the numerical data collected from the performed experiments is presented. The observed damage evolution and obtained diagrams are evaluated and compared with respect to the investigated parameters such as axial stress level, presence of reinforcements and wall typology.

It is important to note that, in this study, the single-leaf term is used as a generic classification, which is used to denote the shear compression test specimens constructed with single material (Küfeki stone units). Although these walls consist of two ashlar leaves they do not inhibit a rubble core and no transverse connection is provided between them.

5.1 Behaviour and Failure of Walls

Since the details of the damage evolution observed and measured during the tests are presented in Appendix C, repetition will be avoided and only the general results will be discussed here. In general, the walls exhibited a shear dominated failure mode, which was a mixture of the shear and flexure type modes. It should be noted that, herein, the flexure type of failure term is used to cover not only the results of bending effects but also overturning of a certain portion of the wall (generally the upper diagonal half) around the toe of the respective loading direction.

5.1.1 Effect of axial stress level

As aforementioned in Chapter 3, in order to investigate the effect of axial stress level, four identical walls were tested under combined axial load and cyclic shear loads. Axial loads were calculated considering the desired axial stress levels and the gross cross-section area of the walls. Namely, these axial stress levels were 0.25,

0.50, 0.75 and 1.00 MPa for M-25-C, M-50-C, M-75-C and M-100-C walls, respectively. The selected axial pre-compression stress values were decided by considering the finite element analysis results of a number of existing historical structures (Appendix A).

Though all specimens exhibited shear type of failure, distinctive differences in the behaviour of the walls could be observed during the tests. The damage distributions of M-25-C, M-50-C, M-75-C and M-100-C walls, observed after the shear compression tests, can be seen in Figure 5.1. In this figure, the (+) sign corresponds to the pushing direction whereas the (-) sign indicates the pulling direction. Details of crack distribution obtained after each displacement level can be found in Appendix C.

The damage evolution generally started with vertical cracks passing through the stone units that could emerge even at small lateral displacement levels. As a result of stress concentrations around the imperfections between the units, these cracks could develop not only around the toes where the compressive stresses were higher, but also at random locations among the wall surfaces. The random distribution of these early vertical cracks was more pronounced in the case of walls with higher axial stresses.

Except the M-100-C specimen, with the highest level of axial stress considered in this study, horizontal flexural cracks were observed between the initial stone courses at the lower half of the specimens. Since the dry bed joints had zero tensile strength in the vertical direction, only the tensile strength of the rubble masonry resisted the tensile stresses at the tension side of the neutral axis. The translation of the neutral axis with the increase in the lateral displacement apparently caused a reduction in the effective shear resisting area, particularly for specimens with relatively lower axial stress levels.

Further increase of the lateral displacement caused the arising of shear damage in the form of diagonal cracks and vertical cracks aligned on a diagonal band. As also seen in Figure 5.1, due to the low tensile strength of the lime stone used in the specimen production and the restraining effects of the cramps interconnecting the head joints of adjacent units, these cracks were crossing the stone units rather than the head joints. Consequently, formation of stepped cracks, that were generally observed and reported in the literature, was prevented.

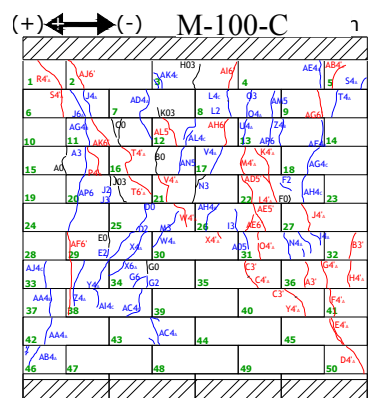
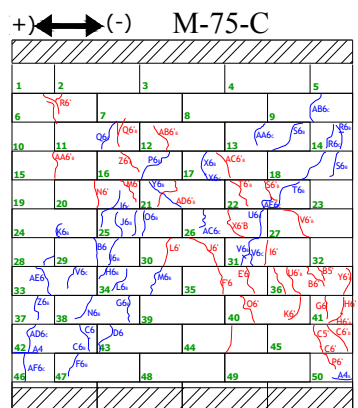
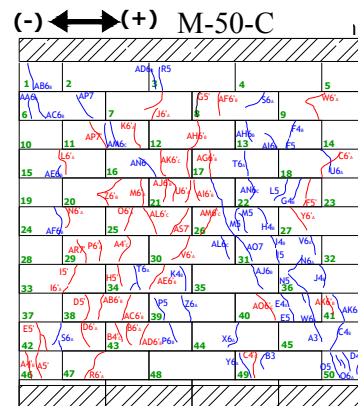
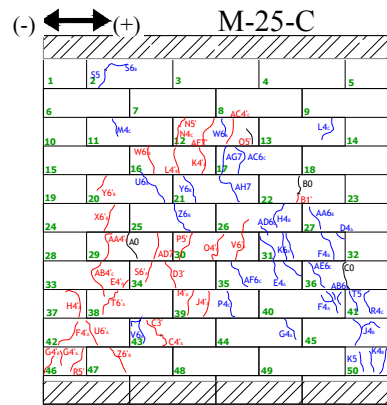


Figure 5.1 : Cracking patterns of M-25-C, M-50-C, M-75-C and M-100-C walls.

During the tests, it was observed that a great majority of the vertical and diagonal cracks crossing the stone units were generally, either at mid-length of the units or near the anchorage sockets of the cramps. Thanks to the existence of the cramps, that were transferring tension forces from one unit to adjacent one at singular points, diagonal cracks were wide spread across the wall faces. It should be noted that, number of cracks was higher in the case of specimens with relatively higher axial stresses, but crack widths were relatively smaller. This may be attributed to the stress concentrations at the imperfections between the bed joints.

After opening of the wide spread diagonal cracks and failure of some of the cramp anchors (due to cracking of stone units), a rocking-type mechanism took place around the lower corners of the walls (toes of the walls). Specimens were separated into two approximately diagonal halves for pushing and pulling directions. The concentrated compressive stresses around the toes caused crushing of these zones. In M-25-C and M-50-C walls, where the vertical compression stress levels were rather small, sliding at some of the stone courses could also be observed.

As indicated above, although shear failure mode was dominating the behaviour different damage patterns could be observed within a single test. Thereby, it was hard to classify the failure modes of the walls under a distinct mode type. Consequently, the failure modes classified and presented in Table 5.1 should be regarded as modes within the vicinity of shear failure mode, with varying amount of flexure/rocking behaviour. In this table, the influence level of flexure/rocking behaviour is indicatively expressed with “+” signs given in the parenthesis. Accordingly, the more the axial load was, the flexure/rocking mode was less eye catching.

Table 5.1 : Observed failure modes of M-25-C, M-50-C, M-75-C and M-100-C.

Specimen	Failure mode
M-25-C	Shear (Flexure+++)
M-50-C	Shear (Flexure++)
M-75-C	Shear (Flexure+)
M-100-C	Shear

5.1.2 Effect of existence of cramps and pins

One of the distinctive characteristics of the investigated wall type is the utilization of reinforcing elements in order to enhance the behaviour of the dry joints. For this purpose, the actual singular reinforcements, namely cramps and pins, were imitated

in this study. In order to investigate the effects of the existence of the cramps and pins, in addition to the cramp reinforced M-50-C wall, one wall was constructed without any reinforcement (Wall M-50) and one was constructed by using cramps and pins (M-50-CP). Pre-compression stress level of M-50, M-50-C M-50-CP specimens was 0.50 MPa. Cracks patterns of these three specimens are presented in Figure 5.2.

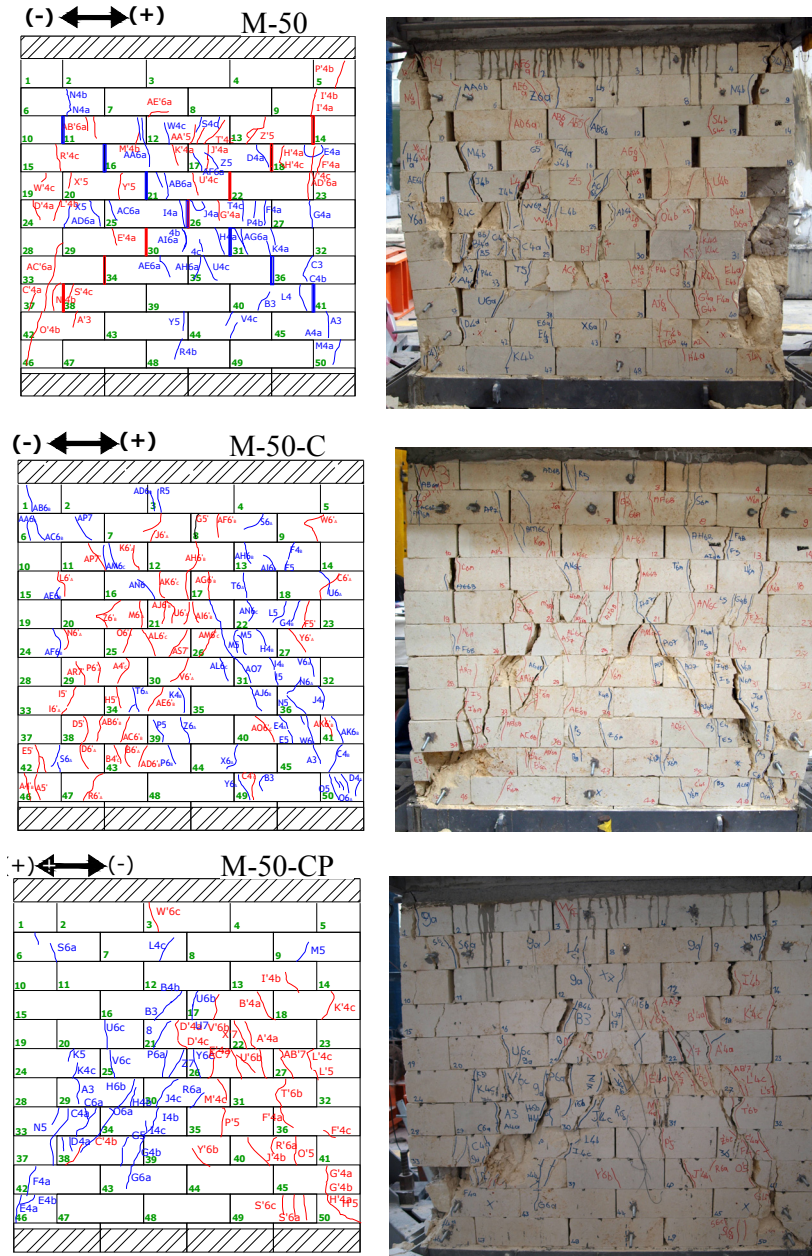


Figure 5.2 : Cracking patterns of M-50, M-50-C and M-50-CP walls.

Once these crack patterns are investigated, it is eye catching that in the case of unreinforced masonry (URM) wall M-50, stepped diagonal cracks passing through

the joints could develop. Since there was not any reinforcement that can limit the deformations, damage was concentrated at these diagonal stepped cracks as soon as they appeared. However, in the case of reinforced masonry walls, cracks were scattered over the wall faces. A great majority of the cracks were particularly crossing near the cramp and pin anchor points where the stone unit cross-sections were disturbed with drilled holes and stress concentrations were high. Introduction of the cramps and pins delayed the formation of the diagonal cracks. Moreover, maximum crack widths at target drift levels were smaller than the unreinforced masonry case.

The introduction of pins did not really affect the evolution of the damage. However, when the damage patterns at the same drift ratios are compared for M-50-C and M-50-CP walls; one can notice that cracks widths are slightly lower for the M-50-CP wall with cramps and pins. The maximum crack widths at target displacement levels can be found in Appendix C. In addition, pins saved the integrity of the M-50-CP wall even at larger displacement levels that M-50-C wall could not reach.

Similar comparisons can be made between the S-50-C and S-50-CP specimens. It might be recalled that these walls were tested to investigate the contribution of the rubble masonry infill (infilled between the ashlar external leaves) to the in-plane behaviour of the multi-leaf walls. S-50-C specimen had cramps between the adjacent stone units, whereas, S-50-CP specimen had cramps and pins. Since there were no rubble masonry infills in these specimens, the contribution of the pins to the overall behaviour could be better observed. Accordingly, when the damage patterns for these specimens are compared (presented in Figure 5.3), one can note that the wall with pins (S-50-CP) experienced more but narrower cracks. The number of diagonal cracks was less in the case of wall S-50-C but their widths were considerably larger than the cracks widths of the S-50-CP wall.

No matter how and to what extent the diagonal cracking occurred; M-50, M-50-C, M-50-CP, S-50-C and S-50-CP specimens with the same pre-compression stress level (0.50 MPa) followed a similar shear dominated damage evolution and failure. As also explained in the case of M-50-C specimen, the damage development of these walls began with an initial flexural behaviour that caused horizontal cracks on the tension side (heel) and vertical cracks in the compressed (toe) areas. At further displacements around the peak strength of the walls, diagonal cracks that were bold

specimens came to failure due to the development of toe crushing as a result of rocking behaviour.

When the crack patterns of M-50-C and S-50-C walls are compared, it can be observed that the specimen with rubble infill had more scattered cracks with smaller crack widths, Figure 5.4. Somehow, the existence of the low strength rubble masonry enhanced the integrity of the wall, so that stresses could be better distributed among the stone units. Eye catching sliding of the stone units could also be observed in the S-50-C specimen.

Interestingly, the crack patterns of the second comparison set that consists of walls M-50-CP and S-50-CP did not reflect the same observations. The damage patterns of these two cramp and pin reinforced masonry walls are presented in Figure 5.5. Both walls had similar crack distribution patterns and crack widths. This contradiction with the first comparison set (M-50-C and S-50-C walls) might be explained with the existence of pins in the second comparison set (M-50-CP and S-50-CP walls), that already enhanced the integrity of the wall.

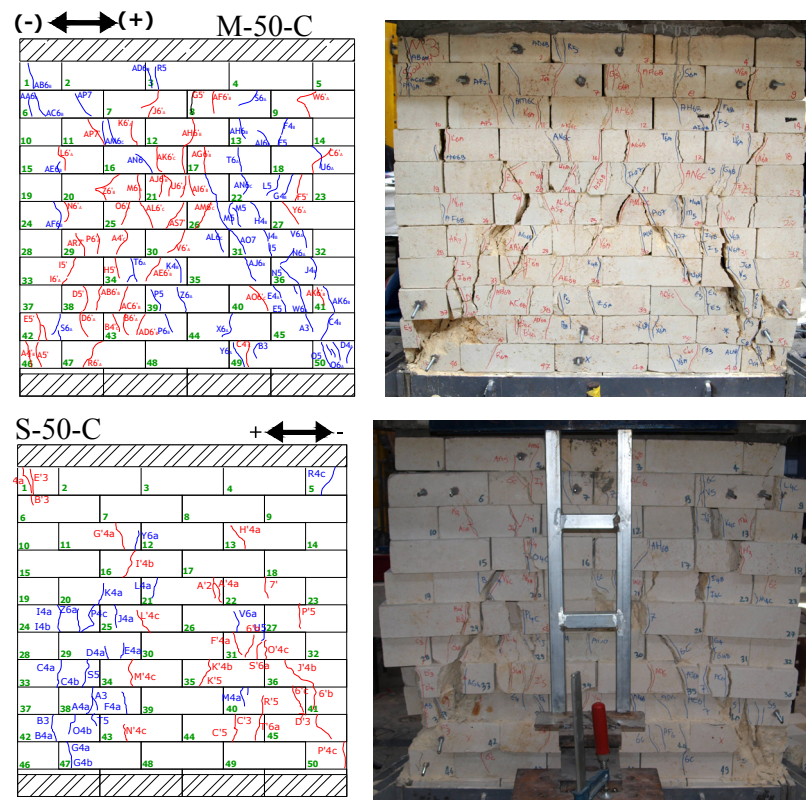


Figure 5.4 : Cracking patterns of M-50-C and S-50-C walls.

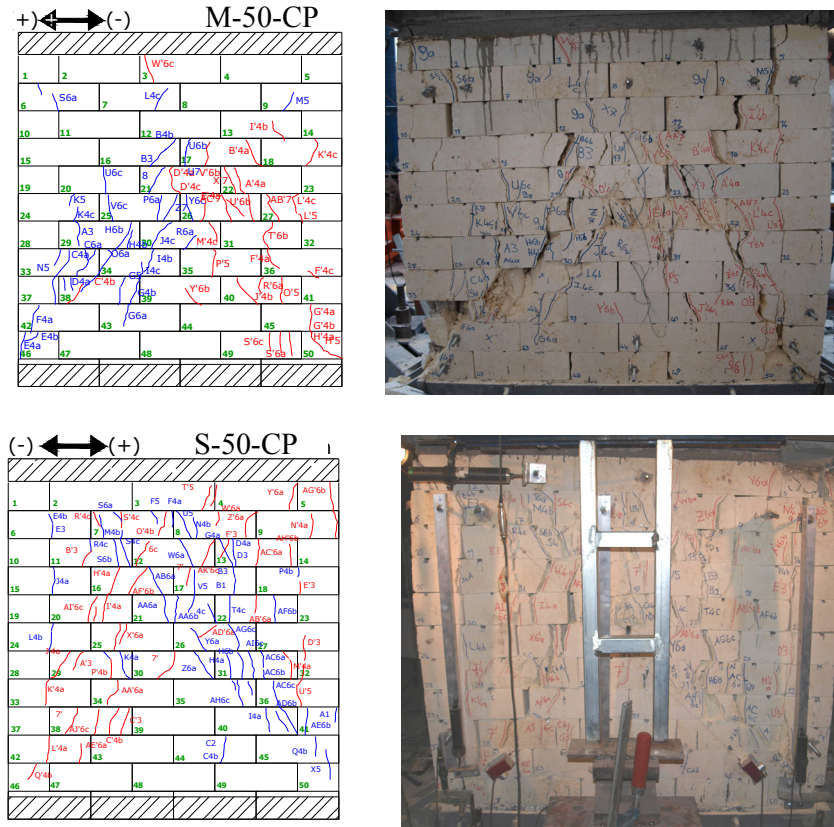


Figure 5.5 : Cracking patterns of M-50-CP and S-50-CP walls.

5.2 Comparison of Hysteresis Diagrams

In addition to observations made during the tests, numerical data was also collected from several measurement devices located at different critical points of the walls. The positions and aims of these instruments are explained in Chapter 3. In this section, the behaviour of the walls, grouped in comparison subsets with respect to investigated tests parameters, will be evaluated quantitatively. Particularly the lateral load-displacement curves will be discussed and critical points on these diagrams will be tabulated. It should be noted that the top displacement values used to plot these curves are averages of top displacement values measured for both external leaves of the walls.

5.2.1 Effect of axial stress level

Lateral load-displacement diagrams of four specimens, tested to investigate the effect of axial stress level on the in-plane behaviour, are presented in Figure 5.6. Applied pre-compression stress levels were 0.25, 0.50, 0.75 and 1.00 MPa for walls M-25-C, M-50-C, M-75-C and M-100-C, respectively. Data points corresponding to first

flexural cracking ($H_{f,cr}$), first diagonal cracking ($H_{dia,cr}$), maximum lateral resistance (H_{max}), 20% strength loss ($H_{0.8max}$) and ultimate displacement ($H_{d,max}$), are also marked in Figure 5.6, where available. The upper horizontal axis of these diagrams gives drift ratio values that correspond to displacement values of the lower horizontal axis.

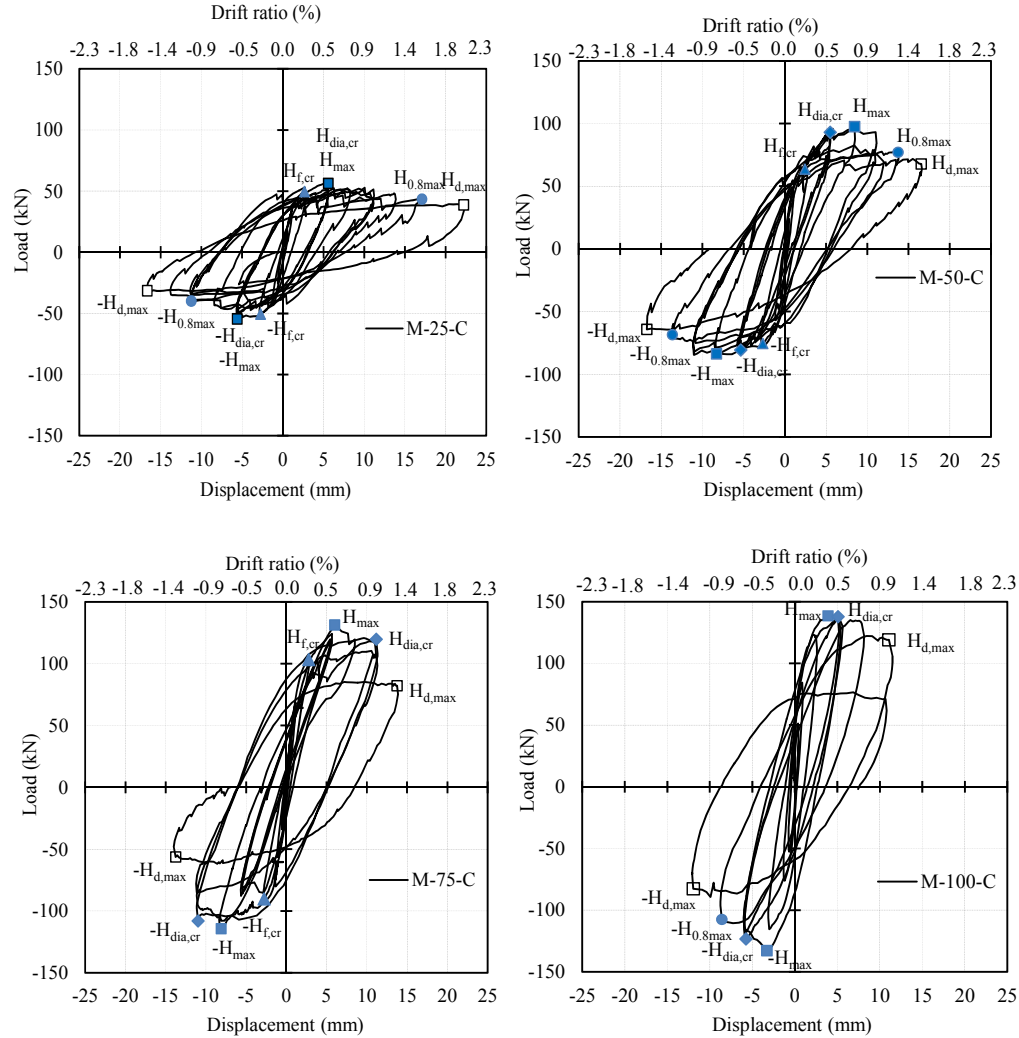


Figure 5.6 : Force-displacement diagrams for M-25-C, M-50-C, M-75-C and M-100-C.

From the lateral load-displacement hysteresis diagrams, it can be observed that, at small displacement levels (i.e. at target drift ratios less than 0.50%) the walls exhibited elastic behaviour and showed negligible residual displacement upon the reversal of the applied horizontal load. With the increase in the achieved lateral displacement, the walls exhibited wider and fatter hysteretic loops and reached the peak load values. Beyond the peak load stage, after which the diagonal cracks and

toe crushing develops, the walls showed further increase in widening of the hysteretic loops and residual displacements. With the substantial loss in the horizontal load capacity and due to formation of extensive damage, the specimens reached their ultimate stages.

The response of the walls in the pushing and pulling directions are not perfectly symmetric, especially for the M-75-C wall, but reasonably close to each other. The degradation in strength and stiffness is generally smooth until the total spalling of the toes.

The obtained hysteresis loops show that the walls exhibited a highly nonlinear response with significant residual displacements after each load reversal. The hysteresis diagrams also back the identification of the failure modes of the specimens as shear dominated, since the hysteresis cycles are not “S” shaped (as generally observed for rocking mechanism, such as the hysteresis reported by Vasconcelos and Lourenço (2009)) and significant amount of energy is dissipated.

It should be noted that the tests were resumed until formation of extensive damage (namely $H_{d,max}$ point), which could cause safety problems during the implementation of the tests. However, in order to define a uniform failure condition, during the interpretation of experimental results of this study, the ultimate state will be assumed as the critical state that corresponds to 20% strength degradation ($H_{0.8max}$ point on the hysteresis diagrams). Assumption of 20% strength loss for ultimate state has also been done by other researchers such as Frumento et al. (2009) and Magenes and Calvi (1997). Envelope curves derived from the hysteresis diagrams by considering this ultimate state definition are presented in Figures 5.7 and 5.8 for the pushing and pulling directions, respectively.

Comparison of lateral load-displacement envelope curves clearly reveals that the variation of the pre-compression stress level has remarkable effect on the behaviour. When submitted to low and moderate axial compression stresses, as in the case of M-25-C and M-50-C walls, larger displacement capacities can be achieved with lower lateral strengths. On the other hand, walls with higher axial pre-compression stresses (walls M-75-C and M-100-C) may reach higher lateral force capacities with less displacement capacities, if the axial stress level is not high enough to cause compression failure or out-of-plane failure. It should be noted that, the only

contradiction to these tendencies are obtained for the pulling direction of the M-25-C wall, which exhibited a premature failure after the crushing of its toe region in the pulling direction. One main reason of this unexpected earlier strength loss in the pulling direction was the extensive spalling and out of plane failure of the external leaves in the compressed zone. Another clear comparison can be made for the stiffness values of the walls. The stiffness apparently increases as long as the pre-compression stress level increases.

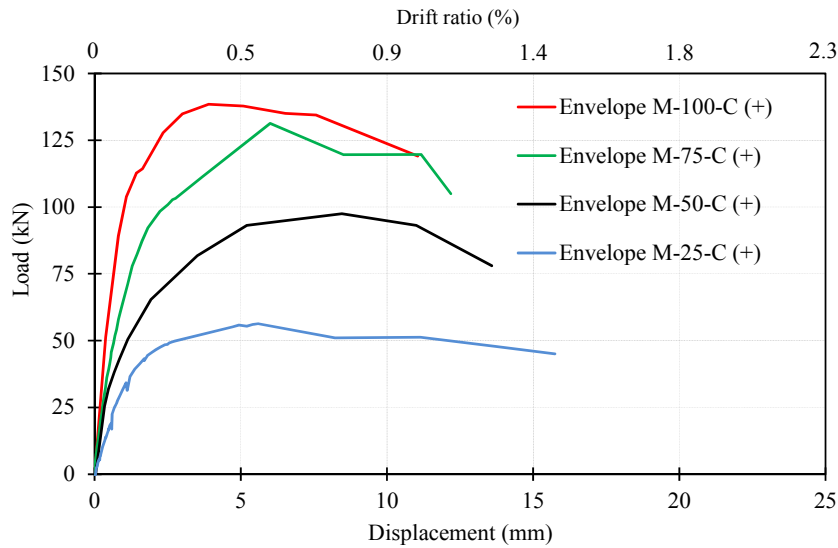


Figure 5.7 : Envelope curves for M-25-C, M-50-C, M-75-C and M-100-C (pushing).

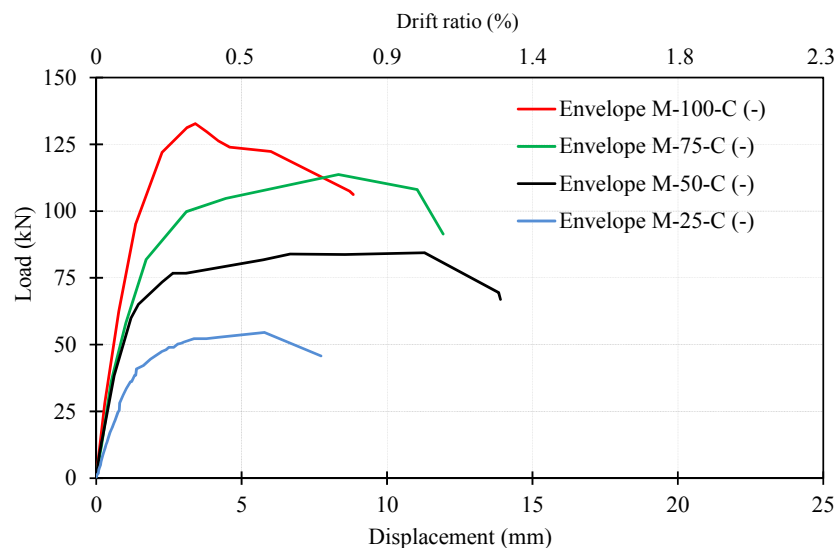


Figure 5.8 : Envelope curves for M-25-C, M-50-C, M-75-C and M-100-C (pulling).

The load and displacement data describing the critical states of the walls are tabulated in Table 5.2. In this table, $H_{f,cr}$, $DR_{f,cr}$, $H_{dia,cr}$, $DR_{dia,cr}$, H_{max} , DR_{Hmax} ,

$H_{0.8\max}$, $DR_{0.8\max}$ and $H_{d,\max}$, $DR_{d,\max}$ columns correspond to lateral load and drift ratio values of first flexural cracking, first diagonal cracking, maximum lateral resistance, 20% strength loss and ultimate displacement critical states. The $H_{0.8\max}$ value marked with a “*” correspond to a state at which strength loss is less than but close to 20%. It should be underlined that these values generally belong to targeted cycle peaks at which observations were done on the specimens, so that the cracks could be identified. Particularly, in the case of first flexural cracking state it is highly probable that the cracks appeared slightly earlier than the displacement peaks at which they were identified and noted.

Table 5.2 : Load and drift ratio data of M-25-C, M-50-C, M-75-C and M-100-C.

Wall	$H_{f,cr}$	$DR_{f,cr}$	$H_{dia,cr}$	$DR_{dia,cr}$	$H_{\max}$	$DR_{H\max}$	$H_{0.8\max}$	$DR_{0.8\max}$	$H_{d,\max}$	$DR_{d,\max}$
	(kN)	(%)	(kN)	(%)	(kN)	(%)	(kN)	(%)	(kN)	(%)
M-25-C (+)	49.9	0.25	56.3	0.50	56.3	0.50	45.0	1.43	38.8	2.00
M-25-C (-)	51.0	0.25	54.5	0.50	54.5	0.50	43.6	0.80	31.8	1.50
M-50-C (+)	64.5	0.25	81.0	0.50	97.5	0.75	77.1	1.25	67.7	1.50
M-50-C (-)	74.8	0.25	80.4	0.50	83.7	0.75	66.5	1.25	64.1	1.50
M-75-C (+)	103.3	0.25	116.3	1.00	131.3	0.55	105.0	1.10	82.1	1.25
M-75-C (-)	90.2	0.25	108.1	1.00	114.3	0.75	91.4	1.10	56.2	1.25
M-100-C (+)	-	-	136.6	0.50	138.5	0.35	110.8*	1.00	71.1	1.00
M-100-C (-)	-	-	116.9	0.50	132.8	0.30	106.8	0.80	83.0	1.00

The load values for the first flexural cracking critical state ($H_{f,cr}$) tend to increase with the increase of the axial stress level. In average, $H_{f,cr}$ loads correspond to 83% of the maximum load capacities in pushing and pulling directions of specimens as given in Table 5.3. However, as mentioned before, the cracks could only be visible and so that identified at the peak displacements of the 0.25% drift ratio cycles. Consequently, a smaller $H_{f,cr}/H_{\max}$ ratio in the order of 70%, as also mentioned by Magenes and Calvi (1997) and Frumento et al. (2009), seems to be acceptable. Investigation of the points at which the stiffness reduces considerably in the load-displacement curves also backs this suggestion.

The first diagonal cracks generally appear near the 0.50% drift ratio, except the M-75-C wall where they appeared at 1.00% drift ratio. They emerged either before, at or after the peak load capacities so that the average $H_{dia,cr}/H_{\max}$ ratio becomes 94%, Table 5.3. Through a force-based interpretation, it can be concluded that increase in axial compression stress level delays the initiation of cracking until larger lateral force is achieved. Voon and Ingham (2006) explains this from a perspective of

principal stresses: a larger lateral force is required to exceed the compressive field resulting from the larger axial load. This compressive field must first be overcome before cracking can initiate.

Tested walls reach their maximum resistance values ($H_{\max}$) between 0.30 and 0.70% drift ratios ($DR_{H_{\max}}$). Assumed ultimate state, which corresponds to 20% strength loss ($H_{0.8\max}$), is generally achieved between 0.80 and 1.50% drift ratios. Drift ratios corresponding to 20% strength loss ($DR_{0.8\max}$) tend to decrease with the increase of the pre-compression stress level. This means that a reduction in the deformation capacity is found for the walls under high axial stress levels. Ultimate displacements at which the tests were ended due to safety concerns, are obtained between 1.00 and 2.00% drift ratios ($DR_{d,\max}$). As expected, $DR_{d,\max}$ values are also inversely proportional with the applied axial stresses.

Typical diagonal deformation vs. lateral displacement hysteresis diagrams of M-25-C, M-50-C, M-75-C and M-100-C walls, obtained by using the diagonally oriented LVDTs, are presented in Figure 5.9. In these diagrams, significant plastic strains can be identified after formation of the diagonal cracks and initiation of the crushing. Since, walls with higher pre-compression stress levels tend to fail at lower drift ratios, total diagonal deformations measured for these walls also become lower.

Table 5.3 : $H_{f,cr}/H_{\max}$ and $H_{dia,cr}/H_{\max}$ ratios (M-25-C, M-50-C, M-75-C and M-100-C).

Wall	$H_{f,cr}/H_{\max}$	$H_{dia,cr}/H_{\max}$
	(%)	(%)
M-25-C (+)	89	100
M-25-C (-)	94	100
M-50-C (+)	66	83
M-50-C (-)	89	96
M-75-C (+)	79	89
M-75-C (-)	79	95
M-100-C (+)	-	99
M-100-C (-)	-	88
Average	83	94
Stdev	10	6
COV	12	7

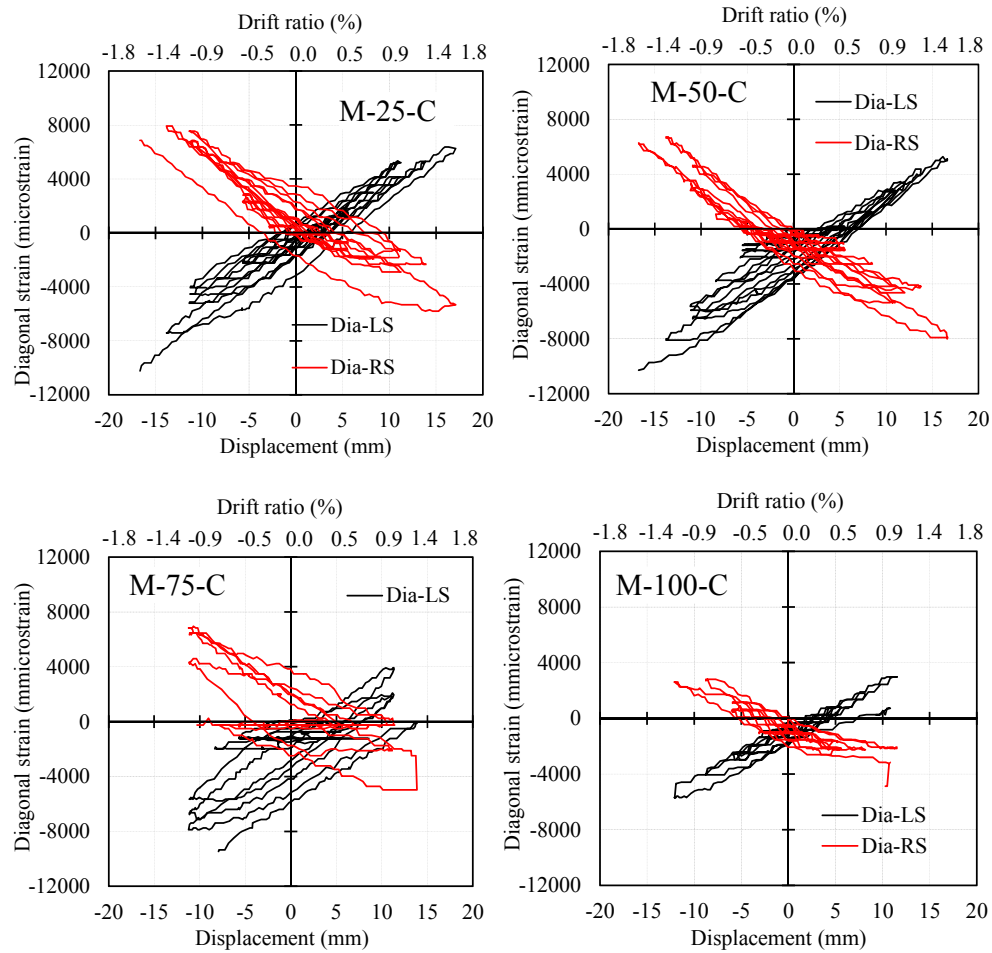


Figure 5.9 : Diagonal strain-lateral displacement hysteresis diagrams for M-25-C, M-50-C, M-75-C and M-100-C.

Variation of vertical deformations near the Eastern side of the walls with respect to lateral displacement is presented in Figure 5.10. In these diagrams, positive sign on the y-axis corresponds to compaction, whereas negative sign corresponds to tension. Positive sign on the x-axis means pushing of the specimen so that the LVDTs near the Eastern side should read compaction and vice versa. Bearing these sign conventions in mind, it is apparent that all specimens exhibited a tendency for compaction of the walls. Though flexural cracks were clearly observed in all specimens except M-100-C wall with the highest pre-compression stress level, it was only possible in M-25-C wall (specimen with the lowest axial stress level) to measure elongation in the pulling direction of the displacement reversals. In other walls, the measured crack widths measured in the gage length were not enough to overcome the residual compression deformations.

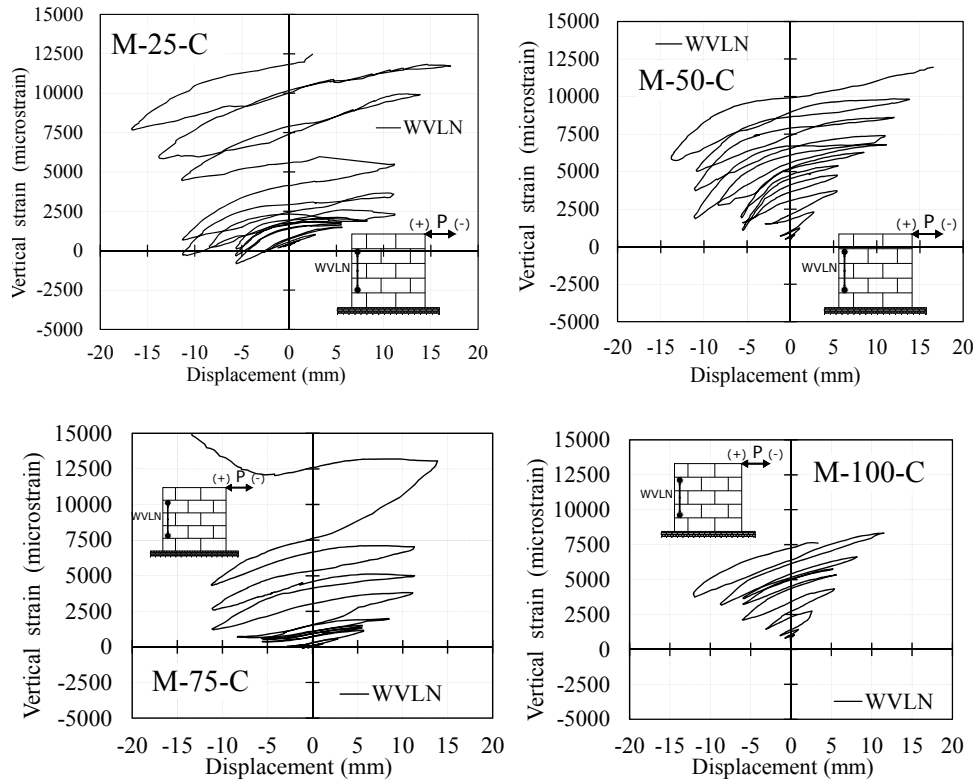


Figure 5.10 : Variation of vertical deformations close to eastern side with respect to lateral displacements.

A typical relation between the vertical displacements measured by LVDTs at the tips of the upper bond beam and the lateral displacement is plotted in Figure 5.11 (M-25-C wall). Diagrams of the other specimens were quite similar to this one. As a result of significant rotation, vertical displacements increase to remarkable values as the lateral displacements increase. The diagrams plotted for both tips of the bond beam are almost symmetric and the general tendency for compaction (negative sign in the vertical axis of Figure 5.11) can be observed. The shortening of the wall observed from the vertical displacement of the upper bond beam also supports the observations done for the vertical displacement measurements indicated in Figure. 5.10. Hysteresis gets wider especially after the diagonal cracking of the wall. It should be noted that the vertical displacements in Figure 5.11 reaches to an asymptote of -10 mm, since the values started to reach the gauge lengths of the LVDTs that were limited to this value.

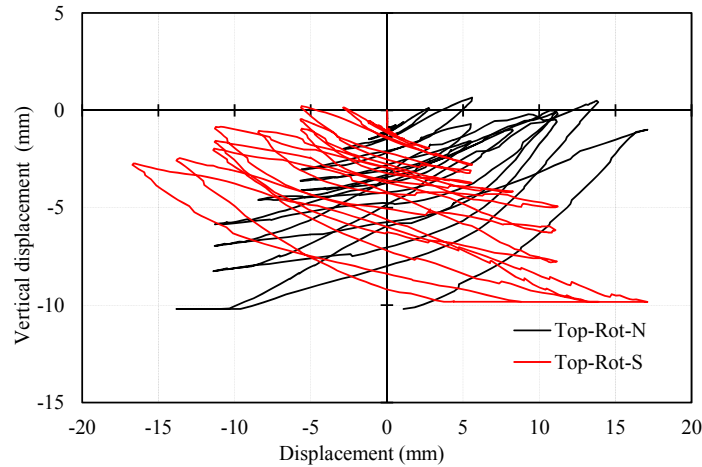


Figure 5.11 : Variation of vertical displacements at bond beam tips, M-25-C.

As observed during the tests, the complete failure of the walls occurs with the rocking of the upper diagonal halves of the walls formed after widening of the diagonal cracks. The crushing of the compressed toes during the rocking behaviour could also be identified by measuring the deformations at these regions. Vertical deformation-lateral displacement diagrams of M-25-C and M-100-C walls measured at the Western toe are presented in Figure 5.12. These diagrams show that, regardless of the acted pre-compression stress level, up to 5.50 mm lateral displacement (0.50% drift ratio) these zones are subjected to small cyclic compression and tension deformations. However, beginning from the 0.50% drift ratio (5.50 mm displacement) vertical compressive deformations increase significantly mainly due to crushing of the stone units at the toes. In the case of M-25-C wall with lower pre-compression axial stress, a portion of the vertical compressive strain can be reduced during displacement cycles. However, M-100-C wall with higher axial stress exhibits a continuously increasing compressive deformation at the toe region.

Separation of the inner and outer leaves of the wall was measured by two LVDTs positioned at mid-heights of the Eastern and Western faces of the specimen. As clearly seen from the transverse displacement versus lateral displacement diagram (Figure 5.13) obtained for the Western side of M-25-C, beginning from the initial loading cycles separation between the outer and inner leaves increased successively. Similar diagrams for out-of-plane bulging of the external leaves were obtained for all walls. However, none of the specimens, even the specimen M-100-C with the highest axial stress level, failed due to separation of external leaves.

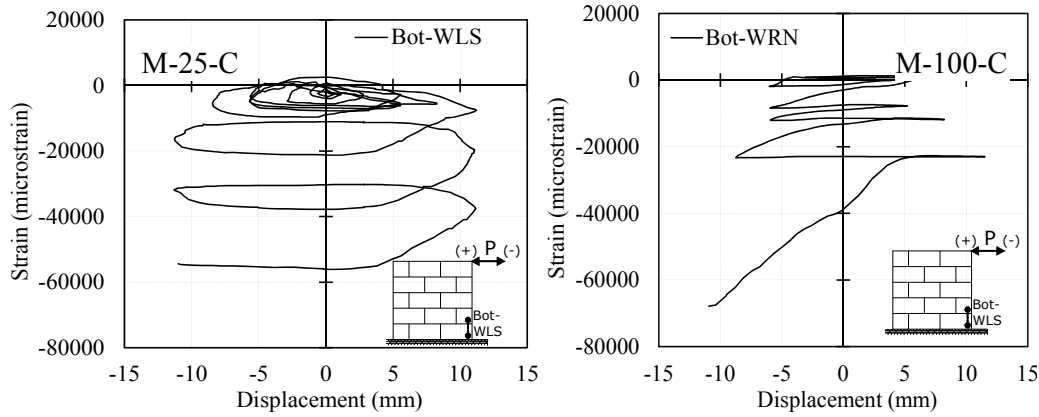


Figure 5.12 : Vertical deformations at the Western toes of M-25-C and M-100-C.

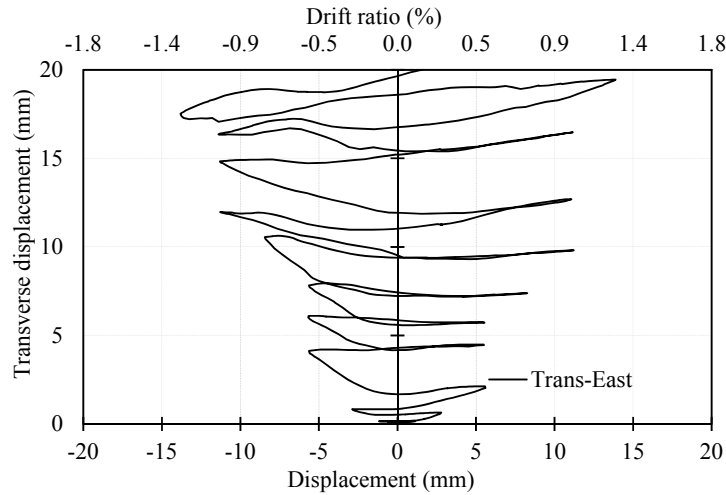


Figure 5.13 : Variation of transverse displacement between exterior leaves, M-25-C.

None of the cramps ruptured during the tests. Three of the cramps on the Northern leaves of the walls were instrumented with strain gauges. One strain gauge was glued to the upper side at the mid-length of the cramps (in the longitudinal direction), while another one was used for the lower side. Measurements logged for the investigated specimens show that compressive and tensile strains on the upper and lower sides of the cramps could reach a maximum value of 2200 microstrain, which was very close to the yield point of the used steel members, Figure 5.14. Interestingly, the strain diagrams turned out to be perfectly symmetric, which means that when upper sides of the cramps were in compression, the lower ones were in tension and vice versa. Consequently, if the average of these symmetric strains is calculated for each instrumented cramp, average elongation values turn out to be very small (less than 500 microstrain). This strain distribution points to a bending behavior for the cramps.

In most cases, the stone units crack at or near the cramp anchors so that transfer of forces is not possible anymore. Consequently, in most cases, the yielding point of the steel members cannot be exceeded.

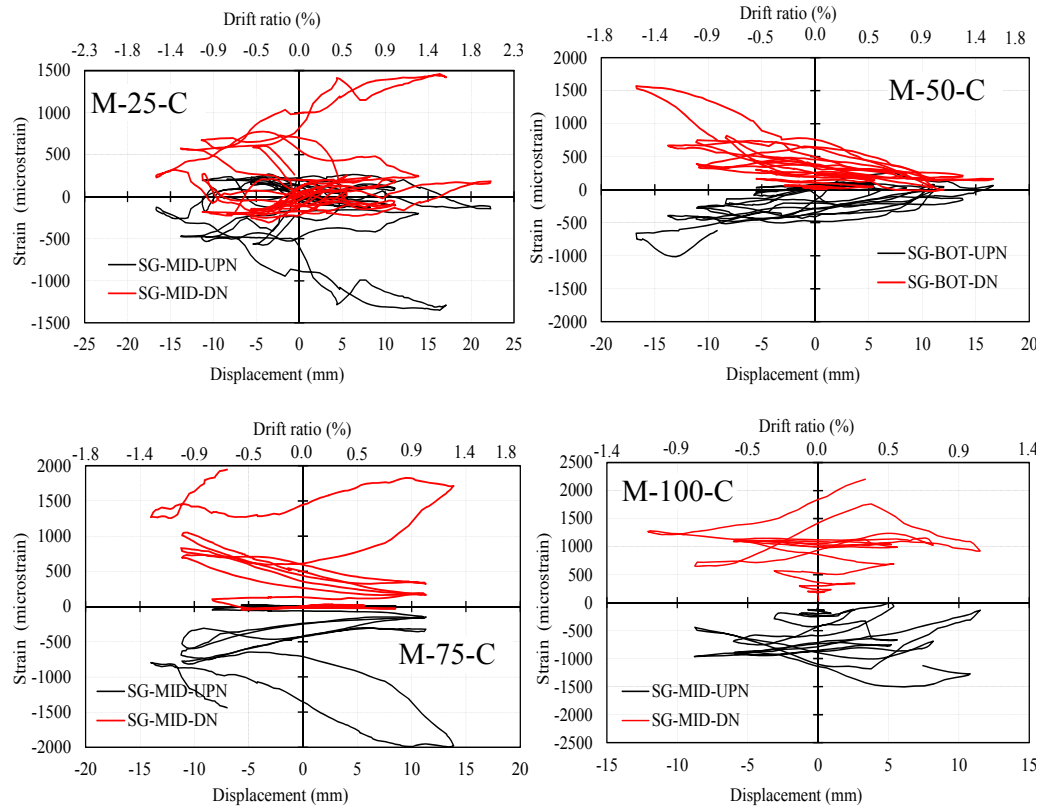


Figure 5.14 : Strain versus lateral displacement curves for instrumented cramps.

5.2.2 Effect of existence of cramps and pins

As mentioned earlier, in order to investigate the effect of the existence of the cramps and pins on the behaviour of the walls, behaviour of M-50-C wall with cramps was compared with unreinforced M-50 and cramp and pin reinforced M-50-CP walls. Axial stress level of these walls was 0.50 MPa. Aiming to disregard the additional contribution of rubble masonry infill, ashlar masonry walls S-50-C (with cramps) and S-50-CP (with cramps and pins) were also tested. Lateral load-displacement diagrams of M-50-C M-50 and M-50-CP specimens are presented in Figure 5.15, whereas the diagrams for S-50-C and S-50-CP are given in Figure 5.16. Data points corresponding to first flexural cracking ($H_{f,cr}$), first diagonal cracking ($H_{dia,cr}$), maximum lateral resistance (H_{max}), 20% strength loss ($H_{0.8max}$) and ultimate displacement ($H_{d,max}$), are also marked in Figures 5.15 and 5.16, where available.

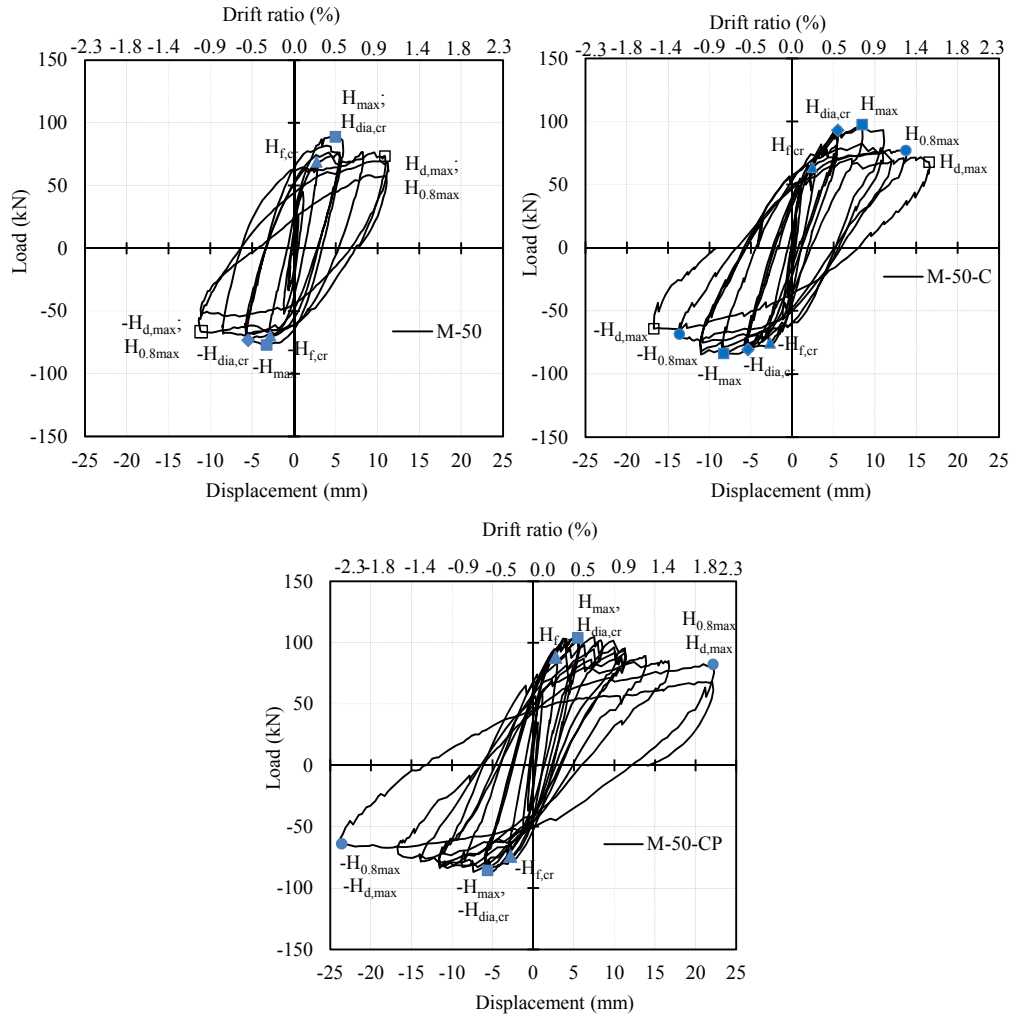


Figure 5.15 : Force-displacement diagrams for M-50, M-50-C and M-50-CP.

From the lateral load-displacement hysteresis diagrams, it can be observed that, at initial small target displacement cycles the walls exhibited elastic behaviour and showed negligible residual displacement upon the reversal of the applied horizontal load. With the increase in the achieved lateral displacement, all walls including the URM M-50 wall followed wider hysteretic loops. With the substantial loss in the horizontal load capacity and due to formation of extensive damage, the specimens reached their ultimate stages.

The response of the walls in the pushing and pulling directions were not perfectly symmetric, but reasonably close to each other, except the S-50-C specimen. The degradation in strength and stiffness was not very sudden so that they could be measured and plotted. Either with or without cramps and pins, the obtained

hysteresis loops show that the walls exhibited a highly nonlinear response with significant residual displacements after each load reversal.

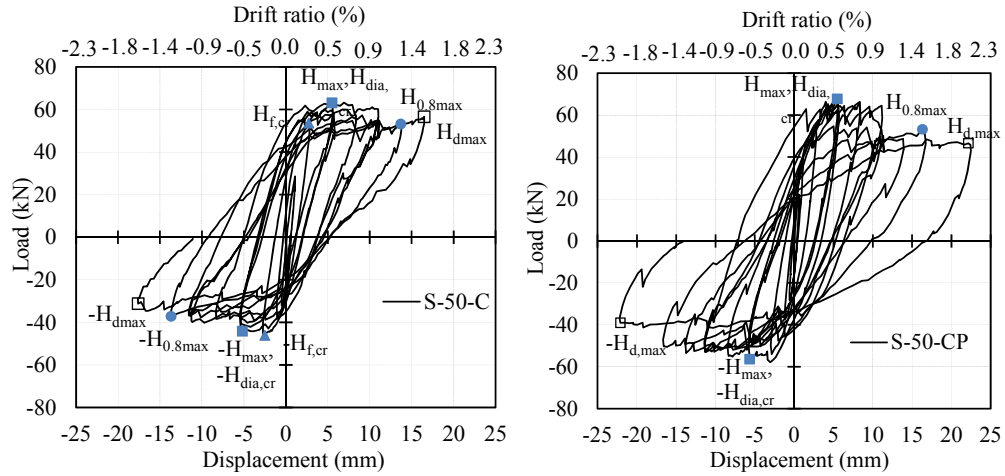


Figure 5.16 : Force-displacement diagrams for S-50-C and S-50-CP.

It should be noted that the tests were resumed until formation of extensive damage ($H_{d,max}$ point on the hysteresis diagrams), after which serious safety problems could emerge. However, during the interpretation of the experimental data it has been assumed that the ultimate state corresponds to 20% strength degradation ($H_{0.8max}$ point). Envelope curves derived from the hysteresis diagrams by considering this ultimate state definition are presented in Figures 5.17 and 5.18 for the pushing and pulling directions, respectively. Similarly, the envelope curves of the second comparison group, that consisted of S-50-C and S-50-CP walls without rubble masonry infill, are given in Figures 5.19 and 5.20 for the pushing and pulling directions, respectively.

The load and displacement data describing the critical states of the walls are tabulated in Table 5.4. In this table, $H_{f,cr}$, $DR_{f,cr}$; $H_{dia,cr}$, $DR_{dia,cr}$; H_{max} , DR_{Hmax} ; $H_{0.8max}$, $DR_{0.8max}$ and $H_{d,max}$, $DR_{d,max}$ columns correspond to lateral load and drift ratio values of first flexural cracking, first diagonal cracking, maximum lateral resistance, 20% strength loss and ultimate displacement critical states. The $H_{0.8max}$ values marked with a “*” correspond to a state at which strength loss is less than but close to 20%.

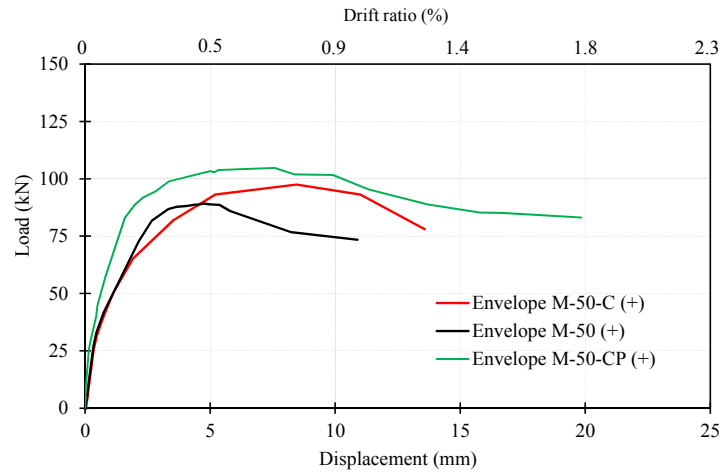


Figure 5.17 : Envelope curves for M-50, M-50-C and M-50-CP (pushing).

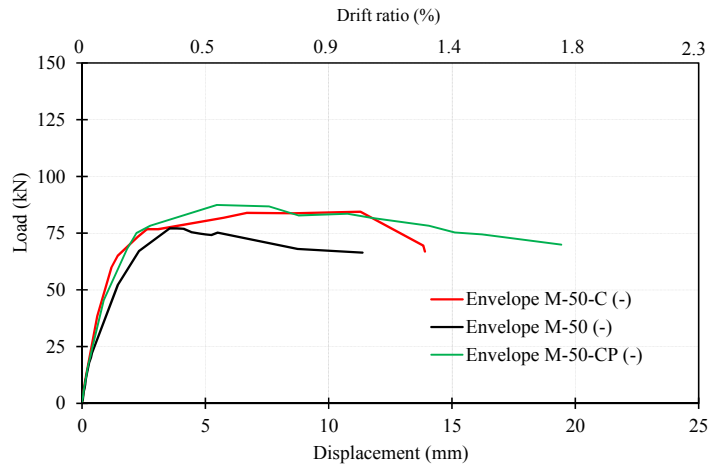


Figure 5.18 : Envelope curves for M-50, M-50-C and M-50-CP (pulling).

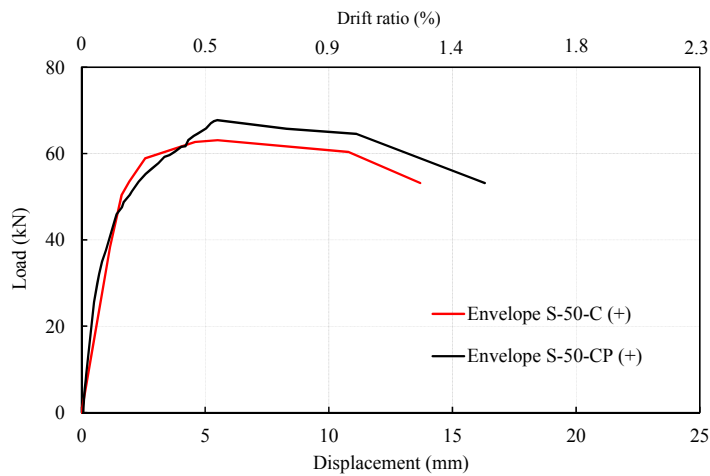


Figure 5.19 : Envelope curves for S-50-C and S-50-CP (pushing).

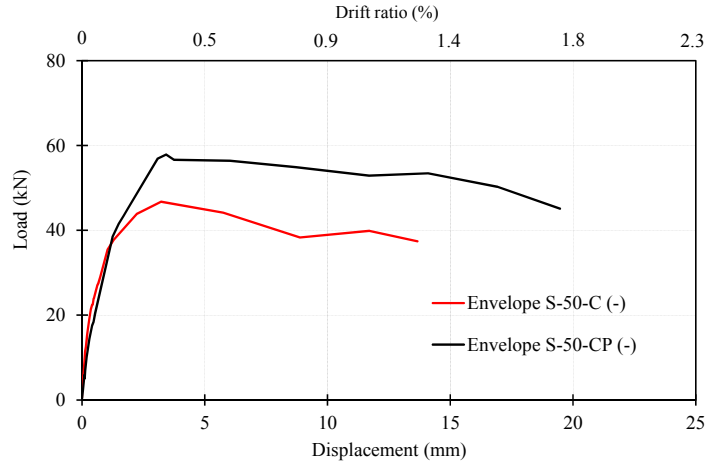


Figure 5.20 : Envelope curves for S-50-C and S-50-CP (pulling).

Table 5.4 : Load and drift ratio data for critical states of M-50, M-50-C, M-50-CP, S-50-C and S-50-CP.

Wall	$H_{f,cr}$	$DR_{f,cr}$	$H_{dia,cr}$	$DR_{dia,cr}$	H_{max}	DR_{Hmax}	$H_{0.8max}$	$DR_{0.8max}$	$H_{d,max}$	$DR_{d,max}$
	(kN)	(%)	(kN)	(%)	(kN)	(%)	(kN)	(%)	(kN)	(%)
M-50 (+)	68.8	0.25	88.7	0.50	89.2	0.43	71.3*	1.00	73.4	1.00
M-50 (-)	69.8	0.25	74.6	0.50	77.1	0.30	66.4*	1.00	66.4	1.00
M-50-C (+)	64.5	0.25	81.0	0.50	97.5	0.75	77.1	1.25	67.7	1.50
M-50-C (-)	74.8	0.25	80.4	0.50	83.7	0.75	66.5	1.25	64.1	1.50
M-50-CP (+)	88.6	0.25	103.9	0.50	103.9	0.50	83.12	1.80	82.3	2.00
M-50-CP (-)	75.0	0.25	87.4	0.50	87.4	0.50	69.9	1.80	65.2	2.00
S-50-C (+)	53.6	0.25	63.1	0.50	63.1	0.50	52.3*	1.25	56.7	1.50
S-50-C (-)	46.8	0.25	44.2	0.50	46.8	0.50	37.4	1.20	34.7	1.50
S-50-CP (+)	-	-	67.8	0.50	67.8	0.50	54.2	1.50	46.6	2.00
S-50-CP (-)	-	-	56.4	0.50	56.4	0.50	45.1	1.80	39.1	2.00

Comparison of lateral load-displacement envelope curves and the data presented in Table 5.4 reveal that the utilization of the cramps and pins has remarkable influence on the behaviour. Introduction of cramps (M-50-C wall) slightly developed the deformability of the walls by increasing the ultimate drift ratio ($DR_{0.8max}$) capacity by 25% with respect to the URM M-50 wall. The deformation capacity was increased by 80% with the combined utilization of cramps and pins (M-50-CP wall), again with respect to the URM M-50 wall. Moreover, strength characteristics of the walls were also developed to a certain extent. M-50-C specimen with cramps reached 9% and the M-50-CP specimen with cramps and pins could reach 15% higher strength in average with respect to the URM M-50 specimen.

When the S-50-C and S-50-CP specimens without rubble masonry infill are taken into account, it can be seen that the force data for pushing and pulling directions remarkably differ. However, if the average values of both directions are utilized, a 14% increase in strength and 35% enhancement in deformability can be pronounced for comparison of S-50-C wall with cramps and S-50-CP wall with pins and cramps. A similar comparison for M-50-C and M-50-CP addresses 5.5% enhancement in strength and 44% enhancement in deformation capacity.

The load values for the first flexural cracking critical state ($H_{f,cr}$) was not remarkably influenced with the existence of the reinforcement. In average $H_{f,cr}$ loads correspond to 85% of the maximum load capacities in pushing and pulling directions of specimens as given in Table 5.3. However, as mentioned before, the cracks could only be visible and so that identified at the peak displacements of the 0.25% drift ratio cycles.

Irrespective of the usage of cramps and pins, in all specimens, the first diagonal cracks appeared about 0.50% drift ratio. They emerged either before, at or after the peak load capacities so that the average $H_{dia,cr}/H_{max}$ ratio becomes 97%, Table 5.5.

The URM M-50 wall reached its maximum resistance (H_{max}) at about 0.37% average drift ratio (DR_{Hmax}), while they were 0.75 and 0.50% for the M-50-C and M-50-CP specimens, respectively. Assumed ultimate states, which correspond to 20% strength loss ($H_{0.8max}$), were achieved at 1.00, 1.25 and 1.80% drift ratios for M-50, M-50-C and M-50-CP, respectively. In other words, integrity of the walls were prolonged by the presence of reinforcements, so that higher displacement levels could be reached by M-50-C and M-50-CP specimens. Similar results can also be drawn for S-50-C and S-50-CP walls. Maximum resistance was attained at 0.50% drift ratio while the assumed ultimate state was reached at 1.23 and 1.65% average drift ratios for S-50-C and S-50-CP, respectively.

Ultimate displacements at which the tests were ended due to safety concerns ($DR_{d,max}$), were achieved at 1.00, 1.50 and 2.00% drift ratios for multi-leaf URM, cramp reinforced and cramp and pin reinforced walls, respectively. Similarly, these values were 1.50 and 2.00% for S-50-C and S-50-CP walls, respectively.

Table 5.5 : $H_{f,cr}/H_{max}$ and $H_{dia,cr}/H_{max}$ ratios for M-50, M-50-C, M-50-CP, S-50-C and S-50-CP walls.

Wall	$H_{f,cr}/H_{max}$	$H_{dia,cr}/H_{max}$
	(%)	(%)
M-50 (+)	77	99
M-50 (-)	91	97
M-50-C (+)	66	83
M-50-C (-)	89	96
M-50-CP (+)	85	100
M-50-CP (-)	86	100
S-50-C (+)	85	100
S-50-C (-)	100	94
S-50-CP (+)	-	100
S-50-CP (-)	-	100
Average	85	97
Stdev	10	5.3
COV	12	5.5

Typical diagonal deformation-lateral displacement hysteresis diagrams of M-50, M-50-C and M-50-CP walls, obtained by using the diagonally oriented LVDTs, are presented in Figure 5.21. In this figure, diagrams of M-50 and M-50-C walls turn out to be quite similar. This is mainly due to the premature cracking of the stone units, so that the tensile capacities of the cramps could not be fully utilized. Given a certain lateral displacement, the corresponding diagonal deformations approximate, which means that the total width of the cracks are also approximate. However, the difference was in the crack configuration. In the case of M-50-C wall, cracks were rather scattered among the wall faces, while deformations were concentrated at stepped diagonal cracks passing through the joints of the M-50 wall. Diagonal strains were slightly smaller in the case of M-50-CP specimen, which was probably due to the deformation restraining effect of the pins.

Similar diagonal strain-lateral displacement graphs are also plotted for S-50-C and S-50-CP walls, Figure 5.22. In a similar fashion with the M-50-C and M-50-CP specimens, the diagonal deformations were approximate and they were quite reversible until 1.00% drift ratio. Significant plastic strains could be recorded after this displacement level at which the diagonal cracks and crushing started to appear.

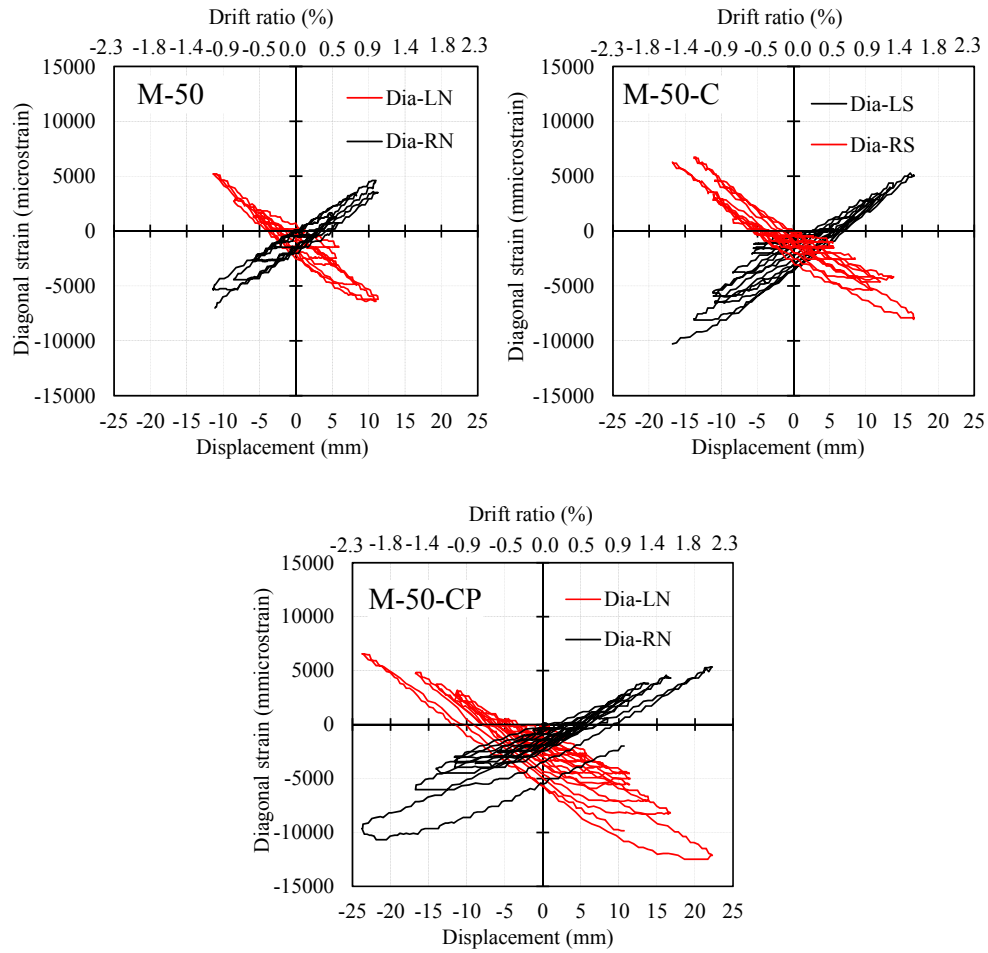


Figure 5.21 : Diagonal strain vs. lateral displacement; M-50, M-50-C and M-50-CP.

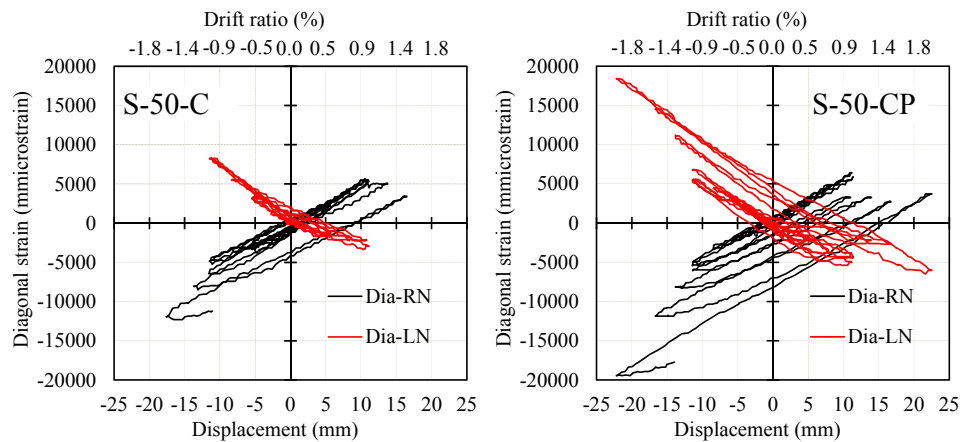


Figure 5.22 : Diagonal strain vs. lateral displacement; S-50-C and S-50-CP.

5.2.3 Effect of existence of rubble masonry infill

Two sets of walls were tested to investigate the contribution of the rubble masonry infilled between the outer ashlar leaves. The first set consists of M-50-C and S-50-C walls which had cramps between the adjacent stone units, while M-50-CP and S-50-CP walls (set two) had pins in addition to the cramps.

Lateral force vs. displacement (and drift ratio) envelope curves of M-50-C and S-50-C specimens are compared in Figures 5.23 and 5.24 for the pushing and pulling directions, respectively.

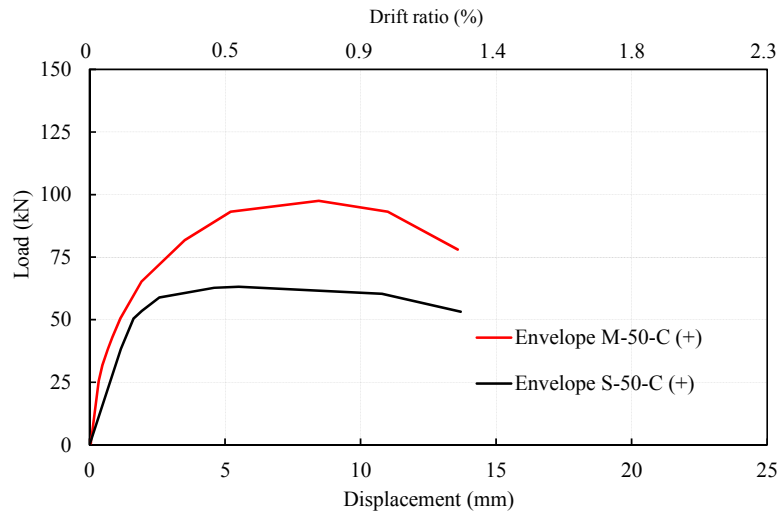


Figure 5.23 : Envelope curves for M-50-C and S-50-C (pushing).

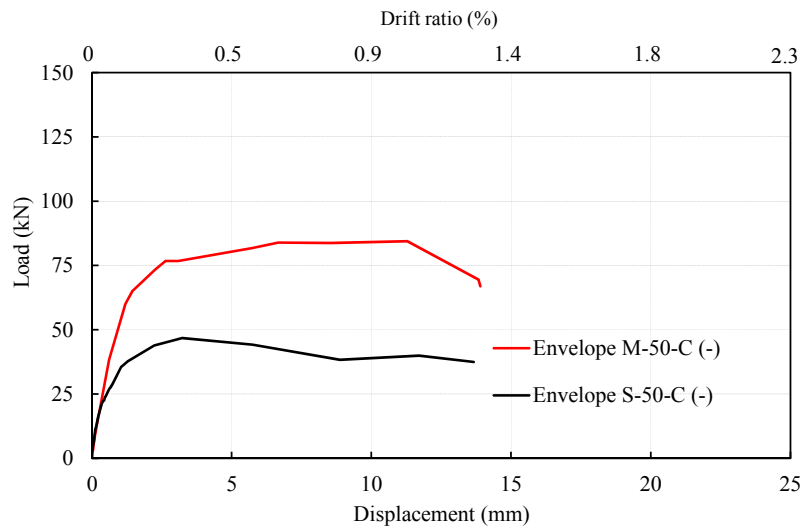


Figure 5.24 : Envelope curves for M-50-C and S-50-C (pulling).

Envelope curves of the second comparison group that consisted of M-50-CP and S-50-CP walls are presented in Figures 5.25 and 5.26 for the pushing and pulling directions, respectively.

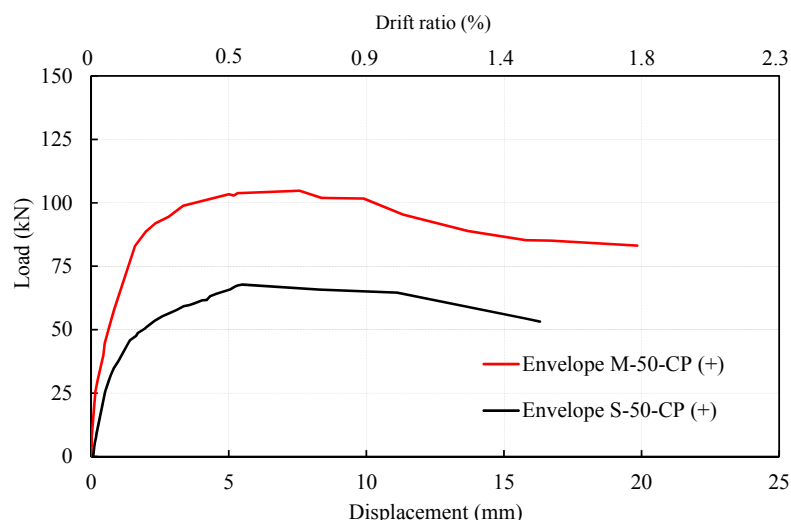


Figure 5.25 : Envelope curves for M-50-CP and S-50-CP (pushing).

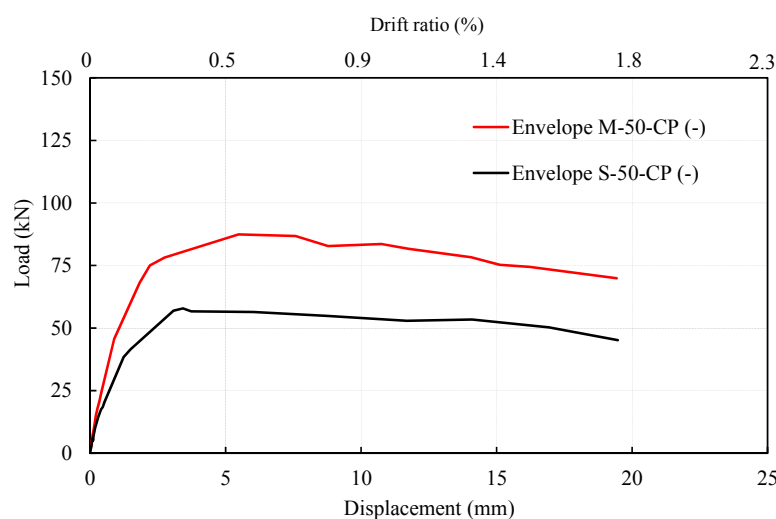


Figure 5.26 : Envelope curves for M-50-CP and S-50-CP (pulling).

The load and displacement data describing the critical states of the compared walls are tabulated in Table 5.6. In this table, $H_{f,cr}$, $DR_{f,cr}$, $H_{dia,cr}$, $DR_{dia,cr}$, H_{max} , DR_{Hmax} , $H_{0.8max}$, $DR_{0.8max}$ and $H_{d,max}$, $DR_{d,max}$ columns correspond to lateral load and drift ratio values of first flexural cracking, first diagonal cracking, maximum lateral resistance, 20% strength loss and ultimate displacement critical states. The $H_{0.8max}$

values marked with a “*” correspond to a state at which strength loss is less than but close to 20%.

Table 5.6 : Load & drift ratio data for critical states of M-50-C, S-50-C, M-50-CP and S-50-CP.

Wall	H _{f,cr}	DR _{f,cr}	H _{dia,cr}	DR _{dia,cr}	H _{max}	DR _{Hmax}	H _{0.8max}	DR _{0.8max}	H _{d,max}	DR _{d,max}
	(kN)	(%)	(kN)	(%)	(kN)	(%)	(kN)	(%)	(kN)	(%)
M-50-C (+)	64.5	0.25	81.0	0.50	97.5	0.75	77.1	1.25	67.7	1.50
M-50-C (-)	74.8	0.25	80.4	0.50	83.7	0.75	66.5	1.25	64.1	1.50
S-50-C (+)	53.6	0.25	63.1	0.50	63.1	0.50	52.3*	1.25	56.7	1.50
S-50-C (-)	46.8	0.25	44.2	0.50	46.8	0.25	37.4	1.20	34.7	1.50
M-50-CP (+)	88.6	0.25	103.9	0.50	103.9	0.50	83.12	1.80	82.3	2.00
M-50-CP (-)	75.0	0.25	87.4	0.50	87.4	0.50	69.9	1.80	65.2	2.00
S-50-CP (+)	-	-	67.8	0.50	67.8	0.50	54.2	1.50	46.6	2.00
S-50-CP (-)	-	-	56.4	0.50	56.4	0.50	45.1	1.80	39.1	2.00

Comparison of lateral load-displacement envelope curves and the data presented in Table 5.6 leads us to an interesting result. Though it consists of a very weak material with respect to stone masonry, rubble masonry infill greatly enhances the strength characteristics of the multi-leaf walls. For both reinforcement configurations, either with cramps or cramps and pins, the lateral resistance capacities of the walls were increased significantly. In average, lateral strength of M-50-C wall was 65% higher than that of S-50-C. In the case of M-50-CP and S-50-CP walls this value reached 54%. Deformation capacity was neither positively nor negatively affected by the introduction of rubble infill. Walls without rubble infill could also sustain their capacities until the assumed ultimate state (20% strength loss).

Typical diagonal deformation-lateral displacement hysteresis diagrams of M-50-C and S-50-C walls, obtained by using the diagonally oriented LVDTs, are presented in Figure 5.27. Similar diagrams for M-50-CP and S-50-CP walls are also presented in Figure 5.28. From these diagrams, it can be noticed that S-50-C and S-50-CP walls tend to have greater plastic deformations (or damage) in the diagonal direction than the M-50-C and M-50-CP walls with rubble infill. This can be identified with the steeper diagonal deformation diagrams obtained for the S-50-C and S-50-CP walls. Lower lateral load capacities of the walls without rubble infill can be attributed to the earlier development of these damage indicator plastic deformations.

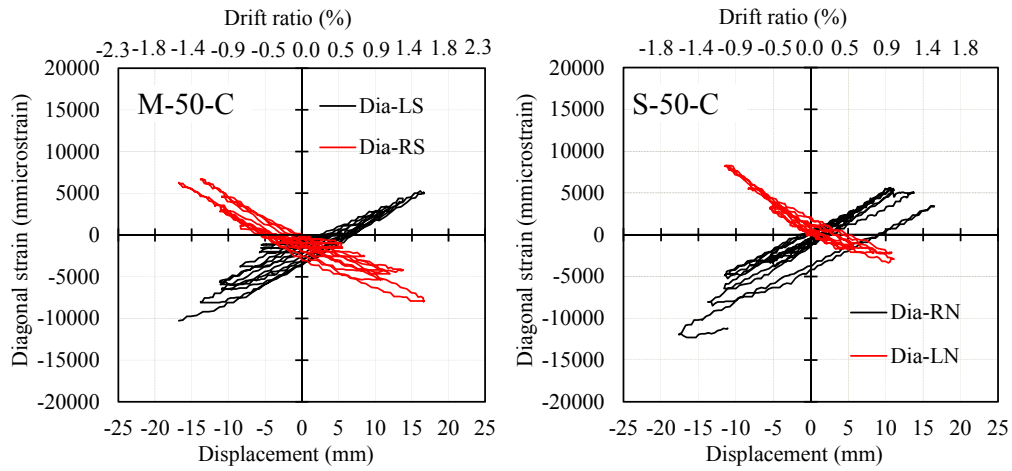


Figure 5.27 : Diagonal strain vs. lateral displacement; M-50-C and S-50-C.

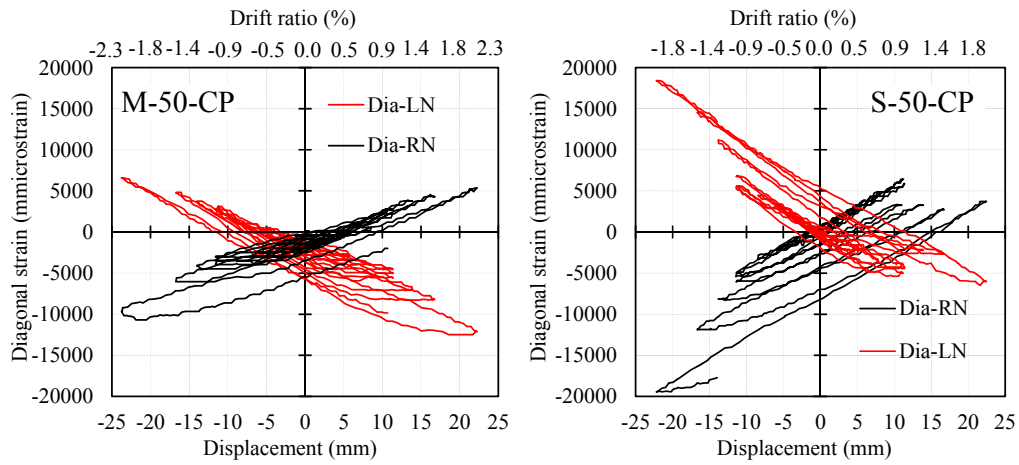


Figure 5.28 : Diagonal strain vs. lateral displacement; M-50-CP and S-50-CP.

6. INTERPRETATION OF THE EXPERIMENTAL DATA

In this chapter, the load-displacement curves are first translated into idealized equivalent bilinear curves. So that, the evaluation of stiffness, strength, deformability and other parameters is simplified. Then the lateral displacements in the form of drift ratios that correspond to critical points of the bilinear idealized diagrams are investigated and the ductility levels of the tested walls are discussed. In order to evaluate the variation of stiffness, firstly typical raw stiffness data obtained from the tests are presented. In the next step, the normalized stiffness-displacement curves are given and compared with the empirical equations given in the literature. Then, the energy dissipation capabilities and damping characteristics of the tested walls are evaluated. Strength and failure mode characteristics are further evaluated in comparison with a number of failure envelopes available in the literature. Finally, the drift ratios corresponding to different performance states are indicated and compared with the code provided performance limits.

6.1 Idealization of the Experimental Envelope Curves

In order to simplify evaluation of stiffness, strength, deformability and make calculations easier; the actual hysteretic behaviour of masonry walls can be idealized by bilinear envelopes. This approach has been used by numerous researchers not only for reinforced concrete structural elements, but also for masonry walls (such as Bosiljkov et al., 2003; Vasconcelos and Lourenço, 2009 and Frumento et al., 2009).

The bilinear idealization process carried out in this study depends on the lateral force-displacement envelope curves presented in the previous chapter. For this purpose, both positive (pushing) and negative (pulling) directions of the tests were considered.

The schematic representation of the bilinear idealization diagram of an experimental envelope curve is shown in Figure 6.1. During the idealisation of the experimental envelope, three main limit states can be defined and used as reference points (Tomazevic 1999). First limit state corresponds to the formation of the first

significant cracks in the wall and can be determined by displacement d_{cr} and lateral load H_{cr} . Maximum resistance limit state is determined by maximum lateral load H_{max} attained at d_{Hmax} displacement. Finally, the last limit state, which corresponds to 20% strength loss, defines the ultimate point of the idealised curve via $H_{0.8max}$ lateral resistance and corresponding $d_{0.8max}$ (or d_u) displacement.

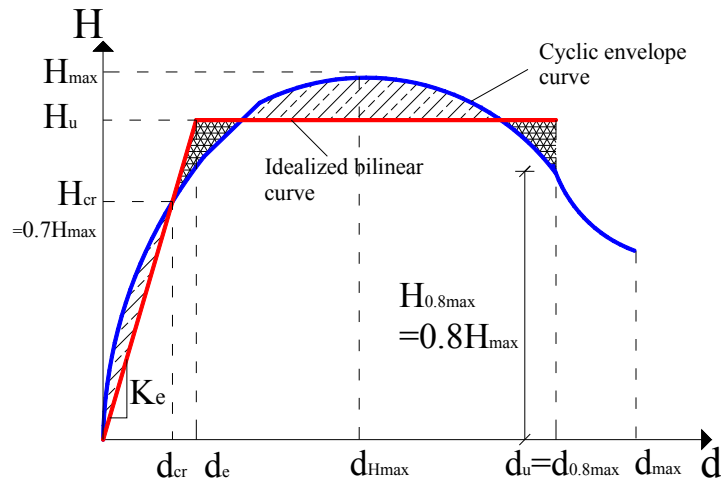


Figure 6.1 : Idealisation of experimental envelope curve with bilinear relationship.

The initial slope of the idealised envelope is generally defined as equivalent elastic or effective stiffness of the wall, $K_e = H_{cr} / d_{cr}$. H_{cr} load and d_{cr} displacement values define the first significant cracking point, where the slope of the load-displacement envelope curve is reduced due to flexural cracking. As mentioned earlier, the average first significant cracking points ($H_{f,cr}$) obtained in this study correspond to 83% of the maximum load capacities (H_{max}). However, since the cracks could only be visible and so that identified at the peak displacements of the 0.25% drift ratio cycles, this percentage is an overestimated value. Consequently, a smaller $H_{f,cr}/H_{max}$ ratio in the order of 70%, as also mentioned by Magenes and Calvi (1997) and Frumento et al. (2009), was utilized in this study. Reliability of this assumption was also checked by investigating the points at which the stiffness reduces considerably in the load-displacement envelope curves.

In this study, the ultimate displacement of the idealised envelope is evaluated as the displacement corresponding to 20% strength degradation, as also assumed by Magenes and Calvi (1997) and Frumento et al. (2009). However, another common

assumption is to accept the ultimate displacement as the displacement value where the idealised line intersects the descending branch of the experimental envelope.

The value of the lateral resistance H_u that corresponds to the horizontal branch of the idealised bilinear curve can be found by taking into account the equal energy dissipation capacity of an actual and idealised wall, which can be translated as equality of the areas below the experimental envelope and idealised bilinear curves. Consequently, the shaded areas with two different hatching patterns shown in Figure 6.1 should be equal to each other. Area under the bilinear curve can be easily computed by Eq. (6.1). If the area under the experimental envelope curve $A_{envelope}$ is equalized to $A_{bilinear}$, Eq. (6.2) is obtained for the H_u ultimate resistance of the idealised diagram.

$$A_{bilinear} = H_u d_u - \frac{H_u^2}{2K_e} \quad (6.1)$$

$$H_u = K_e \left[d_u - \sqrt{d_u^2 - \frac{2A_{envelope}}{K_e}} \right] \quad (6.2)$$

Following the procedure explained above, characteristic values that define the idealised bilinear diagrams of the actual experimental envelope curves can be obtained as summarized in Table 6.1. Bilinear idealization diagrams of experimental envelope curves obtained for each wall can be seen in Appendix C.

In case H_u / H_{max} ratios of M-25-C, M-50-C, M-75-C and M-100-C walls are calculated, an approximate value of 93% can be obtained. Moreover, even if all walls are taken into account this ratio does not change. This finding is in good agreement with the $H_u = 0.90H_{max}$ relationship proposed by Tomazevic (1999) depending on test results of more than 60 walls. Vasconcelos and Lourenço (2009) also report similar ratios varying between 0.90 and 0.97 for tests of dry masonry, irregular masonry and rubble stone masonry walls.

Idealised bilinear load-displacement diagrams for the pushing directions of the walls M-25-C, M-50-C, M-75-C and M-100-C can be compared by using the bilinear curves presented in Figure 6.2. As also observed during the evaluation of the experimental envelopes, the more axial stress is applied the more idealized lateral

resistance is achieved. On the other hand, ultimate displacement capacity is inversely proportional with the pre-compression stress level.

Table 6.1 : Characteristic values of bilinear idealisation.

Wall	H_{cr}	d_{cr}	K_e	H_u	d_e	H_{max}	d_u
	(kN)	(mm)	(kN/mm)	(kN)	(mm)	(kN)	(mm)
M-25-C (+)	39.4	1.38	28.7	51.0	1.78	56.3	15.74
M-25-C (-)	38.2	1.30	29.2	50.5	1.73	54.5	8.46
M-50-C (+)	65.3	1.92	34.0	91.5	2.69	97.5	13.58
M-50-C (-)	58.6	1.16	50.5	80.2	1.59	83.7	13.90
M-75-C (+)	91.9	1.81	50.8	120.5	2.37	131.3	12.18
M-75-C (-)	80.0	1.66	48.3	107.2	2.22	114.3	11.93
M-100-C (+)	97.0	0.95	101.6	131.2	1.29	138.5	11.05
M-100-C (-)	93.0	1.31	70.9	121.2	1.71	132.8	8.83
M-50 (+)	62.4	1.67	37.3	82.9	2.22	89.2	10.89
M-50 (-)	54.0	1.55	34.7	71.7	2.06	77.1	11.37
M-50-CP (+)	72.7	1.28	56.9	94.9	1.67	103.9	19.84
M-50-CP (-)	61.2	1.54	39.9	80.5	2.02	87.4	19.43
S-50-C (+)	44.3	1.38	32.1	60.0	1.87	63.2	13.70
S-50-C (-)	32.7	1.04	31.5	42.1	1.34	46.8	13.66
S-50-CP (+)	47.5	1.61	29.4	63.0	2.14	67.8	15.83
S-50-CP (-)	39.5	1.32	29.8	53.2	1.79	56.4	19.47

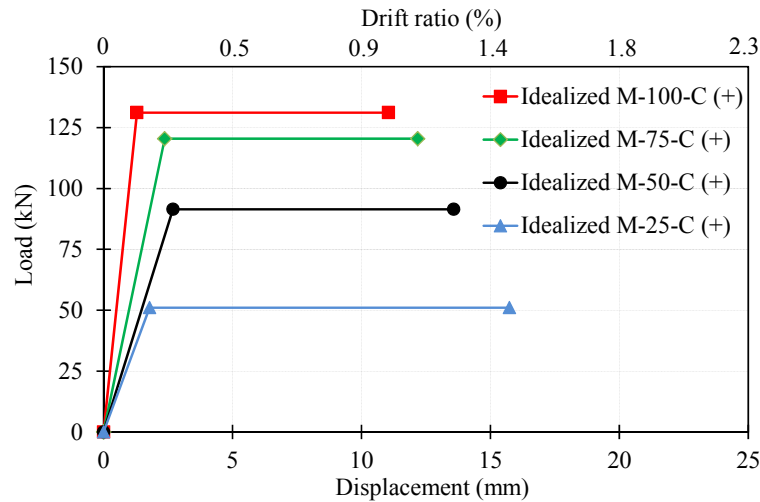


Figure 6.2 : Bilinear idealized diagrams of M-25-C, M-50-C, M-75-C and M-100-C.

Another comparison, regarding the contribution of the reinforcements can be made depending on the bilinear idealized diagrams given in Figures 6.3 and 6.4. Comparing the M-50 and M-50-C walls, it is seen that introduction of cramps clearly enhances the lateral load and deformation capacities. Utilization of pins in addition to

cramps, as in the cases of M-50-CP and S-50-CP, further increases the deformability, while lateral load capacity approximates to M-50-C and S-50-C walls, respectively.

Though it consisted of a weak mortar mixed with stone pieces, existence of rubble masonry core develops the lateral resistance significantly. Load capacities of M-50-C and M-50-CP walls in pushing and pulling directions were approximately 54% higher than that of S-50-C and S-50-CP walls. In conclusion, matching of the bilinear idealized diagrams of M-50-C with S-50-C and M-50-CP with S-50-CP walls shows that rubble core mainly contributes to strength while utilization of pins contribute to deformability.

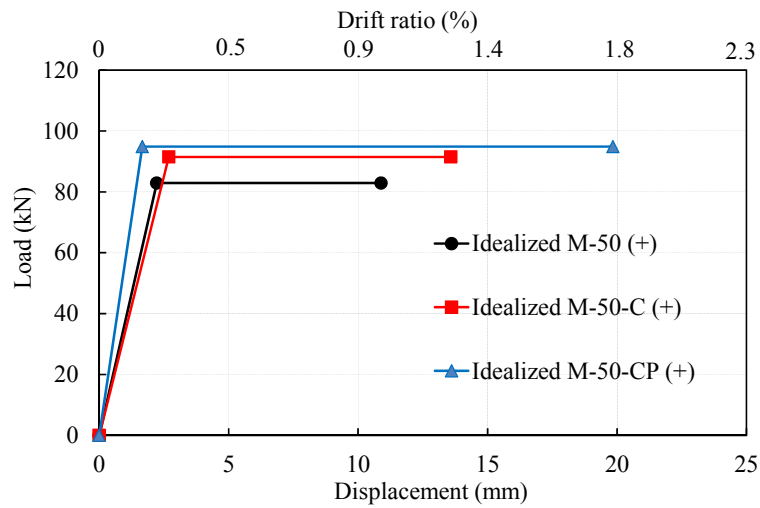


Figure 6.3 : Bilinear idealized diagrams of M-50, M-50-C and M-50-CP.

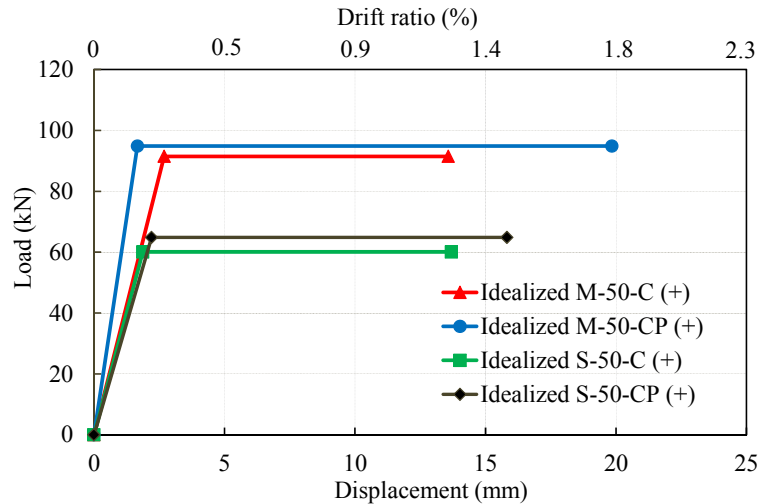


Figure 6.4 : Bilinear idealized diagrams of M-50, M-50-CP, S-50-C and S-50-CP.

In order to ease the perception of the variation of the ultimate resistance of the idealised diagram (H_u) with respect to axial stress level, values for the M-25-C, M-50-C, M-75-C and M-100-C walls are plotted as shown in Figure 6.5. Accordingly, a second order polynomial correlation can be established between ultimate resistance of the idealised diagram H_u and the axial stress level parameters. Apparently, the load capacities of the walls tend to increase due to an increase in the axial stress level. However, the slope of the correlation curve decreases along with the increase in the axial stress level, which means that a direct proportion cannot be defined particularly for high pre-compression stresses where the changing of the failure mode takes place.

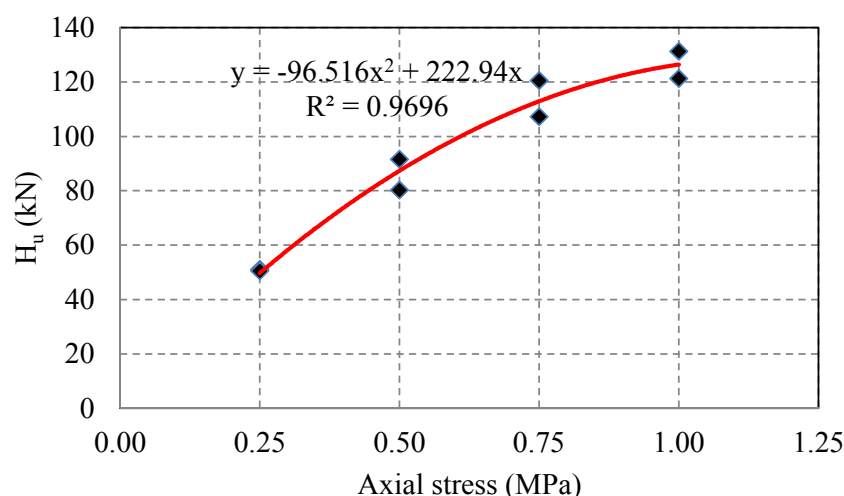


Figure 6.5 : Ultimate resistance of the idealised diagram vs. axial stress level.

6.2 Deformation Capacity

In the case of seismic loads, modern design approaches and codes not only consider the strength of the structural members but also try to define the deformation capabilities and demands. Although masonry, especially the unreinforced masonry (URM), is generally considered as a brittle structural material, it can achieve remarkable deformations prior to failure. Within this topic, lateral displacements in the form of drift ratios that correspond to critical points of the bilinear idealized diagrams will be investigated. Finally, the ductility levels of the tested walls shall be discussed.

First critical point of the bilinear idealized curve corresponds to equivalent yield point of the elastic-perfectly plastic load-displacement diagram (d_e, H_u). The

variation of the drift ratios at this point (DR_e) with respect to axial stresses are plotted in Figure 6.6 for M-25-C, M-50-C, M-75-C and M-100-C walls. Data points of other specimens tested under 0.50 MPa pre-compression (M-50, M-50-CP, S-50-C and S-50-CP walls) also stay in between the data points of M-50-C. From this figure, it can be understood that it is hard to establish a correlation between equivalent yield drift ratio DR_e and axial stress level. The equivalent yield drift ratios vary between 10 and 25% and tend to increase until an approximate axial stress level of 0.60 MPa that is followed by a descending curve.

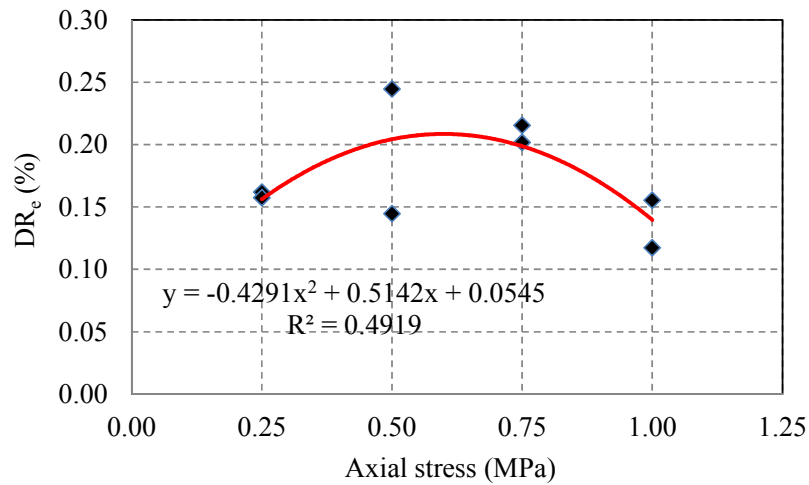


Figure 6.6 : Equivalent yield point drift ratio (DR_e) vs. axial stress level.

As observed previously, a second order polynomial correlation can be established between ultimate resistance of the idealised diagram H_u and the axial stress level parameters. Another correlation, but this time a linear one, between the drift ratio corresponding to ultimate displacement and pre-compression stress level can also be achieved, Figure 6.7. According to this diagram, ultimate drift capacities of the investigated walls are inversely proportional with the axial stresses. As the axial stress increases ultimate drift values decrease, while staying in between 1.4 and 0.8% drift ratios. It should be noted that, in this diagram the ultimate drift capacity of the M-25-C in the pulling direction, which was quite low due to premature crushing of the toes, is neglected.

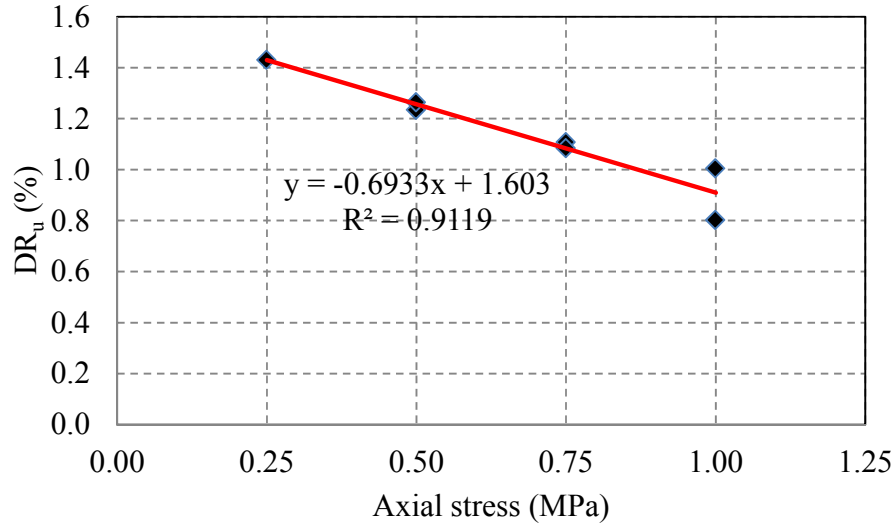


Figure 6.7 : Ultimate drift ratio of the idealised diagram vs. axial stress level.

Ductility of a structural member or system can be defined as the capability to deform beyond its elastic limit without excessive strength decay or stiffness degradation. The ductility term can be used to express the deformation characteristics of materials, cross-sectional behaviour, structural member behaviour and behaviour of structural systems, by utilizing measures such as curvature, rotation, strain and displacement. In this study, lateral displacements for equivalent yield point (d_e) and ultimate condition (d_u) are utilized for derivation of the ductility factors of the tested walls.

The general analytical definition of displacement ductility is as given in Eq. (6.3). However, another ductility factor, based on the cyclic response (Eq. (6.4)) may also be defined (Elnashai and Sarno, 2008). In Eq. (6.4), d_u^+ and d_u^- are the positive and negative ultimate deformations while d_e^+ and d_e^- are the equivalent yield points of the positive and negative cycles, respectively.

$$\mu = \frac{d_u}{d_e} \quad (6.3)$$

$$\mu_{cyc} = \frac{|d_u^+| + |d_u^-|}{|d_e^+| + |d_e^-|} \quad (6.4)$$

In addition to the ductility factors obtained by utilizing the above equations, the average values for pushing and pulling directions (μ_{av}) are also presented in Table

6.2. The results are visualized in Figure 6.8, so that the variation of ductility factors with respect to different parameters is better observed.

From Table 6.2 and Figure 6.8, it can be observed that all walls exhibited considerable deformation capabilities. Both equations give very close ductility values that vary between 5.2 and 10.7. Although the M-100-C wall causes an irregularity, a vague tendency for the effect of the axial stress level on the ductility factor can be observed among the M-25-C, M-50-C and M-75-C walls. Despite the large scatter in the pushing and pulling directions, average ductility decreases as the axial stress level increases. Similar observations are also reported by Vasconcellos and Lourenço (2009) on stone masonry walls with different masonry bond types. The disturbance caused by the M-100-C wall to the mentioned tendency, mainly stems from the fact that initial stiffness is also dependent on the pre-compression level. In the case of M-100-C wall, the ultimate displacement is reduced considerably by the effect of the high axial stress as seen in Figure 6.7. On the other hand, initial stiffness of the wall is increased remarkably, which in return causes a low equivalent yield displacement value, Figure 6.6. Consequently, since the reduction in the equivalent yield displacement is greater than the reduction in the ultimate displacement, M-100-C wall exhibits a higher ductility than expected.

Table 6.2 : Ductility factors of tested walls.

Wall	d_e	d_u	μ	μ_{av}	μ_{cyc}
	(mm)	(mm)			
M-25-C (+)	1.78	15.74	8.8	6.9	6.9
M-25-C (-)	1.73	8.46	4.9		
M-50-C (+)	2.69	13.58	4.5	6.9	6.4
M-50-C (-)	1.59	13.90	8.7		
M-75-C (+)	2.37	12.18	5.1	5.3	5.3
M-75-C (-)	2.22	11.93	5.4		
M-100-C (+)	1.29	11.05	8.6	6.9	6.6
M-100-C (-)	1.71	8.83	5.2		
M-50 (+)	2.22	10.89	4.9	5.2	5.2
M-50 (-)	2.06	11.37	5.5		
M-50-CP (+)	1.67	19.84	11.9	10.7	10.6
M-50-CP (-)	2.02	19.43	9.6		
S-50-C (+)	1.87	13.70	7.3	8.8	8.5
S-50-C (-)	1.34	13.66	10.2		
S-50-CP (+)	2.14	15.83	7.4	9.1	9.0
S-50-CP (-)	1.79	19.47	10.9		

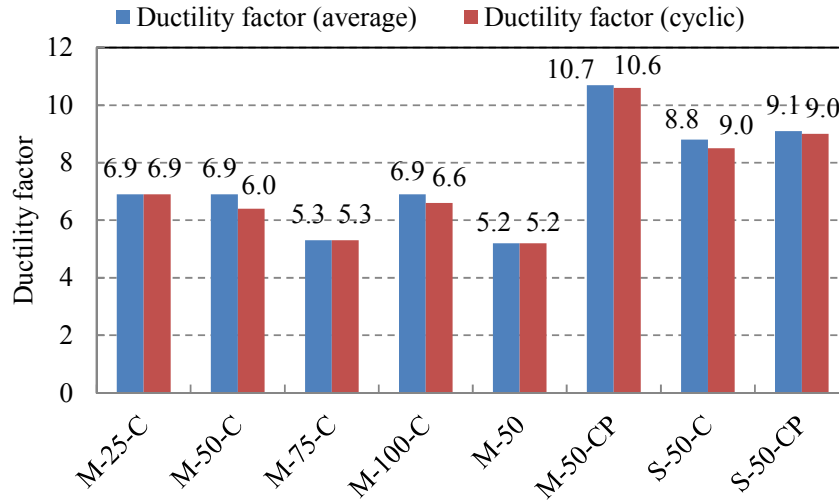


Figure 6.8 : Comparison of average and cyclic ductility factors.

When the ductility factors of M-50 (URM wall), M-50-C (wall with cramps) and M-50-CP (wall with cramps and pins) are compared, it can be observed that introduction of cramps results with 33% increase in average ductility factor and 15% increase in the cyclic ductility factor. Moreover, if pins are used in addition to cramps; a major development in the average ductility factor of URM wall, which is in the order of 106%, is achieved.

Comparison of S-50-C (wall without rubble core, reinforced with cramps) and S-50-CP (wall without rubble core, reinforced with cramps and pins) specimens lead to an unexpected result. Although S-50-C achieves a smaller ultimate displacement than the S-50-CP wall, the differences between the ductility factors of both specimens are much smaller than that of M-50-C and M-50-CP. This result mainly stems from the equivalent yielding values that is obtained by using the initial secant stiffness.

The last comparison can be done between the M-50-C vs. S-50-C and M-50-CP vs. S-50-CP specimen sets, regarding the contribution of the rubble masonry core. Single-leaf S-50-C wall produces an almost 28% higher average ductility factor than the multi-leaf M-50-C wall. This result can be explained with the usage of a single type better quality material in the S-50-C specimen, whereas M-50-C consists of good and weak quality materials. In addition to that, although ultimate displacement capacities of both walls are close to each other, equivalent yield displacements of M-50-C are higher than that of S-50-C (Figure 6.4 and Table 6.2).

In the case of M-50-CP and S-50-CP specimens, unlike the M-50-C and S-50-C walls, M-50-CP multi-leaf wall with composite material seems to have slightly better ductility than the single-leaf wall S-50-CP. It should be recalled that utilization of pins strongly enhances the deformability of the walls. Bearing this positive contribution of pins in mind, it can be concluded that the dominance of pins on the ultimate displacement capacity might have caused this result.

6.3 Variation of Stiffness

Among various parameters, stiffness has a major importance since it associates the developing deformations with the forces that shall act on the structural member. As during seismic events horizontal earthquake forces are shared by the walls with respect to their stiffness's, prediction of the variation of stiffness degradation is important.

Stiffness at a given displacement amplitude (d_i) is generally defined as a secant value that is simply the ratio between the resisting force (H_i) and displacement, so that it can be denoted as $K = H_i / d_i$. In order to enable comparison of specimens tested under different parameters, this secant stiffness can be normalised by the secant stiffness evaluated at the occurrence of the first significant cracks. Finally, the variation of the stiffness ratio, i.e., the ratio between the actual and effective stiffness of the wall K / K_e with respect to lateral displacements can be compared. Similar to the stiffness, the lateral displacements can also be normalised with regard to displacements attained at maximum lateral resistance ($d_{H\max}$).

Tomazevic (1999) states that the shape of stiffness degradation as a function of lateral displacements in a non-dimensional form is quite similar for all types of masonry walls, including plain, confined and reinforced masonry. In addition to that, Tomazevic et al. (1996) proposes a simple equation to correlate the stiffness and deformation by utilizing the normalization approach explained above:

$$\frac{K}{K_e} = \alpha \left(\frac{d_i}{d_{H\max}} \right)^\beta \quad (6.5)$$

where α and β are parameters of stiffness degradation.

In this study, firstly typical raw stiffness data obtained from the tests will be presented and compared with respect to the investigated parameters. Then the normalized stiffness-displacement curves will be given. In Figure 6.9, variation of secant stiffness with respect to displacement at peak values of each cycle is presented for pushing direction of the M-25-C, M-50-C, M-75-C and M-100-C walls, for which the sole parameter is the level of the pre-compression level. As can be observed from this diagram, an increase in the axial stress level leads to higher secant stiffness values. However, the stiffness parameter diminishes remarkably, as soon as the first significant cracks occur in the walls. Stiffnesses of all investigated walls tend to approximate to very small values especially after formation of the diagonal cracks.

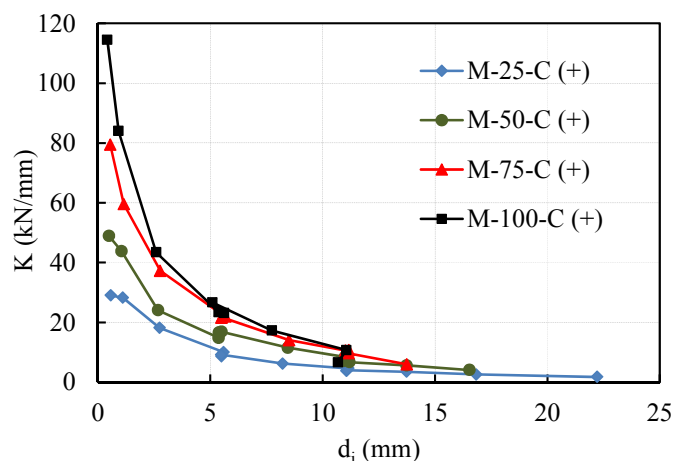


Figure 6.9 : Variation of secant stiffness with respect to axial stress level.

Effect of the reinforcement utilized in the walls to the stiffness variation can be investigated through Figure 6.10, where the secant stiffness vs. displacement diagrams of M-50, M-50-C, M-50-CP, S-50-C and S-50-CP walls in the pushing direction are given. From this figure, it can be realized that the stiffness values of the M-50-C wall with cramps and the URM M-50 wall were very close to each other until the peak lateral resistance at approximately 5.0 mm displacement. After this value M-50-C wall had slightly higher stiffness. In the case of M-50-CP specimen with cramps and pins, stiffness was higher than all other walls until the 5.0 mm displacement (0.50% drift ratio). After the attainment of peak resistance, M-50-CP wall approximated to M-50-C walls.

In the case of S-50-C and S-50-CP walls without rubble masonry infill, S-50-CP had higher stiffness during the first a few displacement cycles. However, with the initiation of the first cracks they become almost identical. As expected, their stiffness

values were clearly less than the walls with rubble masonry core, irrespective of the presence of the reinforcing members.

Considering these results, it can be concluded that existence of pins and cramps as reinforcement plays some role in the stiffness variation particularly prior to occurrence of first significant cracking. After formation of the major diagonal crack bands and crushing of the stone units at the toes, stiffness diminishes significantly and values approximate to each other. It should be noted that, although rubble masonry material is very weak with respect to the stone material, a major difference exists between the single- and multi-leaf walls, particularly until the formation of major damage. This mainly stems from the existence of the rubble masonry core in-between the ashlar outer leaves that supplies additional integrity and moment of inertia to the multi-leaf walls.

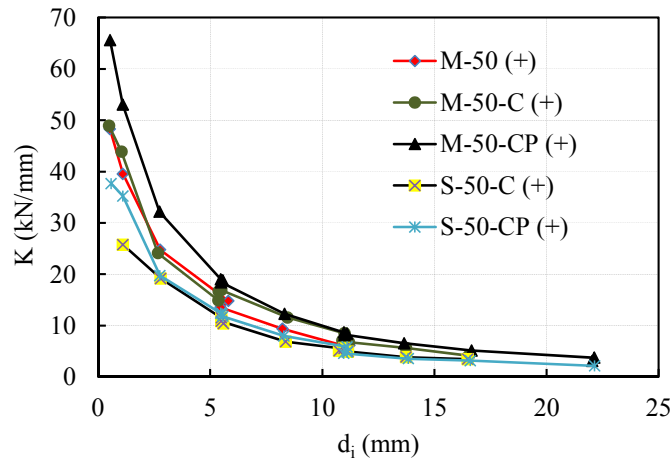


Figure 6.10 : Variation of secant stiffness with respect to reinforcement utilization.

Stiffness deterioration (K/K_e) of all specimens with respect to normalized displacement values ($d_i/d_{H\max}$), as proposed by Tomazevic et al. (1996), are presented in Figure 6.11. Data points of the diagram that belongs to different specimens with different axial stress levels, reinforcement configurations and wall configuration; seems to fit a power type trend line, as also given in Eq. (6.5). α and β parameters used in this equation can be obtained through a power type regression analysis of all data points as 0.28 and -0.80, respectively. Values of α and β parameters and R^2 determination coefficients for each individual wall are tabulated in Table 6.3. From this table it can be observed that the trend line of each individual wall fits well to the data points.

Tomazevic (1999) denotes that these parameters should be determined experimentally, but if there is a lack of experimental data values of 0.30 and -0.85 can be taken into account for the α and β parameters, respectively. However, it is also noted that 0.30 and -0.85 values should be used in the case of normal compression stresses, not exceeding 20% of masonry's compressive strength. It should be noted that these values mainly depend on the test campaign described in Tomazevic et al. (1996).

Interestingly, although the specimens tested by Tomazevic et al. (1996) consist of different materials and reinforcement configurations (concrete masonry blocks with voids and continuous horizontal and vertical reinforcements), the α and β parameters obtained in our study seems to be in good agreement with the suggestions of Tomazevic (1999).

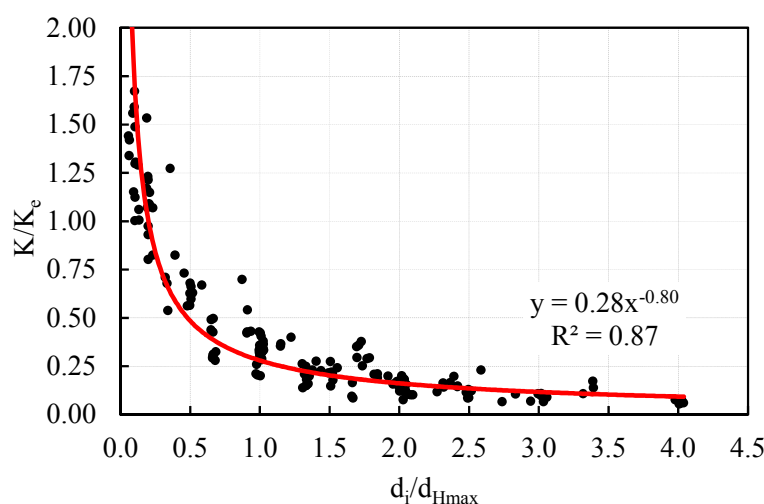


Figure 6.11 : Variation of normalized stiffness with normalized displacements.

Table 6.3 : Parameters for stiffness degradation.

Parameter	M-25-C	M-50-C	M-75-C	M-100-C	M-50	M-50-CP	S-50-C	S-50-CP	All
α	0.26	0.23	0.27	0.31	0.41	0.29	0.25	0.30	0.28
β	-0.87	-0.77	-0.72	-0.75	-0.74	-0.83	-0.91	-0.85	-0.80
R^2	0.93	0.89	0.86	0.85	0.89	0.94	0.94	0.95	0.87

6.4 Energy Dissipation Capacity

The energy dissipation capacity of structural systems is a key parameter for the analysis of their cyclic response against seismic events. In order to deform a material,

a certain amount of energy input E_{imp} is needed. In a similar manner, in order to push or pull the wall to a certain target deformation level (i.e. top displacement of a target cycle) an input energy is required, that is simply equal to the area under the load-displacement curve.

In order to have an overall view and make comparisons between the tested walls, diagrams for variation of dissipated energy with respect to achieved displacement cycle are obtained and shown in Figure 6.12.

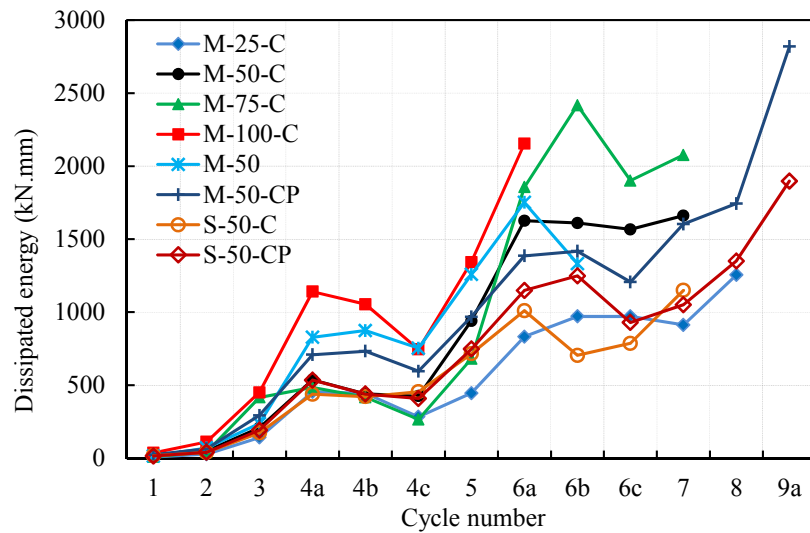


Figure 6.12 : Variation of dissipated energy with respect to each cycle.

As clearly seen from this figure, the amount of dissipated energy at each cyclic loop increases with the increase in the lateral displacements. However, this tendency for increase is not directly proportional with the achieved displacement. At certain cycles or displacement levels, energy dissipation diagrams rises sharply. Comparison with the observations done during the tests reveals that these sharp rises, which generally occurred about 0.5% drift ratio (Cycle 4) and 1.0% drift ratio (Cycle 6), generally correspond to first diagonal cracks and toe crushing damage states. This result is expected since formation of new damage in the form of cracks and crushing means development of further nonlinear deformations, thus the area under the cyclic loops increase remarkably.

If the energy dissipation diagrams of M-25-C, M-50-C, M-75-C and M-100-C specimens are compared, it can be seen that M-100-C wall with the highest axial pre-compression stress dissipates more energy at all cyclic loops. Shing et al. (1989) also report the highly dissipative behaviour of shear dominant walls with higher axial

stress levels. They explain this situation by the positive contribution of the high axial stresses to the enhancement of the interlocking forces particularly in the post-cracking region.

M-50-C and M-75-C walls with hybrid shear/flexure failure modes exhibit similar energy dissipation capacities to each other, until cycle 6a (1.0% drift ratio). However, after this drift ratio, similar to M-100-C, M-75-C wall tends to have more dissipative cycles. Figure 6.12 also reveals that M-25-C wall with the lowest axial stress level, thus with the lowest lateral strength, dissipates the least energy among all multi-leaf walls.

As also mentioned by Shing et al. (1989) and Vasconcelos and Lourenço (2009) these comparisons are not very objective, since lateral strength and configurations of the walls are different. Therefore, an alternative way for a more objective comparison that was proposed by Shing et al. (1989) can be followed. According to this approach, the dissipated energy can be normalized by the elastic energy absorption of the equivalent elastic-perfectly plastic model. So that the normalized cumulative energy at any cycle E_N can be obtained by using Eqs. (6.6) and (6.7):

$$E_N = \frac{1}{E_e} \sum_{i=1}^n E_{dis,i} \quad (6.6)$$

$$E_e = H_u d_e \quad (6.7)$$

where E_e is the total elastic energy absorption of equivalent elastic-perfectly plastic model (sum of pushing and pulling directions), $E_{dis,i}$ is the energy dissipated at cycle i of the test and n is the cycle number corresponding to a certain target drift ratio. The maximum value for the cycle number n can be the number of the cycle after which the peak lateral resistance is reduced to 80% of the ultimate resistance. It should be noted that, in some specimens this 80% reduction does not correspond to any of the target drift ratios and takes place between two adjacent cycle peaks. In such cases, the larger of the adjacent cycles have been considered during the calculations.

Variation of the normalized cumulative energy dissipated at each cycle of M-25-C, M-50-C, M-75-C and M-100-C walls is presented in Figure 6.13. Until the 0.50%

drift ratio (Cycle 4a), where diagonal cracks generally occurred, all specimens dissipated approximate and low amount of energy. After this drift ratio, walls with different pre-compression stress levels began to differentiate from each other. Thus, M-100-C wall with the highest axial stress and shear dominated overall behaviour exhibited a more dissipative character, whilst M-25-C, M-50-C and M-75-C followed it in the same order. Vasconcelos (2005) indicates normalized cumulative energy dissipation values that are inversely proportional with the pre-compression stress levels. A similar observation can be done for the diagrams given in Figure 6.13, with the exception of the M-100-C wall that exhibited a clear dissipative shear failure mode. If the energy values at ultimate states are compared, it can be seen that, overall energy dissipation decreases as the axial stress level increases. This can be attributed to the higher deformability, thus ductility, exhibited by walls with smaller pre-compression stresses. It should be noted that, the rate of energy dissipation increases particularly after the 0.75% drift ratio cycle, around which the strength degradation begins due to toe crushing and extensive diagonal cracks.

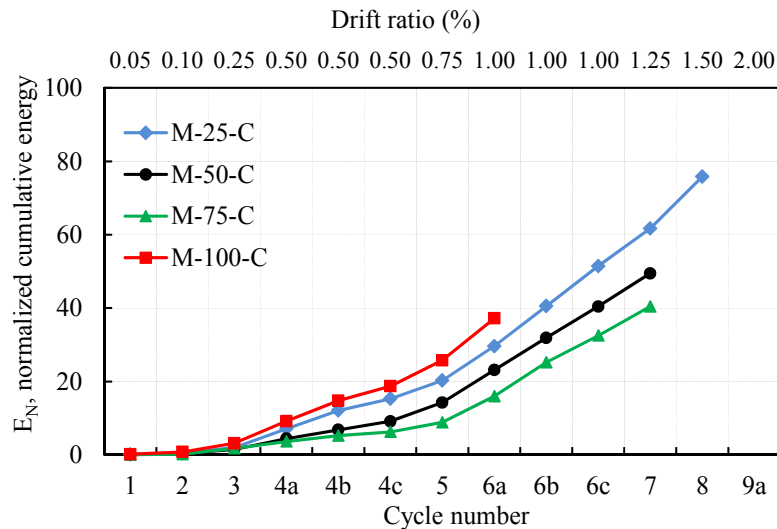


Figure 6.13 : Variation of normalized cumulative energy for M-25-C, M-50-C, M-75-C and M-100-C walls.

Another comparison set can be formed for the investigation of the effect of reinforcement and wall configurations on the energy dissipation characteristics of tested walls. For this purpose, normalized cumulative dissipated energy diagrams of M-50-C, M-50, M-50-CP, S-50-C and S-50-CP walls are plotted in Figure 6.14. From this figure, it can be seen that walls with cramps and pins (M-50-CP and S-50-CP) exhibit the highest energy dissipation capability. The unreinforced masonry wall

M-50 seems to dissipate more energy than the M-50-C wall with cramps. However, in overall, M-50-C dissipates more energy until the ultimate state is reached. Interestingly, if the S-50-C and S-50-CP walls without rubble masonry core are compared with their multi-leaf counterparts (respectively M-50-C and M-50-CP walls), it can be observed that single-leaf walls have better energy dissipation capability. This unexpected result is similar to what has been observed during the ductility factor comparisons. Single leaf walls consisting of single good quality material has smaller equivalent yield displacement values and less cross section area so that elastic energy absorption is much smaller than multi-leaf walls, so that during the normalization process the cumulative energy is divided by a smaller value. Thus, at least numerically they seem to have better energy dissipation.

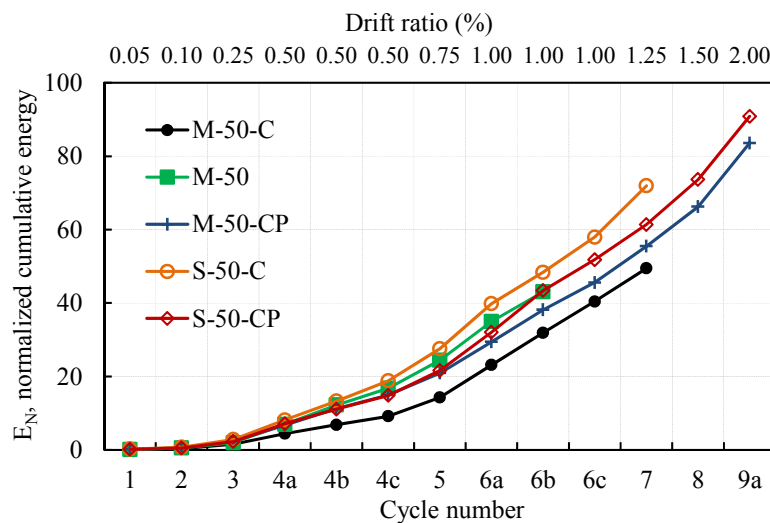


Figure 6.14 : Variation of normalized cumulative energy for M-50-C, M-50, M-50-CP, S-50-C and S-50-CP walls.

One of the main measures of energy dissipation, that is also one of the components of equation of motion, is damping. Seismic energy transmitted to structures can be dissipated through different damping mechanisms that might arise from external, internal and foundation based sources. External sources might include fluid interaction and radiation, whereas foundation damping may occur due to transfer of energy from the vibrating structure to the soil through the foundations. Internal sources of damping can be classified into two main groups, namely natural and supplemental damping. Both groups may include hysteretic, friction and viscous damping components. Apparently, supplemental damping requires utilization of energy dissipation devices. Natural damping arises as the main source of damping

since external damping is negligible compared to other types of damping and foundation damping can also be neglected if soil is infinitely rigid (Elnashai and Sarno, 2008). Consequently, among different types of damping, hysteretic and friction damping are the dominant types encountered in masonry structures. In the case of reinforced concrete structures, friction damping is generally available when the member is cracked and friction occurs along the crack plane. However, due to their nature, masonry walls (especially walls with dry joints) already inhibit friction interfaces in-between the units in the form of head and bed joints.

In order to conform to the mathematical representation of equation of motion, damping is generally expressed in the form of an equivalent viscous damping coefficient c_{eq} that can be expressed in terms of equivalent damping ratio ξ_{eq} and critical damping coefficient c_{cr} . It should be noted that critical damping coefficient c_{cr} is the smallest amount of damping for which no oscillation occurs in free dynamic response (Priestley and Seible, 1996).

Equivalent damping ratio ξ_{eq} can be obtained by using Eqs. (6.8) and (6.9), where $E_{dis,i}$ is the dissipated energy per cycle and E_{es} is the elastic strain energy stored in an equivalent linear elastic system under static conditions (Shibata and Sozen, 1976). It should be noted that, in this study during the calculation of E_{es} elastic strain energy, average peak load and displacement values (H_i and d_i) of pushing and pulling directions are utilized.

$$\xi_{eq} = \frac{E_{dis,i}}{4\pi E_{es}} \quad (6.8)$$

$$E_{es} = \frac{H_i d_i}{2} \quad (6.9)$$

These energy parameters are schematically shown in Figure 6.15, which represents any random cycle obtained during the cyclic displacement reversals. Accordingly $E_{dis,i}$ is equal to the area enclosed by the investigated load-displacement cycle and E_{es} elastic strain energy is the area of the triangle below the line that connects the peak with the origin.

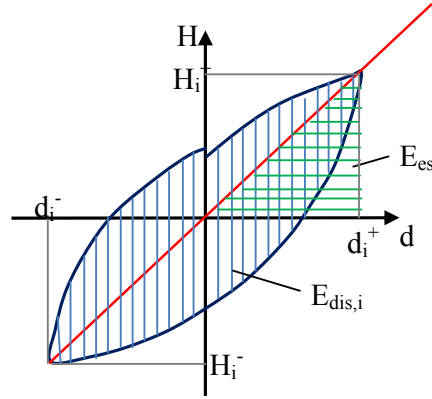


Figure 6.15 : Hysteretic energy dissipation (modified from Priestley et al. (1996)).

Diagrams for the values of the equivalent damping ratio ξ_{eq} at each cycle of M-25-C, M-50-C, M-75-C and M-100-C walls are presented in Figure 6.16. Similarly, the equivalent damping ratios at each cycle of M-50-C, M-50, M-50-CP, S-50-C and S-50-CP walls are compared in Figure 6.17.

Although no clear tendency can be obtained between the equivalent damping ratio and the investigated parameters (such as the pre-compression stress level and existence of reinforcement and rubble core), still interesting observations can be made. Considerable equivalent damping ratios ranging between 8 and 21% were obtained even for the initial cycles in the pre-peak regions (i.e. drift ratios less than 0.50%) of the walls. This shows that small displacements can lead to permanent deformations among the wall. The damping ratios had an increasing trend and climbed up to 38% at the further drift ratios where significant residual deformations occurred during the load reversals. The second and third subsequent cycles at 0.50 and 1.00% drift ratios yielded either to constant or decreasing equivalent damping ratios.

Though these equivalent hysteretic damping ratio values may sound considerably high with respect to other structural systems such as reinforced concrete structures, similar percentages are also indicated for masonry in different sources. Costa et al. (2012) points to a value of 12% at the initial cycles and reports approximately 25% equivalent damping ratio after formation of the diagonal cracks on the double-leaf stone masonry wall with poor infill. Mazzon (2010) indicates values starting from 8%, reaching 30% at ultimate displacement for three-leaf rubble stone masonry specimens. Magenes and Calvi (1997) gives equivalent hysteretic damping ratios increasing from 5% to 16% for brick masonry walls with shear cracking failure

mode, while lower values in the order of 5% are pronounced for specimens with flexural failure mode. ElGawady et al. (2007) indicates values for hollow brick masonry walls that are starting from 5% and jumping to 45% after corner crushing. The New Zealand NZSEE (2012) recommendations proposes an overall equivalent viscous damping ratio of 15% of critical for both the force based and displacement based methods of URM masonry assessment analysis.

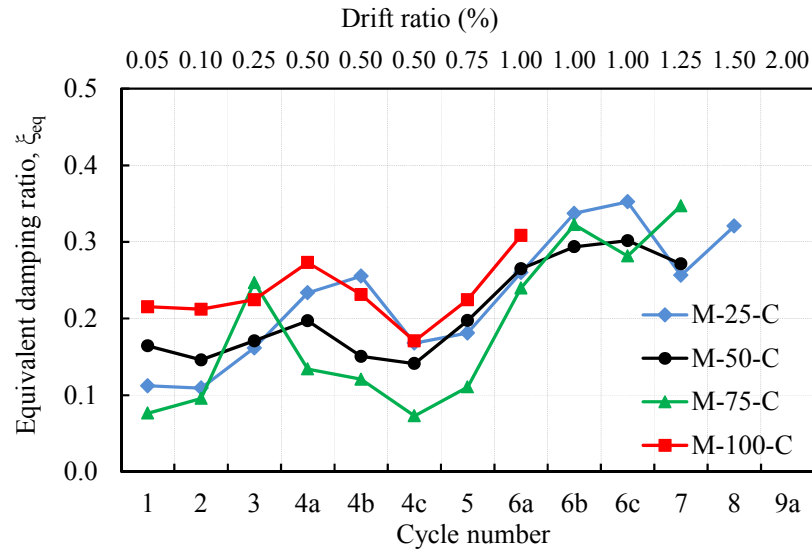


Figure 6.16 : Variation of equivalent damping ratio for M-25-C, M-50-C, M-75-C and M-100-C walls.

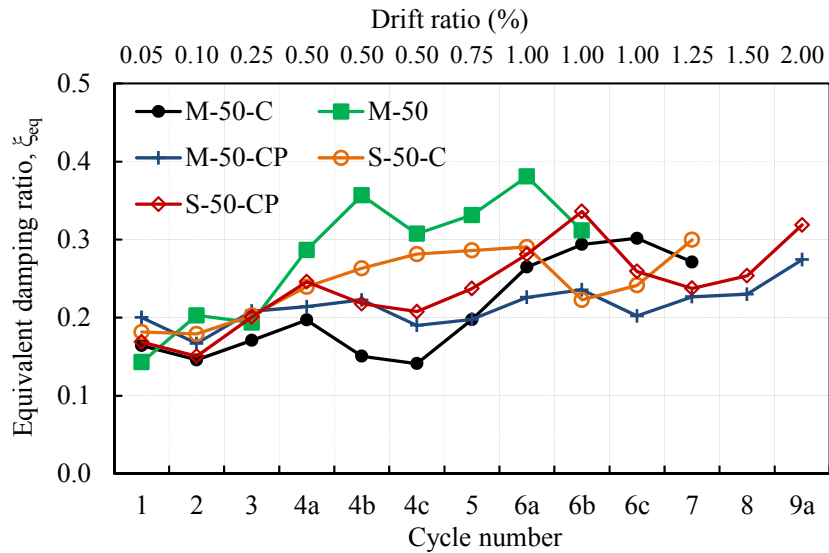


Figure 6.17 : Variation of equivalent damping ratio for M-50-C, M-50, M-50-CP, S-50-C and S-50-CP walls.

6.5 Mechanical Characteristics Obtained From Shear Compression Tests

The experimental program, carried out on the investigated masonry typology, enabled further evaluation of a number of main mechanical characteristics. The pre-compression loading on the shear compression test wallets led way to the measurement of Young's modulus. Moreover, the data collected during the shear loading supplied base for calculation of parameters such as shear strength, tensile strength and shear modulus.

6.5.1 Young's modulus

As also mentioned in Chapter 3, at the initial stage of shear compression tests, the pre-compression load was slowly applied until the desired pre-compression stress level was achieved and then unloaded to almost zero axial force. These loading and unloading cycles were done for three times. During the loading and unloading cycles vertical deformation data was collected via six vertical LVDTs (with an approximate gauge length of 800 mm), so that the Young's modulus of wallets could be calculated. Although their reliability is questionable due to the utilized axial stress range (that was quite lower than the compressive strength of the walls), the Young's modulus values obtained from the pre-compression loading phase are tabulated in Table 6.4 and a typical stress-strain curve obtained from M-50-C wall is presented in Figure 6.18.

Table 6.4 : Mechanical characteristics obtained from shear compression tests.

Wall	Pre-compression stress	E_{wc}	τ_{max}
	(MPa)	(MPa)	(MPa)
M-25-C	0.25	676	0.15
M-50-C	0.50	952	0.25
M-75-C	0.75	2470	0.34
M-100-C	1.00	1440	0.38
M-50	0.50	906	0.23
M-50-CP	0.50	921	0.27
S-50-C	0.50	2008	0.23
S-50-CP	0.50	1590	0.26
Average (Multi-leaf)		1228	
Average (Single-leaf)		1799	

As clearly seen from Figure 6.18, the initial loading cycle caused a certain amount of plastic deformations due to the over closure of the geometric tolerances at the bed joints of the ashlar leaves. However, the following axial loading cycles did not accumulate considerable axial deformations.

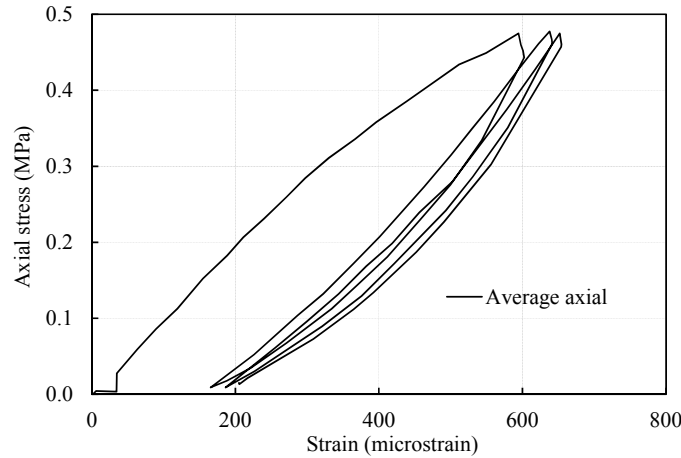


Figure 6.18 : Typical axial stress-strain curve obtained in the pre-compression stage.

The Young's modulus values obtained from the pre-compression loading stages of the shear compression tests of the multi-leaf walls exhibited a tendency to increase as the axial stress level increased. Lourenço et al. (2005) reports a similar observation for dry joint stone masonry specimens and explains the dependency of the joint stiffness to the axial stress by the existence of geometric tolerances and the absence of a levelling mortar between the stone units. Accordingly, as the axial stress increases, the imperfections between the stone units are crushed leading to increased contact area and, consequently, increased joint stiffness.

The only outlier of this tendency stemmed from the M-75-C specimen, which was the first tested specimen. During the establishment of the test setup, this specimen was subjected to small but accidental axial and lateral loads. Consequently, a pre-test joint closure between the stone units was already available and the relatively higher Young's modulus of this specimen can be attributed to this. The Young's modulus of the M-100-C, which was less than that of M-75-C was also verified by an additional loading performed on the M-25-C wall.

Although meaningful values were obtained from the axial pre-loading phase, the obtained average Young moduli were somehow different from the masonry prism uniaxial compression tests. In the case of the single-leaf specimens, a smaller

average Young's modulus value was derived during the pre-loading of the shear compression tests (1799 MPa versus 2615 MPa); whilst it was larger in the multi-leaf walls (1228 MPa versus 576 MPa). This contradiction is not a surprise since the Young's moduli of the shear compression specimens were calculated at the very beginning part of the actual uniaxial stress-strain curves, where the settlement of the joints occurs. The pre-compression stresses at this initial zone were far from the stress range utilized in the evaluation of uniaxial compression tests (about 20 to 50% of the peak stress).

6.5.2 Shear strength

Shear strength is a key mechanical parameter for evaluation of the stress states in masonry walls. The values obtained from the shear compression test specimens are tabulated in Table 6.4. In this table, shear strength values are calculated by simply dividing the maximum lateral resistance $H_{\max}$ (average of pushing and pulling directions) by the gross cross-section area of the walls. The shear strengths, summarized in Table 6.4 and visualized in Figure 6.19, clearly shows that the level of axial stress highly influence the shear strength of the investigated type of walls. The shear strength increases as the axial stress increases. The following expression utilized for the Mohr-Coulomb model can be proposed from the test results of four identical specimens with varying axial stresses between 0.25 and 1.00 MPa (M-25-C, M-50-C, M-75-C and M-100-C):

$$\tau_{\max} = 0.09 + 0.30\sigma \quad (6.10)$$

where $\tau_{\max}$ is the shear strength of masonry and σ is the pre-compression stress. Apparently, an analogy with the expressions given by the codes such as Eurocode 6 (CEN (2005)) and Turkish Seismic Design Code (2007), shows that the 0.09 MPa value corresponds to the shear strength under zero compression strength and 0.30 value corresponds to the constant that defines the contribution of compression stresses (global friction coefficient).

Five walls with different configurations were tested under the same 0.50 MPa axial stress level and the results of the tests in terms of shear and axial stresses are presented in Table 6.4 and Figure 6.20. Three of the walls had a multi-leaf structure (M-50, M50-C and M-50-CP) while the other two had only ashlar outer leaves (S-50-

C and S-50-CP). Among the multi-leaf walls, M-50-CP wall with cramps and pins exhibited the highest shear strength, followed by the M-50-C wall with cramps and M-50 wall without any reinforcement. The increase in shear strength due to the reinforcement was not dramatically high (17% increase between the M-50 and M-50-CP walls). In the case of single-leaf walls, the increase between the shear strengths of S-50-C (with cramps) and S-50-CP walls (with cramps and pins) was 13%. Single-leaf wall with cramps (S-50-C) was comparable with the multi-leaf wall without any reinforcement (M-50), whilst single-leaf wall with cramps and pins (S-50-CP) was between the multi-leaf walls with cramps (M-50-C) and with cramps and pins (M-50-CP).

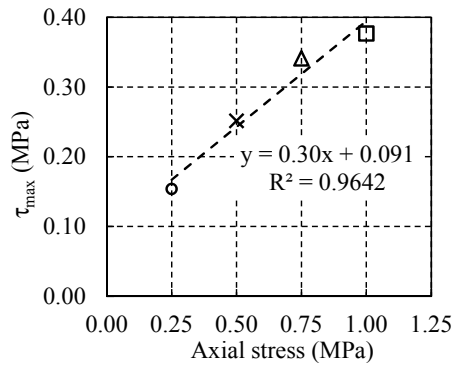


Figure 6.19 : Variation of shear strength with axial stress level.

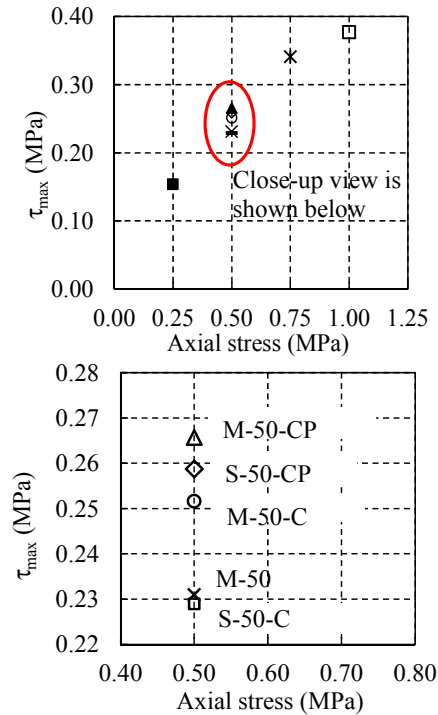


Figure 6.20 : Influence of wall typology on the shear strength.

6.5.3 Tensile strength

Although no diagonal compression tests were planned in scope of this study, valuable data on tensile strength of tested masonry typology can still be evaluated from the shear compression tests. For derivation of the tensile strength from the lateral load data, the equation proposed by Turnsek and Cacovic (1971) and modified by Turnsek and Sheppard (1980) can be utilized. This approach considers the masonry as an elastic, homogenous and isotropic structural element. Under the above-mentioned assumptions, the two dimensional stress states in masonry can be easily visualized by utilizing the Mohr circle as schematically shown in Figure 6.21.

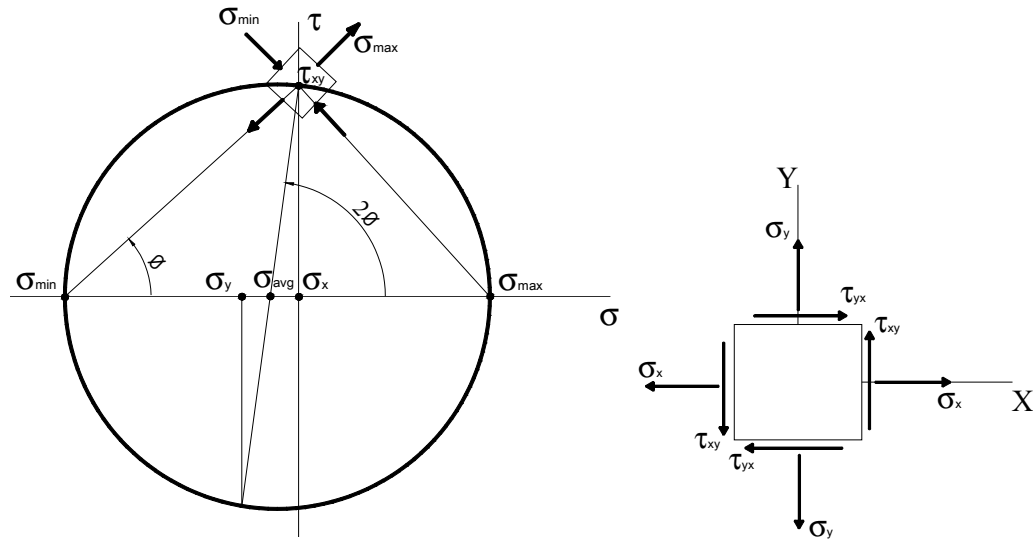


Figure 6.21 : Principal stresses at the given coordinate system.

In this figure, $\sigma_{\max}$ and $\sigma_{\min}$ are the maximum (tensile) and minimum (compressive) principal stresses that would develop in the middle section of the wall (Tomazevic, 1999) and can be evaluated by utilizing Eq. (6.11):

$$\sigma_{\max, \min} = \frac{\sigma_x + \sigma_y}{2} \pm \sqrt{\left(\frac{\sigma_x - \sigma_y}{2}\right)^2 + (\tau_{xy})^2} \quad (6.11)$$

If the σ_y and τ_{xy} are substituted with the average axial (σ) and shear (τ) stresses acting on the wall, respectively, and σ_x is assumed as zero, principal compressive and tensile stress expressions become:

$$\sigma_{\min} = \frac{\sigma}{2} - \sqrt{\left(\frac{-\sigma}{2}\right)^2 + (\tau)^2} \quad (6.12)$$

$$\sigma_{\max} = \frac{\sigma}{2} + \sqrt{\left(\frac{-\sigma}{2}\right)^2 + (\tau)^2} \quad (6.13)$$

For the sake of simplicity and compatibility with the expression proposed by Turnsek and Sheppard (1980), the negative sign of the average axial stress (σ) can be inverted so as to have positive sign in the case of compressive axial loading and denoted as (σ_o). In this case, the principal tensile stress can be assumed as the tensile strength of masonry wall (f_{wt}) and obtained by the following expression:

$$f_{wt} = \sqrt{\left(\frac{\sigma_o}{2}\right)^2 + (\tau)^2} - \frac{\sigma_o}{2} \quad (6.14)$$

Turnsek and Sheppard (1980) modified Eq. (6.14) by a shear stress distribution factor (b) that depends on the geometry (slenderness) of the wall:

$$f_{wt} = \sqrt{\left(\frac{\sigma_o}{2}\right)^2 + (b\tau)^2} - \frac{\sigma_o}{2} \quad (6.15)$$

The stress distribution factor b introduced in Eq. (6.15) is equal to height/width aspect ratio for ratios between 1.0 and 1.5. In case, the aspect ratio is larger than 1.5 it is assumed as 1.5.

Additionally the principal plane orientation (ϕ) can be evaluated by:

$$\phi = 0.5 \arctan \frac{2\tau}{\sigma_o} \quad (6.16)$$

The tensile strength, principal compressive stresses and principal plane orientation values obtained from Eqs. (6.15), (6.12) and (6.16), respectively, are presented in Table 6.5. During the implementation of above expressions, average axial stress (σ_o) is taken as the pre-compression stress acting on the wall. Average shear stress (τ) is simply obtained by dividing the horizontal load value at which the first diagonal

crack occurs to the gross horizontal area of the wall. Since height/length ratio utilized in this study was equal to 1.0, stress distribution factor can be taken as 1.0.

Table 6.5 : Principal stresses obtained from shear compression tests.

Wall	Pre-compression stress	f_{wt}	σ_{min}	ϕ
	(MPa)	(MPa)	(MPa)	(Degree)
M-25-C	0.25	0.07	0.32	25
M-50-C	0.50	0.10	0.60	23
M-75-C	0.75	0.13	0.88	21
M-100-C	1.00	0.13	1.13	19
M-50	0.50	0.09	0.59	21
M-50-CP	0.50	0.11	0.61	23
S-50-C	0.50	0.09	0.59	21
S-50-CP	0.50	0.11	0.61	23
Average (COV)		0.10 (0.19)	0.67 (0.36)	22 (0.09)

The tensile strength of masonry (f_{wt}) calculated by using the Turnsek and Sheppard (1980) approach yields to an average value of 0.10 MPa with a coefficient of variation of 19%. Although it might be deceiving to reach certain judgments at such low values, it can be observed that the tensile strength tends to increase with the increase in the pre-compression stress level (almost 86% between M-25-C and M-100-C). As also indicated by Calderini et al. (2010), in shear compression tests, different values of f_{wt} can be obtained depending on the test type (i.e. diagonal compression and shear compression tests) and stress field associated (i.e. different values of axial loads). The presence of reinforcement in the form of cramps and pins only slightly increases the tensile strength, since the cracking of the stone units controls it. Finally, the internal core with very low tensile strength almost does not have any effect on the masonry tensile strength.

The principal compressive stresses that appear at the middle of the wall tend to increase with the increasing pre-compression stress level (between 0.32 and 1.13 MPa). In all cases, they stay below the 3.71 MPa uniaxial compressive strength of three-leaf masonry. However, it should be noted that, the existence of the complex stress distribution at the middle of the wall makes this statement doubtful. Moreover, the crushing of the masonry along the diagonal directions, observed during the tests, back these doubts.

Finally, the angle of principal stresses exhibit a tendency to decrease as the pre-compression stresses increase, which means that the cracks are likely to form in a more vertical alignment. Since, during the tests, the damage was widely distributed among the wall faces due to the presence of cramps and pins, comparison of principal stress orientations with the actual crack directions is not an easy task. However, still a vague tendency in crack directions similar to that of calculated angles of principal stresses is visible in Fig. 5.1.

6.5.4 Shear modulus

An attempt for evaluation of shear modulus, which is a key mechanical parameter, can be done via further analysis of experimental data. Apart being an important parameter, shear modulus plays a major role in determination of the stiffness that determines the distribution of shear stresses among the walls of the masonry structures.

In this study, experimental data was utilized during the calculation of the shear modulus values of the tested walls. For this purpose, the global shear modulus, which includes the rotation and shear components of the wall, was evaluated. A simple methodology, that was also defined by ASTM E 564-06 (2006) for lightly framed walls, was followed. Accordingly, the global shear modulus can be obtained as a ratio of shear stress (τ) to shear rotation (γ). Here, shear stress is assumed to be uniformly distributed among the wall cross-section and rotation angle is taken as ratio of horizontal displacement to wall height (Δ and a in Figure 6.22, respectively).

ASTM E 564-06 (2006) indicates that the values for shear stress and rotation angle can be selected at a reference load or deflection that is within acceptable performance levels. One recommended reference load is 33% of the maximum lateral resistance $H_{\max}$. In addition to that, other reference load levels or net wall deflections may also be used. Following these recommendations, shear modulus values at 1/3 of the maximum lateral resistance ($G_{1/3}$) and in a range of 30 and 60% of maximum lateral resistance (G_{30-60}) are calculated and presented in Table 6.6. Once more, it should be noted that, these shear modulus values are global values since the total lateral displacement of the walls consists of shear and flexure components. However, they still provide valuable information for determination of lateral stiffness.

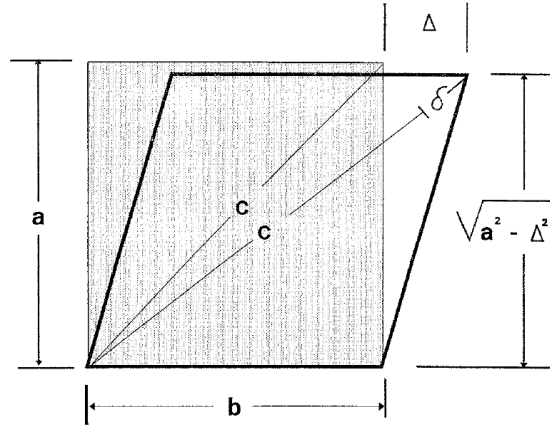


Figure 6.22 : Geometric parameters for shear modulus (ASTM E 564-06, 2006).

A comparison of the shear modulus values presented in Table 6.6 clearly shows that lower pre-compression stress levels lead to smaller shear moduli. Moreover, existence of cramps does not significantly influence the shear modulus. However, existence of pins results with higher moduli. Finally, the inner rubble masonry core causes a considerable increase in the global shear stiffness of the investigated type of walls. Shear modulus values calculated in the range of 30% and 60% of maximum lateral resistance (G_{30-60}) become slightly less than the values calculated at 1/3 of the maximum lateral resistance ($G_{1/3}$).

Table 6.6 : Shear modulus values of tested walls.

Wall	Pre-compression	$G_{1/3}$	G_{30-60}
	(MPa)	(MPa)	(MPa)
M-25-C	0.25	103	102
M-50-C	0.50	205	155
M-75-C	0.75	243	223
M-100-C	1.00	418	377
M-50	0.50	215	195
M-50-CP	0.50	350	291
S-50-C	0.50	127	126
S-50-CP	0.50	230	196
Average (Multi-leaf)		256	224
Average (Single-leaf)		179	161

6.6 Failure Mode Identification

The failure modes of masonry walls subjected to in-plane shear forces mainly depend on the geometry of the wall (squat or slender), the boundary conditions, level of pre-

compression stress, mechanical characteristics of the constituents, masonry geometrical characteristics (such as masonry unit aspect ratio and masonry pattern) and the reinforcement configuration (if available). The damage patterns observed in masonry shear walls can be schematically represented as shown in Figure 6.23.

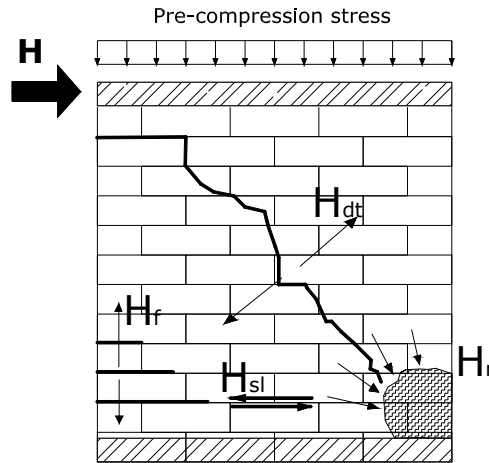


Figure 6.23 : Damage patterns of masonry shear wall.

Accordingly, horizontal flexural cracks appear at a lateral load of H_f , particularly in the case of low to moderate pre-compression stress levels. The horizontal cracks might be located either at single course or be distributed among the height of the wall. After tensile flexural cracking, masonry pier can behave as a nearly rigid body rotating about the toe (rocking). In the case of higher vertical loads, sub-vertical cracks oriented towards the more compressed corners may appear (crushing). In both rocking and crushing cases, the ultimate state is achieved by failure at the compressed corners (H_r in Figure 6.23), Calderini et al. (2010). In the case of slender multi-leaf walls, detachment of the leaves may appear in the compressed region.

Diagonal shear cracking (H_{dt} in Figure 6.23) might cause ultimate limit state particularly at higher levels of pre-compression. The diagonal cracks are oriented in the direction of the principal tensile stress and depending on the material qualities of unit and mortar and geometry of the units; they can be located within the head and bed joints (step like cracking) or can pass along the units (Bosiljkov, 2006).

Sliding (H_{sl} in Figure 6.23) represents a failure mode generally observed at very low levels of pre-compression, particularly in the case of low cohesion supplied by the

mortar between the units of the wall. The sliding plane can be in single course of the bed joint or it can be in several courses (Bosiljkov, 2006).

The above-mentioned limit states can be further investigated numerically by employing failure models. In this study, two different failure models, namely Mann and Müller (1980) and Turnsek and Cacovic (1971) models that define the failure field of masonry with respect to shear and normal stresses, are utilized for comparison of the predicted and experimentally observed failure modes.

6.6.1 Mann and Müller (1980) model

Mann and Müller (1980) model, which has been adopted by the German codes DIN 1053-1 (1996) and DIN 1053-100 (2007), utilizes material characteristics of the constituents, compressive strength of masonry and the level of pre-compression in the compressed part of the wall to predict different types of failure modes (Bosiljkov et al., 2010). The failure envelope is represented by a set of curves mathematically formulated with respect to failure modes such as failure of bed joints due to friction (sliding failure), splitting tensile failure of the units and compressive failure of masonry.

The friction failure can be investigated in two versions, as shown in Figure 6.24 (Jäger and Schöps, 2008). The first friction failure corresponds to shear failure along the bottom bed joint of the masonry wall (H_{sl} in Figure 6.23) and can be expressed by the Mohr-Coulomb criterion:

$$\tau = \frac{l_c}{l} \tau_{vo} + \mu \sigma_d \quad (6.17)$$

where l_c is the length of the compressed cross-section area, l is the total length of the wall, τ_{vo} is the initial shear strength (cohesion), μ is the friction coefficient of the bed joint and σ_d is the average compressive stress over the compressed cross-section area of the wall.

Although the second type of friction failure is a diagonal tension failure (H_{dt} in Figure 6.23), it is also expressed via the Mohr-Coulomb criterion. However, this time, Mann and Müller (1980) model modifies the bed joint cohesion and friction parameters, so that corresponding parameters that are representative of global

behaviour are obtained. Accordingly, the following expressions can be utilized for calculation of the shear strength at first diagonal crack that passes through the joints:

$$\tau = \bar{\tau}_{vo} + \bar{\mu}\sigma_d \quad (6.18)$$

$$\bar{\tau}_{vo} = \frac{\tau_{vo}}{1 + \mu \frac{2h_u}{l_u}} \quad (6.19)$$

$$\bar{\mu} = \frac{\mu}{1 + \mu \frac{2h_u}{l_u}} \quad (6.20)$$

where $\bar{\tau}_{vo}$ and $\bar{\mu}$ are the reduced initial shear strength and friction coefficient parameters, respectively.

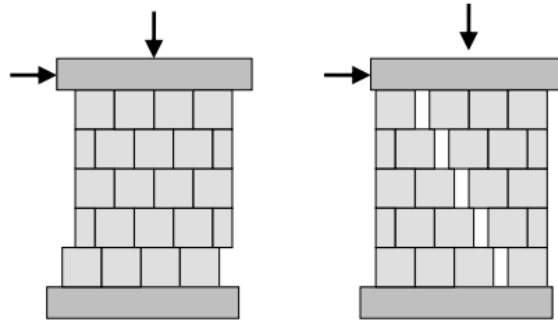


Figure 6.24 : Types of friction failure (Jäger and Schöps, 2008).

Mann and Müller (1980) developed also a criterion for the cracking of units. The criterion adopts the maximum principal stress acting in the centre of a unit as reference stress; so that it must not exceed the tensile strength of the unit itself (Calderini et al., 2010). The criterion for ripping of the units may be expressed as:

$$\tau = 0.45f_{bt} \sqrt{1 + \frac{\sigma_d}{f_{bt}}} \quad (6.21)$$

In this equation, f_{bt} is the unidirectional tensile strength of the units in the longitudinal direction and σ_d is the average compression stress over the compressed cross-section area of the wall. Length of the compressed cross-section area (l_c) of the masonry wall can be evaluated through the following equation for an assumption of linear stress-strain distribution in the compressed zone (Jager and Schöps, 2008):

$$l_c = \frac{3}{2} \left(l - 2e_{ini} - 2 \frac{Hh}{N} \left(\psi - \frac{1}{2} \right) \right) \quad (6.22)$$

where e_{ini} is the initial eccentricity of pre-compression load, H is the acting horizontal load, N is the pre-compression load, h is the height of the wall panel and ψ is a coefficient to define the support conditions (0.5 for fixed-fixed and 1.0 for cantilever support conditions).

In both rocking and crushing modes, failure occurs with attainment of the compressive strength of masonry (f_{wc}). It should be noted that, in some sources such as Graubner and Kranzler (2005) and Bosiljkov et al. (2010), this type of failure mode is not considered as a decisive component of the Mann and Müller failure envelope. Utilizing the beam theory by neglecting the tensile strength of the material and assuming a rectangular stress block at the compressed toe, the shear force at this limit state can be obtained by the following equation:

$$\tau = \frac{\sigma}{2\lambda_v} \left(1 - \frac{\sigma}{f_{wc}} \right) \quad (6.23)$$

where λ_v is the shear slenderness that depends on the type of the supports and becomes equal to h/l for cantilever (full restraint at bottom, free to rotate at top of the panel) and $0.5h/l$ for fixed-fixed (full restraint at top and bottom of the panel) type of support conditions.

Finally, the shear stress at limit state that corresponds to compression failure of masonry due to diagonal compressive stresses can be obtained by the following relationship (Graubner and Kranzler, 2006),

$$\tau = (f_{wc} - \sigma_d) \left(\frac{l_u}{2h_u} \right) \quad (6.24)$$

The resulting failure envelope of the Mann and Müller model, which incorporates the above-mentioned failure modes can be seen in Figure 6.25. It should be noted that, in the presented failure envelope (as also done by some other researchers) bending type of failure, which is decisive particularly in the case of slender walls, is not indicated.

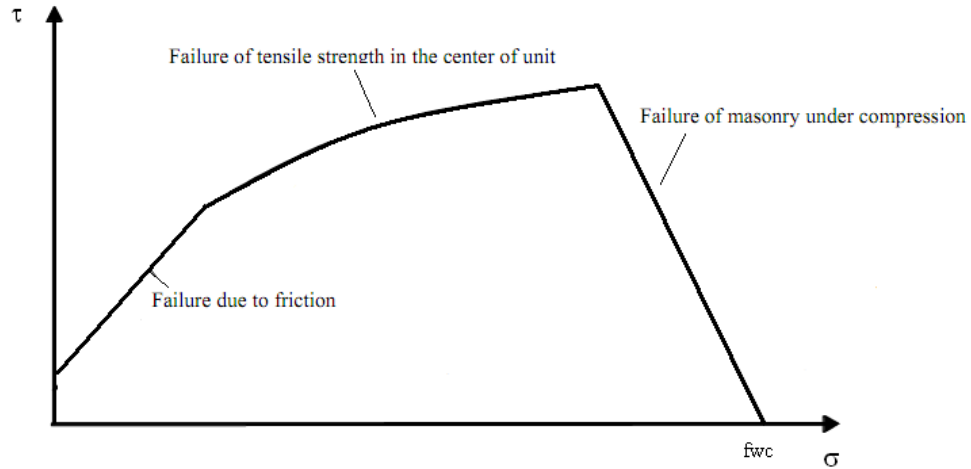


Figure 6.25 : Failure envelope of the Mann and Müller model (modified from Graubner and Kranzler (2005)).

With the substitution of the mechanical characteristics obtained through the tests performed on materials and masonry wallets that were summarized in Chapter 4, the failure field diagrams are derived as shown in Figures 6.26-6.29. Data points of single-leaf wall tests (S-50-C and S-50-CP) are plotted in Figures 6.26 and 6.27, whereas the plots for multi-leaf walls are presented in Figures 6.28 and 6.29. During the implementation of the Mann and Müller model to the shear compression tests, above mentioned equations were utilized with the following parameters: τ_{vo} initial shear strength (cohesion) as zero; μ friction coefficient of the bed joint as 0.76 (obtained from initial shear tests indicated in Chapter 4); h_u height of the stone units as 100 mm; l_u length of the stone units as 300 mm; f_{bt} unidirectional tensile strength of stone units as 1.58 MPa for single-leaf walls and 1.12 MPa for multi-leaf walls; f_{wc} compressive strength of masonry as 7.9 MPa for single-leaf walls and 3.7 MPa for multi-leaf walls and ψ coefficient (to define the support conditions) as 1. Unidirectional tensile strength of stone units f_{bt} was taken as half of the modulus of rupture value obtained from the stone unit bending tests for the case of single-leaf walls where the only constituent was the limestone. However, the weighted average value calculated from the tensile strengths of the stone units in the external leaves and rubble masonry in the inner core (assumed as 80% of the tensile strength from splitting tests) was employed for the f_{bt} value of the multi-leaf wall tests.

Average compression stress over the compressed cross-section area of the wall σ_d was utilized for marking of the data points on the Mann and Müller failure field diagrams, which resulted with slightly higher shear and normal stresses due to employment of reduced wall cross-section area instead of the gross cross-section area. For this purpose, the lengths of the compressed cross-section areas of the masonry walls were obtained by assuming a cantilever type of support condition. However, bending mechanism curves corresponding to fixed-fixed (full restraint at top and bottom of the panel) type of support conditions were also introduced to the diagrams, so that the influence of assumed support conditions can be easily noticed.

Before making comments on the diagrams it should be recalled that, during the tests, the observed damage evolution of the specimens were highly dependent on the pre-compression stress level applied on the walls. M-25-C wall with the least pre-compression stress exhibited a hybrid behaviour that was in between flexural and shear type of mechanism, or in other words, a shear type mechanism with clear signs of flexural behaviour was observed. In the case of the series of specimens with 0.50 MPa axial stress (M-50-C, M-50-CP, S-50-C and S-50-CP), specimens followed a similar damage evolution and failure, where the hybrid behaviour was on the shear side. The damage development of these walls initially began with flexural behaviour that caused horizontal cracks on the tension sides of the horizontal cross-section and vertical cracks in the compressed toe areas. At further displacements around the peak strength of the walls, diagonal cracks that were apparent signs of shear mechanism, could be observed. With the widespread development of diagonal shear cracks, that were mostly passing through the stone units, the effective cross-section area of the walls was remarkably reduced. Finally, the crushing around the toes due to increased compressive stresses led to complete failure of the walls. The only exception to this damage evolution was the development scheme of the diagonal cracks in the case of wall M-50. Since this multi-leaf wall specimen did not have any reinforcement (neither cramps nor pins), the friction failure at the bed joints caused stepped diagonal cracks. M-75-C and M-100-C walls, which were subjected to relatively higher pre-compression stresses, apparently exhibited a shear dominated failure mode. However, higher axial loads also caused earlier and easier crushing of the corners after attainment of the peak lateral strength and formation of several diagonal cracks.

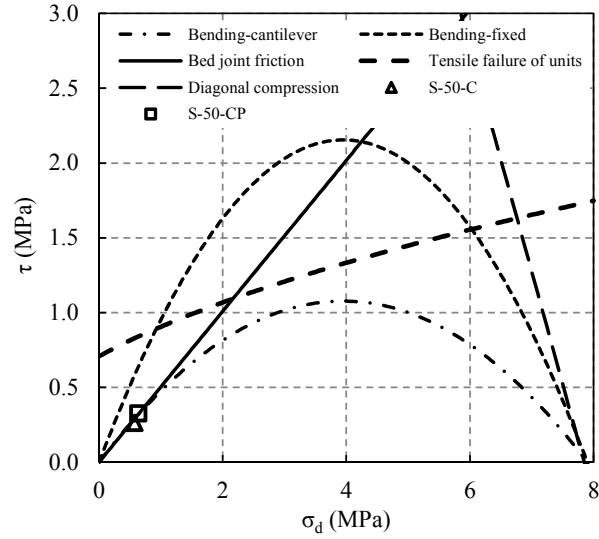


Figure 6.26 : Mann and Müller model for single-leaf specimens.

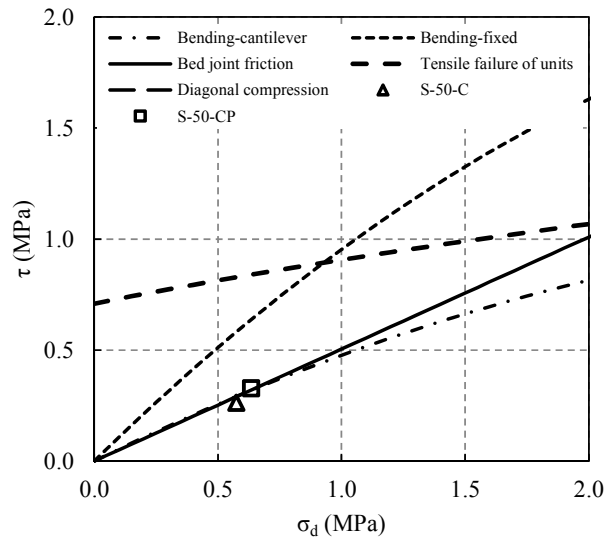


Figure 6.27 : Mann and Müller model for single-leaf specimens (close-up view).

Although the Mann and Müller model was originally developed for unreinforced brick masonry walls, still could successfully predict the above mentioned failure modes of tested specimens, as seen in Figures 6.26-6.29. Data points of all specimens were aligned along the initial rising part of the model that corresponded to loss of bed joint friction and bending failure limit curves, which successfully confirmed the hybrid behaviour observed during the displacement reversals. During the tests, all walls except the unreinforced M-50 specimen, for which the bed joint friction type of failure mode was predicted by the Mann and Müller model, had tensile failure in the units due to restraining effect of the cramps and pins.

Although, bending failure curve for the cantilever support condition seems to be decisive on the behaviour, it should be noted that the actual support condition during the tests was somewhere between the cantilever and fixed-fixed conditions. This observation also verifies the domination of shear damage at the attainment of peak lateral strength.

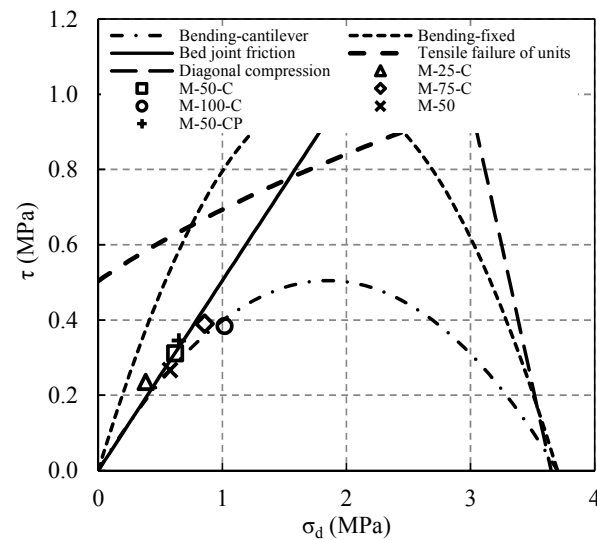


Figure 6.28 : Mann and Müller model for multi-leaf specimens.

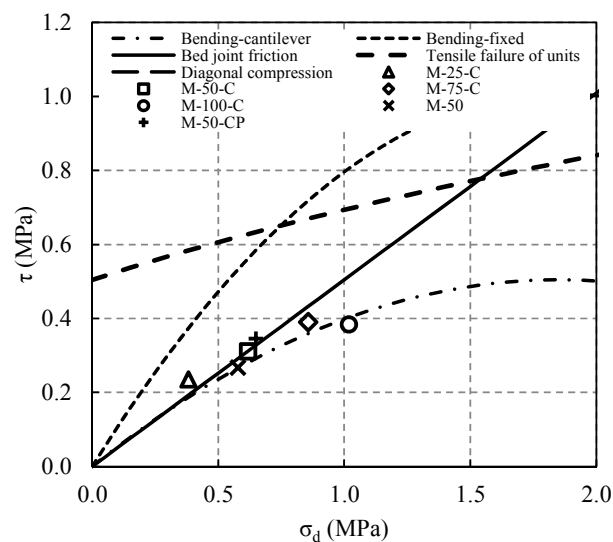


Figure 6.29 : Mann and Müller model for multi-leaf specimens (close-up view).

6.6.2 Turnsek and Cacovic (1971) model

Another widely utilized model for prediction of the shear strength of masonry panels was proposed by Turnsek and Cacovic (1971) and further modified by Turnsek and

Sheppard (1980). In this model, which has been incorporated into Slovenian and Italian codes for almost three decades, the masonry is treated as an elastic, homogenous and isotropic continuum (Bosiljkov et al., 2010). The shear strength of masonry is expressed as a function of diagonal tensile strength of masonry f_{dt} , the average pre-compression stress σ_o and the geometry of the masonry wall

$$\tau = \frac{f_{dt}}{b} \sqrt{1 + \frac{\sigma_o}{f_{dt}}} \quad (6.25)$$

where b is the shear distribution factor, $b = h / l$, $1.0 \leq b \leq 1.5$.

On the other hand, in our study, no diagonal compression tests were conducted and the tensile strength of masonry could only be evaluated by using the shear compression test results as presented in topic 6.5.3. However, since in this topic, the masonry tensile strength values were also derived by using the Turnsek and Cacovic (1971) model, substituting them back into Eq. (6.21) would only be a trivial solution. Consequently, in order to avoid this situation, the diagonal tensile strength of masonry f_{dt} was obtained as a percentage of the masonry compressive strength as indicated by Calderini et al. (2010). Accordingly, the diagonal tensile strength of masonry can be taken as 2.5% of the masonry compressive strength. Given the compressive strengths obtained through masonry prism compression tests carried out for the single- and multi-leaf wall configurations, this assumption yields to diagonal tensile strength values of 0.20 and 0.09 MPa for single- and multi-leaf walls, respectively.

The failure domains obtained for single-leaf walls are presented in Figures 6.30 and 6.31. Although hybrid behaviour with signs of shear and flexure failure modes were observed during the tests, the data points of the single-leaf walls coincided with the failure domain plotted for the bending type failure of a cantilever wall. However, it should be noted that it is not possible to make an exact classification for the support condition of the tested walls despite it was closer to cantilever type. Additionally, if the diagonal tensile strengths obtained from shear compression tests (presented in Table 6.5) are compared with the ones calculated from the masonry compressive strength, it can be seen that the tensile strengths are overestimated for the single-leaf walls (0.20 MPa versus 0.10 MPa); whereas they were close enough for the multi-

leaf walls (0.11 MPa versus 0.09 MPa). Given these two concerns, a margin for the prediction of the failure mode needs to be left and the slight difference observed in prediction of the failure mode by using Figures 6.30 and 6.31 needs to be anticipated.

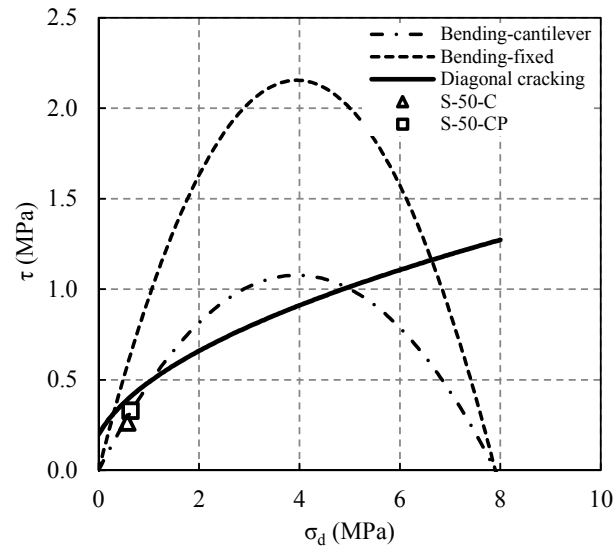


Figure 6.30 : Turnsek and Cacovic model for single-leaf specimens.

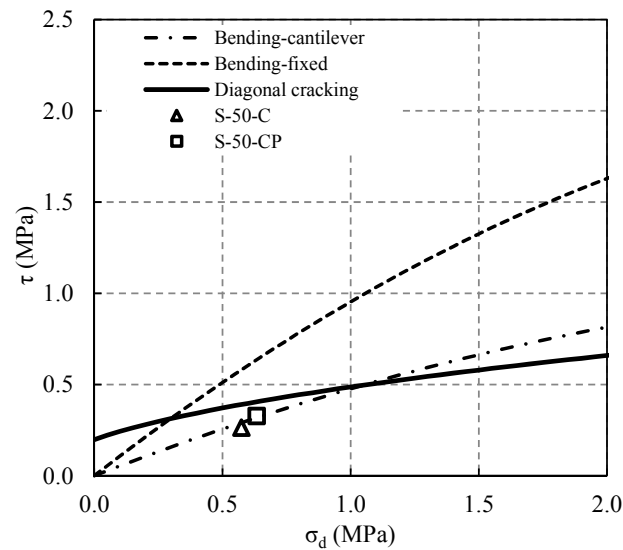


Figure 6.31 : Turnsek and Cacovic model for single-leaf specimens (close-up view).

The performance of the Turnsek and Cacovic model in prediction of the failure modes was better in the case of multi-leaf specimens, as seen in Figures 6.32 and 6.33. Similar to what has been observed during the tests and in the Mann and Müller model, the specimen with the least pre-compression stress level (M-25-C) fails under bending effects while the walls with an intermediate axial stress level (M-50, M-50-C and M-50-CP) coincides to a region where bending and diagonal cracking failure

modes intersect. Apparently, this region addresses to a hybrid type of failure mode for which shear is more pronounced. Finally, the data points of the multi-leaf walls with higher axial stresses (M-75-C and M-100-C) stayed slightly above the diagonal shear cracking failure limit, thus, indicated the shear type of failure mode observed during the tests.

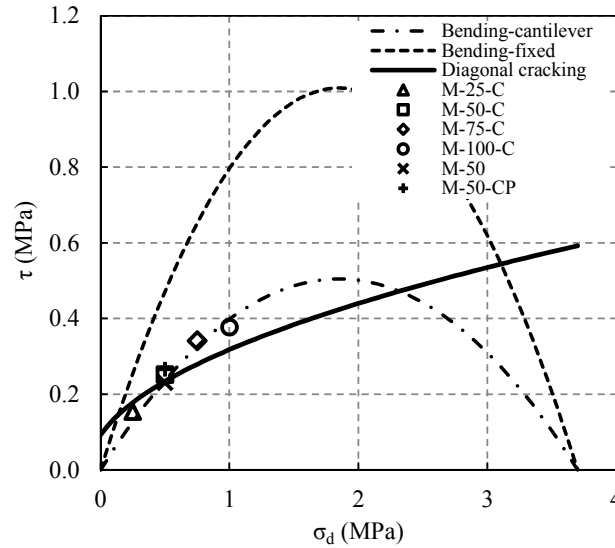


Figure 6.32 : Turnsek and Cacovic model for multi-leaf specimens.

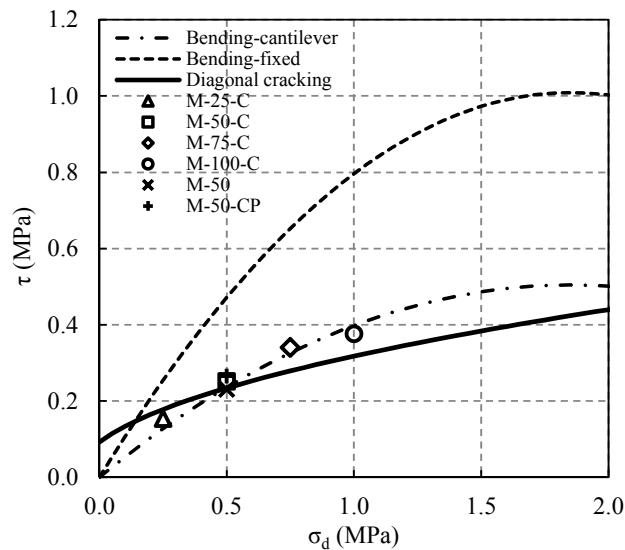


Figure 6.33 : Turnsek and Cacovic model for multi-leaf specimens (close-up view).

6.7 Drift Ratios Corresponding to Performance Levels

In this section, inelastic displacement capabilities of the investigated single- and multi-leaf stone masonry walls are discussed considering the performance based

design concepts. For this purpose, the damage recorded during the shear compression tests are evaluated in correlation with the lateral load-displacement envelope curves. However, it should be noted that, the investigated walls are encountered in the Ottoman Empire monumental structures, whose value and importance cannot be compared with the today's masonry structures. Consequently, different performance levels can be desired for these invaluable structures. However, in the current study, the definitions presented in the seismic assessment guidelines that approach the topic from the viewpoint of occupant safety are followed.

Although the performance based design and assessment of buildings is not a new concept for the academic society of structural engineering; only the last two decades have witnessed penetration of this approach to major seismic design and assessment documents such as ATC 40(1996), FEMA 273 (1997), FEMA 356 (2000), EN 1998-3 (2005) and Turkish Seismic Design Code (2007), that yielded to an increased number of applications in practice. In the performance based seismic assessment of existing structures, the performance objectives for the structure or the components of the structure are defined via limit states, which are shaped by engineering and socio-economic contexts. The majority of the standards define three Structural Performance Levels: Immediate Occupancy (IO), Life Safety (LS) and Collapse Prevention (CP). Other damage state names such as Damage Limitation (DL), Significant Damage (SD) and Near Collapse (NC) are also utilized by some documents such as EN 1998-3 (2005). Although the above mentioned performance levels are widely accepted, it should be noted that several other performance levels can be requested by the owner of the structure depending on the utilization purpose.

For masonry structures, Immediate Occupancy (IO) performance level corresponds to a post-earthquake damage state in which only very limited structural damage has occurred in the form of minor cracks. The load bearing members retain nearly all of their pre-earthquake strength and stiffness. Permanent drifts are very small and can be neglected. Although some minor structural repairs might be required, these would generally not be required prior to re-occupancy.

Life Safety (LS) performance level describes the post-earthquake damage state in which significant damage to the structure has occurred. Extensive cracking, noticeable in-plane offsets and minor out-of-plane offsets can be observed in the case of masonry structures. Despite the significant damage, at least technically, it may still

be possible to repair the structure; however, for economic reasons this may not be practical.

Collapse Prevention (CP) performance level corresponds to the post-earthquake damage state at which the building is heavily damaged, with very limited residual strength and stiffness. However, the structural components continue to carry gravity loads. In masonry structures, in addition to very extensive cracking, large permanent deformations occur. The structure may not be technically practical to repair and is not safe for re-occupancy (FEMA 356, 2000).

In the case of masonry structures, the above mentioned performance levels or damage states are generally associated with the drift ratio limits. As stated by Ghobarah (2004), the definition of comprehensive and realistic drift limits that are associated with known damage states remains as one of the important unresolved issues in performance-based design procedures. Consequently, the classification of the damage and incorporation with the performance levels require extensive experimental data, field observations and theoretical analyses.

A figure, compiled by Bosiljkov et al. (2010), summarizing the performance level definitions and damage states of URM from aspects of damage, drift, behaviour, operability, axial capacity and possible rehabilitation actions, can be seen in Figure 6.34. Accordingly, the residual cracks at the IO performance level are noticeable and can be easily filled. In the case LS performance level, the cracks define a medium level of damage whereas the cracks are extensive and lead to severe damage for the CP performance level. However, these definitions for crack widths need to be quantified for definition of damage states such as none, slight, moderate-heavy and severe-total. For this purpose, in the current study, the identification of the damage states was performed by considering the maximum crack widths of load bearing masonry members and their effects on the stiffness and strength values. A compilation of maximum residual crack width versus damage state data available in literature and a number of post-earthquake damage assessment approaches for masonry structures is presented in Table 6.7. The compilation includes the classification limits proposed by Turkish Catastrophe Insurance Pool (DASK; Ilki, 2011), Standards Australia (AS2870-1996, 1990), Rainer (1983), Anagnostopoulos and Moretti (2008) and Kaminentzky (1985). As seen in this table, except Anagnostopoulos and Moretti (2008) that points to slightly larger crack width limits,

the available sources recommend values which are close to each other. It should be noted that none of the sources indicated above specifically include the investigated type of multi-leaf masonry walls reinforced with cramps and pins.

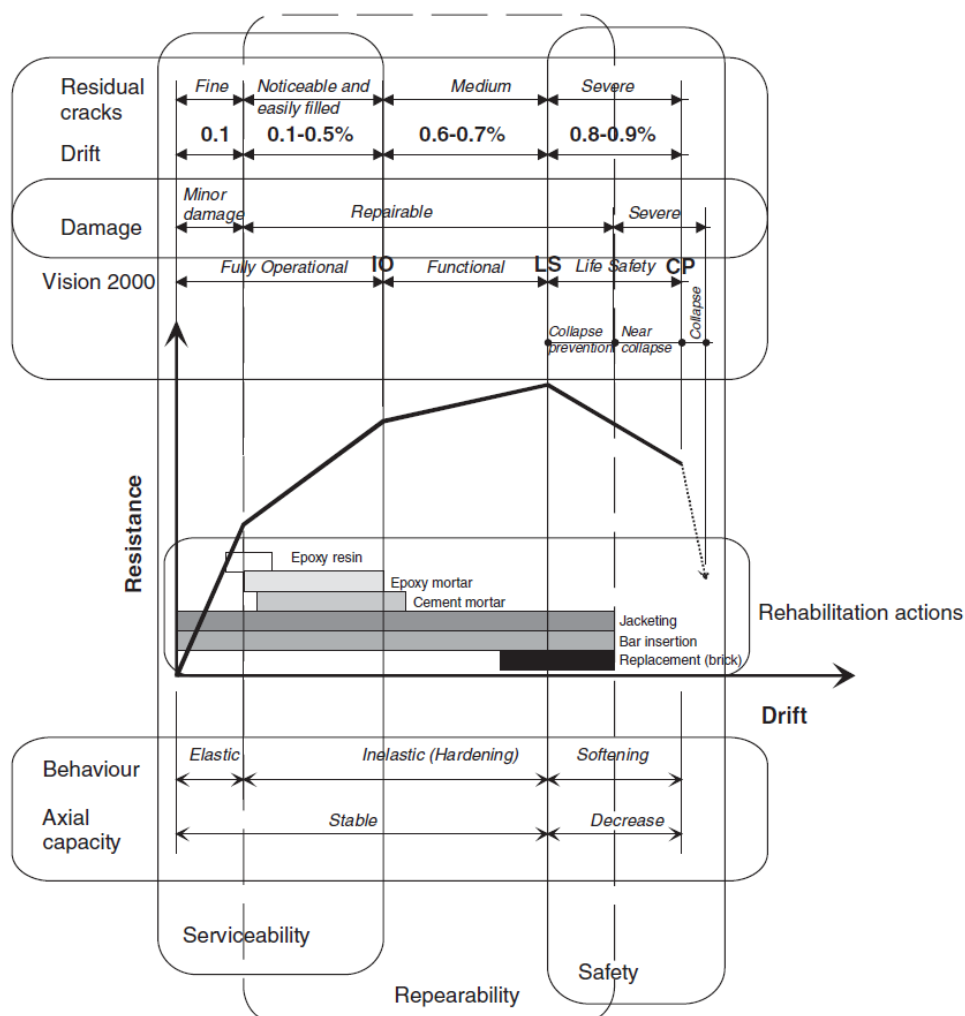


Figure 6.34 : Performance levels for URM (Bosiljkov et al., 2010).

Table 6.7 : Classification of damage based on residual crack widths.

Source	Crack width (mm)			
	None	Slight	Moderate-Heavy	Severe-Total
DASK (Ilki, 2011)	No crack	Hairline	Hairline-10.0	>10.0
AS2870-1996 (1990) and Rainer (1983)	<1.0	1.0-5.0	5.0-15.0	>15.0
Anagnostopoulos and Moretti (2008)	No crack	<3.0	3.0-5.0	>5.0
Kaminentzky (1985)	<0.8	0.8-3.2	3.2-12.7	>12.7

Considering the definitions of the limit states, the crack width limits reported in the literature and the damage evolution observed during the tests; in the current study, the following assumptions were made for designation of the Immediate Occupancy, Life Safety and Collapse Prevention performance levels of the tested walls:

- Immediate Occupancy (IO) performance level is exceeded either when the maximum crack width reaches 3.0 mm or the slope of the lateral load-displacement curve changes significantly, whichever occurs first.
- Life Safety (LS) performance level is reached if the lateral load capacity of the wall is attained, the maximum crack width exceeds 3.0 mm (but less than 10.0 mm) or crushing of the units initiates, whichever occurs first.
- Collapse Prevention (CP) performance level corresponds to the limit state where the specimen loses approximately 20% of its lateral load capacity, the maximum crack width exceeds 10.0 mm, or crushing and spalling of the units become apparent, whichever occurs first.

The drift ratios corresponding to IO, LS and CP limit states, obtained by considering the above listed assumptions, are tabulated for each specimen as seen in Table 6.8. At 0.50 and 1.00 % target drift ratios, for which three consecutive cycles were realized, the crack widths at the initial cycles were utilized unless crack widths increased significantly during the second and third cycles. Observations done during the tests revealed that damage growth was significant particularly at the consecutive cycles of 1.00% drift ratio, which generally stayed in the post-peak region of the lateral load-displacement curves. For instance, a 0.3 mm crack width at the first cycle of 1.00 drift ratio could increase more than ten times and exceed 3.0 mm at the third cycle of the same drift ratio. This observation indicates that the number of load or displacement reversals plays a very important role on the extent of cracking. Consequently, although it is very practical, definition of limit states solely depending on the drift ratios may mislead to overestimation of the performance.

For better perception, the damage state limits decided by using the methodology described above and presented in Table 6.8, are also shown on the pushing direction envelope load-displacement curve of each specimen, as seen in Figures 6.35-6.37.

Table 6.8 : Drift ratios of tested walls corresponding to performance levels.

Wall	Drift ratio (%)		
	IO-Slight	LS-Moderate-Heavy	CP-Severe-Total
M-25-C	0.25	0.50	1.25
M-50-C	0.20	0.75	1.25
M-75-C	0.20	0.60	1.00
M-100-C	0.15	0.35	0.80
M-50	0.25	0.40	1.00
M-50-CP	0.17	0.70	1.50
S-50-C	0.15	0.50	1.00
S-50-CP	0.15	0.75	1.25

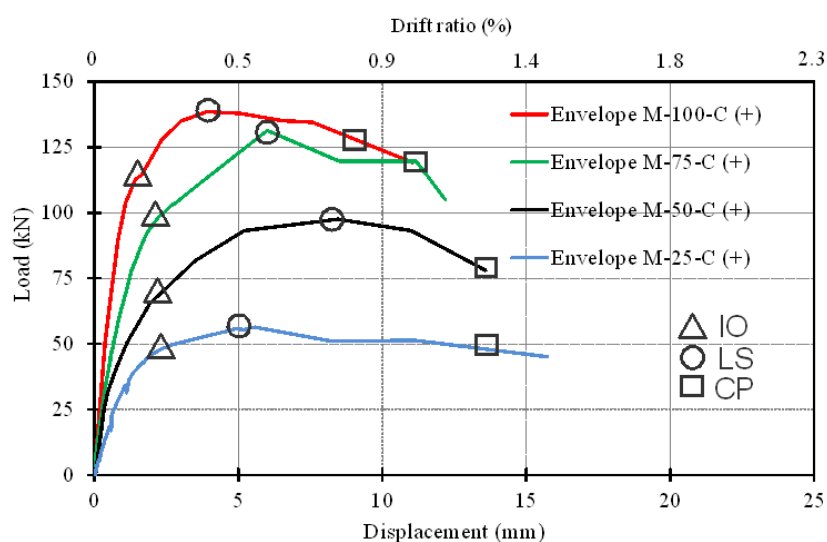


Figure 6.35 : Limit states for M-25-C, M-50-C, M-75-C and M-100-C.

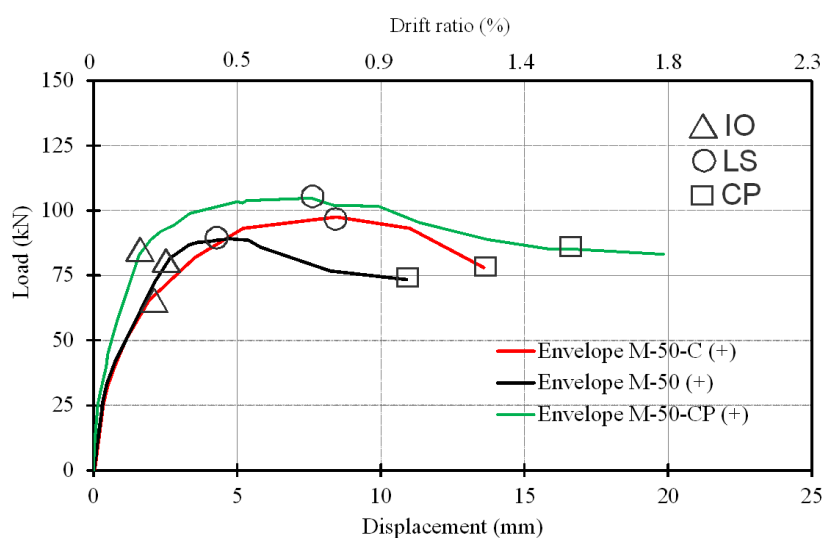


Figure 6.36 : Limit states for M-50-C, M-50 and M-50-CP.

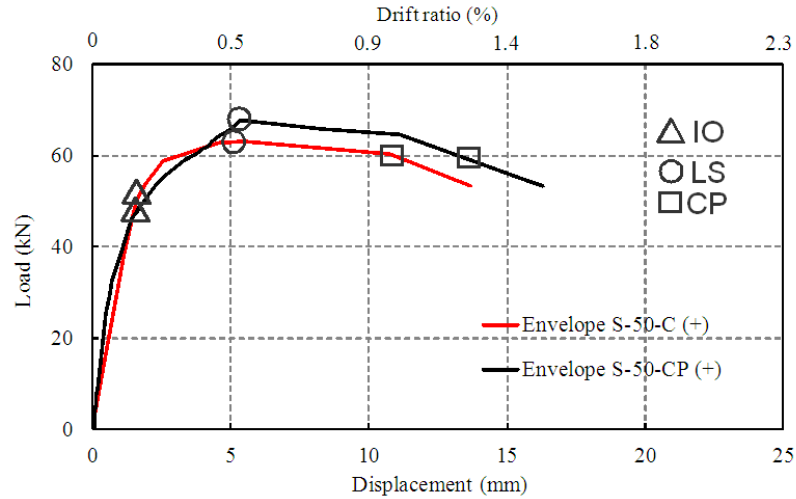


Figure 6.37 : Limit states for S-50-C and S-50-CP.

Accordingly, during the shear compression tests, the Immediate Occupancy limit state was reached around 0.20% drift ratio (varying between 0.15 and 0.25%). These points were mainly coinciding with the significant change in the initial slope of the load-displacement curves. The drift ratios corresponding to Life Safety limit state were scattered between the 0.35 and 0.75% drift ratio values. Finally, the Collapse Prevention state was achieved with the development of wide cracks and crushing of the stone units at drift ratios in between 0.80 and 1.50%.

As seen in Table 6.8 and Figure 6.35, increasing the level of axial stress generally lead to a decrease in the drift ratios corresponding to Life Safety and Collapse Prevention limit states, with the exception of the Life Safety limit obtained for the M-25-C wall. The introduction of the cramp or cramp and pin reinforcements to the multi-leaf walls considerably delayed the achievement of Collapse Prevention limit state (i.e. the M-50, M-50-C and M-50-CP walls in Table 6.8 and Figure 6.36). However, all limits states, except the Collapse Prevention state, corresponded to very close drift ratios for single-leaf S-50-C and S-50-CP specimens, as seen in Table 6.8 and Figure 6.37. In these specimens, similar to the case in multi-leaf walls, the Collapse Prevention limit state was reached at a slightly larger drift ratio, in case pins were introduced between the stone units.

Effect of the axial stress level, which is one main parameters in determination of the failure mode, on the drift ratios corresponding to IO, LS and CP performance levels is also investigated by using the drift ratio-precompression stress level (σ_o / f_{wc}) diagrams of multi-leaf cramp reinforced specimens as seen in Figure 6.38. Although

the number of specimens is limited, clear tendencies can be pointed out and equations can be proposed for relationships between precompression stress level (σ_o / f_{wc}) and drift ratio at IO, LS and CP performance levels (DR_{IO} , DR_{LS} and DR_{CP} , respectively) of investigated multi-leaf walls with cramp reinforcement. In the following equations, σ_o / f_{wc} ratios need to be substituted as percentage values and, consequently, drift ratios are obtained as percentage values.

$$DR_{IO} = -0.0044 \frac{\sigma_o}{f_{wc}} + 0.27 \quad (6.26)$$

$$DR_{LS} = -0.0027 \left(\frac{\sigma_o}{f_{wc}} \right)^2 + 0.083 \frac{\sigma_o}{f_{wc}} + 0.082 \quad (6.27)$$

$$DR_{CP} = -0.0011 \left(\frac{\sigma_o}{f_{wc}} \right)^2 + 0.013 \frac{\sigma_o}{f_{wc}} + 1.23 \quad (6.28)$$

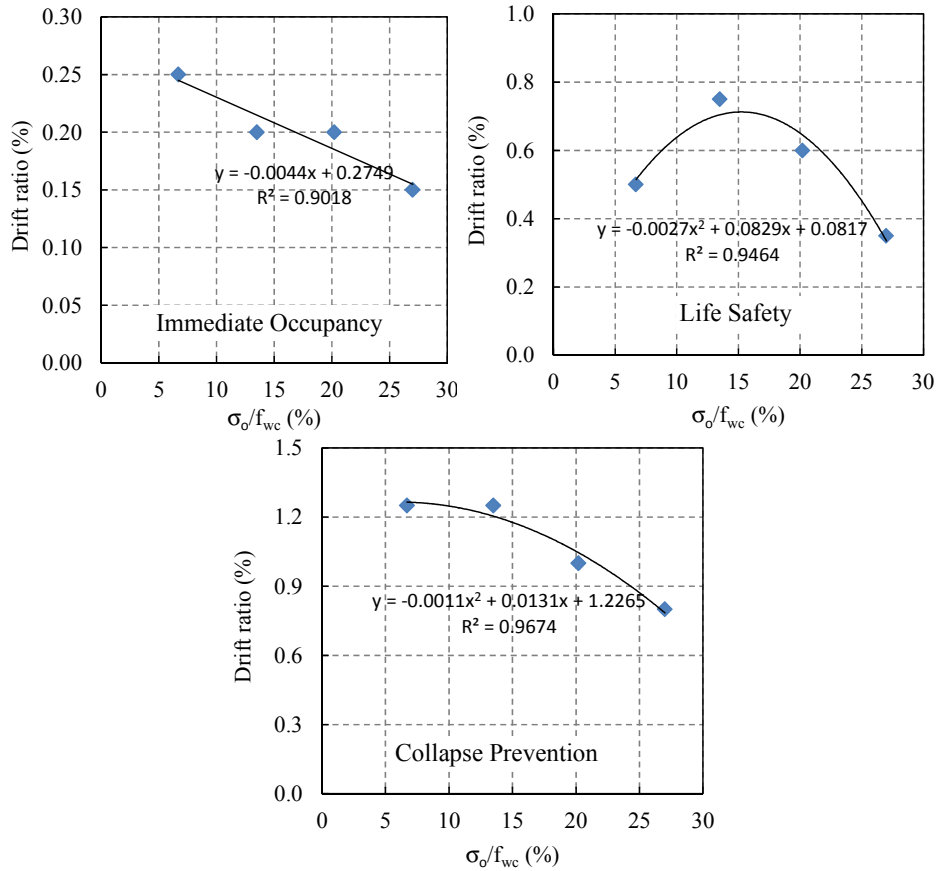


Figure 6.38 : Variation of drift ratio at limit states with respect to precompression.

In order to make comparisons and have a wider angle of view, limit state drift ratios of some other studies such as Tomazevic and Lutman (2007), Vasconcelos and Lourenço (2009), Galasco et al. (2010) and Costa et al. (2010), which were performed on stone masonry walls, are presented in Table 6.9. Although, some of these studies investigate the in-plane behaviour of multi-leaf walls, none of them includes reinforcements like cramps and pins. Percentage of acted precompression stress with respect to masonry compressive strength (σ_o / f_{wc}), observed failure modes and investigated wall typologies are also included in this table. It should be noted that since in the studies of other researchers, the crack widths are not given in detail, mostly the criterion concerning the lateral load-displacement curves were used for designation of the drift ratios corresponding to performance levels.

According to Table 6.9, in the current study, the Immediate Occupancy limit state is reached at drift ratios between 0.15-0.25%, whereas drift ratios between 0.20-0.30%, and 0.41-0.85% can be indicated for Tomazevic and Lutman (2007) and Vasconcelos and Lourenço (2009), respectively. It should be noted that, the axial stress to wall compressive strength ratio selected in the tests of Vasconcelos and Lourenço (2009) is much lower than the ones used in this study. Similarly, the Life Safety state is obtained at drift ratio values between 0.35-0.75%, 0.40-0.60%, 0.58-2.02%, 0.15-0.50% and at 0.50% as observed from the current study, Tomazevic and Lutman (2007) and Vasconcelos and Lourenço (2009), Galasco et al. (2010) and Costa et al. (2010), respectively. Finally, Collapse Prevention level corresponds to drift ratios between 0.80-1.50%, 1.00-1.20%, 0.86-3.12%, 0.86-3.12%, 0.31-1.92% and at 1.10 according to the current study, Tomazevic and Lutman (2007) and Vasconcelos and Lourenço (2009), Galasco et al. (2010) and Costa et al. (2010), respectively. For all limit states, the drift ratios obtained in this study point to a good agreement with the studies carried out by other researchers. At first glance, Vasconcelos and Lourenço (2009) seems as an exception with much larger values, however, if the axial load levels utilized in this study are investigated, it can be seen that they correspond to small percentages of the masonry compressive strength (maximum 6.8%). At these levels, the strength loss was controlled by flexural and rocking behaviour.

A closer look at the influence of axial stress level on the drift capacities at certain limit states can be done via the drift ratio-precompression stress level (σ_o / f_{wc}) diagrams presented in Figure 6.39. Accordingly, the drift ratios corresponding to

Immediate Occupancy, Life Safety and Collapse Prevention performance levels shows a strong tendency to decrease as the amount of axial stress with respect to masonry compressive strength. Bearing in mind that the failure modes in masonry walls are strongly linked with the amount of axial load, it can be concluded that as the failure mode approaches to shear dominated behaviour, observed at relatively higher axial stress levels, the drift ratios of performance levels tend to decrease. Consequently, the drift values indicated by codes or official documents should be given according to the predicted failure modes or they should be on the very safe side by providing the values obtained for shear dominating failure mode.

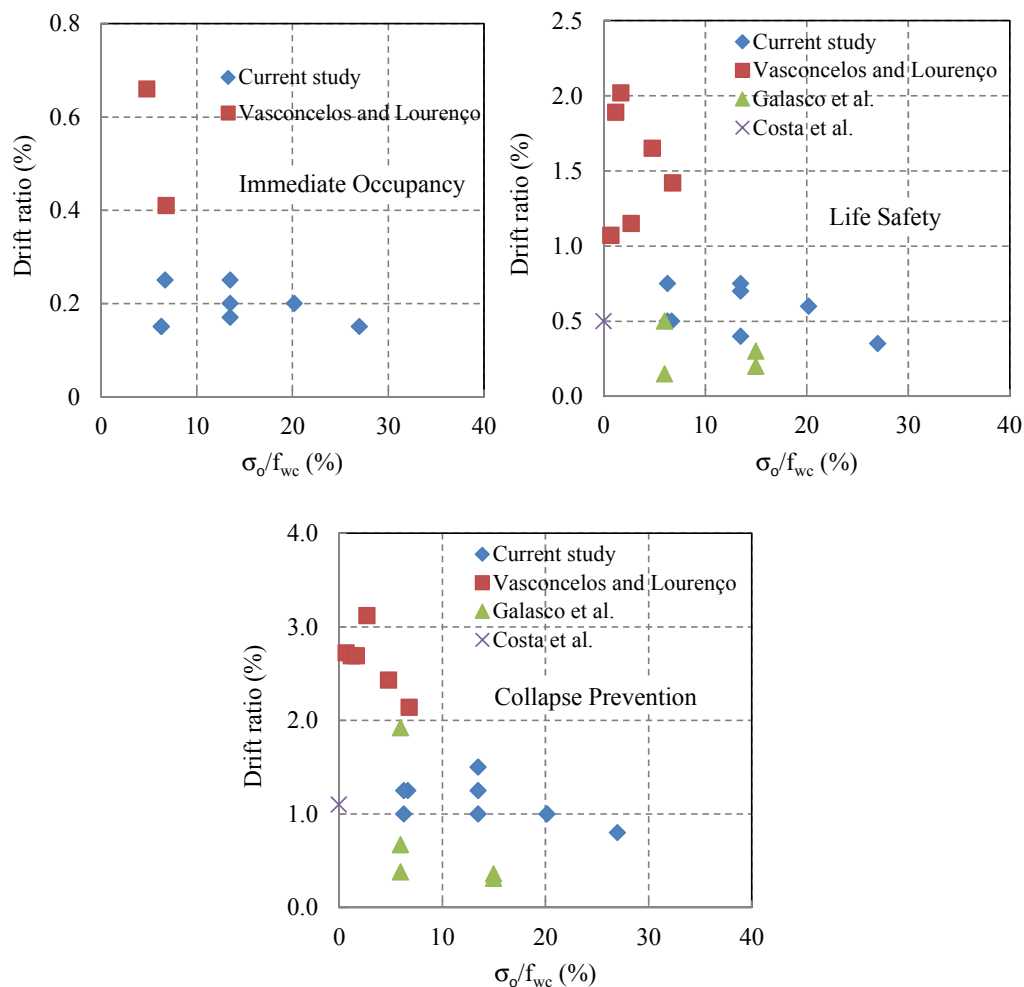


Figure 6.39 : Variation of drift ratio values at limit states with respect to axial stress.

Table 6.9 : Drift ratios of tested walls corresponding to performance levels

Author	Wall	σ_o / f_{wc} (%)	Failure mode	Drift ratio (%)			Notes
				IO	LS	CP	
Current study	M-25-C	6.7	Shear (with flexure)	0.25	0.50	1.25	Three-leaf ashlar, cramp
Current study	M-50-C	13.5	Shear (with flexure)	0.20	0.75	1.25	Three-leaf ashlar, cramp
Current study	M-75-C	20.2	Shear	0.20	0.60	1.00	Three-leaf ashlar, cramp
Current study	M-100-C	27.0	Shear	0.15	0.35	0.80	Three-leaf ashlar, cramp
Current study	M-50	13.5	Shear (with flexure)	0.25	0.40	1.00	Three-leaf ashlar
Current study	M-50-CP	13.5	Shear (with flexure)	0.17	0.70	1.50	Three-leaf ashlar, cramp & pin
Current study	S-50-C	6.3	Shear (with flexure)	0.15	0.50	1.00	Single-leaf ashlar, cramp
Current study	S-50-CP	6.3	Shear (with flexure)	0.15	0.75	1.25	Single-leaf ashlar, cramp & pin
Tomazevic and Lutman (2007)				0.20-0.30	0.40-0.60	1.00-1.20	Shaking table tests
Vasconcelos and Lourenço (2009)	WS.100	0.7	Flexure	–	1.07	2.72	Single-leaf ashlar
Vasconcelos and Lourenço (2009)	WS.175	1.2	Flexure	–	1.89	2.69	Single-leaf ashlar
Vasconcelos and Lourenço (2009)	WS.250	1.7	Flexure	–	2.02	2.69	Single-leaf ashlar
Vasconcelos and Lourenço (2009)	WI.100	2.7	Flexure	–	1.15	3.12	Single-leaf irregular
Vasconcelos and Lourenço (2009)	WI.175	4.8	Flexure	0.66	1.65	2.43	Single-leaf irregular
Vasconcelos and Lourenço (2009)	WI.250	6.8	Flexure	0.41	1.42	2.14	Single-leaf irregular
Vasconcelos and Lourenço (2009)	WR.100	–	Flexure & shear	0.57	1.19	2.90	Single-leaf rubble
Vasconcelos and Lourenço (2009))	WR.175	–	Flexure & shear	0.85	1.19	2.01	Single-leaf rubble
Vasconcelos and Lourenço (2009))	WR.250	–	Shear	0.47	0.58	0.86	Single-leaf rubble
Galasco et al. (2010)	CS00	6.0	Flexure	–	–	1.92	Double-leaf irregular
Galasco et al. (2010)	CS01	15.0	Shear	–	0.30	0.36	Double-leaf irregular
Galasco et al. (2010)	CS02	6.0	Flexure & shear	–	0.50	0.67	Double-leaf irregular
Galasco et al. (2010)	CT01	15.0	Shear	–	0.20	0.31	Double-leaf irregular
Galasco et al. (2010)	CT02	6.0	Shear	–	0.15	0.38	Double-leaf irregular
Costa et al. (2010)	PS_01	0.0	Sliding & shear	–	0.50	1.10	Double-leaf rubble

Among several available codes and guidelines on seismic design and assessment, only a few of them include detailed directions for performance-based assessment of masonry structures. Some of these documents are American FEMA 356 (2000), ASCE/SEI 41-06 (2007), Eurocode 8 Part 3 (EN 1998-3 (2005)), Italian NTC08 (2008) and New Zealand NZSEE (2012). Unfortunately, although it gives very

detailed information on reinforced concrete structures, Turkish Seismic Design Code (2007) lacks such information for masonry structures. Although above mentioned codes and documents include information for reinforced masonry (except Turkish Seismic Design Code (2007)), none of them supplies reference to cramp and pin reinforcements.

The ASCE/SEI 41-06 (2007) guideline which is based on the FEMA 356 (2000) document classifies the actions in a masonry structure as being either deformation-controlled or force-controlled and enables utilization of both linear and nonlinear procedures for assessment of existing masonry buildings. According to the FEMA 356 (2000), the wall components that exhibit failure types such as rocking and bed joint sliding are accepted as deformation controlled whereas the components failing in any of the diagonal tension and toe crushing modes are classified as force-controlled. However, the ASCE/SEI 41-06 (2007) standard uses the same bed joint sliding equation for definition of the shear failure and classifies shear and toe crushing failure modes as force-controlled actions. The force-deformation response of the wall components are represented as shown in Figure 6.40 where the drift ratio is defined as the differential displacement (Δ) between each end of the component over the effective height (h) of the component. In this figure, the drift ratio “d” represents the ultimate deformation for primary components while “e” is for the secondary components, which are given as $0.4 h/l$ and $0.8 h/l$ % for a rocking type of failure, respectively.

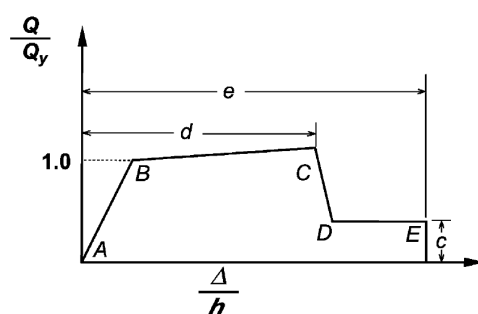


Figure 6.40 : Idealised force-deformation response in ASCE/SEI 41-06 (2007).

In ASCE/SEI 41-06 (2007), the drift ratio limits of unreinforced masonry walls (URM), corresponding to performance levels such as Immediate Occupancy (IO), Life safety (LS) and Collapse Prevention (CP), are only given for the deformation-controlled behavioural mode (namely the rocking mode). However, in the case of reinforced masonry walls, shear actions are considered as deformation-controlled

(unless the axial stress on member is more than 15% of the axial load capacity) and drift limits corresponding to different performance levels are available. The drift limitations of ASCE/SEI 41-06 (2007) and FEMA 356 (2000) obtained for the current study are presented in Table 6.10. Please note that values for reinforced masonry (RM) are also indicated in this table, where available.

The Annex C of Eurocode 8 Part 3 (EN 1998-3 (2005)) provides drift limits for in-plane response of unreinforced masonry walls. This code classifies the behaviour of the walls, either as shear dominated or as flexure dominated with respect to the type of loading conditions, and defines drift ratios accordingly. Three limit states refer to the extent of damage, namely Damage Limitation (DL), Significant Damage (SD), and Near Collapse (NC). The DL limit state corresponds to the point where flexural or shear capacity of the wall is reached (yield point). SD limit state is reached at 0.80 and 0.40% drift ratios for flexure and shear controlled primary seismic walls, respectively (Table 6.10). Finally, the NC limit state arises at drift ratios corresponding to 4/3 of SD limit state leading to 0.53 and 1.07% for shear and flexure dominated walls, respectively. In EN 1998-3 (2005) no further limit state definition exists for reinforced masonry walls.

One of the codes that include several details for masonry construction is the Italian NTC08 (2008) code. This code includes procedures for both linear and non-linear analysis and defines the general criteria for the safety assessment and for the design, execution and control of interventions on existing buildings. The drift ratios corresponding to performance levels can be found in Table 6.10. However, it should be noted that the drift ratio for shear failure of masonry walls are most likely to be reduced to 0.30% from 0.40% in the next revision of the code (private communication with Prof.Dr. Guido Magenes (2011))

Finally, the New Zealand NZSEE (2012) guideline reports the procedures for detailed seismic safety assessment of unreinforced masonry structures. Although higher drift ratio limits for rocking and shear failure modes are recommended with respect to other documents (Table 6.10), a limit on shear strain is imposed to prevent excessive strength degradation (i.e. shear strains should not exceed 0.005).

For the Immediate Occupancy limit state, experimentally obtained drift ratios compiled in Table 6.9 stays above the 0.10% drift ratio value proposed by the FEMA

356 (2000) and ASCE/SEI 41-06 (2007) for URM walls. However, for almost all specimens of this study, the NTC08 (2008) code gives a larger limit for IO level.

The Life Safety performance level drift ratios proposed by FEMA 356 (2000) and ASCE/SEI 41-06 (2007) stays on the safe side for almost all specimens. Only two specimens tested by Galasco et al. (2010) stays below the 0.30% limit. However, since these specimens are reported as shear-dominated walls they should be treated as force controlled members. For the remaining documents such as Eurocode 8 Part 3 (EN 1998-3 (2005)), Italian NTC08 (2008) and New Zealand NZSEE (2012), the proposed drift ratios seem to be reasonable for flexure controlled specimens, while they stay on the unsafe side for shear controlled specimens (particularly the ones tested by Galasco et al. (2010)).

FEMA 356 (2000) and ASCE/SEI 41-06 (2007) documents foresee safe drift ratio values for the Collapse Prevention level of flexure controlled walls. However, Eurocode 8 Part 3 (EN 1998-3 (2005)) becomes unsafe for shear-controlled specimens of Galasco et al. (2010). Such observations done particularly for shear dominated walls also underline the need for further experimental studies on masonry walls with different configurations.

Table 6.10 : Drift ratios given by codes for different performance levels.

Document	IO (DL) (%)	LS (SD) (%)	CP (NC) (%)
FEMA 356 (URM, bed-joint sliding)	0.10	0.30	0.40
FEMA 356 (URM, rocking) ¹	0.10	0.28	0.37
FEMA 356 (RM, flexure) ²	0.10-0.20	0.20-0.30	0.20-0.40
FEMA 356 (RM, shear) ³	0.40	0.60	0.75
ASCE/SEI 41-06 (URM, rocking)	0.10	0.28	0.37
ASCE/SEI 41-06 (RM, flexure) ²	0.10-0.20	0.20-0.30	0.20-0.40
ASCE/SEI 41-06 (RM, shear) ³	0.40	0.60	0.75
EN 1998-3 (URM, flexure)	Yield point	0.80	1.07
EN 1998-3 (URM, shear)	Yield point	0.40	0.53
NTC08 (URM, flexure)	0.30	0.60	N/A
NTC08 (URM, shear)	0.30	0.40	N/A
NTC08 (RM, flexure)	0.40	0.60	N/A
NTC08 (RM, shear)	0.40	0.40	N/A
New Zealand NZSEE (URM, rocking)	N/A	1.00	N/A
New Zealand NZSEE (URM, shear)	N/A	0.50	N/A

¹ For $h/l=0.92$

² Values are for $\sigma_o/f_{wc}=0.075$ and $l/h=1.09$

³ Axial load on member should be less than 15% of the axial load capacity, otherwise the component shall be treated as force-controlled.

7. INVESTIGATION OF THE BEHAVIOUR THROUGH FINITE ELEMENT ANALYSIS

The goal of an advanced numerical analysis is to mimic real world response of an investigated system in the mathematical universe. In order to implement a successful analysis, incorporation of adequate constitutive models is of paramount importance. However, it should be noted that no exact model exists, simply because it is impossible to reflect all natural phenomena into the mathematical universe.

This chapter aims to investigate the behaviour of multi-leaf stone masonry walls through finite element modelling. For this purpose, finite element analysis of tested wall specimens has been carried out by following a micro-modelling approach for building up the geometrical model and defining material characteristics. The analytical and experimental results are evaluated by comparing the damage evolution and load-displacement curves.

7.1 Finite Element Analysis of Masonry Structures

Finite element method, which is a very powerful tool for analysis of masonry structures, heavily depends on well-validated constitutive relationships and an appropriate discretisation of the masonry continuum defining the subject of analysis (Lourenço, 2002).

In terms of mechanical behaviour, masonry is the one of the most complex materials used in construction. Unlike concrete, it is generally not possible to accept it as a homogenous isotropic material. Consequently, challenging characteristics of masonry behaviour has led several researchers to develop constitutive models and analysis approaches of different complexity levels.

The definition of the most suitable method depends on, among other factors, the structure under analysis, the available input data and the analyst's experience and qualifications (Lourenço, 2002). It is possible that different methods may lead to different results, depending on the adequacy of the numerical tool to the sought

information. The best method might be defined as the method that provides the sought information in a reliable manner, within an acceptable error, with the least cost (Oliveira, 2003).

In the case of masonry structures, the existence of joints between the units is the major source of weakness and non-linear behaviour. Depending on the degree of accuracy and the simplicity desired, the modelling strategies presented in Figure 7.1 can be utilized (Lourenço, 2002 and Mistler et al., 2006).

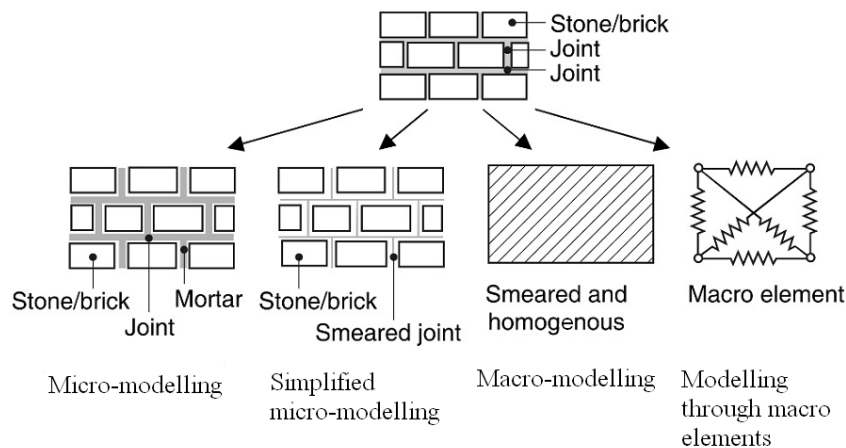


Figure 7.1 : Modelling approaches for masonry structures (modified from Mistler et al., 2006).

In the case of micro-modelling, both units and mortar are discretized and modelled with continuum elements whereas the unit-mortar interface is represented by discontinuum elements. Simplified micro-modelling requires expanded units which are modelled with continuum elements, while the behaviour of the mortar joints and unit-mortar interface is lumped in the discontinuum line interface elements. In the micro- and simplified micro-modelling approaches, Young's modulus, Poisson's ratio and constitutive laws for both units and mortar are considered. Any analysis with this level of refinement is computationally very burdensome and only suitable for the detailed analysis of small sized masonry assemblages (i.e. a part of a structure or laboratory specimens). The large computational effort required even by simplified micro-modelling strategies limits its applicability to the analysis of small structures. Micro-modelling can be accurately used for the calibration of the mechanical parameters of macro-models (Lourenço, 2002).

Macro-modelling utilizes a homogenous material in which the units, mortar joints and unit-mortar interface are smeared out in a continuum. Homogenization process requires utmost care since the results are highly dependent on it.

Since these approaches generally require high computational costs and detailed and hard to obtain information on the mechanical characteristics of constitutive materials; modelling can also be done with macro elements where certain aspects of the masonry behaviour can be reflected by using different types of link elements. This approach is widely utilized during the structural analysis of masonry structures via the equivalent frame method.

In scope of this study, two different 3D finite element models that aim to simulate the behaviour of tested masonry walls are developed by following the micro-modelling approach. The high-level finite element software ABAQUS (ABAQUS, 2009) was chosen for the micro-analysis due to availability of sophisticated computational capabilities together with built-in material models and contact definitions.

A complete finite element analysis consists of three following stages: Pre-processing, simulation and post-processing (ABAQUS, 2009). In the pre-processing stage, the model of the physical problem is defined and an input file is formed. The input file includes information about the geometry (solution mesh), boundary conditions, material constitutive laws, contact definitions (if available) and applied external forces. Herein, external forces term is used as a generic word for any applied point or distributed load, external stress, acceleration, displacement and thermal load. In addition to these definitions, analysis type and options are determined together with the output requests. Depending on the size and complexity of the problem, the pre-processing stage can be carried out via a user interface or simply by using a simple text editor.

The simulation runs as a background process and solves the numerical problem defined in the model. During the simulation stage, the requested output data is stored for post-processing. The analysis run time depends on the complexity of the problem and the computational power of the computer being used.

The results of the simulation can be visualised and investigated in the post-processing stage. The results can be read from the output binary files and displayed as colour contour plots, animations, deformed shape plots and graphs.

7.2 Micro-Modelling of Tested Walls

The micro-modelling of masonry walls requires explicit modelling of each of the constituents such as the units, mortar, core infill and reinforcements with adequate detailing of the geometry. Moreover, an accurate micro-model must include all the basic types of failure mechanisms that characterize masonry (Figure 7.2); such as cracking of the joints, sliding along the bed or head joints, cracking of the units in direct tension, diagonal tensile cracking of the units and masonry crushing (Senthivel and Lourenço, 2009).

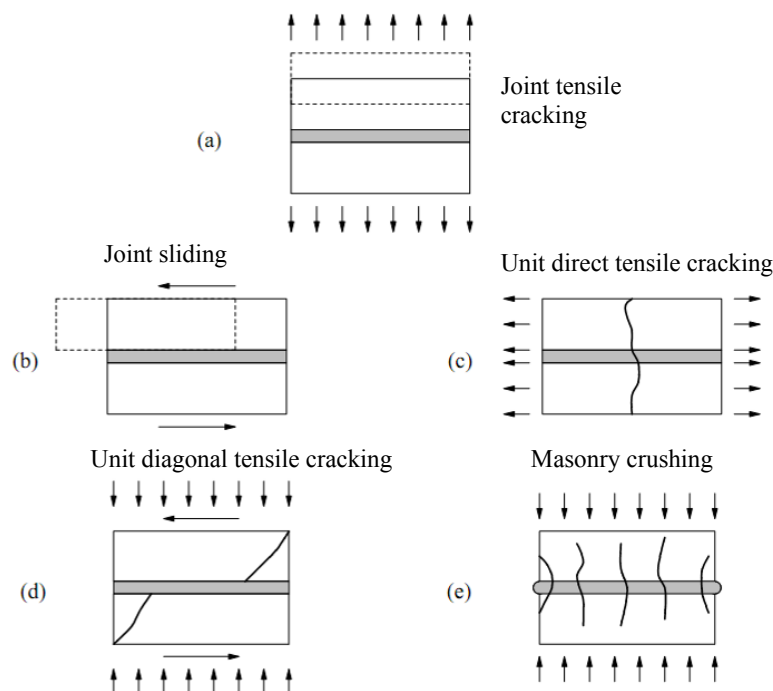


Figure 7.2 : Failure mechanisms in micro-modelling (Senthivel and Lourenço, 2009).

In our case, the constituent materials are the limestone blocks, rubble infill and cramps. Since no mortar joints are available between the units, the interaction of the stone units in the normal and tangential directions needs to be defined via contact definitions.

In scope of this topic, the finite element analysis of tested specimens through micro-modelling approach are handled in two stages. The first attempt is done for the investigation of the behaviour under uniaxial compression loads. For this purpose, the single- and multi-leaf masonry prisms described in Chapter 4 are modelled. In the second stage, the shear compression tests of walls with or without cramps are analysed.

However, since shear compression tests required more advanced modelling and analysis techniques, the following sub-topic will only include shear compression tests. Then, another sub-topic will summarize the analysis of masonry prism uniaxial compression tests. Finally, the results will be delivered again under separate topics.

7.2.1 Micro-modelling of shear compression tests

In commercial finite element analysis software, like ABAQUS, parts are used to create an assembly, which defines the geometry of the subject. In this study, wall assembly consisted of stone blocks and rubble infill (where available) which were formed as 3D deformable solid parts. A different part was prepared for each different dimensioned stone unit type. The four parts for stone units and rubble infill that were used in the model of shear compression test specimens can be seen in Figure 7.3. The rubble infill, which is a highly heterogeneous composite structure, was assumed as an isotropic homogeneous material.

The parts used in the virtual construction of the tested specimens were extruded from 2D drawings prepared in the sketch module of ABAQUS. As seen in Figure 7.3, the parts defined for the stone units were partitioned with respect to the datum points where the springs representing the cramps were fixed. The datum points were placed on the top faces of the parts, at 50 mm distance from the ends. The dimensions of the parts were identical with the stone units used during the construction of the actual masonry wall specimens, so that the alternating widths of the external ashlar leaf could be reflected into the finite element model. Similarly, the part representing the rubble infill had the indentations so that it could perfectly fit to the external ashlar leaves. The reinforced concrete beam that was positioned on top of the walls for transferring the axial load and lateral displacements was also included in the finite element model. A three dimensional elastic part was defined for the addition of the concrete beam to the model.

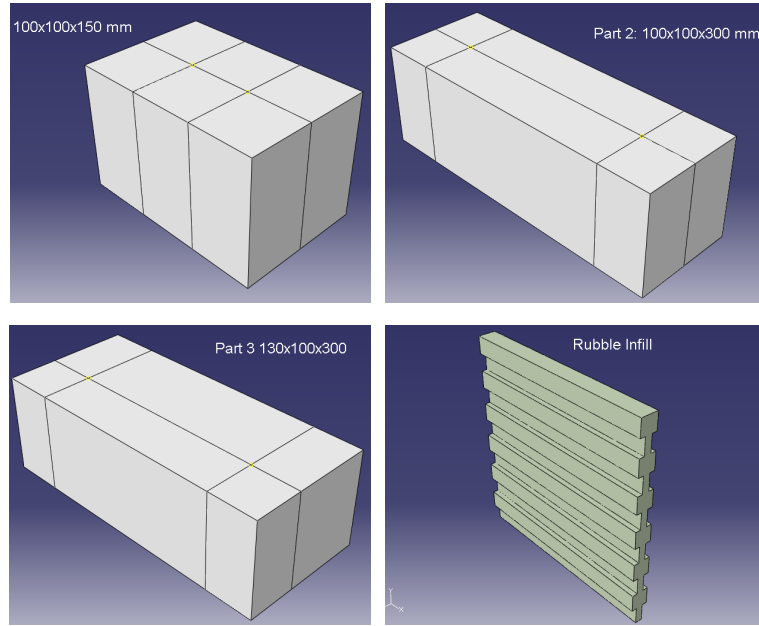


Figure 7.3 : Parts used in the construction of the micro-model.

The complete geometrical model consists of parts which are put together to form an assembly. During construction of the micro-model of the investigated walls, initially, the first course of the stone units was prepared. In case cramps were available in the wall, the datum points at cramp anchor points of adjacent units were connected to each other by using spring elements. Then the similar steps were repeated for the second course and these first two courses were replicated for six times so that the complete ashlar leaf could be generated. Views of the outer leaf assembly are shown in Figure 7.4. The springs representing the cramps that were connecting the stone units can be seen in Figure 7.5.

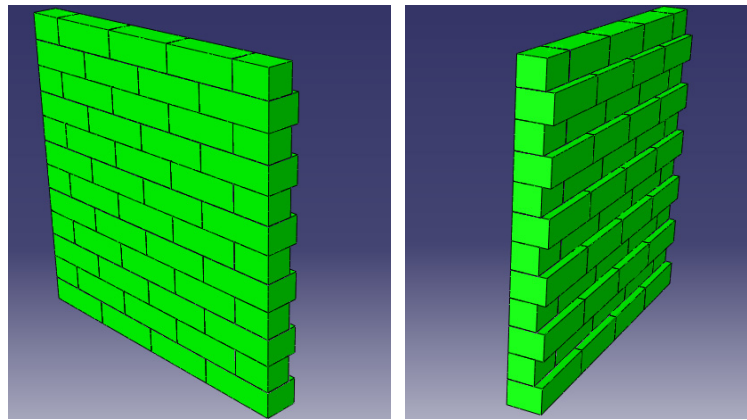


Figure 7.4 : Assembly of the outer leaf in ABAQUS.

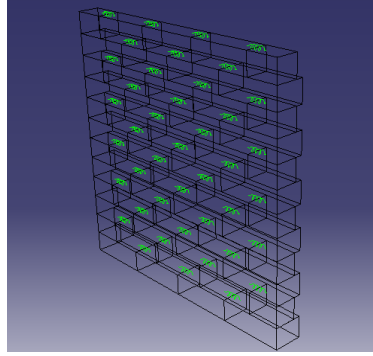


Figure 7.5 : Spring elements representing the cramps.

During the modelling, half of the each wall was included in the analysis, so that computation time and memory requirements were reduced. For this purpose, symmetry plane at mid-thickness along the transverse direction was used. In order to prevent torsion effects, displacements in the transverse direction (z direction in Figure 7.6) were restrained along the symmetry plane. In the final step of the assembly, the rubble infill of the multi-leaf wall was connected to the outer leaf (Figure 7.6). As also assumed during the design of the test setup, the simulated walls were modelled with cantilever type of boundary conditions. Similar to the experimental case, the lowest stone course that was captivated in the U330 steel profile and reaction plates was treated as the foundation of the wall and fixed as shown in Figure 7.6.

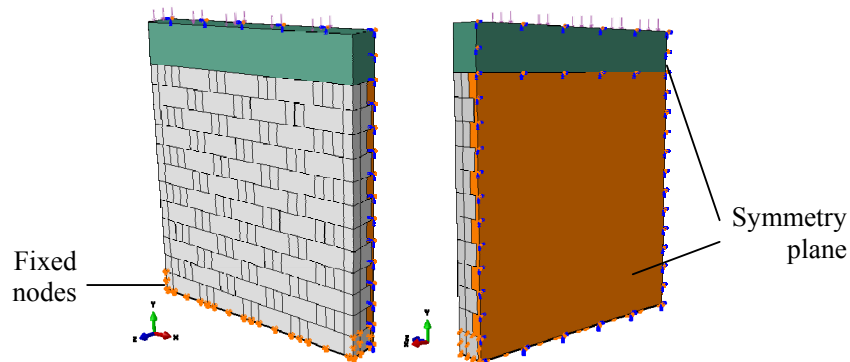


Figure 7.6 : Boundary conditions of the micro-model.

The meshing of the continuum was done by using three-dimensional solid elements, which were 8-node bricks (hexahedra). The selected elements had three translational degree of freedom at each node. Such elements that have nodes only at their corners (Figure 7.7a), use linear interpolation to calculate the displacements at any other point in the element and are often called linear elements (or first-order elements). Elements with mid-side nodes, such as the 20-node brick shown in Figure 7.7b, use

quadratic interpolation and are often called quadratic elements or second-order elements.

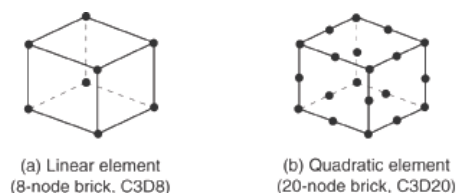


Figure 7.7 : Linear and quadratic brick elements in ABAQUS.

ABAQUS uses numerical techniques to integrate various quantities over the volume of each element. Depending on the number of the Gauss integration points, the elements can be fully or reduced integrated. Reduced integration elements use one fewer integration point in each direction than the fully integrated elements, so that linear elements with reduced integration have single integration point located at the element's centroid. One problematic condition that the linear reduced integration elements suffer is the shear interlocking caused under bending deformations, as seen in Figure 7.8. Since neither of the dotted visualization lines passing through the integration point change their length and the angle between them; all components of stress at the element's single integration point are zero. The element cannot resist bending type of deformation since stiffness in this mode becomes zero. In coarse meshes, this zero-energy mode can propagate through the mesh and produce meaningless results. ABAQUS overcomes this problem by introduction of a small amount of artificial hourglass stiffness that is named as hourglass control. In the ABAQUS User's Manual (ABAQUS, 2009), it is suggested that at least four elements should be used through the thickness when modelling any structure carrying bending loads with this type of element. During the analysis of the masonry shear walls, since bending deformations were limited with respect to shear deformations and since a fine mesh was utilized during the meshing of the walls, C3D8R type elements with reduced integration points and hour glass control was selected to reduce the cost of computation and avoid shear interlocking.



Figure 7.8 : Bending deformation of a linear element with reduced integration (ABAQUS, 2009).

The mesh seeding was done with a spacing of 25 mm so that at least four elements were obtained along the height of the stone units. Instance of each part was meshed independently, so that the nodes at the interfaces of two neighbouring parts were not shared. However, the interaction between the elements was defined via contact algorithms. The meshed models of single- and multi-leaf masonry wall specimens tested under in-plane shear loads can be seen in Figure 7.9.

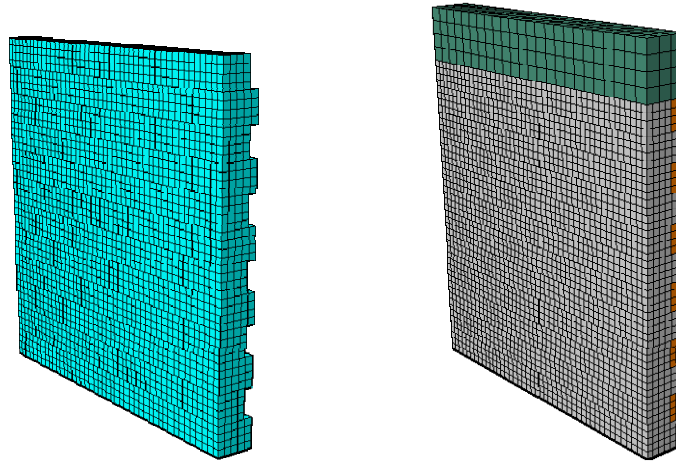


Figure 7.9 : Meshed models of single- and multi-leaf masonry wall specimens.

Apart from basic material properties such as Young's modulus and Poisson's ratio, which are required for definition of the elastic behaviour of the stone units and the rubble core, nonlinear behaviour of the materials was also considered during the analysis. For this purpose, the Concrete Damaged Plasticity Model (CDP Model) available in ABAQUS was used.

The CDP continuum plasticity based model was initially developed by Lubliner et al. (1989) for monotonic behaviour of concrete and further modified by Lee and Fenves (1998) to cover cyclic response. Although it was originally developed for concrete, the CDP model provides a general capability for modelling other quasi-brittle materials such as rock, mortar and ceramics. The main failure mechanisms of the model are tensile cracking and compressive crushing of the material. However, the model does not introduce shear retention assumptions (Dogariu et al, 2009)

According to this material model, under uniaxial tension, the stress-strain response follows a linear elastic relationship until the value of the failure stress σ_{to} is reached (Figure 7.10). The softening branch of the stress-strain curve represents the formation of micro-cracks and localization of strains. Under uniaxial compression,

the response is linear until the value of initial yield stress σ_{co} . Stress hardening after the yield point continues until the peak stress σ_{cu} , and strain softening occurs beyond this peak point (Figure 7.11).

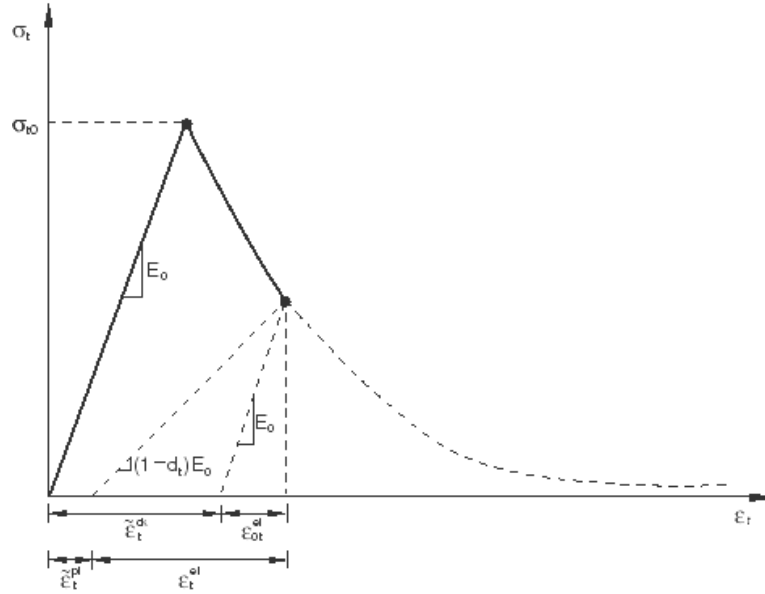


Figure 7.10 : Uniaxial tension response in CDP model (ABAQUS, 2009).

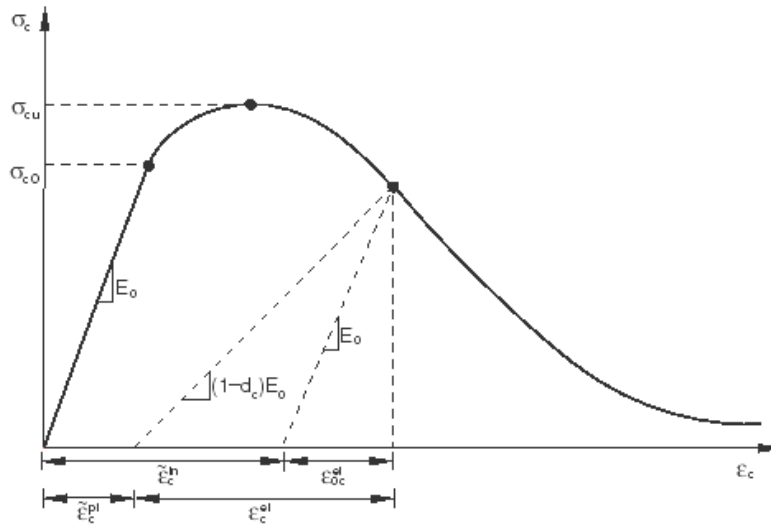


Figure 7.11 : Uniaxial compression response in CDP model (ABAQUS, 2009).

The uniaxial stress-strain curves are input as stress versus inelastic strain data $\tilde{\epsilon}_c^{in}$ and ABAQUS converts the inelastic strains to plastic strains $\tilde{\epsilon}_c^{pl}$. When an unloading occurs from any point on the strain-softening branch of the stress-strain curve, the unloading response is weakened. In other words, the elastic stiffness of the material

appears to be degraded. The degradation of the elastic stiffness is characterized by two damage variables, d_t and d_c , which are assumed to be functions of the plastic strains. The damage variables can take values from zero, representing the undamaged material, to one, which represents total loss of strength. If damage variables are not specified, the model behaves as a plasticity model. So that $\tilde{\epsilon}_t^{pl} = \tilde{\epsilon}_t^{ck}$ and $\tilde{\epsilon}_c^{pl} = \tilde{\epsilon}_c^{in}$ for tension and compression, respectively.

The post-failure behaviour for direct tensile straining can be specified by means of a tension stiffening approach where the post-failure stress-strain relation is input, or by applying a fracture energy-cracking criterion. ABAQUS Analysis User's Manual (ABAQUS, 2009) indicates that, when there is no reinforcement in significant regions of the model, the tension stiffening approach would introduce unreasonable mesh sensitivity into the results. For such cases, Hillerborg's (1976) fracture energy approach is advised and found adequate for many practical purposes. Hillerborg (1976) defines the energy required to open a unit area of crack (G_f) as a material parameter. According to this approach the concrete's brittle behaviour is characterized by a stress-cracking displacement response ($\sigma_t - u_t^{ck}$ response as shown in Figure 7.12) rather than a stress-strain response. Alternatively, the fracture energy (G_f) can be specified directly as a material property. This model assumes a linear loss of strength after cracking, as shown in Figure 7.13. Accordingly, if the tensile failure stress point (σ_{to}) is exceeded, for a known fracture energy (G_f), the cracking displacement at which complete loss of strength takes place becomes:

$$u_{to} = \frac{2G_f}{\sigma_{to}} \quad (7.1)$$

During the modelling of the masonry wall specimens of this study, since no stress-strain data was available for the tensile stress-strain behaviour of stone units and rubble infill, the more practical fracture energy approach was followed.

Typical fracture energy (G_f) values of 0.04 and 0.12 Nmm/mm² are indicated in the ABAQUS Analysis Manual (ABAQUS, 2009) for concrete compressive strengths of approximately 20 MPa and 40 MPa, respectively. However, for the masonry units slightly higher values are reported in the literature. For sand stone specimens Ramos

(2002) reports a fracture energy of 0.11 Nmm/mm^2 for a tensile strength of 3.7 MPa . For the fracture energy of solid clay and calcium-silicate units, Van der Pluijm (1992) indicates values varying between 0.06 and 0.13 Nmm/mm^2 for tensile strength values ranging from 1.5 to 3.5 MPa . Direct tension tests by Vasconcelos (2005) that were performed on different types of granites with tensile strengths varying in a range of 1.56 and 8.08 MPa yielded to fracture energies between 0.15 and 0.27 .

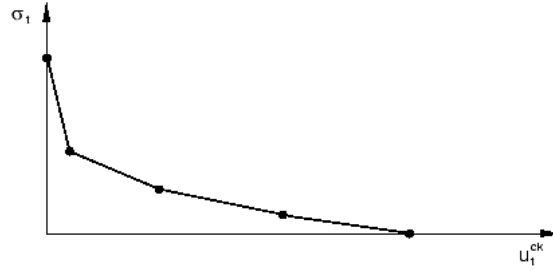


Figure 7.12 : Post failure stress-displacement curve (ABAQUS, 2009).

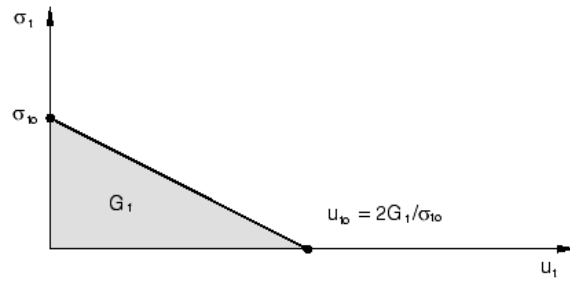


Figure 7.13 : Postfailure stress-fracture energy curve (ABAQUS, 2009).

Since the strength of concrete is related to the state of stress, i.e. biaxial and triaxial stress states, the CDP model uses a yield surface proposed by Lubliner et al. (1989), which is schematically shown in Figure 7.14. Similar to stiffness properties, the strength properties are also dependent on the damage.

The definition of the potential plastic flow and yield function requires the definition of some parameters. For that purpose, ABAQUS requires the user to input f_{bo} / f_{co} ratio, dilation angle ψ , eccentricity and the K parameter. Briefly, f_{bo} / f_{co} is the ratio of the biaxial to uniaxial compressive strength (ratio of biaxial compression σ_{bo} and uniaxial compression σ_{co} stresses in Figure 7.14), dilation angle defines the relationship between the volumetric and shear strains, eccentricity indicates how the

dilation angle changes for different confining pressures and the K parameter decides the shape of the yield function in the deviatoric plane (Minaie, 2009).

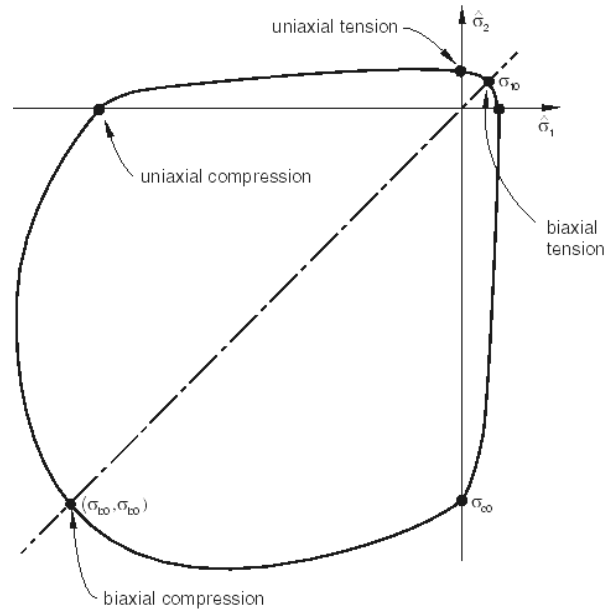


Figure 7.14 : Yield surface in plane stress space (ABAQUS, 2009).

During the finite element modelling of the wall specimens tested in this study, results of the mechanical tests carried out on constituent materials and their assemblages were utilized. The resulting mechanical parameters used for ABAQUS model input are tabulated in Table 7.1. Depending on the modelling approach, the Young's modulus, compressive strength and compressive stress-strain behaviour of the ashlar masonry were obtained from single unit and single-leaf stone masonry compression tests, while the same parameters of the rubble infill were derived from cylinder compression tests. Since rubble material's material properties varied significantly by time, an average age of 150 days was assumed and mechanical characteristics at that age were utilized. Tensile strength of the units was adopted from the indirect tension tests (modulus of rupture tests) made on stone prisms and the resulting average value of 3.2 MPa was divided by two for converting to direct uniaxial tensile strength. Similarly, the uniaxial tensile strength of the rubble infill material was estimated from cylinder splitting tests and divided by 1.2 to convert to the direct uniaxial strength value. Fracture energies were estimated by considering the available information in literature and a number of trials were performed and compared with the actual shear compression test data. Dilation angle was assumed as 35 degree and default values of ABAQUS CDP model was utilized for other parameters. In addition

to these parameters, the uniaxial compressive stress-strain data of the stone leaves and rubble infill was input into the finite element model by using the curves obtained from stone prism and rubble cylinder compression tests. Since ABAQUS required stress versus inelastic strain data after the yield point, the strains obtained from compression tests were converted to inelastic strains by subtracting the elastic strain components (please refer to Figure 7.11 for schematic representation). The stress-inelastic strain curves of utilized materials are presented in Figure 7.15.

Table 7.1 : Mechanical parameters used in the micro-modelling.

Parameter	Stone unit	Stone masonry leaf	Rubble internal leaf
Young's modulus (MPa)	6000	2300	870
Poisson's ratio	0.20	0.20	0.20
Compressive strength (MPa)	17.00	6.90	2.15
Tensile strength (MPa)	1.60	1.60	0.24
Fracture energy (Nmm/mm ²)	0.10	0.10	0.02
Dilation angle (degree)	35	35	35
f_{bo} / f_{co}	1.16	1.16	1.16
Eccentricity	0.10	0.10	0.10
K	0.67	0.67	0.67

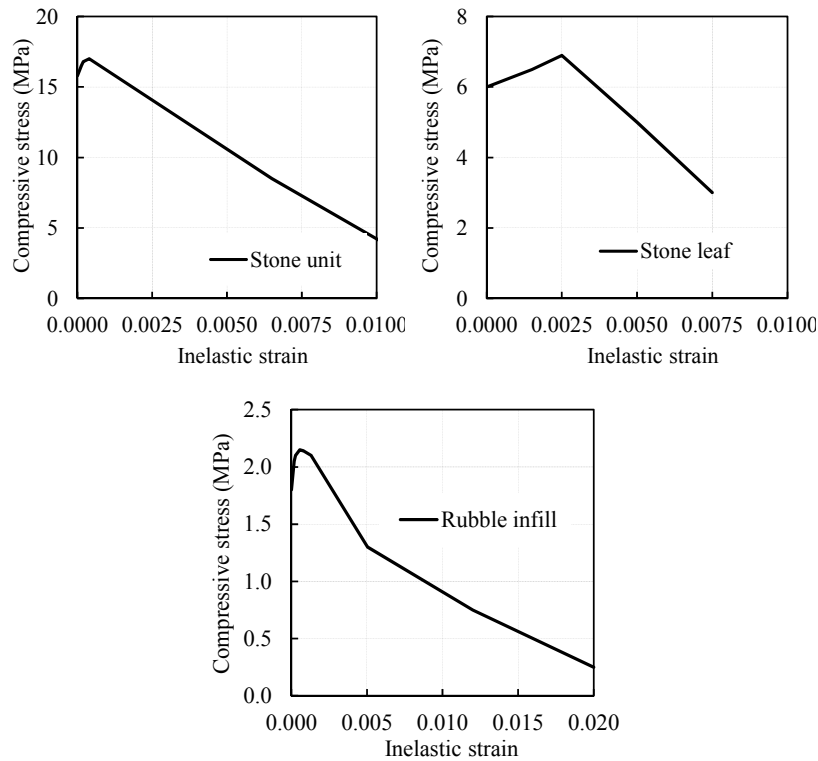


Figure 7.15 : Compressive stress-inelastic strain curves for the micro-modelling.

The cramps used in the external leaves of the ashlar walls were implemented in the finite element models as spring elements. The spring behaviour was linear elastic, which was in parallel with the observations done during the tests by using strain gauges that measured the strains on the cramps. The axial stiffness value of the springs were calculated by using the tensile stress-strain curves obtained from uniaxial tension tests performed on the cramp material. Accordingly, the axial stiffness of the cramp K_{cramp} to be used for the coefficient of elastic spring was evaluated as 21600 N/mm by using the relationship given below:

$$K_{cramp} = \frac{t_{cramp} \times w_{cramp} \times f_{y,cramp}}{\epsilon_{y,cramp} \times l_{o,cramp}} \quad (7.2)$$

where t_{cramp} and w_{cramp} are the cramp cross-section thickness and width values, which are equal to 2.3 and 8.0 mm, respectively. The yielding stress $f_{y,cramp}$ and yielding strain $\epsilon_{y,cramp}$ are available in Table 4.13 as 249 MPa and 0.212%. Finally, $l_{o,cramp}$ is the initial length of the cramps that was equal to 100 mm.

In the micro-modelling approach of masonry walls, the interaction between the constituents of the model (i.e. bed and head joints between the units) plays a major role in defining the overall behaviour. The friction, which is created as a result of compressive forces acting normal to the contacting surfaces, creates shear forces that resist the tangential motion. In our study, the interactions between the stone units and between stone units and rubble core were defined by using the “general contact” algorithm available in ABAQUS. The general contact algorithm automatically detects and pairs the surfaces that may potentially come into contact. The interaction between contacting surfaces consists of two components: One normal to the surfaces and one tangential to the surfaces. Different types of interaction properties can be assigned to both of these components.

During the modelling phase, the interaction property applied in the normal direction to the surfaces was either “softened contact” with a linear function or “hard contact”. In a linear pressure-overclosure relationship, the surfaces transmit contact pressure when the overclosure between them, measured in the contact (normal) direction, is greater than zero. Any magnitude of contact pressure can be transmitted between the surfaces and the surfaces separate when the contact pressure between them becomes

zero or negative. The utilized hard and softened contact interaction properties for the joint normal direction can be visualised as shown in Figure 7.16. In this study, pressure-overclosure relationship of the softened contact was defined based on the finite element analysis of the uniaxial masonry prism compression tests. As a result of these analyses, which are explained in the next section, the average slopes of the pressure-overclosure relationship (k_n contact stiffness in normal direction) were obtained as 50 and 8 N/mm/mm² for single- and multi-leaf walls, respectively. Consequently, these normal stiffness values were assigned to the finite element models of the shear compression tests. In the case of hard contact definition, apparently, no coefficient was required. It should be noted that, the assigned mechanical characteristics of the materials were obtained through single unit compression tests in case the softened contact was used. Whereas, results of the single-leaf masonry compression tests were utilized together with the hard contact interaction property. Please note that, the finite element analysis cases, defined with respect to the definition of contact properties between the stone units, are briefly summarized in Table 7.2.

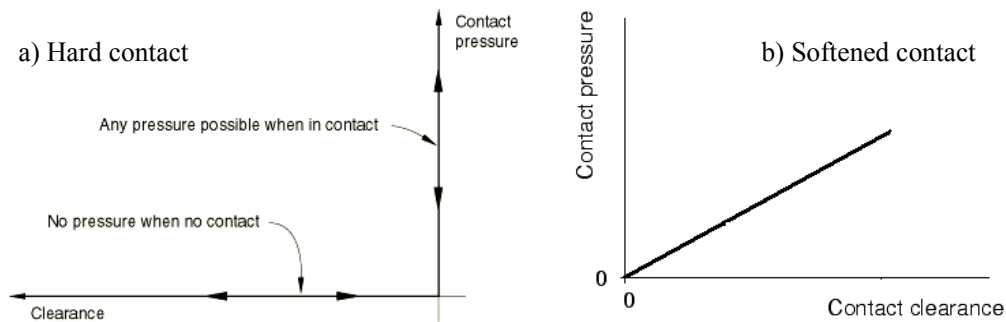


Figure 7.16 : Hard contact and softened contact properties (ABAQUS, 2009).

Penalty type of formulation was used in the tangential direction of the contacting surfaces. In this interaction property type, the common Coulomb friction model defines the frictional behaviour (Eq. 7.3). Accordingly, the tangential motion is zero until the surface traction reaches a critical shear stress value (τ), which depends on the normal contact pressure (σ) via the friction coefficient (μ), Figure 7.17. This approach was quite useful for modelling the frictional behaviour at the bed and head joints of the stone units, since no cohesion existed in the dry joints of the external ashlar leaves. Apparently, this was not valid for the interaction between the stone units and the rubble core. At this type of interface, cohesion exists and probably a different friction coefficient is applicable. However, since no experimental data was

available for stone unit-rubble core interface, same friction coefficient was assigned to all surfaces and cohesion was neglected. The assigned friction coefficient value was equal to 0.74, which was obtained from the initial shear tests whose details are presented in Chapter 4.

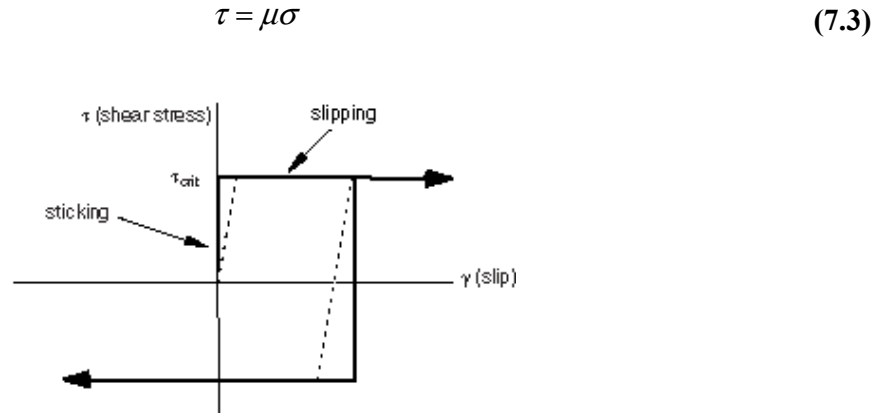


Figure 7.17 : Frictional behaviour in the tangential direction of contact interfaces (ABAQUS, 2009).

Table 7.2 : Interaction properties of analysis cases for shear compression tests.

Analysis Case	Interface direction	Single-leaf	Multi-leaf	External leaf property source
Case 1	Normal	Hard contact	Hard contact	Single-leaf compression tests
	Tangential	Friction, $\mu=0.74$	Friction, $\mu=0.74$	Initial shear tests
Case 2	Normal	Softened contact, $k_n=50$	Softened contact, $k_n=50$	Single unit compression tests
	Tangential	Friction, $\mu=0.74$	Friction, $\mu=0.74$	Initial shear tests
Case 3	Normal	Softened contact, $k_n=8$	Softened contact, $k_n=8$	Single unit compression tests
	Tangential	Friction, $\mu=0.74$	Friction, $\mu=0.74$	Initial shear tests

The loading of the masonry wall models was performed by creating subsequent steps: Initial, axial loading and lateral displacement. At the initial step, the boundary conditions were created. This was followed by uniaxial loading of the wall model to the target normal stress value applied during the tests. The vertical pressure was applied to the top surface of the concrete spreader beam. Finally, after complete loading of the vertical pressure, target lateral displacements were enforced to side surface of the spreader concrete beam, to which the hydraulic actuator was fixed during the shear compression tests. The zones of the finite element model, where axial loading and lateral displacement were implemented, can be seen in Figure 7.18.

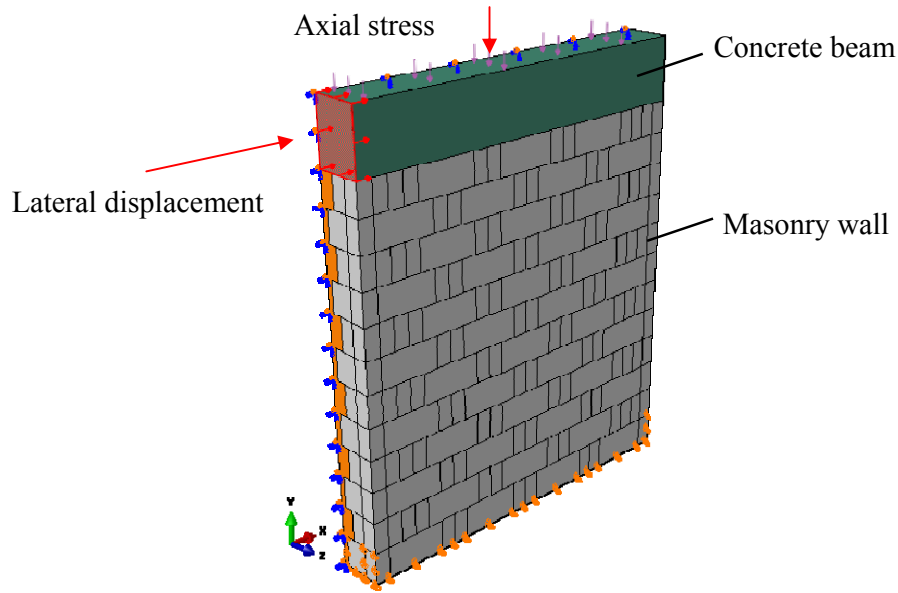


Figure 7.18 : Load and displacement application zones.

In ABAQUS two main analysis types are available for solution of nonlinear problems: Implicit (ABAQUS/Standard) and explicit (ABAQUS/Explicit) analyses. In both integration procedures the equilibrium is defined in terms of the externally applied forces $\mathbf{P}$, the internal forces $\mathbf{I}$ and the nodal accelerations $\ddot{\mathbf{u}}$:

$$\mathbf{M}\ddot{\mathbf{u}} = \mathbf{P} - \mathbf{I} \quad (7.4)$$

where $\mathbf{M}$ is the mass matrix. Both implicit and explicit approaches solve the set of equations for nodal accelerations and determine the internal element forces. However, the difference is in the computation of the accelerations.

In the implicit procedure, a set of linear equations is solved by a direct solution method. ABAQUS/Standard uses the full Newton-Raphson iterative solution method to obtain solutions for nonlinear problems. The solution is found by applying the loads gradually and incrementally toward the final solution. Therefore, in ABAQUS/Standard the simulation is run through a number of load increments and at the end of each load increment approximate equilibrium is found. The sum of all of the incremental responses obtained through several iterations is the approximate solution for the nonlinear problem. Since at each increment the set of equations is formed again (stiffness matrix changes in the tangential stiffness method) and solved, the computational cost is high. If the model contains significant discontinuities such

as contact and frictional sliding, very large number of iterations may be required (ABAQUS, 2009).

The explicit analysis is well-suited for modelling of high speed dynamic events (such as blast and impact), complex contact problems, complex post-buckling problems, highly nonlinear quasi-static problems and for analysis of materials with degradation and failure. Since during the micro-modelling of the tested masonry walls, contact between the stone units has paramount importance, the materials had a degrading character and the tests were actually quasi-static, the analysis of the walls were done via the explicit analysis option available in ABAQUS. Therefore, in scope of this topic more attention is paid on explicit analysis.

It should be noted that explicit analysis has also been used in previous studies such as Dhanasekar and Haider (2008), Dogariu et al. (2009) and Karapitta et al. (2011). However, the nonlinear finite element analysis of masonry walls performed in these studies included only the macro-modelling approach.

In the case of explicit analysis, a central difference rule is used to integrate the equations of motion in all degrees of freedom. The kinematic conditions at one time increment are used to calculate the kinematic conditions at the next increment. At the beginning of each time increment, dynamic equilibrium in Eq. (7.4) is satisfied.

The central difference method is applied with a varying time increment, which is particularly important if the response is nonlinear. The central difference integration scheme for varying time increment is shown in Figure 7.19 (Dunne and Petrinic, 2005).

If Δt_{N+1} is the time increment between t_N and t_{N+1} ; displacement at t_N is $\mathbf{u}_N = \mathbf{u}_{(t_N)}$ as shown in Figure 7.19, then the mid-step velocities are defined by:

$$\dot{\mathbf{u}}_{N-1/2} = \dot{\mathbf{u}}_{(t_{N-1/2})} \quad (7.5)$$

$$\dot{\mathbf{u}}_{N+1/2} = \dot{\mathbf{u}}_{(t_{N+1/2})} \quad (7.6)$$

where

$$t_{N-1/2} = \frac{1}{2}(t_{N-1} + t_N) \quad (7.7)$$

$$t_{N+1/2} = \frac{1}{2}(t_N + t_{N+1}) \quad (7.8)$$

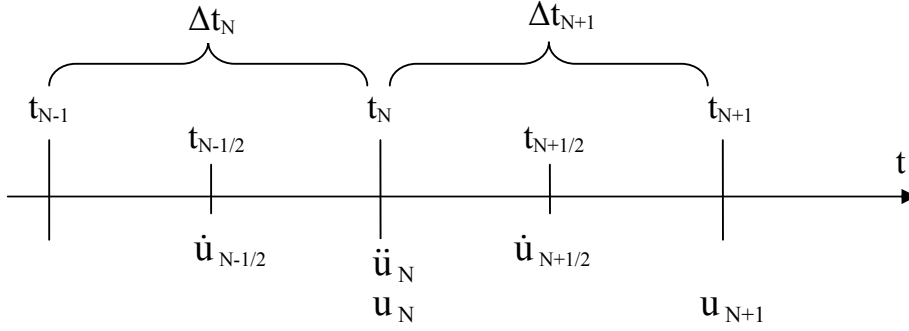


Figure 7.19 : Scheme for central difference integration (Dunne and Petrinic, 2005).

Assuming that the acceleration is constant between subsequent time increments the central difference formula for velocity and acceleration becomes:

$$\dot{\mathbf{u}}_{N+1/2} = \frac{\mathbf{u}_{N+1} - \mathbf{u}_N}{\Delta t_{N+1}} \quad (7.9)$$

$$\ddot{\mathbf{u}}_N = \frac{\dot{\mathbf{u}}_{N+1/2} - \dot{\mathbf{u}}_{N-1/2}}{\Delta t_{N+1/2}} \quad (7.10)$$

where

$$\Delta t_{N+1/2} = \frac{1}{2}(\Delta t_N + \Delta t_{N+1}) \quad (7.11)$$

In case the acceleration term is substituted into the momentum balance equation given in Eq. (7.4), the following expression is obtained:

$$\mathbf{M} \frac{\dot{\mathbf{u}}_{N+1/2} - \dot{\mathbf{u}}_{N-1/2}}{\Delta t_{N+1/2}} = \mathbf{P}_N - \mathbf{I}_N \quad (7.12)$$

which leads to

$$\dot{\mathbf{u}}_{N+1/2} = \mathbf{M}^{-1}(\mathbf{P}_N - \mathbf{I}_N)\Delta t_{N+1/2} + \dot{\mathbf{u}}_{N-1/2} \quad (7.13)$$

and by using Eqs. (7.9) and (7.13), the displacement at t_{N+1} can be obtained (Eq. (7.14)).

$$\mathbf{u}_{N+1} = \mathbf{u}_N + \dot{\mathbf{u}}_{N+1/2} \Delta t_{N+1} \quad (7.14)$$

After this step, the strain increments are obtained from the strain rate, which is followed by computation of the stresses from constitutive equations so that the nodal internal force vector can be assembled. These final calculations complete the N^{th} step and the next step begins.

For the explicit analysis method to produce accurate results, the time increments must be quite small so that the accelerations are nearly constant during an increment. Due to this reason, analyses typically require thousands of increments. However, each increment is computationally inexpensive because there is no need to setup new sets of equations and solve them at each step as in the case of implicit analysis. Most of the computational effort is spent for the element calculations (for strains and stresses) to determine the internal forces.

Although the explicit analysis approach is developed for solution of high-speed dynamic problems, quasi-static problems can also be solved with this approach under certain circumstances.

However, since a static event takes much greater time than a dynamic event, it is often computationally impractical to analyse the simulation in its natural time scale. Apparently, such an effort would require an excessive number of minor increments. In order to obtain an economical and practical solution, somehow the event needs to be accelerated. On the other hand, as the event is accelerated, the static equilibrium transforms into a dynamic equilibrium problem in which inertial forces become more dominant. In quasi-static analysis with explicit approach; the dilemma of processing the problem in the shortest time period while at the same time keeping the inertial forces remain insignificant needs to be overcome.

Instead of artificially increasing the loading rate, the mass (or the material's density) can be scaled up so that the stable time increment Δt , required for an accurate explicit analysis, is also increased and fewer time steps are needed (ABAQUS, 2009). The stable time increment Δt can be obtained by:

$$\Delta t = \frac{L^e}{c_d} \quad (7.15)$$

where L^e is the characteristic element length and c_d is the dilatational wave speed of the material. The dilatational wave speed for a linear elastic material with Young's modulus E and density ρ is given by:

$$c_d = \sqrt{\frac{E}{\rho}} \quad (7.16)$$

The stable time increment for the quasi-static analysis of the masonry walls tested in this study can be obtained as $\Delta t = 1.6 \times 10^{-5} s$ for characteristic element length L^e of 25 mm (average mesh size), Young's modulus of $E = 6000$ MPa and density of $\rho = 2.4 \times 10^{-9} \text{ t/mm}^3$. As also indicated by Karapitta et al.(2011), it is worth to note that this stability time limit increases during a nonlinear analysis, since the Young's modulus of the material decreases after the yielding of the material.

It should be noted that excessive mass scaling may lead to errors in the analysis. If the mass (or the loading speed) is increased more than an optimum value; high kinetic energy values with respect to the input energy, which is a sign of high inertial forces, can be observed. During a quasi-static analysis, the energy balance of the model can be checked so that it can be guaranteed that the inertial forces are negligible.

The energy balance of a system can be expressed with the following equation that constitutes of energy components such as: Internal energy E_I (elastic and plastic strain energy), energy absorbed by viscous dissipation E_V , kinetic energy E_{KE} , energy absorbed by frictional dissipation E_{FD} , work done by external forces E_W and total energy of the system E_{total} :

$$E_I + E_V + E_{KE} + E_{FD} - E_W = E_{total} \quad (7.17)$$

In case of a quasi-static simulation, as a general rule the kinetic energy of the deforming material should not exceed a small fraction (typically 5 to 10%) of its internal energy throughout most of the process (ABAQUS, 2009).

Another concern for a successful quasi-static analysis is the application of the load increment at a certain step. Since sudden movements cause stress waves, which can cause noisy or inaccurate solutions, the loading should be as smooth as possible.

ABAQUS software offers the smooth step option, which automatically creates a smooth step curve (Figure 7.20) and avoids a sudden ramp at each load increment.

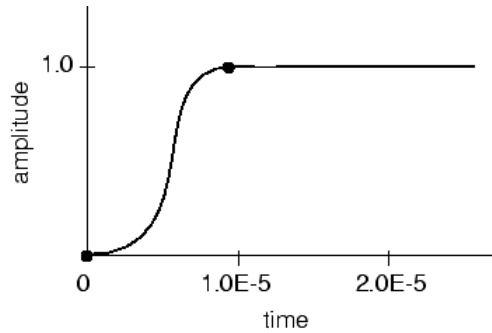


Figure 7.20 : Smooth step amplitude curve (ABAQUS, 2009).

In this study, semi-automatic mass scaling was performed throughout the loading steps, so that the time increments could be kept at desired level. After dozens of trials, an appropriate mass scale factor of four was found and utilized during the whole analysis, so that the kinetic energy of the system could be kept at negligible levels and the energy balance could be sustained. Separate smooth step amplitudes were defined for axial and lateral loading steps.

The post-processing of the obtained data was done with the visualization module of ABAQUS and the required numerical data was transferred to Excel for further evaluation. In the presented strain contours of following topics, the maximum principal plastic strain vectors corresponded to regions where the tensile strengths of the materials were exceeded. Similarly, the minimum principal plastic strain vectors indicated the zones where the materials were close to peak compressive stress or exceeded the peak compressive stress point. In other words, the maximum principal plastic strain vectors pointed the locations of cracks and they were perpendicular to cracking directions, whereas the minimum principal plastic strain vectors marked the crushing zones and they were parallel to compressive stresses that caused crushing.

7.2.2 Micro-modelling of uniaxial compression tests

The micro-modelling of uniaxial compression tests of single- and multi-leaf masonry prisms had many common points with the modelling of the shear compression tests. Consequently, in this topic, only the different aspects of modelling will be given and the previous topic will be addressed at common steps.

The production of the parts and the assembly for finite element model of uniaxial compression tests was similar to that of shear compression tests. The single-leaf prisms only consisted of stone units laid in five courses; while the multi-leaf prisms had rubble infill in addition to the stone leaves. The dimensions of the parts and assemblies were identical with the actual specimens. The part instances used in the assembly process were formed as three dimensional deformable elements. The geometrical models of the both type of prisms are shown in Figure 7.21.

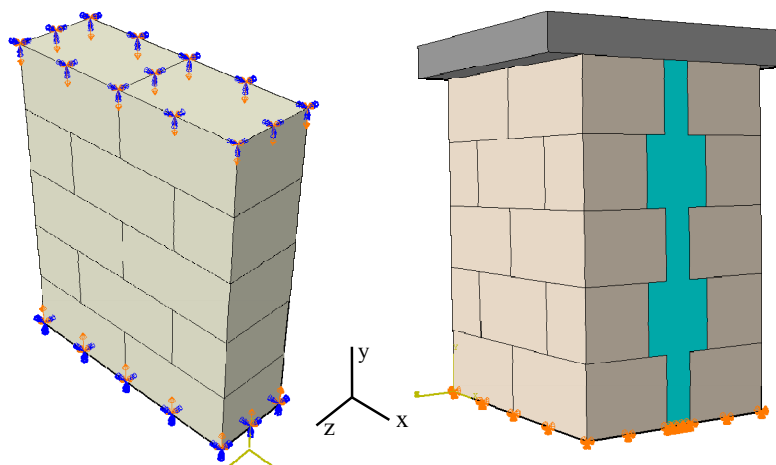


Figure 7.21 : Assemblies for modelling of uniaxial compression tests.

The upper and lower boundary conditions of the single-leaf wallets were assumed to be fixed in the horizontal directions (x- and z-directions) in terms of displacements, as seen in Figure 7.21. The lower boundary conditions of the multi-leaf prisms were identical to that of single-leaf prisms. However, based on the observations done during compression tests that marked presence of out-of-plane movement of the external leaves; it was decided to determine the upper boundary conditions of the multi-leaf prisms by considering the friction between the specimen top face and the steel loading platen. For this purpose, a steel plate was assembled to the top face of the multi-leaf prism as also seen in Figure 7.21, and a friction coefficient of 0.30 was input between the steel plate and specimen top face. Since the analyses were carried out by applying incremental vertical displacements to the top surfaces of the specimen assemblies (in a stepwise manner), the vertical direction (y-direction) was left free.

Solution mesh of the masonry prisms was created by using three-dimensional hexahedra solid elements (named as C3D8R type element in ABAQUS), which were

also utilized in the modelling of shear compression tests. Average size of elements was approximately 20 mm. The views of meshed prisms can be seen Figure 7.22.

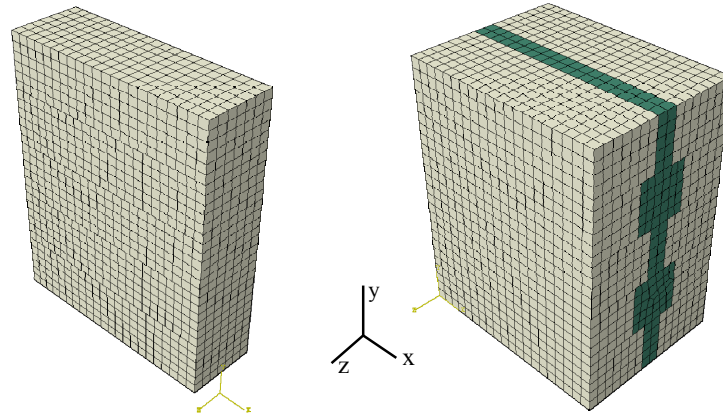


Figure 7.22 : Meshed geometries of uniaxial compression models.

Results of the mechanical tests carried out on constituent materials and single-leaf masonry prisms were used for definition of the material properties. In addition to elastic properties of utilized materials (such as Young's modulus and Poisson's ratio), nonlinear behaviour of the materials was also considered during the analysis. This was achieved by employing the Concrete Damaged Plasticity Model (CDP Model), which was also used in the analyses of shear compression tests. Mechanical parameters input into the finite element model are summarized Table 7.1 and the idealized uniaxial compressive stress-inelastic strain curves of the stone unit and rubble infill are presented in Figure 7.15.

It should be recalled that the material tests presented in Chapter 4 revealed a large scatter in the mechanical characteristics of both materials. Consequently, in the finite element phase, mechanical properties and curves that are representative of the average test results were utilized.

The interaction definition between the stone units and between stone units and rubble core were defined by using the "general contact" option available in ABAQUS. The interaction properties in the tangential direction were identical with the shear compression tests and consisted of a friction coefficient definition.

Since the compression test results of single-leaf prisms indicated that the mechanical properties such as stiffness, strength and deformability were significantly different from that of single units, two different approaches were followed for definition of the contact stiffness in the normal direction of the interfaces. This was mainly due to the

presence of head and bed joints, which included several gaps and imperfections between the stone units. As also explained in Chapter 4, these gaps and imperfections led to stress concentrations on the stone units, so that the damage initiation and development were accelerated. In addition to that; assembling stone blocks, which had a wide scatter in the mechanical characteristics, highly affected the overall behaviour of the masonry assembly. Last but not least, differences between the stone unit qualities of external leaves in the same specimen and the second order effects caused due to larger dimensions of masonry prisms led to lower strength and Young's modulus values.

The first approach used for the definition of contact property in the normal direction of the mating surfaces (such as stone to stone and stone to rubble leaf contact surfaces) was hard contact definition, which required the assignment of the single-leaf stone masonry compression test results (as shown in Figure 7.15) to the external leaves of the analysis model, so that the analysis and test results yielded to similar Young's modulus values. However, since it would only be a back-substitution, the hard contact interaction property was not utilized for modelling of single-leaf masonry compression tests.

In the second approach, as also mentioned in the previous section, a softened contact property with a linear function was utilized for the normal direction of the contacting surfaces. The value for the normal joint stiffness k_n was calculated by considering the masonry pattern as series of two springs, one representing the stone unit while the other spring was representing the joint. According to Oliveria (2003) the overall normal stiffness of these two serially positioned springs can be obtained by,

$$k_n = \frac{1}{h_u \left(\frac{1}{E_{wc}} - \frac{1}{E_{unit}} \right)} \quad (7.18)$$

where h_u is the height of the stone unit (in our case equal to 100 mm), E_{wc} is the Young's modulus of the wall (obtained from stone masonry prism compression tests) and E_{unit} is the Young's modulus of the stone unit (obtained from single unit compression tests). The calculated normal joint stiffness values by considering the stone masonry prism and single unit compression tests, explained in Chapter 4, are

given in Table 7.3. Considering the normal joint stiffness values given in this table, the average k_n value was obtained as 50 N/mm³.

Table 7.3 : Joint stiffness values calculated from stone prism compression tests.

Wall	E_{wc} (MPa)	E_{unit} (MPa)	k_n (N/mm ³)
SL-P-1	3400	6000	78
SL-P-2	2366	6000	39
SL-P-3	2079	6000	32
Average	2615	6000	50

The Coulomb friction model defining the interaction of the stone units in the tangential direction was utilized and as also done in the modelling of shear compression tests, a friction coefficient of 0.76 was assigned to the contacting surfaces.

In summary, two analysis approaches were evaluated for finite element analysis of the single- and multi-leaf wallets subjected to uniaxial compression. The sole differences were in the definition of the interaction properties of the stone joints and the mechanical properties of the stone leaves. In the case of multi-leaf walls, two analysis cases with different softened contact stiffness values were evaluated. A softened contact stiffness value of 50.0 N/mm³ (as obtained from single-leaf compression tests) was utilized for the Analysis Case 2, whereas, the same parameter was assumed as 8.0 N/mm³ for Analysis Case 3. It should be noted that, the 8.0 N/mm³ contact stiffness value in the normal direction of the head and bed joints was obtained after a series of trials, so that the experimental axial stress-strain curves of multi-leaf specimens could be approximated with an acceptable error. The three analysis cases of the two followed analysis approaches are briefly summarized in Table 7.4.

The analysis phase was completed by using ABAQUS/Explicit finite element analysis software and quasi-static analysis procedure was followed by applying a mass scale factor of four. The details of the analysis can be found in the previous topic. After each analysis, the energy balance of the system and the kinetic energy levels were checked.

Table 7.4 : Interaction properties of analysis cases for masonry compression tests.

Analysis Case	Interface direction	Single-leaf	Multi-leaf	External leaf property source
Case 1	Normal	N/A	Hard contact	Single-leaf compression tests
	Tangential	N/A	Friction, $\mu=0.76$	Initial shear tests
Case 2	Normal	Softened contact, $k_n=50$	Softened contact, $k_n=50$	Single unit compression tests
	Tangential	Friction, $\mu=0.74$	Friction, $\mu=0.76$	Initial shear tests
Case 3	Normal	N/A	Softened contact, $k_n=8$	Single unit compression tests
	Tangential	N/A	Friction, $\mu=0.76$	Initial shear tests

7.2.3 Analysis results for micro-modelling of uniaxial compression tests

The finite element analysis performed for single-leaf stone prism compression tests revealed a similar damage evolution to what has been observed during the experiments. During the finite element analysis, the first signs of cracking could be observed at an average axial stress as low as 2.0 MPa, particularly at mid-length of the units at the third course (Figure 7.23a and d). At peak stress, the crushing of the stone units initiated at mid-height of the prism as also shown in Figure 7.23b via red minimum principal plastic strain vectors, concentrated at the upper and lower bed joints of the third stone course. Simultaneously, several cracks appeared at the zones marked with the yellow maximum principal plastic strains as seen in Figure 7.23e. These cracks were vertical at mid-height and inclined at upper and lower stone courses. At post-peak region, the damage near the edges of the prism formed a pyramidal shape while the lowest and uppermost stone courses stayed almost intact, mainly due to the confining effect of the support conditions that represent the friction between the masonry prism and the loading platens (Figure 7.23c and f). The comparison of the actual damage observed during the tests with the finite element analysis showed that, the damaged areas identified by the concentrated tensile and compressive plastic strains were coinciding with the cracks and crushed zones recorded during the tests (Figure 7.23g-i). However, as expected, they were not as symmetrical as obtained in the finite element model.

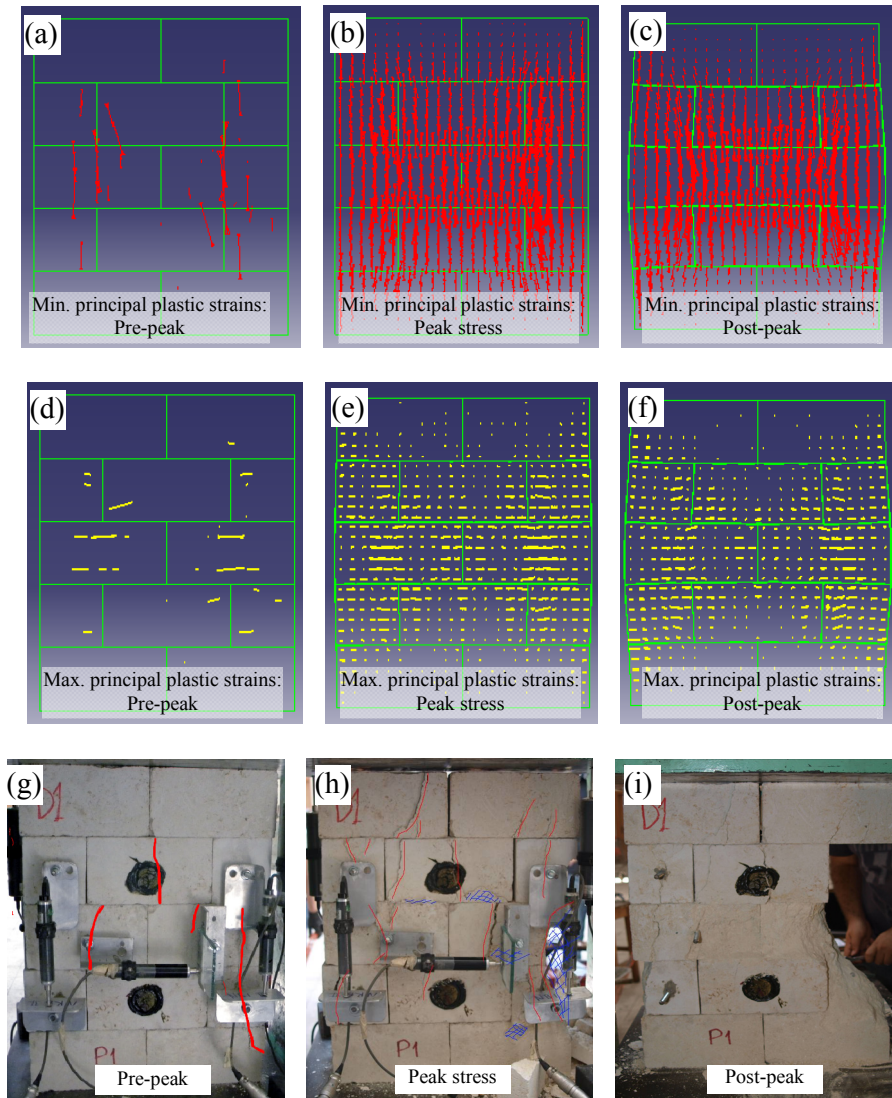


Figure 7.23 : Damage observed in FEA and compression tests of stone prisms.

The axial stress-strain curve obtained from the finite element analysis of the tested stone prisms and the experimental curves can be seen in Figure 7.24. Although it was not possible to mimic the initial hardening behaviour caused by the settlement of the voids and imperfections between the stone courses and loading platens, the presented curves point to a good correlation in means of stiffness in the elastic region. This accuracy is a natural result of the followed analogy for deciding the normal joint stiffness value that primarily aims to fit the stiffness of the masonry wall.

The compressive strength of the ashlar prism, obtained from the finite element analysis, was slightly higher than the average experimental value (9.2 MPa versus 7.9 MPa). Given the wide scatter of the mechanical characteristics of the stone

material, which was clearly observed in the single unit compression tests and presented in Chapter 4, this result can be interpreted as a good prediction.

Shapes of the experimental and analytical curves at the peak and the post-peak regions resembled to each other, particularly in the case of SL-P-1 and SL-P-2 specimens. At peak stress region, both analytical and experimental curves had plateaus with stress rises and drops. Except the SL-P-3 specimen, the post-peak curves had mild slopes.

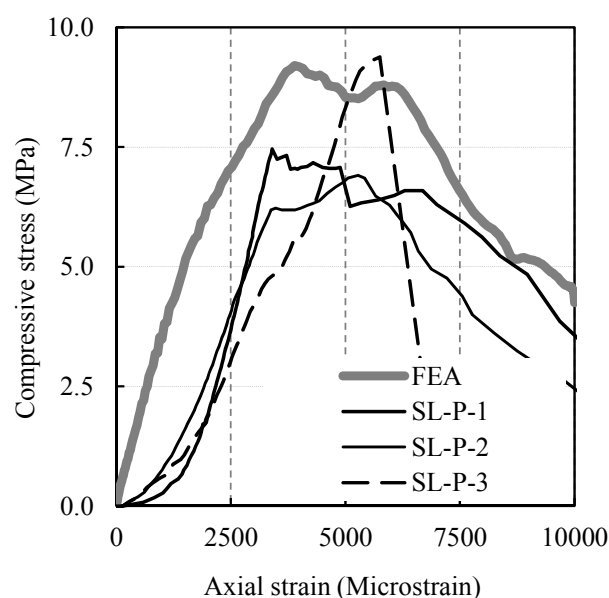


Figure 7.24 : Analytical and experimental compressive stress-strain curves for single-leaf masonry.

The evaluation of the analysis results of the multi-leaf masonry compression tests was not as straightforward as the single-leaf compression tests. As also summarized in Table 7.4, three different analysis cases were followed after performing several trials to approximate the experimental compressive load-displacement curves (Figure 7.25). The first analysis case with hard contact definition in the normal direction of the joint interfaces led to a good approximation of compressive strength since the mechanical properties of the external leaves were derived from single-leaf compression tests. However, the Young's modulus was overestimated. In the second analysis case, a softened contact definition in the normal direction of the joints with a k_n normal stiffness value of 50 N/mm^3 was utilized. It should be noted that, this normal stiffness value was obtained from the compression test results of single-leaf masonry prisms. Although 50 N/mm^3 value led to a good approximation for finite

element analysis of single-leaf prisms, an overestimation of strength and Young's modulus was obtained for compression tests of multi-leaf prisms. Possible reasons for this inconsistency between the finite element analysis results of single- and multi-leaf cases can be attributed to the variation of the material mechanical properties not only between the units of the same exterior leaf but also between the two exterior ashlar leaves. Moreover, the second order effects initiated as a result of the workmanship errors and out-of-plane movement of external leaves, which were observed during the experiments, led to smaller strength and stiffness values in the case of multi-leaf prisms. Apparently, the above-mentioned random variations were not directly considered during the finite element analysis. However, in the third analysis case, they were indirectly considered by using a smaller normal stiffness value of $k_n=8 \text{ N/mm}^3$, that was obtained through a trial and error process. As also seen in Figure 7.25, the third analysis case yielded to a better approximation of the stiffness and strength characteristics to values obtained in the multi-leaf masonry compression tests. In all analysis cases, the post-peak branch of the axial load-displacement curves had steeper slopes than the experimental ones.

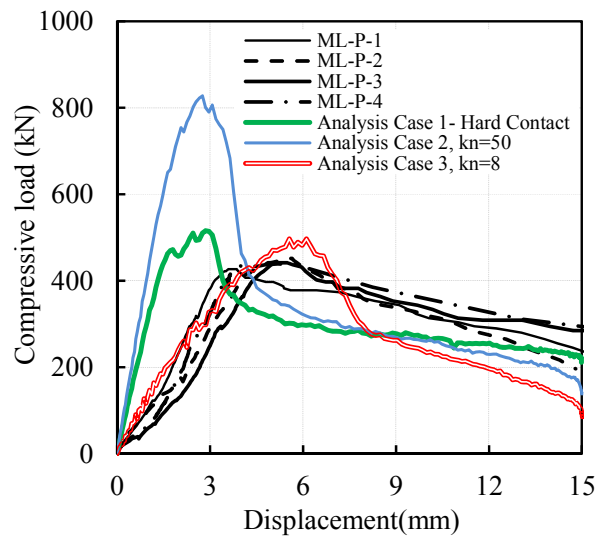


Figure 7.25 : Analytical and experimental compressive load-displacement curves for multi-leaf masonry.

In the pre-peak region, the damage development of the multi-leaf finite element analysis model began with sub-vertical cracks at the stone units at mid-height region (2nd and 3rd stone courses from top), Figure 7.26a and d. Simultaneously, separation between the inner and outer leaves and vertical cracks along the inner rubble core appeared at the transverse section of the model, Figure 7.27d. At peak load level,

crushing of the stone units initiated around the mid-height of the longitudinal and transverse sections. These crushing signs were indicated by accumulation of the minimum principal plastic strains around the mid-height of the prism, particularly at the third stone course level, Figures 7.26b and 7.27b. In parallel with the crushing initiation, vertical cracks were also located at the mid-height level and they were aligned with the head-joints of the 2nd and 3rd stone courses, Figure 7.26e. In the post-peak region of the analysis, maximum and minimum principal plastic strains exhibited an X-like scatter along the longitudinal and transverse directions, pointing to a widespread damage distribution, Figures 7.26c, f and 7.27c and f. In parallel to the single-leaf prisms, the top and bottom stone courses were almost undamaged, particularly due to the confinement supplied by the support conditions.

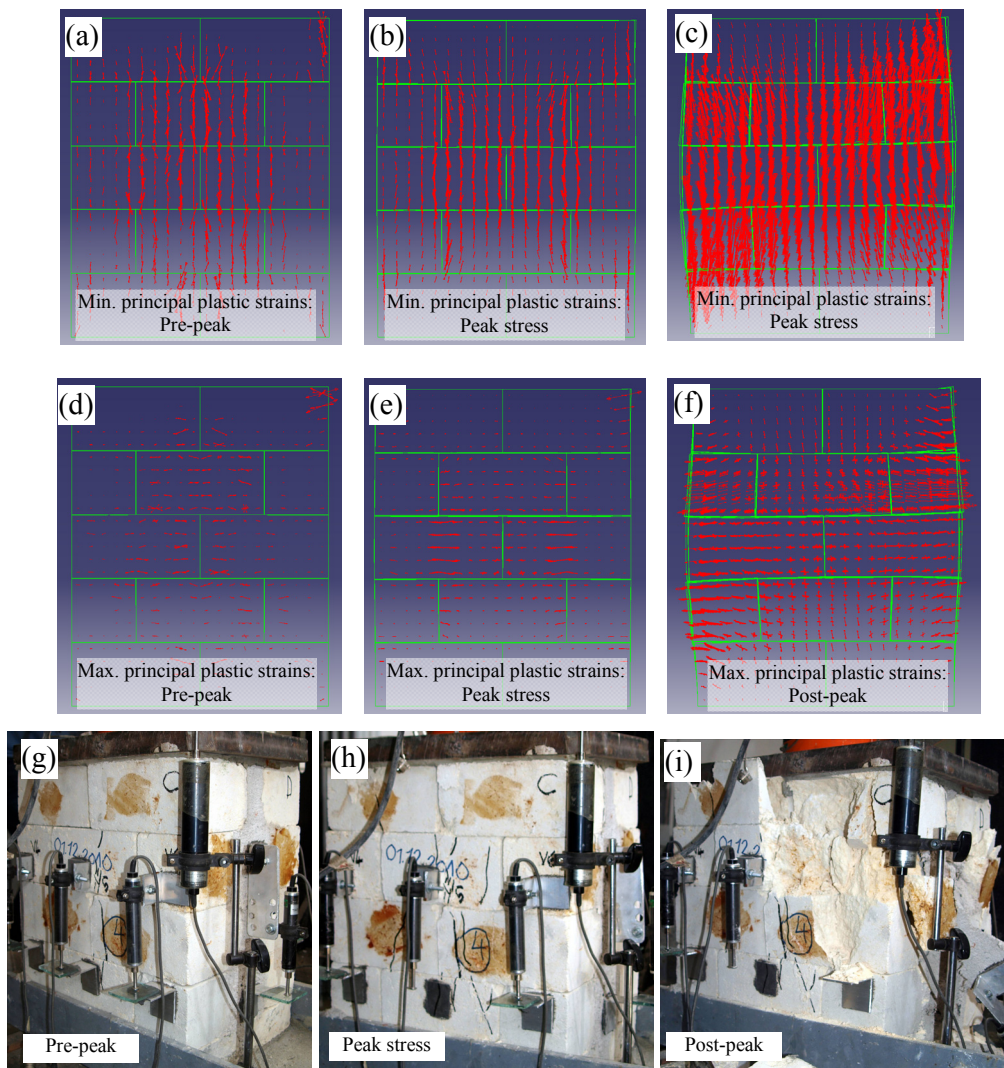


Figure 7.26 : Damage observed in FEA and compression tests of multi-leaf prisms (longitudinal direction).

In summary, comparison of the damage observed during different phases of the tests (Figures 7.26 and 7.27g-i) with the finite element analysis maximum and minimum principal plastic strain distributions (Figures 7.26 and 7.27a-f), reveals the coincidence between the cracked and crushed zones of the model and test specimens. In both cases, the damage evolution started with sub-vertical cracks at the mid-height stone courses and between the rubble-stone interfaces. Further extension of the vertical cracks was followed by signs of crushing, particularly at mid-height, which led to achievement of the peak load level. Finally, during the tests, the post-peak region was accompanied with the spalling of stone and rubble materials by leaving an X-like damage pattern. A similar damage distribution was also identified during the finite element analysis of the multi-leaf prisms.

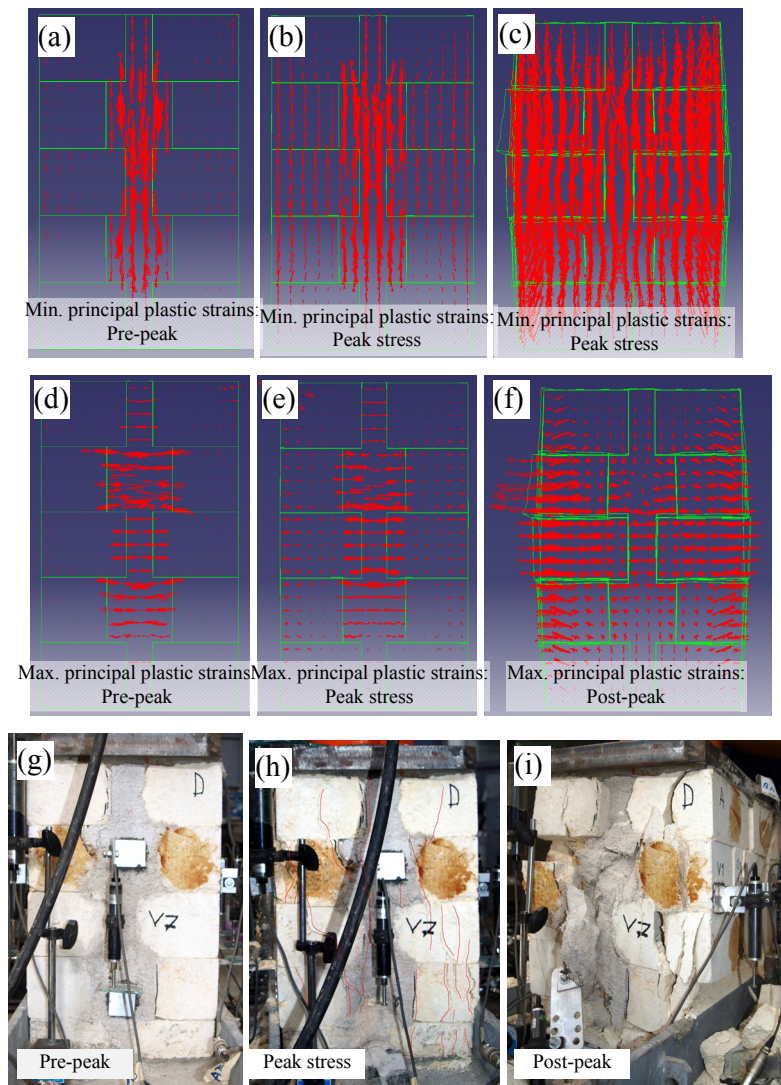


Figure 7.27 : Damage observed in FEA and compression tests of multi-leaf prisms (transverse direction).

As also mentioned earlier, the success of a quasi-static analysis in ABAQUS/Explicit can be quantified by comparing the kinetic and internal energies of the model that are dissipated during the analysis. As a generally accepted rule, the kinetic energy of the deforming material should not exceed a small fraction (typically 5% to 10%) of its internal energy. Otherwise, inertial effects may become significant and risk the success of the quasi-static analysis. In order to make this comparison for the quasi-static analysis of the masonry uniaxial compression tests, typical kinetic and internal energy plots of the analysis for multi-leaf specimens are presented in Figure 7.28. Accordingly, the kinetic energy reaches an oscillating peak zone near and at the strength of the prism. Then a descending branch follows the kinetic energy peak. The spikes in this diagram correspond to occurrence of cracks where drops and rises in the load were observed. The internal energy, which is a sign of elastic and plastic strain energy dissipation, exhibits an increase until the end of the analysis. It is important to note that the order of the kinetic and internal energy magnitudes is highly different. Consequently, comparison of kinetic and internal energies clearly shows that the kinetic energy values are negligibly small with respect to internal energy values. Based on these observations, it can be concluded that, a quasi-static analysis run on ABAQUS/Explicit software can lead to an appropriate and stable solution for analysis of masonry walls subjected to axial compression.

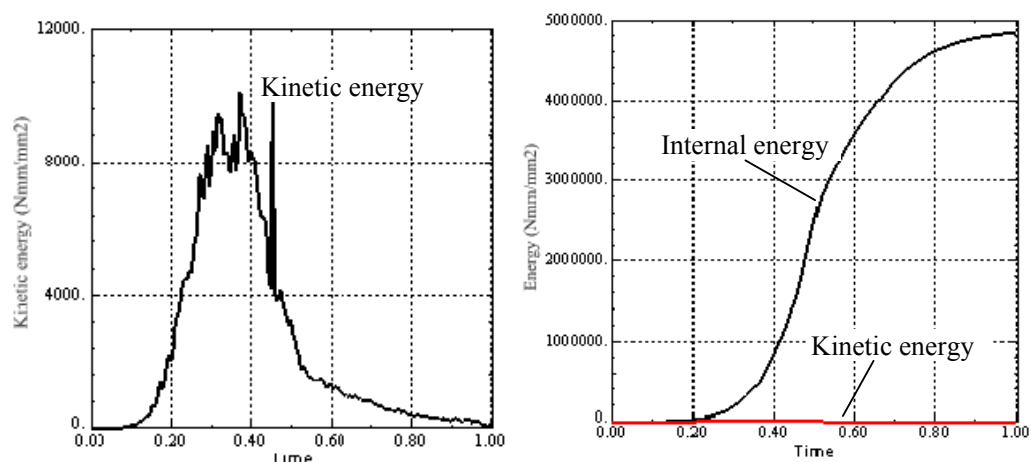


Figure 7.28 : Comparison of the energies for compression loading.

7.2.4 Analysis results for micro-modelling of shear compression tests

In this topic, the results of the finite element analysis, performed on shear compression test specimens, are evaluated and compared with the experimental results. For this purpose, deformed shapes of the analysis models and maximum and minimum principal stresses and plastic strains are utilized. A magnification factor of five was preferred for better perception of deformed shapes. It should be noted that, the figures extracted from finite element analysis and presented in this topic mainly correspond to peak load states. However, figures for other important states that represent the pre-peak, peak and post-peak behaviour are also given in Appendix D.

The minimum and maximum principal stress vectors indicate the formation of the compression and tension struts, so that, the load path inside the wall can be easily visualised. Maximum principal plastic strain vectors appear where cracking of the stone or rubble masonry takes place. However, it should be kept in mind that, in cases where the cracks are not only on the stone units but also between the stone units (in other words, when the head and bed joints open), the deformed shape of the model and the principal stress distributions among the stone masonry can be indicative for damage evolution. Additionally, maximum principal plastic strain vectors on the rubble masonry can also give hint about opening of the dry joints between stone units. Finally, minimum principal plastic strains mark the zones where the compressive strains approach or exceed the compressive strength of the stone or rubble materials.

The S-50-C specimen, which did not have a rubble core in between the stone leaves, was subjected to a constant axial stress of 0.50 MPa. The finite element analysis and the actual test observations figured a shear dominant failure that was identified by the damage development lying along the diagonal direction, as seen in Figures 7.29 and 7.30. It should be noted that the strain and stress data presented in Figure 7.30 and in the corresponding figures of following walls correspond to the state at peak load obtained during the finite element analysis. However, the photographs of specimens shows the after test states. In both analytical and experimental cases, considerable sliding between the bed joints of the stone units could be observed. In the case of FEA, the sliding planes were mainly near the bottom of the wall, while they were distributed along the specimen height during the experiment. As also seen in the maximum principal plastic strain plot given in Figure 7.30, diagonal cracks appear at

the toe region and extend along the diagonal of the wall (with an angle greater than 45 degree with respect to horizontal axis). Similar to the crack pattern obtained during the test, the crack widths were larger at the lower half of the wall. The minimum principal plastic strain vectors indicate the region where crushing of the stone units take place. Both experimental and finite element analysis locate the crushing zone to the toe region.

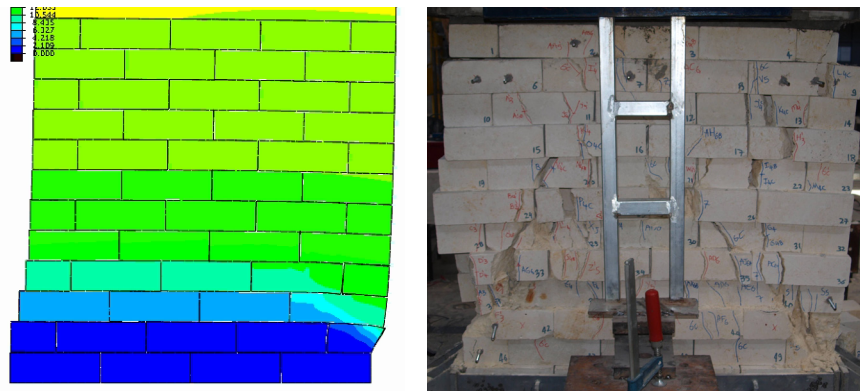


Figure 7.29 : Deformed shapes obtained from FEA and experiment (S-50-C).

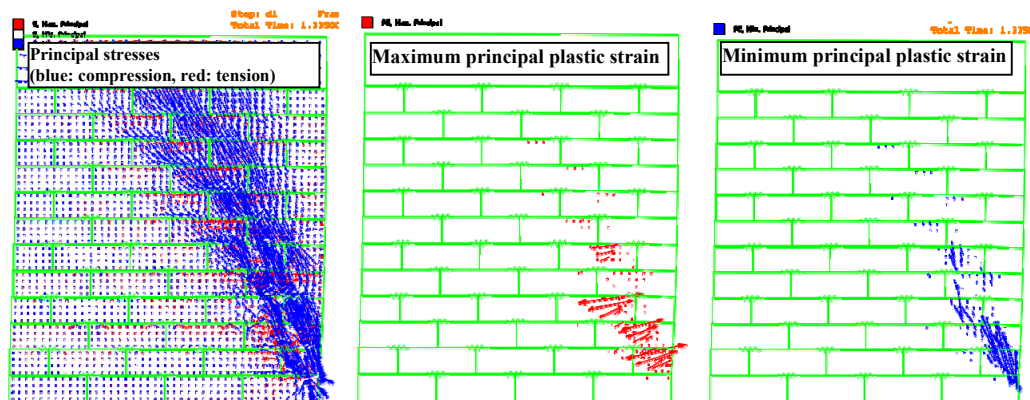


Figure 7.30 : Principal stress and plastic strain distributions at peak load (S-50-C).

The M-50 multi-leaf specimen, which did not have any reinforcement for connection of the stone units, was tested under a constant axial stress of 0.50 MPa. As seen in Figure 7.31, comparison of the deformed shapes of the wall obtained during the FEA and experiment addresses a very similar failure mode. Accordingly, the shear force causes several cracks and, more importantly, leads to wide opening of the head joints between the stone units. The gaps in the flow of the principal stress vectors and the maximum principal plastic strain components, particularly in the distribution among the internal leaf, addresses to a shear dominated failure mode, Figure 7.32. Similar to the experimental response, FEA predicts a failure with extensive crushing of the toe

region. In addition to that, the deformations on the external leaf also coincide with the cracks on the inner leaf as also observed during the experiment.

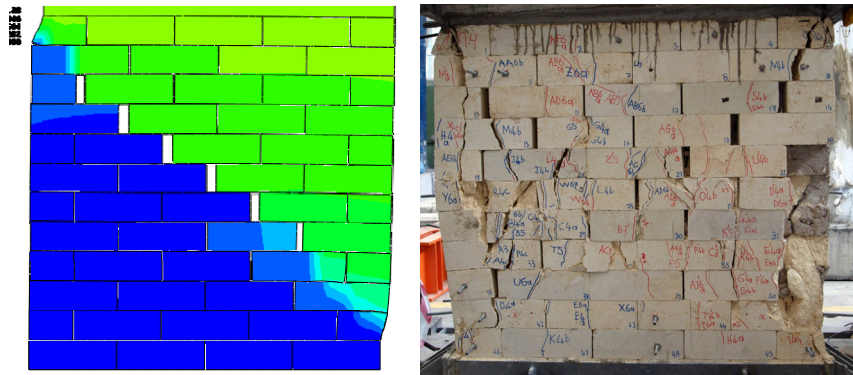


Figure 7.31 : Deformed shapes obtained from FEA and experiment (M-50).

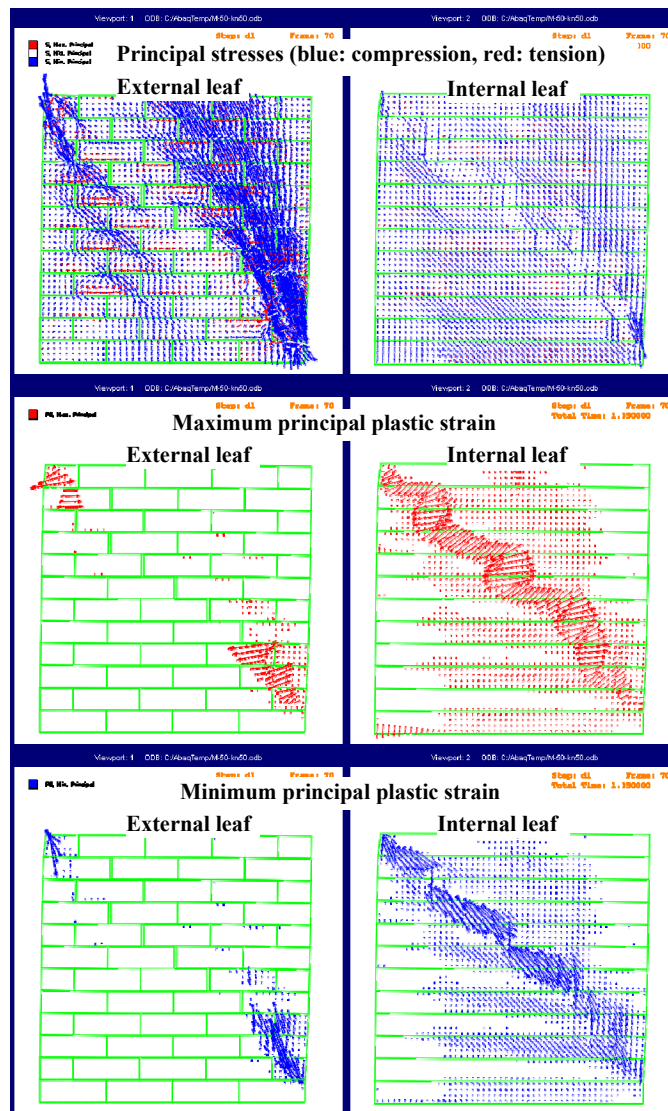


Figure 7.32 : Principal stress and plastic strain distributions at peak load (M-50).

The multi-leaf M-25-C specimen with cramps, tested under the smallest axial stress level of this study (0.25 MPa axial stress), exhibited a shear dominated failure mode with traces of flexural behaviour, not only during the experiment but also during the finite element analysis (Figure 7.33). Vertical separation between the bed joints of the heel region of the wall could be identified particularly at the initial steps of the loading. The stress and strain states presented in Figure 7.34 correspond to peak load level and the horizontal crack at the lower support level can be seen in maximum principal plastic strain distribution of the internal leaf. During the test, the flexural cracks were followed by formation of the diagonal shear cracks. Finally, the failure was observed with the crushing of the lower corners after initiation of a rocking like behaviour (please see test photo in Figure 7.33). The diagonal cracks passing through the stone units and the crushing at the lower corners could also be predicted during the finite element analysis in the form of maximum and minimum principal plastic strain vectors (Figure 7.34). Although disturbances occurred at the principal stress vector distribution among the surfaces of external and internal leaves, mainly due to opening of the joints and cracking of the materials, relatively uniform compression struts could be identified. The compressive stresses, that were aligned along the 45 degree diagonal of the wall, became steeper at the toe region and led to accumulated minimum principal strains which point the occurrence of crushing. In this model and in the following multi-leaf wall models, the deformations (that are indication of cracks) on the external leaves coincide with the cracks on the inner leaf as also observed during the experiment. This match between the damage distributions of inner and outer leaves can be seen in Figures C.46 and C.185 of Appendix C.



Figure 7.33 : Deformed shapes obtained from FEA and experiment (M-25-C).

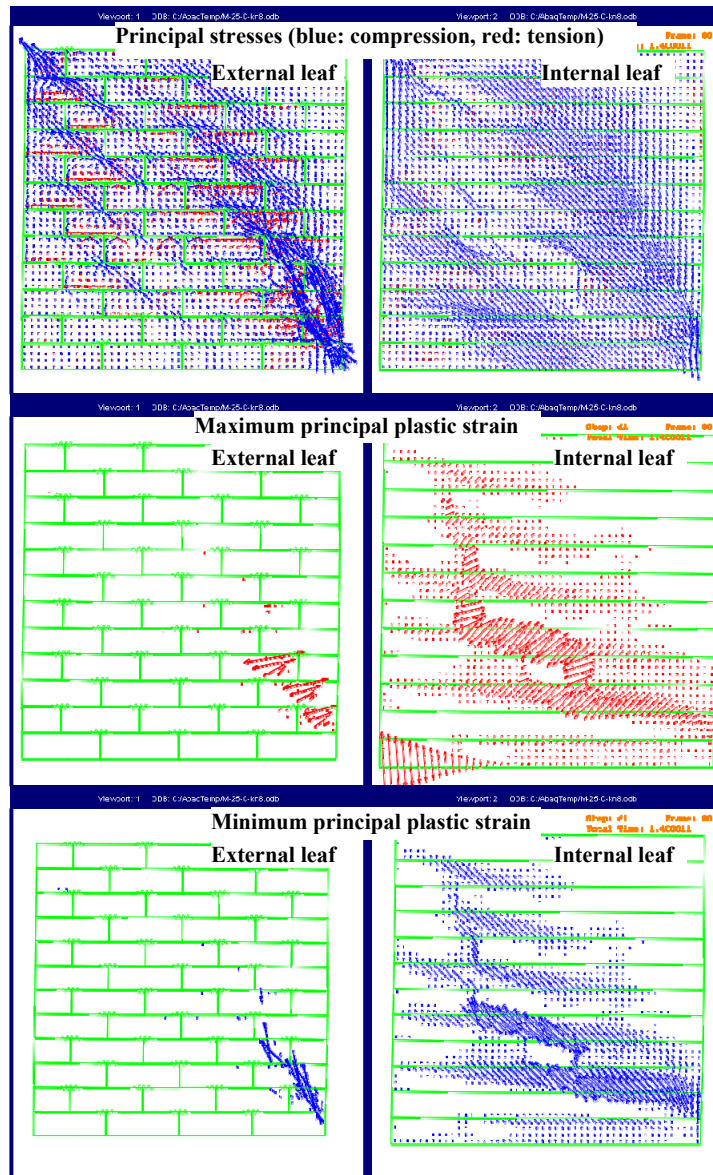


Figure 7.34 : Principal stress and plastic strain distributions at peak load (M-25-C).

The multi-leaf M-50-C specimen with cramps, tested under a medium level of axial stress (0.50 MPa), exhibited a shear dominated failure mode, both during the experiment and the finite element analysis (Figure 7.35). Experimental study and the finite element analysis pointed out that the loss of strength was associated with the crushing of the toe regions after formation of large diagonal cracks. The principal stress vectors at peak load had steeper slopes than the M-25-C wall with lower axial stress level. Moreover, as seen in Figure 7.36, cracks and crushing regions, indicated by maximum and minimum principal plastic strains were more distributed along the diagonal of the wall.

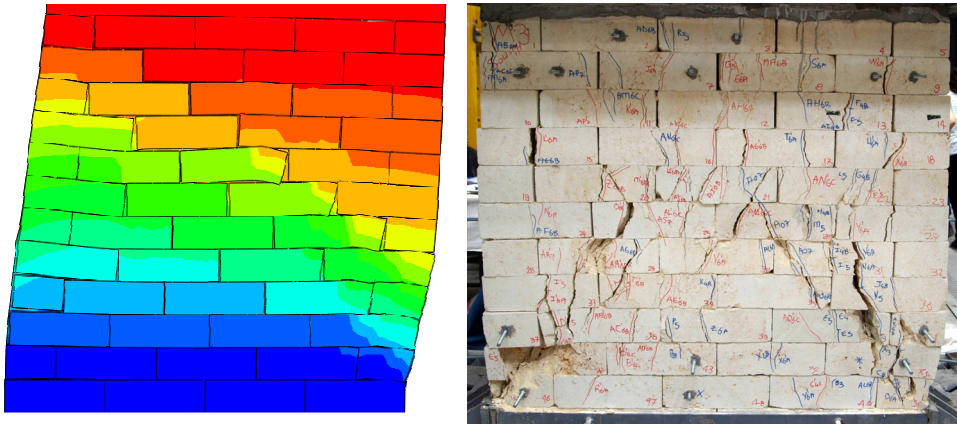


Figure 7.35 : Deformed shapes obtained from FEA and experiment (M-50-C).

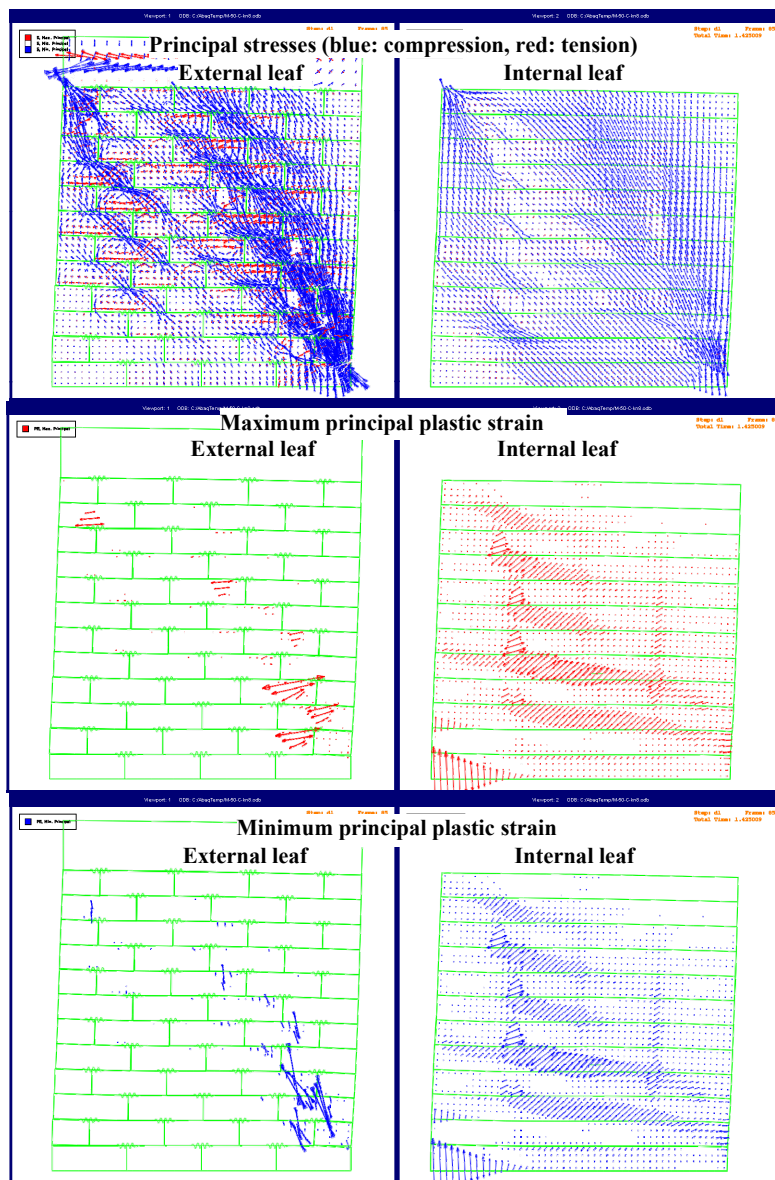


Figure 7.36 : Principal stress and plastic strain distributions at peak load (M-50-C).

Shear controlled failure observed during the experimental study and finite element analysis of the M-75-C specimen with 0.75 MPa axial stress was more apparent than the walls with 0.25 and 0.50 MPa axial stress levels, Figure 7.37. At peak load, the principal compression stress vectors were concentrated at two compression struts with steep slopes, as seen in Figure 7.38. These compression struts were accompanied by widespread maximum and minimum principal plastic strain vectors that pointed to cracking and crushing of the wall constituent materials, Figure 7.38. Although the crushing of the stone units was more concentrated to the toe region, similar to experimental observations, additional crushing along the diagonal cracks could also be identified.

The last specimen investigated by using the micro-modelling approach was M-100-C wall that had the highest axial stress level of this study (1.00 MPa). Similar to previous specimens with lower axial stress levels, the behaviour was dominantly controlled by shear effects, as seen in Figure 7.39. The two main compression struts can be identified in the minimum principal stress distribution of the wall, Figure 7.40. In parallel with the actual crack distribution observed during the test, the cracking pattern obtained from the FEA was quite widespread as indicated by the maximum principal plastic strain vectors plotted in Figure 7.40. The predicted failure state was reached with the extensive crushing of the toe region (please see minimum principal plastic strain distribution in Figure 7.40), as also observed during the experimental study.

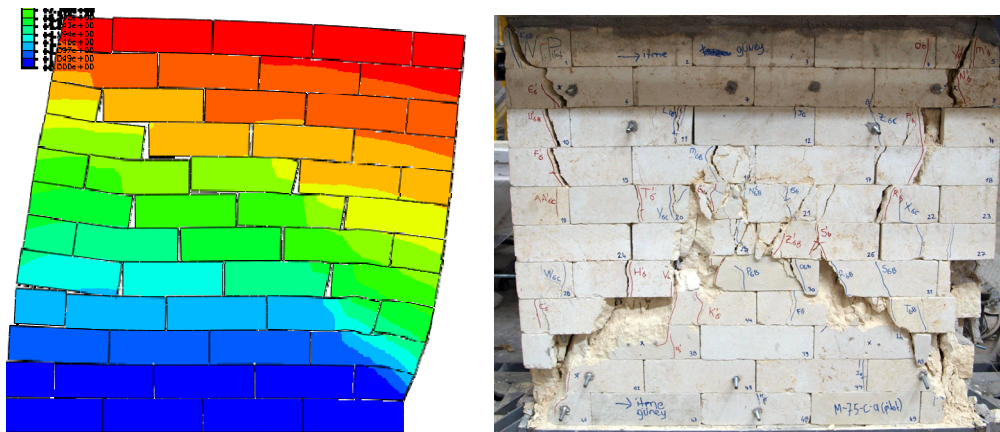


Figure 7.37 : Deformed shapes obtained from FEA and experiment (M-75-C).

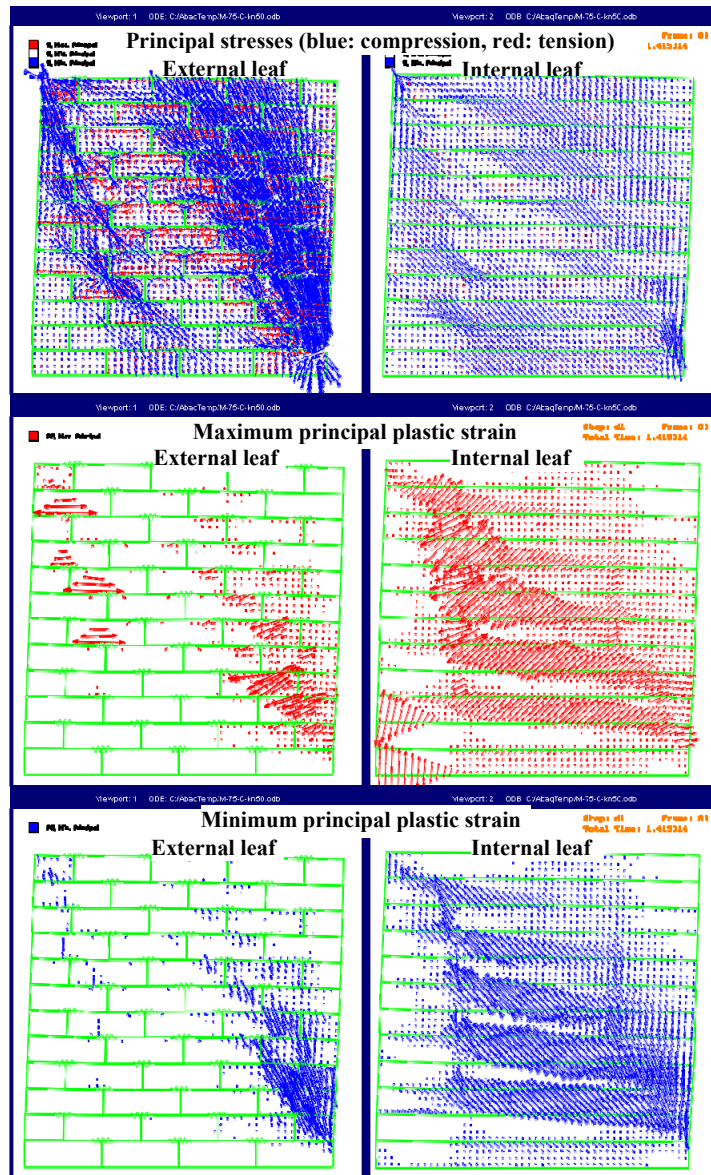


Figure 7.38 : Principal stress and plastic strain distributions at peak load (M-75-C).

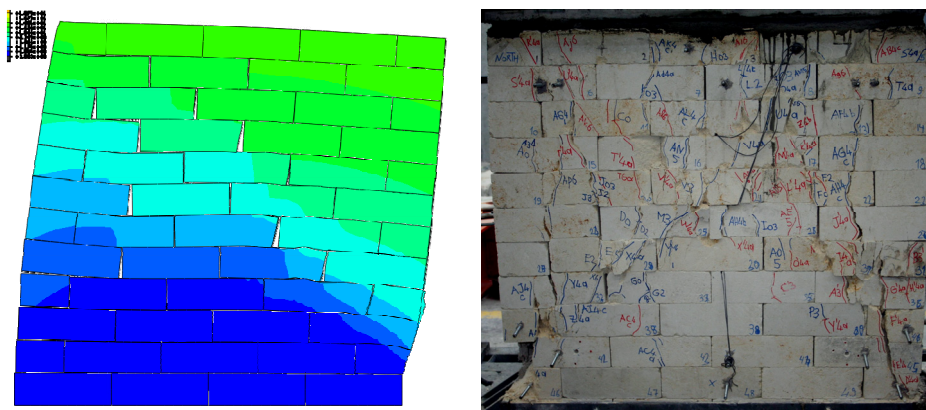


Figure 7.39 : Deformed shapes obtained from FEA and experiment (M-100-C).

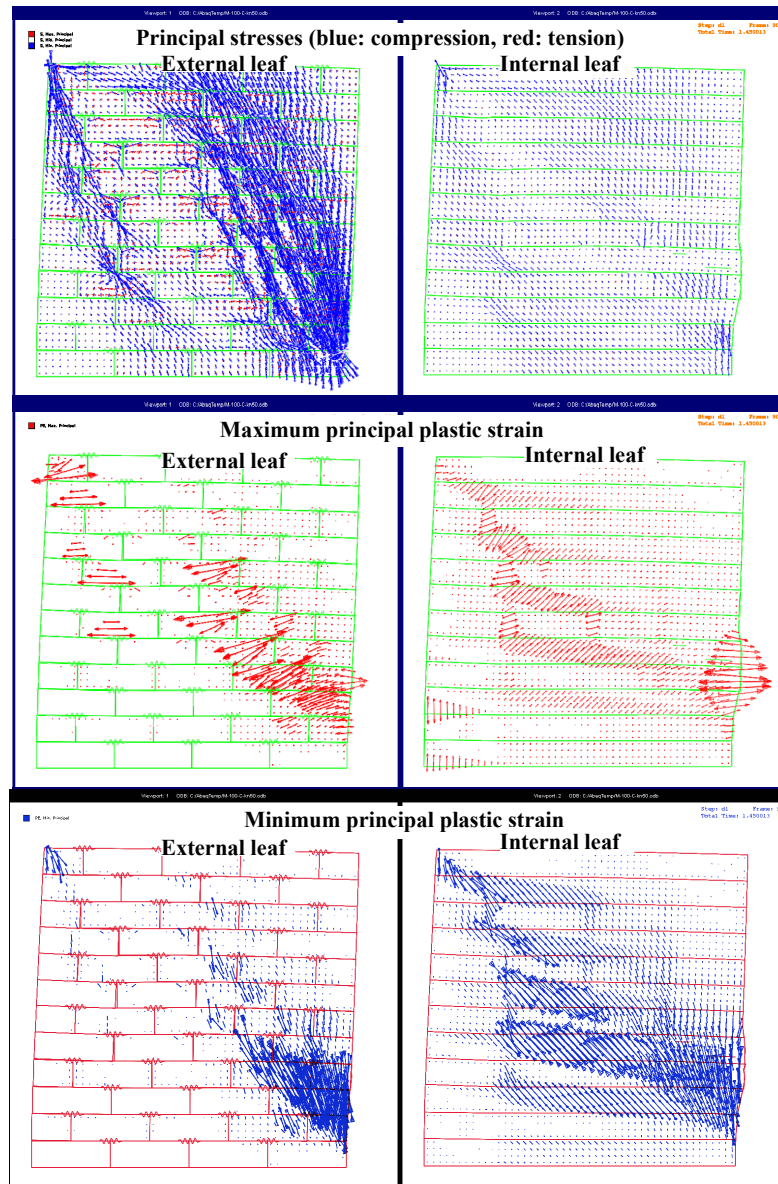


Figure 7.40 : Principal stress and plastic strain distributions at peak load (M-100-C).

The success of the quasi-static analysis concerning the usability of ABAQUS/Explicit for analysis of investigated walls can be quantified by comparing the kinetic and internal energies of the model that are dissipated during the analysis. In order to make this comparison for the quasi-static analysis of the masonry shear compression tests, typical kinetic and internal energy plots are presented in Figure 7.41. As seen in this figure, two peaks occur for the kinetic energy. The first peak corresponds to acting of the axial stress, while the second peak corresponds to initiation of damage. The spikes in the kinetic energy diagram correspond to occurrence of cracks where drops and rises in the load were observed and translation

of the units took place. The internal energy, which is a sign of elastic and plastic strain energy dissipation, exhibits an increase until the end of the analysis and its values are much larger than the kinetic energy. Based on these observations, it can be concluded that, a quasi-static analysis run on ABAQUS/Explicit software can lead to an appropriate and stable solution for analysis of masonry walls subjected to shear compression.

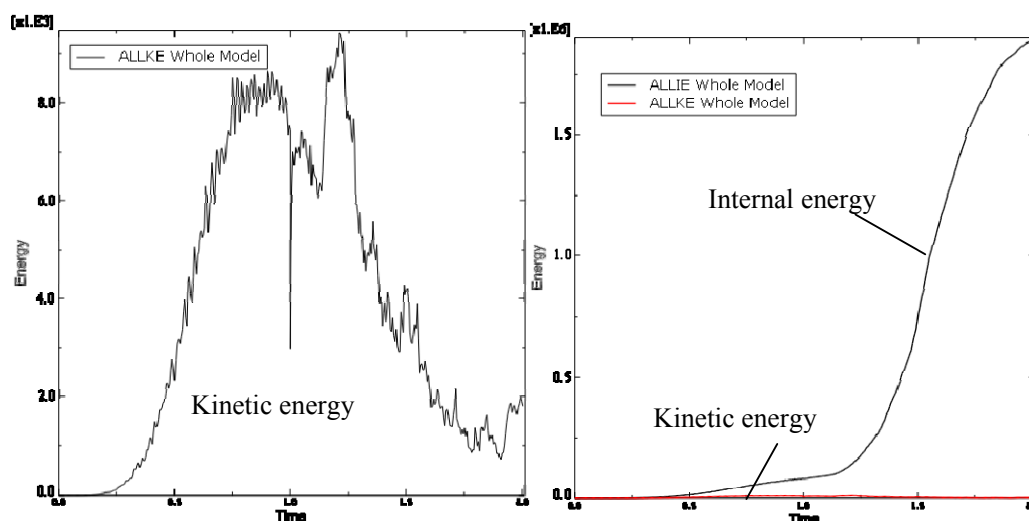


Figure 7.41 : Comparison of typical kinetic and internal energies (M-100-C).

In addition to the damage distribution evaluation presented above, as an indicative of the overall behaviour, lateral load-top displacement curves of the walls were also derived from the finite element analysis and compared with the experimental ones. Since the analyses were carried out under displacement control, the total lateral force corresponding to each lateral displacement value was derived by summing the support reactions in the shear load direction. The top displacement values of the walls in the loading direction were taken as average of the lateral displacements achieved by the nodes of the top loading beam.

Two typical curves obtained for M-25-C and M-75-C specimens are shown in Figure 7.42. In this figure, the red coloured curves, which are drawn by using the raw data obtained from FEA, include several load increases and drops, particularly near and after peak load levels. These spikes occur mainly due to movement of the stone units and formation of damage that is also confirmed by the kinetic energy plots. If the raw data is reorganized by means of a moving average filter, smoother curves drawn in blue colour can be obtained. Please note that, in the following figures, the filtered

curves will be used for the comparison of predicted and experimental load-displacement diagrams.

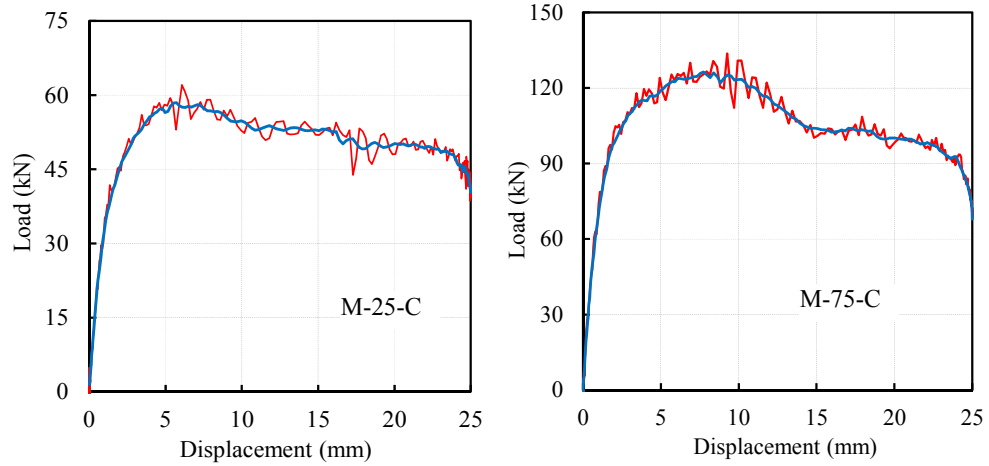


Figure 7.42 : Raw and filtered load-displacement curves for FEA.

Load-displacement responses of the investigated walls obtained through micro-modelling approach are presented in Figure 7.43 for S-50-C and M-50 walls and in Figure 7.44 for M-25-C, M-50-C, M-75-C and M-100-C walls. It should be noted that, for each specimen, three curve predictions are provided. Each of these three curves corresponds to a different analysis case, which were defined for different stone joint axial stiffness definition approaches. As also summarized in Table 7.2, utilized analysis cases include hard and softened contact definitions (dry joint axial stiffness values of $k_n = 8$ and 50 N/mm^3).

Although the micro-modelling of the dry jointed walls included numerous discontinuity surfaces that could cause numerical stability problems, the overall load-displacement response was stable and pre-peak, peak and post-peak branches of the curves could be obtained. Comparison of the experimental curves with the predicted load-displacement curves addresses to a reasonably good agreement for almost all three analysis cases. However, it should be noted that, in some of the specimens (such as S-50-C and M-100-C), considerable difference exists for the strength prediction of the pulling direction. The overestimation of the capacities of these specimens mainly stem from the asymmetry of the experimental response in the pushing and pulling directions.

A closer look at the envelopes of the experimental hysteresis and FEA predicted curves provides better information regarding the accuracy of the analysis approaches

followed for definition of the axial stiffness between the stone interfaces. For this purpose, in Figures 7.43 and 7.44, experimental envelopes of the pushing direction of each specimen are also compared with the numerical curves. Accordingly, it can be noticed that, the stiffness in the ascending branch can be quite accurately predicted by any of the analysis cases, except the M-75-C and M-100-C specimens that had higher axial stresses. However, the analysis case with dry joint axial stiffness value of $k_n = 50 \text{ N/mm}^3$ leads to successful results for these walls. Better performance of the case with higher joint stiffness value ($k_n = 50 \text{ N/mm}^3$) can be attributed to the closure of the gaps between the stone units with higher axial loads applied to M-75-C and M-100-C walls.

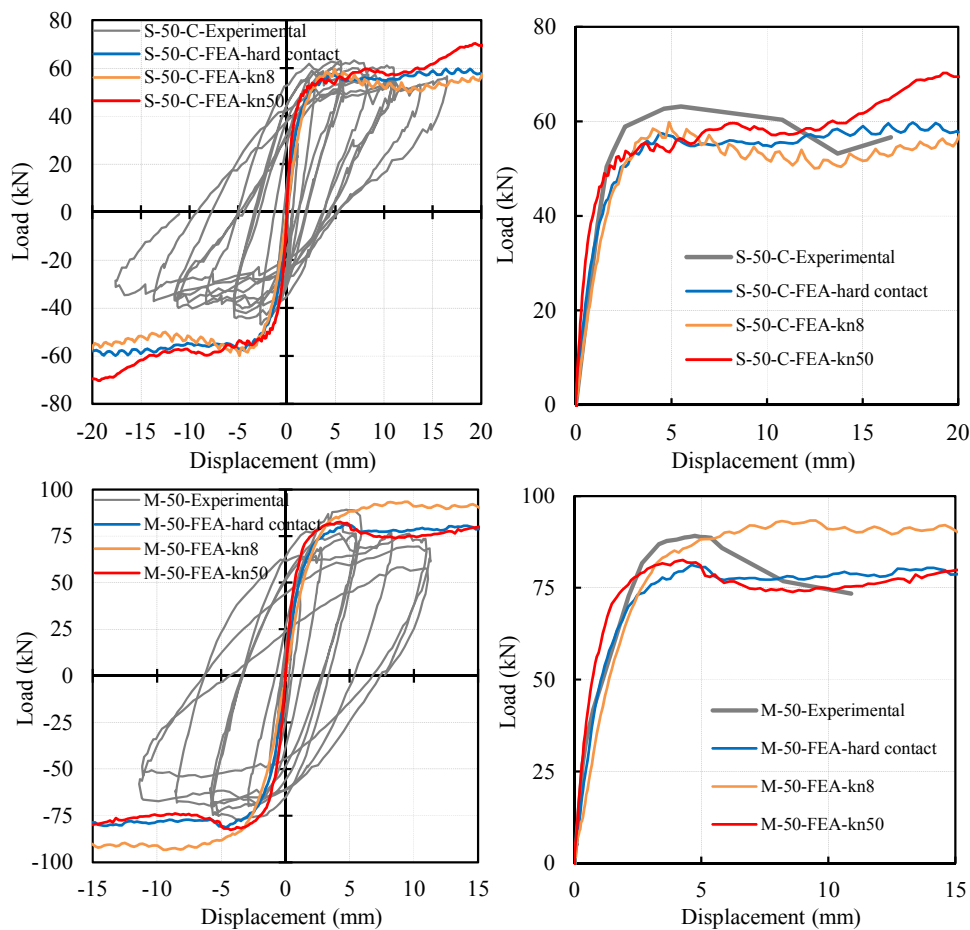


Figure 7.43 : Load-displacement response for micro-modelling (S-50-C and M-50).

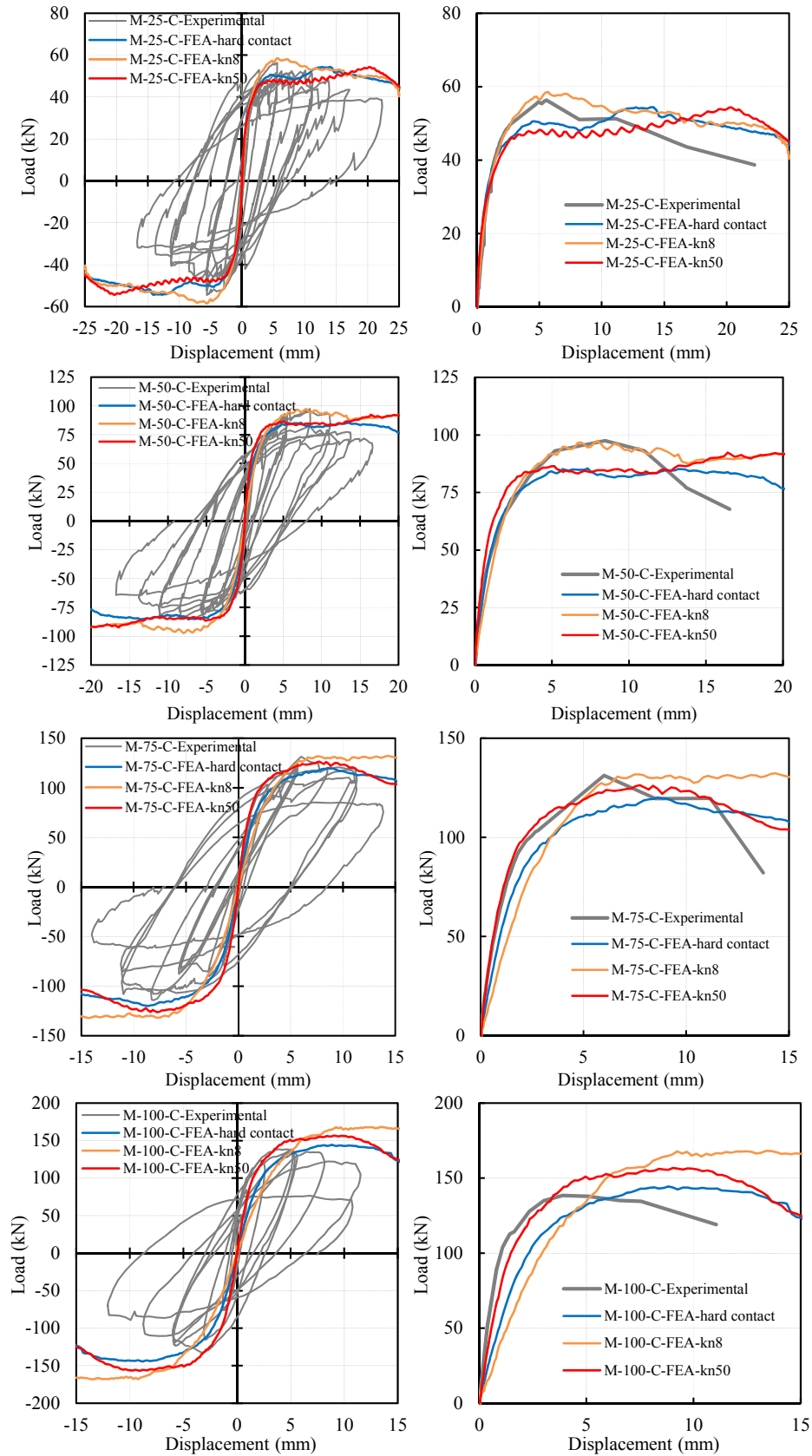


Figure 7.44 : Load-displacement response for micro-modelling (M-25-C, M-50-C, M-75-C and M-100-C).

The discrepancies existing between the experimental and numerical peak load values seem to be reasonable for all analysis cases, particularly, regarding the scatter in the material quality and workmanship. The prediction of the post-peak behaviour is generally acceptable until a point where the load drop gains haste and the slope of the experimental curve becomes steep. Actually, the sudden load drop of the walls due to crushing of the toe regions is also envisaged by the FEA, but at higher displacement ranges as seen in Figure 7.42. One main reason of this earlier strength degradation may be the cyclic loading done during the experiments. Similar to the ascending branch, the descending branch of the curves are better represented by the analysis cases with dry joint axial stiffness value of $k_n = 50 \text{ N/mm}^3$ or hard contact definition. Depending on these observations, it can be concluded that, higher joint axial stiffness values may lead to better predictions for walls with high axial stress levels.

8. CONCLUSIONS

There exists an increasing interest in the structural engineering research community on the rehabilitation and seismic retrofitting of the historical structures. However, due to diversity of the materials and construction techniques, experimental and theoretical studies considering the local conditions and practice become vital for further sustainability of these invaluable assets.

This thesis aims to shed some light on the mechanical characteristics of the materials and masonry walls that were widely utilized in and around the Ottoman capital city Istanbul. In order to achieve this purpose, firstly, wall typologies observed in the classical age Ottoman monumental structures were investigated through literature and site surveys and main characteristics of a prototype wall were determined. Material characteristics were investigated through tests on stone, mortar, iron components and their combinations. During the shear compression tests, eight 1/3 scale walls were constructed and these were subjected to varying levels of pre-compression stresses and cyclic in-plane lateral load reversals. In the next step, the experimental results were evaluated and compared for damage evolution, deformability, strength, stiffness, energy dissipation capability and damping. The experimental load-displacement envelope curves were idealized as equivalent bilinear curves; so that the initial steps for performance based assessment of the investigated type of walls were taken. Then, the load bearing capacities and failure modes obtained from tests were compared with available failure envelopes defined by different researchers. Finally, an attempt for investigating compression and shear compression behaviour of tested walls through explicit finite element analysis and micro-modelling approach was done. The conclusions drawn from this research are presented in this chapter.

8.1 Historical Multi-Leaf Stone Masonry Walls in Turkey

A survey of the existing limited literature on the main aspects of the historical masonry walls of classical age Ottoman period monumental structures reveals that it

is generally typical for Ottoman architecture, particularly for the monumental structures in and around Istanbul, to have thick ashlar layers made of limestone (Küfeki) covering a core leaf that generally consists of rubble stone and mortar. Limestone blocks are generally connected to each other via metal cramps (and in some cases with pins), which are fixed to the stone units with molten lead, in case no mortar is used between the units. The site surveys done for three ancient structures in and around Edirne city, where these details could be partially observed due to existing damage also backed the literature-supplied information and, in addition, supplied some numerical values on the geometrical aspects. Depending on the information collected, in this study, a prototype wall was defined which was then scaled down to obtain the mechanical characteristics of a model multi-leaf wall.

8.2 Characterization of Materials

The mechanical characterization of materials utilized in investigated multi-leaf stone masonry walls required an extensive experimental campaign on the constituent materials and masonry prisms.

The compression tests performed on single limestone units, obtained from a quarry near Istanbul, showed that:

- The mean compressive strength obtained from the single unit compression tests was about 18.0 MPa while standard deviation and coefficient of variation were 5.3 MPa and 29.6%, respectively.
- The compressive strengths varied in a significantly wide range (from 7.5 MPa to 25.5 MPa)
- Mean value of the Young's modulus was approximately 5800 MPa with a standard deviation of 2059 MPa and coefficient of variation of 35.9%.
- Young's modulus of single units was approximately equal to 320 times the compressive strength of single units.

Indirect tensile tests (three-point bending tests) performed on the Küfeki stone samples gave a mean modulus of rupture value of 3.15 MPa with a standard deviation of 0.42 MPa and a variation coefficient of 13.2%.

A mortar mixture, which is representative of historical mortars (such as Khorasan mortar) in means of mechanical characteristics, was sought for resembling the mortar used together with rubble stone in the inner part. For this purpose, a number of trial mixtures were prepared and compression tests were carried out. Eventually a mixture, which has 4.80 MPa compressive strength at 28 days age, was obtained.

Standard cylinder compression and splitting tests were carried out for investigating the main mechanical characteristics of rubble masonry that was used to fill the gap between the two exterior ashlar masonry leaves of the walls. Uniaxial compression tests enabled the derivation of the axial stress-strain curves and related strength and deformation characteristics; while the splitting test, which is an indirect tension test, was performed to have an idea on the tensile strength of the material. Test result and the stress-strain curves indicated that the heterogeneous structure of the composite material, that was formed by a mortar matrix and randomly distributed stone pieces placed without compaction, led to an expected level of variance in the mechanical characteristics. Unexpectedly, the strength did not develop with age. On the contrary, the mean compressive strength dropped from 3.30 MPa at 28 days to 1.70 MPa at 180 days. The Young's modulus values were much lower than that of stone units (1935 MPa at 28 days and 607 MPa at 180 days). Tensile strength of rubble masonry obtained from cylinder splitting tests was around 0.30 MPa at 28 and 90 days.

The usage of metal members in the heritage structures was described briefly and the available information on the mechanical and geometrical characteristics of these reinforcements were compiled from the literature. Then the tests carried out on two cramps discovered at the Edirne II. Bayezid Mosque site during the restoration of the structure were presented and the test results were compared with other studies performed on ancient wrought iron members. The tests on original cramps included chemical, microscopic and metallurgical investigations, which showed that the material had a very low carbon content and exhibited a mainly ferritic character with several slags in the microstructure. By using the empirical equations based on surface hardness, an approximate tensile strength in the order of 300-350 MPa could be obtained. Regarding this information, a commercially available steel material that can represent the original wrought iron cramps during the experimental campaign of this study was selected.

The shear behaviour of the dry joints between the stone units was characterized through a number of initial shear tests. The triplet type tests were performed under three distinctive compressive stress levels. In addition to that, the effect of surface texture on the shear behaviour of dry joints was also investigated. The coefficient of friction value for smooth interfaces was obtained as 0.76, while it was 0.74 for rough interfaces. In order to assess this unexpected result, an attempt was done for characterization of the smooth and rough friction surfaces. The surface profiles were obtained by a setup developed in the laboratory. After the filtering of the waviness and roughness components from the surface profile, it was observed that independent of the surface texture type, the contact between the units was not uniform. Since the net contact area was smaller in the case of rough textured interfaces, shear strength did not increase considerably. Based on these observations, the mechanical constant estimation for friction of such dry joints should be approached with caution and the obtained friction coefficients should be accepted as global parameters rather than exact values. Finally, it was also understood that, usage of smooth surfaced Küfeki stone units during the masonry wall specimen construction was acceptable when the test results were compared with the results of more realistic rough surfaced ones.

In addition to the single stone unit compression tests, three ashlar masonry prisms were tested under uniaxial compression. These prisms were representative of the external leaves of the investigated multi-leaf stone masonry walls. The stress-strain curves of single-leaf prisms were almost linear until the vicinity of the peak stress. The average values for compressive strength, strain corresponding to compressive strength and Young's modulus were obtained as 7.9 MPa, 0.47% and 2615 MPa, respectively. Average masonry prism compressive strength turned out to be 44% of the average single unit compressive strength. Similarly, mean Young's modulus of the masonry prisms was obtained as 45% of the average Young's modulus obtained from single unit compression tests. In average, the ratio of Young's modulus to compressive strength of single-leaf prisms was about 330.

The behaviour of multi-leaf prisms under uniaxial compression was also investigated. Although the damage evolution was similar to that of single-leaf prisms, additionally, vertical cracks along the interfaces of the external and internal leaves were observed even at early steps of the loading. The overall axial deformability was significantly higher than that of single-leaf prisms. Considerable

detachment could be measured between the leaves of the prisms that was more evident particularly after reaching the peak load. The obtained 3.7 MPa average compressive strength was remarkably below the 7.9 MPa strength obtained for the single-leaf masonry prisms. This difference can be attributed to the second order effects caused by the leaf detachment, extensive and unsymmetrical distribution of damage at the outer leaves and wide scatter of the mechanical characteristics of the utilized materials. Similar to the compressive strength, the 576 MPa Young's modulus value was significantly less than that of the single-leaf prisms (2615 MPa). In average, the ratio of Young's modulus to compressive strength of multi-leaf prisms was approximately equal to 155.

8.3 Shear Compression Tests

In order to investigate the in-plane seismic behaviour of multi-leaf stone masonry walls observed in the classical era Ottoman structures, realization of an experimental program was planned. Effects of three main parameters on the shear behaviour of historical stone masonry walls were investigated during the experimental campaign. These parameters were the level of pre-compression stress, existence of reinforcement such as cramps and pins and existence of rubble masonry core. The planned series of experiments consisted of quasi-static cyclic shear tests performed on 1/3 scaled-down model walls. During the application of lateral displacement cycles, vertical stress level was kept constant. The applied axial stress levels were 0.25, 0.50, 0.75 and 1.00 MPa. Specimens tested for investigation of the effects of reinforcements and rubble core were tested under a reference pre-compression stress of 0.50 MPa. The experimental study carried out to investigate the behaviour of multi-leaf stone masonry walls provided data on the failure mode, lateral shear load capacity, ductility, energy dissipation and damage characteristics.

Although all shear compression test specimens clearly exhibited a shear-dominated behaviour, bending effects were also observed particularly for walls with relatively lower axial stress levels. The damage development was as follows:

- The damage development generally started with vertical cracks passing through the stone units that started to occur at initial displacement cycles. Due to stress concentrations around the imperfections between the units, these cracks could develop at random locations among the wall surfaces,

particularly in the case of walls with higher pre-compression stresses. Simultaneously, on the transverse faces of the walls, sub-vertical separation cracks along the interfaces of the inner and external leaves could be observed.

- At early cycles, except the specimen with the highest level of axial stress considered in this study (1.00 MPa), horizontal flexural cracks were observed between the initial stone courses at the lower half of the specimens that were indications of flexure effects.
- Further increase of the lateral displacement caused formation of shear cracks along the diagonals of the walls. Due to the low tensile strength of the lime stone used in the specimen production and the restraining effects of the cramps interconnecting the head joints of adjacent units, these cracks were crossing the stone units rather than the head joints. The first diagonal cracks generally appeared near the 0.50% drift ratio. They emerged either before, at or after the peak load capacities so that the average ratio of the cracking load to lateral strength became 94%. Tested walls reached their maximum resistance between 0.30 and 0.70% drift ratios.
- Thanks to the existence of the cramps, that were transferring tension forces from one unit to adjacent one at singular points, diagonal cracks were wide spread across the wall faces. No yielding was observed for the strain gauge instrumented cramps of the tested specimens. However, in the case of the unreinforced multi-leaf specimen, stair like opening of the joints was apparent.
- Capacities of the cramps and pins could not be fully utilized due to premature cracking of the stone units.
- After opening of the wide spread diagonal cracks and failure of some of the cramp anchors due to cracking of the units, a rocking-type mechanism took place around the lower corners of the walls. The concentrated compressive stresses around the toes caused crushing of these zones. Assumed ultimate state, which corresponded to 20% strength loss, was generally achieved between 0.80 and 1.50% drift ratios. Drift ratios corresponding to 20% strength loss tended to decrease with the increase of the pre-compression stress level.

- In walls, where the vertical compression stress levels were rather small, sliding at some of the stone courses could also be observed.
- Comparison of the shear test results of single-leaf and multi-leaf walls indicated that the existence of the low strength rubble masonry enhanced the integrity of the wall, so that stresses could be better distributed among the stone units.

The obtained hysteresis loops showed that the walls exhibited a highly nonlinear response with significant residual displacements after each load reversal, which is typical for shear-controlled behaviour of masonry walls. When subjected to low and moderate axial compression stresses, larger displacement capacities were achieved with lower lateral strengths. On the other hand, for the obtained failure modes, walls with higher axial pre-compression stresses reached higher lateral force capacities with less displacement capacities.

Introduction of cramps slightly developed the deformability of the multi-leaf wall M-50-C by increasing the ultimate drift ratio capacity by 25% with respect to the URM M-50 wall. The deformation capacity was further increased by 80% with respect to the URM M-50 wall with the combined utilization of cramps and pins in multi-leaf M-50-CP specimen, respectively. Moreover, strength characteristics of the multi-leaf reinforced walls were also developed to some extent (9% increase with cramps, 15% increase with cramps and pins). In the case of single-leaf S-50-C (with cramps) and S-50-CP (with cramps and pins) walls, the introduction of pins led to 14 and 35% increases for lateral load and deformation capacities, respectively.

Though it consisted of a very weak material with respect to stone masonry, presence of rubble masonry infill significantly enhanced the strength characteristics of the multi-leaf walls. The lateral load capacities of multi-leaf cramp and cramp & pin reinforced walls were 65 and 54% higher than load bearing capacities single-leaf walls, respectively.

Experimentally obtained load-displacement curves were translated into idealized equivalent bilinear curves so that parameters concerning the load-displacement response of the walls could be better investigated. The results obtained from hysteresis and idealized bi-linear curves can be summarized as follows:

- For the investigated walls, the more axial stress was applied the more idealized lateral resistance was achieved. On the other hand, ultimate displacement capacity was inversely proportional with the pre-compression stress level.
- All walls exhibited considerable deformation capabilities with ductility values that vary between 5.2 and 10.7. Introduction of cramps to multi-leaf specimens resulted with 33% increase in average ductility factor and 15% increase in the cyclic ductility factor. Moreover, if pins were used in addition to cramps; a major development in the average ductility factor of URM wall, which is in the order of 100%, was achieved.
- Increase in the axial stress level led to higher initial secant stiffness values. Unlike the introduction of pins, the introduction of cramps did not bring an increase in the stiffness parameter. Although rubble masonry material is very weak with respect to the stone material, a major difference existed between the single- and multi-leaf walls, particularly until the formation of major damage. Stiffness values of all tested walls approximated to each other particularly after formation of the diagonal cracks.
- Until the 0.50% drift ratio, where diagonal cracks generally occurred, all specimens dissipated similar and low amount of energy. After this drift ratio, the energy dissipation of walls with different pre-compression stress levels began to deviate from each other. Thus, the wall with the highest axial stress and the most distinctive shear-dominated overall behaviour, exhibited a more dissipative character. In the case of walls with lower axial stress levels, normalized cumulative energy dissipation values were inversely proportional with the pre-compression stresses.
- Considerable equivalent damping ratios varying between 8 and 21% were obtained for the drift ratios less than 0.50%. The damping ratios had an increasing trend and climbed up to 38% at further drift ratios where significant residual deformations occurred during the load reversals. These equivalent damping ratios were in good agreement with the ratios reported in the literature.

The data collected during the shear compression loading of the tested specimens supplied base for calculation of parameters such as shear strength, tensile strength and shear modulus:

- For the investigated walls, regardless of the reinforcement type, the shear strength increased as the axial stress increased.
- An analogy with the Mohr-Coulomb expression given by the Eurocode 6 (1996) and Turkish Seismic Design Code (2007) showed that the shear strength under zero compression strength was 0.09 MPa while the global friction coefficient turned out to be 0.30.
- The tensile strength of masonry calculated by using the Turnsek and Sheppard (1980) approach yielded to an average value of 0.10 MPa. The tensile strength tended to increase with the increase in the pre-compression stress level. The presence of reinforcement in the form of cramps and pins slightly increased the tensile strength, since the cracking of the stone units was decisive. On the other hand, the angle of principal stresses exhibited a tendency to decrease (which means that cracks became more close to vertical) as the pre-compression stresses increased.
- Lower pre-compression stress levels led to smaller shear moduli. Moreover, existence of cramps did not significantly influence the shear modulus. However, existence of pins resulted with higher moduli. The inner rubble masonry core caused a considerable increase in the global shear stiffness.

Two different failure models, namely Mann and Müller (1980) and Turnsek and Cacovic (1971) models that define the failure field of masonry with respect to shear and normal stresses, were utilized for comparison of the predicted and experimentally observed failure modes. Although these models were originally developed for unreinforced brick masonry walls, the predictions for the failure modes of tested specimens were reasonable.

Inelastic displacement capabilities of the investigated single- and multi-leaf stone masonry walls were discussed considering the performance based design concepts. For this purpose, criterion for limit states such as Immediate Occupation, Life Safety and Collapse Prevention was defined based on stiffness, strength and damage. Then

the performance levels of the tested specimens were indicated by considering these limit states. Accordingly:

- Immediate Occupancy limit state was reached around 0.20% drift ratio (varying between 0.15 and 0.25%).
- The drift ratios corresponding to Life Safety limit state were scattered between the 0.35 and 0.75% drift ratio values.
- The Collapse Prevention state was achieved with the development of wide cracks and crushing of the stone units at drift ratios in between 0.80 and 1.50%.
- Increase in the level of axial stress generally led to decrease in the drift ratios corresponding to Life Safety and Collapse Prevention limit states of the wall members.
- The introduction of the cramp or cramp and pin reinforcements to the multi-leaf walls considerably delayed the Collapse Prevention limit state.
- For all limit states, the drift ratios obtained in this study point to a good agreement with the studies carried out by other researchers.
- Comparison of drift ratios of performance levels recommended by a number of major codes and guidelines shows that there exists a strong need for further experimental studies, particularly for shear dominated masonry walls with various typologies.

8.4 Finite Element Analysis

Finite element analysis of tested wall specimens was carried out by following a micro-modelling approach for building up the geometrical model and defining material characteristics. The analytical and experimental results were evaluated by comparing the damage evolution and load-displacement curves. The modelling was done for single- and multi-leaf uniaxial compression and shear compression tests. The following conclusions can be drawn on the obtained results:

- For both types of tests, the development of damage, locations of cracking and crushing, load flow paths and failure modes were successfully predicted by the developed explicit finite element model.

- The predicted axial stress-strain curves of the single-leaf masonry compression tests pointed to a good correlation in terms of elastic stiffness, strength and shape of the descending branch.
- The prediction of the axial stress-strain curve of the multi-leaf masonry compression tests was not as straightforward as the single-leaf compression tests. Determination of the joint normal stiffness required a number of trials until the desired accuracy was achieved. This situation can be attributed to reasons such as; variation of the material mechanical properties not only between the units of the same exterior leaf but also between the two exterior ashlar leaves, second order effects initiated as a result of the workmanship errors and out-of-plane movement of external leaves and asymmetric distribution of damage.
- The plots of principal plastic strain and principal stress vectors obtained from the finite element models of the shear compression tests showed a good match with the crack patterns of the tested walls.
- Although the micro-modelling of the shear compression specimens included several discontinuity surfaces due to presence of dry joints between the stone units that could cause numerical stability problems, the overall load-displacement response was stable and pre-peak, peak and post-peak branches of the curves could be obtained.
- In parallel with the experimental results, stress values observed in the spring elements that represented the cramps of the tested walls did not reach yielding. Consequently, the capacities of the cramps could not be fully reflected to the overall behaviour of the finite element model due to premature cracking of the units, as was the case during the experiments.
- Success of the quasi-static analysis procedure was checked via comparison of the kinetic and internal energies together with the energy balance of the finite element models. The spikes in the kinetic energy diagrams corresponded to occurrence of cracks and translation of the units where drops and rises in the load were observed. The internal energy, which is a sign of elastic and plastic strain energy dissipation, exhibited an increase until the end of the analysis and its values were much larger than the kinetic energy. Accordingly, the

explicit analysis led to an appropriate and stable solution for investigated masonry walls subjected to shear compression loading.

8.5 Recommendations for Further Studies

In order to complete and develop the subjects studied in the thesis, further investigations are proposed as follows:

- Evaluation of the cyclic behaviour of similar masonry stone walls with different height to length ratios.
- Realization of tests under higher axial stress levels.
- Evaluation of the cyclic behaviour of similar masonry stone walls with different leaf thicknesses.
- Evaluation of the cyclic behaviour of similar masonry stone walls with higher strength stone units.
- Evaluation of the cyclic behaviour of masonry walls with alternating rows of stone and brick units.
- Investigation of the possible strengthening techniques that are applicable on the historical structures.
- Experimental characterization of the out-of-plane behaviour of multi-leaf stone masonry walls.

REFERENCES

- ABAQUS** (2009). *Theory manual*, Hibbit, Karlson and Sorenson, Inc, Providence.
- Akman, M.S.** (1978). *Deney ve ölçme tekniğine giriş*, İstanbul Teknik Üniversitesi, İstanbul.
- Almeida, C., Costa, C.Q., Guedes, J., Arede, A., and Costa, A.** (2010). *Mechanical Behaviour Analyzes of One Leaf Stone Masonry Walls*, VI Congreso Internacional Sobre Patologia Recuperacion de Estructuras, Cordoba, Argentina.
- Ambraseys, N.N. and Finkel, C.F.** (1995). *The seismicity of Turkey and adjacent area, a historical review, 1500-1800*, Eren, Istanbul.
- Anagnostopoulos, S.A. and Moretti, M.** (2008). Post-earthquake emergency assessment of building damage, safety and usability- Part 2: Technical issues, *Soil Dynamics and Earthquake Engineering*, **28**, pp. 233-244.
- Arioğlu, E. and Arioğlu, N.** (1999). Mimar Sinan'ın Yaygın Olarak Kullandığı Küfeki Taşının Mühendislik Gizemi (In Turkish), *Mimar Sinan Dönemi Yapı Teknikleri Semineri*, Istanbul, Turkey, 29 May.
- Arioğlu, N. and Arioğlu, E.** (1997). Mimar Sinan'ın Seçtiği Taş : Küfeki ve Çekme Dayanımı (in Turkish), *14. Türkiye İnşaat Mühendisliği Teknik Kongresi*, Izmir, Turkey.
- Arioğlu, N. and Arioğlu, E.** (2005). Engineering mystery of master architect Sinan's "Kufeki" shell limestone, *Architectural Science Review*, **48**, 163-172.
- Arioğlu, N., Tuğrul, A., Zarif, H., Girgin, Z.C. and Arioğlu, E.** (1999a). Küfeki Taşının Dayanıklılık Analizi : Şehzade Camii Örneği-I (in Turkish), *Yapı Dergisi*, Yapı Endüstri Merkezi, September.
- Arioğlu, N., Tuğrul, A., Zarif, H., Girgin, Z.C. and Arioğlu, E.** (1999b). Küfeki Taşının Dayanıklılık Analizi : Şehzade Camii Örneği- II (in Turkish), *Yapı Dergisi*, Yapı Endüstri Merkezi, October.
- AS2870-1996** (1990). Residential Slab and Footings Construction, *Standards Australia*, Sydney, Australia.
- ASCE/SEI 41-06** (2007). ASCE/SEI 41-06 Seismic Rehabilitation of Existing Buildings, *American Society of Civil Engineers*, Reston, VA, USA.
- ASTM C 170-90** (1999). Standard Test Method for Compressive Strength of Dimension Stone, *American Society for Testing Materials*, USA.
- ASTM C 99-87** (2006). Standard Test Method for Modulus of Rupture of Dimension Stone, *American Society for Testing Materials*, USA.

- ASTM D 3148-02** (2002). Standard Test Method for Elastic Moduli of Intact Rock Core Specimens in Uniaxial Compression, *American Society for Testing Materials*, USA.
- ASTM E 384-09** (2009). Standard Test Method for Microindentation Hardness of Materials, *American Society for Testing Materials*, USA.
- ASTM E 564-06** (2006). Standard Practice For Static Load Test for Shear Resistance of Framed Walls for Buildings, *American Society of Testing Materials*, USA.
- ASTM G 1-03** (2003). Standard Practice for Preparing, Cleaning, and Evaluating Corrosion Test Specimens, *American Society for Testing Materials*, USA.
- ATC-40** (1996). Seismic Evaluation and Retrofit of Concrete Buildings, Volume 1-2, *Applied Technology Council*, Redwood City, California, USA.
- Augenti, N., and Parisi, F.** (2010). Constitutive models for tuff masonry under uniaxial compression, *ASCE Journal of Materials in Civil Engineering*, **22**, 11, pp. 1102-1111.
- Augenti, N., Parisi, F., Prota, A., and Manfredi, G.** (2011). In-plane lateral response of a full-scale masonry sub-assembly with and without an inorganic matrix-grid strengthening system, *J. Compos. Constr.*, **15**, 4, pp. 578–590.
- Bal, I.E., Şadan, O.B. and Smyrou, E.** (2007). Structural Analysis of Istanbul Beyazıt II Mosque Retrofitted by Mimar Sinan, *Proc. SHH07: International Symposium on Studies on Historical Heritage*, Antalya, Turkey, September (CD-ROM).
- Bal, I.E.** (2009). Private communication.
- Binda L., Pina Henriques J., Anzani A., Fontana A. and Lourenço P.B.** (2006). A contribution for the understanding of load-transfer mechanisms in multi-leaf masonry walls: Testing and modelling, *Engineering Structures*, **28**, 1132-1148.
- Bosiljkov, V.** (2006). Micro vs. macro reinforcement of brickwork masonry, *Materials and Structures*, **39**, 2, pp. 235-245.
- Bosiljkov, V., Page, A.W., Bokan-Bosiljkov, V. and Zarnic, R.** (2003). Performance based studies of in-plane loaded unreinforced masonry walls, *Masonry International*, **16**, 2, pp. 39–50.
- Bosiljkov, V., Page, A.W., Bokan-Bosiljkov, V. and Zarnic, R.** (2010). Evaluation of the seismic performance of brick masonry walls, *Struct. Control Health Monit.*, **17**, pp. 100–118.
- Buonopane, S. and Kelton, S.** (2008). Mechanical Properties of Wrought Iron from Penns Creek Bridge (1886), *Historic Bridges Conference, In Proc. Historic bridges : preservation, rehabilitation, and maintenance*, editor, Hojjat Adeli, Columbus, Ohio, pp. 167-182.
- Calderini, C., Cattari, S. and Lagomarsino, S.** (2010). The use of the diagonal compression test to identify the shear mechanical parameters of masonry, *Construction and Building Materials*, **24**, 5, pp.677-685.

- CEN EN 1998-3** (2005). EN 1998-3 Eurocode 8: Design of structures for earthquake resistance, Part 3: Assessment and retrofitting of buildings, *European Committee for Standardization*.
- CEN EN-1996-1-1** (2005). EN-1996-1-1 Eurocode 6: Design of masonry structures – Part 1 – Common rules for reinforced and unreinforced masonry structures, *European Committee for Standardization*.
- Chiostrini, S., Galano, L. and Vignoli, A.** (2003). In Situ Shear and Compression Tests in Ancient Stone Masonry Walls of Tuscany, Italy, *Journal of Testing and Evaluation*, ASTM, **31**, 4, pp. 289-304.
- Çimenoglu, H.**, 2009. Private communication.
- Corradi, M., Tedeschi, C., Binda, L. and Borri, A.** (2008). Experimental evaluation of shear and compression strength of masonry wall before and after reinforcement: Deep repointing, *Construction and Building Materials*, **22**, 4, pp.463-472.
- Costa, A.A., Arede, A., Costa, A., Guedes, J. and Silva, B.** (2012). Experimental testing, numerical modelling and seismic strengthening of traditional stone masonry: comprehensive study of a real Azorian pier, *Bulletin of Earthquake Engineering*, **10**, 1, pp.135-159.
- Croci, G.** (2000). *The Conservation and Structural Restoration of Architectural Heritage*, Computational Mechanics Publications, Southampton, UK and Boston, USA.
- Develi, K.** (2006). Süreksizlik içeren ortamların mekanik modellemesi için pürüzlülüğün kuramsal ve deneysel yöntemler ile sayısal tanımı (in Turkish), *PhD Thesis*, Istanbul Technical University.
- Dhanasekar, M. and Haider, W.** (2008). An explicit finite element model for masonry, *Comput. Struct.*, **86**, pp. 15-26.
- DIN 1053-1** (1996). 11-96: Mauerwerk. Teil 1: Berechnung und Ausführung, *Deutsches Institut für Normung e.V.: Beuth Verlag*, Berlin, Germany.
- DIN 1053-100** (2007). 2007-09: Mauerwerk. Teil 100: Berechnung auf der Grundlage des semiprobabilistischen Sicherheitskonzepts. *Deutsches Institut für Normung e.V.: Beuth Verlag*, Berlin, Germany.
- Dogariu, A., Dubina, D., Campitiello, F. and DeMatteis, G.** (2009). Experimental based calibration of a FE Model for Numerical Analysis of Masonry Shear Panels strengthened by Metal Sheathing, *PROHITECH09*, pp. 1133-1138, ISBN 978-0- 415-55803-7.
- Doğu, E.** (2010). Experimental investigation of the seismic behaviour of multileaf historical stone walls, *MSc Thesis*, Istanbul Technical University, Turkey.
- Drysdale, R.G., Hamid, A.A. and Baker, L.R.** (1994). *Masonry Structures Behaviour And Design*, Prentice Hall Inc., Englewood Cliffs, New Jersey, USA.
- Dunne, F.P.D. and Petrinic, N.** (2005). *Introduction to Computational Plasticity*, Oxford University Press, ISBN 0-19-856826-6.

- Egermann, R. and Newald-Burg, C.** (1994). Assessment of the load bearing capacity of historic multiple-leaf masonry walls, *In Proc 10th Int brick/block masonry conf.*, pp. 1603–1612.
- Elban, W.L., Borst, M.A., Roubachewsky, N.M., Kemp, E.L. and Tic P.C.** (1998). Metallurgical Assessment of Historic Wrought Iron: U.S. Custom House, Wheeling, West Virginia, *APT Bulletin*, Vol. **29**, No. 1.
- ElGawady, M.A., Lestuzzi, P. and Badoux, M.** (2007). Static cyclic response of masonry walls retrofitted with fiber-reinforced polymers, *J. Compos. Constr.*, 11, 1, pp. 50–61.
- Elmenshawi, A., Sorour, M., Mufti, A., Jaeger, L.G. and Shrive, N.** (2010). In-plane seismic behaviour of historic stone masonry, *Canadian Journal of Civil Engineering*, **37**, 3, pp.465-476.
- Elnashai, A.S. and Di Sarno, L.** (2008) *Fundamentals of Earthquake Engineering*, John Wiley & Sons Ltd, Chichester, UK.
- EN 10002-1** (2001). Metallic materials - Tensile testing - Part 1: Method of test at ambient temperature, *European Committee for Standardization*, Brussels.
- Erdoğan, M. and Mahmutoglu, Y.** (2006). Barshan Madencilik Sazlıbosna yapı taşı ocağından alınan örneklerin fiziksel ve mekanik özellikleri, *Technical Report*, ITU.
- Federal Emergency Management Agency** (1997). NEHRP guidelines for the seismic rehabilitation of buildings, Rept. FEMA 273 (Guidelines) and Rept. 274 (Commentary), *Technical Report*, Washington, D.C., USA.
- Federal Emergency Management Agency** (2000). “Prestandard and commentary for the seismic rehabilitation of buildings,” Rept. FEMA 356, *Technical Report*, Washington, D.C., USA.
- Flinn, R.A., and Trojan, P.K.** (1990). *Engineering Materials and Their Applications*, 4th ed, Boston, Houghton Mifflin Co., 376.
- Frumento, S., Magenes, G., Morandi, P. and Calvi, G.M.** (2009). Interpretation of experimental shear tests on clay brick masonry walls and evaluation of q-factors for seismic design, *Eucentre Research Report 2009/02*, IUSS Press, Pavia.
- Galasco, A., Magenes, G., Penna, P. and Da Pare, M.** (2010). In-plane cyclic shear tests of undressed double leaf stone masonry panels, *Proc. 14th European Conference on Earthquake Engineering*, Ohrid, Macedonia, 20 August-03 September (CD-ROM).
- Gedik, Y.H. and Celep, Z.** (2008). Earthquake analysis and strengthening of the historical Mehmet Aga Mosque in Istanbul, *Proc. 14th World Conference on Earthquake Engineering*, Beijing, China, October 12-17 (CD-ROM).
- Ghobarah, A.** (2004). On drift limits associated with different damage levels, BLED International Workshop, PEER 2004/05 Report, *Technical Report*, pp. 321–333.

- Gordon, R. and Knopf, R.** (2005). Evaluation of Wrought Iron for Continued Service in Historic Bridges, *Journal of Materials in Civil Engineering*, Graubner, C.-A.; Kranzler, T.: In-Plane Bearing Capacity of Unreinforced Masonry Panels. In: Proceedings of the 7th International Masonry Conference. London, 2006, paper 70.
- Graubner, C.A. and Kranzler, T.** (2005). Shear Design of Unreinforced Masonry Panels – Basic Assumptions and Comparison of Different Standards, *Proceedings of the 10th Canadian Masonry Symposium*, pp. 393-399, Canada.
- Graubner, C.A. and Kranzler, T.** (2006). In-Plane Bearing Capacity of Unreinforced Masonry Panels, *Proceedings of the 7th International Masonry Conference*, London, UK.
- Green, A.W.E.** (1948). Wrought Iron, in Metals Handbook, edited by T. Lyman, Metals Park, Ohio, *American Society for Metals*, pp. 503-505.
- Güler, K., Dabanlı, Ö. and Aköz, A.H.** (2008). Structural and Earthquake Response Analysis of Hirka-i Sharif Mosque, *Proc. 8th International Seminar on Structural Masonry*, Istanbul, pp. 331-339.
- Hillerborg, A., Modeer, M. and Petersson, P.E.** (1976). Analysis of Crack Formation and Crack Growth in Concrete by Means of Fracture Mechanics and Finite Elements, *Cement and Concrete Research*, **6**, pp. 773–782.
- Ilki, A.** (2011). Private communication.
- Ilki, A. and Demir, C.** (2010). Küçük Mustafa Paşa Hamamı, erkekler soyunmalık bölümü için gerçekleştirilen deprem güvenliği analizi ve güçlendirme önerileri konusunda teknik rapor, *Technical Report*, ITU.
- Ilki, A., Ispir, M., Demir, C., Kumbasar, N. and Akman, S.** (2006). An Outline of the Seismic Behavior of Historical Structures in North Western Anatolia, *International Conference on Structural Analysis of Historical Constructions (SAHC2006)*, New Delhi, India, November 6-8 (CD-ROM).
- ISO 3274** (1998). Surface texture: Profile method - Nominal characteristics of contact (stylus) instruments, *International Organization for Standardization*.
- ISO 11562** (1996). Geometrical Product Specifications (GPS), Surface texture: profile method—metrological characteristics of phase correct filters, *International Organization for Standardization*.
- Jager, W. and Schöps, P.** (2008). Proposals for an advanced design model of masonry under lateral loads for the implementation in Eurocode 6-1-1 Technical Report D9.1 of the collective research project ESECMase, *Technical Report*, TU Dresden.
- Jiménez, J.M., Bravo, E., Criado, A.J., Arévalo, R., Dietz, C., Martinez, J.A. and Criado, A.J.** (2004). A new method for dating ancient steel samples using Vickers micro-hardness, *Materials Characterization*, **52**, 2004, pp. 145-151.

- Kaminetzky D.** (1985). Verification of structural adequacy, Rehabilitation, Renovation, and Preservation of Concrete and Masonry Structures, *ACI Special Publication*.
- Karapitta, L., Mouzakis, H. and Carydis, P.** (2011). Explicit finite-element analysis for the in-plane cyclic behavior of unreinforced masonry structures, *Earthquake Engng. Struct. Dyn.*, **40**, pp.175-193.
- Kolay, I.** (1999). *Batı Anadolu XIV. yy. Beylikler Mimarisinde Yapım Teknikleri* (in Turkish), Atatürk Kültür Merkezi Yayınları, Ankara.
- Kolay, I.** (2009). Private communication.
- Lee, J. and Fenves, G.L.** (1998). Plastic-Damage Model for Cyclic Loading of Concrete Structures, *Journal of Engineering Mechanics*, **124**, 8, pp. 892-900.
- Lourenço, P.B.** (1998). Experimental and numerical issues in the modelling of the mechanical behavior of masonry, Roca et al. (eds), *Structural Analysis of Historical Constructions II. CIMNE*, Barcelona, pp. 57-91.
- Lourenço, P.B.** (2002). Computations on historic masonry structures, *Prog. Struct. Engng. Mater.*, **4**, pp.301-319.
- Lourenço, P.B.** (2004). Analysis and restoration of ancient masonry structures: Guidelines and examples, *Innovative Materials and Technologies for Construction and Restoration*, Lecce, Italy, pp. 23-41.
- Lourenço, P.B.** (2005). From fracture mechanics to case studies: The issue of cultural heritage, *Anales de Mecánica de la Fractura*, pp. 10-17.
- Lourenço, P.B., Oliveira, D.V., Roca, P. and Orduña, A.** (2005). Dry stack joint stone masonry walls subjected to in-plane combined loading, *J. Struct. Eng.*, **131**, 11, pp. 1665–1673.
- Lubliner, J., Oliver, J., Oller, S. and Oñate, E.** (1989). A Plastic-Damage Model for Concrete, *International Journal of Solids and Structures*, **25**, pp. 299-329.
- Magenes, G.** (2011). Private communication.
- Magenes, G. and Calvi, G.M.** (1997). In-plane seismic response of brick masonry walls, *Earthq. Engin. and Struct. Dyn.*, Vol. **26**, pp. 1091-1112.
- Mann, W. and Müller, H.** (1980). Failure of shear-stressed masonry – An enlarged theory, tests and application to shear-walls, *Proc. of the International Symposium on Loadbearing Brickwork*, London, UK.
- Mazlum, D.** (2001). The earthquake of 22 May 1766 according to the Ottoman archives and repairing after the earthquake (in Turkish), *PhD Thesis*, Istanbul Technical University, Istanbul, Turkey.
- Mazzon, N.** (2010) Influence of Grout Injection on the Dynamic Behaviour of Stone Masonry Buildings, *PhD Thesis*, Università degli Studi di Padova, Padova, Italy.
- Minaie, E.** (2009). Behavior and vulnerability of reinforced masonry shear walls, *PhD Thesis*, Dept. of Civil, Architectural and Environmental Engineering, Drexel Univ., Philadelphia, USA.

- Mistler, M., Butenweg, C. and Meskouris, K.** (2006). Modelling methods of historic masonry buildings under seismic excitation, *Journal of Seismology*, **10**, pp.497-510.
- Muralikrishnan, B. and Raja, J.** (2009). *Computational Surface and Roundness Metrology*, Springer.
- NTC08 Norme Tecniche per le Costruzioni** (2008). D.M. 14 gennaio 2008, (G.U. n. 29 del 4.02.2008 suppl. ord. N. 30).
- NZSEE** (2012). Recommendations for Assessment and Improvement of the Structural Performance of Buildings in Earthquakes, *New Zealand Society for Earthquake Engineering*, New Zealand.
- Oliveira, D.V.** (2003). Experimental and numerical analysis of blocky masonry structures under cyclic loading, *PhD Thesis*, University of Minho, Portugal.
- Oliveira, D.V., Lourenço, P.B. and Roca, P.** (2006) Cyclic behaviour of stone and brick masonry under uniaxial compressive loading, *Materials and Structures*, **39**, 2, pp. 219–227
- Onaran, K.** (1993). *Malzeme Bilimi* (in Turkish), Bilim Teknik Yayınevi, Istanbul.
- Priestley, M.J.N., Seible, F. and Calvi, G.M.** (1996). *Seismic Design and Retrofit of Bridges*, John Wiley and Sons, New York.
- Rainer P.** (1983). *Movement Control in the Fabric of Buildings*, New York: Nichols
- Ray, A., Dhua, S.K., Prasad, R.R., Jha, S. and Banerjee, S.** (1997). The ancient 11th century iron pillar at Dhar, India: a microstructural insight into material characteristics, *Journal of Materials Science Letters*, **16**, 1997, pp. 371-375.
- Roberti, G.M., Donati, L. and Fontana, A.** (2004). Out-of-plane behaviour of multiple-leaf stone masonry, *Proc. IV Int. Seminar on Structural Analysis of Historical Constructions (SAHC)*, Padua, Italy, 10-12 November, Vol. **1**, pp. 597-602.
- Romano, A.** (2008). Tuff-Brick Listed Masonry: Experimental Behaviour, *Proc. The 14th World Conference on Earthquake Engineering*, Beijing, China, October 12-17.
- Sabnis, G.M., Harris, H.G., White, R.N. and Mirza, M.S.** (1985). *Structural Modeling and Experimental Techniques*, Prentice Hall.
- Senthivel, R. and Lourenço, P.B.** (2009). Finite element modelling of deformation characteristics of historical stone masonry shear walls, *Engineering Structures*, **31**, 9, pp.1930-1943.
- Shibata, A., and M. Sozen.** (1976). Substitute structure method for seismic design in R/C, *J. Struct. Div. ASCE*, **102**, no. ST1, pp. 1–18.
- Shing, P.B., Noland, J.L. and Kiamerus, E.** (1989). Inelastic behavior of concrete masonry shear walls, *Journal of Structural Engineering*, **115**, 9, pp. 2204-2225.

- Silva, R. A., Oliveira, D. V. and Lourenço, P. B.** (2008). On the strengthening of three-leaf stone masonry walls. In *Proceedings of the structural analysis of historic construction*, United Kingdom, Bath.
- Tanyeli, G.** (1990). Osmanlı Mimarlığında Demirin Strüktürel Kullanımı (15.-18. yüzyıl) (in Turkish), *PhD Thesis*, Istanbul Technical University, Istanbul, Turkey.
- Tanyeli, G., Geçkinli, A.E. and Ata, A.** (1990). Osmanlı Mimarisinde Kullanılan Demir Öğelerin Üretim Teknolojisi (in Turkish), *VI. Arkeometri Sonuçları Toplantısı*, Ankara.
- Taş, M.** (2003). Osmanlı'dan Günümüze Yapı Üretiminde Mimarlık Meslek Örgütlenmesinin Gelişimi, *Uludağ Üniversitesi Mühendislik-Mimarlık Fakültesi Dergisi*, **8**, 1.
- Tayla, H.** (2007). *Geleneksel Türk Mimarisinde Yapı Sistem ve Elemanları* (in Turkish), Türkiye Anıt Çevre Turizm Değerlerini Koruma Vakfı, Istanbul.
- Thomas, T.R.** (1999). *Rough Surfaces*, Imperial College Press, London.
- Tomazevic, M.** (1999). *Earthquake Resistant Design of Masonry Buildings*, Vol. **1**, Imperial College Press, London.
- Tomazevic, M. and Lutman, M.** (2007). Heritage Masonry Buildings in Urban Settlements and the Requirements of Eurocodes: Experience of Slovenia, *International Journal of Architectural Heritage*, **1**, pp. 108-130.
- Tomazevic, M., Lutman, M. and Petkovic, L.** (1996). Seismic Behavior of Masonry Walls: Experimental Simulation, *Journal of Structural Engineering*, **122**, 9, pp.1040-1047.
- TS EN 1015-11** (2000). Kagit Harcı - Deney metotları - Bölüm 11: Sertleşmiş harcın basınç ve eğilme dayanımının tayini, *Türk Standardları Enstitüsü*, Ankara.
- TS EN 1052-1** (2000). Kagit - Deney metotları - Bölüm 1: Basınç dayanımı tayini, *Türk Standardları Enstitüsü*, Ankara.
- TS EN 1052-3** (2004). Kâgır - Deney Metotları - Bölüm 3: Başlangıç Kayma Dayanımının Tayini, *Türk Standardları Enstitüsü*, Ankara.
- Tuğrul, A. and Zarif, İ.H.** (1999). Research on limestone decay in a polluting environment, Istanbul-Turkey, *Environmental Geology*, **38**, 2.
- Turkish Seismic Design Code** (2007). Deprem bölgelerinde yapılacak binalar hakkında yönetmelik, *Türk Standardları Enstitüsü*, Ankara.
- Turnsek, V. and Cacovic, F.** (1971). Some experimental results on the strength of brick masonry walls, *Proc., 2nd Int. Brick Masonry Conf.*, British Ceramic Research Association, Stoke-on-Trent, pp. 149-156, April.
- Turnsek, V. and Sheppard, P.F.** (1980). The shear and flexural resistance of masonry walls, *Proc. of the research conf. on earthquake engineering*, Skopje.

- UNDP/UNIDO Project RER/79/01.** (1984). *Building Construction Under Seismic Conditions in the Balkan Region Vol.6, Repair and Strengthening of Historical Monuments and Buildings in Urban Nuclei*, Austria.
- Urekli, F.** (1999). *The 1894 earthquake in Istanbul* (in Turkish), Iletisim, Istanbul.
- Valluzzi, M.R., da Porto, F. and Modena, C.** (2004). Behaviour and modelling of strengthened three-leaf stone masonry walls, *Materials and Structures*, **37**, pp. 184-192.
- Van Der Pluijm, R.** (1992). Material properties of masonry and its components under tension and shear, *Proc. 6th Canadian Masonry Symposium*, eds. V.V. Neis, Saskatoon, Saskatchewan, Canada, pp. 675-686.
- Vasconcelos, G.F.M.** (2005). Experimental investigations on the mechanics of stone masonry: Characterization of granites and behavior of ancient masonry shear walls, *PhD Thesis*, University of Minho, Portugal.
- Vasconcelos, G.F.M. and Lourenço, P.B.** (2009). In-Plane Experimental Behavior of Stone Masonry Walls under Cyclic Loading. *Journal of Structural Engineering*, **135**, 10, pp.1269-1277.
- Vintzileou, E.** (2008). Effect of Timber Ties on the Behavior of Historic Masonry. *Journal of Structural Engineering*, **134**, 6, pp. 961-972.
- Vintzileou, E. and Miltiadou-Fezans, A.** (2008). Mechanical properties of three-leaf stone masonry grouted with ternary or hydraulic lime-based grouts, *Engineering Structures*, **30**, pp. 2265-2276.
- Vintzileou, E. and Tassios, T.P.** (1995). Three-leaf stone masonry strengthened by injecting cement grouts, *Journal of Structural Engineering*, **121**, 5, pp. 848–856.
- Voon, K.C., and Ingham, J.M.** (2006). Experimental in-plane shear strength investigation of reinforced concrete masonry walls, *J. Struct. Eng.*, **132**, 3, pp. 400–408.
- Yorulmaz, M. and Ahunbay, Z.** (1986). Sinan Camilerinde Taşıyıcı Sistem ve Yapım Teknikleri (In Turkish), *The 2nd International Congress on the History of Turkish and Islamic Science and Technology*, Istanbul, Turkey, 28 April-2 May.
- Url-1** <<http://www.gordonengland.co.uk/hardness/vickers.htm>>, date retrieved 24.08.2009.
- Url-2** <http://www.gordonengland.co.uk/hardness/hardness_conversion_1c.htm>, date retrieved 24.08.2009.
- Url-3** < <http://www.efunda.com/units/hardness> >, date retrieved 25.08.2009.
- Bright, S.** (2007). Strength of Victorian Wrought Iron Rail Bridges, message posted to Civil Engineering History & Heritage Exchange, <<http://knowledgelists.ice.org.uk/archives/civilengineering-heritage-1.html>> Sept.18, 2007, date retrieved 25.08.2009.
- NIST National Institute of Standards and Technology**, Reference software for 2D surface analysis, <<http://syseng.nist.gov/VSC/jsp/FileOpen.jsp>>, date retrieved 05.09.2009.

APPENDICES

APPENDIX A: Determination of the Vertical Stress Levels of Tested Walls

APPENDIX B: Characterization of Friction Surfaces

APPENDIX C: Cyclic Shear Tests of Multi-Leaf Masonry Walls

APPENDIX D: Principal Stress and Principal Plastic Strain Distributions Obtained
by Micro-Modelling

APPENDIX A: Determination of the Vertical Stress Levels of Tested Walls

Friction forces in between the masonry units and self-centring effect of the structure's self-weight against the shearing and rocking effects, which are again consequences of structure's weight, plays a major role during lateral actions such as earthquake loads. Consequently, level of vertical stress acting on the wall has paramount importance on the behaviour of masonry walls.

During experimental studies, application of appropriate levels of vertical loads, which are representative of actual stress levels in actual structures, is evident. In this section, a number of studies from different researchers that investigated a number of historical structures from the Ottoman period are compiled. This data is then used for deciding on the appropriate vertical load levels that can be utilized during the experimental study.

Ilki and Demir (2010) investigated the seismic safety of the Küçük Mustafa Paşa Bath (15th century) in Istanbul. Several on site investigations and experiments were carried out before the numerical analysis phase. The walls of the bath complex were multi-leaf walls with roughly hewn stone units. Layout of a typical wall cross-section, obtained through core sampling and videoscope investigations can be seen in Figure A.1. The finite element analysis included several cases for gravity and earthquake loads as well as soil settlement cases. Vertical compressive stresses obtained for the self-weight analysis of the structure were found to be uniformly varying between 0.2 and 1.0 MPa and, as expected, they tended to increase from higher to lower elevations of the structure (Figure A.2). The seismic loads were simulated by applying different levels of lateral accelerations such as 0.05, 0.1, 0.2, 0.3, 0.4 and 0.5 times the acceleration of gravity ($g=9.81 \text{ m/s}^2$). A typical view of the vertical stress distribution resulting from a lateral loading of 0.4g is presented in Figure A.3. Accordingly, the vertical compressive stresses at the compressed parts of the lower elevations of the structure could increase to values near 2.0 MPa. However, they were much smaller at the tension zones of the main walls. During site investigations, wide vertical cracks were observed on the main walls of the structure. Similar damage zones were also present in the laterally loaded finite element models. High tension zones, starting from the dome, were mostly concentrated along the window openings of the main walls as a result of high shear stresses for the walls

parallel to the applied earthquake load direction and as a result out-of-plane actions for walls perpendicular to the loading direction (Figure A.4).

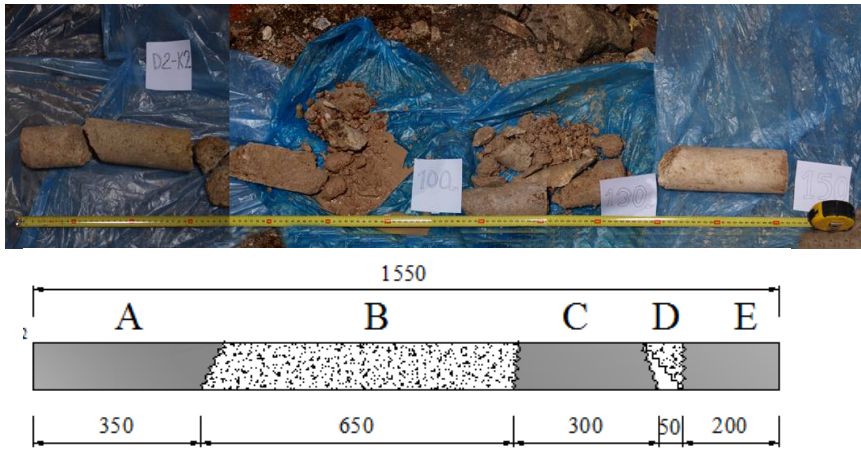


Figure A.1 : Typical layout of the multi-leaf wall.

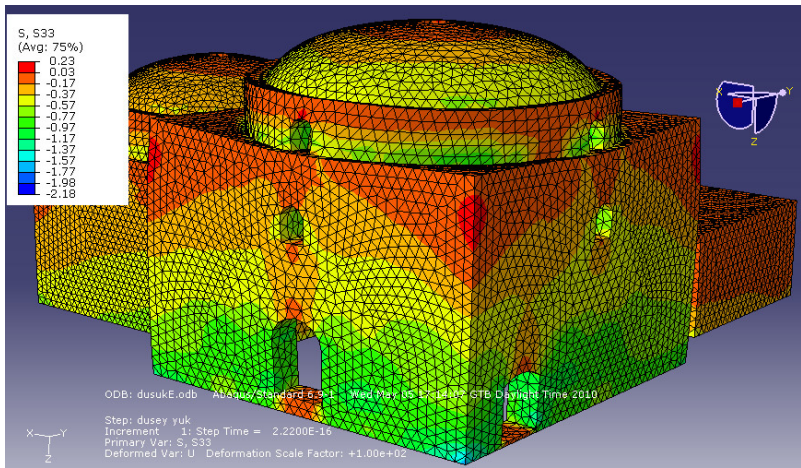


Figure A.2 : Vertical stresses obtained for the self-weight of the structure.

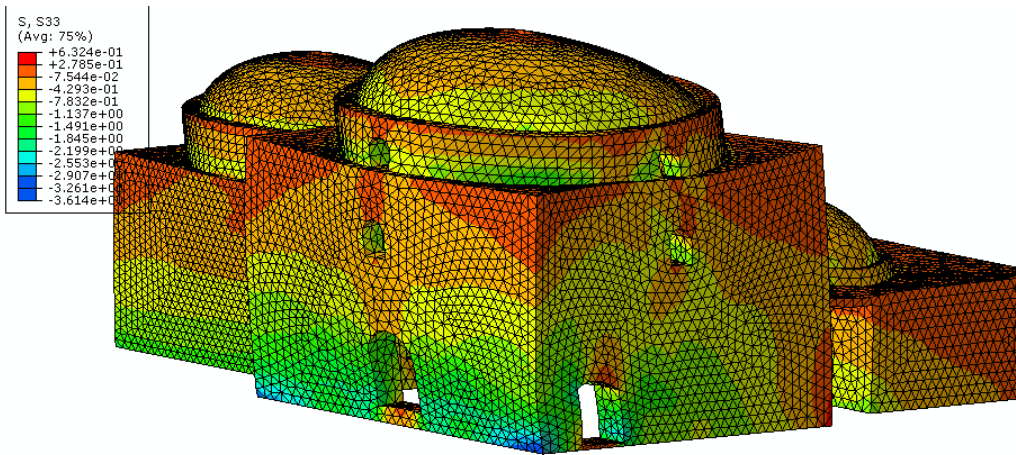


Figure A.3 : Vertical stresses obtained for 0.4g lateral loading.

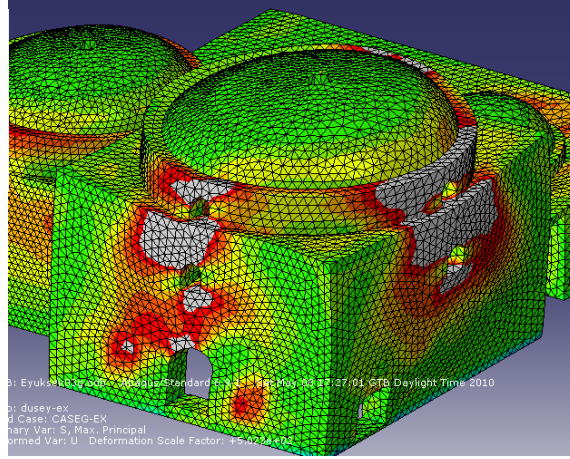


Figure A.4 : Tensile stress distribution obtained for lateral loading.

Gedik and Celep (2008) analyzed the structural behavior of Mehmet Ağa Mosque (late 16th century) under gravity and earthquake loads. As reported by the authors, vertical compressive stresses increases downwards and reaches about 0.20 MPa at the lowest wall level. However due to stress concentrations around the openings, increases up to 0.28 MPa are also observed. Gedik and Celep (2008) also mentioned about the presence of tensile normal stresses occurring at the top levels of the walls due to out of plane loads originating from the dome and tromp. Results of analyses for design (an earthquake having exceeding probability of 10% in 50 years with a return period of 475 years) and maximum (an earthquake having exceeding probability of 20% in 50 years with a return period of 2475 years) spectrums defined in Turkish Seismic Code (2007), shows that vertical normal stresses may increase up to 50% (between 0.3 and 0.6 MPa). Distribution of vertical stresses for self-weight and earthquake loading cases can be seen in Figures A.5 and A.6.

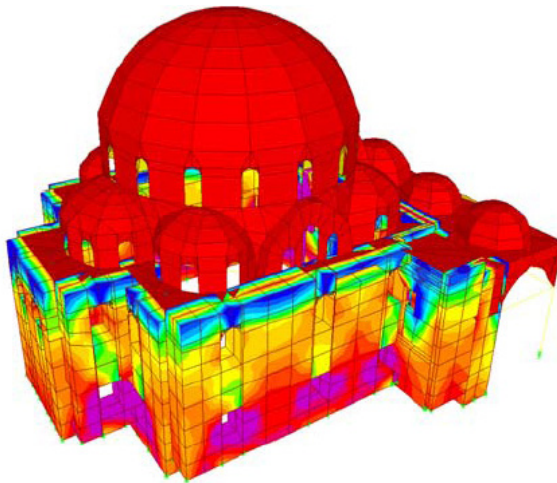


Figure A.5 : Vertical stresses for Mehmet Ağa Mosque under self-weight.

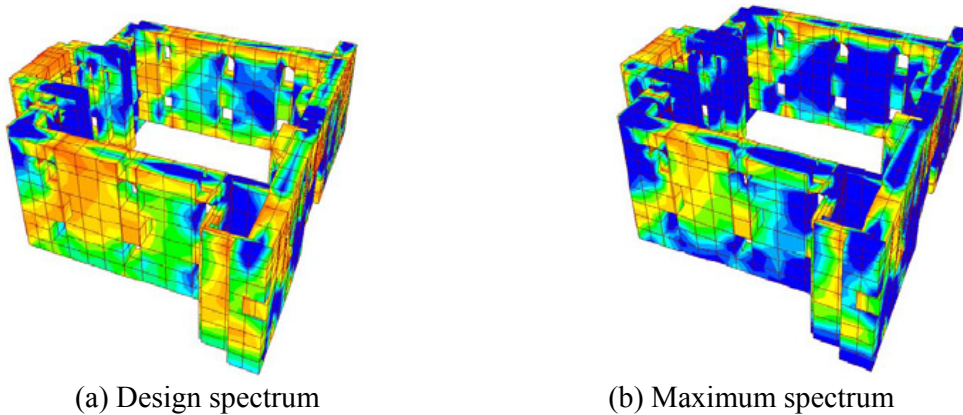


Figure A.6 : Vertical stresses for Mehmet Ağa Mosque under earthquake loads.

Güler et al. (2008) investigated the structural and earthquake response of Hirka-i Sharif Mosque (1851) located in Fatih district, Istanbul. In this structure, wall thicknesses such as 0.80, 1.15 and 1.40 m have been measured. With the utilization of finite element analysis for gravitational and earthquake loads, vertical and lateral stresses have been obtained. Under vertical loads, a maximum compressive stress of 0.80 MPa at the bottom portions of the main walls has been reported. In addition, they also noticed axial stress magnitudes larger than 1.0 MPa, which appeared due to various local geometrical discontinuities possibly caused by windows, doors and piers.

Bal et al. (2007) examined the seismic performance of Beyazıt II Mosque (early 16th century) before and after the retrofitting works done by Architect Sinan. They used linear and nonlinear analysis approaches by employing the finite element method. Distribution of vertical stresses on the main walls of the mosque, self-weight and earthquake loads, are presented in Figures A.7 and A.8, respectively. From these figures, it is clear that the axial stress distribution exhibits large variations due to existence of openings and structural components (such as piers) on the main walls of the structure. Under sustained loads, stress ranges between 0.0 and 0.5 MPa for upper elevations and 0.5 to 1.0 MPa for lower elevations can be identified. In the case of earthquake loads vertical stresses changing between 0.0 to 3.0 MPa (mainly between 0.0 and 1.0 MPa) can be observed.

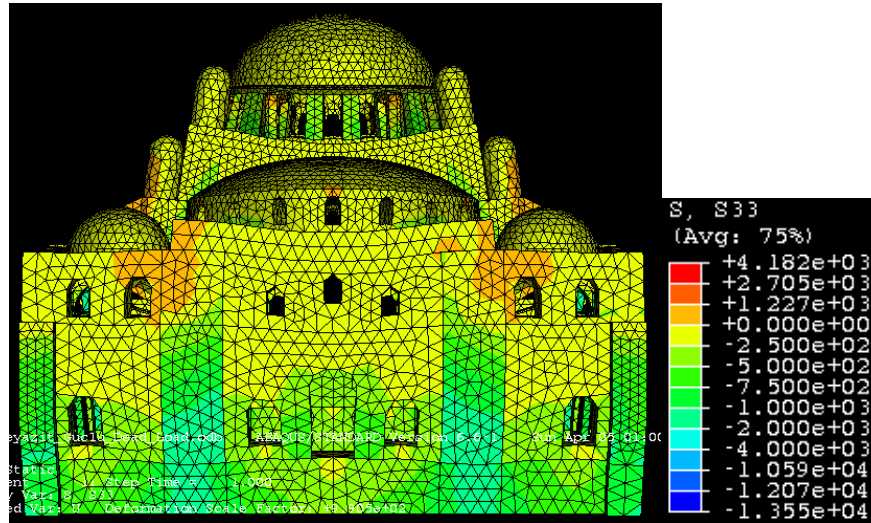


Figure A.7 : Vertical stresses for Beyazıt II Mosque, self-weight (stresses in kPa).

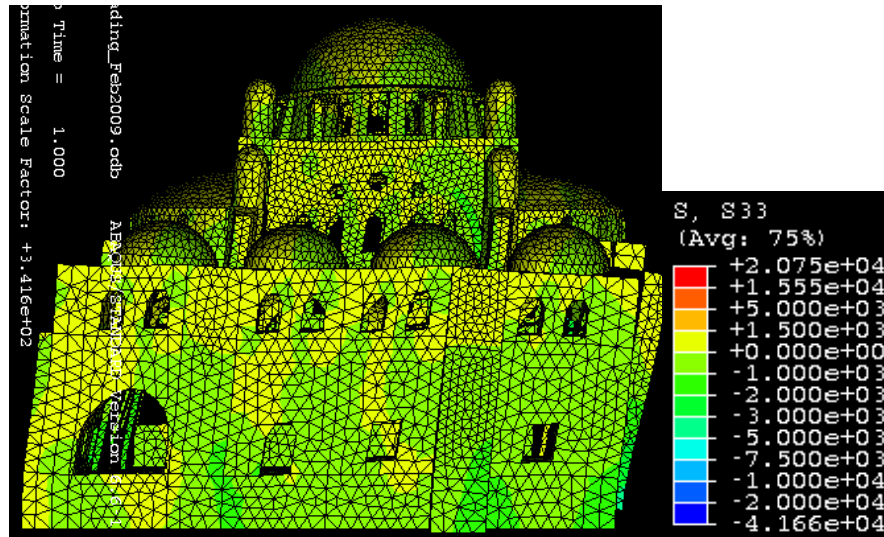


Figure A.8 : Vertical stresses for Beyazıt II Mosque, lateral loads (unit kPa).

Another study by Bal (2009) aimed to investigate Atik Ali Paşa Mosque (late 15th century) in Çemberlitaş Istanbul. Variation of vertical stresses under sustained loads on the exterior main walls of this structure is shown in Figure A.9. From the given view points in the longitudinal and transverse directions one can figure out that the magnitude of axial stresses are in the order of 0.3 MPa for the upper elevations and varying between 0.5 and 1.0 MPa for lower elevations.

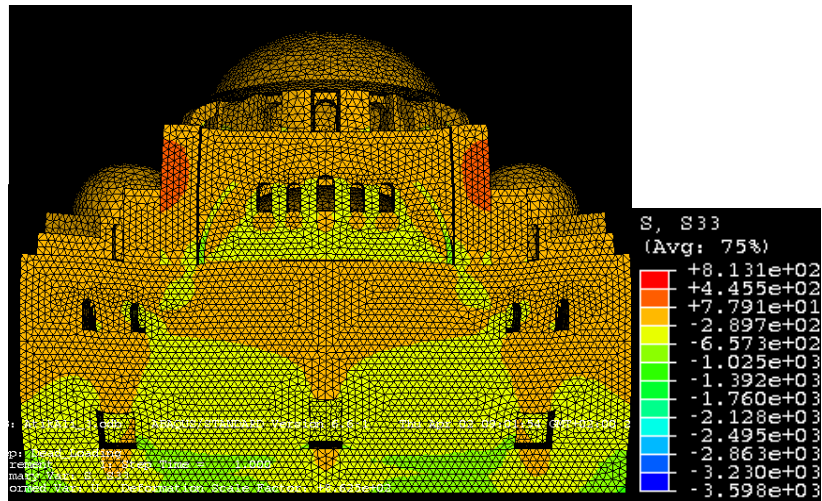


Figure A.9 : Vertical stresses for Atik Ali Paşa Mosque, vertical loads (unit kPA).

APPENDIX B: Characterization of Friction Surfaces

Surface textures of contacting materials play a major role in determination of the interface mechanical characteristics. Texture of a surface consists of three main components: Form, waviness and roughness. Quantification of these components can be done either by using sectional or areal measurement devices (Thomas, 1999). Though several devices based on optical, mechanical, acoustic or laser measurement methods for 3D areal measurement are available, due to technical and financial reasons, a home-made stylus type sectional device was used in this study. Surface roughness measurements of rocks were also carried out by Vasconcelos (2005) and Develi (2006). Vasconcelos (2005) used an optical 3D topographical system for characterization of fractured surfaces of granites after direct tension tests, while Develi (2006) developed a computer controlled surface scanner and used fractal geometry for definition of roughness parameters and performed shear tests.

The developed profilometer was based on two very sensitive Mitutoyo LVDTs with a gauge length of 50 mm. General views of the developed apparatus are shown in Figure B.1. Both LVDTs were fixed on a heavy steel plate which rests on a table. A horizontal LVDT was used for monitoring of the transverse displacement of the stone unit, while a vertical one was used for measurement of the irregularities on the top face of the stone. A needle with a rounded tip was attached to the vertical LVDT, in order to have pin point accuracy for sharp valleys and peaks of the texture. During the measurements, after the needle of the vertical LVDT was lifted, the stone unit was moved very slowly in the longitudinal direction. This very small displacement (kept less than 1 mm) was observed from the digital scope of the horizontal LVDT and registered. Afterwards, the tip of the vertical LVDT was placed upon the surface very slowly, and the position of the tip with respect to initial coordinates was registered. The measurements were done for a total length of 50 mm, on a previously drawn line. A total of 9 profiles (3 smooth and 6 rough textured stone units) were extracted from initial shear test specimen units before the implementations of the tests. Typical profiles obtained for SET I (smooth texture) and SET II (rough texture) specimens are presented in Figure B.2.



Figure B.1 : Stylus type profilometer developed for the current study.

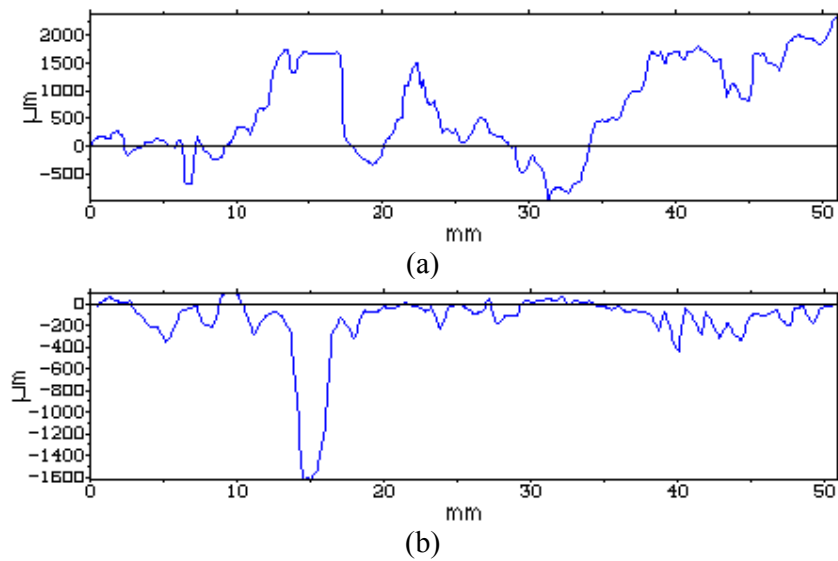


Figure B.2 : Typical profiles obtained for (a) smooth and (b) rough surfaces.

As also indicated by Vasconcelos (2005), the surface profile texture is a composition of structures with different ranges of irregularities and irregularity distributions, or a range of frequency and wavelength components. The longest wavelengths are associated with errors of form (such as curvature); shorter wavelengths constitute waviness; and the shortest wavelengths are called roughness (Thomas, 1999). Of course, the definitions of these components vary depending on the subject. For instance, a range for waviness of an engine cylinder surface might be classified as roughness for a concrete surface.

In order to set apart form, waviness and roughness from each other, the texture profile needs to be processed for appropriate frequency and wavelength components. First step of this process aims to identify the form of the surface that may include

residual slope or roundness. In our case, though, the stone units were cut by a machine and special attention during the texture measurements was paid for alignment of stone units, residual slopes could also be observed quantitatively. These residual slopes left from cutting process or misalignment of the units were set apart by using a best fit least-squares straight line. During this step and in the following steps, an internet-based Surface Metrology Algorithm Testing System available on the National Institute of Standards and Technology web site (NIST, 2009) was utilized for computations.

The waviness of a profile can be extracted via application of filtering methods. Numerous filters for discrete data sets (that define the texture profile), such as electrical filters (like 2-resistor-capacitor 2RC) or digital filters (like 2RC, moving average, spline, multi-scale or Gaussian), are available in the literature (Muralikrishnan and Raja, 2009). Filtering involves partitioning a profile or topography into different wavelength bandwidths. In order to separate out the roughness and the waviness, a cut-off wavelength (λ_{co}) is set. Filters operate depending on the cut-off set for them. All wavelengths below λ_{co} in the profile fall into the roughness regime and all others go into the waviness and form regime (NIST, 2009).

A filter may be thought of as a sequence of weighting terms is moved along the profile, multiplying it term by term and thus smoothing it as it goes (Figure B.3). Gaussian filter is defined as a moving average of the surface profile with a Gaussian weighting function. The weighting function $s(x)$ is given by ISO 11562 (1996) as:

$$s(x) = \frac{1}{\alpha \lambda_{co}} e^{-\pi \left(\frac{x}{\alpha \lambda_{co}}\right)^2} \quad (\text{B.1})$$

where x is the position in relation to the centre of the weighting function, λ_{co} is the cut-off wavelength and $\alpha = \sqrt{\ln(2 / \pi)} = 0.4697$.

The transmission characteristic, which is the ratio of the amplitude of the filtered signal at that wavelength to that of the unfiltered signal (Thomas, 1999), at λ_{co} is 50% for a Gaussian filter. So the Gaussian filter enables one to use a single filter to obtain both the roughness and waviness (NIST, 2009).

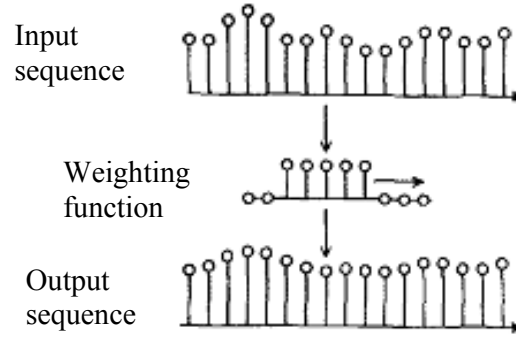


Figure B.3 : Procedure of digital filtering (Thomas, 1999).

It is apparent that the correct definition of cut-off wave length plays a very important role. International standards define the cutoff length in order to evaluate the filter length for waviness and roughness. According to the values obtained for the total surface texture profile (unfiltered roughness data), there are five standard cut-off lengths as shown in Table B.1 (Vasconcelos, 2005 and ISO 3274, 1998).

Table B.1 : Standard cut-off wavelengths (ISO 3274, 1998).

Roughness range (μm)	Cutoff, λ_{co} (mm)
<0.002	0.08
0.002-0.1	0.25
0.1-2	0.80
2-10	2.50
>10	8.00

Since, in this study, the amplitudes of obtained profiles were quite higher than $10\ \mu\text{m}$ as seen in Figure B.2, a λ_{co} value of 8 mm was used during the filtering. The filtering procedure was carried out by utilizing the internet-based Surface Metrology Algorithm Testing System available on the National Institute of Standards and Technology (NIST, 2009) web site. But since these calculations require a set of data at equal intervals, the random displacement and amplitude test data at hand was converted to the required format. For this purpose, interpolations were done between the data points so that a profile data set with $100\ \mu\text{m}$ spacing was obtained. Then, the prepared raw profile data was loaded to internet-based software and processed (Figure B.4). As also explained above, the residual slope (Figure B.4b) was removed by fitting a best fit least-squares straight line (Figure B.4c). Afterwards, the waviness (Figure B.4d) and roughness (Figure B.4e) components were set apart by applying

the Gaussian filter with a cut-off length of 8 mm. A sample profile of a smooth surface texture before and after the application of the same procedure is presented in Figure B.5. In Figures B.4 and B.5, we can observe that not only roughness, but also waviness, plays an important role in defining the texture and consequently, in defining parameters related with the surface texture. Especially, in the case of rough surface textures, effect of waviness is more pronounced.

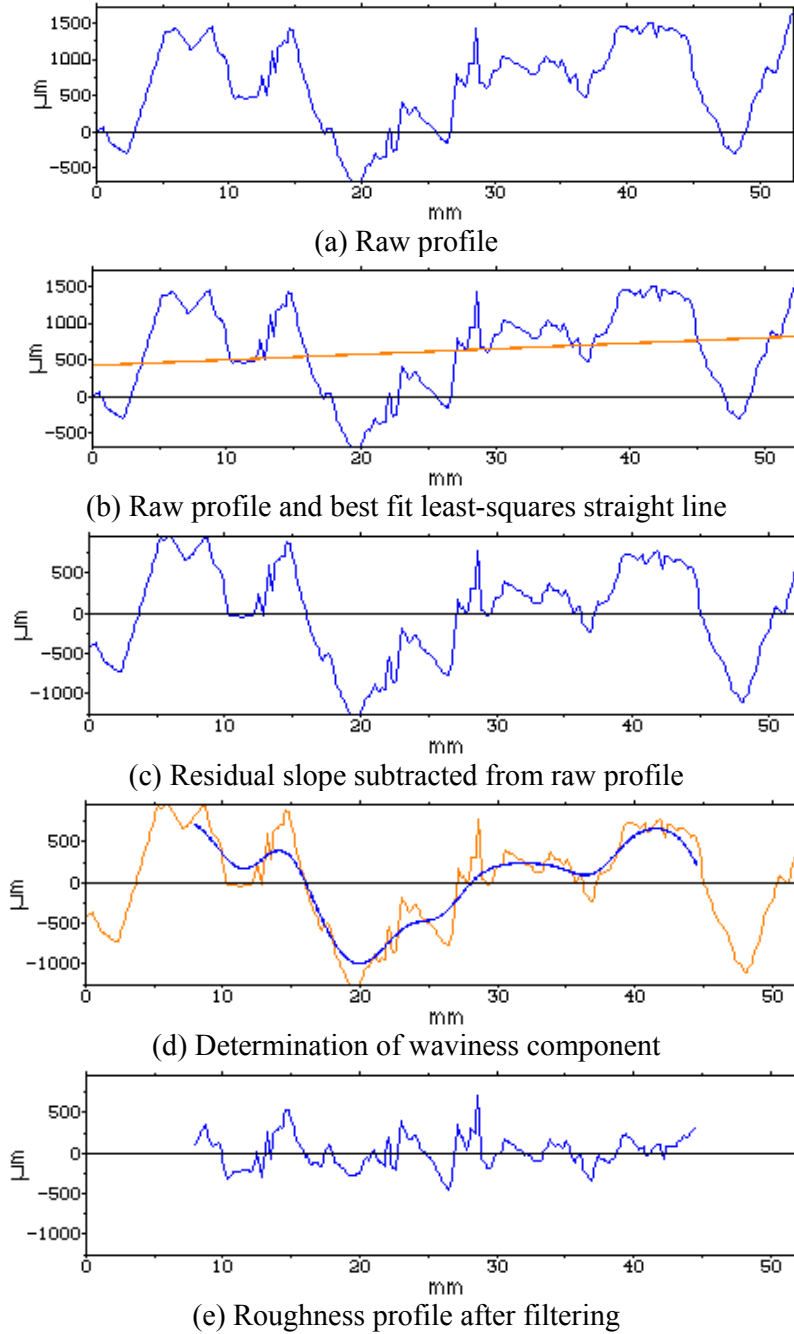


Figure B.4 : Obtaining texture components for a rough texture (5-S3-C stone unit).

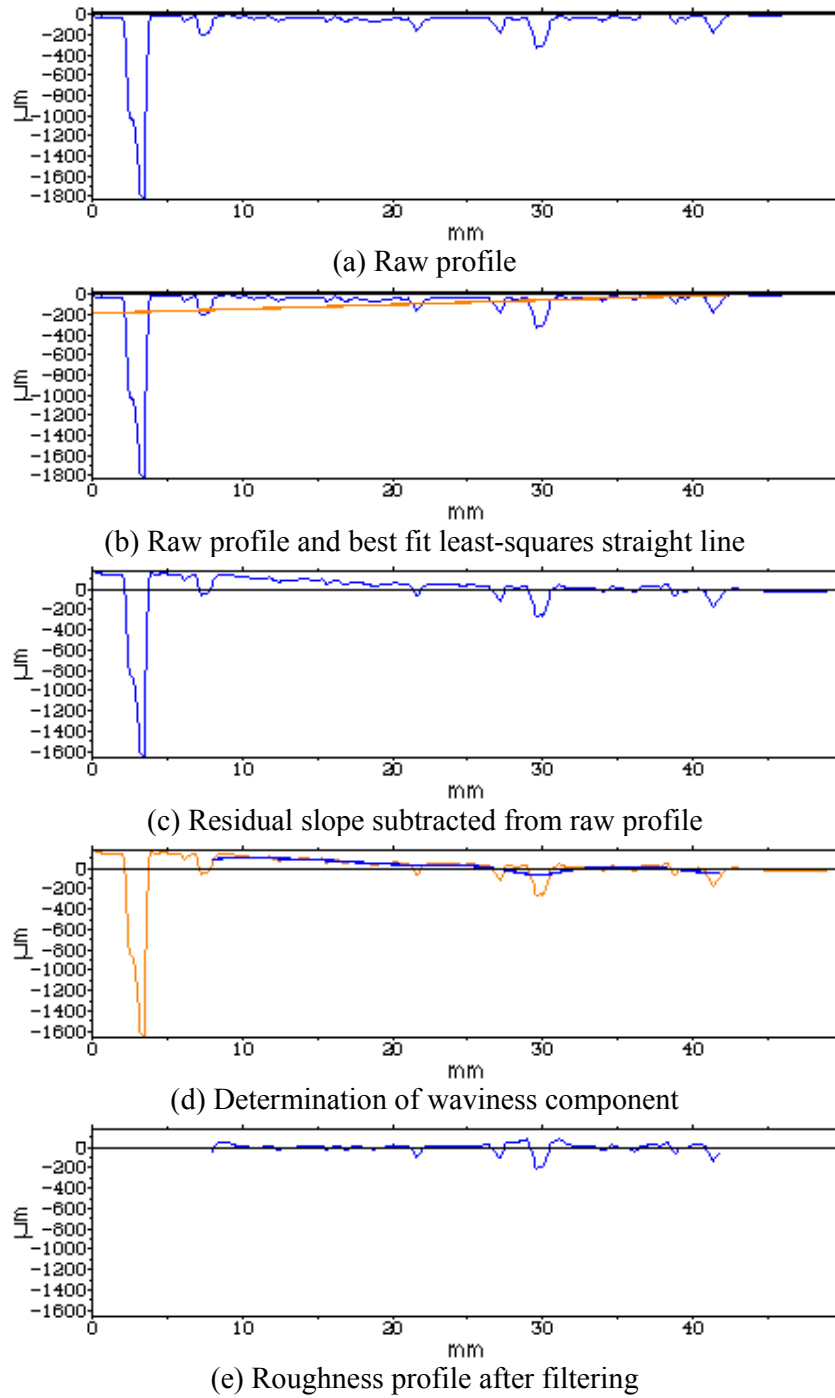


Figure B.5 : Obtaining texture components for a smooth texture (5-5-A stone unit).

APPENDIX C: Cyclic Shear Tests of Multi-Leaf Masonry Walls

C.1 M-25-C Wall

The M-25-C specimen consisted of two ashlar outer leaves and a rubble masonry core. The stones were connected to each other via horizontally positioned cramps. Axial stress level was kept as 0.25 MPa during the test. The test was conducted 141 days after the production of the M-25-C wall.

C.1.1 Initial compression test (M-25-C)

Four successive axial loading and unloading cycles were applied prior to reversed cyclic shear loading. The first target axial load level was 500 kN. However, since initiation of vertical cracks at cut stones and at the interfaces of inner and outer leaves was observed; loading was ceased at 260 kN (almost 0.72 MPa). The axial load reversals for the next three cycles were limited to 90 kN that corresponds to 0.25 MPa. The axial stress-axial strain diagrams of these four cycles plotted by using the average of the measurements of the six LVDTs (with an approximate gauge length of 800 mm) on both sides of the specimen are presented in Figure C.1.

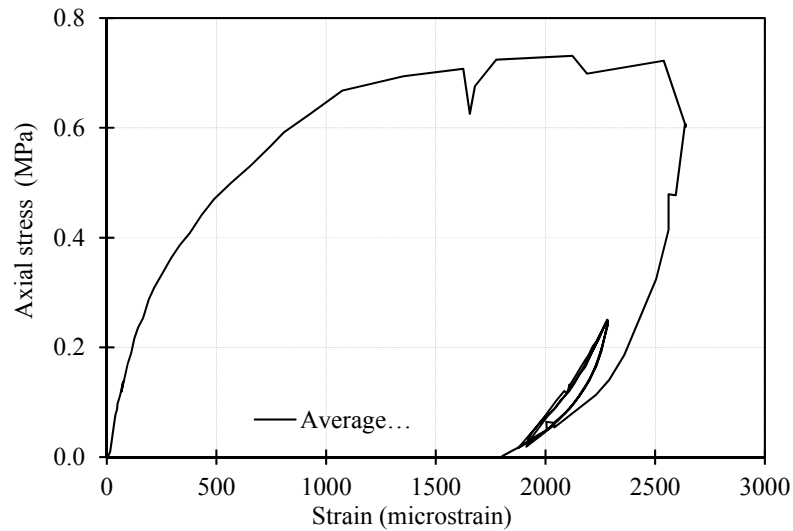


Figure C.1 : Axial stress-axial strain behaviour of M-25-C specimen.

C.1.2 Shear compression test (M-25-C)

The M-25-C specimen was subjected to a vertical stress of 0.25 MPa (lowest axial stress level of this study). After performing the pre-loading session that consisted of four successive axial loading cycles, all channels were initialized and hydraulic

power supply of the lateral actuator was activated. The lateral loading began after applying the targeted 0.25 MPa axial load.

Up to 0.25% drift ratio (2.75 mm displacement), a small number of randomly distributed vertical cracks on the outer leaves (North and South sides) could be observed, Figures C.2 and C.3. Simultaneously, cracks visible on the east and west sides, that were previously formed during the first axial pre-loading cycle, extended and widened up to 0.3 mm thickness, Figure C.3.

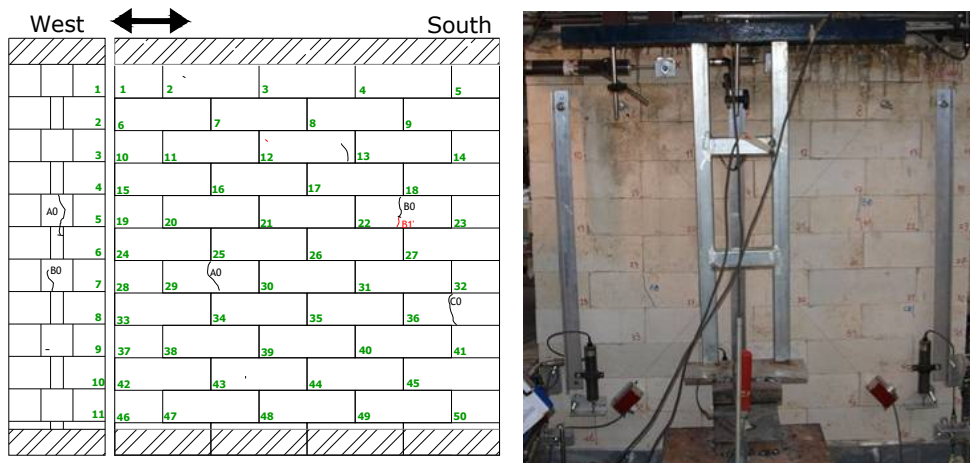


Figure C.2 : Cycle 1, M-25-C after 0.55 mm displacement (0.05% drift ratio).

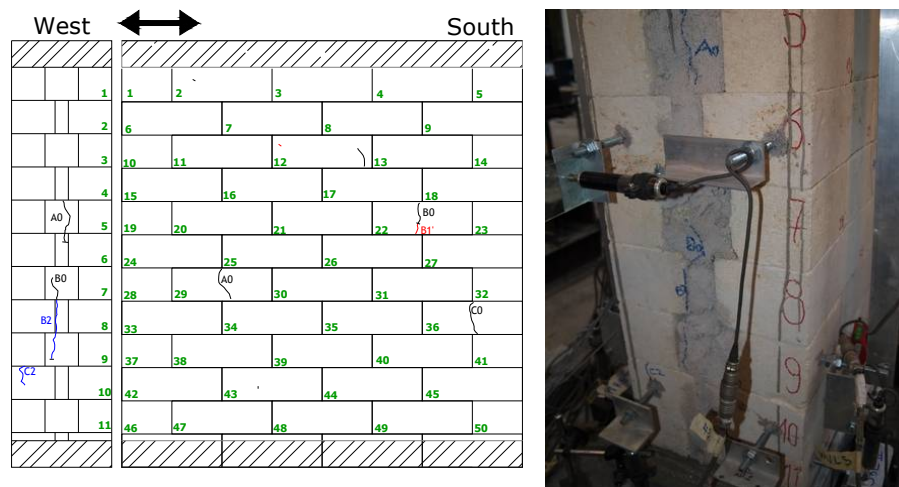


Figure C.3 : Cycle 2, M-25-C after 1.10 mm displacement (0.10% drift ratio).

At the pulling part of the 2.75 mm (0.25% drift ratio) cycle, first horizontal crack at the East side could be observed, Figure C.4. In addition to that, a number of vertical cracks with widths ranging between 0.1 and 1.0 mm (0.5 mm in average) occurred at the outer leaves.

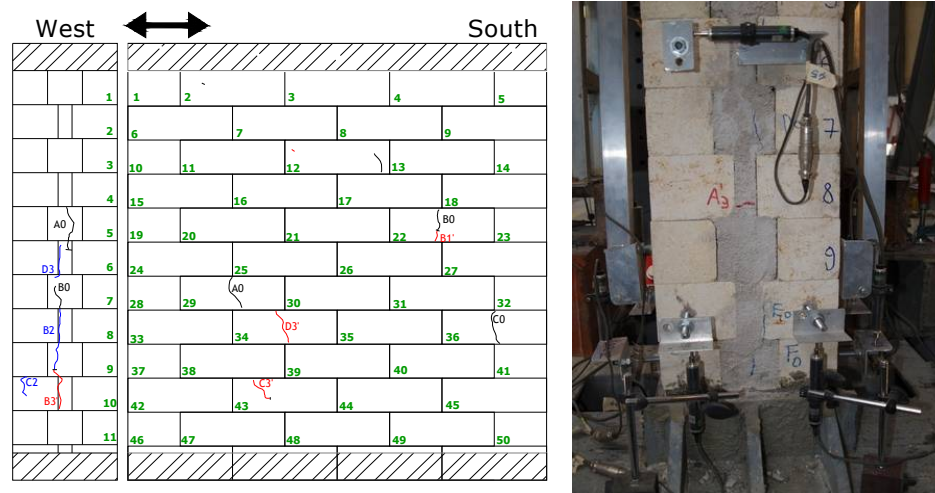


Figure C.4 : Cycle 3, M-25-C after 2.75 mm displacement (0.25% drift ratio).

At the pushing part of the 5.50 mm (0.50% drift ratio) cycle (cycle 4a), first horizontal crack at lower 1/3 of the specimen height of the West side could be observed. Moreover, a number of vertical and diagonal cracks started to appear around the compression zones at the toe parts of each loading direction, Figure C.5. Maximum crack width at peaks of this cycle could reach 1.2 mm. In addition to the traces of separation between the outer leaves and the rubble masonry core, a bulge formation started to appear at mid-height of the East side.

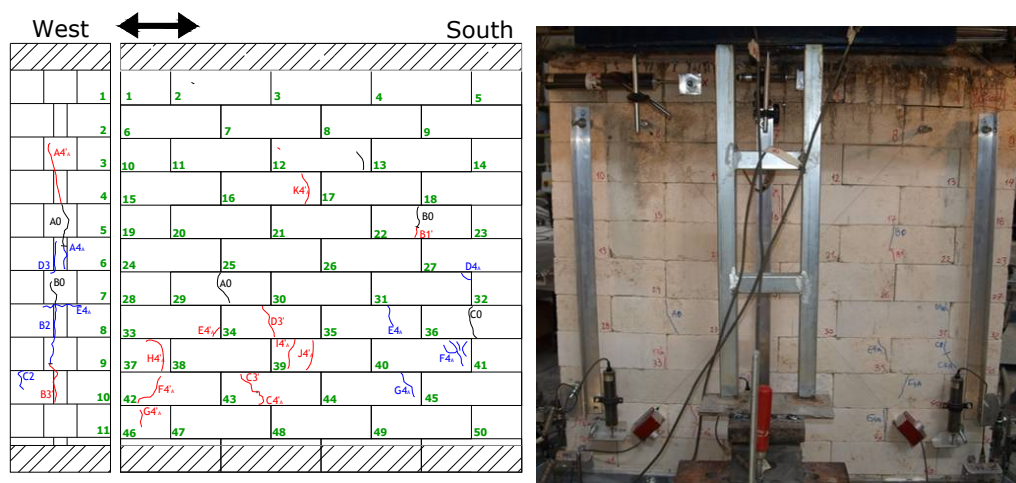


Figure C.5 : Cycle 4a, M-25-C after 5.50 mm displacement (0.50% drift ratio).

During the second 5.50 mm (0.50% drift ratio) cycle (cycle 4b), diagonal cracks passing through the stone units and stepped cracks following the vertical and horizontal joints occurred. Number and width of diagonal cracks increased, especially for the compression zone of the pushing direction, Figure C.6. Rubble core

exhibited signs of crushing at the compression zones at the toes of each loading direction.

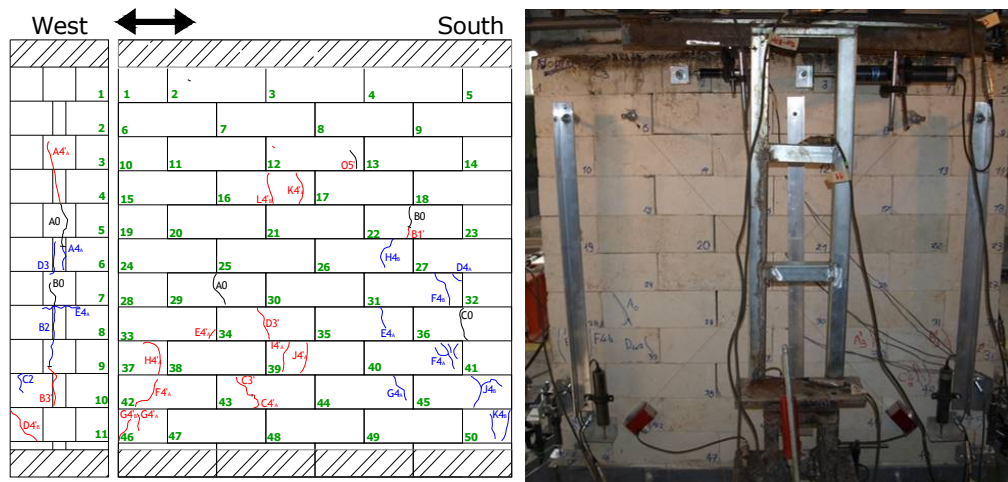


Figure C.6 : Cycle 4b, M-25-C after 5.50 mm displacement (0.50% drift ratio).

Observations at the peaks of the third cycle of 5.50 mm (0.50% drift ratio, cycle 4c) revealed the initiation of crushing signs at the toe of pulling direction, Figure C.7. Diagonal crack widths could reach up to 2.3 mm. It was also observed that, considerable number of the cracks passing through the stone units started from the points where cramps were bent and anchored to the stone.

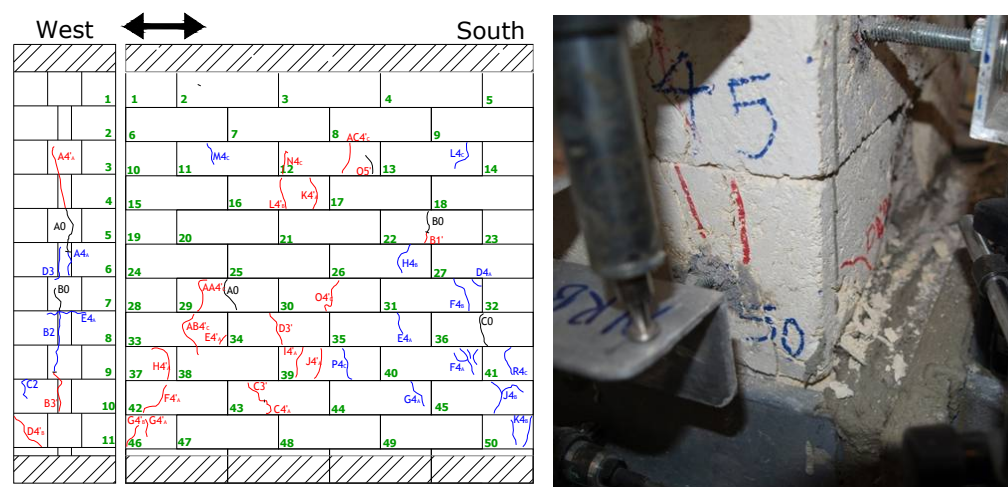


Figure C.7 : Cycle 4c, M-25-C after 5.50 mm displacement (0.50% drift ratio).

Cycle 5, targeting 8.25 mm horizontal displacement (0.75% drift ratio), caused a visible amount of sliding between horizontal joints of the lower courses (Figure C.8) and crushing of the stone units at the toe of the pushing direction (Figure C.9a). The

separation occurred between the outer leaves and the rubble masonry core reached to 8 mm in transverse direction along the wall thickness, Figure C.9b.

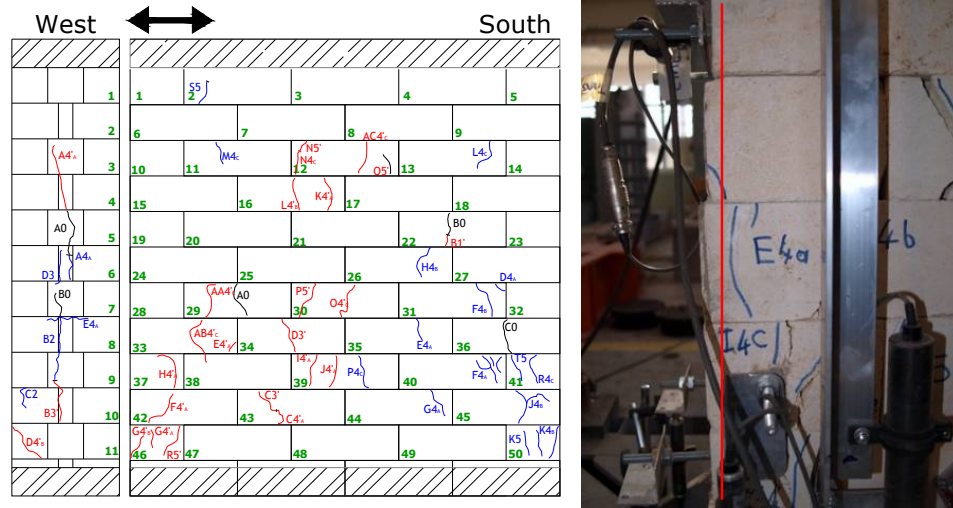


Figure C.8 : Cycle 5, M-25-C after 8.25 mm displacement (0.75% drift ratio).

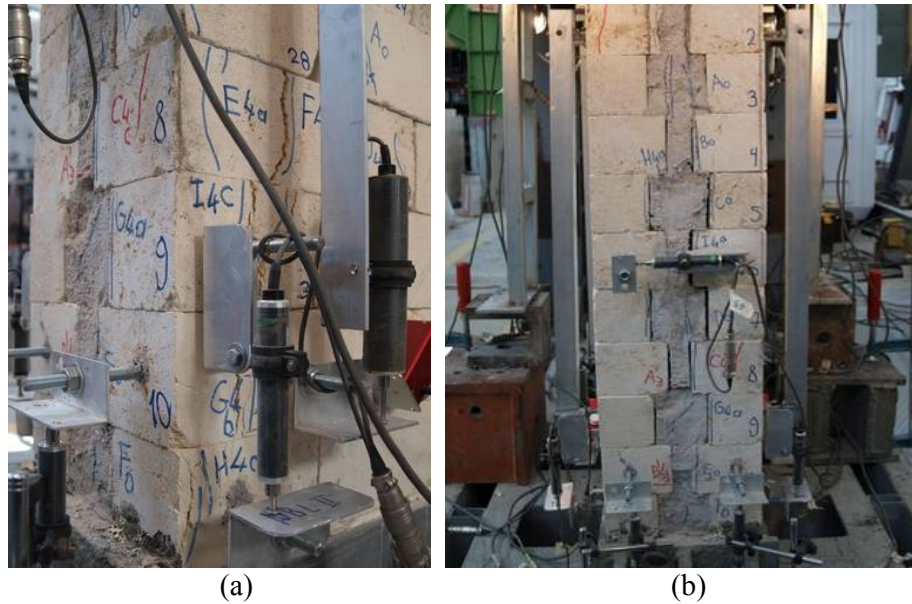


Figure C.9 : Cycle 5, M-25-C after 8.25 mm displacement (0.75% drift ratio).

During the first, second and third 11.0 mm (1.0% drift ratio) cycles (cycles 6a, 6b and 6c), number and width of the diagonal cracks increased noticeably (maximum crack width over 5 mm). Rocking behaviour became apparent with the horizontal cracks observed between the courses of stone units at the lower half of the specimen height (Figure C.10) and the spalling began at the crushed toes of each loading direction, Figure C.11. However, with the spalling of the compression zones during

the successive displacement cycles, the horizontal cracks between the stone unit courses propagated upwards, Figure C.12. The horizontal slip measured on North and South faces of the wall reached 30 and 15 mm, respectively; while the horizontal opening between the stone courses was about 10 mm.

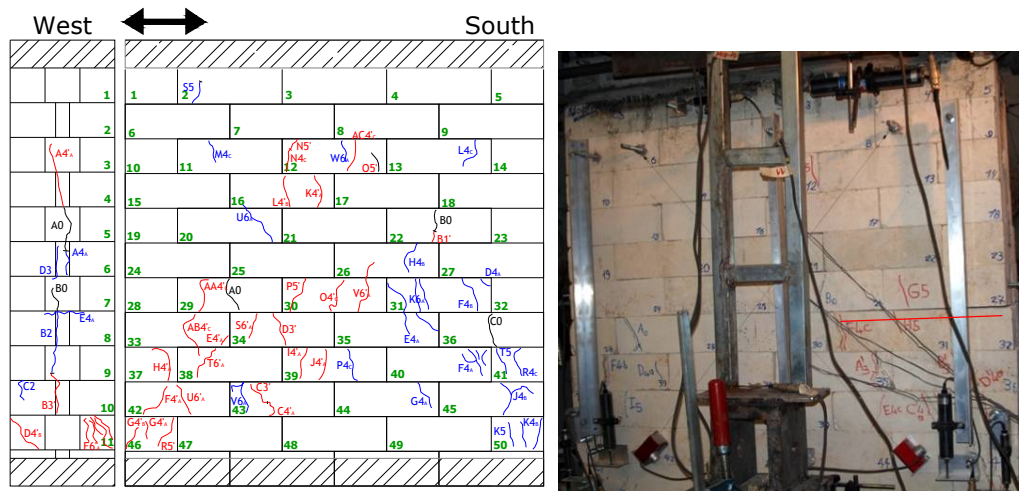


Figure C.10 : Cycle 6a, M-25-C after 11.0 mm displacement (1.00% drift ratio).

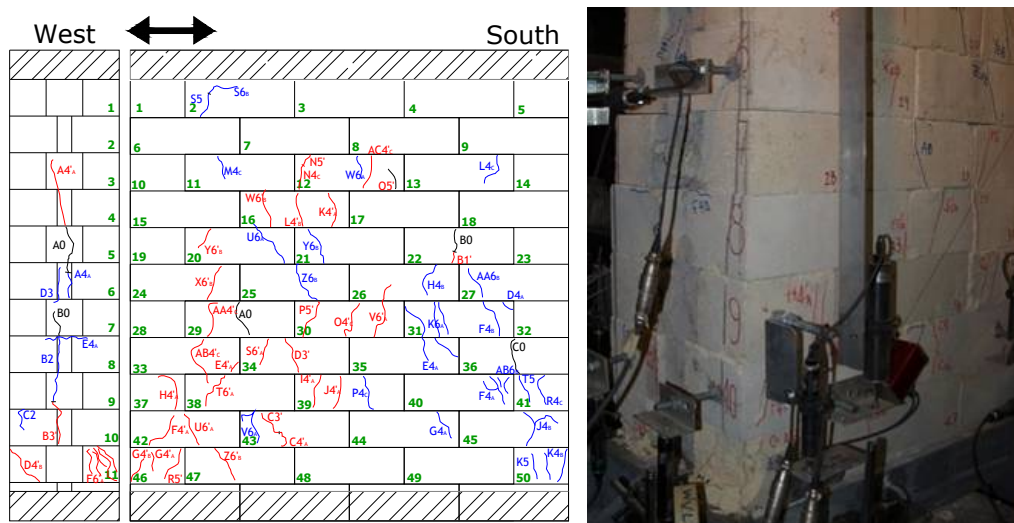


Figure C.11 : Cycle 6b, M-25-C after 11.0 mm displacement (1.00% drift ratio).

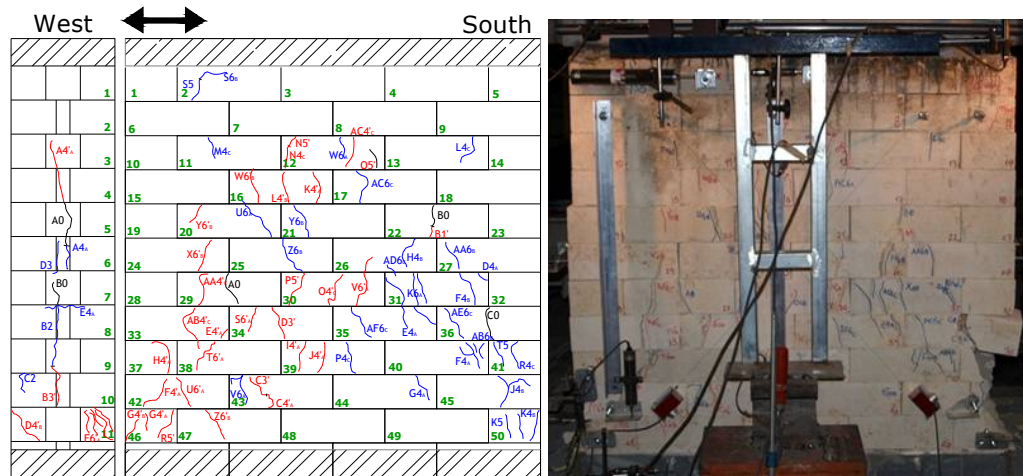


Figure C.12 : Cycle 6c, M-25-C after 11.0 mm displacement (1.0% drift ratio).

The cracking and spalling of stone units continued during the 7th and 8th cycles (corresponding to 1.25 and 1.50% drift ratios, respectively), Figures C.13 and C.14. In cycle 8, the horizontal sliding between the stone courses reached eye-catching values in the order of 50 mm, Figure C.14.

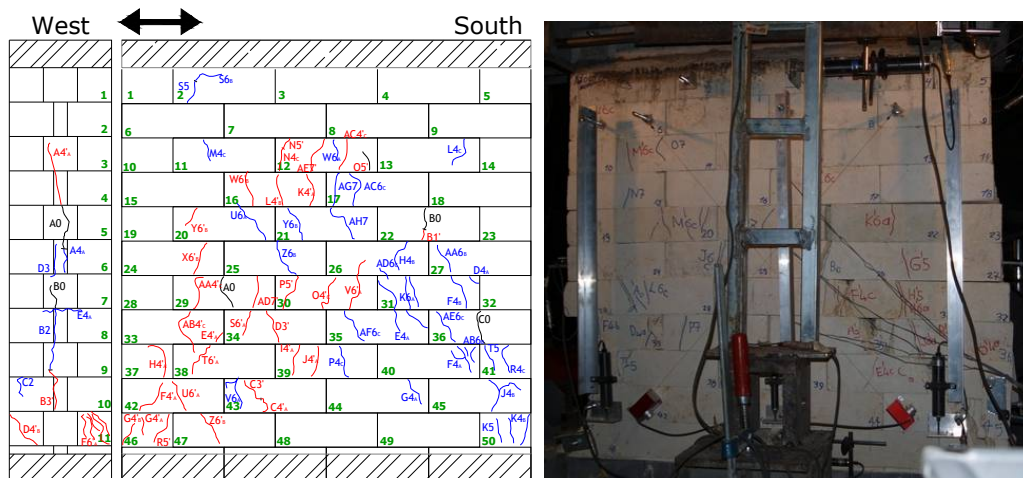


Figure C.13 : Cycle 7, M-25-C after 13.75 mm displacement (1.25% drift ratio).

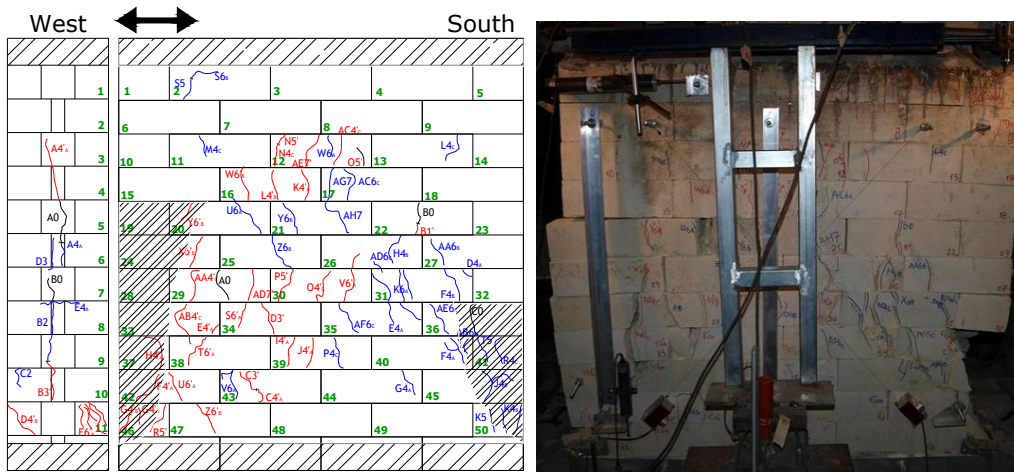


Figure C.14 : Cycle 8, M-25-C after 16.50 mm displacement (1.50% drift ratio).

After completing the pushing part of the 22.0 mm (2.0% drift ratio) cycle (cycle 9), the test was stopped due to heavy damage and significant drop (31% drop) in the load bearing capacity. After test views of the North, South, East and West faces of the specimen are presented in Figures C.15-C.17.



Figure C.15 : Cycle 9, North face of M-25-C after 2.0% drift ratio.



Figure C.16 : Cycle 9, South face of M-25-C after 2.0% drift ratio.

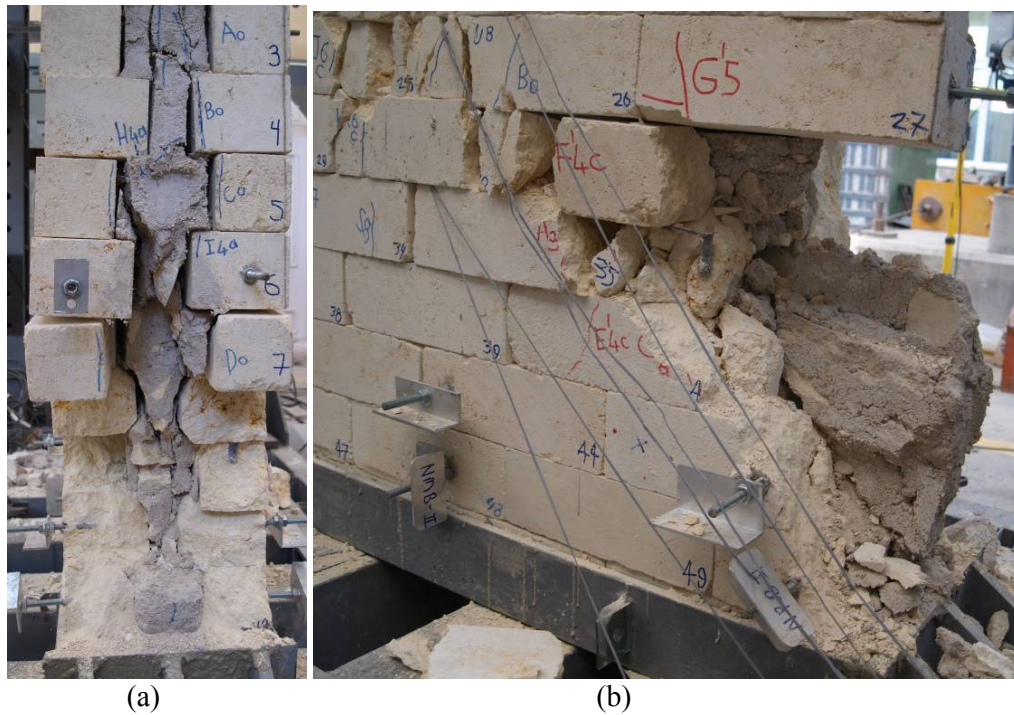


Figure C.17 : Cycle 9, (a) East and (b) West faces of M-25-C after 22.0 mm displacement (2.0% drift ratio).

Drift ratios, displacements, peak loads, peak load to maximum strength ratios (Peak Load/ H_{max}) and notes on damage development corresponding to each performed cycle are summarized in Table C.1. From this table it can be pointed out that

maximum load capacity of the M-25-C specimen is attained at the first cycle of 0.50% drift ratio. The damage evolution started with the horizontal bending cracks at lower levels of lateral displacement and followed by diagonal cracks either passing through the stone units or joints. Cracks passing through the stone units were generally concentrated at the points where the cramps were bent and anchored to the stone units. A sample of these crack concentrations at the cramp tips are marked with white arrows and presented in Figure C.18.

Damage development continued with the rocking of the upper half of the specimen and crushing (after cracking) of the compressed units around which the rocking occurred. Meanwhile, a considerable amount of sliding among the horizontal joints and separation of the inner and outer leaves were also observed. Strength degradation reached to 20% at the 1.50% drift ratio for the pushing direction and 0.75% drift ratio for the pulling direction. The test was stopped at 2.00% drift ratio after extensive spalling of the compressed parts that caused significant loss of strength.

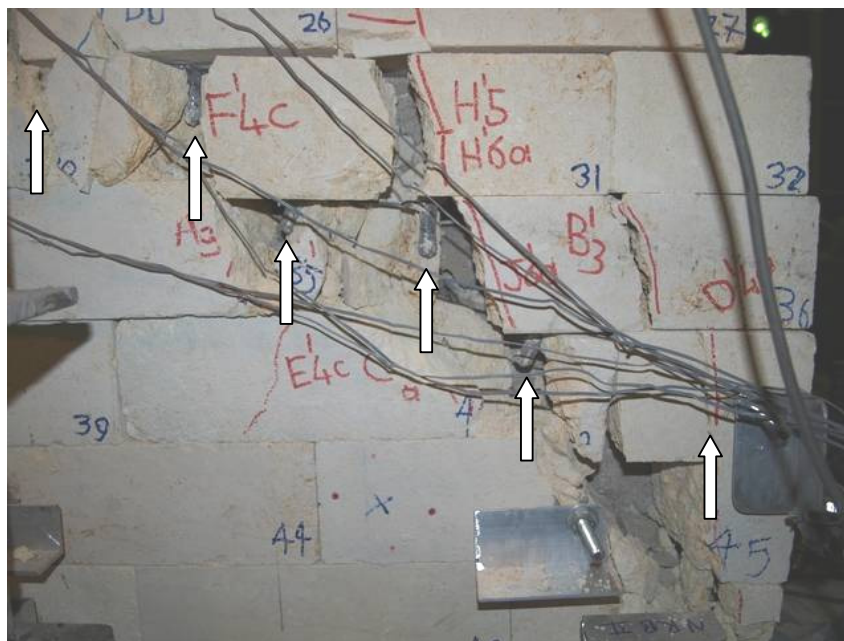


Figure C.18 : Cracks passing through the tips of the cramps.

Table C.1 : M-25-C specimen summary table.

Cycle	Drift ratio (%)	Displacement (mm)	Peak Load (kN)	Peak Load/ H_{max} (%)	Note
1. Push	0.05	0.55	18.9	34	
1. Pull	-0.05	-0.55	-25.5	47	
2. Push	0.10	1.10	34.2	61	
2. Pull	-0.10	-1.10	-38.6	71	
3. Push	0.25	2.75	49.9	89	First horizontal crack-West side
3. Pull	-0.25	-2.75	-51.0	94	First horizontal bending crack visible-East side
4. Push (a)	0.50	5.50	56.3	100	Max. load, first diagonal cracks
4. Pull (a)	-0.50	-5.50	-54.5	100	Max. load, first diagonal cracks
4. Push (b)	0.50	5.50	47.6	85	Stepped cracks, rubble core started crushing-East side
4. Pull (b)	-0.50	-5.50	-52.3	96	Stepped cracks, rubble core started crushing-West side
4. Push (c)	0.50	5.50	50.5	90	Toe crushing started-East side
4. Pull (c)	-0.50	-5.50	-46.7	86	Toe crushing started-West side
5. Push	0.75	8.25	51.0	91	Sliding, separation between the outer leaves and the inner core on the East side
5. Pull	-0.75	-8.25	-43.2	79	Sliding, separation between the outer leaves and the inner core on the West side
6. Push (a)	1.00	11.00	51.3	91	Increased number of diagonal cracks
6. Pull (a)	-1.00	-11.00	-39.8	73	Increased number of diagonal cracks
6. Push (b)	1.00	11.00	48.4	86	Rocking, spalling started at the toe-East side
6. Pull (b)	-1.00	-11.00	-40.6	75	Rocking, spalling started at the toe-West side
6. Push (c)	1.00	11.00	44.2	79	Significant spalling at the toe-East side
6. Pull (c)	-1.00	-11.00	-34.1	63	Significant spalling at the toe-West side
7. Push	1.25	13.75	47.8	85	Toe crushing, sliding
7. Pull	-1.25	-13.75	-34.6	63	Toe crushing, sliding
8. Push	1.50	16.50	43.5	77	Heavy damage
8. Pull	-1.50	-16.50	-31.8	58	Heavy damage
9. Push (a)	2.00	22.00	38.8	69	Loss of stability

The lateral load-displacement hysteresis diagram describing the in-plane response of the M-25-C specimen is presented in Figure C.19. Data points corresponding to first flexural cracking ($H_{f,cr}$), first diagonal cracking ($H_{dia,cr}$), maximum lateral resistance (H_{max}), 20% strength loss ($H_{0.8max}$) and ultimate displacement ($H_{d,max}$), are marked in the figure. The obtained hysteresis loops show that the wall exhibited a highly nonlinear response with significant residual displacements after each load reversal.

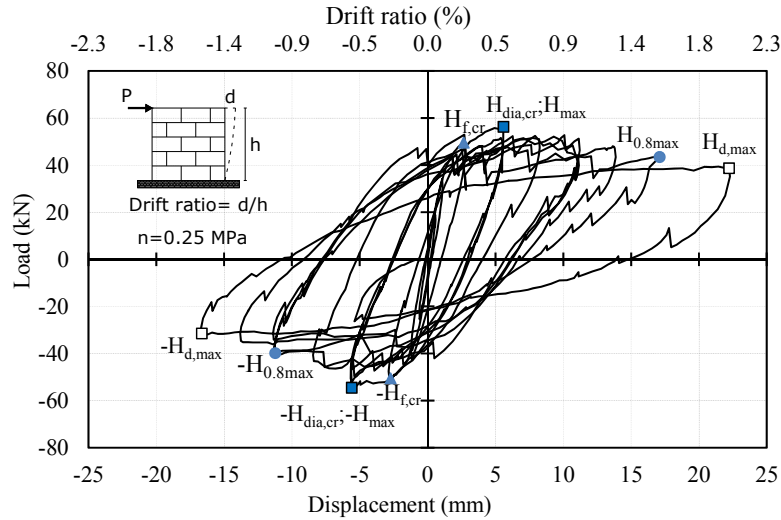


Figure C.19 : Lateral load-displacement diagram for specimen M-25-C.

The variation of the dissipated energy by means of the percentage of the total dissipated energy ($E_{dis,i}/E_{dis,cumulative}$, i denoting the cycle number) corresponding to each loading cycle is presented in Figure C.20. This presentation enables the identification of the critical points corresponding to particular events. A steep increase in the dissipated energy is a sign of increase in the nonlinear deformation, which can be directly associated with the damage growth. In this particular case, the first sudden increase in the dissipated energy occurs at Cycle 3 (0.25% drift ratio), where the first flexural cracks becomes visible. The second increase at Cycle 4 (0.50% drift ratio) corresponds to first diagonal cracks. With further formation of the diagonal cracks and crushing and spalling of the compressed zones, another sudden increase happens at Cycle 6 (1.00% drift ratio).

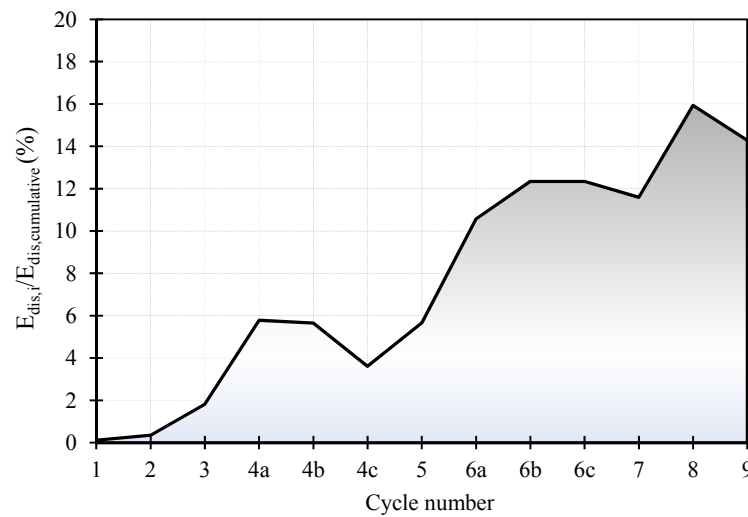


Figure C.20 : Ratio of dissipated energy at each cycle to total, M-25-C.

Though evaluation of in-plane behaviour of masonry walls through hysteresis curves is inevitable, envelope curves are also very beneficial for simplified investigation of force-displacement diagrams. Derivation of envelope curves can serve as a simplified method for evaluation of characteristics such as stiffness, strength and deformability. The envelope curve obtained for the M-25-C specimen can be seen in Figure C.21.

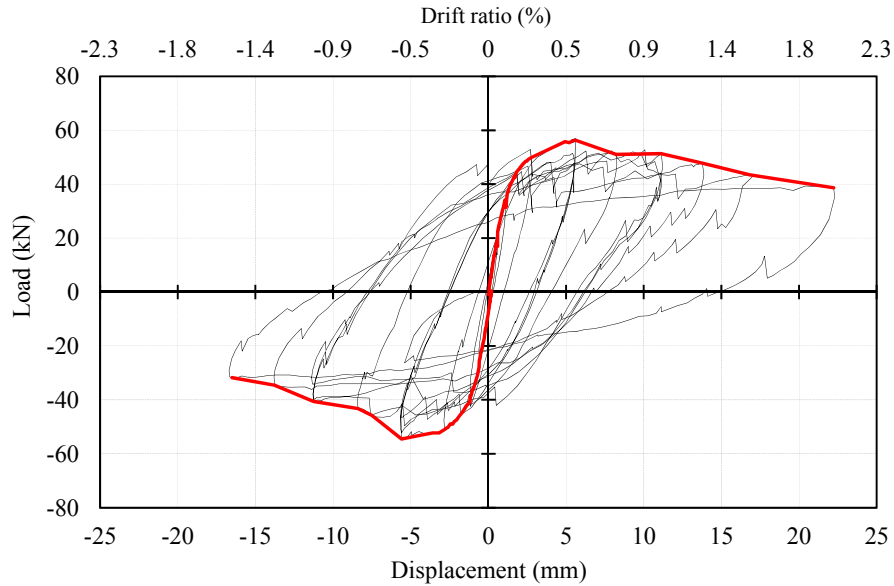


Figure C.21 : Envelope curve of lateral load-displacement diagram, M-25-C.

Apart from lateral load and displacement data, various other measurements were also collected from the specimen. Figure C.22 shows the variation of the vertical displacements measured at the tips of the top bond beam with respect to lateral displacement. As can be observed, the wall specimen tends to shorten although the axial force is quite low for this specimen. This compaction increases with the formation of further damage (such as crushing and spalling of compressed zones) at larger lateral displacements.

Vertical deformations measured by channels 36, 37, 40 and 41 (as shown in Figures 3.18 and 3.19) with respect to lateral displacement are presented in Figure C.23. In these figures, positive sign direction of the vertical axis corresponds to the shortening of the measured length. Accordingly, although in overall the wall tends to shorten, elongation can be observed for the zones that stay in tension during the cyclic displacement cycles.

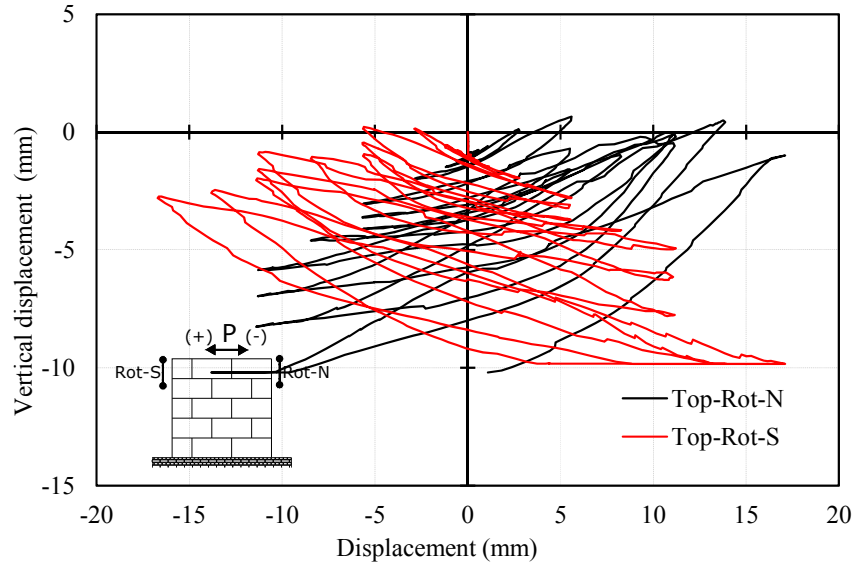


Figure C.22 : Vertical displacements at the bond beam end-points, M-25-C.

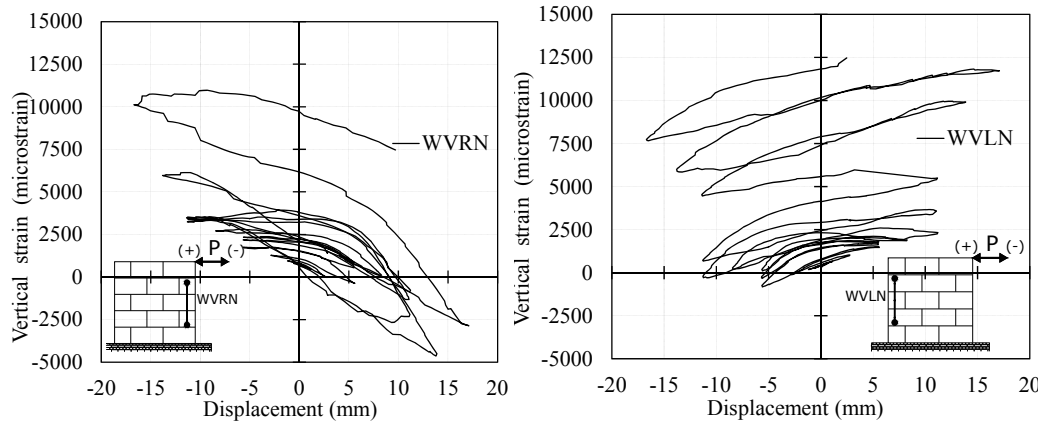


Figure C.23 : Vertical deformations near the Western and Eastern sides, M-25-C.

Vertical deformation measurements at toes and heels (depending on the loading direction) with respect to lateral displacements show that these zones are almost under compression throughout the test, Figure C.24. The deformations increase significantly with the increase in lateral drift, particularly after 8.25 mm (0.75% drift ratio), and considerable amount of plastic deformations occur.

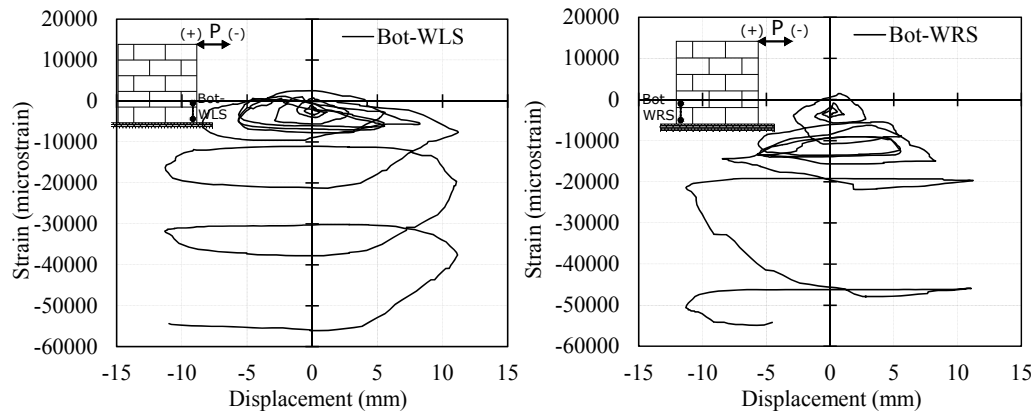


Figure C.24 : Vertical deformations at the toes and heels of the wall M-25-C.

Three of the cramps on the Northern leaf of the wall were instrumented with strain gauges. One strain gauge was glued to the upper side at the mid-length of the cramp (in the longitudinal direction), while another one was used for the lower side. Measurements logged for the cramp at mid-height of the specimen show that compressive and tensile strains on the upper and lower sides of the cramp could reach 1500 microstrain which are almost 500 microstrain below the yield point of the steel, Figure C.25. Interestingly, the strain diagram turns out to be perfectly symmetric, which means that when upper side is in compression the lower one is tension and vice versa. Consequently, if the average of these symmetric strains is calculated for each instrumented cramp, an average elongation value less than 100 microstrain is obtained for the duration of the whole test process, as seen in Figure C.26. Similar results were observed for upper and lower instrumented cramps of the specimen.

Detachment between the exterior leaves of the wall was measured by two LVDTs positioned at mid-heights of the Eastern and Western faces of the specimen. As clearly seen from the transverse displacement versus lateral displacement diagram (Figure C.27) obtained for the Eastern side, outer leaves tend to open in the out-of-plane direction.

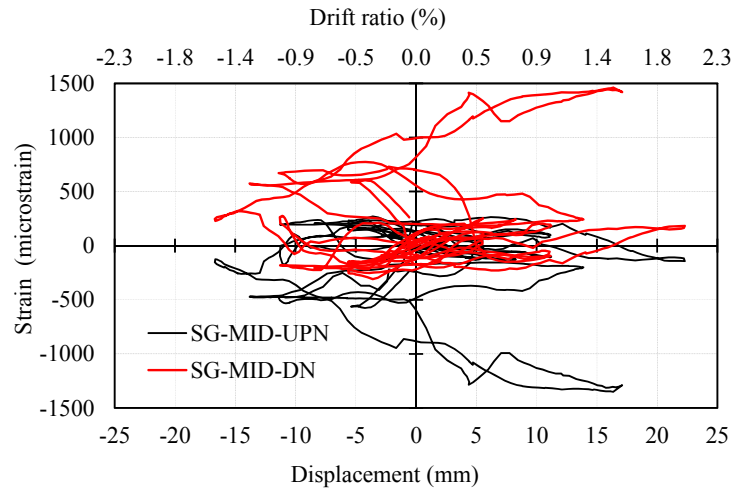


Figure C.25 : Strain variation for mid-height cramp (M-25-C wall).

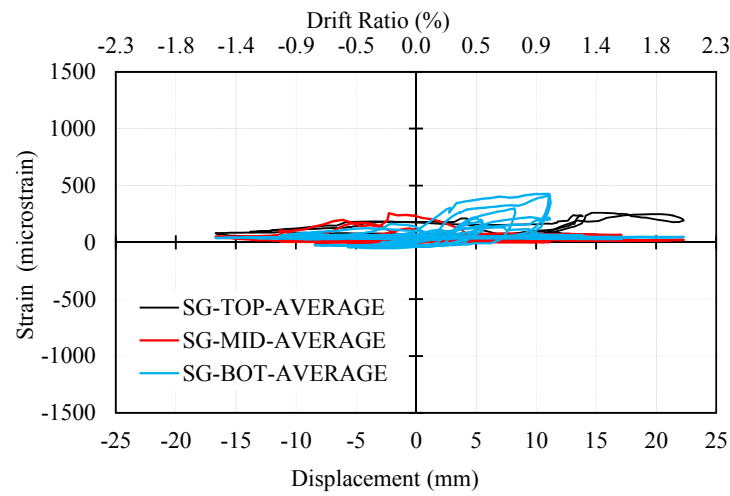


Figure C.26 : Average strains of instrumented cramps of wall M-25-C.

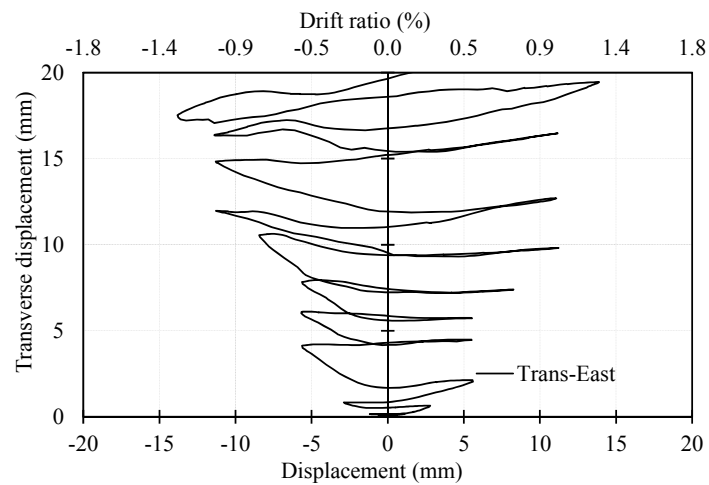


Figure C.27 : Detachment between the exterior leaves (Eastern side, M-25-C).

C.2 M-50-C Wall

The M-50-C specimen consisted of two ashlar outer leaves and a rubble masonry core. The stones were connected to each other via horizontally positioned clamps. Axial stress level was kept as 0.50 MPa throughout the test. The test was conducted 155 days after the production of the M-50-C wall.

C.2.1 Initial compression test (M-50-C)

Three successive axial loading and unloading cycles were applied prior to cyclic lateral loading. The target vertical load level was 180 kN that corresponded to 0.50 MPa axial stress. The axial stress-axial strain diagrams of these three cycles plotted by using the average of the measurements of the six LVDTs (with an approximate gauge length of 800 mm) on both sides of the specimen are presented in Figure C.28. As also seen in Figure C.28, the first applied axial loading cycle caused considerable amount of plastic deformation in the axial direction.

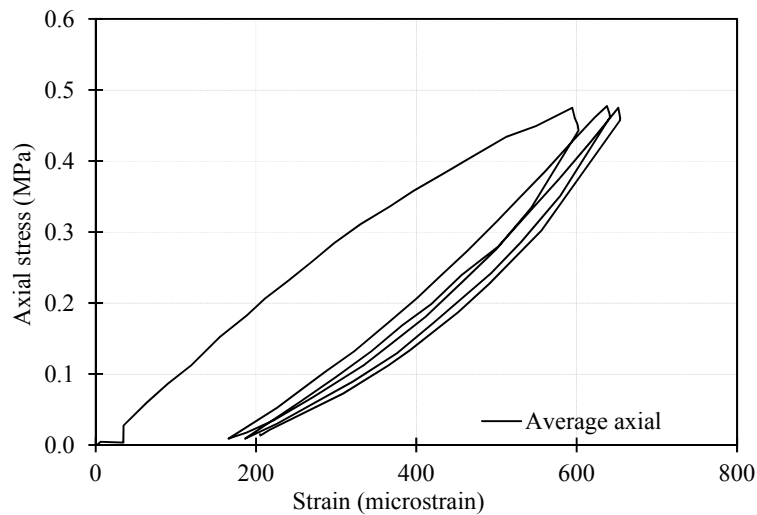


Figure C.28 : Axial stress-axial strain behaviour of M-50-C specimen.

C.2.2 Shear compression test (M-50-C)

The M-50-C specimen, which was subjected to a vertical stress level of 0.50 MPa (medium vertical stress level of this study), exhibited a shear governed response during the application of in-plane lateral displacement cycles. After performing the pre-loading session that consisted of three successive axial loading cycles, all channels were initialized and hydraulic power supply of the lateral actuator was activated. Then lateral loading began after applying the targeted 0.50 MPa axial load.

Up to 0.25% drift ratio (2.75 mm displacement), no damage development could be observed, Figures C.29 and C.30.

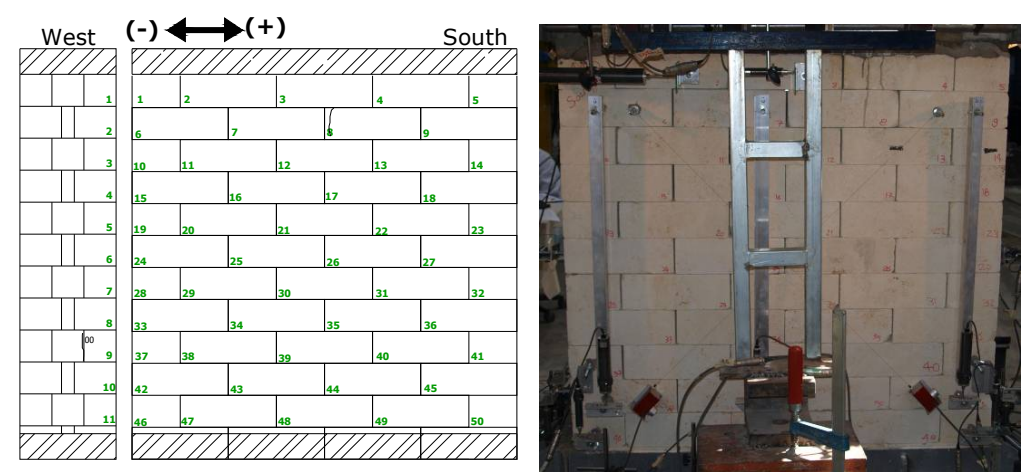


Figure C.29 : Cycle 1, M-50-C after 0.55 mm displacement (0.05% drift ratio).

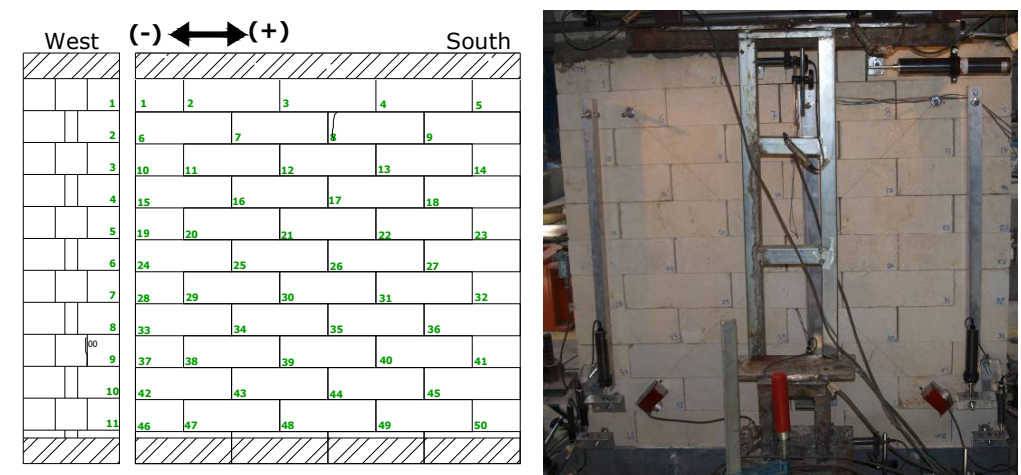


Figure C.30 : Cycle 2, M-50-C after 1.10 mm displacement (0.10% drift ratio).

At the 2.75 mm (0.25% drift ratio) cycle, first horizontal cracks at the lower parts of the Western and Eastern sides could be observed, Figure C.31. In addition to that, vertical cracks between the interfaces of outer leaves and the rubble core could be seen. During the pushing of the specimen, first vertical cracks passing through the stone units occurred at the toe of the wall, Figure C.31.

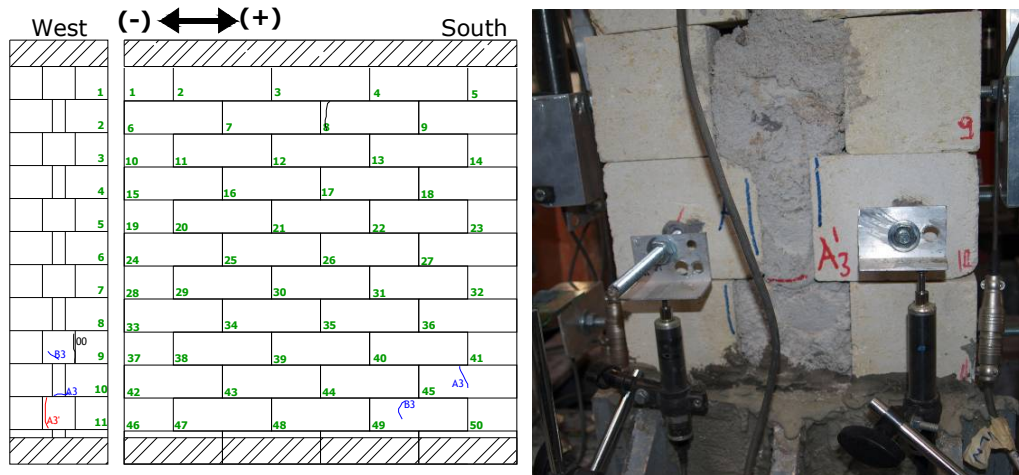


Figure C.31 : Cycle 3, M-50-C after 2.75 mm displacement (0.25% drift ratio).

At the 5.50 mm cycle (0.50% drift ratio, cycle 4a), a number of vertical cracks started to appear around the compression zones at the toe parts of each loading direction, Figure C.32. Maximum crack width at peaks of this cycle could reach 0.3 mm. Bed joints of the lower half of the specimen opened in the vertical direction due to rocking behaviour of the upper 2/3 of the specimen. Vertical separation cracks between the inner and outer leaves not only followed the interface, but also continued through the rubble core where the stone units intended into the core, Figure C.33.

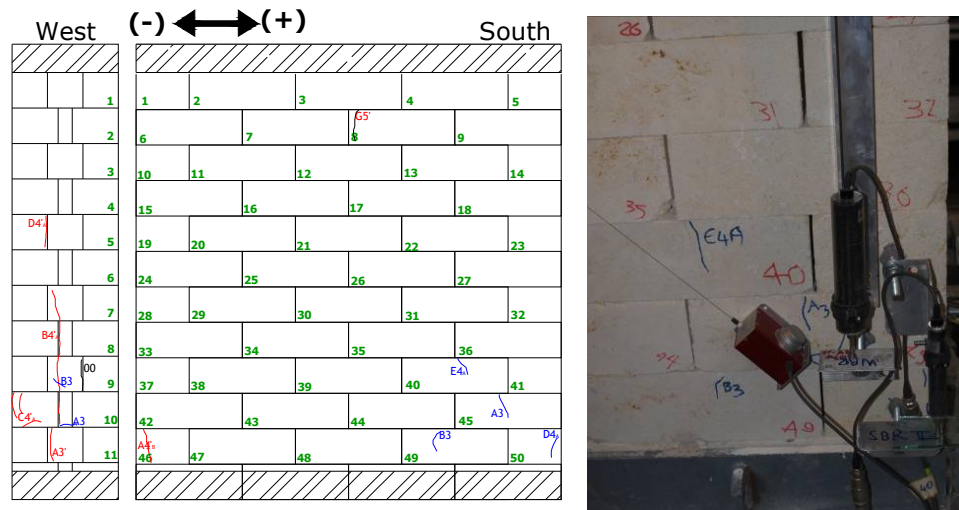


Figure C.32 : Cycle 4a, M-50-C after 5.50 mm displacement (0.50% drift ratio).



Figure C.33 : Cycle 4a, vertical cracks between the inner and outer leaves, M-50-C.

During the second 5.50 mm (0.50% drift ratio) cycle (cycle 4b), vertical cracks passing through the stone units started to appear on a diagonal band. Number of vertical cracks at the heels and toes increased, especially for the compression zone of the pushing direction, Figure C.34. Vertical cracks in the rubble core further extended upwards, Figure C.34.

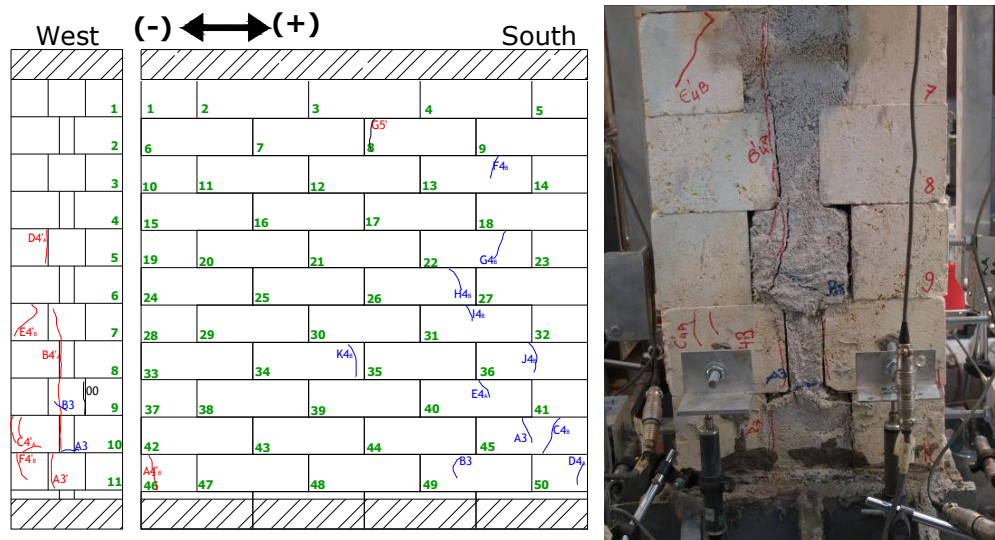


Figure C.34 : Cycle 4b, M-50-C after 5.50 mm displacement (0.50% drift ratio).

During the third cycle of 5.50 mm (0.50% drift ratio, cycle 4c) crack widths could reach up to 0.7 mm. A great majority of the vertical and diagonal cracks on the stone units were either passing through the mid-length of the unit or the anchor points of the cramps, Figure C.35.

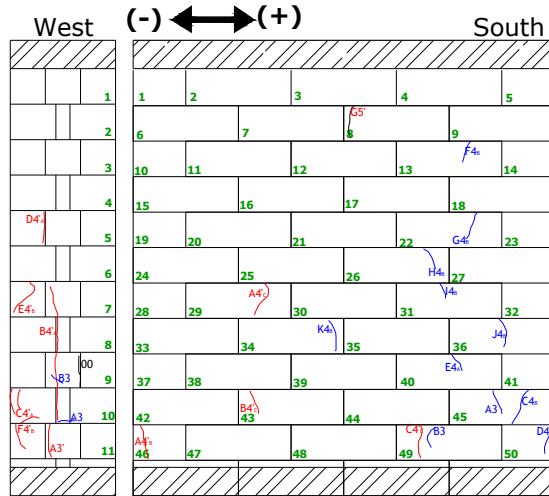


Figure C.35 : Cycle 4c, M-50-C after 5.50 mm displacement (0.50% drift ratio).

During Cycle 5, targeting 8.25 mm horizontal displacement (0.75% drift ratio), new cracks at the compressed areas and at the upper half of the specimen appeared, Figure C.36. Length and width of the existing cracks extended. Maximum crack widths reached 2.2 mm and 1.0 mm on the Southern and Northern sides, respectively.

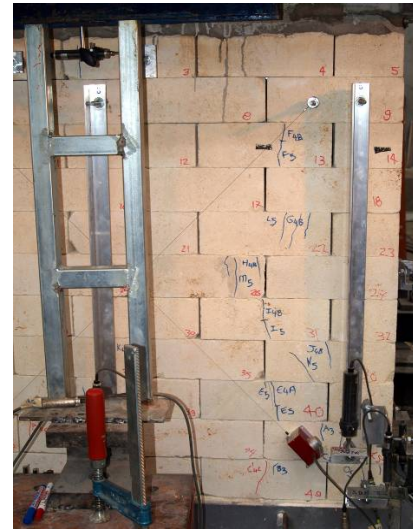
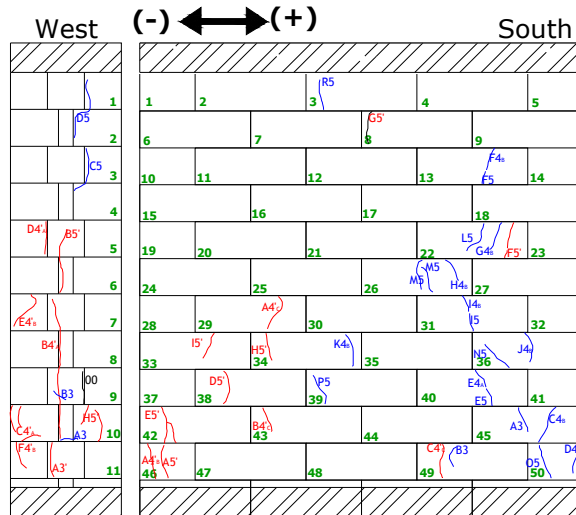
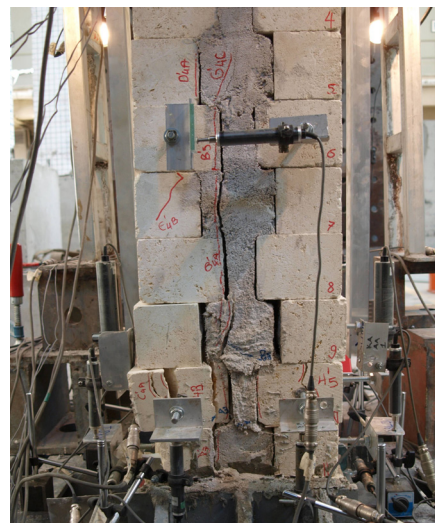
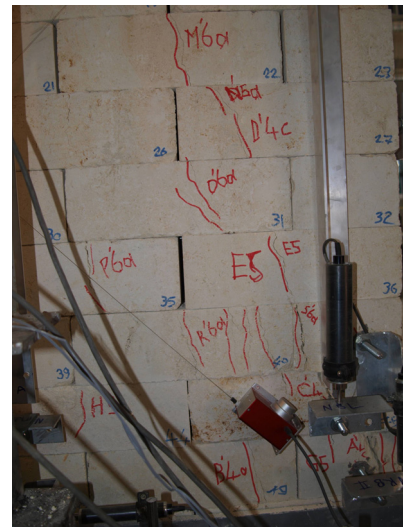


Figure C.36 : Cycle 5, M-50-C after 8.25 mm displacement (0.75% drift ratio).

During the first 11.0 mm cycle (1.0% drift ratio, cycle 6a), number and width of the diagonal cracks increased noticeably (maximum crack width over 3.5 mm), Figure C.37. Signs of crushing of the toes of each loading direction became visible during cycle 6a, Figure C.37. The second and third cycles of the 1.0% drift ratio further extended the damage, Figures C.38 and C.39. The crushed units started spalling



330

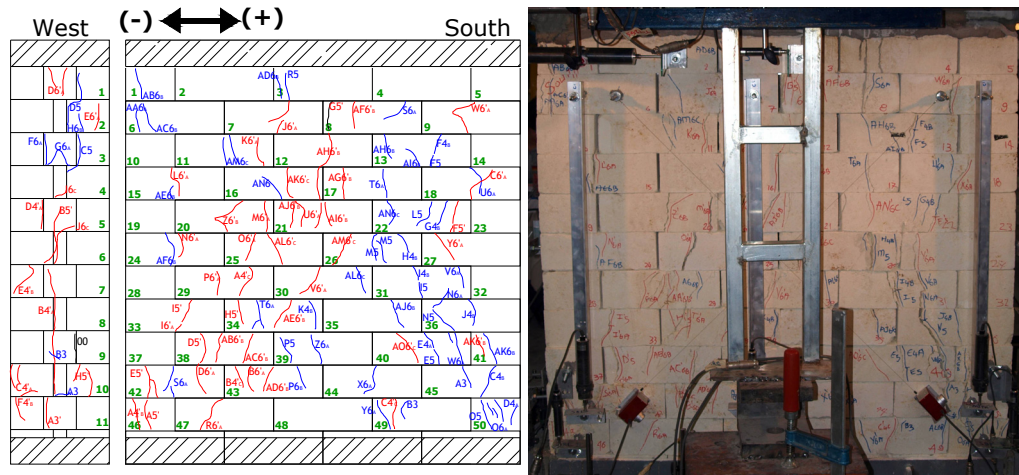


Figure C.39 : Cycle 6c, M-50-C after 11.0 mm displacement (1.0% drift ratio).

The cracking and spalling of stone units continued during the 7th and 8th cycles (corresponding to 1.25 and 1.50% drift ratios, respectively), Figures C.40 and C.41. In addition to cracks on the stone units, diagonal stepped cracks following the head and bed joints formed and the upper diagonal halves of the wall rotated around the toes for each respective loading direction.

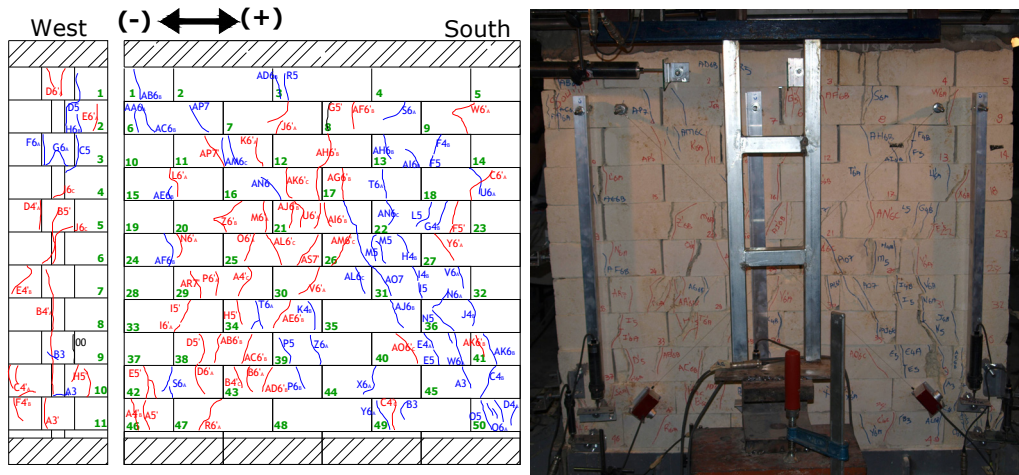


Figure C.40 : Cycle 7, M-50-C after 13.75 mm displacement (1.25% drift ratio).

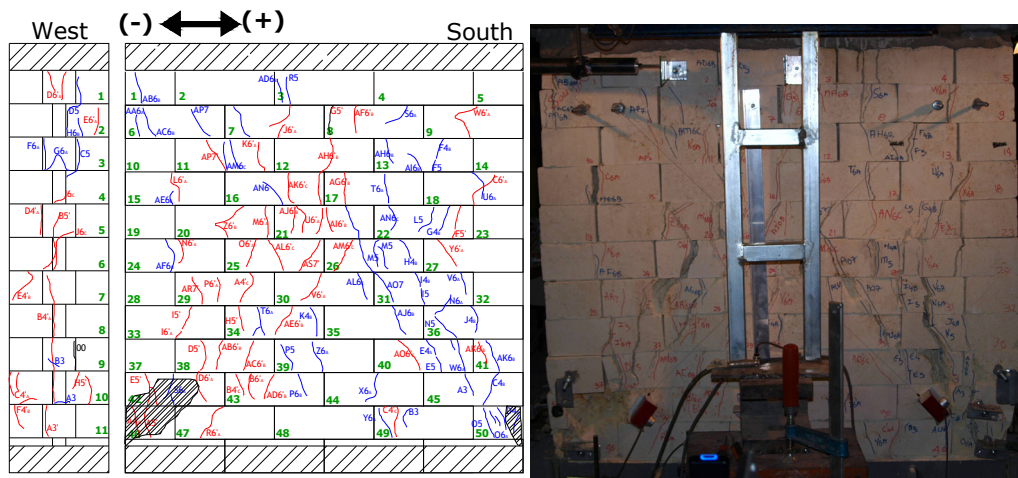


Figure C.41 : Cycle 8, M-50-C after 16.50 mm displacement (1.50% drift ratio).

After completing the pushing and pulling parts of the 16.5 mm (1.5% drift ratio) cycle, the test was stopped due to 25% drop in the load bearing capacity. After test views of the North, South, East and West faces of the specimen are presented in Figures C.42-C.44.



Figure C.42 : Cycle 8, North face of M-50-C after 1.50% drift ratio.



Figure C.43 : Cycle 8, South face of M-50-C after 1.50% drift ratio.



Figure C.44 : Cycle 8, (a) East and (b) West faces of M-50-C after 16.50 mm displacement (1.50% drift ratio).

Drift ratios, displacements, peak loads, peak load to maximum strength ratios (Peak Load/ H_{max}) and notes on damage development corresponding to each performed cycle are summarized in Table C.2. From this table it can be pointed out that load capacity of the M-50-C specimen is attained at of 8.25 mm target displacement (0.75% drift ratio) for pushing and pulling directions. The damage evolution started with horizontal bending cracks at lower levels of lateral displacement and followed by vertical and diagonal cracks passing through the stone units (especially at the toes). Cracks passing through the stone units were generally concentrated at mid-length of the units and the points where the cramps were bent and anchored to the stone units. A typical view of these crack concentrations at the cramp tips is presented in Figure C.45.

Damage evolution continued with the development of diagonal cracks. After the formation of the diagonal crack bands, upper diagonal halves of the wall started rocking. The test was stopped at 1.50% drift ratio after extensive spalling of the compressed parts that caused significant loss of strength (30% strength loss at 1.50% drift ratio).



Figure C.45 : Damage in the vicinity of cramp anchors, M-50-C.

Table C.2 : M-50-C specimen summary table.

Cycle	Drift ratio (%)	Displacement (mm)	Peak Load (kN)	Peak Load/H _{max} (%)	Note
1. Push	0.05	0.55	24.0	25	
1. Pull	-0.05	-0.55	-39.4	47	
2. Push	0.10	1.10	46.7	48	
2. Pull	-0.10	-1.10	-56.9	67	
3. Push	0.25	2.75	64.5	66	
3. Pull	-0.25	-2.75	-76.8	91	First horizontal cracks due to flexure, first vertical cracks at the toes
4. Push (a)	0.50	5.50	81.0	83	First diagonal cracks, cracks between inner and outer leaves
4. Pull (a)	-0.50	-5.50	-80.4	95	First diagonal cracks, cracks between inner and outer leaves
4. Push (b)	0.50	5.50	89.7	92	New vertical cracks on a diagonal band
4. Pull (b)	-0.50	-5.50	-81.8	97	New vertical cracks on a diagonal band- North side
4. Push (c)	0.50	5.50	93.0	95	Number and width of cracks increased
4. Pull (c)	-0.50	-5.50	-79.4	94	Number and width of cracks increased
5. Push	0.75	8.25	97.5	100	Maximum load, new diagonal cracks
5. Pull	-0.75	-8.25	-83.7	94	New diagonal cracks
6. Push (a)	1.00	11.00	93.1	96	Increased number of diagonal cracks, signs of toe crushing
6. Pull (a)	-1.00	-11.00	-84.5	100	Increased number of diagonal cracks, signs of toe crushing
6. Push (b)	1.00	11.00	77.3	79	Spalling started at the toe-East side
6. Pull (b)	-1.00	-11.00	-82.1	97	Spalling started at the toe-West side
6. Push (c)	1.00	11.00	75.2	77	Spalling at the toe-East side
6. Pull (c)	-1.00	-11.00	-75.2	89	Spalling at the toe-West side
7. Push	1.25	13.75	77.1	79	Upper diagonal half rocking
7. Pull	-1.25	-13.75	-66.5	79	Upper diagonal half rocking
8. Push	1.50	16.50	67.7	69	Spalling at the toe-East side
8. Pull	-1.50	-16.50	-64.1	76	Spalling at the toe-West side

The M-50-C specimen was further investigated after completion of the test by removing the stone units one by one. Surprisingly, despite the separation of inner and outer leaves, the damage pattern observed among them was almost coinciding. As seen in Figure C.46, damage on the outer leaves were also continuing in the rubble masonry core.

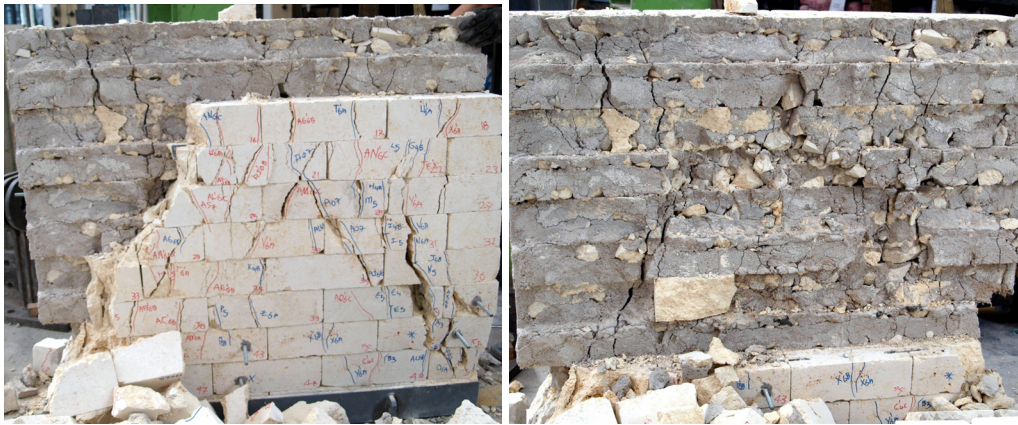


Figure C.46 : Damage on the outer and inner leaves, M-50-C.

The lateral load-displacement hysteresis diagram describing the in-plane response of the M-50-C specimen is presented in Figure C.47. Data points corresponding to first flexural cracking ($H_{f,cr}$), first diagonal cracking ($H_{dia,cr}$), maximum lateral resistance (H_{max}), 20% strength loss ($H_{0.8max}$) and ultimate displacement ($H_{d,max}$), are marked in the figure. The highly dissipative hysteresis loops show that the wall exhibited a significant nonlinear response with large residual displacements after each load reversal. The envelope curve obtained for the M-50-C specimen can be seen in Figure C.48.

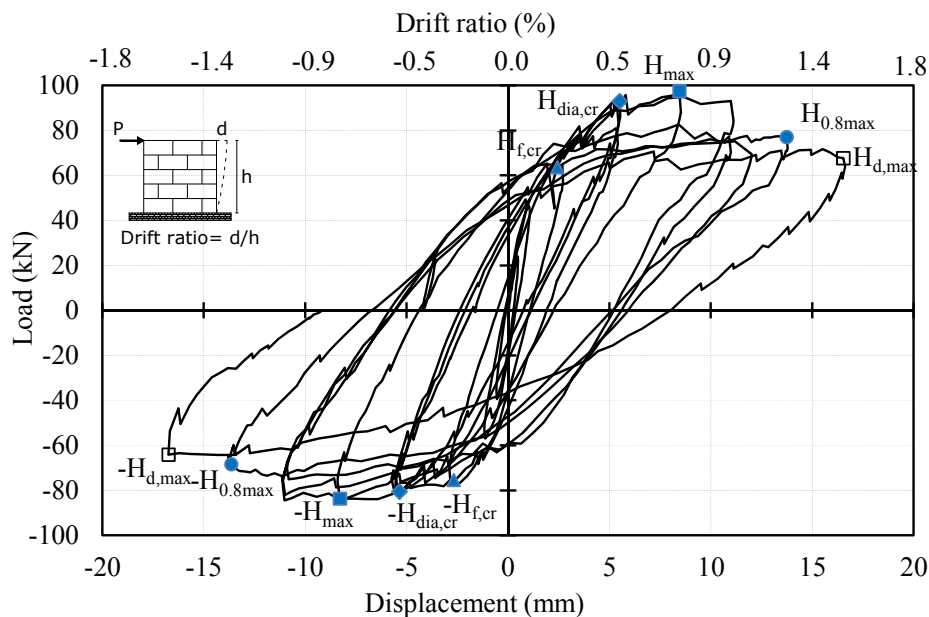


Figure C.47 : Lateral load-displacement diagram for specimen M-50-C.

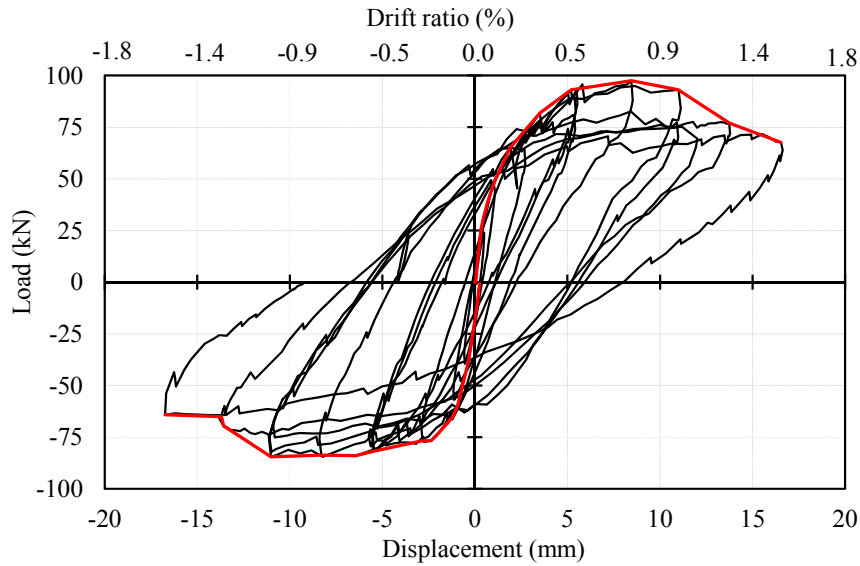


Figure C.48 : Envelope curve of lateral load-displacement diagram of M-50-C.

The variation of the dissipated energy by means of the percentage of the total dissipated energy ($E_{dis,i}/E_{dis,cumulative}$, i denoting the cycle number) corresponding to each loading cycle is presented in Figure C.49. This presentation enables the identification of the critical points corresponding to particular events. A steep increase in the dissipated energy is a sign of increase in the nonlinear deformation, which can be directly associated with the damage growth. In this particular case, the steep increase at Cycle 4 (0.50% drift ratio) corresponds to first diagonal cracks. With further formation of the diagonal cracks and crushing and spalling of the compressed zones, another sudden increase happens at Cycle 6 (1.00% drift ratio).

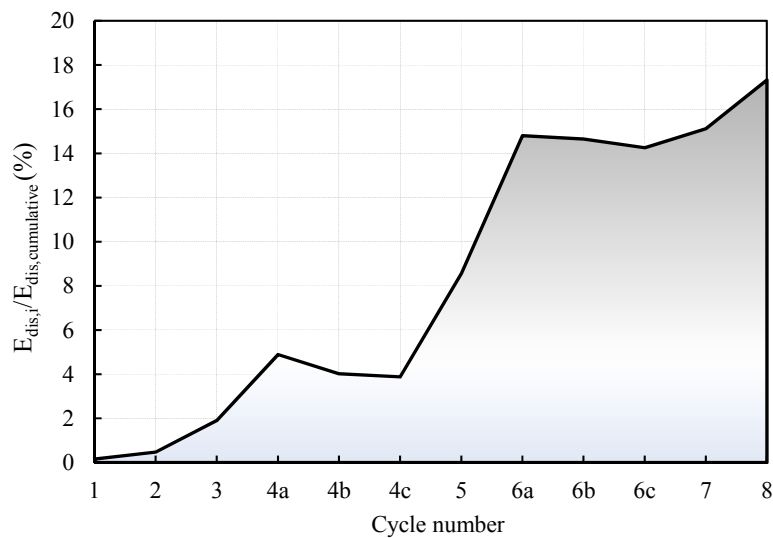


Figure C.49 : Ratio of the dissipated energy to total dissipated energy, M-50-C.

Figure C.50 shows the variation of the vertical displacements measured at the tips of the top bond beam with respect to lateral displacement. As can be observed, the general tendency for compaction is more pronounced for larger lateral displacements.

Vertical deformations measured by channels 36, 37, 40 and 41 (as shown in Figures 3.18 and 3.19) with respect to lateral displacement are presented in Figure C.51. In these figures, positive sign direction of the vertical axis corresponds to the shortening of the measured length. It is interesting to note that even when the LVDTs stayed in the tension zone, the measured total vertical deformation was pointing to an overall compaction.

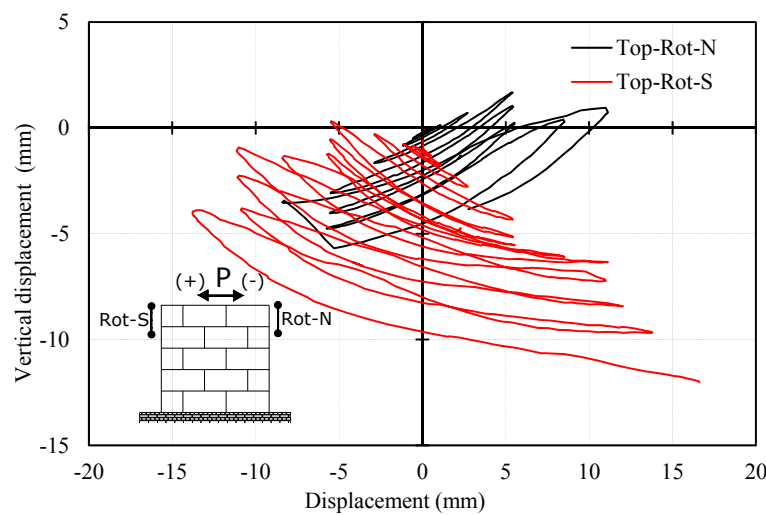


Figure C.50 : Variation of vertical displacements at the bond beam, M-50-C.

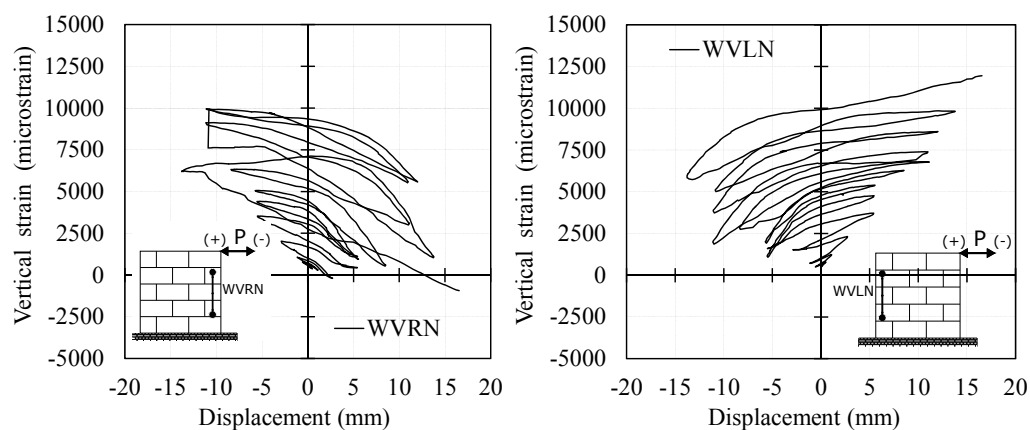


Figure C.51 : Vertical deformations near the Western and Eastern sides, M-50-C.

Vertical deformation measurements at toes and heels (depending on the loading direction) with respect to lateral displacements show that up to 11.0 mm (1.0% drift

ratio) these zones are subjected to cyclic compression and tension deformations, Figure C.52. However, beginning from the 1.0% drift ratio vertical compressive deformations make a sharp increase due to crushing of the stone units at the toes.

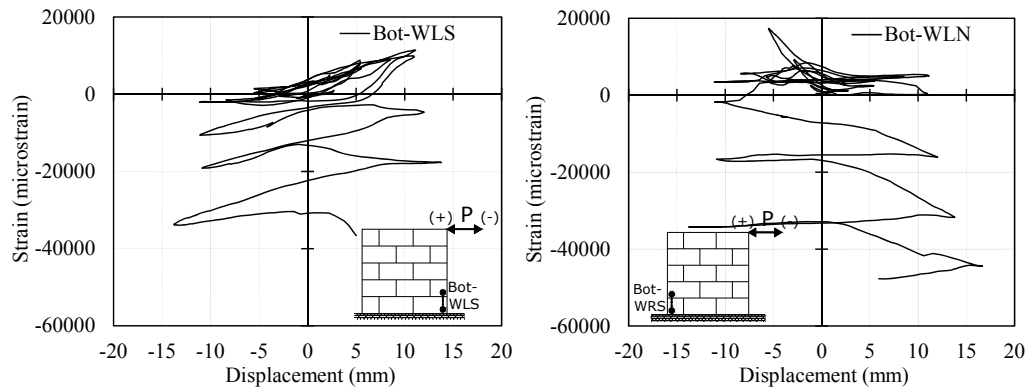


Figure C.52 : Vertical deformations at the toes and heels of the wall M-50-C.

Three of the cramps on the Northern leaf of the wall were instrumented with strain gauges. Measurements logged for the cramp at the lower part of the specimen show that strains could reach 1600 microstrain which are almost 400 microstrain below the yield point of the steel, Figure C.53. The strain diagram turns out to be symmetric until the last loading cycle, which means that when upper side is in compression the lower one is tension and vice versa. Consequently, if the average of these symmetric strains is calculated for each instrumented cramp, an average elongation value less than 500 microstrain is obtained for the duration of the whole test process, as seen in Figure C.54. Similar results were observed for upper and lower instrumented cramps of the specimen.

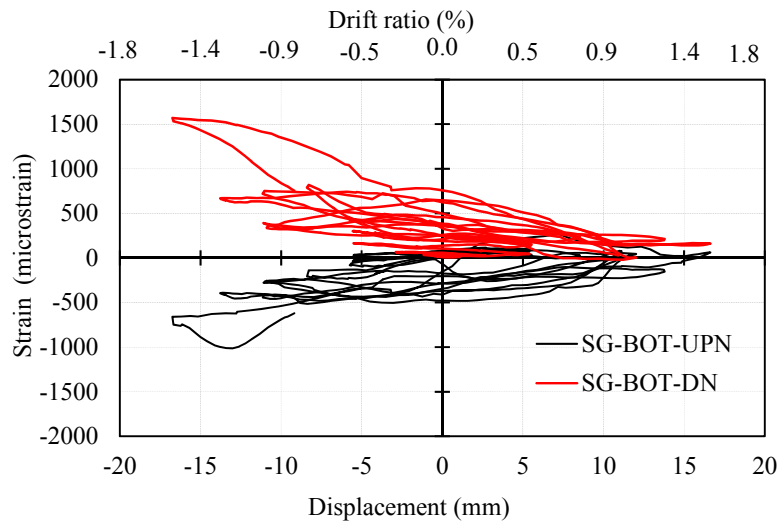


Figure C.53 : Strain variation for lower cramp, M-50-C.

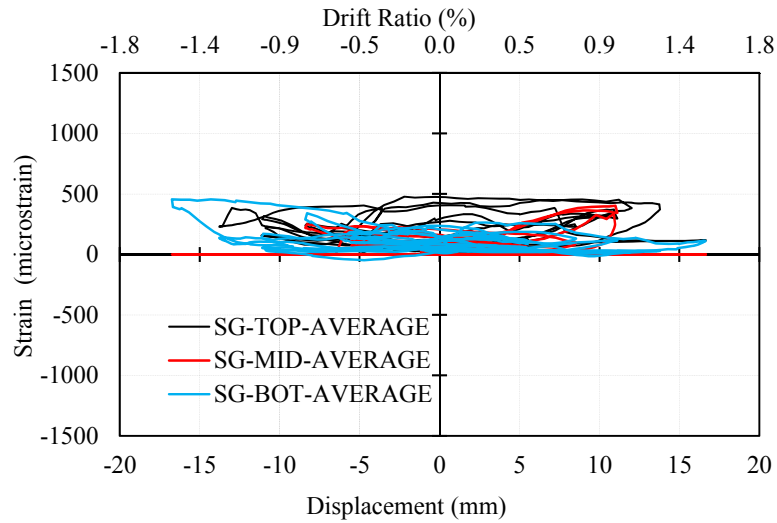


Figure C.54 : Average strains of instrumented cramps of wall M-50-C.

Comments done based on the strain gauge measurements regarding the bending of the cramps can be further backed by the observations carried out after the test. For instance, two different cramps revealed during the after test investigations are presented in Figure C.55. As it is clearly seen, both cramps are bent at mid-length of the steel member. It is also noticeable that the molten lead used for anchoring the reinforcements to stone units are still attached to the members.



Figure C.55 : Bending of the cramps, M-50-C.

Detachment between the external leaves of the wall was measured by two LVDTs positioned at mid-heights of the Eastern and Western faces of the specimen. As clearly seen from the transverse displacement versus lateral displacement diagram (Figure C.56) obtained for the Western side, particularly after 8.25 mm lateral displacement (0.75% drift ratio) separation between the outer and inner leaves increased rapidly.

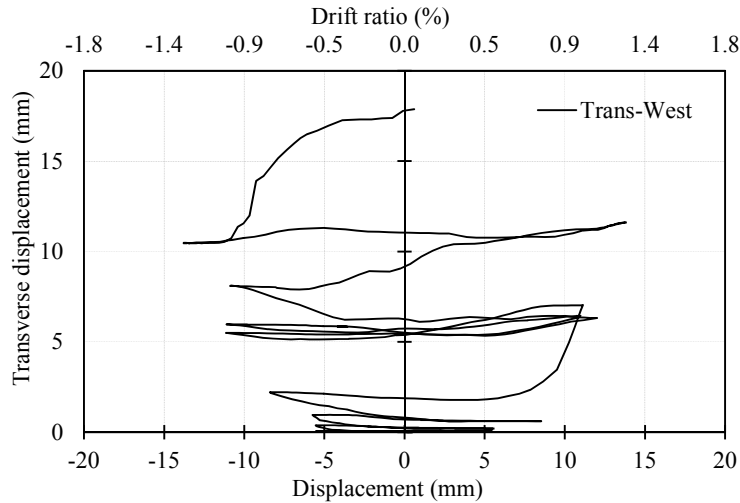


Figure C.56 : Detachment between the exterior leaves (Western side, M-50-C).

C.3 M-75-C Wall

The M-75-C specimen consisted of two ashlar external leaves and a rubble masonry core. Cramps were positioned in the horizontal dry joints of the wall. Vertical pre-compression stress level was kept as 0.75 MPa during the test. The test was conducted 129 days after the production of the M-75-C wall.

C.3.1 Initial compression test (M-75-C)

Three successive axial loading and unloading cycles were applied prior to cyclic lateral loading. The target vertical load level was 270 kN that corresponded to 0.75 MPa axial stress. The axial stress-axial strain diagrams of these three cycles plotted by using the average of the measurements of the six LVDTs (with an approximate gauge length of 800 mm) on both sides of the specimen are presented in Figure C.57.

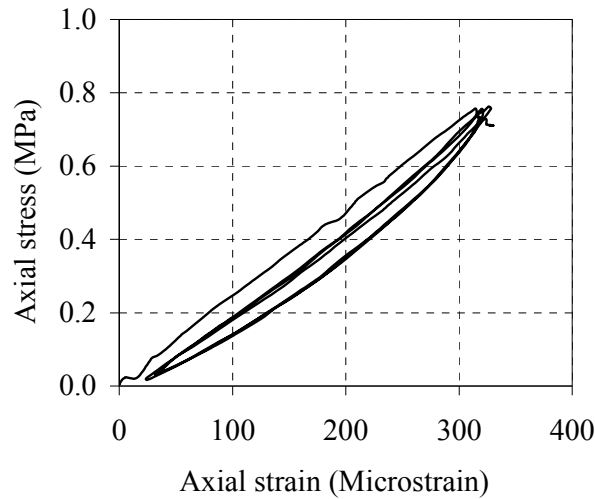


Figure C.57 : Axial stress-axial strain behaviour of M-75-C specimen.

C.3.2 Shear compression test (M-75-C)

The M-75-C specimen, which was subjected to axial stress level of 0.75 MPa, exhibited a shear governed response during the application of in-plane lateral displacement cycles. After performing the pre-loading session that consisted of three successive axial loading cycles, all channels were initialized and hydraulic power supply of the lateral actuator was activated. The lateral loading began after applying the targeted 0.75 MPa axial stress.

Up to 0.25% drift ratio (2.75 mm displacement), no damage could be observed on the Northern and Southern sides of the specimen. However, vertical cracks at the rubble masonry and the rubble core-stone unit interfaces appeared due to higher level of axial load, Figures C.58 and C.59.

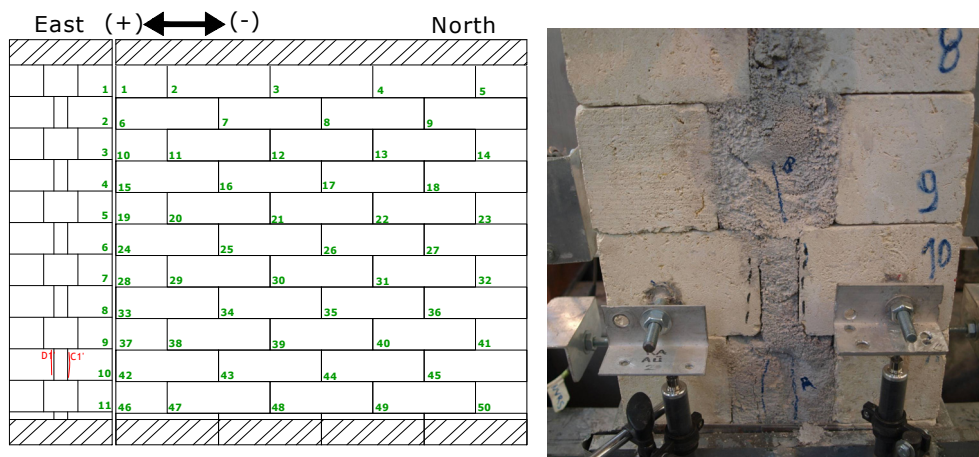


Figure C.58 : Cycle 1, M-75-C after 0.55 mm displacement (0.05% drift ratio).

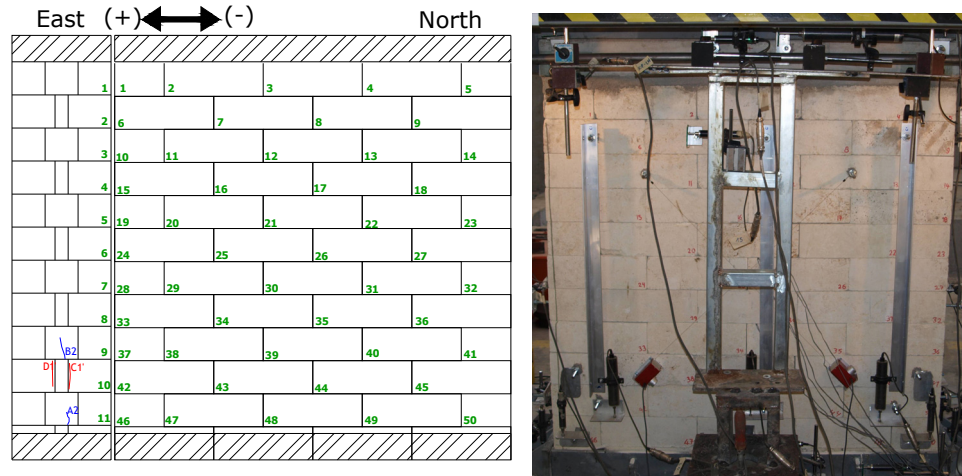


Figure C.59 : Cycle 2, M-75-C after 1.10 mm displacement (0.10% drift ratio).

At the pushing and pulling parts of the 2.75 mm (0.25% drift ratio) cycle, first horizontal cracks near the foundation level of the Western and Eastern sides could be observed, Figure C.60. These flexure based horizontal cracks were only visible on the rubble masonry core. In addition to that, vertical cracks between the interfaces of outer leaves and the rubble core could be seen.

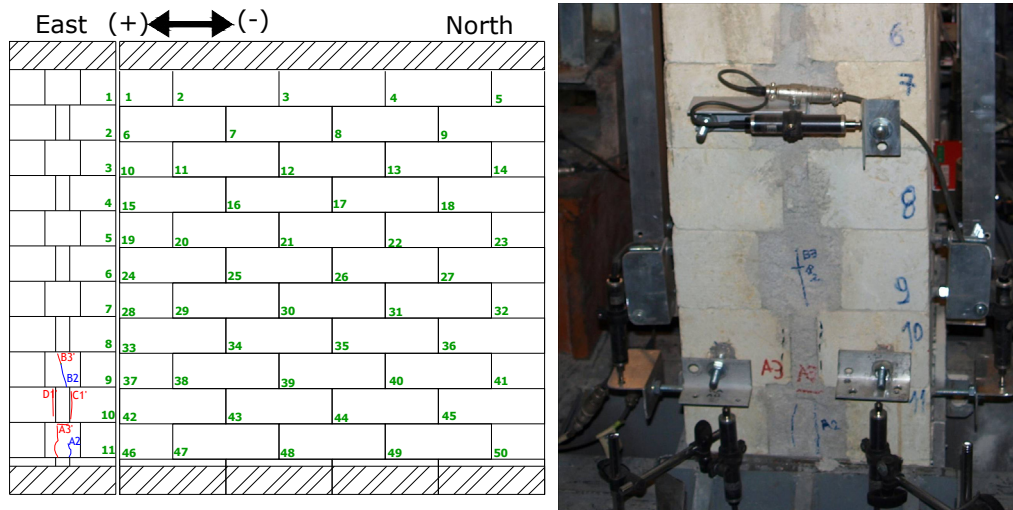


Figure C.60 : Cycle 3, M-75-C after 2.75 mm displacement (0.25% drift ratio).

During the first, second and third 5.50 mm cycles (0.50% drift ratio, cycles 4a, 4b and 4c), horizontal flexure cracks, previously seen on the rubble masonry, could be observed between the first and second stone courses of Eastern side and at the foundation level of Western side, Figures C.61-C.63.

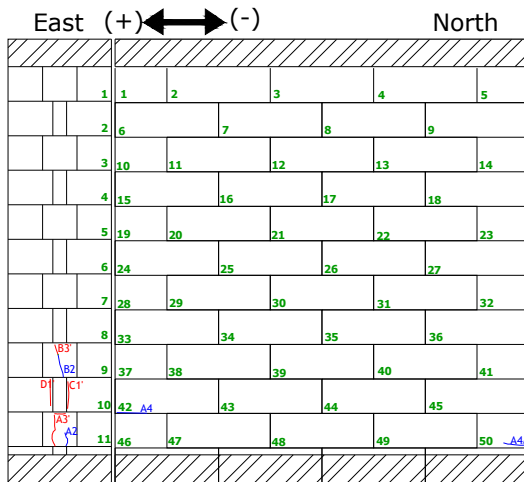


Figure C.61 : Cycle 4a, M-75-C after 5.50 mm displacement (0.50% drift ratio).

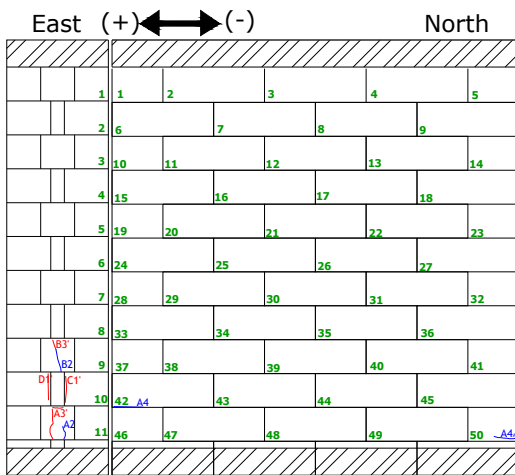


Figure C.62 : Cycle 4b, M-75-C after 5.50 mm displacement (0.50% drift ratio).

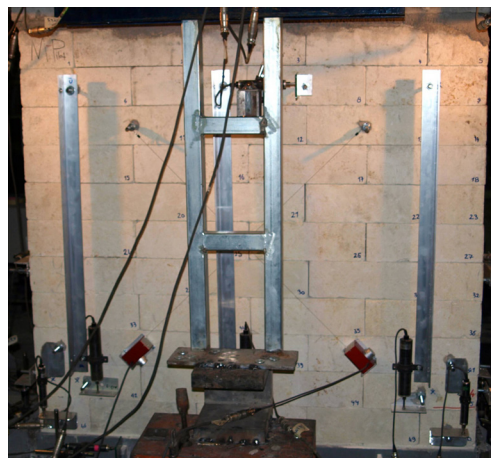
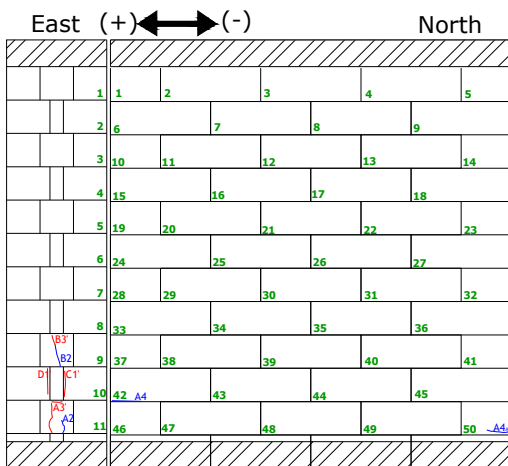


Figure C.63 : Cycle 4c, M-75-C after 5.50 mm displacement (0.50% drift ratio).

During cycle 5, targeting 8.25 mm horizontal displacement (0.75% drift ratio), vertical cracks passing through the stone units started to appear at the compressed parts of the specimen, Figure C.64. On the Western face of the wall, vertical separation cracks between the inner and outer leaves extended up to 2/3 of the specimen height.

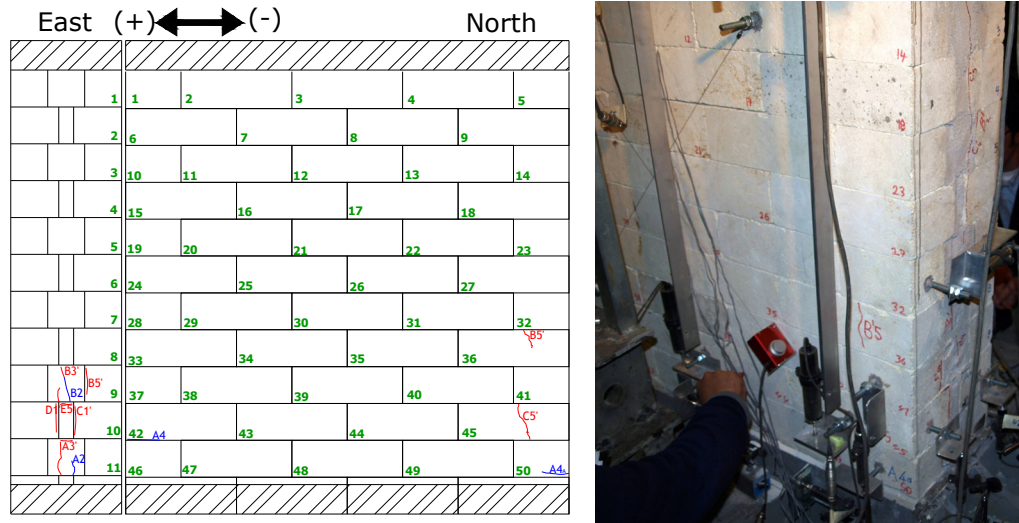
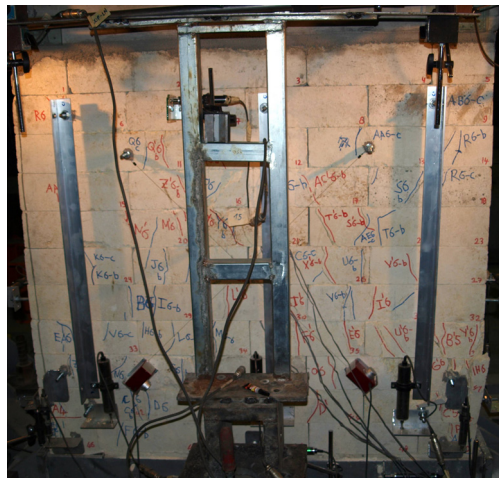
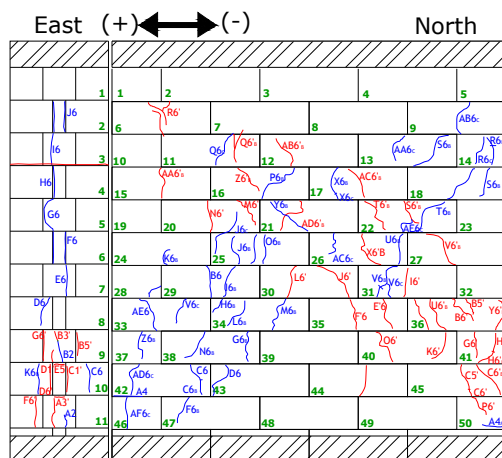
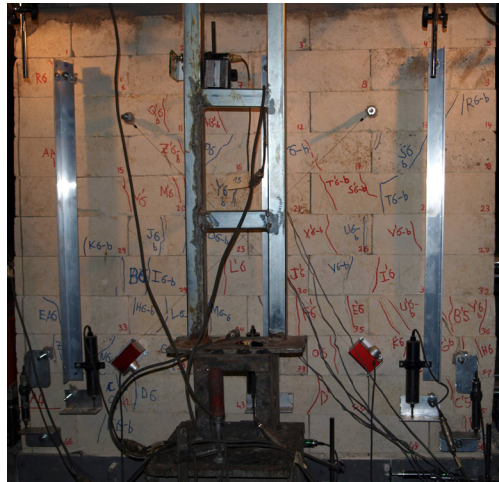
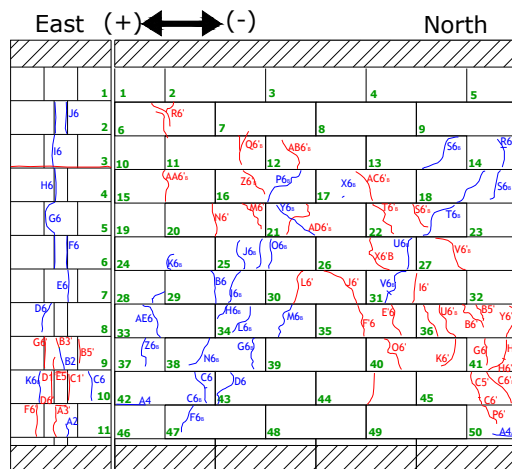
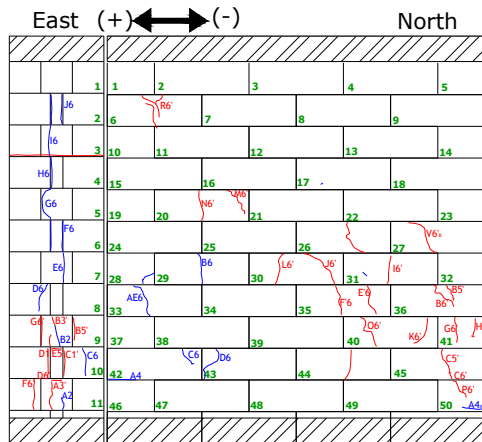


Figure C.64 : Cycle 5, M-75-C after 8.25 mm displacement (0.75% drift ratio).

During the first 11.0 mm cycle (1.0% drift ratio, cycle 6a), several diagonal cracks passing through the stone units appeared, Figure C.65. Damage, consisting of vertical cracks and crushing of the stone units, increased rapidly around the toes of the wall.

The second and third cycles of the 1.0% drift ratio further extended the damage, Figures C.66 and C.67. After each successive loading cycle, number and width of the cracks increased gradually. Though the mechanical properties of limestone and rubble masonry were very different, vertical cracks visible on the Eastern and Western sides that were generally running through the rubble core could continue on the indentations of stone units, Figure C.68 . Crushing of the stone units around the toes of the wall was followed by initiation of spalling, Figure C.69.



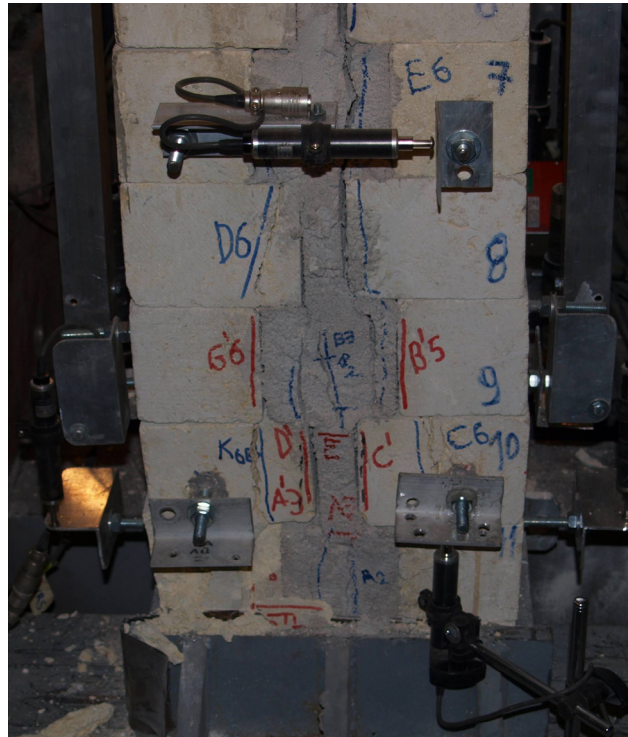


Figure C.68 : Cycle 6c, Eastern side of M-75-C (11.0 mm, 1.00% drift ratio).

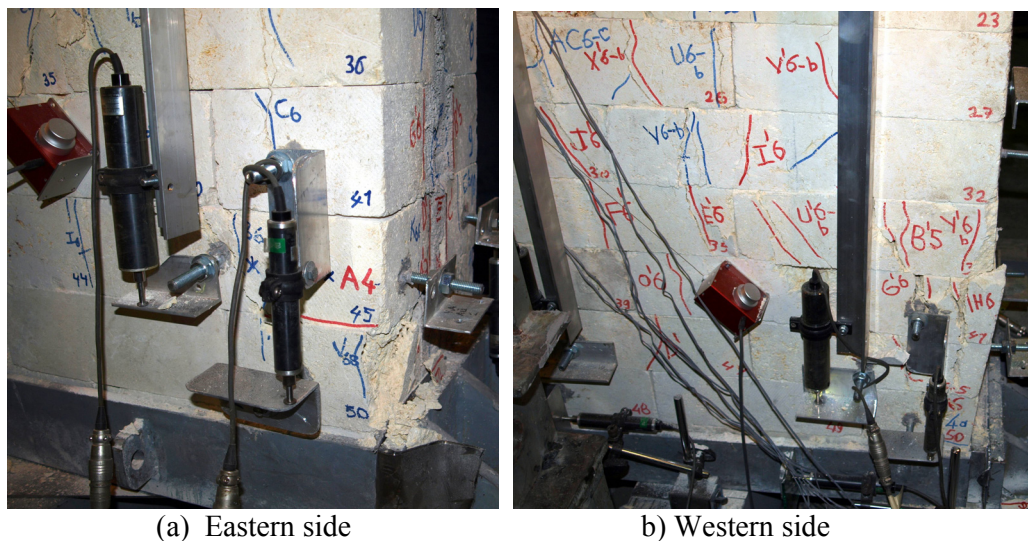


Figure C.69 : Cycle 6c, crushing of stone units after 1.0% drift ratio (M-75-C).

The cracking and spalling of stone units continued during the 13.75 mm displacement cycle (cycle 7, corresponding to 1.25% drift ratio), Figure C.70. Severe crushing and spalling of stone units were observed around the toes.

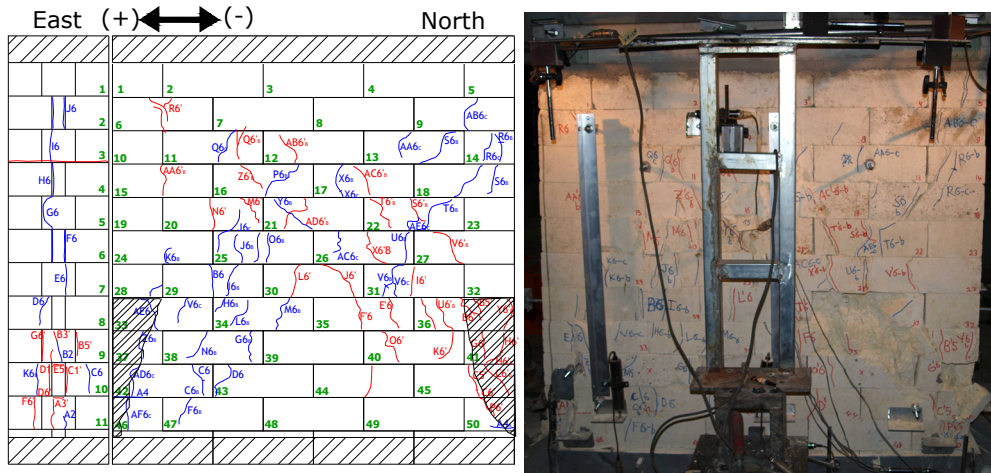


Figure C.70 : Cycle 7, M-75-C after 13.75 mm displacement (1.25% drift ratio).

After completing the pushing and pulling parts of the 13.75 mm (1.25% drift ratio) cycle, the test was stopped due to heavy damage which might cause total loss of stability. After test views of the North, South, East and West faces of the specimen are presented in Figures C.71- C.73. If the damage distribution on the Northern and Southern faces are compared, one can notice that cracks on the Northern face are more dispersed than the ones on the Southern face. However, cracks on the Southern face are concentrated along certain paths and they have much larger crack widths.

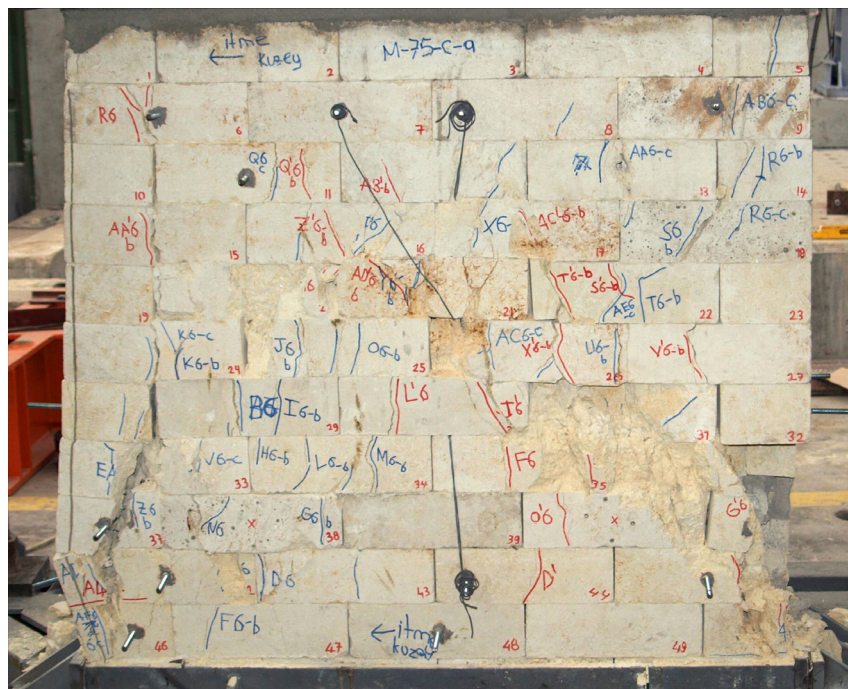


Figure C.71 : Cycle 7, North face of M-75-C (13.75 mm, 1.25% drift ratio).



Figure C.72 : Cycle 7, South face of M-75-C (13.75 mm, 1.25% drift ratio).



(a) Eastern side



(b) Western side

Figure C.73 : Cycle 7, (a) East and (b) West faces of M-75-C after 1.25% drift ratio.

Drift ratios, displacements, peak loads, peak load to maximum strength ratios (Peak Load/ $H_{\max}$) and notes on damage development corresponding to each performed cycle are summarized in Table C.3. The damage evolution started with the vertical cracks in the rubble masonry core at 0.10% drift ratio. At 0.25% drift ratio, horizontal bending cracks appeared near the foundation level and could only be observed on the rubble masonry. During the successive cycles of 0.50% drift ratio, these horizontal cracks could be seen on the stone masonry bed joints. Vertical cracks around the compressed toes of the wall started to appear at 0.75% drift ratio and followed by diagonal cracks passing through the stone units (especially at 1.00% drift ratio). Cracks passing through the stone units were generally concentrated at mid-length of the units and the points where the cramps were bent and anchored to the stone units. A typical sample of these crack concentrations at the cramp tips is presented in Figure C.74. After formation of the diagonal crack bands, upper diagonal halves of the wall started rocking. The test was stopped at 1.25% drift ratio after extensive spalling of the compressed parts that caused significant and rapid loss of strength.



Figure C.74 : Damage in the vicinity of cramp anchors, M-75-C.

Table C.3 : M-75-C specimen summary table.

Cycle	Drift ratio (%)	Displacement (mm)	Peak Load (kN)	Peak Load/ H_{max} (%)	Note
1. Push	0.05	0.55	43.7	33	
1. Pull	-0.05	-0.55	-34.0	30	
2. Push	0.10	1.10	68.5	52	Vertical cracks at rubble masonry, Eastern side
2. Pull	-0.10	-1.10	-56.4	49	Vertical cracks at rubble masonry, Western side
3. Push	0.25	2.75	103.3	79	First horizontal cracks at rubble core due to flexure, Western side
3. Pull	-0.25	-2.75	-90.2	79	First horizontal cracks at rubble core due to flexure, Eastern side
4. Push (a)	0.50	5.50	124.3	95	First horizontal cracks visible at outer leaves, Western side
4. Pull (a)	-0.50	-5.50	-81.6	71	First horizontal cracks visible at outer leaves, Eastern side
4. Push (b)	0.50	5.50	118.0	90	
4. Pull (b)	-0.50	-5.50	-85.8	75	
4. Push (c)	0.50	5.50	119.8	91	
4. Pull (c)	-0.50	-5.50	-87.7	77	
5. Push	0.75	8.25	119.6	91	First vertical cracks at the toe
5. Pull	-0.75	-8.25	-113.7	99	First vertical cracks at the toe
6. Push (a)	1.00	11.00	116.3	89	First diagonal cracks, signs of toe crushing
6. Pull (a)	-1.00	-11.00	-108.1	95	First diagonal cracks, signs of toe crushing
6. Push (b)	1.00	11.00	119.7	91	Crushing started at the toe-East side, diagonal cracks extended
6. Pull (b)	-1.00	-11.00	-95.2	83	Crushing started at the toe-West side, diagonal cracks extended
6. Push (c)	1.00	11.00	107.8	82	Spalling at the toe-East side
6. Pull (c)	-1.00	-11.00	-85.0	74	Spalling at the toe-West side
7. Push	1.25	13.75	82.1	63	Heavy damage, rapid loss of strength
7. Pull	-1.25	-13.75	-56.2	49	Heavy damage, rapid loss of strength

The lateral load-displacement hysteresis diagram describing the in-plane response of the M-75-C specimen is presented in Figure C.75. Data points corresponding to first flexural cracking ($H_{f,cr}$), first diagonal cracking ($H_{dia,cr}$), maximum lateral resistance (H_{max}) and ultimate displacement ($H_{d,max}$), are marked in the figure. Maximum load capacities of the M-75-C specimen are attained while pushing to target displacement of 8.25 mm (0.75% drift ratio) and pulling to -8.25 mm (0.75% drift ratio). Maximum load capacity is 131.3 kN for the pushing direction while it is -114.3 kN for the pulling direction. The envelope curve obtained for the M-75-C specimen can be seen in Figure C.76.

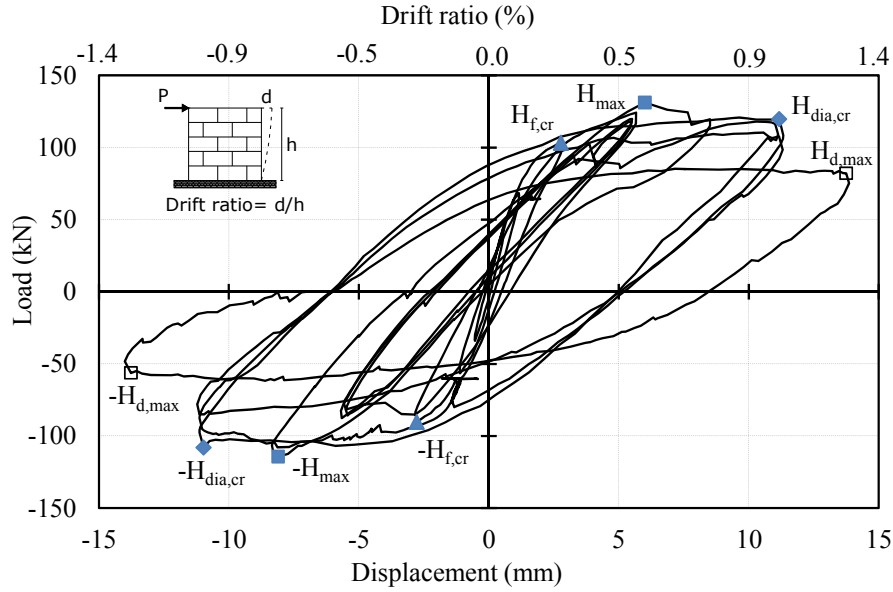


Figure C.75 : Lateral load-displacement diagram for specimen M-75-C.

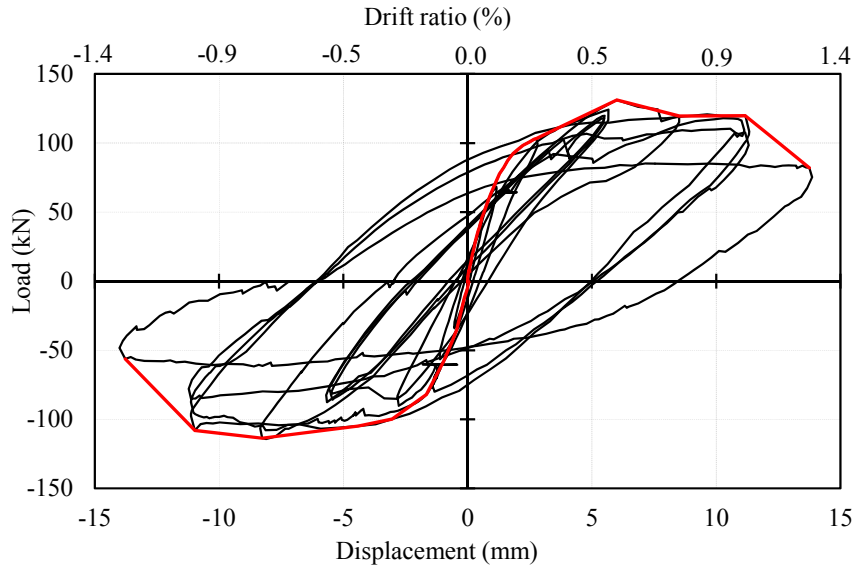


Figure C.76 : Envelope curve of lateral load-displacement diagram of M-75-C.

The highly dissipative hysteresis loops show that the wall exhibited a significant nonlinear response with large residual displacements after each load reversal. The variation of the dissipated energy by means of the percentage of the total dissipated energy ($E_{dis,i}/E_{dis,cumulative}$, i denoting the cycle number) corresponding to each loading cycle is presented in Figure C.77. In this figure, the steep increase at Cycle 3 (0.25% drift ratio) corresponds to first horizontal flexure cracks. The second step increase in dissipated energy occurs between the 5th and 6th cycles where the first diagonal

cracks and toe crushing initiates. With occurrence of the diagonal cracks and crushing and spalling of the compressed zones, another sudden increase happens at Cycle 6b (1.00% drift ratio).

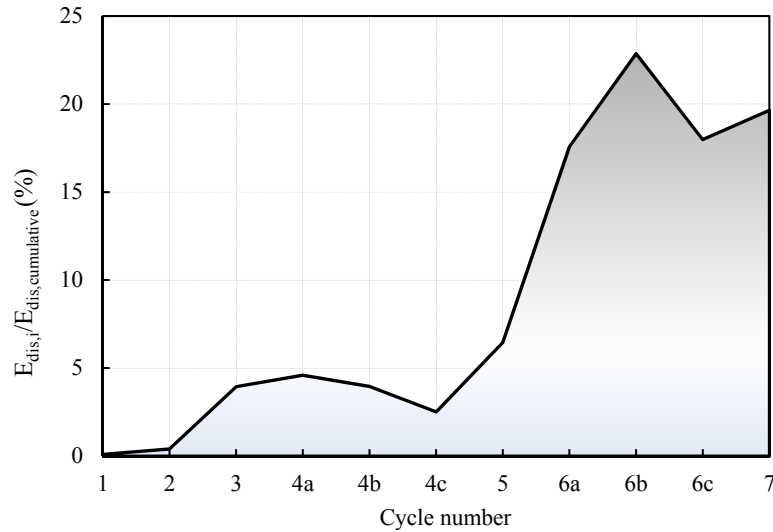


Figure C.77 : Ratio of the dissipated energy to total dissipated energy, M-75-C.

Vertical deformations measured by channels 36, 37, 40 and 41 (as shown in Figures 3.18 and 3.19) with respect to lateral displacement are presented in Figure C.78. In these figures, positive sign direction of the vertical axis corresponds to the shortening of the measured length. It is interesting to note that, except the first a few cycles, even when the LVDTs stayed in the tension zone, the measured total vertical deformation was generally pointing to an overall compaction.

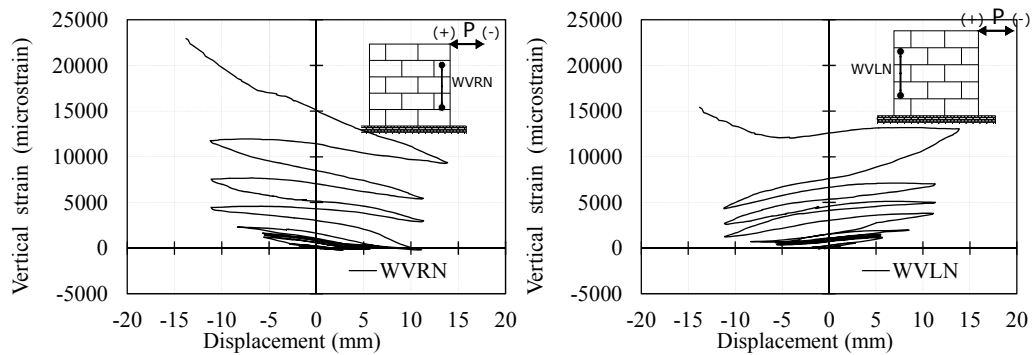


Figure C.78 : Vertical deformations near the Western and Eastern sides, M-75-C.

Vertical deformation measurements at toes and heels (depending on the loading direction) with respect to lateral displacements show that up to 11.0 mm (1.0% drift ratio) these zones are subjected to cyclic compression and tension deformations,

Figure C.79. However, beginning from the 1.0% drift ratio vertical compressive deformations make a sharp increase due to crushing of the stone units at the toes.

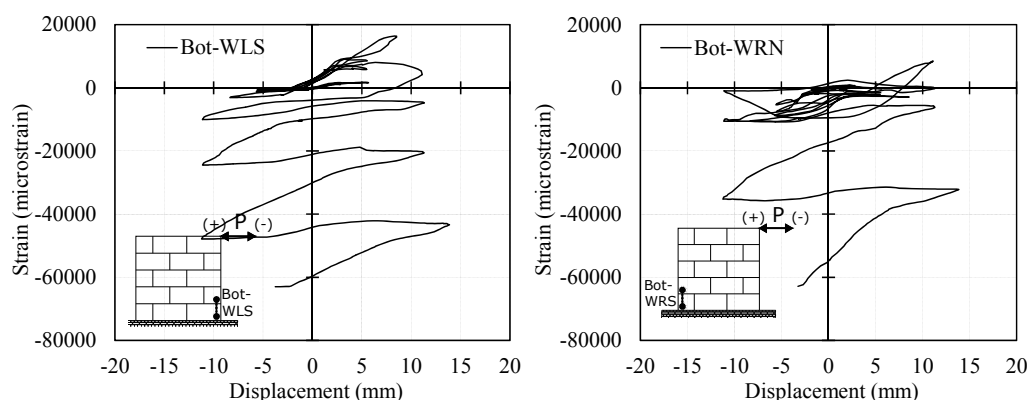


Figure C.79 : Vertical deformations at the toes and heels of the wall M-75-C.

Three of the cramps on the Northern leaf of the wall were instrumented with strain gauges. Measurements logged for the cramp at the mid-height of the specimen show that strains could reach 2000 microstrain which is very close to the yield point of the steel, Figure C.80. However, the strain diagram turns out to be symmetric until the last loading cycle, which means that when upper side is in compression the lower one is tension and vice versa. Consequently, if the average of these symmetric strains is calculated for each instrumented cramp, an average elongation value less than 300 microstrain is obtained for the duration of the whole test process, as seen in Figure C.81. Similar results were observed for upper and lower instrumented cramps of the specimen.

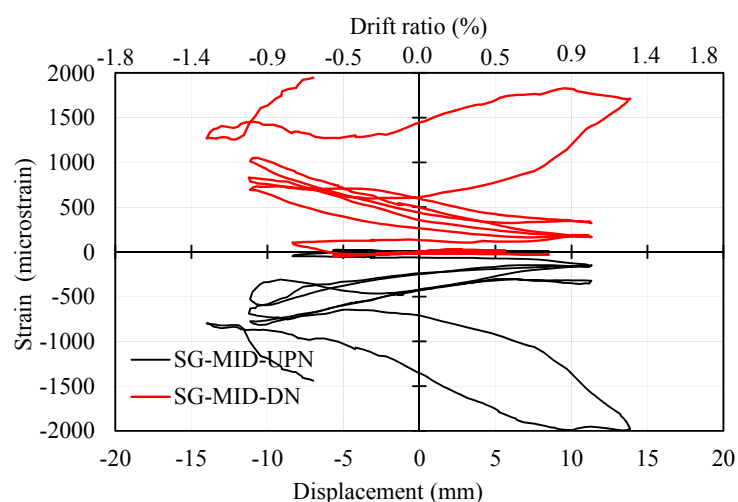


Figure C.80 : Strain variation for the cramp at mid-height of the wall, M-75-C.

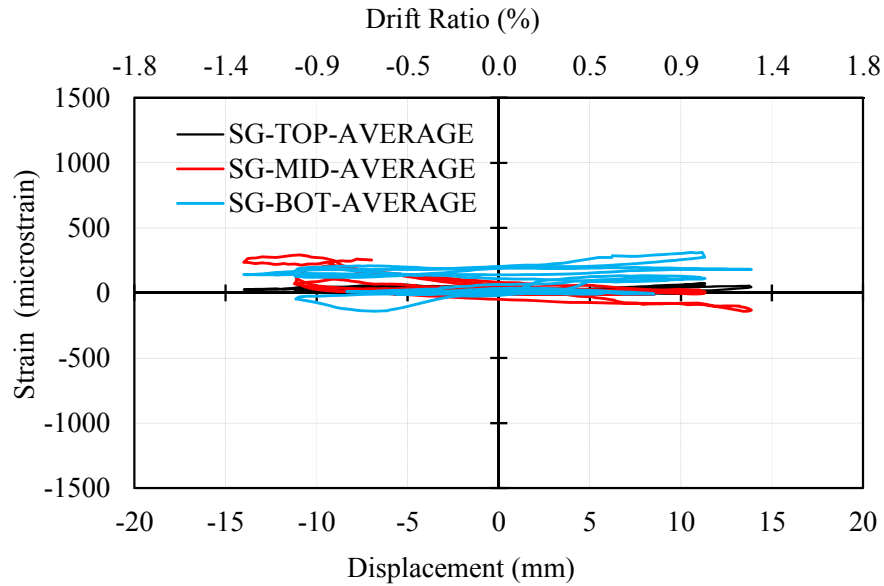


Figure C.81 : Average strains on instrumented cramps of wall M-75-C.

Transverse displacement between the exterior leaves of the wall was measured by two LVDTs positioned at mid-heights of the Eastern and Western faces. As clearly seen from the transverse displacement versus lateral displacement diagram (Figure C.82) obtained for the Western side, particularly after 2.75 mm lateral displacement (0.25% drift ratio) separation between the outer and inner leaves increased rapidly.

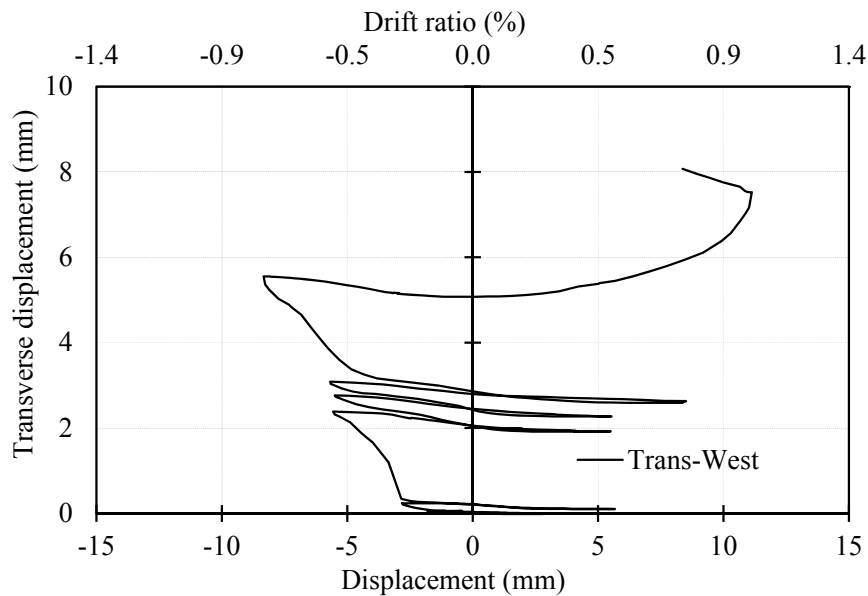


Figure C.82 : Detachment between the external leaves of M-75-C.

C.4 M-100-C Wall

The M-100-C specimen consisted of two ashlar external leaves and a rubble masonry core. Cramps were positioned in the horizontal dry joints of the wall. Vertical pre-compression stress level was kept as 1.0 MPa during the test. The test was conducted 165 days after the production of the M-100-C wall.

C.4.1 Initial compression test (M-100-C)

Three successive vertical loading and unloading cycles were applied prior to reversed cyclic lateral loading. The target vertical load level was 360 kN that corresponded to 1.0 MPa vertical stress. The axial stress-axial strain diagrams of these three cycles plotted by using the average of the measurements of the six LVDTs (with an approximate gauge length of 800 mm) on both sides of the specimen are presented in Figure C.83.

As also seen in Figure C.83, the first applied axial loading cycle caused a high amount of plastic deformation in the axial direction. During the application of the axial load, at about 330 kN (approximately 0.90 MPa axial stress), suddenly several vertical cracks occurred in the rubble masonry core that could be observed on the Eastern and Western sides of the wall. In addition to that, randomly distributed vertical cracks on the stone units could also be observed on the Northern and Southern sides, Figure C.84.

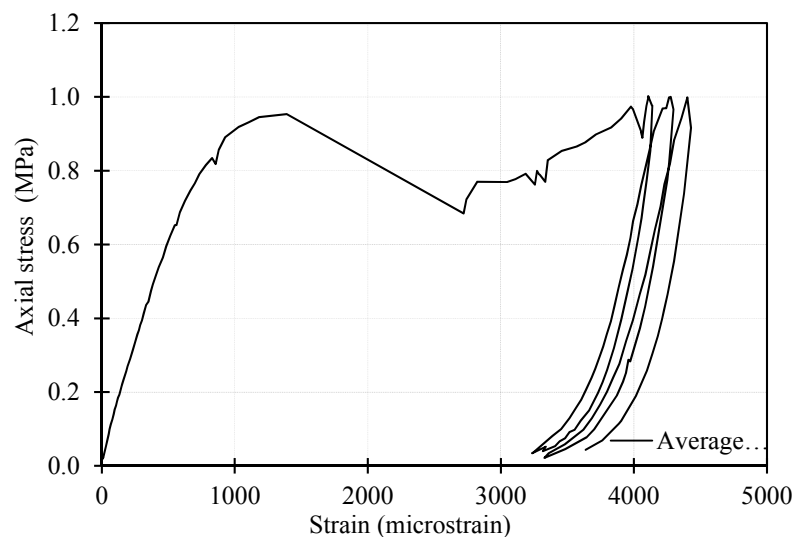


Figure C.83 : Axial stress-axial strain behaviour of M-100-C specimen.



Figure C.84 : Observed damage during the axial loading of M-100-C specimen.

C.4.2 Shear compression test (M-100-C)

The M-100-C specimen, which was subjected to axial stress level of 1.0 MPa (highest axial stress level of this study), exhibited a shear governed response during the application of in-plane lateral displacement cycles. After performing the pre-loading session that consisted of three successive axial loading cycles, all channels were initialized and hydraulic power supply of the lateral actuator was activated. The lateral loading began after applying the targeted 1.0 MPa axial stress.

At 0.55 and 1.10 mm displacement cycles (0.05 and 0.10% drift ratios, respectively), new vertical cracks, in addition to pre-damage cracks left from the axial loading, appeared on the Northern and Southern faces of the wall, Figures C.85-C.87. In contrary to previous specimens these cracks were not accumulated at the toes but randomly distributed on the wall. At 0.10% drift ratio, maximum crack width could reach 0.5 mm width.

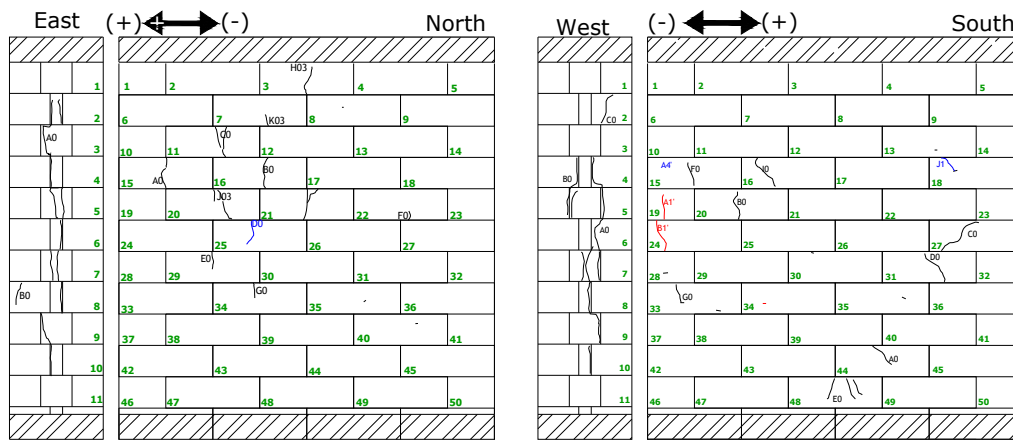


Figure C.85 : Cycle 1, M-100-C after 0.55 mm displacement (0.05% drift ratio).

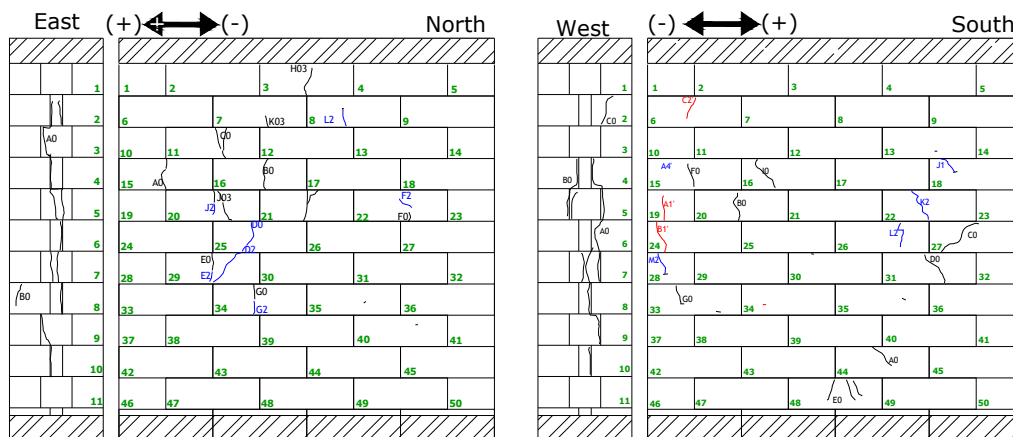


Figure C.86 : Cycle 2, M-100-C after 1.10 mm displacement (0.10% drift ratio).



Figure C.87 : Cycle 2, Southern face of M-100-C after 0.10% drift ratio.

During the pushing and pulling components of the 2.75 mm (0.25% drift ratio) cycle, number and width of the cracks increased visibly, particularly on the Northern face, Figures C.88 and C.89. Those cracks, whose crack width could reach 1.0 mm, were not accumulated at the compressed toes of the wall, but distributed among the Southern and Northern faces. The random distribution of cracks, which occurred as a result of bending effects on the limestone units, was mainly due to imperfections between the units. Under high axial stresses these imperfections caused bending effects on the stone units.

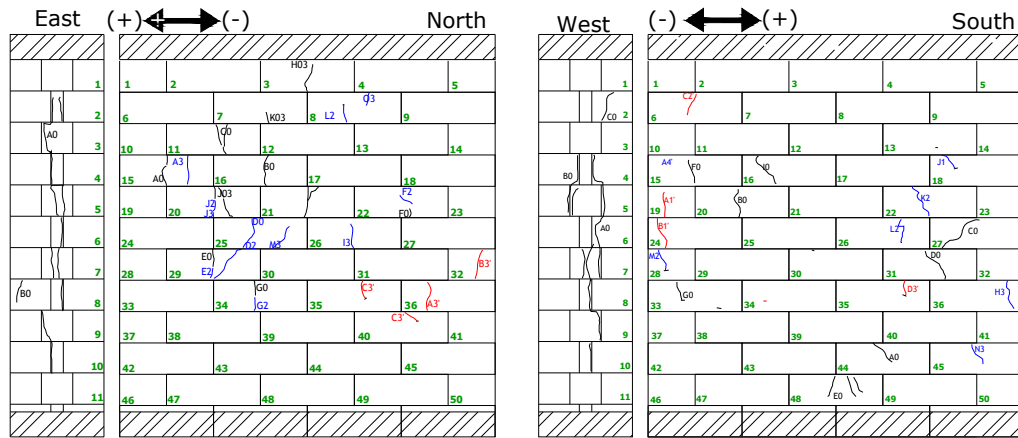


Figure C.88 : Cycle 3, M-100-C after 2.75 mm displacement (0.25% drift ratio).

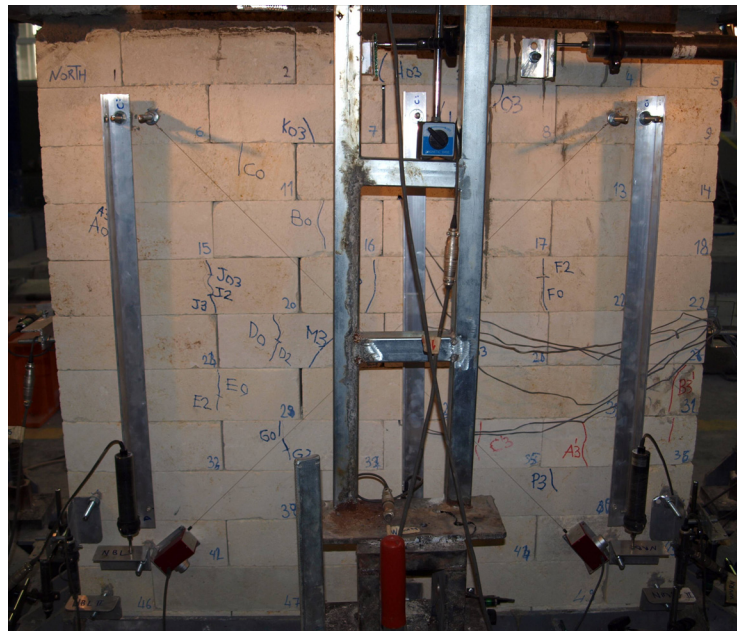


Figure C.89 : Cycle 3, Northern face of M-100-C after 0.25% drift ratio.

360

During the second 5.50 mm cycle (0.50% drift ratio, cycle 4b), number of diagonal cracks increased marginally, Figure C.92. An out-of-plane movement of stone units was clearly visible on the Northern face of the specimen.

During the third cycle of 5.50 mm (0.50% drift ratio, cycle 4c) number of vertical and diagonal cracks increased, especially at the regions close to the Western side of the Southern face, Figures C.93 and C.94. Observed crack widths could reach up to 5 mm. Crushing of the toes was followed by spalling of small stone pieces. Separation between the external leaves reached values more than 10 mm, consequently, the bulging at mid-height was easily perceptible.

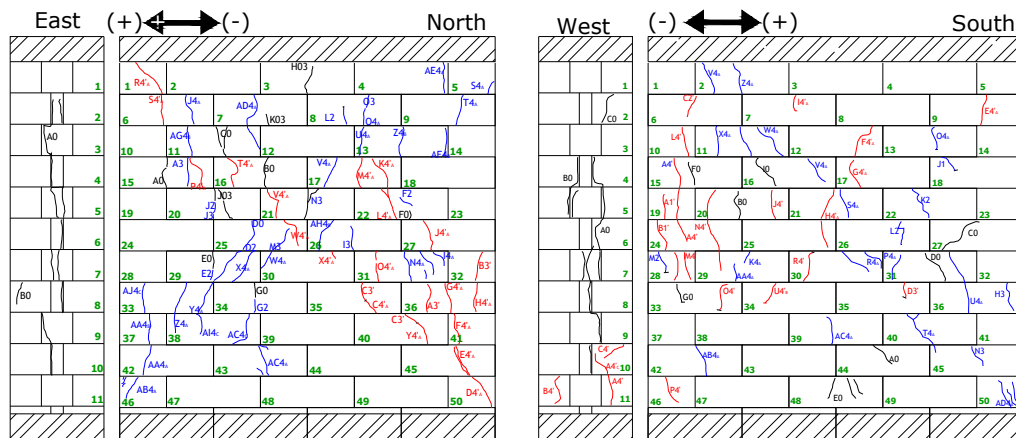


Figure C.92 : Cycle 4b, M-100-C after 5.50 mm displacement (0.50% drift ratio).

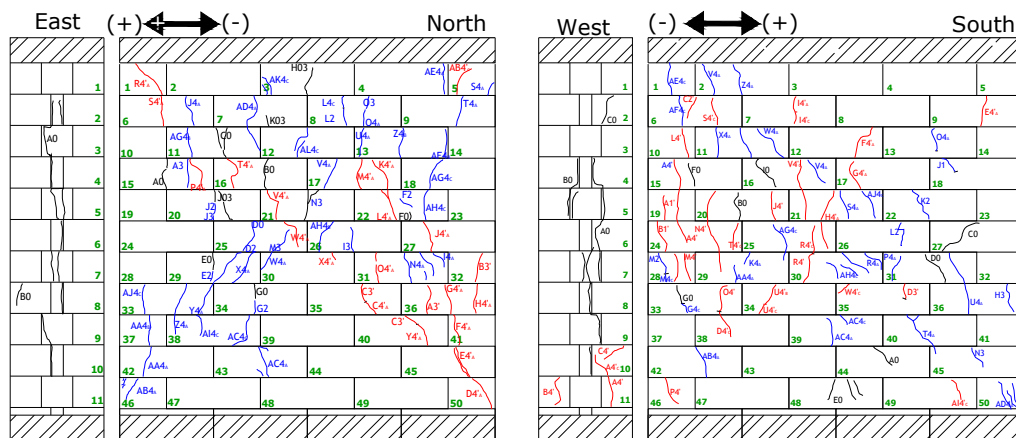


Figure C.93 : Cycle 4c, M-100-C after 5.50 mm displacement (0.50% drift ratio).



Figure C.94 : Cycle 4c, Southern face of M-100-C after 0.50% drift ratio.

Crushing and spalling of the toes continued during the 5th cycle, which targeted 8.25 mm horizontal displacement (0.75% drift ratio), Figure C.95. In addition to toe crushing, on the Southern face of the specimen, vertical cracks passing through cramp ends and mid-length of the stone units extended along the wall height and formed a column, Figures C.96 and C.97.



Figure C.95 : Cycle 5, toe crushing and spalling of M-100-C after 0.75% drift ratio.

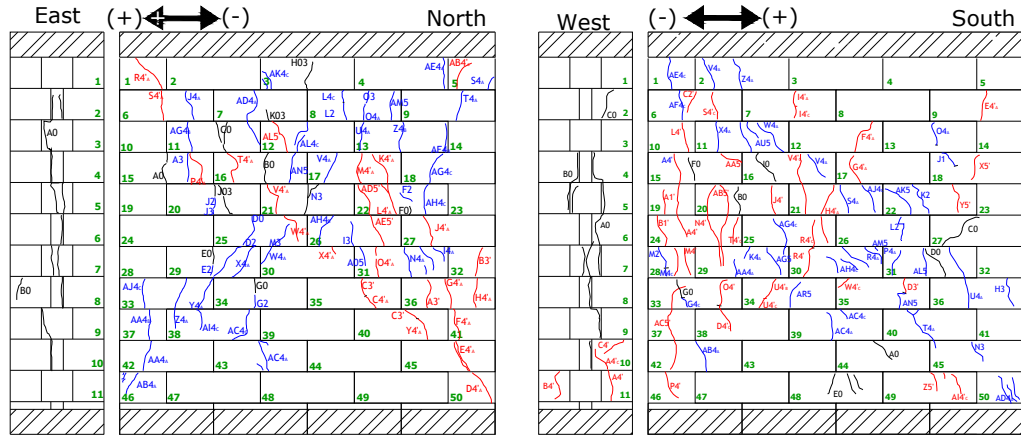


Figure C.96 : Cycle 5, M-100-C after 8.25 mm displacement (0.75% drift ratio).

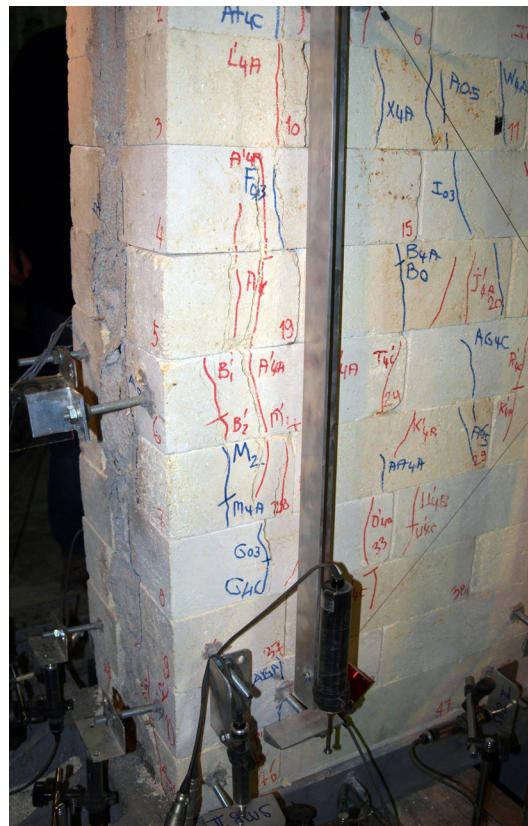


Figure C.97 : Vertical cracks on the Southern face of M-100-C (0.75% drift ratio).

During the first 11.0 mm cycle (1.0% drift ratio, cycle 6a), apart from several diagonal and vertical cracks (as seen in Figure C.98), damage evolved with crushing and spalling of the stone units not only at the toes, but also along the diagonal crack bands (particularly on the Northern face). Moreover, as seen in Figure C.99, vertical column formed on the Southern face buckled under compression.

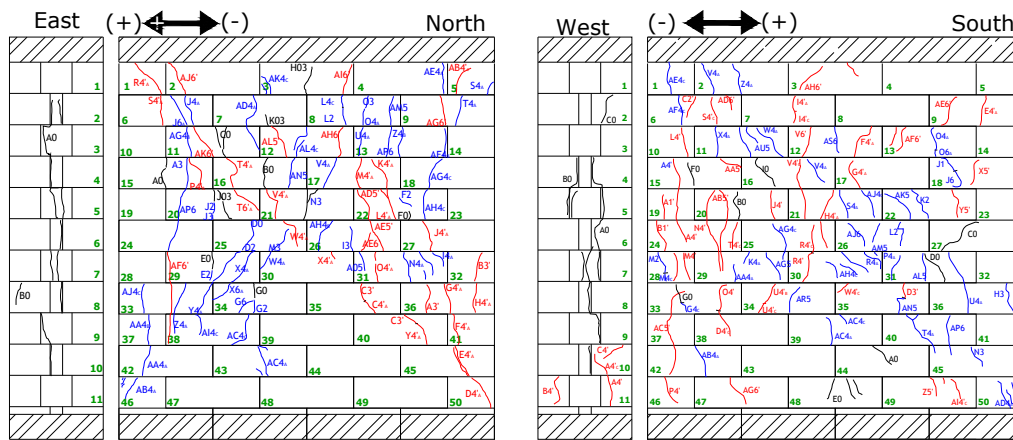


Figure C.98 : Cycle 6a, M-100-C after 11.0 mm displacement (1.00% drift ratio).



Figure C.99 : Cycle 6a, Buckling of the vertical column after 1.00% drift ratio.

After completing the pushing part of the 11.0 mm (1.0% drift ratio) cycle, the test was stopped due to extensive damage and drop in the load bearing capacity. After test views of the North, South, East and West faces of the specimen are presented in Figures C.100-C.102, respectively.



Figure C.100 : Northern face of M-100-C specimen after the test.



Figure C.101 : Southern face of M-100-C specimen after the test.



Figure C.102 : (a) East and (b) West faces of M-100-C after the test.

Drift ratios, displacements, peak loads, peak load to maximum strength ratios (Peak Load/ H_{max}) and notes on damage development corresponding to each performed cycle are summarized in Table C.4. Load capacities of the M-100-C specimen were attained at displacements of 3.90 mm and -3.29 mm while travelling to 5.50 mm target displacement (0.50% drift ratio) for pushing and pulling directions, respectively. Load capacity was 138.5 kN for the pushing direction and -132.8 kN for the pulling direction. Damage evolution started with the rapid separation of the inner and outer leaves during the pre-loading phase. Until 5.50 mm lateral displacement (0.50% drift ratio) only random vertical cracks passing through the stone units could be observed. However, several diagonal cracks appeared while loading to 5.50 mm target displacement. Moreover, crushing and spalling of the toes started at successive cycles of this target displacement level. Damage reached significant levels at 11.0 mm lateral displacement (1.00% drift ratio), where extensive spalling of stone units caused a sharp drop in the load bearing capacity.

The lateral load-displacement hysteresis diagram describing the in-plane response of the M-100-C specimen is presented in Figure C.103. Data points corresponding to first diagonal cracking ($H_{dia,cr}$), maximum lateral resistance (H_{max}), 20% strength loss ($H_{0.8max}$) and ultimate displacement ($H_{d,max}$), are marked in the figure. The envelope curve obtained for the M-100-C specimen is presented in Figure C.104.

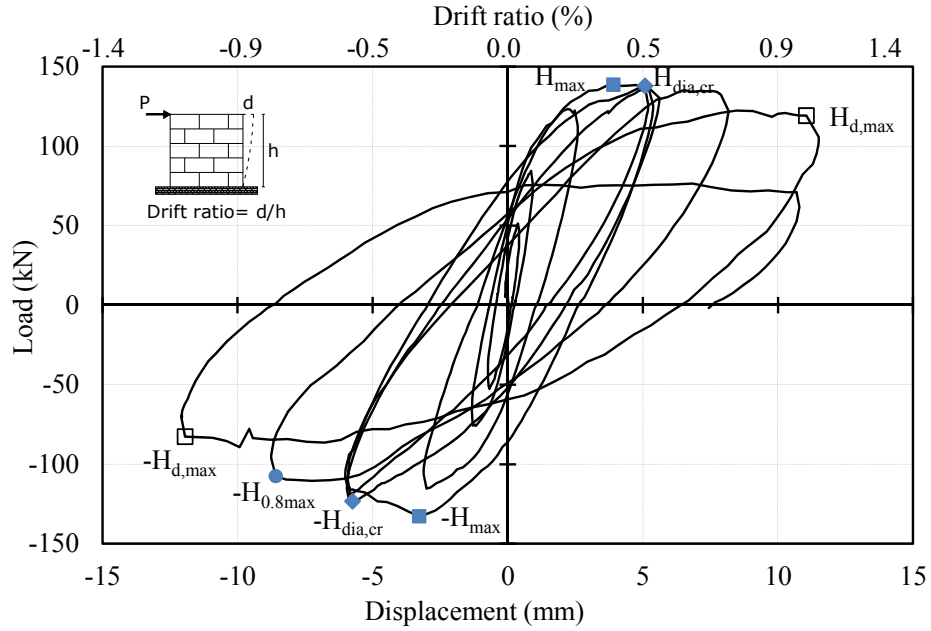


Figure C.103 : Lateral load-displacement diagram for specimen M-100-C.

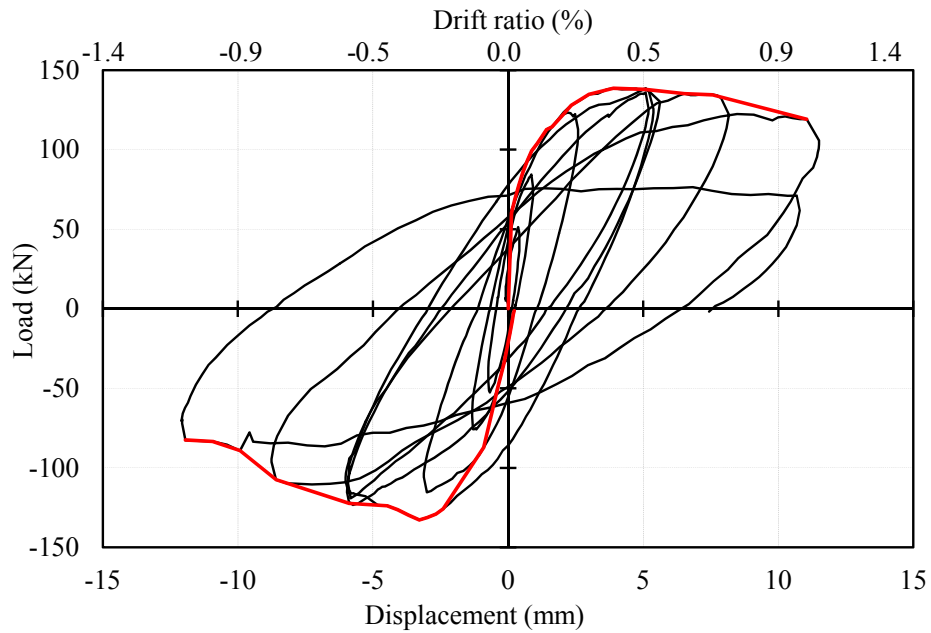


Figure C.104 : Envelope curve of lateral load-displacement diagram of M-100-C.

Table C.4 : M-100-C specimen summary table.

Cycle	Drift ratio (%)	Displacement (mm)	Peak Load (kN)	Peak Load/ H_{max} (%)	Note
1. Push	0.05	0.55	47.5	34	Randomly distributed cracks on stone units
1. Pull	-0.05	-0.55	-50.4	38	Randomly distributed cracks on stone units
2. Push	0.10	1.10	84.3	61	Randomly distributed cracks on stone units
2. Pull	-0.10	-1.10	-75.9	57	Randomly distributed cracks on stone units
3. Push	0.25	2.75	122.4	88	Randomly distributed cracks on stone units
3. Pull	-0.25	-2.75	-115.3	89	Randomly distributed cracks on stone units
4. Push (a)	0.50	5.50	136.6	99	First diagonal cracks, signs of crushing, maximum load
4. Pull (a)	-0.50	-5.50	-116.9	88	First diagonal cracks, signs of crushing, maximum load
4. Push (b)	0.50	5.50	130.1	94	New diagonal cracks
4. Pull (b)	-0.50	-5.50	-123.3	93	New diagonal cracks
4. Push (c)	0.50	5.50	135.9	98	Spalling started, diagonal cracks
4. Pull (c)	-0.50	-5.50	-119.2	90	Spalling started, diagonal cracks
5. Push	0.75	8.25	132.3	96	Spalling at the toe
5. Pull	-0.75	-8.25	-109.7	83	Spalling at the toe
6. Push (a)	1.00	11.00	119.1	86	Increased number of diagonal cracks, crushing
6. Pull (a)	-1.00	-11.00	-83.0	63	Increased number of diagonal cracks, extensive spalling at Southern face
6. Push (b)	1.00	11.00	71.1	51	Rapid loss of load capacity

Hysteresis loops obtained during the test indicates a highly dissipative behaviour with large residual displacements. The variation of the dissipated energy by means of the percentage of the total dissipated energy ($E_{dis,i}/E_{dis,cumulative}$, i denoting the cycle number) corresponding to each loading cycle is presented in Figure C.105. This presentation enables the identification of the critical points corresponding to particular events. A steep increase in the dissipated energy is a sign of increase in the nonlinear displacement, which can be directly associated with the damage growth. The steep increase from Cycle 2 to Cycle 4a (0.50% drift ratio) corresponds to first diagonal cracks. With further formation of the diagonal cracks and crushing and spalling of the compressed zones, another sudden increase happens at Cycles 5 and 6.

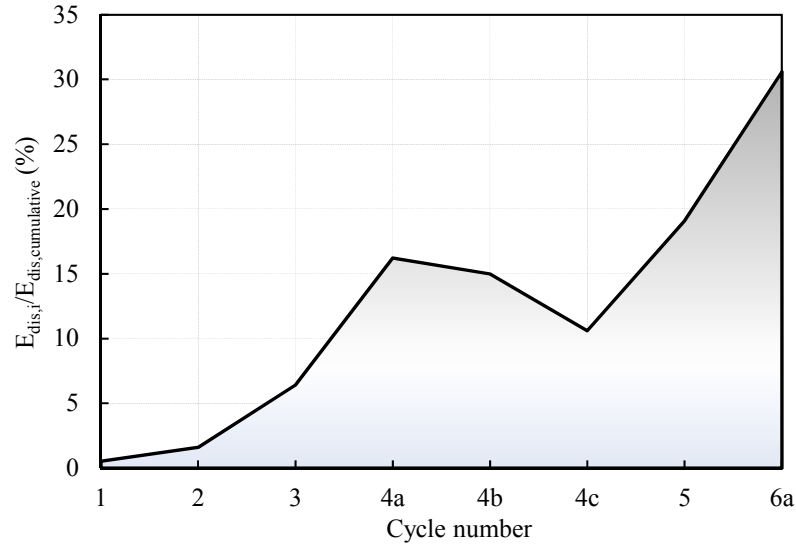


Figure C.105 : Ratio of the dissipated energy to total dissipated energy, M-100-C.

Apart from lateral load and displacement data, various other measurements were also collected from the specimen. Figure C.106 shows the variation of the vertical displacements measured at the tips of the top bond beam with respect to lateral displacement. Apparently, specimen shortens during the test, mainly due to high level of vertical load.

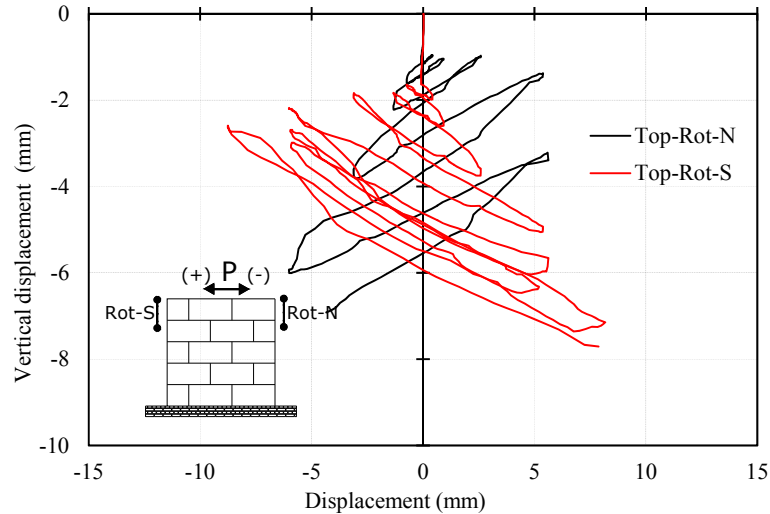


Figure C.106 : Variation of vertical displacements at the bond beam tips, M-100-C.

Vertical deformations measured by channels 36, 37, 40 and 41 (as shown in Figures 3.18 and 3.19) with respect to lateral displacement are presented in Figure C.107. In these figures, positive sign direction of the vertical axis corresponds to the shortening of the measured length. It is interesting to note that even when the LVDTs stayed in

the tension zone, the measured total vertical deformation was pointing to an overall compaction.

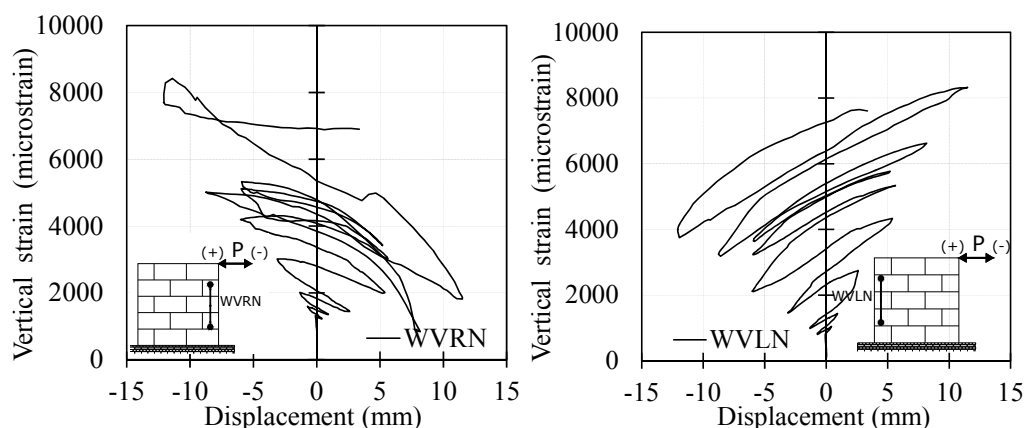


Figure C.107 : Vertical deformations near the Western and Eastern sides, M-100-C.

Vertical deformation measurements at toes and heels (depending on the loading direction) with respect to lateral displacements show that up to 5.50 mm lateral displacement (0.50% drift ratio) these zones are subjected to small cyclic compression and tension deformations, Figure C.108. However, beginning from the 0.50% drift ratio (5.50 mm displacement) vertical compressive deformations increase significantly mainly due to crushing of the stone units at the toes.

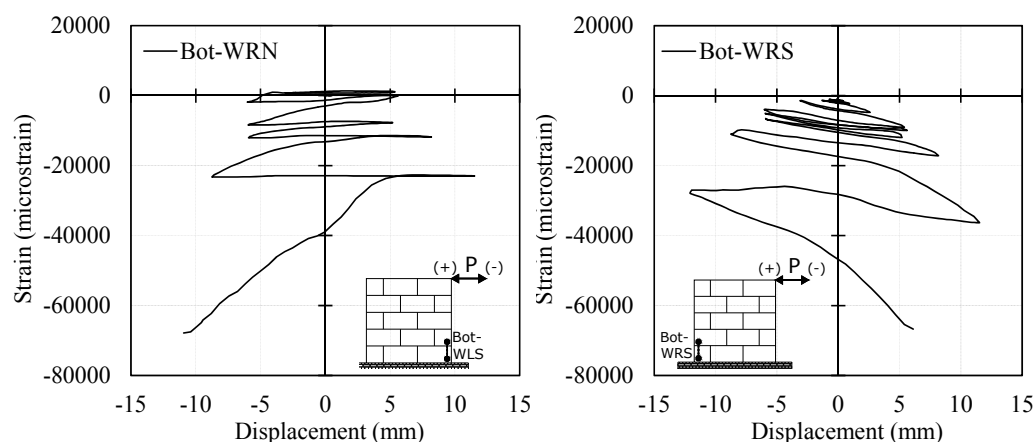


Figure C.108 : Vertical deformations at the toes and heels of the wall M-100-C.

Three of the cramps on the Northern leaf of the wall were instrumented with strain gauges. Measurements logged for the cramp at the mid-height of the specimen show that strains could reach 2200 microstrain which is very close to the yielding strain of the steel, Figure C.109. The strain diagram is almost symmetric which means that when upper side is in compression the lower one is in tension and vice versa.

Consequently, if the average of these symmetric strains is calculated for each instrumented cramp, average elongation values does not exceed the 25% of the yielding strain, as seen in Figure C.110.

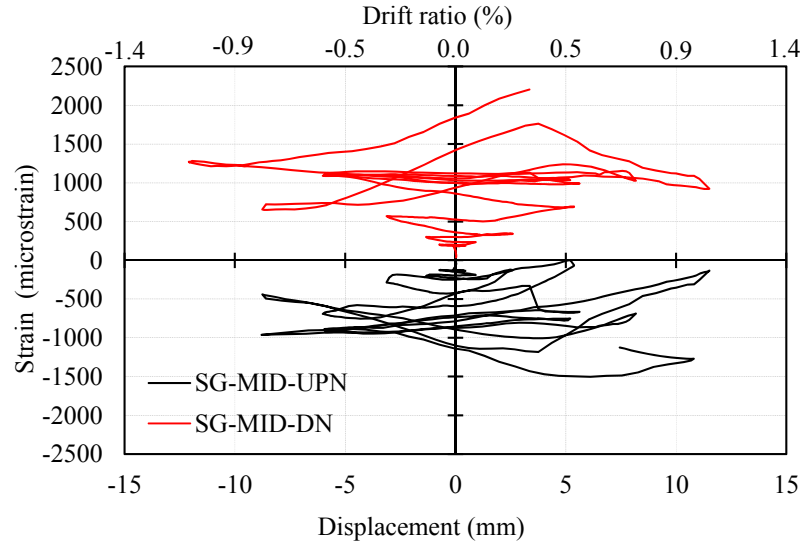


Figure C.109 : Strain variation for the cramp at mid-height of the wall, M-100-C.

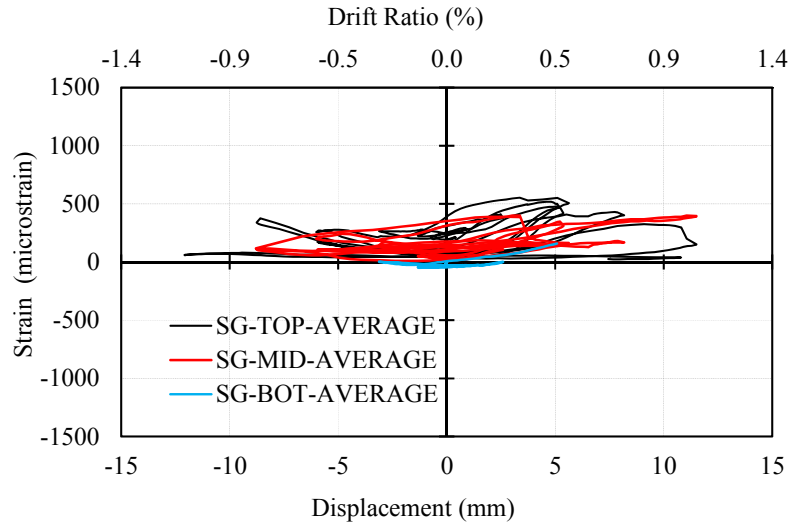


Figure C.110 : Average strains of instrumented cramps of wall M-100-C.

Detachment between the exterior leaves of the wall was measured by two LVDTs positioned at mid-heights of the Eastern and Western faces of the specimen. As clearly seen from the transverse displacement versus lateral displacement diagram (Figure C.111) obtained for the Western side, beginning from the initial loading cycles separation between the outer and inner leaves increased successively.

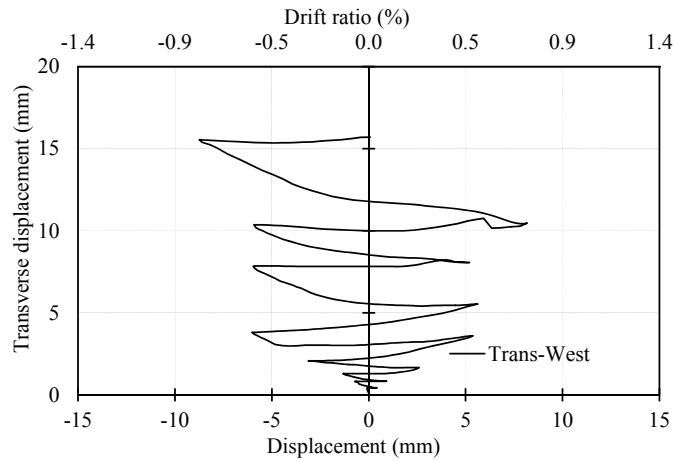


Figure C.111 : Detachment between the exterior leaves (Western side, M-100-C).

C.5 M-50 Wall

The M-50 specimen consisted of two ashlar outer leaves and a rubble masonry core. No cramps or pins were used for the connection of the stone units to each other. Axial stress level was kept as 0.50 MPa during the test. The test was conducted 185 days after the production of the M-50 wall.

C.5.1 Initial compression test (M-50)

Three successive vertical loading and unloading cycles were applied prior to reversed cyclic lateral loading. The target axial load level was 180 kN that corresponded to 0.50 MPa axial stress. The axial stress-axial strain diagrams of these three cycles plotted by using the average of the measurements of the six LVDTs (with an approximate gauge length of 800 mm) on both sides of the specimen are presented in Figure C.112.

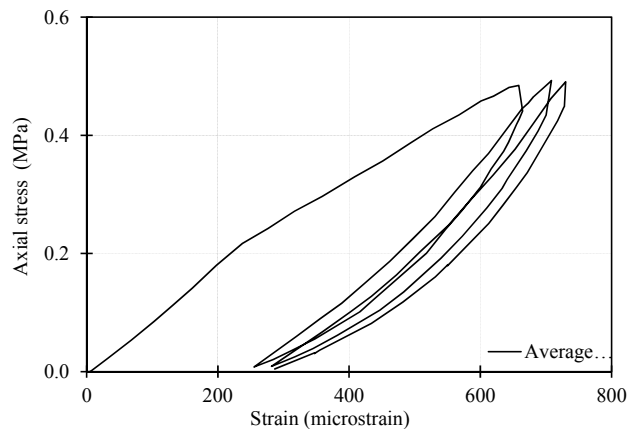


Figure C.112 : Axial stress-strain behaviour of M-50 specimen.

C.5.2 Shear compression test (M-50)

After performing the pre-loading session that consisted of three successive axial loading cycles, all channels were initialized and hydraulic power supply of the lateral actuator was activated. The lateral loading began after applying the targeted 0.50 MPa axial load.

No damage could be observed during the 0.55 mm target displacement cycle; except the signs of detachment at the interfaces of inner and outer leaves (Figure C.113), which were left from the transportation of the wall. At 1.10 mm displacement cycle (0.10% drift ratio), first vertical cracks appeared at the compressed parts of the Northern and Western sides of the wall, Figures C.114 and C.115.

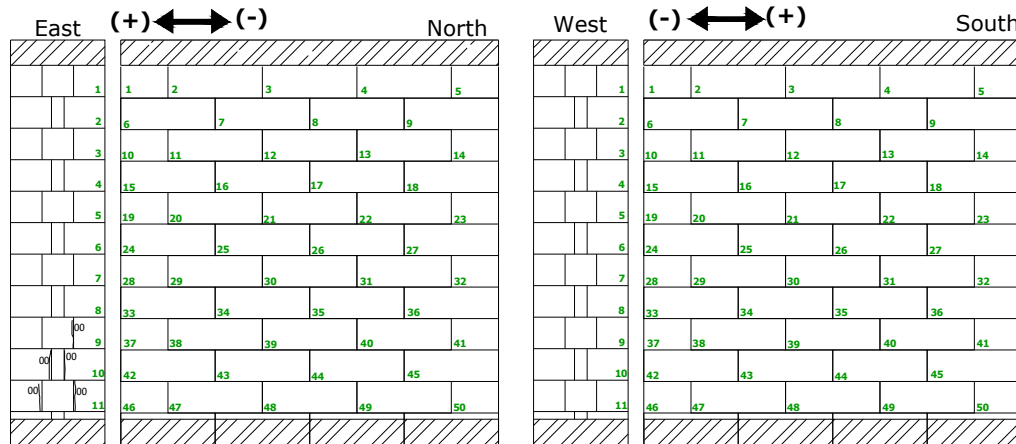


Figure C.113 : Cycle 1, M-50 after 0.55 mm displacement (0.05% drift ratio).

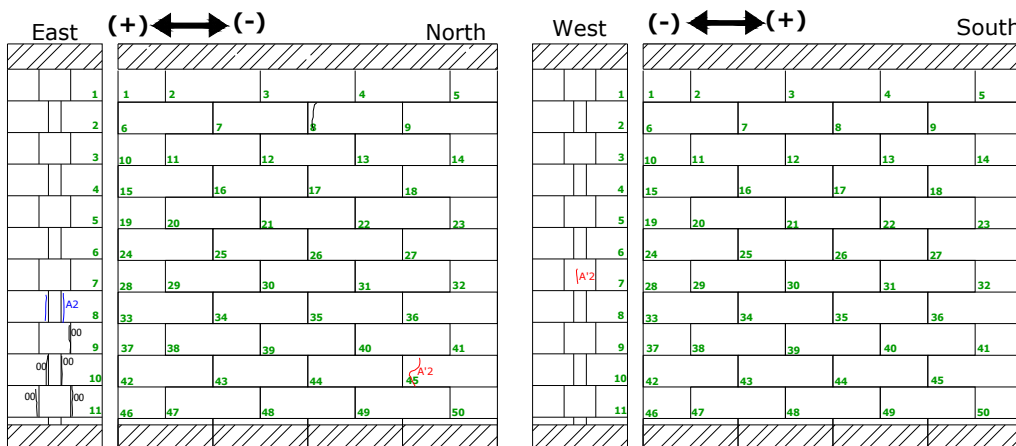


Figure C.114 : Cycle 2, M-50 after 1.10 mm displacement (0.10% drift ratio).



Figure C.119 : Cycle 4a, Northern face of M-50 after 0.50% drift ratio.

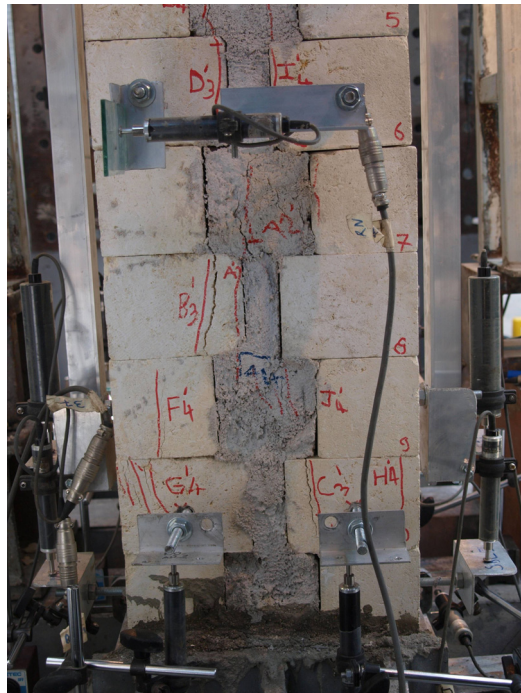


Figure C.120 : Cycle 4a, Western side of M-50 after 0.50% drift ratio.

During the second 5.50 mm cycle (0.50% drift ratio, cycle 4b), number of diagonal cracks increased visibly, Figure C.121. The stepped cracks passing through the joints were clearly visible, Figure C.122. Signs of crushing could be observed around the toes of the wall. The third cycle of 5.50 mm (0.50% drift ratio, cycle 4c) brought marginal increase to the extent of damage observed on the specimen, Figure C.123. Maximum crack width on the Northern and Southern faces could reach 3.5 mm.

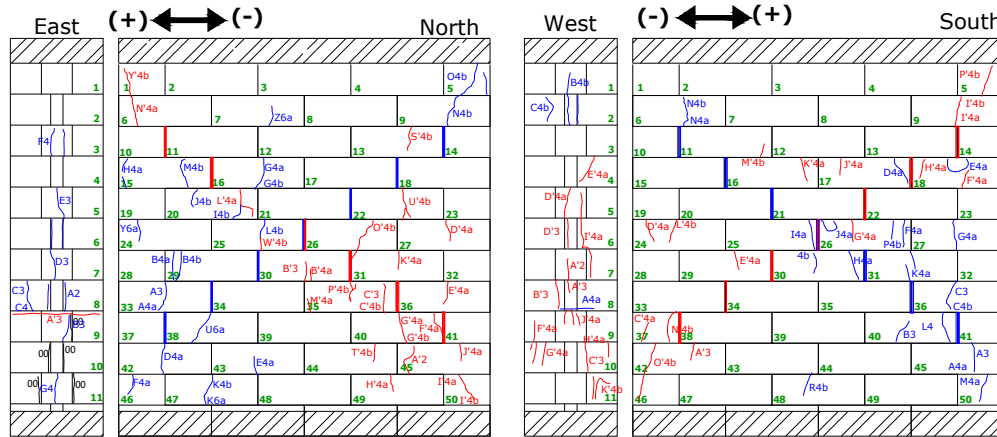


Figure C.121 : Cycle 4b, M-50 after 5.50 mm displacement (0.50% drift ratio).

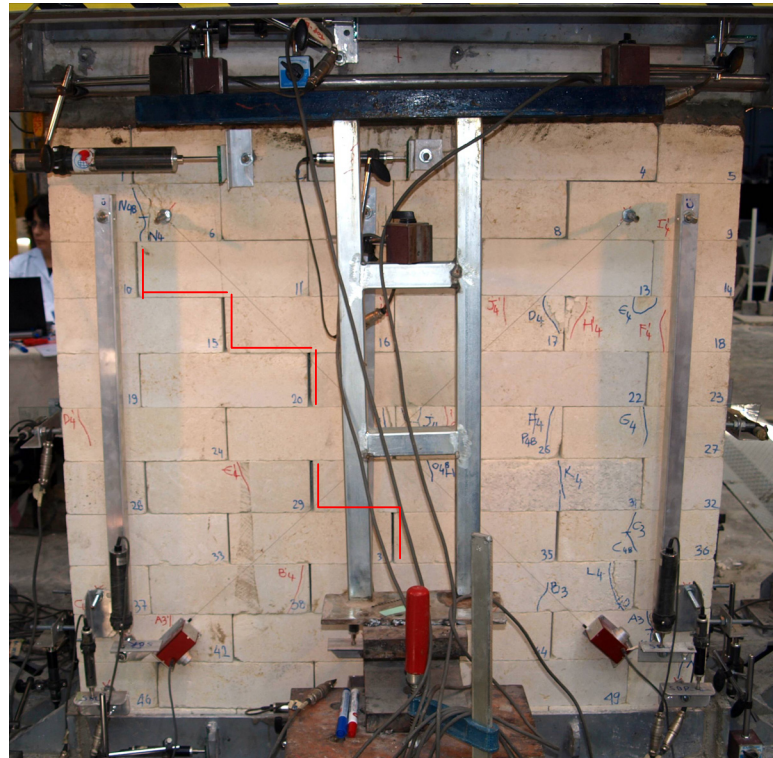


Figure C.122 : Cycle 4b, Southern face of M-50 after 0.50% drift ratio.

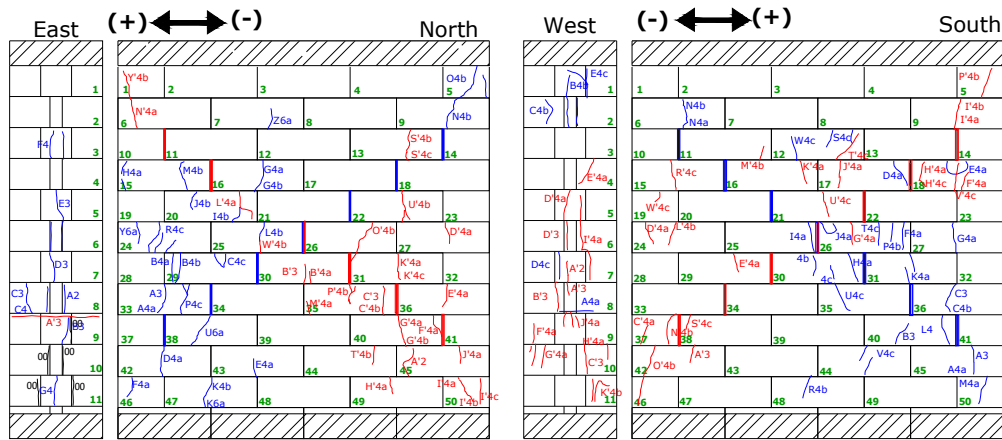


Figure C.123 : Cycle 4c, M-50 after 5.50 mm displacement (0.50% drift ratio).

Stepped diagonal cracks became apparent during the 5th loading cycle which targeted 8.25 mm horizontal displacement (0.75% drift ratio), Figure C.124. Since the deformations were accumulated at these stepped cracks, almost no increase in the cracks passing through the stone units could be identified (Figure C.125). The separation of the head joints on the path of the stepped diagonal cracks could reach 15 mm width. Crushing and spalling continued at the corner stone units of the toes, as seen in Figure C.126.



Figure C.124 : Cycle 5, Northern face of M-50 after 0.75% drift ratio.

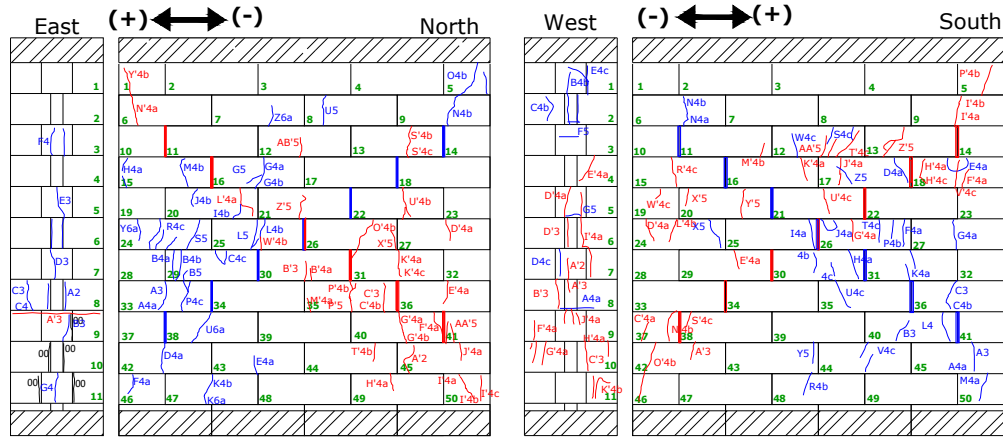


Figure C.125 : Cycle 5, M-50 after 8.25 mm displacement (0.75% drift ratio).

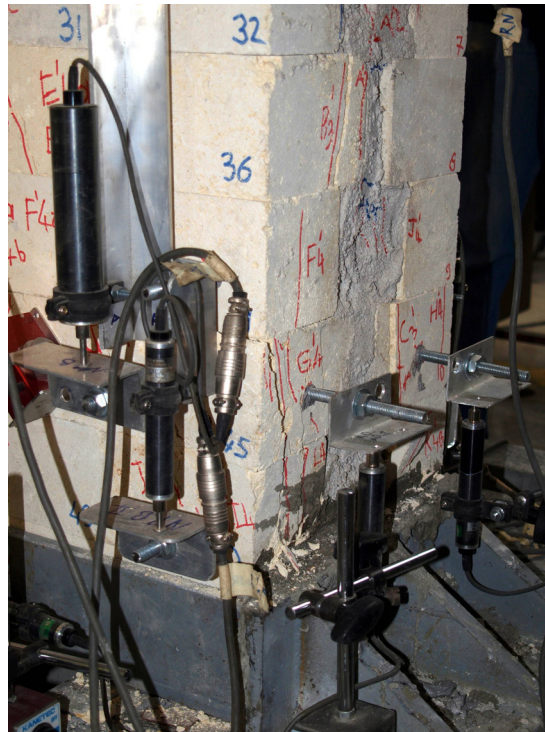
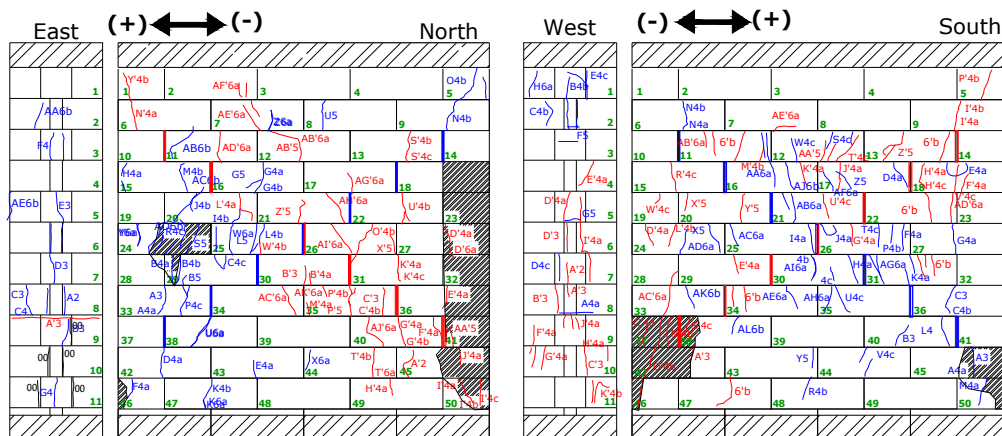
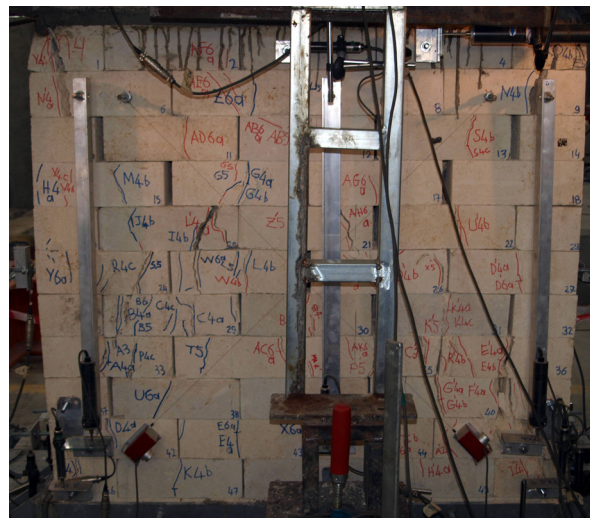
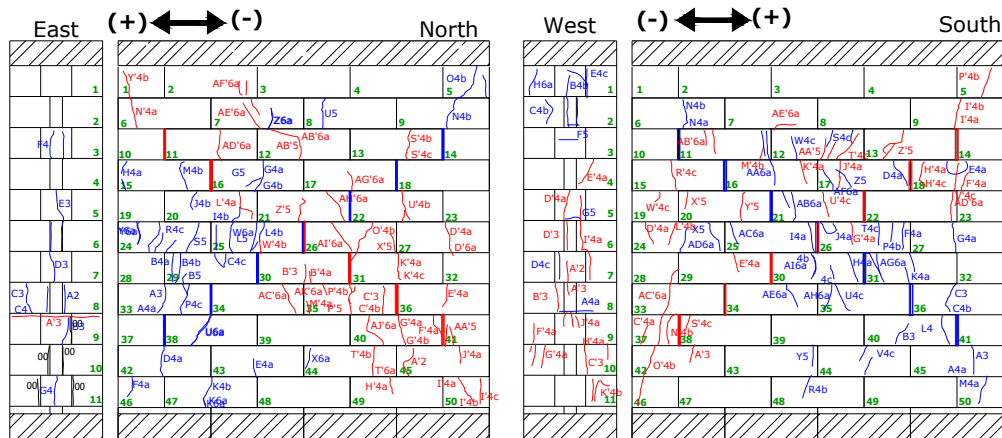


Figure C.126 : Cycle 5, Toe crushing after 0.75% drift ratio (M-50).

During the first 11.0 mm cycle (1.0% drift ratio, cycle 6a), damage evolved with further widening of the stepped diagonal cracks and crushing of the stone units at the toes, Figures C.127 and C.128. The opening of the head joints on the path of the stepped diagonal cracks could reach 22 mm width. This value exceeded 30 mm during the second 11.0 mm lateral displacement cycle. In addition to that, during the second cycle (cycle 6b, 11.0 mm), toe regions of the exterior ashlar masonry leaves spalled completely, Figures C.129 and C.130.



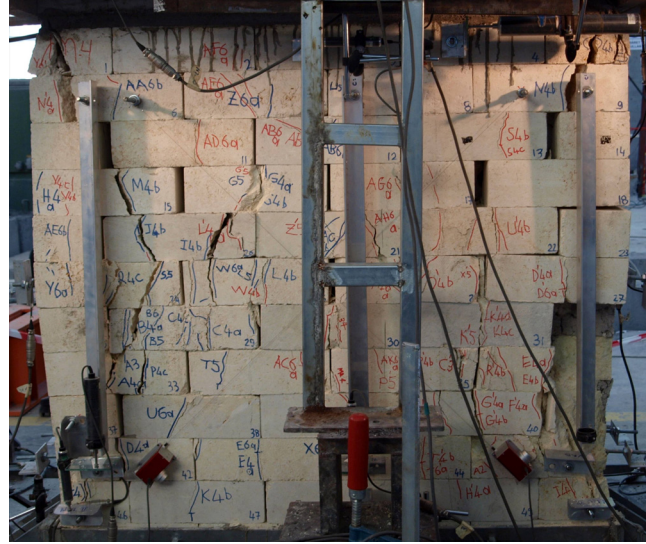


Figure C.130 : Cycle 6b, Extensive spalling of toe after 1.00% drift ratio (M-50).

After completing the pushing part of the third 11.0 mm (1.0% drift ratio) cycle, the test was ended due to significant drop in the load bearing capacity that occurred in parallel with the development of extensive damage. After test views of the North, South, East and West faces of the specimen are presented in Figures C.131-C.133.



Figure C.131 : Northern face of M-50 specimen after the test.



Figure C.132 : Southern face of M-50 specimen after the test.



Figure C.133 : (a) Eastern and (b) Western faces of M-50 after the test.

The lateral load-displacement hysteresis diagram describing the in-plane response of the M-50 specimen is presented in Figure C.134. Data points corresponding to first diagonal cracking ($H_{dia,cr}$), maximum lateral resistance (H_{max}), 20% strength loss ($H_{0.8max}$) and ultimate displacement ($H_{d,max}$), are marked in the figure. The envelope curve obtained for the M-50 specimen is presented in Figure C.135.

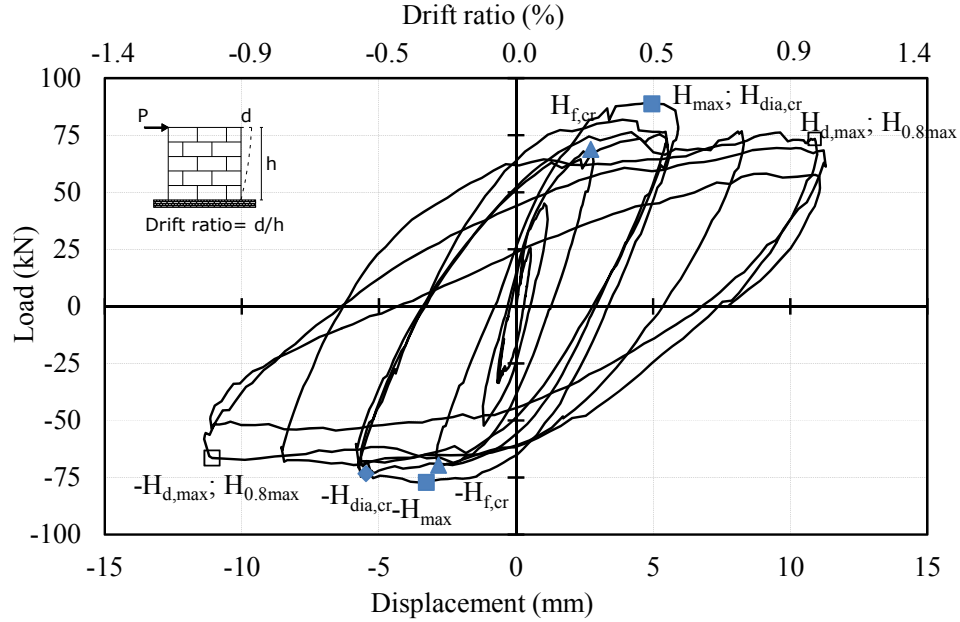


Figure C.134 : Lateral load-displacement diagram for specimen M-50.

Drift ratios, displacements, peak loads, peak load to maximum strength ratios (Peak Load/ H_{max}) and notes on damage development corresponding to each performed cycle are summarized in Table C.5. Maximum load capacities of the M-50 specimen were attained at displacements of 4.74 mm and -3.27 mm while travelling to 5.50 mm target displacement (0.50% drift ratio) for pushing and pulling directions, respectively. Load capacity was 89.2 kN for the pushing direction and -77.1 kN for the pulling direction. Damage evolution started with the slight detachment of the inner and external leaves during the pre-loading phase. Until 5.50 mm lateral displacement (0.50% drift ratio) only random vertical cracks passing through the stone units could be observed. However, several diagonal cracks appeared while loading to 5.50 mm target displacement. Moreover, crushing and spalling of the toes started at successive cycles of this target displacement level. Damage reached significant levels at 11.0 mm lateral displacement (1.00% drift ratio), where

extensive spalling of stone units caused a significant drop in the load bearing capacity.

Table C.5 : M-50 specimen summary table.

Cycle	Drift ratio (%)	Displacement (mm)	Peak Load (kN)	Peak Load/H _{max} (%)	Note
1. Push	0.05	0.55	25.6	29	No damage
1. Pull	-0.05	-0.55	-33.5	44	No damage
2. Push	0.10	1.10	45.1	51	Vertical cracks at compressed parts
2. Pull	-0.10	-1.10	-52.3	68	Vertical cracks at compressed parts
3. Push	0.25	2.75	68.8	77	Vertical cracks at the toes and the rubble core, first horizontal bending crack on the Western side
3. Pull	-0.25	-2.75	-69.8	91	First horizontal bending crack visible at the Eastern side, vertical cracks at the toes and the rubble core
4. Push (a)	0.50	5.50	88.7	99	First diagonal cracks, stepped diagonal cracks between units,
4. Pull (a)	-0.50	-5.50	-74.6	97	First diagonal cracks, stepped diagonal cracks between units
4. Push (b)	0.50	5.50	71.3	80	New diagonal cracks, crushing started at the toe
4. Pull (b)	-0.50	-5.50	-70.0	91	New diagonal cracks, crushing started at the toe
4. Push (c)	0.50	5.50	73.9	83	Diagonal cracks increased
4. Pull (c)	-0.50	-5.50	-67.9	88	Diagonal cracks increased
5. Push	0.75	8.25	76.8	86	Wider diagonal stepped cracks, spalling started at the toe
5. Pull	-0.75	-8.25	-68.0	88	Wider diagonal stepped cracks, spalling started at the toe
6. Push (a)	1.00	11.00	66.3	74	Significant spalling of the stone units, wider diagonal cracks
6. Pull (a)	-1.00	-11.00	-66.4	86	Significant spalling of the stone units, wider diagonal cracks
6. Push (b)	1.00	11.00	68.4	77	Complete spalling of stone units at the toe
6. Pull (b)	-1.00	-11.00	-53.6	70	Complete spalling of stone units at the toe
6. Push (c)	1.00	11.00	57.2	64	Test stopped due to extensive damage

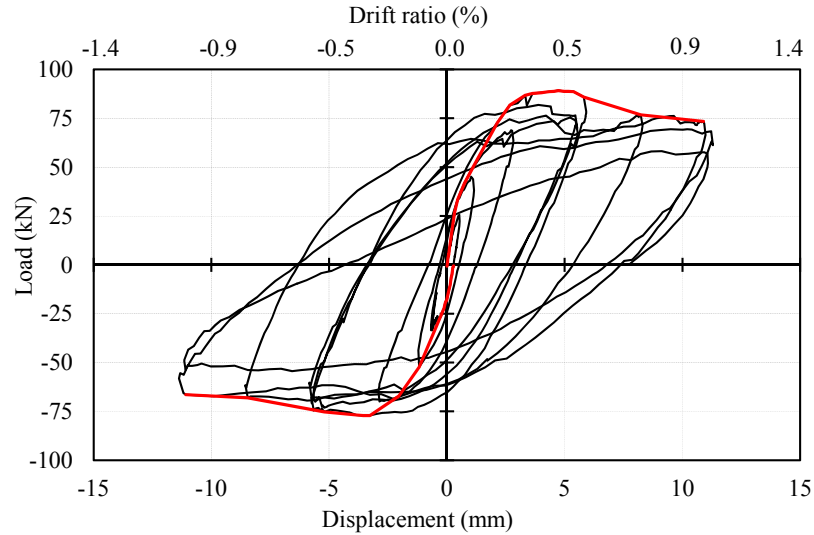


Figure C.135 : Envelope curve of lateral load-displacement diagram of M-50.

Hysteresis loops obtained during the test indicates a highly dissipative behaviour with large residual displacements. The variation of the dissipated energy by means of the percentage of the total dissipated energy ($E_{dis,i}/E_{dis,cumulative}$, i denoting the cycle number) corresponding to each loading cycle is presented in Figure 5.136. This presentation enables the identification of the critical points corresponding to particular events. A steep increase in the dissipated energy is a sign of increase in the nonlinear displacement, which can be directly associated with the damage growth. In this particular case, the steep increase from Cycle 3 to Cycle 4a (0.50% drift ratio) corresponds to first diagonal cracks either passing through the stone units or joints. With further formation of the diagonal cracks and crushing and spalling of the compressed zones, another sudden increase occurs at Cycle 6a (1.00% drift ratio).

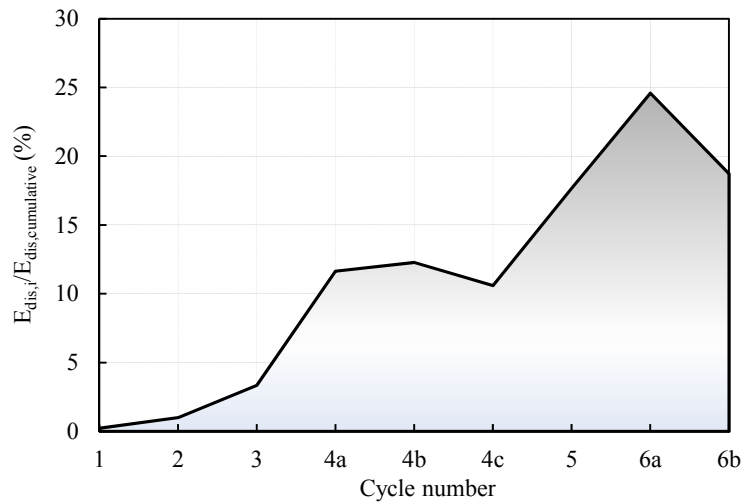


Figure 5.136 : Ratio of the dissipated energy of each cycle to total dissipated, M-50.

Variation of the vertical displacements measured at the tips of the top bond beam with respect to lateral displacement is presented in Figure C.137. Accordingly, specimen exhibits a tendency for compaction during the application of the lateral load reversals.

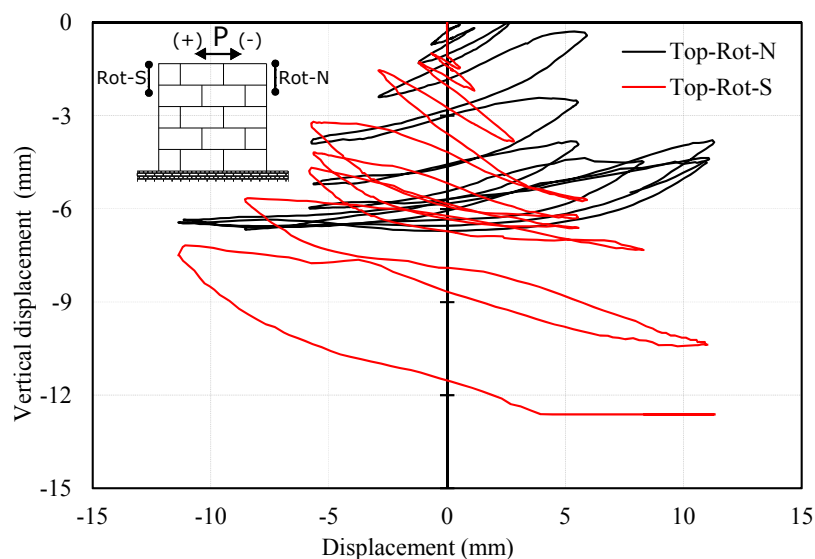


Figure C.137 : Variation of vertical displacements at the bond beam tips, M-50.

Vertical deformations measured by channels 36, 37, 40 and 41 (as shown in Figures 3.18 and 3.19) with respect to lateral displacement are presented in Figure C.138. In these figures, positive sign direction of the vertical axis corresponds to the shortening of the measured length. It is interesting to note that even when the LVDTs stayed in the tension zone, the measured total vertical deformation was pointing to an overall compaction.

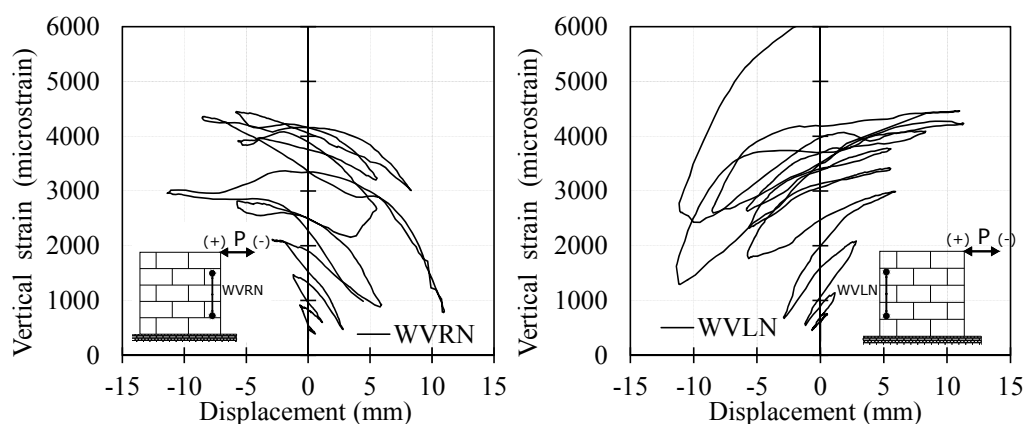


Figure C.138 : Vertical deformations near the Western and Eastern sides, M-50.

Vertical deformation measurements at toes and heels (depending on the loading direction) with respect to lateral displacements show that especially after 5.50 mm horizontal displacement (0.5% drift ratio) vertical compressive deformations make a sharp increase due to crushing of the stone units at the toes, Figure C.139.

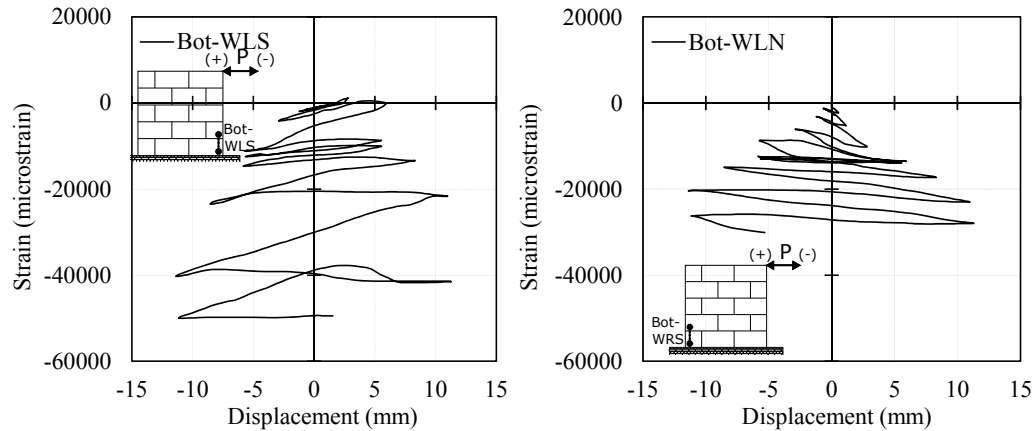


Figure C.139 : Vertical deformations at the lower corners of the M-50 wall.

Detachment between the exterior leaves of the wall was measured by two LVDTs positioned at mid-heights of the Eastern and Western sides of the specimen. As clearly seen from the transverse displacement versus lateral displacement diagram (Figure C.140) obtained for the Eastern side, beginning from the initial loading cycles separation between the outer and inner leaves increased successively.

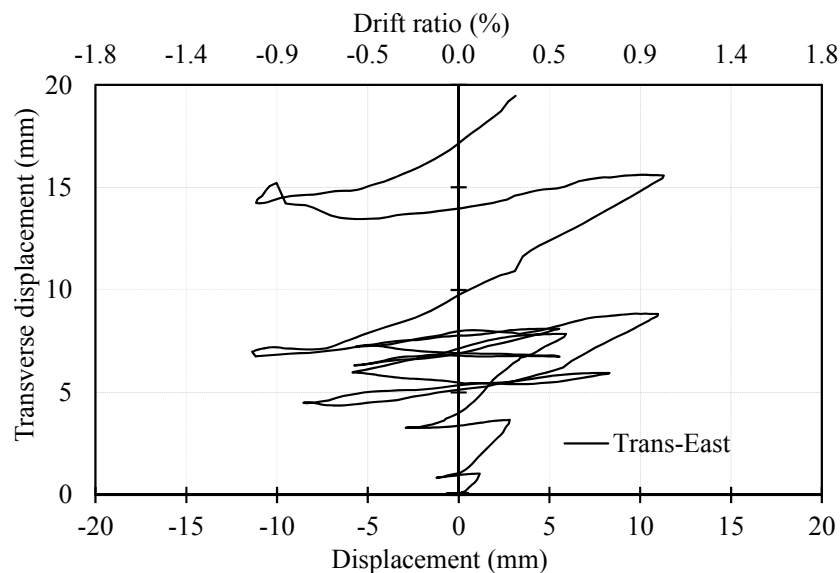


Figure C.140 : Detachment between the external leaves of the M-50 wall.

C.6 S-50-C Wall

The S-50-C wall consisted of two ashlar outer leaves. The stone units were connected to each other via horizontally positioned cramps. Axial stress level was kept as 0.50 MPa during the test. The test was conducted 207 days after the production of the S-50-C wall.

C.6.1 Initial compression test (S-50-C)

Three successive vertical loading and unloading cycles were applied prior to reversed cyclic lateral loading. The target axial load level was 120 kN that corresponded to 0.50 MPa axial stress. The axial stress-axial strain diagrams of these three cycles plotted by using the average of the measurements of the six LVDTs (with an approximate gauge length of 800 mm) on both sides of the specimen are presented in Figure C.141.

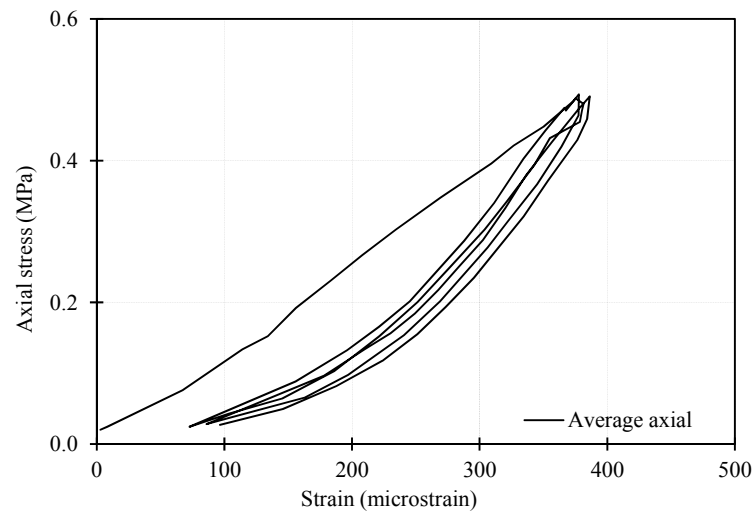


Figure C.141 : Axial stress-axial strain behaviour of S-50-C specimen.

C.6.2 Shear compression test (M-50)

After performing the pre-loading session that consisted of three successive axial loading cycles, all channels were initialized and hydraulic power supply of the lateral actuator was activated. The lateral loading began after applying the targeted 0.50 MPa axial load.

Up to 0.25% drift ratio (2.75 mm displacement), no significant damage could be observed except a few vertical cracks at lower corners of the wall that occurred

during the drilling of the stone units for mounting of the LVDTs, Figures C.142 and C.143.

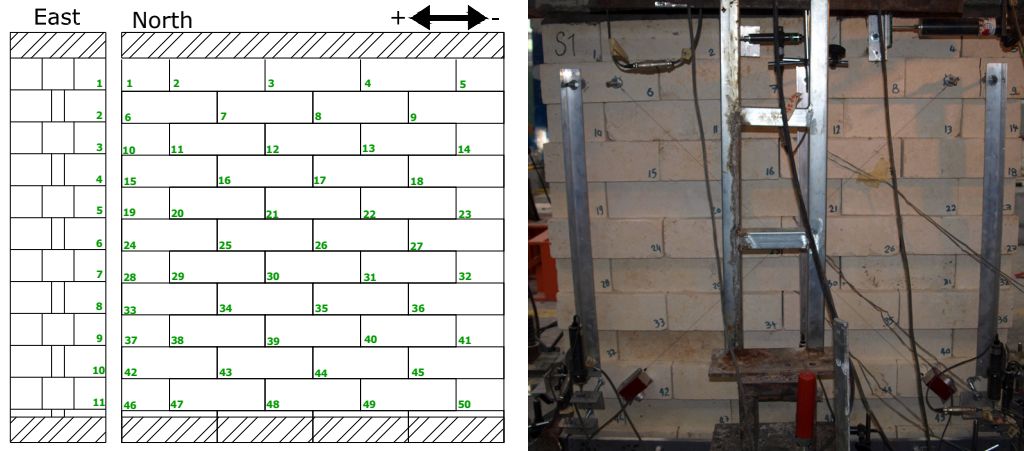


Figure C.142 : Cycle 1, S-50-C after 0.55 mm displacement (0.05% drift ratio).

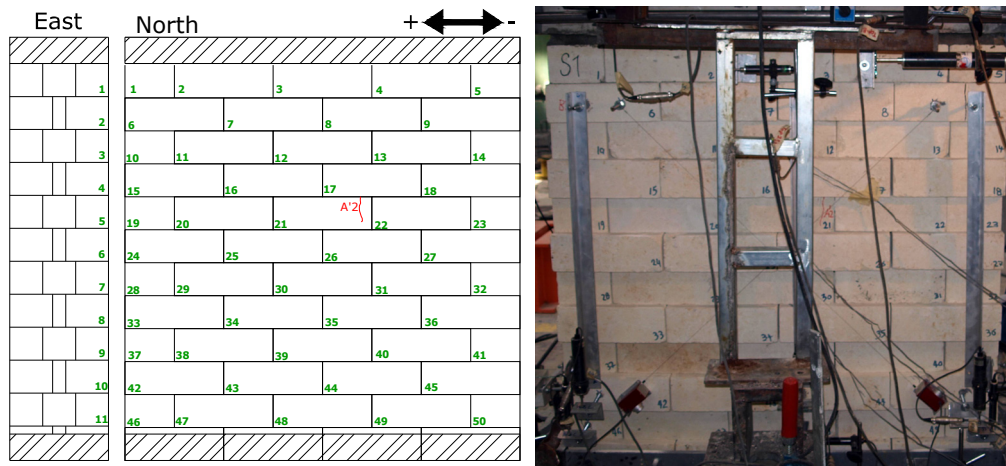


Figure C.143 : Cycle 2, S-50-C after 1.10 mm displacement (0.10% drift ratio).

At the pushing and pulling parts of the 2.75 mm (0.25% drift ratio) cycle, first vertical cracks passing through the stone units occurred at the compression zones (toe regions) of the wall, Figure C.144.

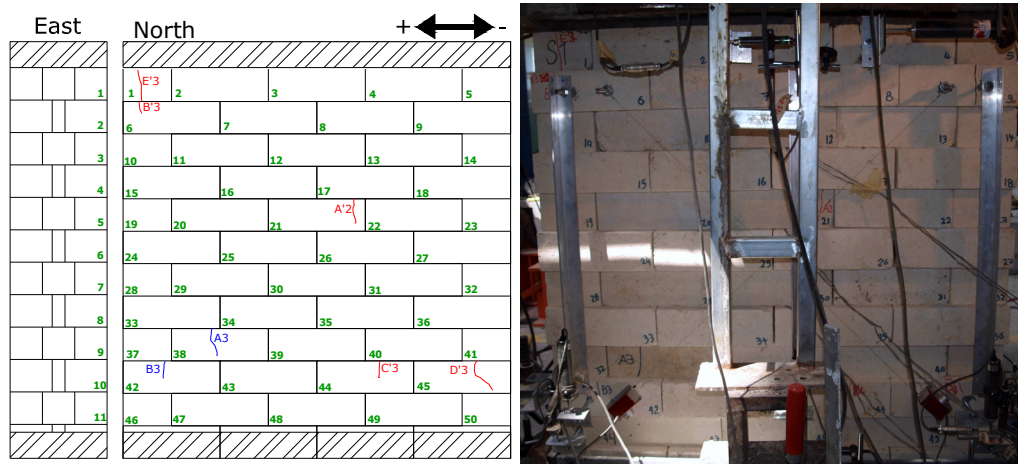


Figure C.144 : Cycle 3, S-50-C after 2.75 mm displacement (0.25% drift ratio).

At 5.50 mm cycle (0.50% drift ratio, cycle 4a), a number of vertical and diagonal cracks appeared among the external leaves, Figure C.145. Maximum crack width at peaks of this cycle could reach 2 mm.

During the second 5.50 mm cycle (cycle 4b, 0.50% drift ratio), in addition to the increase in the number of vertical and diagonal crack passing through the stone units (Figure C.146), crushing signs started to appear at the Eastern side of the Northern face and at the Western side of the Southern face.

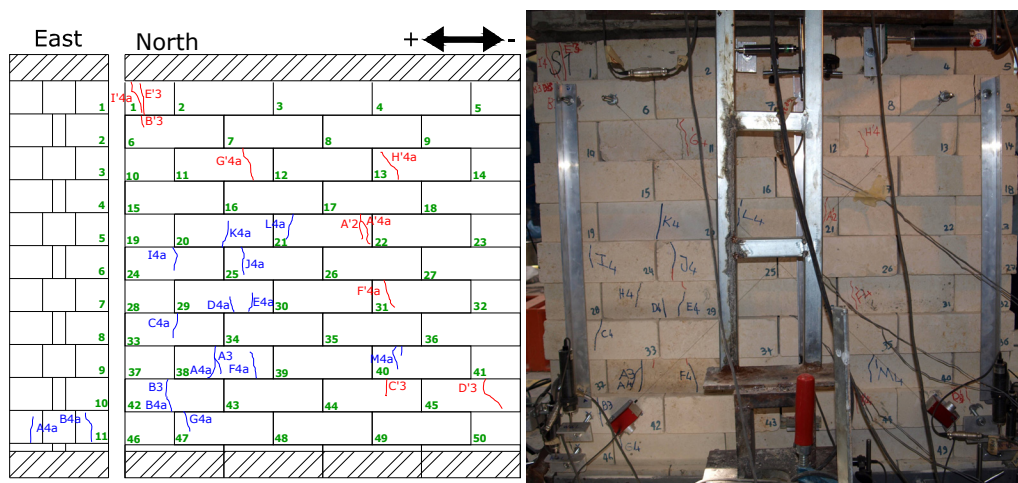


Figure C.145 : Cycle 4a, S-50-C after 5.50 mm displacement (0.50% drift ratio).

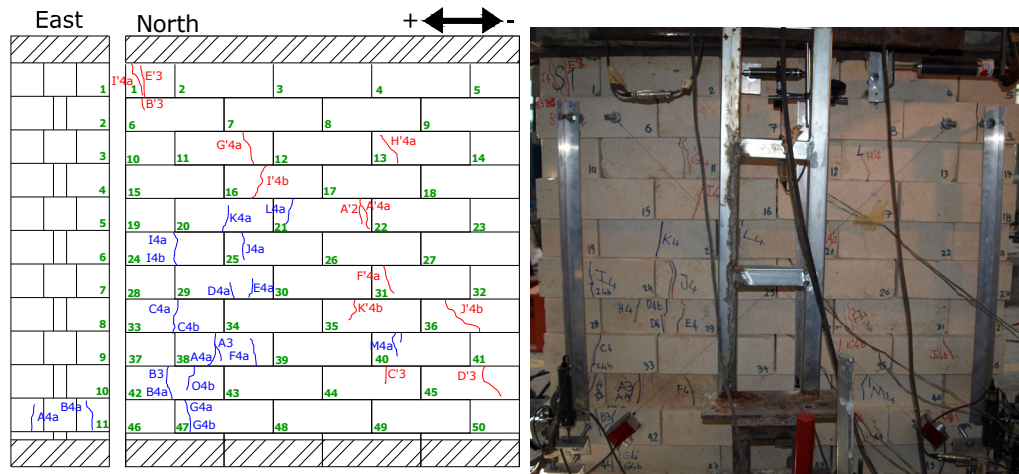


Figure C.146 : Cycle 4b, S-50-C after 5.50 mm displacement (0.50% drift ratio).

During the third cycle of 5.50 mm (cycle 4c, 0.50% drift ratio) crushing of the stone units became easily visible (Figure C.147) and maximum crack width could reach 15 mm. Horizontal cracks due to opening of the bed joints in the vertical direction (flexure cracks) could be observed just above the 4th stone course. Probably these cracks were formed in the previous displacement cycles but could only become visible in this one.

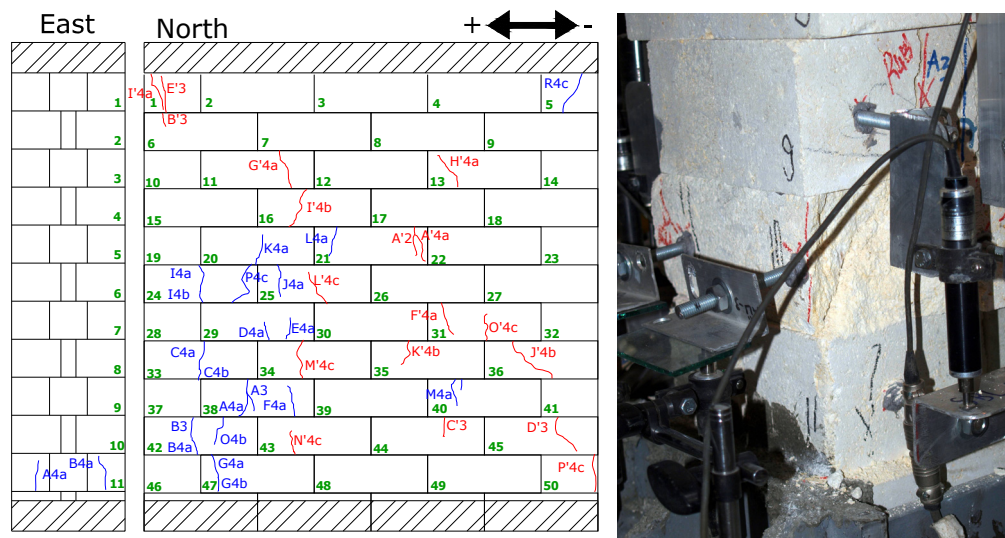


Figure C.147 : Cycle 4c, S-50-C after 5.50 mm displacement (0.50% drift ratio).

During cycle 5, that targeted 8.25 mm horizontal displacement (0.75% drift ratio), length and width of the existing cracks increased, Figure C.148. Maximum crack width reached 20 mm. Residual displacements due to sliding of the stone courses on top of each other could be observed at the lower heights of the wall, as seen in Figure C.149.

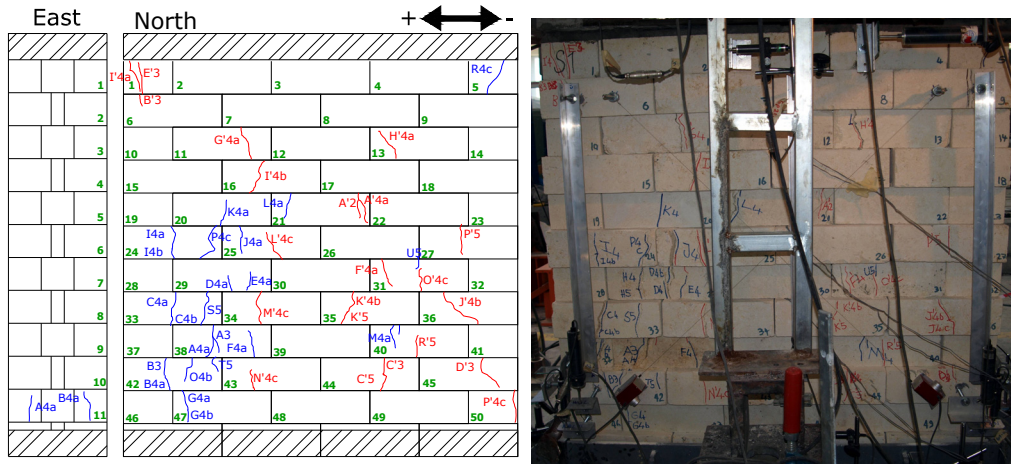


Figure C.148 : Cycle 5, S-50-C after 8.25 mm displacement (0.75% drift ratio).



Figure C.149 : Sliding between the stone courses, S-50-C (0.75% drift ratio).

During the first 11.0 mm cycle (1.0% drift ratio, cycle 6a), number and width of the diagonal cracks increased noticeably, Figure 5.156. Large portions of crushed stone units around the toes spalled, Figure C.150. The second and third cycles of the 1.0% drift ratio further increased the damage, Figures C.151 and C.152. At the end of the displacement reversals of 1.0% drift ratio, maximum crack width reached 40 mm and largest residual displacement between stone courses due to sliding was in the order 10 mm.

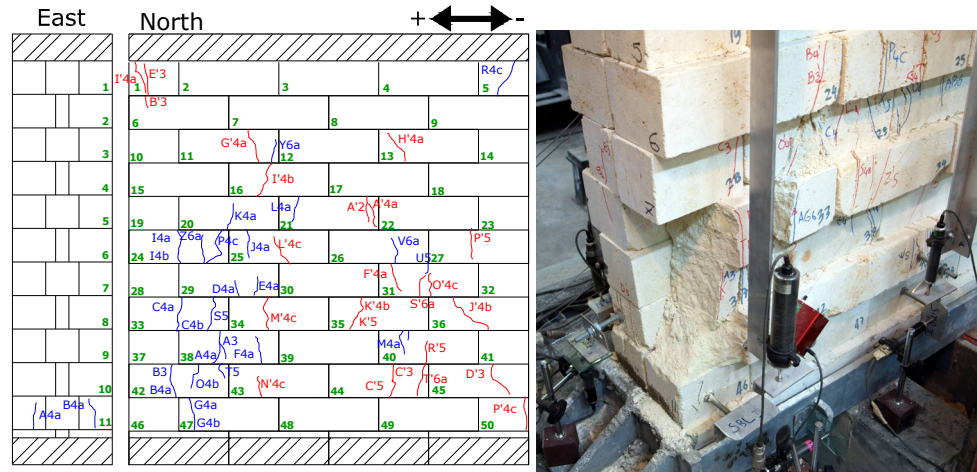


Figure C.150 : Cycle 6a, S-50-C after 11.0 mm displacement (1.00% drift ratio).

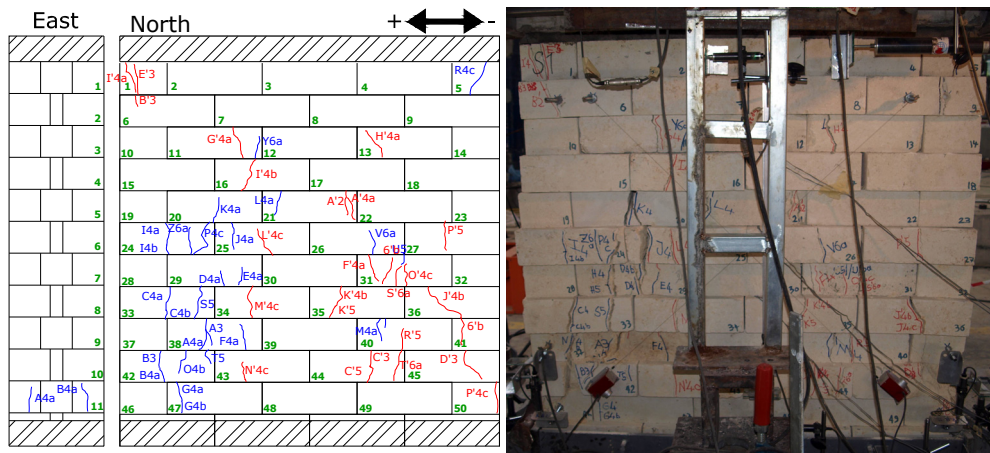


Figure C.151 : Cycle 6b, S-50-C after 11.0 mm displacement (1.00% drift ratio).

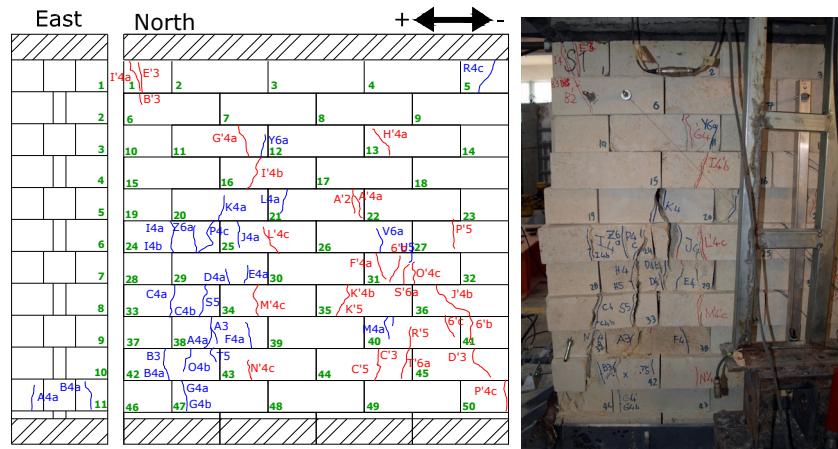


Figure C.152 : Cycle 6c, S-50-C after 11.0 mm displacement (1.0% drift ratio).

The cracking and spalling of stone units continued during the 7th and 8th cycles (corresponding to 1.25 and 1.50% drift ratios, respectively), Figures C.153 and C.154.

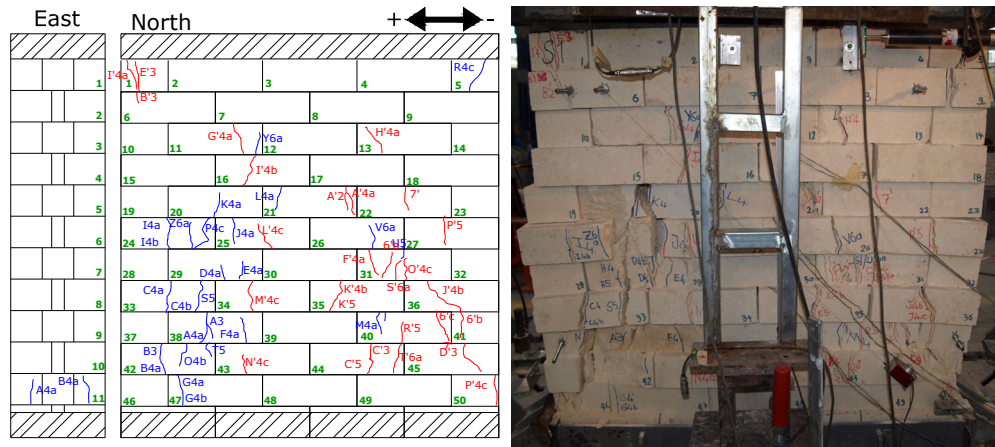


Figure C.153 : Cycle 7, S-50-C after 13.75 mm displacement (1.25% drift ratio).

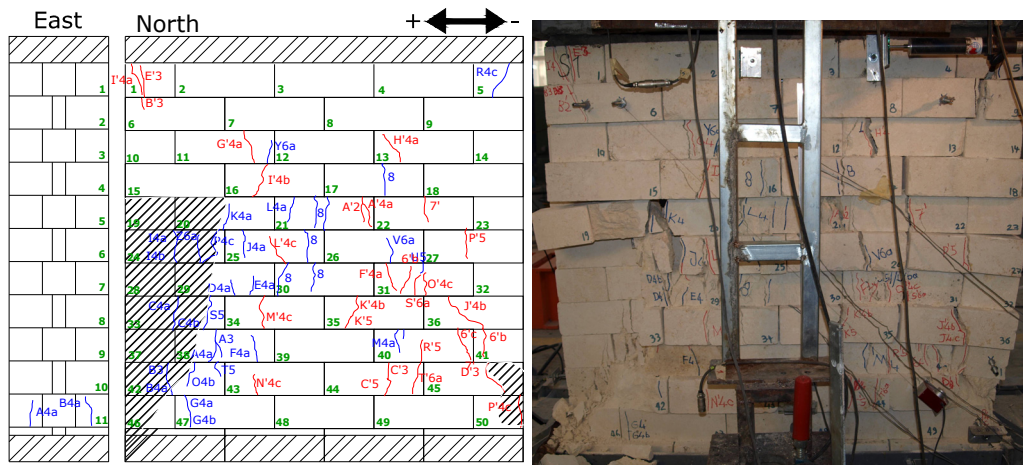


Figure C.154 : Cycle 8, S-50-C after 16.50 mm displacement (1.50% drift ratio).

After completing the pushing and pulling parts of the 16.5 mm (1.5% drift ratio) cycle, the test was stopped due to heavy damage, which caused safety concerns. After test views of the North, South, East and West faces of the specimen are presented in Figures C.155-C.157.



Figure C.155 : Cycle 8, North face of S-50-C at the end of the test.



Figure C.156 : Cycle 8, South face of S-50-C at the end of the test.



Figure C.157 : Cycle 8, (a) East and (b) West faces of S-50-C at the end of the test.

Drift ratios, displacements, peak loads, peak load to maximum strength ratios (Peak Load/ H_{max}) and notes on damage development corresponding to each performed cycle are summarized in Table C.6. From this table it can be pointed out that maximum load capacities of the S-50-C specimen are attained at of 5.50 and -2.75 mm target displacement (0.50% and 0.25% drift ratios) for pushing and pulling directions, respectively. The damage evolution started with vertical and diagonal cracks passing through the stone units (especially at the toes) at 0.50% drift ratio.

Damage evolution continued with the further development of diagonal cracks and crushing of the toes. Residual displacements due to sliding and dislocation of stone units could be observed at 0.75% drift ratio. After the formation of the diagonal cracks, upper diagonal halves of the wall started rocking. The test was stopped at 1.50% drift ratio after extensive spalling of the compressed parts that caused significant loss of strength.

Table C.6 : S-50-C specimen summary table.

Cycle	Drift ratio (%)	Displacement (mm)	Peak Load (kN)	Peak Load/H _{max} (%)	Note
1. Push	0.05	0.55	0.8	1.3	
1. Pull	-0.05	-0.55	-32.0	68	
2. Push	0.10	1.10	28.6	45	
2. Pull	-0.10	-1.10	-36.2	77	
3. Push	0.25	2.75	53.6	85	First vertical cracks around the toe
3. Pull	-0.25	-2.75	-46.8	100	First vertical cracks around the toe
4. Push (a)	0.50	5.50	63.1	100	New vertical and diagonal cracks
4. Pull (a)	-0.50	-5.50	-44.2	95	New vertical and diagonal cracks
4. Push (b)	0.50	5.50	57.8	92	Crushing begins, new vertical and diagonal cracks
4. Pull (b)	-0.50	-5.50	-40.8	87	Crushing begins, new vertical and diagonal cracks
4. Push (c)	0.50	5.50	59.4	94	Number and width of cracks increased, horizontal flexure cracks visible
4. Pull (c)	-0.50	-5.50	-38.5	82	Number and width of cracks increased, horizontal flexure cracks visible
5. Push	0.75	8.25	57.5	91	Residual displacements due to sliding of the stone courses at lower heights
5. Pull	-0.75	-8.25	-38.3	82	Residual displacements due to sliding of the stone courses at lower heights
6. Push (a)	1.00	11.00	60.3	96	Increased number of diagonal cracks, spalling of stone units at the toe
6. Pull (a)	-1.00	-11.00	-39.3	84	Spalling of stone units at the toe
6. Push (b)	1.00	11.00	54.4	86	Spalling at the toe-East side
6. Pull (b)	-1.00	-11.00	-36.3	78	Spalling at the toe-West side
6. Push (c)	1.00	11.00	54.6	87	Spalling at the toe-East side
6. Pull (c)	-1.00	-11.00	-37.0	79	Spalling at the toe-West side
7. Push	1.25	13.75	52.3	83	Upper diagonal half rocking, spalling at the toe
7. Pull	-1.25	-13.75	-37.2	80	Upper diagonal half rocking, spalling at the toe
8. Push	1.50	16.50	56.7	90	Upper diagonal half rocking, spalling at the toe
8. Pull	-1.50	-16.50	-34.7	74	Upper diagonal half rocking, spalling at the toe

The lateral load-displacement hysteresis diagram describing the in-plane response of the S-50-C specimen is presented in Figure C.158. Data points corresponding to first flexural cracking ($H_{f,cr}$), first diagonal cracking ($H_{dia,cr}$), maximum lateral resistance (H_{max}), 20% strength loss ($H_{0.8max}$) and ultimate displacement ($H_{d,max}$), are marked in

the figure. The envelope curve obtained for the S-50-C specimen can be seen in Figure C.159.

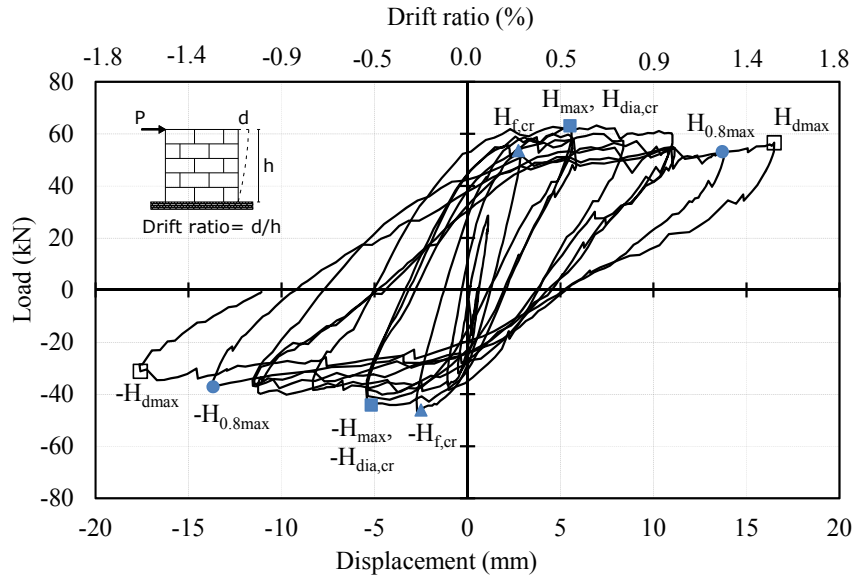


Figure C.158 : Lateral load-displacement diagram for specimen S-50-C.

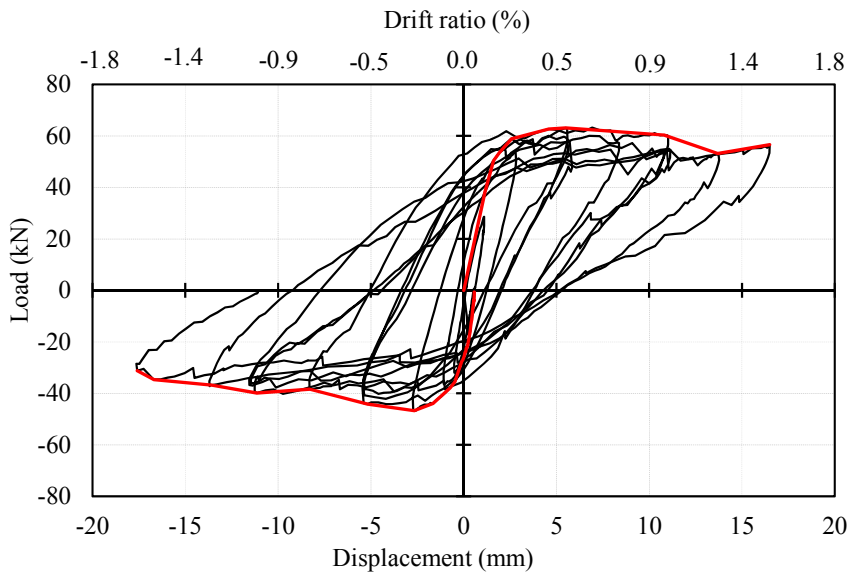


Figure C.159 : Envelope curve of lateral load-displacement diagram of S-50-C.

The variation of the dissipated energy by means of the percentage of the total dissipated energy ($E_{dis,i}/E_{dis,cumulative}$, i denoting the cycle number) corresponding to each loading cycle is presented in Figure C.160. This presentation enables the identification of the critical points corresponding to particular events. A steep increase in the dissipated energy is a sign of increase in the nonlinear deformations, which can be directly associated with the damage growth. In this particular case, the

steep increase at Cycle 4 (0.50% drift ratio) corresponds to first diagonal cracks. With further formation of the diagonal cracks and crushing and spalling of the compressed zones, another sudden increase happens at Cycle 6 (1.0% drift ratio).

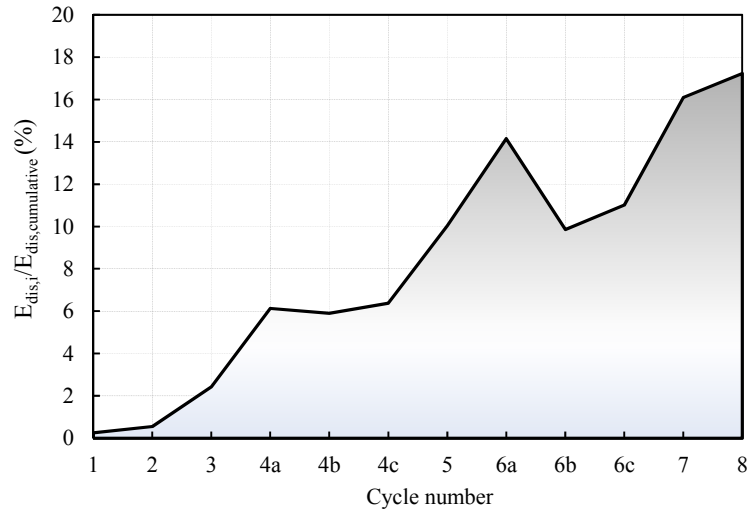


Figure C.160 : Ratio of the dissipated energy to total dissipated energy, S-50-C.

In addition to lateral load and displacement data, various other measurements were also collected from the specimen. Figure C.161 shows the variation of the vertical displacements measured at the tips of the top bond beam with respect to lateral displacement. As can be observed, the specimen tends to shorten throughout the lateral loading.

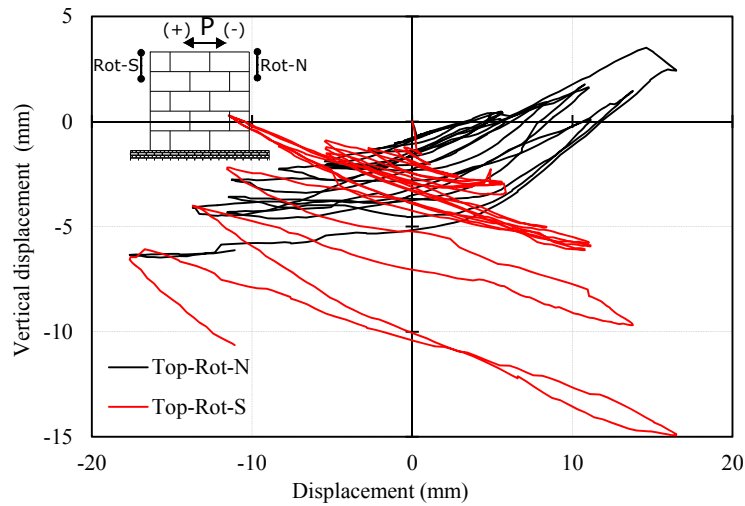


Figure C.161 : Variation of vertical displacements at the bond beam tips, S-50-C.

Vertical deformation measurements at toes and heels (depending on the loading direction) with respect to lateral displacements show that especially after 5.50 mm

horizontal displacement (0.5% drift ratio) vertical compressive deformations make a sharp increase due to crushing of the stone units at the toes, Figure C.162.

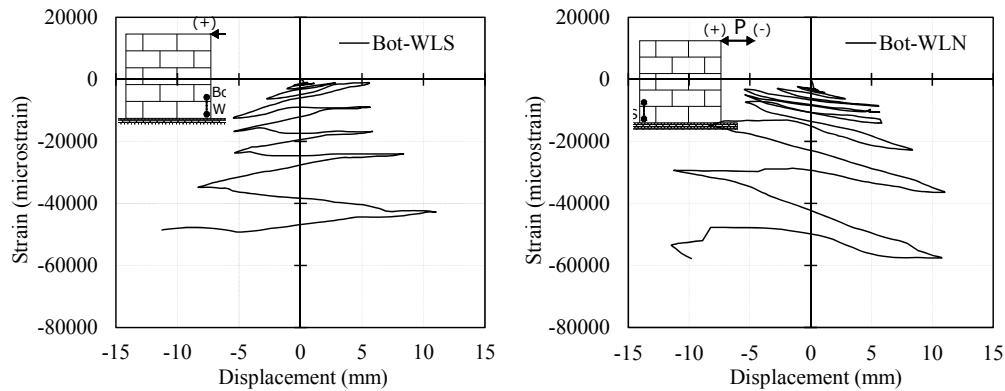


Figure C.162 : Vertical deformations at the toes and heels of the wall S-50-C.

As shown in Figure 3.20, three of the cramps on the Northern leaf of the wall were instrumented with strain gauges. Measurements logged for the cramp at the lower half of the specimen show that strains could reach 1000 microstrain which are definitely below the yield point of the steel, Figure C.163. The strain diagram turns out to be symmetric until the last loading cycle, which means that when upper side of the cramp is in compression the lower one is tension and vice versa. Consequently, if the average of these symmetric strains is calculated for each instrumented cramp, an average elongation value less than 300 microstrain is obtained for the duration of the whole test process, as seen in Figure C.164.

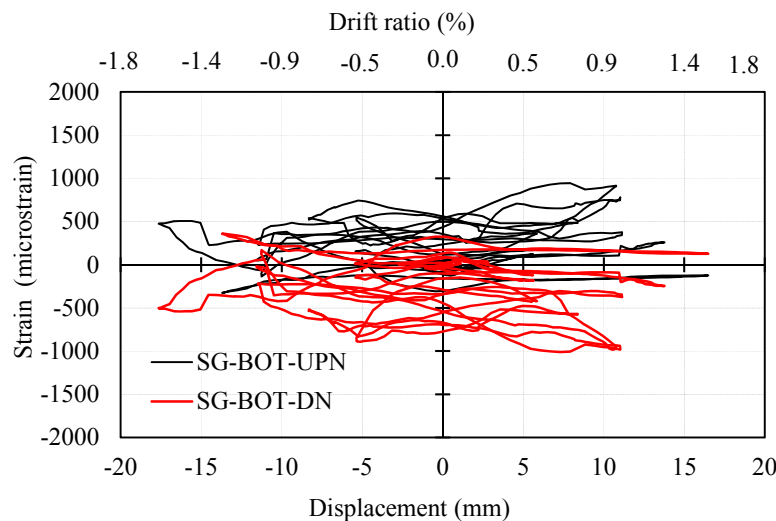


Figure C.163 : Strain variation for the cramp at lower half of the wall, S-50-C.

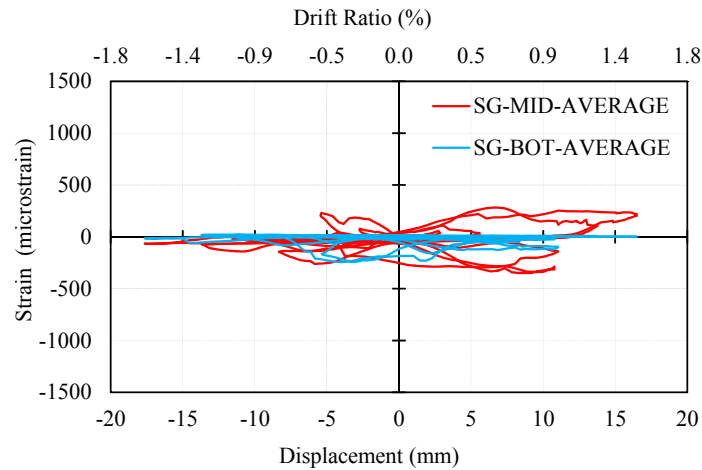


Figure C.164 : Average strains of instrumented cramps of wall S-50-C.

C.7 M-50-CP Wall

The M-50-CP specimen consisted of two ashlar outer leaves and a rubble masonry core. The stone units were connected to each other via horizontally positioned clamps and vertical pins. Axial stress level was kept as 0.50 MPa during the test. The test was conducted 212 days after the production of the M-50-CP wall.

C.7.1 Initial compression test (M-50-CP)

Four successive axial loading and unloading cycles were applied prior to reversed cyclic shear loading. The target axial load level was 180 kN. The axial stress-axial strain diagrams of these four cycles plotted by using the average of the measurements of the six LVDTs on both sides of the specimen are presented in Figure C.165.

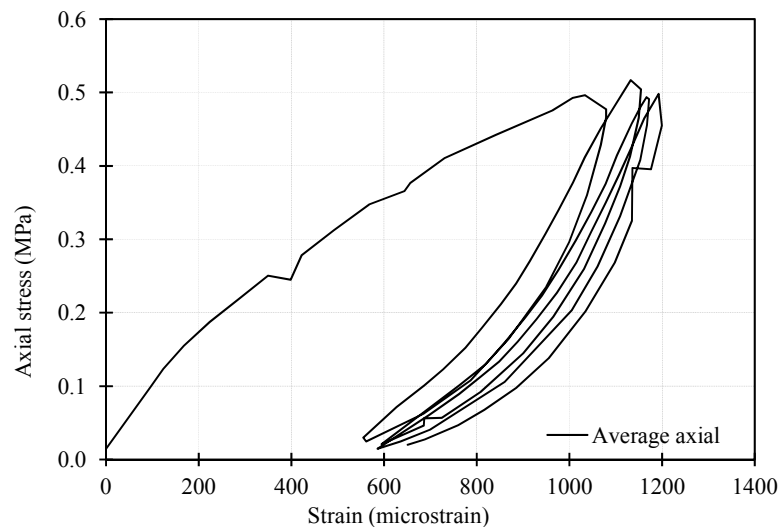


Figure C.165 : Axial stress-strain behaviour of M-50-CP specimen.

C.7.2 Shear compression test (M-50-CP)

After performing the pre-loading session that consisted of four successive axial loading cycles, all channels were initialized and hydraulic power supply of the lateral actuator was activated. The lateral loading began after applying the targeted 0.50 MPa axial load.

Up to 0.25% drift ratio (2.75 mm displacement), no significant damage formation could be observed, Figures C.166 and C.167. At the pushing and pulling parts of the 2.75 mm (0.25% drift ratio) cycle, first vertical cracks passing through the stone units occurred on the Northern and Southern faces of the wall, Figure C.168. During this cycle, first horizontal flexure cracks at the lower parts of the Eastern side could be observed, Figure C.169.

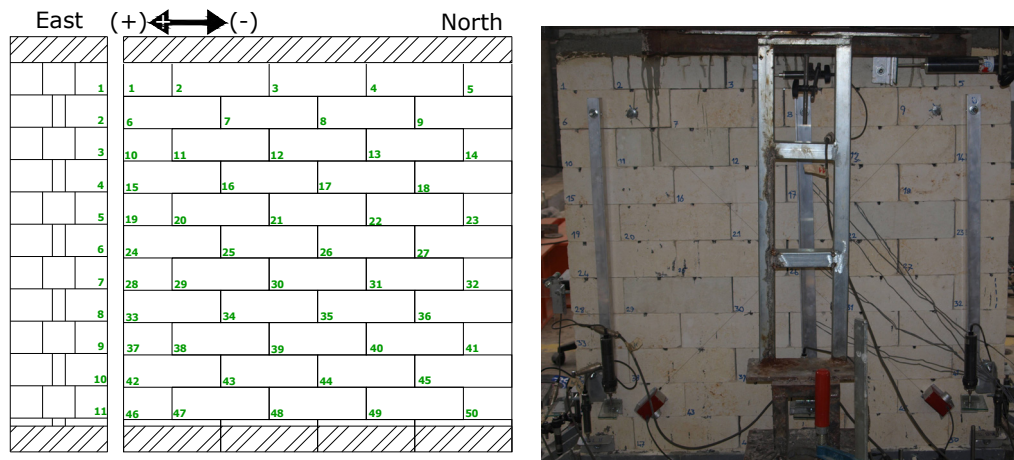


Figure C.166 : Cycle 1, M-50-CP after 0.55 mm displacement (0.05% drift ratio).

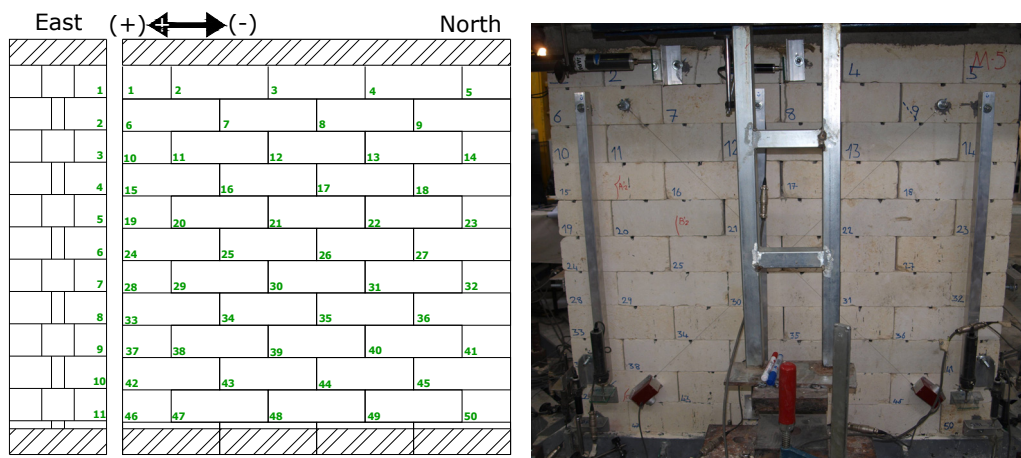


Figure C.167 : Cycle 2, M-50-CP after 1.10 mm displacement (0.10% drift ratio).

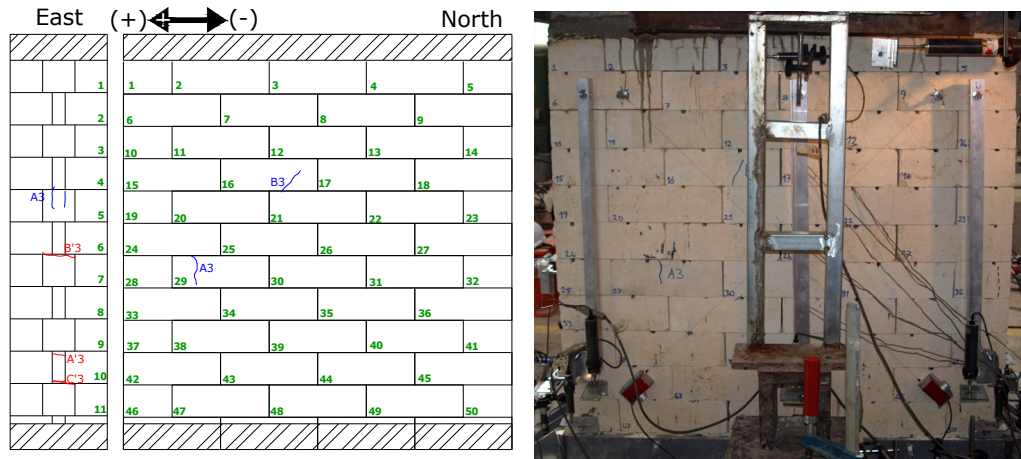


Figure C.168 : Cycle 3, M-50-CP after 2.75 mm displacement (0.25% drift ratio).

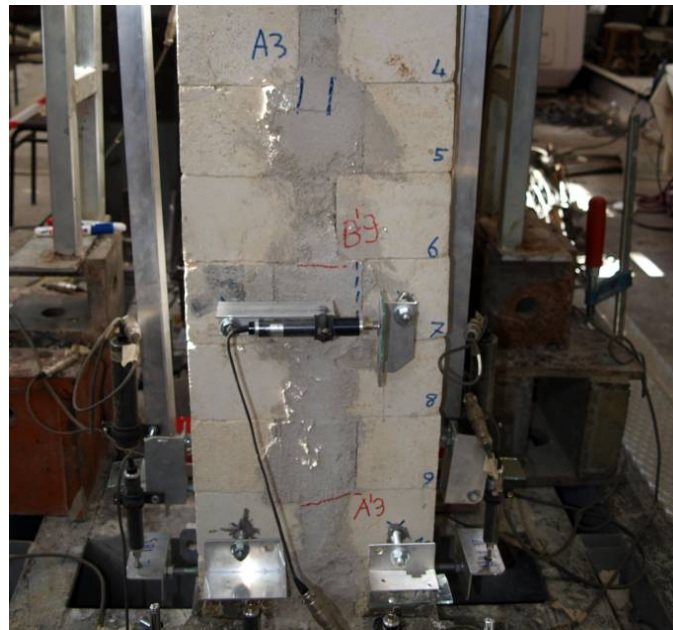


Figure C.169 : Cycle 3, M-50-CP after 2.75 mm displacement (0.25% drift ratio).

At the 5.50 mm cycle (0.50% drift ratio, cycle 4a), a number of vertical and diagonal cracks started to appear across the diagonals of each loading direction, Figure C.170. Many of these cracks were crossing from points where the pins or cramps were anchored, Figure C.171a. Maximum crack width at peaks of this cycle could reach 0.8 mm. Vertical cracks passing either through the inner and outer leaves interfaces or the rubble masonry became easily visible and extended upwards, Figure C.171b.

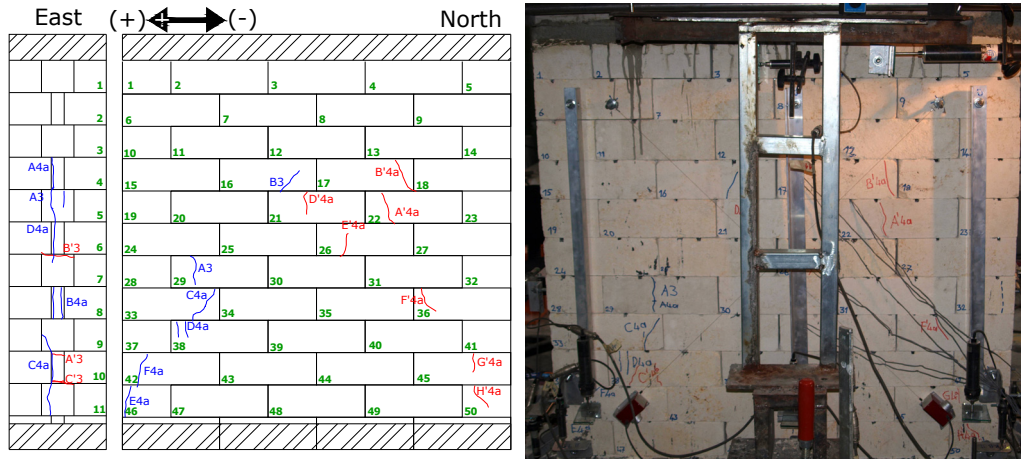


Figure C.170 : Cycle 4a, M-50-CP after 5.50 mm displacement (0.50% drift ratio).

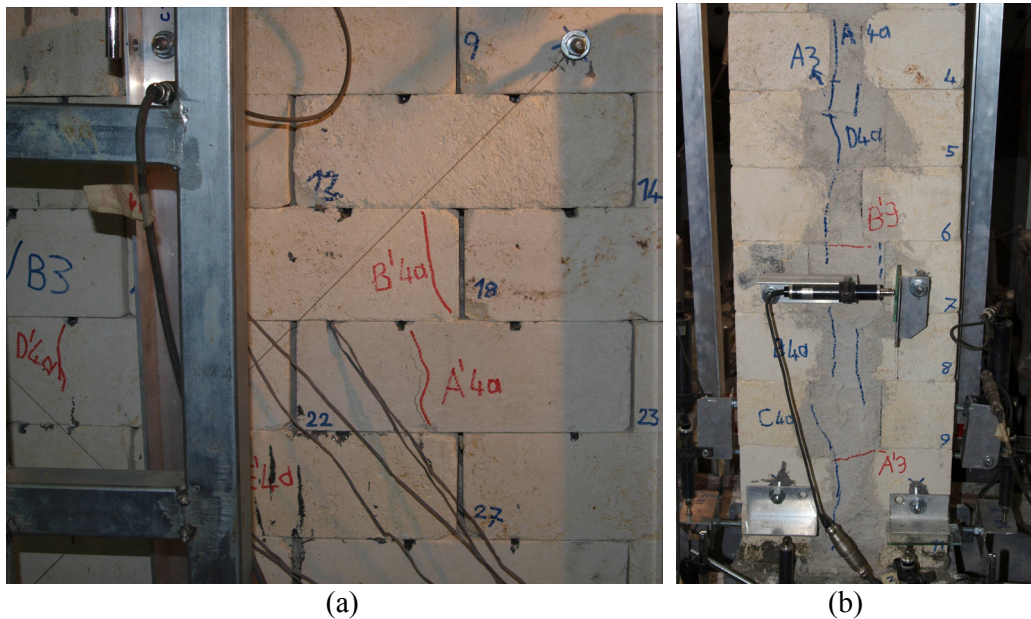


Figure C.171 : Cycle 4a, (a) cracks near the pins and cramps, (b) vertical cracks between the inner and outer leaves, M-50-CP.

During the second 5.50 mm cycle (cycle 4b, 0.50% drift ratio), more vertical and diagonal cracks passing through the stone units started to appear on a diagonal band, Figure C.172. The weak mortar in the rubble masonry mix started to crush at the Eastern and Western toes. Stone units at the toes started to show signs of crushing during the third cycle of 5.50 mm (0.50% drift ratio, cycle 4c). At this displacement cycle, widths of the cracks on the Northern and Southern sides could reach up to 2.0 mm, Figure C.173.

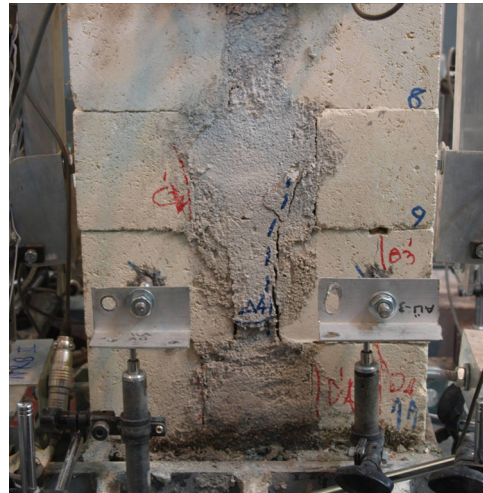
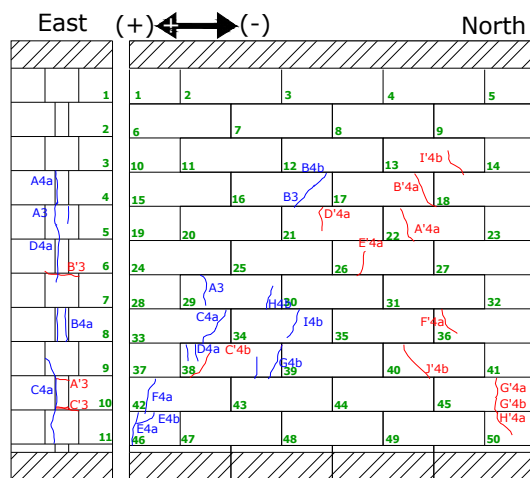


Figure C.172 : Cycle 4b, M-50-CP after 5.50 mm displacement (0.50% drift ratio).

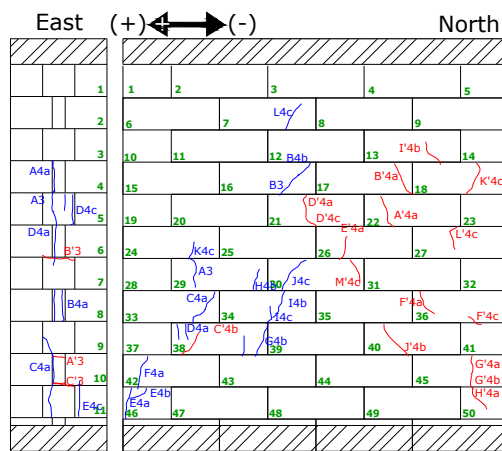


Figure C.173 : Cycle 4c, M-50-CP after 5.50 mm displacement (0.50% drift ratio).

During Cycle 5 (8.25 mm horizontal displacement, 0.75% drift ratio), the crushing of the stone units became evident at the toes of the wall. New cracks at the compressed areas and diagonal bands of the specimen appeared, Figure C.174. Length and width of the existing cracks extended. Maximum crack width could reach 3.5 mm thickness.

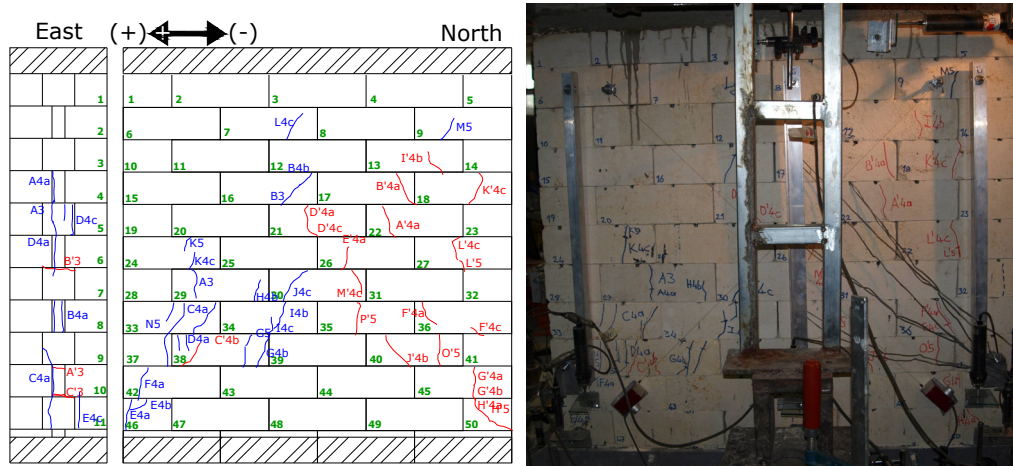


Figure C.174 : Cycle 5, M-50-CP after 8.25 mm displacement (0.75% drift ratio).

During the first 11.0 mm cycle (1.0% drift ratio, cycle 6a), number and width of the diagonal cracks increased noticeably (maximum crack width slightly over 5.0 mm), Figure C.175. Crushed stone units of the toes began spalling at the lowest stone course. Interestingly, apart from the shear dominated behaviour, horizontal cracks at the lower half of the specimen and bending type deformation of the wall became easily visible. The second and third cycles of the 1.0% drift ratio further extended the damage, Figures C.176 and C.177.

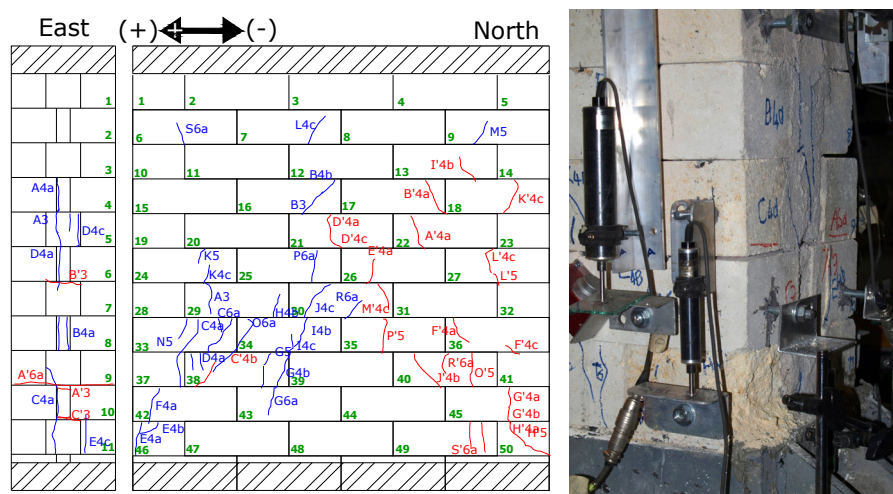
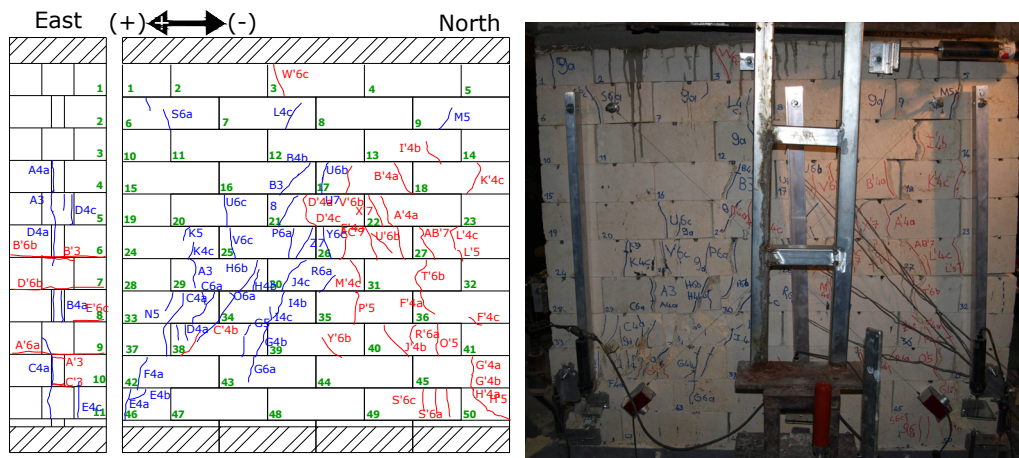
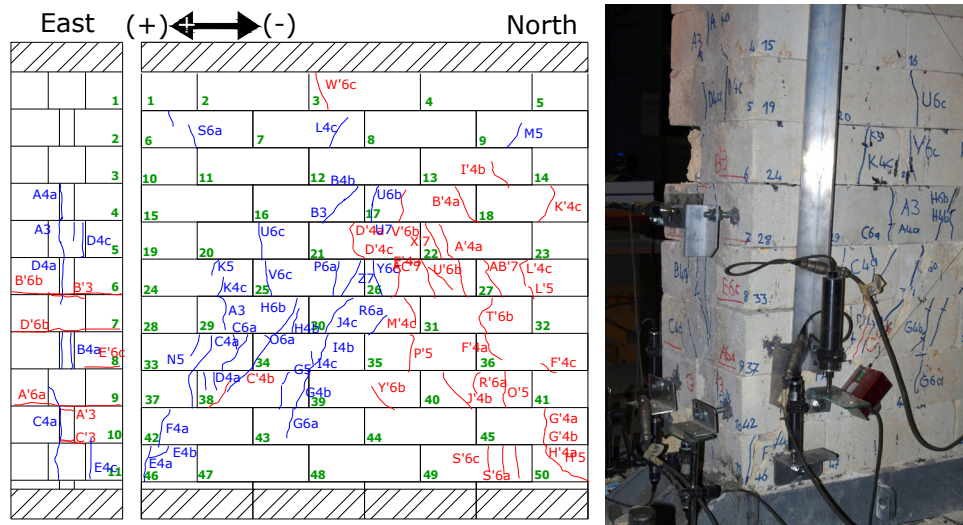


Figure C.175 : Cycle 6a, M-50-CP after 11.0 mm displacement (1.00% drift ratio).



After completing the pushing and pulling parts of the first 22.0 mm cycle (Cycle 9a, 2.0% drift ratio) and the pushing part of the second 22.0 mm cycle, the test was stopped due to 35% drop in the load bearing capacity (Figure C.180). After test views of the North, South, East and West faces of the specimen are presented in Figures C.181-C.183.

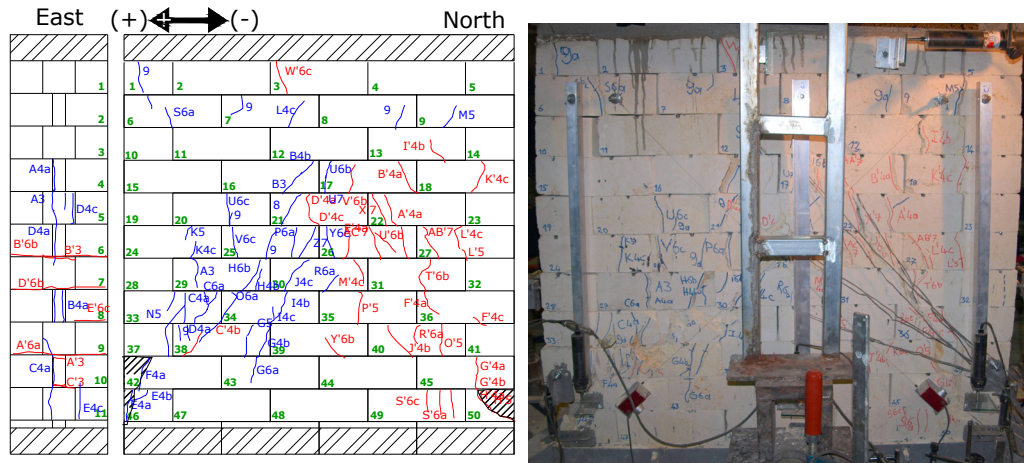


Figure C.180 : Cycle 9a, M-50-CP after 22.0 mm displacement (2.0% drift ratio).



Figure C.181 : Northern face of M-50-CP after 2.0% drift ratio.



Figure C.182 : Southern face of M-50-CP after 2.0% drift ratio.

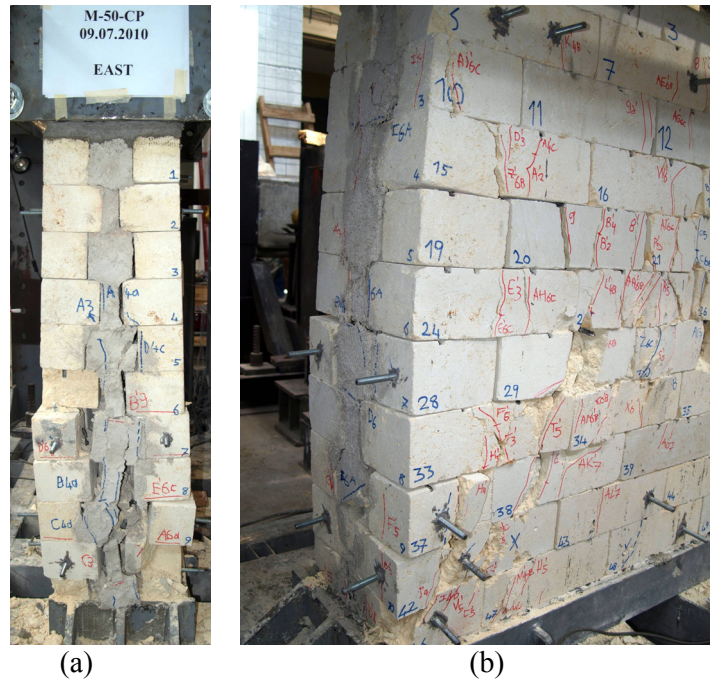


Figure C.183 : (a) Eastern and (b) Western faces of M-50-CP after 2.0% drift ratio.

Drift ratios, displacements, peak loads, peak load to maximum strength ratios (Peak Load/ H_{max}) and notes on damage development corresponding to each performed cycle are summarized in Table C.7. From this table it can be pointed out that, maximum load capacities of the M-50-CP specimen is attained at of 5.50 mm target displacement (0.50% drift ratio) for pushing and pulling directions. The damage evolution started with the horizontal bending cracks at lower levels of lateral displacement and followed by vertical and diagonal cracks passing through the stone units (especially at the toes). Cracks passing through the stone units were generally concentrated at mid-length of the units, at pins and the points where the cramps were bent and anchored to the stone units. Typical views for cracks crossing the pin sockets are presented in Figure C.184. From this figure it is also interesting to see that the molten lead could properly fill the gaps around the pins.

Damage evolution continued with the development of diagonal cracks. After the formation of the diagonal crack bands, upper diagonal halves of the wall started rocking. The test was stopped at 2.00% drift ratio after extensive spalling of the compressed parts that caused significant loss of strength (35% strength loss at 2.00% drift ratio).

Table C.7 : M-50-CP specimen summary table.

Cycle	Drift ratio (%)	Displacement (mm)	Peak Load (kN)	Peak Load/H _{max} (%)	Note
1. Push	0.05	0.55	35.1	33	
1. Pull	-0.05	-0.55	-37.0	42	
2. Push	0.10	1.10	58.6	56	
2. Pull	-0.10	-1.10	-54.1	62	
3. Push	0.25	2.75	88.6	85	
3. Pull	-0.25	-2.75	-75.0	86	First horizontal cracks due to flexure, first vertical cracks near the toes
4. Push (a)	0.50	5.50	103.9	99	First diagonal cracks, cracks between inner and outer leaves
4. Pull (a)	-0.50	-5.50	-85.6	98	First diagonal cracks, cracks between inner and outer leaves
4. Push (b)	0.50	5.50	104.6	100	New vertical and diagonal cracks on a diagonal band, crushing of rubble masonry
4. Pull (b)	-0.50	-5.50	-83.7	96	New vertical and diagonal cracks on a diagonal band, crushing of rubble masonry
4. Push (c)	0.50	5.50	100.9	96	Crushing of units started at the toe, number and width of cracks increased
4. Pull (c)	-0.50	-5.50	-80.7	92	Crushing of units started at the toe, number and width of cracks increased
5. Push	0.75	8.25	101.9	97	New diagonal cracks, crushing of stone units
5. Pull	-0.75	-8.25	-82.8	95	New diagonal cracks, crushing of stone units
6. Push (a)	1.00	11.00	95.4	91	Increased number of diagonal cracks, spalling at the Eastern toe, bending type deformation visible
6. Pull (a)	-1.00	-11.00	-82.0	94	Increased number of diagonal cracks, spalling at the Western toe, bending type deformation visible
6. Push (b)	1.00	11.00	93.3	89	Rocking, spalling at the East toe
6. Pull (b)	-1.00	-11.00	-82.4	94	Rocking, spalling at the West toe
6. Push (c)	1.00	11.00	92.0	88	Rocking, spalling at the East toe
6. Pull (c)	-1.00	-11.00	-83.7	96	Rocking, spalling at the West toe
7. Push	1.25	13.75	89.2	85	Upper diagonal half rocking, crushing and spalling on the diagonal band
7. Pull	-1.25	-13.75	-78.3	90	Upper diagonal half rocking, crushing and spalling on the diagonal band
8. Push	1.50	16.50	85.0	81	Upper diagonal half rocking
8. Pull	-1.50	-16.50	-75.3	86	Upper diagonal half rocking
9. Push (a)	2.00	22.00	82.3	79	Upper diagonal half rocking
9. Pull (a)	-2.00	-22.00	-65.2	75	Upper diagonal half rocking
9. Push (b)	2.00	22.00	68.1	65	Upper diagonal half rocking



Figure C.184 : Damage in the vicinity of pins, M-50-CP.

The M-50-CP specimen was further investigated after completion of the test by removing the stone units one by one. Despite the separation of inner and outer leaves, the damage pattern observed among them was almost coinciding, Figure C.185.



Figure C.185 : Damage on the outer and inner leaves, M-50-CP.

The lateral load-displacement hysteresis diagram describing the in-plane response of the M-50-CP specimen is presented in Figure C.186. Data points corresponding to first flexural cracking ($H_{f,cr}$), first diagonal cracking ($H_{dia,cr}$), maximum lateral resistance (H_{max}), 20% strength loss ($H_{0.8max}$) and ultimate displacement ($H_{d,max}$), are marked in the figure. The highly dissipative hysteresis loops show that the wall exhibited a significant nonlinear response with large residual displacements after

each load reversal. The envelope curve obtained for the M-50-CP specimen can be seen in Figure C.187. It should be noted that the lateral force-displacement curves in pushing and pulling directions were not symmetric and the bearing capacity in the pushing direction was higher.

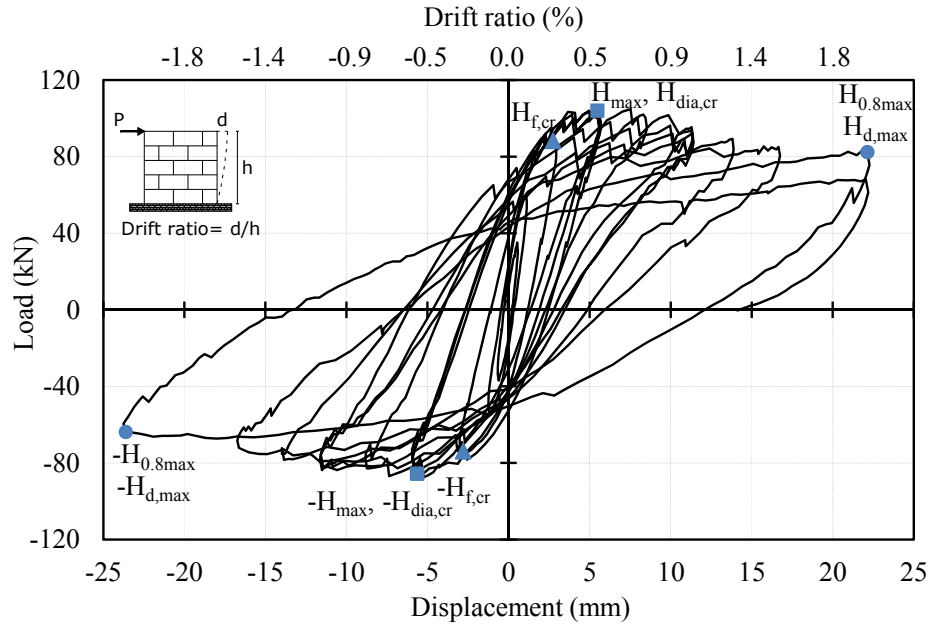


Figure C.186 : Lateral load-displacement diagram for specimen M-50-CP.

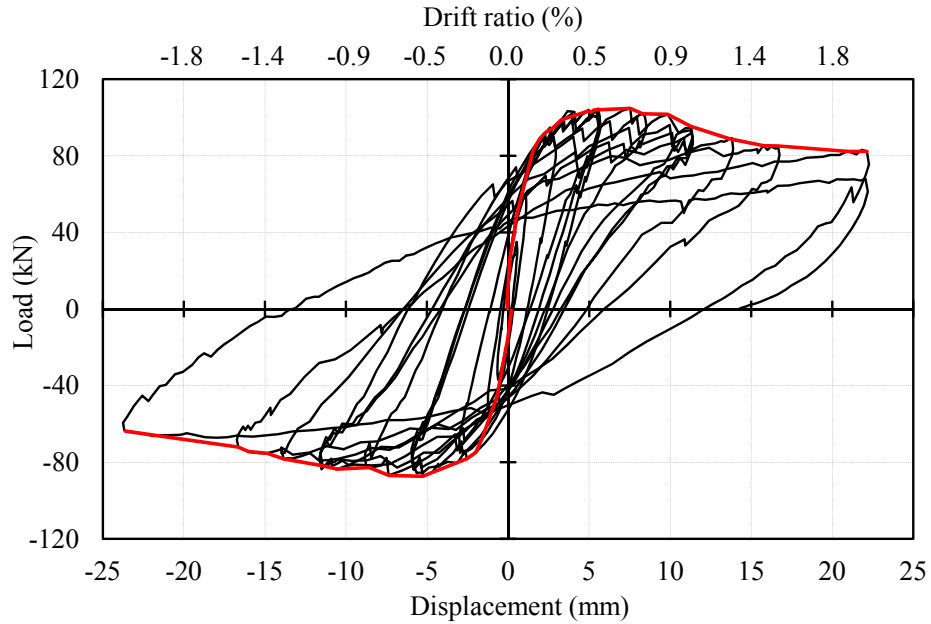


Figure C.187 : Envelope curve of lateral load-displacement diagram of M-50-CP.

The variation of the dissipated energy by means of the percentage of the total dissipated energy ($E_{dis,i}/E_{dis,cumulative}$, i denoting the cycle number) corresponding to each loading cycle is presented in Figure C.188. In this figure, the steep increase at Cycle 4 (0.50% drift ratio) corresponds to first diagonal cracks and initiation of toe crushing. With further formation of the diagonal cracks and crushing and spalling of the compressed zones, another sudden increase happens at Cycle 6 (1.00% drift ratio).

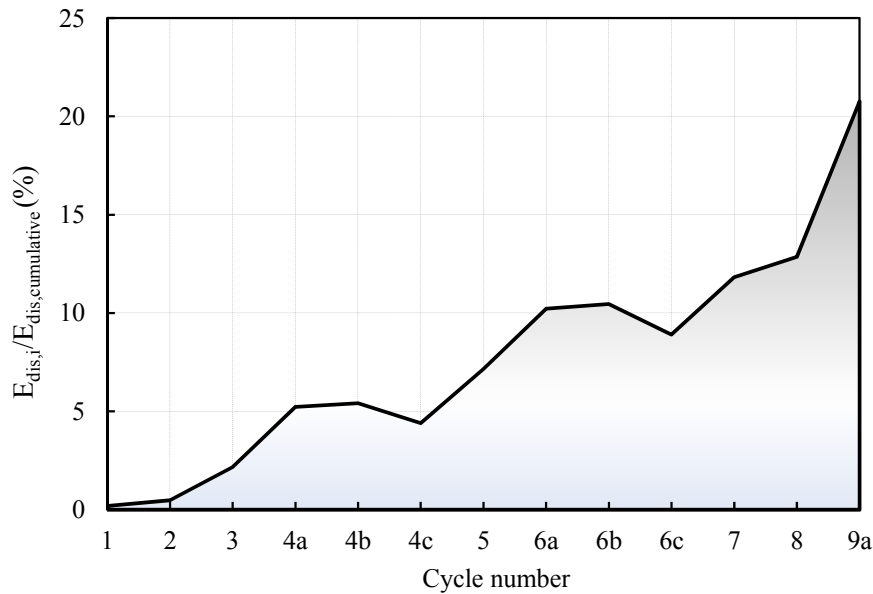


Figure C.188 : Ratio of dissipated energy to total dissipated energy, M-50-CP.

Apart from lateral load and displacement data, various other measurements were also collected from the specimen. Figure C.189 shows the variation of the vertical displacements measured at the tips of the top bond beam with respect to lateral displacement. As seen in this figure, a general tendency for compaction can be pronounced for larger lateral displacements. In parallel with the observations done during the test (especially after 11.0 mm lateral displacement), flexural effects cause shortening due to crushing of the lower stone courses and lengthening due to elongation of the zones under tension (horizontal cracks).

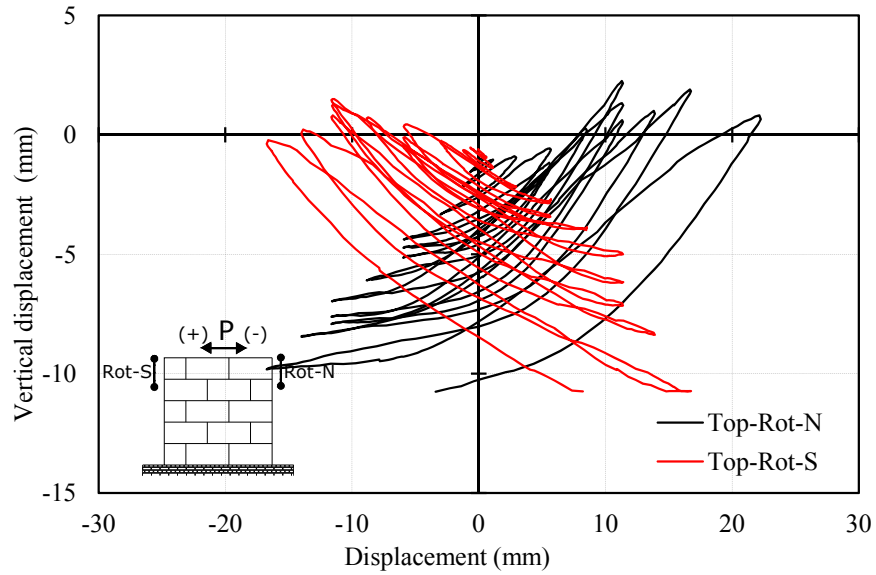


Figure C.189 : Variation of vertical displacements at the bond beam tips, M-50-CP.

Vertical deformations measured by channels 36, 37, 40 and 41 (as shown in Figures 3.19 and 3.20) with respect to lateral displacement shows that the horizontal cracks at Eastern and Western sides of the specimen increases significantly as the deformations occur, Figure C.190.

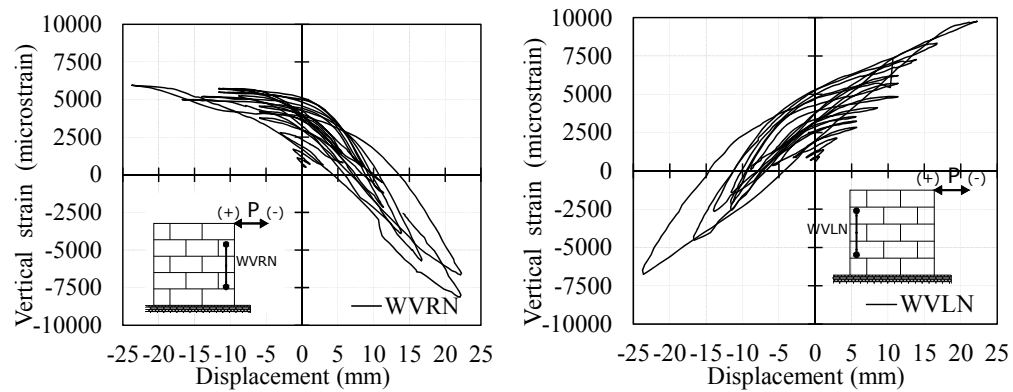


Figure C.190 : Vertical deformations near the Western and Eastern sides, M-50-CP.

Vertical deformation measurements at toes and heels (depending on the loading direction) with respect to lateral displacements show that up to 5.50 mm (0.50% drift ratio) these zones are subjected to reversible cyclic compression and tension deformations, Figure C.191. However, as also observed during the test, beginning from the 0.50% drift ratio vertical compressive deformations make a sharp increase due to initiation of the crushing of the stone units at the toes.

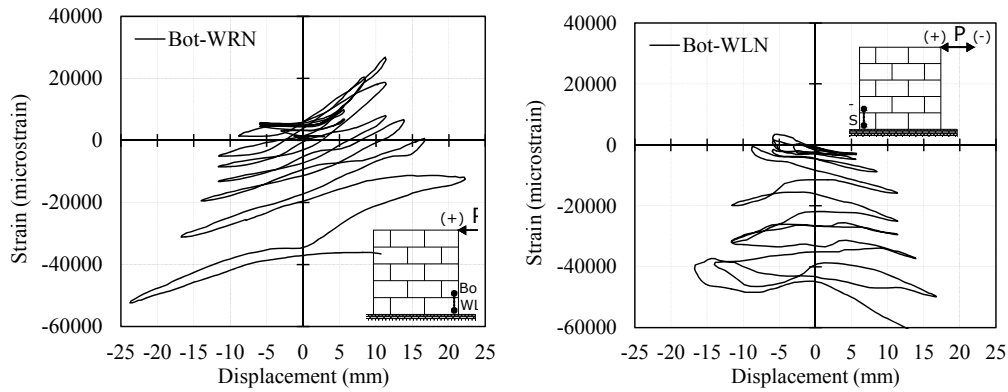


Figure C.191 : Vertical deformations at the lower corners of the wall M-50-CP.

Three of the cramps on the Northern leaf of the wall were instrumented with strain gauges as shown in Fig. 3.20. Measurements logged for the cramp at the lower half of the specimen show that strains could reach 400 microstrain which was quite below the yield point of the steel, Figure C.192.

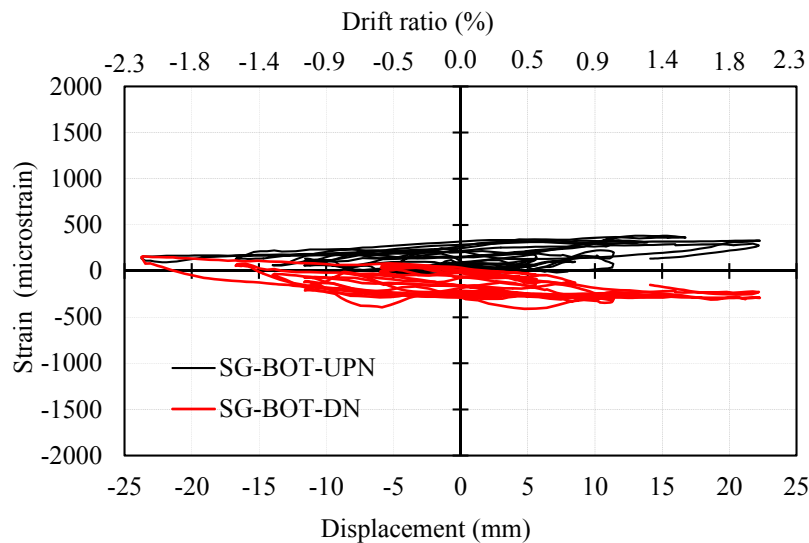


Figure C.192 : Strain variation for the cramp at lower half of the wall, M-50-CP.

Detachment between the external leaves of the wall was measured by two LVDTs positioned at mid-heights of the Eastern and Western faces of the specimen. As clearly seen from the transverse displacement versus lateral displacement diagram (Figure C.193) obtained for the Western side, particularly after the first 11.00 mm lateral displacement (1.00% drift ratio) separation between the outer and inner leaves increased rapidly.

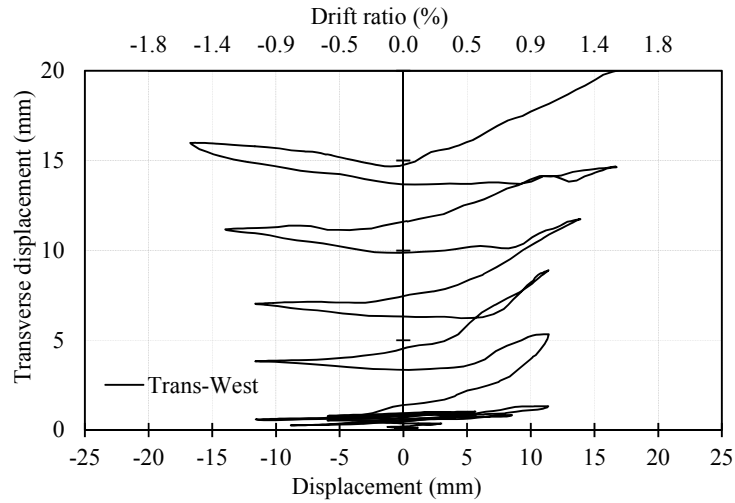


Figure C.193 : Detachment between the external leaves of the M-50-CP wall.

C.8 S-50-CP Wall

The S-50-CP specimen consisted of two ashlar leaves without a rubble masonry core. The stones were connected to each other via vertical pins and horizontally positioned cramps. The test was conducted 225 days after the production of the S-50-CP wall.

C.8.1 Initial compression test (S-50-CP)

Three successive axial loading and unloading cycles were applied prior to reversed cyclic shear loading. The target axial load level was 120 kN that corresponds to 0.50 MPa axial stress. The axial stress-axial strain diagram plotted by using the average displacement measurements of the six vertical LVDTs on both sides of the specimen are presented in Figure C.194.

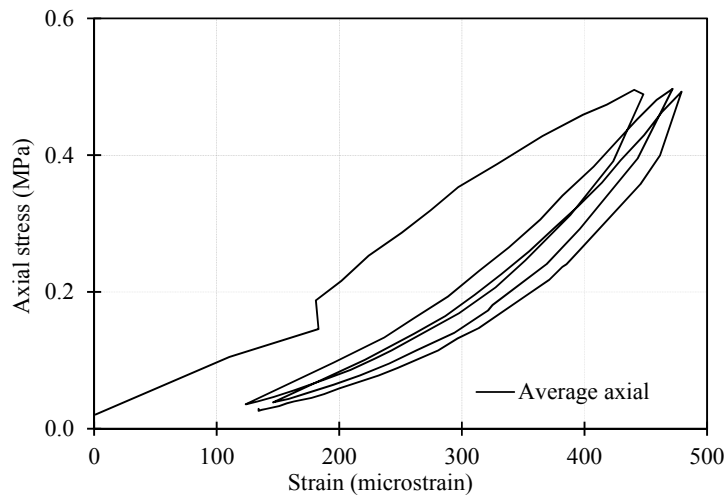


Figure C.194 : Axial stress-axial strain behaviour of S-50-CP specimen.

C.8.2 Shear compression test (S-50-CP)

After performing the pre-loading session that consisted of three successive axial loading cycles, all channels were initialized and hydraulic power supply of the lateral actuator was activated. The lateral loading began after applying the targeted 0.50 MPa axial load.

Up to 0.25% drift ratio (2.75 mm displacement), no significant damage could be observed except a few vertical cracks on the stone units which probably occurred during the axial loading due to stress concentrations around the imperfections of the bed joints, Figures C.195 and C.196.

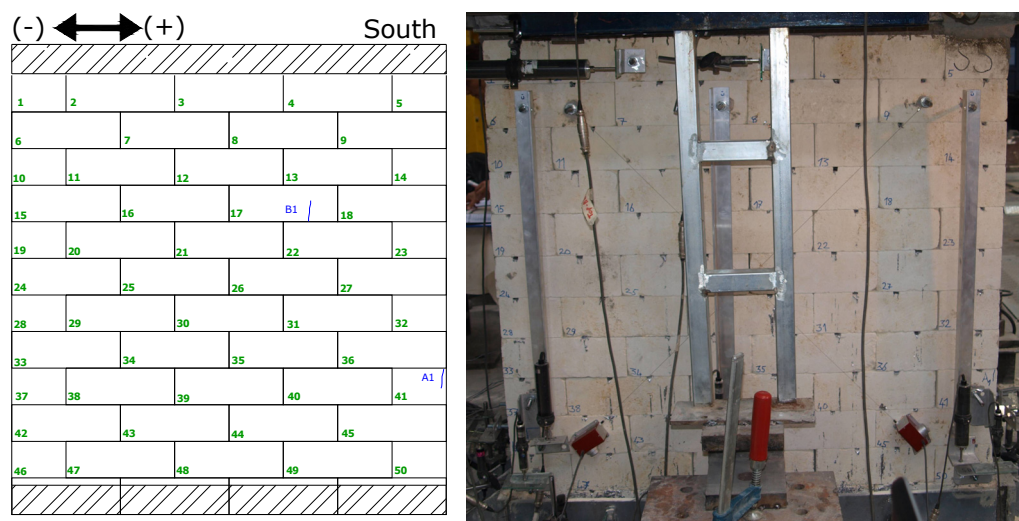


Figure C.195 : Cycle 1, S-50-CP after 0.55 mm displacement (0.05% drift ratio).

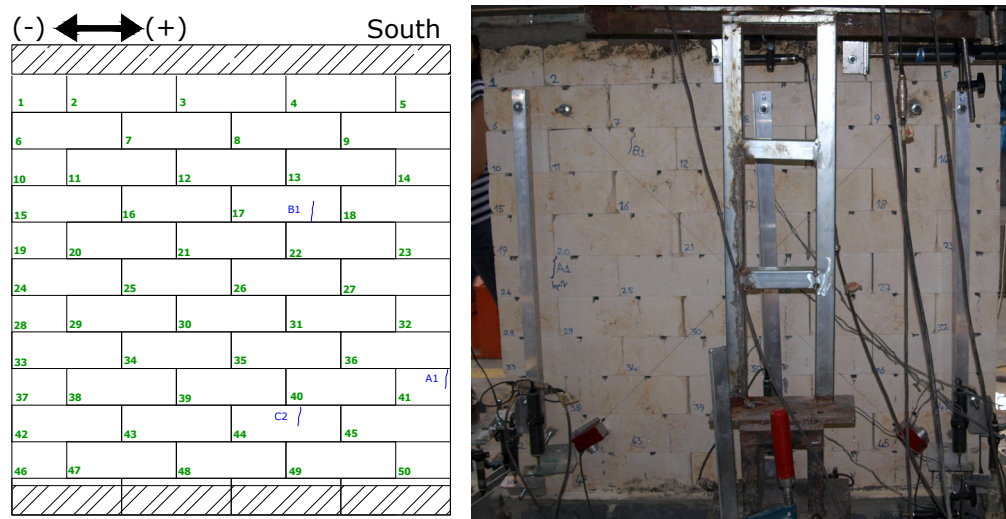


Figure C.196 : Cycle 2, S-50-CP after 1.10 mm displacement (0.10% drift ratio).

At the pushing and pulling parts of the 2.75 mm (0.25% drift ratio) cycle, more vertical cracks passing through the stone units occurred, Figure C.197. These cracks were not localized around the toes but distributed among the wall faces.

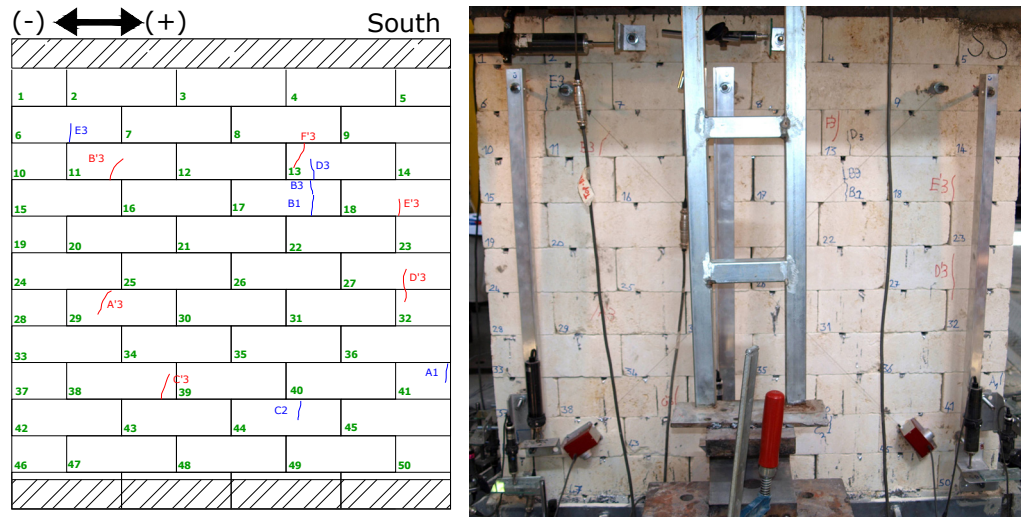


Figure C.197 : Cycle 3, S-50-CP after 2.75 mm displacement (0.25% drift ratio).

At 5.50 mm cycle (0.50% drift ratio, cycle 4a), a number of vertical and diagonal (mostly vertical) cracks appeared among the wall outer faces, Figure C.198. These cracks were randomly distributed among the outer faces of the wall. Maximum crack width at peaks of this cycle could reach 1.2 mm.

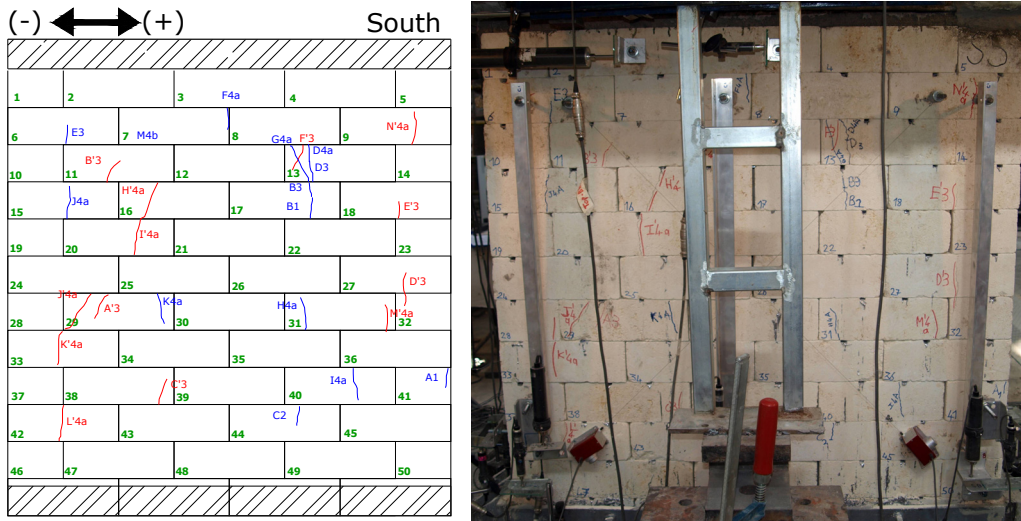


Figure C.198 : Cycle 4a, S-50-CP after 5.50 mm displacement (0.50% drift ratio).

During the second 5.50 mm cycle (0.50% drift ratio, cycle 4b), in addition to the increase in the number of vertical and diagonal crack passing through the stone units

(Figure C.199), crushing signs started to appear at the Eastern and Western sides of the wall.

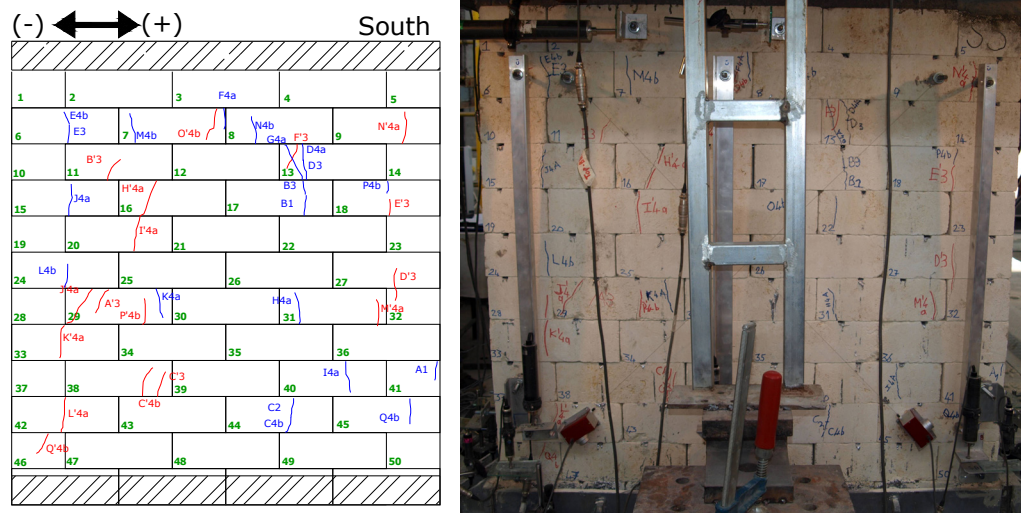


Figure C.199 : Cycle 4b, S-50-CP after 5.50 mm displacement (0.50% drift ratio).

During the third cycle of 5.50 mm (0.50% drift ratio, cycle 4c) crack widths could reach up to 3 mm. Most of the cracks passing through the stone units were either in the vicinity of the pins and cramps or at mid-length of the cut stones, Figure C.200.

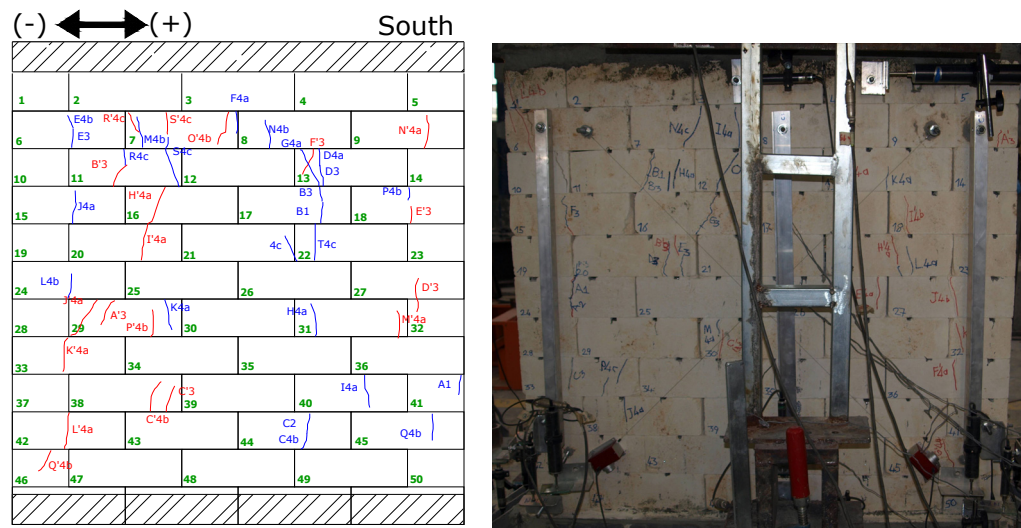


Figure C.200 : Cycle 4c, S-50-CP after 5.50 mm displacement (0.50% drift ratio).

During the Cycle 5, targeting 8.25 mm horizontal displacement (0.75% drift ratio), length and width of the existing cracks increased. Crushing of the stone units at the toes of pulling and pushing loading directions became easily visible, Figure C.201.

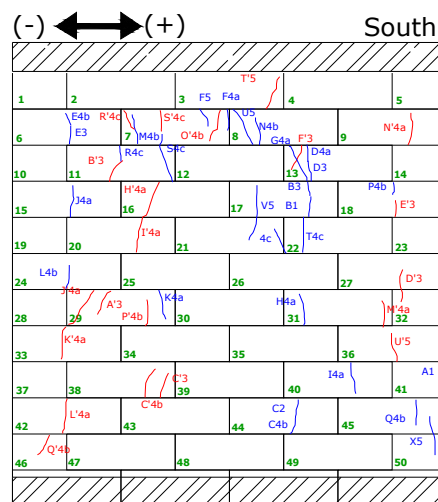


Figure C.201 : Cycle 5, S-50-C after 8.25 mm displacement (0.75% drift ratio).

During the first 11.0 mm cycle (1.0% drift ratio, cycle 6a), number and width of the diagonal cracks increased noticeably, so that the diagonal crack could be easily identified, Figure C.202. Maximum crack width was in the order of 15 mm. Following the failure at some of the cramp anchors, some of the head joints expanded especially on the Northern face of the wall (marked with white arrows on Figure C.202). Crushed stone units around the toes began spalling, especially at the Eastern toe, which was under compression during the pushing direction loading.

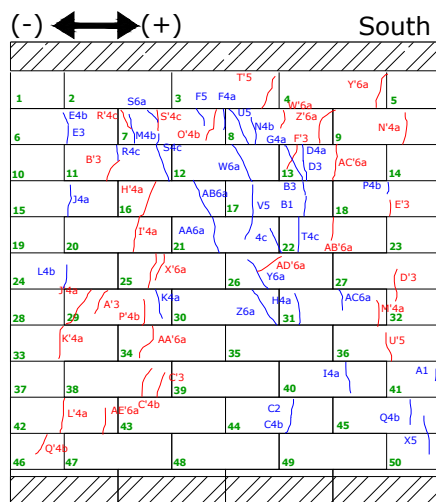


Figure C.202 : Cycle 6a, S-50-C after 11.0 mm displacement (1.00% drift ratio).

The second and third cycles of the 1.0% drift ratio further extended the damage, Figures C.203 and C.204. At the displacement reversals of 1.0% drift ratio, maximum opening at the head joints reached 25 mm and spalling at the toes

continued. The horizontal load capacity dropped more than 20% in the pushing direction, whose Eastern toe was more damaged than the pulling direction's Western toe. However, the test was continued since the capacity in the pulling direction was not affected.

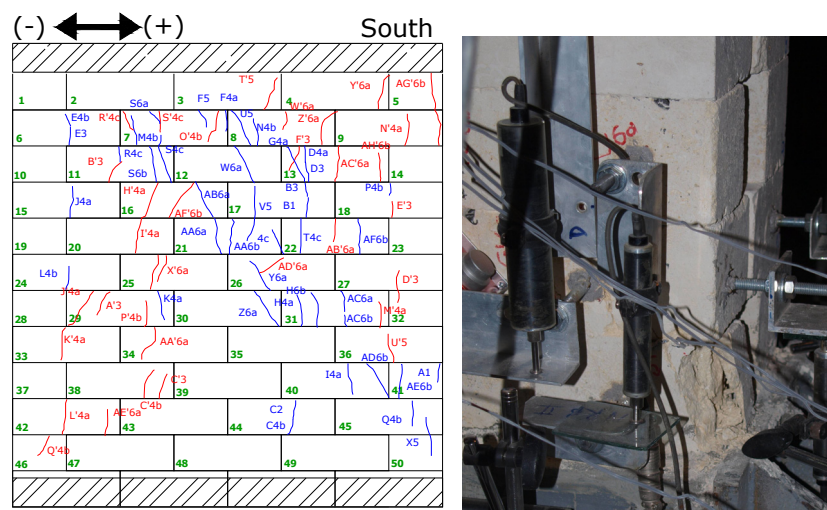


Figure C.203 : Cycle 6b, S-50-CP after 11.0 mm displacement (1.00% drift ratio).

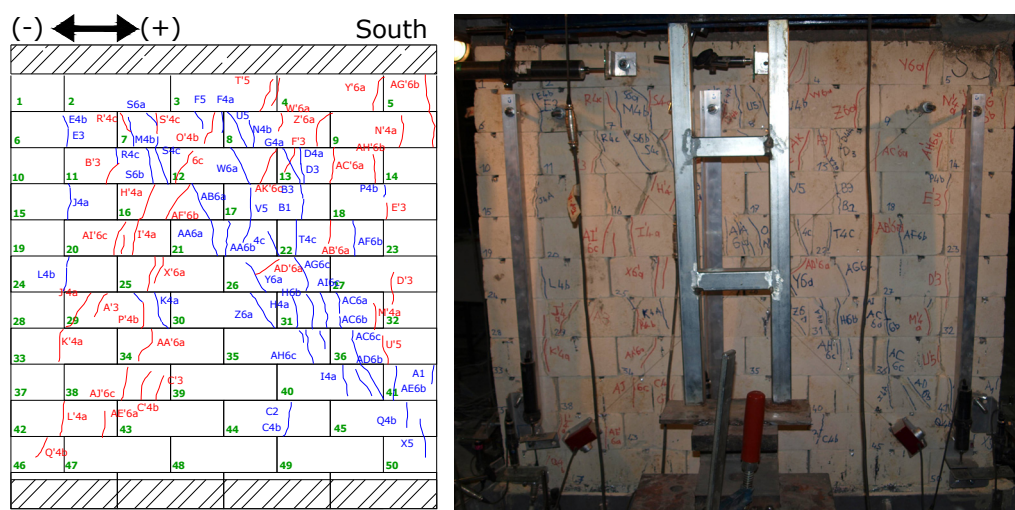


Figure C.204 : Cycle 6c, S-50-CP after 11.0 mm displacement (1.0% drift ratio).

The cracking and crushing of stone units along the diagonals of the specimen continued during the 7th and 8th cycles (corresponding to 1.25 and 1.50% drift ratios, respectively), Figures 17 and 18. Rocking of the upper diagonal halves of the wall, around the crushing and spalling toes of each respective loading direction, was apparent. The horizontal cracks between the stone courses (such as the one marked in Figure 17) could reach 10 mm width. It should be noted that the crushing of the

stone units was not only observed around the toes but also along the diagonal compression struts.

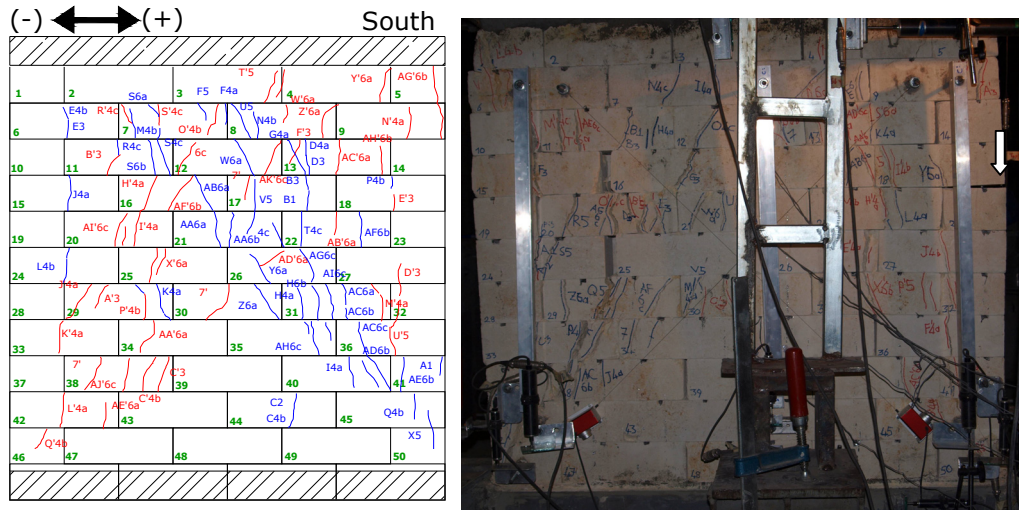


Figure C.205 : Cycle 7, S-50-CP after 13.75 mm displacement (1.25% drift ratio).

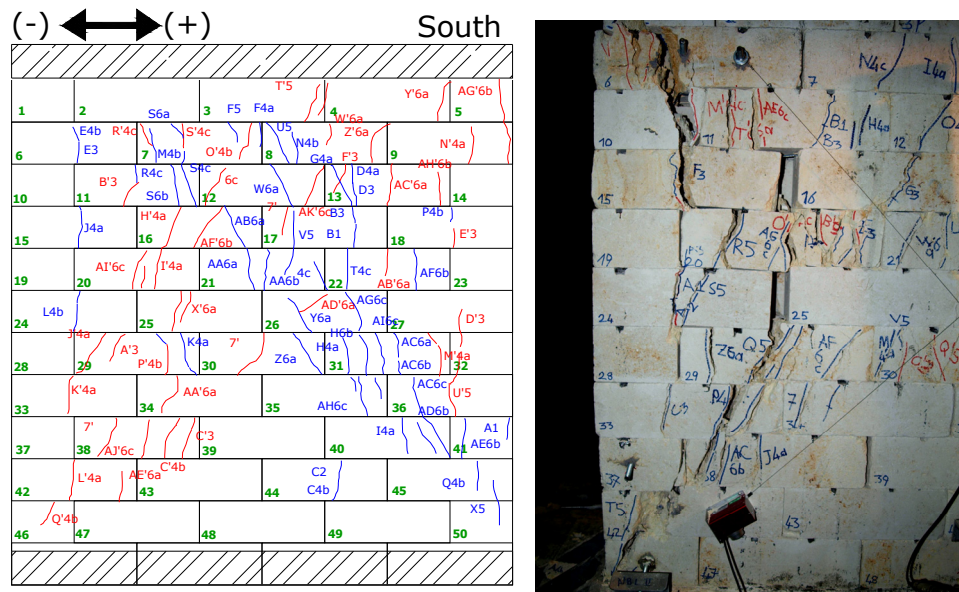


Figure C.206 : Cycle 8, S-50-CP after 16.50 mm displacement (1.50% drift ratio).

After completing the pushing and pulling parts of the first 22.0 mm cycle (Cycle 9a, 2.0% drift ratio), the test was stopped due to extensive damage and 30% drop in the load bearing capacity. At this cycle, the maximum crack width could reach 40 mm width, Figure C.207. After test views of the North, South, East and West faces of the specimen are presented in Figures C.208-C.210.

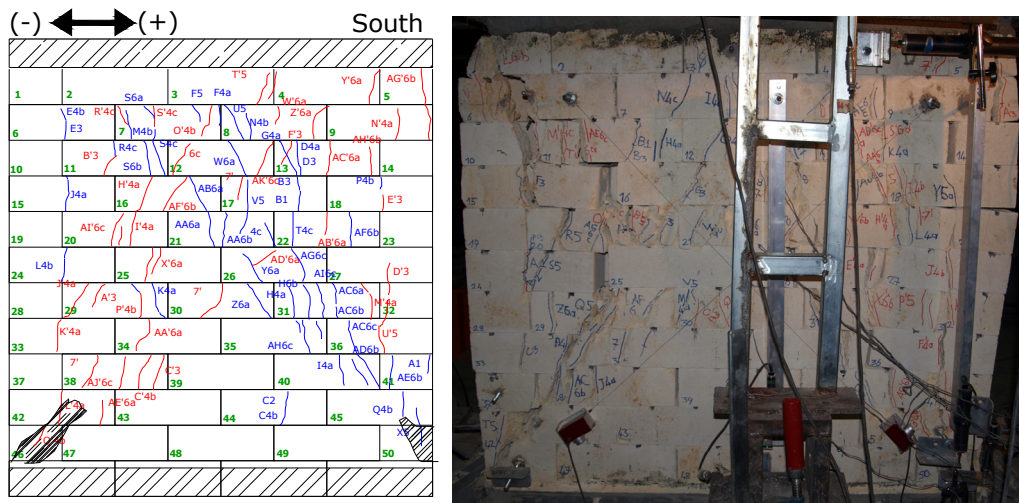


Figure C.207 : Cycle 9a, S-50-CP after 22.00 mm displacement (2.00% drift ratio).



Figure C.208 : Cycle 9a, North face of S-50-CP after 1.50% drift ratio.



Figure C.209 : Cycle 8, South face of S-50-CP after 1.50% drift ratio.

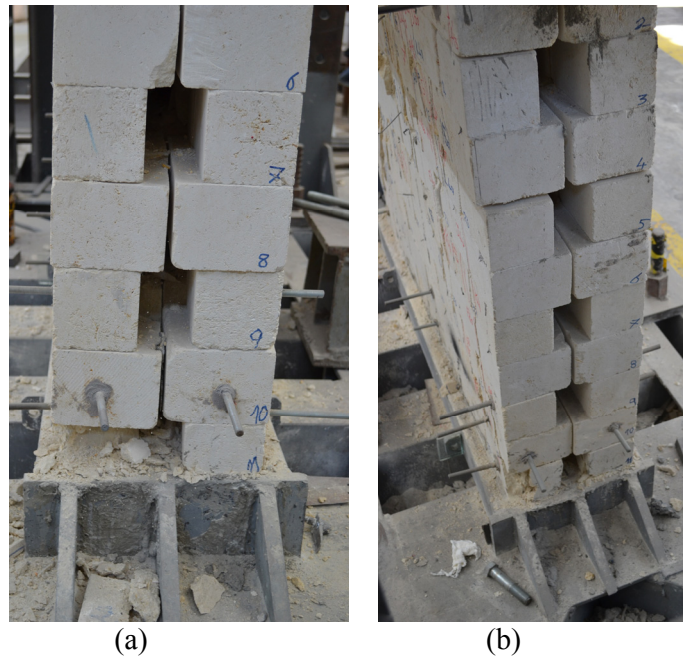


Figure C.210 : Cycle 8, (a) East and (b) West faces of S-50-CP at the end of the test.

Drift ratios, displacements, peak loads, peak load to maximum strength ratios (Peak Load/ H_{max}) and notes on damage development corresponding to each performed cycle are summarized in Table C.8. From this table it can be pointed out that

maximum load capacities (67.8 kN for pushing, -56.4 kN for pulling directions) of the S-50-CP specimen are attained at 5.50 mm target displacement (0.50% drift ratio) for pushing and pulling directions.

Table C.8 : S-50-CP specimen summary table.

Cycle	Drift ratio (%)	Displacement (mm)	Peak Load (kN)	Peak Load/H _{max} (%)	Note
1. Push	0.05	0.55	22.2	33	Few vertical cracks
1. Pull	-0.05	-0.55	-28.3	49	Few vertical cracks
2. Push	0.10	1.10	39.1	58	Few vertical cracks
2. Pull	-0.10	-1.10	-37.6	65	Few vertical cracks
3. Push	0.25	2.75	54.4	80	Vertical cracks
3. Pull	-0.25	-2.75	-54.0	93	Vertical cracks
4. Push (a)	0.50	5.50	67.8	100	Vertical cracks on a diagonal band, a few diagonal cracks
4. Pull (a)	-0.50	-5.50	-56.4	100	Vertical cracks on a diagonal band, a few diagonal cracks
4. Push (b)	0.50	5.50	67.6	99	Crushing signs, new vertical and diagonal cracks
4. Pull (b)	-0.50	-5.50	-48.6	84	Crushing signs, new vertical and diagonal cracks
4. Push (c)	0.50	5.50	65.4	96	Number and width of cracks increased
4. Pull (c)	-0.50	-5.50	-47.2	82	Number and width of cracks increased
5. Push	0.75	8.25	65.8	97	Crushing at toe
5. Pull	-0.75	-8.25	-54.9	95	Crushing at toe
6. Push (a)	1.00	11.00	64.6	95	Diagonal cracks easily visible, spalling of stone units at the toe
6. Pull (a)	-1.00	-11.00	-52.9	91	Diagonal cracks easily visible, spalling of stone units at the toe
6. Push (b)	1.00	11.00	53.3	79	Spalling at the toe-East side, sudden loss of strength
6. Pull (b)	-1.00	-11.00	-52.5	91	Spalling at the toe-West side
6. Push (c)	1.00	11.00	49.8	73	Spalling at the toe-East side
6. Pull (c)	-1.00	-11.00	-53.0	92	Spalling at the toe-West side
7. Push	1.25	13.75	48.7	72	Upper diagonal half rocking, spalling at the toe
7. Pull	-1.25	-13.75	-53.5	92	Upper diagonal half rocking, spalling at the toe
8. Push	1.50	16.50	53.3	79	Upper diagonal half rocking, spalling at the toe
8. Pull	-1.50	-16.50	-50.3	87	Upper diagonal half rocking, spalling at the toe
9. Push (a)	2.00	22.00	46.6	69	Upper diagonal half rocking, spalling at the toe
9. Pull (a)	-2.00	-22.00	-39.1	68	Upper diagonal half rocking, spalling at the toe

The lateral load-displacement hysteresis diagram describing the in-plane response of the S-50-CP specimen is presented in Figure C.211. Data points corresponding to first diagonal cracking ($H_{dia,cr}$), maximum lateral resistance (H_{max}), 20% strength loss ($H_{0.8max}$) and ultimate displacement ($H_{d,max}$), are marked in the figure. The highly dissipative hysteresis loops show that the wall exhibited a significant nonlinear response with large residual displacements after each load reversal. The envelope curve obtained for the S-50-C specimen can be seen in Figure C.212.

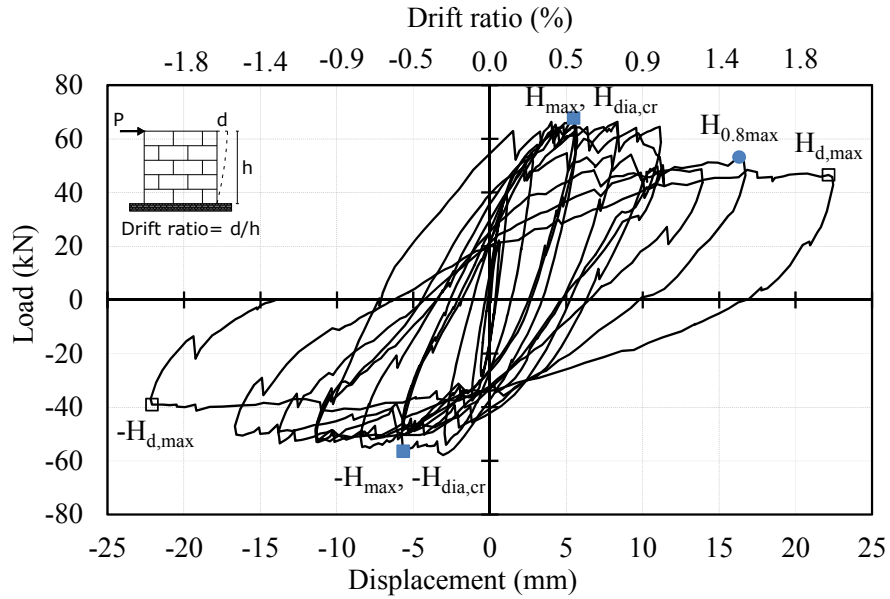


Figure C.211 : Lateral load-displacement diagram for specimen S-50-CP.

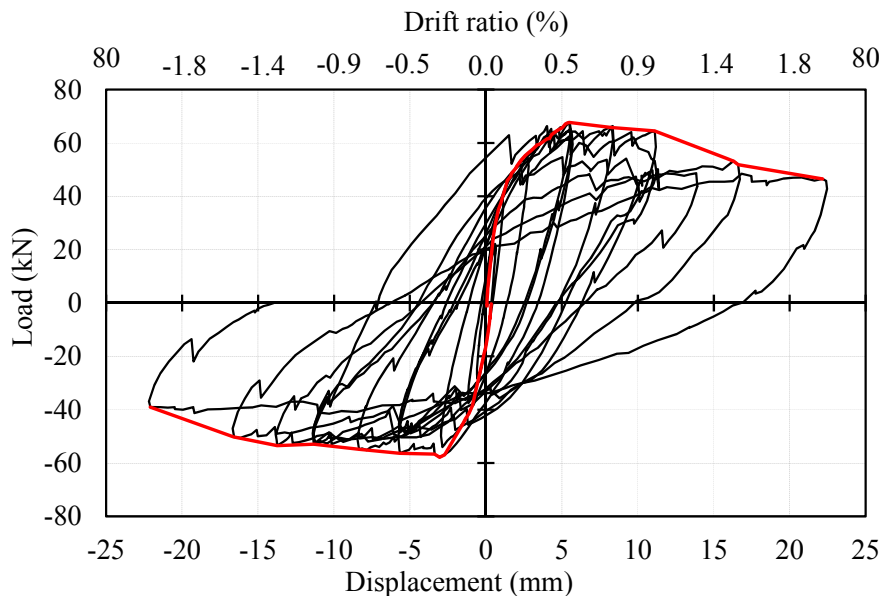


Figure C.212 : Envelope curve of lateral load-displacement diagram of S-50-CP.

The variation of the dissipated energy by means of the percentage of the total dissipated energy ($E_{dis,i}/E_{dis,cumulative}$, i denoting the cycle number) corresponding to each loading cycle is presented in Figure C.213. In this figure, the steep increase at Cycle 4 (0.50% drift ratio) corresponds to first diagonal cracks. With further formation of the diagonal cracks and crushing and spalling of the compressed zones, another sudden increase happens at Cycle 6 (1.00% drift ratio). With the initiation of the rocking behaviour extensive damage occurs so that the dissipated energy reaches its last peak value at Cycle 9a (2.00% drift ratio).

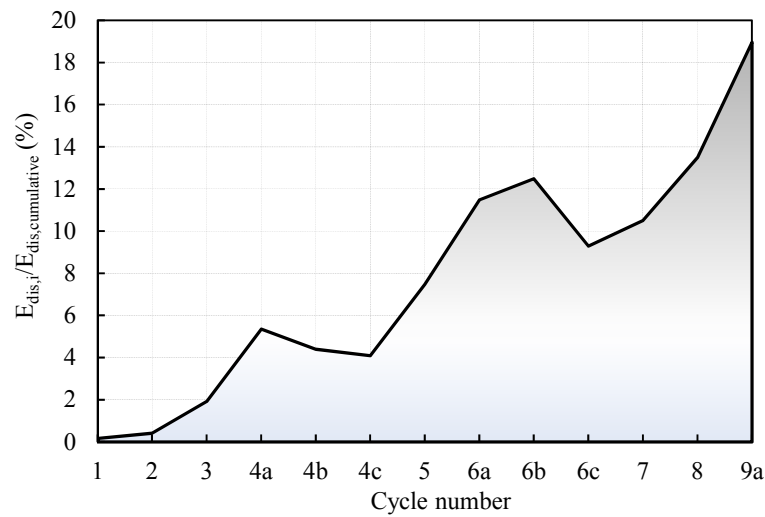


Figure C.213 : Ratio of dissipated energy to total dissipated energy, S-50-CP.

Figure C.214 shows the variation of the vertical displacements measured at the tips of the top bond beam with respect to lateral displacement. As can be observed, the specimen (especially the southern leaf) tends to compact during the lateral loading.

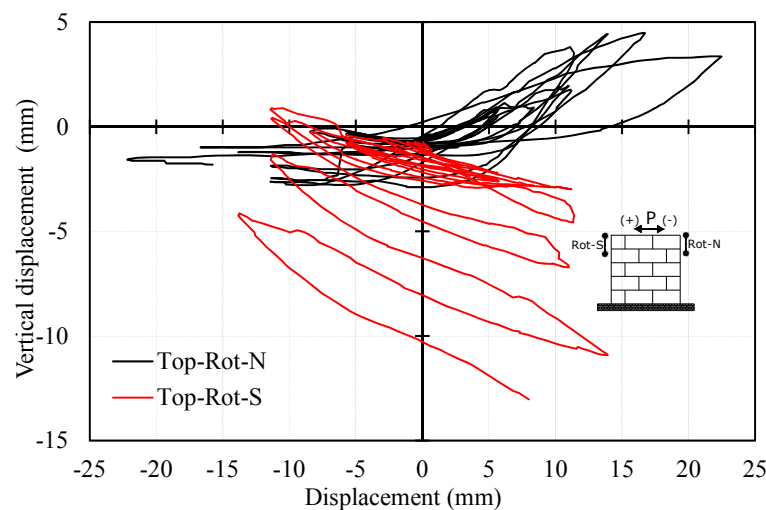


Figure C.214 : Variation of vertical displacements at the bond beam tips, S-50-CP.

Vertical deformation measurements at toes and heels (depending on the loading direction) with respect to lateral displacements show that especially after 5.50 mm horizontal displacement (0.5% drift ratio) vertical compressive deformations make a sharp increase due to crushing of the stone units at the toes, Figure C.215.

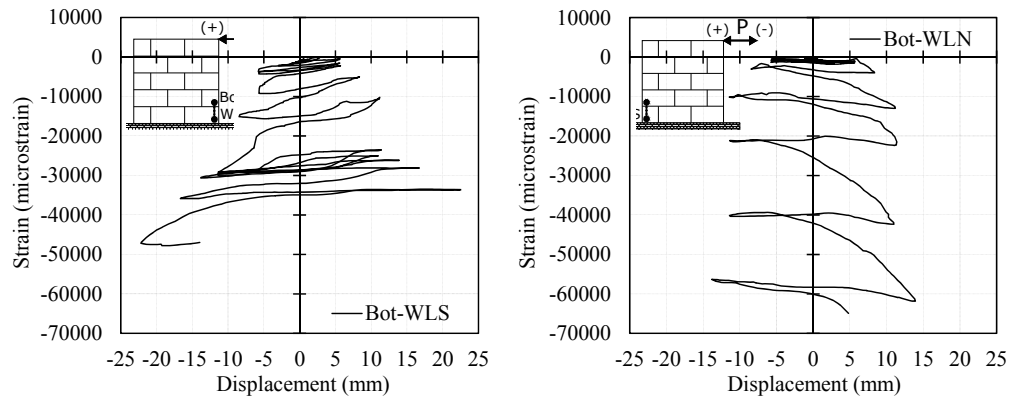


Figure C.215 : Vertical deformations at the toes and heels of the wall S-50-CP.

Three of the cramps on the Northern leaf of the wall were instrumented with strain gauges. Measurements logged for the cramp at mid-height of the specimen show that strains could reach 3000 microstrain value (which is above the 2000 microstrain yielding strain limit) during the last lateral displacement cycle, Figure C.216. Prior to the 22.0 mm (2.0% drift ratio) none of the cramps could reach the yielding point. The strain diagram turns out to be symmetric, which means that when upper side of the cramp is in compression the lower one is tension and vice versa. Consequently, if the average of these symmetric strains is calculated for each instrumented cramp, an average elongation value less than 300 microstrain is obtained for the duration of the whole test process, as seen in Figure C.217.

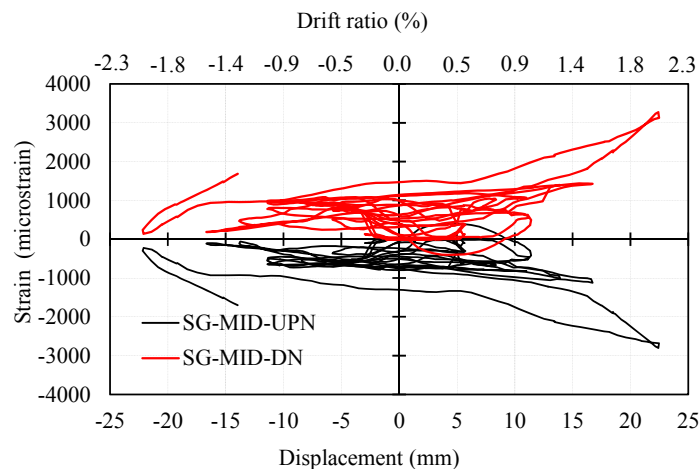


Figure C.216 : Strain variation for mid-height cramp, S-50-CP.

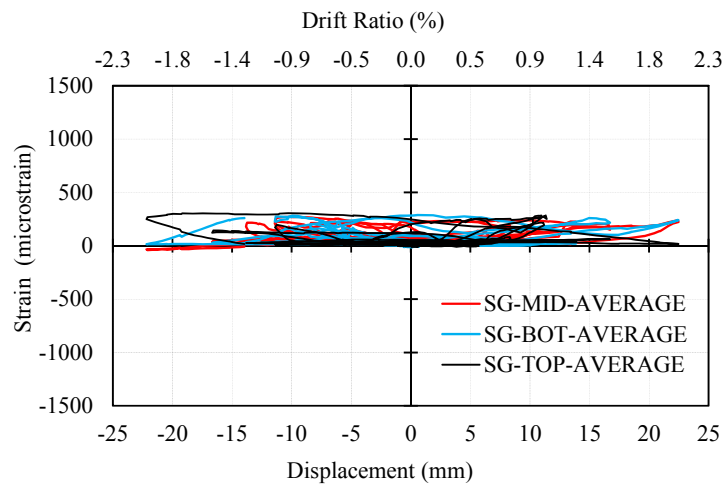


Figure C.217 : Average strains of instrumented cramps of wall S-50-CP.

APPENDIX D: Principal Stress and Principal Plastic Strain Distributions Obtained by Micro-Modelling

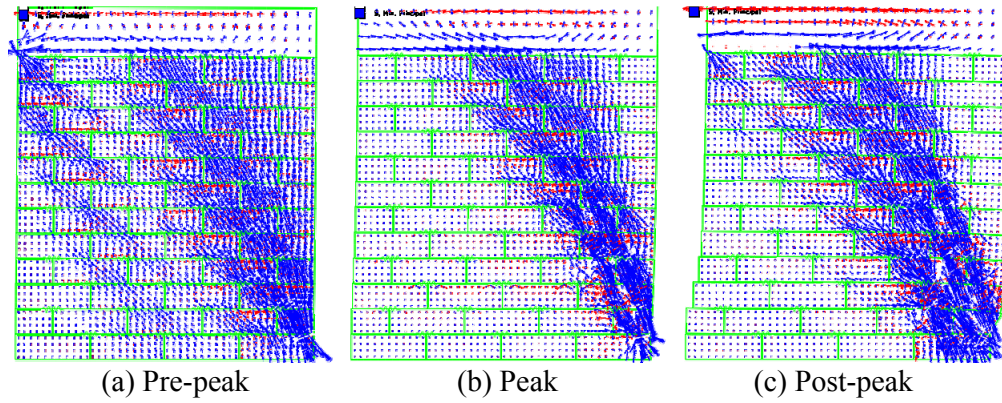


Figure D.1 : Principal stress distributions of S-50-C model.

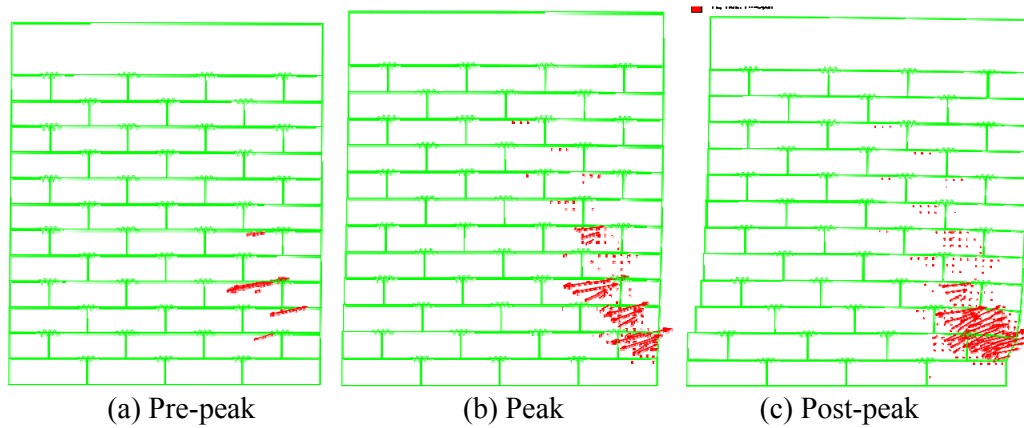


Figure D.2 : Maximum principal plastic strain distributions of S-50-C model.

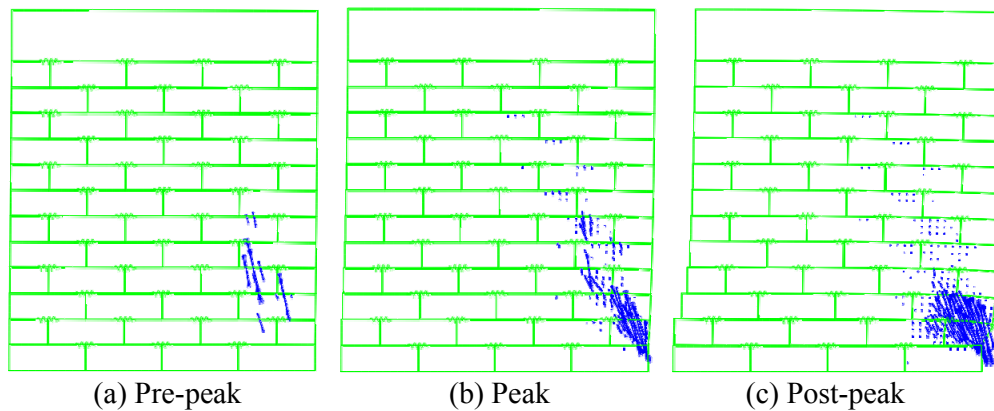
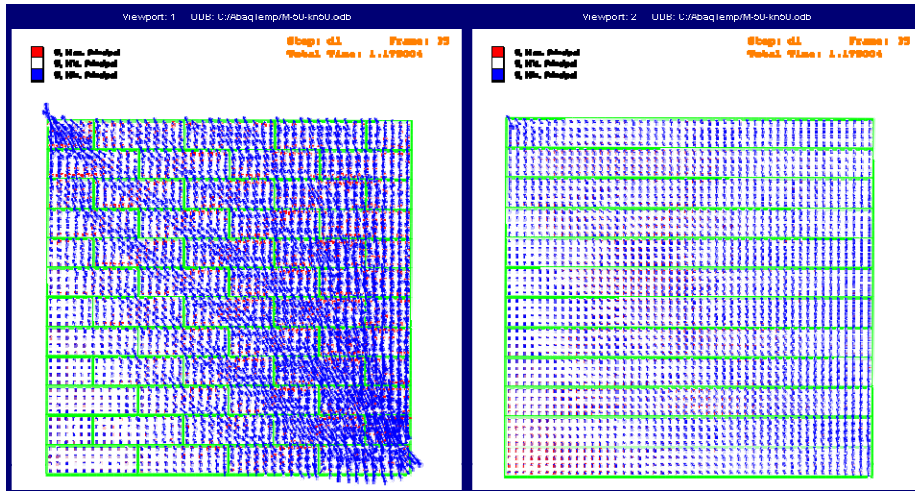
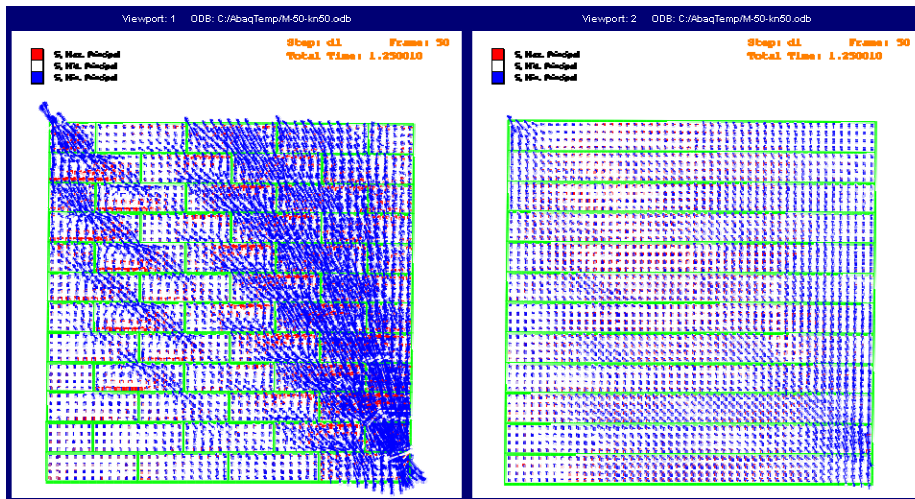


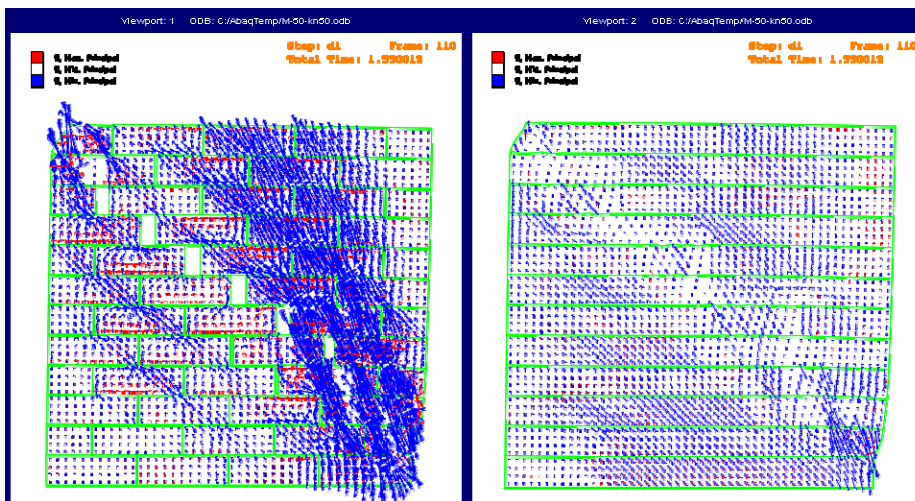
Figure D.3 : Minimum principal plastic strain distributions of S-50-C model.



(a) Pre-peak



(b) Peak

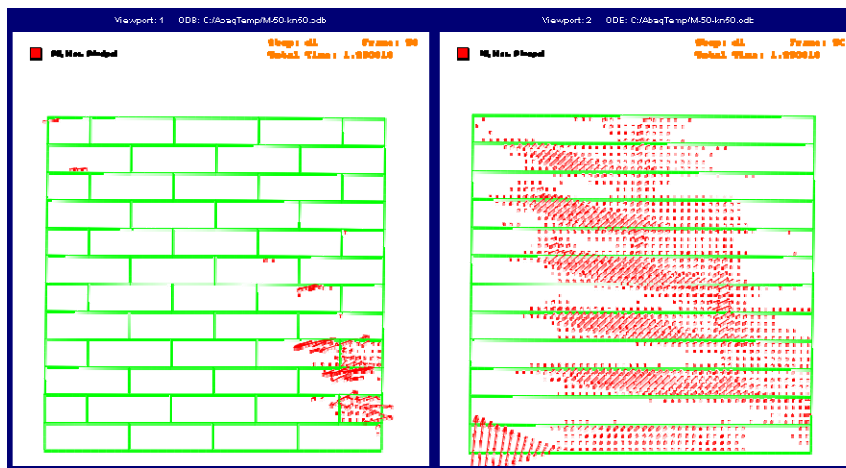


(c) Post-peak

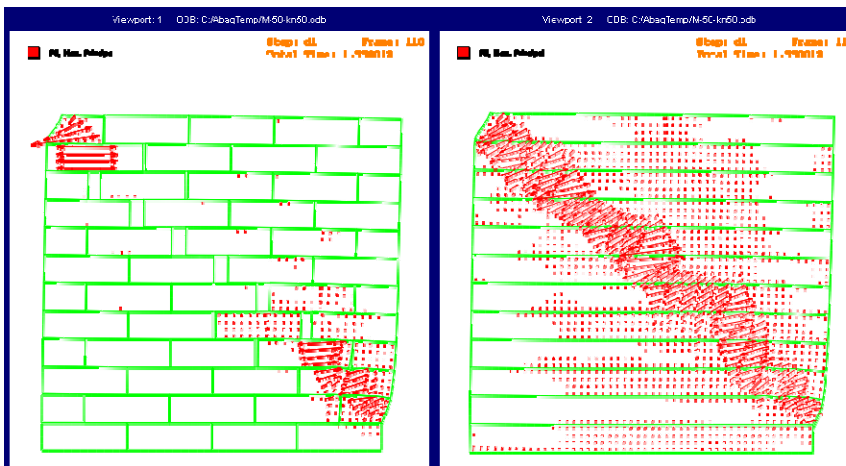
Figure D.4 : Principal stress distributions of M-50 model.



(a) Pre-peak

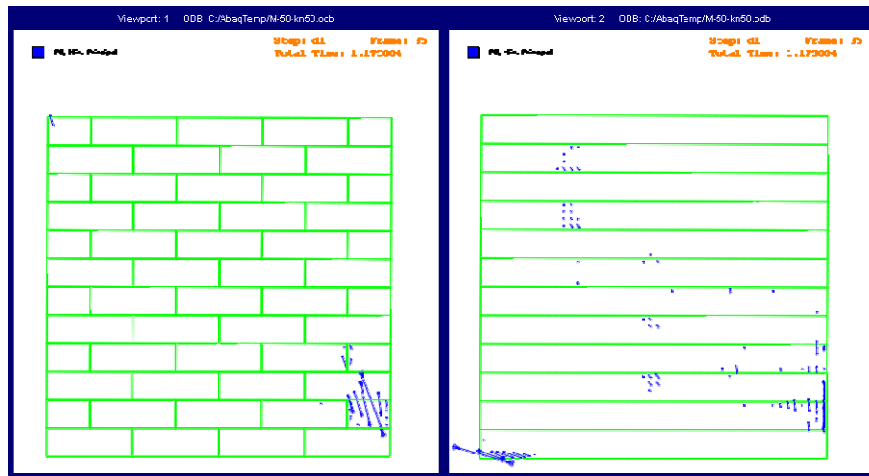


(b) Peak

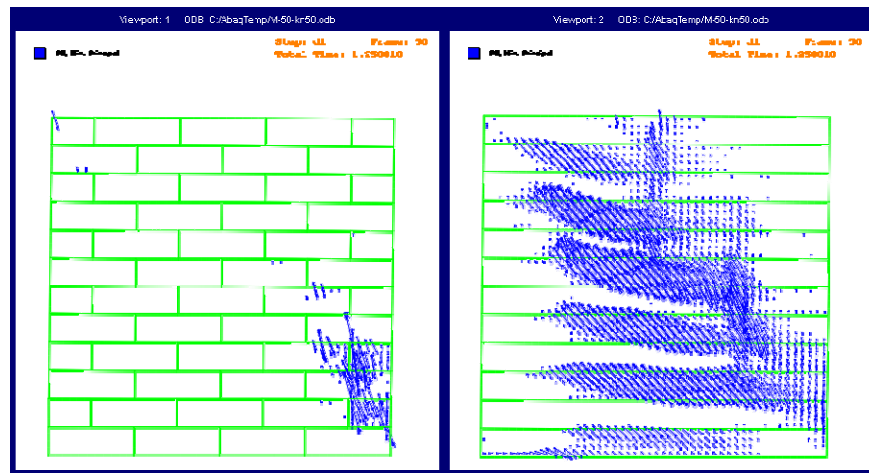


(c) Post-peak

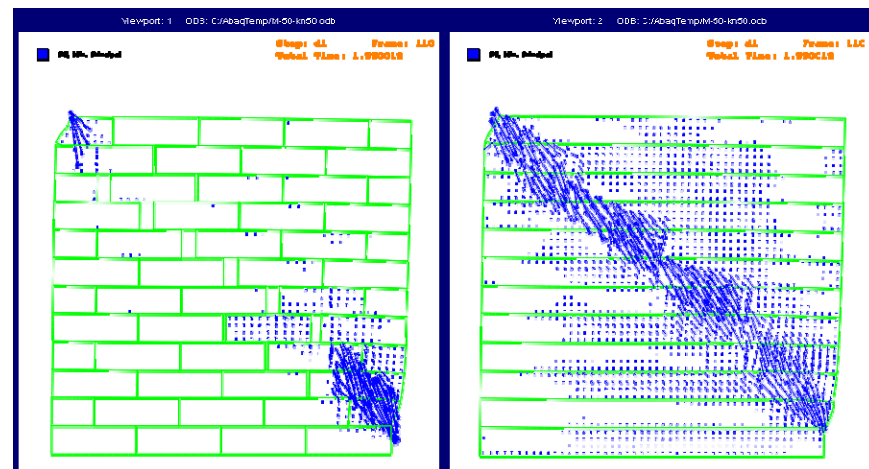
Figure D.5 : Maximum principal plastic strain distributions of M-50 model.



(a) Pre-peak

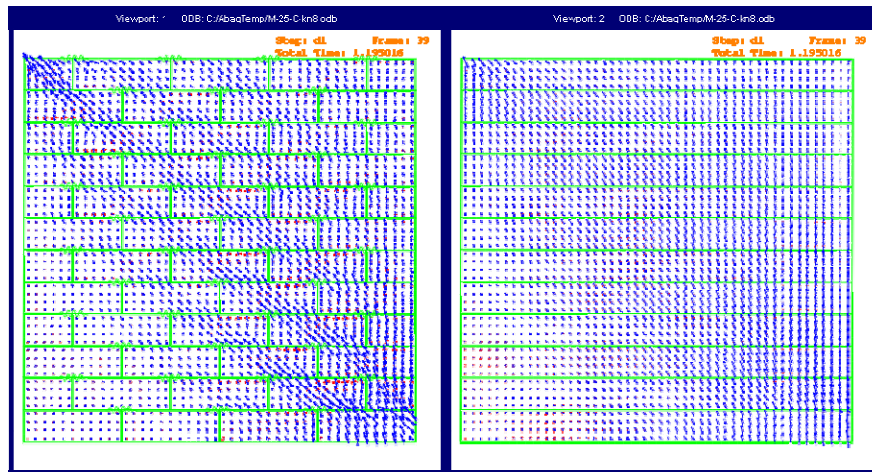


(b) Peak

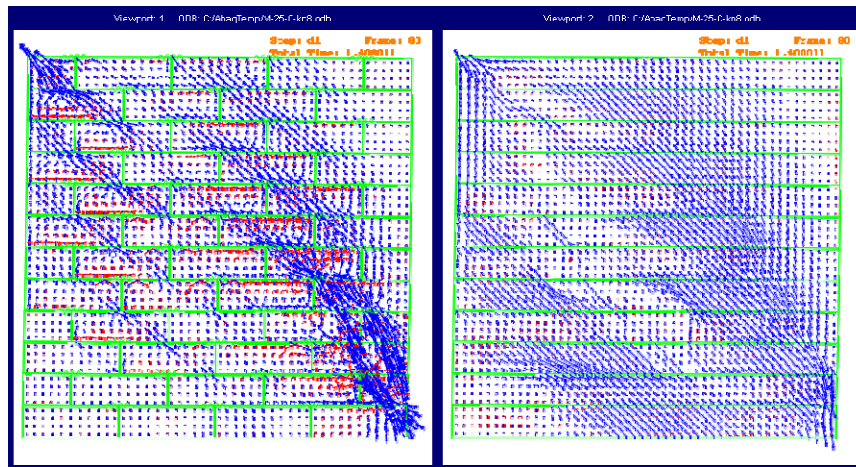


(c) Post-peak

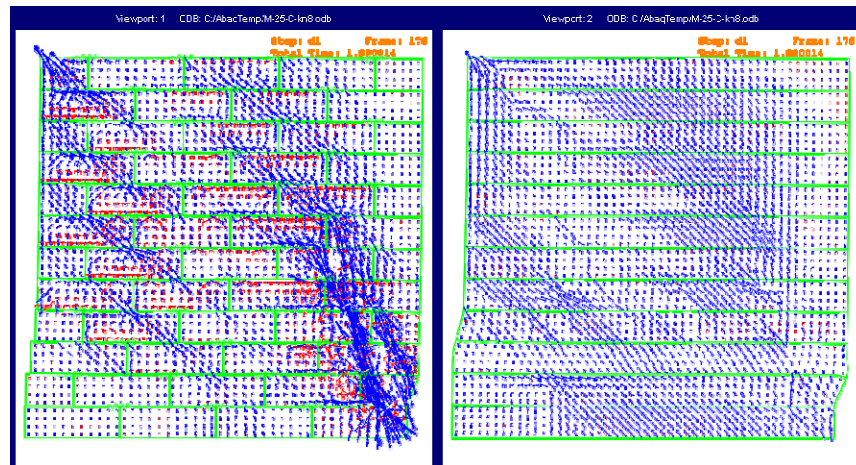
Figure D.6 : Minimum principal plastic strain distributions of M-50 model.



(a) Pre-peak

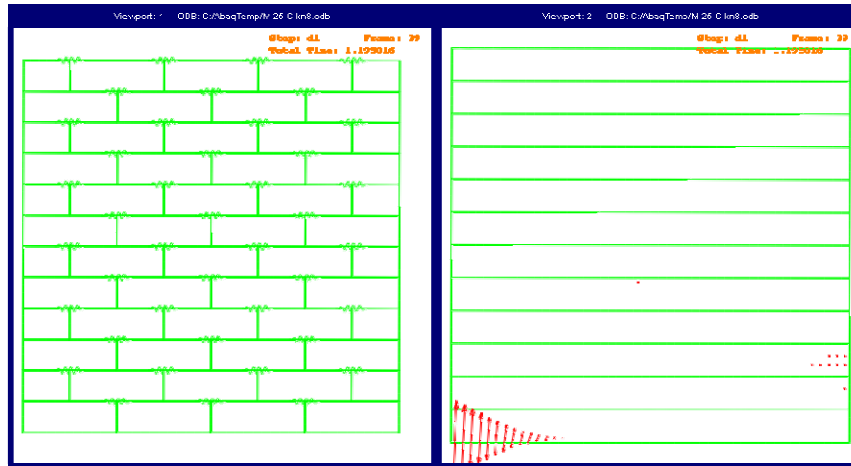


(b) Peak

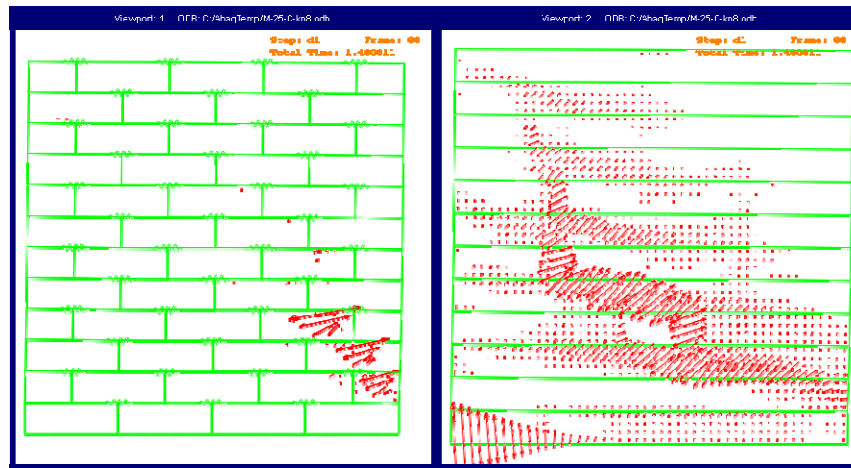


(c) Post-peak

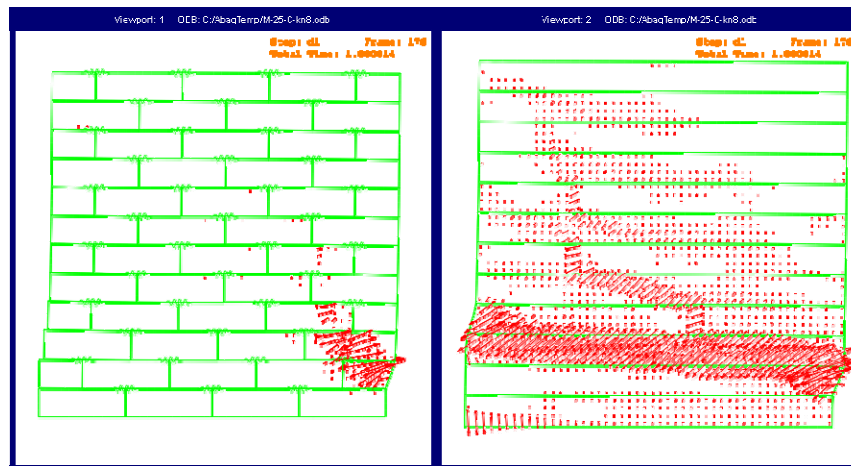
Figure D.7 : Principal stress distributions of M-25-C model.



(a) Pre-peak

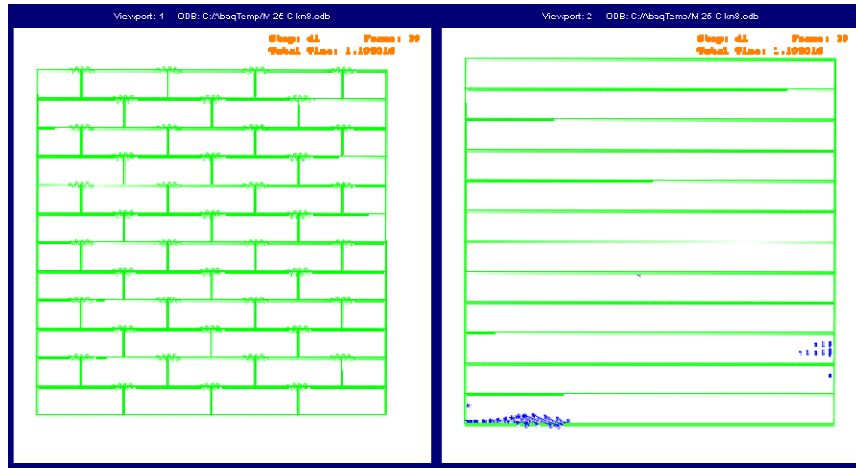


(b) Peak

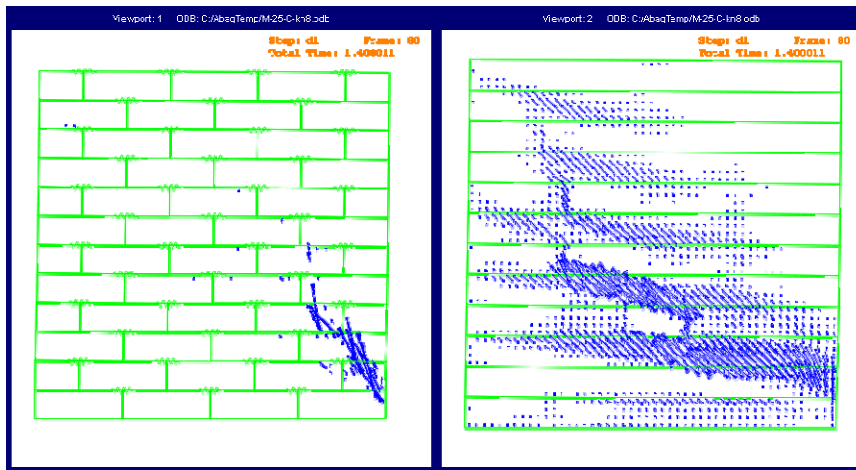


(c) Post-peak

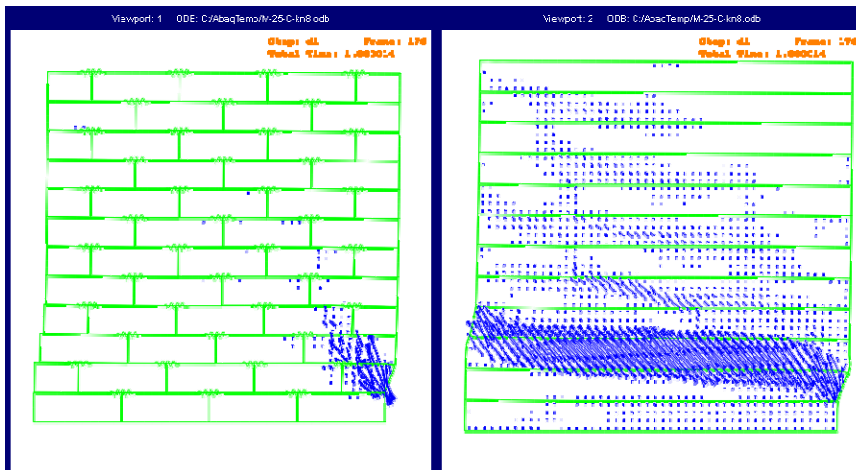
Figure D.8 : Maximum principal plastic strain distributions of M-25-C model.



(a) Pre-peak

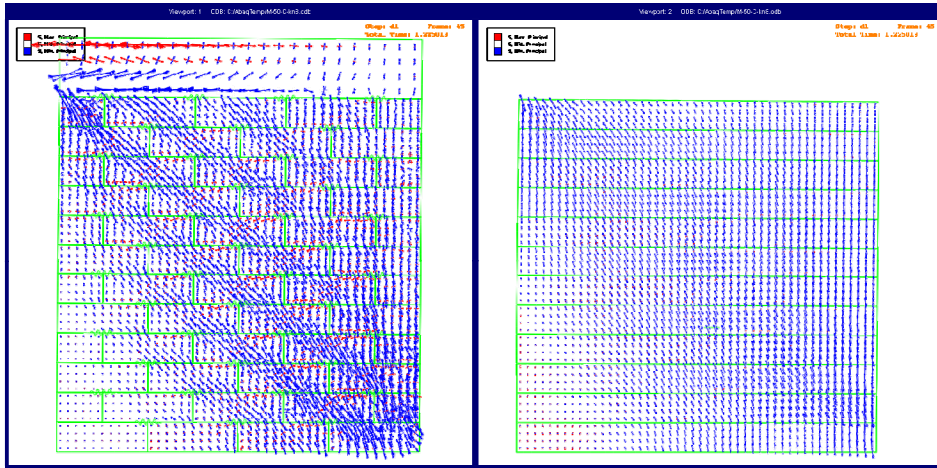


(b) Peak

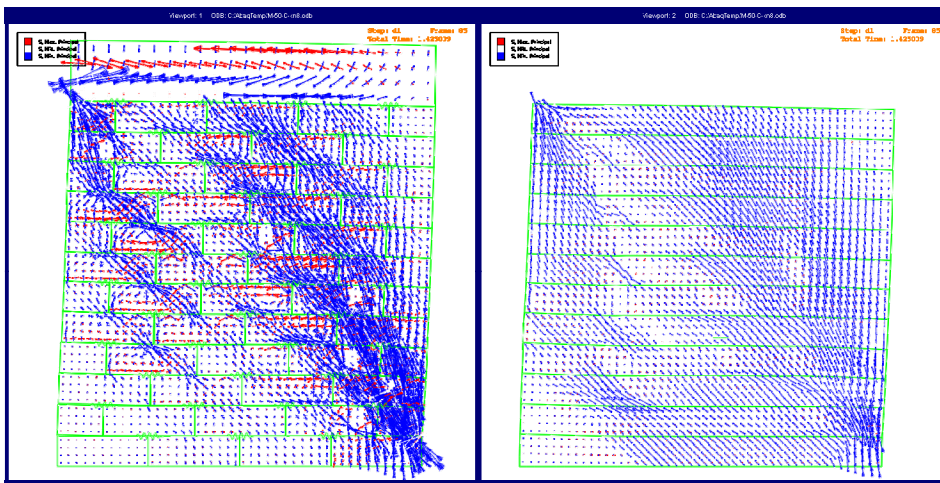


(c) Post-peak

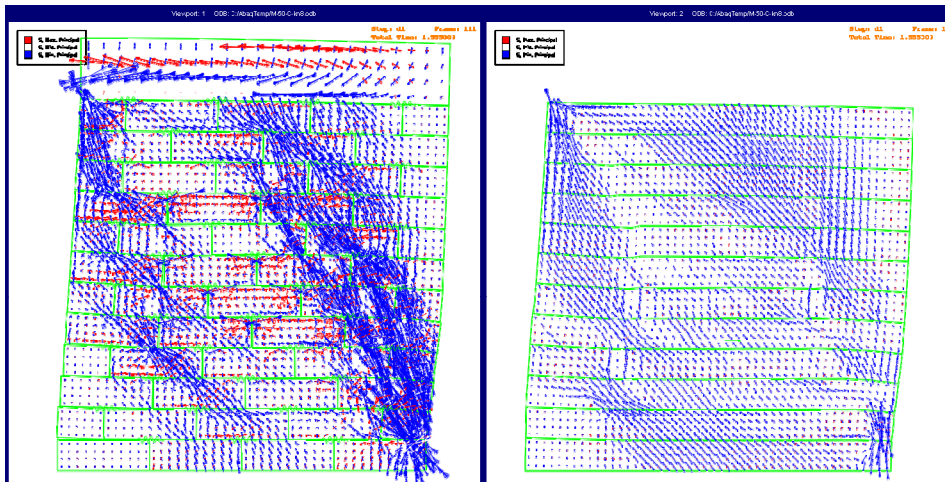
Figure D.9 : Minimum principal plastic strain distributions of M-25-C model.



(a) Pre-peak

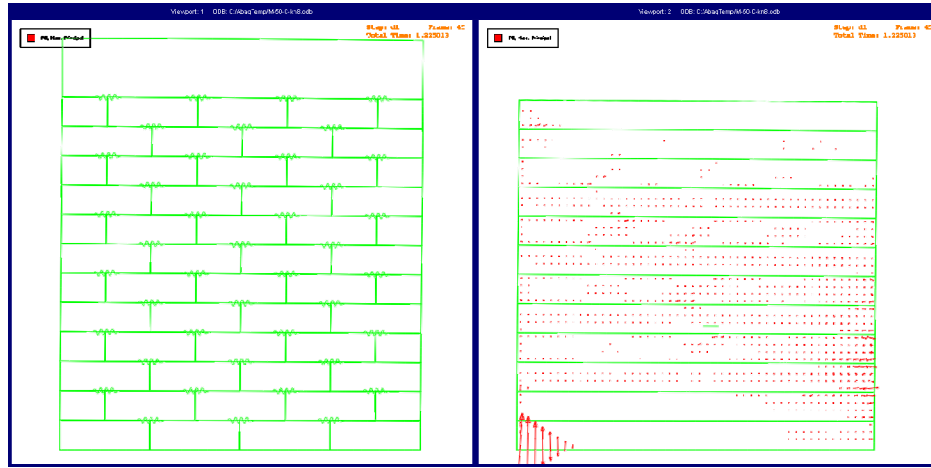


(b) Peak

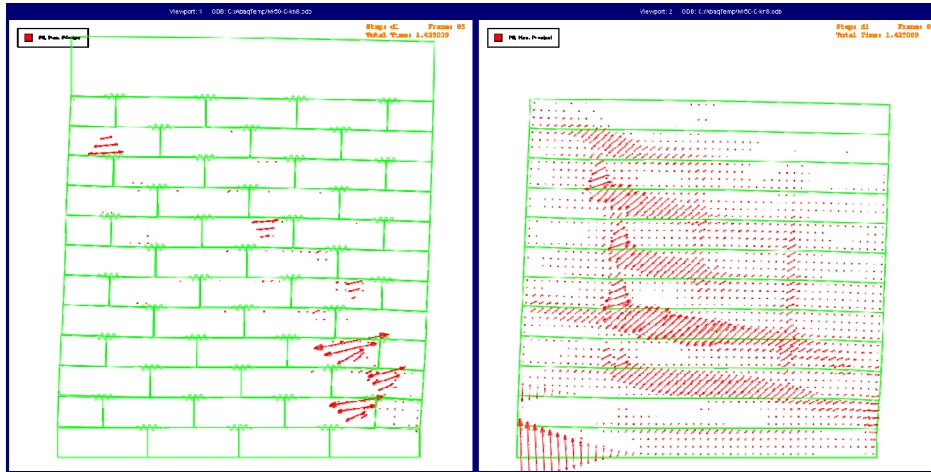


(c) Post-peak

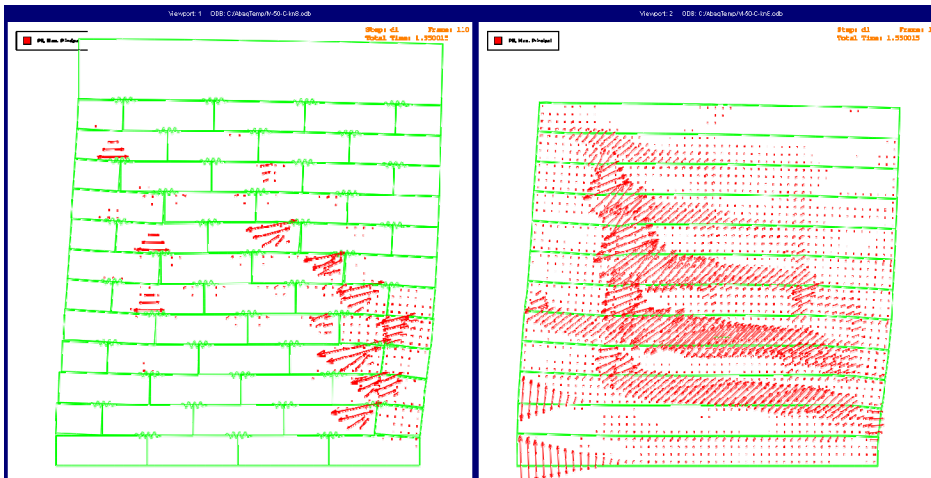
Figure D.10 : Principal stress distributions of M-50-C model.



(a) Pre-peak



(b) Peak

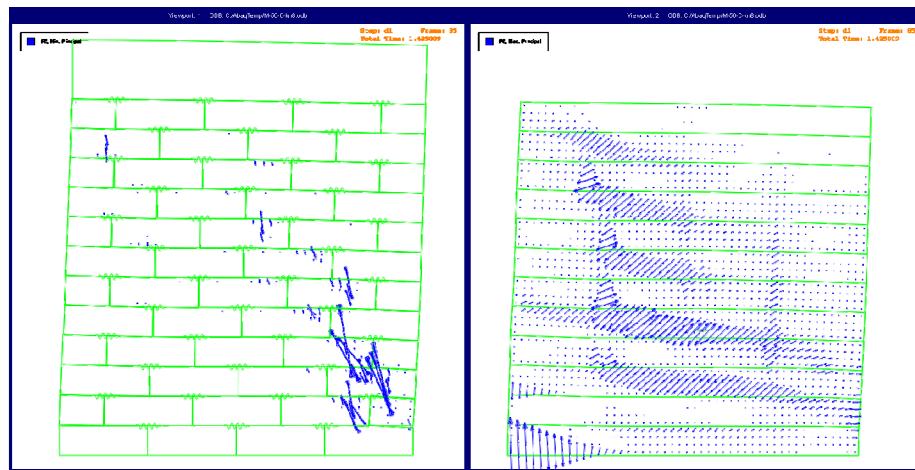


(c) Post-peak

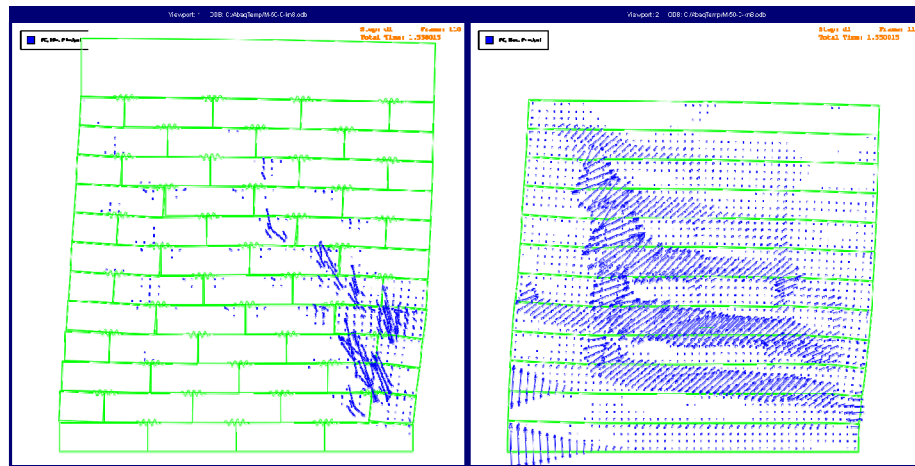
Figure D.11 : Maximum principal plastic strain distributions of M-50-C model.



(a) Pre-peak

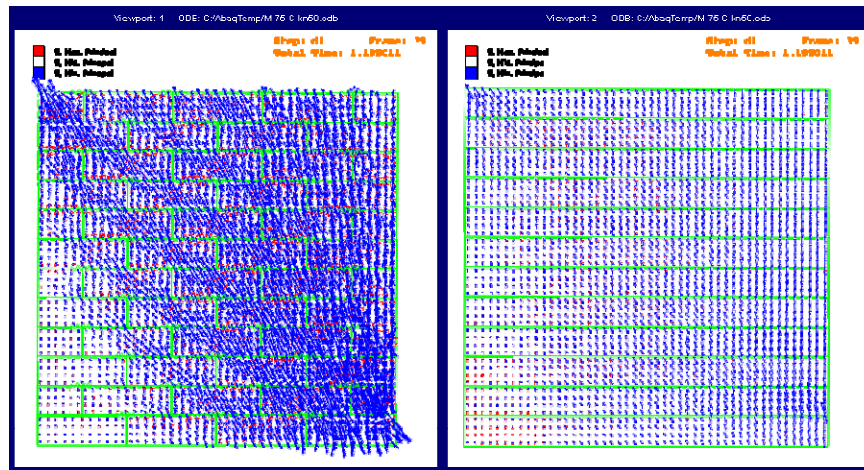


(b) Peak

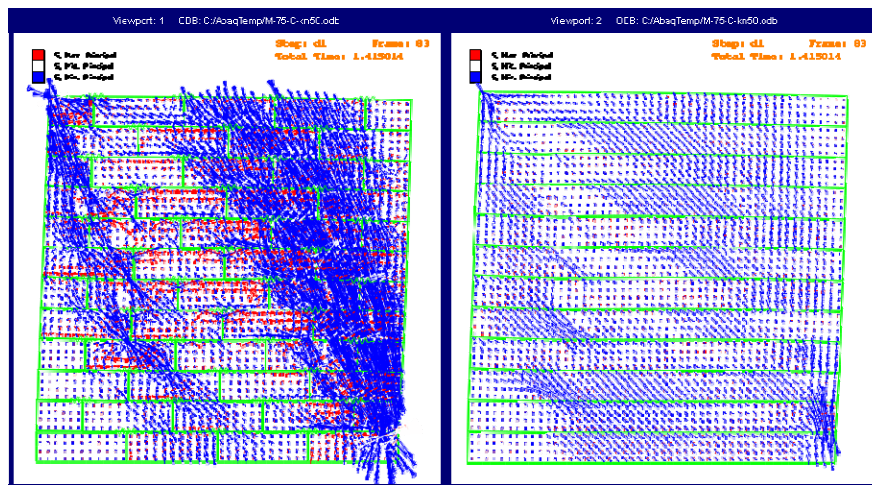


(c) Post-peak

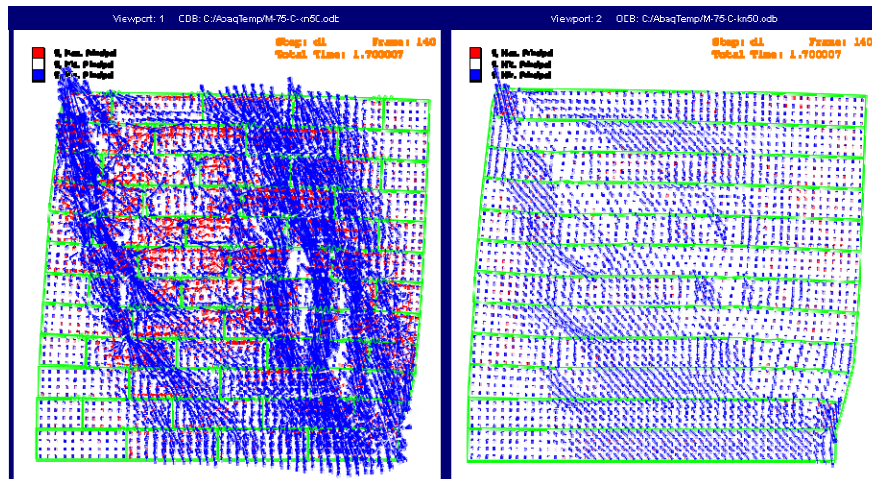
Figure D.12 : Minimum principal plastic strain distributions of M-50-C model.



(a) Pre-peak

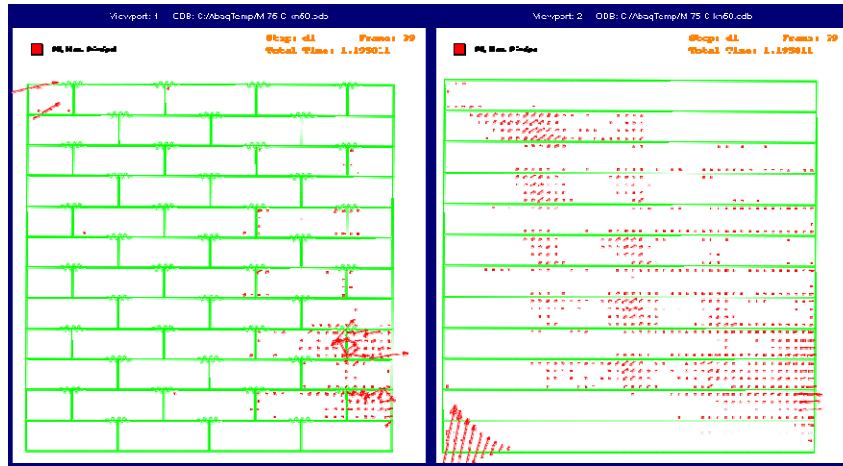


(b) Peak

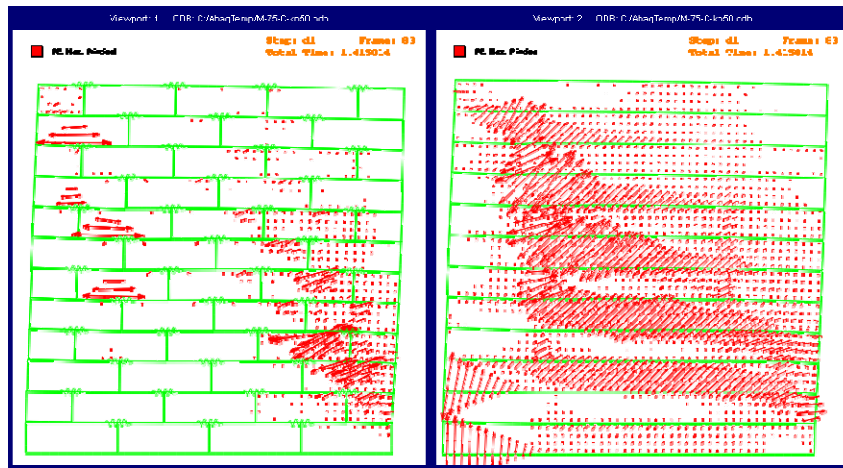


(c) Post-peak

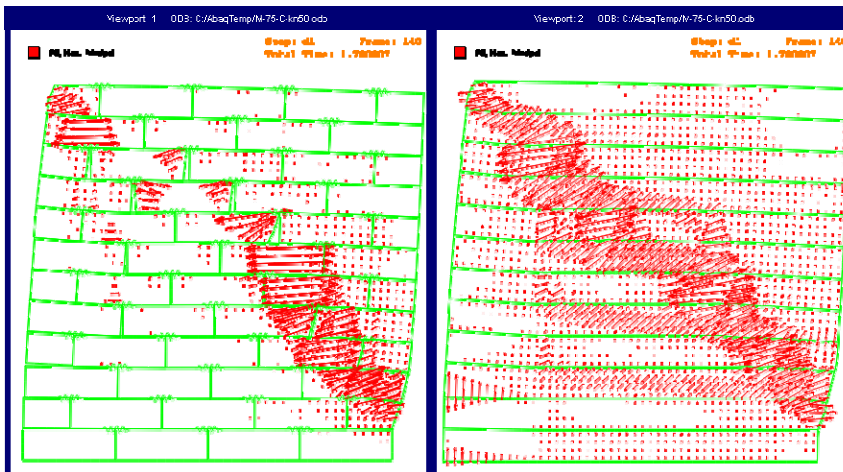
Figure D.13 : Principal stress distributions of M-75-C model.



(a) Pre-peak

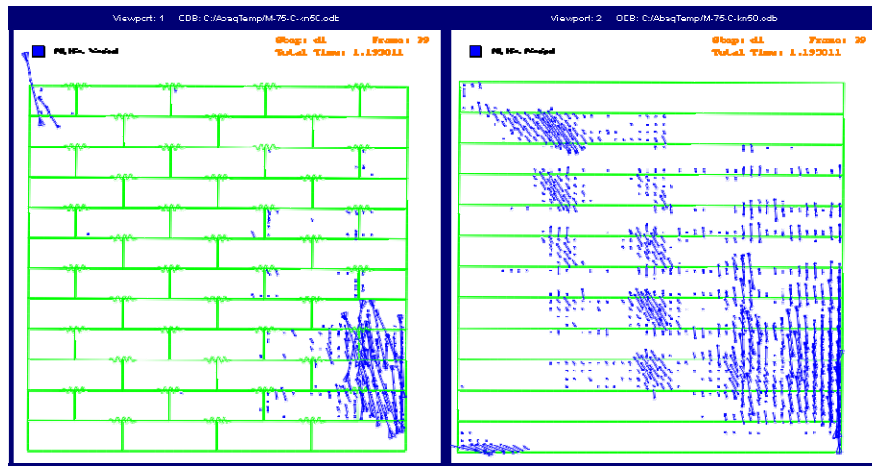


(b) Peak

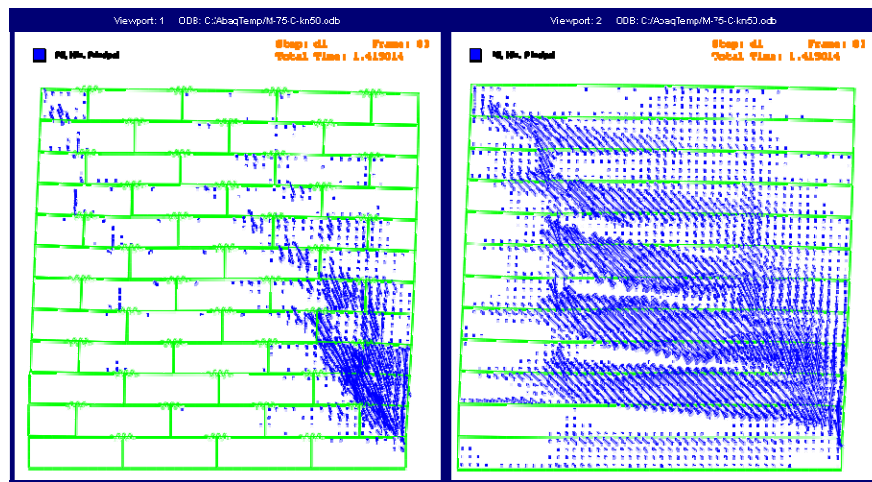


(c) Post-peak

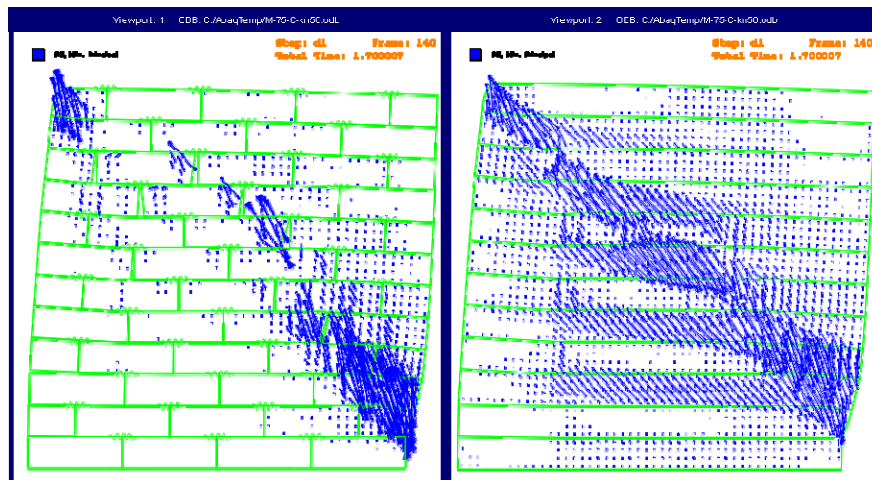
Figure D.14 : Maximum principal plastic strain distributions of M-75-C model.



(a) Pre-peak

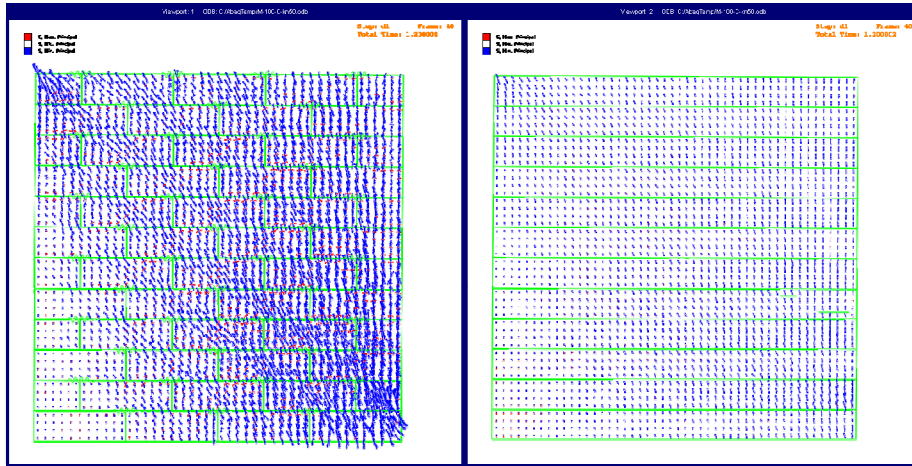


(b) Peak

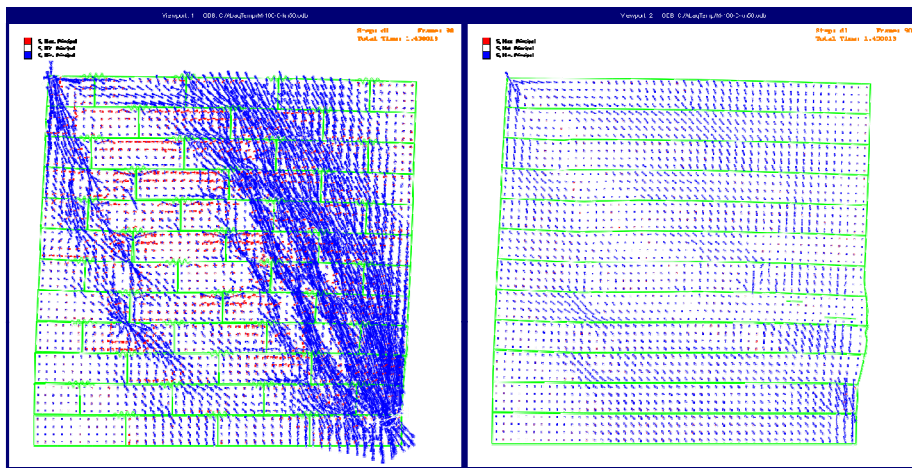


(c) Post-peak

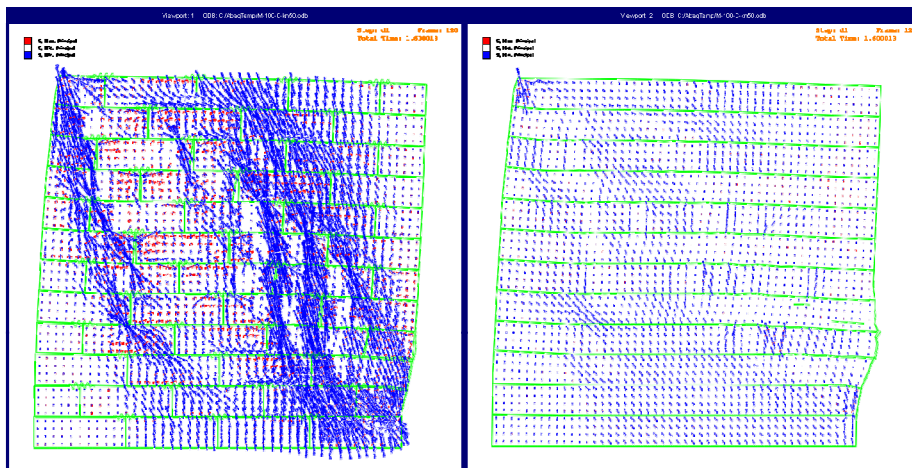
Figure D.15 : Minimum principal plastic strain distributions of M-75-C model.



(a) Pre-peak

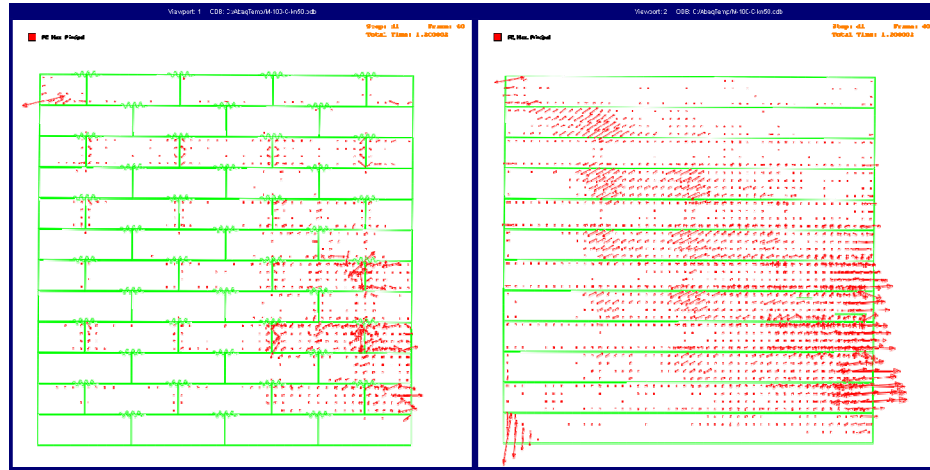


(b) Peak

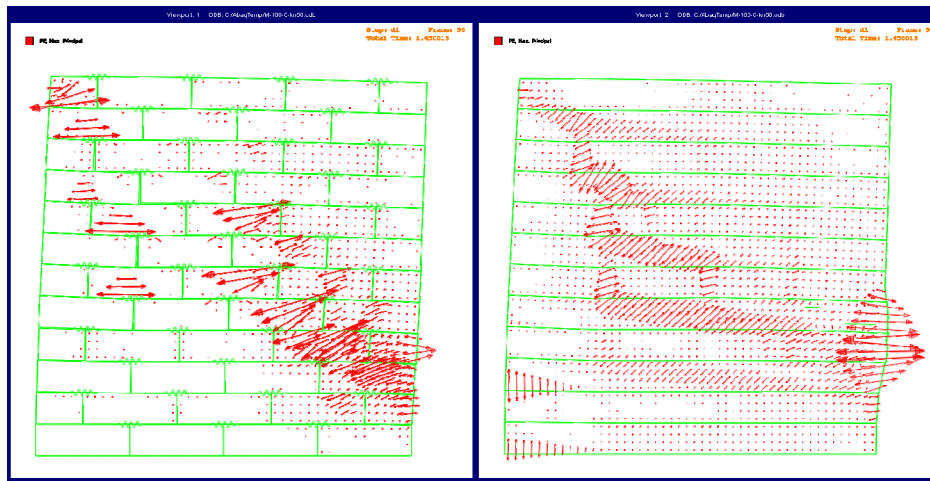


(c) Post-peak

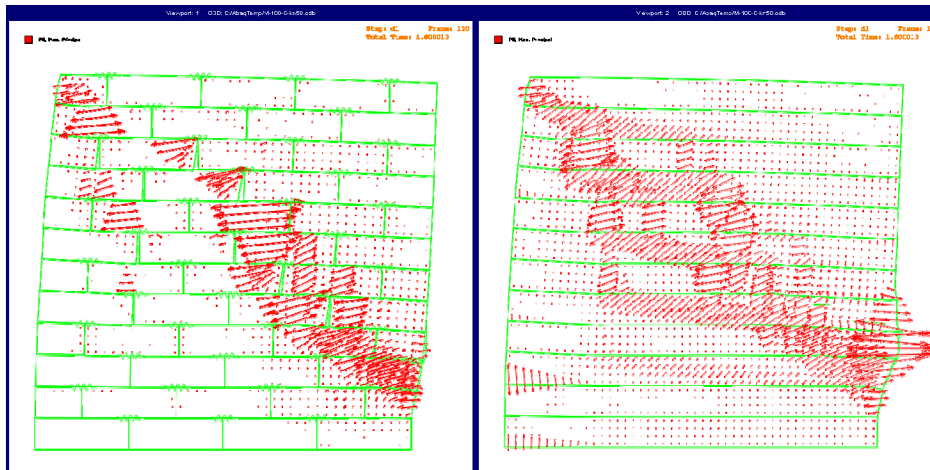
Figure D.16 : Principal stress distributions of M-100-C model.



(a) Pre-peak

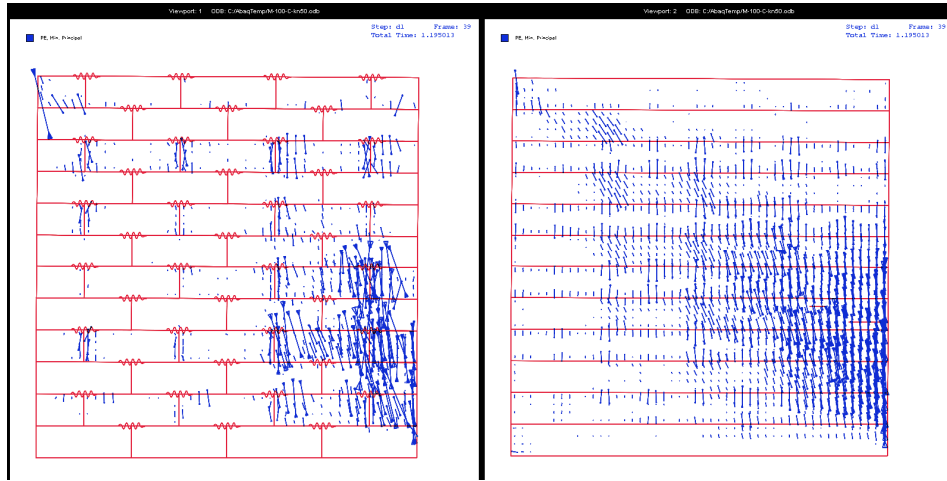


(b) Peak

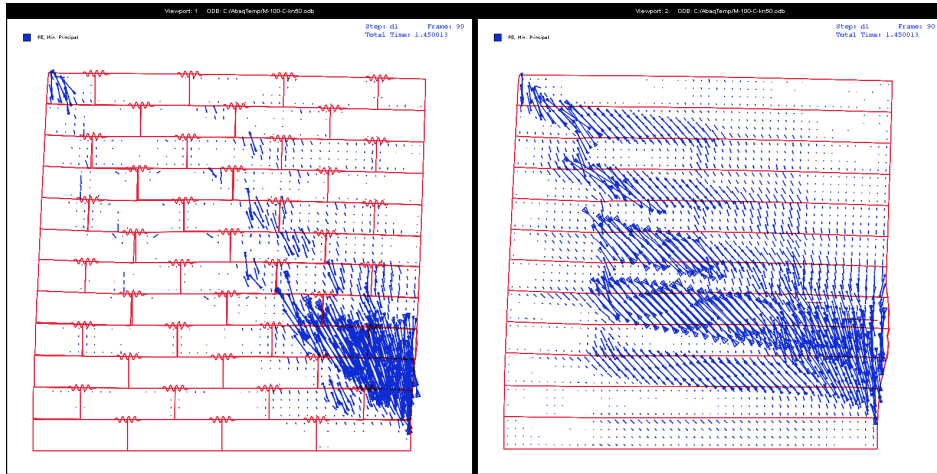


(c) Post-peak

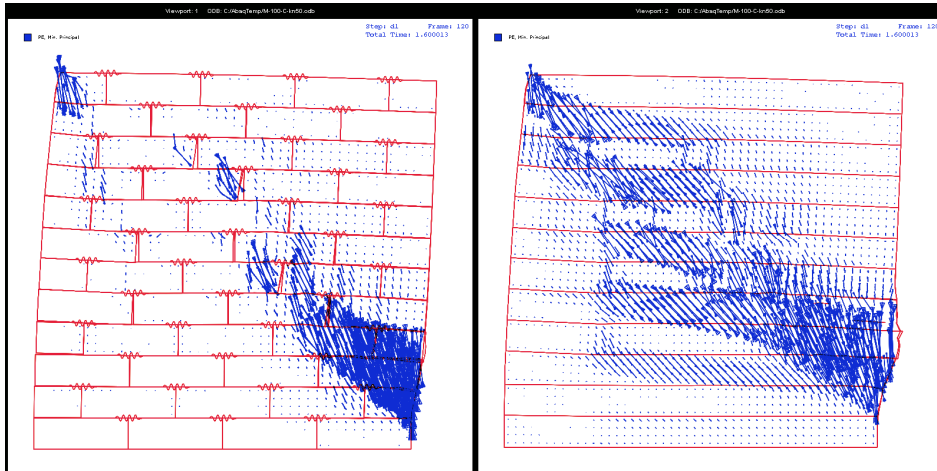
Figure D.17 : Maximum principal plastic strain distributions of M-100-C model.



(a) Pre-peak



(b) Peak



(c) Post-peak

Figure D.18 : Minimum principal plastic strain distributions of M-100-C model.

CURRICULUM VITAE

Name Surname: Cem Demir

Place and Date of Birth: Samandağ, 11.11.1978

Address: İstanbul Teknik Üniversitesi, İnşaat Fakültesi, Yapı ve Deprem
Mühendisliği Laboratuvarı, Ayazağa, Sarıyer, İstanbul

E-Mail: demirce@itu.edu.tr

B.Sc.: Istanbul Technical University, Civil Engineering Department

M.Sc.: Istanbul Technical University, Structural Engineering Program

Professional Experience and Rewards:

Research Assistant at ITU Civil Engineering Faculty, Civil Engineering
Department since 2002.

Participation to several studies carried out at ITU Structural and EQ Engineering
Laboratory.

Best Paper Award, Advances in Structural Engineering Journal, 2009.

List of Publications:

- **Demir, C.**, Kolcu, K. and Ilki, A. (2010). Effects of Loading Rate and Duration on Axial Behavior of Concrete Confined by Fiber Reinforced Polymer Sheets, *ASCE, J. Comp. Const.*, **14**, 2, pp. 146-151.
- Ispir, M., **Demir, C.**, Ilki, A., Kumbasar, N. (2010). Material Characterization of the Historical Unreinforced Masonry Akaretler Row Houses in Istanbul, *ASCE, J. Mat. in Civ. Engrg.*, **22**, 7, pp. 702-713.
- Ilki, A., **Demir, C.** and Bedirhanoglu, I. (2009). Seismic Retrofit of Brittle and Low Strength RC Columns using Fiber Reinforced Polymer and Cementitious Composites, *Advances in Structural Engineering*, **12**, 3, pp. 325-347.
- Ilki, A., Peker, O., Karamuk, E., **Demir, C.** and Kumbasar, N. (2008). FRP Retrofit of Low and Medium Strength Circular and Rectangular Reinforced Concrete Columns, *ASCE, J. Mat. Civ. Eng.*, **20**, 2, pp. 169-188.
- **Demir, C.**, Aydogmus, A. and Ilki, A. (2012). Short Term Creep Tests of Low Strength Rectangular Concrete Members Jacketed with Carbon FRP Sheets, *Proc. APFIS2012 The Third Asia-Pacific Conference on FRP in Structures*, Sapporo, Japan, 2012.
- Ilki, A., **Demir, C.**, Karadeniz, A. and Kumbasar, N. (2003). Hasarlı Betonarme

Elemanların Çelik Köşebent ve Lamalar ile Güçlendirilmesi, *İTÜ Dergisi/d*, 2(3), pp. 3-16.

- Akgun, D., **Demir, C.** and Ilki, A. (2010). Axial Behavior of FRP Jacketed Extended Rectangular Members, Proc. CICE 2010 5th International Conference on FRP Composites in Civil Engineering, Beijing, China.
- Gedik, O., **Demir, C.** and Ilki, A. (2009). Effect of unconfined concrete strength on the strength and deformability predictions of frp confined concrete, *Proc. Asia-Pacific Conference on FRP in Structures*, Seoul, Korea.
- Ispir, M., **Demir, C.**, Ilki, A. and Kumbasar, N. (2008). Evaluation of Test Methods for Seismic Safety Assessment of Masonry Buildings, *Proc. 8th International Seminar on Structural Masonry ISSM08*, Istanbul, Turkey.
- Goksu, C., Demirtas, B., **Demir, C.**, Ilki, A. and Kumbasar, N. (2008). Seismic Behavior of Low Strength RC Columns with Corroded Plain Reinforcing Bars, *Proc. 14th World Conference on Earthquake Engineering 14WCEE*, Beijing, China.
- Ilki, A., Ispir, M., As, F., **Demir, C.** and Kumbasar, N. (2008). FRP Retrofit of Walls Constructed with Historical Bricks, *Challenges for Civil Construction CCC2008*, Porto, Portugal.
- Ilki, A., **Demir, C.** and Kumbasar, N. (2007). Effects of pre-damage and cyclic compression on axial behavior of FRP jacketed concrete, *Asia-Pacific Conference on FRP in Structures*, Hong Kong.
- **Demir, C.**, Acar, D., Terzi, S.O., Ispir, M., Demirtas, B., Ilki A. and Kumbasar, N. (2007). Preliminary Structural Assessment of Kariye Monument-Northern Annex, *Proc. Int. Symp. Studies on Historical Heritage*, Antalya, Turkey.
- Ilki, A., Goksu, C., **Demir, C.** and Kumbasar, N. (2007). Seismic Analysis of a RC Frame Building with FRP Retrofitted Infill Walls, *Proc. 6th International Conference on Fracture Mechanics for Concrete and Concrete Structures, FraMCoS-6*, Catania.
- Ilki, A., Ispir, M., **Demir, C.**, Karamuk, E., As, F., Cini, N., Aydin, M., Toponder, G., Tulun, T., Kumbasar, N. and Akman, S. (2007). Seismic Safety of a Historical Row House Complex Built during Ottoman Period, *Proc. 9th Canadian Conference on Earthquake Engineering*, Ottawa, Canada.
- Kuran, F., **Demir, C.**, Koroglu, O., Kocaman, C. and Ilki, A. (2007). Seismic Safety Analysis of an Existing 1502 Type Disaster Building Using New Version of Turkish Seismic Design Code, *Proc. ECCOMAS Thematic Conference on Computational Methods in Structural Dynamics and Earthquake Engineering*, Crete, Greece.
- Ilki, A., Akgun, D., Goray, R.O., **Demir, C.** and Kumbasar, N. (2006). Seismic Strengthening of Low Strength Concrete Columns by External Confinement using Prefabricated SFRCC Panels, *Proc. 19th Australasian Conference on the Mechanics of Structures and Materials*, Canterbury, New Zealand.
- Ilki, A., Bedirhanoglu, I., Basegmez, I.H., **Demir, C.**, and Kumbasar, N. (2006). Shear Retrofit of Low Strength Reinforced Concrete Short Columns with GFRP Composites, *Proc. 3rd International Conference on Composites in Civil Engineering (CICE2006)*, Miami, USA.

- Ilki, A., Ispir, M., **Demir, C.**, Kumbasar, N. and Akman, S. (2006). An Outline of the Seismic Behavior of Historical Structures in North Western Anatolia, *International Conference on Structural Analysis of Historical Constructions (SAHC2006)*, New Delhi, India.
- Ilki, A., Bedirhanoglu, İ., **Demir, C.** and Kumbasar, N. (2006). Prediction of the Behavior of FRP Confined RC Columns under Axial Load and Flexure, *Proc. 2nd International fib Congress*, Naples, Italy.
- Ilki, A., **Demir, C.**, Goksu, C. and Kumbasar, N. (2006). Prefabricated SFRC Panels for Occupant Friendly Seismic Retrofit of Reinforced Concrete Columns, *Proc. 8th National Conference on Earthquake Engineering*, San Francisco, USA.
- Goksu, C., **Demir, C.**, Darilmaz, K., Ilki, A. and Kumbasar, N. (2006). Static Nonlinear Analysis of a Retrofitted Typical Reinforced Concrete Building in Turkey, *Proc. 8th National Conference on Earthquake Engineering*, San Francisco, USA.
- Erol, G., **Demir, C.**, Ilki, A., Yuksel, E. and Karadogan, F. (2006). Effective Strengthening of RC Frames with and without Lap Splice Problems, *Proc. 8th National Conference on Earthquake Engineering*, San Francisco, USA.
- Ilki, A., **Demir, C.**, Akgün, D., Göray, O. and Kumbasar, N. (2006). Retrofit of Concrete Members with Externally Bonded Prefabricated SFRCC Jackets, *Proc. 16th European Conference on Fracture*, Alexandropolis, Greece.
- Ilki, A., **Demir, C.**, Bedirhanoglu, İ. and Kumbasar, N. (2005). Flexural Behavior of FRP Jacketed Reinforced Concrete Columns, *Proc. Third International Conference on Composites in Construction*, Lyon, France.
- Ilki, A., Darilmaz, K., **Demir, C.**, Bedirhanoglu, İ. and Kumbasar, N. (2005). Non-linear Seismic Analysis of a Reinforced Concrete Building with FRP Jacketed Columns, *Proc. fib Symposium Keep Concrete Attractive*, Budapest, Hungary.
- **Demir, C.**, Ilki A. and Kumbasar, N. (2005). Moment-Curvature Relationships for Columns Jacketed with FRP Composites, *Proc. fib Symposium Keep Concrete Attractive*, Budapest, Hungary.
- Yuksel, E., Ilki, A., Erol, G., **Demir, C.** and Karadoğan, F. (2005). Seismic Retrofit of Infilled Reinforced Concrete Frames with CFRP Composites, *Nato Workshop on Seismic Assessment and Rehabilitation of Existing Buildings*, Istanbul, Turkey.
- Ilki, A., Yilmaz, E., **Demir, C.** and Kumbasar, N. (2004). Prefabricated SFRC Jackets for Seismic Retrofit of Non-ductile Rectangular Reinforced Concrete Columns, *Proc. 13th World Conference on Earthquake Engineering*, Vancouver, Canada.
- Boduroğlu, H., Özdemir, P., Ilki, A., Şirin, S., **Demir C.** and Baysan, F. (2004). Towards a Modified Rapid Screening Method for Existing Medium Rise RC Buildings in Turkey, *Proc. 13th World Conference on Earthquake Engineering*, Vancouver, Canada.
- Ilki, A., **Demir, C.** and Kumbasar, N. (2004). Inelastic Behaviour of RC Jacketed Damaged Concrete Sections under Reversed Cyclic Flexure, *Proc.*

2nd Int. Conference on Structural Engineering, Mechanics and Computation, Cape Town, South Africa.

- Ilki, A., **Demir, C.** and Kumbasar, N. (2004). Effect of Confined Concrete Stress-Strain Model on the Moment-Curvature Relationships of Reinforced Concrete Members, *Proc. 2nd Int. Conference on Structural Engineering, Mechanics and Computation, Cape Town, 2004.*

PUBLICATIONS/PRESENTATIONS ON THE THESIS

- **Demir, C.** and Ilki, A. (2011). Taş Yığma Duvarların Yatay Yükler Altında Davranışı, *Ulusal Yapı Mekaniği Laboratuvarları Toplantısı - 2011*, 14-15 May, Kocaeli, Turkey.