

BEHAVIOR OF ENGINEERED CEMENTITIOUS COMPOSITE-
REPAIRED SUPERELASTIC-SHAPE MEMORY ALLOY
REINFORCED SHEAR WALLS

MICHAEL ARMANDO SOTO ROJAS

A THESIS SUBMITTED TO
THE FACULTY OF GRADUATE STUDIES
IN PARTIAL FULFILLMENT OF THE REQUIREMENTS
FOR THE DEGREE OF
MASTER OF APPLIED SCIENCE

GRADUATE PROGRAM IN CIVIL ENGINEERING
YORK UNIVERSITY
TORONTO, ONTARIO

AUGUST 2020

© MICHAEL ARMANDO SOTO ROJAS, 2020

Abstract

Ductility capacity and energy dissipation are two primary characteristics well-designed reinforced concrete structures are expected to achieve to endure a seismic event. The design objective is to guarantee the life safety performance level. The objective is readily realized with the use of conventional construction materials; however, the structural performance can be typified by significant residual deformation due to yielding of reinforcement and damage from the response of the concrete. The implementation of innovative materials such as Shape Memory Alloys (SMAs) and High-Performance Fiber Reinforced Concrete (HPFRC) provide an opportunity to improve the post-earthquake state of structures, while achieving the design objective. Moreover, by limiting the post-earthquake residual damage, occupant removal and subsequent demolition of structures are avoided, and minor repairs for a relatively immediate occupancy become possible. The combination of these two materials lead to self-centering with improved damage tolerance. In this research, previously tested, Superelastic-Shape Memory Alloy (SE-SMA) and typical-steel reinforced concrete shear walls are repaired and tested under the same simulated seismic loading. The repairing method involved the removal of the heavily damaged concrete in the plastic hinge zone, replacement of ruptured and buckled steel reinforcement, and casting of Engineered Cementitious Composite (ECC) where the previous concrete was removed. The numerical modelling and experimental testing of the repair technique highlight that the brittle behavior of concrete in tension and its deformation incompatibility with reinforcing steel bars, and yielding of steel reinforcement are suppressed by establishing a composite system that integrates the self-centering phenomenon of SE-SMA and the distinctive ductility properties of ECC.

Acknowledgments

Foremost, I would like to express my sincere gratitude to my supervisor Professor Dr. Dan Palermo, for the continuous support and mentorship throughout the two years of this research project. His dynamism, vision, sincerity and motivation have deeply inspired me. It was a great privilege and honor to work and study under your guidance.

The completion of this project could not have been possible without the support of my fellow graduate coworkers; enormous thanks go to my good friends Mena and Adrien, part of the structural group in the High Bay Lab at York University, who spent with me countless hours and long nights of work during the execution of the experimental program of this study. The camaraderie spirit and team effort that we developed will forever be appreciated,

To my caring and supporting girlfriend, Rita, my deepest gratitude. Your encouragement, when the times got rough, is much appreciated and duly noted. It was a great comfort and relief to know that you were always there to help with every challenge this research project presented. Nothing of this would have been possible without you. Thank you for everything.

To my family, that even far away in Costa Rica, always found means to make me feel close to them and kept me pushing to achieve my goals. To my mother Ana and my father Felix, there are no words to express how grateful I am for everything that you sacrificed to give me a better future, for prioritizing my education and for the endless emotional support. Everything that I have done and accomplished is because of you. Thanks to my sister Karla, my brothers: Manuel and Andrés, and last but not least, to my grandmother, who is not here with us anymore, Rosa. To all of them, thank you.

Table of Contents

ABSTRACT	II
ACKNOWLEDGMENTS	III
TABLE OF CONTENTS	IV
LIST OF TABLES	IX
LIST OF FIGURES	X
LIST OF ABBREVIATION	XX
LIST OF SYMBOLS	XXI
CHAPTER 1. INTRODUCTION	1
1.1 Background	1
1.2 Scope of the Research	4
1.2.1 Shear Walls	4
1.2.2 Shape Memory Alloys	12
1.2.3 High-Performance Fiber Reinforced Concrete (HPFRC)	14
1.3 Scope and Objective of Thesis.....	17
1.4 Novelty of the research	18
1.5 Thesis Layout.....	18
Chapter 2 – Literature Review	19
Chapter 3 – Preliminary Analysis	19
Chapter 4 - Experimental Program	19
Chapter 5 – Experimental Results.....	20
Chapter 6 – Discussion of Results	20
Chapter 7 – Numerical Study.....	20

Chapter 8 – Conclusions and Recommendations.....	21
CHAPTER 2. LITERATURE REVIEW	22
2.1 Introduction.....	22
2.2 SMA Reinforced Concrete Structures	22
2.2.1 Introduction.....	22
2.2.2 Behavior of SMA Reinforced Concrete Structures	23
2.3 Behavior of Repaired Reinforced Concrete Shear Walls	33
2.4 Introduction to High-Performance Fiber-Reinforced Concrete (HPFRC).....	39
2.4.1 Evolution and Classification of HPFRC	39
2.4.2 Composition of HPFRC	41
2.4.3 Examples of Previously Developed HPFRCC.....	45
2.5 SMA-HPC Hybrid Composite	50
CHAPTER 3. PRELIMINARY ANALYSIS.....	58
3.1 Background of the Walls Under Study	58
3.1.1 Original Walls.....	58
3.1.2 Material Properties.....	62
3.1.3 Previous Damage and Visual Inspection	66
3.2 Proposed Repairing Options	68
3.2.1 Concrete Removal and Replacement/Addition of Reinforcement.....	68
3.3 Preliminary Numerical Analysis.....	72
3.3.1 Introduction.....	72
3.3.2 Conceptual Basis.....	72
3.3.3 Modelling Methodology	74
3.3.4 Concrete Constitutive Models.....	74
3.3.5 Reinforcement Constitutive Models	78
3.3.6 Definition of Regions for the FE Model Formulation	80

3.3.7 Original Walls - FE Models	81
3.3.8 FE Models Validation and Analysis	82
3.3.9 Repaired Walls - FE Models.....	85
CHAPTER 4. EXPERIMENTAL PROGRAM	91
4.1 Material Characterization Tests	91
4.1.1 Flow Test	91
4.1.2 Uniaxial Compression Test.....	91
4.1.3 Flexural Beam Test	92
4.1.4 Uniaxial Tensile Test	94
4.2 Development of Engineering Cementitious Composite (ECC) Mixture	95
4.2.1 Materials	95
4.2.2 Trials	97
4.2.3 Mechanical Behavior	99
4.3 Repairing Procedure.....	102
4.3.1 Bracing System	104
4.3.2 Concrete Removal.....	106
4.3.3 Replacement and Addition of Reinforcing Bars	109
4.3.4 Formwork Design and Construction	116
4.3.5 Batching, Casting and Curing of the In-house Design Mix.....	119
4.3.6 Repaired Walls.....	123
4.4 Experimental Setup, Instrumentation and Testing.....	124
4.4.1 Global Test Assembly.....	124
4.4.2 Instrumentation	127
4.4.3 Loading Protocol.....	135
CHAPTER 5. EXPERIMENTAL RESULTS	139
5.1 Material characterization	139
5.1.1 Concrete Properties.....	139

5.1.2 Reinforcement Properties.....	143
5.2 Repaired Shear Walls Tests	146
5.2.1 Crack Characteristics	146
5.2.2 Load – Displacement Response	160
5.2.3 Mid-height Displacement.....	167
5.2.4 Reinforcement Strains.....	169
CHAPTER 6. DISCUSSION OF RESULTS.....	175
6.1 Introduction.....	175
6.1.1 Stiffness and Strength	175
6.1.2 Vertical Displacements	177
6.1.3 Rotations	180
6.1.4 Effect of Starter Bars	182
6.1.5 Shear Distortions.....	184
6.2 Pre- and Post-Repairing Behavior	186
6.2.1 Global Load-Displacement Responses	186
6.2.2 Secant Stiffness.....	188
6.2.3 Lateral Load Capacity.....	189
6.2.4 Recovery Capacity	191
6.2.5 Energy Dissipation.....	194
6.2.6 Failure Modes	195
CHAPTER 7. NUMERICAL STUDY	198
7.1 Updated FE models.....	198
7.2 Materials Properties and Constitutive Models.....	199
7.3 Load-Displacement Response.....	200
7.4 Energy Dissipation.....	203
7.5 Recovery Capacity	204

7.6 Failure Mode and Cracking Pattern	205
CHAPTER 8. CONCLUSIONS AND RECOMMENDATIONS	208
8.1 Conclusions.....	208
8.1.1 Preliminary Numerical Analysis.....	208
8.1.2 ECC Mixture.....	209
8.1.3 Structural Performance of Repaired Walls	209
8.1.4 Comparison Between Pre- and Post-Repairing Behavior of Walls	212
8.1.5 Numerical Study	215
8.2 Recommendations	216
REFERENCES.....	218
APPENDICES	227
Appendix A Load - Deformation Responses from Cable and Linear Potentiometers.....	227
A.1 Lateral Displacements.....	227
A.2 Vertical Displacements	232
A.3 Diagonal Displacements	238
A.4 Sliding and Uplift of Foundation Block.....	240
Appendix B. Strain gauges' Response.....	243
B.1 Strain Gauges Used in Wall RSWS-R	243
B.2 Strain Gauges Used in Wall RSWN.....	259

List of Tables

CHAPTER 2. LITERATURE REVIEW

Table 2.1 Mix proportions for ECC (Georgiou and Pantazopoulou, 2016)	45
Table 2.2 Mix proportions for ECC (Eshghi, 2018)	46
Table 2.3 Proposed mix design proportions for a water to binder ratio of 0.2 and 0.18 (Shao, 2016)	48
Table 2.4 Design mix proportions (Saikali, 2019)	49

Chapter 3. PRELIMINARY ANALYSIS

Table 3.1 Concrete material properties for original shear walls (Morcos and Palermo, 2019)	62
Table 3.2 Mechanical properties of steel reinforcement used in the original walls (Morcos, 2020)	65
Table 3.3 Mechanical properties of SE-SMA reinforcement used in the original Wall SWN (Morcos, 2020)	66
Table 3.4 Concrete constitutive models employed in the analyses with VecTor2	75
Table 3.5 Reinforcement constitutive models employed in VecTor2.....	79

CHAPTER 4. EXPERIMENTAL PROGRAM

Table 4.1 Materials used for ECC trials	96
Table 4.2 Com-K mix proportions with the addition of PVA fibres.....	98
Table 4.3 Modified-Com K mix proportions	98
Table 4.4 In-house mix proportions	99
Table 4.5 Volume required per wall.....	119
Table 4.6. Modified - Mix proportions for Engineered Cementitious Composite (ECC)	119

CHAPTER 5. EXPERIMENTAL RESULTS

Table 5.1 Cylinder compression strength of walls	139
Table 5.2 Tensile flexural strength properties of mix ECC-RSWS-R	142
Table 5.3 Performance points for repaired shear walls	166

CHAPTER 6. DISCUSSION OF RESULTS

Table 6.1 Performance points of original and repaired walls	188
--	-----

CHAPTER 7. NUMERICAL STUDY

Table 7.1 Performance points of numerical (FE models) and experimental responses of Walls RSWS-R and RSWN.....	203
---	-----

List of Figures

CHAPTER 1. INTRODUCTION

Figure 1.1 Damage to shear walls: (a) concrete damage; and (b) rupture of the reinforcing bars (Wallace et al., 2012)	1
Figure 1.2 Cost of bridgework present in Ontario and tendered between 2009 and 2014 (Mermigas, 2018)	2
Figure 1.3 Typical reinforced concrete shear wall.....	4
Figure 1.4 Slender shear wall failure modes: (a) crack pattern; and (b) critical zones (Medhekar, M. S.; Jain, 1993)	6
Figure 1.5 Squat shear wall failure modes: (a) crack pattern; and (b) critical zones (Medhekar, M. S.; Jain, 1993).....	7
Figure 1.6 Yield displacement Δ_y based on the reduced stiffness equivalent elasto-plastic method (Park, 1988).....	8
Figure 1.7 Ultimate displacement Δ_u based on a significant load capacity after peak load (Park, 1988)	9
Figure 1.8 Energy dissipated for a complete cycle of a hysteretic response (Park and Eom, 2004)	10
Figure 1.9 Ductile failure mode of a beam-column joint due to flexural plastic hinging of the beams (Park, 1988).....	11
Figure 1.10 Brittle failure modes: (a) shear failure of an RC structural wall; and (b) beam-column joint due to bond deterioration (Park, 1988).....	11
Figure 1.11 Austenite Stress-Strain Transformation (Abdulridha and Palermo, 2013).....	12
Figure 1.12 Salient SMA Properties for Structural Applications (McCormick, 2007)	13
Figure 1.13 Stress-strain response under direct tension for strain hardening and softening materials (Naaman and Reinhardt, 2003)	15
Figure 1.14 Different types of steel and synthetic fibers added to the concrete matrix (Buttignol, Sousa and Bittencourt, 2017).....	16
Figure 1.15 Mechanical Properties of UHPFRC and ECC: (a) Uniaxial compression; and (b) direct tension	17

CHAPTER 2. LITERATURE REVIEW

Figure 2.1 SMA-RC column reinforcement detailing (Saiidi and Wang, 2006)	23
Figure 2.2 Finite element model of the SMAC-1 columns (Alam, Youssef and Nehdi, 2008).....	24
Figure 2.3 Base shear versus top displacement response: (a) experimental test (Saiidi and Wang, 2006); and (b) FE model (Alam, Youssef and Nehdi, 2008).....	24
Figure 2.4 Building layout for testing: (a) location of the joint; and (b) reinforcement detailing for the SMA-BCJ (JBC-2). (Youssef, Alam and Nehdi, 2008).....	25
Figure 2.5 Responses of beam: (a) JBC-1 and (b) JBC-2; crack pattern at 72 mm displacement of: (c) JBC-1 and (d) JBC-2 (Youssef, Alam and Nehdi, 2008)	26
Figure 2.6 Numerical model for JBC-2: (a) FE model; and (b) reverse-cyclic response (Alam, Youssef and Nehdi, 2008).....	27
Figure 2.7 Reinforcement details and dimensions of W2-NR: (a) elevation view; (b) section 1-1; and (c) Section 2-2 (Abdulridha and Palermo, 2017).....	27
Figure 2.8 Responses of Walls W1-SR and W2-NR: (a) lateral load-drift envelope responses; and (b) displacement recovery capacity-lateral drift responses.....	28
Figure 2.9 Conditions of the walls at failure: (a) W1-SR; (b) W2-NR (Abdulridha and Palermo, 2013); (c) rupture and buckling of steel reinforcement in the web of W2-NR; and (d) buckling of SMA bars in the boundary zone for W2-NR (Zaidi, Palermo and Dragmeriscu, 2016)	29
Figure 2.10 Lateral load-displacement response for W2-NR: (a) experimental; and (b) numerical (Abdulridha and Palermo, 2017).....	30
Figure 2.11 Mechanical couplers: (a) Original study by Abdulridha and Palermo (2013); and (b) study by Morcos and Palermo (2019).	31
Figure 2.12 Experimental load-displacement response for Wall: (a) SWS-R; and (b) SWN (Morcos and Palermo, 2019)	31
Figure 2.13 Numerical load-displacement response for Wall: (a) SWS-R(Morcos and Palermo, 2019)(Morcos and Palermo, 2019); and (b) SWN (Morcos and Palermo, 2018).....	32
Figure 2.14 FRP strengthening after removal of damaged concrete, replacement of ruptured steel bars and casting of new mortar in the bottom region of the walls	33
Figure 2.15 DP1 and DP1R: (a) repair components for DP1R; (b) load-displacement for DP1; and (c) load-displacement for DP1R	34
Figure 2.16 Beam tip load-storey drift responses: (a) JBC-2; and (b) JBC-3.....	35

Figure 2.17 Damage in JBC-3: (a) crack pattern at the rear face; and (b) cracking in the repair concrete	36
Figure 2.18 Details of RW2-NR: (a) schematic of reinforcement layout; and (b) photo before casting of SCC (Zaidi, Palermo and Dragmeriscu, 2016).....	37
Figure 2.19 Stress-strain response of original and retested SMA bars (Zaidi, Palermo and Dragmeriscu, 2016).....	37
Figure 2.20 Responses of walls before and after repair: (a) lateral load-drift envelope response for the positive direction; and (b) displacement recovery-drift response (Cortés-Puentes et al., 2018) .	38
Figure 2.21 Evolution of terminology of Fiber Reinforced Concrete (Naaman, 2018).....	39
Figure 2.22 FRC classification: (a) classification based on tensile and bending behavior (Naaman and Reinhardt, 2006); and (b) sub-division of cementitious materials based on enhanced properties (Habel, 2004)	40
Figure 2.23 Relationship between maximum aggregate size and fiber length (Johnston, 1996)	42
Figure 2.24 Different shapes of steel fibers (Naaman, 2003)	43
Figure 2.25 Properties of straight, twisted and hooked steel fibers: (a) energy absorption capacity; and (b) fracture energy (Wille, El-Tawil and Naaman, 2014).....	44
Figure 2.26 Test results: (a) stress-strain from direct tensile for SHCC; (b) dog-bone failure mode; (c) comparison of the four-point bending response for HVFA (control) and SHCC; and (d) failure mode of HVFA (top) and SHCC (bottom)	46
Figure 2.27 Prism testing: (a) load-midpoint deflection for prisms under flexure; (b) failure mode of prism S1; and (c) prism S2 (Eshghi, 2018).....	47
Figure 2.28 Test results: (a) stress-strain relationship of mix design under direct tension; and (b) cracking at failure (Shao, 2016).....	48
Figure 2.29 Prism test results: (a) load-mid-deflection response; and (b) mode of failure (Saikali, 2019)	49
Figure 2.30 Dimensional and reinforcement details: (a) elevation view of Wall CW; (b) elevation view of Wall SMA-SFRC; (c) cross-section of CW; (d) cross-section of SMA-SFRC above 780 mm; and (e) cross-section of SMA-SFRC below 550 mm (Tolou Kian and Noguez, 2018)	50
Figure 2.31 Behaviour of walls: (a) envelope response; (b) hysteresis response for CW; (c) hysteresis response for SMA-SFRC; and (d) observed damaged at failure (Tolou Kian and Noguez, 2018) ...	51

Figure 2.32 Specimens details: (a) nominal dimensions and longitudinal reinforcement; (b) cross-section details for semi-column with steel reinforcement; (c) cross-section details for semi-column with SMA reinforcement	52
Figure 2.33 Experimental lateral load-drift response: (a) HPC; and (b) VHPC (Pereiro-Barceló et al., 2018)	53
Figure 2.34 Numerical modelling: (a) FE model of the specimen; (b) hysteretic responses for HPC; and (c) hysteretic responses for VHPC (Pereiro-Barceló et al., 2018)	54
Figure 2.35 Highway 99 in Seattle: (a) Off-ramp exit bridge; and (b) column reinforcement detailing	54
Figure 2.36 Comparison between SMA-RC, SMA-ECC, ECC and RC beams under cyclic loading: (a) self-centering factor; and (b) spatial strain measured by DIC after unloading of the 7 th cycle....	55
Figure 2.37 SMA-ECC test results: (a) stress-strain behavior of composites under direct tension; (b) impact energy enhancement: (c) ECC2-0 at failure; and (d) ECC2-1.5 at failure.....	56
CHAPTER 3. PRELIMINARY ANALYSIS	
Figure 3.1 Three-dimensional representation of Wall SWN: (a) nominal dimensions; and (b) reinforcement distribution in plastic hinge	59
Figure 3.2 Nominal dimensions and reinforcement details for Wall SWN (Morcos, 2020)	60
Figure 3.3 Nominal dimensions and reinforcement details for Wall SWS-R (Morcos, 2020).....	61
Figure 3.4. Walls after casting: SWS-R (behind) and SWN (forward) (Morcos and Palermo, 2018)	62
Figure 3.5 Typical stress-strain response of 10M reinforcement for Wall SWS-R (Morcos, 2020)	63
Figure 3.6 Typical stress-strain response of 10M reinforcement for Wall SWN (Morcos, 2020) ...	63
Figure 3.7 Cyclic stress-strain response for SMA reinforcement (Morcos, 2020)	64
Figure 3.8 Cyclic response for 10 M bar for Wall SWS-R (Morcos, 2020).....	64
Figure 3.9 Cyclic response for 10 M bar for Wall SWN (Morcos, 2020)	65
Figure 3.10 Conditions of the original walls after testing: (a) SWS-R; and (b) SWN	67
Figure 3.11 Damage in the plastic hinge: (a) SWS-R; and (b) SWN	68
Figure 3.12 Concrete removal options measured from the base of the wall: (a) 300 mm; (b) 500 mm; and (c) 900 mm	69
Figure 3.13 Reinforcement detailing proposed based on the height of concrete removed: (a),(b),(c) for SWS-R; and (d),(e),(f) for SWSN	71

Figure 3.14 Material properties for UHPFRC: (a) input material properties for compression response; and (b) input values representing the custom-input-strain based tension softening in Vector2 (Saikali (2019) and Wang (2003)).....	77
Figure 3.15 Experimental and numerical responses: (a) 280 mm long-prisms (P1); and (b) 500 mm-long prisms (P2) (Saikali, 2019)	77
Figure 3.16 FE modelling of ECC prisms: (a) material properties for ECC used in VecTor2; (b) experimental and numerical responses for 280 mm long-prisms (P1); and (c) experimental and numerical responses for 500 mm-long prism (P2).....	78
Figure 3.17 SMA plastic offset constitutive model (Abdulridha and Palermo, 2013)	79
Figure 3.18 Defined regions for FE models.....	80
Figure 3.19 FE models: (a) SWS-R; and (b) SWN.....	81
Figure 3.20 Comparison of load-displacement responses: (a) numerical SWS-R; (b) experimental SWS-R; (c) numerical SWN; and (d) experimental SWN (Morcos and Palermo, 2018).....	83
Figure 3.21 Cracking pattern at 2.5% drift predicted by the numerical models: (a) SWS-R; and (b) SWN.....	84
Figure 3.22 Stress levels in longitudinal reinforcing bars at peak load: (a) SWS-R; and (b) SWN.....	85
Figure 3.23 Numerical models for Wall RSWN: (a) one-half of plastic hinge removed; and (b) entire plastic hinge removed	86
Figure 3.24 Load-displacement responses for models with half of the plastic hinge replaced: (a) RSWS-R with UHPFRC; (b) RSWS-R with ECC; (c) RSWN with UHPFRC; and (d) RSWN with ECC.....	87
Figure 3.25 Cracking pattern of numerical models for Wall RSWN: (a) without starter bars; and (b) with the addition of starter bars	89
Figure 3.26 Load-displacement responses of repaired numerical models with replacement of the entire plastic hinge zone: (a) conventional steel wall; and (b) SMA wall	90
CHAPTER 4. EXPERIMENTAL PROGRAM	
Figure 4.1 Uniaxial compression test setup	92
Figure 4.2 Prisms dimensions for four-point bending test (all units shown in millimeters) (Saikali, 2019)	93
Figure 4.3 Four-point bending test on Prism P1	93
Figure 4.4 Uniaxial tensile test setup	95

Figure 4.5 Load - actuator displacement response.....	100
Figure 4.6 Cracking patterns for prism with: (a) Modified-Com K mix; and (b) In-House mix....	101
Figure 4.7 Micro cracking at the bottom surface of prisms with In-House mix design: (a) without marking; and (b) with marking	101
Figure 4.8 General repairing procedure for damaged shear walls: (a) previous damage; (b) bracing system; (c) concrete removal in the plastic hinge and replacement/addition of reinforcing bars; (d) formwork and casting; and (e) repaired wall	103
Figure 4.9 Preliminary design of the bracing system: (a) general view; (b) front view; (c) top view; and (d) lateral view	104
Figure 4.10 Materials employed for the bracing system prior to repairing	105
Figure 4.11 Bracing system for the walls: (a) front view; and (b) side view.....	105
Figure 4.12 First stage of concrete removal and damage of reinforcing bars: (a) SWS-R; and (b) SWN.....	106
Figure 4.13 Second stage of concrete removal: (a) SWS-R; and (b) SWN.....	107
Figure 4.14 Current state of couplers for SWN: (a) west side; and (b) east side.....	108
Figure 4.15 Third stage of concrete removal: (a) SWS-R; and (b) SWN.....	108
Figure 4.16 Segments of steel reinforcement removed in plastic hinge region: (a) SWS-R; and (b) SWN.....	110
Figure 4.17 Mechanical coupler placed at the base of wall: (a) SWS-R; and (b) SWN.....	111
Figure 4.18 Concrete removal in the foundation: (a) SWS-R; and (b) SWN	111
Figure 4.19 Reinforcing bars at the base of wall: (a) SWS-R; and (b) SWN	112
Figure 4.20 Flattening of existing reinforcing bars at the top of the plastic hinge: (a) SWS-R; and (b) SWN.....	112
Figure 4.21 Drilled holes for starter bars	113
Figure 4.22 Installation of starter bars: (a) SWS-R; and (b) SWN.....	114
Figure 4.23 Mechanical coupler installation.....	115
Figure 4.24 Final reinforcement layout: (a),(b) Wall SWS-R; and (c),(d) Wall SWN.....	116
Figure 4.25 Formwork design for the plastic hinge	117
Figure 4.26 Constructred formwork: (a) top view; and (b) side view	118
Figure 4.27 Formwork installed on Wall SWN	118

Figure 4.28 ECC mixing procedure: (a) mixing of dry material; (b) addition of water with superplasticizer; (c) addition of PVA fibers; and (d) flowability of the mix.....	120
Figure 4.29 Procedure in Casting: (a) Casting of the wall; (b) pouring of ECC through the designed compartment; and (c) additional prisms and cylinders	121
Figure 4.30 Deficiency in Wall S: (a) Poor distribution at the top boundary of the plastic hinge region; and (b) casting of SCC in the affected region	122
Figure 4.31 Demolding of formwork: (a) Wall SWS-R; and (b) Wall SWN	122
Figure 4.32 Curing setup: (a) Wall SWS-R; and (b) Wall SWN.....	123
Figure 4.33 Repaired walls: (a) RSWS-R; and (b) RSWN.....	124
Figure 4.34 Global test setup assembly for repaired walls	125
Figure 4.35 Assembled experimental test setup.....	126
Figure 4.36 Top view of the test assembly: (a) graphic representation; and (b) actual experimental setup	127
Figure 4.37 Installation of strain gauges on new segments of steel bars	128
Figure 4.38 Strain gauge location: (a) Wall RSWS-R (front); (b) Wall RSWS-R (back); (c) Wall RSWN (front); and (d) Wall RSWN (back)	129
Figure 4.39 Instrumentation: (a) schematic illustration; (b) location and; (c) photo of instrumentation on the wall.....	131
Figure 4.40 Location of cable potentiometers for lateral displacement measurements: (a) CP1; (b) CP2; (c) CP3; and (d) CP4.....	132
Figure 4.41 Location of cable potentiometers for vertical displacement measurements: (a) CP5 and CP7; and (b) CP6 and CP8	132
Figure 4.42 Cable potentiometers: (a) sliding of foundation block (CP9); and (b) top lateral displacement relative to the strong floor (CP10)	133
Figure 4.43 Cable potentiometers utilized to measure vertical and diagonal displacements in the plastic hinge	134
Figure 4.44 Linear potentiometers employed to measure uplift of the foundation block.....	134
Figure 4.45 Loading protocol for Wall RSWS-R	136
Figure 4.46 Loading protocol for Wall RSWN.....	136
Figure 4.47 Summarized flow chart of the loading protocol programmed in the MTS - MPE software	137

CHAPTER 5. EXPERIMENTAL RESULTS

Figure 5.1 Compression cylinder failure mode: (a) ECC-RSWS-R; (b) ECC-RSWN; and (c) SCC-RSWS-R.....	141
Figure 5.2 Load-mid-span displacement of ECC prisms: (a) P1 (280 mm); and (b) P2 (500 mm)	142
Figure 5.3 State of prisms at the end of testing: (a) P1 bottom; (b) P2 bottom; (c) P1 side; and (d) P2 side	143
Figure 5.4 Typical monotonic stress-strain for 10M steel coupon	143
Figure 5.5 Stress-strain response with 6% strain cycle of 10M steel and SMA bars (Morcos, 2020)	144
Figure 5.6 Cyclic stress-strain response of reinforcement utilized in repaired walls	145
Figure 5.7 Milestone points for crack width and length measurements	146
Figure 5.8 Crack pattern in RSWN at 0.2% drift.....	147
Figure 5.9 Crack pattern at 0.3% drift: (a) RSWS-R; and (b) RSWN.....	148
Figure 5.10 Crack pattern at 0.4% drift: (a) RSWS-R; and (b) RSWN.....	149
Figure 5.11 Crack pattern at 0.5% drift: (a) RSWS-R crack; and (b) RSWN	151
Figure 5.12 Crack pattern at 1% drift: (a) RSWS-R; and (b) RSWN.....	153
Figure 5.13 Crack pattern at 1.5 %drift: (a) RSWS-R; and (b) RSWN.....	154
Figure 5.14 Crack pattern at 2% drift: (a) RSWS-R; and (b) RSWN.....	156
Figure 5.15 Crack pattern at 2.5 %drift: (a) RSWS-R; and (b) RSWN.....	157
Figure 5.16 Crack pattern at 3% drift: (a) RSWS-R; and (b) RSWN.....	158
Figure 5.17 Wall RSWN crack pattern at 3.5 % drift.....	159
Figure 5.18 Crack pattern at 4% drift for Wall RSWN	159
Figure 5.19 Crack pattern at the end of testing: (a) 3% drift for RSWS-R; and (b) 4% drift for RSWN	160
Figure 5.20 Load – displacement response of Wall RSWS-R.....	161
Figure 5.21 Load – displacement response of Wall RSWN	161
Figure 5.22 Response of Wall RSWS-R: a) peak load cycle; and b) imposed damage.....	163
Figure 5.23 Response of Wall RSWN-R: a) peak load cycle; and b) imposed damage.....	164
Figure 5.24 Ruptured reinforcement in Wall RSWS-R: (a) left boundary; and (b) right boundary	165

Figure 5.25 Damage in Wall RSWN: (a) ruptured steel bar in web region; and (b) undamaged SMA bars	165
Figure 5.26 Load-displacement response relative to the base (CP1) and the strong floor (CP10): (a) Wall RSWS-R; and (b) Wall RSWN	167
Figure 5.27 Mid-height lateral displacement of Wall RSWS-R	168
Figure 5.28 Mid-height lateral displacement of Wall RSWN	168
Figure 5.29 Load – strain response of Strain Gauge SG1 for Wall RSWS-R	170
Figure 5.30 Load – strain response of Strain Gauge SG1 for Wall RSWN.....	170
Figure 5.31 Load – strain response of Strain Gauge SG14 for Wall RSWS-R	171
Figure 5.32 Load – strain response of Strain Gauge SG14 for Wall RSWN.....	172
Figure 5.33 Load – strain response of Strain Gauge SG30 for Wall RSWS-R	173
Figure 5.34 Load – strain response of Strain Gauge G30 for Wall RSWN.....	174
CHAPTER 6. DISCUSSION OF RESULTS	
Figure 6.1 Secant stiffness-displacement ductility response for repaired walls	175
Figure 6.2 Effect of cyclic loading degradation for repaired walls	176
Figure 6.3 Vertical displacements in Wall RSWS-R: loading beam (CP5); b) plastic hinge (CP11);base of wall (CP7).....	177
Figure 6.4 Vertical displacements in Wall RSWN: loading beam (CP5); b) plastic hinge (CP11);base of wall (CP7).....	178
Figure 6.5 Envelope of vertical displacements for Walls RSWS-R and RSWN.....	179
Figure 6.6 Load-rotation response for Wall RSWS-R at the top loading beam	180
Figure 6.7 Load-rotation response for Wall RSWN at the top loading beam.....	181
Figure 6.8 Load-rotation response for Wall RSWS-R at: (a) base of the wall; (b) upper limit of the plastic hinge	181
Figure 6.9 Load-rotation response for Wall RSWN at: (a) base of the wall; (b) upper limit of the plastic hinge	182
Figure 6.10 Lateral displacement at the base of the repaired walls: (a) RSWS-R; and (b) RSWN.....	183
Figure 6.11 State of the base of the walls at the end of testing: (a) SWN; and (b) RSWN	183
Figure 6.12 Load – shear strain response of Wall RSWS-R.....	184
Figure 6.13 Load – shear strain response for Wall RSWN.....	185
Figure 6.14 Load-displacement response of Walls SWS-R and RSWS-R.....	187

Figure 6.15 Load-displacement response of Walls SWN and RSWN.....	187
Figure 6.16 Secant stiffness – drift responses: (a) conventional steel wall; and (b) SMA wall	189
Figure 6.17 Lateral load-drift envelope for Walls SWS-R and RSWS-R	190
Figure 6.18 Lateral load-drift envelope for Walls SWN and RSWN	190
Figure 6.19 Comparison of hysteretic response at 2.5% drift for Walls RSWS-R and RSWN	191
Figure 6.20 Displacement recovery capacity - drift responses for Walls SWS-R and RSWS-R ...	193
Figure 6.21 Displacement recovery capacity - drift responses for Walls SWN and RSWN	193
Figure 6.22 Hysteretic response at 1.5% drift: (a) steel walls; and (b) SMA walls.....	194
Figure 6.23 Energy dissipated at different drift levels for steel and SMA walls	195
Figure 6.24 Crack pattern at failure: (a) Wall SWS-R; and (b) Wall RSWS-R	196
Figure 6.25 Crack pattern at failure: (a) Wall SWN; and (b) Wall RSWN	196
Figure 6.26 Pulling out of PVA fibers at the localized cracks: (a) boundary zone; and (b) web region	197
CHAPTER 7. NUMERICAL STUDY	
Figure 7.1 Updated FE models: (a) RSWS-R; and (b) RSWN.....	198
Figure 7.2 ECC constitutive model: (a) updated material properties used in VecTor2; b) response curves of Prism P1; and response curves of Prism P2	199
Figure 7.3 Load-displacement response for Wall RSWS-R	200
Figure 7.4 Load-displacement response for Wall RSWN.....	200
Figure 7.5 Lateral load-drift envelope responses for Wall RSWS-R	202
Figure 7.6 Lateral load-drift envelope responses for Wall RSWN.....	202
Figure 7.7 Energy dissipation-drift response for FE model and experimental response of Walls RSWS-R and RSWN	203
Figure 7.8 Displacement recovery-drift response for FE model and experimental testing of Wall RSWS-R.....	205
Figure 7.9 Displacement recovery-drift response for FE model and experimental testing of Wall RSWN	205
Figure 7.10 Cracking pattern at the end of the testing for Wall RSWS-R: (a) Experimental; and (b) Numerical.....	206
Figure 7.11 Cracking pattern at the end of the testing for Wall RSWN: (a) Experimental; and (b) Numerical.....	207

List of Abbreviation

CFRP	Carbon Fiber Reinforced Polymer
CSA	Canadian Standards Association
DFRCC	Ductile Fiber Reinforced Cement Composited
DSP	Densified Small Particles
ECC	Engineered Cementitious Composites
FEM	Finite Element Modelling
FRC	Fiber-Reinforced Concrete
GGBS	Ground granulated blast furnace slag
HPFRC	High Performance Fiber Reinforced Concrete
HPFRCC	High Performance Fiber Reinforced Cementitious Composite
MTO	Ministry of Transportation of Ontario
MDF	Macro-Defect-Free
NiTi	Nickel-Titanium
PE	Polyethylene
PP	Plastic Polypropylene
PVA	Polyvinyl Alcohol
RPC	Reactive Powder Concrete
RC	Reinforced Concrete
SCC	Self-Compacted Concrete
SIFCON	Slurry Infiltrated Fiber Concrete
SIMCON	Slurry Infiltrated Mat Concrete
SF	Steel Fiber
SP	Superplasticizer
SMA	Shape Memory Alloy
SE-SMA	Super-elastic Shape Memory Alloy
UHPC	Ultra-High-Performance Concrete
UHPFRC	Ultra-High-Performance Fiber Reinforced Concrete
UHPSFRC	Ultra-High-Performance Steel Fiber Reinforced Concrete

List of Symbols

A_s	Cross-section of the reinforcing bar (mm ²)
a	Shear span (mm)
a_{max}	Maximum aggregate size (mm)
c	Weight of the cement (kg)
d_f	Diameter of the steel fibers (mm)
d_h	Diameter of the hole (mm)
D	Diameter of the flow table (mm)
E_D	Dissipated energy
E_i	Initial Elastic Modulus
F_b	Force in the bar (kN)
f_t	Tensile strength (psi)
f'_c	Concrete compressive strength at 28 days (MPa)
G_f	Fracture energy
ℓ	Length (mm)
ℓ_f	Length of fiber (mm)
P	Applied load (kN)
P_{max}	Maximum applied load (kN)
P_u	Ultimate load applied (kN)
RVC	Recovery capacity
w	Weight of the water (kg)
w/b	Water-binder ratio
w/c	Water-cement ratio
V_f	Volumetric fraction of fibers (%)
V_y	Yield load
$\emptyset$	Diameter (mm)
$\emptyset_f$	Diameter of fiber (mm)
σ_{cc}	Tensile strength corresponding to first crack in direct tension(MPa)
$\sigma_{flexure}$	Bending stress (MPa)

ε_{cc}	Tensile strain corresponding to first crack in direct tension
ε_{ct}	Matrix tensile strain
ε_{pc}	Tensile strain corresponding to peak load in direct tension
ε_s	Reinforcement strain
μ	Displacement ductility
Δ_{peak}	Maximum displacement (mm)
Δ_{res}	Residual displacement (mm)
Δ_u	Ultimate displacement (mm)
Δ_y	Yield displacement (mm)
σ_L	Forward transformation
σ_{UL}	Reverse transformation
ε_R	Residual strain,
ξ_{eq}	Equivalent Viscous Damping

CHAPTER 1. INTRODUCTION

1.1 Background

In seismic regions, it is common to expect the presence of significant telluric events such as earthquakes. This type of event often causes extensive human and material losses. New design codes and non-standard documents such as the Eurocode (2004) and FEMA P-1050 (2015), respectively, mainly concentrate on the structure's ductility and energy dissipation to accommodate higher seismic demands, eliminating their potential collapse and guaranteeing a life safety performance level. However, designing based on the inelastic response of conventional construction materials results in significant residual deformation due to the yielding of reinforcement and damage from the response of the concrete (441 ACI Committee, 1996). Although the structures built in the major cities of Chile were designed based on relatively current codes (ACI 318-05), they experienced severe permanent damages during the 2010 earthquake. Figure 1.1 (a) and (b) present the damage endured by the reinforced concrete shear walls located in the first story of new high-rise buildings in the cities of Santiago and Concepcion (Wallace *et al.*, 2012).

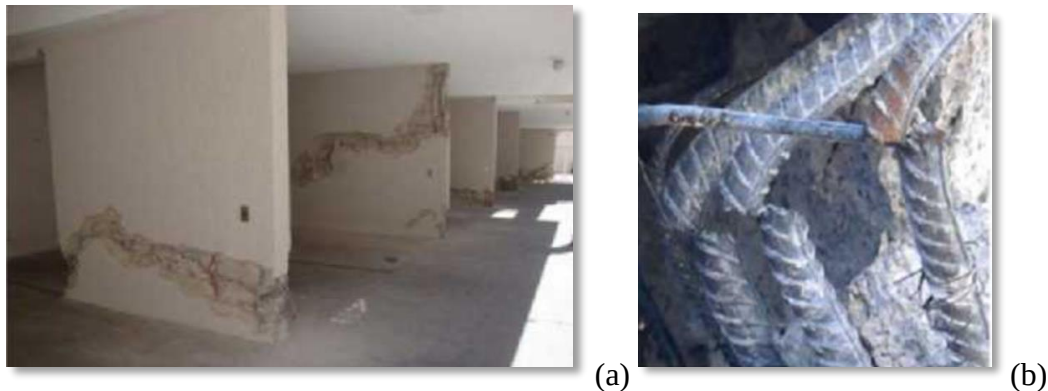


Figure 1.1 Damage to shear walls: (a) concrete damage; and (b) rupture of the reinforcing bars (Wallace *et al.*, 2012)

Thus, the infrastructure post-earthquake may be compromised in both structural integrity and functionality, leading to enormous economic deficits that may take years to be compensated. Faced with these facts, there are two clear alternatives: the first is eviction and subsequent demolition of the structure; an option that is economically expensive in terms of demolition costs, transporting and

discarding waste material, and then preparing the site to rebuild. The second option involves the application of repairing methods aiming to recover the initial strength, stiffness, and ductility of the structure.

Furthermore, performance-based and capacity-based design approaches are not applicable to all existing structures, and thus an upgrade is necessary to meet the requirements of current standards for adequate seismic performance. An example is the previously designed shear wall system in Reinforced Concrete (RC) structures that are currently considered under-designed in terms of strength, ductility, and energy dissipation capacities. Moreover, material degradation is another common type of damage in existing structures and superstructures that require rehabilitation to extend their life service. The Ministry of Transportation of Ontario (MTO) is liable for 2850 bridges, mostly constructed before the year 2000 and currently in need of enhancement. Figure 1.2 illustrates that bridge replacement would be 65% more expensive than proceeding with rehabilitation methods (Mermigas, 2018). In fact, most of the bridges built in the 1960s are being rehabilitated for the second time due to the limited properties of the conventional materials previously used.

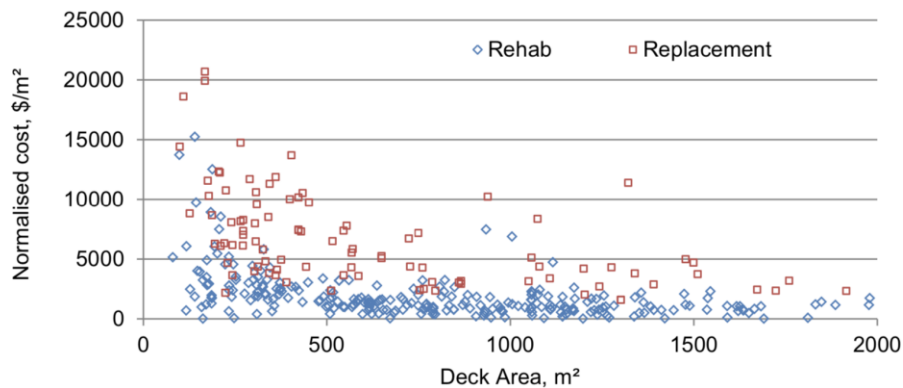


Figure 1.2 Cost of bridgework present in Ontario and tendered between 2009 and 2014 (Mermigas, 2018)

Therefore, both ductility and durability are two primary components that need to be considered when designing or rehabilitating new or existing reinforced concrete structures. A current research option to achieve this objective is the implementation of novel materials to replace traditional steel reinforcement and conventional concrete in reinforced shear walls and thus overcome their mechanical and performance limits. In this study, the focus will be on Shape Memory Alloys (SMAs)

to replace steel reinforcement and Engineering Cementitious Composites (ECC) to replace traditional concrete.

An attractive material is the Shape Memory Alloy (SMA), a smart material that has applications in retrofitting, repairing, and in new construction as a replacement of conventional steel reinforcement in critical regions owing to their self-centering ability. These metallic alloys can undergo large deformations and return to their original shape upon unloading at room temperature (Superelastic form) or heated (Shape Memory Effect) (Lagoudas, 1989). The ability of the material to experience a reversible hysteretic shape under cyclic loading provides energy dissipation capacity, while its pseudo yielding (phase transformation) phenomenon provides ductility.

High-Performance Fiber Reinforced Concrete (HPFRC) such as Engineered Cementitious Composite (ECC) and Ultra-High Steel-Fiber Reinforced Concrete (UHP-SFRC) are innovative concrete matrices composed of two main constituents: the high strength concrete matrix and fibers. This concrete category is mainly characterized by their behavior under tension with the ability to sustain tension cracking, the main feature that conventional concrete lacks resulting in its deterioration once subjected to loading. The high mechanical properties of both UHPSFRC and ECC make them attractive materials for repairing retrofitting and new applications.

The use of SMA reinforcing bars provides the structural element with higher displacement capacity and self-centering that requires a concrete, such as UHPSFRC and ECC, with the ability to sustain these deformations and limit the extent of damage to provide a self-centering and damage controlled structure. Limited research has been conducted on investigating the advantages of repairing involving the use of emerging High-Performance Fiber-Reinforced Concrete (HPFRC) combined with SMAs to enhance the seismic resistance of reinforced concrete shear walls along with controlling permanent damage in the concrete and promoting global self-centering of lateral displacements.

1.2 Scope of the Research

1.2.1 Shear Walls

1.2.1.1 *Definition*

Lateral forces induced by wind, earthquake, and uneven settlement can lead to substantial twisting (torsional) forces. Such loading, if not properly accounted for, can lead to shear failure of the structure. Shear walls are vertical elements of the lateral force-resisting system, used to sustain lateral forces parallel to the plane of the wall. These elements are one of the most commonly used in the lateral-load resisting systems in high-rise buildings, with a rigid vertical diaphragm capable of transferring lateral forces from exterior walls, floors, and roofs to the ground foundation in a direction parallel to their planes (Massone and Wallace, 2004). Generally, shear walls are either plane or flanged in section. They also provide adequate strength and stiffness to control lateral displacements. Figure 1.3 illustrates an example of a structure with perimeter reinforced-concrete walls.

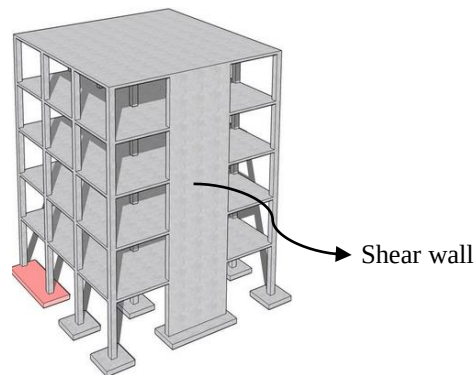


Figure 1.3 Typical reinforced concrete shear wall

Flexural shear walls are designed to respond to earthquake loading in a ductile mode with the ability to undergo large lateral deformations accompanied by energy dissipation. The plastic hinge of a reinforced concrete shear wall is the length where it is assumed that the inelastic curvatures are constant (Bohl and Adebar, 2011). It must be designed to suppress any type of brittle failure mode. During an earthquake event, the shear wall's main objective is to dissipate the energy through deformation. Nevertheless, the post-earthquake behavior of the structure could be compromised due

to the damage presented, causing uncertainty on the service levels of the whole structure and possibly restricting repair procedures (Abdulridha and Palermo, 2017).

1.2.1.2 *Type of Shear Walls and Failure Modes*

In general, the height-to-width ratio of a shear wall determines its behavior. For a ratio greater than 2, the wall will behave as a flexural (slender) wall. For a ratio not exceeding 2, the wall will respond as a squat wall. Moreover, each type of shear wall experiences different potential modes of failure. Based on previously damaged RC buildings from seismic activities or experimental research programs, several modes of failures have been identified.

1.2.1.2.1 *Slender Wall*

Slender walls present a height-to-width ratio exceeding 2, mainly characterized by high bending deformation capacity. These types of walls resist loads by acting as a vertical cantilever beam. The strength of properly designed slender walls is governed by flexure.

At the onset of inelastic response, horizontal cracking initiates in the plastic hinge region of slender shear walls subjected to cyclic lateral loading. The stress in the concrete surrounding the reinforcing bars at the bottom of the wall exceeds its tensile capacity and develops cracks. Owing to the cyclic loading, the first potential failure mode of slender shear walls can be dominated by the yielding and buckling of the longitudinal reinforcing bars due to the tensile and compressive stresses, respectively. In this case, the wall would fail by either rupture or buckling of the vertical flexural reinforcement. Due to the large lateral displacements that slender walls can accommodate, the second potential failure mode is the concrete crushing occurring at the toes of the wall at the extreme fibers when subjected to compressive stresses. However, this failure mode is not expected to occur based on design codes unless the flexural reinforcement is over-designed, the wall presents an asymmetrical geometry, or carries a large axial load. Once the concrete cover surrounding the reinforcing bars spalls, the third potential failure mode is due to buckling of the web reinforcing bars in the compression zone of the wall. Note that the presence of shear reinforcement or ties provides confinement or resistance to buckling of the bars. Given that significant damage tends to be

concentrated within the plastic hinge region (Figure 1.4), repairing methodologies for slender shear walls is focused on the plastic hinge at the bottom of the wall.

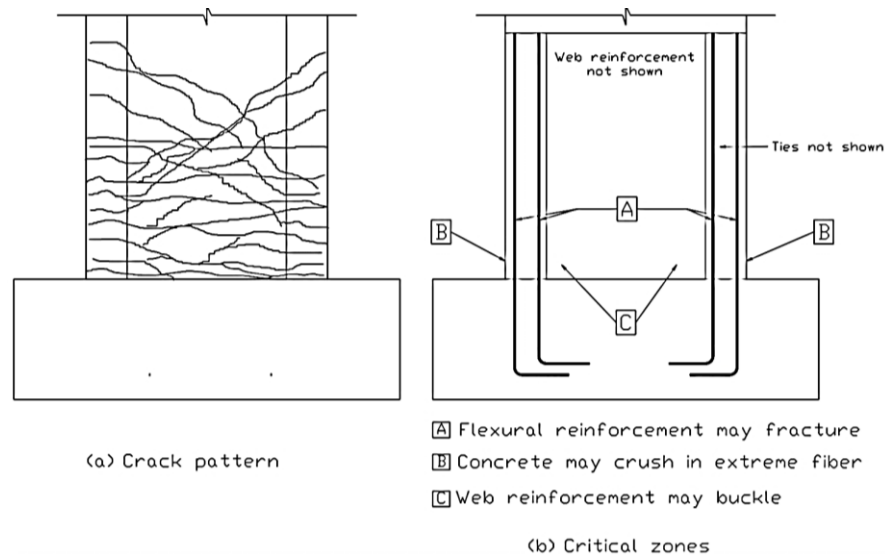


Figure 1.4 Slender shear wall failure modes: (a) crack pattern; and (b) critical zones (Medhekar, M. S.; Jain, 1993)

1.2.1.2.2 Squat Wall

Squat walls are characterized by a height-to-width ratio not exceeding 2, and the behavior is dominated by shear-related mechanisms. Shear failure can also occur when a lower quantity of shear reinforcement is considered than the design capacity. This failure mode is considered brittle and, thus, circumvented by ensuring the flexural strength of the wall controls the response, such that the design shear strength exceeds the shear demand corresponding to the flexural capacity. As shown in Figure 1.5, a squat wall can develop three modes of shear failure: diagonal tension, diagonal compression, or sliding shear (Paulay, Priestley and Syngé, 1982).

Based on equilibrium, the shear force from the in-plane horizontal loading generates tensile stresses, leading to the formation of diagonal tensile cracks. In case of an insufficient amount of shear reinforcement to prevent the propagation of the cracks, diagonal tension shear failure arises. However, in case of an adequate amount of shear reinforcement resisting the tensile stresses, higher compressive stresses are transferred to the compression struts parallel to the diagonal cracks. If the compressive stresses exceed the compressive capacity, concrete crushing and web failure may occur.

This is defined as diagonal compression failure. Sliding shear failure can occur following several cycles of loading, as a result of the propagation of tensile cracks that initiate at each boundary of the wall leading to a single horizontal crack at the base of the wall. Due to the opening of the crack, the shear force is resisted by shear friction of the concrete along the horizontal plane and dowel action of the vertical reinforcing bars. A low ratio of vertical reinforcement or a low applied axial load increases the probability of failure from shear sliding.

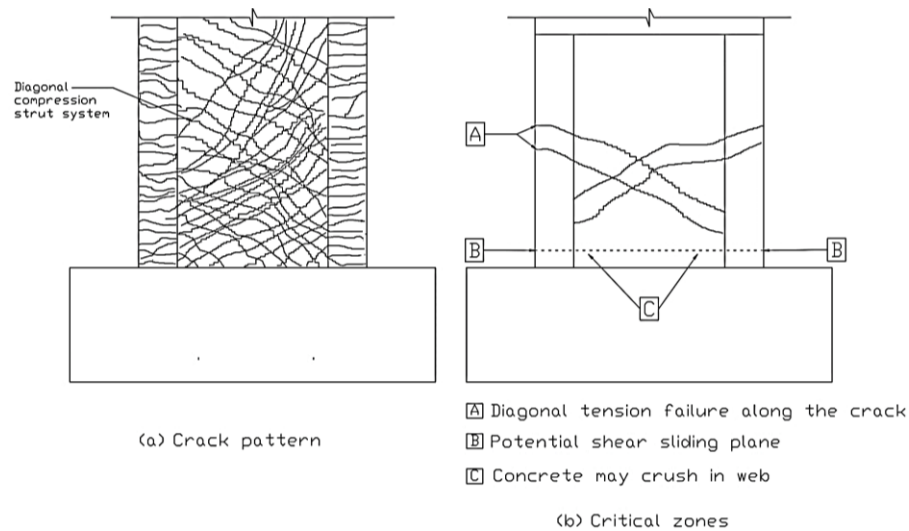


Figure 1.5: Squat shear wall failure modes: (a) crack pattern; and (b) critical zones (Medhekar, M. S.; Jain, 1993)

For both slender and squat shear walls, the anchorage of the longitudinal reinforcement from the wall into the foundation, if not adequately considered, can lead to an adverse response, such as rocking. This type of behaviour leads to rigid body motion adjacent to the foundation, which prevents the wall from realizing its lateral capacity. Another phenomenon that both types of walls can experience when subjected to cyclic loading, is out of plane movement. This behavior should be prevented to eliminate lateral buckling of the compression zone (Galal and El-Sokkarry, 2008).

1.2.1.3 Salient Response Parameters of Shear Walls

Several experimental methods have previously been used in laboratory settings to study and analyze the behavior of shear walls under cyclic loading for enhancement of design codes. The most commonly used procedure is the quasi-static load testing by means of hydraulic actuators and displacement-controlled cycles. The hysteretic response is then recorded and used to estimate the

ductility of the structure to assess the adequacy of the design criteria proposed. The method proposed by Park (1988) is used to establish the critical values of the experimental response curves to determine the ductility. The following sections describe the methods used to determine salient parameters of response from experimental responses.

1.2.1.3.1 Performance Parameters

1.2.1.3.1.1 Yield Displacement

Figure 1.6 illustrates the actual pre-peak response of a structure experience during pushover loading. Following the method by Park (1998), the yield displacement is obtained based on the reduced stiffness equivalent elasto-plastic yield method, where the nonlinear response of the structure is represented by an approximated bilinear response. It is defined by the intersection of a secant line starting from the origin and projecting through the response curve at 75% of the peak load with the horizontal plateau along the peak load as shown in Figure 1.6. At the point of intersection, a vertical line intersecting with the pushover curve represents the yield displacement, Δ_y , and the yield load, V_y .

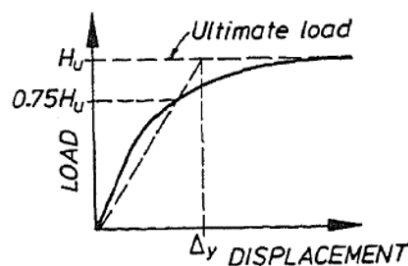


Figure 1.6 Yield displacement Δ_y based on the reduced stiffness equivalent elasto-plastic method (Park, 1988)

1.2.1.3.1.2 Ultimate Displacement

Typically, in ductile systems, the lateral response will reach its peak value followed by a gradual degradation in lateral resistance due to the influence of concrete crushing. In such cases, the ultimate displacement, Δ_u , corresponds to a 20% drop in the peak lateral load in the post-peak region. Figure 1.7 represents this method based on a significant loss in load capacity after peak load (Park, 1988). In

cases where the reinforcement fractures, the ultimate displacement is defined as the displacement at the onset of fracture and prior to a sudden drop in load capacity.

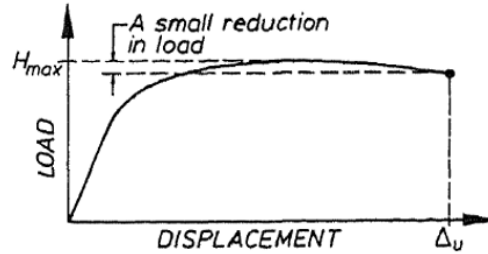


Figure 1.7 Ultimate displacement Δ_u based on a significant load capacity after peak load (Park, 1988)

1.2.1.3.1.3 Available and Cumulative Displacement Ductility

Displacement ductility, μ , by definition, is the ultimate displacement Δ_u divided by the yield displacement Δ_y obtained from Figure 1.6 and Figure 1.7, respectively. For the case of reverse cyclic loading, shear walls can be assessed based on its cumulative displacement ductility which is the sum of the ductility displacement obtained from each cycle for each direction $\sum \mu$. Alternatively, many studies use the average of the displacement ductility from each direction of loading.

1.2.1.3.1.4 Energy Dissipation

The energy dissipation capacity of a structure provides a measure of the ability to absorb the energy from a seismic event. Typical ductile reinforced concrete systems experience wide hysteretic loops due to the yielding of the reinforcement, which results in significant energy dissipation. By definition, the energy dissipated, E_D , by a structure during reverse cyclic loading is calculated as the area inside of a complete loop of the load-displacement response (Park and Eom, 2004), as shown in Figure 1.8.

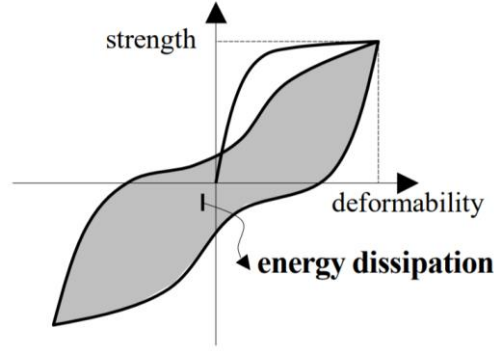


Figure 1.8 Energy dissipated for a complete cycle of a hysteretic response (Park and Eom, 2004)

Recent post-earthquake investigations have highlighted that the residual deformation state of a structure is a good indicator of the structural performance. Therefore, in addition to the importance of the structure to sustain large cyclic deformations, the capacity to recover the deformations is also critical to ensure a structure can be serviceable. The latter also provides a measure of the structural damage and whether repair is viable. The recovery capacity has become a critical parameter for new structures incorporating innovative materials since these materials are typically more expensive and intended to provide damage control and self-centering capacity. The displacement recovery capacity, RVC, is calculated as the ratio of the difference between the peak displacement and the residual displacement to the peak displacement for each drift level (Cortés-Puentes et al., 2018) and presented in Equation 1.1.

$$RVC = \frac{\Delta_{peak} - \Delta_{res}}{\Delta_{peak}} \times 100 \quad [1.1]$$

1.2.1.3.2 Hysteresis Response Behavior of Shear Walls

The hysteresis response represents the load-displacement behavior of a structure subjected to cyclic loading. Based on previous research, the characteristics of the hysteresis loops can be correlated to the prevailing failure mode of the structure. The inelastic deformations experienced by reinforced concrete structural elements under cyclic loading that is controlled by flexural failure with the development of plastic hinges are shown in Figure 1.9.

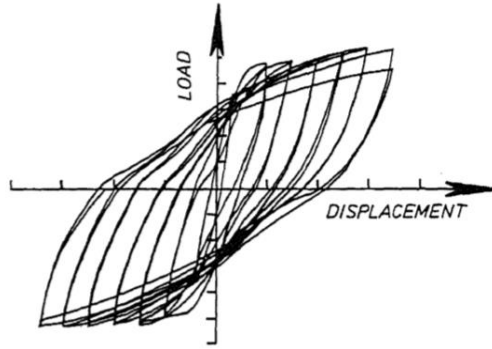


Figure 1.9 Ductile failure mode of a beam-column joint due to flexural plastic hinging of the beams (Park, 1988)

Brittle failure modes such as shear and bond mechanisms are depicted in Figure 1.10 (a) and (b), respectively. The bond or anchorage deterioration at the interface of the shear wall element and the foundation is described previously as a rocking failure mode that is controlled by the debonding of the longitudinal bars at the base of the wall. The cyclic response of a structure controlled by this brittle failure mode experiences a rapid degradation of the strength with increasing permanent deformations. In addition, the response is characterized by a significant decrease in stiffness. The pinching of the hysteresis loops is an indication of a response that is controlled by shear-related mechanisms (Figure 1.10 (a)). The presence of pinching is directly related to a low energy dissipation index and thus reduced ductility of the structural element. This can be explained by the narrower loops in Figure 1.10 (a) and (b), compared to those illustrated in Figure 1.9, explaining the lower ability to dissipate energy under cyclic loading.

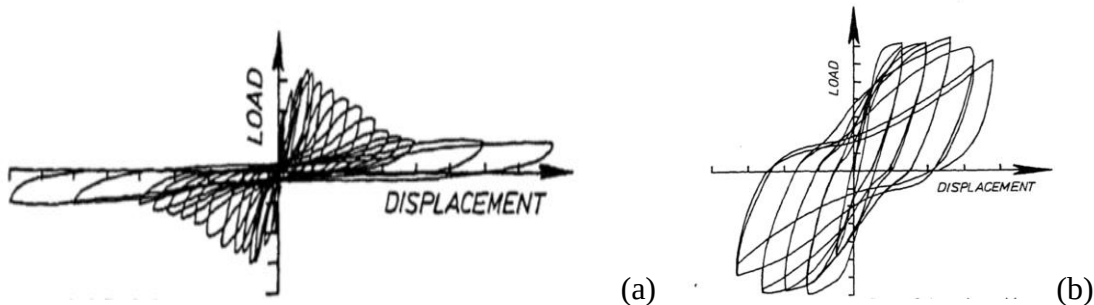


Figure 1.10 Brittle failure modes: (a) shear failure of an RC structural wall; and (b) beam-column joint due to bond deterioration (Park, 1988)

Research has intensified on developing novel materials to control shear and bond failure modes in shear walls, leading toward more ductile structural elements controlled by flexural yielding with high recovery capacity. For shear walls, future innovative designs are focused on sustaining seismic

activities with limited post-earthquake repairing. In addition, existing buildings that might experience damage during an earthquake could be repaired or retrofitted to become more resilient for future seismic events.

1.2.2 Shape Memory Alloys

Within this study, one of the novel and emerging materials being investigated are Shape Memory Alloys (SMAs), which are metallic alloys that, after apparent plastic deformation, return to their original shape through a heating process. The same materials within a specific temperature range can be deformed up to almost 6-8% strain, returning to their original shape upon unloading. These unusual effects are called the memory of thermal form (shape memory effect) and memory of elastic form (superelasticity), respectively (Lagoudas, 1989). Both effects are due to a phase change called thermoelastic-martensitic transformation presented in Figure 1.11. SMAs exhibit two distinct crystalline structures or phases: Martensite and Austenite. The phase that takes place in low temperatures is called Martensite and the mother phase that gives rise to it, at higher temperatures, is called Austenite. At this phase, a phenomenon called pseudo-elasticity occurs; it refers to a transformation from Austenite to Martensite by increasing external stress without modifying the temperature. Thereafter, the SMA is capable of re-transforming back to the Austenite state by unloading the stress (Abdulridha and Palermo, 2013).

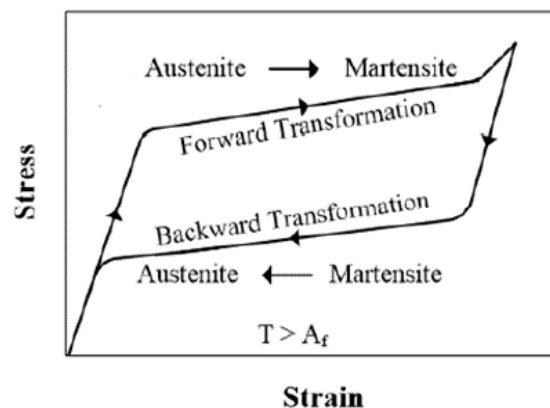


Figure 1.11 Austenite Stress-Strain Transformation (Abdulridha and Palermo, 2013)

Given the novel response of SMAs to changes in temperature and strain, they have been classified as "smart materials". The potential applications of these two main behaviors are widespread and can be

used to generate force or movement (through the shape memory effect) or to store energy (super-elasticity). If implemented in structural and civil applications, the following properties are of interest: forward transformation stress σ_L , reverse transformation σ_{UL} stress, initial elastic modulus E_i , residual strain ε_R , and the equivalent viscous damping ξ_{eq} . The forward and reverse transformation stresses correspond to 3% strain in the SMA during loading and unloading, respectively, as illustrated in Figure 1.12.

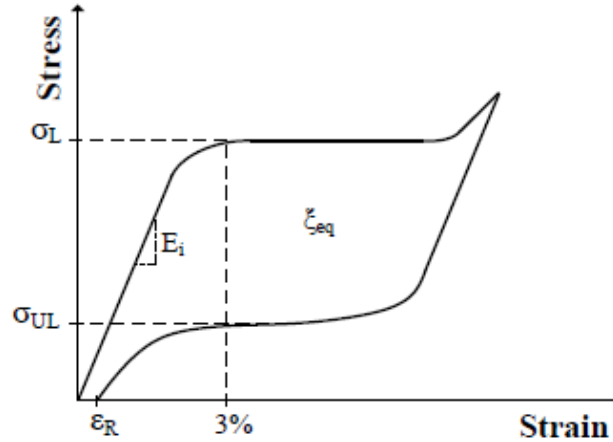


Figure 1.12 Salient SMA Properties for Structural Applications (McCormick, 2007)

Forward transformation (σ_L) is the stress level at which the martensitic phase transformation initiates. At this stage, the SMA experiences a significant stiffness change. At the reverse transformation stress (σ_{UL}), the SMA returns to the austenitic phase, defining the size of the hysteresis loop. The initial elastic modulus (E_i) provides an indication of the expected elastic behavior of the structure during a seismic event. The initial stiffness affects the magnitude of the structure's acceleration when subjected to seismic loading. The residual strain (ε_R), referred to as plastic deformation, provides a measure of the re-centering capabilities of super-elastic SMAs. The viscous damping (ξ_{eq}) is the measure of damping provided by the hysteretic behaviour of SMA and is used to compare the energy dissipation with other energy dissipating systems. The complete cycle of the stress-strain relationship includes a hysteretic area that represents dissipated energy, which results due to changes at the crystalline level that manifests in the case of the super-elastic effect (Klemas, 2002). The metal alloy of Nickel and Titanium, Nickel-Titanium (Ni-Ti), is the most common type of SMA. This alloy, also known as Nitinol (Nickel-Titanium Naval Ordnance Laboratory), was discovered in the early 1960s at the Naval Ordnance Laboratory (Buehler, Gilfrich and Wiley, 1963) and patented in 1965. It provides a recovery strain limit of 6-8%. Beyond this threshold, residual strains begin to accumulate

but are significantly smaller than those developed in traditional deformed reinforcing steel (Zaidi, Palermo and Dragmeriscu, 2016). These mechanical properties, along with its high dissipation energy, make it attractive for structural applications (Tazarv and Saiid Saiidi, 2015). The ability of NiTi to experience a reversible hysteretic shape under cyclic loading provides a structure with ductility and energy dissipation, in addition to the recovery capacity that improves the seismic resilience.

1.2.3 High-Performance Fiber Reinforced Concrete (HPFRC)

High-Performance Fiber Reinforced Concrete (HPFRC) is an innovative material composed of two main constituents: the high strength concrete matrix and fibers. The difference between the matrix of this novel material and conventional concrete relies on the presence of only fine aggregates (nominal diameter less than 1mm), high content of cementitious material (cement, inert fillers and mineral additives), a low water-to-binder ratio that fluctuates between 0.16 and 0.2 and chemical additives known as superplasticizers. The absence of coarse aggregate eliminates the stress concentration typically observed in conventional concrete and creates a very dense matrix with low capillarity increasing its durability. According to Naaman's Fiber Reinforced Composites (FRC) classification (2003), HPFRC corresponds to the highest level with a tensile behavior characterized by a strain-hardening phase with the formation of multiple cracks prior to crack localization (Figure 1.13). In this stage, the ultimate values of energy absorption are reached. Fibers are added in general to bridge the tensile cracks, increasing the load-carrying capacity and thus the flexural strength with the elimination of the brittle failure mode and damage usually observed in the case of conventional concrete.



Figure 1.14 Different types of steel and synthetic fibers added to the concrete matrix (Buttignol, Sousa and Bittencourt, 2017)

The specific bond between the fibers and the cementitious matrix for both ECC and UHPSFRC results in their own distinctive properties and mechanical behavior under compression and tension loading. UHPSFRC presents enhanced behavior under compression loading compared to ECC, specifically the compressive stress capacity. The optimized packing procedure followed to develop a dense UHPSFRC matrix leads to an extremely high compressive strength, often more than 120 MPa in comparison to ECC ranging between 60 and 80 MPa, as depicted in Figure 1.15 (a). The same applies to the tensile strength of greater than 10 MPa for UHPSFRC and 3-6 MPa for ECC, as illustrated in Figure 1.15 (b).

However, ECC is mainly characterized by its behavior under direct tension presenting a large strain-hardening plateau reaching 8% accompanied by the development of multiple cracks (Li, Wang, and Wu, 2001). This is known as ultra, pseudo-strain-hardening tensile behavior of ultra-ductility. As depicted in Figure 1.15 (b), ECC known as ultra-ductile is capable of sustaining much higher straining during its strain-hardening phase prior to crack localization in comparison to UHPSFRC (Van Zijl *et al.*, 2012).

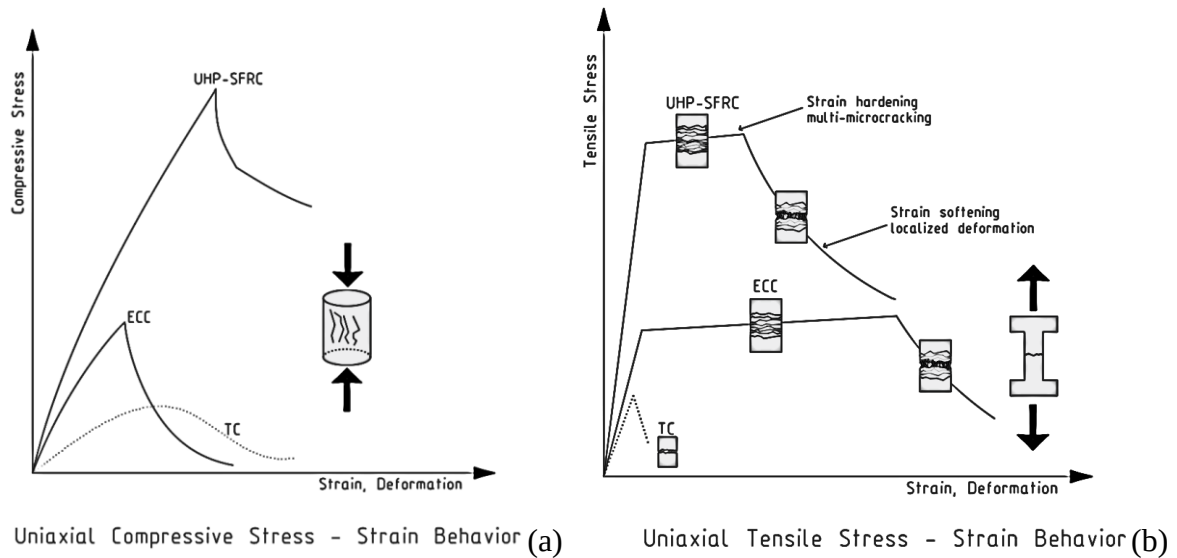


Figure 1.15 Mechanical Properties of UHP-SFRC and ECC: (a) Uniaxial compression; and (b) direct tension

1.3 Scope and Objective of Thesis

This thesis intends to expand the existing repairing alternatives and experimentally assess the performance under reverse cyclic loading of previously tested shear walls, using High-Performance Fiber Reinforced Concrete (HPFRC) and Superelastic-Shape Memory Alloy (SE-SMA) bars in the plastic hinge region. The study incorporates experimental testing and complementary nonlinear modelling.

To achieve the objectives, the following main tasks were completed:

- Visual inspection, extensive review, and analysis of the damage reported that defined the extent of deterioration of the previously tested shear walls.
- Preliminary nonlinear numerical modelling using the FEM platform VecTor2, to evaluate the different repairing techniques and materials that led to the most viable repairing methodology.
- Implementation of identical repairing procedures on the two previously tested shear walls based on the results of the numerical modelling: a reinforced wall with SE-SMA bars in the plastic hinge zone and a companion wall entirely reinforced with steel reinforcement that served as a benchmark structure.

- Numerical assessment of the combined interaction of SE-SMA bars and HPFRC as repairing materials in the plastic hinge region.
- Performance comparison of the experimental results of the shear walls in their pre and post repaired stages when subjected to the same loading protocol.
- Updated nonlinear numerical modelling with VecTor2 utilizing the known materials properties of the experimental shear walls, along with the comparisons between the experimental and numerical results.

1.4 Novelty of the research

The research presented in this thesis investigates experimentally the behavior of HPFRC's as part of a repairing method in the plastic hinge zone of a hybrid SMA-Steel reinforced concrete shear wall previously tested and damaged. It aims to highlight the benefits of implementing HPFRC and SMA materials to enhance the post-repaired behavior under simulated seismic loading, by reducing the permanent deformation and damage typically experienced by shear walls. Furthermore, the complementary nonlinear numerical modelling will verify the ability of the Finite Element Modelling (FEM) tool, VecTor2, to predict the behavior of shear walls utilizing constitutive models simulating the hysteretic behavior of the SMA reinforcing bars, and an alternative procedure to simulate the tensile properties of the HPFRC through the implementation of a user-defined constitutive model.

1.5 Thesis Layout

The thesis comprises eight chapters. Chapter 1 introduces the type of structure under study and a brief description of the novel materials to be used: SMA and HPFRC. Chapter 2 is formed by a literature review of the different hybrid SMA-HPFRC structures and their behavior, in addition to examples of repairing procedures. Chapter 3 provides a description of the damage experienced by the original shear walls and a preliminary analysis to define the extent of damage and the repairing process. Chapter 4 exposes the experimental program followed. Chapter 5 and Chapter 6 presents the experimental results and their respective analysis, correspondingly. Chapter 7 explains the detailed numerical analysis and provides a comparison to the experimental results. Lastly, Chapter 8 presents conclusions and recommendations from the research program.

Chapter 2 – Literature Review

This chapter contains information relative to the novel materials employed in this study: HPFRC and SMA, along with repairing/retrofitting methods that are of interest to this research. The first section of this chapter provides a review of the implementation of SMA in structural engineering. The second section includes repairing and retrofitting procedures in critical sections of RC structures employing smart materials. The third section involves a detailed description of the HPFRC. The chapter concludes with applications of hybrid SMA-HPFRC structures and their respective performance.

Chapter 3 – Preliminary Analysis

The first section of this chapter introduces the two previously tested shear walls under study, its detailing, and an extensive visual inspection of the damage sustained. In the second part, the chapter focuses on preliminary numerical modelling of the shear walls to define the extent of the repairing procedure, and the viability of incorporating UHPSFRC and ECC as the primary repairing material.

Chapter 4 - Experimental Program

This chapter focuses on the execution of the experimental program. The first section involves a description of the different types of tests conducted to obtain the required material properties for the concrete and reinforcement. Thereafter, the second section provides results of the trial mixes based on previously developed mixtures of HPFRC to improve the overall behavior. Based on the visual inspection and the preliminary numerical analysis, a repairing procedure for both shear walls is presented in the third section. Following a chronological order, this section includes the designing of the bracing system, concrete removal, replacement and addition of new reinforcing bars, construction of the formwork, and batching and casting of the chosen in-house HPFRC mixture. The last section explains the assembly of the test setup, the logic behind the instrumentation and the loading protocol.

Chapter 5 – Experimental Results

Chapter 5 contains the results of the material characterization tests performed and utilized as the primary repairing materials. Thereafter, the experimental results for each wall are presented. The damage sustained at each drift is discussed along with the deterioration levels for both the concrete and reinforcement. The load-displacement responses, mid-height displacements, and the reinforcement strains are presented.

Chapter 6 – Discussion of Results

The structural performance of the repaired walls is discussed in this section. The stiffness and the lateral load capacities are analyzed along with the vertical displacements and global rotations. The shear distortions generated in the plastic hinge region are explained, and the effect of the addition of the starter bars is discussed. Furthermore, a detailed comparison of the overall performance of the repaired and the original walls is established, emphasizing key parameters such as energy dissipation, recovery capacity, failure modes and the use of ECC.

Chapter 7 – Numerical Study

In this chapter, the numerical models of both shear walls developed in the preliminary analysis are updated with the material properties established from the experimental program. Subsequently, the FE models were subjected to the repairing method, while carrying forward the residual state of the original walls, followed by the application of the loading protocol as implemented in the experimental test. Finally, a comparison is provided between the experimental and the numerical results to analyze the viability of the numerical method to predict the behavior of repaired reinforced concrete structures.

Chapter 8 – Conclusions and Recommendations

The final chapter summarizes the key conclusions from this study, including the experimental program and the complementary numerical modelling. Additionally, it offers recommendations to be adopted in future research projects.

CHAPTER 2. LITERATURE REVIEW

2.1 Introduction

This chapter contains four main sections with the first focusing on the implementation of Superelastic-Shape Memory Alloy (SE-SMA) reinforcement in new construction to highlight its advantages over conventional reinforcement in terms of seismic response and experienced damage. The second section is an extension of the state-of-the-art, comprising the application of this smart material in repairing procedures to restore the damage experienced by the structure and to improve the response to future seismic activity. The second section also provides a review of repairing and rehabilitation of existing structures that are no longer considered to be at the life-safety or service levels. The third section is a detailed description of the innovative High-Performance Fiber Reinforced Concrete, including the different classifications, composition, mechanical properties, and current applications. Lastly, examples of applications for the combined implementation of SMA and HPFRC in structures is provided.

2.2 SMA Reinforced Concrete Structures

2.2.1 Introduction

Several recent research activities have focused on the implementation of SE-SMA in critical sections as a replacement for conventional steel reinforcement. Examples of tested structures include shear walls, column-beam connections, and bridge piers, to name a few. The main goal is to show the adequacy of Nitinol reinforcement for future designs, addressing earthquake loading from the perspective of reinstating the structural elements back to their original shape in a post-earthquake scenario in addition to proper energy dissipation capacity. A variety of experimental testing programs conducted in laboratories simulated the reverse cyclic loading generally experienced by the structural elements in seismic regions through shake table testing, pseudo-dynamic testing, and quasi-static load testing. Moreover, numerical analyses were conducted to simulate the experimental results and assess the behavior of SMA reinforced concrete structures for future implementation in design software. The literature provides evidence of the ability of

finite element programs to predict the behavior of SMA-reinforced concrete structures, taking into consideration the material's super-elasticity. The main characteristics of interest are the failure mode and hysteresis response of the new and repaired reinforced concrete structures, the yield and ultimate lateral strength capacities with their corresponding displacements, the energy dissipated, and the recovery of displacements.

2.2.2 Behavior of SMA Reinforced Concrete Structures

Several researches focused on partially replacing a portion of the conventional reinforcing steel with SMA bars in the plastic hinge regions. Saiidi and Wang (2006) were among the first who incorporated SMA bars as a replacement of conventional steel reinforcement in the plastic hinge region of reinforced concrete spiral columns representing bridge piers, as shown in Figure 2.1. The Ni-Ti rods were then joined to steel reinforcing bars using mechanical couplers as depicted in Sections A, B and C of Figure 2.1. The study focused on two spiral RC columns subjected to shake table testing: the first column was used as a control specimen, and the second column labelled as SMAC-1 comprises the Ni-Ti SMA bars. It was shown that SMAC-1 experienced a lower top lateral displacement and was able to recover the displacements, with a 0.2% residual drift at the end of testing.

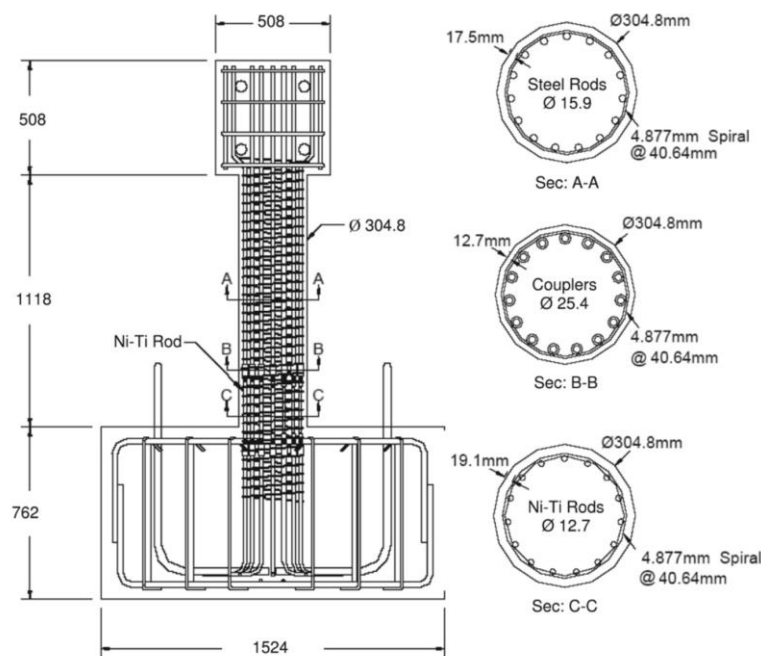


Figure 2.1 SMA-RC column reinforcement detailing (Saiidi and Wang, 2006)

This experimental testing was numerically modelled, as shown in Figure 2.2 to simulate and compare the experimental and numerical results (Alam, Youssef and Nehdi, 2008) using SeismoSoft (2019). No bond-link elements were considered in the FE model to represent the couplers since the experimental results suggested a full load transfer mechanism between the SMA rods and the steel reinforcement. Both the numerical and experimental SMA-RC columns experienced the same failure mode governed by concrete crushing and yielding of the SMA bars. The Nonlinear Finite Element Model (NLFEM) developed by Alam et al. (2008) (Figure 2.3 (b)) was able to predict the experimental response (Figure 2.3 (a)) of the SMA-columns tested by Saiidi and Wang (2006). The experimental and numerical maximum base shear and corresponding top displacement were similar: 77.2 kN and 66 mm and 81.5 kN and 62 mm, respectively. In addition, the calculated numerical cumulative dissipated energy was 9.4% higher than the measured from the experimental response.

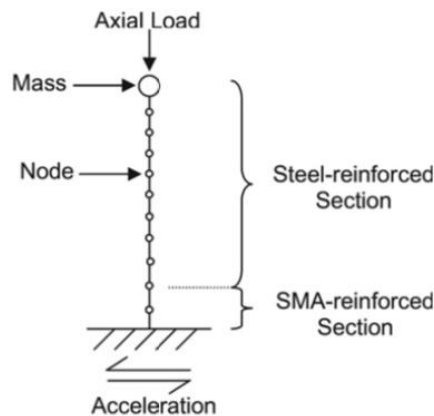


Figure 2.2 Finite element model of the SMAC-1 columns (Alam, Youssef and Nehdi, 2008)

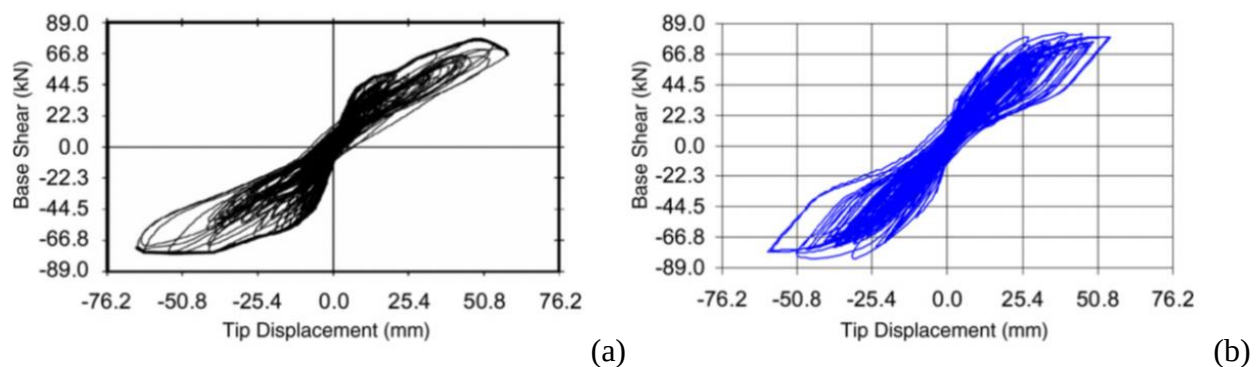


Figure 2.3 Base shear versus top displacement response: (a) experimental test (Saiidi and Wang, 2006); and (b) FE model (Alam, Youssef and Nehdi, 2008)

Youssef, Alam and Nehdi (2008) demonstrated significant improvement in the behavior of a beam adjacent to a beam-column joint under seismic loading when reinforced with hybrid steel-SMA in place of traditional steel in terms of residual displacements, crack development and relocation of the plastic hinge. An eight-storey prototype frame building (Canadian Standards Association, 2004) was designed from which two, $\frac{3}{4}$ -scaled exterior beam-column joints were constructed, shown in Figure 2.4 (a). The tip of the beam was subjected to a reverse quasi-static load while applying a constant axial load on the column. The two specimens were identical in geometry and shear reinforcement distribution. However, the control specimen was exclusively reinforced with steel reinforcement (denoted JBC-1). The second specimen (JBC-2) incorporated SE-SMA bars in the plastic hinge region of the beam, extending 360 mm from the face of the column (Figure 2.4 (b)). The SMA bars were connected to regular steel reinforcement on both ends using single barrel screw-lock couplers.

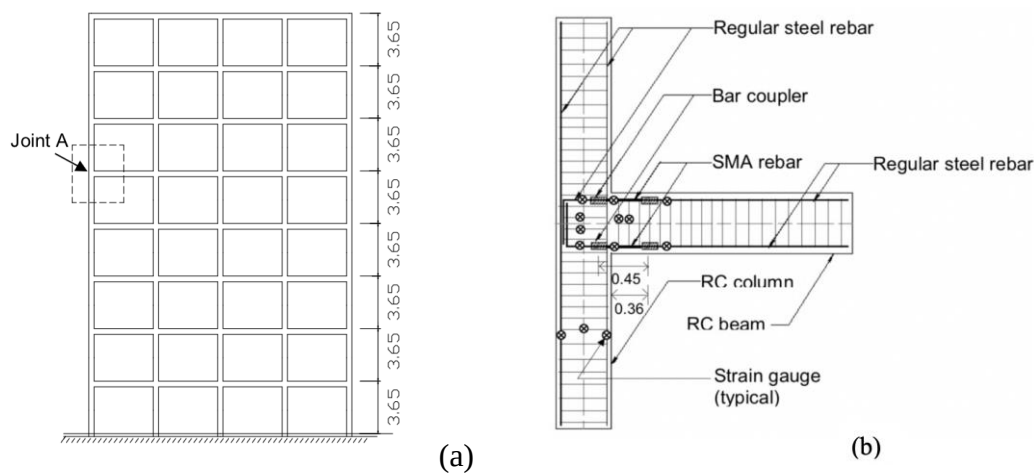


Figure 2.4 Building layout for testing: (a) location of the joint; and (b) reinforcement detailing for the SMA-BCJ (JBC-2). (Youssef, Alam and Nehdi, 2008)

The hysteretic behavior of the steel and SMA-reinforced specimens sustained similar load capacities for large storey drift values, as shown in Figure 2.5 (a) and (b), respectively. However, JBC-1 experienced higher stiffness in the post-first flexural crack due to the higher modulus of elasticity of the steel reinforcing bars relative to the SMA. The control specimen developed larger hysteretic loops compared to the SMA-specimen, resulting in higher cumulative dissipated energy. For a 7.9% storey drift, JBC-2 dissipated 16.7 kNm of cumulative energy compared to 26.5 kNm

for JBC-1, corresponding to a reduction of 26%. This is apparent in the high level of concrete damage and cracking experienced by the steel-reinforced specimen (Figure 2.5 (c)) relative to the SMA-reinforced specimen (Figure 2.5 (d)). An interesting observation was the shifting of the plastic hinge region away from the column in the SMA specimen, approximately half the depth of the beam.

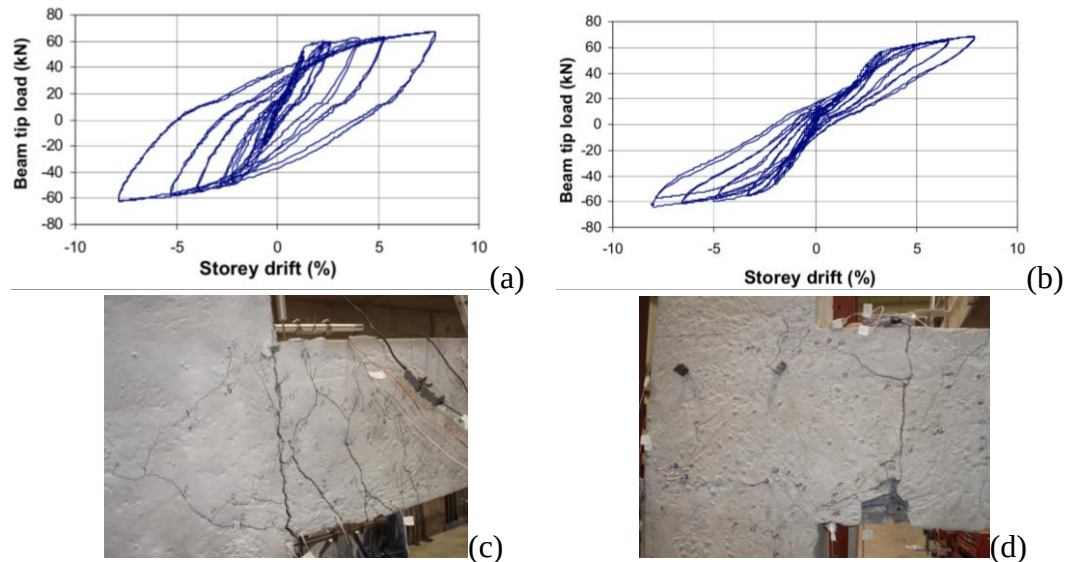


Figure 2.5 Responses of beam: (a) JBC-1 and (b) JBC-2; crack pattern at 72 mm displacement of: (c) JBC-1 and (d) JBC-2 (Youssef, Alam and Nehdi, 2008)

Alam, Youssef and Nehdi (2008) validated the experimental results of JBC2 numerically using the FE model depicted in Figure 2.6 (a) using SeismoSoft (2019). The primary analysis was conducted without considering the bond-slip behaviour of the SMA bar. The analysis resulted in a lower initial stiffness with a 17% reduction and greater energy dissipation than that recorded during the experimental testing. For the secondary analysis, the bond-slip model was implemented as a rotational spring at the joint. The numerical response (Figure 2.6 (b)) from this model was in proper agreement with the experimental response (Figure 2.5 (b)) simulating similar initial stiffness with an 11% increase of beam tip load at a tip displacement of 72 mm. The numerical model under-estimated the energy dissipation by 16.2%.

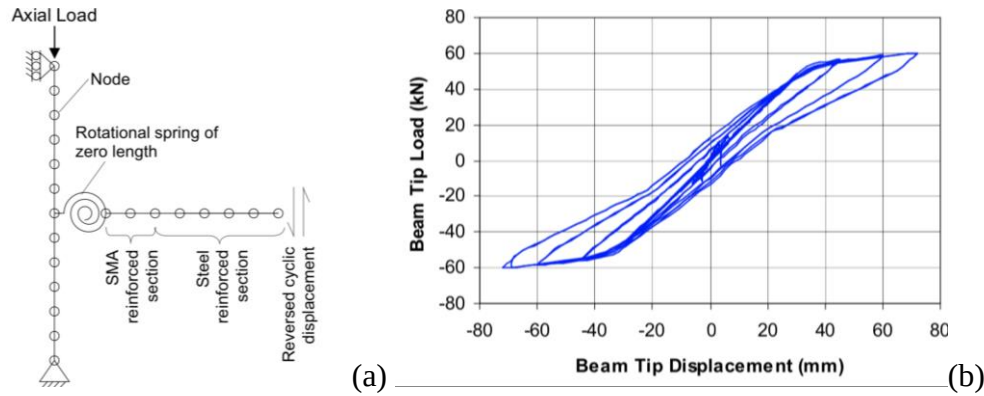


Figure 2.6 Numerical model for JBC-2: (a) FE model; and (b) reverse-cyclic response (Alam, Youssef and Nehdi, 2008)

Abdulridha and Palermo (2013) assessed the behavior of a hybrid SMA-steel reinforced concrete shear wall under reverse-cyclic lateral loading. Two different shear walls were constructed as part of the experimental work. The only difference between these two walls was the longitudinal reinforcement utilized in the plastic hinge region within the end boundary zones: deformed steel bars (W1-SR) and superelastic SMA bars (W2-NR). The walls were designed according to the seismic detailing provisions for ductile walls prescribed by the Canadian Standards Association Standard A23.3-04 Design of Concrete Structures. Figure 2.7 shows the dimensional and reinforcement details for the W2-NR.

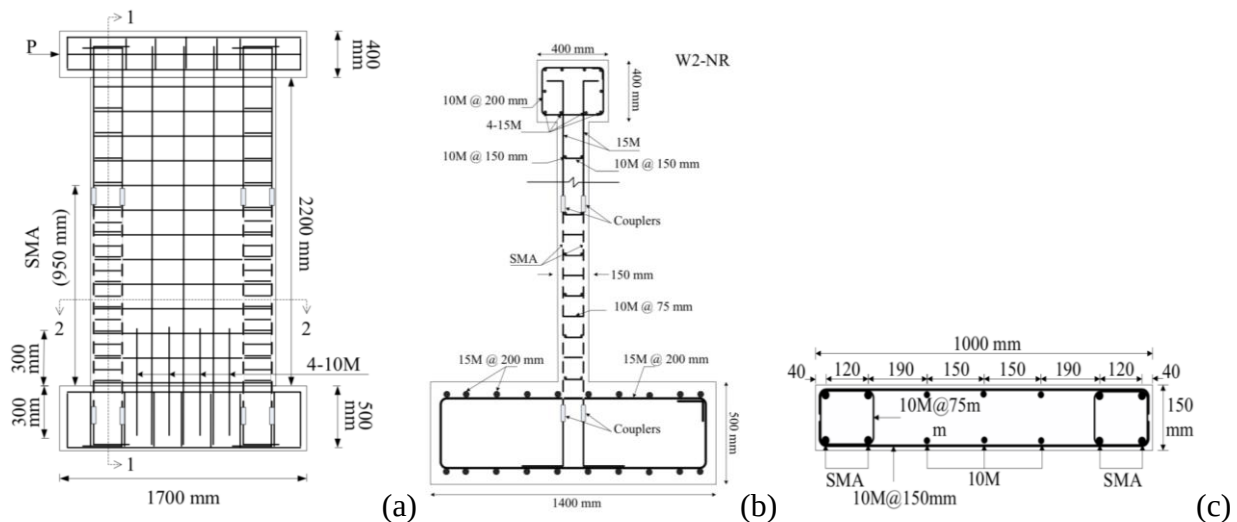


Figure 2.7 Reinforcement details and dimensions of W2-NR: (a) elevation view; (b) section 1-1; and (c) Section 2-2 (Abdulridha and Palermo, 2017)

Both walls sustained similar yield loads of 110 kN. However, the SMA wall experienced larger displacement at yield (26.4 mm) compared to 9.4 mm for the steel wall due to the lower Young Modulus of Ni-Ti bars. As depicted in Figure 2.8 (a), the SMA wall exhibited a lower stiffness at the yielding, approximately 35% of the control wall's stiffness, which translates to a higher fundamental period for the structure and thus lower seismic forces. Prior to failure, the hybrid wall (W2-NR) sustained a drift capacity of 3%; meanwhile, the steel wall failed at 3.2% drift. At the end of testing, the SMA wall experienced a re-centering capacity of 92%, three times greater than the steel wall as depicted in Figure 2.8 (b). For a drift of 3%, the NiTi-SMA bars in the W2-NR wall were able to re-center and recover up to 85% of the lateral displacement, even though the steel reinforcing bars present in the web region had yielded. For larger drifts, after rupturing of the steel reinforcement in the web region, the SMA wall experienced an increase of the displacement recovery capacity from 85% to 92%. In addition, the SMA wall dissipated less energy due to the reverse transformation that occurs in the SMA reinforcement during unloading (self-centering), while sustaining similar peak lateral load and displacement capacities.

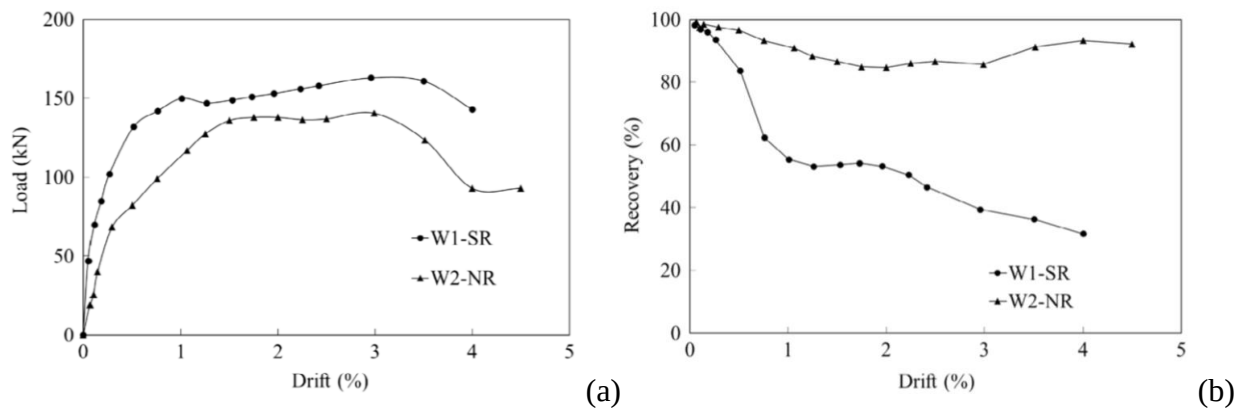


Figure 2.8 Responses of Walls W1-SR and W2-NR: (a) lateral load-drift envelope responses; and (b) displacement recovery capacity-lateral drift responses

Shear cracks were predominant in Wall W1-SR, while flexural cracking was more prevalent in Wall W2-NR. Moreover, both walls experienced widespread damage; the concrete located in the plastic hinge region was severely deteriorated, with more cracking surfacing in Wall W1-SR. The reduced cracking in W2-NR was attributed to the smooth surface of the SMA bars, which reduced bonding to the surrounding concrete, resulting in fewer but broader cracks. The SMA wall was able to recover up to 88% of the average crack opening compared to 24% for the steel wall. Figure

2.9 (a) and (b) show the conditions of the walls after testing. Both walls developed a wide crack spreading horizontally along the wall at the termination of the starter bars where the behavior of the walls was controlled. The walls failed due to the rupture of longitudinal reinforcing bars in the plastic hinge region, near the base of the wall. The steel reinforcement located in the boundary regions of W1-SR ruptured, whereas for W2-NR, the steel reinforcement in the web region ruptured. A subsequent research study repaired the damaged walls, where it was observed after concrete removal in the plastic hinge area that the SMA bars in the boundary zones experienced local buckling (Zaidi, Palermo and Dragmeriscu, 2016). It was reported that one SMA bar ruptured at the location of the mechanical couplers due to the sharp end screws. Figure 2.9 (c) and (d) illustrate the damage in the reinforcement of the SMA wall.



Figure 2.9 Conditions of the walls at failure: (a) W1-SR; (b) W2-NR (Abdulridha and Palermo, 2013); (c) rupture and buckling of steel reinforcement in the web of W2-NR; and (d) buckling of SMA bars in the boundary zone for W2-NR (Zaidi, Palermo and Dragmeriscu, 2016)

Abdulridha and Palermo (2017) modelled W2-NR in the finite element platform VecTor 2, using the SMA hysteretic constitutive model developed by Abdulridha and Palermo (2013) and the default constitutive bond model for a smooth bar to represent the force interaction between the SMA bars and the surrounding concrete. The numerical load-lateral displacement response shown in Figure 2.10 (a) was in agreement with the response of the wall observed in the experimental program (Figure 2.10 (b)), with an overestimation of the yield load by 3.8% and the peak load by 12.8%. Moreover, the numerical model was able to capture the rupture of the steel reinforcement in the web region but not the buckling of the SMA bars. Some divergences are evident, such as the top lateral displacements; however, the model was able to capture the residual displacements.

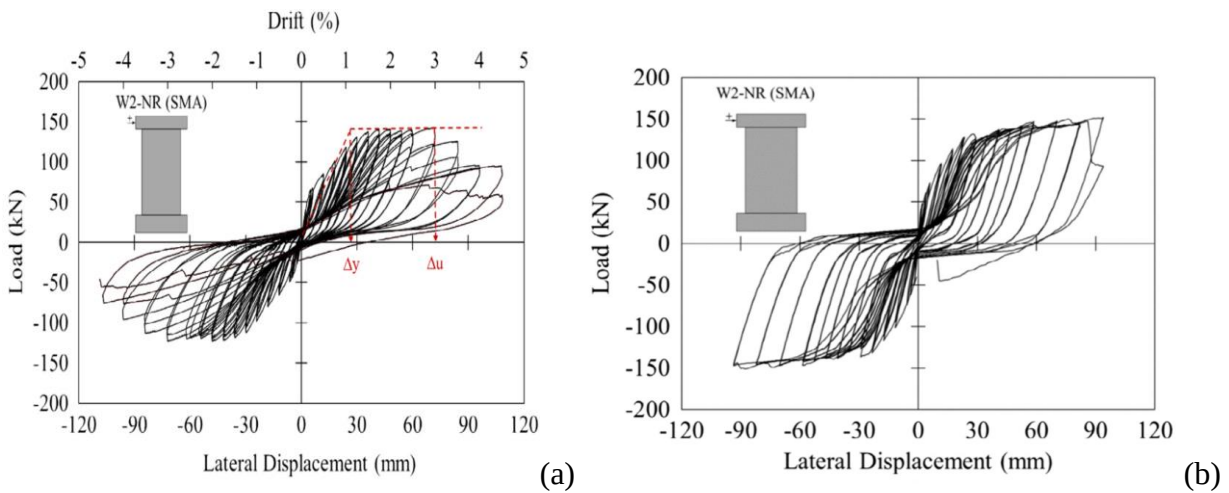


Figure 2.10 Lateral load-displacement response for W2-NR: (a) experimental; and (b) numerical (Abdulridha and Palermo, 2017)

The research conducted by Abdulridha and Palermo (2013) served as the benchmark study for Morcos and Palermo (2019), where the behavior of a hybrid SMA-steel reinforced concrete shear wall under reversed cyclic lateral loading was assessed. Two slender walls were constructed as part of the experimental work, with the same approach previously discussed in terms of design, dimensions, reinforcement details, and mechanical couplers. The main differences included: (1) Reduction of the length of the SMA bars within the plastic hinge region from 950 mm to 900 mm; (2) No starter bars between the foundation and the wall as shown in Figure 2.7; and (3) Replacement of the modified single-barrel screw lock coupler (Figure 2.11 (a)) with mechanical couplers joining headed deformed steel reinforcement and Ni-Ti bars (Figure 2.11 (b)). The control conventional steel wall was denoted SWS-R and the hybrid SMA-steel wall SWN.



Figure 2.11 Mechanical couplers: (a) Original study by Abdulridha and Palermo (2013); and (b) study by Morcos and Palermo (2019).

Figure 2.12 depicts the experimental lateral load-displacement responses for Wall SWS-R and SWN. Both structures sustained the same yield load of approximately 90 kN, with Wall SWN experiencing larger lateral displacement at yield (24 mm) in comparison to 8.7 mm for Wall SWS-R. Similarly, the peak load was approximately 118 kN for both walls. However, the corresponding displacement for SWN was 91 mm, three times more than SWS-R. Wall SWN failed at a drift of 4.29% compared to 2.5% for the control wall. The displacement ductility capacity of SWN (2.3) was lower than SWS-R (6.9) due to the lower initial secant stiffness and larger yield displacement. The average crack width recovery was 82% for SWN and 36% for SWS-R. The cracks that were wider than 8.5 mm, recovered by 40%. For a drift of 2.5%, the displacement recovery capacity of the SMA wall was 66% compared to 26% for the steel wall.

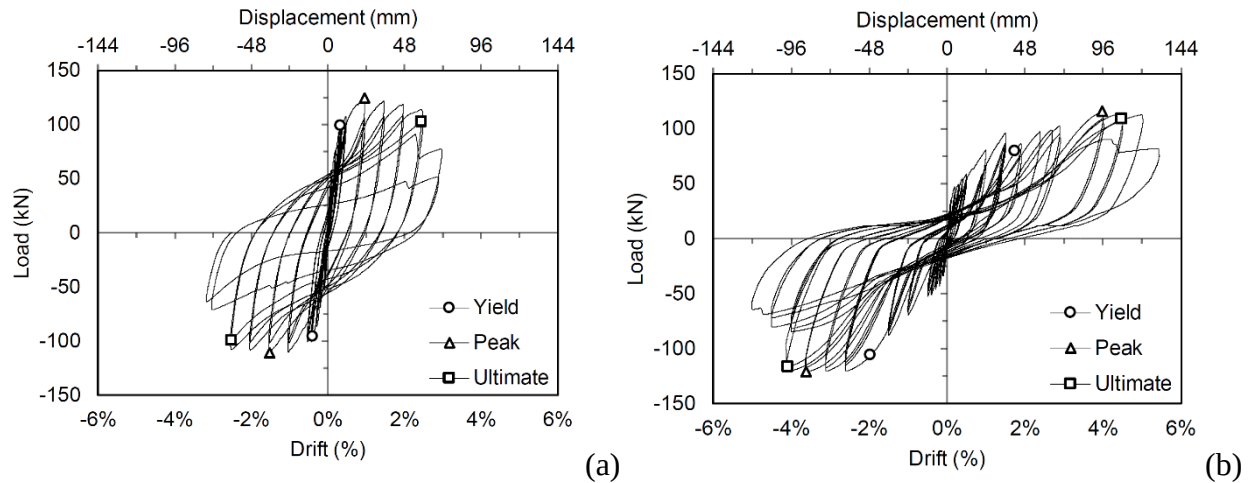


Figure 2.12 Experimental load-displacement response for Wall: (a) SWS-R; and (b) SWN (Morcos and Palermo, 2019)

Morcos and Palermo (2018) modelled the walls using VecTor2 prior to conducting the experimental testing. The hysteretic responses of the FE models for Walls SWS-R and SWN are illustrated in Figure 2.13. For Wall SWS-R, the numerical response (Figure 2.13 (a)) was in agreement with the experimental data (Figure 2.12 (a)), capturing the wide hysteretic loops

attributed to the yielding of the steel reinforcement in the plastic hinge region. However, significant discrepancies were noticed in the performance points. The yield load was overestimated by 28%, with an 11% reduction in the yield displacement when compared to the experimental values. At peak, similar behavior was observed, the FE model overestimated the lateral load by 13% and underestimated the displacement by 44%. At 2.5% drift, the numerical and experimental results for Wall SWS-R provided similar displacement recoveries of approximately 26%. The numerical response for Wall RSWN was similar to the experimental response in terms of pinching with the hysteretic loops dilating with increasing lateral displacements (Figure 2.12 (b) and Figure 2.13 (b)). This demonstrates that the recovery capacity of the SMA bars was adequately captured in the FE model. The model experienced stiffer behavior than the actual wall with greater secant stiffness at the yield point. The yield load was overestimated by 42%, while the yield displacement was underestimated by 62%. This discrepancy was reduced during higher displacements. The FE model overestimated the peak load by 24% and understated the peak displacement by 6%. Both experimental and numerically-modelled walls experienced similar displacement recovery capacity of approximately 60% at a drift of 2.5%. In terms of failure mode, the FE model for Wall SWN was able to predict the yielding of the steel longitudinal bars in the web without any damage to the SMA bars in the boundaries. However, the model did not capture the rocking experienced by the wall nor the major horizontal crack at the bottom of the wall. For Wall SWS-R, the FE model captured the rupturing of the longitudinal reinforcing bars in the boundary zone.

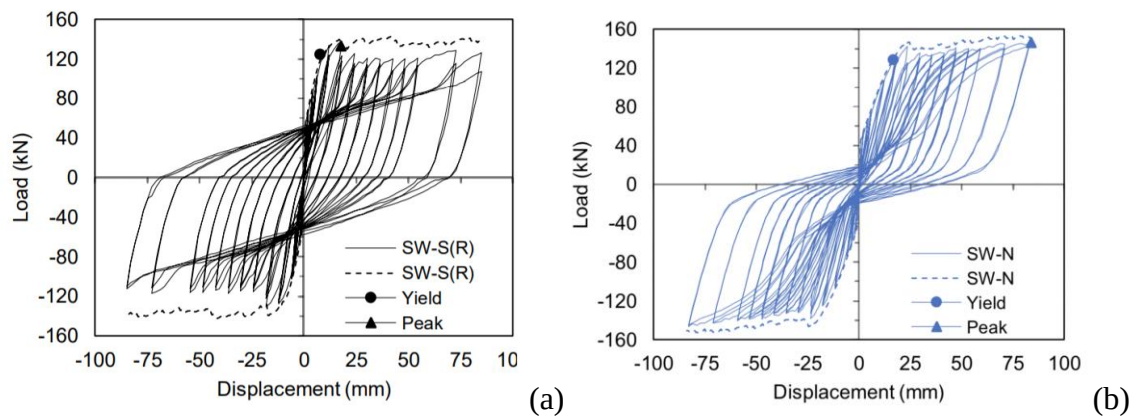


Figure 2.13 Numerical load-displacement response for Wall: (a) SWS-R(Morcos and Palermo, 2019)(Morcos and Palermo, 2019); and (b) SWN (Morcos and Palermo, 2018)

2.3 Behavior of Repaired Reinforced Concrete Shear Walls

The repairing and rehabilitation of reinforced concrete shear walls should not only restore the structural element's strength, stiffness and energy dissipation but also should meet recent design standards. There have been several studies, experimental and numerical, to investigate repairing methodologies, a number of which are discussed in the subsequent sections.

Antoniades, Salonikios and Kappos (2005) built five concrete shear walls that were subjected to a quasi-static cyclic loading protocol, and that experienced a flexural failure mode dominated by heavy concrete damage and the buckling of reinforcing bars. The repairing procedure considered the removal of the damaged concrete with a depth of 250 mm into the foundation, removal of the ruptured reinforcing bars and lap-welding of new steel bar segments. The bottom section of the walls was cast with high-strength, non-shrinking mortar. Walls LSW1, LSW2, LSW4 and LSW5, were then wrapped with FRP jackets and FRP strips as depicted in

Figure 2.14. Moreover, the repairing methodology for LSW3 denoted as a control-repaired wall, excluded the FRP-reinforcement. The FRP-strengthening repairing procedure provided improved performance compared to the control-repaired wall in terms of increased stiffness and higher strength capacity. However, the strength was still lower than the original walls. Furthermore, both repairing procedures were inefficient in terms of restoring the energy dissipation.

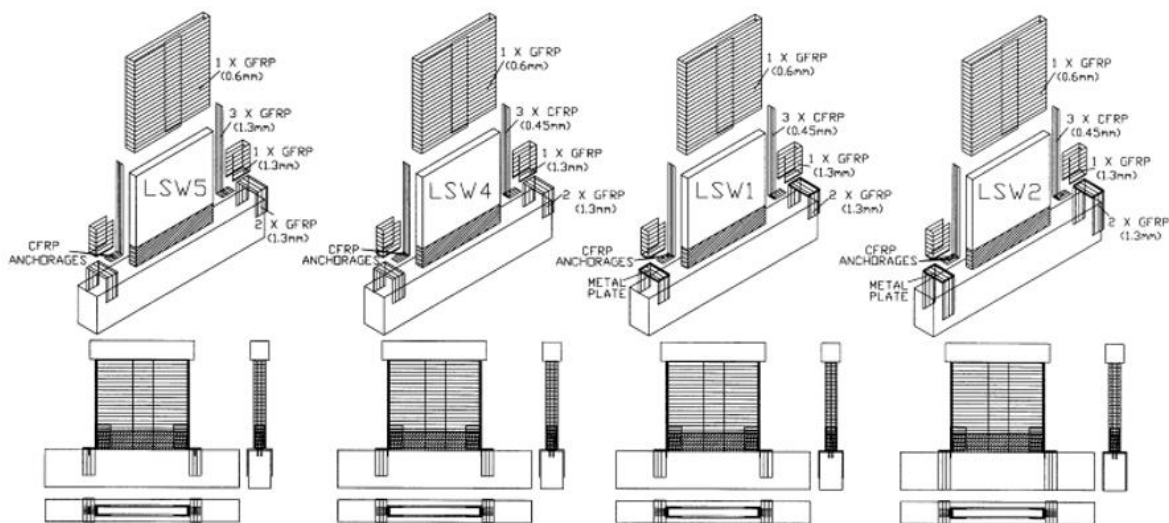


Figure 2.14 FRP strengthening after removal of damaged concrete, replacement of ruptured steel bars and casting of new mortar in the bottom region of the walls

A large-scale squat shear wall, denoted as DP1, was subjected to quasi-static reversed cyclic loading and a constant axial load of 1200 kN (Palermo and Vecchio, 2002). The damaged shear wall was then repaired by removing the damaged concrete in the web region and replaced with new concrete, as illustrated in Figure 2.15 (a) (Vecchio *et al.*, 2002). The concrete removal extended 15 mm in depth through the flange, top and bottom slabs of the wall. Lastly, the 15 mm of the web was filled with epoxy grout to ensure adequate bonding and structural continuity. No repair was considered for the flexural reinforcement. The concrete used for repairing had a compressive strength of approximately 40 MPa, nearly twice that of the original concrete used in the web region. DP1R, where R stands for repair, was then subjected to the same loading as the original wall to compare the efficiency of the repairing procedure. The comparison of the load-displacement responses for DP1 (Figure 2.15 (b)) and DP1R (Figure 2.15 (c)) demonstrates the ability of this repairing procedure to redevelop the strength, stiffness and energy dissipation of the original wall, where for both walls, the pinching in the hysteretic response indicated a shear-dominated failure mode.

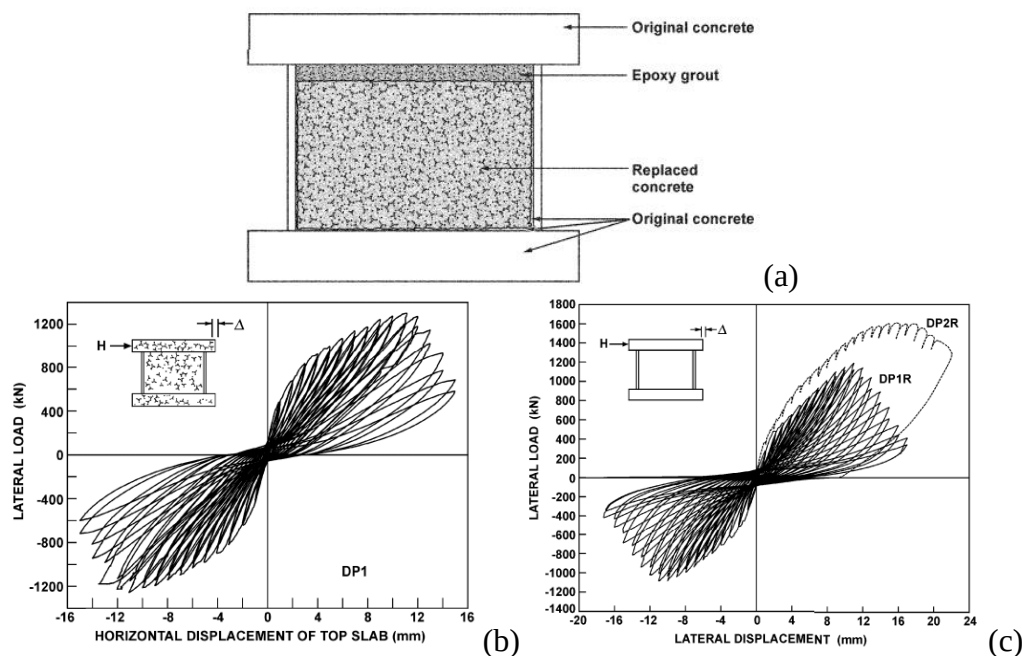


Figure 2.15 DP1 and DP1R: (a) repair components for DP1R; (b) load-displacement for DP1; and (c) load-displacement for DP1R

Nehdi, Alam and Youssef (2011) repaired the SMA-reinforced beam-column joints JBC-2 (Figure 2.4) previously tested and discussed (Youssef, Alam and Nehdi, 2008). The repairing procedure included the removal of the damaged concrete, the preparation of the exposed concrete surface,

casting of the Structural Repair Concrete (SRC), and then sealing all minor cracks with epoxy injection. The repaired beam-column joint denoted as JBC-3 was subsequently subjected to a reverse cyclic loading protocol. The compressive and split cylinder tensile strength of the traditional concrete used in JBC-2 was 53.7 MPa and 2.8 MPa, respectively. For JBC-3, the SRC concrete had a compressive strength of 61.6 MPa and a split cylinder tensile strength of 3.8 MPa. The hysteretic response of JBC-3 (Figure 2.16 (a)) was similar to JBC-2 (Figure 2.16 (b)). However, it is evident that the strength of JBC-3 was reduced after a 6.5% storey drift, while the strength of JBC-2 was increasing. The peak load of JBC-3 was 5.4% lower than JBC-2.

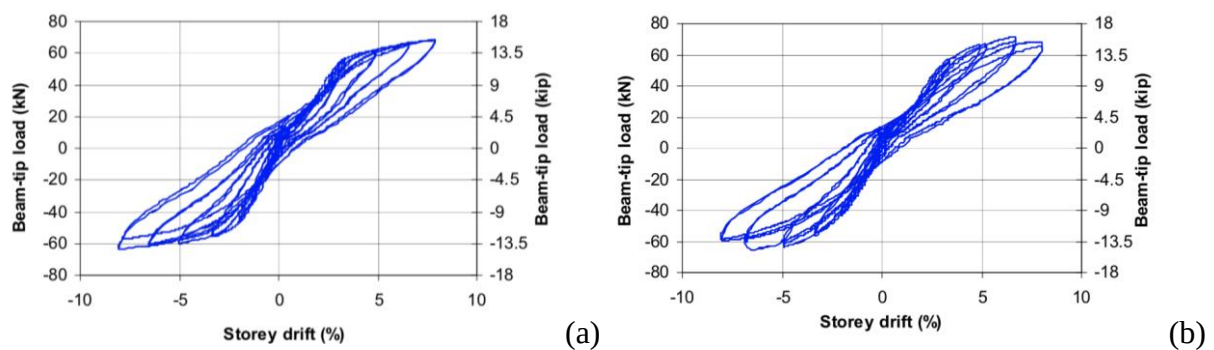


Figure 2.16 Beam tip load-storey drift responses: (a) JBC-2; and (b) JBC-3

The residual strains of the SMA bars were greater for JBC-3 due to the repetitive cyclic loading. For a storey drift of 3%, the cumulative dissipated energy for the repaired beam-column joint was lower than the original joint by 11.5%. At 7.9% storey drift, both beam-column joints dissipated the same cumulative energy; however, JBC-3 experienced greater concrete damage. This can be observed by comparing Figure 2.5 (a) representing JBC-2 and Figure 2.17(a) for JBC-3. The repaired SMA beam-column joint was able to recover the post-yielding displacement with very minimal damage. The repairing procedure did not alter the development of the flexural crack at a distance of the beam's depth from the column, as shown in Figure 2.17 (a). It is to be noted that the bonding between the repair and original concrete was adequate since the flexural crack developed in the new concrete and not at the cold joint (Figure 2.17 (b)). However, subjecting the joint to further reverse cyclic loading led to yielding.

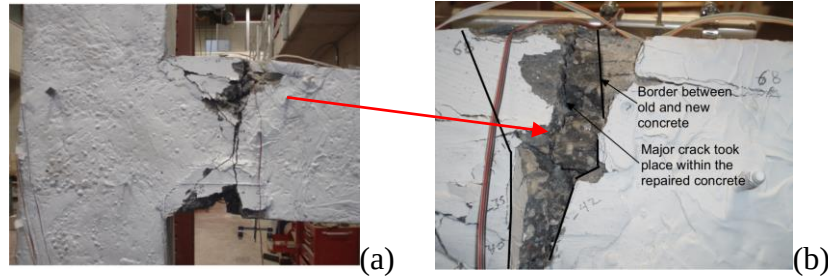


Figure 2.17 Damage in JBC-3: (a) crack pattern at the rear face; and (b) cracking in the repair concrete

Zaidi, Palermo and Dragmeriscu (2016) repaired the two previously-damaged concrete slender shear walls reported by Abdulridha and Palermo (2013) to study their behavior under reverse cyclic loading and to compare them with their original walls. The original walls of this study named W1-SR and W2-NR were renamed RW1-SR and RW2-NR, respectively, where the R stands for “repaired”. In contrast to W1-SR, no external loads were required to re-align W2-NR due to the self-centering that took place at the end of testing, which is one of the primary advantages for repairing SMA-reinforced members. The repair strategy involved: removal of the heavily damaged concrete located in the plastic hinge region, replacement of the ruptured and buckled steel reinforcing bars, shortening of the SMA bars located in the boundary zones to remove local buckling, and the addition of mechanical couplers to connect existing bars with the new reinforcement. Thus, some modifications of the original walls were considered: 150 mm-long mechanical couplers with 18 screws were used to connect the SMA bars with the steel bars to prevent bar slip. In addition, the repaired SMA-wall considered shorter SMA bars. The implementation of the additional mechanical couplers led to a reduction of the spacing between the shear reinforcement. No starter bars were considered. Self-Consolidating Concrete (SCC), with a compressive strength of 80 MPa, was used as the repair material to replace the removed concrete, Figure 2.18 demonstrates the reinforcement layout for RW2-NR.

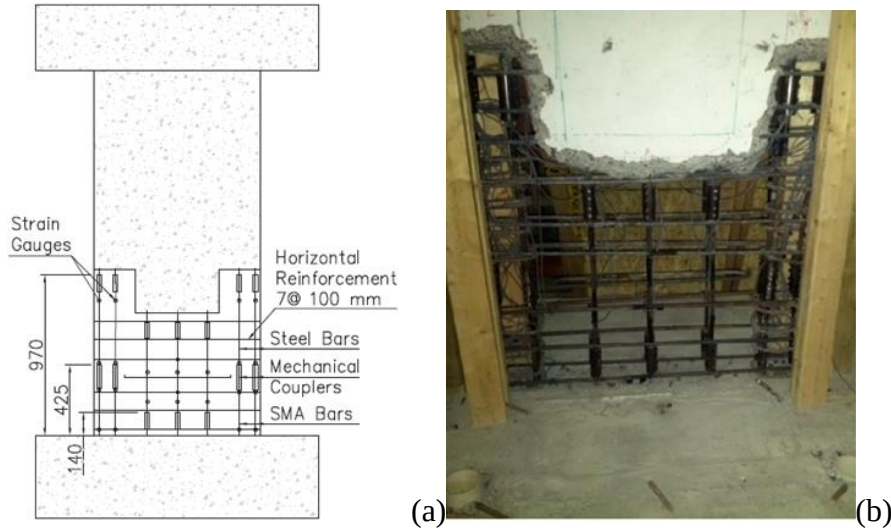


Figure 2.18 Details of RW2-NR: (a) schematic of reinforcement layout; and (b) photo before casting of SCC (Zaidi, Palermo and Dragmeriscu, 2016)

The portions of the SMA bars that were removed from the original wall were re-tested under cyclic loading, illustrating the similarity of the stress-strain response to the original state, as shown in Figure 2.19. Hence, approximately the same mechanical properties of the SMA were assumed for both original and repaired walls.

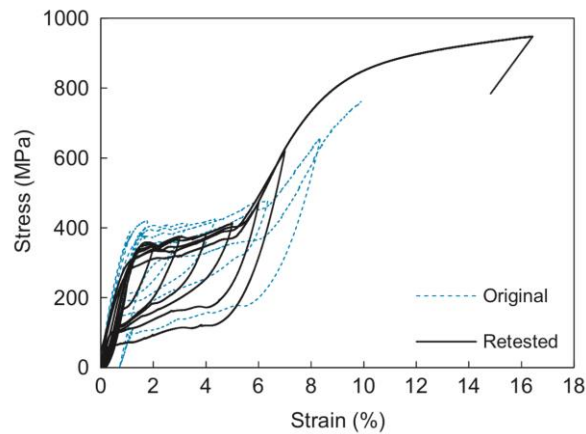


Figure 2.19 Stress-strain response of original and retested SMA bars (Zaidi, Palermo and Dragmeriscu, 2016)

The repaired walls were subjected to reverse cycles at multiples of the experimental yield displacement recorded by Abdulridha and Palermo (2013). It was proven that the process of only removing the damaged concrete was sufficient since the intact concrete above the plastic hinge region did not sustain any additional cracks nor damage. Furthermore, the horizontal crack shown in Figure 2.9 (b) shifted towards the base of the wall due to the elimination of the starter bars. The

repaired SMA walls developed concentrated cracks in the vicinity of the couplers, whereas widespread cracking was noticed in the repaired steel wall, which can be related to the smooth surface of the SMA bars that manifest in concerned concrete damage. In the post-peak phase, three longitudinal reinforcing bars in the left boundary region and all the longitudinal reinforcement present in the right boundary zone ruptured in Wall RW1-SR. For Wall RW2-NR, one longitudinal reinforcing bar in the web region accompanied by the two outermost SMA bars in each boundary region ruptured. As presented in Figure 2.20 (a), both repaired walls experienced a similar pre-yielding phase as their original state. For the same yield load of approximately 112 kN, RW2-NR developed a lower yield drift of 0.9 % compared to 1.1 % for W2-NR. However, the repaired SMA-wall developed a greater yield drift than both original and repaired steel walls of 0.39% and 0.6%, respectively. RW2-NR did not develop the post-yield plateau experienced by RW1-SR. Beyond the peak load, RW1-SR experienced a significant decrease in strength due to the rupturing of the longitudinal bars at a drift of 2.63% compared to an ultimate drift of 3.5% for the original steel wall. The reduction of the ultimate drift from 3% for W2-NR to 2.5% for RW2-NR is due to the premature rupture of the SMA bars. However, the repairing procedure was considered efficient since both repaired walls experienced a drift capacity greater than the design range of 2%. The stiffness of the repaired steel wall was reduced due to the damage retained in the original wall; meanwhile, the repaired SMA wall experienced a higher initial stiffness due to the shortened SMA bars. As illustrated in Figure 2.20 (b), RW2-NR was able to recover up to 80% of its displacement capacity for drifts below 2%. The repaired walls dissipated more energy than the original walls. This was due to the wider hysteretic cycles with higher equivalent damping for the majority of lateral drift, specifically in the post-yield range.

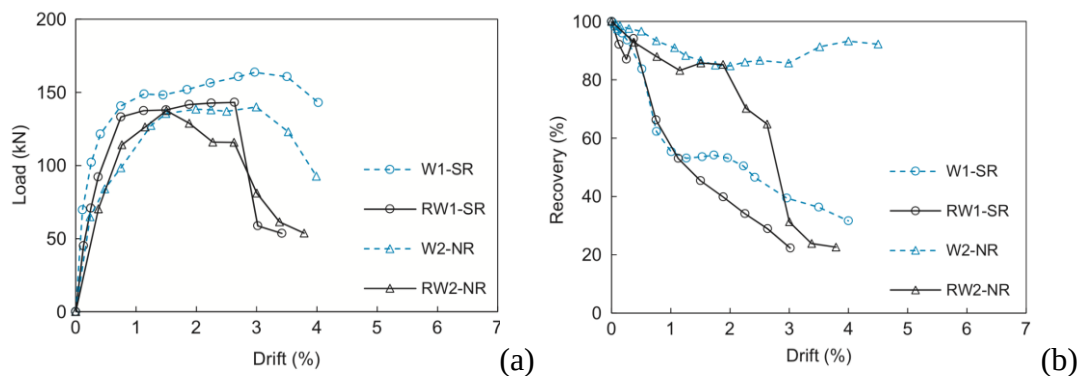


Figure 2.20 Responses of walls before and after repair: (a) lateral load-drift envelope response for the positive direction; and (b) displacement recovery-drift response (Cortés-Puentes et al., 2018)

The repaired walls experienced slightly lower lateral strength, displacement and less pinching than the original walls. In general, the responses of the walls restored the yield and ultimate lateral load capacities but sustained lower drift capacities that are acceptable for design codes. Using SMA bars results in localized damaged and high recovery capacity up to a 2% drift ratio.

2.4 Introduction to High-Performance Fiber-Reinforced Concrete (HPFRC)

2.4.1 Evolution and Classification of HPFRC

The evolution of Fiber Reinforced Concrete (FRC) composites have spanned the past half-century. As presented in Figure 2.21, new “FRC terminologies” are given for each timeline based on the different design mixes developed and characterized by material properties and mechanical strength (Naaman, 2018). In some cases, undesirable results lead to a shift or alteration in the methodology followed. An example is the Slurry Infiltrated Fiber Concrete (SIFCON), known by its high volumetric fraction of steel fibers (20%) and low tensile strength where a mat of fibers is added manually, making it a very difficult procedure (Breitenbucher, 1999). SIMCON was thereafter developed, recognized as the improved SIFCON, where the fiber fraction was reduced to 5%, where a pre-fabricated fiber mat can be easily placed (Krstulovic-Opara *et al.*, 1997). In other scenarios, the results are desirable, and further research was conducted to enhance the properties.

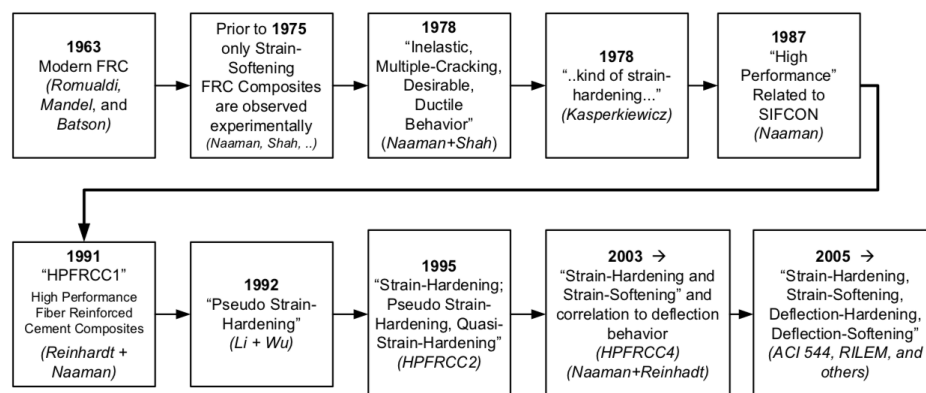


Figure 2.21 Evolution of terminology of Fiber Reinforced Concrete (Naaman, 2018)

Naaman and Reinhardt (2006) classified FRC composites into four performance categories, as depicted in Figure 2.22 (a) depending on the volumetric fraction of fibers and the behavior under

direct tension and then under flexure. Based on the stress-strain response under direct tension, FRC can exhibit a strain hardening comportment known as High-Performance Fiber-Reinforced Cementitious Composite (HPFRCC) or a strain-softening behavior. The latter is then categorized based on the load-deflection response in flexure, which can follow a softening or a hardening response. Deflection-hardening materials, so-called Ductile Fiber Reinforced Cement Composites (DFRCC), are marked by their ability to develop multiple-cracks under flexure, yet not in tension. Deflection hardening composites are used to limit crack propagation from the structural response mechanism, while deflection softening is more common to prevent cracking from shrinkage. Even though all HPFRCC exhibits a deflection-hardening response, the same is not true for DFRCC materials. Therefore, HPFRCC is prominent with strain-hardening under direct tension and the ability to develop multiple cracks between the occurrence of the first crack and the peak load prior to crack localization. Examples of HPFRCC are Engineered Cementitious Composites (ECC) and Slurry Infiltrated Fibrous Concrete (SIFCON). As presented in Figure 2.22 (b), Ultra-High-Performance Fiber-Reinforced Concrete (UHPFRC) is an enhanced group within HPFRCC, providing the highest mechanical properties in terms of tensile, compressive strengths, and energy absorption.

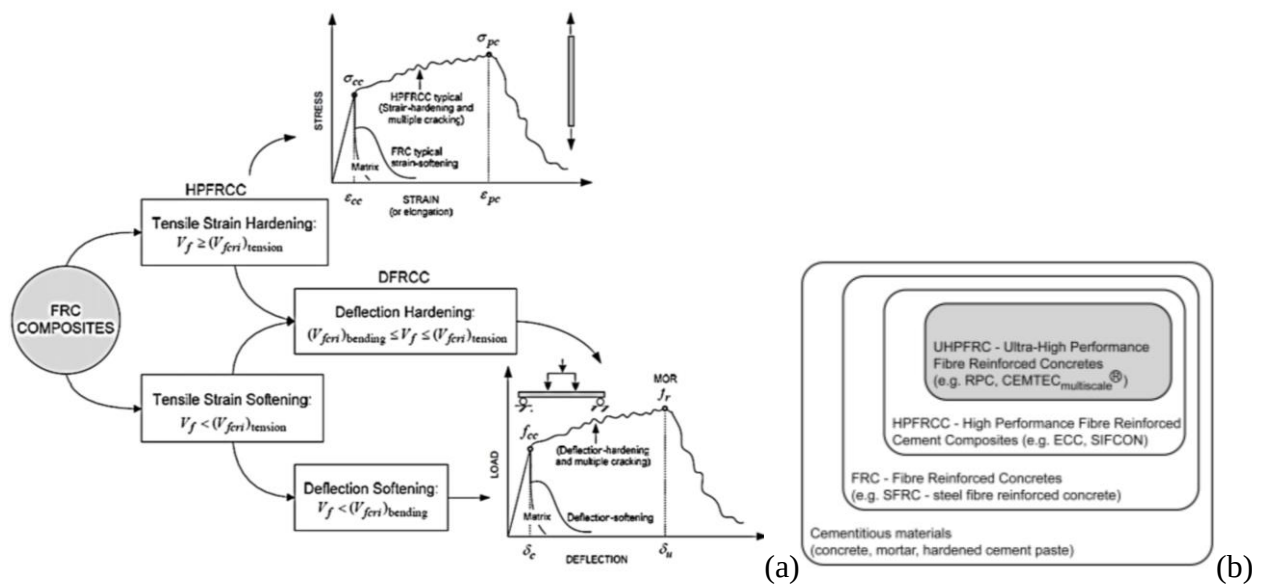


Figure 2.22 FRC classification: (a) classification based on tensile and bending behavior (Naaman and Reinhardt, 2006); and (b) sub-division of cementitious materials based on enhanced properties (Habel, 2004)

2.4.2 Composition of HPFRC

All HPFRC is mainly composed of two constituents: the concrete matrix and the fibers. The behavior primarily depends on the mechanical properties of the cementitious matrix; secondly, on the type of fibers, the volumetric fraction, and tensile strength; and lastly, on the interaction between the two main constituents.

2.4.2.1 *Cementitious Matrix*

All HPFRC matrixes are composed of similar components: sand, cement, water, mineral and chemical additives. Silica sand is usually used, containing solely fine aggregates with a nominal particle size less than 0.5mm. Mineral additives comprising cementitious materials and inert materials are exploited as substitutes for Portland cement to improve the fresh properties of the concrete and its compressive strength. The supplementary cementitious material is classified as pozzolans (silica fume, fly ash Type F) and hydraulic materials (ground granulated blast furnace slag (GGBS), fly ash Type C) where both participate in the cement hydration process in an indirect and direct fashion, respectively. The finest pozzolan is silica fume, also known as micro-silica, which is a rapid reactive that tends to increase the speed of the cement hydration reaction. Generally, a 10% portion of cement replacement by silica fume results in higher strengths (Shao, 2016a). Conversely, Type F fly ash is considered a less reactive component with particle size larger than cement. It has been demonstrated that the substitution of 20% of the cement content by fly ash lowers the concrete setting rate with high compressive strength (Ibrahim *et al.*, 2017). The hydraulic cementitious additive GGBS is a slow reactive normally used to improve the flowability where the optimal substitute portion was calculated to be around 30% of the cement content. Inert materials, as their name indicates, are inactive elements that can fill the voids in the concrete matrix, increasing its workability, density and, therefore, its compressive strength. Limestone is an example of an inert material currently used in cement packing known as “Portland-Limestone Cement” to create a stronger binder with eco-friendly qualities (Hooton, Nokken and Thomas, 2007). The addition of superplasticizers (SPs) improves the workability of the fresh matrix and develops a self-consolidating concrete without compromising the strength. For HPFRCC, the water is added to target a water to binder ratio of 0.2 (Zhou *et al.*, 2012).

2.4.2.2 *Fibers*

Unlike regular reinforcement, fiber reinforced concrete is classified as discontinuous reinforcement added to the cement matrix following a random distribution. Fibers are divided based on their geometry (length, aspect ratio (l_f / d_f), and anchorage mechanism) and mechanical properties (Modulus of Elasticity and tensile strength). In terms of geometry, fibers are classified as macro-fibers and micro-fibers depending on the length and diameter. Macro-fibers are longer than 18 mm with a diameter greater than 0.3 mm and low aspect ratio, while micro-fibers are shorter than 18 mm with a diameter of less than 0.3 mm and a high aspect ratio (Lawler, Zampini and Shah, 2005). It was previously proven that the ability of the fibers to bridge through the cracks is affected by the maximum aggregate size present in the matrix. ACI 544 (1998) states that the length of the fiber should not exceed 2-4 times the maximal aggregate diameter (Figure 2.23). For the HPFRCC matrix, where fine aggregates are only implemented with an aggregate size below 1 mm, the 13 mm long micro-fibers ($d_f = 0.2$ mm) are the optimal choice.

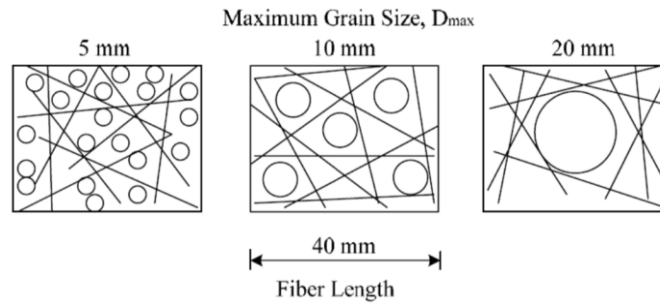


Figure 2.23 Relationship between maximum aggregate size and fiber length (Johnston, 1996)

In terms of mechanical properties, steel, carbon and glass fibers provide high Modulus of Elasticity in comparison to polypropylene, polyethylene and Polyvinyl Alcohol (PVA) fibers. Steel and PVA fibers develop similar tensile strength of 1250 MPa and 1300 MPa, respectively. However, steel fibers exhibit Modulus of Elasticity of 210 GPa, which is significantly greater than the 38 GPa of the PVA fibers (Abbass and Iqbal Khan, 2018). Fibers are added as a fraction of the total volume of concrete known as the fiber volumetric fraction. Micro-steel fibers are utilized in Ultra-High-Performance Fiber Reinforced concrete (UHPFRC) with an optimal fiber content of 2.5% (Wille, El-Tawil and Naaman, 2014), while PVA fibers are employed in Engineered Cementitious Composites (ECC) with a volumetric fraction of 2% (Ahmed, Maalej and Paramasivam, 2007). For cases where the optimal PVA fiber content is exceeded, the fibers tend to clump together,

lowering the fresh properties of the concrete. This can be interpreted as a “balling effect” for steel fibers. Furthermore, the length of steel fibers is directly related to the workability of the concrete matrix, where longer fibers lead to lower flowability creating larger entrapped air and reducing the strength of the concrete. Steel fibers are known for corrosion sensitivity. Therefore, brass-coating steel fibers or synthetic fibers are more favorable (Marcos-Meson *et al.*, 2018). However, the PVA fibers influence the water content in the matrix. This is related to the hydrophilic nature of them which attracts water molecule cells, reducing the water content and postponing the initiation of cement hydration. This challenge is normally adjusted by taking account of this effect in the design proportions. Synthetic PVA fibers are generally more expensive than steel fibers. However, the higher density of steel fibers compared to synthetic fibers results in greater overall weight and thus higher cost (Dopko *et al.*, 2018).

2.4.2.3 **Bond strength**

The main component that ensures a strain-hardening behavior of an HPFRC is the force interaction between the matrix and the steel fibers (Zollo, 1997). This is achieved by the ability of the fiber to bridge through the concrete cracks ensuing a pull-out failure mode with a high toughness index. If the strength of the matrix is higher than the tensile strength of the fibers, the fibers tend to rupture without the ability to develop a ductile behavior (Yoo, Yoon and Banthia, 2015). Unlike other fibers, steel is malleable to be cast into different forms while preserving its stiffness. This is considered an advantage over other types of fibers since they develop mechanical bond strength from the different geometries as shown in Figure 2.24.



Figure 2.24 Different shapes of steel fibers (Naaman, 2003)

It was shown that the geometry of macro-steel fibers such as hooked-end and twisted fibers increase the fiber energy absorbing mechanism due to mechanical bonding (Wille and Naaman, 2012). However, it was proven that micro-short straight steel brass-coated fibers developed similar

energy absorption capacity (Figure 2.25 (a)) and fracture energy (Figure 2.25 (b)) as hooked and twisted fibers due to the high interfacial interaction and group effect parameter with ultra-high-performance concrete matrix compared to macro-steel fibers (Wille, El-Tawil and Naaman, 2014). Moreover, this can be explained by the larger contact surface for fibers with higher aspect ratios.

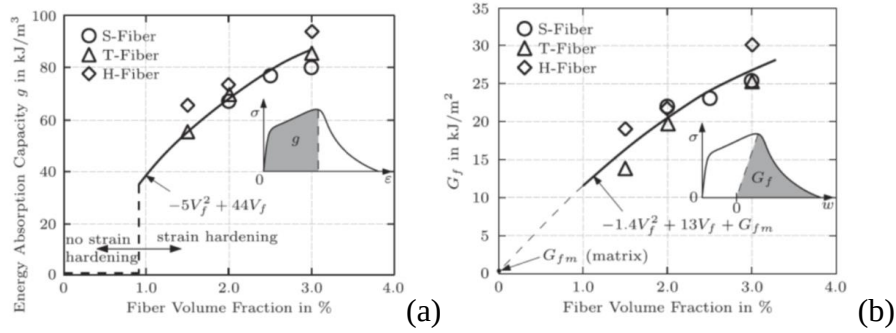


Figure 2.25 Properties of straight, twisted and hooked steel fibers: (a) energy absorption capacity; and (b) fracture energy (Wille, El-Tawil and Naaman, 2014)

PVA fibers are widely used, given their robust interaction with the concrete matrix due to their hydrophilic characteristics, forming hydrogen connections with the cement molecules (Zheng and Feldman, 1995). The full development of strength of PVA fibers, when embedded in a cementitious matrix, eliminates the need to alternate forms; they possess mechanical bonding comparable to steel. Therefore, they are mostly found in single-straight stranded form without the need of deformations. However, if the chemical bond is very strong, the failure mode can shift from ductile to brittle where fiber de-bonding is eliminated, causing rupture instead of pullout with low tensile strain capacities (Kanda and Li, 1998; Li, 2002b). Thus, fiber coating such as an oiling agent is used (Li, 2002a). Another methodology that was demonstrated to reduce the strong bond strength between PVA fibers and cement with fiber-pullout is the addition of fly ash, by increasing the porosity around the fibers (Georgiou and Pantazopoulou, 2016). The mechanical properties of ECC-PVA composites are considered time-dependent, where their strain hardening performance can change to a ductile response when the specimens become older (Li *et al.*, 2004). In terms of the concrete matrix, the addition of silica fume tends to improve the bond interface between the binder and the fibers (Habel *et al.*, 2006).

2.4.3 Examples of Previously Developed HPFRCC

2.4.3.1 Engineered Cementitious Composites (ECC)

Various design mixtures and mixing procedures have been developed for ECC (Li *et al.*, 2003), (Lepech *et al.*, 2008). Among them, Lepech and Li (2008) focused on design mixes concerning large-scale castings, which are of interest to this current research. The mixture ECC-M45 was replicated by Georgiou and Pantazopoulou (2016) according to the materials available in Europe to mainly show the improvement of the mechanical properties of this strain-hardening material when Fly Ash is added. The authors developed two design mixes, SHCC and HVFA (control specimens), as presented in Table 2.1.

Table 2.1 Mix proportions for ECC (Georgiou and Pantazopoulou, 2016)

Mix	Cement	Fly Ash	Sand	Water	Super Plasticizer	Fibers
SHCC	1.00	1.20	0.80	0.60	0.017	2% (per volume)
HVFA	1.00	1.20	0.80	0.55	0.012	-

The mixture SHCC developed by Georgiou and Pantazopoulou (2016) exhibited a very defined strain-hardening phase, as shown in Figure 2.26 (a), where the fracture energy, G_f , was calculated around 1065 N/m, approximately eight-times greater than normal concrete. The specimens developed multiple cracks, as depicted in Figure 2.26 (b). The authors were not able to test the HVFA specimens (control) under direct tension since they cracked during demolding. However, the brittle behavior of HVFA was captured in the four-point bending test (Figure 2.26 (c)) with a sudden drop in the resistance curve once the yield load was reached, and a splitting failure mode (Figure 2.26 (d)). This was not the case for specimen SHCC-4PB, where the load-deflection response provided a very ductile performance with a yield load of 24.3 kN and an ultimate load of 40.25 kN (Figure 2.26 (c)). Multiple cracking can be observed in Figure 2.26 (d).

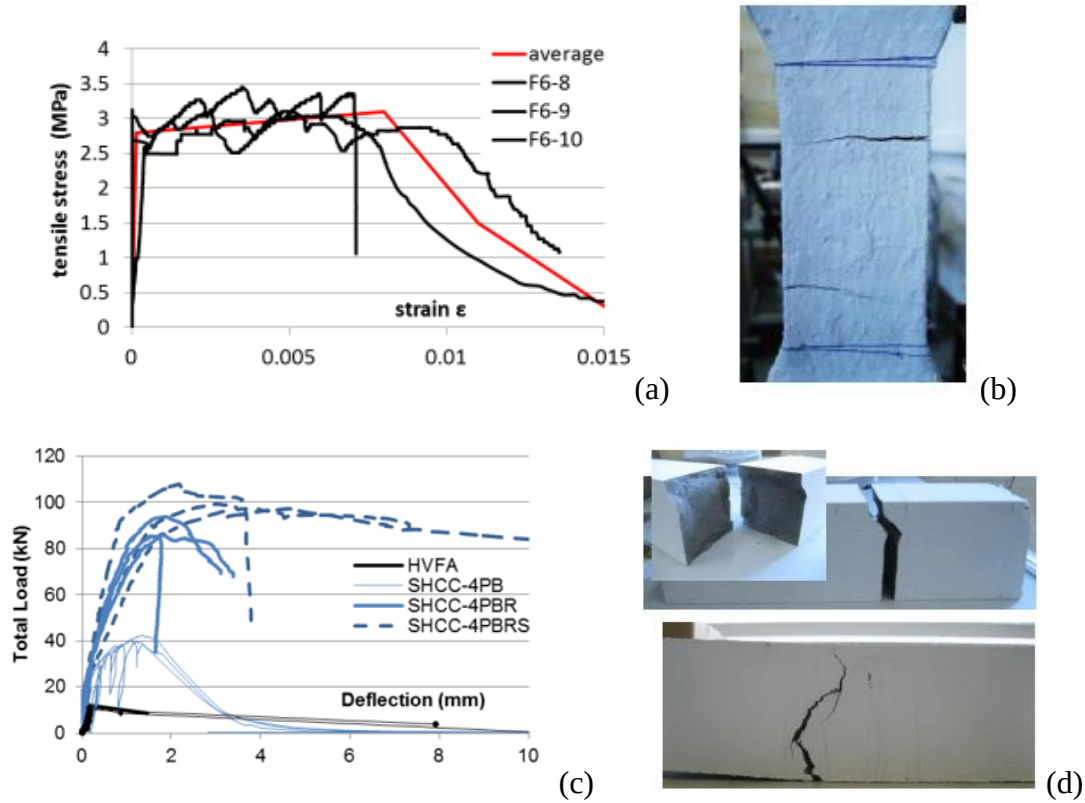


Figure 2.26 Test results: (a) stress-strain from direct tensile for SHCC; (b) dog-bone failure mode; (c) comparison of the four-point bending response for HVFA (control) and SHCC; and (d) failure mode of HVFA (top) and SHCC (bottom)

The same mix design was replicated in North America (Eshghi, 2018) following almost identical proportions, using the products available in Toronto, Canada. The mixing proportions are presented in Table 2.2. The mixing procedure consisted of mixing all the dry material together for 2 minutes. The water content was divided into two portions. SP was introduced to the first half of the water and then gradually poured into the dry materials and mixed for 3 minutes. Thereafter, the fibers were spread slowly into the moist mixture while the mixing proceeded between 5-10 minutes. Lastly, the second portion of the water was added and mixed until a proper fresh consistency was obtained. The cube (50 mm x 50 mm x 50 mm) compressive strength was approximately 70 MPa (age greater than 93 days).

Table 2.2 Mix proportions for ECC (Eshghi, 2018)

Series Name	Fiber (Kg)	Cement (Kg)	Fly Ash (Kg)	Silica Fume (Kg)	Slag (Kg)	Sand (Kg)	Water (Kg)	SP (Kg)	Total Volume (L)
M1-M5	1	16	26.7	4.4	1.8	17.8	12.44	0.4	40

Two sample prisms S1 and S2 (280 mm x 50 mm x 50 mm) were tested under four-point loading, where the load versus mid-deflection illustrates a ductile response with strain-hardening as shown in Figure 2.27 (a). The calculated tensile strength was 5.9 MPa while the flexural tensile strength was 11.3 MPa. S1 and S2 experienced multi-cracking as illustrated in Figure 2.27 (b) and (c), respectively.

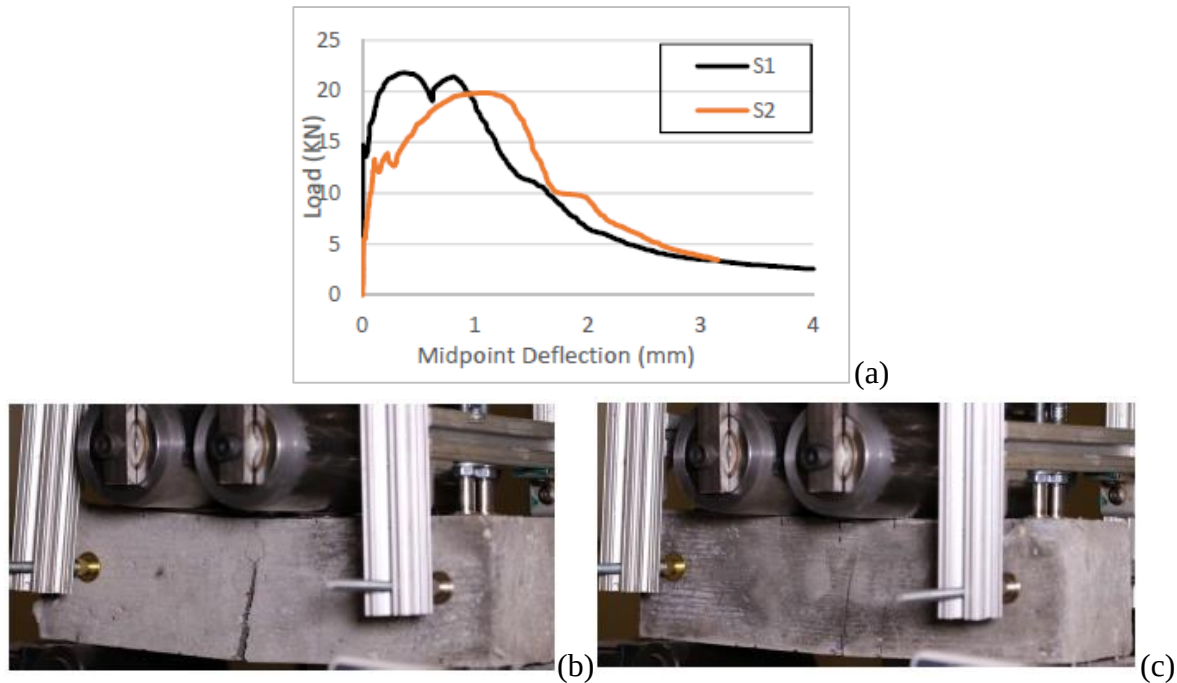


Figure 2.27 Prism testing: (a) load-midpoint deflection for prisms under flexure; (b) failure mode of prism S1; and (c) prism S2 (Eshghi, 2018)

2.4.3.2 Ultra-High-Performance Fiber-Reinforced Concrete (UHPFRC)

UHPSFRC has recently been used as a commercial material, with economical repairing solution features (Doiron, 2016). In addition, in-house design mixes have been developed using local materials such as those by Yu, Spiesz and Brouwers, (2014) and Shao (2016). The latter developed a design mix with the constituents in Table 2.3 for a water to binder ratio of 0.18 and 0.2 using materials that were available in Ontario.

Table 2.3 Proposed mix design proportions for a water to binder ratio of 0.2 and 0.18 (Shao, 2016)

Constituents	Weight (kg/m ³)	
	W/B: 0.20	W/B: 0.18
GUL cement (10 to 12% limestone powder)	724.13	757.81
Ground granulated blast-furnace slag (Grancem)	362.06	378.90
Densified silica fume	120.69	126.30
Silica Sand (#530) (grain size less than 1mm)	668.60	699.70
Polycarboxylate Superplasticizer (Approx. 40% solid content) MasterGlenium 3400NV	17.01	25.83
Steel fibre (13 mm of length, 0.2 mm of diameter)	195.75	195.75
Tap Water	241.13	227.34
Total	2329.37	2411.64
Fibre vol.%	2.5%	2.5%
Sand/Binder	55.4%	55.4%
Superplasticizer Dry Extract/Binder (DE/B)	0.62%	0.90%
SF/Binder	10%	10%
GGBFS/Binder	30%	30%

The design mix presented adequate fresh and hard properties. It exhibited a ductile stress-strain response under direct tension obtained from testing slender prismatic shapes (585 mm x 50 mm x 50 mm) with a cracking strength and corresponding strain of 7.57 MPa and 0.00017, respectively. The strain-hardening phase is evident in Figure 2.28 (a) with the cracking pattern at failure depicted in Figure 2.28 (b).

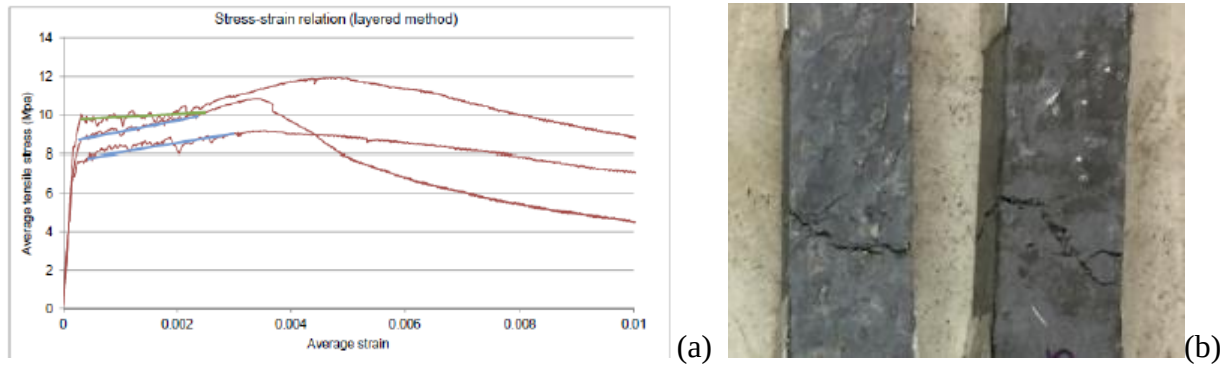


Figure 2.28 Test results: (a) stress-strain relationship of mix design under direct tension; and (b) cracking at failure (Shao, 2016)

This design mixture was then replicated (Saikali, 2019 and Tsiotsias, 2019) using the same materials to perform beam and pullout tests to extract the bond strength developed with regular reinforcement. Saikali (2019) followed the same design mix proposed in Table 2.3 for a water to binder ratio of 0.2 but varied the amount of superplasticizer to obtain proper workability (Table 2.4).

Table 2.4 Design mix proportions (Saikali, 2019)

Cement (kg/m ³)	Fine Sand (kg/m ³)	Slag (kg/m ³)	Silica Fume (kg/m ³)	Superplasticizer (kg/m ³)	Water (kg/m ³)	Steel Fibers (kg/m ³)	w/b
724.13	668.6	362.06	120.69	11.1	241.13	195.75	0.2

The mixing procedure was divided into four stages: the first is the dry state, where the cementitious materials were added and mixed for 5 minutes. Then, the water and half of the superplasticizer were gradually poured and mixed for 5 minutes. Subsequently, the mix was rested for 10 minutes to transform into the “wet state”. The second portion of superplasticizer is then slowly added until the mix was in the “liquid state”. When the mix presented a proper flowability, the brass-coated steel fibers were added and mixed for 5 minutes to reach its final state. Prisms (280 mm x 50 mm x 50 mm) were tested under flexure and responded in a ductile failure mode with a tensile flexural strength of 28.8 MPa and a corresponding mid-deflection of 0.56 mm. In addition, multiple cracks were observed in the specimens. Figure 2.29 (a) and (b) provide the load-mid-deflection response for the prisms and the failure modes, respectively.

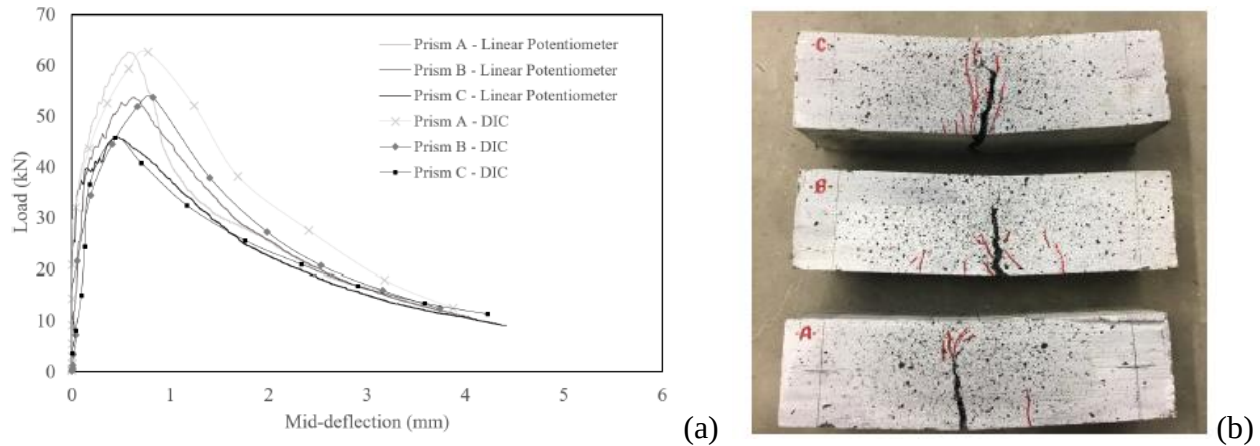


Figure 2.29 Prism test results: (a) load-mid-deflection response; and (b) mode of failure (Saikali, 2019)

Based on the satisfying mechanical properties of the design mix under flexure captured by Eshghi (2018) and Saikali (2019), these design mixes were used as a reference point for the ECC and for the UHPSFRC mix designs considered for this study, respectively.

2.5 SMA-HPC Hybrid Composite

The combination of two novel materials, such as SMA and HPC, each one characterized by their mechanical properties, provide opportunities to develop structures with enhanced performance. Current research is investigating the performance of High-Performance Concrete and Ni-Ti (SMA) under cyclic loading, to incorporate these two smart materials in the critical regions of reinforced concrete structural elements.

Two slender shear walls were constructed based on CSA A23.3-14 (Canadian Standards Association, 2004) and ACI 318-14 (ACI Committee 318, 2014), with an aspect ratio of 2 (Tolou Kian and Noguez, 2018). The first was a control wall (CW) as shown in Figure 2.30 (a) while the second was a hybrid SMA-SFRC wall containing four additional #4 SMA longitudinal bars in each boundary as illustrated in Figure 2.30 (b) up to a height of 550 mm (Figure 2.30 (e)). The SMA bars were connected to regular steel bars using mechanical couplers (Figure 2.30 (d)). The research included the study of Steel Fiber Reinforced Concrete with a fiber volumetric fraction of 0.75% and a compressive strength of 51 MPa. Note that the SFRC containing hooked-end, 50 mm long steel fibers were mainly dominated by strain-softening behavior without developing strain-hardening under direct tension. Thus, the SFRC provided improved post-cracking behavior compared to conventional concrete.

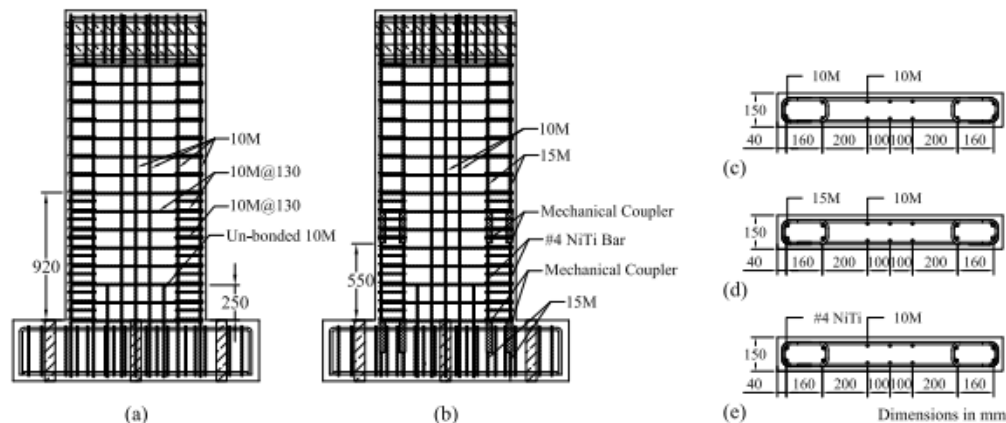


Figure 2.30 Dimensional and reinforcement details: (a) elevation view of Wall CW; (b) elevation view of Wall SMA-SFRC; (c) cross-section of CW; (d) cross-section of SMA-SFRC above 780 mm; and (e) cross-section of SMA-SFRC below 550 mm (Tolou Kian and Noguez, 2018)

Both walls failed due to the rupture of the conventional reinforcing steel, detected by the 20% drop in the lateral load from the peak. However, the difference in the mechanical properties between

the conventional concrete and SFRC is reflected in the post-yielding phase of the lateral load-displacement responses, where the first bar ruptured at a drift of 5% for Wall SMA-SFRC in comparison to 3.3% for Wall CW (Figure 2.31 (a)). Both walls sustained similar base shear strength of 170 kN (Figure 2.31 (b) and (c)). However, the SMA-SFRC wall experienced a higher drift capacity with reduced residual drifts. Moreover, it was observed that concrete spalling surfaced once the SMA bars yielded in SMA-SFRC at a drift of 1.4% while spalling became evident at a drift of 2.7% in CW prior to rupturing of the steel reinforcement. Furthermore, concrete spalling was more evident in CW than SMA-SFRC due to the presence of steel fibers. The stiffness of SMA-SFRC was initially higher than CW, however, it decreased with increasing load due to the concrete cracking observed at yielding of the SMA bars.

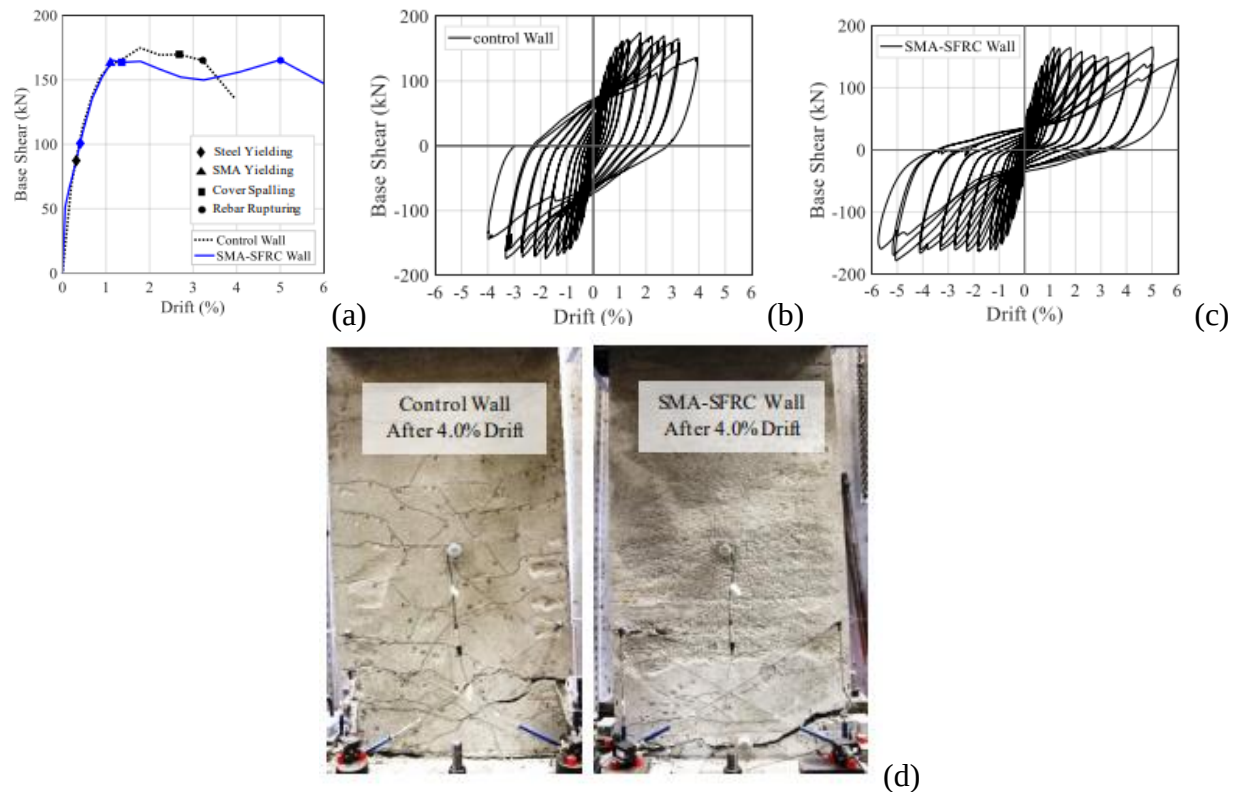


Figure 2.31 Behaviour of walls: (a) envelope response; (b) hysteresis response for CW; (c) hysteresis response for SMA-SFRC; and (d) observed damaged at failure (Tolou Kian and Noguez, 2018)

Pereiro-Barceló *et al.* (2018) constructed seven specimens of two semi-columns connected by a stub that simulated the effect of an intermediate slab or column-foundation joint. The structures were tested under compression and cyclic lateral loading. NiTi reinforcing bars were placed on one side of the stub element, anchored 150 mm into the joint and extending 750 mm beyond the

joint that was defined as the plastic hinge zone, as shown in Figure 2.32 (a). The cross-section detailing is shown in Figure 2.32 (c). The other side of the stub element, contained regular steel bars only, the cross-section detailing of this section is depicted in Figure 2.32 (b). The SMA bars were connected to the steel reinforcing bars with shear screw couplers. Two high-performance fiber-reinforced concretes employing steel fibers were used in the critical region, VHPC and HPC, with compressive strengths of 123 MPa and 80 MPa, respectively. The testing was terminated when the peak load dropped by 20%. Hairline cracking was observed in the specimens with HPC, and to a lesser degree, in the specimens with VHPC. However, all the specimens developed a major crack within the plastic hinge zone. The additional damage that surfaced in the concrete cover of HPC specimens resulted in the buckling of the SMA bars and the termination of the testing. Moreover, the limited concrete cracking observed in the VHPC specimens prevented the yielding of the longitudinal reinforcement.

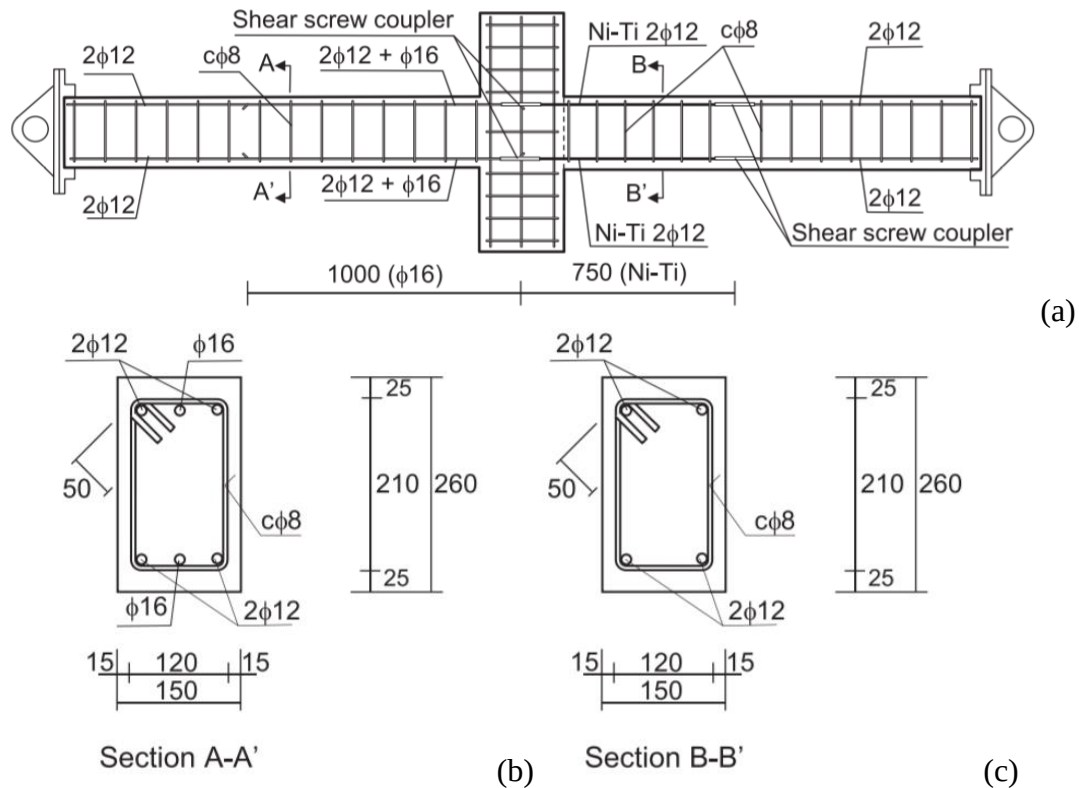


Figure 2.32 Specimens details: (a) nominal dimensions and longitudinal reinforcement; (b) cross-section details for semi-column with steel reinforcement; and (c) cross-section details for semi-column with SMA reinforcement

The hysteretic response of VHPC (Figure 2.33 (a)) was stiffer by 38% compared to HPC (Figure 2.33 (b)). Moreover, the VHPC specimens developed strength capacities greater than HPC by 40%.

The former dissipated less energy than the latter, explained by the reduced damage. However, the high concrete strength of VHPC specimens led to a lower displacement ductility of approximately 34% in comparison to HPC specimens.

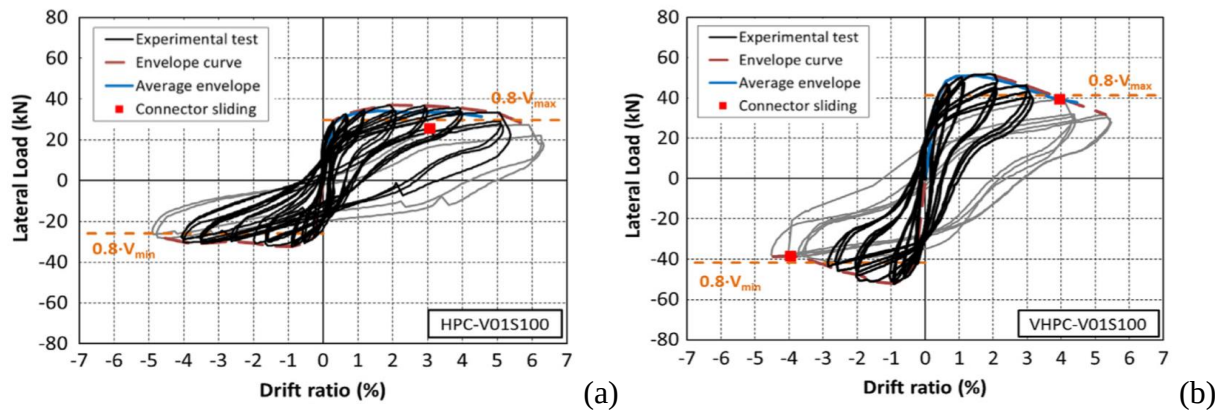


Figure 2.33 Experimental lateral load-drift response: (a) HPC; and (b) VHPC (Pereiro-Barceló et al., 2018)

The structures were modeled in one-dimension, using the FEM software OpenSees (McKenna F, Fenves GL, 2018) under static cyclic loading, as shown in Figure 2.34 (a). The constitutive models for the SMA and the HPFRC were considered using the Self Centering Material and the Engineered Cementitious Composites material models available in OpenSees. The behavior of the concrete materials under tension was based on input values extracted from the inverse analysis of previously conducted flexural tests. Figure 2.34 (b) and (c) provide the experimental and numerical hysteretic responses for one specimen containing HPC and VHPC, respectively. The numerical model was not able to replicate the reverse cyclic response adequately, including the residual drifts and energy dissipation. However, the model predicted similar peak and ultimate lateral loads, and displacement ductility.

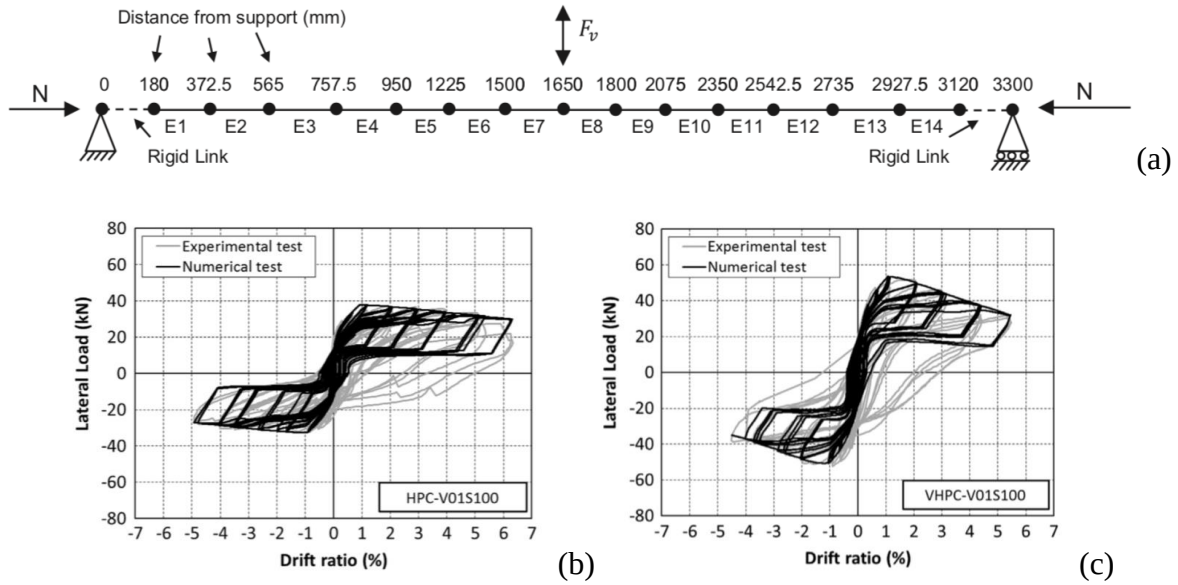


Figure 2.34 Numerical modelling: (a) FE model of the specimen; (b) hysteretic responses for HPC; and (c) hysteretic responses for VHPC (Pereiro-Barceló et al., 2018)

Saiidi and Wang (2006) repaired the concrete columns previously tested and illustrated in Figure 2.1 using ECC. This repair procedure resulted in the combined use of two novel materials ECC/SMA, which significantly diminished the concrete damage. These results demonstrate that the SMA combined with ECC can reduce repairing required after an earthquake event. This research lead hybrid SMA-ECC principles that were applied for the first time in practice in the construction of a bridge in Seattle as part of an exit ramp from Highway 99, as shown in Figure 2.35 (a). SE-SMA bars were placed in the plastic hinge region of the bridge columns and coupled with conventional reinforcement, as depicted in Figure 2.35 (b). The columns were filled with ECC in the critical region, and traditional concrete for the rest of the structure was utilized. In the case of an earthquake scenario, the bridge is expected to present a ductile behavior (Doughton, 2017).

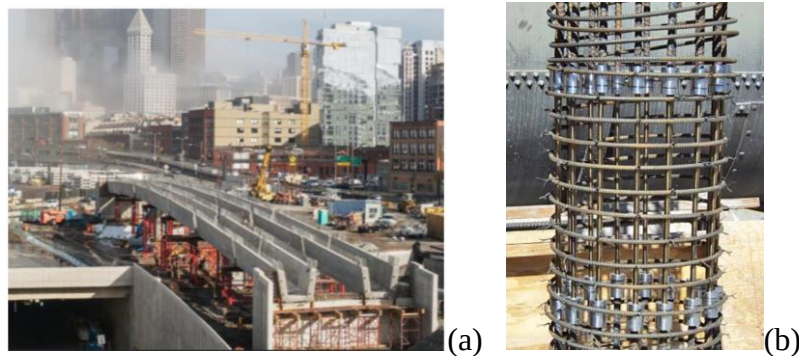


Figure 2.35 Highway 99 in Seattle: (a) Off-ramp exit bridge; and (b) column reinforcement detailing

Recent research focused on spatial distribution of Ni-Ti SMA in structural elements with the incorporation of SMA-fibers as opposed to discrete large-diameter bars. This procedure enhances the bond strength between the SMA and the concrete. SMA-ECC fiber-reinforced composites present high deformation compatibility, energy dissipation and post-recovery where the high tensile strain capacities of ECC accommodate the super-elastic properties of NiTi-SMA. Li, Li and Song (2015) researched the behavior of SMA-ECC beams under dynamic loading with comparison to SMA-RC (SMA fibers with conventional concrete), ECC (without SMA fibers) and RC (without SMA fibers). As illustrated in Figure 2.36 (a), SMA-RC and SMA-ECC beams exhibit similar self-centering greater than the two other beams owing to the presence of the SMA fibers. However, the main difference between SMA-RC and SMA-ECC beams relies on the spatial strains presented in Figure 2.36 (b) after several loadings. It was evident that the combination of these two novel materials altered the development of one localized crack (0.35 mm wide) in SMA-RC to uniformly distributed micro-cracks with “self-repairing” properties in SMA-ECC.

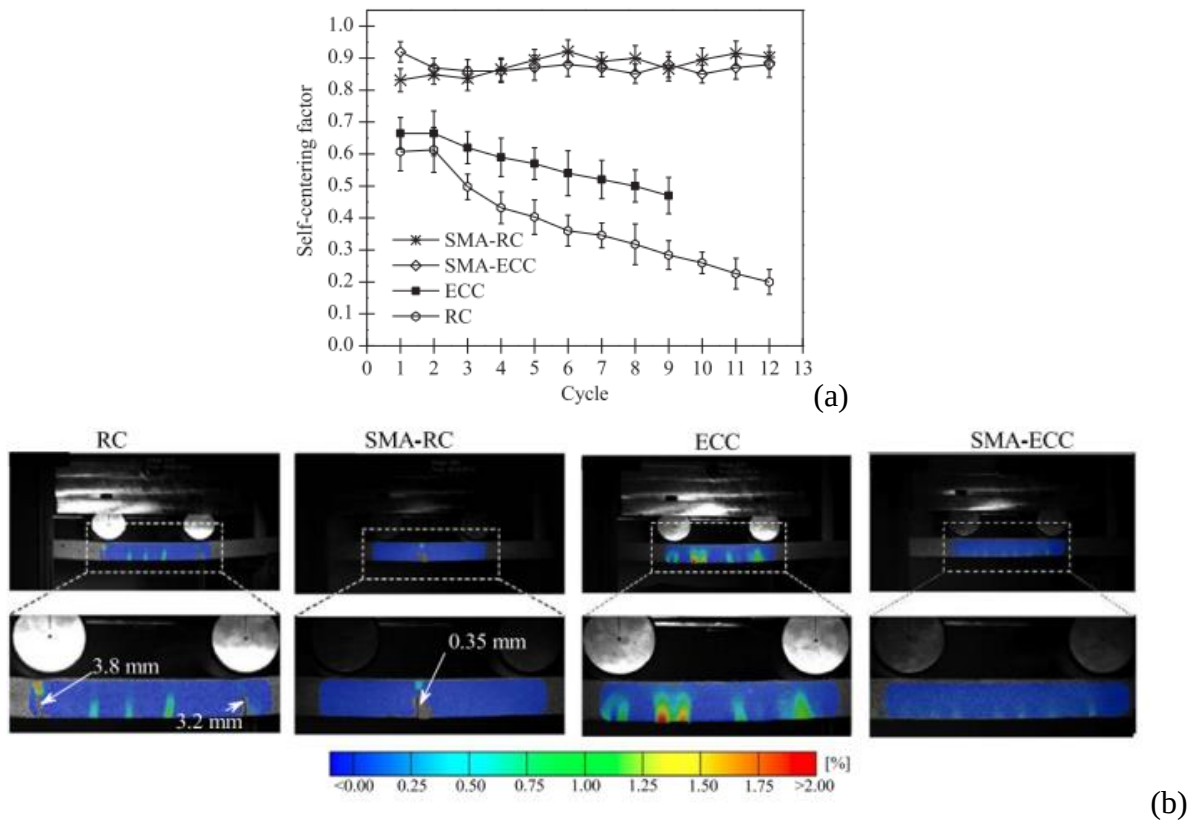


Figure 2.36 Comparison between SMA-RC, SMA-ECC, ECC and RC beams under cyclic loading: (a) self-centering factor; and (b) spatial strain measured by DIC after unloading of the 7th cycle

Ali, Soliman and Nehdi (2017) incorporated NiTi-SMA in short fibers in the Engineered Cementitious Composites (ECC) matrix as a methodology to enhance the impact resistance and self-healing capacities. NiTi-SMA fibers (16 mm long) with a volumetric fraction varying between 0% to 1.5% were added to the ECC matrix with a 2% volume fraction of 8 mm – long PVA fibers to study their performance under impact loading. The hybrid matrix provided a compressive strength of 70 MPa and a ductile stress-strain curve under direct tension reaching tensile strengths varying between 6 MPa and 8 MPa as shown in Figure 2.37 (a). The addition of 1.5% volumetric fraction of SMA fibers (ECC2-1.5) increased the ultimate tensile strength of the matrix by 39.8% compared to the control ECC2-0 with the ability to reach high tensile strains up to 8%. Comparison of the impact energy at failure between the concrete matrix with and without SMA fibers reveals that the energy dissipation of ECC-SMA composites increased by 284%, 1687% and 1628% from the control matrix with the addition of 0.5%, 1% and 1.5% of SMA fibers, respectively (Figure 2.37 (b)). Furthermore, the contrast in impact energy between the hybrid composites at first crack and at failure demonstrates ductile behavior defined by the strain-hardening phase with the ability to sustain multiple cracks with limited propagation of cracking prior to failure. This is illustrated by the comparison of the failure modes of ECC2-0 (Figure 2.37 (c)) and ECC2-1.5 (Figure 2.37 (d)).

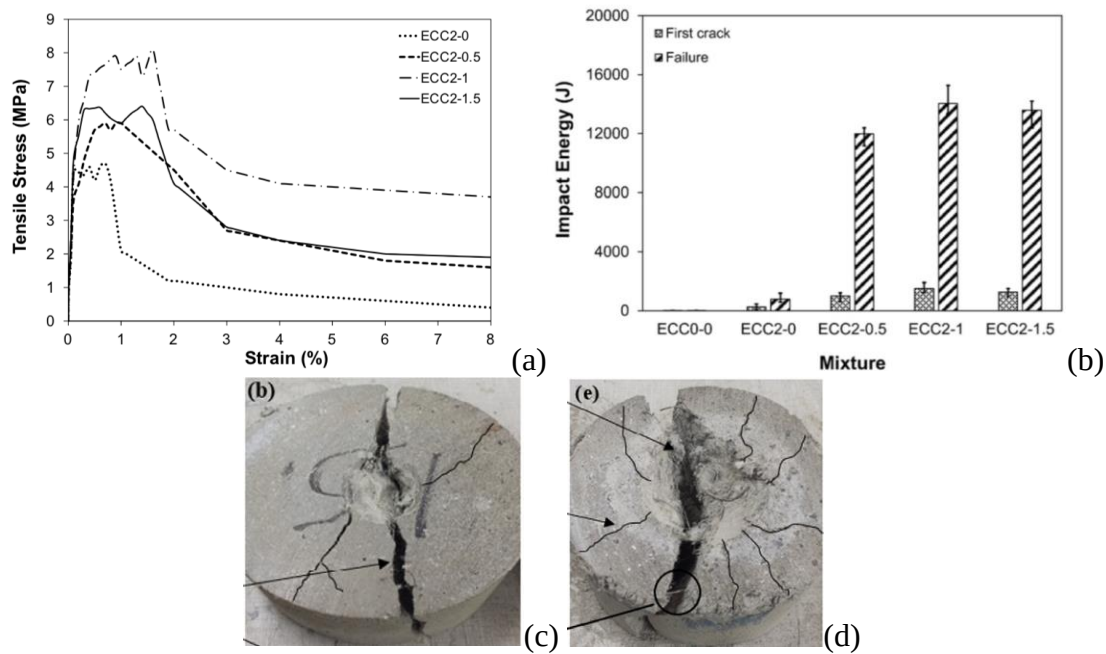


Figure 2.37 SMA-ECC test results: (a) stress-strain behavior of composites under direct tension; (b) impact energy enhancement; (c) ECC2-0 at failure; and (d) ECC2-1.5 at failure

The implementation of Superelastic-Shape Memory Alloy (SE-SMA) reinforcement and High-Performance Fiber Reinforced Concrete (HPFRC) in new construction and as part of repairing/rehabilitation procedures was introduced in this chapter. The application of the self-centering phenomenon of SMAs and the distinctive tensile properties of HPFRCs into reinforced concrete structures was demonstrated to enhance the seismic response and mitigate damage. However, limited studies have been reported on the combined application of SE-SMA and HPFRC in critical regions. The potential for merging these two novel materials is presented in the research herein. A repairing process is investigated that involves the employment of emerging High-Performance Fiber-Reinforced Concrete (HPFRC) combined with the innovative Shape Memory Alloy (SMA) reinforcement with the intention to enhance the seismic performance of reinforced concrete slender shear walls. After repairing, the walls were subjected to simulated earthquake loading. In addition, the experimental testing provides benchmark data, which will be used to develop and corroborate numerical models to evaluate concrete structures repaired with these new materials. The dual use of these novel materials promotes self-centering capacity with damage control.

CHAPTER 3. PRELIMINARY ANALYSIS

3.1 Background of the Walls Under Study

3.1.1 Original Walls

The structures investigated as part of this research, correspond to two reinforced concrete, ductile shear walls, previously tested by Morcos and Palermo (2019) at York University, Canada. The study aimed to investigate the self-centering capacity of Superelastic Shape Memory Alloy reinforced concrete shear walls. The walls were designed based on the seismic provisions of the Canadian Standards Association (CSA) A23.3 Design of Reinforced Concrete Structures (Morcos and Palermo, 2018).

The original steel-reinforced shear wall was constructed with conventional deformed steel reinforcement in the longitudinal direction of the web and boundary zones, and in the transverse direction for shear reinforcement and buckling-prevention ties in the boundary zones. It was designated as SWS-R, where the 'SW' denotes Shear Wall, the letter 'S' represents Steel-Reinforced, and the letter 'R' indicates Repair. The latter is due to a repair at the base of the wall after casting due to honeycombing (Morcos and Palermo, 2018)(Mena and Palermo, 2018)(Mena and Palermo, 2018)(Mena and Palermo, 2018)(Mena and Palermo, 2018). The Superelastic – Shape Memory Alloy (SE-SMA) reinforced shear wall named SWN, where the 'N' represents Nitinol-reinforced, differed by the replacement of the longitudinal steel reinforcement with SE-SMA bars in the boundary regions, within the plastic hinge zone.

Figure 3.1 presents Wall SWN utilizing SE-SMA bars in the boundary zones inside the plastic hinge region, using a non-invasive method of mechanical coupling to the steel reinforcement. The companion wall, SWS-R, was constructed entirely using deformed mild steel. The dimensions are the same for both walls, as shown in Figure 3.1 (a). The shear walls have a rectangular cross-section: 150 mm in thickness and 1000 mm in length. The height of the walls is 2200 mm, resulting in an aspect ratio of 2.2, which was selected to promote ductile flexural response.

Furthermore, these walls were constructed on a 1600 mm long, 500 mm high, and 1000 mm wide foundation blocks. The top of the walls was capped with a stiff 400 mm by 400 mm beam that measured 1600 mm in length. For the conventional steel wall, each boundary contained four 10M longitudinal bars, and the web had three pairs of evenly spaced 10M longitudinal bars. The same approach was followed for Wall SWN, except for the longitudinal steel reinforcement in the boundary zones that was replaced with SE-SMA smooth bars with a diameter of 12.7 mm, as illustrated in Figure 3.1 (b). The SMA bars were limited to a length of 900 mm within the plastic hinge region and extended 300 mm into the foundation block.

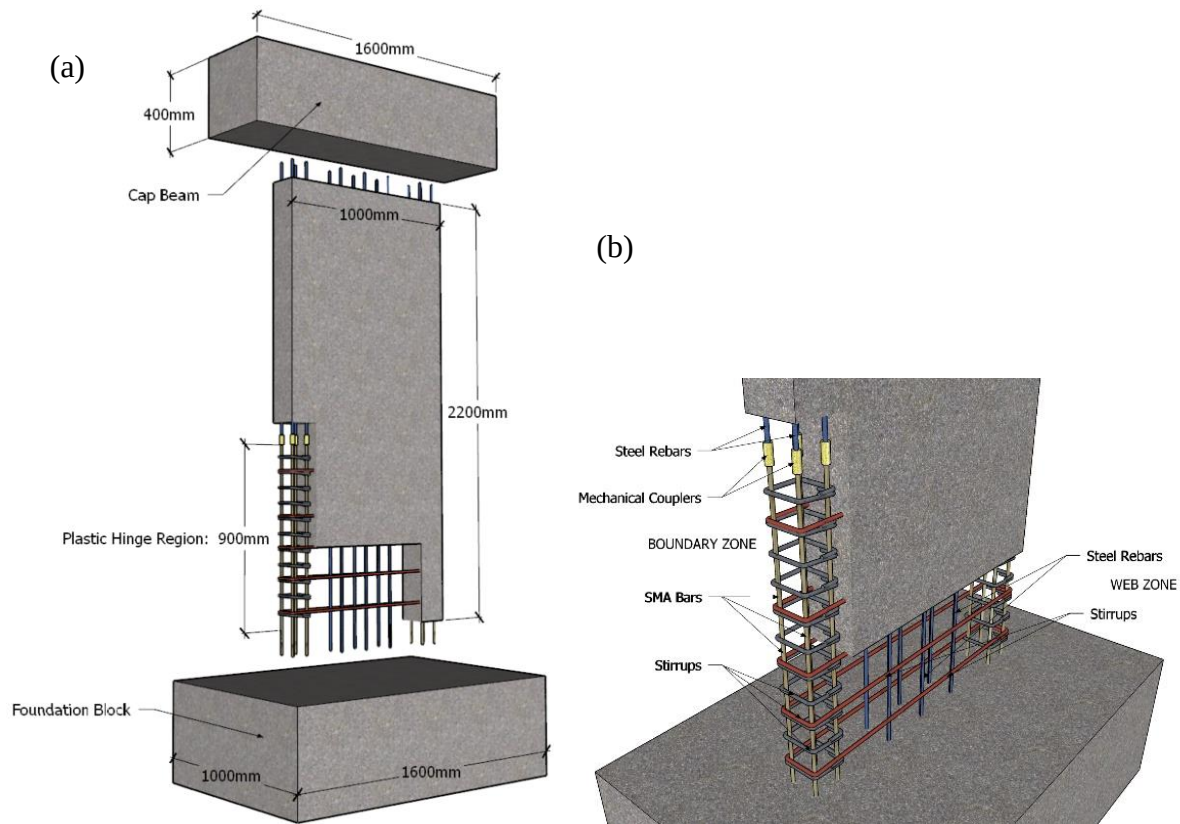


Figure 3.1 Three-dimensional representation of Wall SWN: (a) nominal dimensions; and (b) reinforcement distribution in plastic hinge

The specific detailing for the reinforcement location, spacing and distribution for Wall SWN are illustrated in Figure 3.2 Wall SWS-R had similar reinforcement detailing and is presented in Figure 3.3. Figure 3.4 provides photos of the walls after casting and before testing.

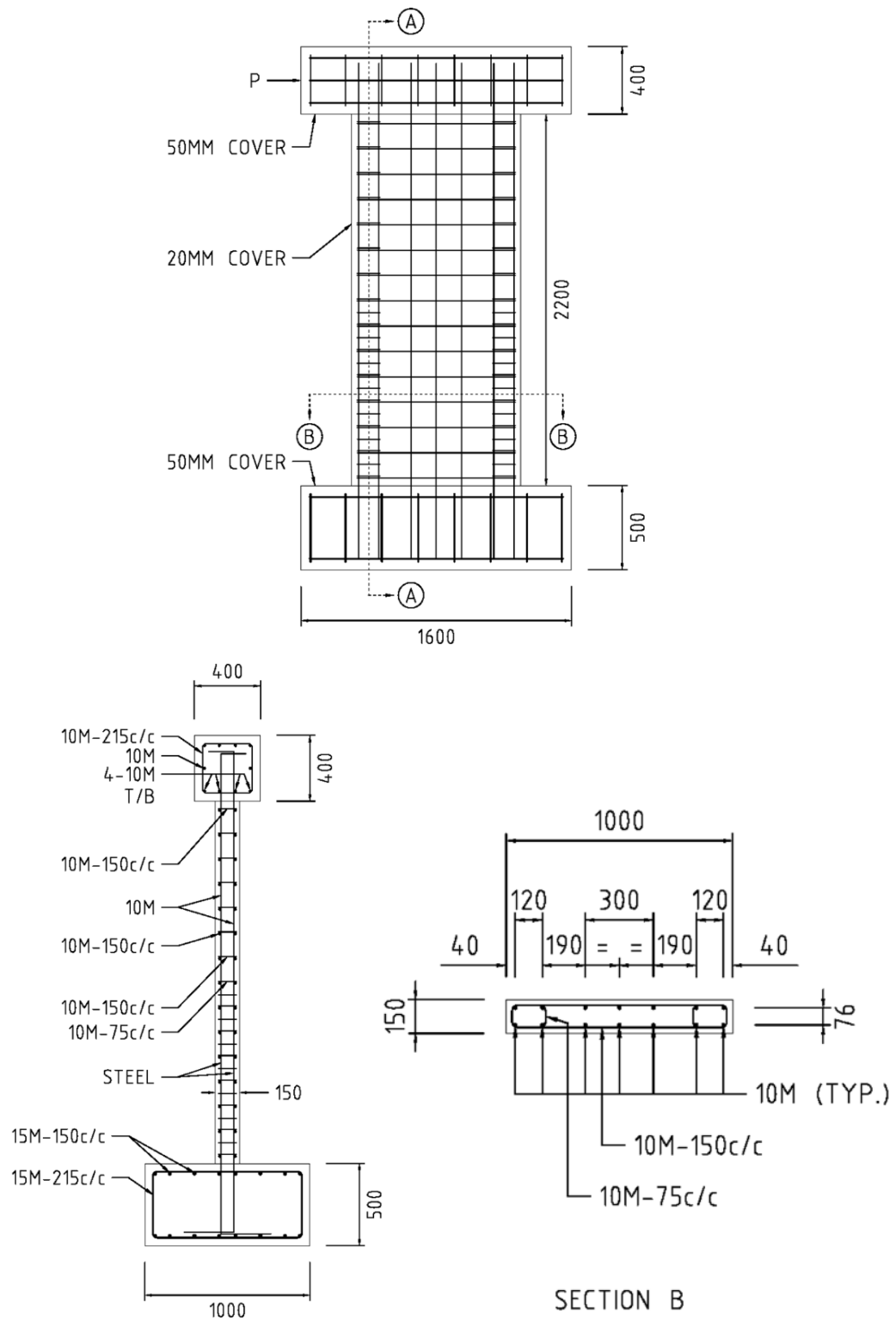


Figure 3.3 Nominal dimensions and reinforcement details for Wall SWS-R (Morcos, 2019)



Figure 3.4. Walls after casting: SWS-R (behind) and SWN (forward) (Morcos and Palermo, 2018)

3.1.2 Material Properties

Both walls utilized normal concrete, with the exception of the base of Wall SWS-R. As reported by Morcos and Palermo (2018), Self-Consolidated Concrete (SCC) was employed to repair the base of the wall due to honeycombing after casting of Wall SWS-R. Table 3.1 provides the material properties of each concrete.

Table 3.1 Concrete material properties for original shear walls (Morcos and Palermo, 2019)

Wall	Concrete Type	f'_c (MPa)	Age (days)	Aggregate Size (mm)
SWS-R	30MPa Normal Concrete	50.76	500	14
	Sika Sikacrete-08 SCC	73.42	405	8
SWN	30MPa Normal Concrete	39.28	410	14

Morcos (2020) conducted experimental testing on the primary reinforcement used in the walls to establish the main properties. Typical monotonic stress-strain responses are shown in Figure 3.5 for Wall SWS-R and in Figure 3.6 for Wall SWN. Moreover, the cyclic stress-strain response for the

NiTi SMA reinforcement is depicted in Figure 3.7. Figure 3.8 and Figure 3.9 show the cyclic response for the steel reinforcement in Wall SWS-R and SWN, respectively. Lastly, Table 3.2 and Table 3.3 presents the main mechanical properties of the steel reinforcement and SE-SMA bars employed, respectively.

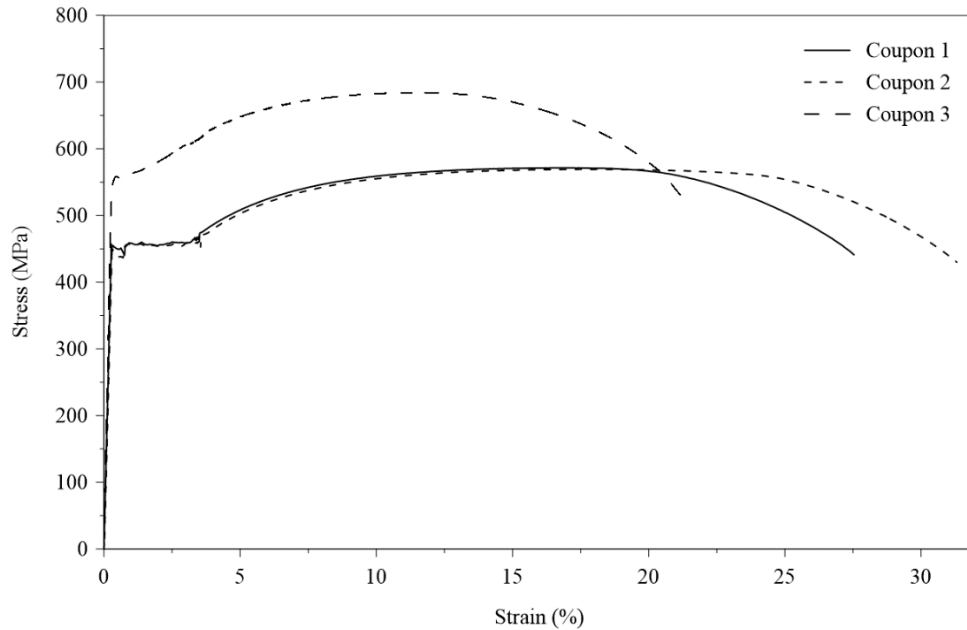


Figure 3.5 Typical stress-strain response of 10M reinforcement for Wall SWS-R (Morcos and Palermo, 2019)

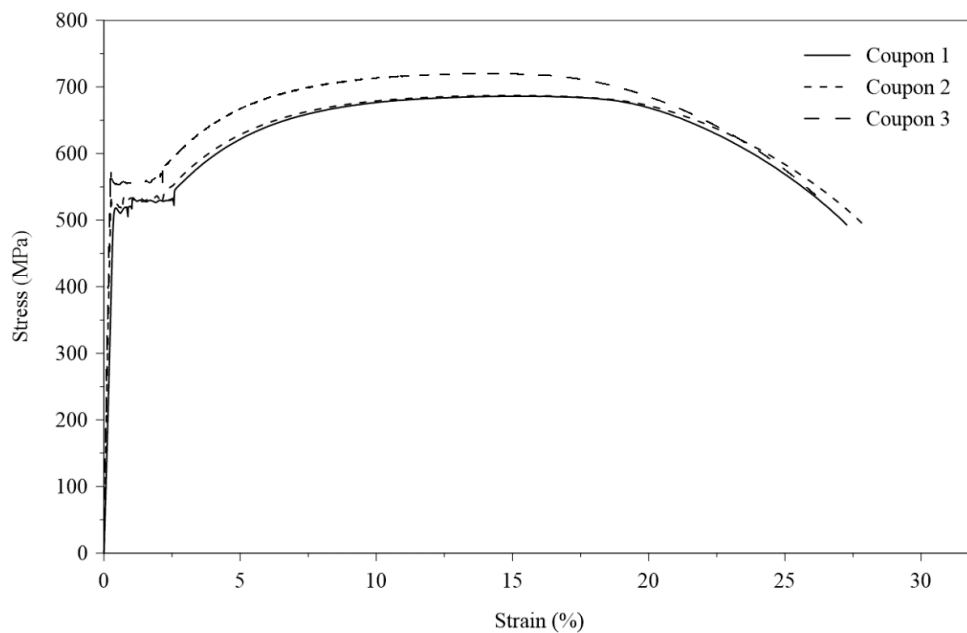


Figure 3.6 Typical stress-strain response of 10M reinforcement for Wall SWN (Morcos and Palermo, 2019)

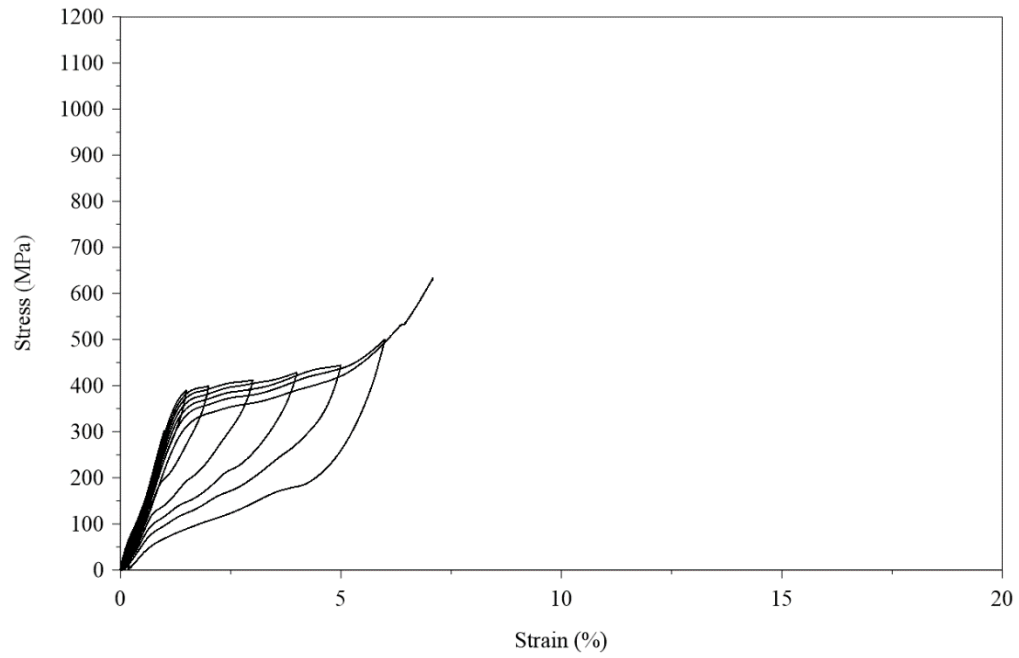


Figure 3.7 Cyclic stress-strain response for SMA reinforcement (Morcos and Palermo, 2019)

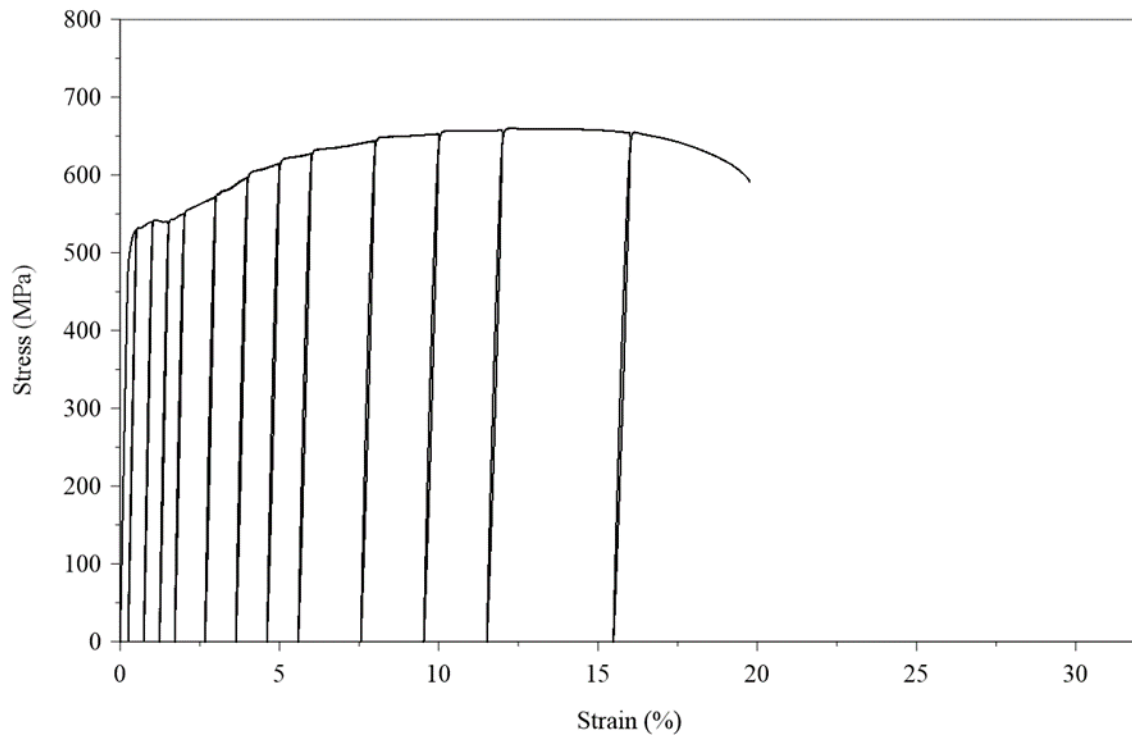


Figure 3.8 Cyclic response for 10 M bar for Wall SWS-R (Morcos and Palermo, 2019)

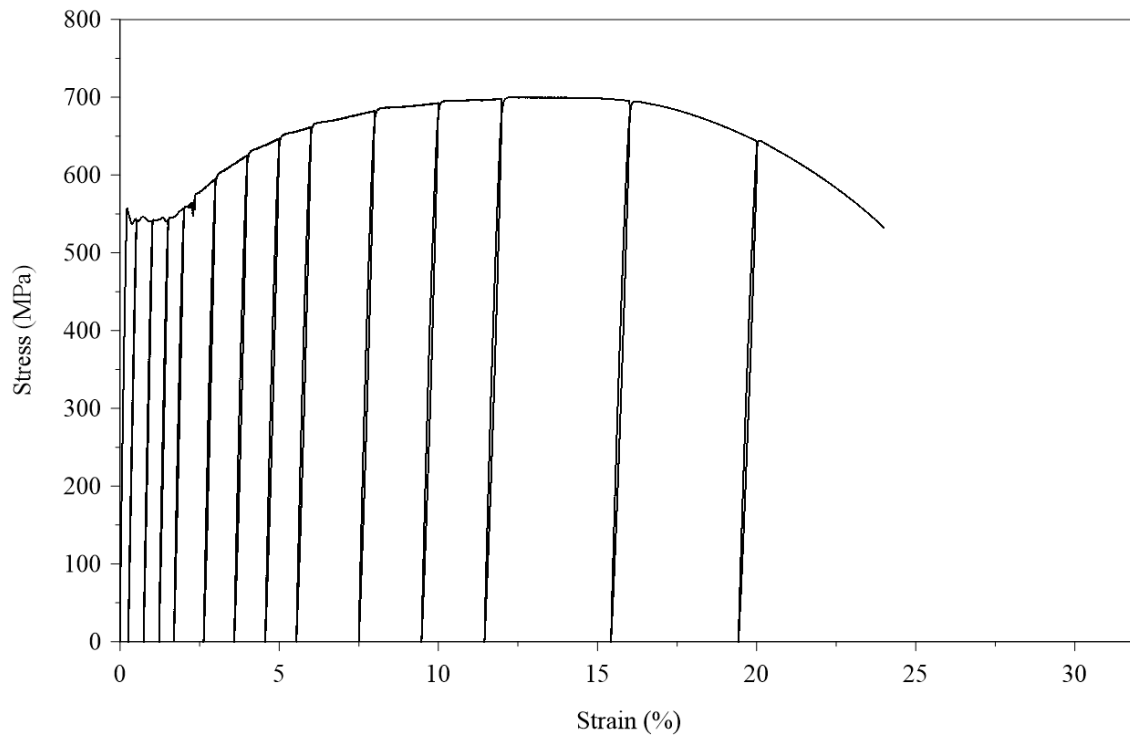


Figure 3.9 Cyclic response for 10 M bar for Wall SWN (Morcos and Palermo, 2019)

Table 3.2 Mechanical properties of steel reinforcement used in the original walls (Morcos and Palermo, 2019)

Rebar	Diameter (mm)	Yield		Modulus of Elasticity (GPa)	Strain Hardening (%)	Tensile		Rupture (%)	
		Strength (MPa)	Strain (%)			Strength (MPa)	Strain (%)		
10M	a	11.3	488	0.3	163	2.7	608	15.4	26.7
	b	11.3	530	0.3	162	2.3	698	14.8	27.1
15M	16		480	0.3	170	3.1	581	14.1	26.3
#13	12.7		482	0.3	161	2.1	645	14.8	27.5

Table 3.3 Mechanical properties of SE-SMA reinforcement used in the original Wall SWN (Morcos and Palermo, 2019)

Upper Plateau (at 3% strain during initial loading)	435	MPa
Lower Plateau (at 2.5% strain during unloading)	195	MPa
Residual Elongation (difference at 7.0MPa stress, after 6% strain)	0.25	%
Yield Strength	338	MPa
Modulus of Elasticity (mechanical)	42	GPa
Tensile Strength	1034	MPa
Uniform Elongation	16	%

3.1.3 Previous Damage and Visual Inspection

Slender shear walls behave as vertical cantilever beams with their strength controlled by flexure when properly designed and subjected to lateral loads. The National Building Code of Canada (NBCC) 2015 and Canadian Standard Association (CSA)-A23.3-19 provide current estimates of seismic loading, acceptable analysis methods, and provisions that are based on capacity design principles, respectively. For slender shear walls, dissipation of the earthquake energy is controlled by yielding of the tension reinforcement located in the boundary regions and, to some degree, the vertical reinforcement spread along the web of the wall. The non-linear behaviour of the wall is assumed, in most cases, to be governed by the formation of a single plastic hinge at their base, while keeping the upper part of the wall in the elastic regime (Chaallal and Gauthier, 2001). However, recent studies have demonstrated that the formation of a second plastic hinge in walls is possible under design ground motions due to the higher mode actions on tall RC shear walls (Ghorbanirenani *et al.*, 2012)

The two walls were tested under reverse-cyclic loading to simulate the effects of a seismic event following a loading protocol established by FEMA 461 (2007) and ATC-24 (1992), in which gradual increments of drift were applied to the walls leading to failure as illustrated in Figure 3.10 (a) and (b) for Wall SWS-R and Wall SWN, respectively.

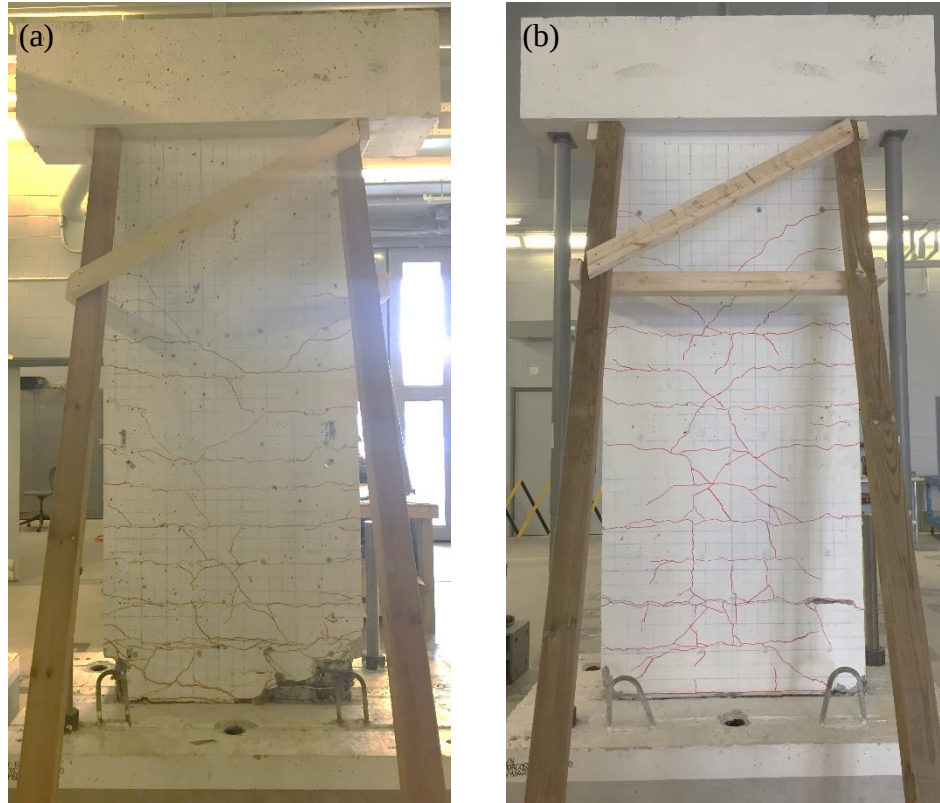


Figure 3.10 Conditions of the original walls after testing: (a) SWS-R; and (b) SWN

Based on the damage reported (Morcos and Palermo, 2019), Wall SWN sustained an ultimate drift capacity of 4.3%. Meanwhile, Wall SWS-R failed at 2.5% drift. Both walls experienced widespread damage, including hairline-flexural cracks above the plastic hinge region. Moreover, the concrete located in the plastic hinge region was severely deteriorated with the development of significant flexural and shear cracks. Figure 3.11 (a) illustrates the base of Wall SWS-R, specifically the severe concrete spalling and crushing. In addition, the reinforcing bars in the boundary regions ruptured. Wall SWN depicted in Figure 3.11 (b) developed significant flexural-shear cracks, with one major crack surfacing approximately 300 mm above the base of the wall. No SE-SMA bars ruptured in the boundary region; however, steel reinforcement in the web regions ruptured. Additionally, Wall SWN experienced milder concrete spalling with reduced cracking, attributed to the smooth surface of the SMA bars, which reduced bonding to the surrounding concrete, resulting in fewer but wider cracks. Another main difference was observed in the substantial crack along the base of Wall SWN, promoting rocking/sliding during testing, which ultimately controlled the behaviour of the wall (Morcos and Palermo, 2019).

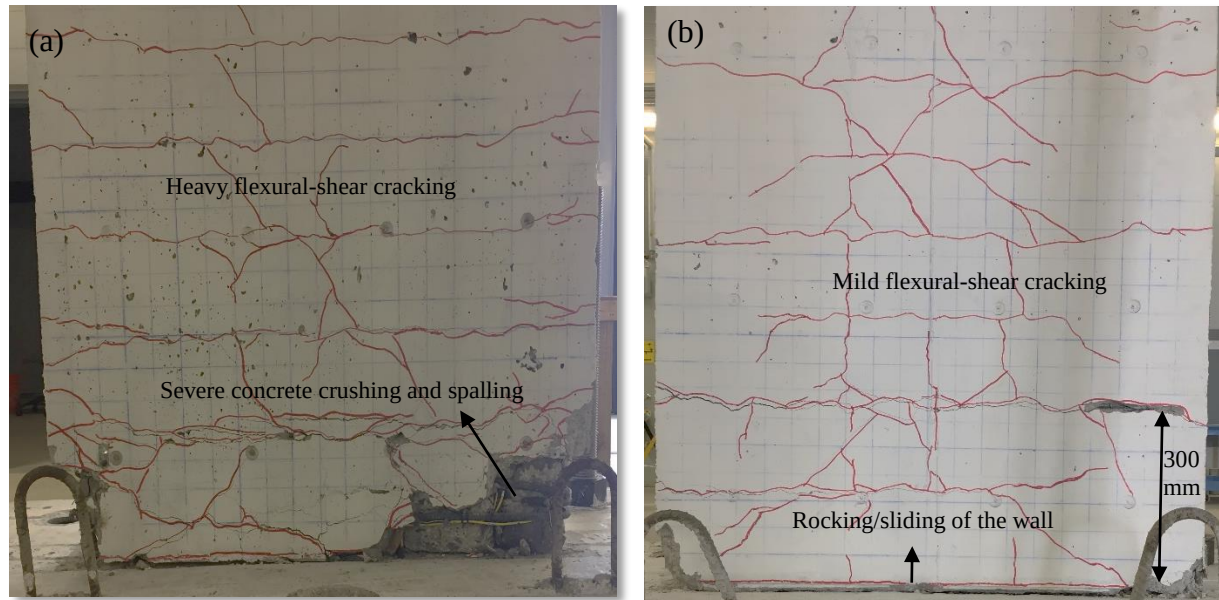


Figure 3.11 Damage in the plastic hinge: (a) SWS-R; and (b) SWN

3.2 Proposed Repairing Options

In the following sections, various approaches to repair the walls are explained and numerically investigated to provide an assessment of the most suitable repairing strategy.

3.2.1 Concrete Removal and Replacement/Addition of Reinforcement

As reported in the visual inspection in Section 3.1.3, the concrete experienced severe deterioration at the base of the wall, and the steel reinforcing bars ruptured or buckled. The damage was considered to define the extent of the concrete removal for both walls.

Three different options were proposed and further analyzed. Figure 3.12 depicts the sections of concrete to be removed from the base of the wall.

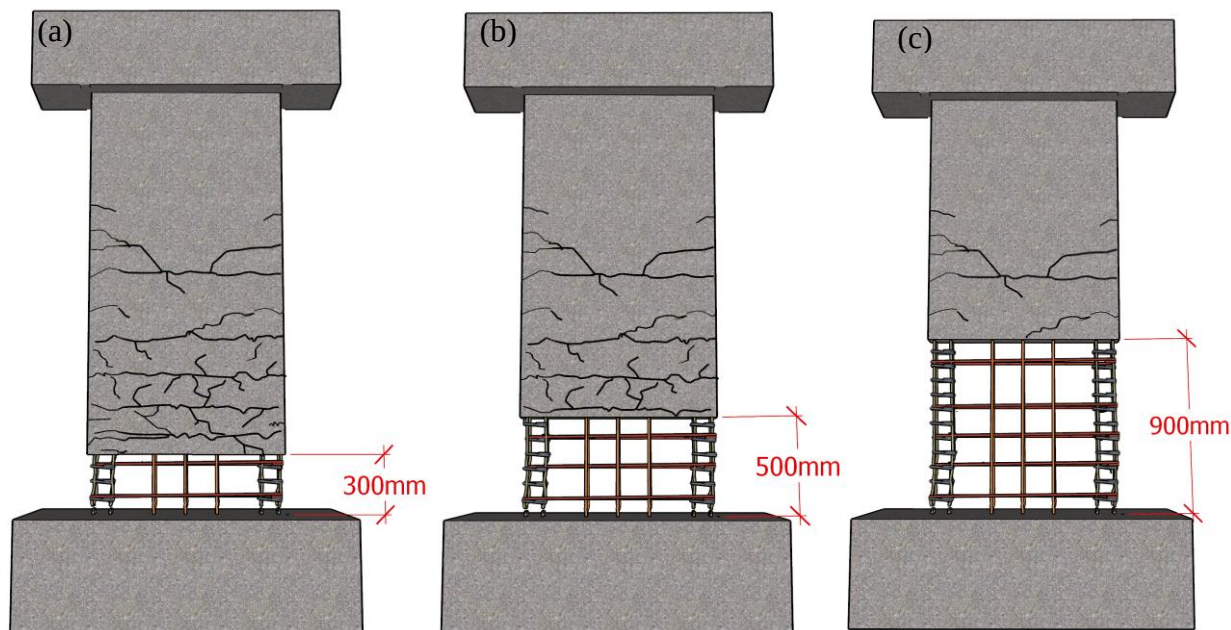


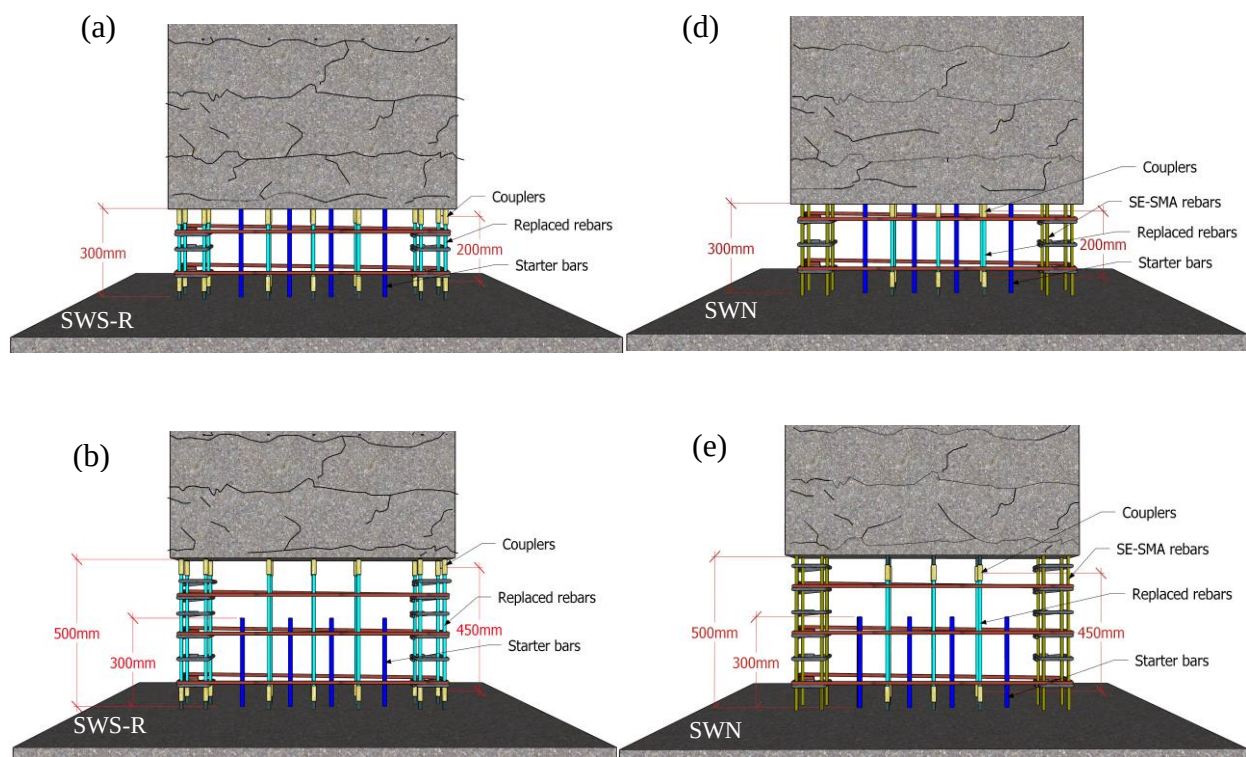
Figure 3.12 Concrete removal options measured from the base of the wall: (a) 300 mm; (b) 500 mm; and (c) 900 mm

The options were selected keeping in mind cost and time to repair the walls without compromising the overall behaviour. The removal of 300 mm of damaged concrete, as shown in Figure 3.12 (a), represents one-third of the height of the plastic hinge, and in theory, should reduce labour and materials in the repair process. In addition, it considers the zone within the plastic hinge that experienced most of the damage and offers sufficient space to replace the damaged reinforcing bars. However, the current condition of the reinforcement above 300 mm would remain unknown and potentially impact the behaviour of the structure. The removal of 500 mm of concrete from the base of the wall implies a replacement of approximately one-half of the plastic hinge, as shown in Figure 3.12 (b). This option provides repair to a broader section, including replacing ruptured and buckled bars over a larger height. The last option (Figure 3.12 (c)) involves removing the concrete over the entire plastic hinge, eliminating any plastic deformation experienced by the original walls. For each option, HPFRC is used to replace the removed concrete.

For Wall SWS-R, the reinforcing bars located in the web region experienced buckling, and the longitudinal steel in the boundary regions ruptured. In Wall SWN, SE-SMA reinforcing bars in the boundary zone were not damaged. However, the longitudinal steel reinforcement in the web region ruptured. As part of the investigation of the repair options, the damaged steel reinforcement will be removed and replaced with identical 10M reinforcing bars. Four-inch, shear-off mechanical couplers

will be employed to connect the new segments of steel to the remaining reinforcement in the original walls. The location and the extension of the new reinforcing bars and couplers will be established based on the extent of concrete removed within the plastic hinge and the location of damage of the steel reinforcement. The SE-SMA bars will remain the same and will not be replaced. Furthermore, damage to the shear reinforcement and anti-buckling ties is not expected. During the actual repair process, the stirrups and anti-buckling ties will be removed to permit replacement of the damaged longitudinal reinforcement. Thereafter, the same shear reinforcement and anti-buckling will be reused.

The rocking and sliding that manifested in the original walls will be addressed by the addition of 10M starter bars along the base of the wall, that will function as dowels, connecting the foundation block and the wall. The length of the starter bars will be less than the development length to curtail any contribution to the flexural capacity while improving the sliding shear capacity. Figure 3.13 provides schematic drawings of the different reinforcement layout for each of the repair options being considered.



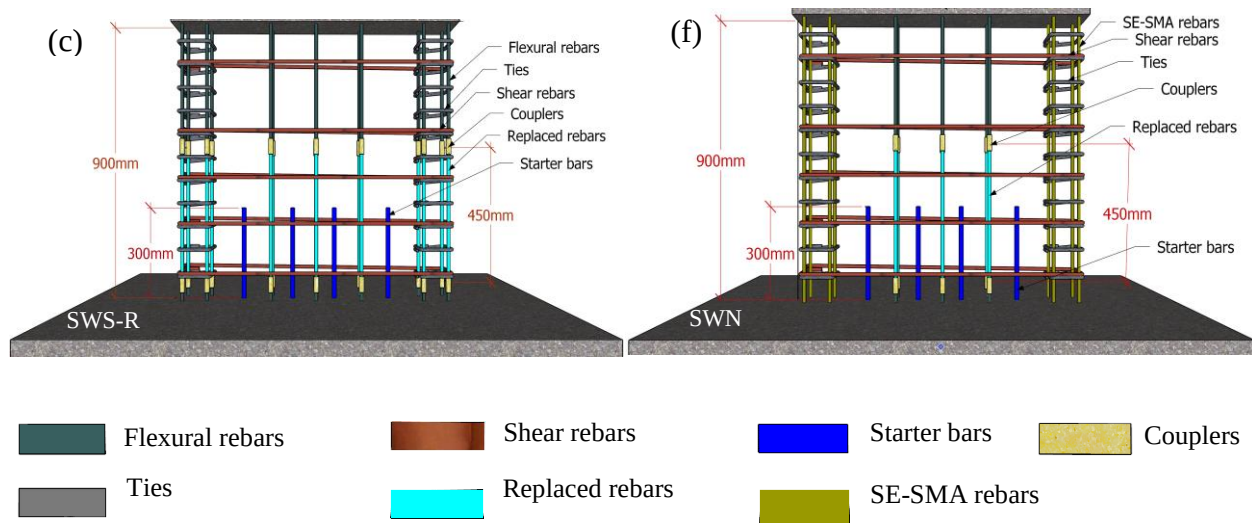


Figure 3.13 Reinforcement detailing proposed based on the height of concrete removed: (a),(b),(c) for SWS-R; and (d),(e),(f) for SWN

The reinforcement detailing depicted in Figure 3.13 varies depending on the height of concrete removed: one-third (300 mm), one-half (500 mm) or the entire plastic hinge zone (900 mm). The process will be identical: four starter bars will be placed between the longitudinal web reinforcement, spaced 75 mm within the web region and 100 mm from the boundary zone. Additionally, the starter bars would be embedded 200 mm into the foundation block and extended 300 mm above the base of the walls.

For Wall SWS-R, a total of 28 mechanical couplers will be used, and their location will be conditional to the repair option chosen, as is shown in Figure 3.13 (a), (b) and (c). Note that for Wall SWN, the quantity of couplers is only 12. The couplers will only be required for the damaged longitudinal web reinforcement since the SE-SMA bars in the boundary regions remained undamaged. Likewise, the length of the new segments of steel reinforcement to be connected to the original, undamaged reinforcement, will depend on the repair option.

Three major factors will determine the scope of the repairing methodology: the amount of concrete removed, the addition of the starter bars, and the choice of HPFRC. Therefore, preliminary numerical analyses were performed to define the extent of the repairing procedure and to establish the viability of UHPSFRC or ECC as the primary repairing material. In addition, the numerical approach consisted

of Nonlinear Finite Element Modelling (NLFEM) to achieve a detailed understanding of the behaviour of the walls, prior to and post repairing.

3.3 Preliminary Numerical Analysis

3.3.1 Introduction

Finite Element Analysis (FEA) is a numerical method used to solve engineering problems utilizing an array of mathematical techniques. The method takes a structure and subdivides it into smaller and simpler parts called Finite Elements (FE) interconnected by nodes, forming a mesh. The equations that model these FE are solved and assembled back into the larger system of equations that modelled the entire structure and gave an approximate solution. The finer the discretization, the more accurate the results. The process extends to a more sophisticated analytical approach that generates a broad system of equations that is generally too extensive to be solved by hand calculations. Different computational tools implement this method, including VecTor2, which is the software selected for this study.

3.3.2 Conceptual Basis

VecTor2 is a two-dimensional nonlinear finite element program used for the numerical analyses in this research. The program employs the Modified Compression Field Theory (MCFT) (Vecchio and Collins, 1986) and the Disturbed Stress Field Model (DSFM) (Vecchio, 2000). It is based on an incremental, total load method with a smeared, rotating crack model for reinforced concrete, utilizing concrete as an orthotropic material. Additionally, the convergence of results uses a secant stiffness formulation with an iterative procedure (Wong, Vecchio and Tamm, 2013).

Initially, the Compression Field Theory (CFT) did not consider the transmission of tensile stresses between the concrete and the reinforcement once the concrete cracked (Collins and Mitchell, 1980). The theory was modified and named the MCFT by Vecchio and Collins (1986), by establishing a formulation where the tensile stresses of the concrete between cracks were taken into consideration. The addition of the Disturbed Stress Field Model (DSFM) (Vecchio, 2000) was implemented later as

an extension of the MCFT to improve the overall response of structures by explicitly allowing slip along the crack in the elements (neglected in MCFT) and removing the restriction that the principal stress and strains directions remain coincident.

The structures that are discretized in the software make use of different element types: a 3-node constant strain triangular element, a 4-node plane stress rectangular element, and a 4-node quadrilateral element. These elements are chosen depending on the complexity of the structure. Reinforcement can be smeared within the elements. For discrete reinforcement, a two-node truss bar element is used. Lastly, to represent the interaction (bond-slip) between concrete elements and discrete reinforcement, a two-node bond-link and a 4-node contact element can be created.

VecTor2 comprises several constitutive models for smeared reinforced concrete, discrete reinforcement, and bond materials. The ability to simulate second-order mechanisms in reinforced concrete is possible through available models representing tension stiffening, tension softening, splitting effects, compression softening, dilation, confinement, crack width and slip, and bond-slip. Other models can be selected to simulate: reinforcement buckling, dowel action, concrete hysteresis, and simulation of the chronology of construction and loading of structures for retrofitting and forensic engineering purposes through the theory of engaging and disengaging of the elements (Vecchio and Bucci, 1999).

The preprocessor and postprocessor for VecTor2 are FormWorks and Augustus, respectively. FormWorks was developed by Wong (2002) as the user-friendly, graphical interface and pre-analysis tool that contains the necessary components to build the FE model and generates the input files for VecTor2. A complete description of the software can be found elsewhere (Wong, Vecchio and Trommels, 2013). Augustus, developed by Bentz (1996), provides the graphical output of the results, including element stresses and strains, various load-deformation responses, cracking patterns, deformation states, and other structural responses.

3.3.3 Modelling Methodology

In this study, finite element models were developed to analyze the viability of the repairing options previously described. The modelling methodology follows a chronological order. First, FE models were developed to simulate and validate the damage experienced in the walls prior to repairing. Second, additional FE models are developed to simulate the repairing methodologies considered, utilizing as basis the numerical models of the original walls.

The development of the FE models initiates by defining the constitutive models to represent the materials and their mechanical properties that formed the numerical analysis of the walls. Thereafter, the mesh is subdivided into zones to differentiate changes in geometry and material properties, followed by discretization. The models of the original walls were generated and analyzed under the same loading protocol used during testing by Morcos and Palermo (2019) as means to corroborate the models. Thereafter, FE models of the repaired walls were employed to simulate the repair methodologies discussed in Section 3.2. Each repair strategy was analyzed to assess the most viable option.

3.3.4 Concrete Constitutive Models

The default constitutive models of VecTor 2, where possible, were selected to capture a satisfactory response. However, due to the selection of the repair materials (UHPC and ECC), two different approaches were considered to simulate the behaviour of the concretes used in the original and repaired walls. Table 3.4 lists the constitutive models selected for the FE models.

Table 3.4 Concrete constitutive models employed in the analyses with VecTor2

Mechanical behavior	Constitutive model
Compression Pre-Peak	Hognestad / Popovics High-Strength *
Compression Post-Peak	Modified Park-Kent
Compression Softening	Vecchio 1992-A (e1/e2-Form)
Tension Stiffening	Modified Bentz 2003 / Not considered *
Tension Softening	Bilinear / Custom Input*
FRC Tension	Not Consider*
Confined Strength	Kupfer - Richart
Dilation	Variable - Orthotropic
Cracking Criterion	Mohr-Coulomb (Stress)
Crack Stress Calculation	Basic (DSFM/MCFT)
Crack Width Check	Agg/2.5 Max Crack Width
Crack Slip Calculation	Walraven
Creep and Relaxation	Not Considered
Hysteretic Response	Palermo 2002 (w/Decay)*

*Represents non-default model

In the analysis of the original walls, the Hognestad model was selected for the compression pre-peak response of the concrete (Hognestad, 1951); it is a simple parabola-shaped response, suitable for concrete with compressive strength under 50 MPa. The Modified Park-Kent default model was selected as the compression post-peak response (Scott, Park and Priestley, 1982). The Vecchio 1992-A (e1/e2-Form) model was employed to simulate the compression softening effects, considering the decrease of compression strength and stiffness (Vecchio, 1992) due to the presence of tensile strains in the transverse direction. After cracking, the concrete tensile stresses diminish virtually to zero at the free surface of cracks. However, owing to bond action with the reinforcement, average tensile stresses continue to exist in the concrete between cracks; this mechanism was captured with the tension stiffness model: Modified Bentz 2003 (Bentz, 2003). For plain concrete regions or in zones where the reinforcement ratio is low, the tension softening may govern for the post-cracking response of the concrete, and the CEB-FIP (bilinear) model (Telford, 1990) was used. It is able to capture the brittle fracture of concrete, with a steep but finite gradient to capture the decay of strength with crack development and propagation. The Kupfer-Richart model was selected to account for the enhancement in the confinement strength in a triaxially compressed concrete (Kupfer H, Hilsdorf H and Rusch H, 1969). For the cracking criterion, the Mohr-Coulomb stress model is used. The crack slip was represented by the Walraven model (Walraven, 1981). The concrete crack width check serves to reduce the average compressive stresses in the element when the crack width exceeds a specified

limit, a limit of 25% of the aggregate size was used. Finally, the degradation of strength and stiffness in the cyclic response of the concrete was captured with the Palermo model (Palermo, 2002). Detailed information on the models is available elsewhere (Vecchio, Wong and Tamm, 2013).

The concrete material properties for the analyses conducted herein were adopted from the experimental testing of the original walls (SWS-R and SWN) reported by Morcos and Palermo (2019) and listed in Table 3.1. Wall SWS-R had a concrete compressive strength of 50.76 MPa, while the reported strength for Wall SWN was 39.28 MPa. The concrete in both walls contained a maximum aggregate size of 14 mm.

For the repaired walls, the non-default models presented in Table 3.4 were selected in addition to the default models previously discussed. The compression pre-peak response was simulated with the Popovics High-Strength concrete model (Popovics, 1973); it encompasses a variation of the original Popovics curve that was modified to better reflect the response of concretes with a compressive strength higher than 50 MPa. Due to the complexity of the matrix composition of HPFRC, including different types of fibres and the variety of mixtures, a user-defined model was employed. (Note that most FEM platforms do not contain built-in constitutive models to capture the tensile behaviour of this type of material). The approach taken with VecTor2 involved a custom-input, multi-linear response to define the tensile stress-strain behaviour of the respective HPFRC utilized.

Saikali (2019) proposed a methodology to represent HPFRC using the available constitutive models in VecTor2. The mechanical properties such as compressive strength and corresponding strain, and Young Modulus were input as known parameters to capture the behaviour of the material in compression (Figure 3.14 (a)). To represent the response of the material in tension, an iterative, custom input of the existing tension softening model in VecTor2, as shown in Figure 3.14 (b), was used. The f_{cr} and ϵ_{cr} represents the first crack of concrete through a linear ascending branch, which marks the initiation of the pseudo-strain-hardening tensile behaviour (Van Zijl *et al.*, 2012) characterized by multi-cracking. $(f_c, \epsilon_c)_1$ captures the peak tensile stress and corresponding strain and the end of the multi-cracking stage, chosen as (10 MPa, 3 me) in this case. Points $(f_c, \epsilon_c)_2$, $(f_c, \epsilon_c)_3$ and $(f_c, \epsilon_c)_4$ capture the crack localization phase with a strain-softening behaviour through a trilinear descending branch taken as (9 MPa, 10 me), (5 MPa, 225 me) and (1.75 MPa, 325 me), respectively.

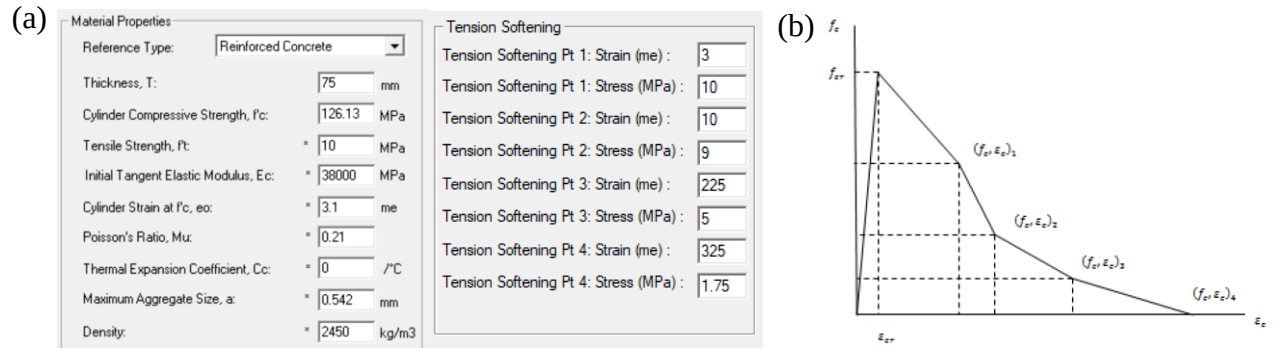


Figure 3.14 Material properties for UHPFRC: (a) input material properties for compression response; and (b) input values representing the custom-input-strain based tension softening in Vector2 (Saikali (2019) and Wang (2003))

The final input values used in the custom tension-softening model were fixed once the load-displacement response of two flexural prisms denoted herein as P1 (75 mm x 75 mm x 280 mm) and P2 (75 mm x 75 mm x 500 mm) were satisfactorily simulated as presented in Figure 3.15 (a) and (b), respectively, based on the work of Saikali (2019). These values were chosen to represent the UHPFRC in the preliminary numerical analyses of the repaired models.

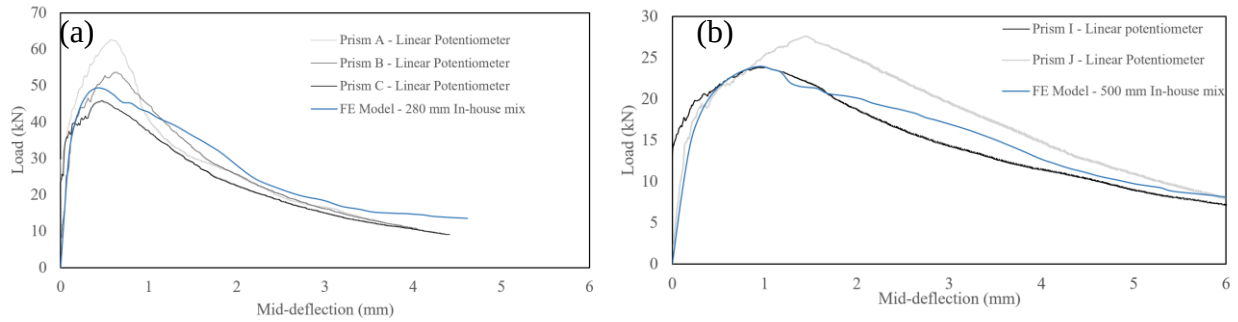


Figure 3.15 Experimental and numerical responses: (a) 280 mm long-prisms (P1); and (b) 500 mm-long prisms (P2) (Saikali, 2019)

Eshghi (2018) developed an ECC mix with PVA fibres that resulted in compressive strength of 60 MPa and tensile strength of approximately 6 MPa (presented in Section 2.4.3.1). Moreover, the researcher conducted four-point bending tests on the flexural prisms P1 and P2 to extract the flexural behaviour of the concrete. However, Eshghi (2018) did not represent them in Vector2. Thus, the research herein followed the same procedure as Saikali (2019) to obtain the numerical material properties to simulate the load-displacement responses of prisms P1 and P2 tested by Eshghi (2018). Figure 3.16 (a) provides the material properties utilized to simulate the behavior of the prisms tested

under four-point bending by Eshghi (2018). The experimental and numerical responses are presented in Figure 3.16 (b) and (c).

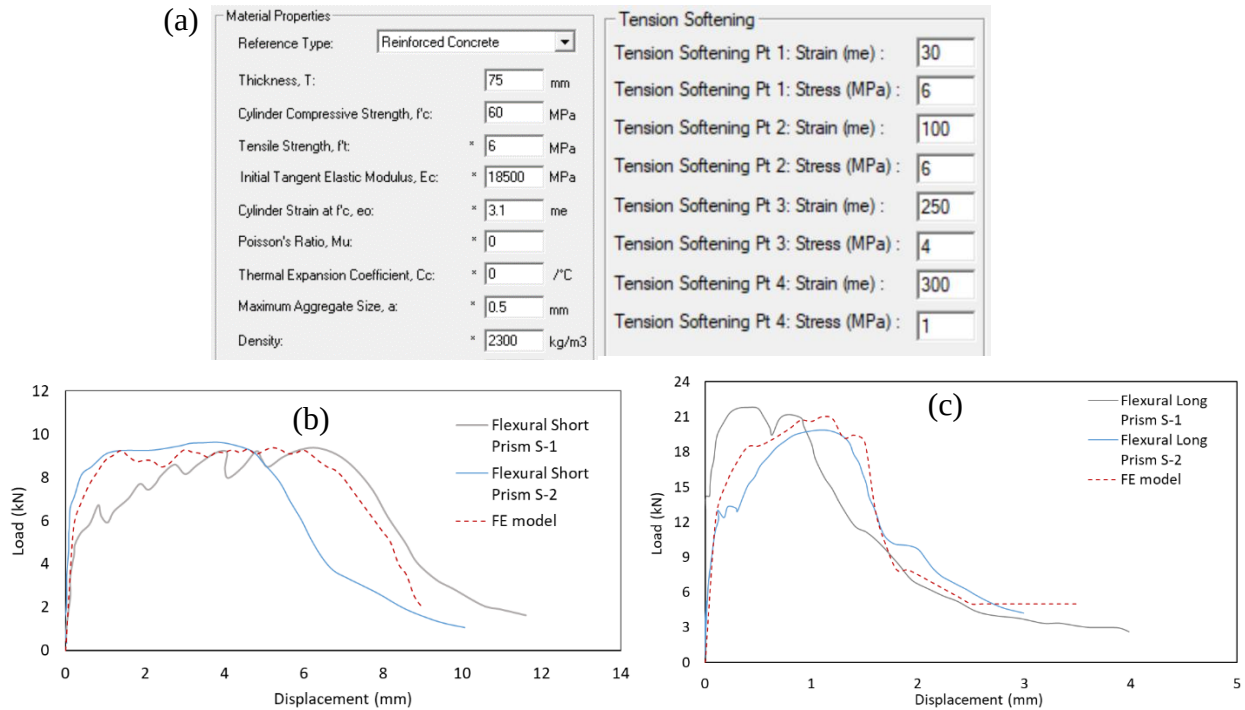


Figure 3.16 FE modelling of ECC prisms: (a) material properties for ECC used in VecTor2; (b) experimental and numerical responses for 280 mm long-prisms (P1); and (c) experimental and numerical responses for 500 mm-long prism (P2)

The material properties depicted in Figure 3.16 (a) were employed in the preliminary numerical models of the walls to represent the ECC material that replaced the concrete removed in each of the repair options.

3.3.5 Reinforcement Constitutive Models

The backbone envelope for the reinforcing steel was based on a default response that includes a linear elastic region, a yield region and a nonlinear strain hardening zone. The steel hysteresis response was captured with the Seckin model (Seckin, 1981). The default Tassios model was selected for dowel action effects. Furthermore, buckling of the reinforcing bars was not considered in the analysis. The Eligehausen model (Eligehausen, Popov and Bertero, 1982) was selected to simulate the bond-slip response between the concrete and reinforcement. Table 3.5 lists the constitutive reinforcement models selected for the analysis with VecTor2.

Table 3.5 Reinforcement constitutive models employed in VecTor2

Mechanical behavior	Constitutive model
Hysteric Response	Bauschinger Effect (Seckin)
Dowel Action	Tassios (Crack Slip)
Buckling	Not considerer
Bond	Eligehausen

The behaviour of the SE-SMA follows a built-in model in VecTor2 based on the work of Abdulridha et al. (2013), as illustrated in Figure 3.17.

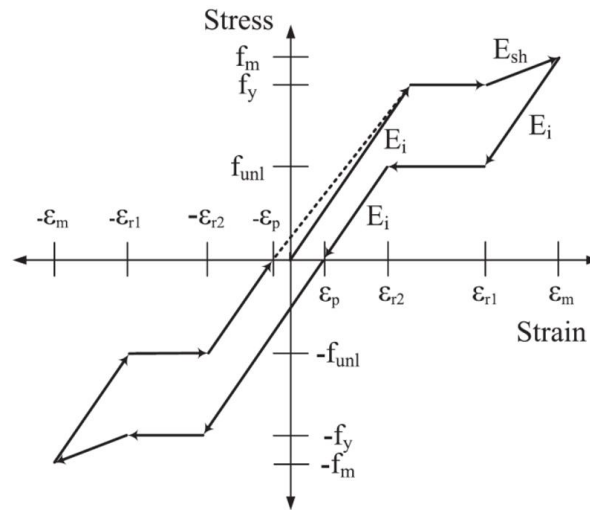


Figure 3.17 SMA plastic offset constitutive model (Abdulridha and Palermo, 2013)

The model is formed by a series of multi-linear branches representing the hysteresis of SE-Nitinol while capturing the self-recovery phenomenon and plastic deformations that accumulate beyond 6% of strain. The loading phase of the model embodies the transformation from Austenite to Martensite. In the unloading segment, the model simulates the re-transformation to Martensite, capturing the superelasticity.

The reinforcement properties employed in this chapter are taken from the manufacturer. (Note that the updated properties obtained from material testing are utilized later on in the numerical study presented in Chapter 7). The steel reinforcement within Wall SWS-R included 15M bars ($\varnothing = 16$ mm, $f_y = 472.5$ MPa, and $f_u = 578$ MPa) in the foundation block and 10M bars ($\varnothing = 11.3$ mm, $f_y = 454.4$ MPa, and $f_u = 570$ MPa) for the wall and the cap beam. Wall SWN utilized the same 15M bars for the foundation block. The steel reinforcement for the wall and cap beam was sourced from a different

supplier, including 10M bars ($\varnothing = 11.3$ mm, $f_y = 529.4$ MPa, and $f_u = 686.5$ MPa) and #13 bars ($\varnothing = 12.7$ mm, $f_y = 474.5$ MPa, and $f_u = 639.5$ MPa). Note that the #13 bars were connected via couplers to the SMA bars. This reinforcement was used by the supplier of the couplers used in the original walls, where both the #13 and SMA bars were forged to accommodate the coupler. The properties of the SE-SMA bars reported by the manufacturer were: $\varnothing = 12.7$ mm, $f_y = 380$ MPa and $f_u = 900$ MPa. Finally, the mechanical couplers had the following properties: $\varnothing = 35$ mm, $f_y = 690$ MPa and $f_u = 795$ MPa. The location of the reinforcement within the FE models replicated that used in the construction of the original walls, as illustrated in Figure 3.3 for Wall SWS-R and Figure 3.2 for Wall SWN. Furthermore, the new segments of 10M bars used as starter bars and to replace the damaged bars, for the purposes of the preliminary analysis of the repaired FE models, were assumed to have typical values of: $\varnothing = 11.3$ mm, $f_y = 420$ MPa, and $f_u = 580$ MPa (Rai, Jain and Chakrabarti, 2012), while the properties of the mechanical couplers reported by the manufacturer were: $\varnothing = 33$ mm, $f_y = 620$ MPa and $f_u = 760$ MPa.

3.3.6 Definition of Regions for the FE Model Formulation

A total of 9 different regions were defined in the FE models. Figure 3.18 provides a graphic representation of the subdivisions defined in the walls to define the regions.

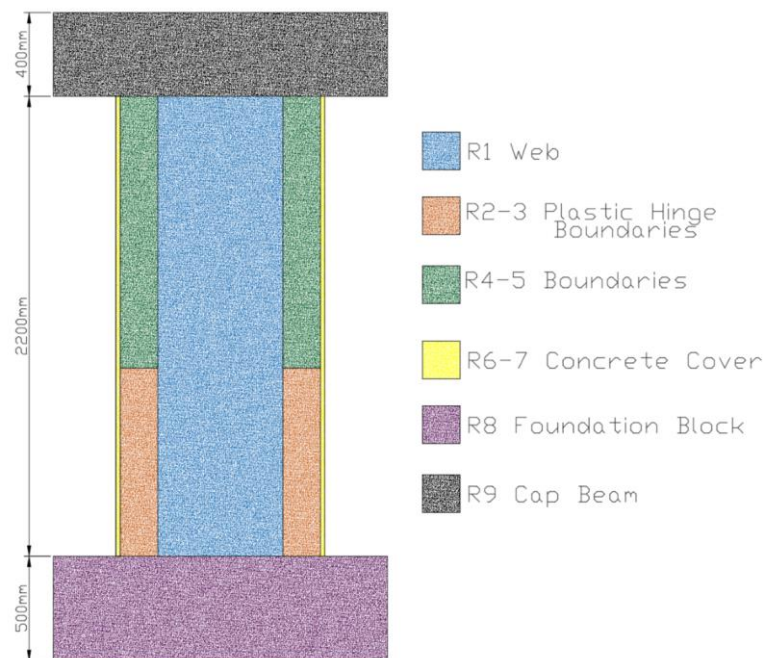


Figure 3.18 Defined regions for FE models

The subdivisions were defined according to the thickness of the components, and the smeared reinforcement ratios of the shear reinforcement and anti-buckling ties. Furthermore, the subdivisions considered the zones from which the concrete was removed as part of the repairing methodology.

3.3.7 Original Walls - FE Models

Figure 3.19 depicts the FE models generated for the original walls. A sensitivity analysis was performed to determine the size of the elements. The mesh size maintained an optimal aspect ratio of 1:1 covering most of the regions of the original walls with rectangular elements of 50 mm by 50 mm. Due to geometric constraints, the concrete cover was the only region that required an aspect ratio of 2.5, with 20 mm by 50 mm elements. The description of the models for the repaired walls is provided in Section 3.3.9.

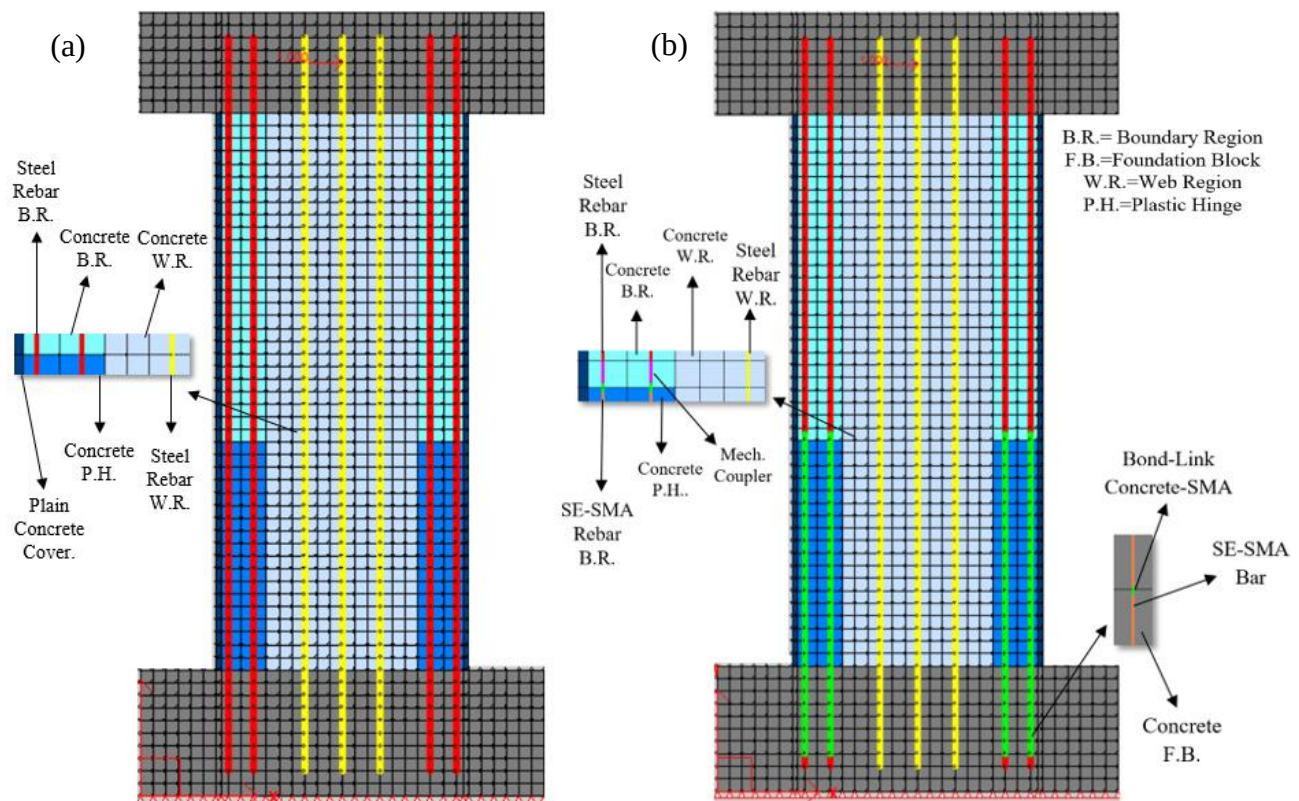


Figure 3.19 FE models: (a) SWS-R; and (b) SWN

For both models, the shear reinforcement and buckling-prevention ties were smeared within the concrete elements. In the web region, the transverse reinforcement ratio is 0.89%. Within the boundary regions in the plastic hinge zone, the reinforcement ratio in the transverse direction is 2.67%, and the out-of-plane reinforcement ratio is 1.85%, which accounts for shear reinforcement and buckling-prevention ties. Note that the legs of the ties in the horizontal direction contribute to the shear reinforcement, while the legs of the ties in the out-of-plane direction provide confinement. The top boundary zones, above the plastic hinge region, have reinforcement ratios of 1.78% and 1.11% in the transverse and out-of-plane direction, respectively. The foundation block and the cap beam were assumed as stiff elements since no damage is expected in this region; a reinforcement ratio of 3% was assigned for both transverse and out-of-plane direction. The cover regions were assumed to be plain concrete.

The longitudinal reinforcing steel for both walls and SE-SMA bars in SWN were modelled as discrete reinforcement, with deformed and smooth truss-bar elements, respectively. The deformed-mild steel was assumed fully bonded to the concrete. Bond link elements were used in Wall SWN to represent the interaction between the SE-SMA bars in the plastic hinge with the concrete elements. The mechanical couplers inside the plastic hinge region were modelled with 50 mm-long smooth truss-bar elements that were considered fully bonded to the concrete.

3.3.8 FE Models Validation and Analysis

The FE models in their pre- and post-repaired states were subjected to the same loading protocol as imposed on the original walls during testing and presented in Section 4.4.3. The numerical results are assessed against the experimental results reported by Morcos and Palermo (2019) for the original walls. Figure 3.20 (a) and (c) depicts the load-displacement responses predicted by the FE models of Wall SWS-R and SWN, and Figure 3.20 (b) and (d) provide the respective experimental responses of the original walls.

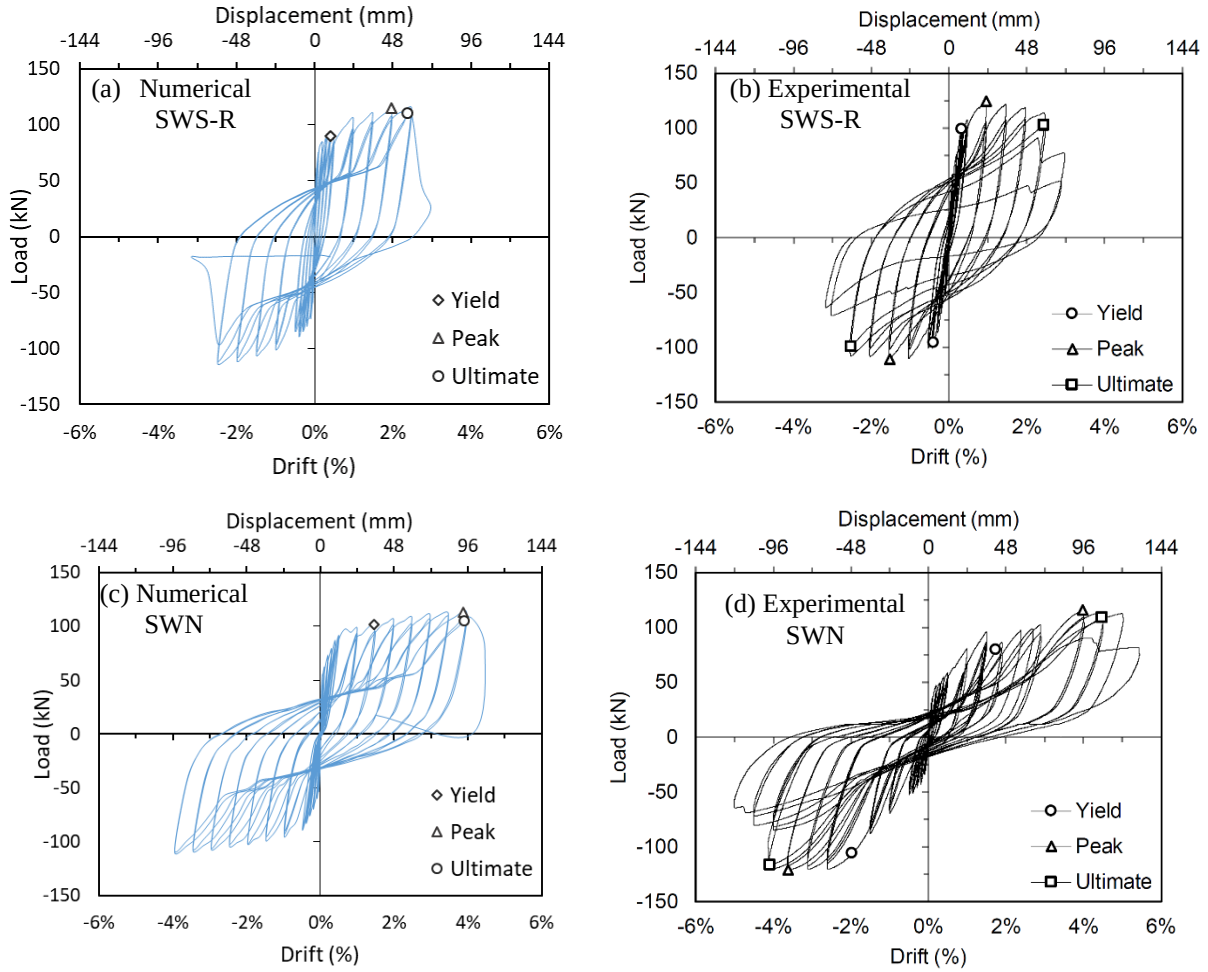


Figure 3.20 Comparison of load-displacement responses: (a) numerical SWS-R; (b) experimental SWS-R; (c) numerical SWN; and (d) experimental SWN (Morcos and Palermo, 2018)

The yield, peak and ultimate points predicted by the FE models are identified in Figure 3.20 and correspond to the positive direction of loading using the reduced stiffness, equivalent elastoplastic method proposed by Park (1988). Note that the ultimate point is based on a 20% reduction in the lateral load capacity or the displacement cycle prior to fracturing of the reinforcement. In this study, for the 20% reduction, at least one full cycle was required. The numerical response of SWS-R in Figure 3.20 (a) experienced a yield load of 89.4 kN at a displacement of 9.5 mm, which represents a 7.8% and 9% difference, respectively, compared to the experimental results reported for Wall SWS-R (95 kN at 8.7 mm displacement). The numerical peak load attained for SWS-R was 114.6 kN, with a corresponding displacement of 45.3 mm. A similar peak load of 118 kN was reported for Wall SWS-R at a displacement of 30 mm. The difference in displacements is attributed to the gradual post-yield stiffness predicted in the model. Furthermore, the numerical model attained an ultimate load of

110.47 kN, with a displacement of 57 mm. The observed ultimate load during experimental testing was 101 kN at a displacement of 60 mm.

The numerical model for Wall SWN experienced yielding at 101.3 kN and 34.7 mm displacement, which are comparable to the reported experimental values 93 kN of load at 45 mm of displacement. The FE model reached a peak load of 112.7 kN with 93 mm displacement, which represents a negligible difference of 5.3% and 2.4 %, respectively, in comparison to the experimental values (118 kN, 91 mm). Finally, the numerical model for SWN reached the ultimate load (105.78 kN) at a displacement of 94 mm, while the observed values for Wall SWN were 113 kN at 103 mm displacement. Both models predicted extensive flexural cracking near the base, with significantly more damage developing in Wall SWS-R up to a height of 700 mm from the base of the wall. Figure 3.21 illustrates the cracking pattern of the FE models at 2.5% drift.

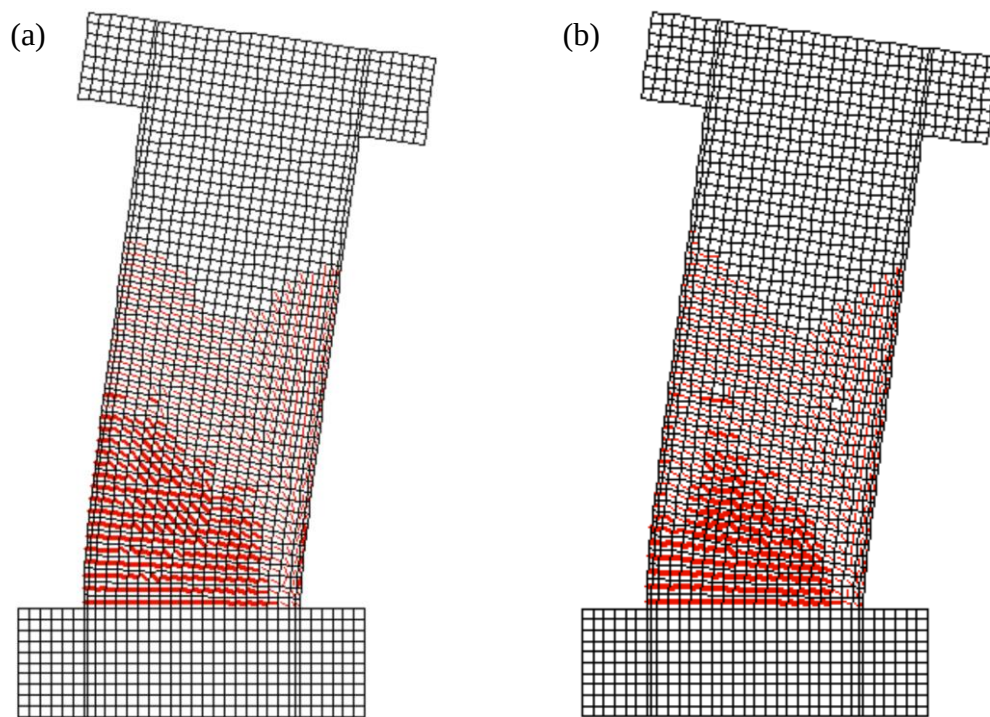


Figure 3.21 Cracking pattern at 2.5% drift predicted by the numerical models: (a) SWS-R; and (b) SWN

In terms of failure modes, the numerical models were able to replicate that reported from the experimental testing. Figure 3.22 illustrates the stress levels experienced by the longitudinal reinforcing from the numerical models at peak load.

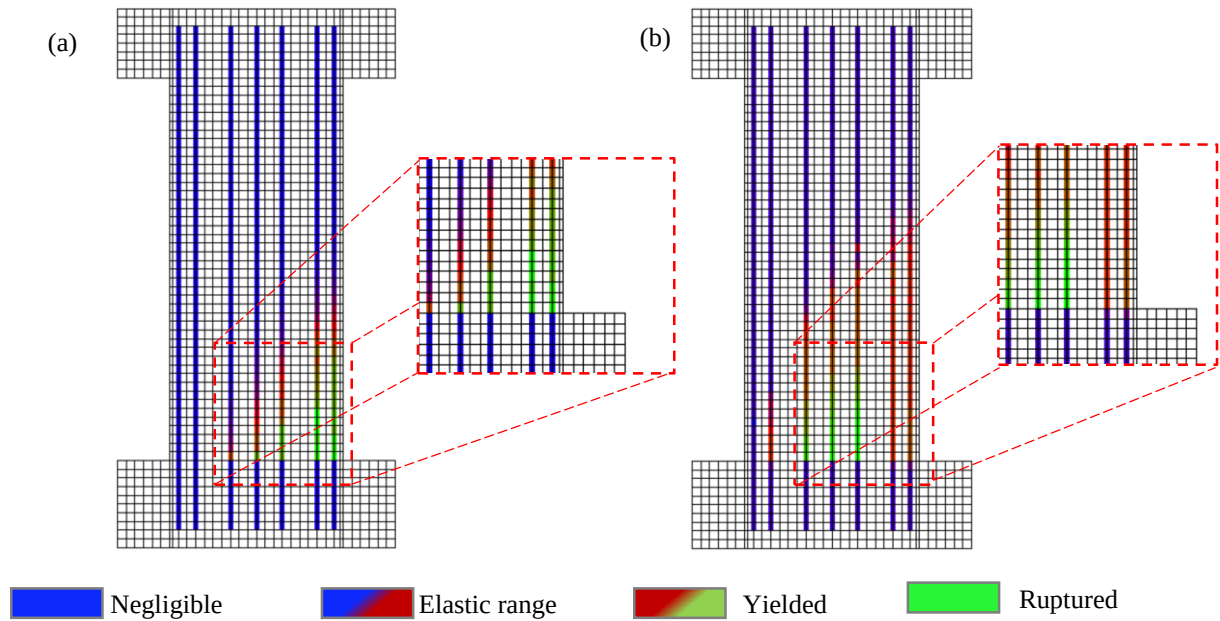


Figure 3.22 Stress levels in longitudinal reinforcing bars at peak load: (a) SWS-R; and (b) SWN

The numerical model for Wall SWS-R experienced failure due to the rupturing of the longitudinal reinforcing bars in the boundary regions at the base of the wall, as depicted in Figure 3.22 (a), which was consistent with the test results. Likewise, the numerical model for Wall SWN experienced rupturing of the longitudinal steel reinforcing bars in the web region with no evident damage to the SE-SMA bars in the boundary zones, as shown in Figure 3.22 (b). The damage to the reinforcement predicted by the numerical model for Wall SWN agreed with that reported during testing.

The overall behaviour provided by the numerical models for the original walls was assumed satisfactory; performance points, cracking patterns, and failure modes were all considered within the ranges of acceptable results.

3.3.9 Repaired Walls - FE Models

The longitudinal reinforcing bars experienced yielding in the numerical models of the original walls, 300 mm above the base of the wall up to approximately 500 mm, as shown in Figure 3.22. Therefore, the first repair option that considered removing the concrete corresponding to one-third of the plastic hinge, as shown in Figure 3.12 (a), was not further investigated, given that the longitudinal bars

experienced yielding above the proposed concrete repair zone. The other two repair options are further analyzed herein. Figure 3.23 provides graphic representations of the numerical models of the repaired wall RSWN, where the letter R indicates repaired. Note that similar models were developed for Wall RSWS-R.

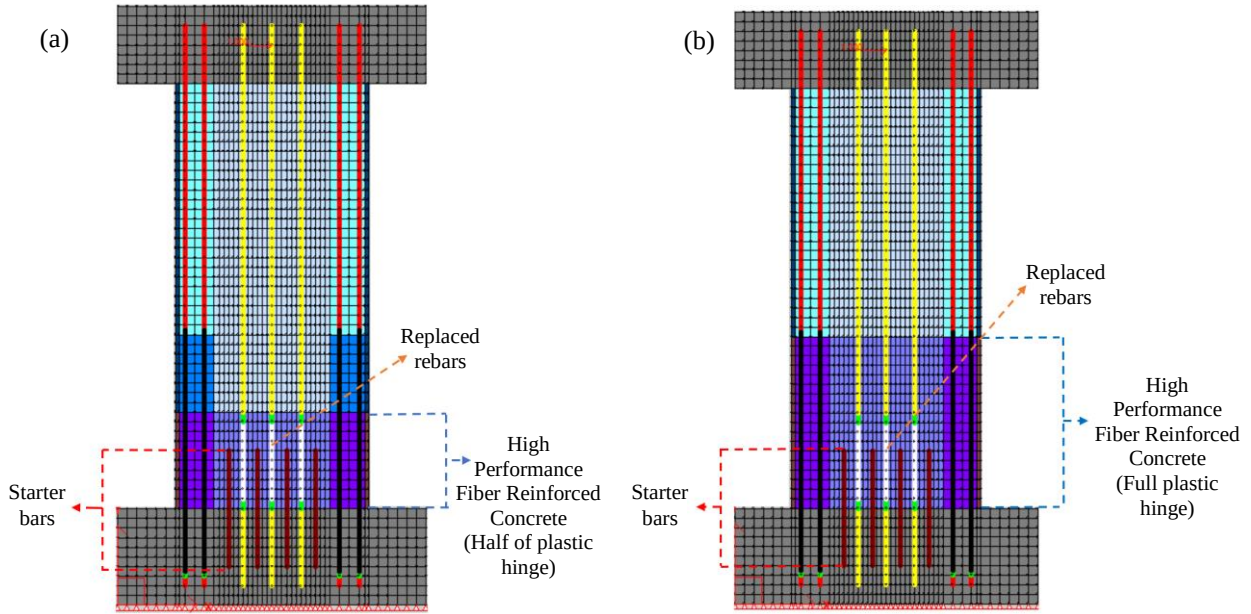


Figure 3.23 Numerical models for Wall RSWN: (a) one-half of plastic hinge removed; and (b) entire plastic hinge removed

The finite element models for the repaired walls WRE based on the FE models of the original walls presented previously in Section 3.3.7. The element size was reduced in the mesh discretization to accommodate the different sections of concrete that were removed and replaced with HPC. Additionally, the starter bars were added, located at the base of the wall, extending 300 mm into the plastic hinge and 200 mm into the foundation block and were simulated with deformed truss bar elements fully bonded to the concrete. The development length of the starter bars was considered by changing the area of the reinforcement over each truss bar segment. Thus, the last segment at the termination of the starter bars within the wall had the smallest area, increasing towards the base of the wall.

Analyses were performed on the two different repair options for RSWS-R and RSWN. The models were subjected to the same loading protocol as the original walls. The following sections provide a

rationale for the choice of the repair concrete, the effect of the addition of starter bars, and the extent of concrete removed. These results were used to select the most viable option to repair the walls in the experimental program.

3.3.9.1 Choice of HPFRC

Figure 3.24 illustrates the load-displacement responses based on the numerical models of RSWS-R and RSWN repaired by removing 500 mm of damaged concrete measured from the base of the wall, as presented in Figure 3.12 (b), with the addition of the reinforcement detailing shown in Figure 3.13 (c) and (d), respectively. Each set of repaired models incorporated either the ECC or the UHPFRC properties, and are compared against the numerical models of the original walls SWS-R and SWN.

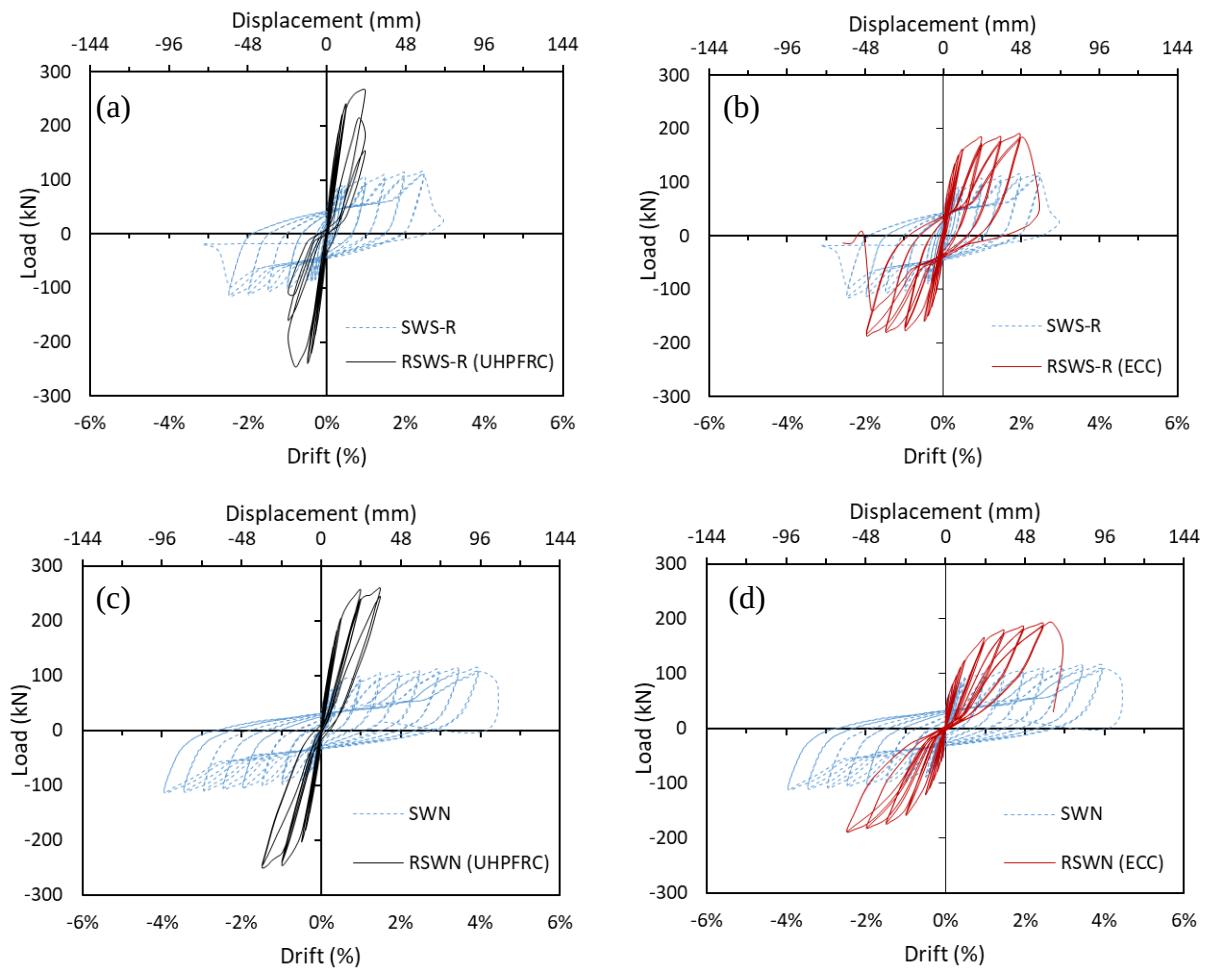


Figure 3.24 Load-displacement responses for models with half of the plastic hinge replaced: (a) RSWS-R with UHPFRC; (b) RSWS-R with ECC; (c) RSWN with UHPFRC; and (d) RSWN with ECC

The higher strength and stiffness properties of the UHPFRC mix resulted in a more stiff and brittle behavior for the numerical models of the repaired walls. Wall RSWS-R presented in Figure 3.24 (a) reached a peak load of 265.4 kN at a displacement of 23.67 mm (1% drift). Likewise, the repaired Wall RSWN with UHPFRC (Figure 3.24 (c)), attained a similar peak load of 258.6 kN at 35.54 mm (1.5% drift) of displacement. However, the numerical models of the repaired walls employing ECC developed a more ductile behaviour, experiencing a lower load but higher displacement capacity. RSWS-R with ECC (Figure 3.24 (b)) sustained a load of 188.1 kN with a displacement of 47.16 mm (2% drift); while the numerical model for RSWN utilizing ECC (Figure 3.24 (d)) developed a peak load of 190.9 kN at a displacement of 65.2 mm (2.7% drift).

In comparing the responses of the numerical models for the repaired and original walls, it becomes evident that the ECC repair concrete is a suitable option for this study. The ECC repaired wall is capable of developing larger lateral displacements, which enables the superelastic characteristic of the SMA materials to be exploited in the overall behaviour of the wall. This becomes evident in the pinching effect experienced by the repaired wall employing SE-SMA bars and ECC (Figure 3.24 (d)).

3.3.9.2 Effect of the Addition of Starter Bars

To study the effect of the addition of the starter bars, the numerical model for RSWN using ECC, with half of the plastic hinge zone replaced (Figure 3.12 (b)), was further analyzed by removing the starter bars and comparing to the same model containing the full repair strategy. Figure 3.25 shows the cracking pattern at failure with and without starter bars.

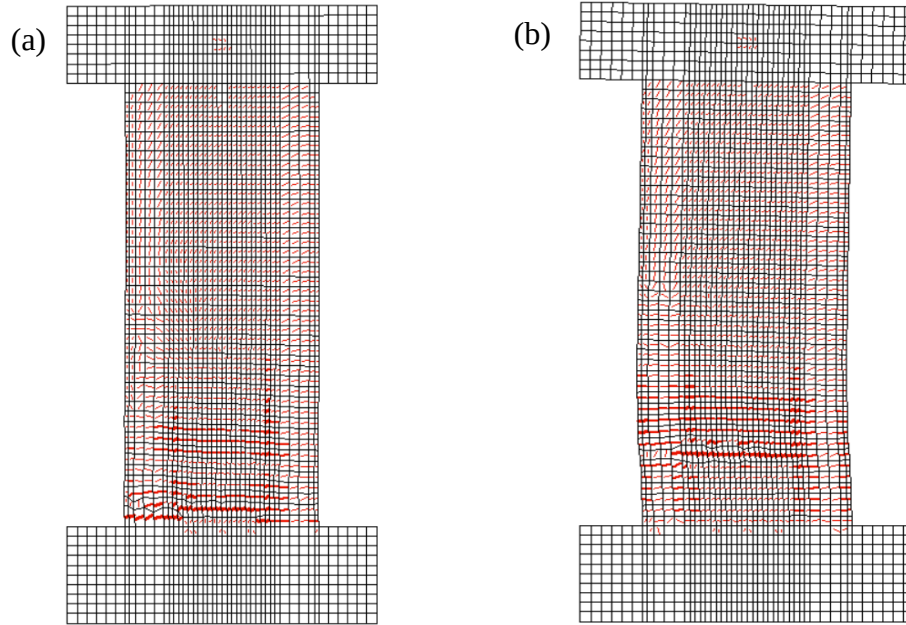


Figure 3.25 Cracking pattern of numerical models for Wall RSWN: (a) without starter bars; and (b) with the addition of starter bars

Wall RSWN with no starter bars depicted in Figure 3.25 (a) sustained significantly wide flexural cracking just above the couplers located at the base of the wall. The model failed in this region by rupturing of the steel reinforcement in the web zone. Conversely, the model of RSWN using starter bars resulted in a shift in the location of failure. As observed in Figure 3.25 (b), the damage surfaced in the model at 300 mm above the base of the wall, corresponding to the termination of the starter bars. Note that the starter bars had a negligible effect on the flexural strength of the wall and succeeded in eliminating the rocking/sliding that was present in the original walls. Therefore, the addition of the starter bars was deemed necessary in the repair of the walls in the experimental program to ensure the full capacity of the ECC and SMA bars would be utilized.

3.3.9.3 *Extent of Concrete Removal*

The extent of concrete to be removed for the repair of the walls in the experimental program was based on a comparison established between the numerical models. The load-displacement response of the models for RSWN and RSWS-R repaired by replacing the entire plastic hinge zone with ECC, and the addition of starter bars (Figure 3.23 (b)) is presented in Figure 3.26, superimposed over the numerical results of the original walls.

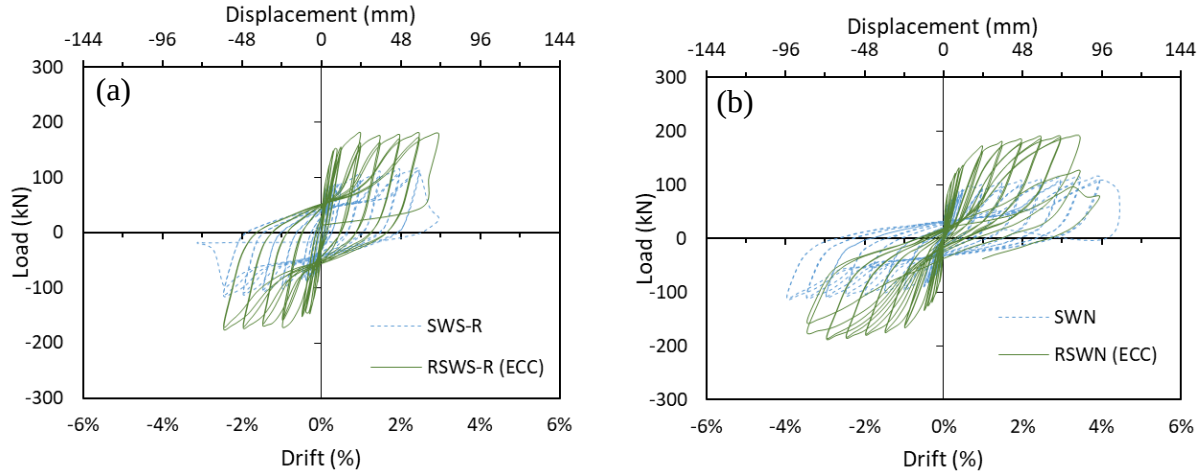


Figure 3.26 Load-displacement responses of repaired numerical models with replacement of the entire plastic hinge zone: (a) conventional steel wall; and (b) SMA wall

The numerical repaired models with the replacement of the entire plastic hinge zone were able to sustain similar displacements as the original walls, with augmented lateral load capacity. When compared to the numerical response of the original walls, it is evident that the simulated tensile capacities of the ECC input in the repaired FE models generated a stiffer behavior with adequate levels of deformation. Moreover, the repaired models under this approach exhibited a similar peak load with a higher displacement prior to failure when compared to the additional repaired models with half of the plastic hinge removed, as showed in Figure 3.24 (b) and (d). The results presented in Figure 3.26 evidenced that the addition of ECC in an area where plastic deformation previously occurred prevented the model from suffering extensive damage in critical regions that affected the overall behavior of the walls. This was translated in higher lateral displacements prior to reaching performance points of yield, peak and ultimate loads. Thus, it was determined to adopt the removal of the entire plastic hinge region in the experimental program presented in the following chapter.

CHAPTER 4. EXPERIMENTAL PROGRAM

This chapter discusses the execution of the experimental program with a step-by-step approach, comprising all the stages to perform the experimental testing of the walls.

4.1 Material Characterization Tests

The material characterization tests procedures are presented in this section and comply with the guidelines associated with the American Society for Testing and Materials (ASTM). The results are presented in Chapter 6.

4.1.1 Flow Test

The flowability test of the fresh ECC mixture follows the requirements specified by the Standard Specification for Fabricating and Testing Specimens of Ultra-High Performance Concrete (ASTM C1856, 2017), in which a flow table in compliance with ASTM C230/C230M (2014) is utilized. The flow table supports a mold which is filled in a single layer of fresh concrete. Thereafter, the mold is removed, allowing the fresh mixture to rest and expand for two minutes. Once this is complete, the diameter of the ECC mixture is measured along the lines of maximum and minimum diameter, from which the average flow value is determined.

4.1.2 Uniaxial Compression Test

A compression testing machine (Controls Pilot) is used to perform uniaxial compression tests on 150 mm-long, 75 mm-diameter concrete cylinders to evaluate the compressive strength (ASTM C1856, 2017). Prior to testing, the cylinders were grinded on both faces to produce a smooth surface and to ensure a uniform stress distribution during the test. Furthermore, three different values of length and diameter were measured to determine the average dimensions for each specimen prior to testing. Figure 4.1 illustrates the uniaxial compression test setup prior to testing of the standard cylinder.



Figure 4.1 Uniaxial compression test setup

4.1.3 Flexural Beam Test

Four-point bending tests are conducted to obtain indirectly the tensile strength of the ECC. Ideally, the tensile strength can be extracted directly through a Direct Tension Test (DTT). However, significant challenges such as the gauge length and complications with the end grips generating additional stresses can compromise the results. Hence, flexural beam tests are adopted to obtain the flexural (tensile) strength by evaluating prisms under four-point bending, according to ASTM C1856 (2017).

Two sets of prisms with identical square cross-sections (75 mm by 75 mm) were cast with different shear span-to-depth ratio: $a/d = 1$ and $a/d = 2$ for Prism P1 and Prism P2, respectively. Figure 4.2 illustrates a simplified representation of the test setup, with the nominal dimensions, and the location of the supports and point loads on the set of prisms tested.

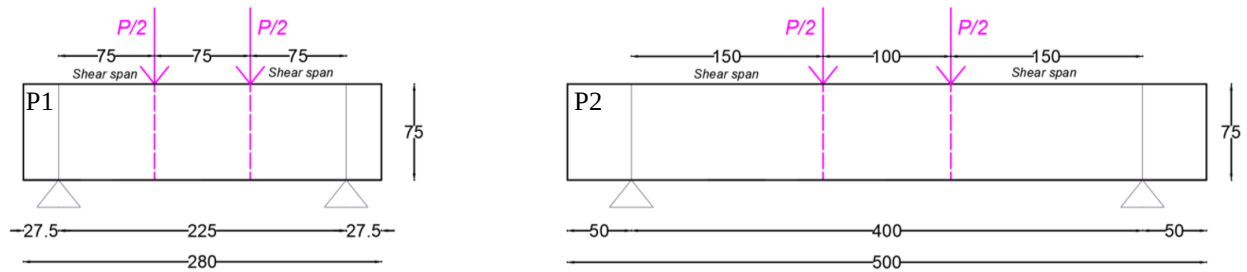


Figure 4.2 Prisms dimensions for four-point bending test (all units shown in millimeters) (Saikali, 2019)

The MTS universal testing machine is used with displacement control at a test rate of 0.005 mm/s to determine the load-displacement response of the prisms. The load is applied with bearing rollers attached to the MTS testing machine. The mid-span displacement is captured through a linear potentiometer attached to a rigid instrumentation frame that was pinned to the prisms, as depicted in Figure 4.3.

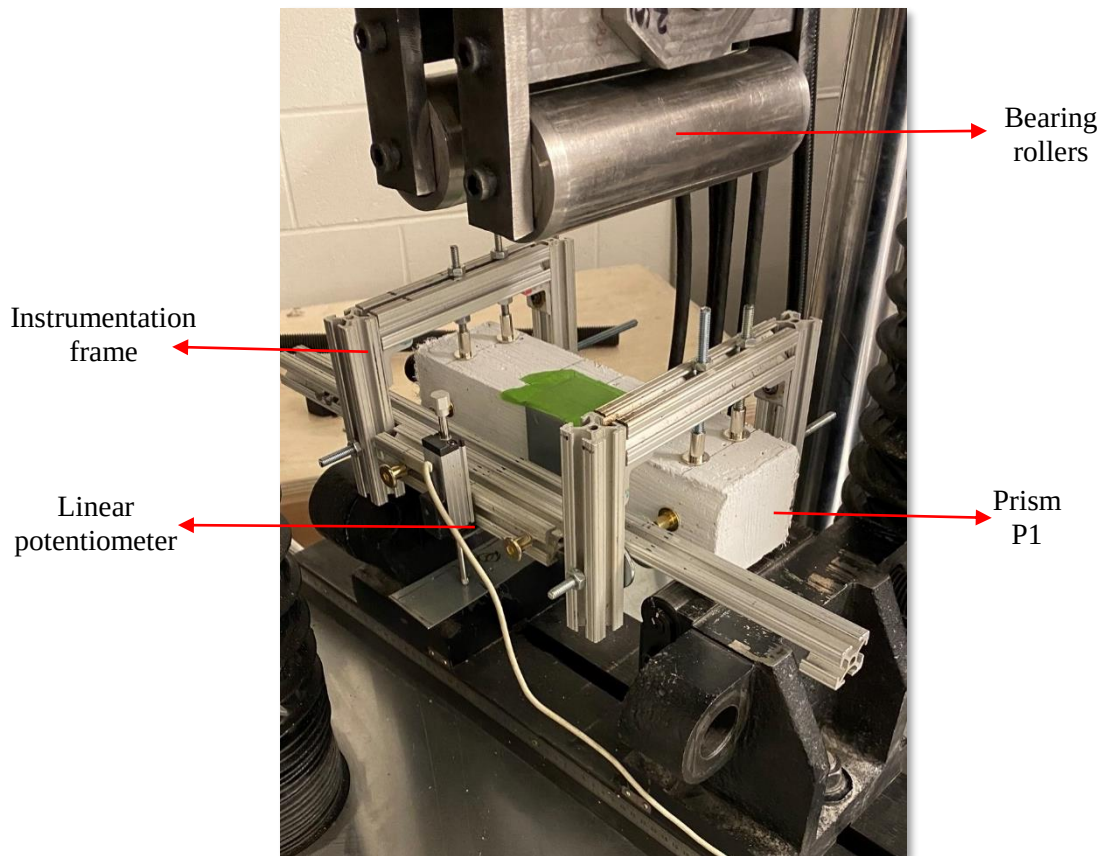


Figure 4.3 Four-point bending test on Prism P1

The flexural strength ($\sigma_{flexure}$) is calculated based on the response obtained from the prisms. Assuming a linear distribution of stress and strain, $\sigma_{flexure}$ is calculated from Equation 2.1 showed below:

$$\sigma_{flexure} = \frac{My}{I} = \frac{6M}{bh^2} = \frac{6M}{h^3} \quad [2.1]$$

Where,

$\sigma_{flexure}$: Flexural strength (MPa)

M : Moment at peak load (Nmm)

b : Width of the prism (mm)

h : Overall depth of the prism (mm)

4.1.4 Uniaxial Tensile Test

Coupons measuring 500 mm in length of the 10M steel reinforcing bars used to replace the damaged reinforcement in both walls were tested under direct tension according to the ASTM Standard E8/E8M (2013) to determine the stress-strain response under monotonic, uniaxial tensile loading.

Furthermore, based on the Standard Test Method for Tension Testing of Nickel-Titanium Superelastic Materials (ASTM F2516, 2014), cyclic, uniaxial tensile loading was applied to an additional 10M bar coupon to allow comparison with the response of the SE-SMA bars used in Wall SWN, tested by Morcos (2020) and presented in Figure 3.7.

Prior to testing, the cross-sectional area of each 10M coupon was reduced with a belt-sander to generate a localized, controlled failure at the mid-span of the bar. The strain was measured with an extensometer with a 2-in. (50 mm) gauge length attached to the coupon and connected to the MTS universal testing machine, as depicted in Figure 4.4. The results and analyses of the tests are presented in Chapter 5.

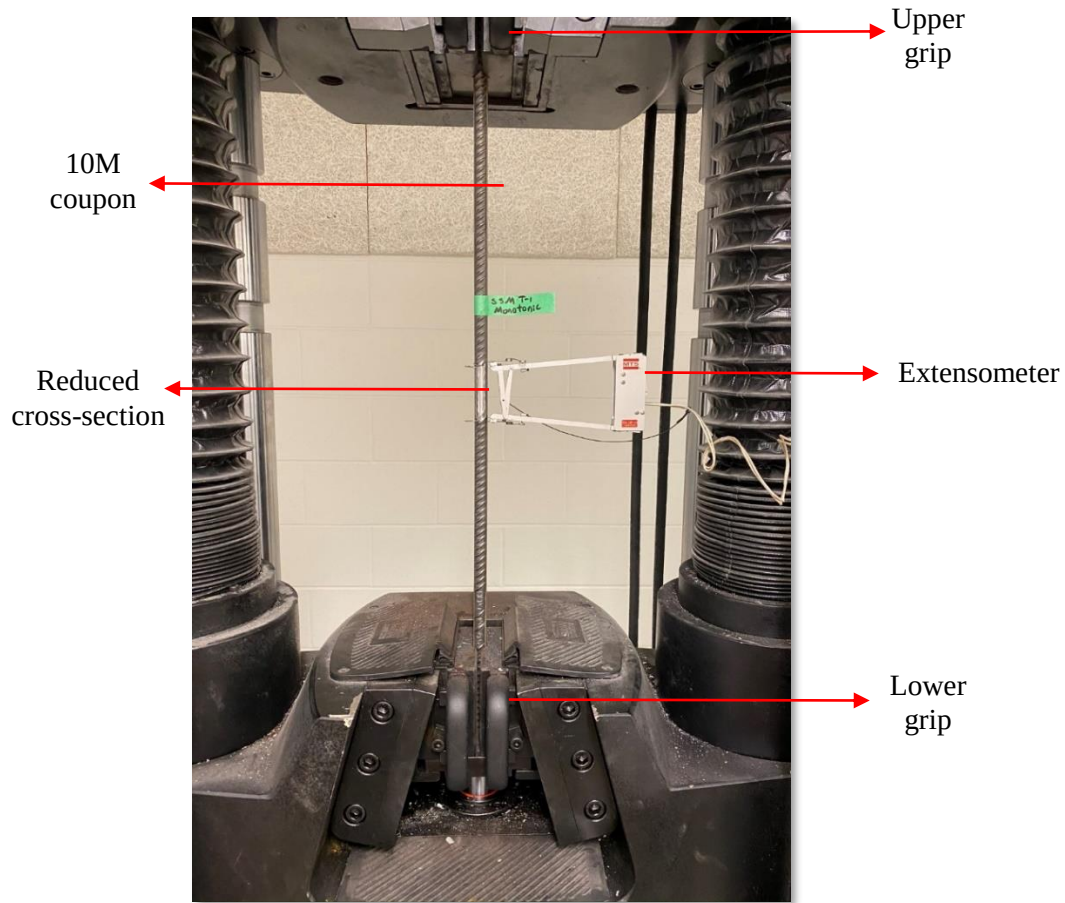


Figure 4.4 Uniaxial tensile test setup

4.2 Development of Engineering Cementitious Composite (ECC) Mixture

The preliminary numerical analysis determined that the Engineering Cementitious Composite (ECC) represents the most viable option as a primary repairing material for the damaged walls. In this section, a series of trials were evaluated to determine an in-house ECC mixture design, based on two previously developed mixes.

4.2.1 Materials

In general, the first mix design used a proprietary concrete commercial design mix denoted herein as Com K. The mix contained mainly a prepackaged HPC dry mix and two liquid admixtures. The Modified-Com K mix considered the addition of PVA fibres and Superplasticizer (SP). The second

mix design followed nearly identical proportions as the In-House mix developed by Eshghi (2018), which was previously presented in Table 2.2. The mix was adjusted by changing the superplasticizer employed and the addition of smaller diameter PVA fibres to obtain the desired workability.

As previously discussed, ECC is formed by mixing cement, pozzolans, inner materials, superplasticizer and PVA fibres. Table 4.1 lists the materials employed for ECC mix trials. Note that the materials utilized in Com K are not listed due to confidentiality.

Table 4.1 Materials used for ECC trials

Material	Picture	Properties	Supplier
Cement		Type GUL – General use Portland-Limestone Cement	CRH Company
Fly Ash		Type F	Lafarge Company
Silica Fume		MasterLife® SF 100 Densified Silica Fume Admixture	BASF Company
Slag		Ground Granulated Blast Furnace SLAG (GGBFS)	St. Mary's Cement

Sand		Bell-730 Max grain size: 0.3 mm	Bell & McKenzie
Synthetic Fibres		Kuralon RECS100 x 12 $\varnothing = 0.1$ mm Length = 12 mm	Kuraray Company
		Kuralon REC15 x 12 $\varnothing = 0.04$ mm Length = 12 mm	
Superplasticizer		MasterGlenium® 3400	BASF Company
		MasterGlenium® 7700	

4.2.2 Trials

A series of trials were prepared for the In-House and the Modified Com-K mix aiming for proper workability in the mixture based on ASTM C1856 (2017). The methodology to test for flowability is discussed in Section 4.1.1.

Table 4.2 presents the reference design proportions of Com-K with the Kuralon RECS100 x 12 ($\varnothing = 0.1$ mm) synthetic fibres included in concrete paste. All the dry materials were initially mixed for 3 minutes to obtain a full blend. The water was divided into two portions, where the first half

was placed with Admixture A to be added gradually to the dry mix. After 1 minute and 30 seconds of mixing, the second portion of the water was combined with ice and poured to be mixed for an additional 6 minutes and 30 seconds. The ice prevents overheating of the mix. Admixture B is then gradually added and mixed for approximately 5 minutes until reaching a liquid state. Lastly, the fibres are added. Owing to the hydrophilic nature of the PVA fibres, it was evident that a significant portion of the water was absorbed rather than participating in the hydration of the cementitious material. Thus, the low residual water resulted in poor workability.

Table 4.2 Com-K mix proportions with the addition of PVA fibres

Material	Amount	Flowability
Dry mix (Kg)	3.82	 Flow test: 140 mm
Admixture A (g)	115.92	
Admixture B (g)	80.24	
Water (g)	294.22	
Ice (g)	132.68	
PVA fibers (g)	50.00	
Ø = 0.1 mm		
Volume (L)	2.00	

A proposed solution was the addition of the MasterGlenium® 3400 superplasticizer with the intention to increase the flowability. The chemical admixture was included after the addition of PVA fibres to be mixed for 3 minutes and with continuous flow testing. The trials with the Modified Com-K mixture involved the addition of superplasticizer by 2 grams with each 2-liter mix trial, until reaching the proper flowability. The final modified Com-K proportions are presented in Table 4.3.

Table 4.3 Modified-Com K mix proportions

Material	Amount	Flowability
Dry mix (Kg)	3.82	 Flow test: 225 mm
Admixture A (g)	115.92	
Admixture B (g)	80.24	
Water (g)	294.22	
Ice (g)	132.68	
Superplasticizer (g)	20.00	
PVA fibers (g)	50.00	
Ø = 0.1 mm		
Volume (L)	2.00	

The In-House design mixing proportions and procedures were identical to the reference mixture, as discussed in Section 2.4.3.1 (Eshghi 2018). For this study, the PVA fibres utilized in the original In-House mix design ($\varnothing = 0.1$ mm) were reduced by 50% of their weight. The same amount was then substituted by smaller diameter fibres ($\varnothing = 0.04$ mm). The objective was to provide a higher concentration of fibres in the mix, maintaining the same volume, leading to a larger tensile strain-hardening plateau of ECC prior to failure, which translates to higher ductility. However, the flowability of the mixture did not adhere to the requirements of ASTM C1856 (2017). To address the workability challenges, MasterGlenium® 3400 superplasticizer was used in place of MasterGlenium® 7700, since it was proven to provide a higher level of workability in previous trials. As depicted in Table 4.4, following the proposed modification to the superplasticizer, an adequate flow test was obtained.

Table 4.4 In-house mix proportions

Material	Amount	Flowability
Cement (Kg)	0.80	
Fly Ash (Kg)	1.33	
Silica Fume (g)	220.00	
Slag (g)	90.00	
Sand (g)	890.00	
Water (g)	622.00	
Superplasticizer (g)	20.00	
PVA fibers (g) $\varnothing = 0.04$ mm	25.00	
PVA fibers (g) $\varnothing = 0.1$ mm	25.00	
Volume (L)	2.00	
		Flow test: 221 mm

4.2.3 Mechanical Behavior

Four batches of two-liters each were cast for the Modified-Com K and In-House mixes. Three cylinders and prisms for each mix design were cast to test for their mechanical properties and evaluate their behaviour. The cylinders had a diameter of 75 mm and a length of 150 mm, while the prisms had a square cross-section of 50 mm and a length of 280 mm.

Uniaxial compression tests were performed on the concrete cylinders at 28 days; the average compressive strength obtained for Modified-Com K was 102.59 MPa. The In-house mix provided

an average compressive strength of 57.38 MPa. Furthermore, four-point bending tests were conducted on the small prisms to study their flexural-tensile strength, failure mode, and cracking patterns. The load-actuator displacement responses for one prism of each mix design is illustrated in Figure 4.5. An additional set of prisms with typical concrete (compressive strength of 45 MPa) were cast for comparison. The responses presented are typical of the sets of prisms.

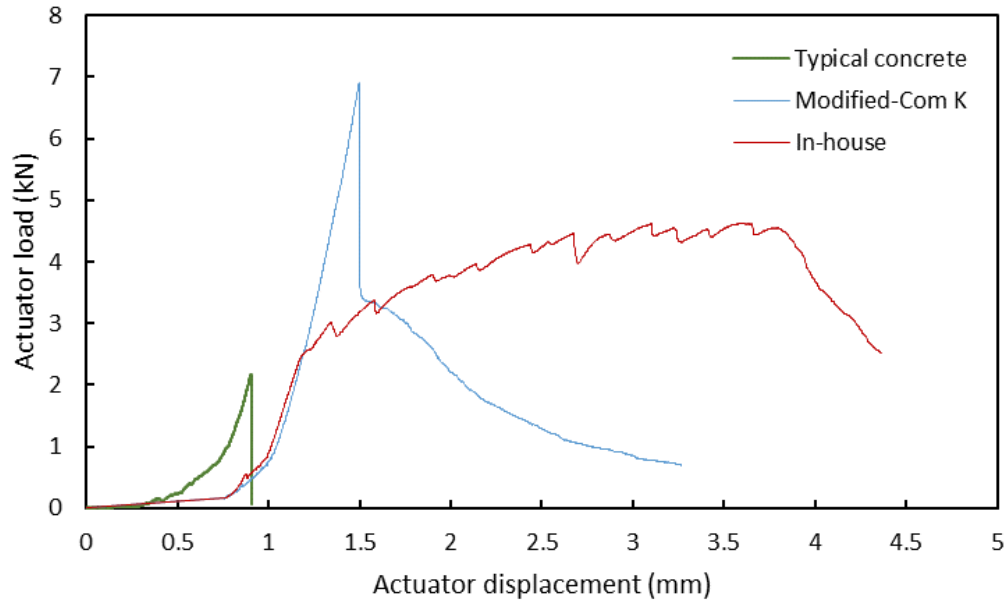


Figure 4.5 Load - actuator displacement response

The prisms with the Modified-Com K mix developed a steep pre-peak response, reaching a peak strength of approximately 7 kN, followed by a sudden decay in strength and a mild softening behaviour towards the end of testing. The response was controlled by a single major crack near the mid-span of the prism. The In-House mix was able to develop a very ductile behaviour before reaching the peak load (approximately 4.5 kN). A well-sustained strain-hardening behaviour was captured as a result of the microcracking prior to crack localization. The cracking patterns are illustrated in Figure 4.6. Both HPFRC mixes developed a much higher flexural strength and displacement than the prisms containing the typical concrete.

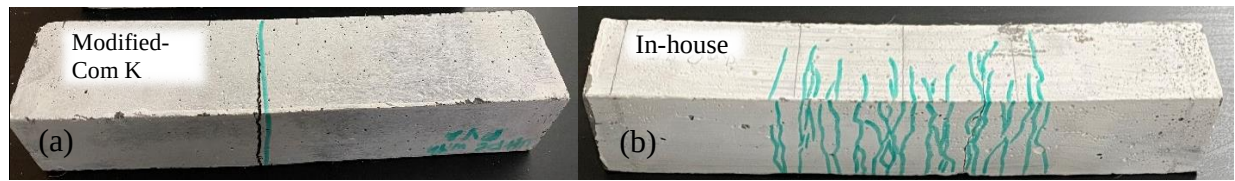


Figure 4.6 Cracking patterns for prism with: (a) Modified-Com K mix; and (b) In-House mix

Based on the results presented above, the Modified-Com K mix does not qualify as an ECC mixture. The prepackaged dry material mix does not specify the contents; however, it appears to be formed mostly by reactive materials that created a stiff concrete matrix, not allowing the PVA fibers to bridge the cracks, and therefore, promoted rupture instead of pullout failure. The high compressive and flexural strengths classify this mix between an ECC and UHPC mix, thus not appropriate for this study, where the objective is for a ductile response. Conversely, the In-House mix with two different types of fibers represents the ideal mix for this research. The sustained strain-hardening behavior, multiple cracking development, and compressive strength classify it as ECC. Therefore, the In-house mix was selected as the repair concrete material for both walls. Figure 4.7 provides an enlarged view of the microcracking characteristics of the prism with the In-house mix.

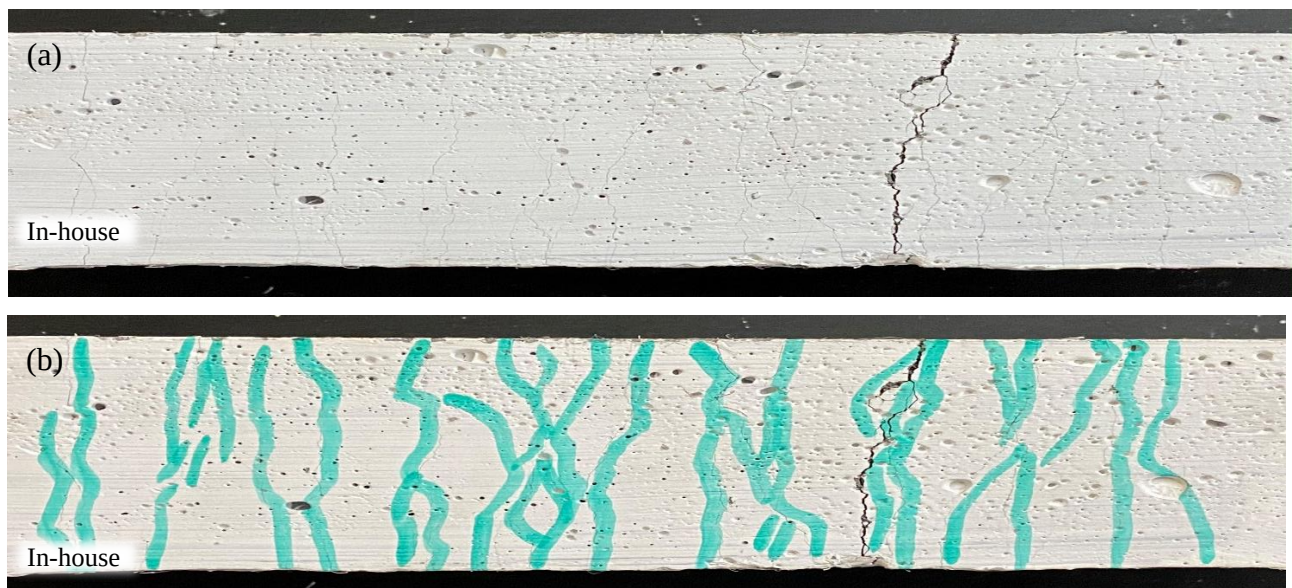


Figure 4.7 Micro cracking at the bottom surface of prisms with In-House mix design: (a) without marking; and (b) with marking

4.3 Repairing Procedure

Based on the visual inspection, the damage reported, and the preliminary numerical analyses performed on the walls, a general repairing procedure was developed, aiming for a full recovery and strengthening of the walls when subjected to lateral loading. The critical deterioration experienced by the walls within the plastic hinge region and the opportunity to establish a consistent comparison led to an identical repairing procedure in this specific zone for both walls. Figure 4.8. illustrates an overview of the proposed repair method in chronological order: (a) visual inspection of the previously damaged walls to assess their state; (b) bracing system built to ensure safety and restrict the movement of the walls at the time of removing the concrete; (c) removal of deteriorated concrete and replacement/addition of reinforcing bars; (d) formwork construction and casting of HPFRC where the previous concrete was removed; and (e) curing of the repaired walls prior to testing.

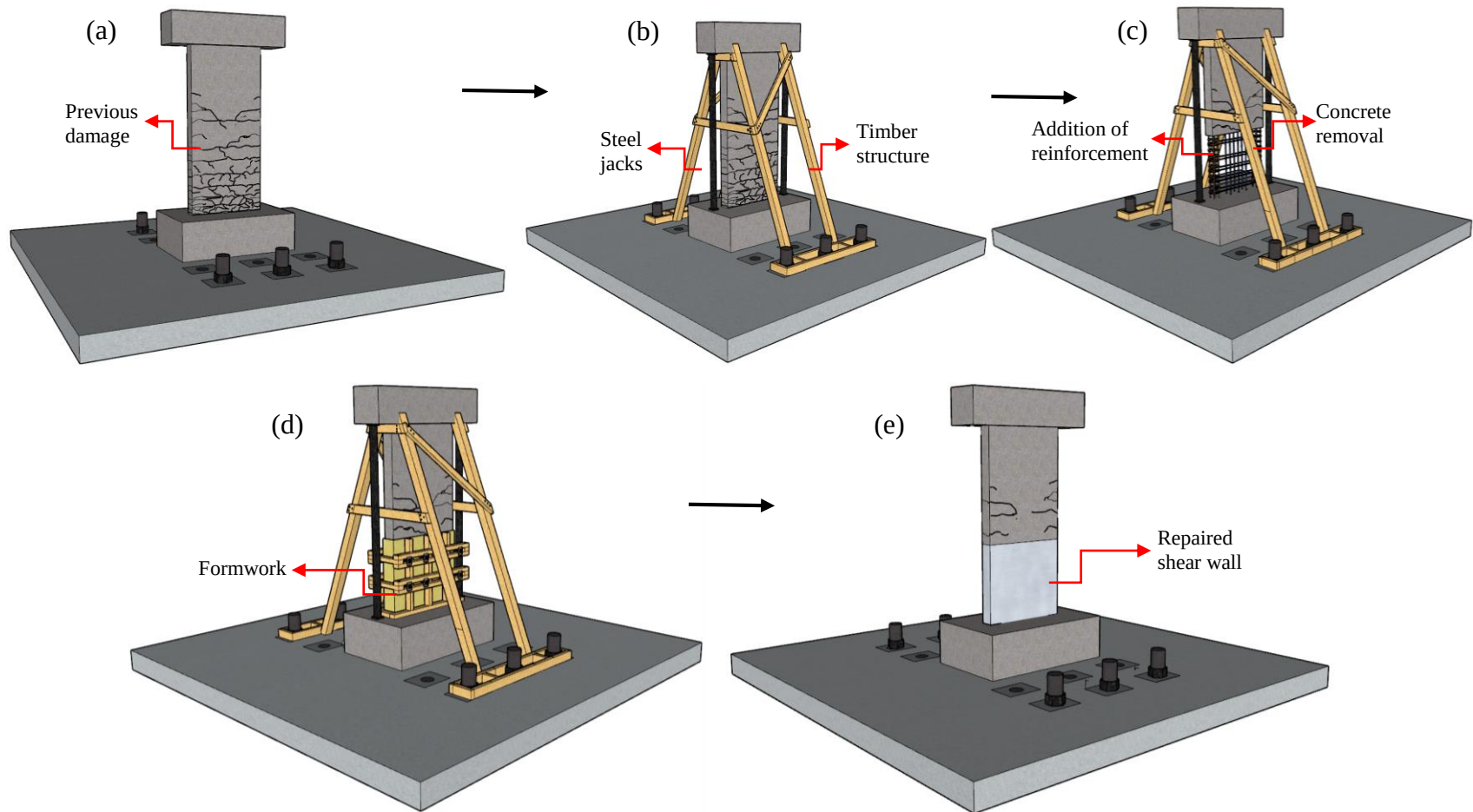


Figure 4.8 General repairing procedure for damaged shear walls: (a) previous damage; (b) bracing system; (c) concrete removal in the plastic hinge and replacement/addition of reinforcing bars; (d) formwork and casting; and (e) repaired wall

4.3.1 Bracing System

The construction of a structure capable of bracing the walls was required to provide a safety mechanism prior to the concrete removal. Dimensional lumber and steel jack posts represented the most viable option to build a structure able to fulfill this purpose. The most economical and efficient method to construct the temporary structure was selected, as illustrated in the various views in Figure 4.9.

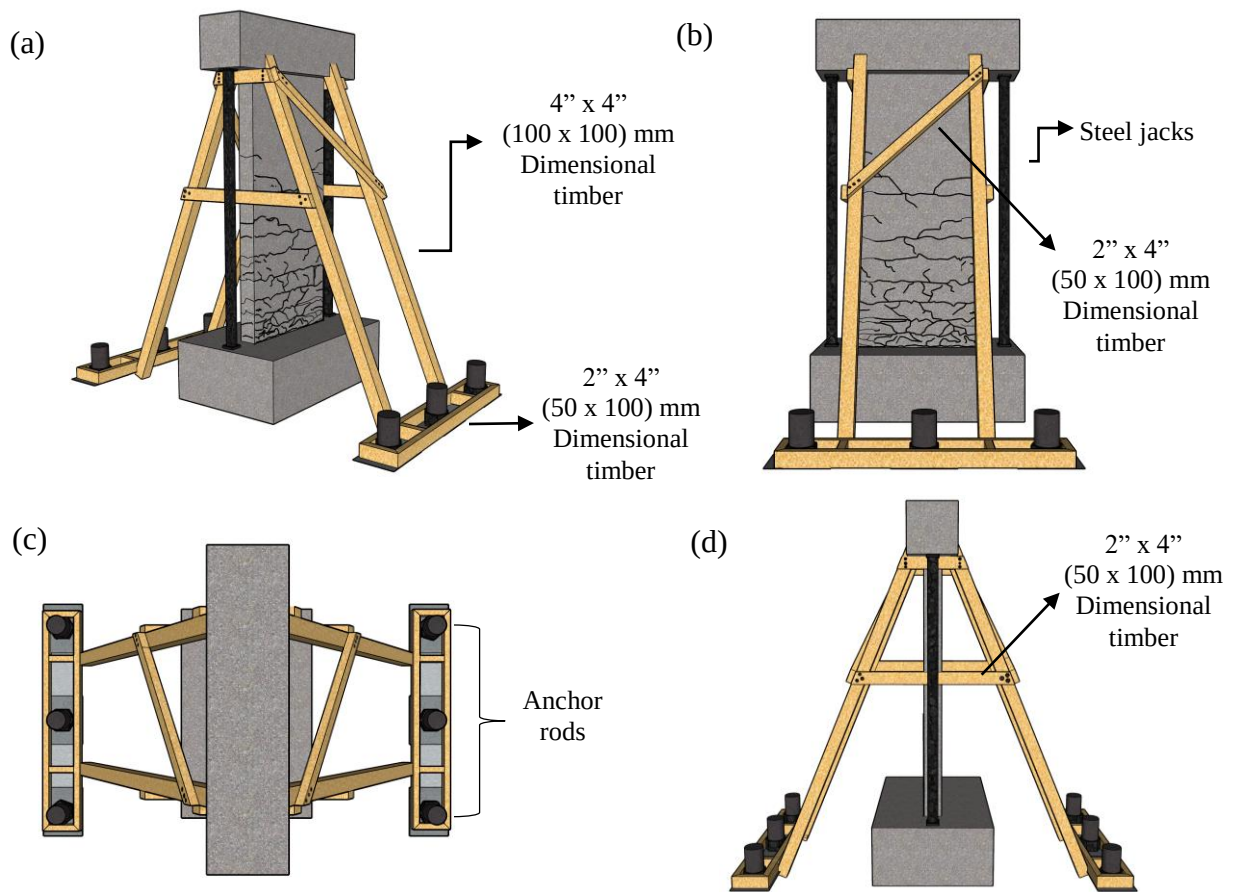


Figure 4.9 Preliminary design of the bracing system: (a) general view; (b) front view; (c) top view; and (d) lateral view

Pressure-treated lumber with a cross-section of 4 inches (100 mm) by 4 inches (100 mm) and a length of 10 feet (3000 mm) was employed as the principal elements to prevent lateral and out-of-plane movement of the walls, as shown in Figure 4.9 (a). Four pieces were utilized, one at each edge for both faces of the walls. They were secured at the bottom with an additional timber grid

built with dimensional lumber of 2 inches (50 mm) by 4 inches (100 mm) held by anchor rods fixed to the strong floor, as represented in Figure 4.9 (c). Additional 2 inches (50 mm) by 4 inches (100 mm) pieces were used to brace the main elements holding the walls, as shown in Figure 4.9 (b) and (d). Moreover, steel jacks with a weight capacity of 8500 lb and a maximum extension of 96 inches (2400 mm) were used to sustain the weight of the walls; one on each side of the wall, bearing against the foundation block and the cap beam, as shown in Figure 4.9 (b). Figure 4.10 is a photo of the materials utilized to build the bracing structure.



Figure 4.10 Materials employed for the bracing system prior to repairing

Figure 4.11 presents the constructed bracing system for both walls. The assembly was made with the sole objective of preventing lateral movement and rotation prior to the concrete removal.



Figure 4.11 Bracing system for the walls: (a) front view; and (b) side view

4.3.2 Concrete Removal

After the installation of the bracing system, the concrete removal phase was initiated and subdivided into three stages to further investigate and define the actual extent of damage in the plastic hinge region for the previously tested walls. The equipment utilized for all stages comprised a heavy-duty hammer for the plain-concrete regions and a pneumatic hammer for a more precise concrete removal near the reinforcement and in the boundary regions.

4.3.2.1 First Stage of Concrete Removal (300 mm Strip)

A strip of 300 mm measured from the base of the walls was first removed, where heavily damaged concrete was evident. Wall SWS-R, shown in Figure 4.12 (a), experienced rupture of the deformed steel reinforcing bars in the boundary region of the plastic hinge. In addition, buckling of the steel bars was evident in the web zone. Figure 4.12 (b) illustrates that Wall SWN experienced no apparent damage to the SE-SMA bars. Nonetheless, the steel reinforcing bars located in the web region ruptured.

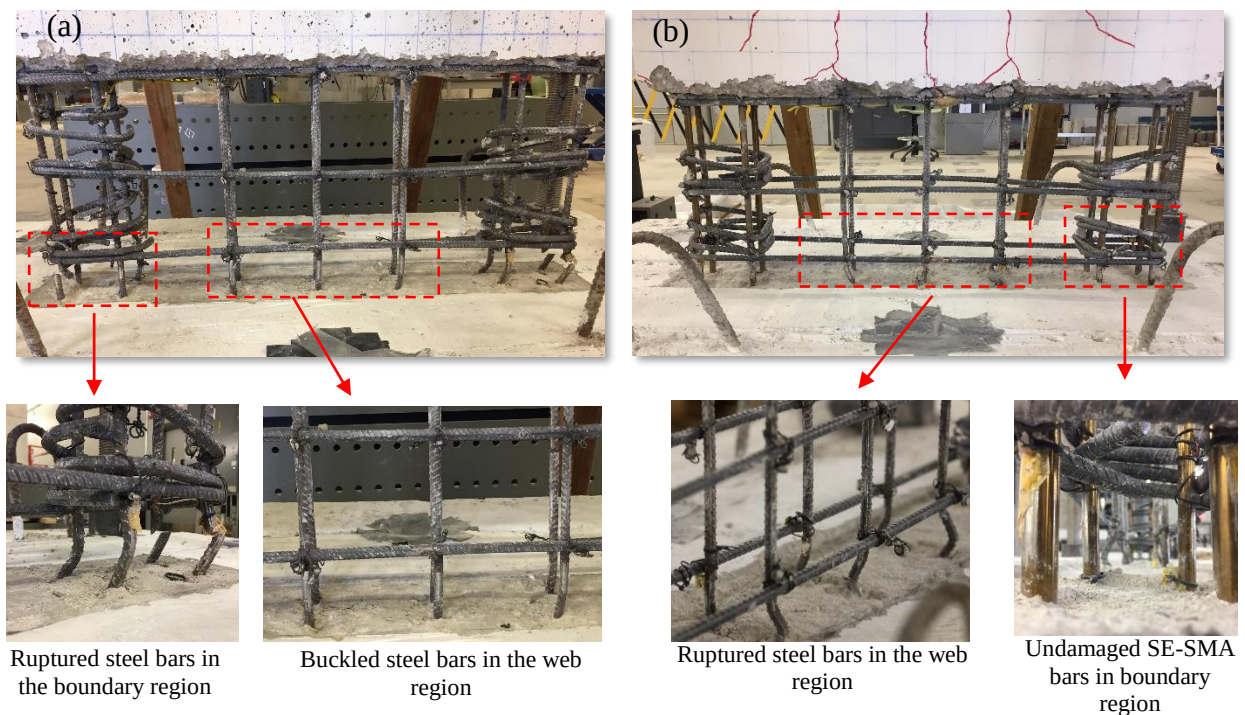


Figure 4.12 First stage of concrete removal and damage of reinforcing bars: (a) SWS-R; and (b) SWN

4.3.2.2 Second Stage of Concrete Removal (U-shape)

The boundary regions are typically susceptible to experience greater damage during testing; therefore, it was decided to continue with the concrete removal for the full plastic hinge length in these areas. Figure 4.13 (a) and Figure 4.13 (b) depicts the U-shape concrete removed for Walls SWS-R and SWN, respectively. In addition to the damage noted in the reinforcing bars at the base of the walls in the first stage, no other damage was visible in the boundary zones for either the vertical reinforcement or the shear reinforcement.

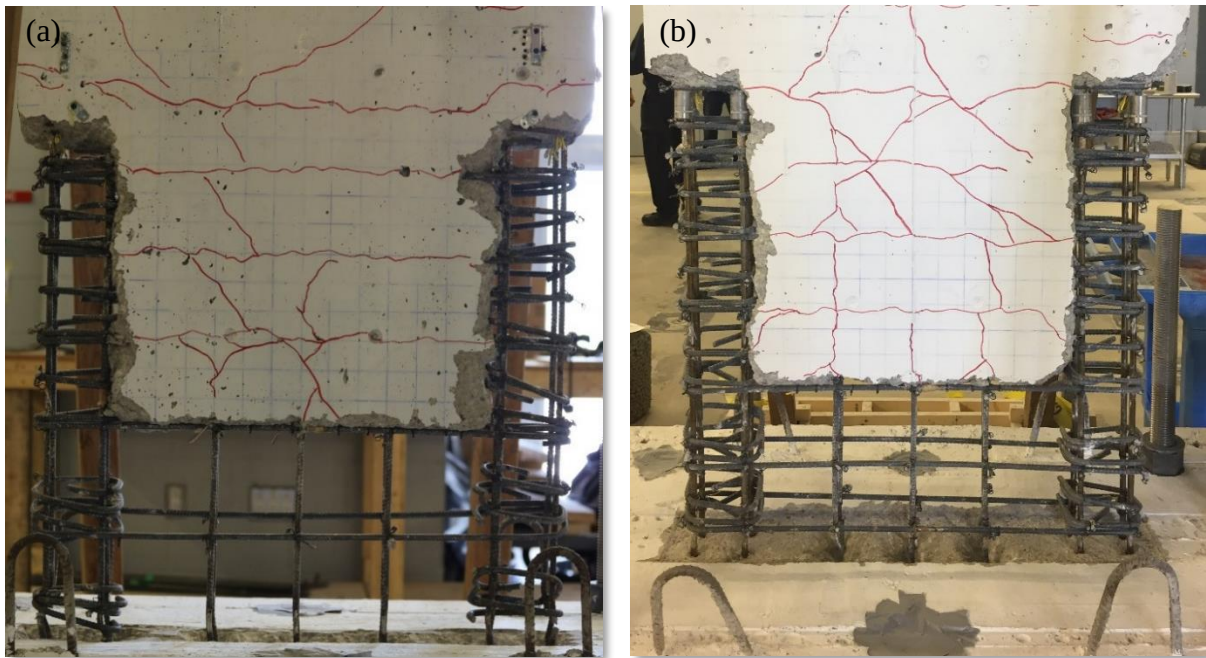


Figure 4.13 Second stage of concrete removal: (a) SWS-R; and (b) SWN

The current state of the mechanical couplers utilized in Wall SWN required a visual assessment. Figure 4.14 provides the state of the couplers used in both ends of Wall SWN. The coupling mechanism suffered no evident damage. Additionally, the SE-SMA bars and the steel bars that are connected to the couplers appeared to be in good condition.

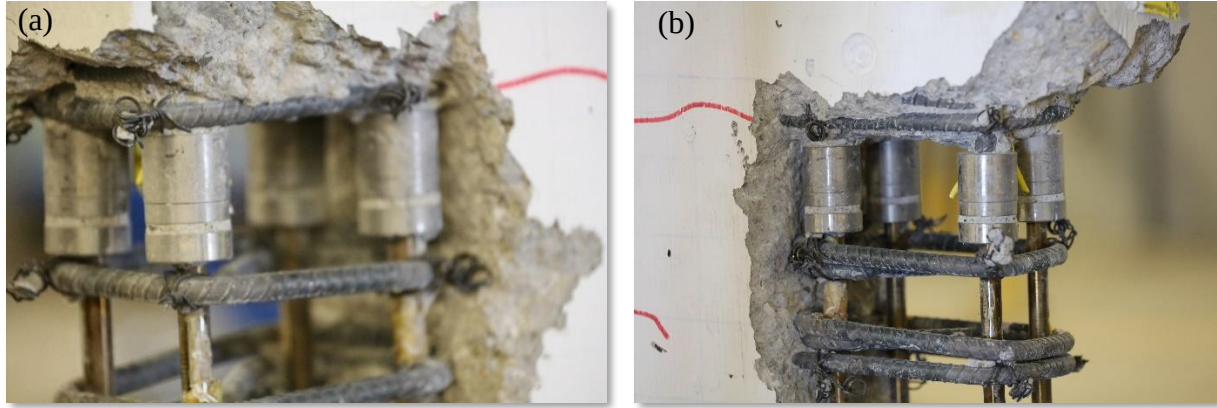


Figure 4.14 Current state of couplers for SWN: (a) west side; and (b) east side

4.3.2.3 Third Stage of Concrete Removal (Full Plastic Hinge)

The final stage involved the removal of concrete over the entire plastic hinge height, as determined in the preliminary numerical analyses. The extent of damage in this region and the opportunity to assess the behavior of the walls repaired with ECC in the critical region as part of a hybrid system were significant reasons in choosing this procedure. The same approach was taken for both walls to allow comparison. Figure 4.15 illustrates the concrete removed in the two walls.

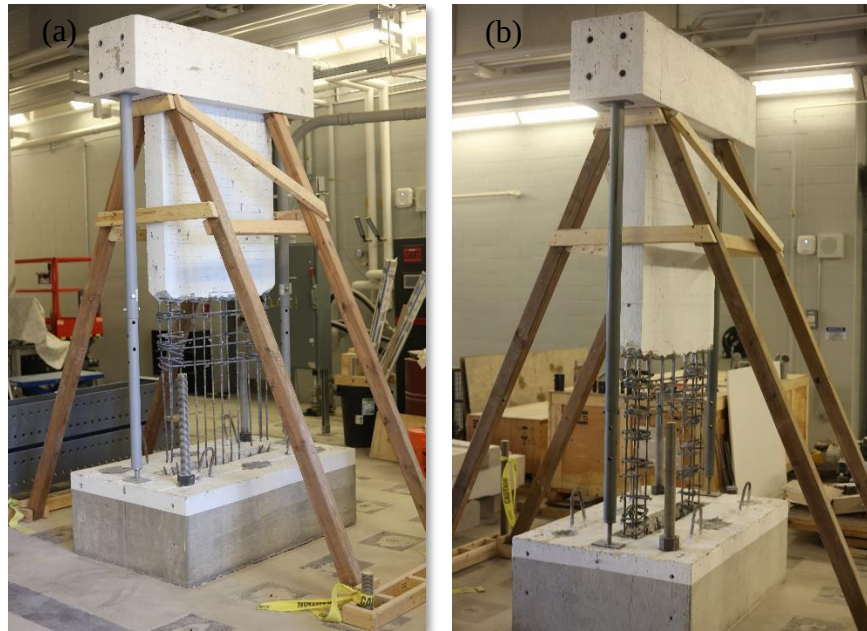


Figure 4.15 Third stage of concrete removal: (a) SWS-R; and (b) SWN

For Wall SWS-R (Figure 4.15 (a)), two reinforcing bars located 400 mm above the base of the wall, in the web region, experienced buckling. Wall SWN (Figure 4.15 (b)) did not experience this damage at the same level in the reinforcement. Furthermore, a rough surface was maintained at the top of the plastic hinge zone to improve the behaviour of the cold joint between the repair material (ECC) and the existing concrete.

4.3.3 Replacement and Addition of Reinforcing Bars

As presented previously, both walls experienced substantial damage of their mild-steel reinforcing bars within the plastic hinge. The boundary and web reinforcement in Wall SWS-R ruptured and buckled respectively, whereas the bars in the web zone of Wall SWN ruptured. All ruptured and buckled bars were removed and subsequently replaced with identical 10M mild-steel deformed bars.

4.3.3.1 Removal of Ruptured/Buckled Reinforcing Bars

Cutting and removal of damaged reinforcing bars involved the utilization of an angle grinder with a 7-inch (175 mm) diameter metal cut-off disk. Figure 4.16 (a) and (b) illustrate the segments of bars removed from Wall SWS-R and SWN, respectively.

The different types of damage in the reinforcement extended up to a height of 400 mm measured from the base of Wall SWS-R. Therefore, 500 mm of the damaged reinforcement measured from the base were removed from both walls where the deterioration was concentrated. The remaining reinforcement was repaired by placing new segments of reinforcement that were connected with mechanical couplers. Figure 4.16 (a) illustrates the state of Wall SWS-R after the reinforcing bars within the boundaries and web regions were removed. For Wall SWN shown in Figure 4.16 (b), only the reinforcement in the web region was removed, while the original SE-SMA bars in the boundary region remained intact. The shear and anti-buckling reinforcement for both walls was removed prior to removing the longitudinal reinforcement and subsequently reused after the new longitudinal bars were installed.

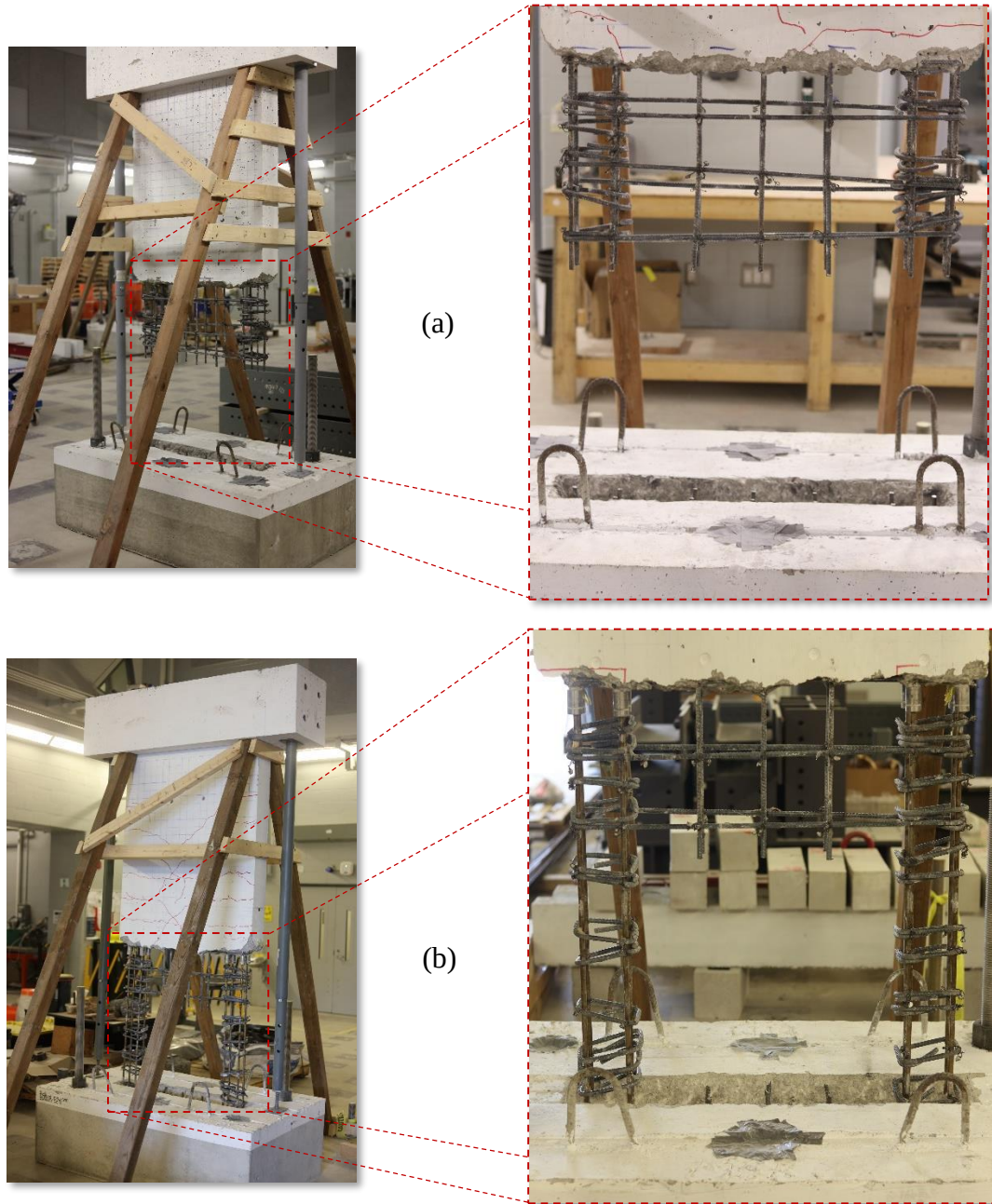


Figure 4.16 Segments of steel reinforcement removed in plastic hinge region: (a) SWS-R; and (b) SWN

4.3.3.2 Surface Preparation for New Reinforcement

The addition of the new reinforcing bar segments involved the addition of mechanical couplers within the walls. This process required the remaining reinforcement to be vertically straight and to have a flat cross-sectional surface such that the new reinforcing bars could be butt-ended with the

aid of mechanical couplers Figure 4.17 illustrates that the height of the fracturing and buckling of the reinforcement provided a challenge to accommodate the coupling of the remaining and new reinforcement.

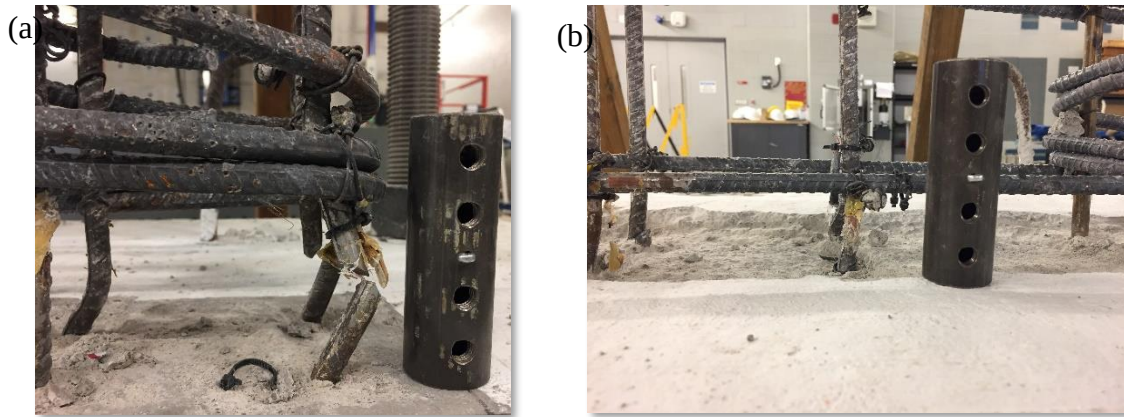


Figure 4.17 Mechanical coupler placed at the base of wall: (a) SWS-R; and (b) SWN

Figure 4.17 (a) and (b) illustrate the mechanical couplers adjacent to the boundary and web reinforcement at the base of the walls, respectively. The state and the length of the reinforcing bars at the base of the wall were not appropriate for the coupling mechanism. To address this challenge, additional concrete below the wall, in the foundation was removed. Figure 4.18 shows the state of the walls after concrete in the foundations was removed.

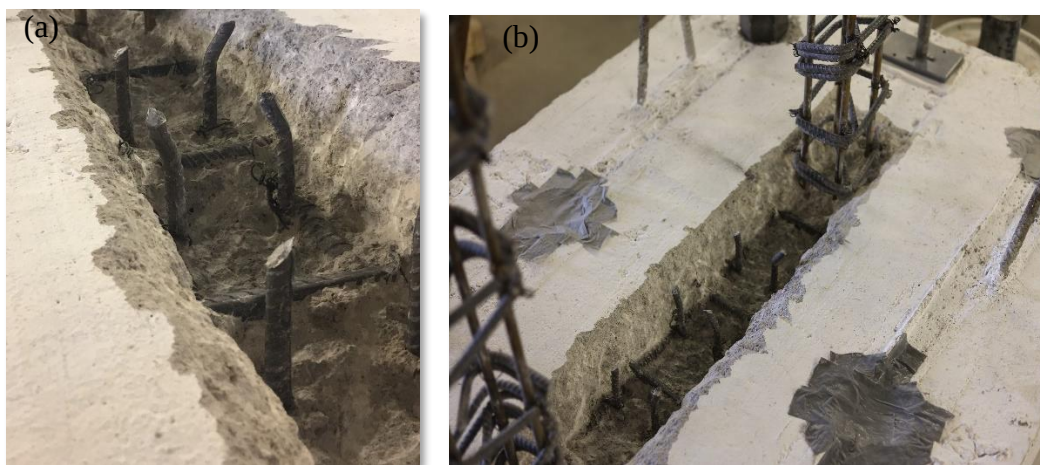


Figure 4.18 Concrete removal in the foundation: (a) SWS-R; and (b) SWN

Approximately 60 mm of concrete was removed from both walls, while maintaining the original dimensions of the wall above (1000 mm wide by 150 mm thick). Thereafter, the longitudinal reinforcing bars protruding from the foundation were cut to an appropriate length to satisfy the coupling mechanism to the new segments of reinforcement. The remaining bars were cut with an angle grinder using a 7-inch (175 mm) diameter metal cut-off disk, while the ends of the bars were made flush with a 3/8-inch (9.5 mm) by 2-inch (50 mm) belt sander. Figure 4.19 shows the longitudinal bars protruding from the foundation and ready for the mechanical couplers.

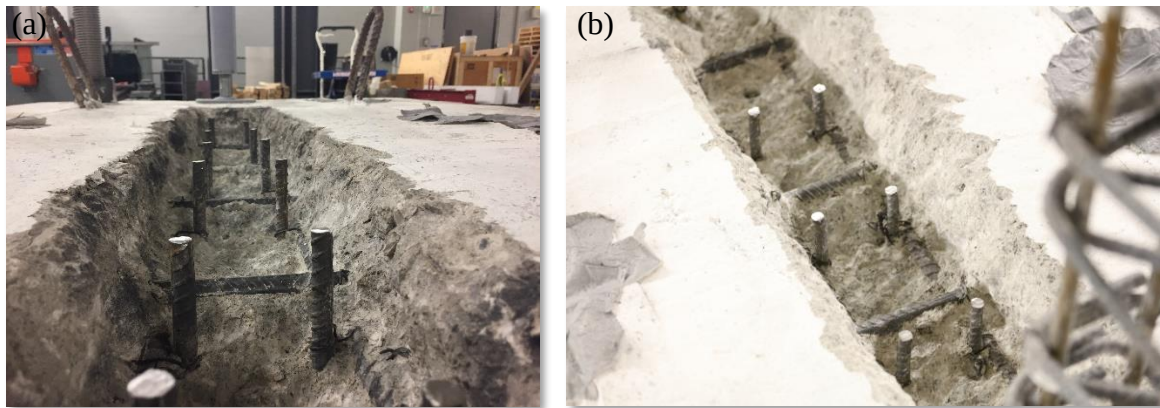


Figure 4.19 Reinforcing bars at the base of wall: (a) SWS-R; and (b) SWN

Similarly, the remaining reinforcing bars at the top of the plastic hinge were made flush as shown in Figure 4.20.

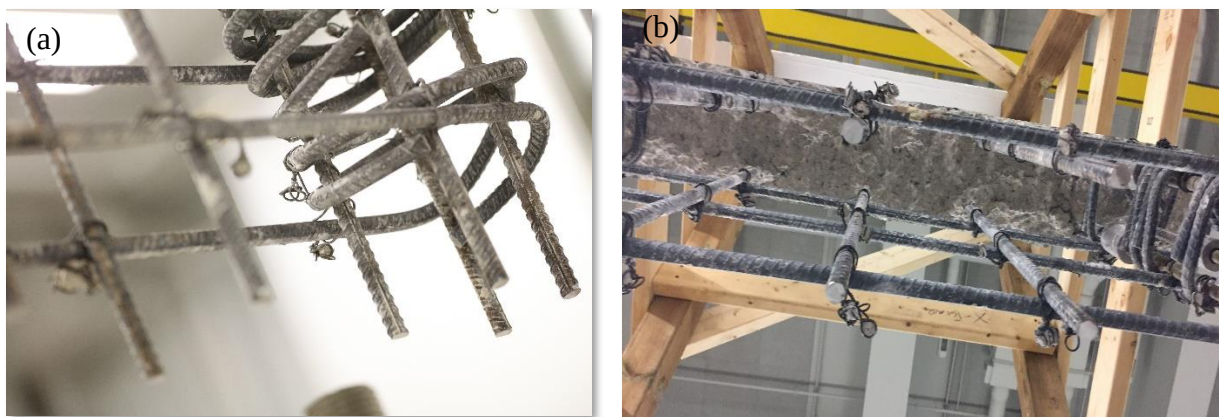


Figure 4.20 Flattening of existing reinforcing bars at the top of the plastic hinge: (a) SWS-R; and (b) SWN

A 5/8-inch (16 mm) SDS-Plus rotatory hammer was used to drill four holes into the foundation for the starter bars, as presented in Figure 4.21. The depth of each hole was 5 inches (125 mm)

with ½ inch (12.5 mm) diameter. The extent of the drilling was based on the specifications required by the epoxy grout to ensure the bars were fully bonded to the concrete.



Figure 4.21 Drilled holes for starter bars

4.3.3.3 Placement of the Starter Bars

Four 10M reinforcing bars spaced at 150 mm, were installed at the base of both walls. Sika AnchorFix-300, a high-performance, 2-component adhesive epoxy was used to bond the reinforcing bars into the concrete, where the holes were drilled. As depicted in Figure 4.22, these reinforcing bars served as starter bars aiming to suppress rocking/sliding at the base of the walls, which was evident during the testing of the original walls. Each starter bar extended 300 mm above the foundation.

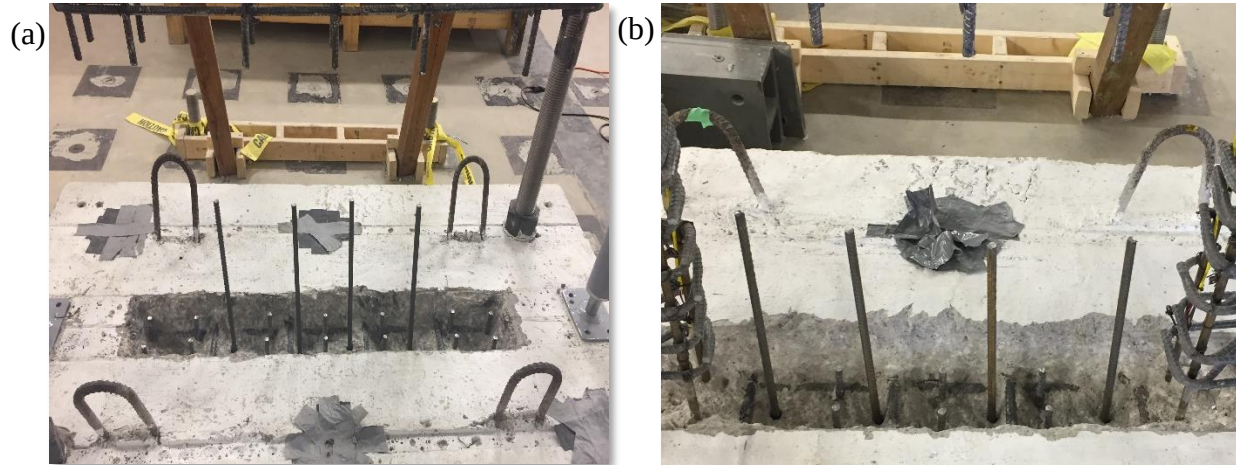


Figure 4.22 Installation of starter bars: (a) SWS-R; and (b) SWN

4.3.3.4 New Reinforcement and Coupling Mechanism

The new segments of reinforcing bars used to replace the fractured and/or buckled reinforcing bars were connected to the remaining reinforcement with mechanical couplers. The couplers guaranteed continuity without slip between the new and remaining deformed reinforcement. The couplers used for the deformed reinforcing steel differed from those by Morcos (2020) in the original SWN wall to connect the SE-SMA bars with the deformed steel bars. The 4 inches (100 mm) - Dayton Superior Bar Lock Coupler was selected; it uses lock-shear bolts and serrated grip rails to mechanically splice the reinforcing bars that are inserted from either end. The heads of the shear bolts are designed to shear off at a required torque to ensure proper installation. Therefore, an air impact wrench was utilized to tight the bolts and connect the new segments of steel bars and the existing bars. Figure 4.23 shows the mechanical coupler installed in the wall.

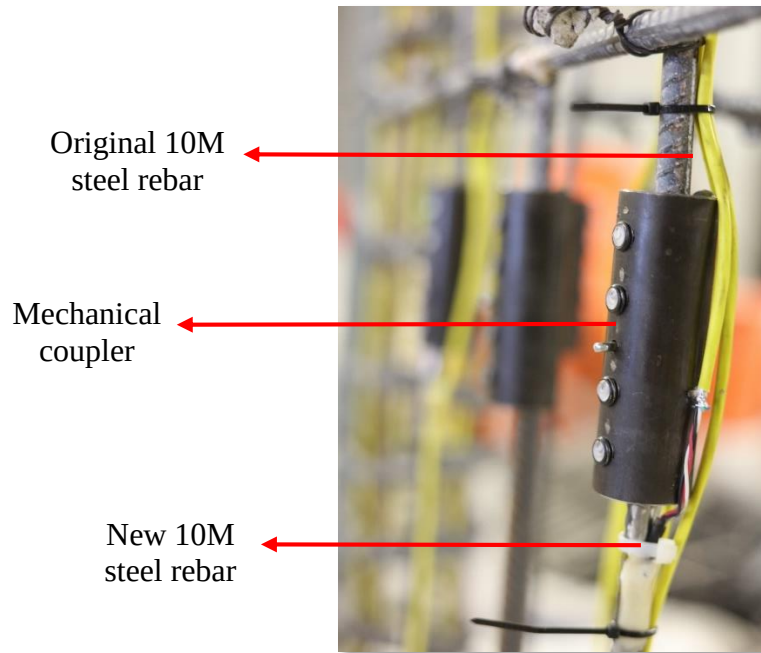
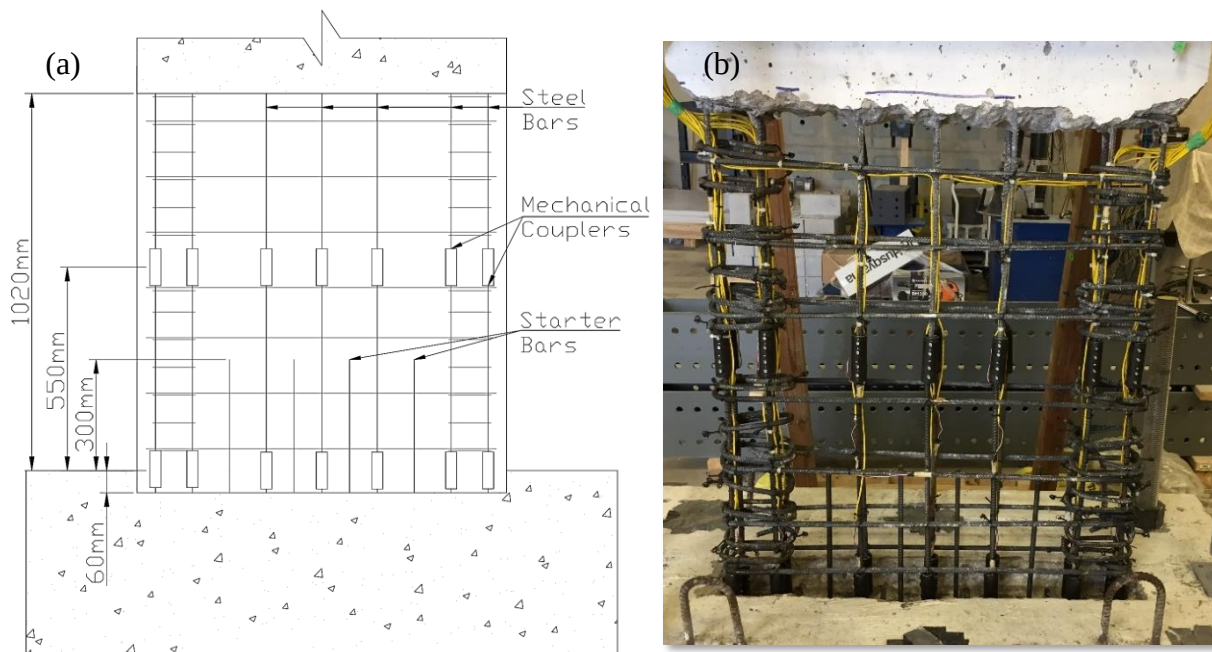


Figure 4.23 Mechanical coupler installation

A total of 28 couplers were used for Wall SWS-R, and 12 couplers for SWN, given that only the steel bars in the web region were replaced. In addition, the new segments of steel bars used in both walls were 550 mm in length. The location of the new bar segments and mechanical couplers are shown in Figure 4.24 (a) and (b) for Wall SWS-R, and in Figure 4.24 (c) and (d) for Wall SWN.



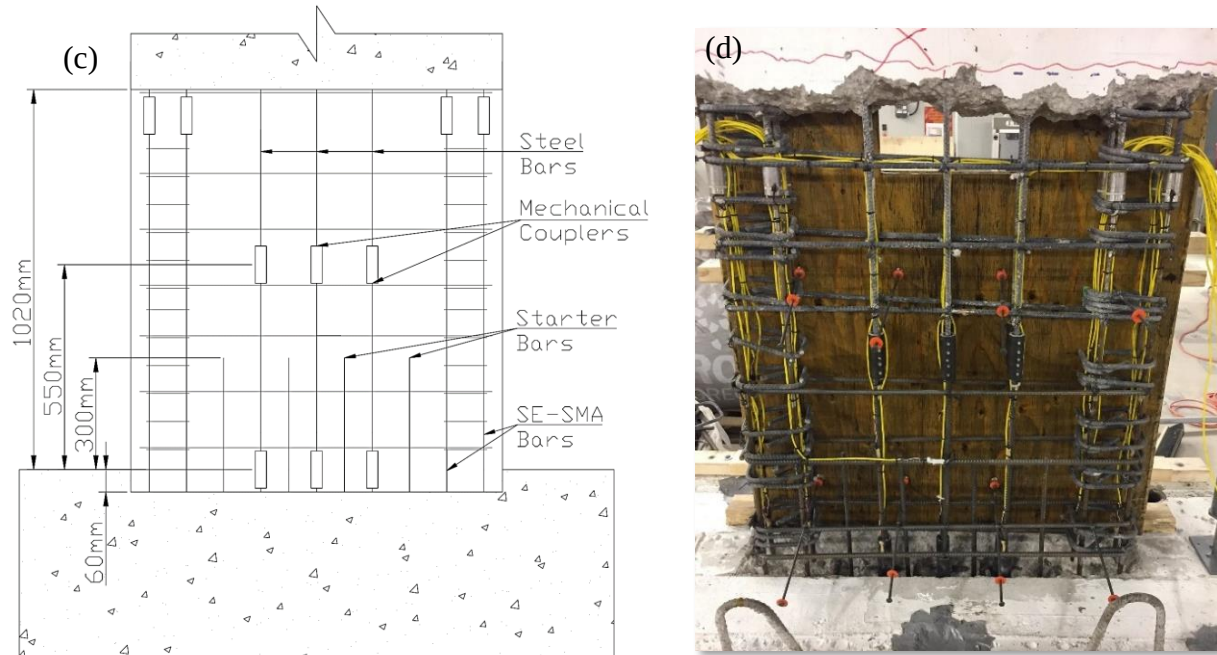


Figure 4.24 Final reinforcement layout: (a),(b) Wall SWS-R; and (c),(d) Wall SWN

4.3.4 Formwork Design and Construction

Figure 4.25 portrays a graphic representation of the formwork design, assuming concrete removal for the entire plastic hinge length in the walls. The design involved the use of $\frac{1}{2}$ inch (12.5 mm) thick plywood as the main component to cover the walls in which the ECC would be poured, as shown in Figure 4.25 (a). Dimensional studs of 2 inches (50 mm) by 4 inches (100 mm) are employed to brace the plywood vertically and horizontally to prevent it from bulging as presented in Figure 4.25 (b) and (c). Snap-ties add additional resistance from over-pressure effects and ensure the proper performance of the formwork (Figure 4.25(b)).

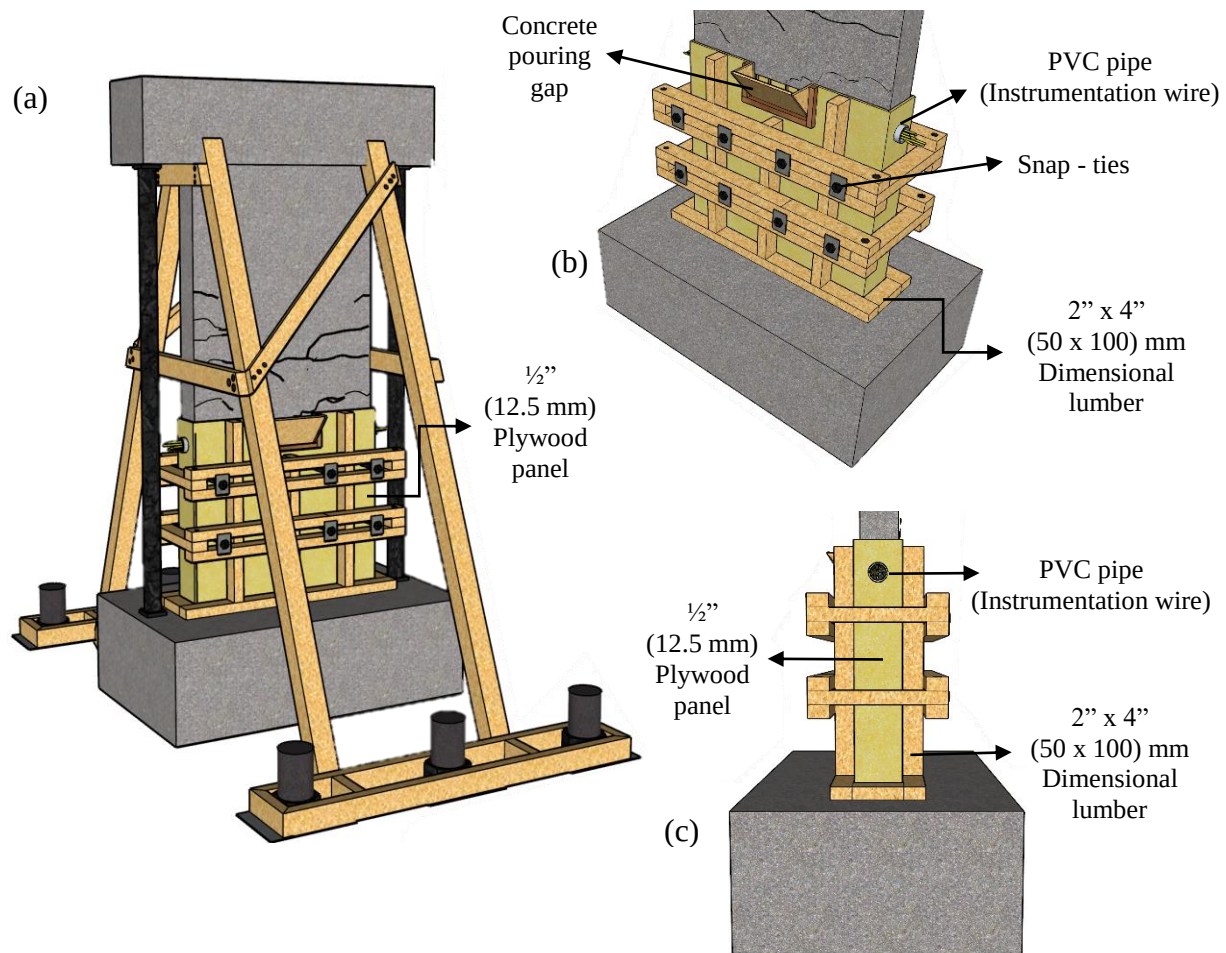


Figure 4.25 Formwork design for the plastic hinge

Two inches (50 mm) diameter holes were drilled on each lateral face of the formwork, in which a Polyvinyl Chloride (PVC) pipe was installed (Figure 4.25 (b) and (c)) to provide a conduit for the strain gauge wires that were attached to the strain gauges on the reinforcement.

ECC is recognized for its self-consolidating properties; therefore, it does not require external vibration. A 200 mm long by 300 mm wide opening was designed to be centered at the top-front side of the formwork where the concrete was poured, as shown in Figure 4.25 (b). Figure 4.26 illustrates the built formwork.



Figure 4.26 Constructed formwork: (a) top view; and (b) side view

Furthermore, the bracing system described in Section 4.3.1 was utilized to provide additional support during casting. Figure 4.27 shows the formwork installed in Wall SWN prior to casting.



Figure 4.27 Formwork installed on Wall SWN

4.3.5 Batching, Casting and Curing of the In-house Design Mix

4.3.5.1 *Batching and Mixing of the In-house Mix*

The ECC was batched and mixed in-house, based on the mix design proposed by Eshghi (2018), with two notable modifications: the addition of smaller-diameter PVA fibers and the use of a different superplasticizer as presented in Section 4.2. In addition to the ECC required to repair the plastic hinge zone of the walls, a set of long and small prisms, along with cylinders, were prepared to obtain the material properties of the mix for each wall. Table 4.5 quantifies the estimated volume required per wall.

Table 4.5 Volume required per wall

Specimen Type	Dimensions (mm)		Volume (L)
Wall	<i>b</i>	150	162
	<i>h</i>	1080	
	<i>l</i>	1000	
Long Prisms	<i>b</i>	75	2.81
	<i>h</i>	75	
	<i>l</i>	500	
Short Prisms	<i>b</i>	75	1.58
	<i>h</i>	75	
	<i>l</i>	280	
Compression Cylinders	ϕ	75	0.6
	<i>h</i>	150	

The pan-type mixer used to batch the ECC has a capacity of 225 liters; however, to avoid the sudden expansion and overflowing of the mixture, it was decided to prepare three batches of 75 liters each to produce the required amount of ECC per wall, including the additional specimens. Table 4.6 lists the design proportions for a batch of 75 liters.

Table 4.6. Modified - Mix proportions for Engineered Cementitious Composite (ECC)

PVA Fibers (Kg)		Cement (Kg)	Fly Ash (Kg)	Silica Fume (Kg)	Slag (Kg)	Sand (Kg)	Water (Kg)	Superplasticizer (Kg)	Volume (L)
$\phi=0.1\text{mm}$	$\phi=0.04\text{mm}$								
0.94	0.94	30	50	8.25	3.38	10.73	23.33	0.75	75

ϕ = fibers diameter

Given the fact that each wall was cast in three stages, all the materials were batched the day prior to mixing to avoid cold joints in the plastic hinge zone. The mixing procedure was initiated by adding all the dry materials in the pan and mixing it for five minutes. Thereafter, a blend of one-half of the amount of water with one-half of the superplasticizer was gradually added to the dry mix for five minutes. The next step involved the addition of the PVA fibers; both types were previously mixed and subsequently added, gradually, for 10 minutes. Finally, the second blend of water with the superplasticizer was progressively added to the mix for an extra five minutes, until reaching the required flowability. Figure 4.28 illustrates the main steps of the mixing procedure of the ECC mixture.

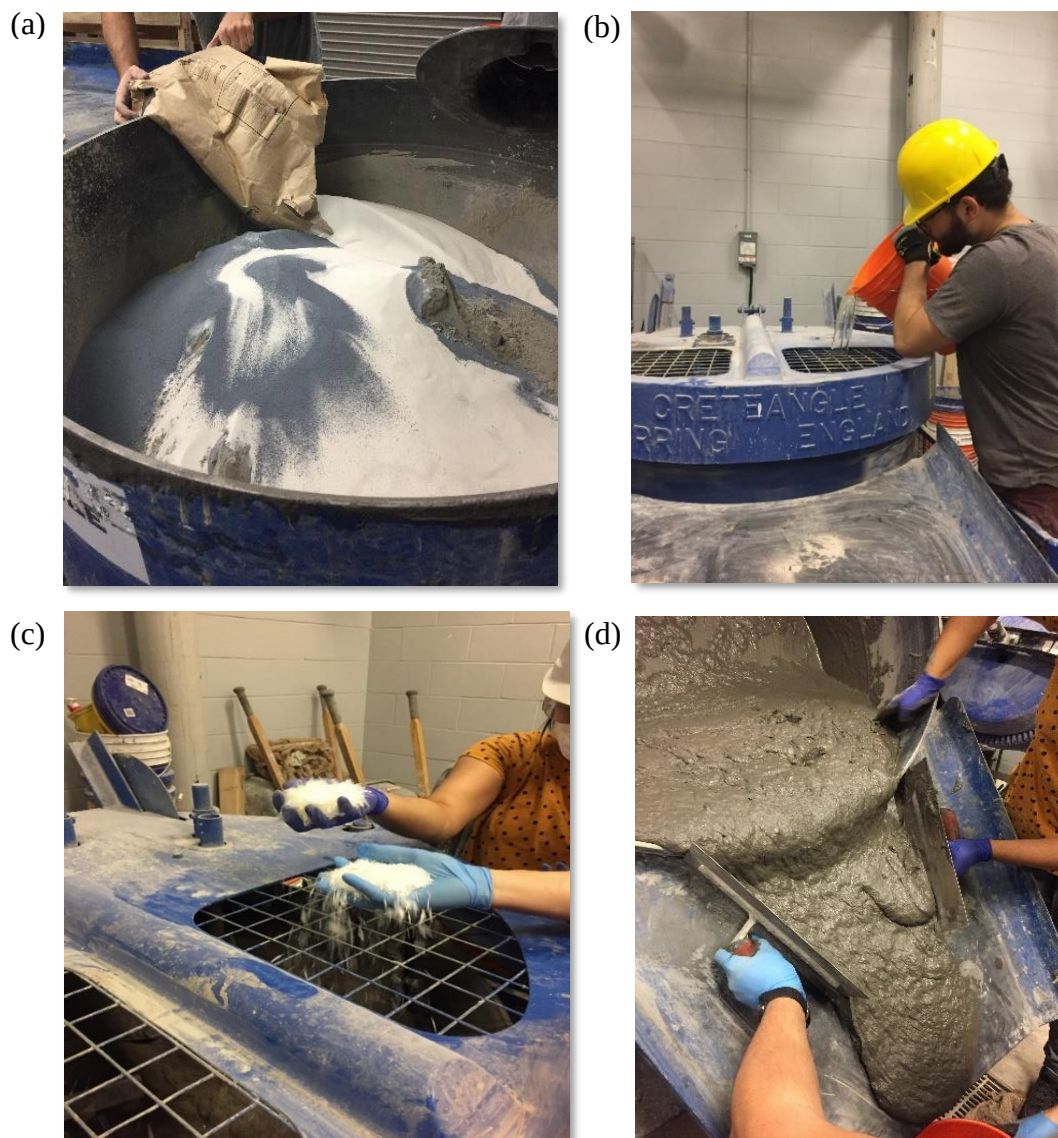


Figure 4.28 ECC mixing procedure: (a) mixing of dry material; (b) addition of water with superplasticizer; (c) addition of PVA fibers; and (d) flowability of the mix

4.3.5.2 Casting of the Walls

Once the mixture attained the targeted flowability, the mixture was poured in 20-gallon buckets to cast the walls and additional specimens. Figure 4.29 depicts the procedure followed for the casting.



Figure 4.29 Procedure in Casting: (a) Casting of the wall; (b) pouring of ECC through the designed compartment; and (c) additional prisms and cylinders

4.3.5.3 Deficiencies in Wall SWS-R

Deficiencies were encountered when the formwork was removed from Wall SWS-R. The ECC mixture was not adequately distributed at the top boundary of the plastic hinge, and therefore, gaps were evident at both extremes of the wall, as presented in Figure 4.30 (a). The problem was solved by casting Self-Consolidating Concrete (SCC) in the affected area, utilizing the same formwork. Moreover, additional concrete cylinders were cast containing SCC to obtain its compressive properties. Figure 4.30 (b) shows Wall SWS-R after casting of the SCC.

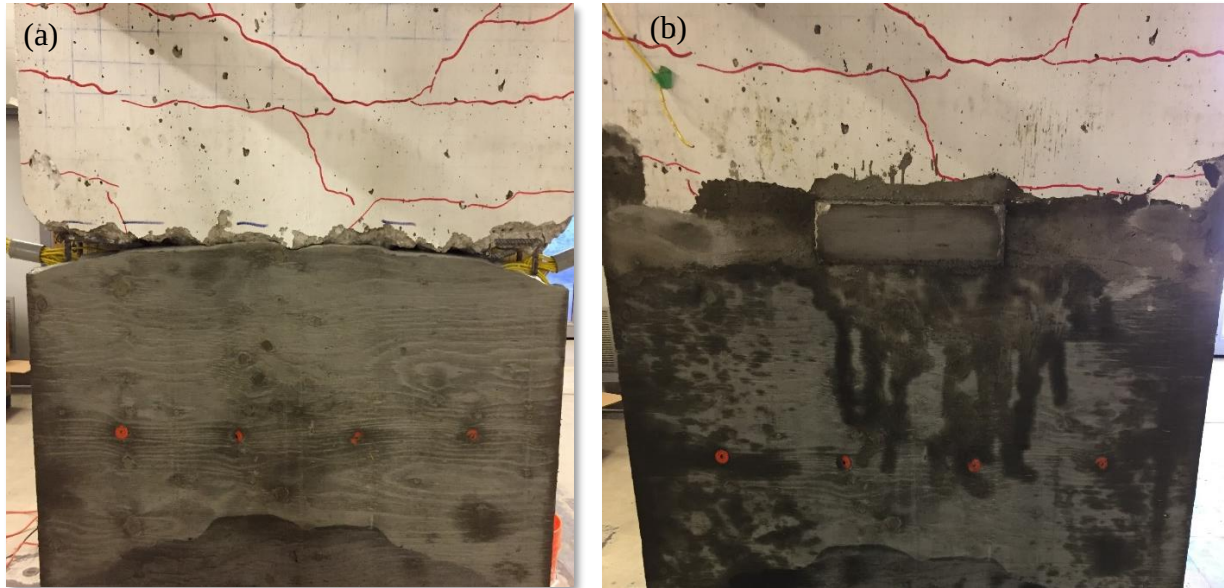


Figure 4.30 Deficiency in Wall S: (a) Poor distribution at the top boundary of the plastic hinge region; and (b) casting of SCC in the affected region

4.3.5.4 Demolding and curing of the walls

The formwork was removed three days after casting of each wall. Figure 4.31 shows the state of the walls on the day of demolding.



Figure 4.31 Demolding of formwork: (a) Wall SWS-R; and (b) Wall SWN

The curing procedure commenced immediately and extended for 28 days. Each wall was covered with wet burlap sheets and a vapor barrier. Every day, the walls were hosed with water until the end of the curing period. Figure 4.32 shows the walls being cured.

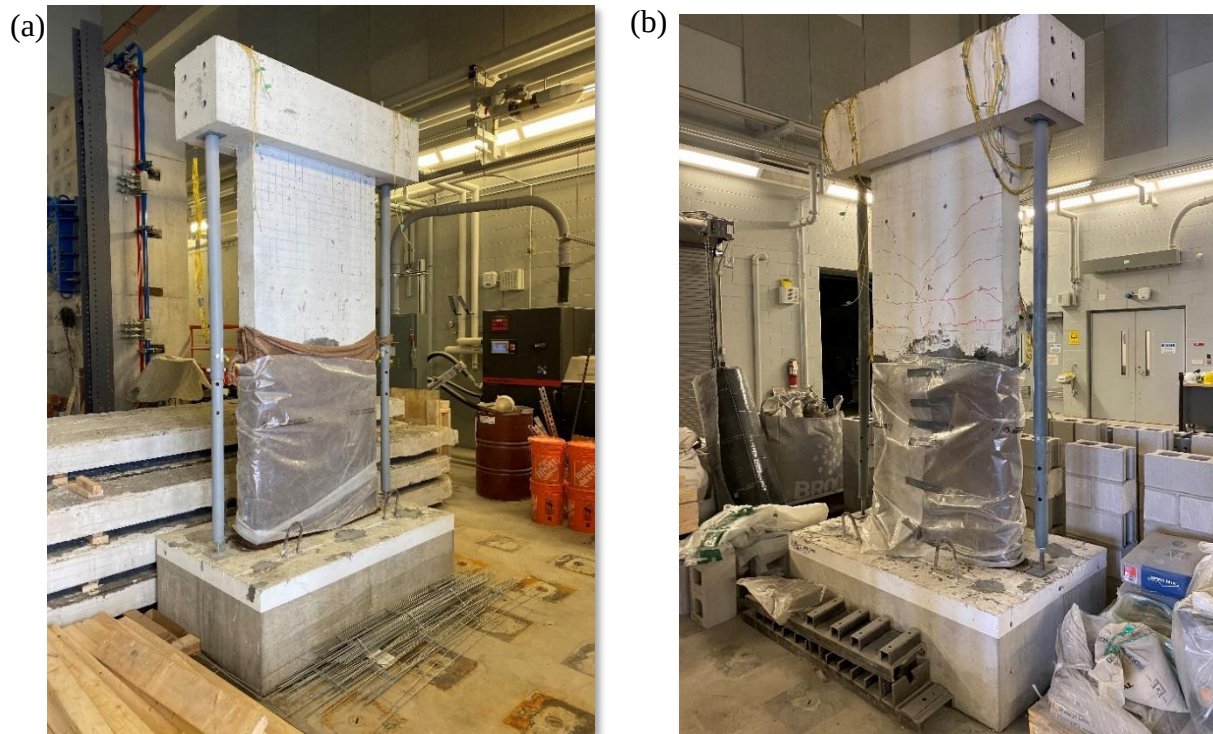


Figure 4.32 Curing setup: (a) Wall SWS-R; and (b) Wall SWN

4.3.6 Repaired Walls

Once the curing was completed, the walls were prepared for testing. White paint was applied on the walls, where the ECC was cast. Afterwards, a grid of 50 mm by 50 mm was drawn on the walls. The procedure was intended to locate, during the experimental testing, cracking patterns and widths that could be compared against the finite element models. Figure 4.33 shows the final repaired walls referred to as RSWS-R and RSWN, where the letter “R” indicates repaired.

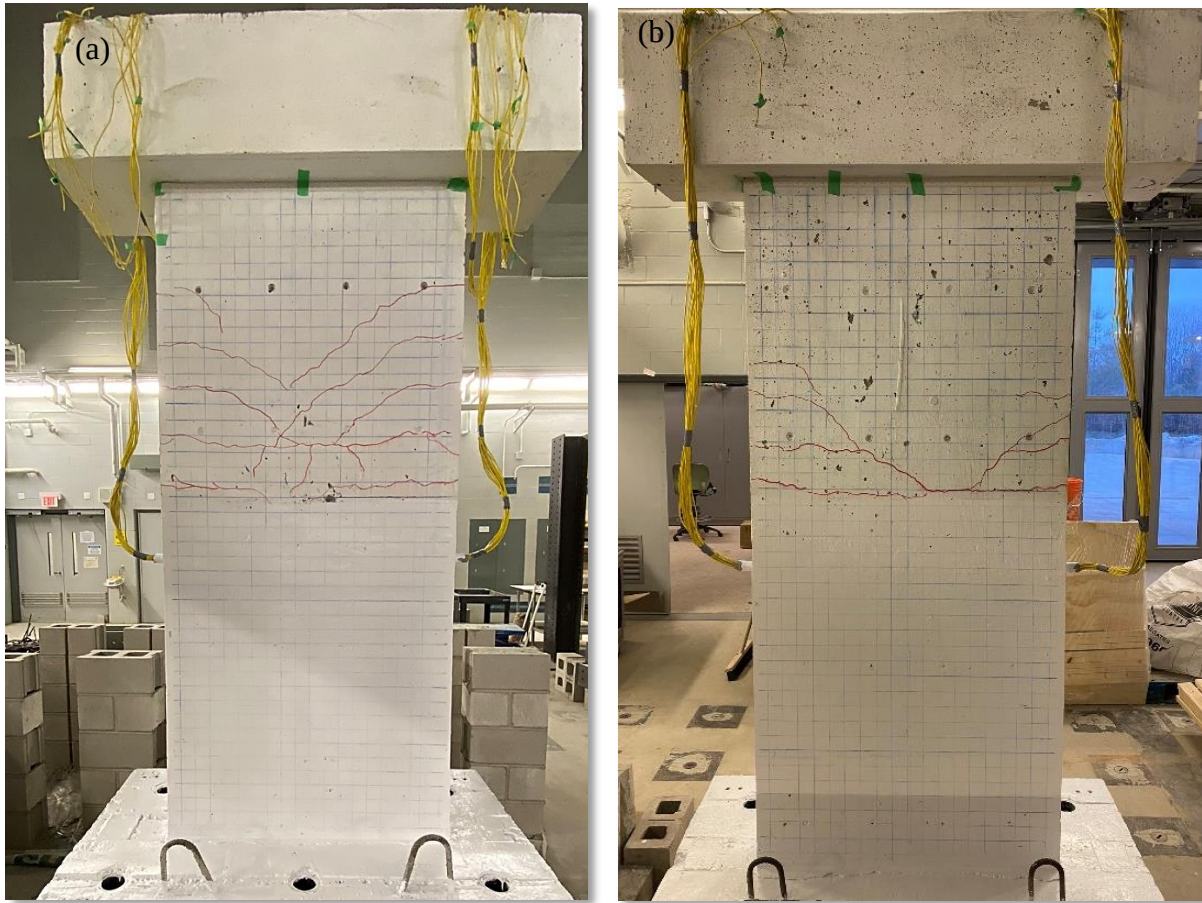


Figure 4.33 Repaired walls: (a) RWS-R; and (b) RSWN

4.4 Experimental Setup, Instrumentation and Testing

4.4.1 Global Test Assembly

The experimental test setup designed by Morcos and Palermo (2019) for the original walls was adopted in this study. Testing was conducted in the High Bay Structural Laboratory, located at York University, Canada. Figure 4.34 illustrates a general overview of the assembled test setup.

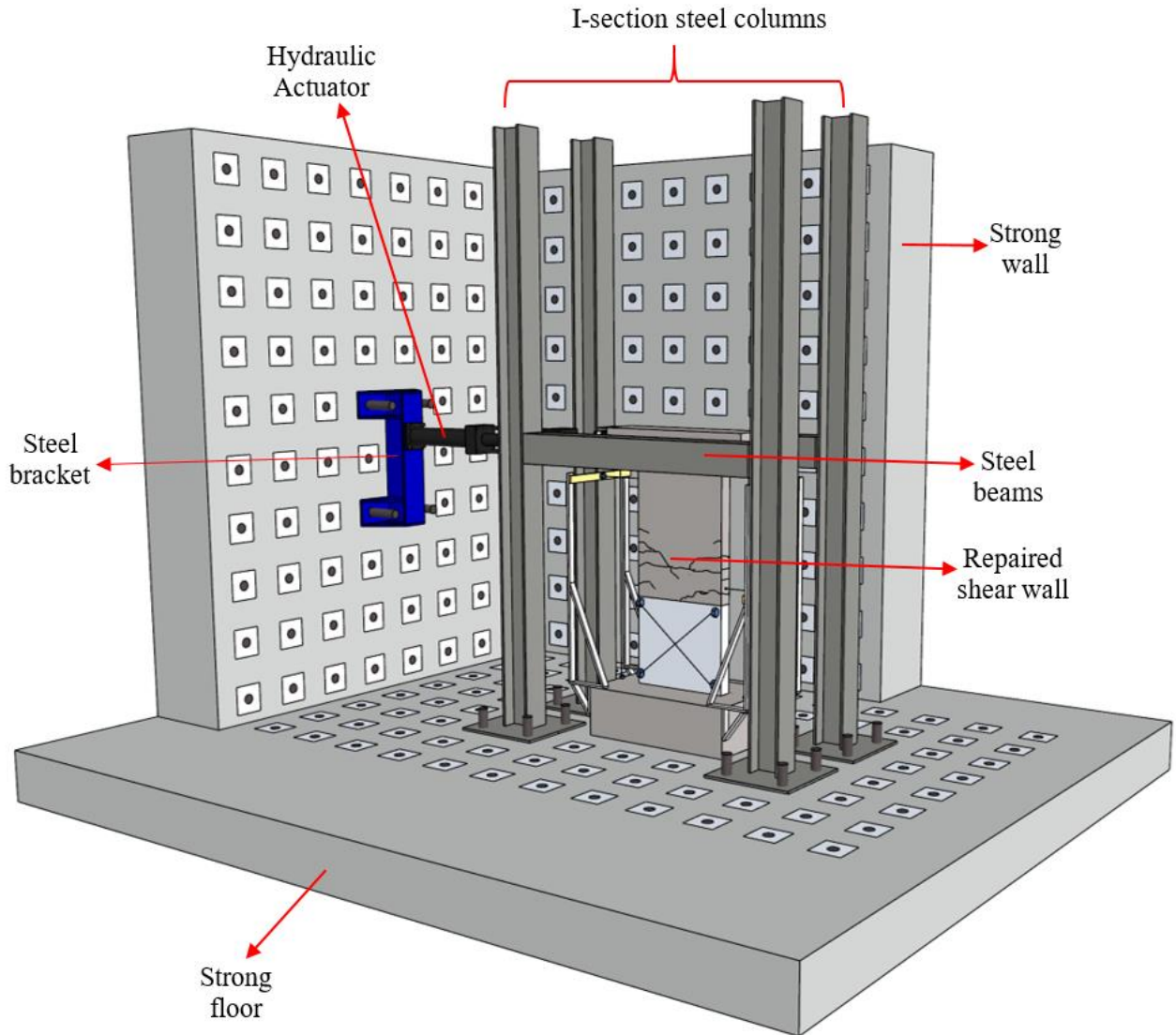


Figure 4.34 Global test setup assembly for repaired walls

Lateral loading was applied in a displacement-controlled mode through a single hydraulic actuator positioned at the center of the cap loading beam of the repaired walls. The actuator was mounted on a steel bracket that, in turn, was fixed to the laboratory strong wall. A lateral supporting frame was assembled. A set of four, I-section steel columns were vertically positioned and fixed to the strong floor. The columns were located at each corner of the repaired wall and connected to each other with four steel beams to provide a rigid restraining system. The test frame was built with the intention of eliminating out-of-plane displacements during testing. Figure 4.35 shows the constructed experimental test setup.



Figure 4.35 Assembled experimental test setup

Figure 4.36 illustrates the top view of the test setup. The two steel beams that are parallel to the length of the cap loading beam were positioned at the same elevation of the loading point (mid-height of cap beam). Each of these beams has four customizable rollers that were adjusted to make contact with the cap loading beam to restrain out-of-plane movement during the experimental test, as depicted in Figure 4.36 (a). Moreover, Figure 4.36 (b) represents the top view of the actual test assembly.

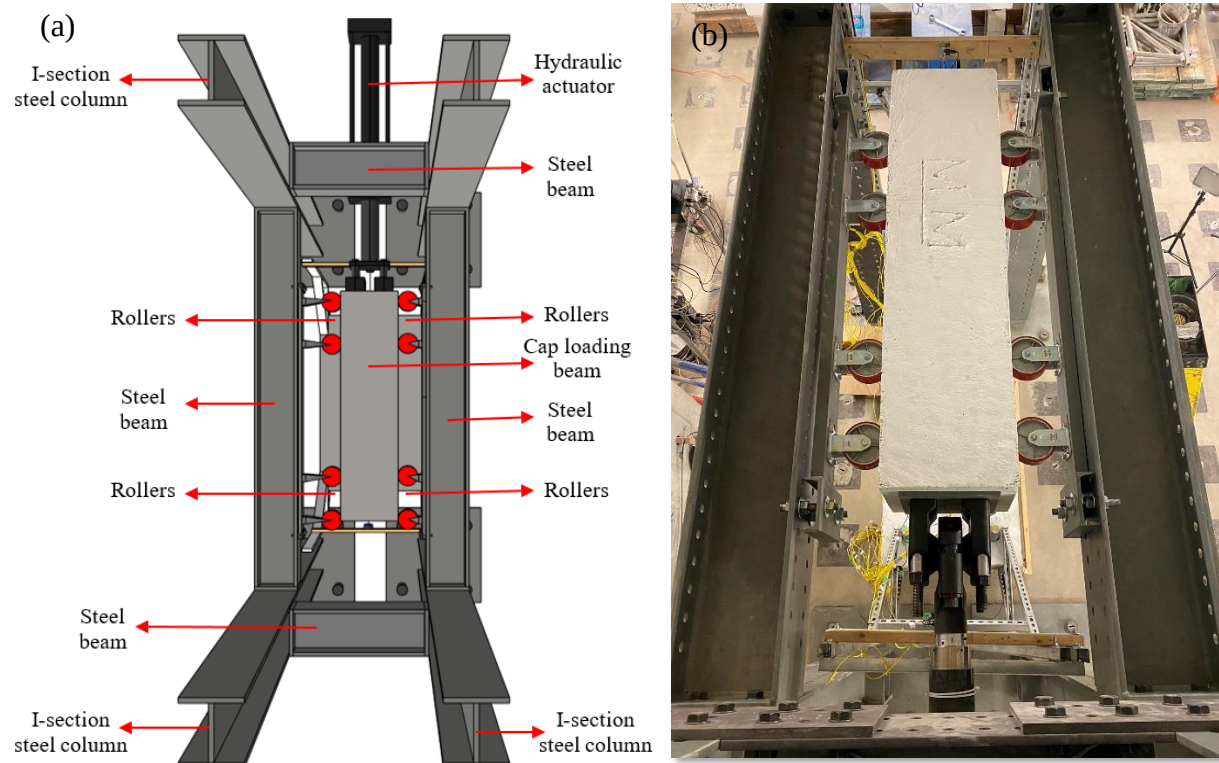


Figure 4.36 Top view of the test assembly: (a) graphic representation; and (b) actual experimental setup

4.4.2 Instrumentation

The response of each wall was monitored and recorded via strain gauges, linear and cable potentiometers. Lateral and vertical displacements at different heights, as well as diagonal elongations in the plastic hinge zone of the repaired walls, were measured.

4.4.2.1 Strain Gauges

A total of 34 strain gauges were installed in each wall at strategic locations where the damage was expected to be substantial, with the objective to evaluate the behavior of the reinforcing bar during testing. With a gauge length of 5 mm and a width of 2 mm, the general purpose, FA-series strain gauges were selected. Furthermore, these strain gauges are characterized with a gauge factor of 2 and are capable of measuring strains up to 4%.

Furthermore, prior to the installation of the gauges, lead wires were welded to the gauges, and a surface preparation was required on the reinforcing bars, where the strain gauges were attached. First, using a belt sander, the surface of the bars were smoothened. Second, the smoothened surface was cleaned with alcohol to remove dirt. Third, the gauges were attached to the bars utilizing liquid glue. Finally, the strain gauges were covered with liquid rubber and subsequently with a two-component epoxy to protect them during the casting of the ECC. The strain gauges installed on the new segments of steel reinforcing bars placed in the repaired walls are shown in Figure 4.37.



Figure 4.37 Installation of strain gauges on new segments of steel bars

Moreover, the layout and numbering of the strain gauges for Wall RSWS-R and Wall RSWN are provided in Figure 4.38.

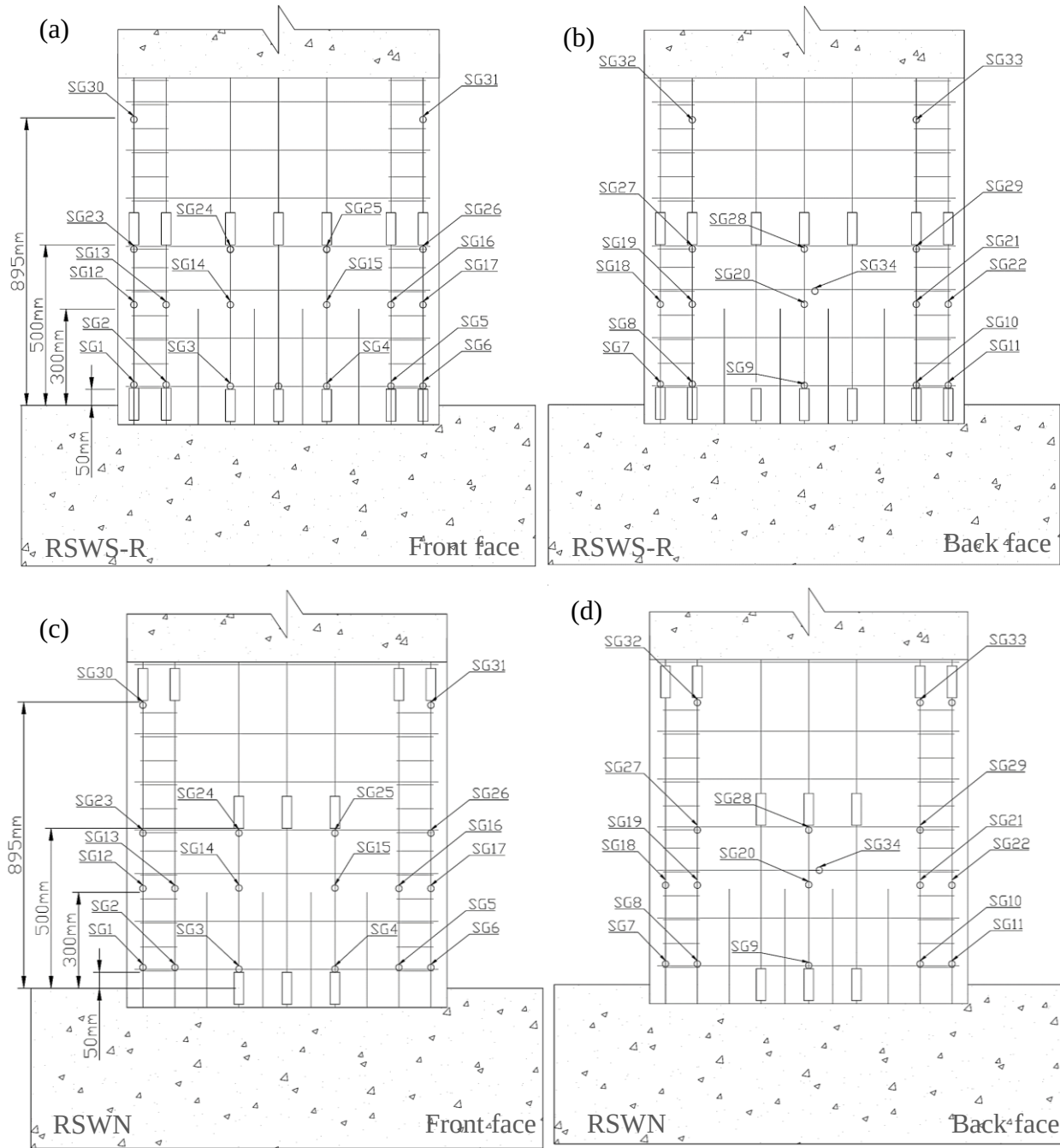


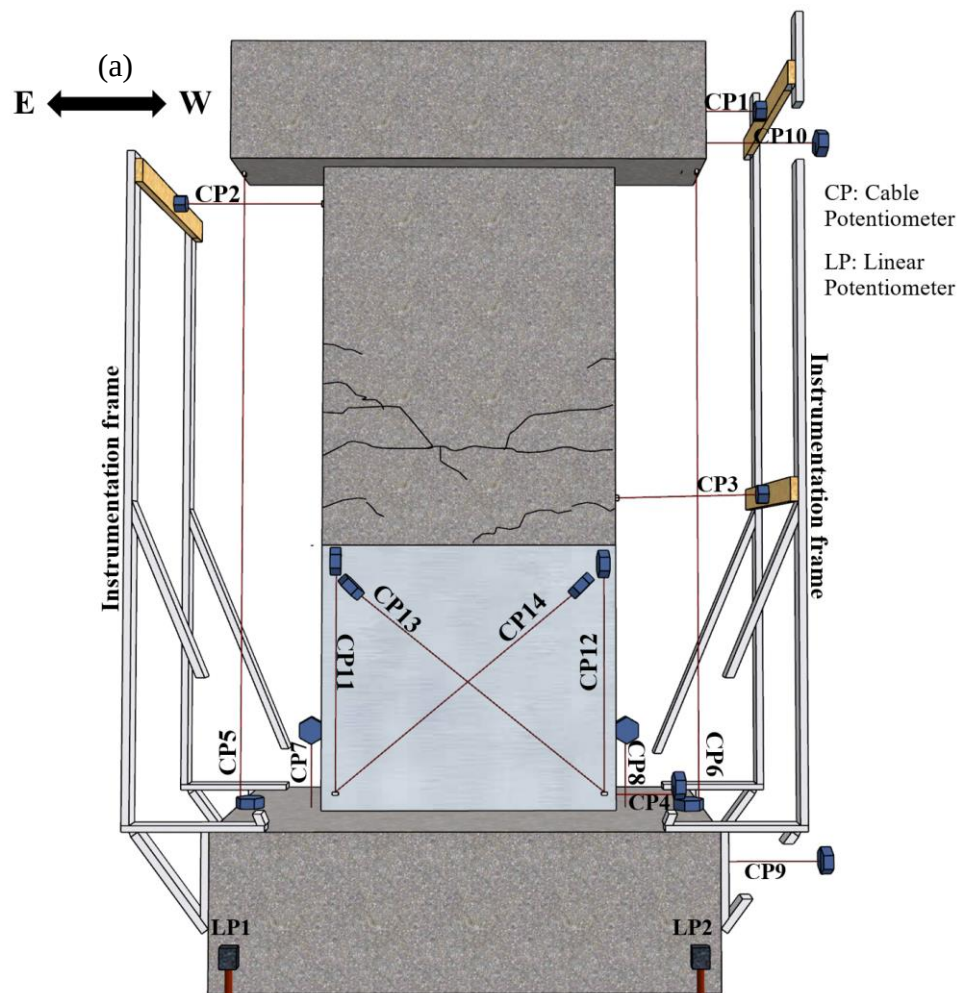
Figure 4.38 Strain gauge location: (a) Wall RSWS-R (front); (b) Wall RSWS-R (back); (c) Wall RSWN (front); and (d) Wall RSWN (back)

The strain gauges (SG) were located in the same position for both walls to allow comparison. The first row of strain gauges, comprising SG1 to SG11 were located 50 mm above the base of the walls, above the bottom mechanical couplers to analyze the behavior of the reinforcement at this location. The second row of strain gauges was located at 300 mm height measured from the base

of the walls (SG12 to SG22). It was assumed that a critical section would develop above the ends of the starter bars, due to the change of stiffness given the amount of reinforcement present at this location. Similarly, the third row of strain gauges (SG23-SG29) was installed at the height of 500 mm, below the upper mechanical couplers. Finally, the last row of strain gauges (SG30-SG33) was attached at the limit of the plastic hinge region to measure the strains at this location. No damage was expected in the shear reinforcement; nevertheless, SG34 was installed in one of the stirrups at the height of 450 mm measured from the base of the wall to study its behavior.

4.4.2.2 Cable and Linear Potentiometers

Figure 4.39 (a) shows a graphic representation of the instruments utilized on the repaired walls, Figure 4.39 (b) depicts the exact location of each instrument, and Figure 4.39 (c) provides a photo of the instrumentation on the walls.



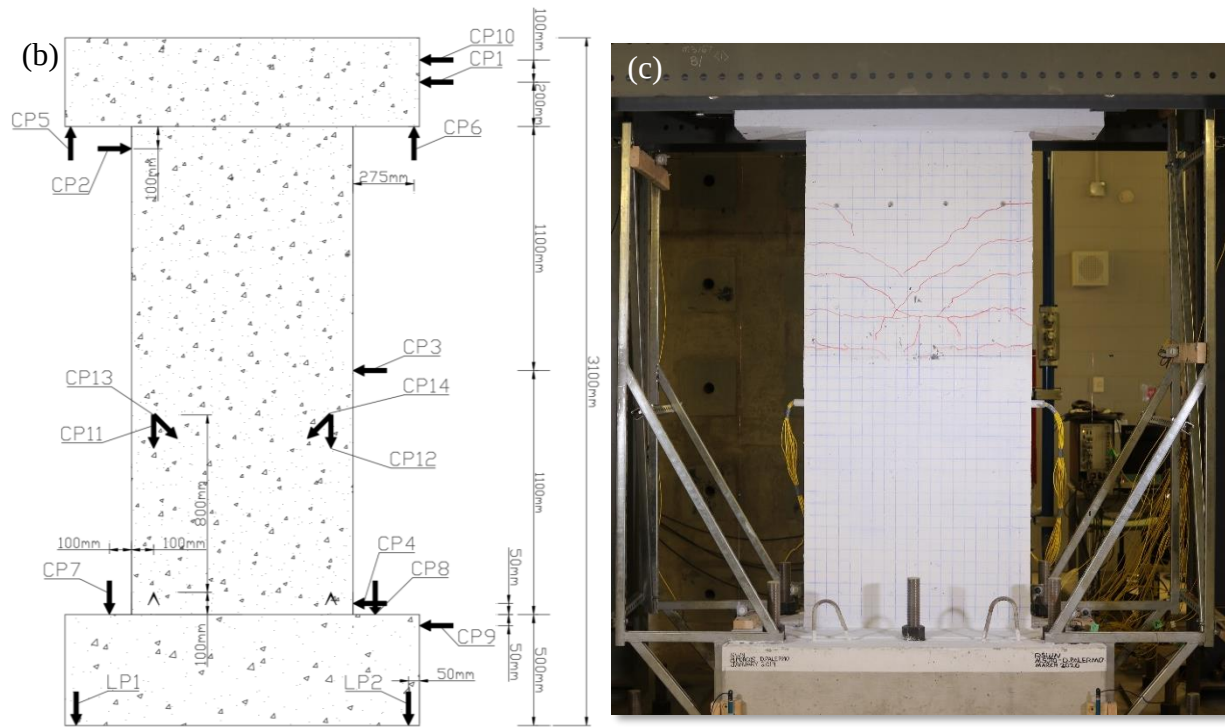
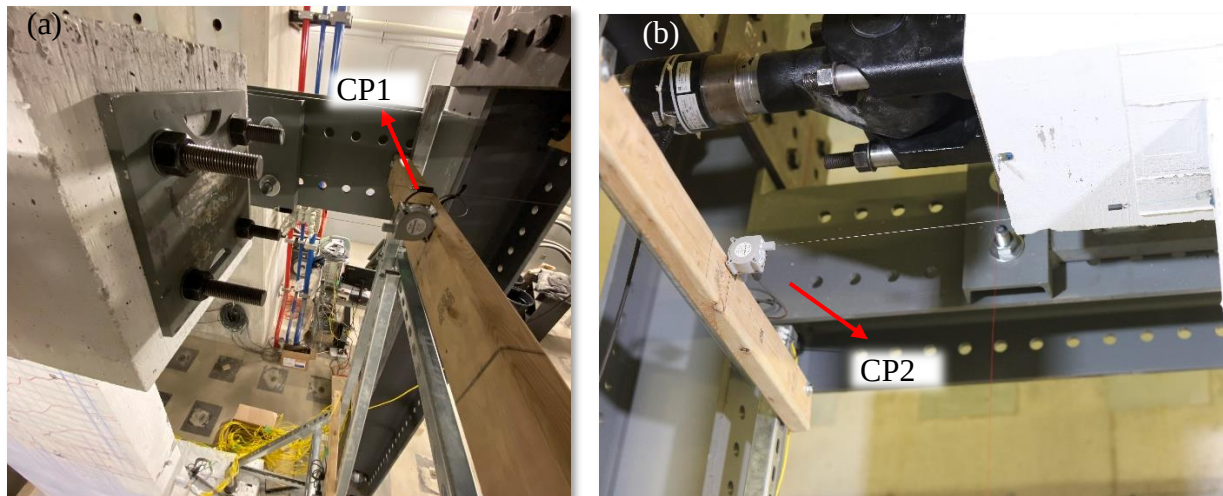


Figure 4.39 Instrumentation: (a) schematic illustration; (b) location and; (c) photo of instrumentation on the wall

A stiff instrumentation frame was installed on the repaired walls to enable measurements relative to the foundation block. CP1, CP2, CP3 and CP4 recorded horizontal displacements at different heights under this premise. Figure 4.40 shows the location of each of these cable potentiometers.



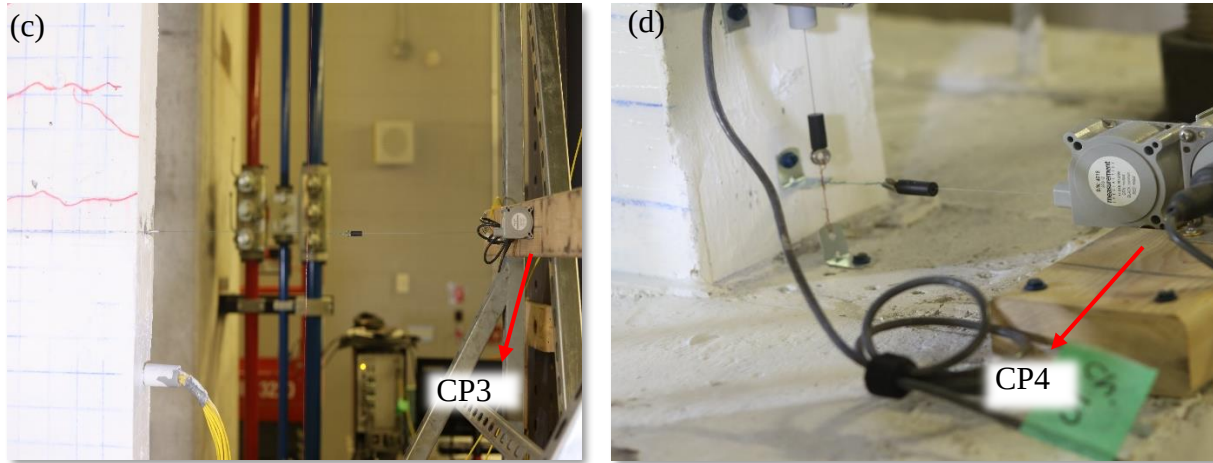


Figure 4.40 Location of cable potentiometers for lateral displacement measurements: (a) CP1; (b) CP2; (c) CP3; and (d) CP4

CP1 (Figure 4.40 (a)) was utilized to measure the lateral displacement at the loading point at the mid-span of the cap beam. Moreover, CP1 represented the reference point in which the target displacements were determined to perform the loading protocol in the experimental testing. CP2 (Figure 4.40 (b)) recorded the horizontal displacement at the top of the wall, under the cap beam. CP3, as shown in Figure 4.40 (c), monitored the lateral displacement at the mid-height of the wall. Finally, CP4 was employed to measure the horizontal displacement at the base of the wall (slip relative to the foundation), as represented in Figure 4.40 (d). The location of cable potentiometers CP5, CP6, CP7 and CP8 are shown in Figure 4.41.

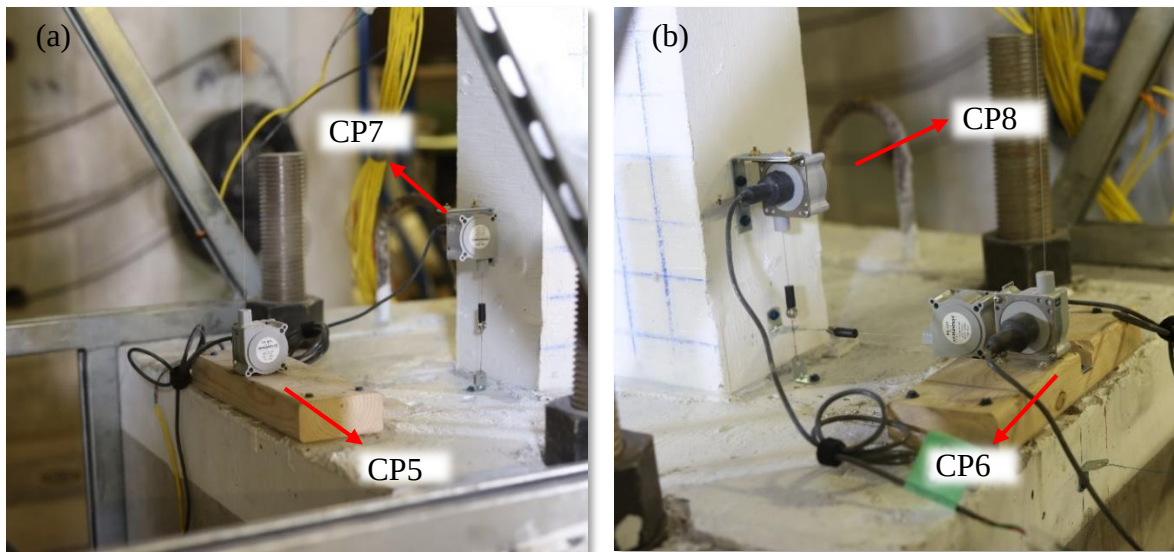


Figure 4.41 Location of cable potentiometers for vertical displacement measurements: (a) CP5 and CP7; and (b) CP6 and CP8

CP7 and CP5 shown in Figure 4.41 (a) were used to measure the vertical displacements relative to the foundation block on the east side of the repaired walls. Likewise, CP6 and CP8 presented in Figure 4.41 (b) followed the same approach on the west side of the repaired structures. CP7 and CP8 measured the local vertical separation of the wall from the foundation, while CP5 and CP6 measured the vertical elongation over the entire height of the walls.

CP9 shown in Figure 4.42 (a) measured sliding of the foundation block, and CP10, represented in Figure 4.42 (b), recorded the horizontal displacements 100 mm above the loading point (where CP1 is located), relative to the strong floor.

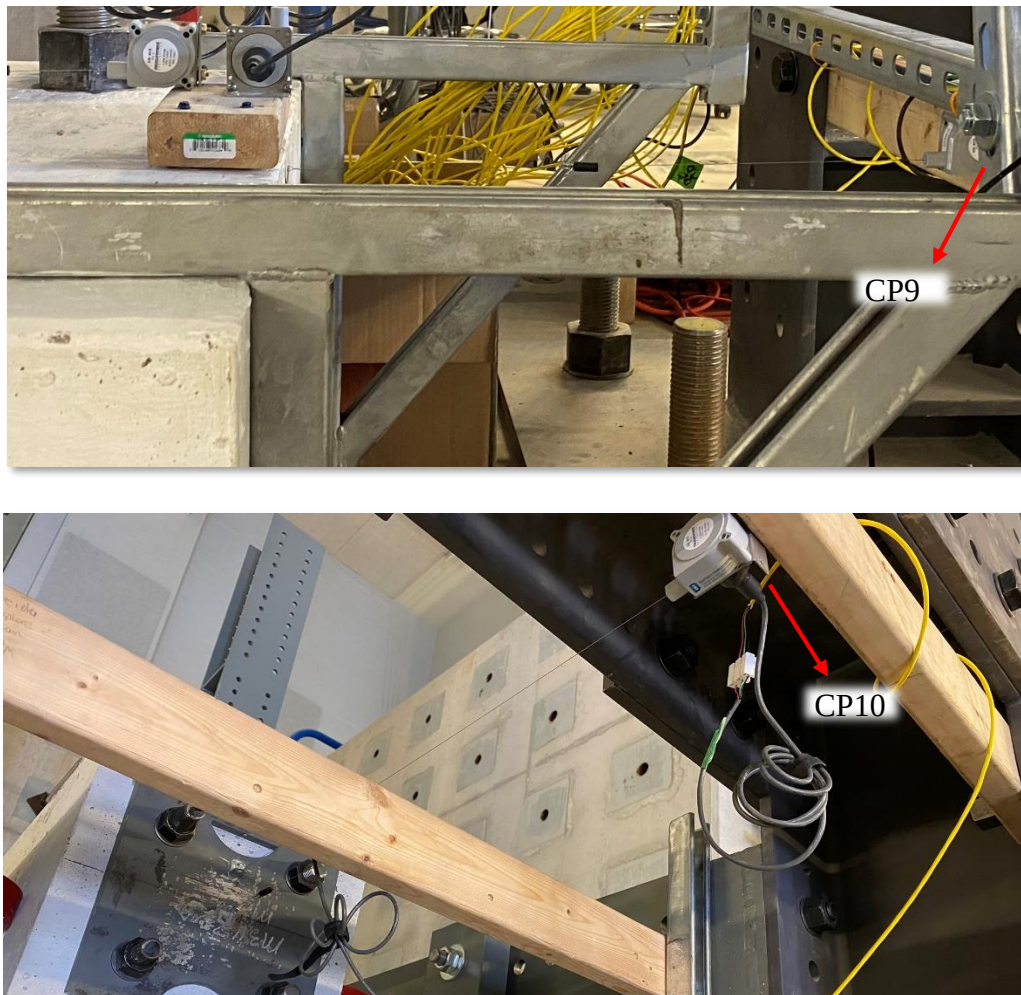


Figure 4.42 Cable potentiometers: (a) sliding of foundation block (CP9); and (b) top lateral displacement relative to the strong floor (CP10)

Vertical displacements over the plastic hinge were recorded with CP11 and CP12, located 800 mm above the base of the wall. Additionally, CP13 and CP14 were connected along the same line of action as CP11 and CP12 to calculate diagonal deformations. Figure 4.43 shows the cable potentiometers installed in the plastic hinge zone.

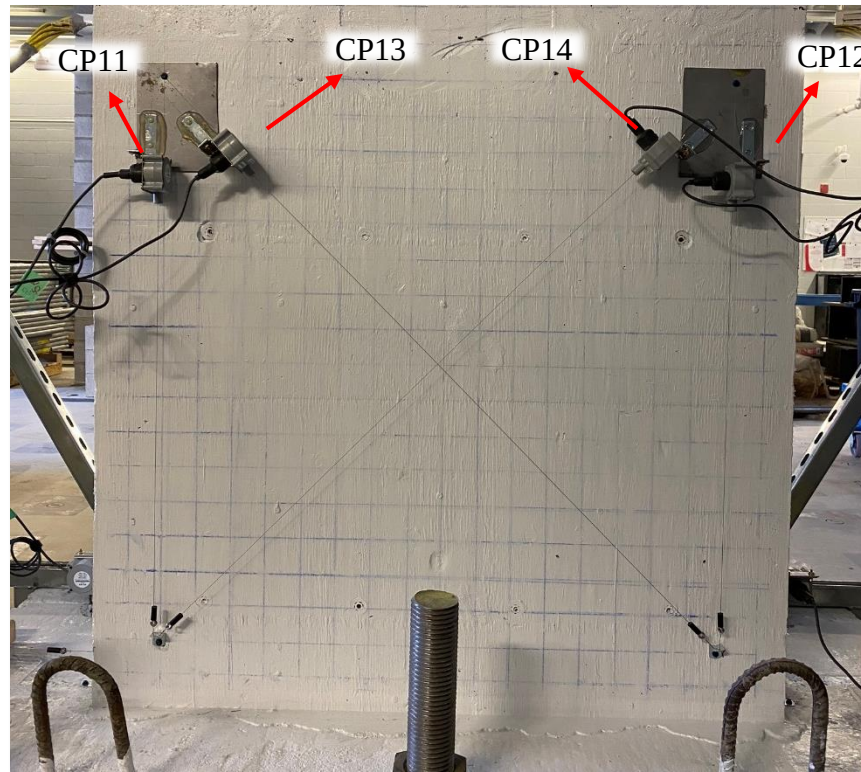


Figure 4.43 Cable potentiometers utilized to measure vertical and diagonal displacements in the plastic hinge
 Lastly, the linear potentiometers LP1 and LP2 monitored the uplift of the repaired structures on the east and west side, respectively, as shown in Figure 4.44.

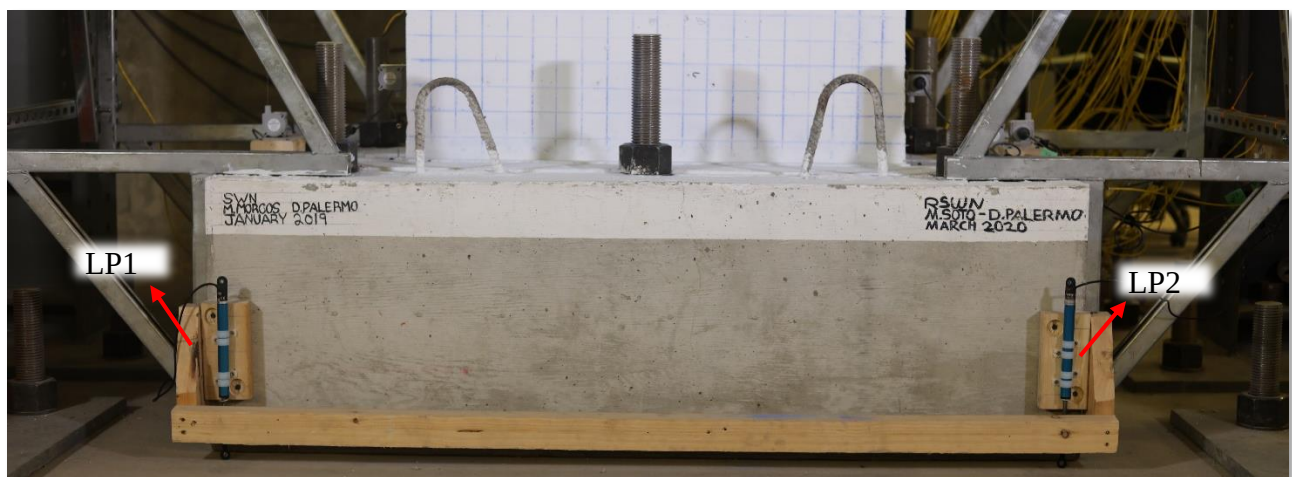


Figure 4.44 Linear potentiometers employed to measure uplift of the foundation block

The readings from the instrumentation (strain gauges, and cable and linear potentiometers) were recorded through a series of Data Acquisition Systems (DAQ's) connected to the Central Processing Unit (CPU) used to control the experimental testing of the repaired walls.

4.4.3 Loading Protocol

The majority of research projects involving experimental testing of reinforced concrete structures under seismic load simulations employ quasi-static cyclic loading through hydraulic actuators. This approach does not follow a strain rate or specific displacement history subjected by a particular earthquake; it rather exerts to the structure predetermined numbers of displacement controlled quasi-static loading cycles to pre-established displacement ductility factors (Park, 1988).

The loading protocol was identical to that one applied to the original walls by Morcos and Palermo (2019). The approach taken is intended to evaluate the pre- and post-repairing conditions of the walls under the same loading. The suggestions provided by FEMA 461 (2007) and recommendations of the ATC-24 (1992) formed the loading/unloading displacements utilized during testing. Morcos and Palermo (2018) performed preliminary pushover analysis for Walls SWS-R and SWN to estimate the yield displacements. The target displacements were adapted as a single loading history suitable for both walls based on the yield displacement of SWN ($\Delta y = 24$ mm) (Morcos and Palermo, 2019). The loading cycles were divided into two groups: pre- and post-yield lateral displacement phases. The first loading phase consisted of 3 repetitions of seven targeted symmetrical displacements of $1/20 \Delta y$, $1/10 \Delta y$, $1/5 \Delta y$, $3/10 \Delta y$, $2/5 \Delta y$ and, $1/2 \Delta y$ to reach the yield displacement Δy , thereafter, two repetitive cycles were imposed with displacement increments of $0.5 \Delta y$ until $5 \Delta y$ for Wall SWS-R. Wall SWN continued with three repetitive cycles for each target displacement up to $2 \Delta y$ and two cycles per subsequent displacement levels thereafter. Figure 4.45 and Figure 4.46 depicts a graphic representation of the loading protocol for Walls RSWS-R and RSWN, respectively.

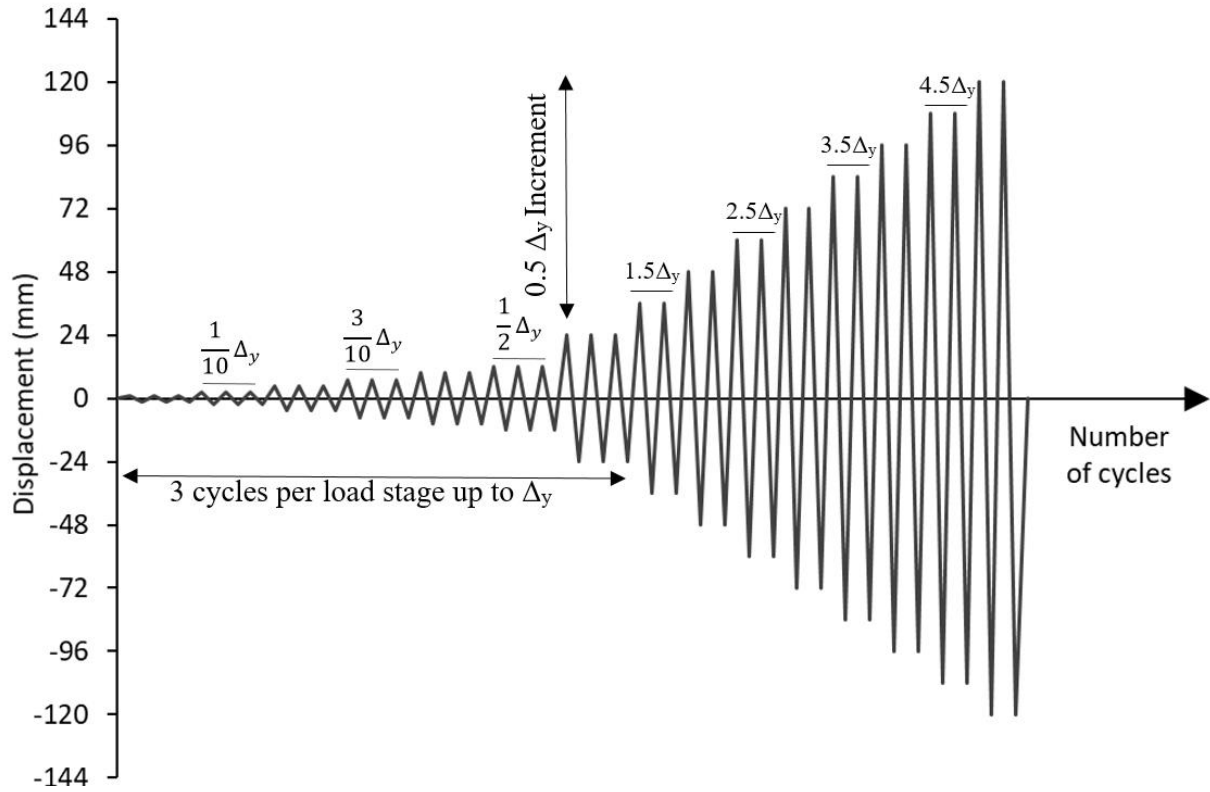


Figure 4.45 Loading protocol for Wall RSWS-R

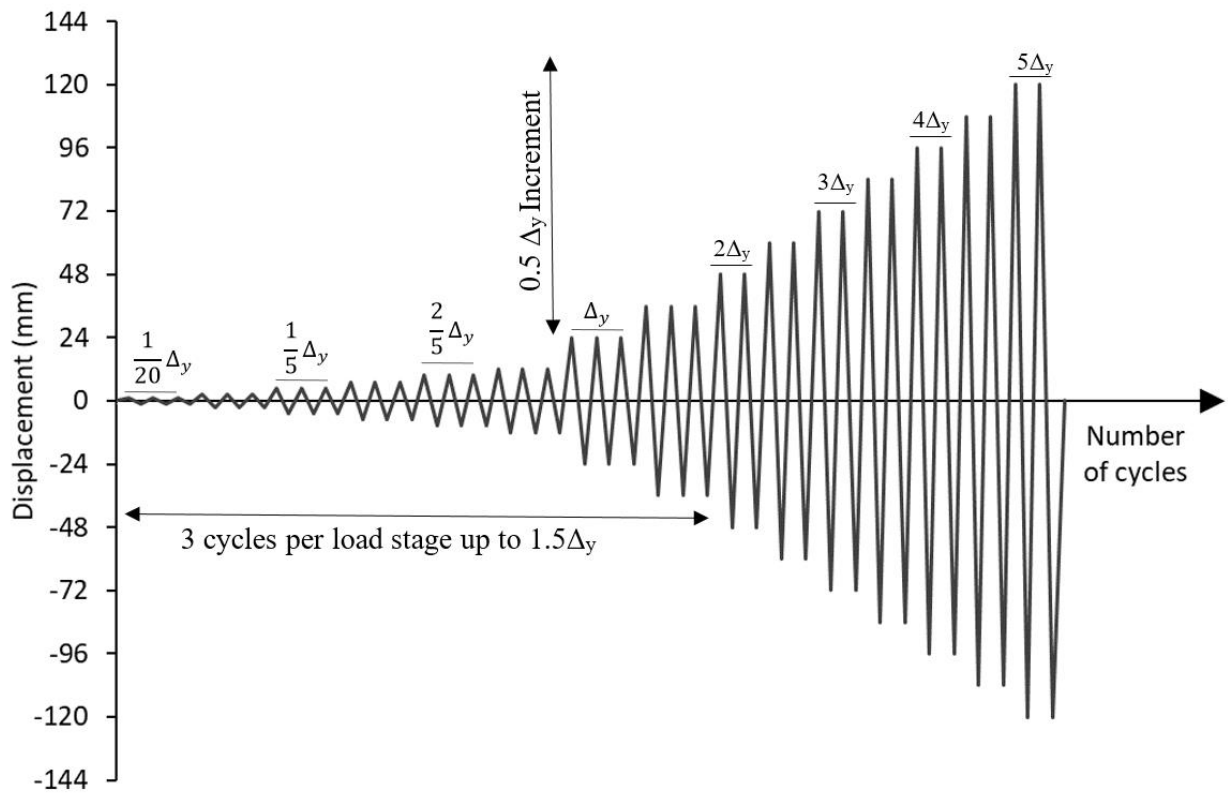


Figure 4.46 Loading protocol for Wall RSWN

The MTS Test-Suite Multipurpose Elite (MPE) Software was the computational tool used to control the force and stroke of the hydraulic actuator during the experimental testing of both walls. It requires the input of the loading protocol. Morcos (2020) designed the guidelines tailored for the testing of the original walls inside the software. The same protocol was used for the repaired walls. Notice that the actuator stroke does not represent the actual displacement during testing of the walls. Therefore, it became necessary to manually control the loading protocol through the entire experimental testing. Figure 4.47 summarizes in a flow chart the programmed procedure inside the software.

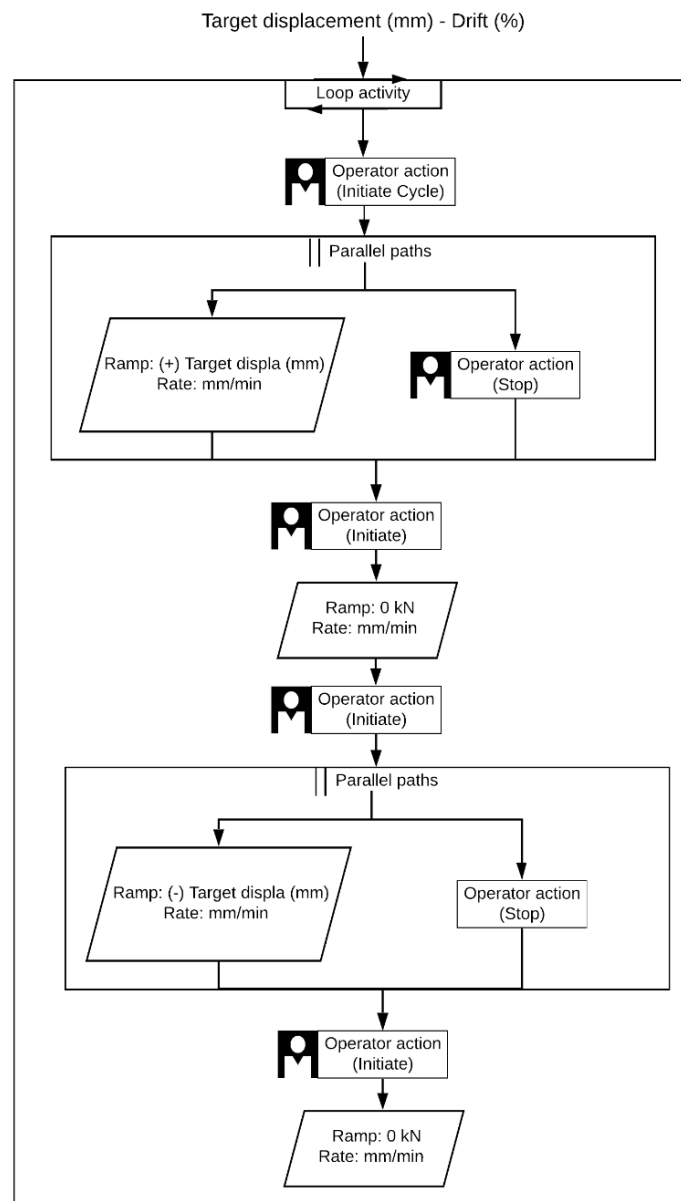


Figure 4.47 Summarized flow chart of the loading protocol programmed in the MTS - MPE software

Going into a specific target displacement, the software enters into a loop activity which requires the input of the number of cycles to be performed. Thereafter, the operator initiates the loading manually through a displacement-controlled rate in the positive direction with a previously established rate in millimeters per minute. At the same time, the option to manually stop the loading becomes available. The operator is required to stop the loading at the moment of reaching the (+) target displacement based on the readings from the cable potentiometer CP1, explained previously in Section 4.4.2.2. Thereafter, the operator initiates the unloading phase, ramping down to zero kN with a predefined unloading rate in millimeters per minute. The software automatically stops when the load in the actuator reaches zero kN. Subsequently, the same procedure initiates in the negative direction; the operator will be required to start the loading in the negative direction heading to the (-) target displacement, thereafter, manually stopping the loading once it reaches the required displacement. Finally, the unloading phase initiates and terminates when it reaches zero kN. Once the number of repetitions is completed, the next target displacement commences. The same approach continues until the end of the programmed loading protocol.

CHAPTER 5. EXPERIMENTAL RESULTS

The chapter presents the experimental results obtained from the different tests performed to characterize the mechanical properties of the Engineered Cementitious Composite (ECC) and the reinforcement used as the primary repairing materials. Thereafter, the experimental results recorded from testing the repaired walls are presented, including the crack widths and patterns at the target displacements, lateral load-displacement responses and lateral displacements at along the height of the walls.

5.1 Material characterization

5.1.1 Concrete Properties

5.1.1.1 Cylinder Compression Strength and Failure Crack Pattern

Two sets of concrete cylinders were tested for Wall RSWS-R: the first set represents the ECC in-house mix, and the second set considers the SCC mixture used to repair the voids encountered within the wall. Each set contains six cylinders: three for the 28-day compressive tests and the remaining for the day of testing. Wall RSWN has two sets of ECC with three cylinders for the 28-day test and three for the test day. The average compressive strength of the three cylinders per set is listed in Table 5.1. It can be noted that the in-house mix used in two different batches resulted in similar compressive strengths, proving consistency.

Table 5.1 Cylinder compression strength of walls

Designation	Compressive Strength f'_c (MPa)	
	28 Days	Test Day
ECC-RSWS-R	49.07	64.03 (152 days)
SCC-RSWS-R	37.20	59.38 (148 days)
ECC-RSWN	46.06	62.94 (151 days)

After testing, the ECC cylinders for Wall RSWS-R (Figure 5.1 (a)) and RSWN (Figure 5.1 (b)) developed similar crack patterns. The presence of PVA fibers bridged through the cracks eliminating brittle failure and maintaining the original integrity of the specimen. Most cylinders exhibited a ductile shear crack failure mode varying between 45° to 90° . Few cylinders experienced longitudinal columnar vertical cracks starting from the top horizontal surface. All the cracks varied from hairline to hairline cracks proving a well-confined specimen. The Self-Consolidating Concrete (SCC) cylinders shown in Figure 5.1 (c) presented a less ductile behaviour than the ECC cylinders yet remained intact during testing. The absence of fibers resulted in concrete crushing and spalling on both edges of the cylinder accompanied with medium to wide vertical cracks. The shear cracks were observed on both sides of the specimen, reducing the strength of the cylinder prior to failure.

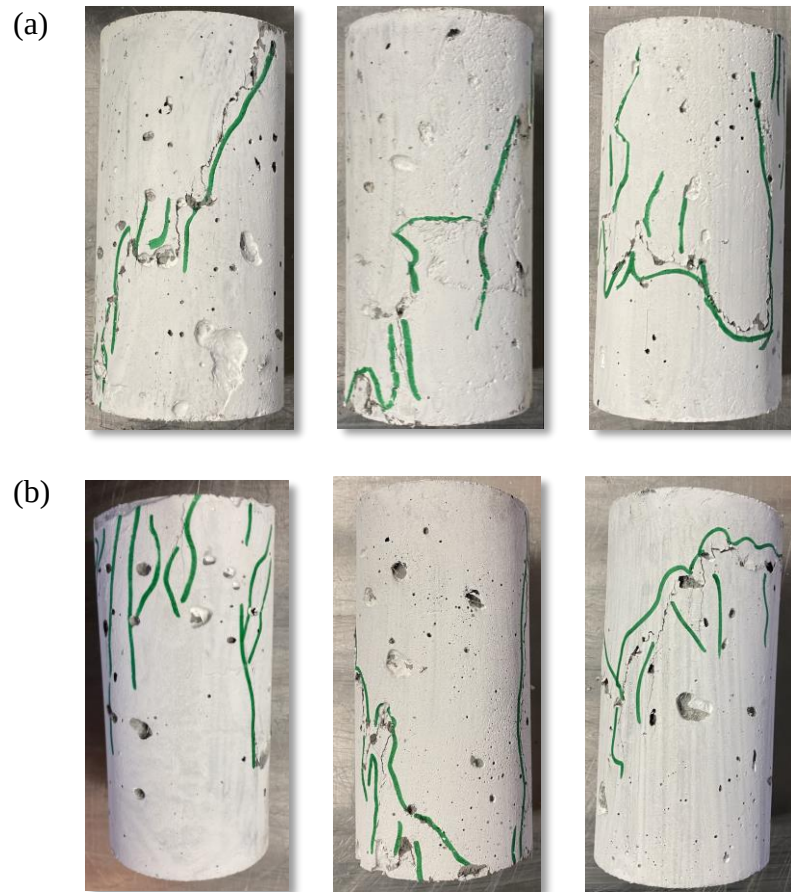
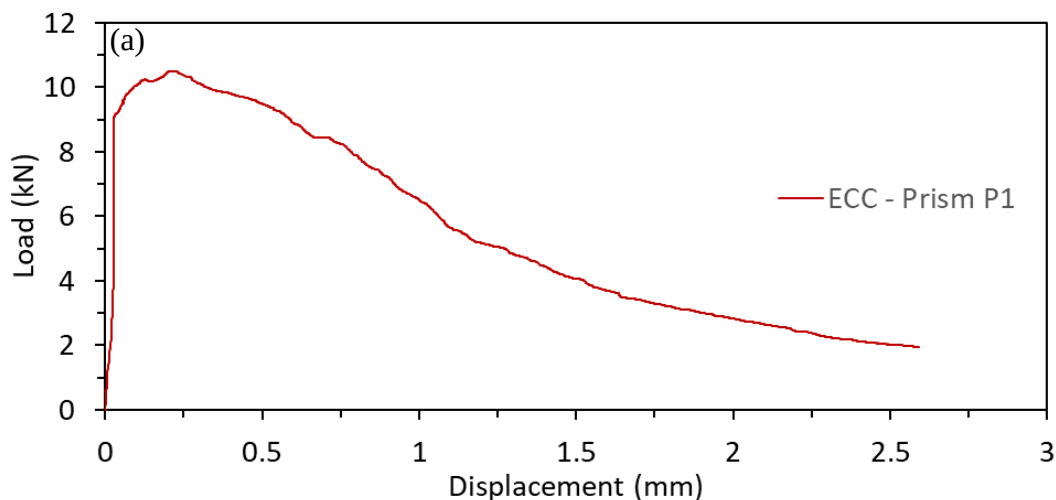




Figure 5.1 Compression cylinder failure mode: (a) ECC-RSWS-R; (b) ECC-RSWN; and (c) SCC-RSWS-R

5.1.1.2 Flexural Strength

The typical total vertical load versus mid-displacement response of Prism P1 and Prism P2 are presented in Figure 5.2 (a) and (b), respectively. The prisms were cast from the ECC batch for Wall RSWS-R and tested at the age of 28-days. The responses present a noticeable ductile behavior, characterized by a plateau stage at the initiation of cracking and prior to reaching the peak load. Additionally, both prisms developed a stiff initial response where the first crack developed at 85% of the peak load for Prism P1 and 75% for Prism P2. Both curves are indicative of the development of several cracks during loading marked by small local disturbances occurring upon the opening of cracks. The descending branch of both prisms follows a gradual softening behavior. The ultimate point was defined by a 20% drop of peak strength. At this stage, Prisms P1 and P2 experienced displacements of 0.72 mm and 1.48 mm, respectively.



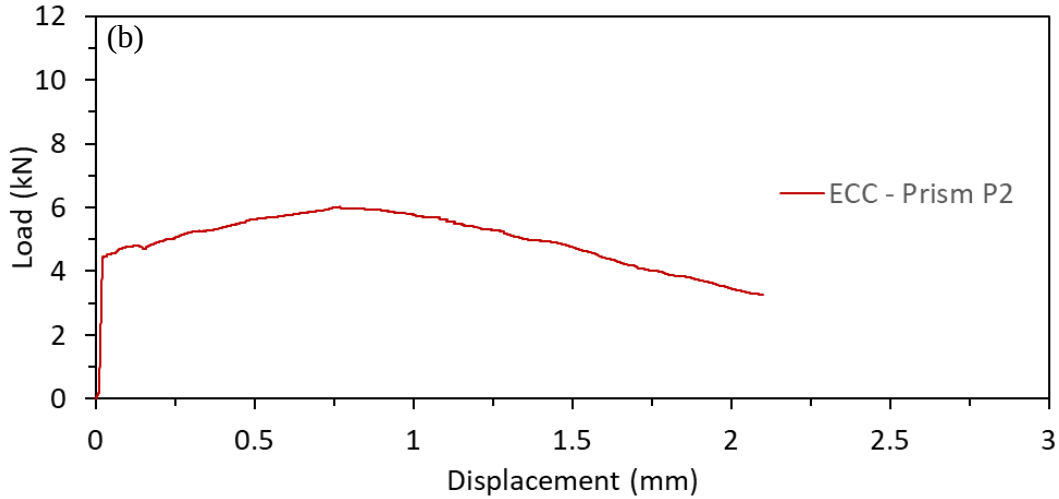


Figure 5.2 Load-mid-span displacement of ECC prisms: (a) P1 (280 mm); and (b) P2 (500 mm)

The salient points of the load-displacement responses are presented in Table 5.2, including the peak load (F_{peak}) and corresponding mid-span displacement (Δ_{peak}), and the maximum flexural strength (f_{max}) calculated based on Section 4.1.3.

Table 5.2 Tensile flexural strength properties of mix ECC-RSWS-R

Designation	Prism	Dimensions (mm)	F_{peak} (kN)	Δ_{peak} (mm)	f_{max} (MPa)
ECC-RSWS-R	Prism P1	75 x 75 x 280	10.50	0.22	5.60
	Prism P2	75 x 75 x 500	6.02	0.77	6.42

Figure 5.3 illustrates the condition of the prisms on the bottom and side faces at the end of testing. The presence of the PVA fibers eliminated the split brittle failure mode of the prisms. Both prisms developed a localized dominant crack within the constant moment region. The major crack for Prism P1 was at the same vertical location as the loading roller, while for Prism P2 it was in the center of the prism. Multiple fine cracks were observed along the length of the prisms and in correspondence with the flexural response curve provided in Figure 5.2. The fibers were able to bridge through and control the opening of the dominant crack, allowing for the formation of other finer cracks.

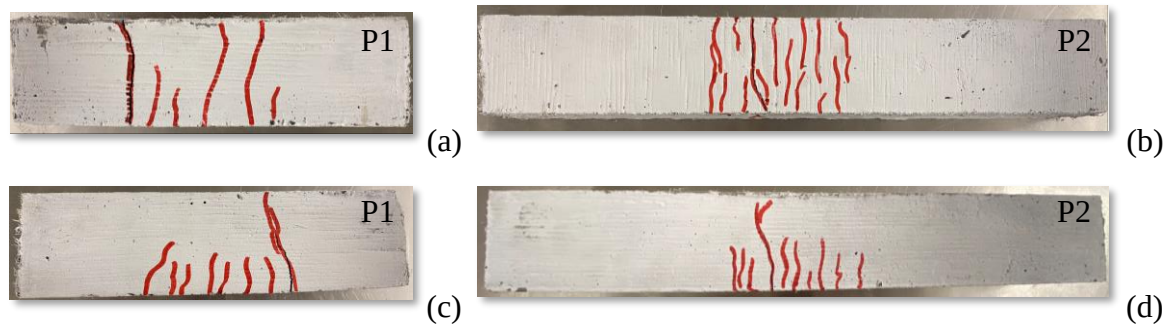


Figure 5.3 State of prisms at the end of testing: (a) P1 bottom; (b) P2 bottom; (c) P1 side; and (d) P2 side

5.1.2 Reinforcement Properties

The 10M bars utilized in the repaired walls were all obtained from the same source; a series of uniaxial mechanical tests were performed to establish the mechanical properties. Figure 5.4 presents the typical stress-strain response of the reinforcing steel under monotonic loading.

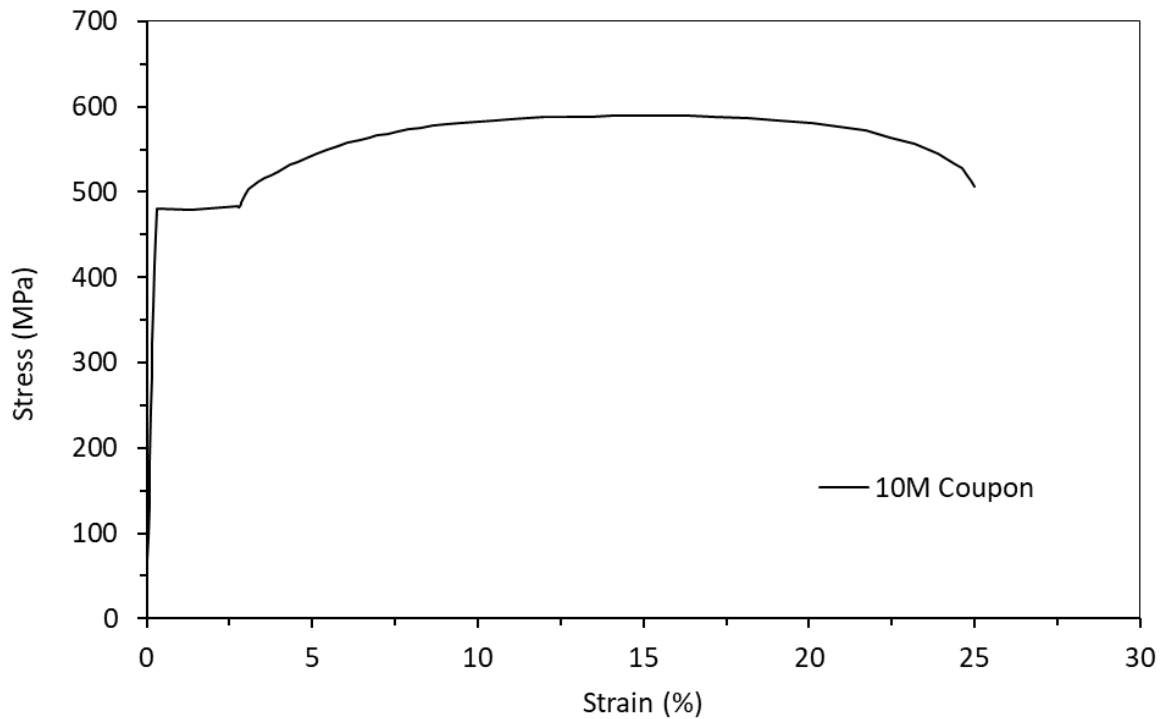


Figure 5.4 Typical monotonic stress-strain for 10M steel coupon

The response shows the traditional trilinear response for mild reinforcing steel, initiating with a linear-elastic ascending portion prior to reaching the yielding point, passing through the plastic region and finalizing with a strain hardening that led to the necking effect and subsequent rupture of the

coupon. The Young's Modulus (E) was calculated as the slope of the steep elastic section of the curve, with a value of approximately 195 GPa. The yield strength (f_y) was 485 MPa with 0.21% strain, and the maximum tensile strength (F_u) was 590 MPa with 14.32% strain. The bar ruptured at the mid-span, where the cross-section was reduced, in between the 2-inch (50 mm) gauge of the extensometer, as depicted in Figure 4.4. The rupture strain was 24.3%.

As discussed in Section 1.2.2, the Superelastic – Shape Memory Alloy (SE-SMA) has the capacity to fully recover from up to 6% strain when unloaded. Morcos (2020) performed a hybrid monotonic loading with a 6% strain cycle on the SE-SMA bars coupons used in the original Wall SWN. Figure 5.5 illustrates the stress-strain response with an identical 6% strain cycle of the 10M reinforcing bars used in the repaired walls.

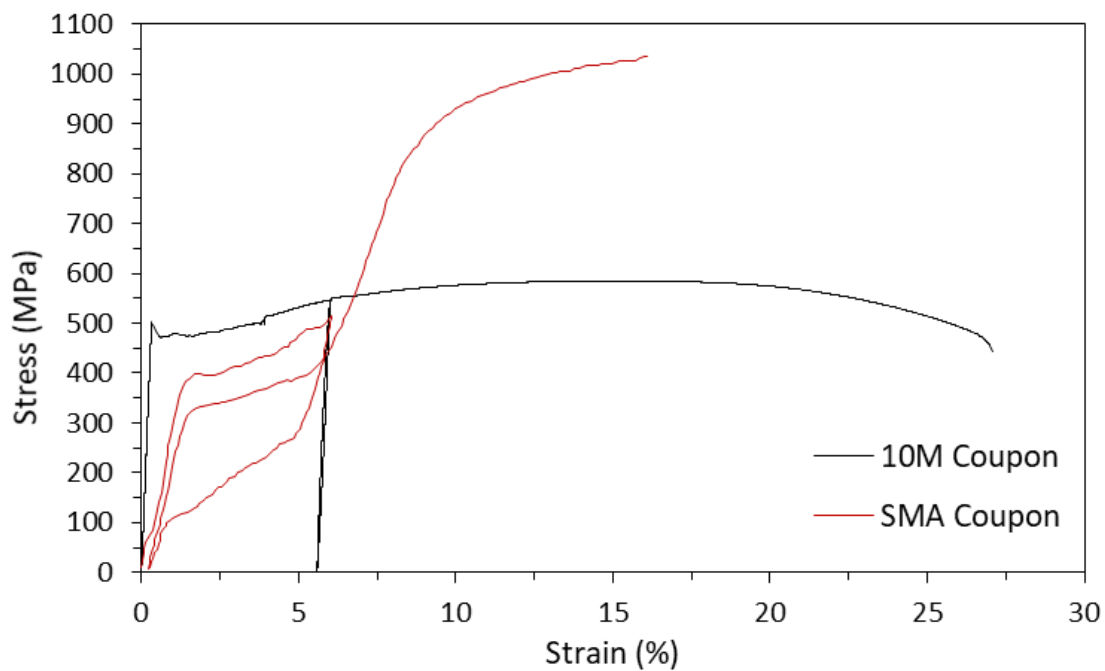


Figure 5.5 Stress-strain response with 6% strain cycle of 10M steel and SMA bars (Morcos, 2020)

The superelasticity property of the SMA coupon is evident at 6% strain, the SMA developed a stress of 495 MPa and thereafter, when unloaded to zero stress, the SMA coupon experienced a negligible plastic offset of 0.18% strain prior to reloading to failure. The SMA failed prematurely at a stress of 1034 MPa with 16% strain by rupture of the top conical head of the mechanical coupler used in the original SWN wall (Morcos, 2020). At 6% strain, the 10M steel coupon was located in the strain

hardening region, with a stress of 534 MPa. When unloading to zero stress, the steel coupon experienced a residual strain of approximately the same value (6%). Thereafter, the steel coupon was loaded to failure, reaching a maximum tensile strength of 584 MPa with a 16.6% strain.

Morcos (2020) tested an additional set of SMA coupons under several cyclic excursions. The same loading protocol was followed by the steel reinforcement used in the repaired walls. The results are plotted in Figure 5.6 and confirmed during the unloading and reloading branches that the curve of the SMA bars experienced considerably more nonlinearity in comparison to the 10M bar.

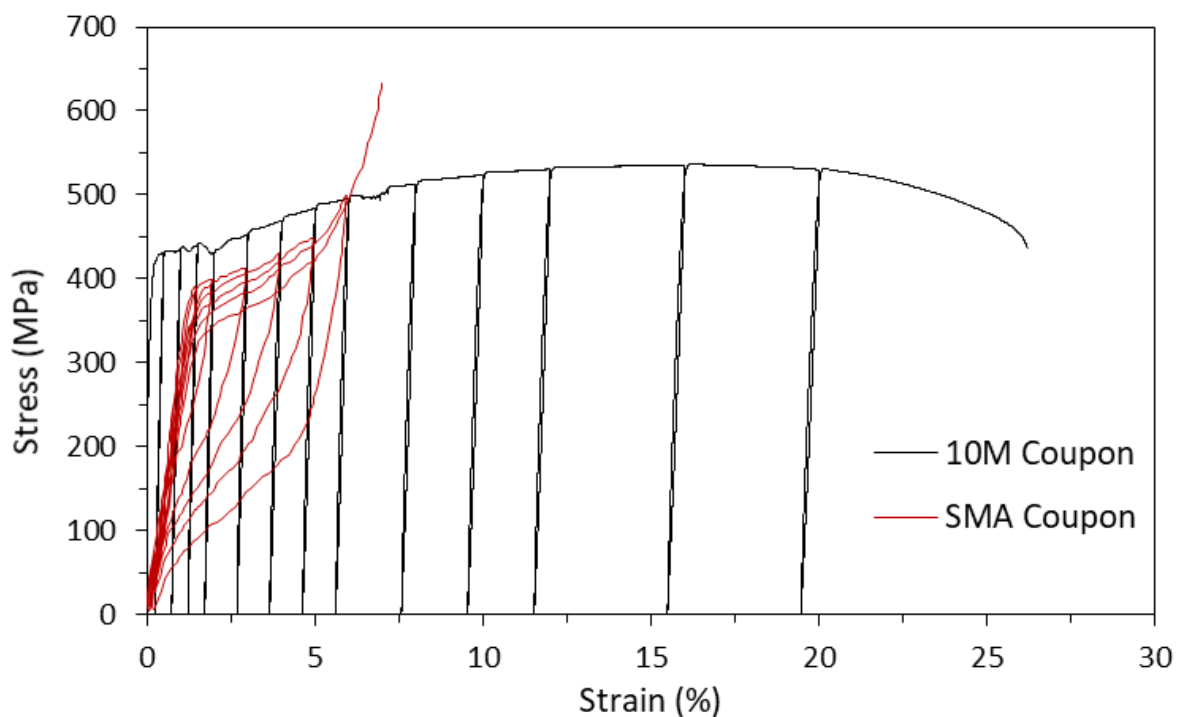


Figure 5.6 Cyclic stress-strain response of reinforcement utilized in repaired walls

The SMA coupon experienced premature failure due to rupturing at the top of the reduced cross-section (Morcos, 2020) during loading to the 6% strain cycle. However, the response of the SMA was notably different to the 10M coupon, specifically during unloading with negligible plastic strains.

5.2 Repaired Shear Walls Tests

5.2.1 Crack Characteristics

The crack characteristics of Wall RSWS-R and RSWN were monitored during testing. Detailed description at each target displacement is presented in this section. The crack pattern identified with the red color in the photos represents the damage sustained by the original walls (Morcos, 2020). The green color was used to mark the cracks developed during the experimental testing of the repaired walls. During each drift level, cracks were detected and then recorded in terms of width and length using a crack gauge and the grid drawn on the walls, respectively. The width and crack extensions were measured at four milestone points taken from the last repetition of the concerned target displacement (drift level): at maximum applied displacement in the positive direction $D_{max}^+ = (w_{D_{max}^+}; l_{D_{max}^+})$, at displacement when no load is applied $D_{zero}^+ = (w_{D_{zero}^+}; l_{D_{zero}^+})$, at maximum displacement in the negative direction $D_{max}^- = (w_{D_{max}^-}; l_{D_{max}^-})$, and at displacement when no load is applied $D_{zero}^- = (w_{D_{zero}^-}; l_{D_{zero}^-})$, as shown in Figure 5.7.

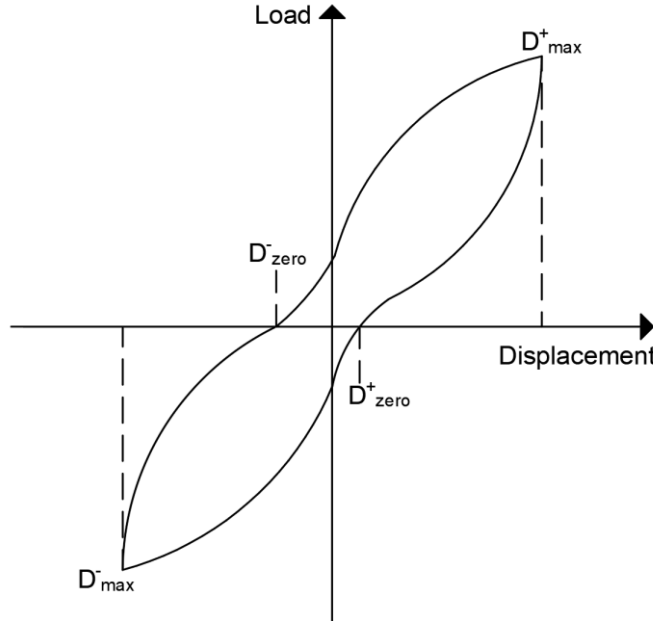


Figure 5.7 Milestone points for crack width and length measurements

No cracks were observed in Walls RSWS-R and RSWN during the three cycles of the 0.05% and 0.1% drift levels. During the applied drift of 0.2%, no crack appeared in Wall RSWS-R. For the SMA

reinforced wall in the positive loading direction of 0.2% drift, a 0.3 mm wide crack surfaced on the left side of the wall located 425 mm above the base of the wall, which extended for 300 mm. This flexural crack was denoted as **N1**. During the unloading phase of Wall RSWN, the same crack closed to a width of $w_{D_{zero}^+} = 0.1$ mm, as presented in Figure 5.8 (a). During loading of the last cycle in the negative direction, the previously existing flexural cracks located above the plastic hinge started to widen. The crack was denoted as **N2**, presented on the right side of the wall at a distance of 1250 mm from the base. The width and length of the D_{max}^- crack were 0.25 mm and 250 mm, respectively. The crack width reduced to 0.15 mm at the end of unloading in the negative direction, as depicted in Figure 5.8 (b).

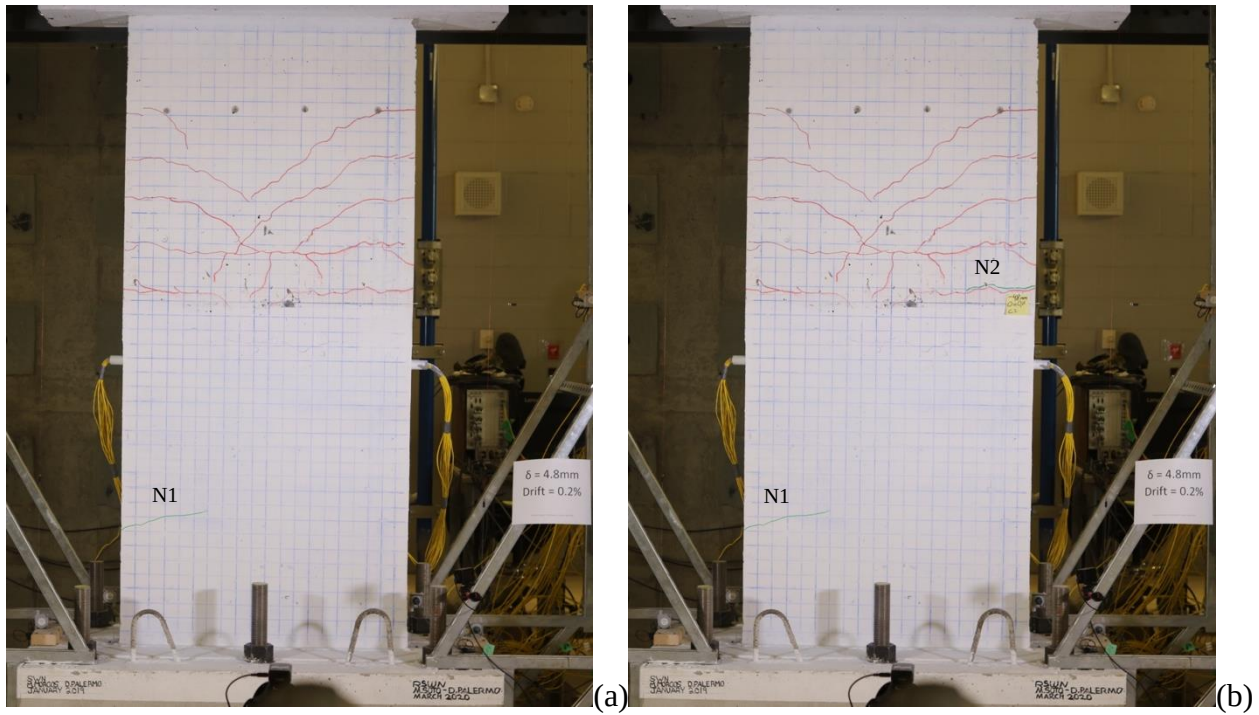


Figure 5.8 Crack pattern in RSWN at 0.2% drift: (a) initial crack N1 at $D_{zero}^+ = (0.1 \text{ mm}; 300 \text{ mm})$; and (b) second crack N2 at $D_{zero}^- = (0.15 \text{ mm}; 250 \text{ mm})$

At a drift of 0.3% (7.2 mm displacement), both walls experienced crack growth. The width of the existing crack located on the left side at 1250 mm from the base of Wall RSWS-R re-opened to a width of 0.15 mm. The crack was denoted as **S1**. During the same loading phase, the crack **N1** on Wall RSWN, introduced in Figure 5.8 (a) widened to $w_{D_{max}^+} = 0.6$ mm and extended a further 50 mm to reach $l_{D_{max}^+} = 350$ mm. During the unloading phase in the positive direction, the width of crack **S1**

on Wall RSWS-R reduced to 0.1 mm. Similarly, the width of crack **N1** for Wall RSWN reduced to 0.15 mm. Loading in the negative direction for Wall RSWS-R resulted in another previously existing flexural crack located at the same height as crack **S1** but on the opposite end to re-open to a width of $w_{D_{max}^-} = 0.15$ mm. This crack, at the end of unloading, had a width of $w_{D_{zero}^-} = 0.1$ mm. These two cracks merged in the middle of the wall and were labelled **S1**, as shown in Figure 5.9 (a). During the loading of Wall RSWN in the negative direction, crack **N2** introduced in Figure 5.8 (b) re-opened to $w_{D_{max}^-} = 0.2$ mm and propagated to reach a length of $l_{D_{max}^-} = 350$ mm. Simultaneously, the wall experienced an additional crack **N3** at the height of 400 mm above the base and opposite to crack **N1**. The crack characteristics of **N3** were $D_{max}^- = (0.4$ mm; 200 mm). In the unloaded position in the negative direction, the width $w_{D_{zero}^-}$ of cracks **N2** and **N3** were 0.15 mm and 0.10 mm, respectively. The three cracks **N1**, **N2** and **N3** for the SMA wall are shown in Figure 5.9 (b).

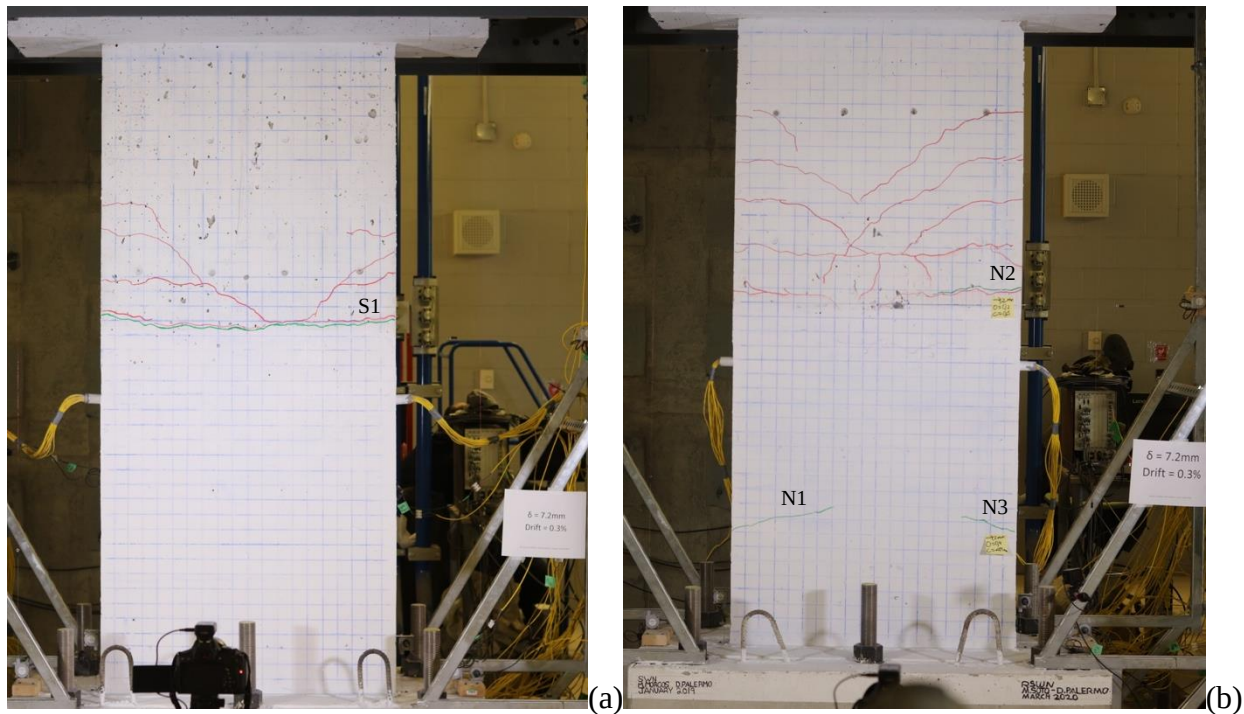


Figure 5.9 Crack pattern at 0.3% drift: (a) RSWS-R crack **S1** at $D_{zero}^- = (0.15$ mm; 1000 mm); and (b) RSWN crack **N1** at $D_{zero}^- = (0.15$ mm; 350 mm), **N2** at $D_{zero}^- = (0.15$ mm; 350 mm) and **N3** at $D_{zero}^- = (0.10$ mm; 200 mm)

At the completion of the third repetition at 0.4% drift, the conventional repaired steel wall did not develop any additional cracks. During the loading in the positive direction, the left side of crack **S1** widened to 0.18 mm and then recovered to 0.1 mm in the unloaded state. Similarly, in the negative

direction of loading, the width of crack **S1** on the right side opened to 0.18 mm, with a residual width of 0.1 mm when the load was removed. The widening of crack **S1** on Wall RSWS-R is depicted in Figure 5.10 (a). The SMA wall, during the positive direction of loading, experienced widening of a pre-existing crack, denoted **N4**. The crack was located at the left side and above the plastic hinge, at 1025 mm from the base of the wall. The crack characteristics of **N4** were $D_{max}^+ = (1.8 \text{ mm}; 350 \text{ mm})$. Moreover, crack **N1** widened to 1.25 mm and extended to a length of 400 mm. Upon completion of unloading, the width of crack **N1** and **N4** were 0.1 mm and 0.25 mm, respectively. The crack **N2** located above the plastic hinge did not change during the loading and unloading of the negative phase for the last repetition of 0.4% drift. Crack **N3**, which surfaced at the cold joint of casting of the ECC, was stable regarding length and widened to $w_{D_{max}^-} = 1 \text{ mm}$. Crack **N3** had a residual width of $w_{D_{zero}^-} = 0.2 \text{ mm}$. The four cracks **N1**, **N2**, **N3** and **N4**, are illustrated in Figure 5.10 (b).

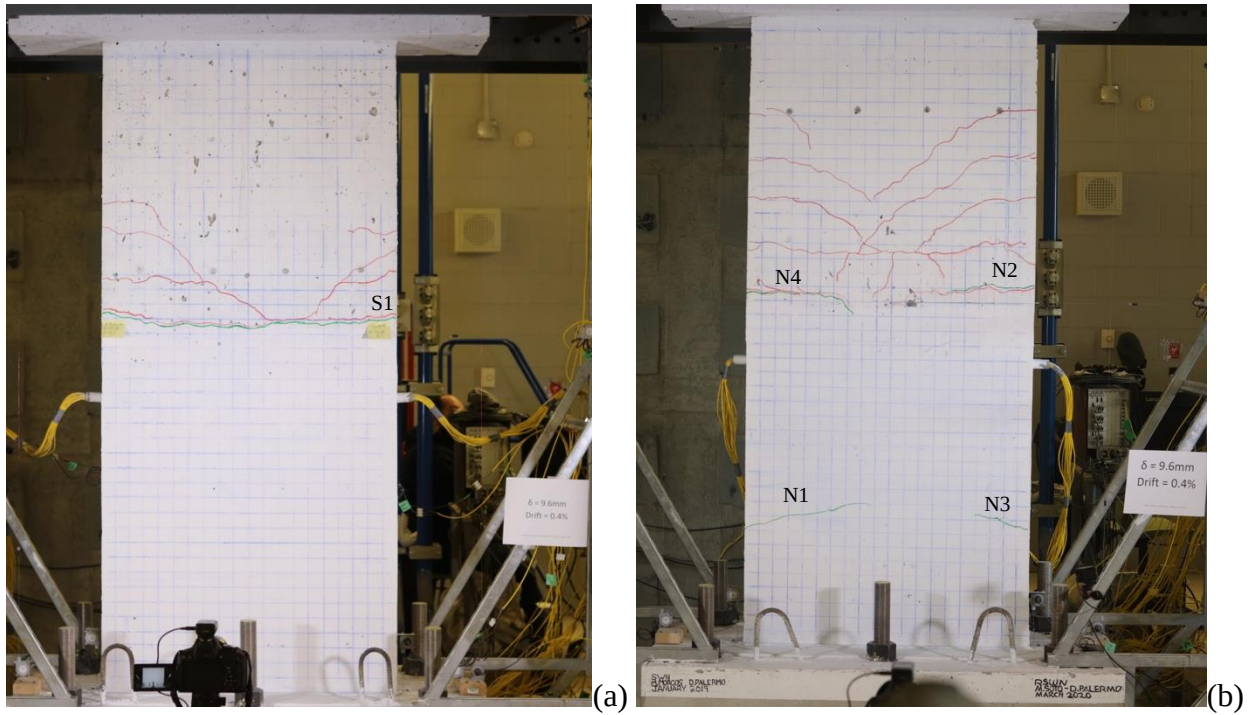


Figure 5.10 Crack pattern at 0.4% drift: (a) RSWS-R crack **S1** at $D_{zero}^- = (0.10 \text{ mm}; 1000 \text{ mm})$; and (b) RSWN crack **N1** at $D_{zero}^- = (0.1 \text{ mm}; 350 \text{ mm})$, **N2** at $D_{zero}^- = (0.15 \text{ mm}; 350 \text{ mm})$, **N3** at $D_{zero}^- = (0.20 \text{ mm}; 200 \text{ mm})$ and **N4** at $D_{zero}^- = (0.25 \text{ mm}; 400 \text{ mm})$

As the lateral displacements increased, flexural cracks were the dominant type of cracks observed in the SMA wall, while shear cracks surfaced in the steel reinforced wall. Wall RSWS-R experienced a 250 mm long crack, **S2**, during the first cycle of 0.5% drift in the negative direction, which was

located at the 25 mm from the base of the wall. Another crack, **S3**, appeared during the second repetition in the positive direction on the left side of the wall at 250 mm from the base. During the last repetition of the 0.5% drift cycle in the positive direction, crack **S3** widened to $w_{D_{max}^+} = 0.15$ mm with a length of $l_{D_{max}^+} = 25$ mm. Once unloaded, crack **S3** recovered to with of $w_{D_{zero}^+} = 0.1$ mm. During loading to the last repetition in the positive direction of 0.5% drift, the cracks in Wall RSWN continued to appear with the formation of one additional crack **N5**. The new crack **N5** was located on the left side of the wall, at 850 mm from the base. Moreover, **N5** was 50 mm long and 0.15 mm wide. Crack **N1** widened to 1.5 mm and extended an additional 100 mm. Crack **N4** presented no additional growth. Once unloaded, the width of cracks **N1** and **N5** constricted to $w_{D_{zero}^+} = 0.4$ mm and $w_{D_{zero}^+} = 0.1$ mm, respectively. When the conventional steel wall was loaded in the negative direction, the previously identified flexural crack **S2** widened to $w_{D_{max}^-} = 0.15$ mm. Furthermore, two shear cracks denoted **S4** and **S5** initiated at 1550 mm and 1650 mm from the base of the wall with the same crack width of $w_{D_{max}^-} = 0.1$. Crack **S4** propagated diagonally to 1350 mm from the base. Similarly, crack **S5** extended at a 45° angle and terminated at 1500 mm height measured from the bottom of the wall. Once unloaded, crack **S4** and **S5** closed to a width less than 0.1 mm. For the SMA wall concerning loading in the negative direction, crack **N2** existing from the original test widened to 0.3 mm and extended to the middle of the wall with a total length of 500 mm. **N3** located 400 mm above the wall base, widened to $w_{D_{max}^-} = 1.25$ mm and extended an additional 150 mm. At the end of this drift level, cracks **N2** and **N3** recovered to a $w_{D_{zero}^-} = 0.1$ mm and $w_{D_{zero}^-} = 0.25$ mm, correspondingly. Figure 5.11 (a) and (b) illustrate the crack patterns at the end of the 0.5% drift cycle for Walls RSWS-R and RSWN, respectively.

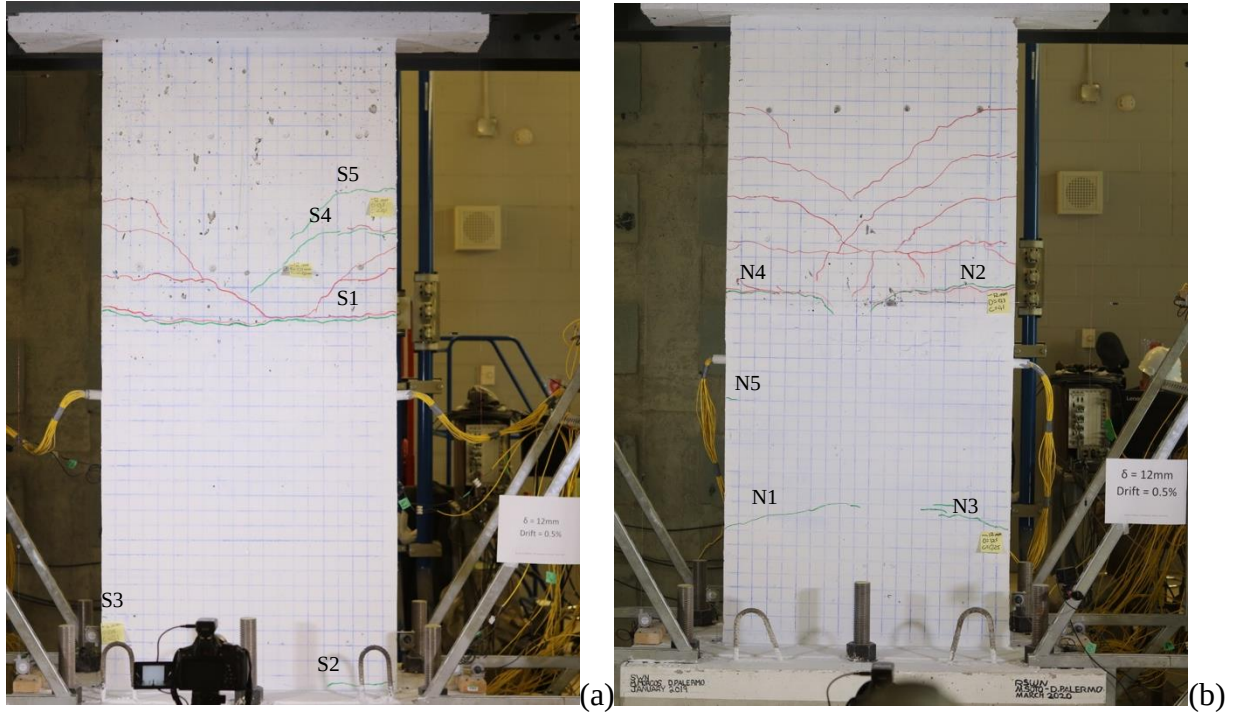


Figure 5.11 Crack pattern at 0.5% drift: (a) RSWS-R crack S1 at $D_{zero}^- = (0.10 \text{ mm}; 1000 \text{ mm})$, S2 at $D_{zero}^- = (0.1 \text{ mm}; 250 \text{ mm})$, S3 at $D_{zero}^- = (0.1 \text{ mm}; 25 \text{ mm})$, S4 at $D_{zero}^- = (0.1 \text{ mm}; 500 \text{ mm})$ and S5 at $D_{zero}^- = (0.1 \text{ mm}; 350 \text{ mm})$; and (b) RSWN crack N1 at $D_{zero}^- = (0.4 \text{ mm}; 450 \text{ mm})$, N2 at $D_{zero}^- = (0.10 \text{ mm}; 500 \text{ mm})$, N3 at $D_{zero}^- = (0.25 \text{ mm}; 350 \text{ mm})$, N4 at $D_{zero}^- = (0.25 \text{ mm}; 400 \text{ mm})$ and N5 at $D_{zero}^- = (0.1 \text{ mm}; 50 \text{ mm})$

The walls developed one major crack at a displacement of 24 mm; however, each one was localized at a different region, as shown in Figure 5.12. For the steel wall, during the first cycle at 1% drift in the positive direction, flexural crack **S6** initiated at the left side at 275 mm from the base. The crack extended 600 mm with a width of approximately 3 mm. In addition, two supplementary, 0.1 mm-wide shear cracks denoted **S7** and **S8** developed above the plastic hinge on the left side at 1850 mm and 2000 mm from the base, respectively. Through the unloading of the first cycle, crack **S6** did not recover while cracks **S7** and **S8** decreased to less than 0.1 mm. When Wall RSWS-R was pushed to the negative direction during the first cycle, another 1.5 mm-wide crack initiated adjacent to the starter bars at approximately 275 mm and propagated to connect with crack **S6** to form one major horizontal crack extending the width of the wall. The initiation of this localized damage was attributed to the sudden change in stiffness in the wall due to the starter bars and mechanical couplers. Simultaneously, another crack **S9** initiated from the right edge of the wall at the height of 275 mm and extended 160 mm without intersecting with crack **S6**. On the right side of the wall, a short 75 mm-long crack **S10** with a width of 0.15 mm surfaced. In addition, another 0.1 mm-wide shear crack

S11 developed above the plastic hinge at the height of 1800 mm from the base of the wall. Crack **S11** propagated downwards to join with crack **S5**. Once the wall was unloaded during the first cycle, crack **S10** recovered to a width of 0.1 mm. It is noted that Wall RSWS-R yielded during the first cycle to 1% drift. The crack characteristics of the conventional reinforcing wall were then evaluated for the last repetition of the imposed displacement of 24 mm. During the positive direction, the crack above the starter bars remained intact. An additional flexural crack **S12** was observed on the left side at 400 mm from the base of the wall that extended for 150 mm with a crack width of 1.5 mm. At 75 mm from the left base of the wall, a 0.1 mm-wide crack, denoted as **S13**, appeared and extended for 100 mm. Once unloaded, the crack width of crack **S12** reduced to $w_{D_{zero}^+} = 0.9$ mm. For Wall RSWN, at a drift of 1%, during the last cycle in the positive direction, cracks **N1** and **N4** widened to 5 mm and 0.18 mm, respectively, while remaining the same length. During loading, at 250 mm along the major crack **N1**, two hairline shear cracks branched downwards at a 45° angle. Crack **N5** widened an additional 0.2 mm and extended to 260 mm long. Two previously existing hairline shear cracks reported by Morcos (2020), reopened and extended for an additional 5 mm. These cracks were located above the plastic hinge region at the height of 1555 mm and 1575 mm measured from the base of the wall, and at the end of the unloading phase, these cracks had a width less than 0.1 mm; while the major crack **N1** recovered to 1.5 mm.

During loading to the negative direction for the last repetition at 1% drift, crack **N3** in Wall RSWN propagated to intersect crack **N1** at the height of 500 mm from the base of the wall. Thus, here on Crack **N3** and **N1** are denoted **N1**, which extended the entire width of the wall with an opening of 5 mm. Once unloaded, crack **N1** recovered to a width of 1.25 mm. For the conventional shear wall, crack **S12** above the starter bars remained the same during the negative direction of loading. An additional 50 mm-long crack **S14** surfaced at 325 mm from the right base and opened to 0.15 mm. At the unloaded state, of Wall RSWS-R, crack **S14** recovered a width of 0.1 mm, while crack **S10** remained the same. The final crack pattern at the end of the third repetition at 1% drift is shown in Figure 5.12 (a) and (b) for the steel and SMA walls, respectively.

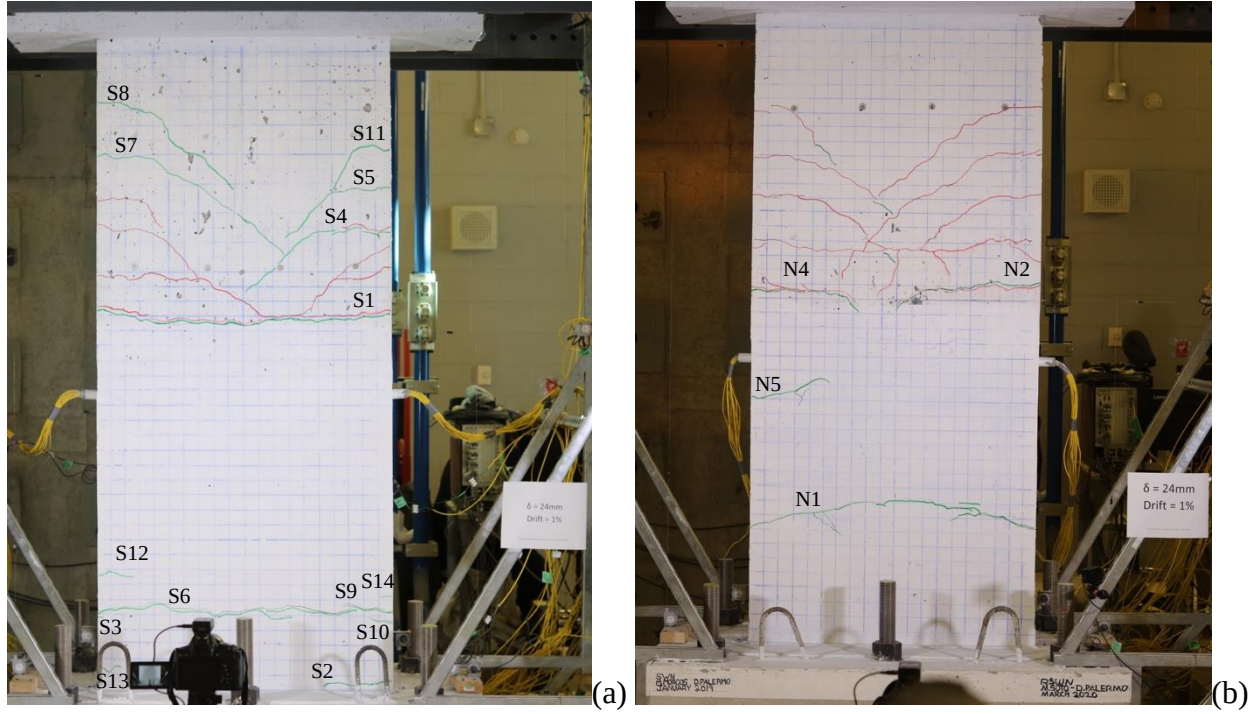


Figure 5.12 Crack pattern at 1% drift: (a) RSWS-R crack, S6 at $D_{zero}^- = (3 \text{ mm}; 875 \text{ mm})$, S9 at $D_{zero}^- = (3 \text{ mm}; 175 \text{ mm})$, S10 at $D_{zero}^- = (0.1 \text{ mm}; 75 \text{ mm})$, S12 at $D_{zero}^- = (0.9 \text{ mm}; 150 \text{ mm})$, S14 at $D_{zero}^- = (0.1 \text{ mm}; 50 \text{ mm})$; and (b) RSWN crack N1 at $D_{zero}^- = (1.25 \text{ mm}; 1000 \text{ mm})$, and N5 at $D_{zero}^- = (0.1 \text{ mm}; 260 \text{ mm})$

During the first cycle of 1.5% drift, crack **S12** extended 200 mm in Wall RSWS-R. It became visually evident that the PVA fibers located along crack **S12** were pulling out during the second repetition of 36 mm of displacement in the positive direction. In addition, an additional shear crack **S15** surfaced between the starter bars and the couplers located at 300 mm and 400 mm from the base of the wall, respectively. Crack **S15** propagated to a length of 650 mm with a width $w_{D_{max}^+}$ of 3.5 mm on the left end and 0.15 mm at the right end. It is to be noted that crack **S6** maintained a crack width greater than 3 mm. Once the conventional steel wall was unloaded, the previously stretched fibers in tension that bridged through cracks **S12** and **S6** on the left side were stress-free. The SMA wall experienced three cycles at 1.5% drift value. Crack **N1** widened to 10 mm when loaded in the positive direction, making the Ni-Ti bars visible. A shear crack **N6** appeared on the left side of the wall at 375 mm above the base and propagated at a 45° angle to meet Crack **N1**. Crack **N6** developed a width $w_{D_{max}^+}$ of 1.5 mm. Another crack **N7** initiated just above the starter bars approximately at the mid-width of the wall and then propagated upwards to the right end at the height of 400 mm. **N7** presented a crack width $w_{D_{max}^+}$ of 0.8 mm. Crack **N5** widened to 0.3 mm. Above the plastic hinge, two, 0.1 mm wide-shear cracks developed at 2000 mm and 2150 mm above the base of the wall. Once unloaded, the residual

crack width $w_{D_{zero}^+}$ for **N1** was 5 mm, for **N6** was 1 mm, for **N7** was 0.5 mm and for **N5** was 0.1 mm. In addition, all the shear cracks above the plastic hinge closed to hairline cracks, as shown in Figure 5.13 (b).

During loading of Wall RSW-S-R in the negative direction to the second repetition of 1.5% drift, crack **S9** widened to a width $w_{D_{max}^-} = 7$ mm at the right end. During this stage, the PVA fibers were pulling out and rupturing. Moreover, crack **S9** (Figure 5.12(a)) extended to connect with crack **S6** and formed one wide crack extending across the entire wall. The width of crack **S6** on the left side was approximately 0.8 mm. Here on these two cracks were denoted **S6**. An additional crack S16 initiated on the right side of the wall at a height of 400 mm from the base with a crack width of 0.6 mm. No additional cracks were observed above the plastic hinge region. Once the wall was unloaded, the right side of crack **S6** closed to a residual width of 5 mm. During the last cycle in the negative direction of the SMA wall, the right side of cracks **N1** and **N7** widened to 7 mm and 5 mm, respectively. The region above the plastic hinge did not change. During the unloading phase of Wall RSWN, crack **N1** recovered to 1.5 mm while **N7** closed to 1.25 mm. Figure 5.13 (a) and (b) represent the cracking pattern at the end of 1.5% drift for walls RSW-R and RSWN, respectively.

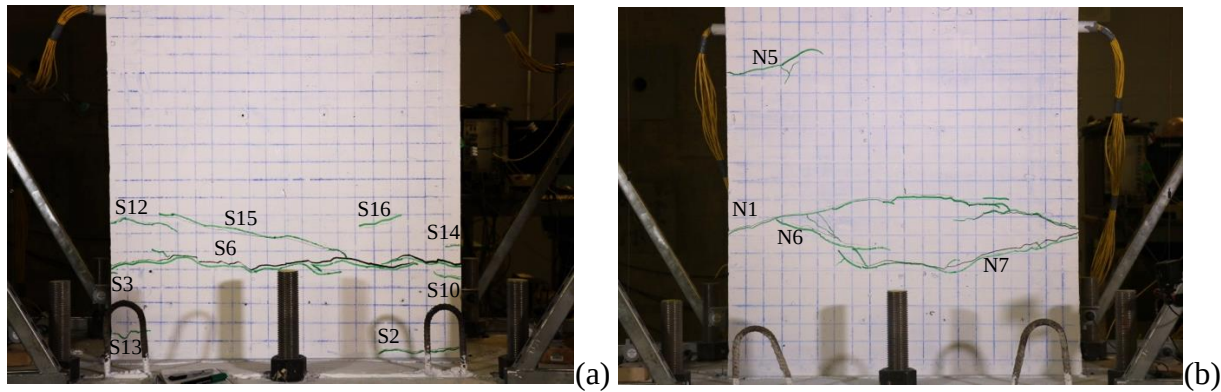


Figure 5.13 Crack pattern at 1.5 %drift: (a) RSW-S-R crack, S12 at $D_{zero}^- = (0.9 \text{ mm}; 200 \text{ mm})$, S15 at $D_{zero}^- = (3.5 \text{ mm}; 650 \text{ mm})$, S6 at $D_{zero}^- = (5 \text{ mm}; 1000 \text{ mm})$; and (b) RSWN crack N1 left side at $D_{zero}^- = (5 \text{ mm}; 1000 \text{ mm})$, crack N1 right side at $D_{zero}^- = (1.5 \text{ mm}; 1000 \text{ mm})$, N5 at $D_{zero}^- = (0.1 \text{ mm}; 260 \text{ mm})$, N6 at $D_{zero}^- = (1 \text{ mm}; 300 \text{ mm})$, N7 at $D_{zero}^- = (1.25 \text{ mm}; 700 \text{ mm})$

During the last cycle at 48 mm displacement (2% drift) for the conventional wall, in the positive direction, crack **S13** widened to 0.8 mm and extended to a length of 150 mm. The left side of crack **S6** formed at the end of the starter bars and had a width of 8 mm. Most of the fibers crossing crack

S6 were pulled out. Similarly, crack **S12** developed at the location of the upper couplers and widened to 6 mm. It was evident that both cracks **S12** and **S6** penetrated throughout the thickness of the wall. The shear crack **S15** extended to the left end of the wall to meet crack **S6**. The shear crack located at the height of 1400 mm from the base of the wall propagated diagonally downwards for a length of 150 mm. During the unloading phase, crack **S6** did not recover and maintained a width of 8 mm, while crack **S12** was recovered to a width of 4 mm.

During loading of the SMA wall to the positive direction during the last repetition at 2% drift, crack **N1** widened to 16 mm where most of the fibers pulled out, and the SMA bar in the boundary region was visible. The width of crack **N6** reached 5 mm. The horizontal section of crack **N7**, located in the middle of the wall, was affected and widened to 1.25 mm. Crack **N5** opened to 0.4 mm. Once unloaded, cracks **N1**, **N6**, **N7** and **N5** recovered to 8 mm, 3 mm, 0.9 mm, and 0.1 mm, respectively.

While loading Wall RSWS-R in the negative direction to 2% drift, crack **S2** opened to 1.5 mm. The right side of crack **S6** widened to 10 mm while extending along the entire wall. Also, the width of the shear crack **S16** increased to 3 mm and extended to the middle of the wall to intersect with crack **S15**. Simultaneously, cracks **S14** and **S10** propagated to join with crack **S6**. During the unloading phase, crack **S2** in the conventional steel wall was able to recover to 1 mm. However, cracks **S6** and **S16** remained the same in terms of width. For Wall RSWN, two main cracks extended while the wall was loaded in the negative direction; the right side of crack **N1** opened to 5 mm, and the right side of crack **N7** widened to 10 mm. Upon unloading, **N1** and **N7** recovered to widths of 1.25 mm and 5 mm, respectively. Figure 5.14 (a) and (b) depict the cracking pattern at the end of 2% drift for Walls RSWS-R and RSWN, respectively.

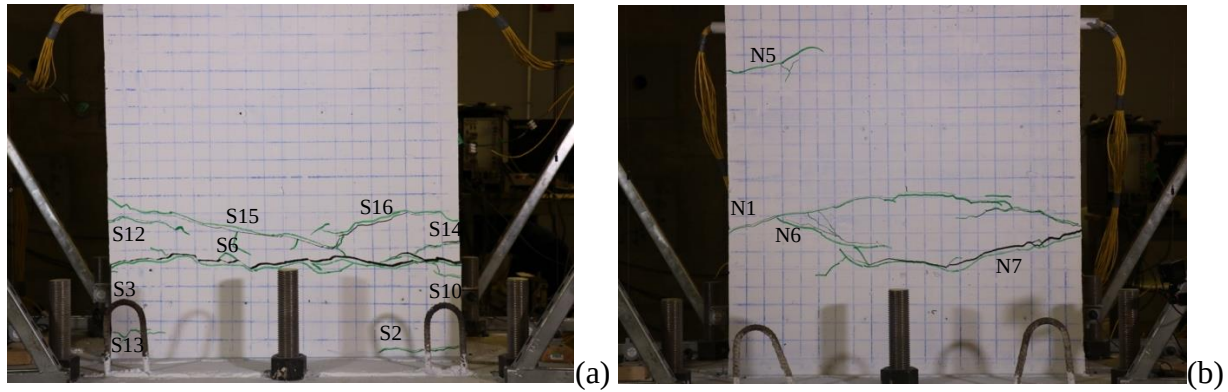


Figure 5.14 Crack pattern at 2% drift: (a) RSWs-R crack, S12 at $D_{zero}^- = (4 \text{ mm}; 225 \text{ mm})$, S6 left side at $D_{zero}^- = (8 \text{ mm}; 1000 \text{ mm})$, S6 right side at $D_{zero}^- = (10 \text{ mm}; 1000 \text{ mm})$ and S16 left side at $D_{zero}^- = (3 \text{ mm}; 400 \text{ mm})$; and (b) RSWN crack N1 left side at $D_{zero}^- = (8 \text{ mm}; 1000 \text{ mm})$, crack N1 right side at $D_{zero}^- = (1.25 \text{ mm}; 1000 \text{ mm})$, N5 at $D_{zero}^- = (0.1 \text{ mm}; 260 \text{ mm})$, N6 at $D_{zero}^- = (3 \text{ mm}; 300 \text{ mm})$, N7 left side at $D_{zero}^- = (0.9 \text{ mm}; 700 \text{ mm})$, N7 right side at $D_{zero}^- = (5 \text{ mm}; 700 \text{ mm})$

For an imposed displacement of 60 mm (2.5% drift), the repaired conventional wall experienced crack widening with out-of-plane movements due to concrete interlocking in the critical cracked region (Figure 5.15 (a)). For the first cycle in the negative direction 2.5% drift, crack **S6** extended along the entire width of Wall RSWs-R and through the thickness, dividing the wall into two sections that were mainly connected by the longitudinal steel reinforcement. Crack **S6** experienced a width of 20 mm. The section of Wall RSWs-R below crack **S6** remained intact to the foundation block, while rocking of the wall at this crack location became evident. Moreover, the section of the wall above crack **S6** experienced sliding along the crack, extending outwards with an offset of 10 mm for each cycle. At the end of the first cycle, the right side of cracks **S6** and **S14** were 10 mm and 11 mm wide, respectively. Crack **S14** spread and intersected with crack **S16**. Noteworthy it that two longitudinal reinforcing steel bars located in the right boundary region ruptured. During the second repetition in the positive direction, the left side of cracks **S6** and **S12** opened to 15 mm and 7 mm, respectively. No recovery of the crack widths was observed during the unloading phase.

At 2.5% drift, during the second repetition in the positive direction, the cracks present within the plastic hinge of the SMA wall became wider without undergoing any propagation. Crack **N1** opened to 20 mm, crack **N6** to 10 mm, crack **N7** on the left side to 3 mm, and crack **N5** to 0.5 mm. During the unloading phase, all cracks recovered; **N1** closed to 13 mm, **N6** to 8 mm, the left side of **N7** to 1.5 mm, and **N5** to less than 0.1 mm. Unlike the conventional steel wall, some vertical cracks appeared on the right side of Wall RSWN attributed to the debonding of the smooth surface of the

SMA bars located in the boundary region. During loading of Wall RSWN to the negative direction, crack **N1** on the right side opened to 0.4 mm while the right side of **N7** opened to 18 mm. Once unloaded, crack **N1** recovered to 0.1 mm while crack **N7** closed to 10 mm. The same crack pattern was visible on the left side of the wall due to the loss of bond between the SMA bar and the surrounding concrete. During the last repetition in the negative direction for the steel wall, the dominant crack **S6** developed a width of 17 mm without any recovery upon unloading. Figure 5.15 (a) and (b) illustrate the cracking patterns at the end of 2.5% drift in the plastic hinge zone, for Walls RSW-S-R and RSWN, respectively

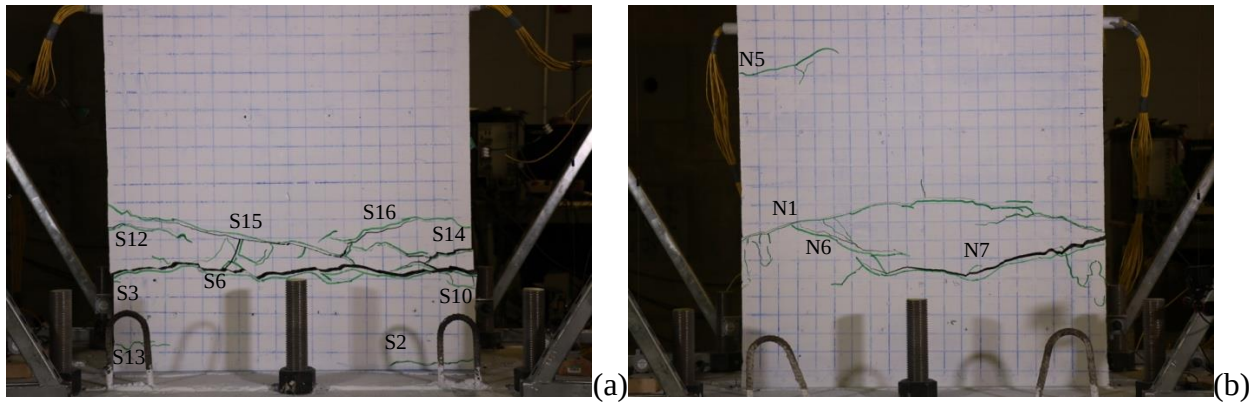


Figure 5.15 Crack pattern at 2.5 %drift: (a) RSW-S-R crack, S12 at $D_{zero}^- = (7 \text{ mm}; 225 \text{ mm})$, S6 left side at $D_{zero}^- = (15 \text{ mm}; 1000 \text{ mm})$, S6 right side at $D_{zero}^- = (17 \text{ mm}; 1000 \text{ mm})$; and (b) RSWN crack N1 left side at $D_{zero}^- = (13 \text{ mm}; 1000 \text{ mm})$, crack N1 right side at $D_{zero}^- = (0.1 \text{ mm}; 1000 \text{ mm})$, N5 at $D_{zero}^- = (0.1 \text{ mm}; 260 \text{ mm})$, N6 at $D_{zero}^- = (8 \text{ mm}; 300 \text{ mm})$, N7 left side at $D_{zero}^- = (1.5 \text{ mm}; 700 \text{ mm})$, N7 right side at $D_{zero}^- = (10 \text{ mm}; 700 \text{ mm})$

At the 3% drift cycle, the experimental testing of the conventional steel wall was terminated. The cracking characteristics of the wall did not change throughout this cycle with the exception of crack **S6** that extended to a width of 30 mm without recovery. A total of three longitudinal steel bars ruptured in the boundary region by the end of the second repetition. At the end of testing, Wall RSW-S-R experienced a lateral offset of approximately 20 mm along crack **S6**. The final crack pattern of the wall inside the plastic hinge region is presented in Figure 5.16 (a).

The testing of the SMA wall proceeded with higher lateral displacements. During the positive direction of the last repetition to 3% drift, the left side of crack **N1** widened to 25 mm, crack **N6** to 18 mm, and crack **N7** on the left side to 4 mm. It was evident that one additional shear crack branched out from the right side of crack **N7**, extending diagonally downwards. The PVA fibers were visible

along the opening of crack **N7**. Once the wall was unloaded, the left side of crack **N1** recovered to 20 mm, crack **N6** to 10 mm, while the left side of crack **N7** closed to 7 mm. When the wall was loading in the negative direction, the right side of crack **N1** widened to 4 mm, and crack **N7** to 21 mm. During the unloading phase, cracks **N1** and **N7** closed to 1 mm and 17 mm, respectively, as can be seen in Figure 5.16 (b).

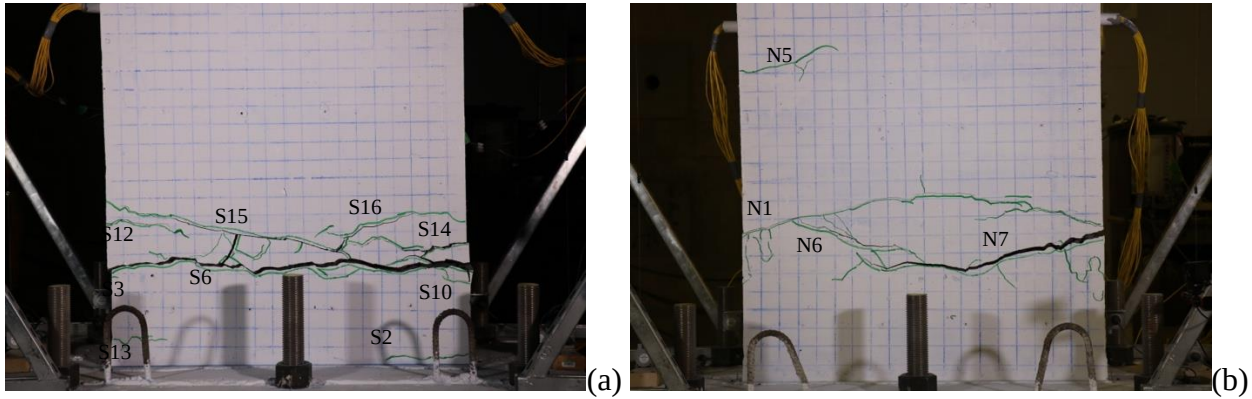


Figure 5.16 Crack pattern at 3% drift: (a) RSWS-R failure mode with crack S6 at $D_{zero}^- = (30 \text{ mm}; 1000 \text{ mm})$; and (b) RSWN crack N1 left side at $D_{zero}^- = (20 \text{ mm}; 1000 \text{ mm})$, crack N1 right side at $D_{zero}^- = (1 \text{ mm}; 1000 \text{ mm})$, N6 at $D_{zero}^- = (10 \text{ mm}; 300 \text{ mm})$, N7 left side at $D_{zero}^- = (7 \text{ mm}; 700 \text{ mm})$, N7 right side at $D_{zero}^- = (17 \text{ mm}; 700 \text{ mm})$

Wall RSWN was subjected to a displacement of 84 mm (3.5% drift) in the positive direction, opening the left side of crack **N1** to 30 mm. The crack was sufficiently wide to expose the SMA bars in the left boundary of the wall. Most of the PVA fibers pulled out in this area. The width of crack **N6** reached 7 mm, while the left side of crack **N7** widened to 8 mm. The state of the wall during the second repetition in the positive direction is shown in Figure 5.17 (a). Once unloaded, the left side of **N1** closed to 21 mm, while crack **N6** closed to 5 mm. During loading to the negative direction, the longitudinal steel reinforcing bar in the web region and adjacent to the SMA bars ruptured. The right side of crack **N7** opened to 31 mm. The ruptured steel bar resulted in a significant reduction in the recovery of crack **N7**, to a width of only 21 mm at the end of the cycle. The crack pattern of the wall at the end of 3.5% drift is presented in Figure 5.17 (b).

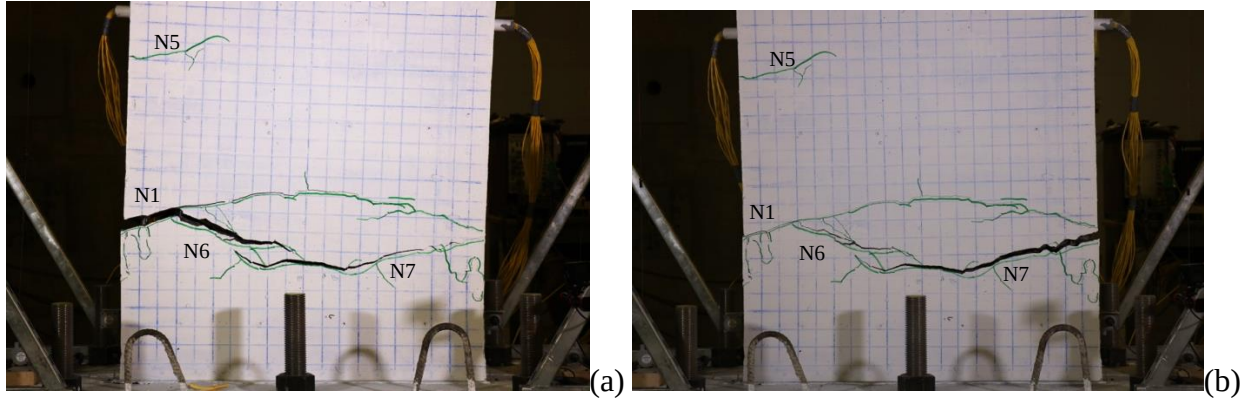


Figure 5.17 Wall RSWN crack pattern at 3.5 % drift: (a) maximum opening in the positive direction; and (b) at crack N1 left side at $D_{zero}^{-} = (21 \text{ mm; } 1000 \text{ mm})$, N6 at $D_{zero}^{-} = (5 \text{ mm; } 300 \text{ mm})$, N7 right side at $D_{zero}^{-} = (21 \text{ mm; } 700 \text{ mm})$

Testing was terminated at the end of the first repetition at 4% drift. During loading to the positive direction, a longitudinal steel bar ruptured on the left side of the web region. Furthermore, in the final excursion in the negative direction, two longitudinal reinforcing bars located on the right side of the web region ruptured. Figure 5.18 depicts the final crack pattern in the plastic hinge region of the SMA wall.

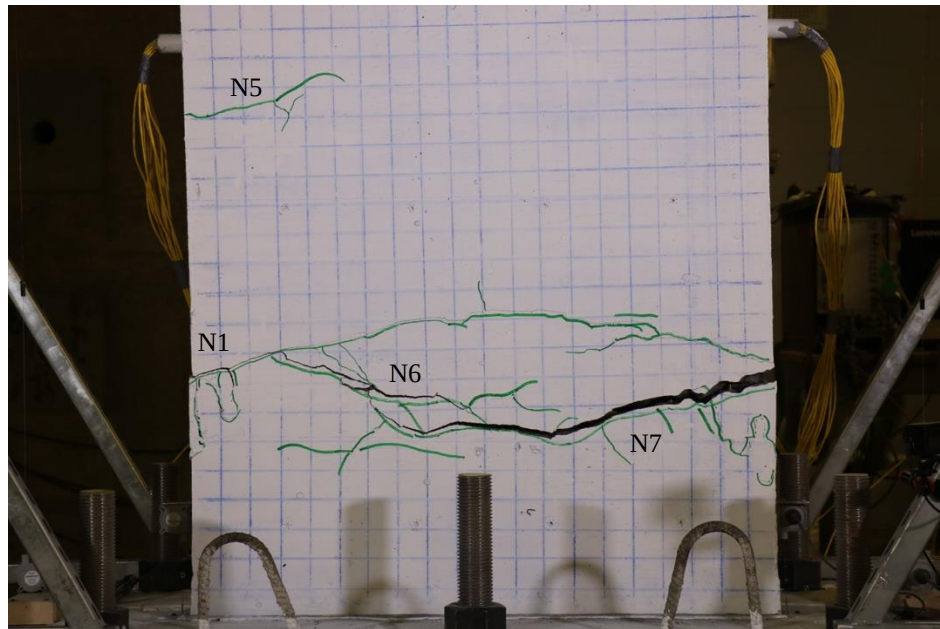


Figure 5.18 Crack pattern at 4% drift for Wall RSWN

The final state of the repaired walls after testing is illustrated in Figure 5.19.

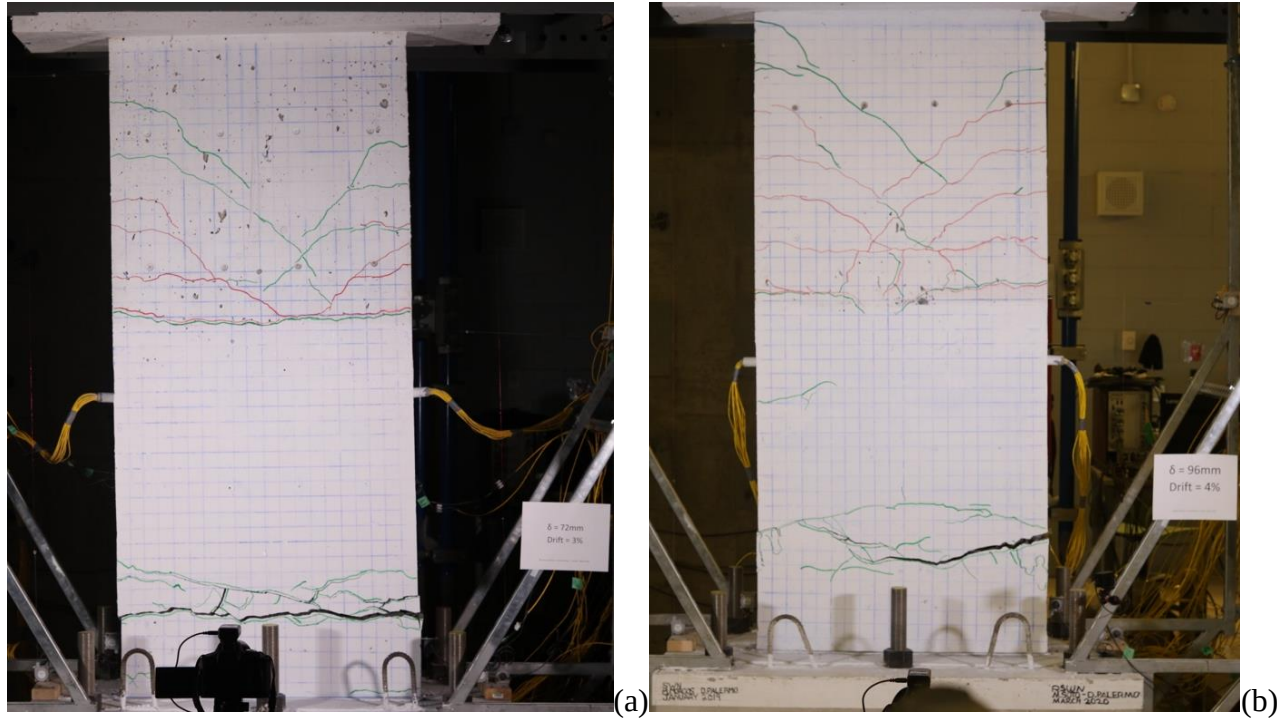


Figure 5.19 Crack pattern at the end of testing: (a) 3% drift for RSWs-R; and (b) 4% drift for RSWN

5.2.2 Load – Displacement Response

As discussed in Section 4.4.2, cable and linear potentiometers were utilized to record salient displacement behaviors of the wall at various locations.

The overall hysteresis response was recorded based on cable potentiometer CP1 (Figure 4.39) located at the mid-height of the cap loading beam, which recorded measurements relative to the foundation block. Strategic performance parameters, including yield, peak, and ultimate points, were identified. Furthermore, they were calculated as an average from the positive and the negative directions based on the reduced stiffness equivalent elasto-plastic method (Park, 1988). The ultimate point was established based on a 20% reduction in the lateral load capacity or the displacement cycle prior to fracturing of the reinforcement. Figure 5.20 and Figure 5.21 depict the lateral load-displacement response of Walls RSWs-R and RSWN, respectively.

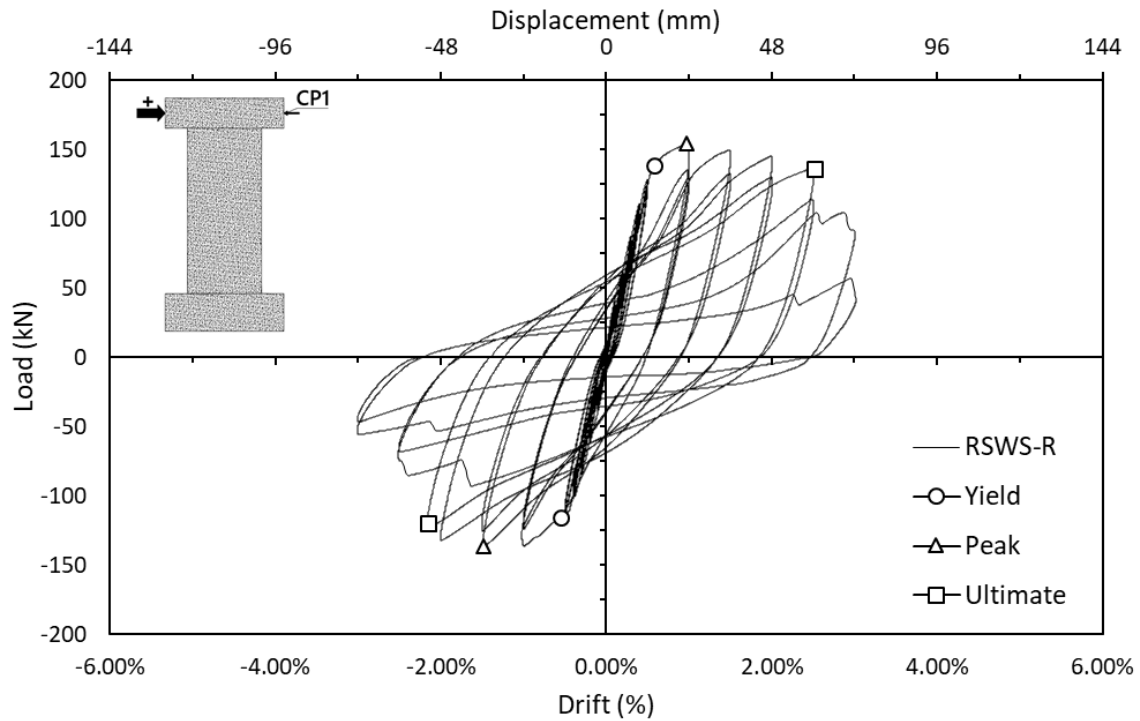


Figure 5.20 Load – displacement response of Wall RSWS-R

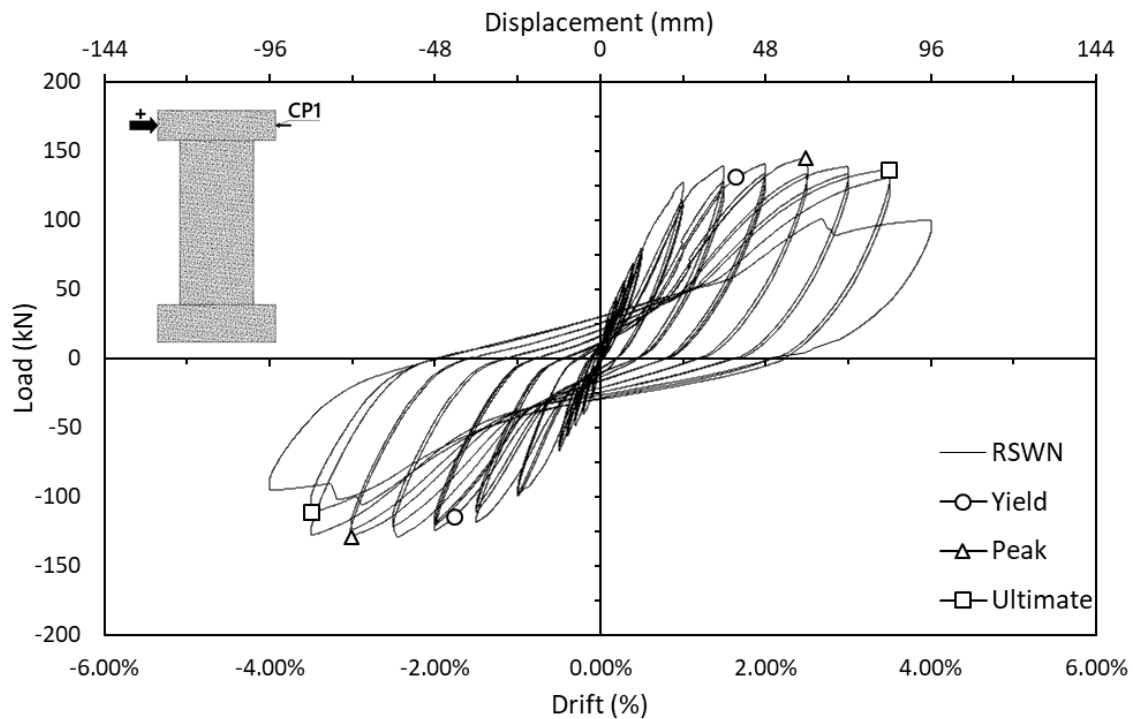


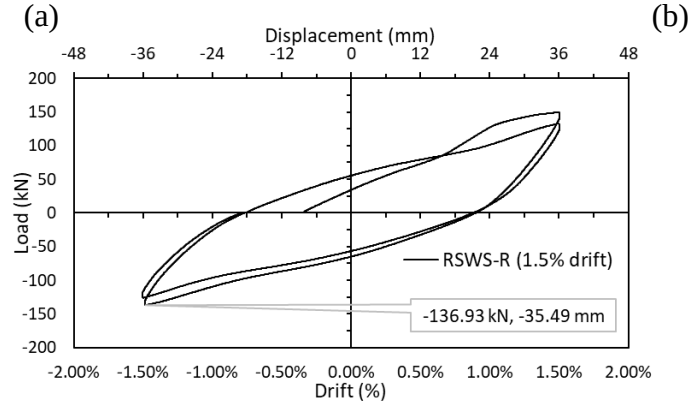
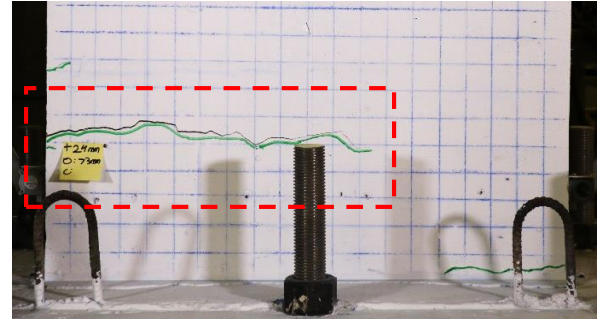
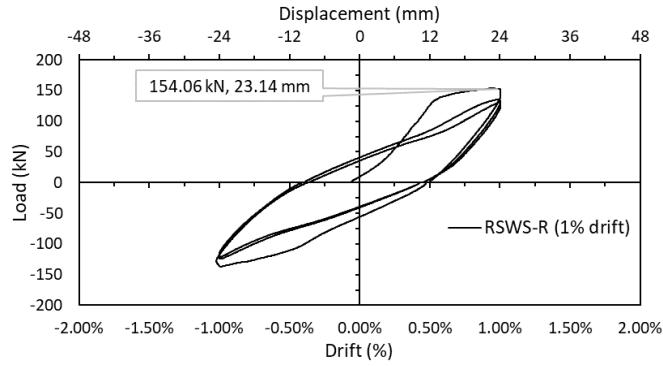
Figure 5.21 Load – displacement response of Wall RSWN

In general, the effect created by the cyclic loading resulted in degradation of the stiffness for the second and third repetitions for the same displacement level. The hysteresis loops of the first two

loading stages (0.05% and 0.1% drift) exhibited linear-elastic behavior with negligible residual displacements. Thereafter, Wall RSWN experienced a softer behavior with higher displacements, attributed to the low modulus of elasticity of the SMA rebars located in the boundary regions. Conversely, Wall RSWS-R achieved higher loads.

Prior to the yield point, Wall RSWS-R experienced a stiffer behavior in comparison to Wall RSWN. The average global yield point was reached at a load of 127.34 kN and a displacement of 13.56 mm (0.57% drift) for Wall RSWS-R. Wall RSWN experienced average global yielding at 122.91 kN with a displacement of 40.93 mm (1.71 % drift). The post-yield behavior of Wall RSWS-R was marked by a steep development of lateral load capacity prior reaching the peak load, contrary to Wall RSWN, that after the yield point, it generated a near-constant plateau with a gradual increase of lateral load to the peak. Moreover, the displacement recovery capacity of the Wall RSWN was apparent in comparison to RSWS-R; this is exemplified at the displacement level of 24 mm, where Wall RSWN presented a residual displacement of 4.9 mm, while the residual displacement in Wall RSWS-R was 11 mm.

Wall RSWS-R experienced an average peak lateral load capacity of 145.49 kN with an average displacement of 29.32 mm (1.22% drift). During loading to the positive direction at the first excursion of 1% drift, Wall RSWS-R developed a major crack located 300 mm above the base of the wall that corresponded to the peak load for the positive direction. The peak load in the negative direction was captured during the first repetition in the subsequent displacement of 36 mm (1.5% drift) when a similar crack appeared on the opposite side. Figure 5.22 presents the cycles pertaining to 1% and 1.5% drift for Wall RSWS-R where the peak load was attained with the damage that was experienced.



(b)

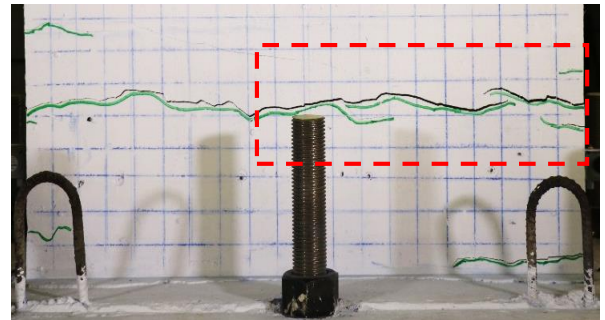


Figure 5.22 Response of Wall RSWS-R: a) peak load cycle; and b) imposed damage

Wall RSWN experienced an average peak load of 136.98 kN with an average displacement of 83.81 mm (2.74% drift). The peak load in the positive direction was recorded at 2.5% drift which was marked by an existing crack that widened and spread at 400 mm height measured from the base of the wall. The peak load in the negative direction was captured at the following displacement of 72 mm (3% drift) with a similar cracking pattern on the opposite side. Figure 5.23 illustrates the cycles where Wall RSWN sustained the peak load in both directions with their respective damage.

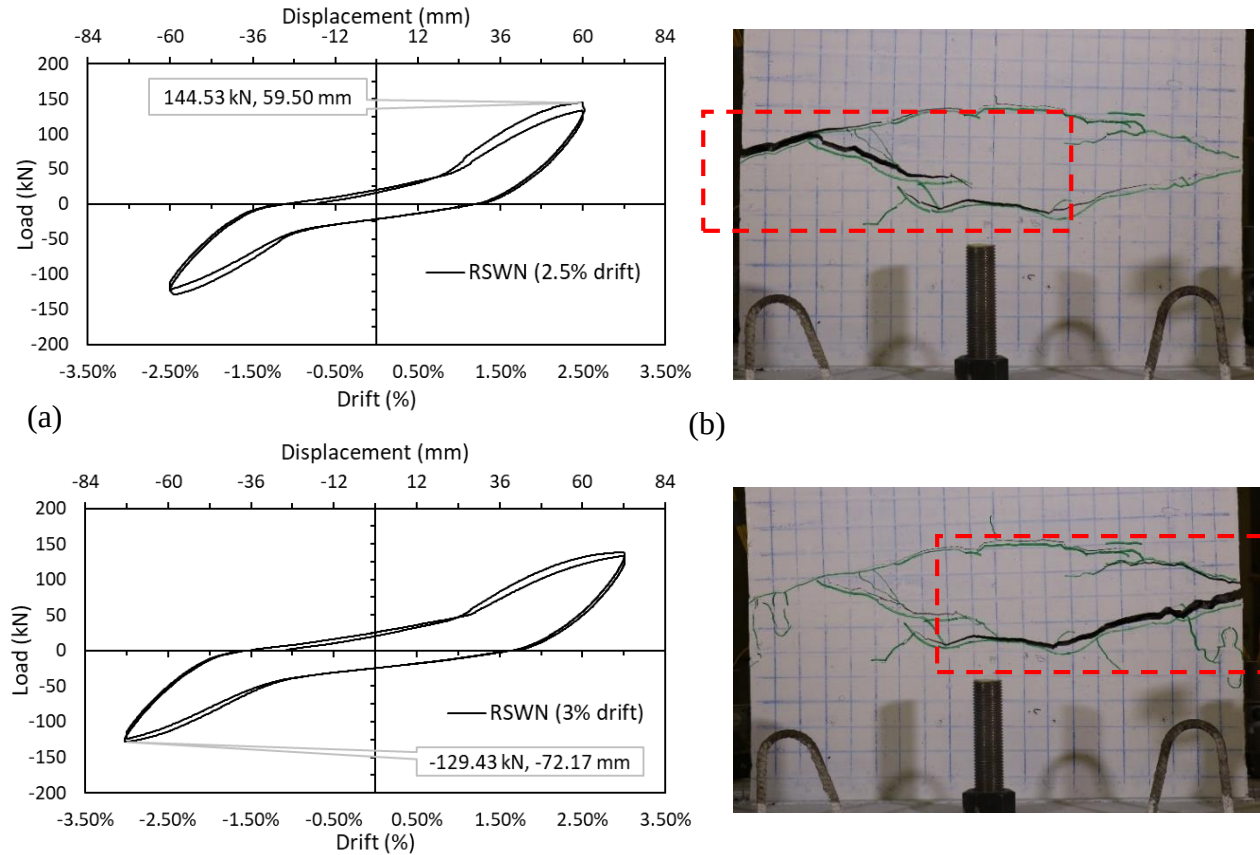


Figure 5.23 Response of Wall RSWN-R: a) peak load cycle; and b) imposed damage

The major cracks shown in Figure 5.22 and Figure 5.23 controlled the overall behavior of the walls. In the post-peak response, it was evident that a localized plastic hinge developed just above the starter bars in both walls. Moreover, after the peak point, Wall RSWS-R experienced a gradual degradation of lateral load capacity leading to an ultimate load of 128.18 kN at an average displacement of 56.19 mm (2.34% drift), which represented the last cycle prior to rupturing of the longitudinal steel reinforcement located adjacent to the starter bars in the boundary region. Two steel bars ruptured in the right boundary zone during the second excursion to 2.5% drift. Thereafter, loading to the first repetition of 72 mm displacement (3% drift cycle), resulted in the rupturing of two steel rebars in the left boundary region. Furthermore, an additional bar in the right boundary region ruptured during loading at this displacement level. Testing was terminated after the 3% drift cycle. Figure 5.24 provides photos at the major cracks where the longitudinal steel reinforcement ruptured in the boundary regions.

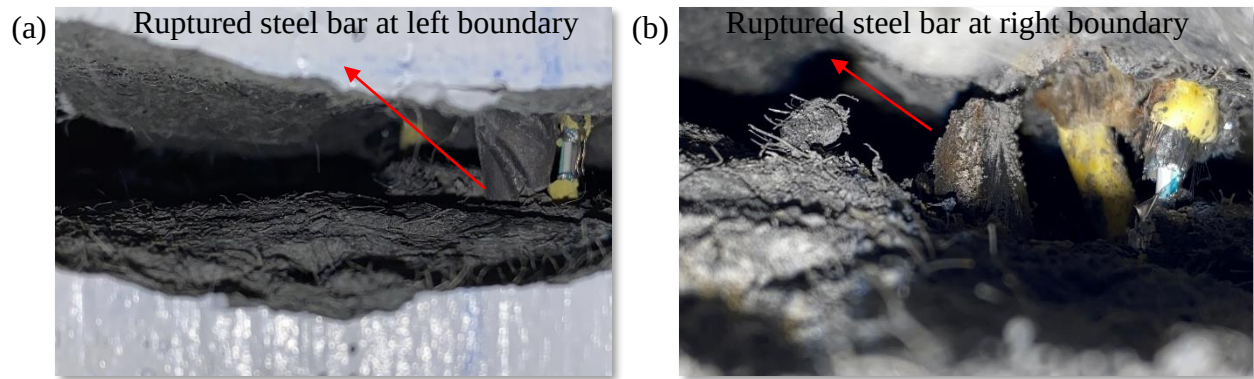


Figure 5.24 Ruptured reinforcement in Wall RSWS-R: (a) left boundary; and (b) right boundary

Wall RSWN sustained an average ultimate load of 136.98 kN with an average ultimate displacement of 83.81 mm (3.49%). The SE-SMA rebars located in the boundary region experienced no damage; however, the longitudinal steel reinforcing bars in the web region ruptured and marked the termination of the test. One steel bar adjacent to the boundary SMA reinforcement ruptured during the first repetition of loading in the negative direction at 3.5% drift. Thereafter, during the first repetition to 4% drift, two additional steel bars in the web region fractured, which resulted in the termination of the test at this stage.

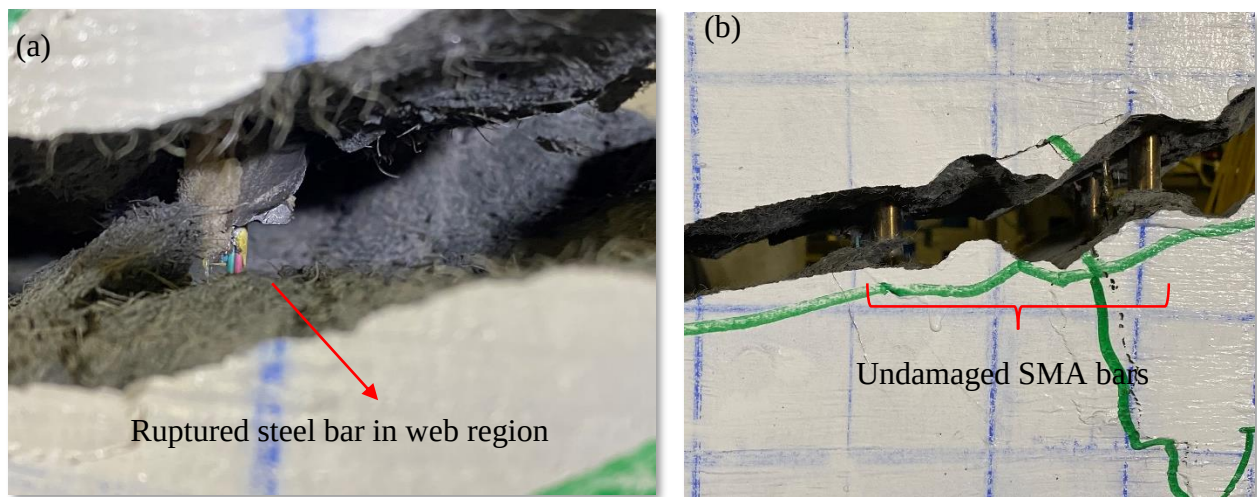


Figure 5.25 Damage in Wall RSWN: (a) ruptured steel bar in web region; and (b) undamaged SMA bars

The wide hysteretic loops exhibited by Wall RSWS-R translated into a higher level of dissipated energy and greater damage in comparison to the SMA. On the contrary, the hysteretic loops

experienced by Wall RSWS-R developed a pinching effect, attributed to the self-centering capacity of the SE-SMA bars that generated high displacement recoveries and reduced energy dissipation, which will further be discussed in Chapter 6. Table 5.3 summarizes the performance points for the repaired walls.

Table 5.3 Performance points for repaired shear walls

Wall	Yield			Peak			Ultimate			μ
	δ mm	Drift (%)	Load kN	δ mm	Drift (%)	Load kN	δ mm	Drift (%)	Load kN	
RSWS-R	13.56	0.57%	127.34	29.32	1.22%	145.49	56.19	2.34%	128.18	4.14
RSWN	40.93	1.71%	122.91	65.83	2.74%	136.98	83.81	3.49%	123.69	2.05

Note: the values are based on an average from the positive and negative directions of loading

To monitor the movement of the foundation block, the lateral displacement of the loading beam was measured relative to the base of the wall (CP1) and relative to the strong floor (CP10). The displacements recorded from the two cable potentiometers are illustrated in Figure 5.26 (a) and (b) for Wall RSWN and Wall RSWS-R, respectively. The responses of CP10 and CP1 for both walls are similar, indicating that the walls did not experience significant movement along the base of the foundation throughout the testing. Note, however, the greater horizontal displacement of the foundation is evident in the negative direction of loading. During the last loading stage for Wall RSWS-R, CP1 and CP10 recorded 71.63 mm, -72.20 mm and 76.09 mm, -82.52 mm displacements in the positive and negative directions of loading, respectively. In Wall RSWN, CP1 and CP10 captured 96.07 mm, -95.19 mm and 95.38 mm, -105.63 displacements in the positive and negative directions, respectively.

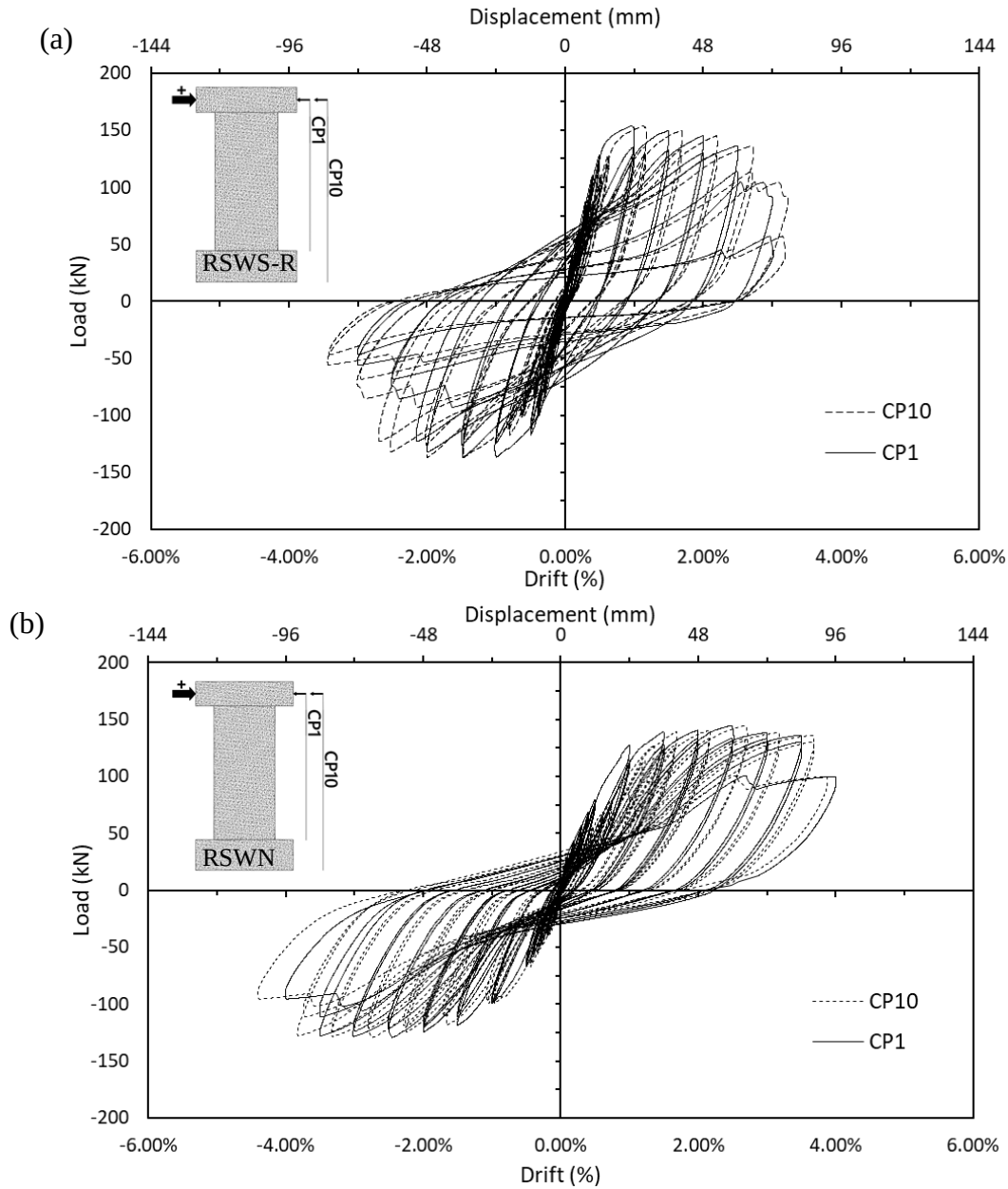


Figure 5.26 Load-displacement response relative to the base (CP1) and the strong floor (CP10): (a) Wall RSWS-R; and (b) Wall RSWN

5.2.3 Mid-height Displacement

Cable potentiometer (CP3) was installed at the height of 1100 mm, measured from the base of the walls. The location served to study the mid-height displacement relative to the foundation block. The data obtained from this measurement is compared to the lateral displacements recorded by CP1,

located at the top loading beam. Figure 5.27 and Figure 5.28 provide the load-displacement response at mid-height for the repaired walls.

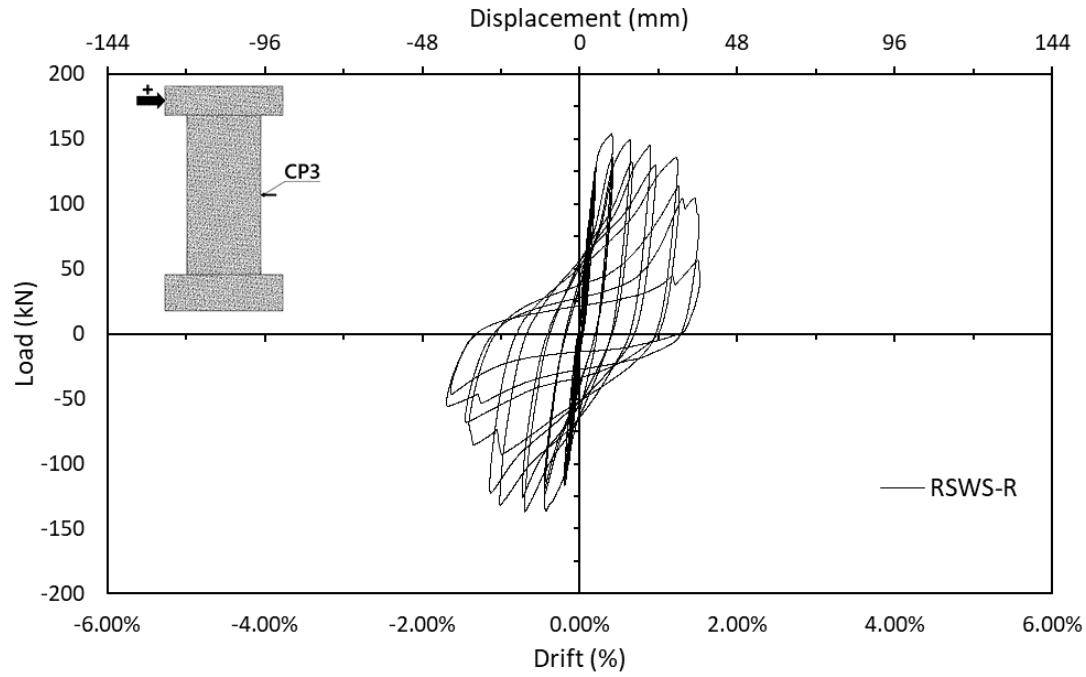


Figure 5.27 Mid-height lateral displacement of Wall RSWS-R

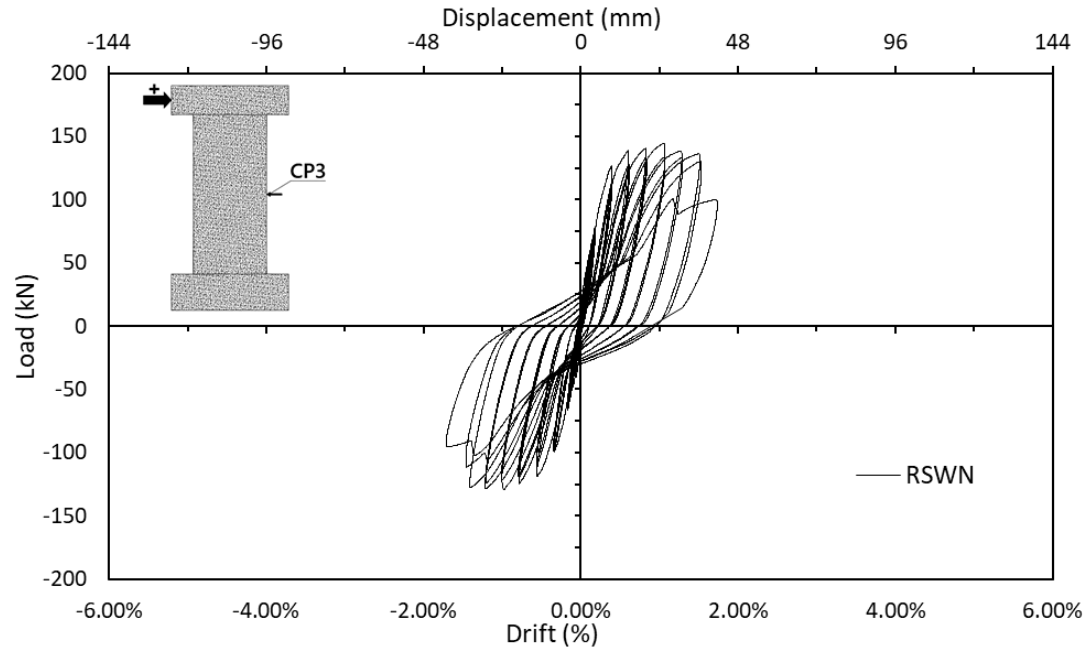


Figure 5.28 Mid-height lateral displacement of Wall RSWN

The mid-height lateral displacement at yielding for Wall RSWS-R represented 44% of the displacement recorded by CP1. Wall RSWN experienced a mid-height displacement of 21.54 mm, which represented 52% of the displacement captured by the displacement at the top loading beam. Moreover, at the peak lateral capacity, displacements of 11.43 mm and 29.60 mm were recorded by CP3 for Walls RSWS-R and RSWN, respectively, which represented a 45% of the CP1 displacement in both walls. Finally, at the ultimate point, 28.43 mm of mid-height displacement for Wall SWS-R and 38.49 mm for Wall RSWN were captured by CP3, which resulted in 50% and 46%, respectively, of the displacement monitored by CP1. The results demonstrate similar trends in the distribution of the lateral displacements in both walls.

The lateral displacement at the base of the walls, vertical displacements, plastic hinge development, foundation block sliding, and uplift are discussed in Chapter 6. Furthermore, the complete load-displacement responses recorded by the cable and linear potentiometers are available in Appendix A.

5.2.4 Reinforcement Strains

Strain gauges were placed at critical locations throughout the vertical and horizontal reinforcement inside the plastic hinge region of the repaired walls to record the strains. Most of the strain gauges installed just above the starter bars and just below the upper mechanical couplers exceeded their strain capacity due to the high lateral displacements levels imposed on the walls, while other gauges were capable of monitoring the strains throughout testing.

Strain Gauge SG1 was located on the outer longitudinal bars in the plastic hinge boundary region, just above the bottom mechanical couplers, for both repaired walls. The lateral load-strain responses from SG1 is plotted in Figure 5.29 and Figure 5.30 for Wall RSWS-R and Wall RSWN, respectively.

The first crack (S2) (Figure 5.11) initiated in the conventional wall at a drift of 0.5% with an applied load of 128.12 kN. This drift level resulted in yielding of Wall RSWS-R at an average load of 127.34 kN. The formation of the crack can be identified in Figure 5.29 by the sudden increase in strain values from 0.2% to 0.9% that yielded the steel bars near the base of the wall. The cracked concrete was no longer able to carry the load. Thus, the stresses were transferred to the surrounding steel located in

between cracks through bond. Therefore, in this region, the stresses in the cracked concrete were reduced while the steel bar experiences an increase in stresses and strains. No cracks were observed in this region for Wall RSWN throughout the entire testing.

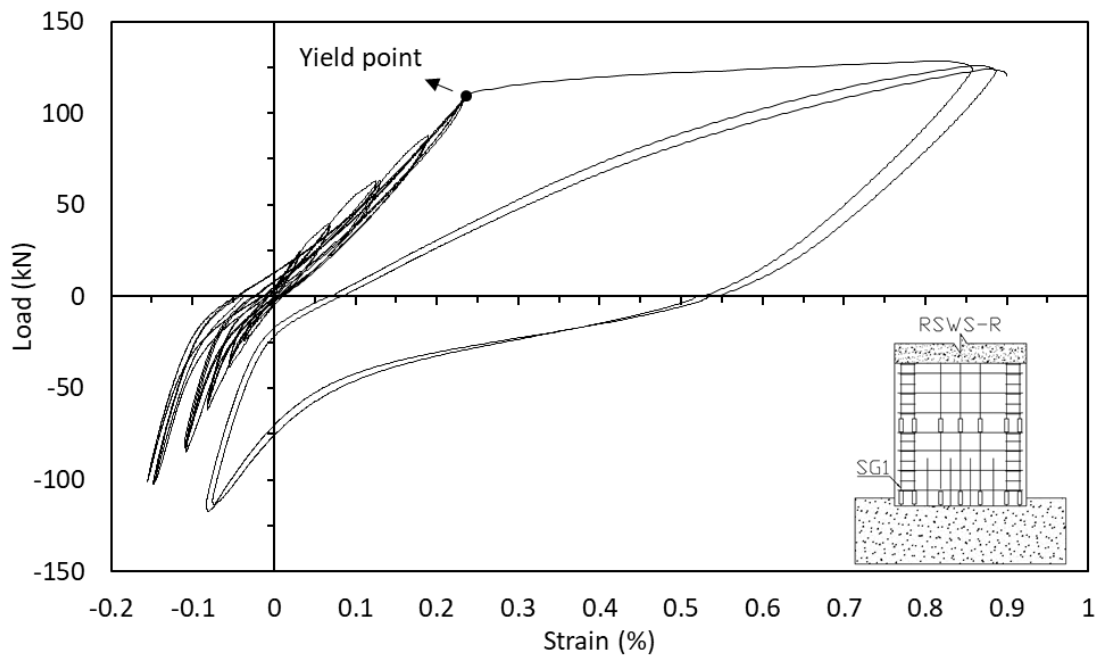


Figure 5.29 Load – strain response of Strain Gauge SG1 for Wall RWS-R

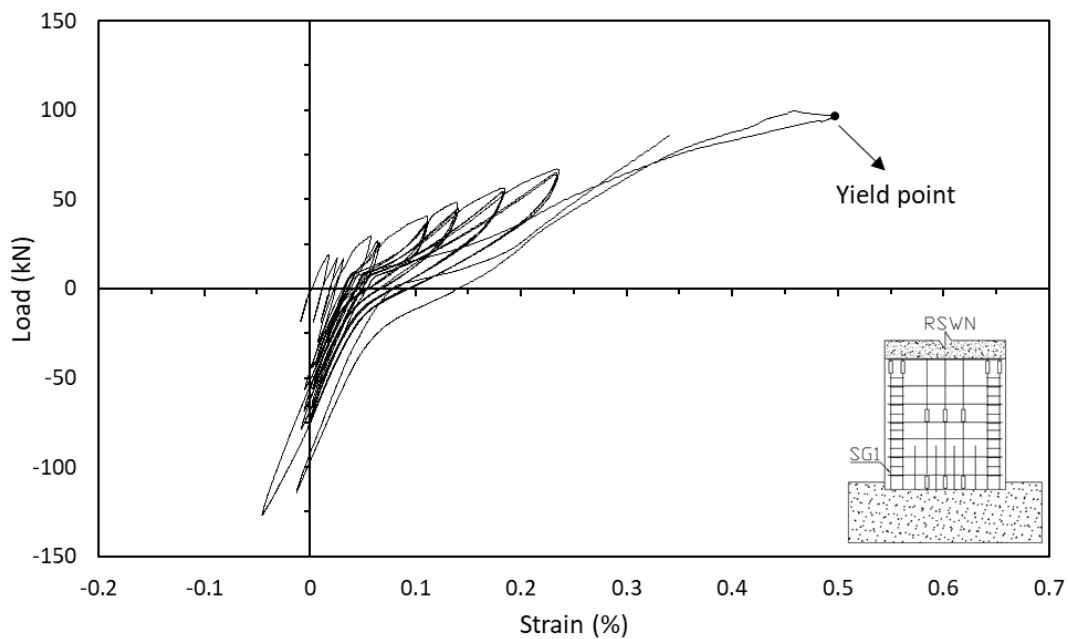


Figure 5.30 Load – strain response of Strain Gauge SG1 for Wall RSWN

The data permitted a comparison between the reinforcement utilized in the boundary regions for both walls (Figure 5.29 and Figure 5.30). It became evident the effect of the reduced stiffness of the SMA bars in the wall relative to the steel reinforcing bars. The yield point of the strain gauge placed on the steel reinforcement in the bottom boundary region of Wall RSWS-R yielded at 0.2% at an applied load of 127.34 kN and drift of 0.5%, while the strain gauge placed at the same location on the SMA rebar for Wall RSWN, yielded at almost two and half times the strain of the steel in Wall RSWN. This corresponds to a lateral load of 96.23 kN and a drift of approximately 1%.

As discussed in Section 5.1, the heavy damage in the repaired walls occurred in the region between the termination of the starter bars and the lower end of the upper mechanical couplers. Strain Gauge SG14 was mounted in the web region reinforcement within this area. Figure 5.31 and Figure 5.32 illustrate the load – strain responses of Strain Gauge SG14 for Walls RSWS-R and RSWN, respectively.

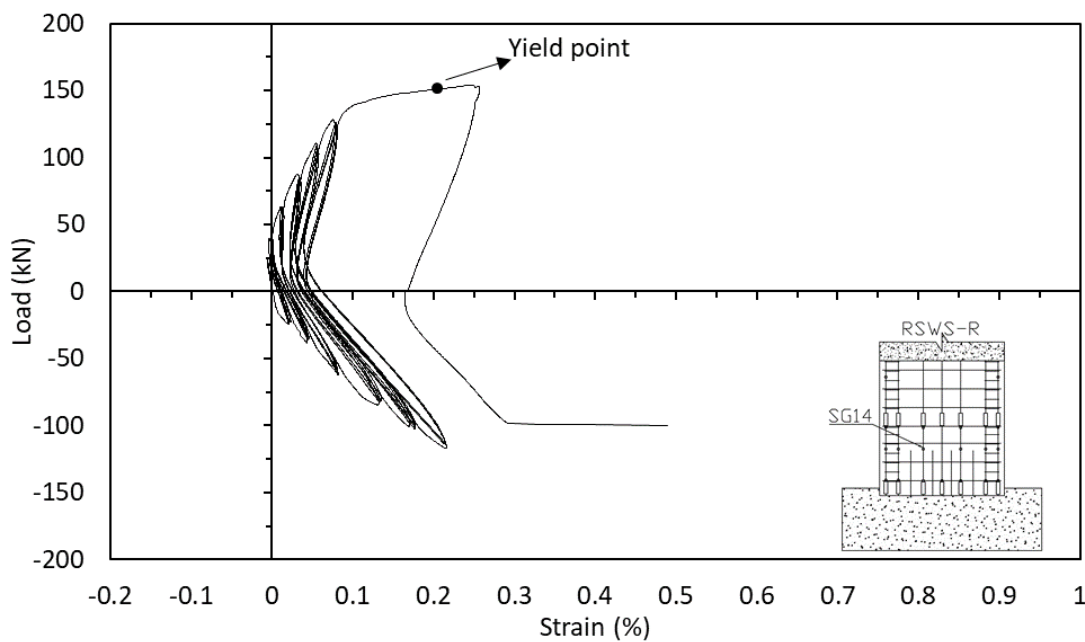


Figure 5.31 Load – strain response of Strain Gauge SG14 for Wall RSWS-R

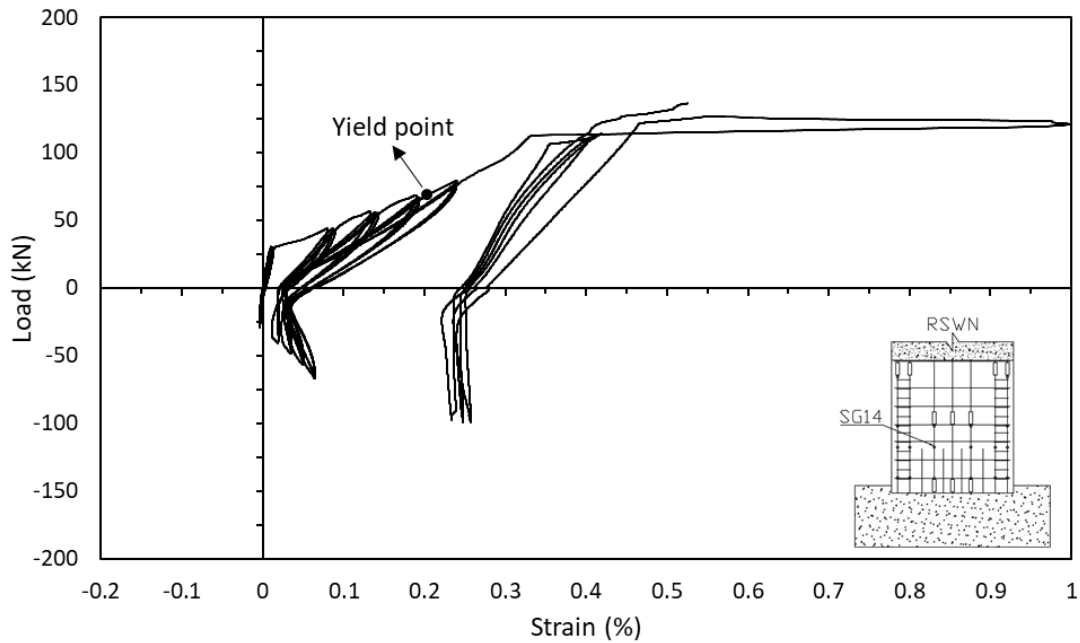


Figure 5.32 Load – strain response of Strain Gauge SG14 for Wall RSWN

For Wall RSWS-R, the web steel reinforcement located in the critical region yielded at a load of 150 kN. This load was attained near the completion of the 1% drift cycle for the overall response of the conventional steel wall. The yield point (SG14) of the reinforcement is in agreement with the appearance of substantial damage that was experienced at this level. Note that the steel bars in the web region experienced yielding after the reinforcement in the boundary zone (Figure 5.29). The strains experienced by the longitudinal web reinforcement (SG14) in Wall RSWN reached yielding at a load of 68 kN and drift of 0.4%, prior to the yielding of the SMA bars located in the boundary region (Figure 5.30). The behavior matches the global performance of Wall RSWN, where the steel web reinforcement suffered rupture along the main flexural cracks, with no apparent damage to the SMA bars.

The response of the reinforcement located at the top of the plastic hinge region is provided in Figure 5.33 and Figure 5.34, illustrating the load–strain response of Strain Gauge SG30, located approximately 895 mm measured from the base of the walls. The strain gauge was able to capture for Wall RSWS-R yielding of the boundary steel reinforcement at a load of 107.5 kN and drift of approximately 0.5%; at the same location, the SMA bars in Wall RSWN yielded at a lateral load of

110 kN and drift of 1%, but with a significantly larger strain. The results indicated yielding in the vertical reinforcement throughout the entire height of the plastic hinge region. Furthermore, the superelastic capacity of the SMA bars was fairly evident in comparison to the steel reinforcement. Upon unloading from the peak strain recorded by SG30 in Wall RSWS-R (0.3%), the plastic strain was 0.09%. For the same strain level, SG30 in Wall RSWN recorded a residual strain of 0.06%. Furthermore, the maximum strain capture from SG30 in the Wall RSWSN was 0.7%, in which the SMA bar was able to recover approximately 65%.

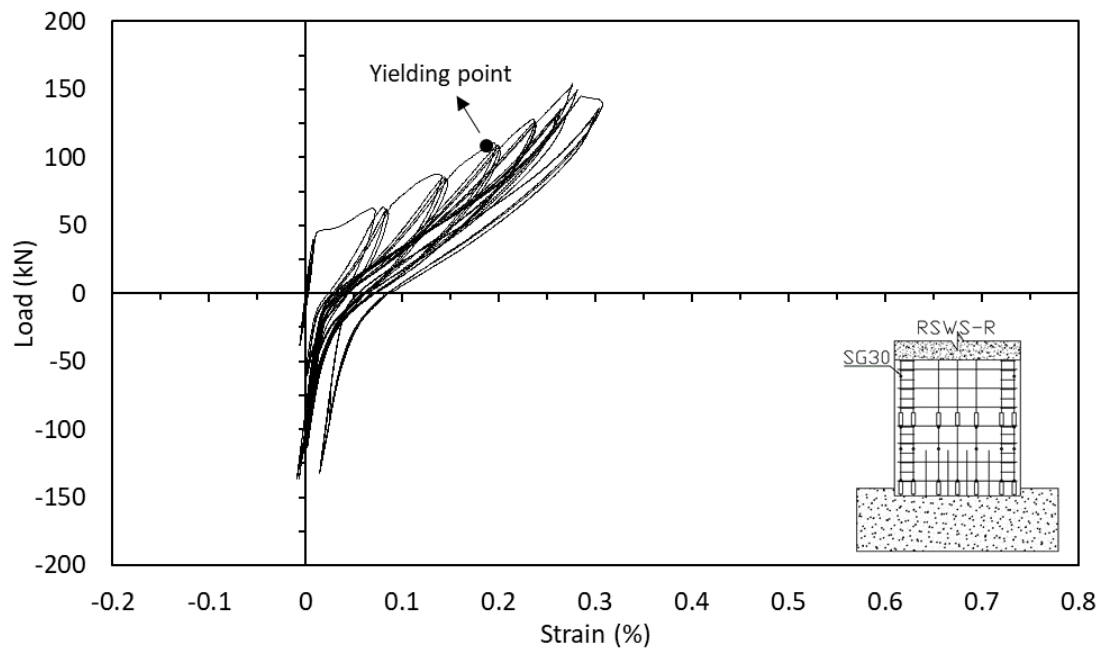


Figure 5.33 Load – strain response of Strain Gauge SG30 for Wall RSWS-R

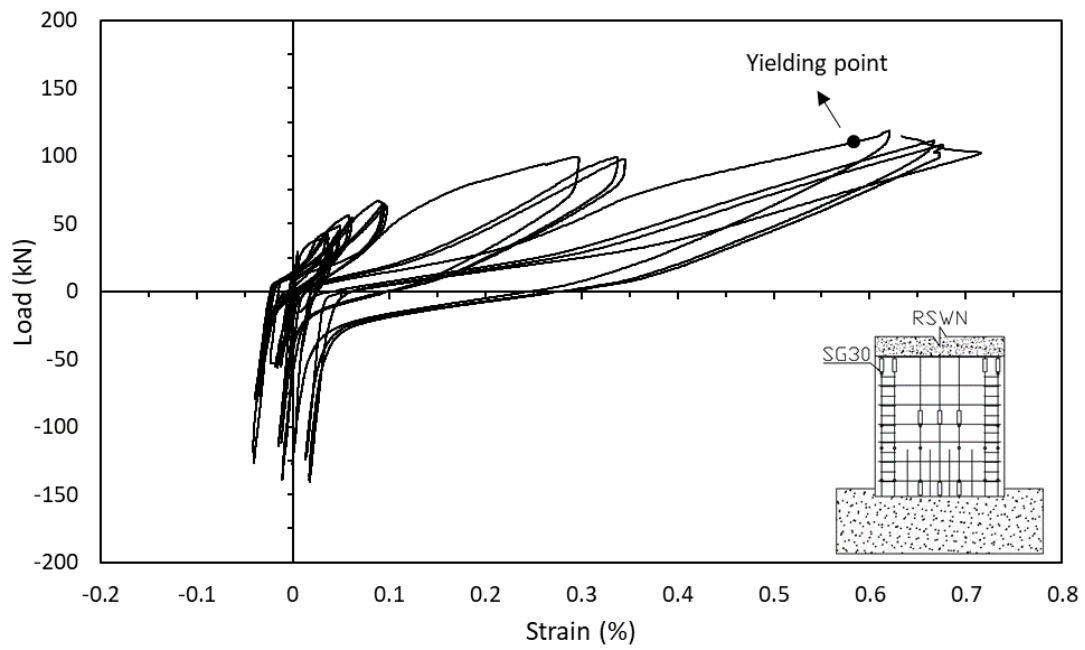


Figure 5.34 Load – strain response of Strain Gauge G30 for Wall RSWN

The load-strain responses of all the strain gauges used in the repaired walls are available in Appendix B.

CHAPTER 6. DISCUSSION OF RESULTS

6.1 Introduction

The global responses of the repaired walls were further analyzed to compare their performance in terms of strength and stiffness. Thereafter, a detailed description and further comparison are established between the pre and post-repaired behavior of the walls, including energy dissipation, recovery capacity, overall load-displacement response, and failure modes.

6.1.1 Stiffness and Strength

The secant stiffness can be utilized to characterize the nonlinear response of the repaired walls. It was calculated as the ratio of the peak load to the corresponding displacement in the positive direction for the first repetition of each drift level. Figure 6.1 presents the secant stiffness-displacement ductility response for the repaired walls.

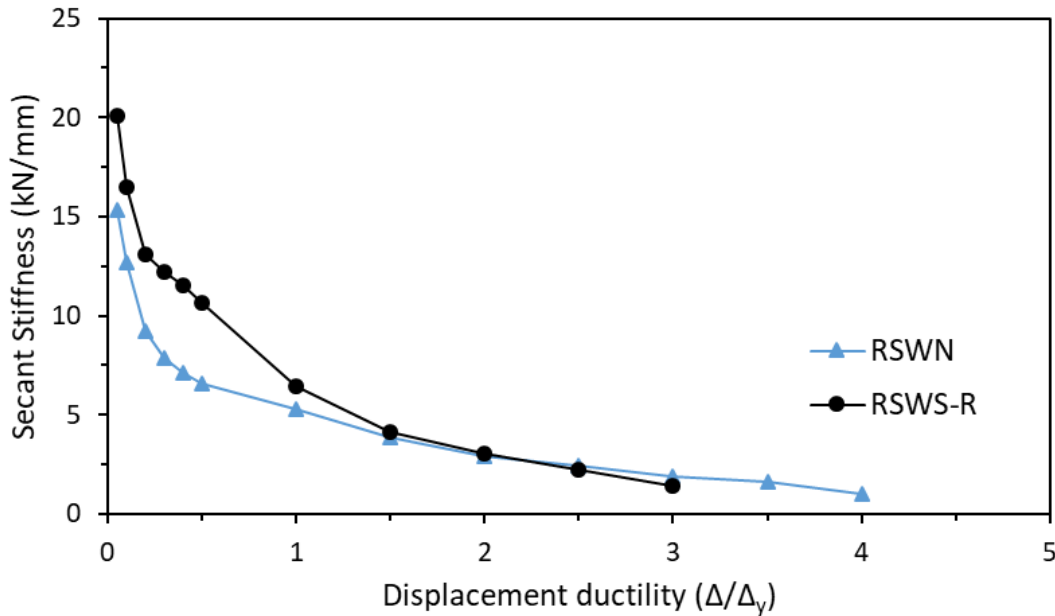


Figure 6.1 Secant stiffness-displacement ductility response for repaired walls

Wall RSWN developed a lower initial stiffness, and therefore, higher yield displacement. The initial stiffness attained by Wall RSWs-R was 20.09 kN/mm, while Wall RSWN experienced stiffness of 15.37 kN/mm (23.5% reduction). Moreover, at a displacement ductility of 1, the secant stiffness of Wall RSWN is marked by a 36.5% reduction in comparison to Wall RSWs-R due to the yielding of the steel reinforcement within the web region prior to the yielding of the SMA bars in the boundary zone. Thereafter, the secant stiffness values behaved in a similar manner for both walls and were able to develop a ductile behavior with a gradual decrease of stiffness prior to failure.

The overall load-displacement responses previously presented in Figure 5.20 for Wall RSWs-R, and Figure 5.21 for Wall RSWN, illustrated significant strength degradation during the second and third repetitions for the same target displacement. To assess the effect of cyclic degradation in strength, the strength ratio of the second repetition to the first repetition for the same drift level is provided in Figure 6.2.

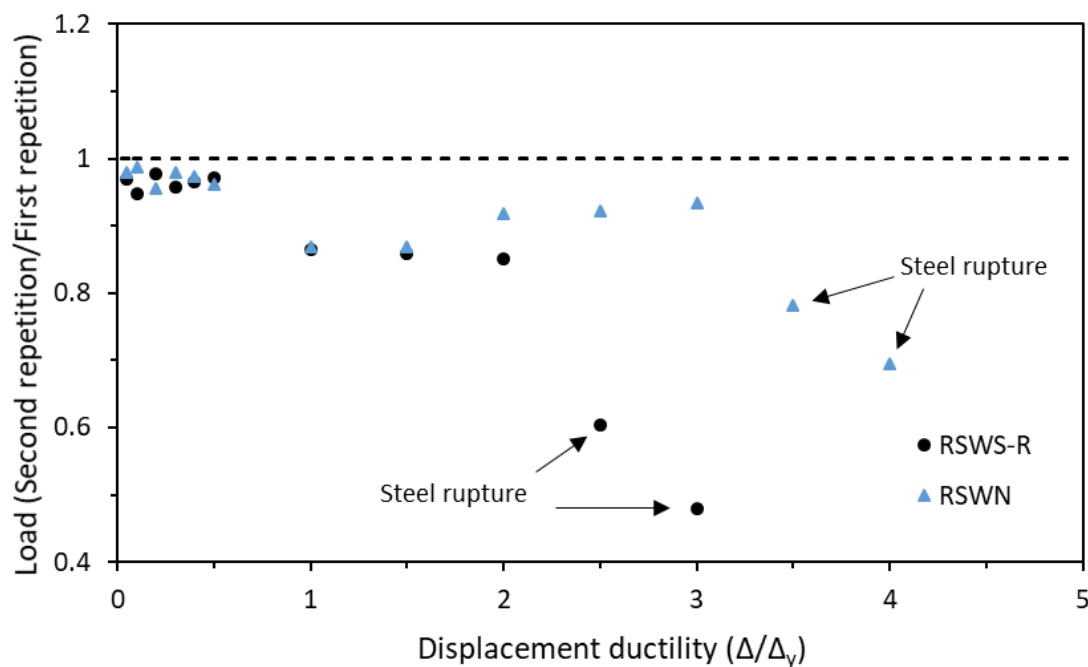


Figure 6.2 Effect of cyclic loading degradation for repaired walls

During the second repetition of loading, both walls experienced a strength degradation of 3% up to the displacement ductility of 0.5. Thereafter, the development of the first major crack for both walls marked a lateral load degradation of approximately 14% for the second repetition from a displacement

ductility of 1 up to failure. The sudden decays in the strength ratios are attributed to fracturing of the steel reinforcement in the boundary and web regions for Walls RSWS-R and RSWN, respectively.

6.1.2 Vertical Displacements

The localized damage that surfaced at approximately 400 mm from the base of the walls is evident through the vertical displacements that were recorded along the height of the walls. The repaired walls were equipped with a series of cable potentiometers that monitored the vertical displacements: at the top beam (CP5 and CP6) relative to the base; at the top of the plastic hinge region relative to the base of the wall (CP11 and CP12); and at the base of the wall (CP7 and CP18). The location of the instruments is depicted in Figure 4.39. Figure 6.3 and Figure 6.4 present the vertical displacements for RSWS-R and RSWN, respectively.

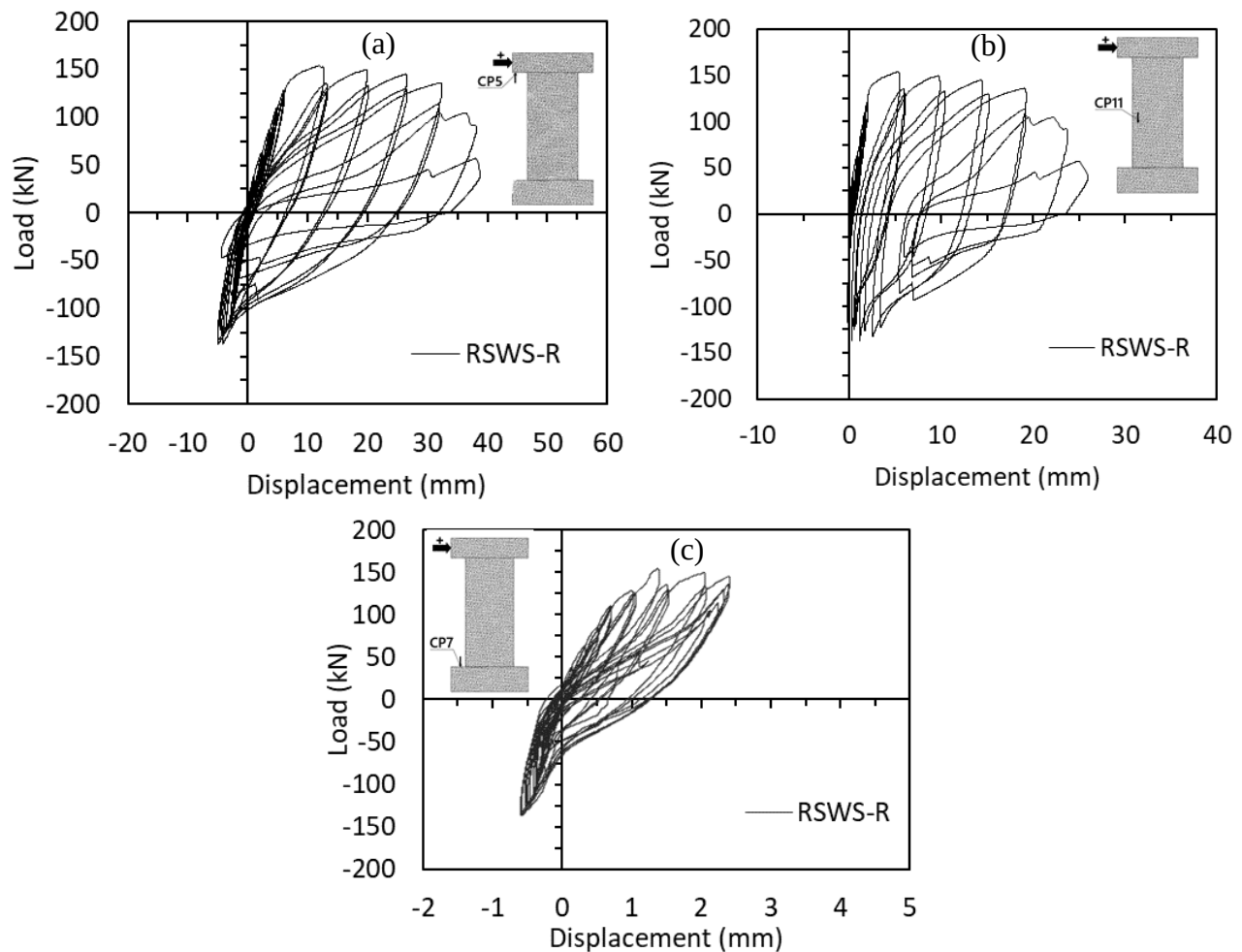


Figure 6.3 Vertical displacements in Wall RSWS-R: loading beam (CP5); b) plastic hinge (CP11); base of wall (CP7)

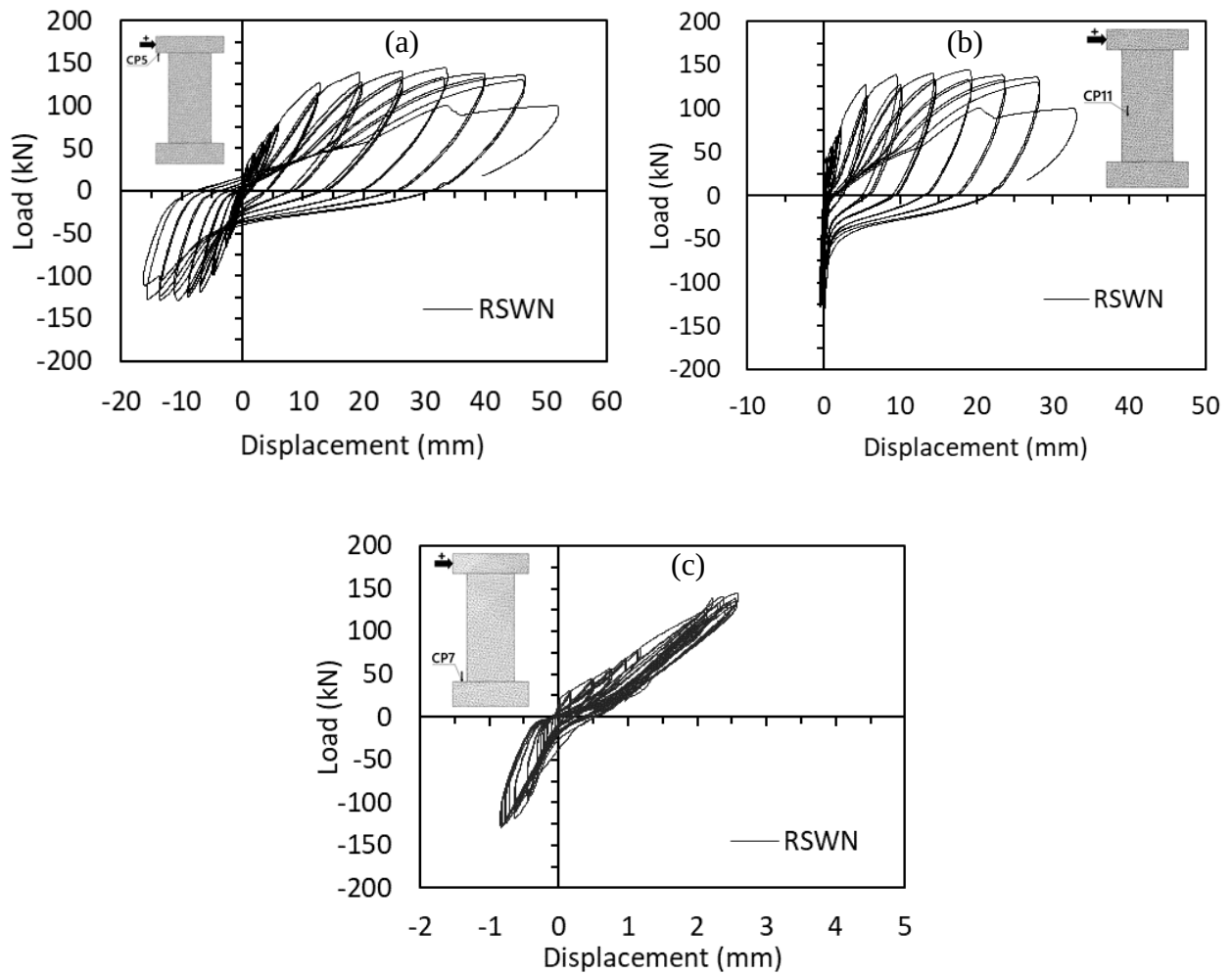


Figure 6.4 Vertical displacements in Wall RSWN: loading beam (CP5); b) plastic hinge (CP11); base of wall (CP7)

The vertical elongations resulted from the concrete cracking and yielding of the longitudinal reinforcement. To further compare the results, the envelope of the vertical displacements for the first repetition of loading in the positive direction at the top loading beam (CP5), at a height of 800 mm from the base (CP11), and 50 mm from the base (CP7) are plotted against the lateral displacement ductility in Figure 6.5.

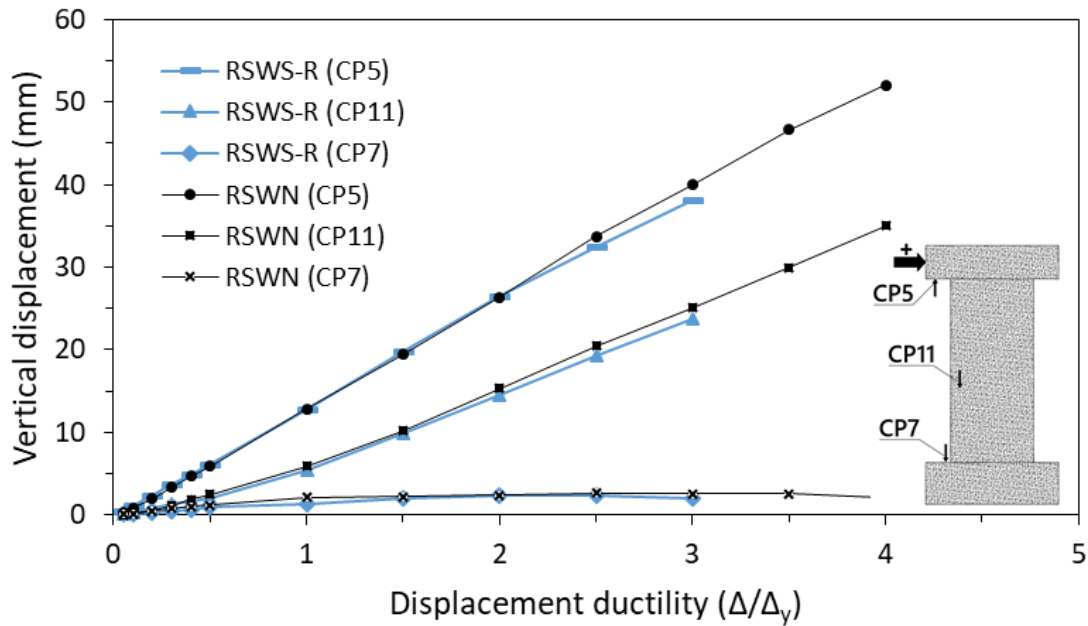


Figure 6.5 Envelope of vertical displacements for Walls RSWS-R and RSWN

The results demonstrate that during similar lateral displacement ductility levels, the repaired walls behave in an almost identical manner, with slightly higher vertical displacements experienced by Wall RSWN. During the displacement ductility level of 2.5, the vertical displacement in Wall RSWS-R was 32.43 mm at the top beam, 19.23 mm at the top of the plastic hinge, and 2.44 mm at the base of the wall. Similarly, for Wall RSWN the vertical displacement was 33.73 mm at the top loading beam, 19.29 mm at the top of the plastic hinge, and 2.51 mm at the base of the wall, which represented a 3.85%, 0.31%, and 2.86% increase in vertical displacements, respectively, in comparison to Wall RSWS-R. Note that the relatively small vertical displacements recorded near the base of the wall confirm that the base of the wall remained intact with the foundation and that rocking along the base was prevented.

Furthermore, the difference in the vertical displacements between the top beam and the plastic hinge demonstrates that substantial vertical displacements were experienced over the height of the plastic hinge due to the localized cracking that surfaced in this region. In the post-yield response, the vertical displacements recorded at the top of the plastic hinge zone represented approximately 60% of the displacements at the top beam for the repaired walls. In addition, this trend continued with increasing displacement ductility, where the walls experienced larger vertical displacements over the plastic

hinge, indicating that the inelastic rotation was concentrated above the base of the walls. This is attributed to the single predominant cracking that appeared just above the starter bars.

6.1.3 Rotations

The rotations were determined using the vertical displacements recorded at the top loading beam, upper limit of the plastic hinge, and base of the wall to evaluate the flexural response of the repaired walls considering the effect of the damage (concrete cracking and reinforcement yielding). The hysteretic load-rotation behaviors at the top loading beam are depicted in Figure 6.6 and Figure 6.7 for Walls RSWS-R and RSWN, respectively. Wall RSWN experienced a higher rotation throughout the experimental testing. The rotation at yielding was 10.7×10^{-3} rad and 29.9×10^{-3} rad for Walls RSWS-R and RSWN, respectively. The larger rotation in Wall RSWN is attributed to the larger displacement at yielding. The inelastic rotation, calculated as the difference between the ultimate and the yielding rotations, was 28.5×10^{-3} rad and 32.2×10^{-3} rad for Walls RSWS-R and RSWN, respectively. The difference lies in the early development of the peak lateral load in Wall RSWS-R, at a rotation of 17.2×10^{-3} rad, while Wall RSWN experienced a rotation of 44.7×10^{-3} rad at peak load. A notable difference is the capacity of Wall RSWN to recover the large rotations.

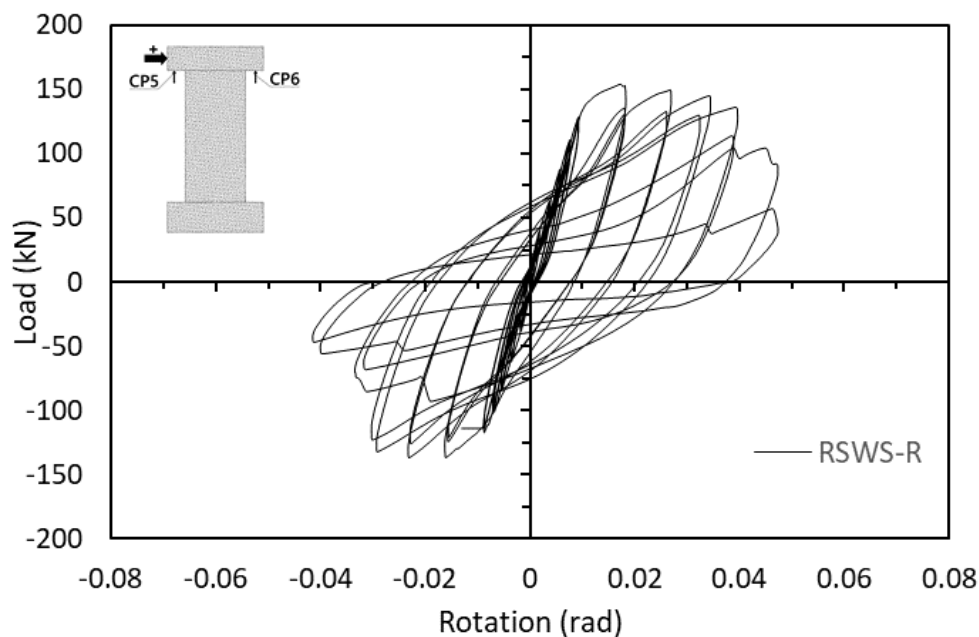


Figure 6.6 Load-rotation response for Wall RSWS-R at the top loading beam

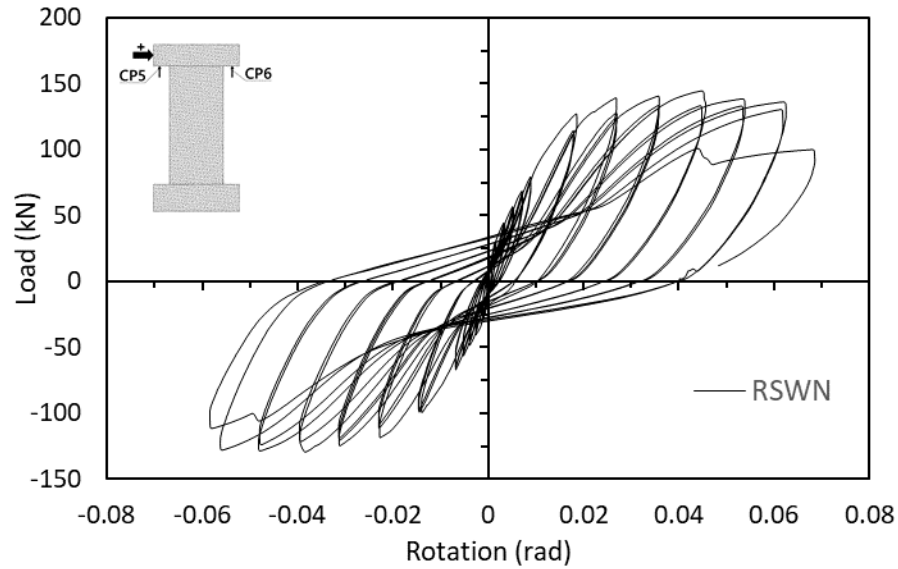


Figure 6.7 Load-rotation response for Wall RSWN at the top loading beam

The rotation of the walls was also recorded over a height of 800 mm from the base, corresponding to the assumed plastic hinge region. Additionally, rotation was calculated 50 mm from the base of the wall itself. Figure 6.8 and Figure 6.9 illustrates the calculated rotations for Wall RSWS-R and Wall RSWN, respectively. Both walls experienced negligible rotations, near the base, as depicted in Figure 6.8 (a) and Figure 6.9 (a). At the plastic hinge level, it is noteworthy that the repaired walls experience comparable maximum rotations, where Wall RSWS-R developed 3.02×10^{-3} rad and Wall RSWN achieved 3.50×10^{-3} rad.

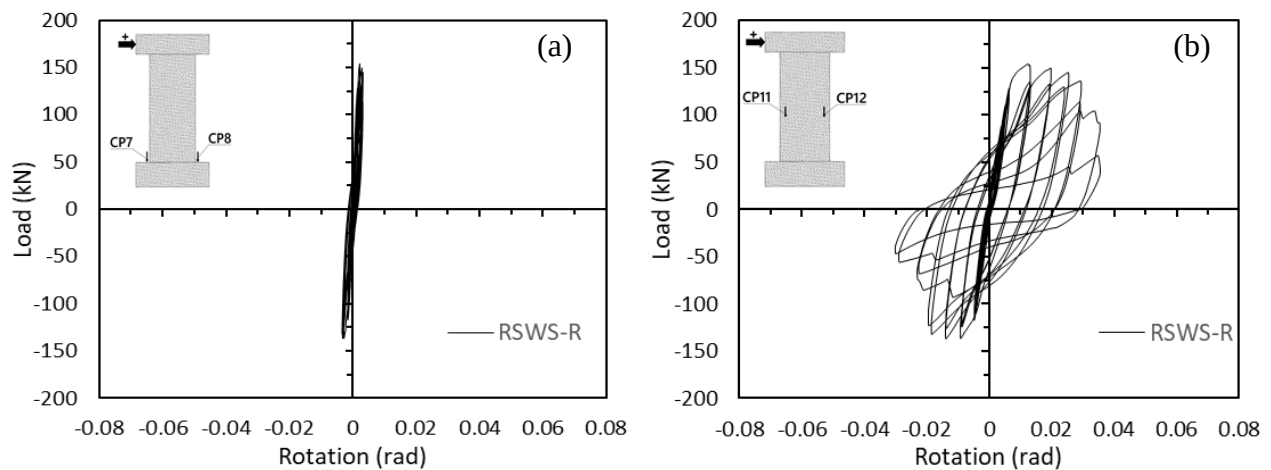


Figure 6.8 Load-rotation response for Wall RSWS-R at: (a) base of the wall; (b) upper limit of the plastic hinge

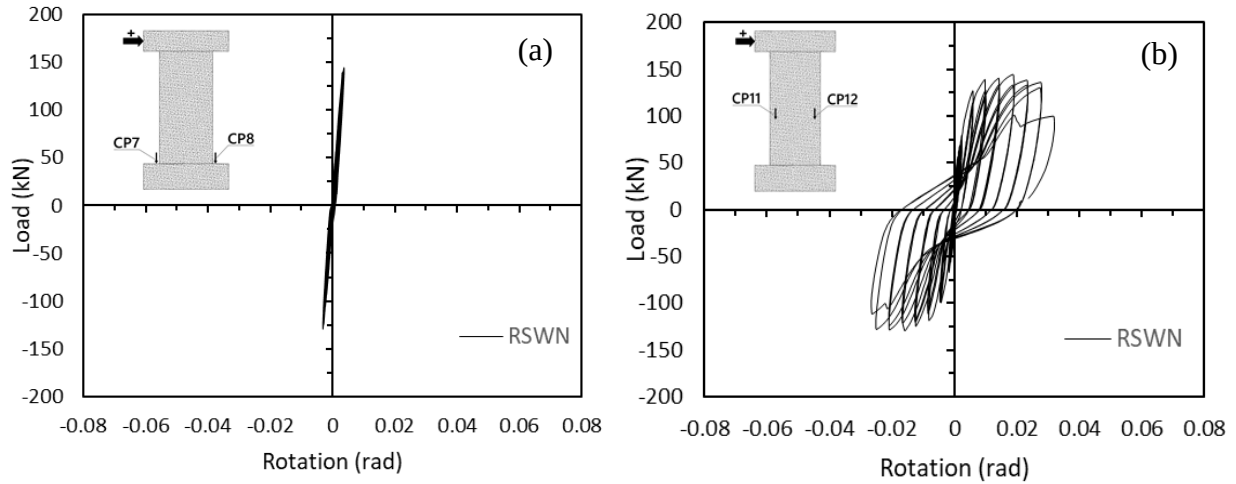


Figure 6.9 Load-rotation response for Wall RSWN at: (a) base of the wall; (b) upper limit of the plastic hinge

6.1.4 Effect of Starter Bars

The addition of the starter bars in the web region of the repaired walls aimed to control sliding of the walls along the cold joint between the ECC repair concrete and the existing concrete at the foundation level. In addition, the starter bars were used to eliminate rocking that was experienced by the original walls along their base that had a significant effect on the response of Wall SWN (Morcos, 2020). Controlling both sliding and rocking along the base would result in a response that was controlled within the wall panel. This, in turn, would ensure that the benefits of the ECC are realized. Cable potentiometer CP4 was located 50 mm above the base of the walls to monitor the horizontal displacements throughout the experimental testing (Figure 4.39). Figure 6.10 illustrates the hysteretic load-displacement response of CP4 for the repaired walls. The highest lateral displacement recorded by CP4 was 0.38 mm for Wall RSWN and 0.31 mm for Wall RSWS-R. The negligible displacements validated that the addition of the starter bars eliminated sliding along the base of the walls.

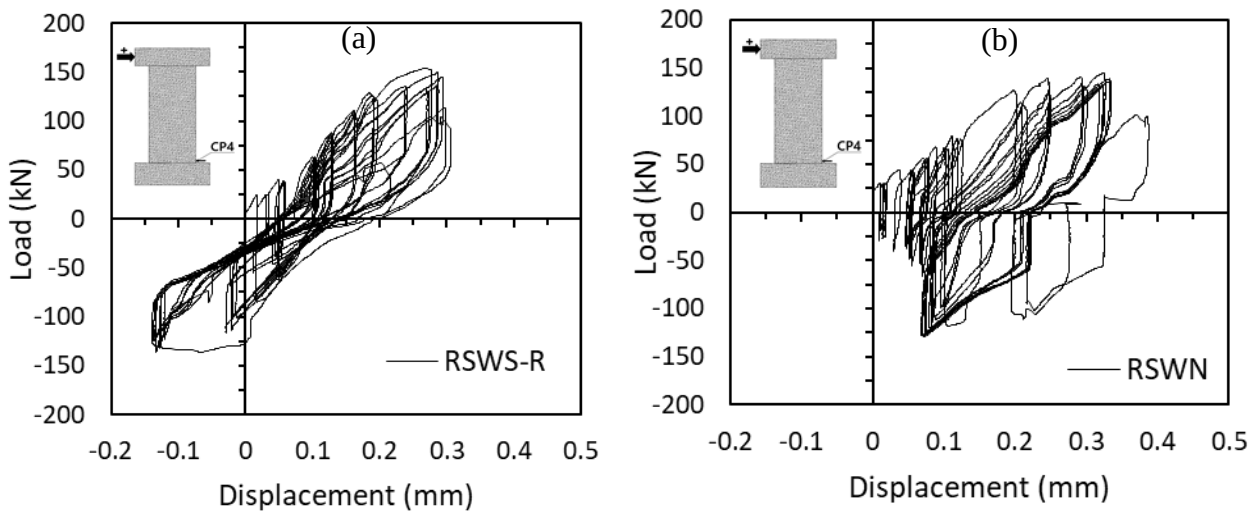


Figure 6.10 Lateral displacement at the base of the repaired walls: (a) RWS-R; and (b) RSWN

Furthermore, the starter bars eliminated the damage that was experienced at the base of the original walls. Figure 6.11 illustrates the state of the base of the walls at the end of testing of SWN and RSWN. It became evident that no significant cracking surfaced in the repaired walls along the height of the starter bars (300 mm), which was attributed to a high concentration of steel in this region that created a rigid section, shifting the failure mode higher into the walls.

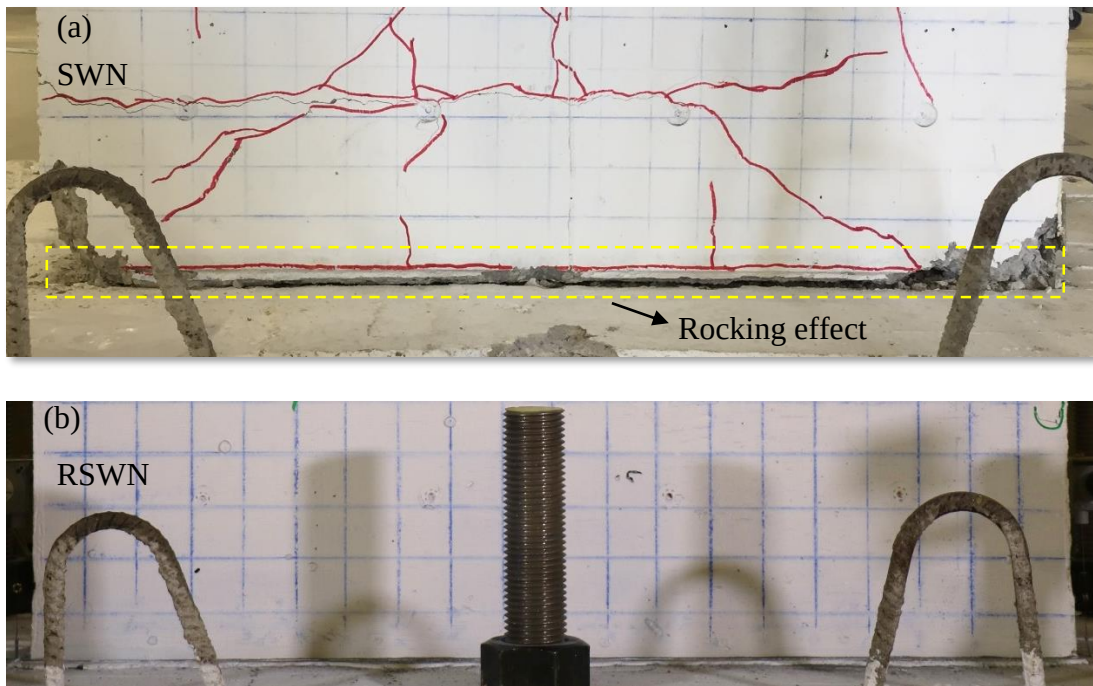


Figure 6.11 State of the base of the walls at the end of testing: (a) SWN; and (b) RSWN

6.1.5 Shear Distortions

The average shear strains were calculated over the plastic hinge for the repaired walls following the approach established by Oesterle et al. (1976) using Equation 6.1:

$$\gamma_{avg} = (\delta_1 d_1 - \delta_2 d_2) / 2hl \quad [\text{Eq. 6.1}]$$

Where γ_{avg} represents the calculated average shear strain; d_1 and d_2 denote the reference length of the cable potentiometers CP13 and CP14 located diagonally over the plastic hinge (Figure 4.39); and δ_1 and δ_2 refer to the change in length of CP13 and CP14, respectively. Finally, h and l are the height and the width of the cable potentiometers. The dimensions were the same for the repaired walls, d_1 and d_2 were 1130 mm in length, and a square-area was previously defined to study the shear distortions, with identical dimensions of $h = l = 800$ mm. Figure 6.12 and Figure 6.13 depicts the load-shear strain response for Walls RSWS-R and RSWN, respectively.

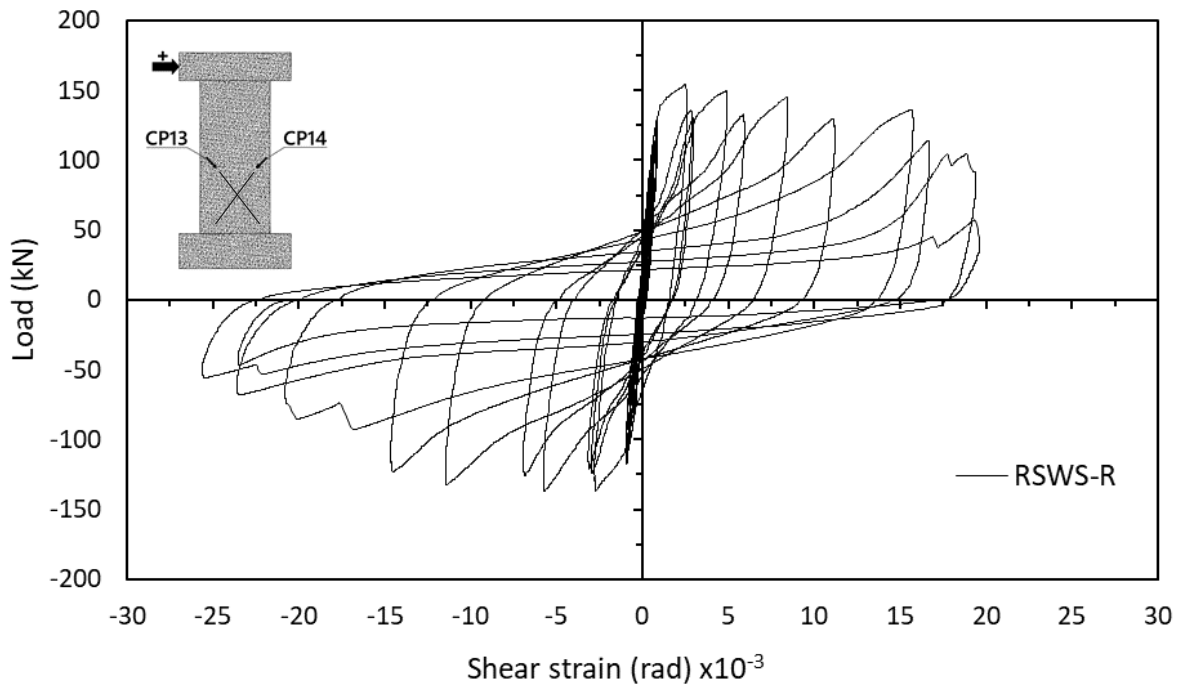


Figure 6.12 Load – shear strain response of Wall RSWS-R

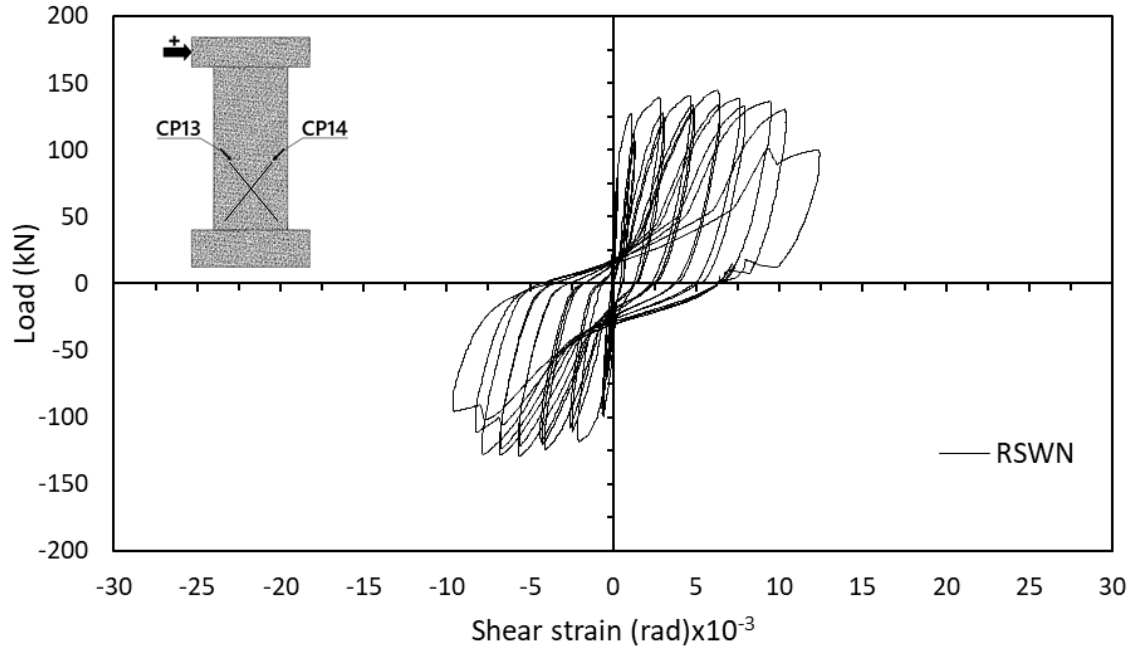


Figure 6.13 Load – shear strain response for Wall RSWN

Both walls experienced similar average shear strains at yielding, 0.80×10^{-3} rads for Wall RSWS-R and 0.86×10^{-3} rads for Wall RSWN. The latter experienced higher average shear strains at the peak lateral load of 2.44×10^{-3} rads as compared to 1.46×10^{-3} rads for Wall RSWS-R. However, as the lateral displacements increased, Wall RSWS-R experienced significantly higher shear strains, reaching a maximum average shear strain of 19.66×10^{-3} rads, while Wall RSWN was subject to 12.43×10^{-3} rads. The high strains can be attributed to the increased shear resistance of ECC and its ability to sustain high shear deformations (Shimizu et al. 2005). In fact, none of the walls experienced shear failure or significant shear damage, indicating that the shear capacity of ECC combined with the transverse reinforcement was not exceeded. The main difference between both walls is denoted by the higher shear strain recovery capacity of the ECC-SMA wall, whereas the ECC-conventional wall sustained high residual shear straining. The re-centering characteristics of the SMA bars promoted the symmetrical behavior of Wall RSWN, whereas the conventional steel wall experienced asymmetrical response, with an average shear strain of 19.66×10^{-3} rads in the positive direction and 25×10^{-3} rads in the negative side. This phenomenon is characterized as ratcheting, where the behaviour accumulates higher deformation in one direction of loading. In addition, no accentuated

pinching was observed in the walls explained by the lack of shear cracks within the plastic hinge region.

6.2 Pre- and Post-Repairing Behavior

The feasibility of the repairing procedure proposed in this research is evaluated in this section. A comparison through the different performance parameters is established to determine the behavior of the walls in their pre- and post-repairing state. Moreover, this section highlights the influence of the Engineered Cementitious Composites (ECC) in the response of the walls as a primary repairing material.

6.2.1 Global Load-Displacement Responses

The hysteretic lateral load-displacement response for the walls in their pre- and post-repairing state are depicted in Figure 6.14 and Figure 6.15 for the conventional steel wall and SMA wall, respectively. The performance of the repaired walls is marked by a higher lateral load resistance and comparable displacement capacity in comparison to the original walls. Wall RSWS-R dissipated more energy than Wall SWS-R, with slightly wider hysteretic curves characterized by the development of higher loads for the same lateral displacements, as shown in Figure 6.14. Similarly, a significant increase in the lateral strength was observed for Wall RSWN, which was attained at a lower lateral displacement (Figure 6.15). Wall RSWN experienced a reduction in the drift capacity relative to the original wall. No evident difference in pinching was observed in the repaired SMA wall in comparison to the original wall. Table 6.1 summarizes the performance points for the original and repaired walls.

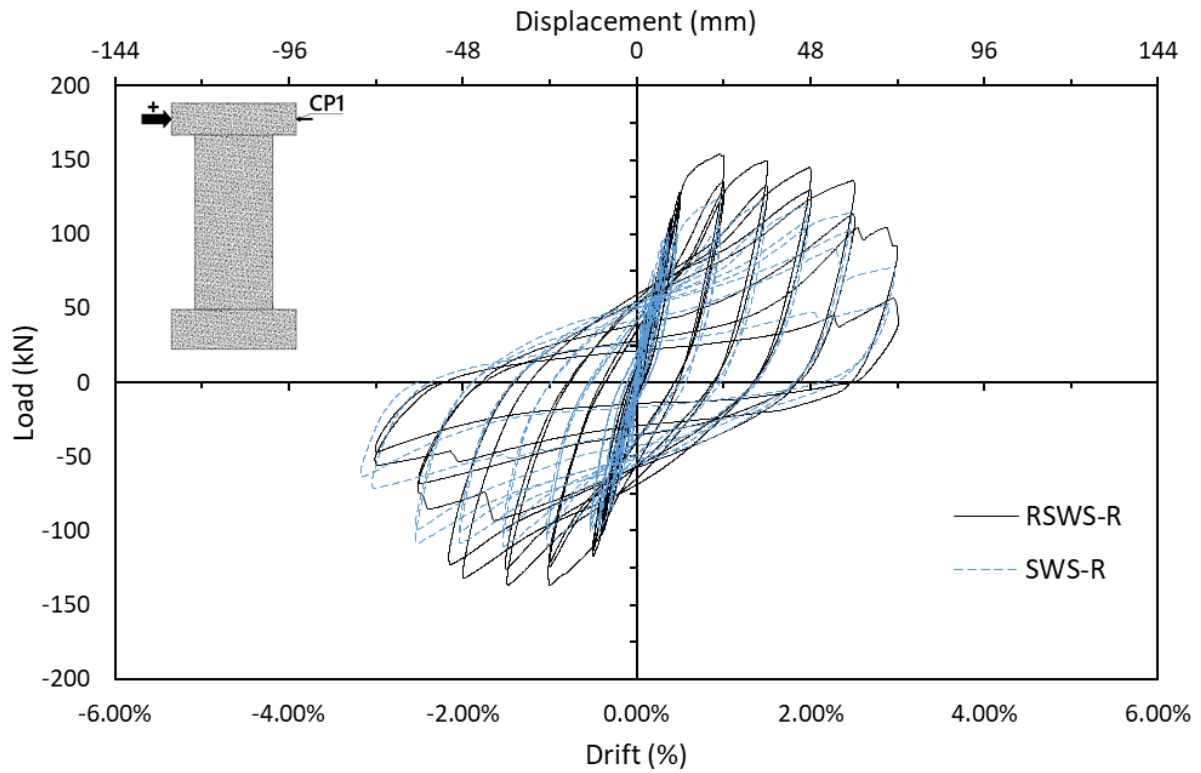


Figure 6.14 Load-displacement response of Walls SWS-R and RSWs-R

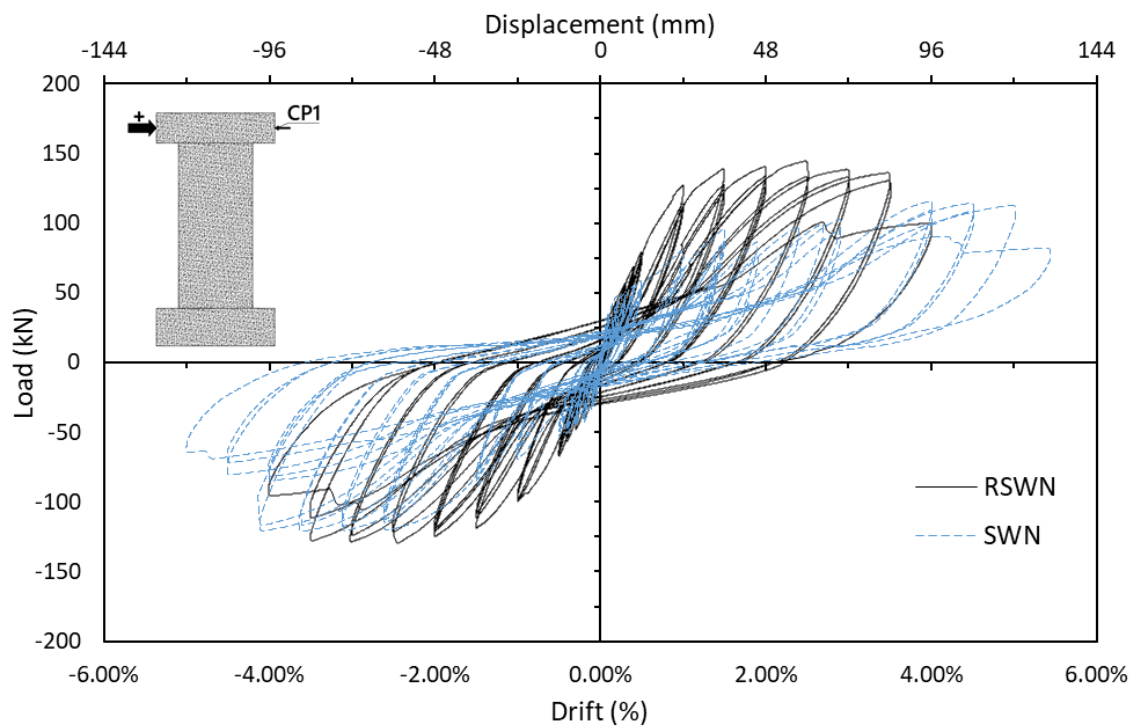


Figure 6.15 Load-displacement response of Walls SWN and RSWN

Table 6.1 Performance points of original and repaired walls

Wall	Yield			Peak			Ultimate			μ
	δ mm	Drift (%)	Load kN	δ mm	Drift (%)	Load kN	δ mm	Drift (%)	Load kN	
SWS-R	8.7	0.4%	97	30	1.3%	118	60	2.5	101	6.9
RSWS-R	13.6	0.6%	127	29.3	1.2%	145	56	2.3%	128	4.1
SWN	45.0	1.9%	93	91	3.8%	118	103	4.3%	113	2.3
RSWN	40.9	1.7%	123	65.8	2.7%	137	84	3.5%	124	2.0

6.2.2 Secant Stiffness

The secant stiffness-drift response for the original and repaired conventional steel wall and the SMA wall are illustrated in Figure 6.16 (a) and (b), respectively. In general, the walls in the pre- and post-repaired state exhibited a steady decay of the secant stiffness with increased drift up to failure, resulting in a low stiffness at the end of testing. Moreover, a lower initial stiffness was exhibited by the repaired walls in comparison to the original walls. The initial stiffness for Walls SWS-R and SWN were 32.4 kN/mm and 20.32 kN/mm, respectively, while Walls RSWS-R and RSWN developed stiffness of 20.1 kN/mm and 15.4 kN/mm, respectively. The probable cause is the pre-existing damage in the repaired walls in the concrete above the plastic hinge that was not repaired. However, once cracking initiated, the secant stiffness values of original walls promptly decreased to the stiffness of the repaired walls. With increased drift, the repaired walls exhibited slightly higher stiffness; this was attributed to the bridging effect of the PVA fibers in the ECC repair material, which controlled the cracking and delayed the degradation of stiffness and load capacity of the repaired walls. The effect is evident at a drift level of 1%, where the original steel wall developed a secant stiffness of 5.20 kN/mm, while Wall RSWS-R attained a stiffness of 6.50 kN/mm. Similarly, the original SMA wall, at 1% drift, experienced a secant stiffness of 2.90 kN/mm, whereas Wall RSWN sustained a stiffness of 5.29 kN/mm.

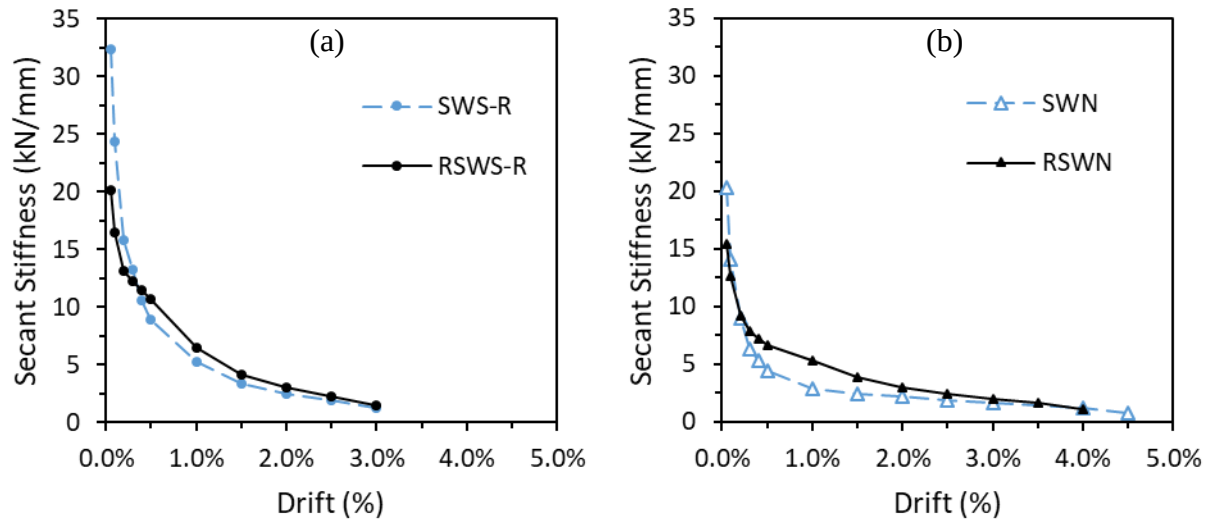


Figure 6.16 Secant stiffness – drift responses: (a) conventional steel wall; and (b) SMA wall

6.2.3 Lateral Load Capacity

The lateral load-drift envelopes of the repaired walls were developed utilizing the peak loads and corresponding displacements of the first repetition for the same target displacement in the positive direction. Figure 6.17 and Figure 6.18 illustrate the responses for Walls SWS-R and RSWS-R, and Walls SWN and RSWN, respectively. The original walls exhibited a higher initial stiffness in comparison to the repaired walls, which were tested with existing damage above the plastic hinge region. The repaired walls experienced yielding at a relatively similar drift level as the original walls, but with a significant increase of 25% in the corresponding lateral strength. Thereafter, Wall RSWS-R developed its peak lateral strength (145 kN) at a similar drift level, which represented a 19% increase in comparison to Wall SWS-R (Figure 6.17). Wall RSWN developed a 14% increase in lateral strength at the peak at a displacement of 65.8 mm (2.7% drift) in comparison to Wall SWN, which attained its peak strength at a displacement of 91 mm (3.8% drift) (Figure 6.18). The same trend continued up to ultimate, with an increase of 12% and 9% in lateral load resistance for Wall RSWS-R and Wall RSWN with respect to Wall SWS-R and Wall SWN, respectively. It is noteworthy, the shapes of the envelopes are similar for RSWS-R and SWS-R. The envelope of RSWN exhibits a response that can be characterized by a bilinear behavior; whereas SWN provides a tri-linear response.

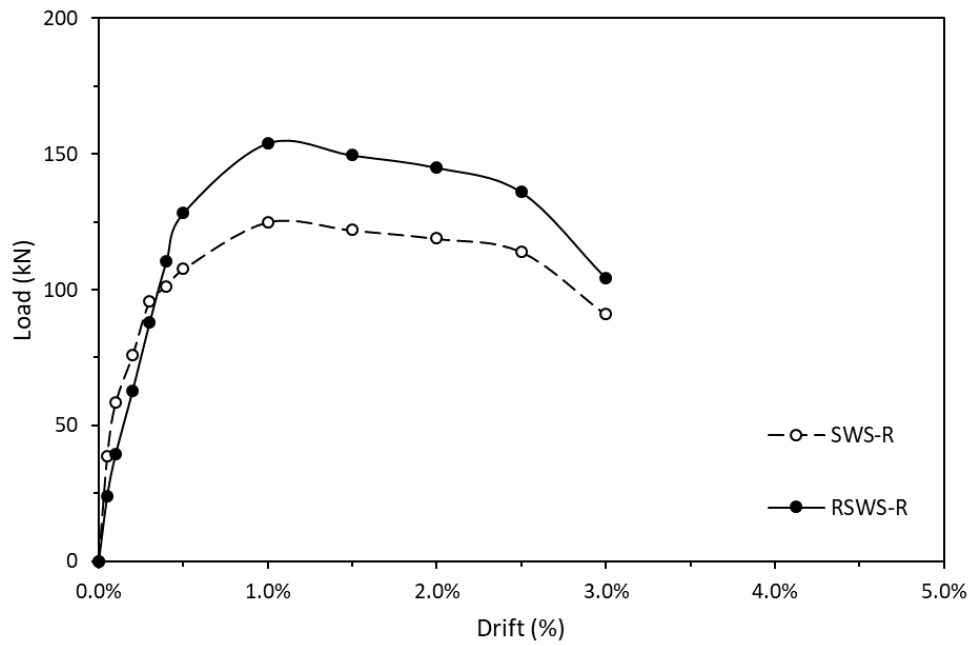


Figure 6.17 Lateral load-drift envelope for Walls SWS-R and RSWS-R

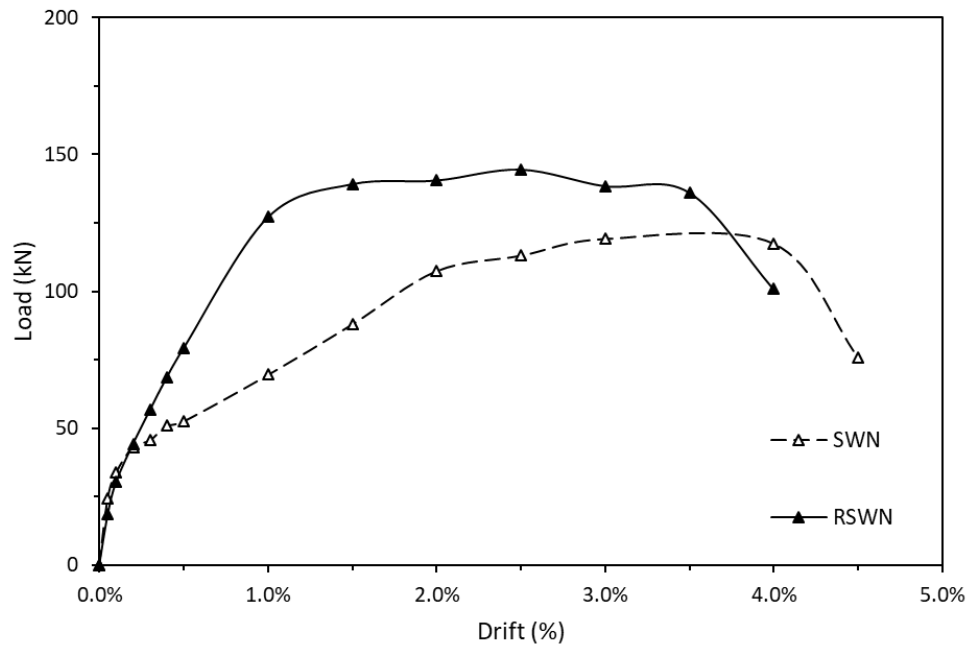


Figure 6.18 Lateral load-drift envelope for Walls SWN and RSWN

6.2.4 Recovery Capacity

The ability of the repaired walls to recover from large inelastic displacements is illustrated in Figure 6.19 for the same drift level (2.5%). The responses of Walls RSWS-R and RSWN are based on the first repetition of loading. During the unloading phase in the positive direction, Wall RSWS-R experienced a plastic offset of 44.8 mm, whereas Wall RSWN developed a residual displacement of 27.5 mm, which corresponds to a 55% reduction of the imposed displacement (60 mm). The same trend was observed in the negative direction. Note that Wall RSWS-R experienced rupture of the longitudinal steel reinforcement, provoking a decay in lateral strength. The plastic offsets remained symmetrical for both walls at this drift level.

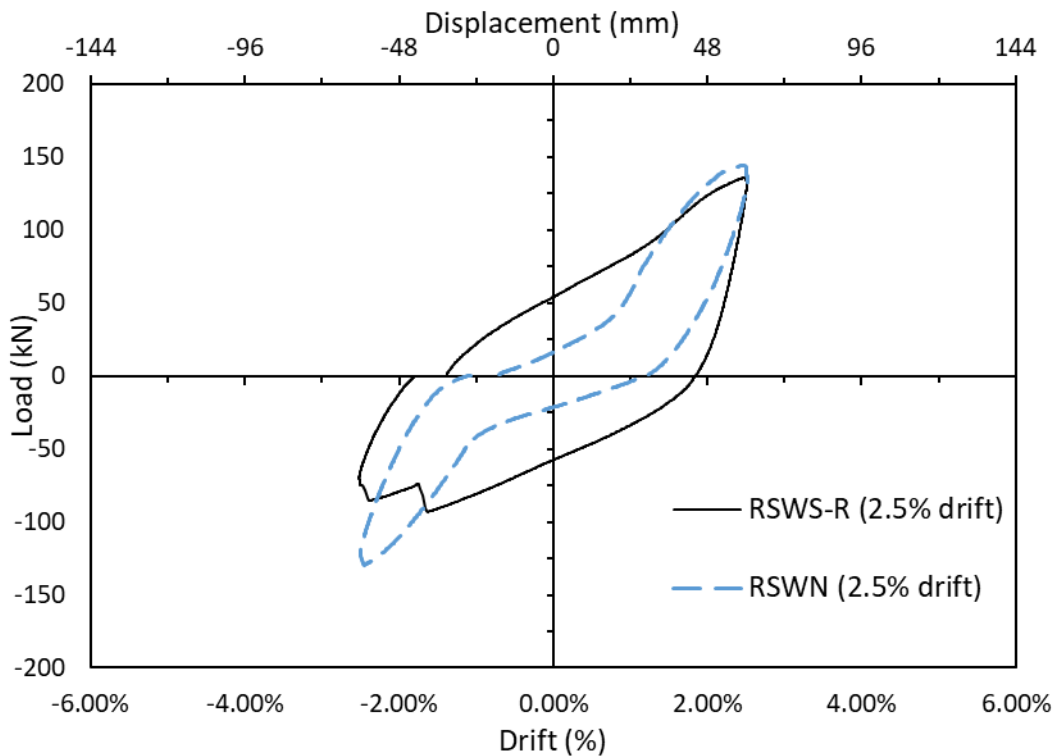


Figure 6.19 Comparison of hysteretic response at 2.5% drift for Walls RSWS-R and RSWN

The substantial difference in displacement recovery between the repaired SMA wall and the repaired steel wall is attributed to the self-centering capacity of the SMA bars within Wall RSWN. However, the use of ECC in both walls furthered modified the recovery capabilities in comparison to the original

walls. Figure 6.20 and Figure 6.21 present the recovery capacity-drift response for the conventional steel walls and SMA walls, respectively.

The enhanced recovery attained by the repaired walls is partly due to the compatible deformation of the ECC with the reinforcement and the reverse transformation of the SMA bars upon unloading for the SMA walls. In addition, the placement of the starter bars shifted the critical section away from the base of the wall, preventing both sliding and rocking, which in turn, permitted the characteristics of the ECC and SMA materials to better contribute to the response of the walls. At the onset of testing up to 0.3% drift, the original and repaired steel walls behaved in a similar manner. At 0.4% drift, Wall SWS-R recovered 76% from the imposed displacement, while Wall RSWS-R retrieved 85.4%. Similar behavior was retained throughout testing; at 1% drift Wall SWS-R recovered 40.2%, whereas Wall RSWS-R recuperated 51.6%. At 2% drift, Wall SWS-R recovered 29%, while Wall RSWS-R was able to retrieve 37.6%. The difference of approximately 10% in recovery capacities was maintained up to failure, as presented in Figure 6.20.

The SMA walls exhibited similar behavior during the first two target displacements (Figure 6.21). Thereafter, a clear difference was observed. At 0.2% drift, Wall SWN recovered 69%, while Wall RSWN was able to retrieve 90% of the imposed drift level. The largest difference was recorded at 1% drift where Wall RSWN registered a recovery capacity of 80%, while the original SMA wall recovered 47%. The slightly reduced recovery capacity of Wall RSWN relative to Wall SWN at 3% and 3.5% drift was attributed to the localized damage and yielding of the steel reinforcement in the web region above the height of the starter bars that diminished the recovery capacity. The increase of recovery for Wall RSWN at 4% drift was the result of fracturing of the longitudinal web bar, reducing the resistance to recovery.

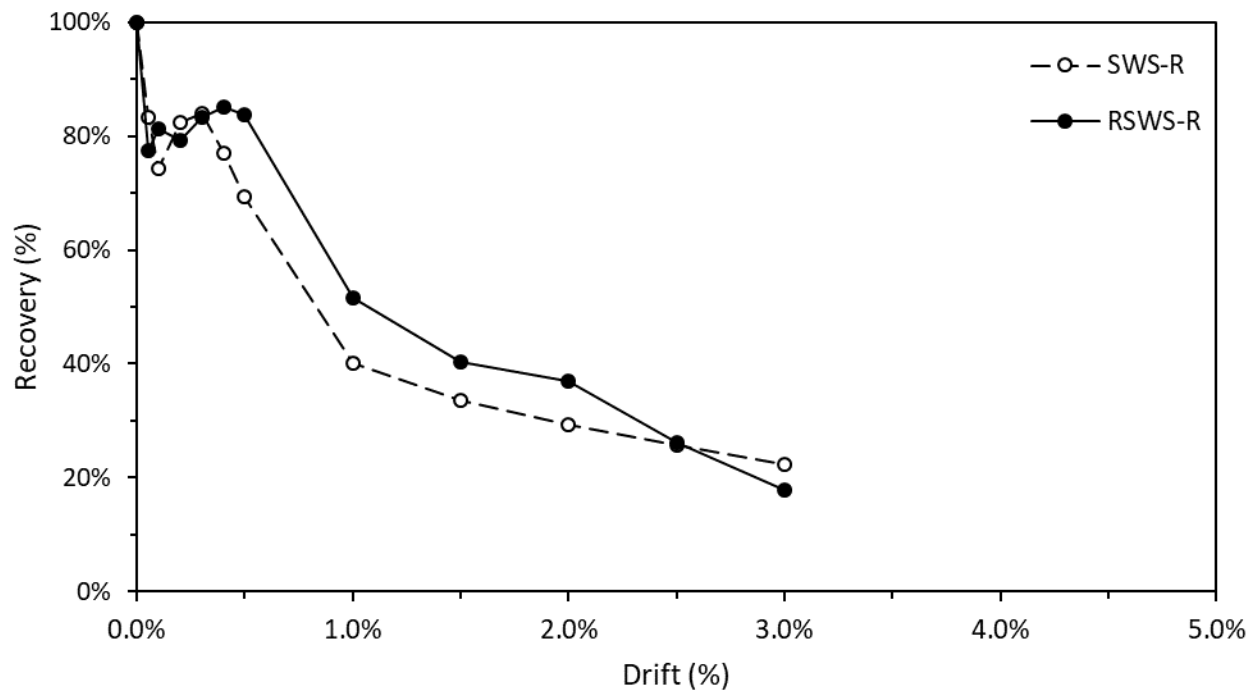


Figure 6.20 Displacement recovery capacity - drift responses for Walls SWS-R and RSWS-R

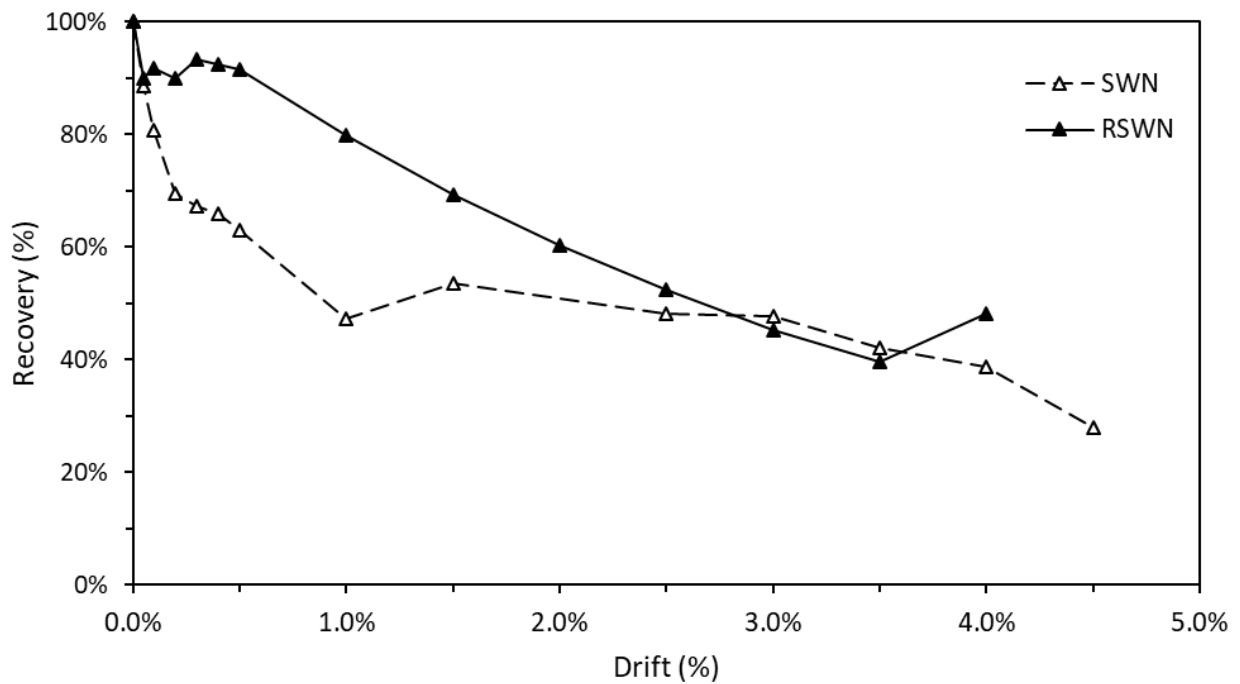


Figure 6.21 Displacement recovery capacity - drift responses for Walls SWN and RSWN

6.2.5 Energy Dissipation

The dissipated energy developed at 1.5% drift level (average peak lateral displacement for the steel walls) was computed for the walls in the pre- and post-repaired state from the hysteretic response shown in Figure 6.22. The energy was calculated from the area encompassed by the loop for the first repetition of loading.

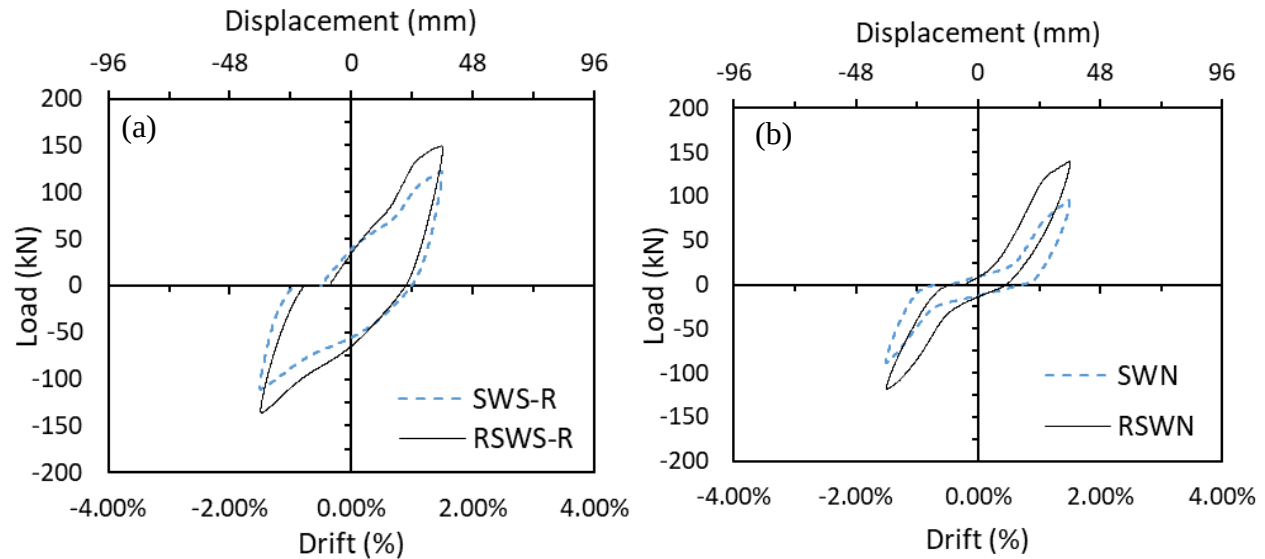


Figure 6.22 Hysteretic response at 1.5% drift: (a) steel walls; and (b) SMA walls

Wall RSWS-R was able to dissipate 6240 Nm, which represented 8% more energy than the original steel wall (5765 Nm) at the peak displacement. Similarly, Wall RSWN dissipated 2700 Nm, a 22% increase over Wall SWN (2130 Nm). In both cases, the ECC used in the repaired walls contributed to a strength enhancement in the walls, leading to higher dissipated energy throughout testing.

To further study the dissipated energy, Figure 6.23 depicts the energy dissipated at each drift for the original and repaired walls.

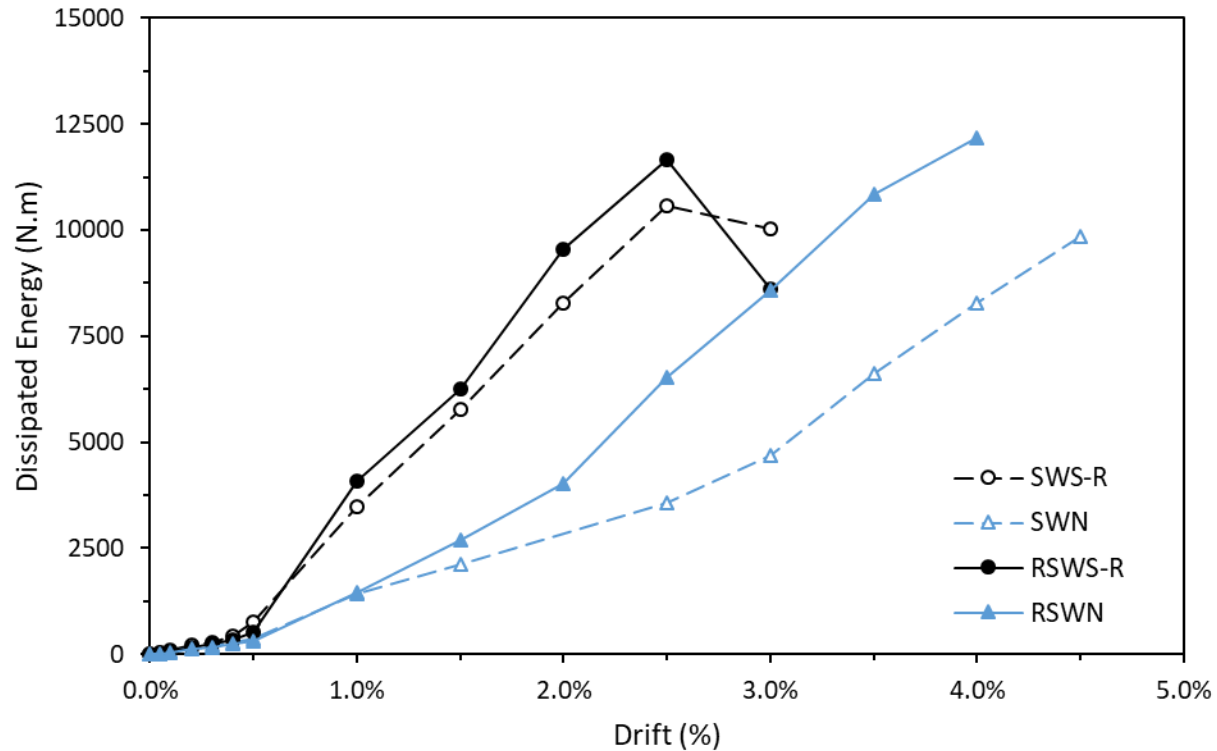


Figure 6.23 Energy dissipated at different drift levels for steel and SMA walls

Up to 0.5% drift, the energy dissipation for all the walls was comparable. Thereafter, the repaired walls dissipated more energy than the original walls for the most part. At 2.5% drift, Wall RSWS-R dissipated 11645 Nm, whereas Wall SWS-R dissipated 10585 Nm. Moreover, the repaired SMA wall was able to dissipate 6515 Nm, while Wall SWN dissipated approximately 45% less energy (3553 Nm).

6.2.6 Failure Modes

Figure 6.24 (a) and (b) depict the crack characteristics of Wall SWS-R and Wall RSWS-R at the end of testing. The crack pattern of Wall SWN is presented in Figure 6.25 (a) and in Figure 6.25 (b) for Wall RSWN. The influence of the PVA-fibers in the ECC mix is evident when comparing the state of the walls at failure. The combination of the high tensile strength and strain capacities of ECC with the transverse stirrups prevented shear failure, and controlled both shear and flexural cracking in Walls RSWS-R and RSWN; whereas the original Walls SWS-R and SWN experienced widespread cracking. The application of the ECC as a repairing material in the plastic hinge zone shifted the

widespread cracking to a more localized damage zone. Nevertheless, the repaired walls experienced a predominant crack that extended across the length of the wall. Delamination of the concrete is evident in Walls SWN and SWS-R, as shown in Figure 6.24 (a) and Figure 6.25 (a), respectively. The addition of the PVA fibers eliminated the fanning crack pattern, and concrete spalling and crushing at the base of the wall experienced by the original walls. The bond between the synthetic fibers and the surrounding concrete reduced additional cracks from propagating and widening that enhanced the damage tolerance in the repaired walls.

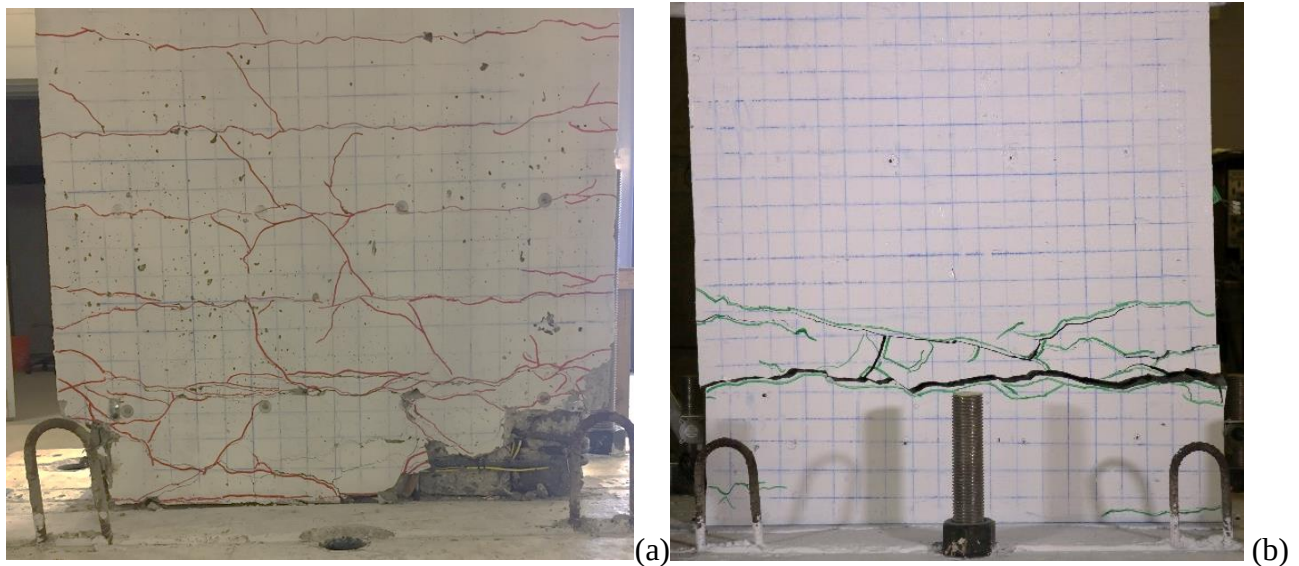


Figure 6.24 Crack pattern at failure: (a) Wall SWS-R; and (b) Wall RSWS-R

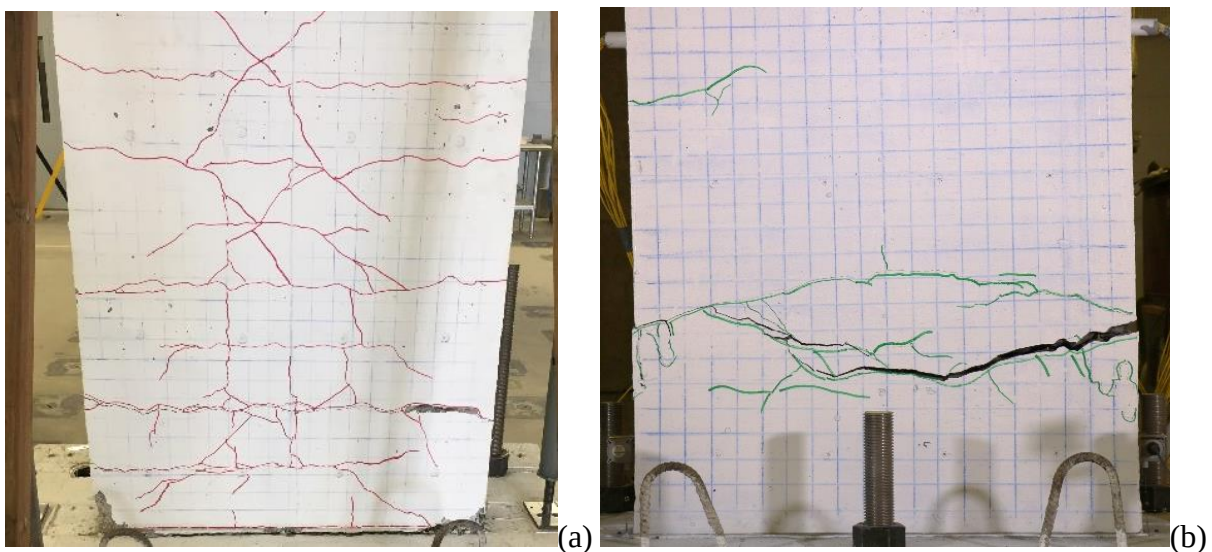


Figure 6.25 Crack pattern at failure: (a) Wall SWN; and (b) Wall RSWN

Replacing the conventional concrete with the ECC in the plastic hinge resulted in a delay of the damage experienced by the repaired walls at larger drifts. Moderate damage was detected up to 1.5% drift for both repaired walls compared to 0.5% drift for Wall SWS-R and 0.3% drift for Wall SWN (Morcos, 2019). This can be associated with the effectiveness of the fibers in bridging through the crack at large displacements. The damage retention and crack recovery sustained up to drifts of 2% for Wall RSWS-R and 2.5% for Wall RSWN. Thereafter, the fibers began pulling out from the localized major crack. The PVA pulled out once the crack exceeded the anchorage length of the fibers, approximately 6 mm. The major crack that formed in Wall RSWN at a height of 400 mm from the base illustrated the benefits of the fibers, which prevented the crack from widening up drifts of 2.5%. Figure 6.26 (a) and (b) show a close-up view of the PVA fibers pulling out at the localized cracks located in the boundaries and web region of the walls, respectively. Note that similar behaviors were observed in both repaired walls.

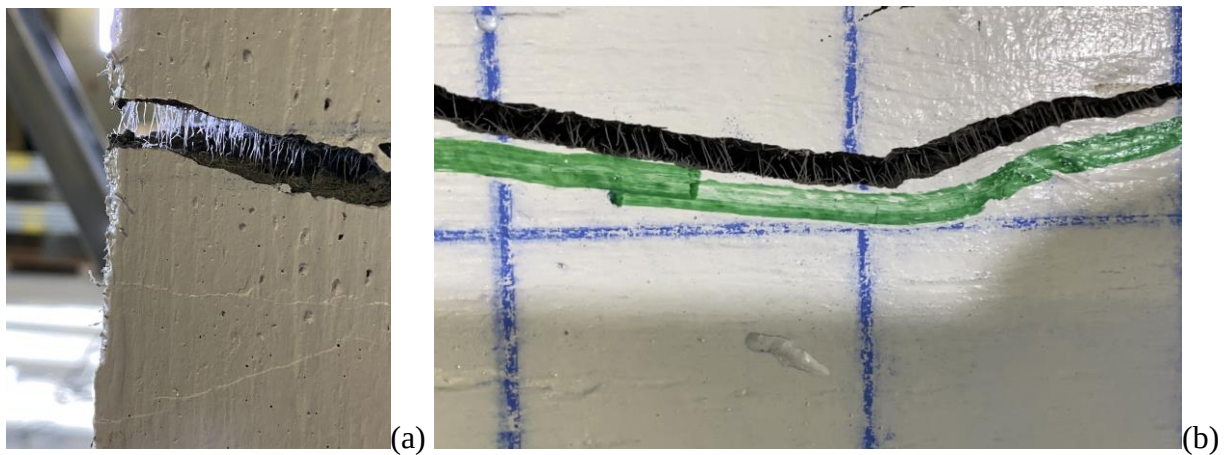


Figure 6.26 Pulling out of PVA fibers at the localized cracks: (a) boundary zone; and (b) web region

CHAPTER 7. NUMERICAL STUDY

Numerical models of the repaired walls were developed in VecTor2 to model the behavior of reinforced concrete structures with non-traditional materials. The finite element (FE) models presented in this chapter are based on the models presented in the preliminary numerical analysis presented in Chapter 3.

7.1 Updated FE models

The preliminary numerical models did not account for major modifications encountered throughout the experimental repairing procedure. As discussed in Section 4.3, during the experimental program, it became necessary to remove additional concrete in the foundation to allow for the proper installation of the mechanical couplers and to provide an improved interlock between the repair concrete and existing concrete. The latter was intended to control sliding along the base. In addition, the location of the base couplers and the length of the new segments of 10 M bars placed in the repaired walls were altered. These modifications were taken into consideration in developing the FE models presented in Figure 7.1 (a) and (b) for Walls RSWS-R and RSWN, respectively.

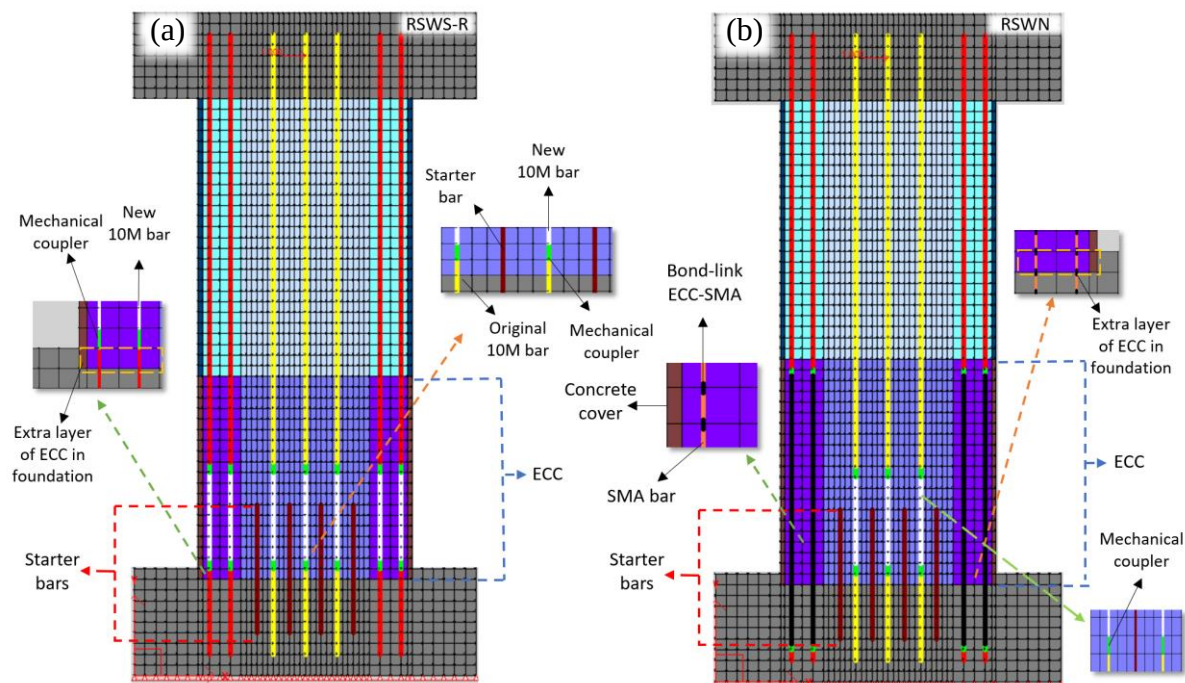


Figure 7.1 Updated FE models: (a) RSWS-R; and (b) RSWN

7.2 Materials Properties and Constitutive Models

The concrete and reinforcement properties obtained from the experimental testing and used as the primary repairing materials presented in Chapter 5, were used as the input for the material properties required in the final FE models. Furthermore, the selection of constitutive models was identical to the original selection, as discussed in Section 3.3.4. Special consideration was given to the development of the tensile stress-strain response for the ECC mixture used in VecTor2. Following the same approach described in Section 3.3.4, preliminary curve fitting of prism tests was performed on the FE models of Prim P1 and Prism P2 (Figure 3.16). The FE models were subjected to iterative four-point bending until a satisfactory response was obtained to establish the stress-strain data required for the custom-input tensile response of the ECC used in the experimental walls and presented in Figure 7.2.

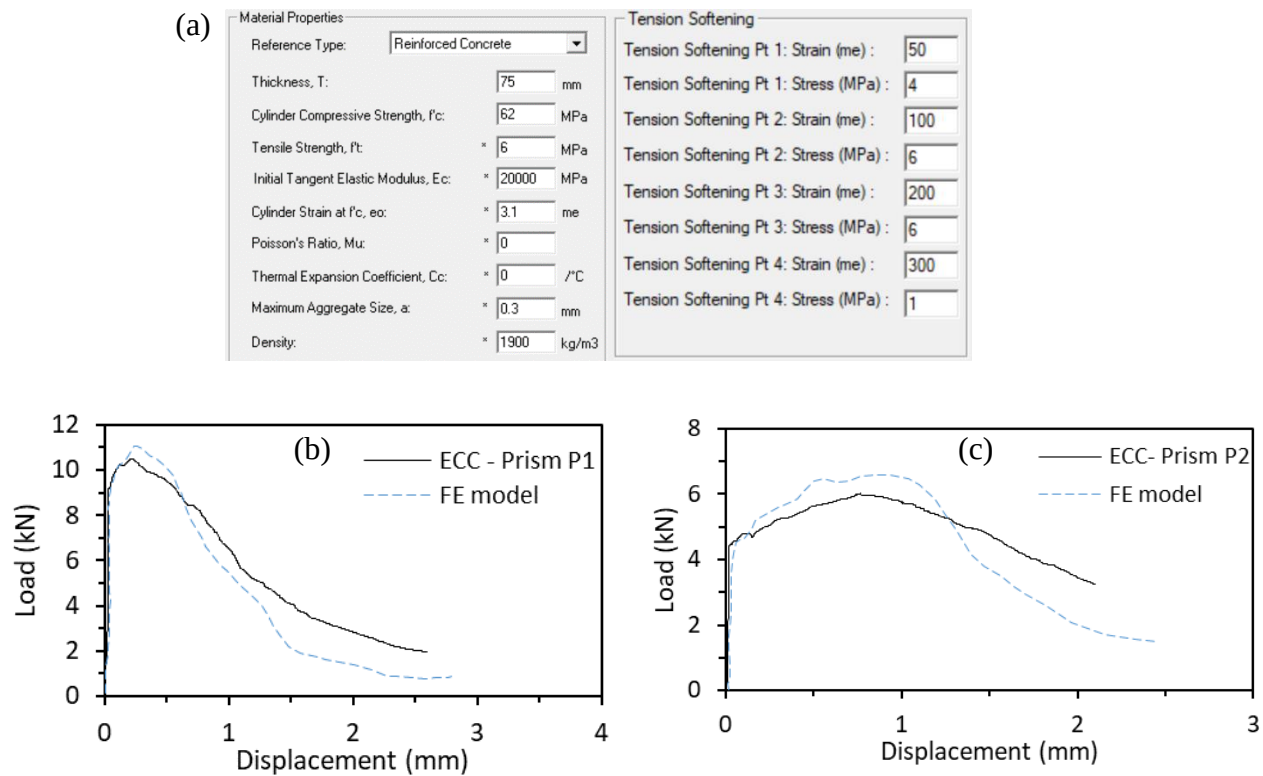


Figure 7.2 ECC constitutive model: (a) updated material properties used in VecTor2; b) response curves of Prism P1; and response curves of Prism P2

7.3 Load-Displacement Response

The updated FE models for Walls RSWS-R and RSWN were subjected to the same loading protocol used during testing. The experimental and numerical lateral load-displacement responses are provided in Figure 7.3. and Figure 7.4, for Wall SWS-R and RSWN, respectively.

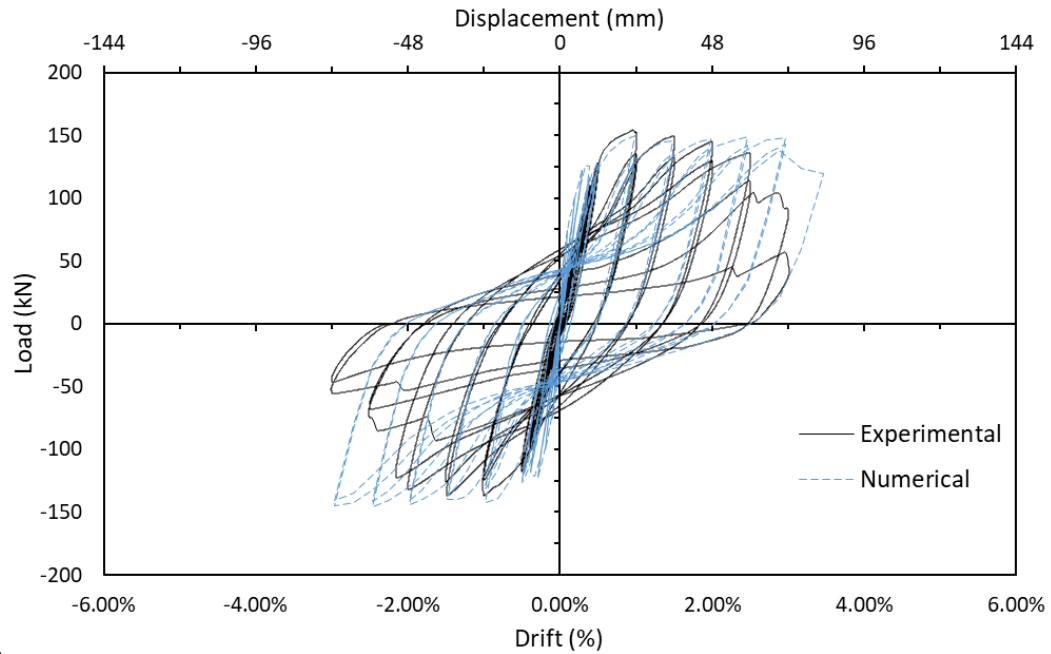


Figure 7.3 Load-displacement response for Wall RSWS-R

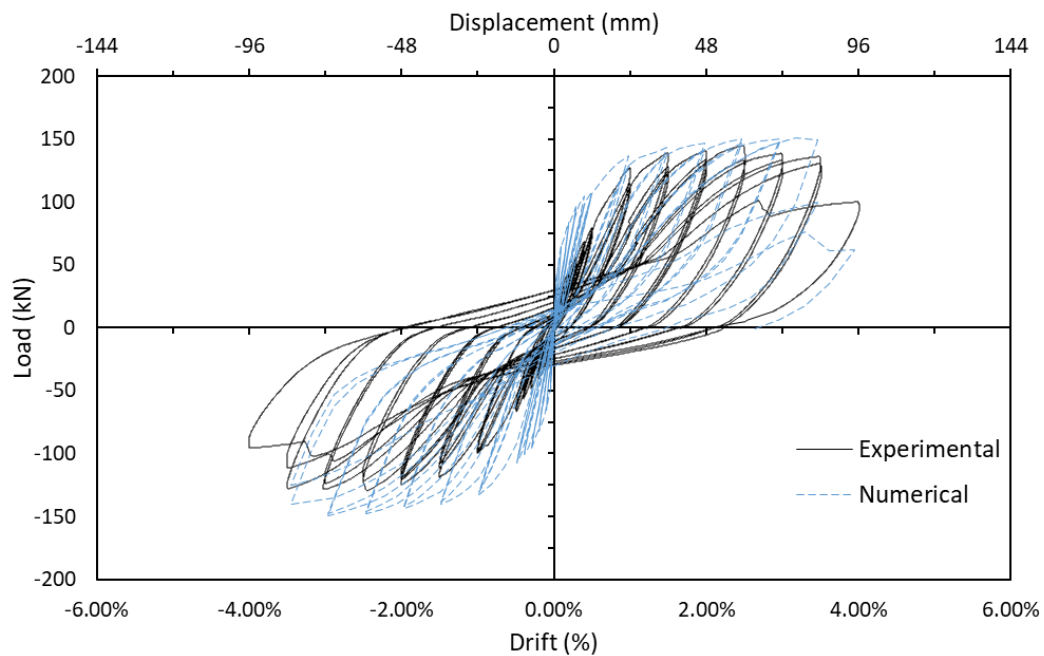


Figure 7.4 Load-displacement response for Wall RSWN

The overall hysteretic behavior of the numerical responses were in close agreement with what was captured during the experimental testing, specifically in lateral load resistance, residual displacements, performance points, and failure modes. However, a discrepancy was also noted in the numerical responses; the underestimation of the cyclic degradation effect during the second and third repetitions of a cycle for the same target displacement. For Wall RSWS-R, the numerical response experienced a milder decay in lateral strength after the peak in comparison to the experimental response, which resulted in a larger ultimate displacement and higher dissipated energy. The difference was attributed to the delay in the fracture of the longitudinal steel reinforcement in the boundary region in the FE element model. Furthermore, for Wall RSWN, the numerical response developed a more accentuated pinching effect than the experimental response, which demonstrated higher recovery capacity from plastic deformations. This was attributed to a delayed development of yielding in the steel reinforcing bars in the web region and localized cracking, which allowed the SMA bars to maintain the self-centering property into higher drift levels.

The lateral load-drift envelop responses for the first repetition of loading in the positive direction based on the numerical models, and experimental testing of Walls RSWS-R and RSWN are presented in Figure 7.5 and Figure 7.6, respectively. An overestimation of initial stiffness for both numerical models was evident. The delay in simulating first cracking in the FE models resulted in a significant difference prior to yielding. Furthermore, at yielding, the numerical lateral load was approximately 2.5% greater for Wall RSWS-R and 7% smaller for Wall RSWN. At the peak point, the numerical and experimental responses were in excellent agreement. The peak displacements from the numerical models differed by 0.5% in comparison to the experimental results. In addition, the simulated peak load for Wall RSWS-R was the same as the experimental value, while for Wall RSWN the model resulted in a 15% greater lateral strength. The ultimate point based on the models was recorded with 20% greater displacement and 7% larger lateral load capacity for Wall RSWS-R. The FE model of Wall RSWN, was able to replicate the ultimate displacement with a 2% difference, and an ultimate load 10% greater than the experimental response. Moreover, the numerical responses were able to capture the rupturing of the steel reinforcement in the boundary region for Wall RSWS-R and in the web zone for wall RSWN, with no damage to the SMA bars. Table 7.1 lists the performance points for the experimental results and the FE models.

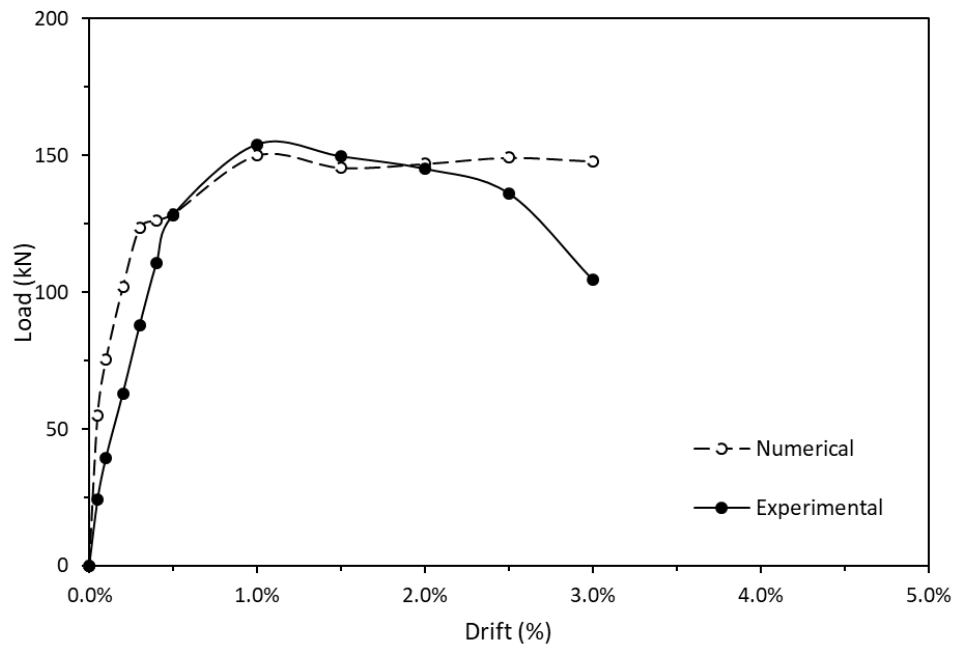


Figure 7.5 Lateral load-drift envelope responses for Wall RSWS-R

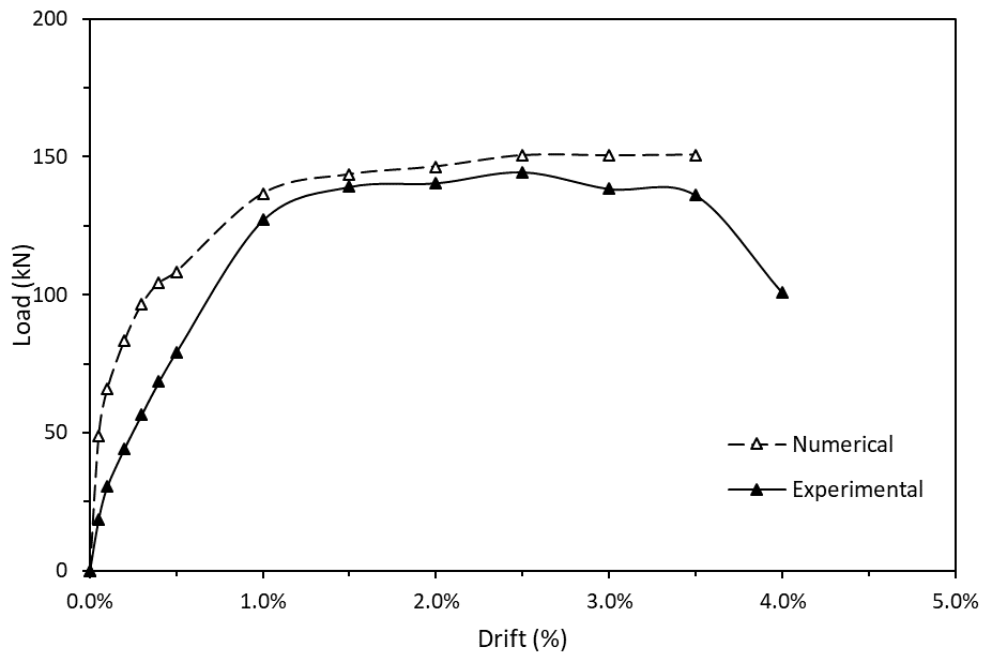


Figure 7.6 Lateral load-drift envelope responses for Wall RSWN

Table 7.1 Performance points of numerical (FE models) and experimental responses of Walls RSWS-R and RSWN

Wall	Yield			Peak			Ultimate		
	δ mm	Drift (%)	Load kN	δ mm	Drift (%)	Load kN	δ mm	Drift (%)	Load kN
RSWS-R	13.56	0.57%	127	29.32	1.22%	145	56.19	2.34%	128
FE RSWS-R	7.89	0.33%	124	29.64	1.23%	145	71.16	2.97%	138
RSWN	40.93	1.71%	123	65.83	2.74%	137	83.81	3.49%	124
FE-RSWN	21.06	0.88%	114	65.22	2.72%	150	82.00	3.42%	137

7.4 Energy Dissipation

The energy dissipation–drift response was calculated from the positive direction of the first repetition of each target displacement for the finite element (FE) models and experimental responses of Walls RSWS-R and RSWN and are presented in Figure 7.7.

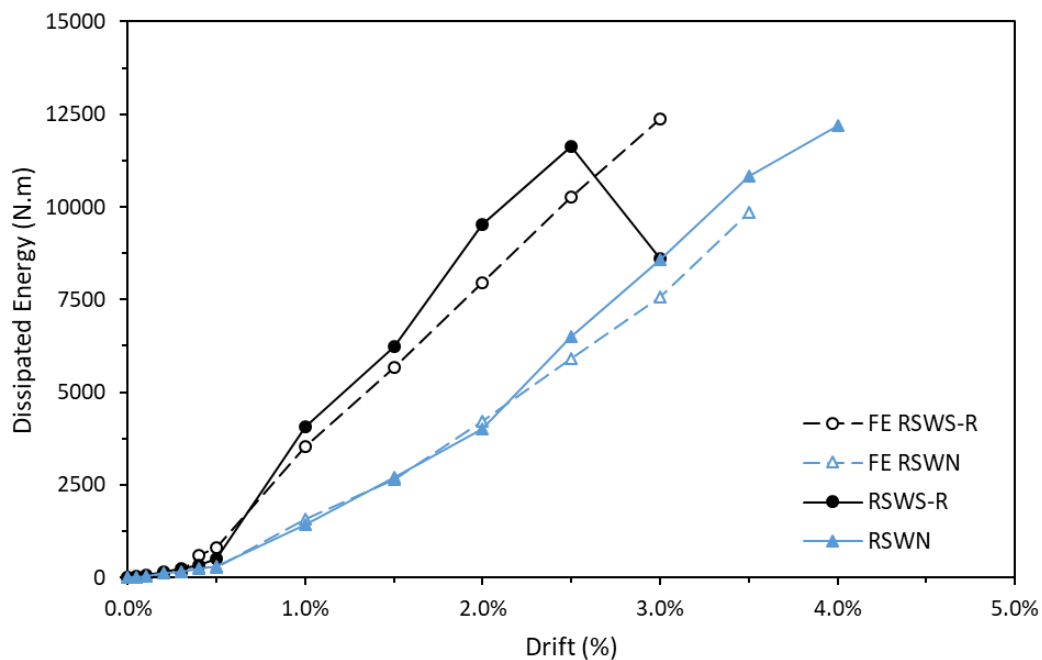


Figure 7.7 Energy dissipation-drift response for FE model and experimental response of Walls RSWS-R and RSWN

The finite element model provided comparable dissipated energy throughout the drift levels. This is attributed to the robust constitutive models employed for the different materials in VecTor2. At the peak lateral load capacity, the predicted dissipated energy for Wall RSWS-R was 4005 Nm, 11% less of what was experienced by the wall during testing. For Wall RSWN, the dissipated energy captured by the model was 9% lower than the actual response, with a value of 6640 Nm. Note that during the

3% drift level, there is a significant difference between the numerical and experimental responses of Wall RSWS-R; this was attributed to a discrepancy of the drift to initiate rupture of the longitudinal reinforcement in the boundary zone. The experimental wall suffered ruptured of steel reinforcement at 2.5% drift, which provoked a sudden decay of energy dissipated at the subsequent drift level. The same scenario was encountered for the numerical response at a larger drift (3%). Similarly, the modelled response of Wall RSWN experienced a comparable behavior at the last drift level.

7.5 Recovery Capacity

The recovery capacity from the numerical models was calculated from the positive direction for the first repetition of each drift level to allow comparison with the experimental results. Figure 7.7 and Figure 7.8 depicts the recovery capacity-drift response for Walls RSWS-R and RSWN, respectively.

The modelled response of Wall RSWS-R presented full recovery up to 0.4% drift, thereafter, with the initiation of flexural cracking in the plastic hinge region, the numerical model suffered a gradual decay in its recovery capabilities that were comparable with the experimental results. At the peak lateral capacity, the predicted response calculated a recovery capacity of 50%, while the experimental wall developed a recovery of 47%. Thereafter, the FE model overestimated the recovery capacity by approximately 15% to failure. Furthermore, the well-defined pinching effect simulated in the FE model for Wall RSWN resulted in a significant overestimation of recovery capacity after the 0.5% drift level that remained until failure.

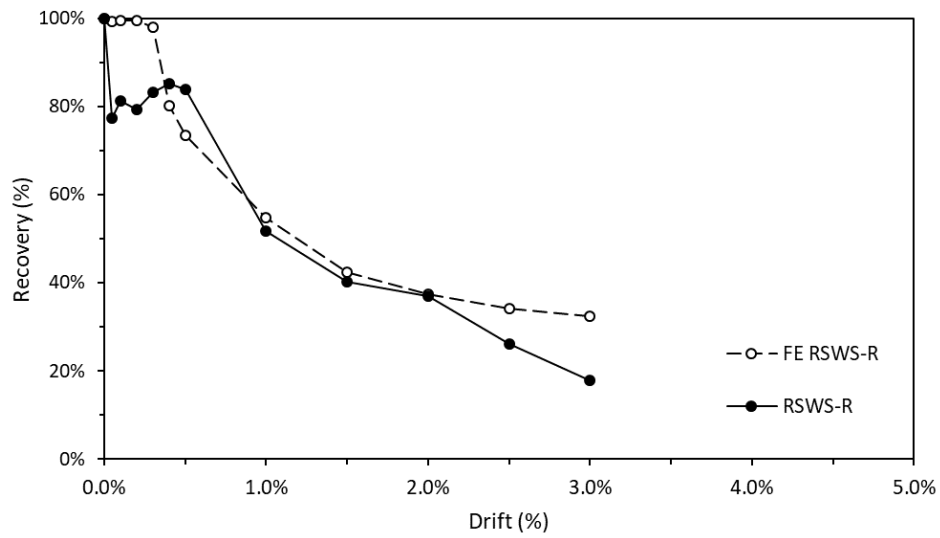


Figure 7.8 Displacement recovery-drift response for FE model and experimental testing of Wall RSWS-R

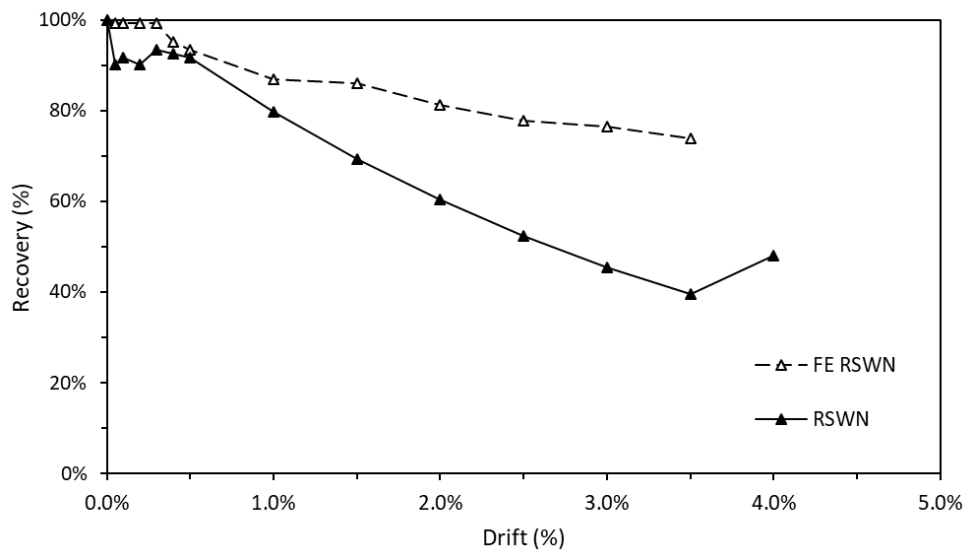


Figure 7.9 Displacement recovery-drift response for FE model and experimental testing of Wall RSWN

7.6 Failure Mode and Cracking Pattern

A comparison is established between the experimental walls and numerical models at the end of testing and analysis, respectively, in Figure 7.10 and Figure 7.11. The predicted crack patterns in both walls were in good agreement with what was observed during the experimental testing. The FE models developed major flexural cracking adjacent to the termination of the starter bars and extended

to a height of 450 mm from the base that controlled their overall behavior. Flexural-shear hairline cracking developed above this region and spread through the walls. For the experimental walls, this pattern was captured above the plastic hinge region, where the existing normal concrete was located. However, due to the cracking recovery characteristic of the ECC used in the plastic hinge region, the hairline flexural cracking was not visually evident. Furthermore, the numerical models were able to correctly simulate the failure mode of the walls. The FE models captured the rupturing of the longitudinal steel reinforcement within the plastic hinge area; in the boundary regions for Wall RSWS-R and in the web for Wall RSWN, with no damage experienced by the truss bars simulating the SMA bars in the boundary zone.

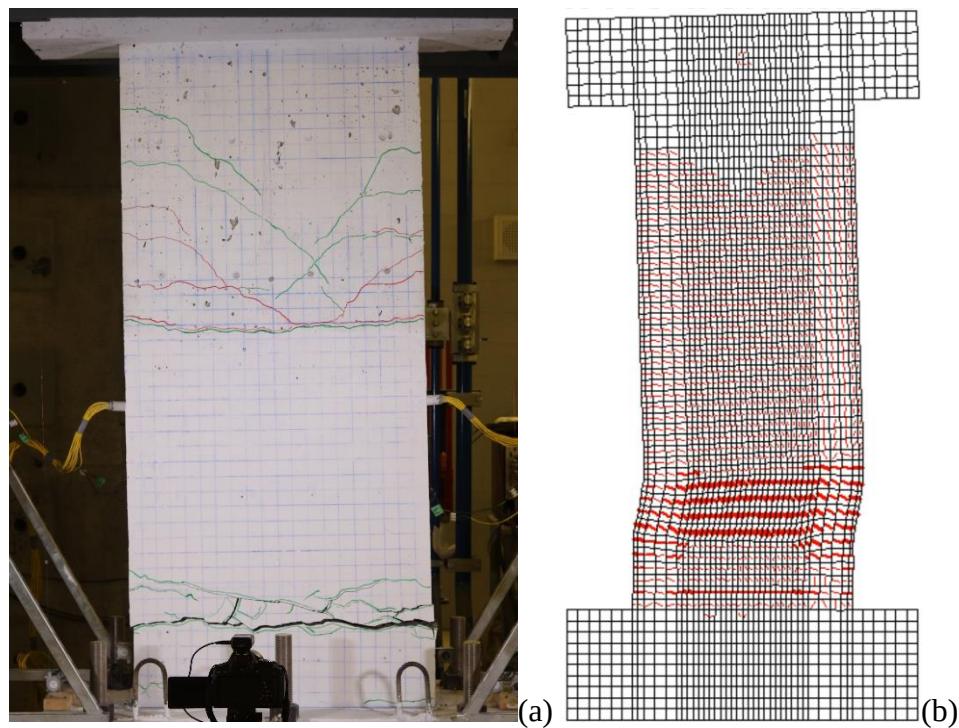


Figure 7.10 Cracking pattern at the end of the testing for Wall RSWS-R: (a) Experimental; and (b) Numerical

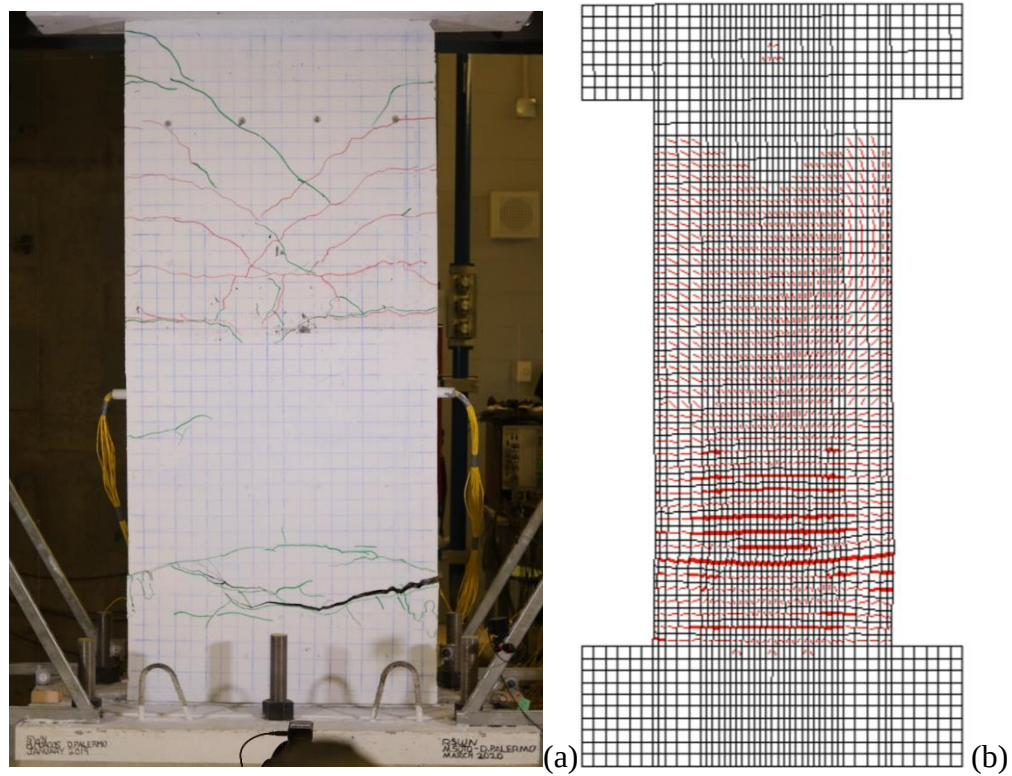


Figure 7.11 Cracking pattern at the end of the testing for Wall RSWN: (a) Experimental; and (b) Numerical

CHAPTER 8. CONCLUSIONS AND RECOMMENDATIONS

8.1 Conclusions

Previously-tested, Superelastic-Shape Memory Alloy (SE-SMA) and typical-steel reinforced concrete shear walls were repaired and tested under the same simulated seismic loading. The repairing method involved the removal of the heavily damaged concrete in the plastic hinge zone, replacement of ruptured and buckled steel reinforcement, addition of starter bars and casting of Engineered Cementitious Composite (ECC) as the primary repairing material where the previous concrete was removed. Numerical modelling was implemented to verify the viability of the repairing procedure and to provide a modelling approach to simulate the response of reinforced concrete walls with emerging, high-performance materials. The self-centering phenomenon of SE-SMA and the distinctive ductility properties of ECC were capable of restoring and further enhancing the capacity of the walls and reducing damage relative to the original walls. The following sections provide conclusions drawn from this study.

8.1.1 Preliminary Numerical Analysis

Preliminary analyses were performed through the nonlinear finite element method to define the extent of the repairing procedure and to assess the merits of using either UHPFRC (Ultra-High Performance Fibre Reinforced Concrete) and ECC (Engineered Cementitious Composites) as the main repairing material in the experimental program.

- The numerical models developed for the original walls were capable of simulating the response and damage experienced during testing of the walls prior to repairing as reported by Morcos and Palermo (2019) in terms of performance points, cracking patterns, and failure mode.
- The current research verified the previously proposed methodology to represent HPFRC (High-Performance Fibre Reinforced Concrete) in the software VecTor2 (Saikali, 2019), employing a custom-input, multi-linear response that defined the tensile stress-strain behaviour of the HPFRC.

- Through the preliminary numerical analyses, it was determined that the ECC provided the most viable option as the primary repairing material for the damaged walls. The numerically modelled repaired ECC walls developed a more ductile behaviour without an excessive over-strength in comparison to the wall models with UHPFRC.

8.1.2 ECC Mixture

A series of trials were batched and cast based on a previously-developed ECC mix by Eshghi (2018) to understand the behaviour and enhance the mechanical properties.

- The reference mix was modified to improve the strain hardening behavior with the development of micro-cracking by implementing two different types of PVA fibers (Kuralon RECS100x12 and Kuralon REC15x12) and an alternative superplasticizer (MasterGlenium® 3400).
- The In-house mix developed in this study exhibited an average cylinder compressive strength of 63 MPa on the day of testing (150 days). The presence of the PVA fibers bridged through the cracks eliminating any brittle failure and maintaining the integrity of the cylinders.
- The flexural tensile strength was approximately 6 MPa. The response curves of prism tests indicated the development of several cracks during loading, marked by small local disturbances occurring upon the opening of cracks. This showcased a ductile behavior with a plateau phase once the cracks initiated and prior to reaching the peak load. The descending branch of the flexural prisms followed a gradual softening behavior.

8.1.3 Structural Performance of Repaired Walls

The repair strategy involved the removal of heavily damaged concrete in the plastic hinge zone, replacement of ruptured and buckled steel reinforcement, the addition of starter bars at the base of the wall, and casting of Engineering Cementitious Composites (ECC) to replace the removed concrete.

- The first crack appeared at a drift of 0.2% and 0.1% for Walls RSWS-R (steel wall) and RSWN (SMA wall), respectively. Both walls developed one major crack at 1% drift that surfaced between the ends of the starter bars and the upper mechanical couplers. This crack controlled the overall behavior and failure of the walls. The addition of the starter bars strengthened and stiffened the lower portion of the walls and shifted the failure location.
- Previously-developed and new shear cracks appeared above the plastic hinge region and were attributed to the low tensile properties of the existing concrete used in this region.
- The effect of cyclic degradation on the lateral strength was evident after 0.5% drift for the repaired walls. The development of the first major crack in the walls marked a lateral load degradation of approximately 14% for the second repetition of loading that continued until failure.
- The average global yield point was reached at a load of 127.34 kN with a yield displacement of 13.56 mm (0.57% drift) for Wall RSWS-R. Wall RSWN experienced global yielding at 122.91 kN with a displacement of 40.93 mm (1.71% drift).
- The peak lateral strength in Wall RSWS-R arose at an average load of 145.49 kN with an average displacement of 29.32 mm (1.22% drift). Wall RSWN experienced an average peak load of 136.98 kN with an average displacement of 83.81 mm (2.74% drift). Note that the average was based on the first the positive and negative directions of loading.
- In the post-peak behavior, Wall RSWS-R experienced a gradual degradation of lateral load capacity leading to an average ultimate load of 128.18 kN at an average displacement of 56.19 mm (2.34% drift). Wall RSWN sustained an average ultimate load of 136.98 kN with an average ultimate displacement of 83.81 mm (3.49%).
- Testing was terminated for Wall RSWS-R after the 3% drift cycle due to rupturing of three longitudinal steel bars in the right boundary region and two steel bars in the left boundary region. For Wall RSWN, testing was terminated at 4% drift following the rupturing of three longitudinal

steel reinforcing bars in the web region. At the end of testing none of the SE-SMA rebars located in the boundary regions of Wall RSWN experienced damage.

- The hysteretic response of Wall RSWS-R exhibited wide loops that translated into significant energy dissipation in comparison to the SMA wall. On the contrary, the hysteretic loops in Wall RSWN developed more pinching, attributed to the self-centering capacity of the SE-SMA bars that generated high displacement recovery and reduced energy dissipation.
- The strain gauges captured yielding in the vertical reinforcement throughout the entire height of the plastic hinge in the boundary region for both walls.
- For Wall RSWS-R, the web longitudinal steel reinforcement located at the critical section yielded at a load of 130 kN. This load was attained near the end of the 1% drift cycle. The strains experienced by the longitudinal web reinforcement Wall RSWN reached yielding at a load of 68 kN, preceding yielding of the SMA bars located in the boundary region. The behavior matches the global performance of Wall RSWN, where the steel web reinforcement ruptured along the main flexural crack, with no apparent damage in the SMA bars.
- Wall RSWN developed a lower initial stiffness, and therefore, higher yield displacement. The post-yield secant stiffness of Wall RSWN was marked by a 36.5% reduction in comparison to Wall RSWS-R due to the yielding of the steel reinforcement within the web region prior to yielding of the SMA bars in the web boundary zone. Both walls developed a ductile behavior with a gradual decrease of stiffness prior to failure.
- The small difference between the vertical displacements at the top beam and at the top of the plastic hinge in both walls demonstrated a disproportional distribution along the height of the walls due to the localized cracking that surfaced inside the plastic hinge region. During higher lateral displacement ductility levels, the walls experienced larger vertical displacements indicating inelastic rotation was concentrated above the base of the walls and along the single predominant cracking that appeared just above the starter bars and the lower end of the upper mechanical couplers.

- Wall RSWN experienced higher rotations throughout testing. Once the yield point was reached, the rotation of both walls increased significantly, with a higher degree for Wall RSWN, which was accredited to the lower modulus of elasticity of the SMA bars relative to the steel bars used in Wall RSWS-R.
- The high shear strains experienced inside the plastic hinge in both walls without any visible shear damage was attributed to the increased shear resistance of ECC and its ability to sustain high shear deformations.
- The substantial difference in displacement recovery between Walls RSWN and RSWS-R is attributed to the compatible deformation of the ECC with the SMA bars and the reverse transformation of the SMA reinforcement upon unloading.
- The reduced recovery capacity of Wall RSWN during the larger applied displacements was attributed to the localized damage and yielding of the steel reinforcement in the web region that developed above the height of the starter bars that provided resistance to the recovery.

8.1.4 Comparison Between Pre- and Post-Repairing Behavior of Walls

The feasibility of the repairing procedure performed on the repaired walls and the influence of the Engineered Cementitious Composites (ECC) on the overall response as a primary repairing material was evaluated. The main observed comparable and differing parameters between the original and the repaired walls are presented below:

- For Wall SWS-R, the concrete located in the plastic hinge region was severely deteriorated with the development of significant flexural and shear cracks. Similarly, Wall SWN developed reduced flexural-shear cracks, yet one major crack was observed at 300 mm above the base of the wall. The application of the ECC as a repairing material altered the widespread cracking to a more localized damage limited within one-third of the plastic hinge region.

- The starter bars in the repaired walls eliminated the damage encountered at the base that was observed in the original walls. It became evident that along the height of the starter bars (300 mm), no significant cracking surfaced in the repaired walls, which was attributed to a high concentration of steel in this region that created a rigid section, shifting the critical section away from the base.
- The reduced vertical displacements, and subsequent rigid body rotation, recorded at the base of the walls validated the implementation of the starter bars. In addition, the starter bars prevented sliding that was present in the original walls along the base.
- Walls SWS-R and SWN experienced widespread damage, with hairline to narrow flexural cracks above the plastic hinge region. Some of these cracks reopened during testing of the repaired walls. Contrary to the original walls, no shear damage was detected in Walls RSWS-R and RSWN within the plastic hinge zone.
- The rupture of the longitudinal steel reinforcement in the web region of Wall RSWN and in the boundary zones of Wall RSWS-R shifted to a height of 300 mm from the base of the walls (termination point of the starter bars); whereas, this similar damage in the original walls arose along the base.
- The repaired walls experienced yielding at a similar drift level as the original walls, but with a significant increase of 25% in lateral strength.
- Wall RSWS-R developed its peak lateral strength at a similar drift and a load of 145 kN, which represented a 19% increase relative to Wall SWS-R. Wall RSWN developed a 14% increase in lateral strength at the peak with a lower displacement of 65.8 mm (2.7% drift) in comparison to the displacement of 91 mm (3.8% drift) for Wall SWN.
- The repaired walls experienced an ultimate point characterized by an increment of 12% and 9% in lateral load resistance for Wall RSWS-R and Wall RSWN with respect to Wall SWS-R and

Wall SWN. Wall SWN sustained an ultimate drift capacity of 4.3% as to 3.5% for Wall RSWN. Wall SWS-R failed at 2.3% drift as to 2.5% for RSWS-R.

- The walls in the pre- and post-repaired state exhibited a gradual decay of the secant stiffness with increased drift up to failure, resulting in a stagnant low stiffness. The original Walls SWS-R and SWN exhibited higher initial stiffness of 32.4 kN/mm and 20.32 kN/mm, respectively, while Walls RSWS-R and RSWN developed stiffness of 20.1 kN/mm and 15.4 kN/mm, respectively. The difference is attributed to the damage in the concrete above the plastic hinge region that was not repaired.
- Once cracking initiated, the secant stiffness of the original walls promptly dropped below the values of the repaired walls. The bridging effect of the PVA fibers in the ECC controlled crack widening and promoted crack recovery, which delayed the degradation of stiffness and load capacity of the repaired walls.
- The ECC used in the repaired walls developed higher resistance to shear and compression with compatible deformation to the reinforcement, leading to higher load capacities at the same target displacements as the original walls, which was translated to higher dissipated energy. Wall RSWS-R was able to dissipate 6240 Nm, which represented 8% more energy than the original steel wall (5765 Nm) at peak displacement. Similarly, Wall RSWN dissipated 2700 Nm, demonstrating a 22% increase over Wall SWN (2130 Nm).
- Replacing the conventional concrete with ECC resulted in postponing the damage experienced by the walls at large drifts. Moderate damage was detected up to 1.5% drift for both repaired walls compared to 0.5% drift for Wall SWS-R and 0.3% drift for Wall SWN (Morcos, 2019). This can be associated with the effectiveness of the fibers in bridging through the cracks at large displacements.

8.1.5 Numerical Study

The numerical models of the repaired walls, developed through the preliminary analyses, were updated with the mechanical properties obtained from the experimental results. The models were then subjected to the same loading protocol as the experimental testing. The following conclusions result from the numerical analyses.

- The overall hysteretic behavior of the numerical responses were in close agreement with that obtained during testing, specifically the lateral load resistance, residual displacements, performance points, and failure modes.
- Discrepancies were noted in the numerical responses in the underestimation of the cyclic degradation effect during the second and third repetitions of a cycle for the same target displacement.
- The numerical response for Wall RSWS-R experienced a milder decay in strength after the peak compared to the experimental response, which translated to a larger ultimate displacement and higher energy dissipation. The difference was attributed to the delay in the fracture of the longitudinal steel reinforcement in the boundary region for the FE model.
- The numerical response of Wall RSWN developed greater pinching than the experimental response, which demonstrated higher capacities of recovery from plastic deformations. This was attributed to a late development of yielding in the longitudinal steel reinforcing bars in the web region and localized cracking, which allowed the SMA bars to enable and maintain the self-centering capacity into higher drift levels.
- The FE models obtained comparable energy dissipation throughout the drift levels imposed on the walls. This was attributed to the array of robust constitutive models employed for the different materials, which resulted in good agreement in the overall load-displacement responses.

- The simulated crack patterns in both walls were in correct agreement with what was observed during testing. The FE models were able to simulate the major flexural cracking adjacent to the termination of the starter bars and extended to a height of 450 mm from the base, which controlled the overall behavior.
- The numerical models were able to simulate the failure mode of the walls correctly. The FE models captured the rupturing of the longitudinal steel reinforcement within the plastic hinge area, in the boundary regions for Wall RSWS-R and in the web for Wall RSWN, with no damage experienced by the truss bars simulating the SMA bars in the boundary zone.
- The implementation of a constitutive model in VecTor2 capable of simulating an appropriate tensile behavior for the Engineered Cementitious Composites (ECC) would improve the prediction capabilities of the FE models.

8.2 Recommendations

Based on the work completed during this research and the lessons learned, the following aspects are to be considered for future research in this area:

- Additional mechanical testing of the ECC mix is required for detailed characterization of the concrete to be used as input in numerical modelling.
- Further numerical analyses to determine the effect of cyclic degradation in the FE models
- A systematic assessment of the starter bars and their height to avoid the development of a critical section at the termination point of the bars.
- Assessment of behavior of SMA reinforced walls under reverse cyclic excursions with axial loading.
- Study of the bond-slip behavior between ECC with smooth-SMA reinforcing bars.

- Nonlinear dynamic analysis of walls with ECC and reinforced with SMAs to understand the response relative to walls with normal concrete and steel reinforcement under more realistic seismic loading.

REFERENCES

- 441 ACI Committee (1996) '441R-96: Report on High-Strength Concrete Columns', *Technical Documents*.
- 544, A. C. (1998) 'Guide for Specifying, Proportioning, Mixing, Placing, and Finishing Steel Fiber Reinforced Concrete, ACI 544.3R-93', in *Manual of Concrete Practice*.
- Abbass, W. and Iqbal Khan, M. (2018) 'Mechanical properties of Hybrid steel/PVA fibers reinforced high strength concrete', *MATEC Web of Conferences*, 199, pp. 1–5. doi: 10.1051/mateconf/201819911005.
- Abdulridha, A. *et al.* (2013) 'Behavior and modeling of superelastic shape memory alloy reinforced concrete beams', *Engineering Structures*. doi: 10.1016/j.engstruct.2012.12.041.
- Abdulridha, A. and Palermo, D. (2013) *Performance of Superelastic Shape Memory Alloy Reinforced Concrete Elements Subjected to Monotonic and Cyclic Loading*. University of Ottawa.
- Abdulridha, A. and Palermo, D. (2017) 'Behaviour and modelling of hybrid SMA-steel reinforced concrete slender shear wall', *Engineering Structures*. doi: 10.1016/j.engstruct.2017.04.058.
- ACI Committee 318 (2014) *Building Code Requirements for Structural Concrete*, American Concrete Institute.
- Ahmed, S. F. U., Maalej, M. and Paramasivam, P. (2007) 'Flexural responses of hybrid steel-polyethylene fiber reinforced cement composites containing high volume fly ash', *Construction and Building Materials*. doi: 10.1016/j.conbuildmat.2006.01.002.
- Alam, M. S., Youssef, M. A. and Nehdi, M. (2008) 'Analytical prediction of the seismic behaviour of superelastic shape memory alloy reinforced concrete elements', *Engineering Structures*. doi: 10.1016/j.engstruct.2008.05.025.
- Ali El-Sayed Mahmoud Ali, M. (2017) *Tensile and Impact Behaviour of Shape Memory Alloy Fibre Reinforced Engineered Cementitious Composites*. University of Western Ontario.
- Ali, M. A. E. M., Soliman, A. M. and Nehdi, M. L. (2017) 'Hybrid-fiber reinforced engineered cementitious composite under tensile and impact loading', *Materials and Design*. doi: 10.1016/j.matdes.2016.12.047.
- American Concrete Institute - ACI Committee 318 (2005) *Building Code Requirements for Structural Concrete(ACI 318S-05) and Commentary (ACI 318SR-05)*, *ACI Structural Journal*. doi:

10.1016/S1130-6343(05)73662-X.

Antoniades, K. K., Salonikios, T. N. and Kappos, A. J. (2005) 'Tests on seismically damaged reinforced concrete walls repaired and strengthened using fiber-reinforced polymers', *Journal of Composites for Construction*. doi: 10.1061/(ASCE)1090-0268(2005)9:3(236).

ASTM C1856 (2017) 'Standard Practice for Fabricating and Testing Specimens of Ultra-High Performance Concrete', *ASTM International*. doi: 10.1520/C1856.

ASTM C230 (2014) 'Standard Specification for Flow Table for Use in Tests of Hydraulic Cement 1', *Annual Book of ASTM Standards*. doi: 10.1520/C0230.

ASTM F2516–14 (2014) 'ASTM F2516–14: Standard Test Method for Tension Testing of Nickel-Titanium Superelastic Materials', *ASTM International*. doi: 10.1520/F2516-14.2.

ASTM Standard E8/E8M-13a (2013) '*Standard Test Methods for Tension Testing of Metallic Materials*', *ASTM International*. doi: 10.1520/E0008_E0008M-13A.

ATC-24 (1992) *Guidelines for Cyclic Seismic Testing of Components of Steel Structures*. AISI.

Bentz, E. C. (2005) 'Explaining the riddle of tension stiffening models for shear panel experiments', *Journal of Structural Engineering*. doi: 10.1061/(ASCE)0733-9445(2005)131:9(1422).

Bohl, A. and Adebar, P. (2011) 'Plastic hinge lengths in high-rise concrete shear walls', *ACI Structural Journal*. doi: 10.14359/51664249.

Breitenbucher, R. (1999) 'High performance fibre concrete SIFCON for repairing environmental structures', in *HIGH PERFORMANCE FIBER REINFORCED CEMENT COMPOSITES (HPFRCC3)*.

Buehler, W. J., Gilfrich, J. V. and Wiley, R. C. (1963) 'Effect of Low-Temperature Phase Changes on the Mechanical Properties of Alloys near Composition TiNi', *Journal of Applied Physics*. doi: 10.1063/1.1729603.

Buttignol, T. E. T., Sousa, J. L. A. O. and Bittencourt, T. N. (2017) 'Ultra High-Performance Fiber-Reinforced Concrete (UHPFRC): a review of material properties and design procedures', *Revista IBRACON de Estruturas e Materiais*. doi: 10.1590/s1983-41952017000400011.

Canadian Standards Association (2004) 'CSA A23.3-04 Design of Concrete Structures'. Rexdale, Canada.

Chaallal, O. and Gauthier, D. (2001) 'Deformation and ductility considerations of ductile coupled shear walls located in Canadian seismic regions', *Canadian Journal of Civil Engineering*. doi: 10.1139/cjce-28-4-738.

- Collins, M. P. and Mitchell, D. (1980) 'Shear and Torsion Design of Prestressed and Non-Prestressed Concrete Beams', *PCI Journal*. doi: 10.15554/pcij.09011980.32.100.
- Cortés-Puentes, L. *et al.* (2018) 'Cyclic loading testing of repaired SMA and steel reinforced concrete shear walls', *Engineering Structures*. doi: 10.1016/j.engstruct.2018.04.044.
- Doiron, G. (2016) 'Pier Repair/Retrofit Using UHPC – Examples of Completed Projects in North America', (1), pp. 983–992. doi: 10.21838/uhpc.2016.99.
- Dopko, M. *et al.* (2018) 'Flexural performance evaluation of fiber-reinforced concrete incorporating multiple macro-synthetic fibers', *Transportation Research Record*. doi: 10.1177/0361198118798986.
- Eligehausen, R., Popov, E. P. and Bertero, V. V. (1982) 'LOCAL BOND STRESS-SLIP RELATIONSHIPS OF DEFORMED BARS UNDER GENERALIZED EXCITATIONS.', in.
- Eshghi, N. (2018) *Behaviour and Analysis of Strain Hardening Fiber Reinforced Cementitious Composites Under Shear and Flexure*. York University. Available at: <https://www.library.yorku.ca/find/Record/10315-35898>.
- Eurocode, C. (2004) '8: Design of structures for earthquake resistance—Part 1: General rules, seismic actions and rules for buildings (EN 1998-1: 2004)', *European Committee for Normalization, Brussels*.
- Fema 461 (2007) 'Interim Testing Protocols for Determining the Seismic Performance Characteristics of Structural and Nonstructural Components', *Applied Technology Council*.
- FEMA P-1050 (2015) *NEHRP Recommended seismic provisions for new buildings and other structures*, *Building Seismic Safety Council*. doi: 10.1017/CBO9781107415324.004.
- Galal, K. and El-Sokkarry, H. (2008) 'Recent Advancements in Retrofit of Rc Shear Walls', *The 14th World Conference on Earthquake Engineering*.
- Georgiou, A. V. and Pantazopoulou, S. J. (2016) 'Use of Waste Fly Ash from Power Plants for Use in Cementitious Composites for Structural Elements', *4th International Conference on Sustainable Solid Waste Management*, (June). doi: 10.1017/CBO9781107415324.004.
- Ghorbanirenani, I. *et al.* (2012) 'Shake table testing of slender RC shear walls subjected to eastern North America seismic ground motions', *Journal of Structural Engineering (United States)*. doi: 10.1061/(ASCE)ST.1943-541X.0000581.
- Habel, K. (2004) 'Structural Behaviour of Elements Combining Ultra-High Performance Fibre Reinforced Concretes (UHPFRC) and Reinforced Concrete', *Fifth RILEM Symposium on Fibre-*

Reinforced Concretes (FRC) - BEFIB' 2000. doi: 10.5075/epfl-thesis-3036.

Habel, K. *et al.* (2006) 'Development of the mechanical properties of an Ultra-High Performance Fiber Reinforced Concrete (UHPFRC)', *Cement and Concrete Research*. doi: 10.1016/j.cemconres.2006.03.009.

Hognestad, E. (1951) 'A Study of Combined Bending and Axial Load in Reinforced Concrete Members', *Bulletin Series No. 399*. doi: 10.14359/7785.

Hooton, R. D., Nokken, M. and Thomas, M. D. A. (2007) 'Portland-Limestone Cement : State-of-the-Art Report and Gap Analysis for CSA A 3000', *Cement Association of Canada*. doi: 10.1016/j.fss.2012.08.014.

Ibrahim, M. A. *et al.* (2017) 'Effect of material constituents on mechanical & fracture mechanics properties of ultra-high-performance concrete', *ACI Structural Journal*. doi: 10.14359/51689717.

Johnston, C. D. (1996) 'Proportioning, mixing and placement of fibre-reinforced cements and concretes', in *Proceedings of the International RILEM Conference*.

Kanda, T. and Li, V. C. (1998) 'Interface property and apparent strength of high-strength hydrophilic fiber in cement matrix', *Journal of Materials in Civil Engineering*. doi: 10.1061/(ASCE)0899-1561(1998)10:1(5).

Klemas, J. (2002) 'Materiales inteligentes Aleaciones metálicas y polímeros con memoria de forma **', *Revista CES MEDICINA*.

Krstulovic-Opara, N. *et al.* (1997) 'Flexural behavior of composite R.C.-Slurry Infiltrated Mat Concrete (SIMCON) members', *ACI Structural Journal*. doi: 10.14359/500.

KUPFER H, HILSDORF HK and RUSCH H (1969) 'BEHAVIOR OF CONCRETE UNDER BIAXIAL STRESSES', *Am Concrete Inst-J*.

Lagoudas, D. . (1989) *Shape Memory Alloys - Modeling and Engineering Applications*, *Journal of Chemical Information and Modeling*. doi: 10.1017/CBO9781107415324.004.

Lawler, J. S., Zampini, D. and Shah, S. P. (2005) 'Microfiber and macrofiber hybrid fiber-reinforced concrete', *Journal of Materials in Civil Engineering*. doi: 10.1061/(ASCE)0899-1561(2005)17:5(595).

Lepech, M. D. *et al.* (2008) 'Design of green engineered cementitious composites for improved sustainability', *ACI Materials Journal*. doi: 10.14359/20198.

Lepech, M. D. and Li, V. C. (2008) 'Large-scale processing of engineered cementitious composites', *ACI Materials Journal*. doi: 10.14359/19897.

- Li, V. C. (2002a) ‘Advances in ECC Research’, *ACI Special Publication on Concrete: Material Science to Applications*.
- Li, V. C. (2002b) ‘Reflections on the research and development of Engineered Cementitious Composites (ECC)’, *JCI International Workshop on Ductile Fiber Reinforced Cementitious Composites (DFRCC)—Application and Evaluation*.
- Li, V. C. et al. (2003) *Final Report on Durable Link Slabs for Jointless Bridge Decks Based on Strain-Hardening Cementitious Composites*, The Advanced Civil Engineering Material Research Laboratory, MDOT Project Manager: Roger Till.
- Li, V. C. et al. (2004) ‘Micromechanics-based durability study of polyvinyl alcohol-engineered cementitious composite’, *ACI Materials Journal*. doi: 10.14359/13120.
- Li, X., Li, M. and Song, G. (2015) ‘Energy-dissipating and self-repairing SMA-ECC composite material system’, *Smart Materials and Structures*. doi: 10.1088/0964-1726/24/2/025024.
- Mahin, S. (2008) ‘Sustainable Design Considerations in Earthquake Engineering’, *The 14th World Conference on Earthquake Engineering*.
- Marcos-Meson, V. et al. (2018) ‘Corrosion resistance of steel fibre reinforced concrete - A literature review’, *Cement and Concrete Research*. doi: 10.1016/j.cemconres.2017.05.016.
- Massone, L. M. and Wallace, J. W. (2004) ‘Load-Deformation Responses of Slender Reinforced Concrete Walls’, *ACI Structural Journal*.
- McKenna F, Fenves GL, F. F. (no date) *OpenSees*.
- Medhekar, M. S.; Jain, S. K. (1993) ‘Seismic Behaviour, design and detailing of RC shear walls, Part I: Behaviour and strength’, *The Indian Concrete Journal*, pp. 311–318.
- Mena, M. and Palermo, D. (2018) ‘MEMORY ALLOY REINFORCED CONCRETE SHEAR WALLS’, in *CSCE Fredericton Annual Conference*. Fredericton, Canada: CSCE Fredericton Annual Conference, p. 11.
- Mermigas, K. K. (2018) *Evolution of Bridge Practices in Ontario, Canada*. Saskatoon, Canada.
- Morcos, M. and Palermo, D. (2019) ‘SMA-Reinforced Concrete Shear Walls Subjected to Reverse Cyclic Loading’, in *SMAR 2019 - Fifth Conference on Smart Monitoring, Assessment and Rehabilitation of Civil Structures*. Potsdam, Germany: SMAR 2019. Available at: <https://www.smar2019.org/>.
- Naaman, a. E. and Reinhardt, H. W. (2003) ‘High Performance Fiber Reinforced Cement Composites HPRCC-4: International RILEM Workshop’, *Materials and Structures*. doi: 10.1007/BF02479507.

- Naaman, A. E. (2003) 'Engineered Steel Fibers with Optimal Properties for Reinforcement of Cement Composites', *Journal of Advanced Concrete Technology*. doi: 10.3151/jact.1.241.
- Naaman, A. E. (2018) 'Fiber reinforced concrete: five decades of progress', in. doi: 10.21452/bccm4.2018.02.01.
- Naaman, A. E. and Reinhardt, H. W. (2006) 'Proposed classification of HPFRC composites based on their tensile response', *Materials and Structures/Materiaux et Constructions*. doi: 10.1617/s11527-006-9103-2.
- Nehdi, M., Alam, M. S. and Youssef, M. A. (2011) 'Seismic behaviour of repaired superelastic shape memory alloy reinforced concrete beam-column joint', *Smart Structures and Systems*. doi: 10.12989/sss.2011.7.5.329.
- Oesterle, R. G. *et al.* (1976) 'Earthquake Resistant Structural Walls. Tests of Isolated Walls.', *Pca*.
- Palermo, D. (2002) *Behaviour and analysis of reinforced concrete walls subjected to reversed cyclic loading, ProQuest Dissertations and Theses*.
- Palermo, D. and Vecchio, F. J. (2002) 'Behavior of three-dimensional reinforced concrete shear walls', *ACI Structural Journal*. doi: 10.14359/11038.
- Park, H. and Eom, T. (2004) 'Energy Dissipation Capacity of Flexure-Dominated Reinforced Concrete Members', *13th World Conference on Earthquake Engineering World Conference on Earthquake Engineering*.
- Park, R. (1988) 'Ductility evaluation from laboratory and analytical testing', in *Proceedings of the 9th World Conference on Earthquake Engineering, Tokyo-Kyoto, Japan*. doi: 1.
- Paulay, T., Priestley, M. J. N. and Syngé, A. J. (1982) 'DUCTILITY IN EARTHQUAKE RESISTING SQUAT SHEARWALLS.', in *Journal of the American Concrete Institute*. doi: 10.14359/10903.
- Pereiro-Barceló, J. *et al.* (2018) 'Ductility of high-performance concrete and very-high-performance concrete elements with Ni-Ti reinforcements', *Construction and Building Materials*. doi: 10.1016/j.conbuildmat.2018.04.172.
- Popovics, S. (1973) 'A numerical approach to the complete stress-strain curve of concrete', *Cement and Concrete Research*. doi: 10.1016/0008-8846(73)90096-3.
- Rai, D. C., Jain, S. K. and Chakrabarti, I. (2012) 'Evaluation of Properties of Steel Reinforcing Bars for Seismic Design', *15 Wcee*.
- Saiidi, M. S. and Wang, H. (2006) 'Exploratory study of seismic response of concrete columns with

shape memory alloys reinforcement’, *ACI Structural Journal*. doi: 10.14359/15322.

Saikali, R. (2019) *Bond Behaviour of Steel Reinforcing Bars Embedded in Ultra-High-Performance Steel Fiber Reinforced Concrete*. York University.

Scott, B. D., Park, R. and Priestley, M. J. N. (1982) ‘STRESS-STRAIN BEHAVIOR OF CONCRETE CONFINED BY OVERLAPPING HOOPS AT LOW AND HIGH STRAIN RATES.’, *Journal of the American Concrete Institute*. doi: 10.14359/10875.

Seckin, M. (1981) *HYSTERETIC BEHAVIOR OF CAST-IN-PLACE EXTERIOR BEAM COLUMN SUB-ASSEMBLIES*. University of Toronto.

SeismoSoft (2019) ‘A computer program for static and dynamic nonlinear analysis of framed structures’, Available from URL: www.seismosoft.com. doi: <http://www.seismosoft.com>.

Shao, B. (2016a) ‘Mix Development of PLC-based Ultra-high Performance Fibre Reinforced Concrete and Characterization of Key Mechanical Properties and Time-Dependent Behaviour’, pp. 1–130. Available at: <https://tspace.library.utoronto.ca/handle/1807/76013>.

Shao, B. (2016b) ‘Mix Development of PLC-based Ultra-high Performance Fibre Reinforced Concrete and Characterization of Key Mechanical Properties and Time-Dependent Behaviour’, pp. 1–130.

Tazarv, M. and Saiid Saiidi, M. (2015) ‘Reinforcing NiTi Superelastic SMA for Concrete Structures’, *Journal of Structural Engineering (United States)*. doi: 10.1061/(ASCE)ST.1943-541X.0001176.

Telford, T. (1990) ‘Model Code 1990’, *Comite Euro-International Du Beton, Paris*. doi: 10.1680/ceb-fipmc1990.35430.

Tolou Kian, M. J. and Noguez, C. C. (2018) ‘Damage mitigation in reinforced concrete shear walls detailed with SMA and SFRC’, in *International Conference on Civil, Structural and Transportation Engineering*. doi: 10.11159/iccste18.103.

Tsiotsias, K. (2019) ‘Bond Strength of Ultra-High-Performance Concrete Under Direct Tension Pullout’, (March).

Vecchio, F. J. (1992) ‘Finite element modeling of concrete expansion and confinement’, *Journal of Structural Engineering (United States)*. doi: 10.1061/(ASCE)0733-9445(1992)118:9(2390).

Vecchio, F. J. (2000) ‘Disturbed Stress Field Model for Reinforced Concrete: Formulation’, *Journal of Structural Engineering*. doi: 10.1061/(ASCE)0733-9445(2000)126:9(1070).

Vecchio, F. J. et al. (2002) ‘Behavior of repaired cyclically loaded shearwalls’, *ACI Structural Journal*. doi: 10.14359/11916.

- Vecchio, F. J. and Bucci, F. (1999) 'Analysis of repaired reinforced concrete structures', *Journal of Structural Engineering*. doi: 10.1061/(ASCE)0733-9445(1999)125:6(644).
- Vecchio, F. J. and Collins, M. P. (1986) 'The Modified Compression Field Theory for reinforced concrete elements subjected to shear', *ACI Structural Journal*.
- Vecchio, F. J., Wong, P. and Trommels, H. (2013) 'Vector2 & Formworks User's Manual.', *Second Edition*. doi: 10.1007/s007690000247.
- Wallace, J. W. *et al.* (2012) 'Damage and implications for seismic design of RC structural wall buildings', *Earthquake Spectra*. doi: 10.1193/1.4000047.
- Walraven, J. C. (1981) 'FUNDAMENTAL ANALYSIS OF AGGREGATE INTERLOCK', *ASCE J Struct Div*.
- Wille, K., El-Tawil, S. and Naaman, A. E. (2014) 'Properties of strain hardening UHPFRC under direct tensile loading', *Cement and Concrete Composites*. doi: 10.1016/j.cemconcomp.2013.12.015.
- Wille, K. and Naaman, A. E. (2012) 'Pullout behavior of high-strength steel fibers embedded in ultra-high-performance concrete', *ACI Materials Journal*. doi: 10.14359/51683923.
- Wong, P. S.-L. (2002) 'User facilities for two-dimensional nonlinear finite element analysis of reinforced concrete', *ProQuest Dissertations and Theses*.
- Wong, P. S., Vecchio, F. J. and Trommels, H. (2013) *VecTor2 and FormWorks user's manual*, University of Toronto, Toronto, ON, Canada. doi: 10.1037/023990.
- Yoo, D. Y., Yoon, Y. S. and Banthia, N. (2015) 'Flexural response of steel-fiber-reinforced concrete beams: Effects of strength, fiber content, and strain-rate', *Cement and Concrete Composites*. doi: 10.1016/j.cemconcomp.2015.10.001.
- Youssef, M. A., Alam, M. S. and Nehdi, M. (2008) 'Experimental investigation on the seismic behavior of beam-column joints reinforced with superelastic shape memory alloys', *Journal of Earthquake Engineering*. doi: 10.1080/13632460802003082.
- Yu, R., Spiesz, P. and Brouwers, H. J. H. (2014) 'Mix design and properties assessment of Ultra-High Performance Fibre Reinforced Concrete (UHPFRC)', *Cement and Concrete Research*. doi: 10.1016/j.cemconres.2013.11.002.
- Zaidi, M., Palermo, D. and Dragmeriscu, E. (2016) *Experimental Testing and Reliability Analysis of Repaired SMA Shear Wall*. University of Ottawa.
- Zheng, Z. and Feldman, D. (1995) 'Synthetic fibre-reinforced concrete', *Progress in Polymer Science*. doi: 10.1016/0079-6700(94)00030-6.

- Zhou, J. *et al.* (2012) 'Improved fiber distribution and mechanical properties of engineered cementitious composites by adjusting the mixing sequence', *Cement and Concrete Composites*. doi: 10.1016/j.cemconcomp.2011.11.019.
- Van Zijl, G. P. A. G. *et al.* (2012) 'Durability of strain-hardening cement-based composites (SHCC)', *Materials and Structures/Materiaux et Constructions*. doi: 10.1617/s11527-012-9845-y.
- Zollo, R. F. (1997) 'Fiber-reinforced concrete: An overview after 30 years of development', *Cement and Concrete Composites*. doi: 10.1016/s0958-9465(96)00046-7.

APPENDICES

Appendix A Load - Deformation Responses from Cable and Linear Potentiometers

A.1 Lateral Displacements

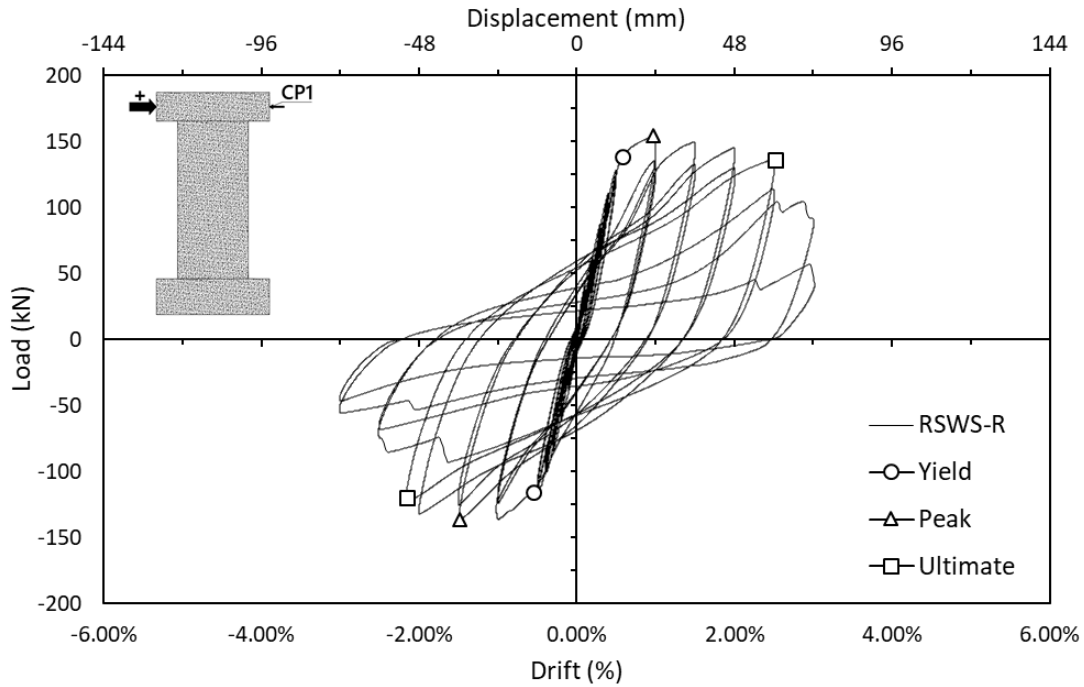


Figure A.1 Load-deformation response of CP1 in wall RSWS-R

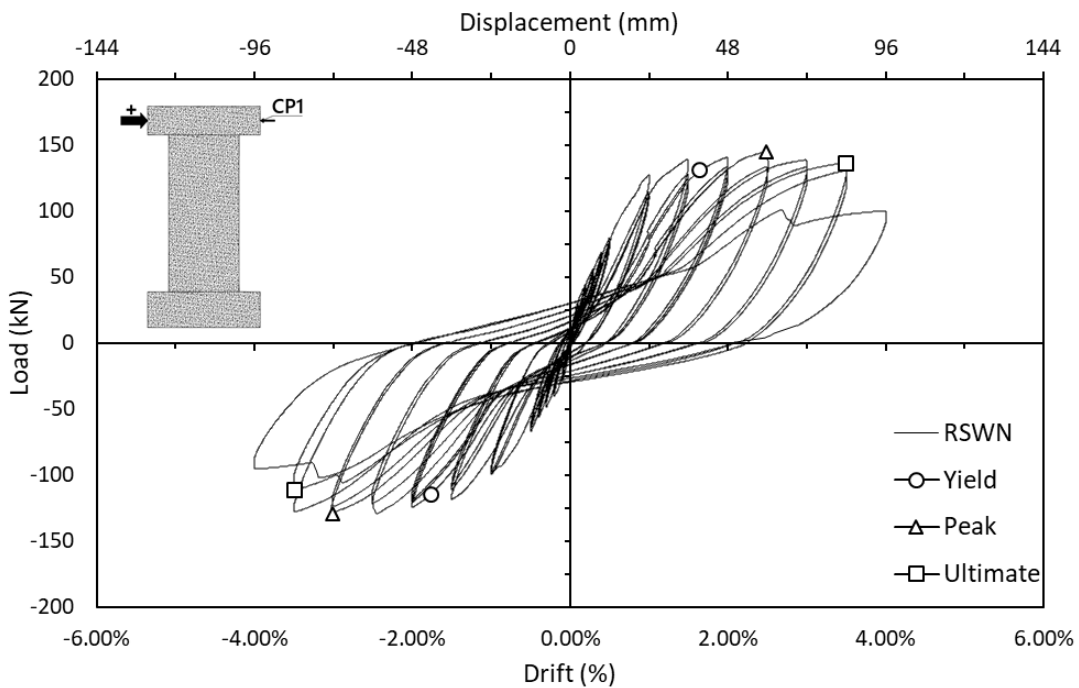


Figure A.2 Load-deformation response of CP1 in wall RSWN

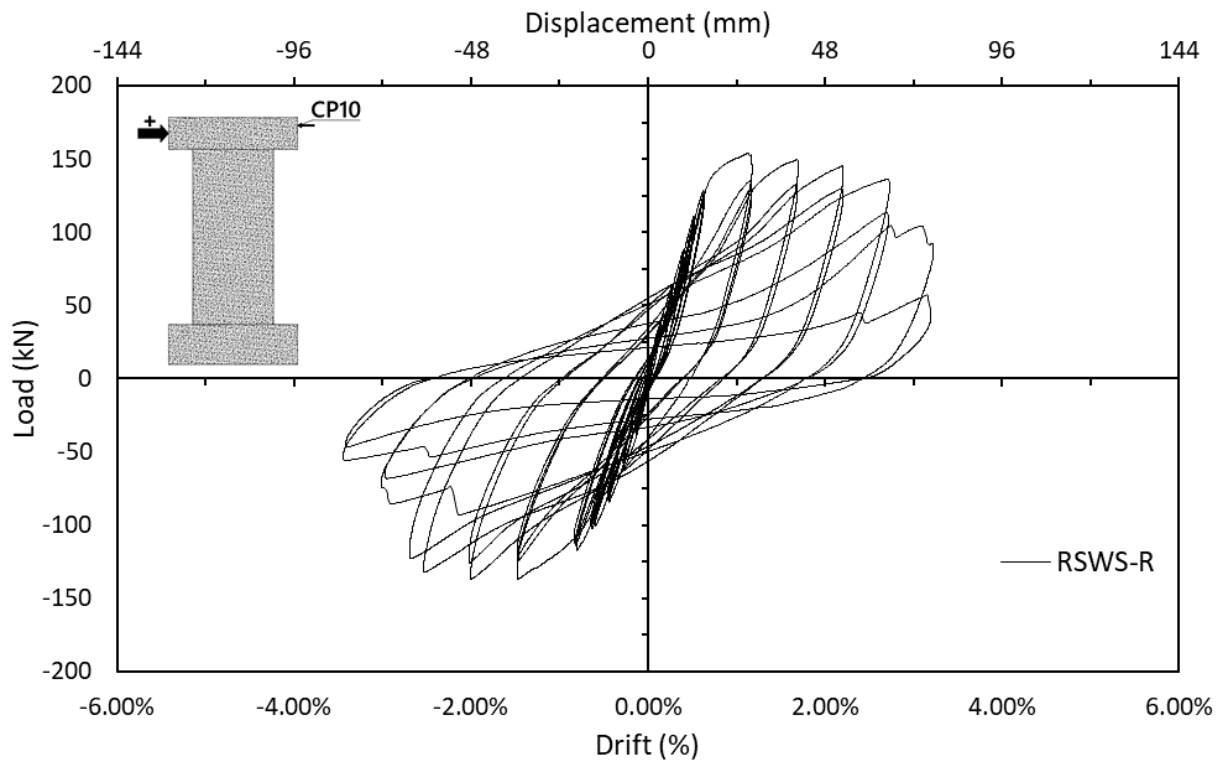


Figure A.3 Load-deformation response of CP10 in wall RSWS-R

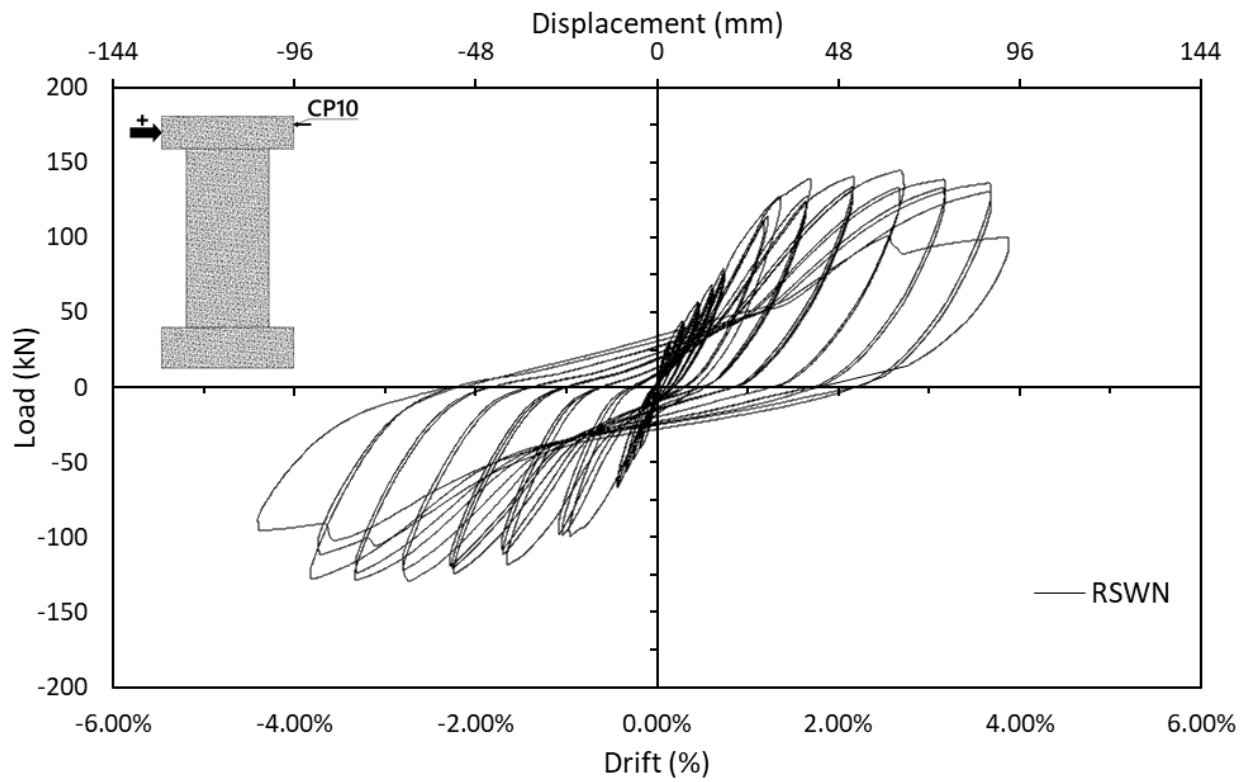


Figure A.4 Load-deformation response of CP10 in wall RSWN

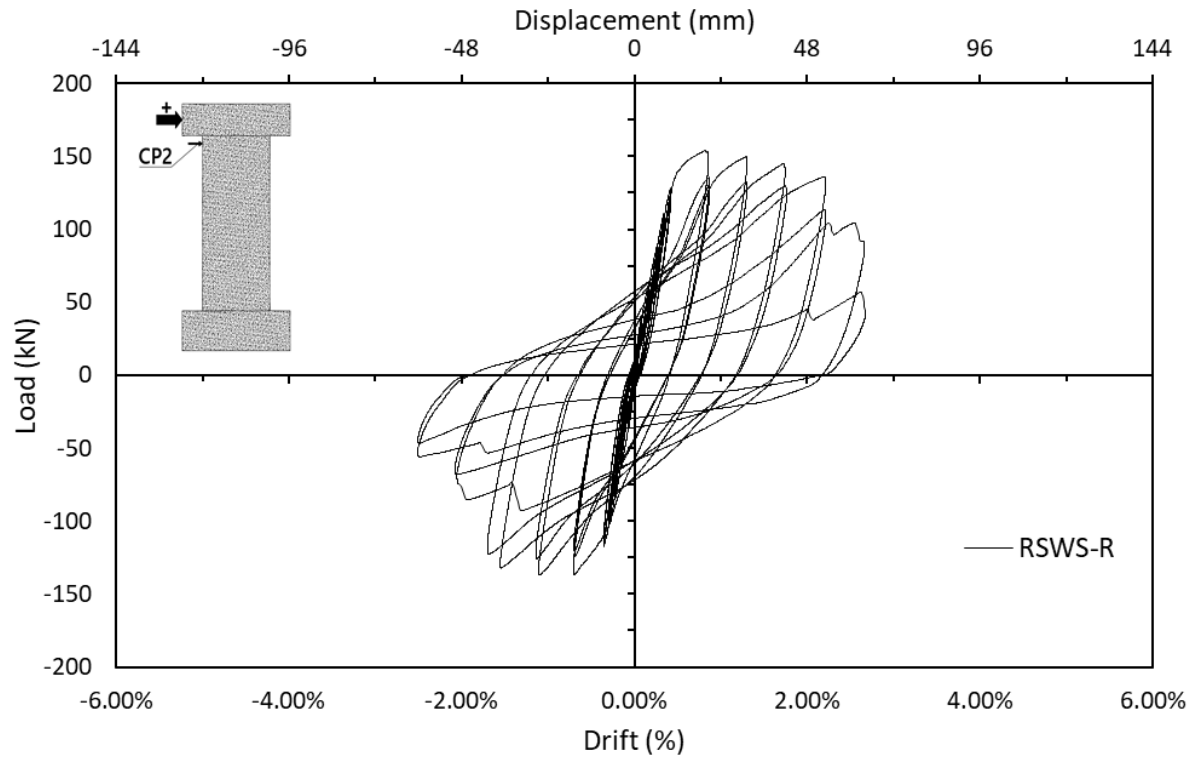


Figure A.5 Load-deformation response of CP2 in wall RSWS-R

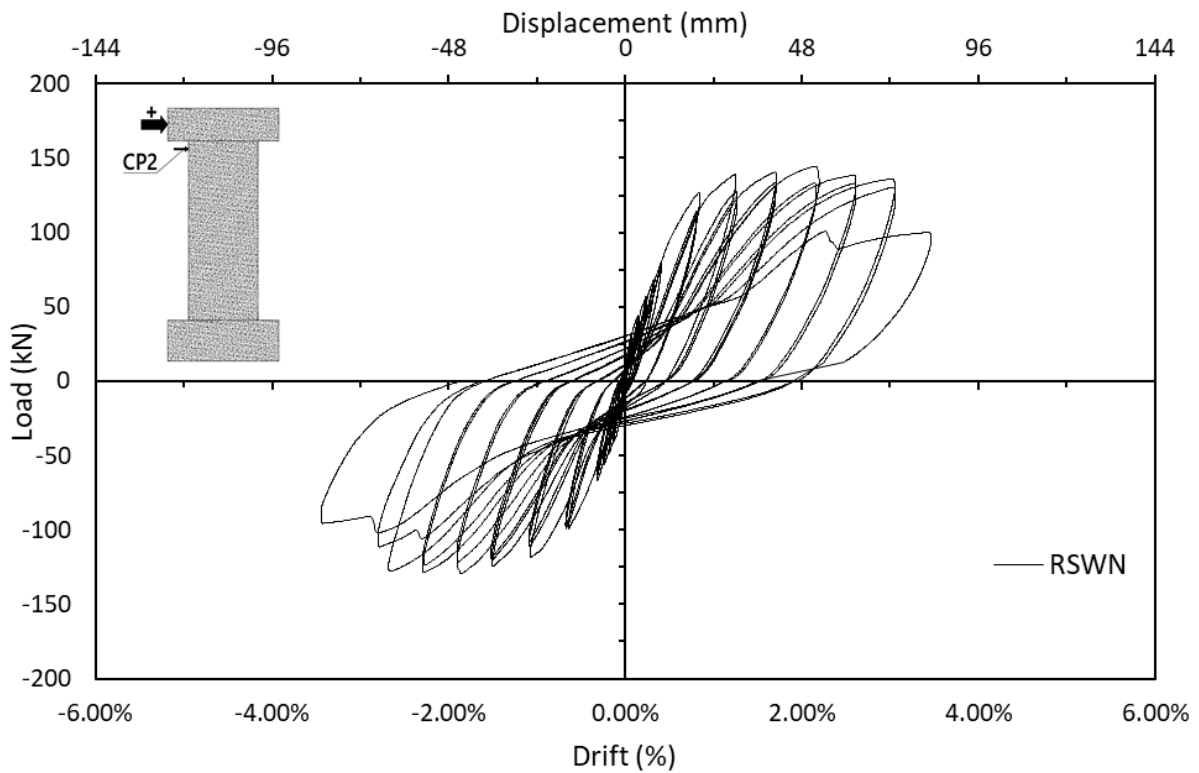


Figure A.6 Load-deformation response of CP2 in wall RSWN

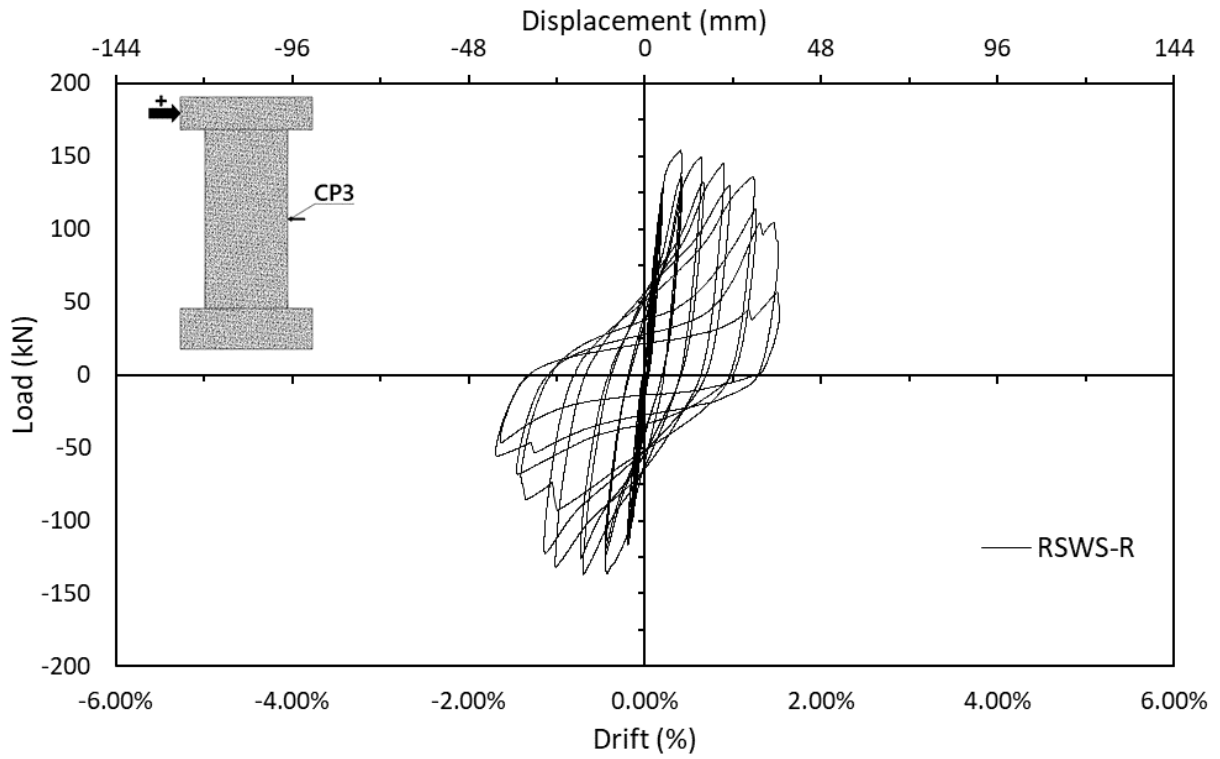


Figure A.7 Load-deformation response of CP2 in wall RSWs-R

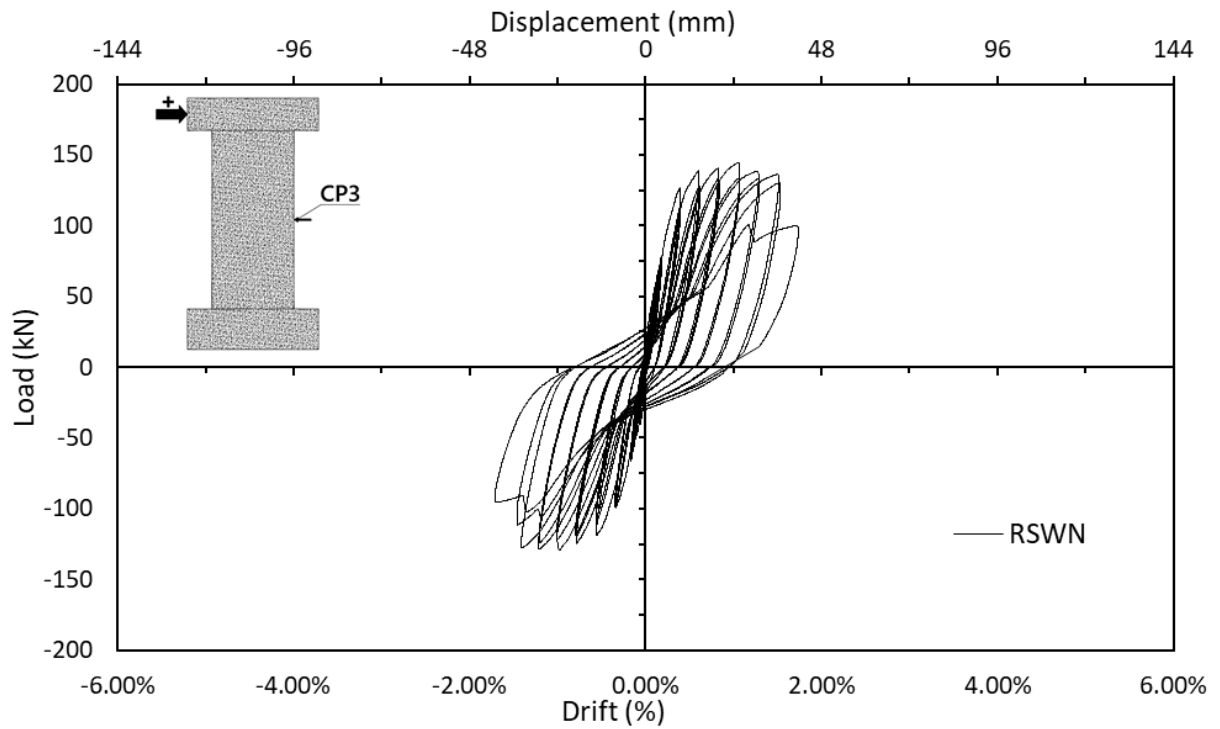


Figure A.8 Load-deformation response of CP3 in wall RSWN

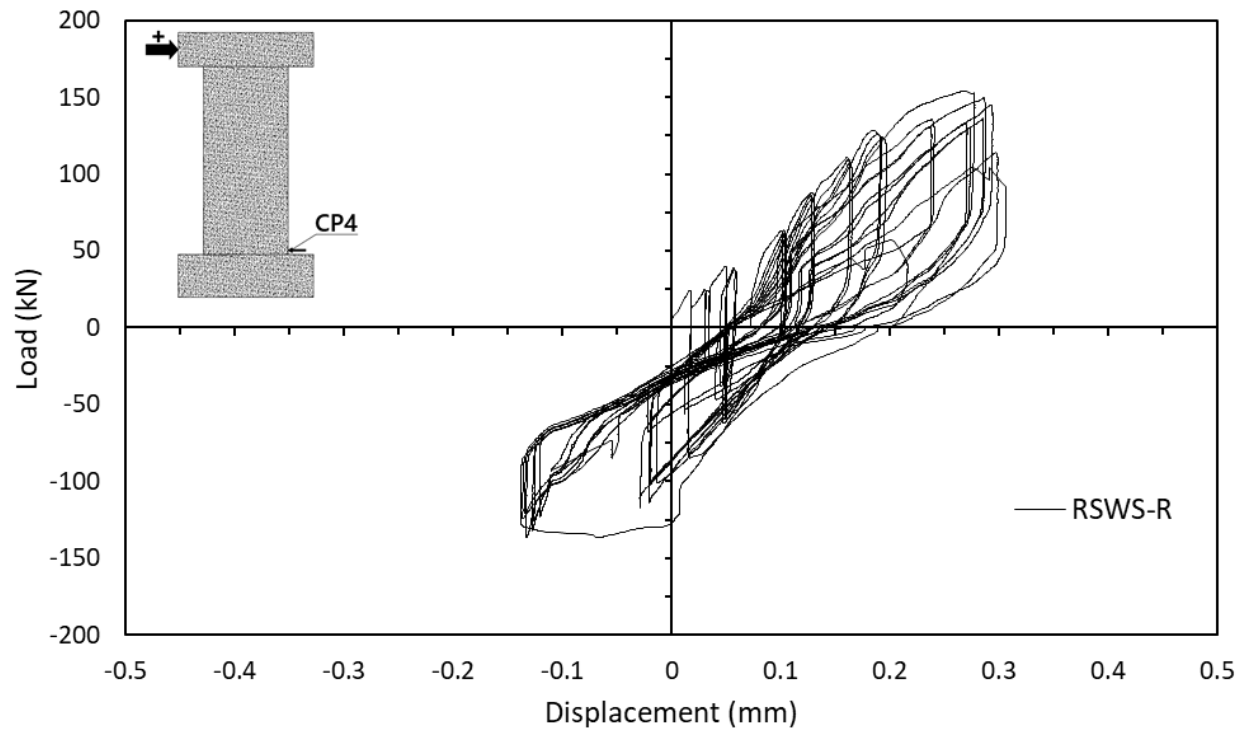


Figure A.9 Load-deformation response of CP4 in wall RSWS-R

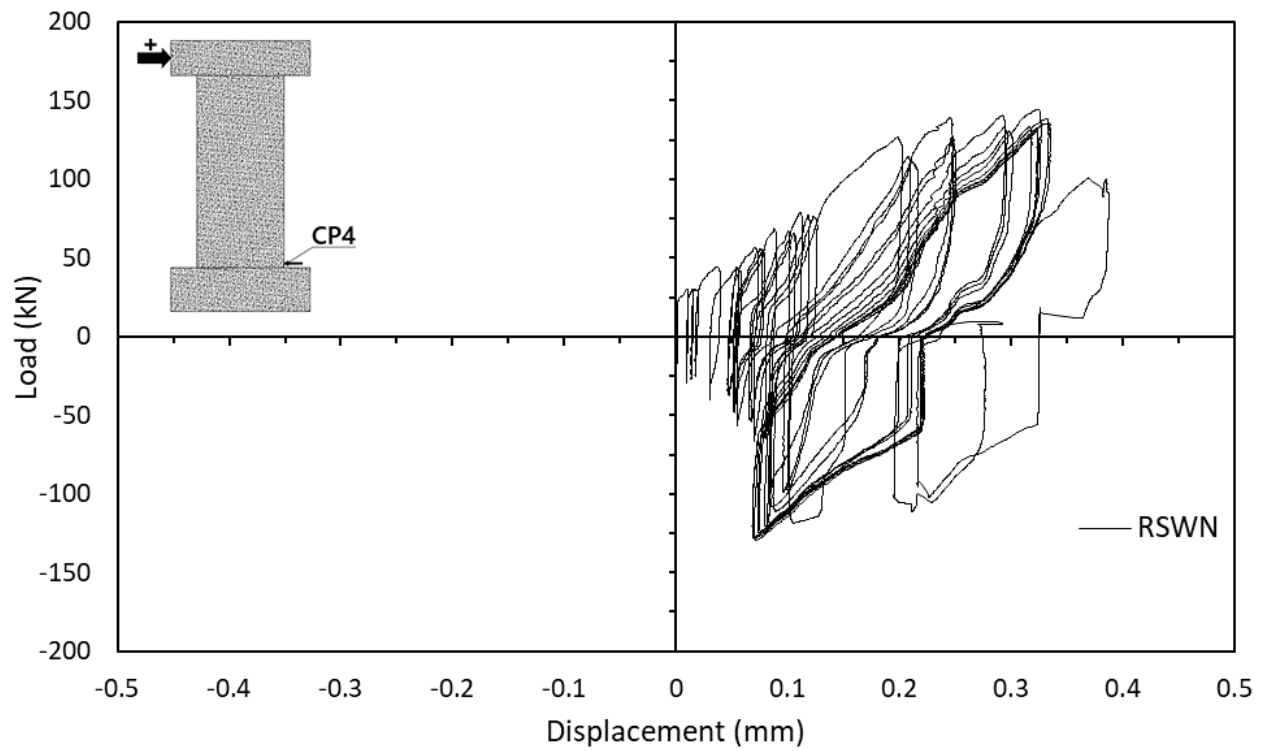


Figure A.10 Load-deformation response of CP4 in wall RSWN

A.2 Vertical Displacements

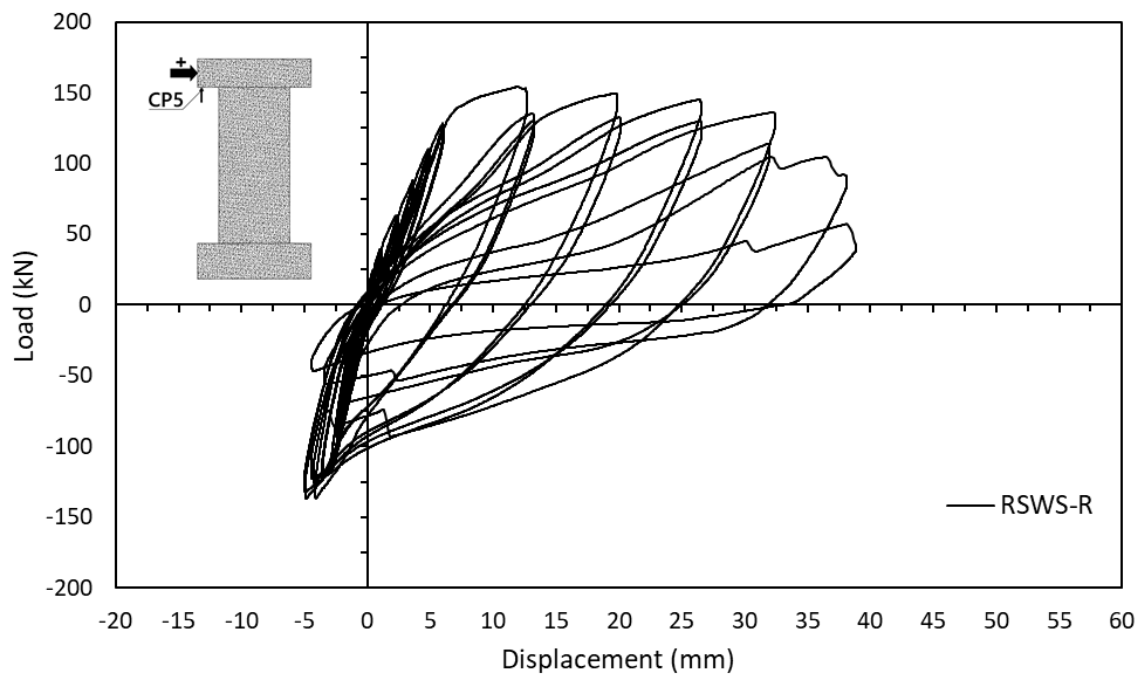


Figure A.11 Load-deformation response of CP5 in wall RSWs-R

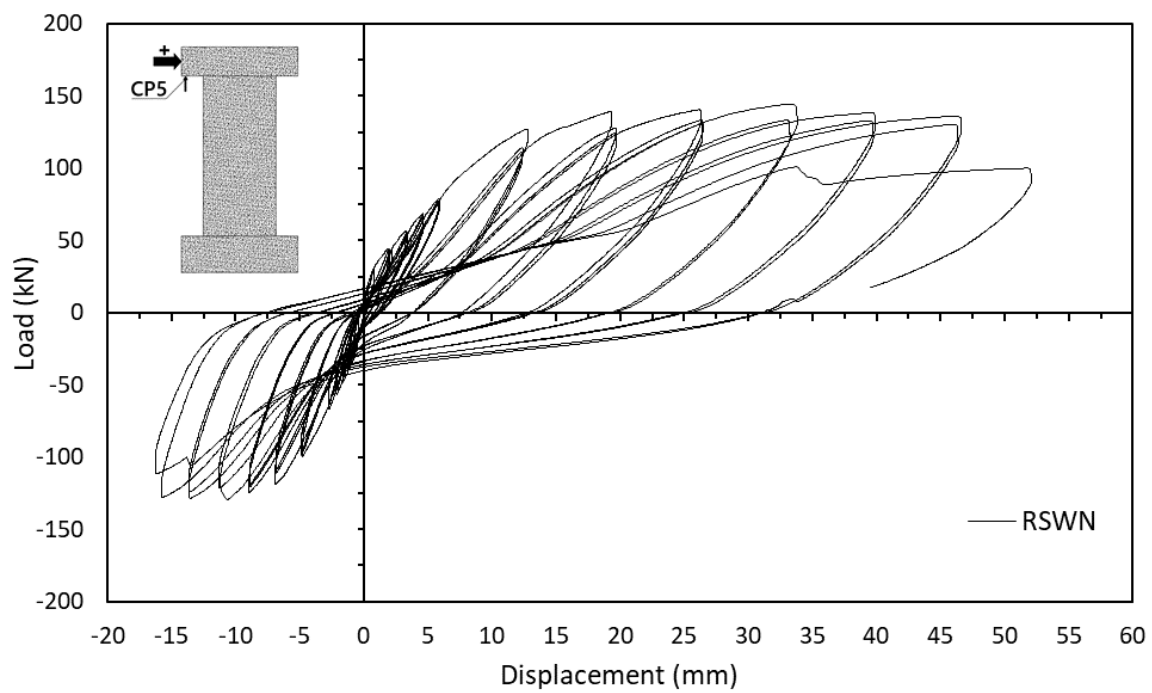


Figure A.12 Load-deformation response of CP5 in wall RSWN

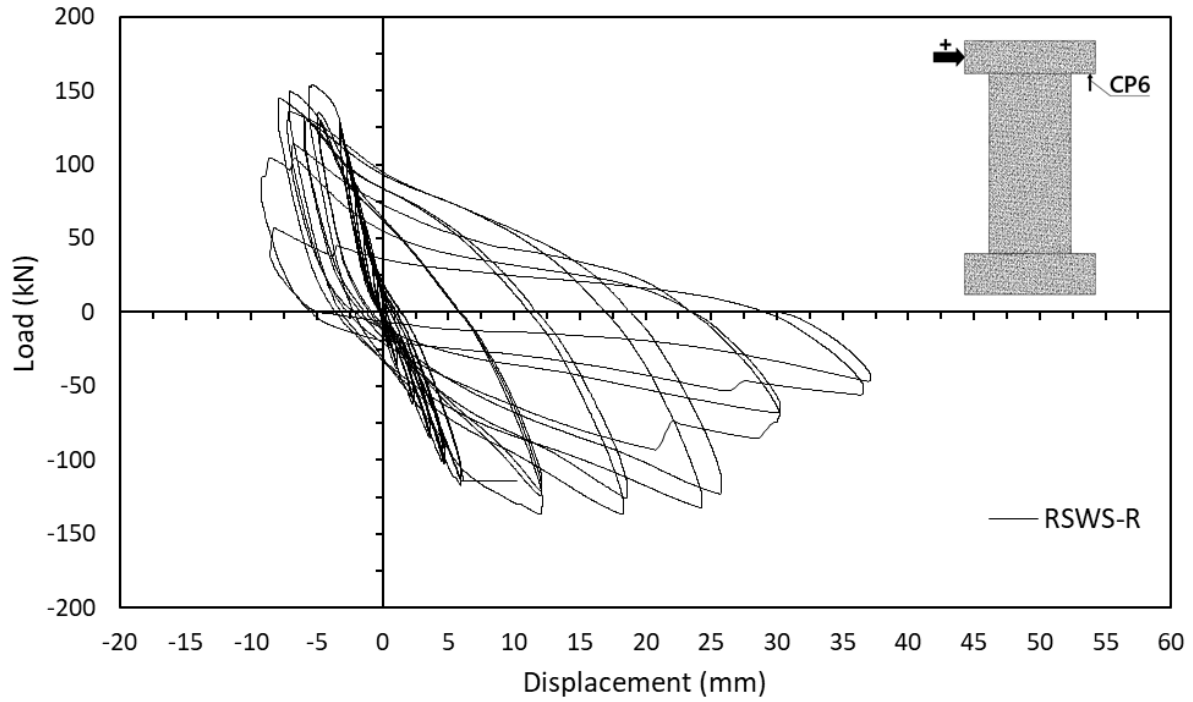


Figure A.13 Load-deformation response of CP6 in wall RSW-S-R

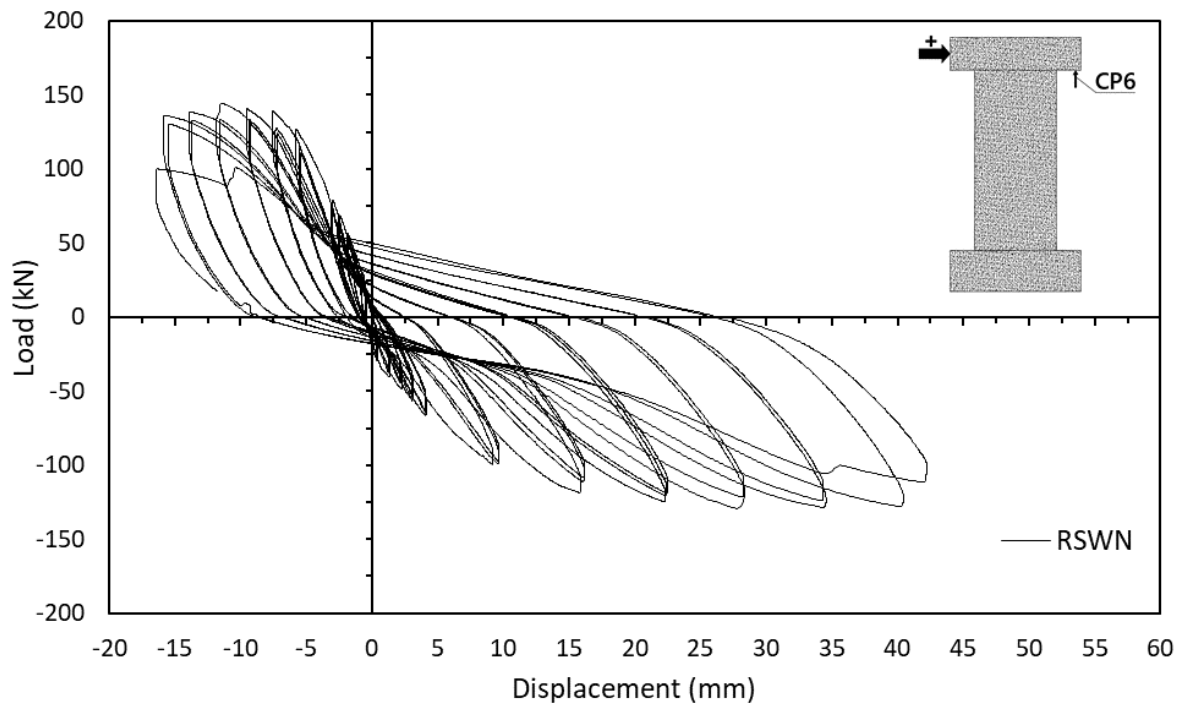


Figure A.14 Load-deformation response of CP6 in wall RSW-N

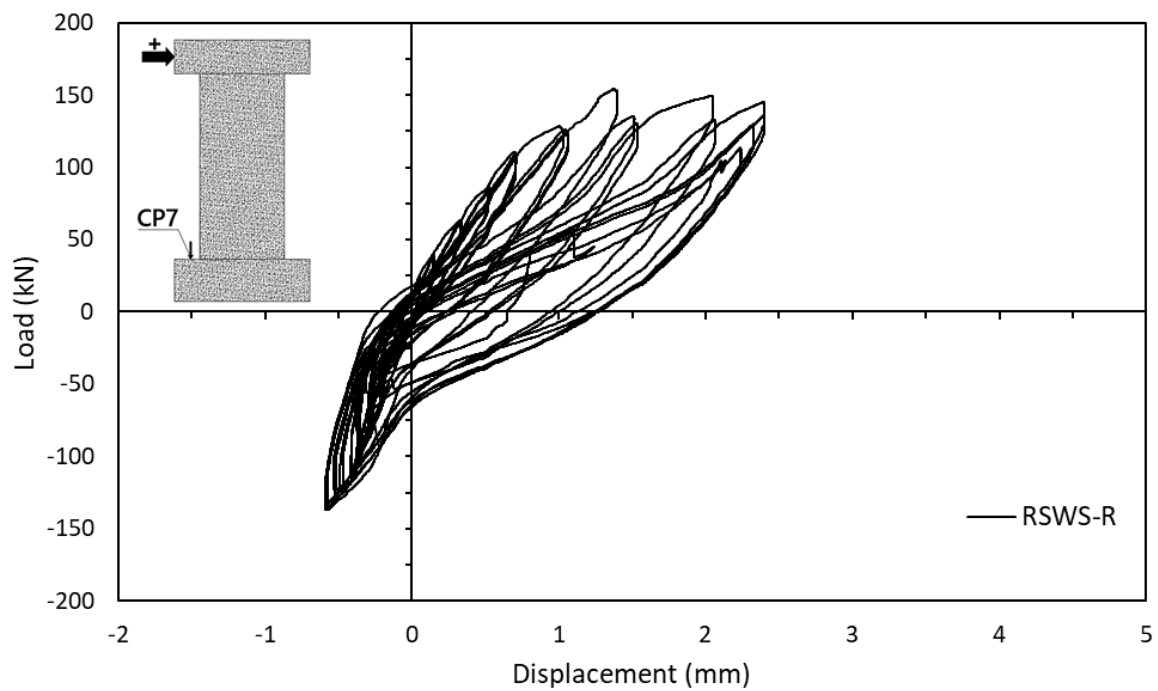


Figure A.15 Load-deformation response of CP7 in wall RSWS-R

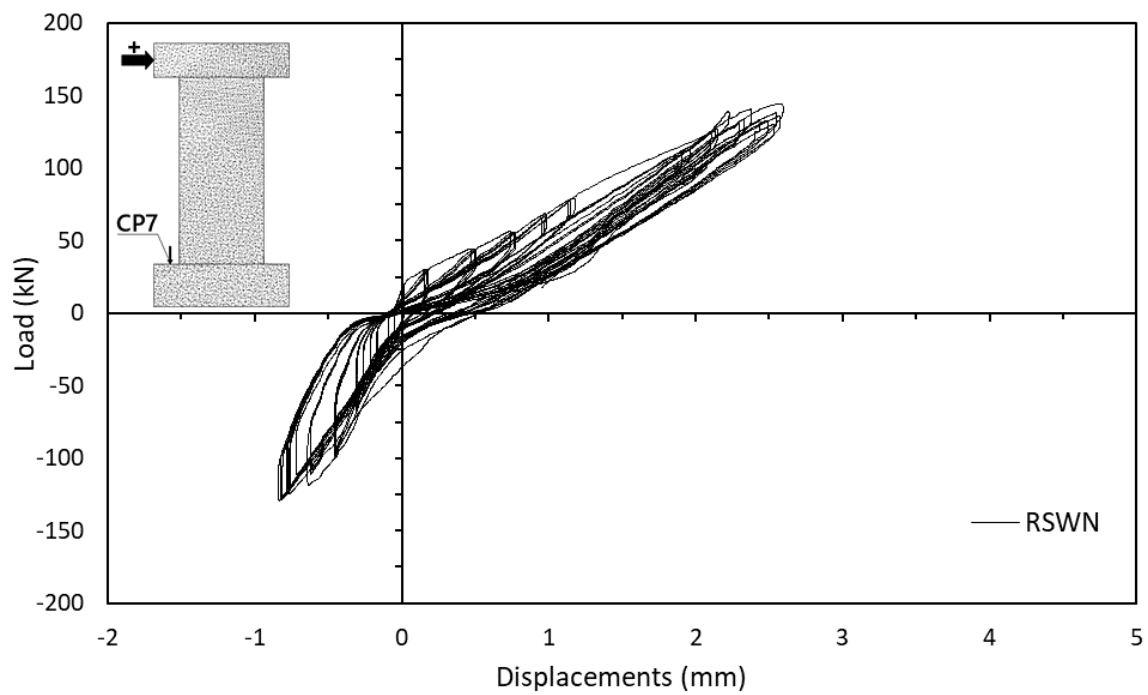


Figure A.16 Load-deformation response of CP7 in wall RSWN

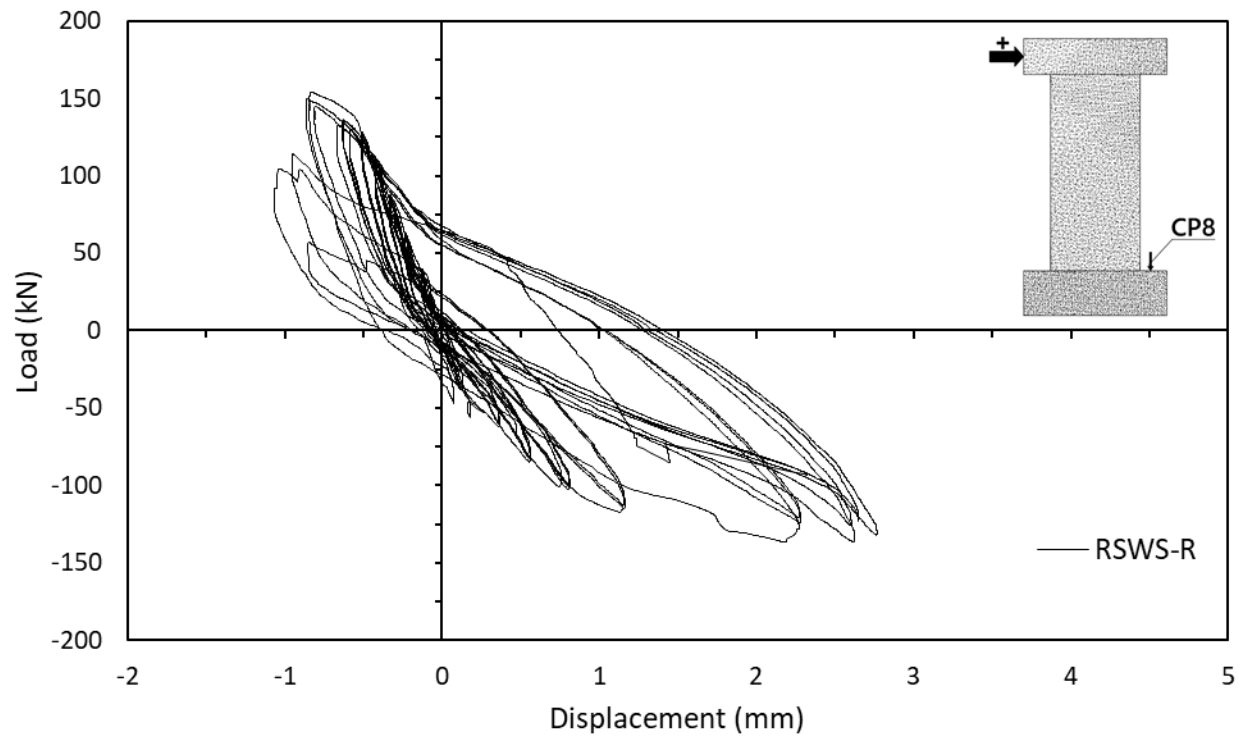


Figure A.17 Load-deformation response of CP8 in wall RSW

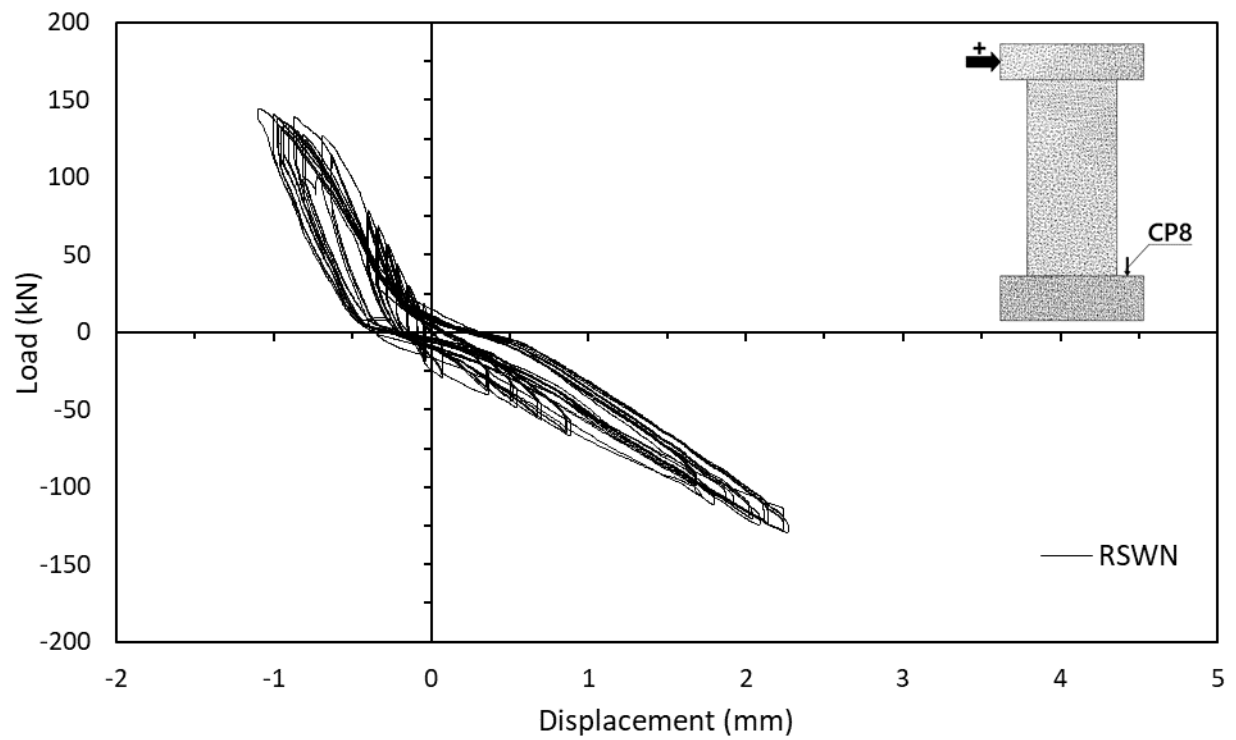


Figure A.18 Load-deformation response of CP8 in wall RSWN

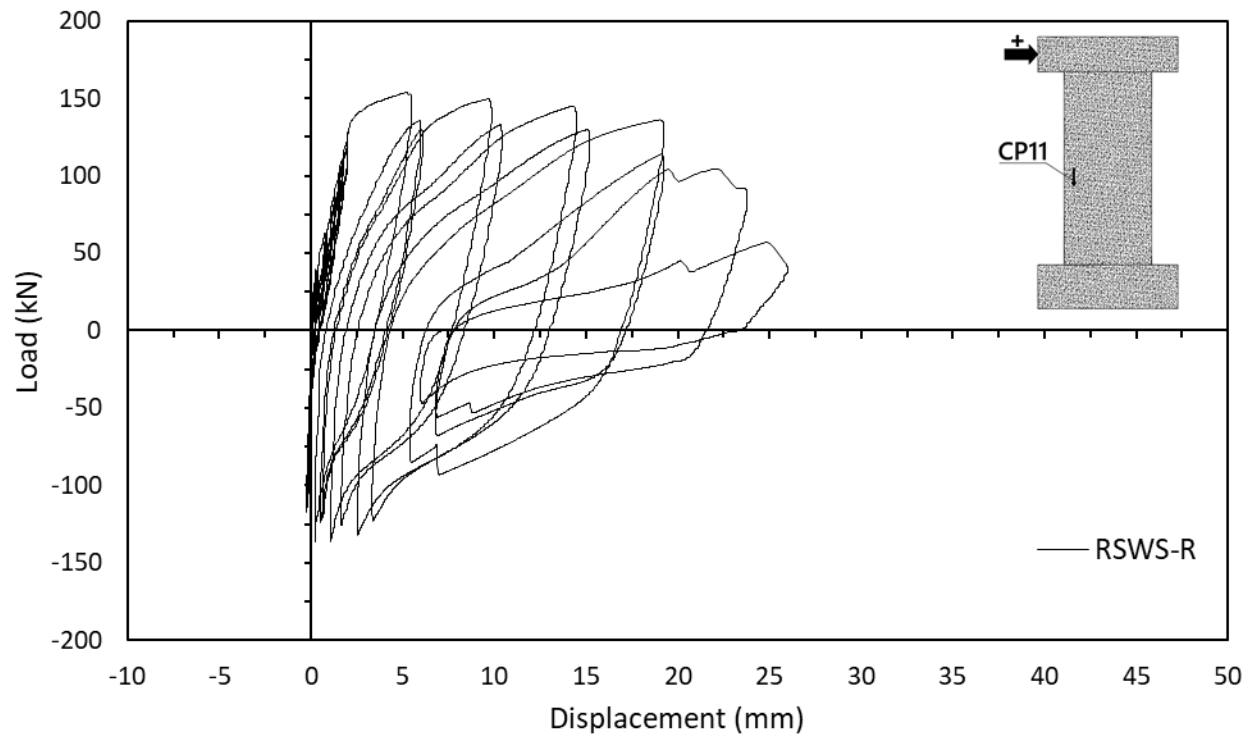


Figure A.19 Load-deformation response of CP11 in wall RSWS-R

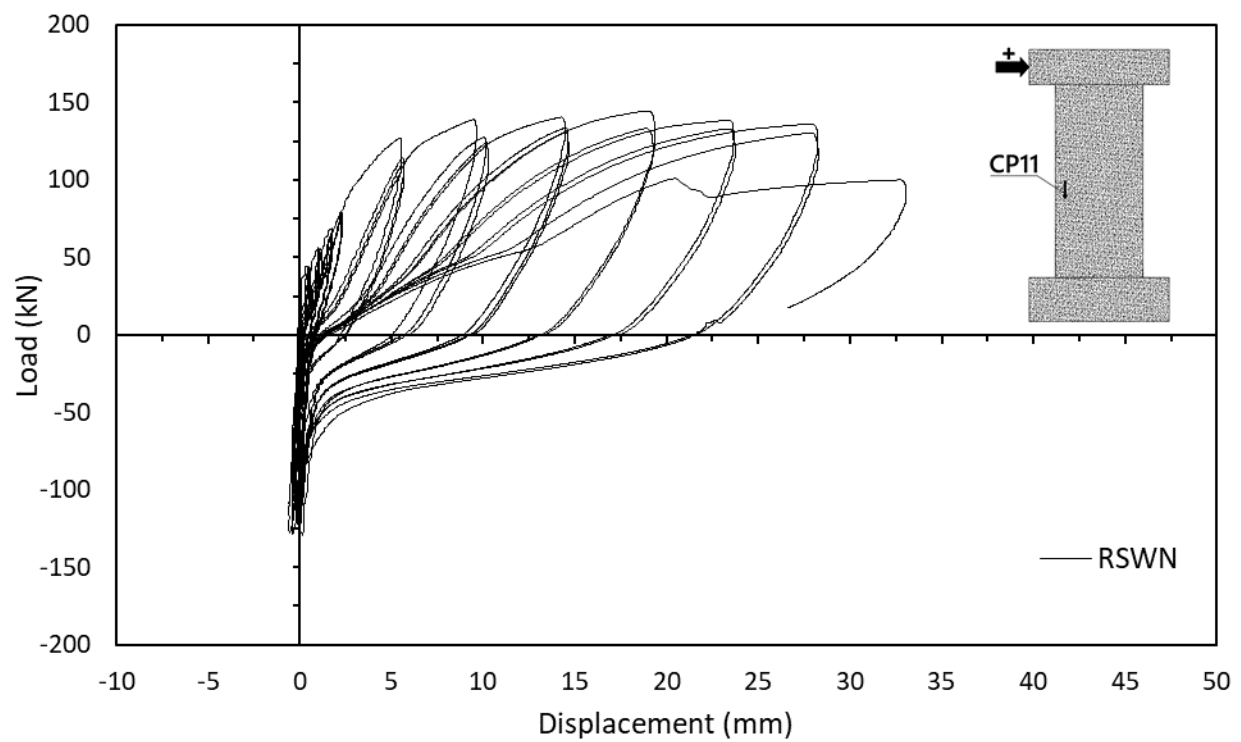


Figure A.20 Load-deformation response of CP11 in wall RSWN

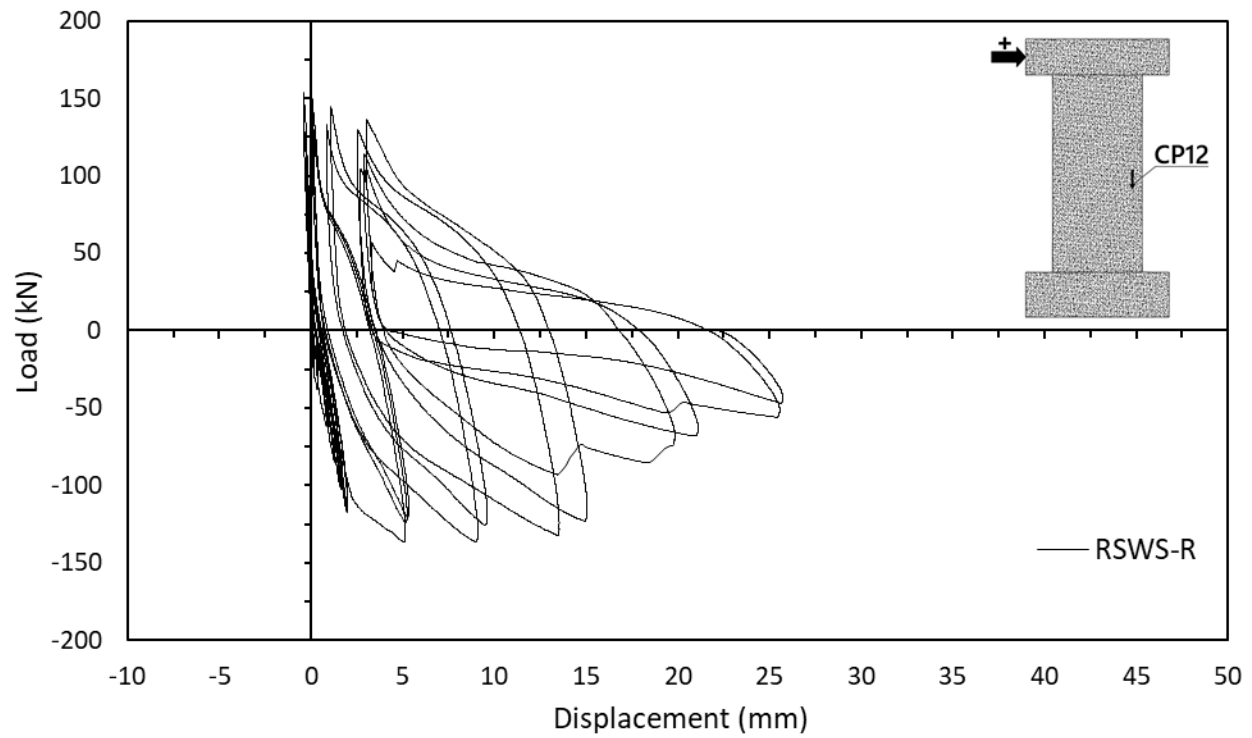


Figure A.21 Load-deformation response of CP12 in wall RSWS-R

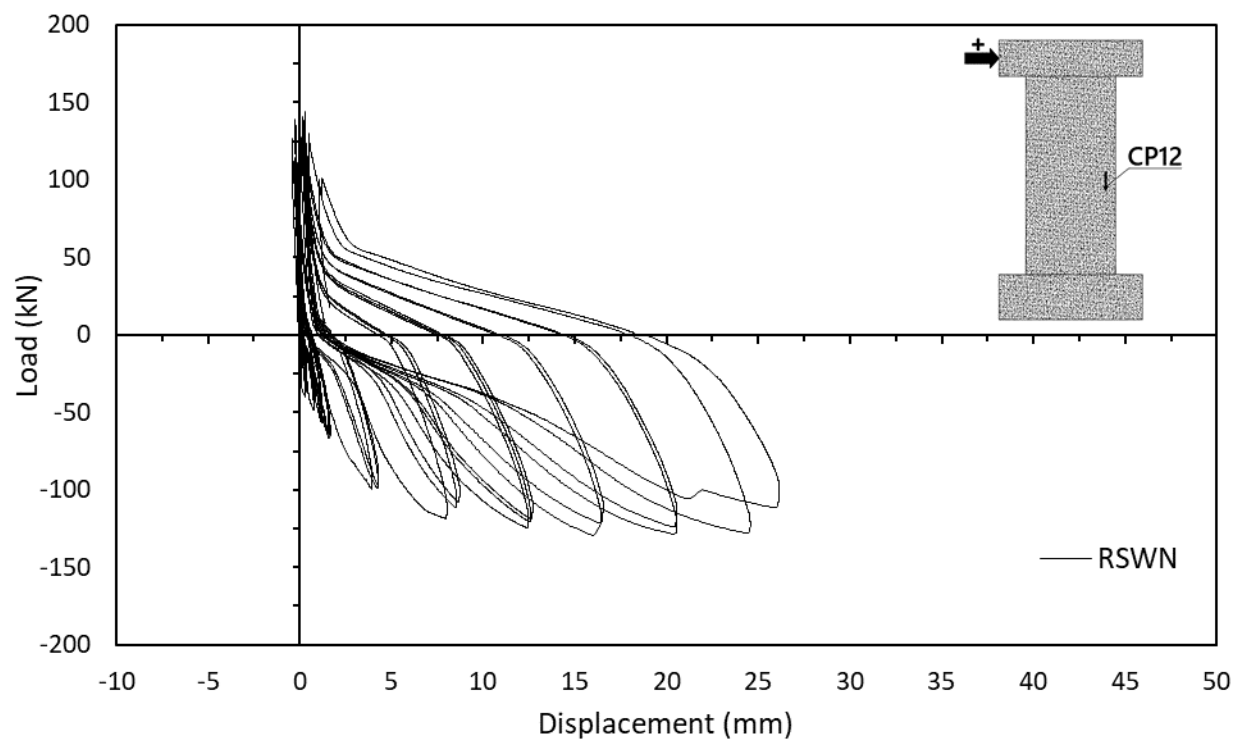


Figure A.22 Load-deformation response of CP12 in wall RSWN

A.3 Diagonal Displacements

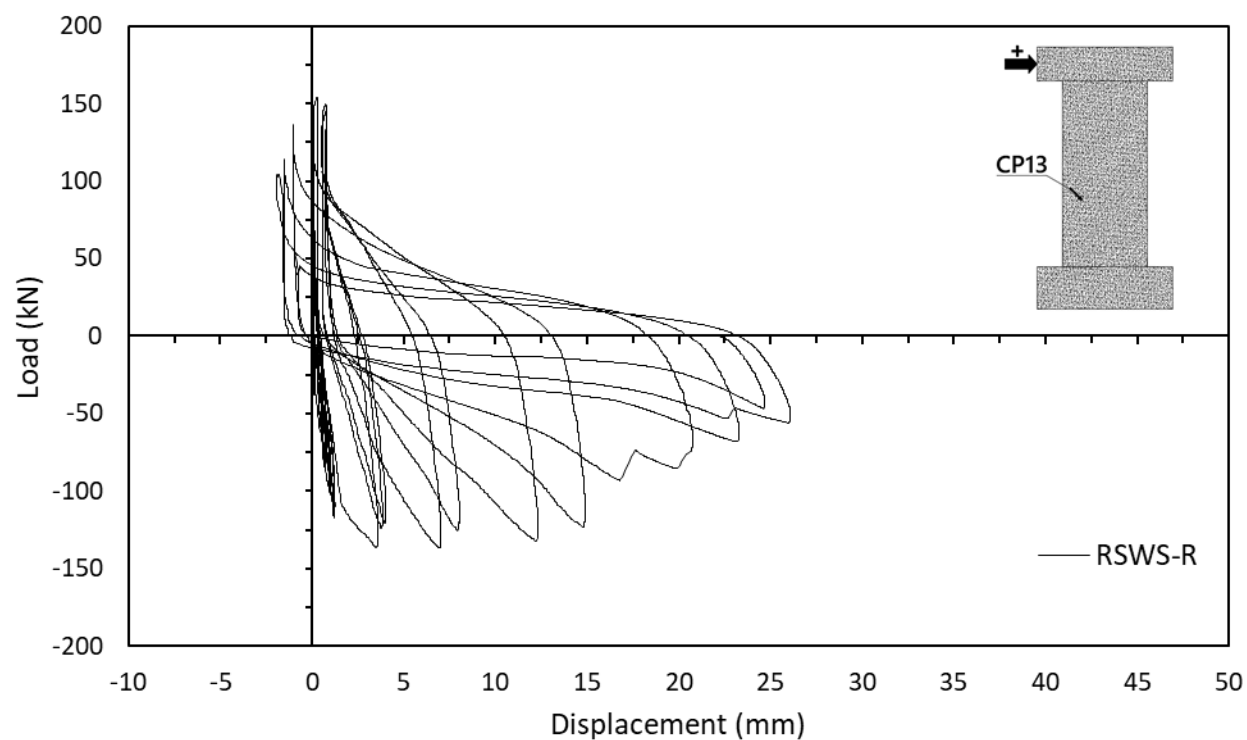


Figure A.23 Load-deformation response of CP13 in wall RSWs-R

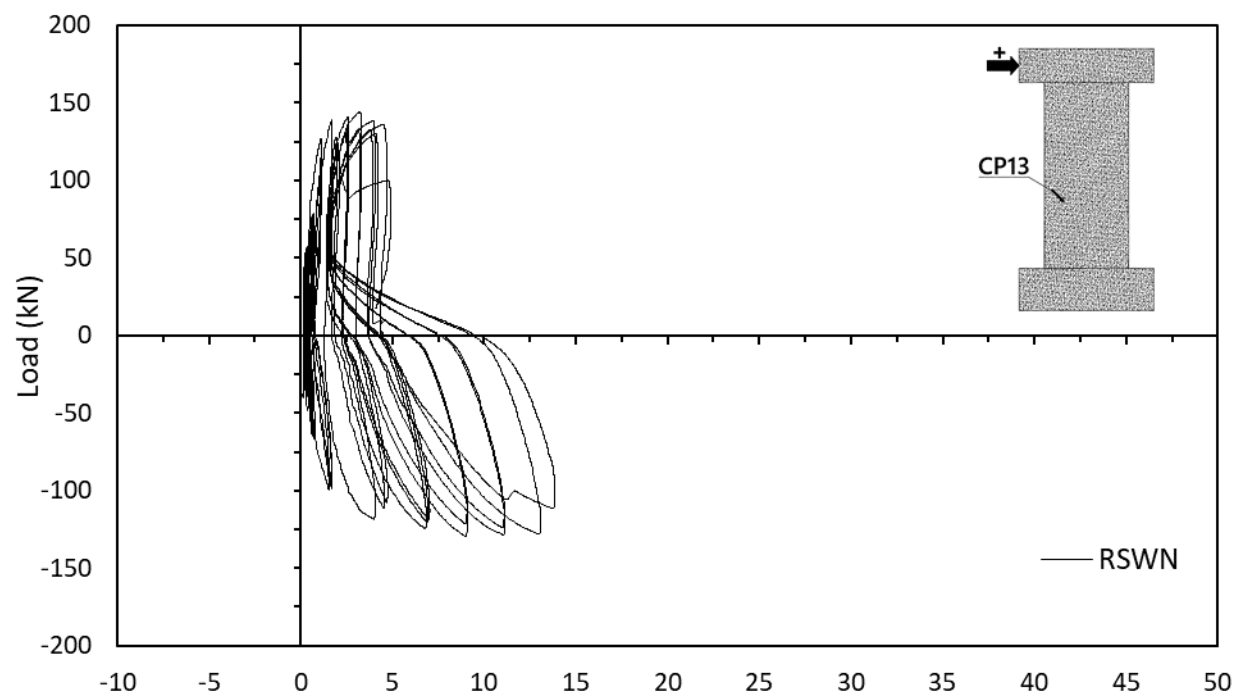


Figure A.24 Load-deformation response of CP13 in wall RSWN

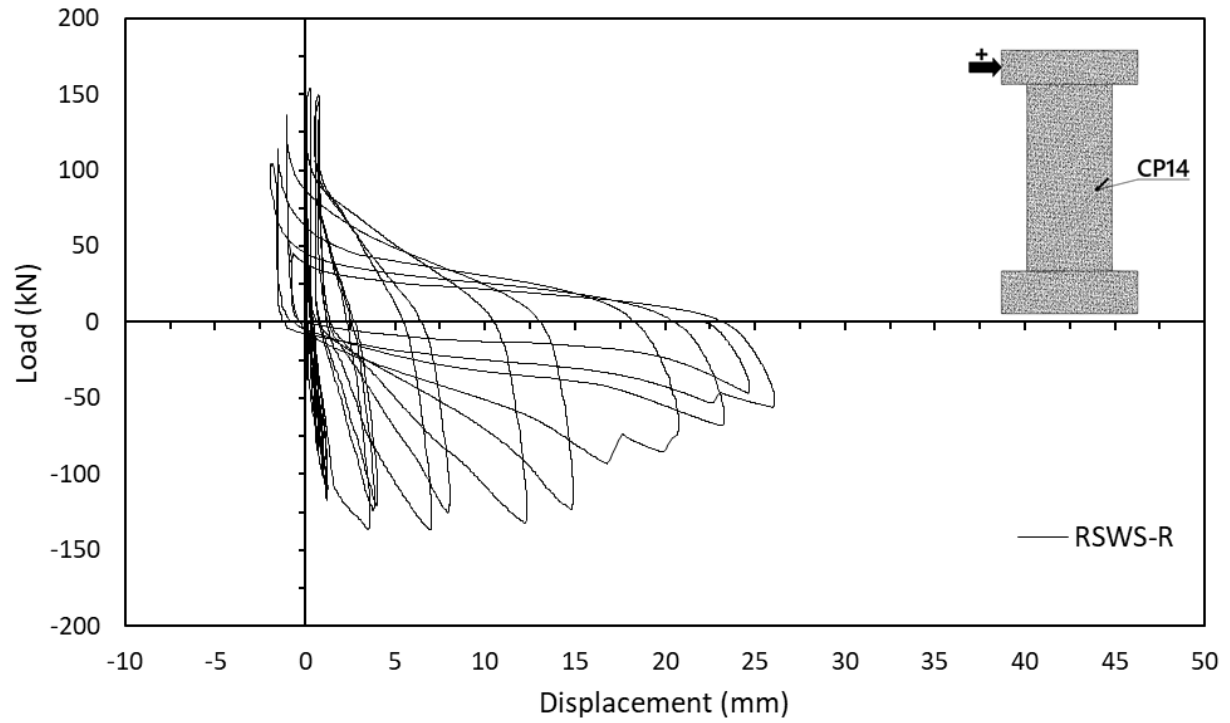


Figure A.25 Load-deformation response of CP14 in wall RSWS-R

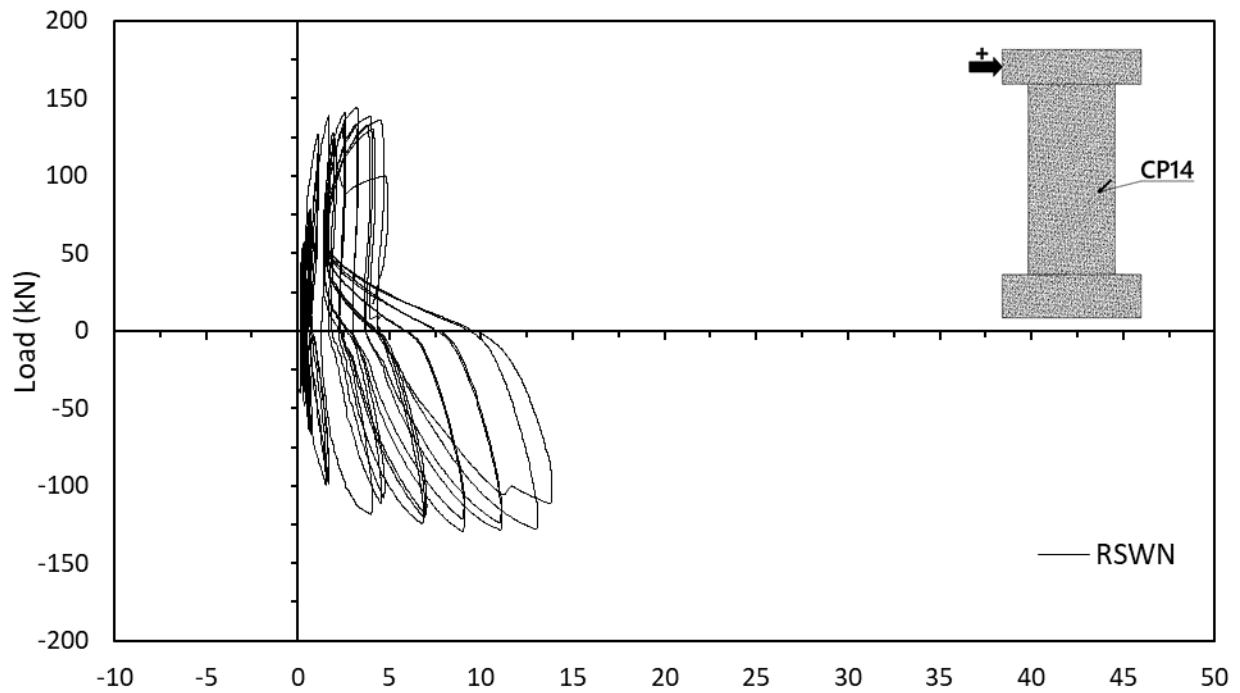


Figure A.26 Load-deformation response of CP14 in wall RSWN

A.4 Sliding and Uplift of Foundation Block

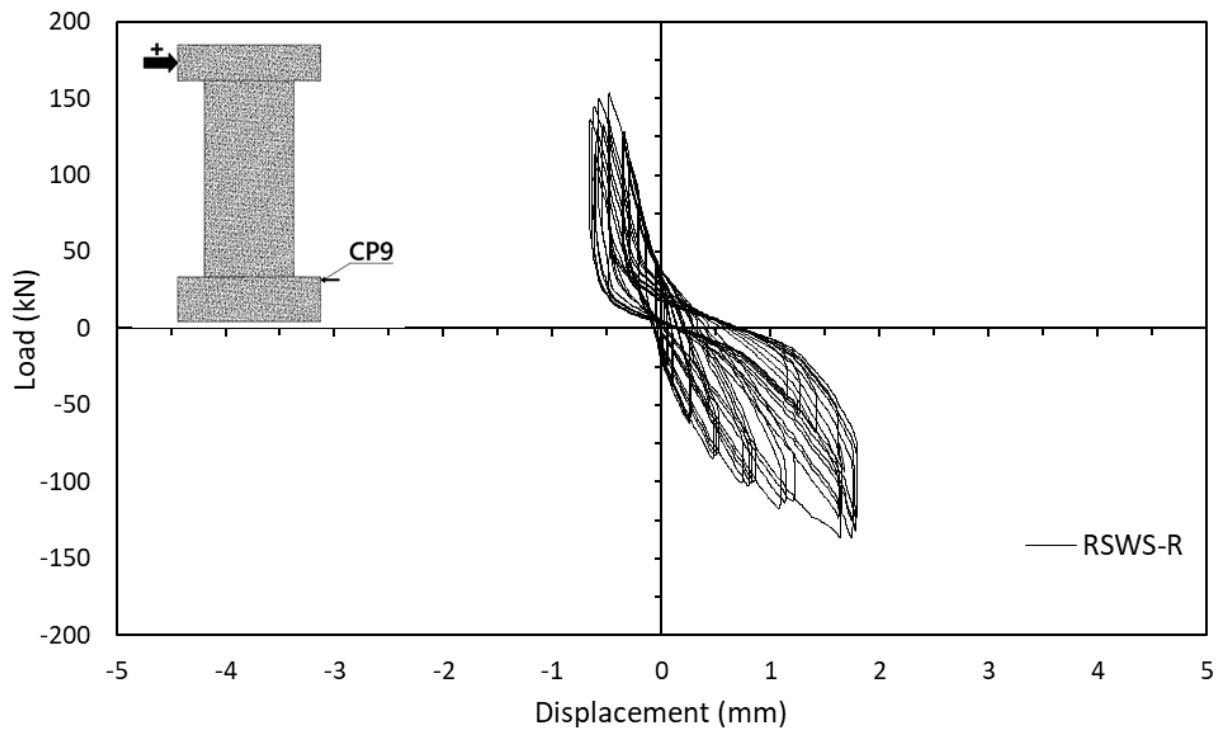


Figure A.27 Load-deformation response of CP9 in wall RSW-R

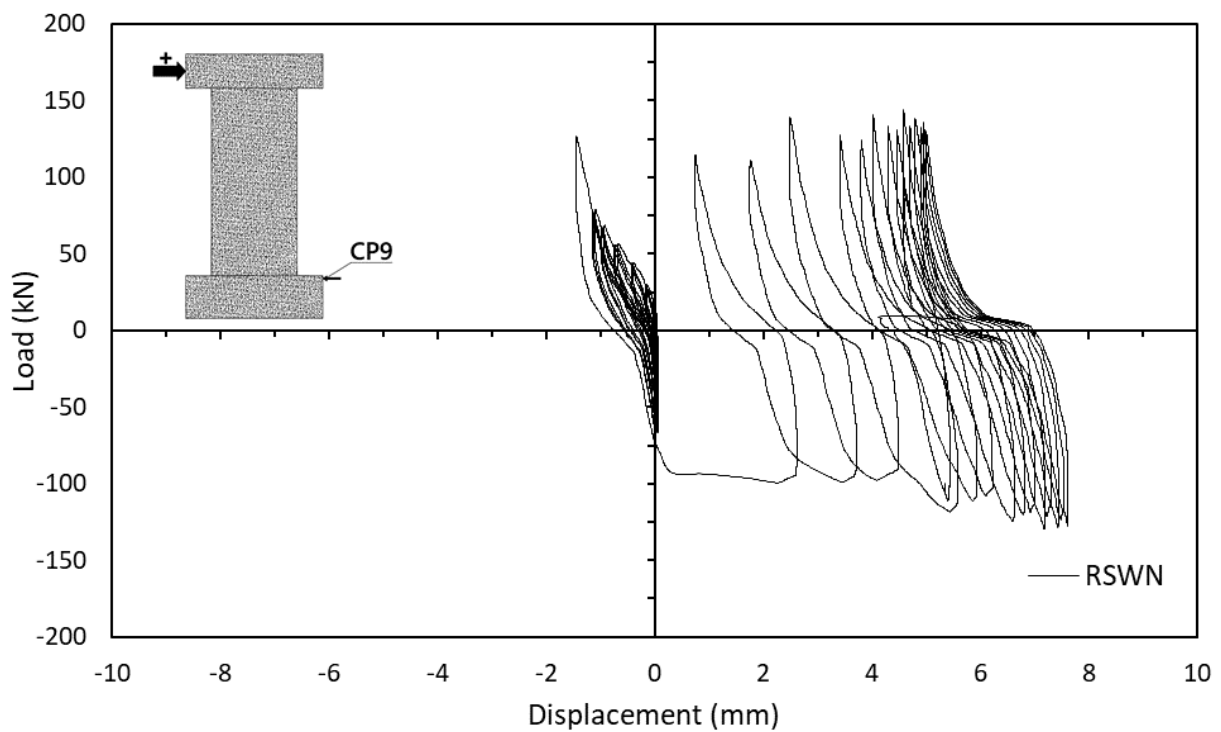


Figure A.28 Load-deformation response of CP9 in wall RSWN

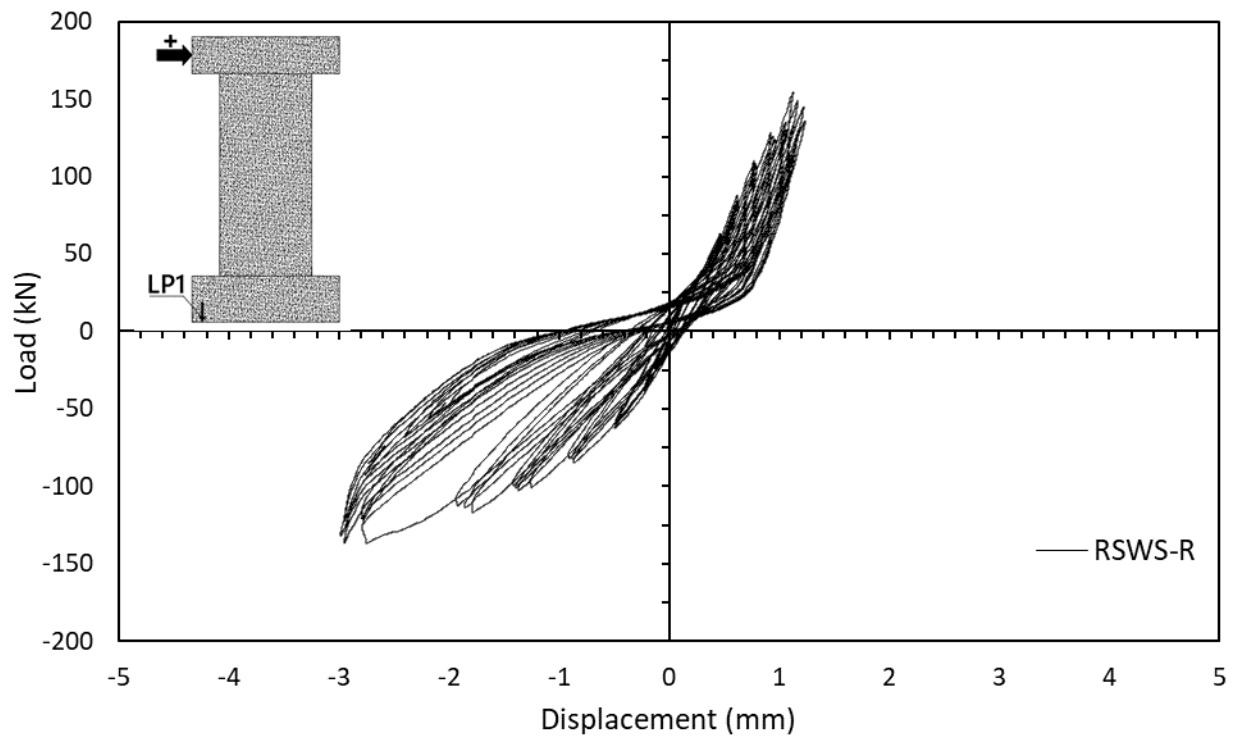


Figure A.29 Load-deformation response of LP1 in wall RSWS-R

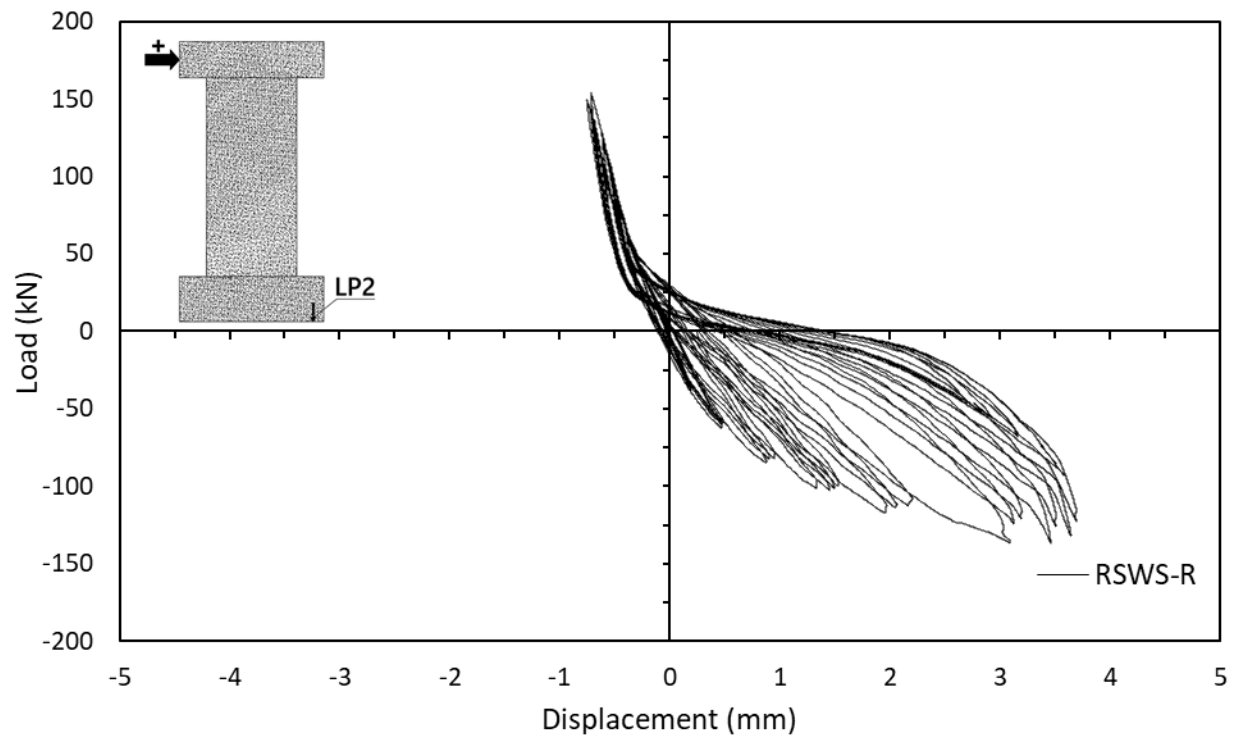


Figure A.30 Load-deformation response of LP2 in wall RSWN

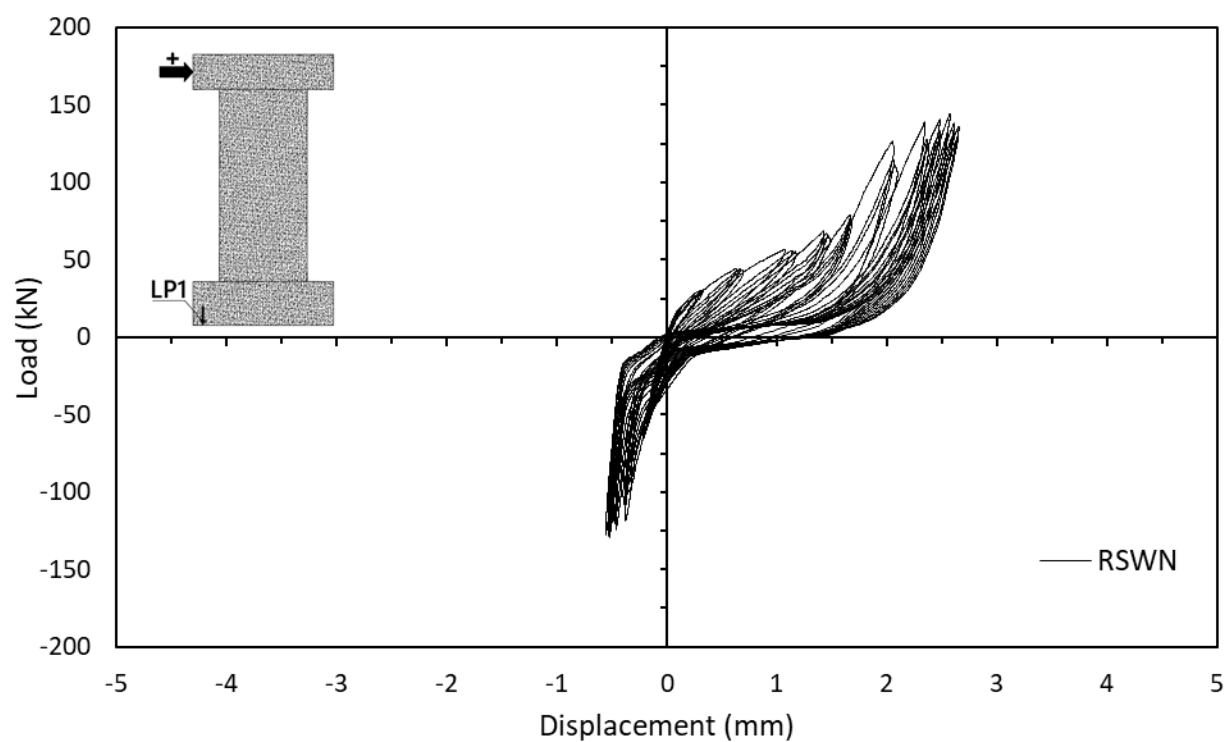


Figure A.31 Load-deformation response of LP1 in wall RSWN

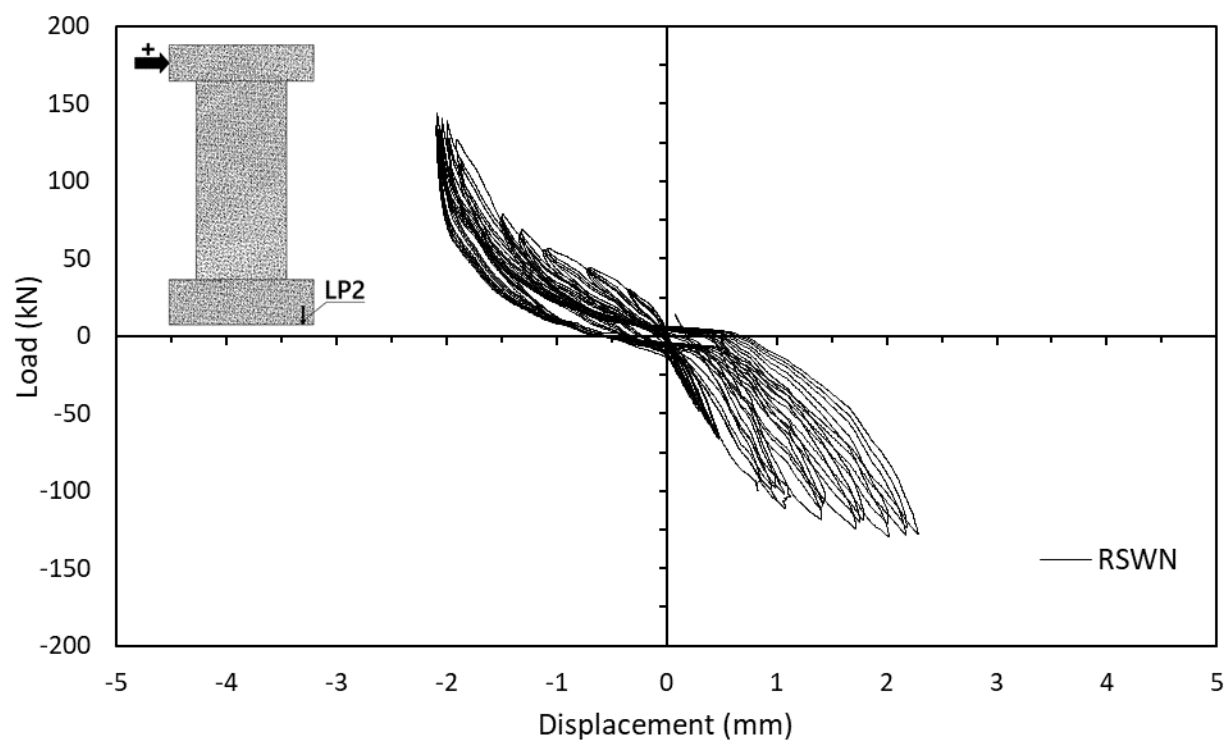


Figure A.32 Load-deformation response of LP2 in wall RSWN

Appendix B. Strain gauges' Response

B.1 Strain Gauges Used in Wall RSWS-R

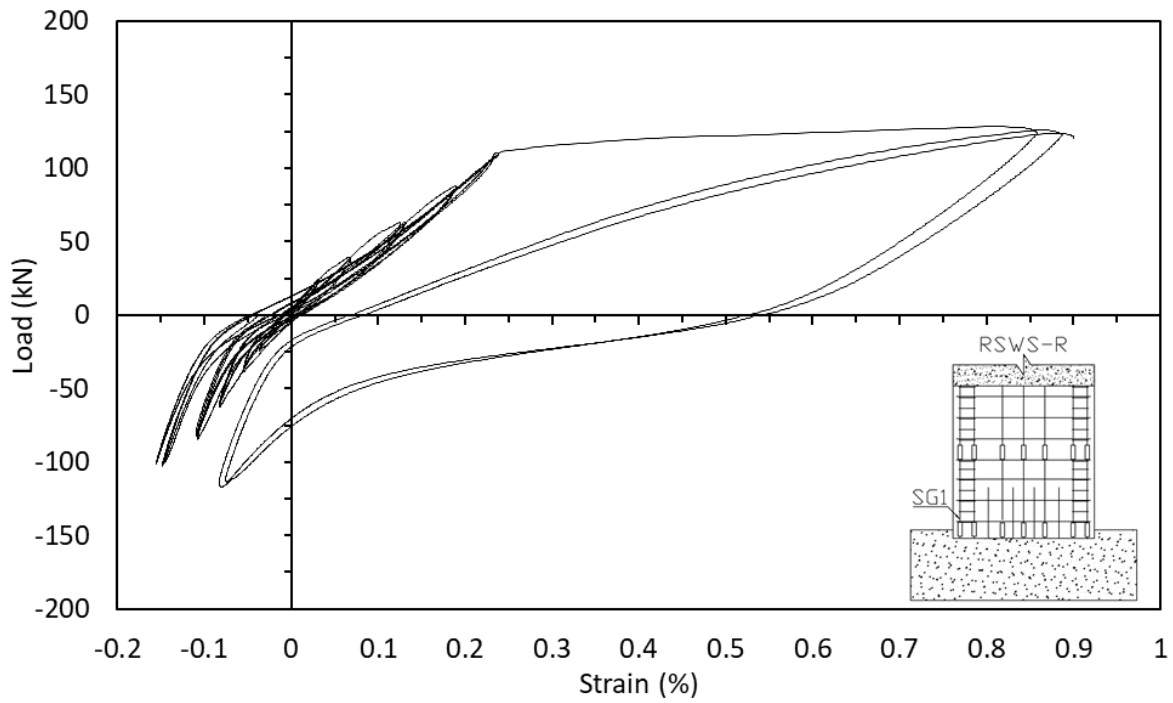


Figure B.1 SG1 Response for wall RSWS-R

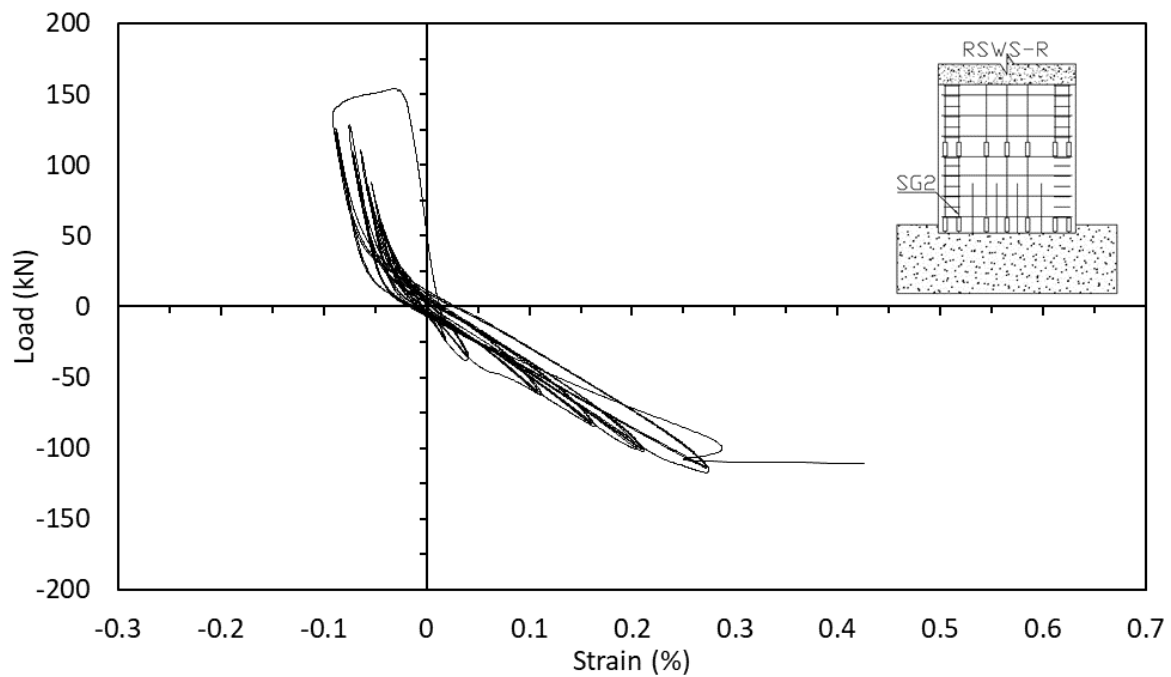


Figure B.2 SG2 Response for wall RSWS-R

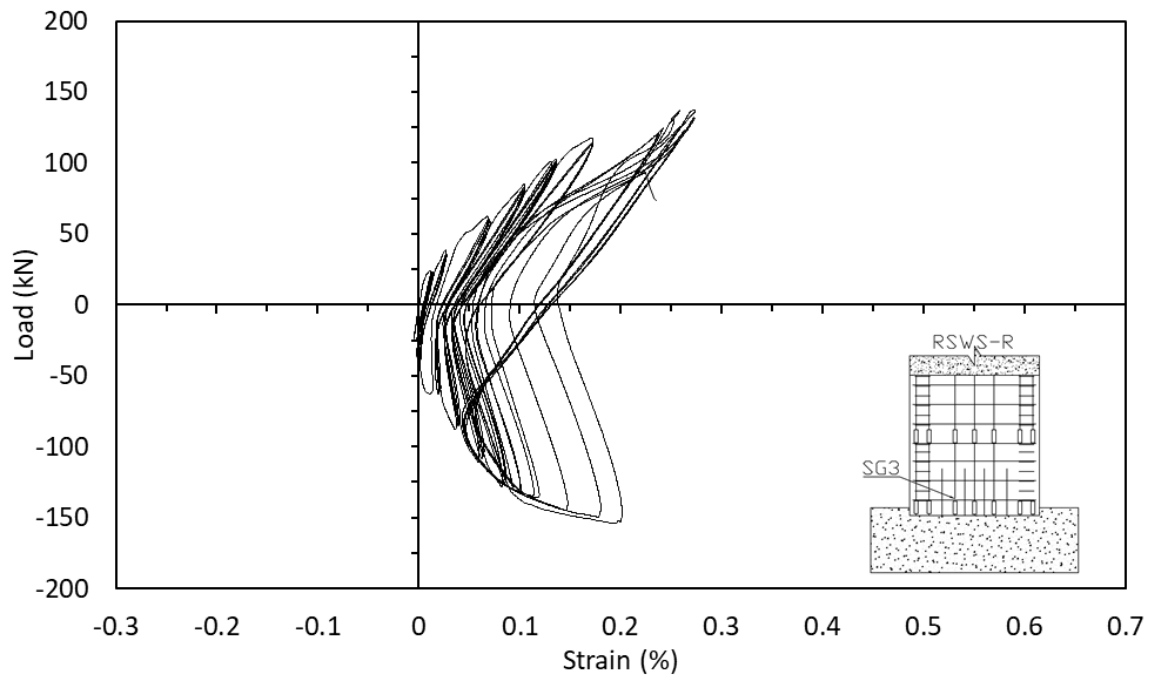


Figure B.3 SG3 Response for wall RWS-R

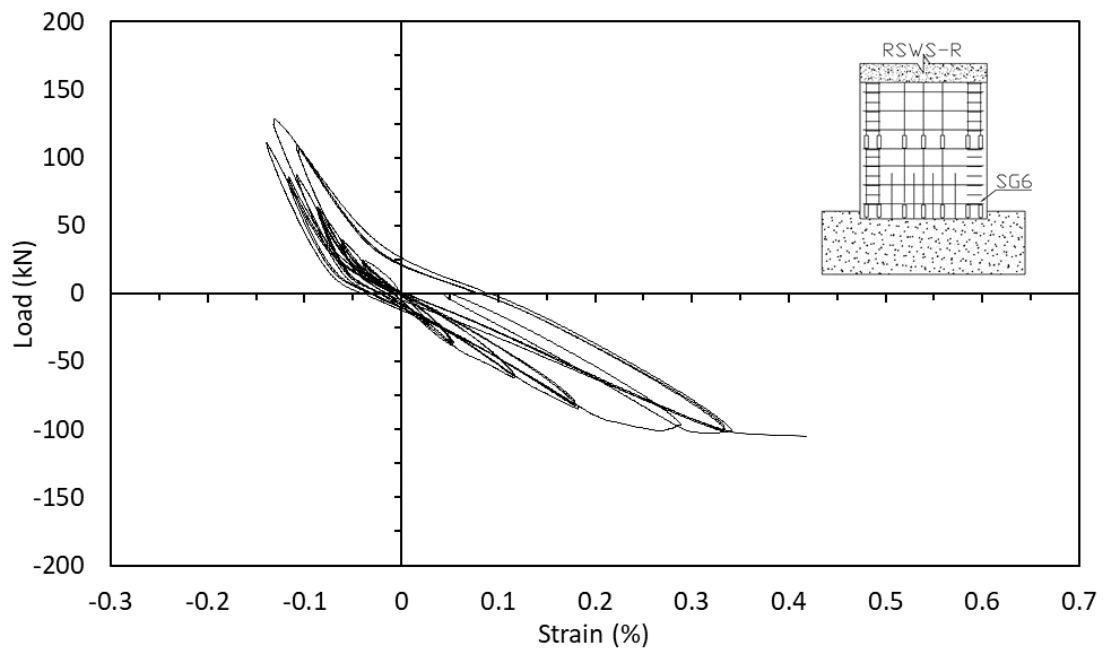


Figure B.4 SG6 Response for wall RWS-R

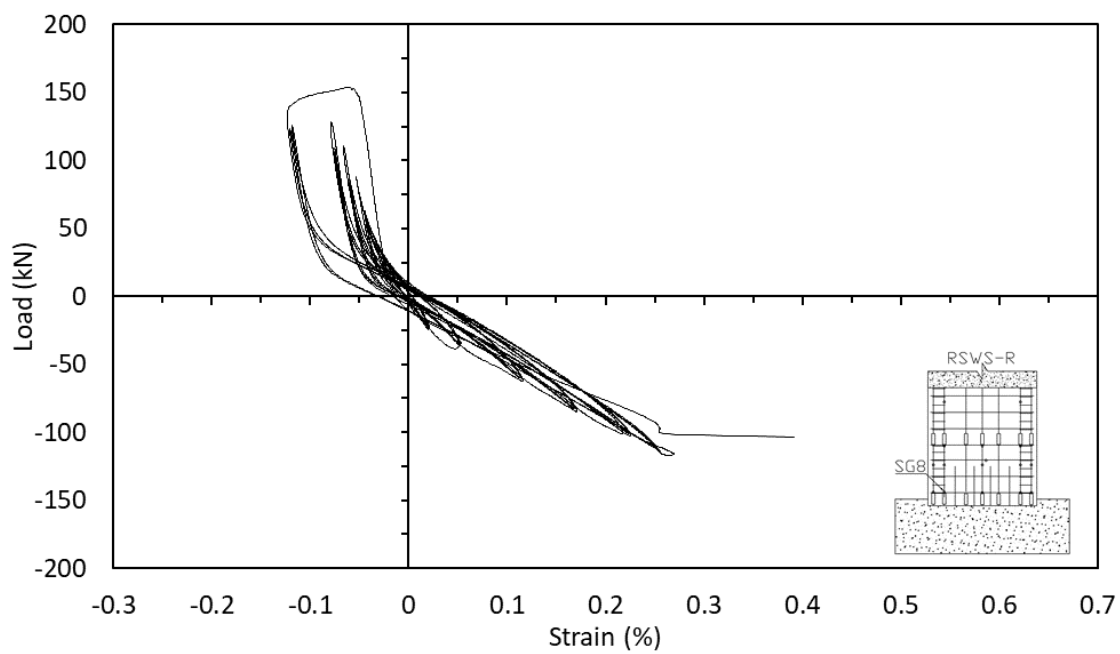


Figure B.5 SG8 Response for wall RWS-R

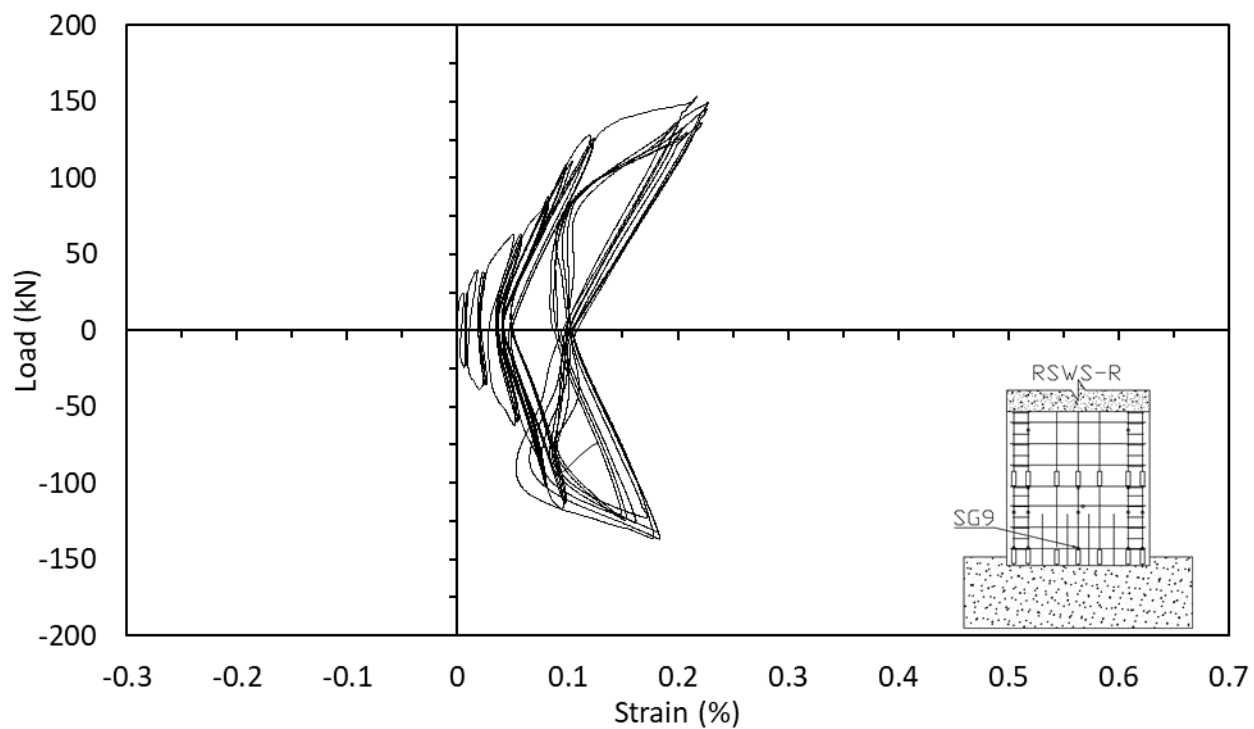


Figure B.6 SG9 Response for wall RWS-R

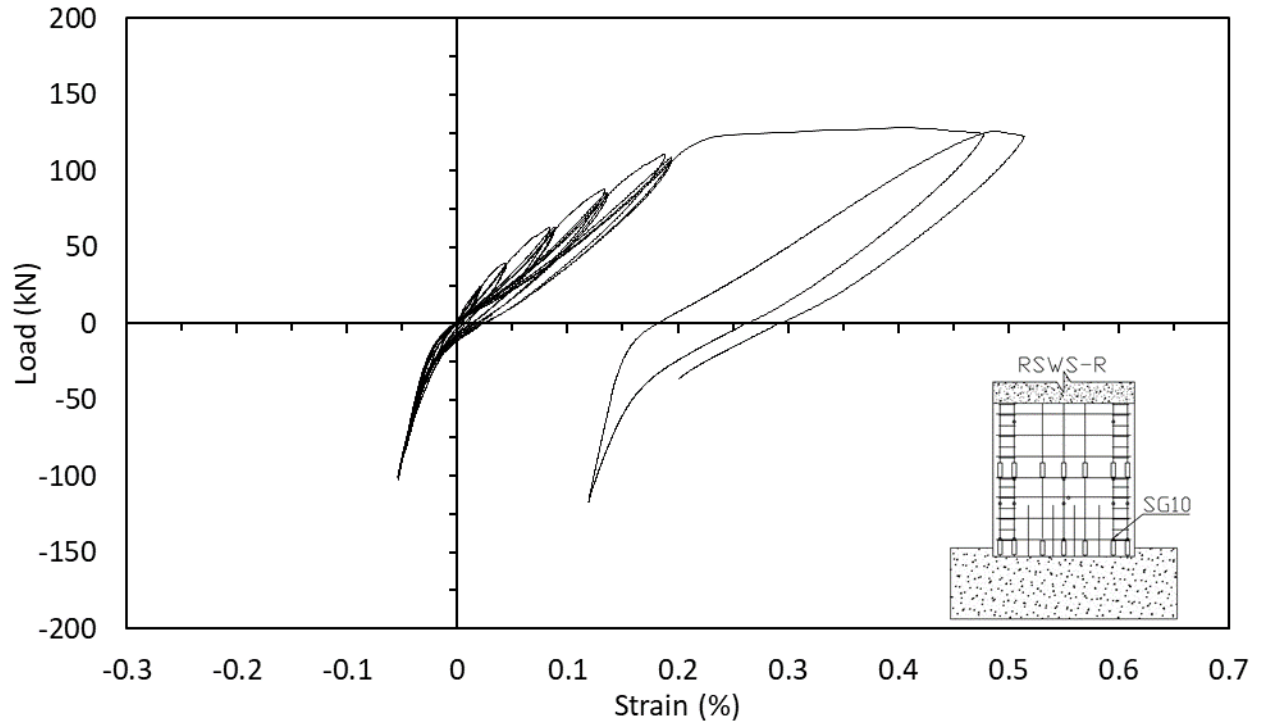


Figure B.7 SG10 Response for wall RSWs-R

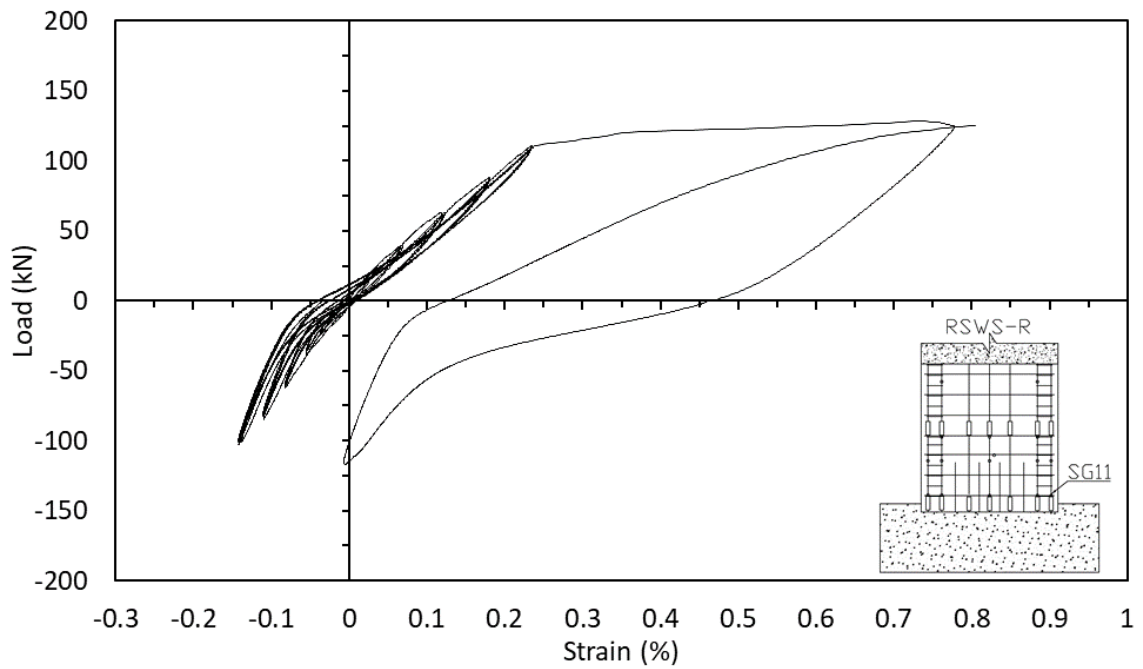


Figure B.8 SG11 Response for wall RSWs-R

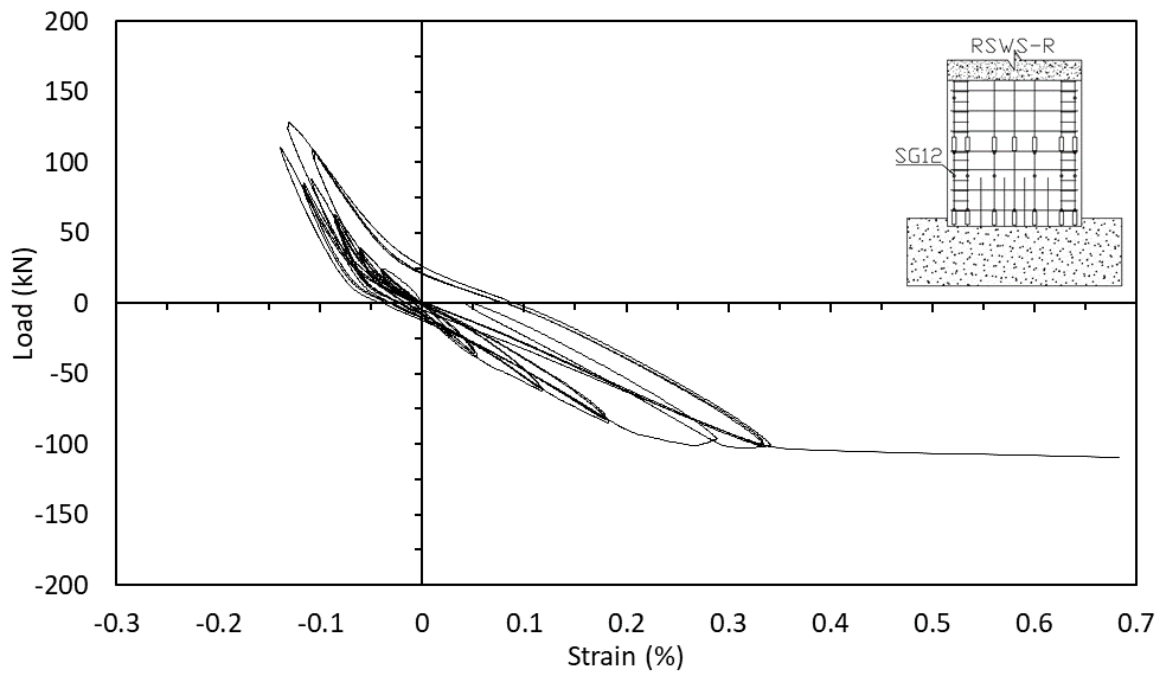


Figure B.9 SG12 Response for wall RSWs-R

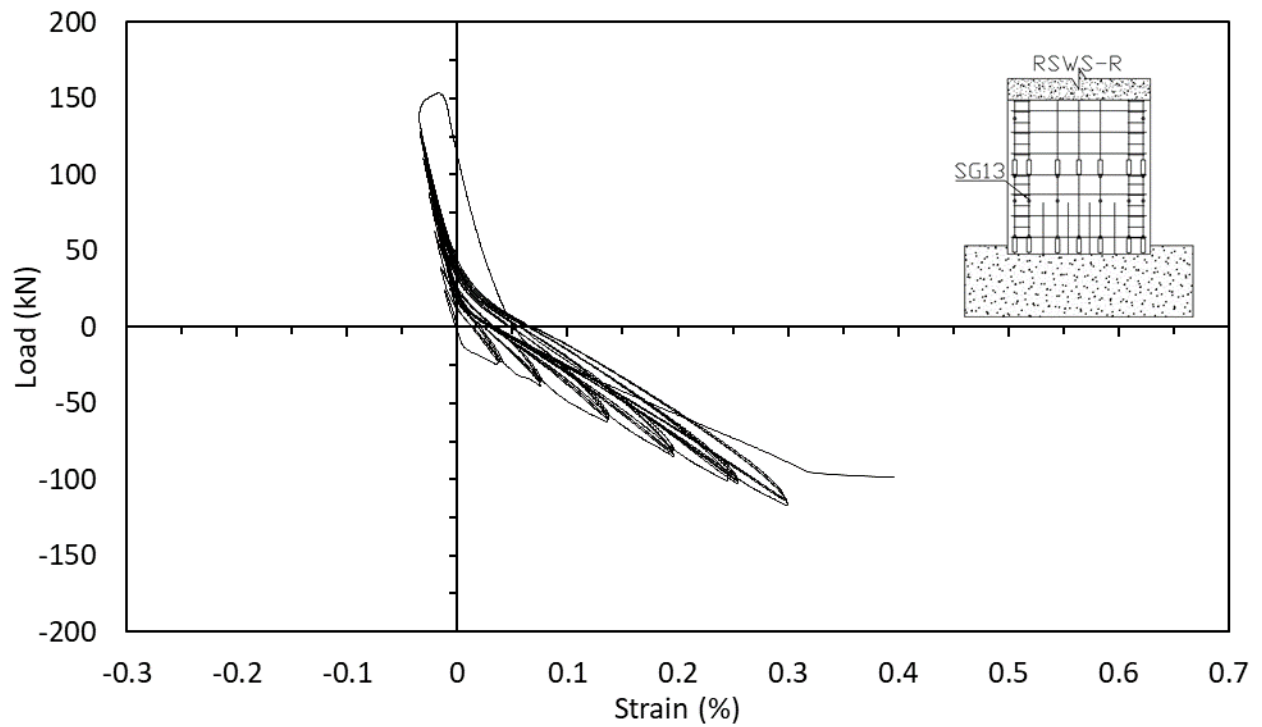


Figure B.10 SG13 Response for wall RSWs-R

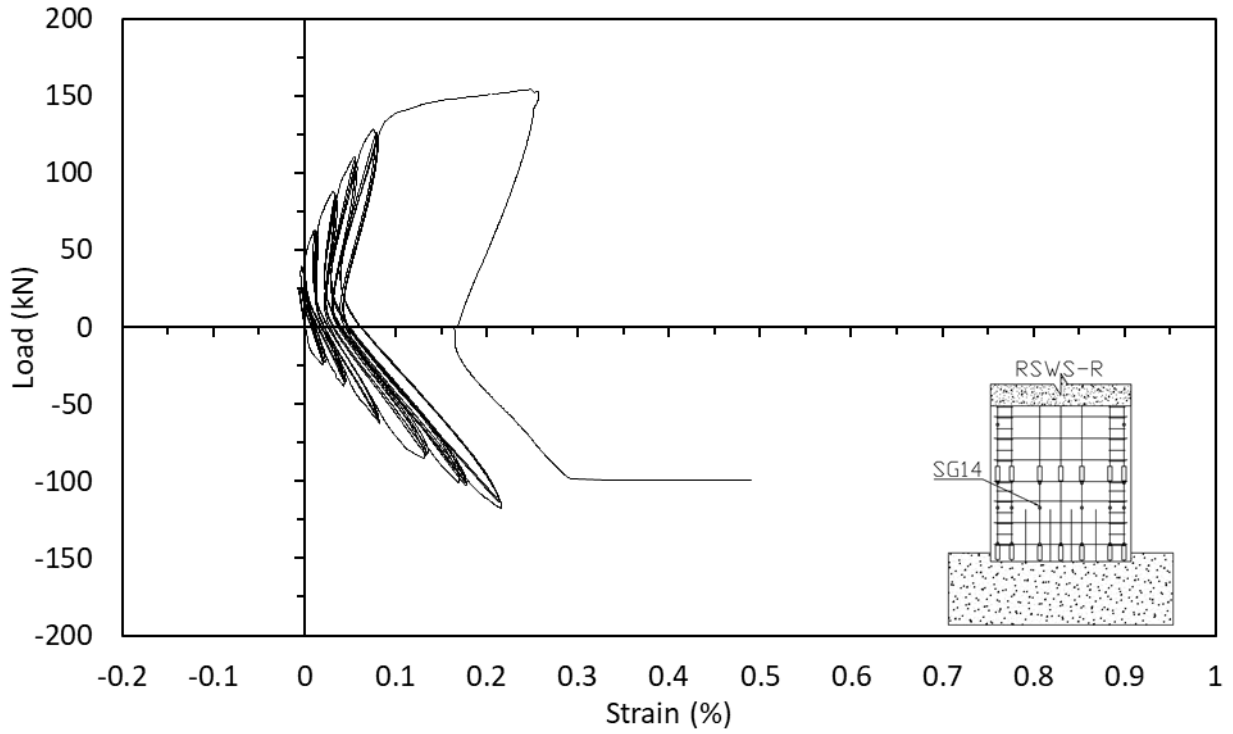


Figure B.11 SG14 Response for wall RSWs-R

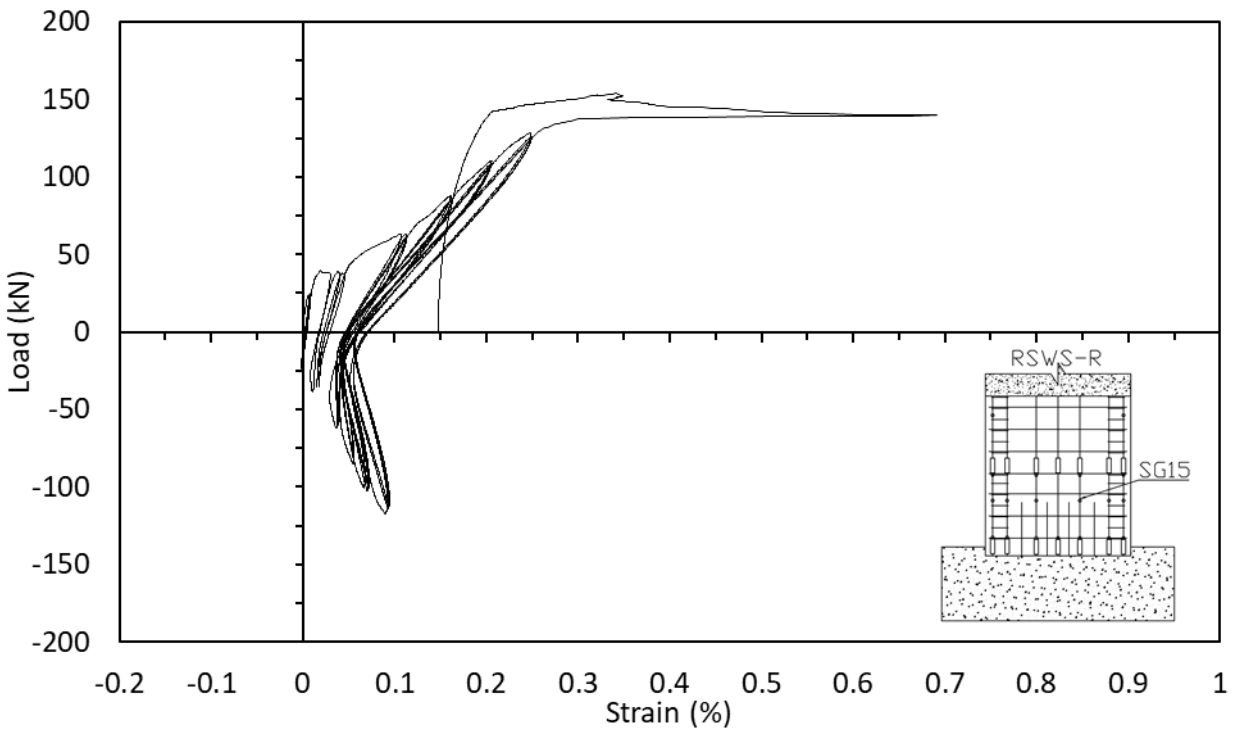


Figure B.12 SG15 Response for wall RSWs-R

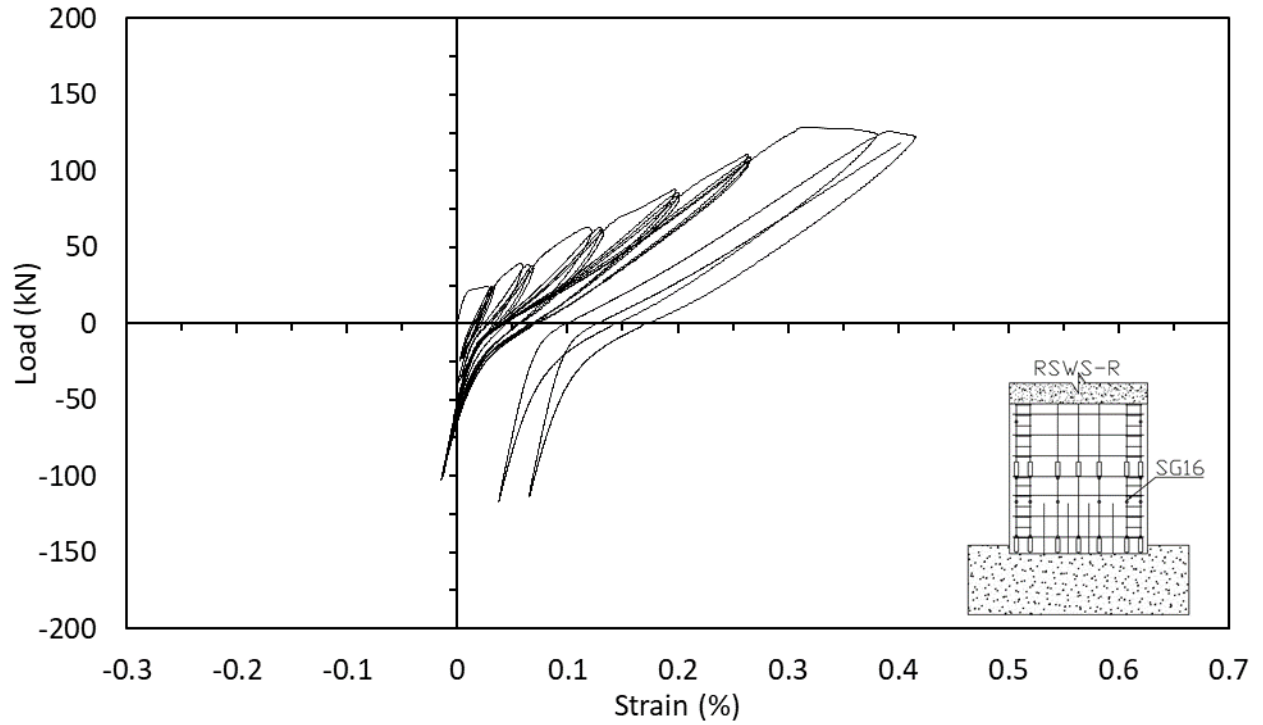


Figure B.13 SG16 Response for wall RSW-S-R

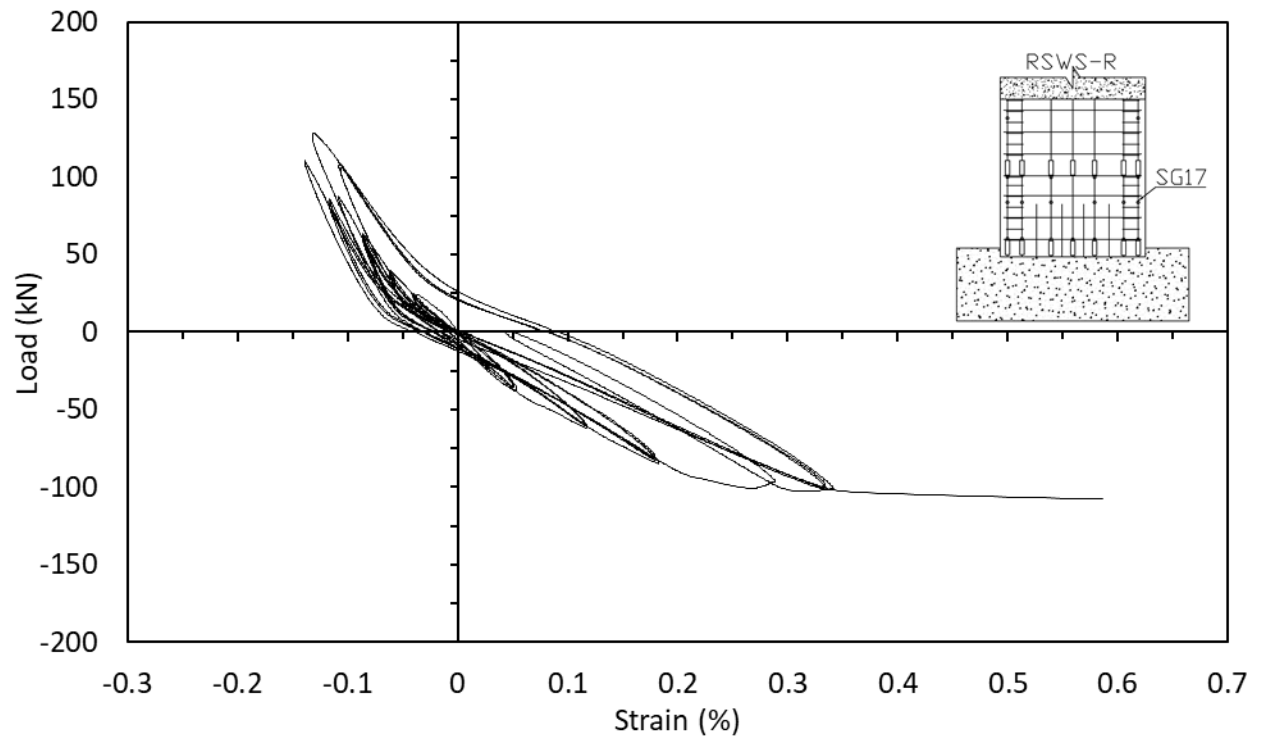


Figure B.14 SG17 Response for wall RSW-S-R

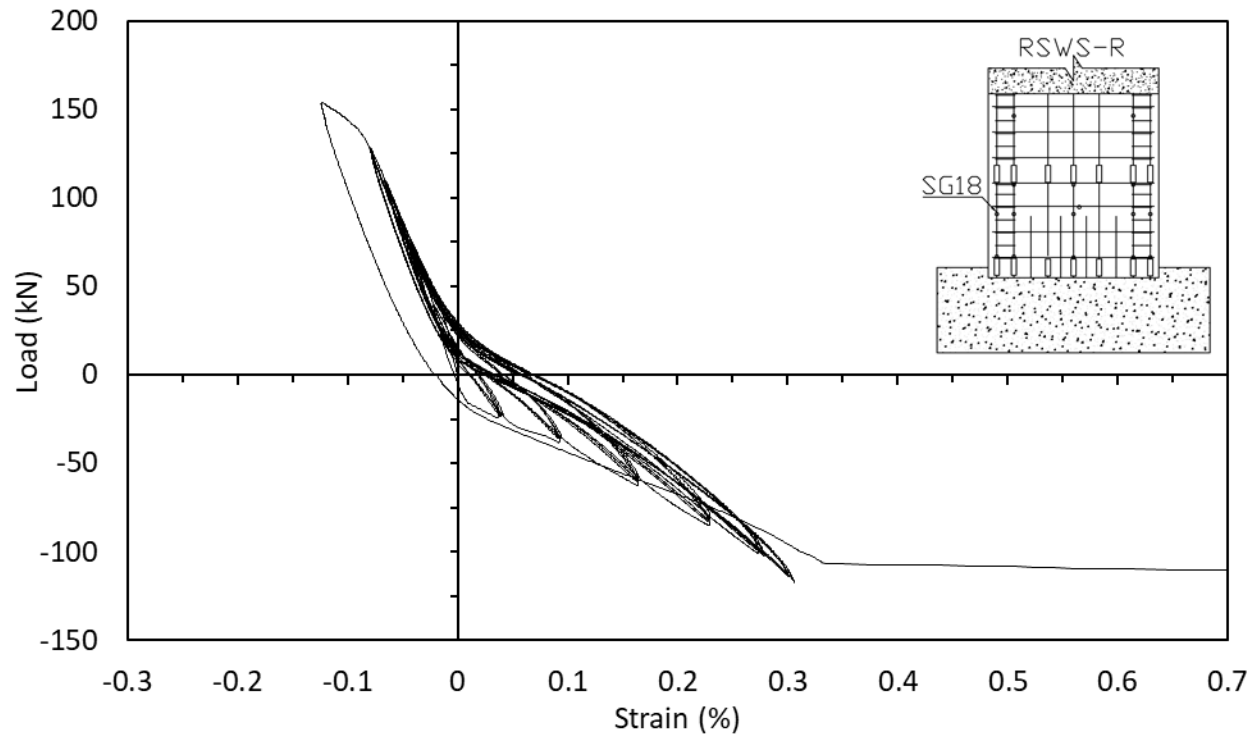


Figure B.15 SG18 Response for wall RSWs-R

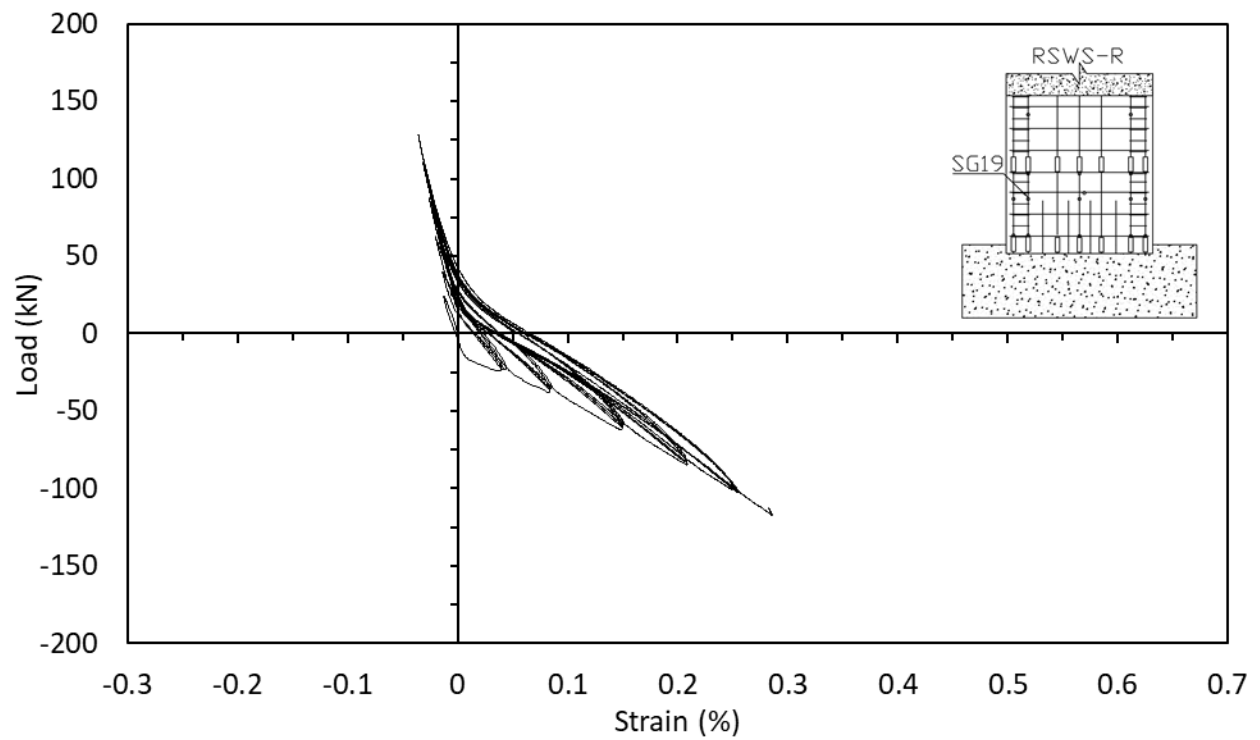


Figure B.16 SG19 Response for wall RSWs-R

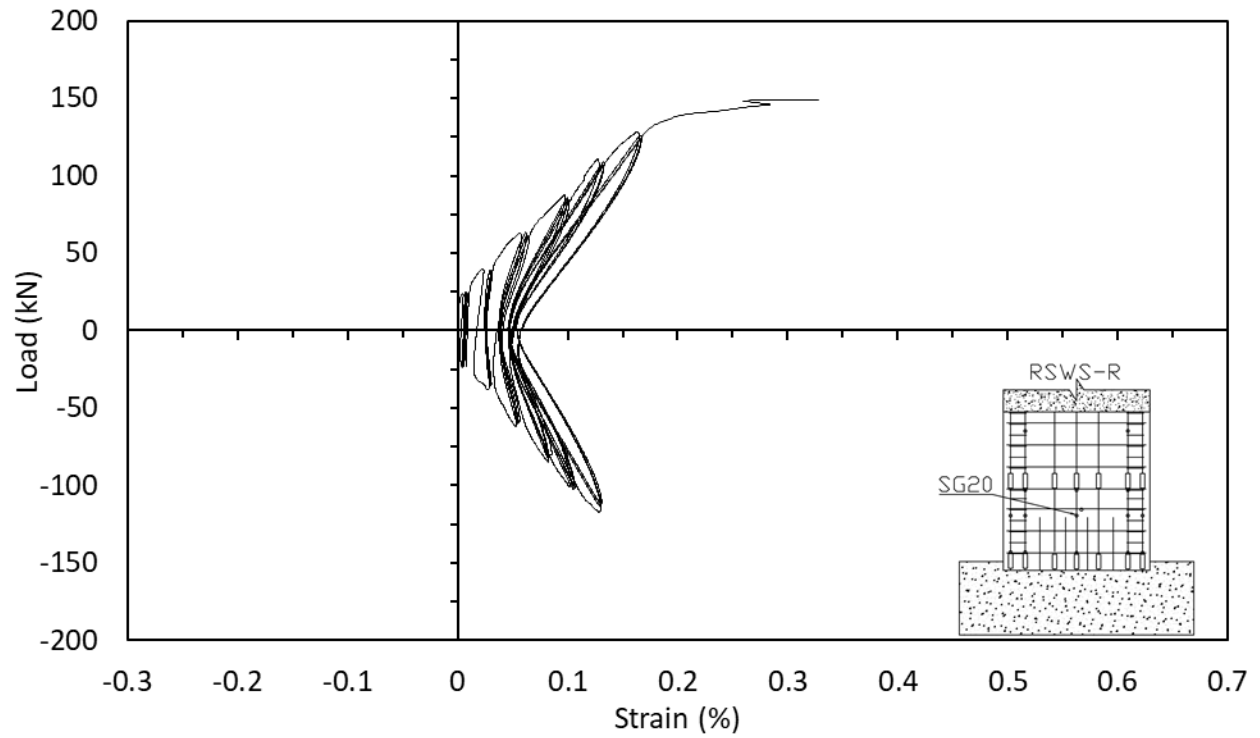


Figure B.17 SG20 Response for wall RWS-R

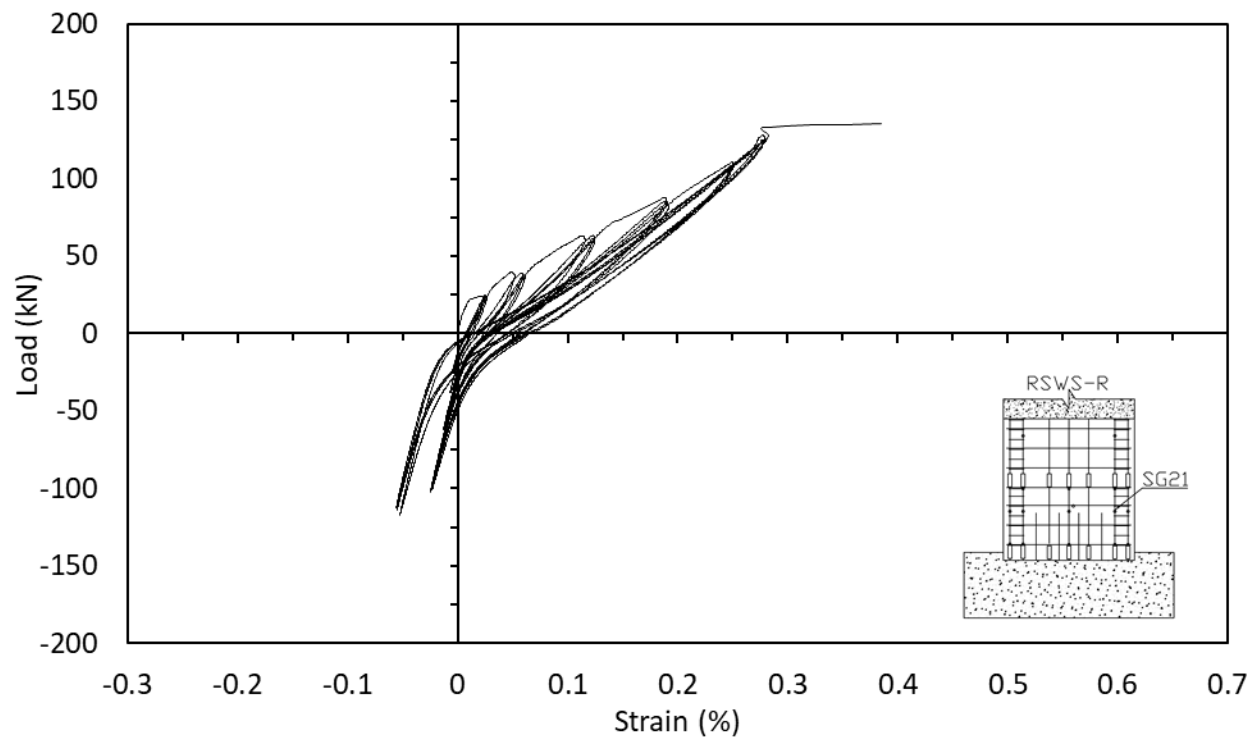


Figure B.18 SG21 Response for wall RWS-R

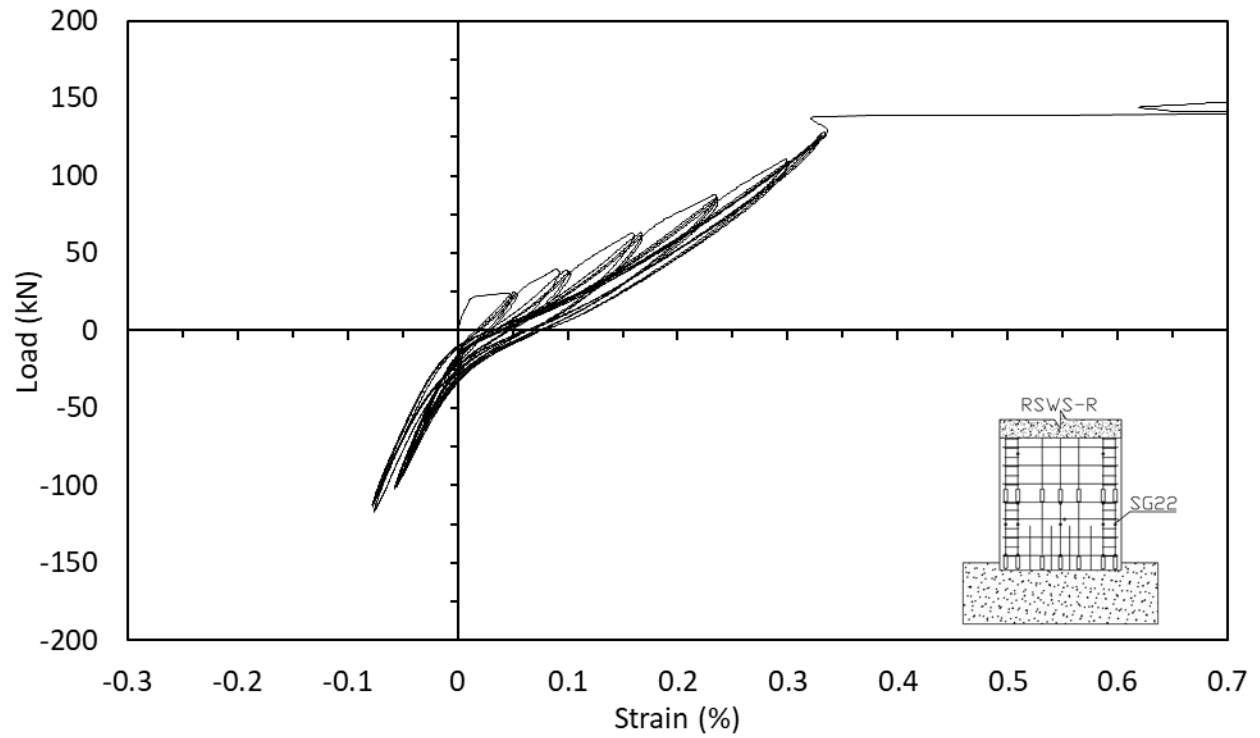


Figure B.19 SG22 Response for wall RWS-R

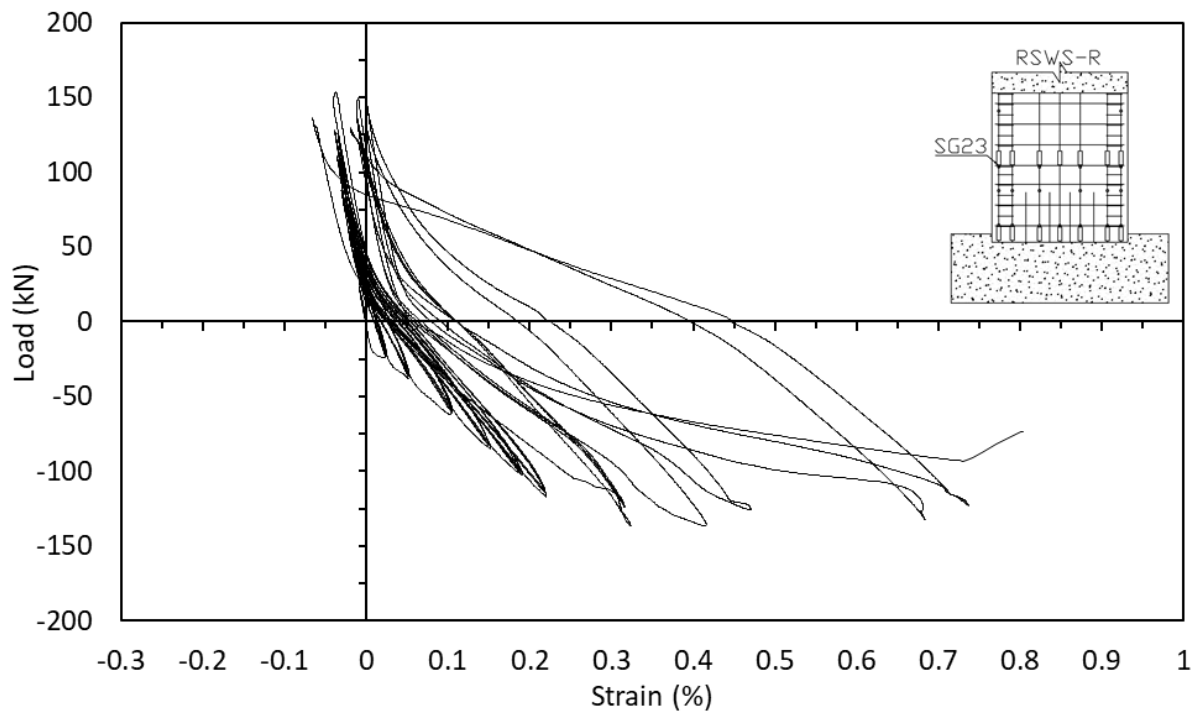


Figure B.20 SG23 Response for wall RWS-R

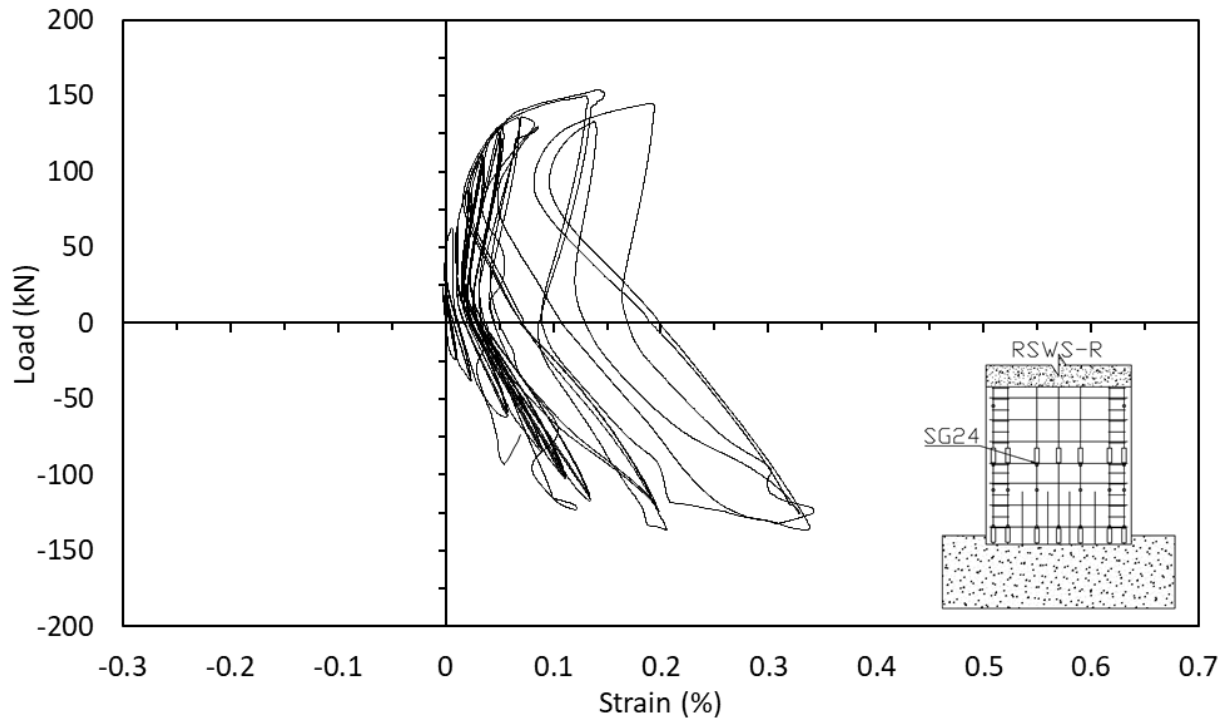


Figure B.21 SG24 Response for wall RSW-S-R

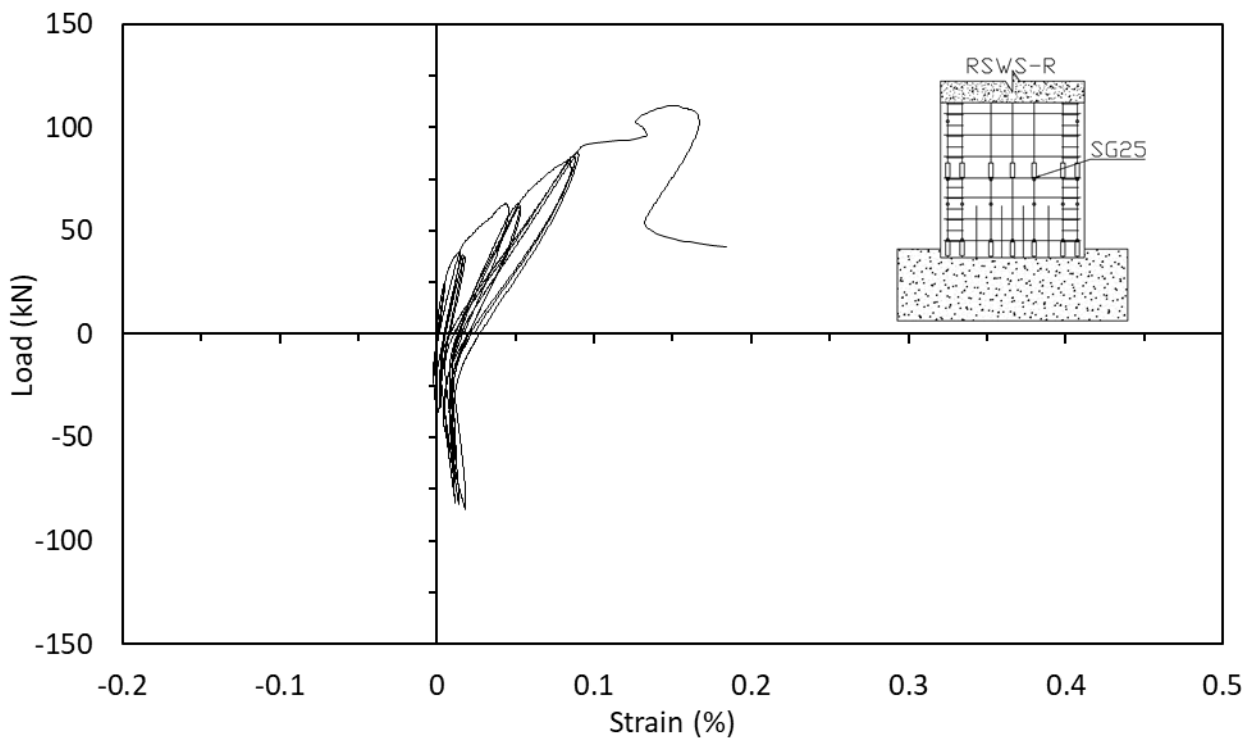


Figure B.22 SG25 Response for wall RSW-S-R

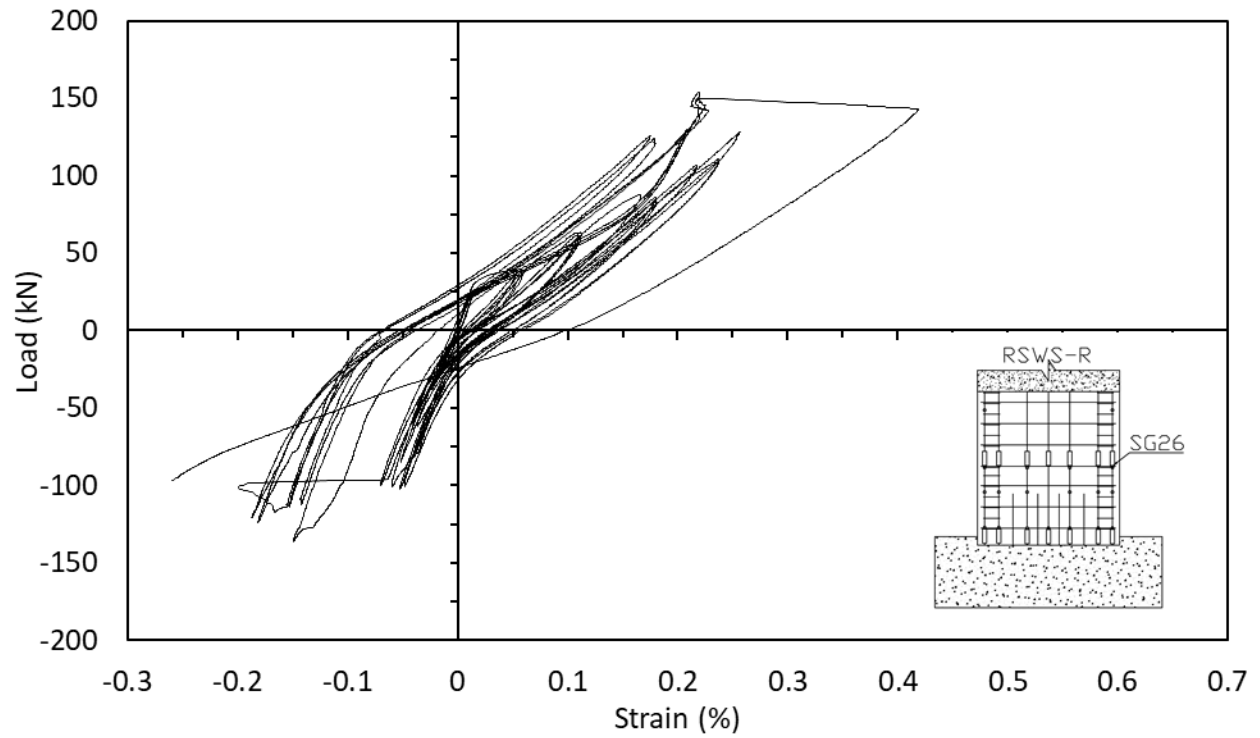


Figure B.23 SG26 Response for wall RSWs-R

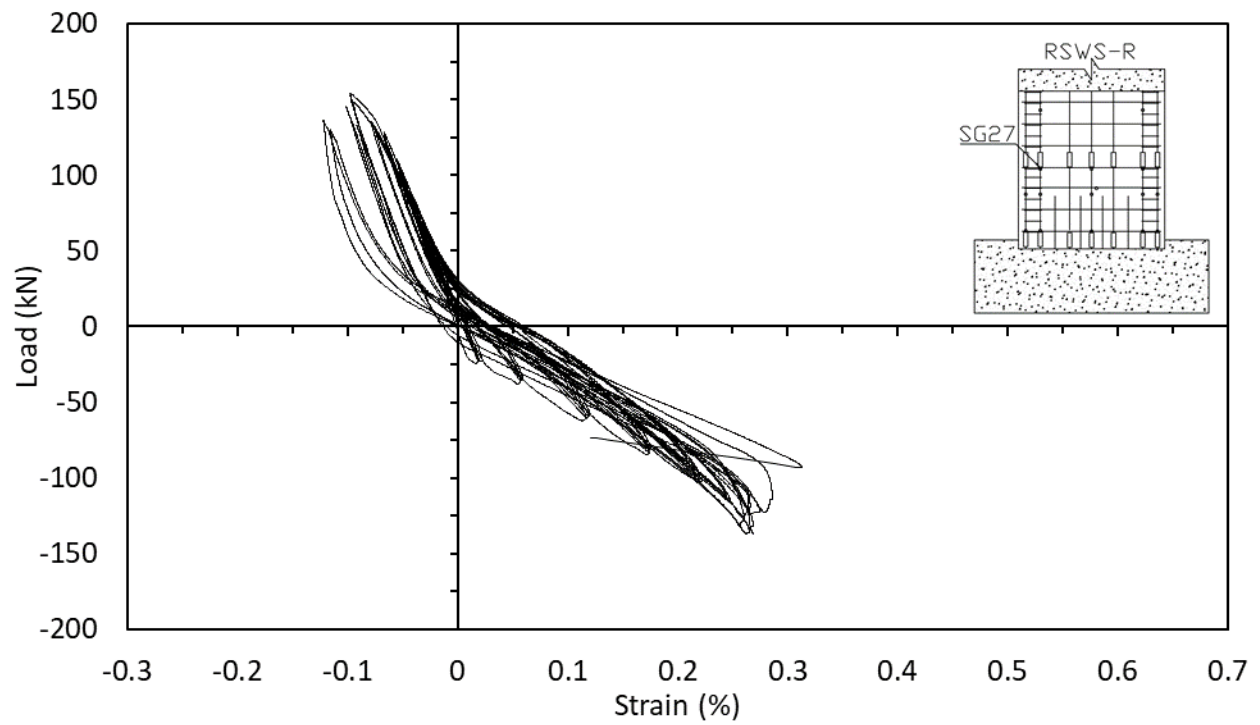


Figure B.24 SG27 Response for wall RSWs-R

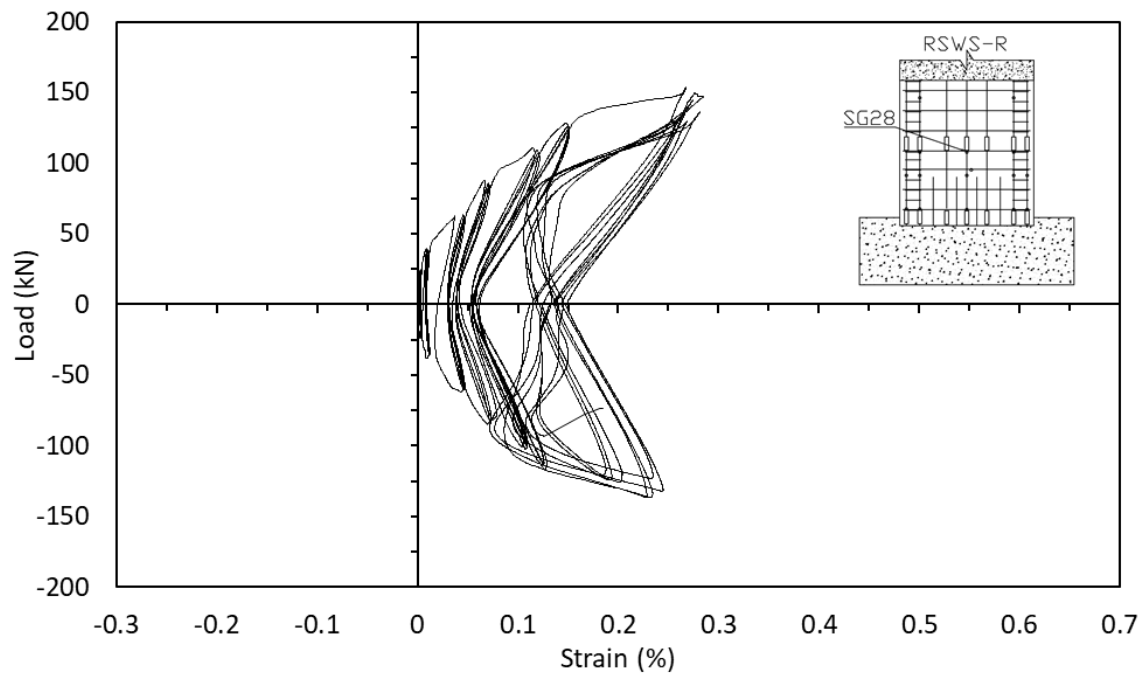


Figure B.25 SG28 Response for wall RSWs-R

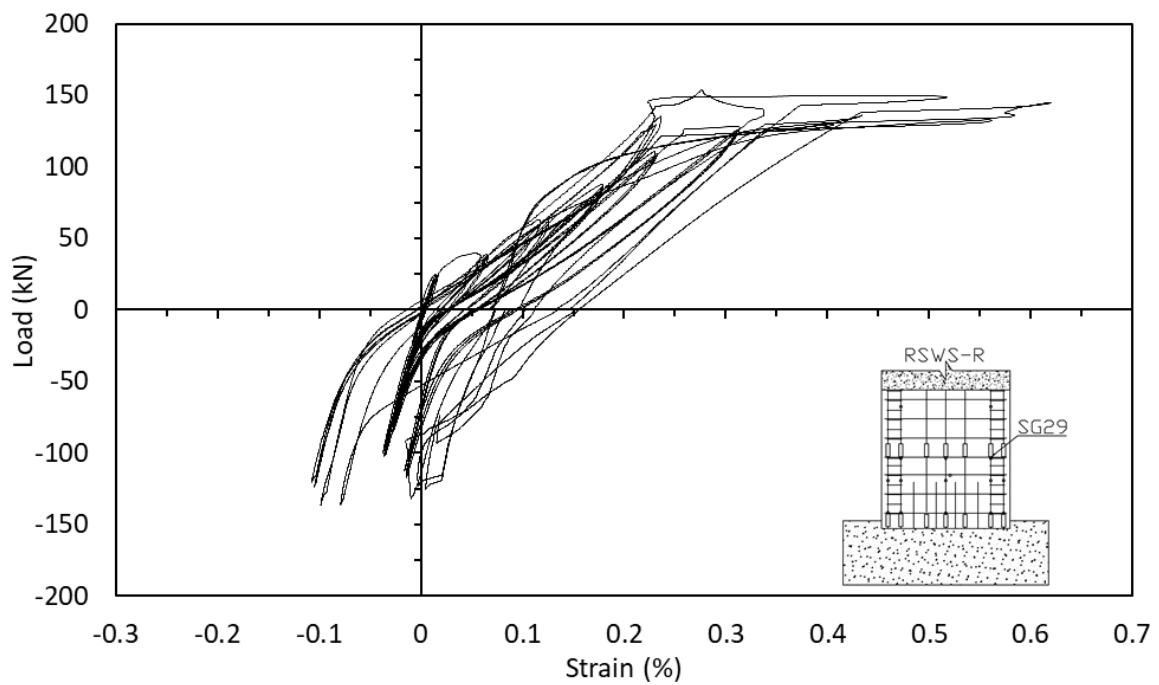


Figure B.26 SG29 Response for wall RSWs-R

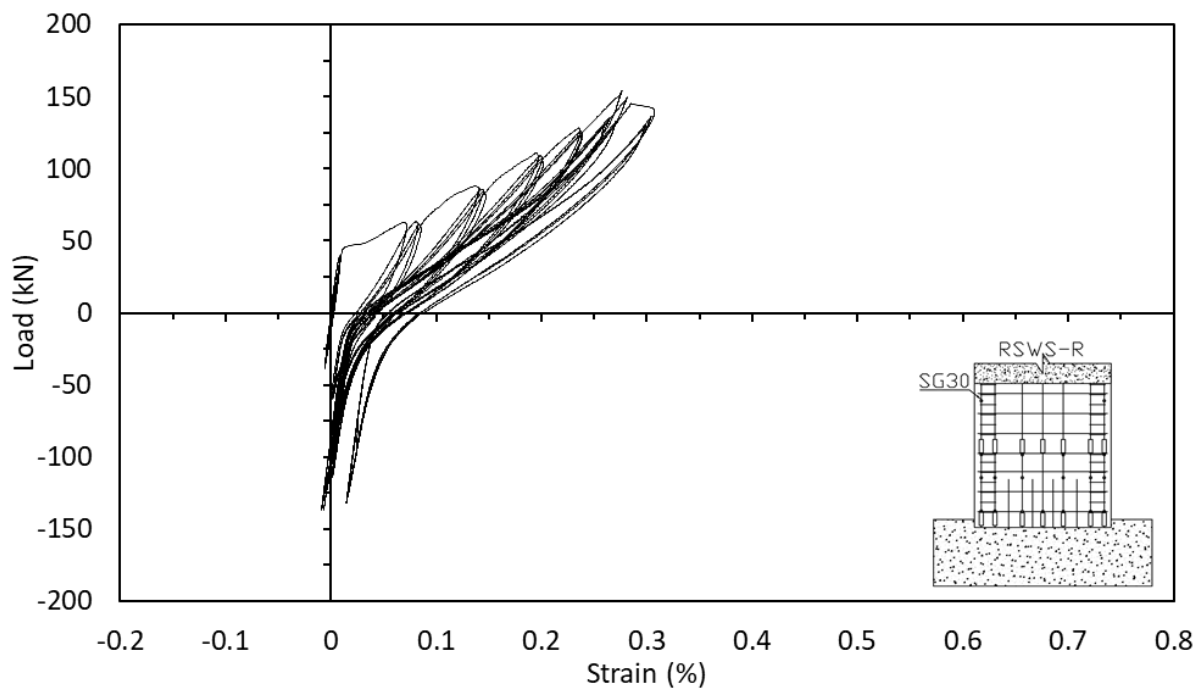


Figure B.27 SG30 Response for wall RWS-R

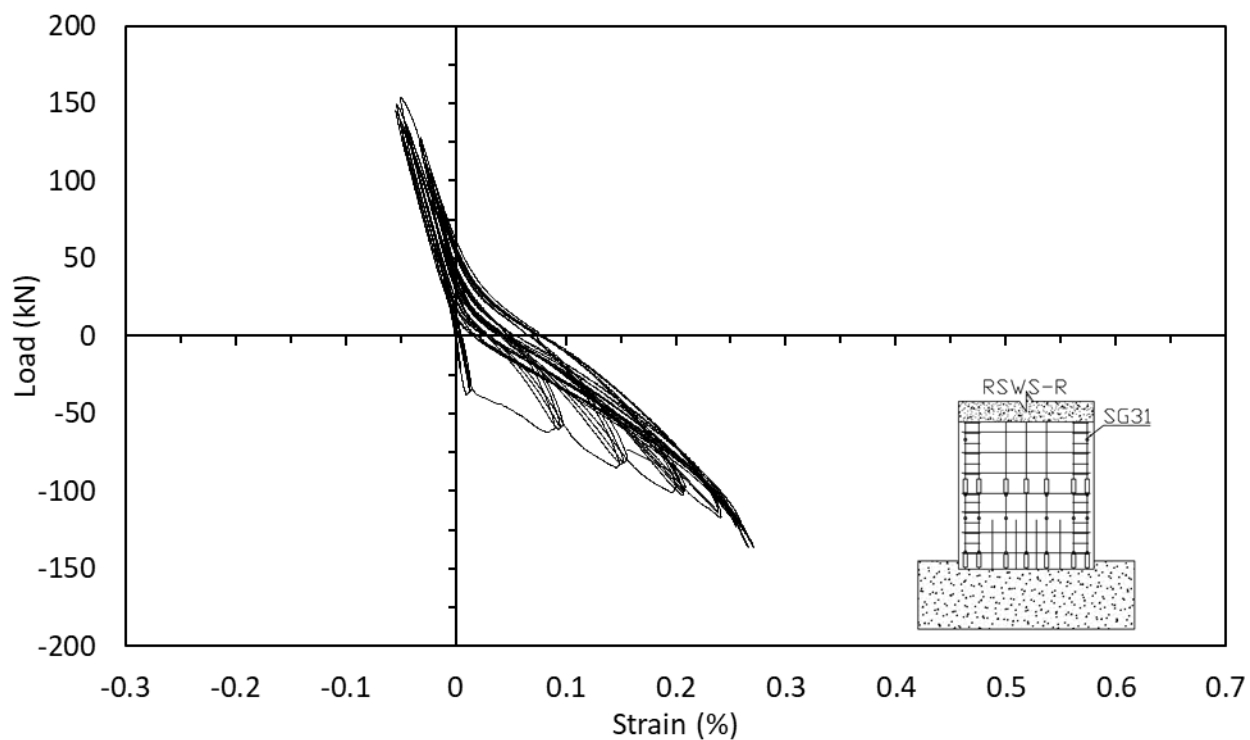


Figure B.28 SG31 Response for wall RWS-R

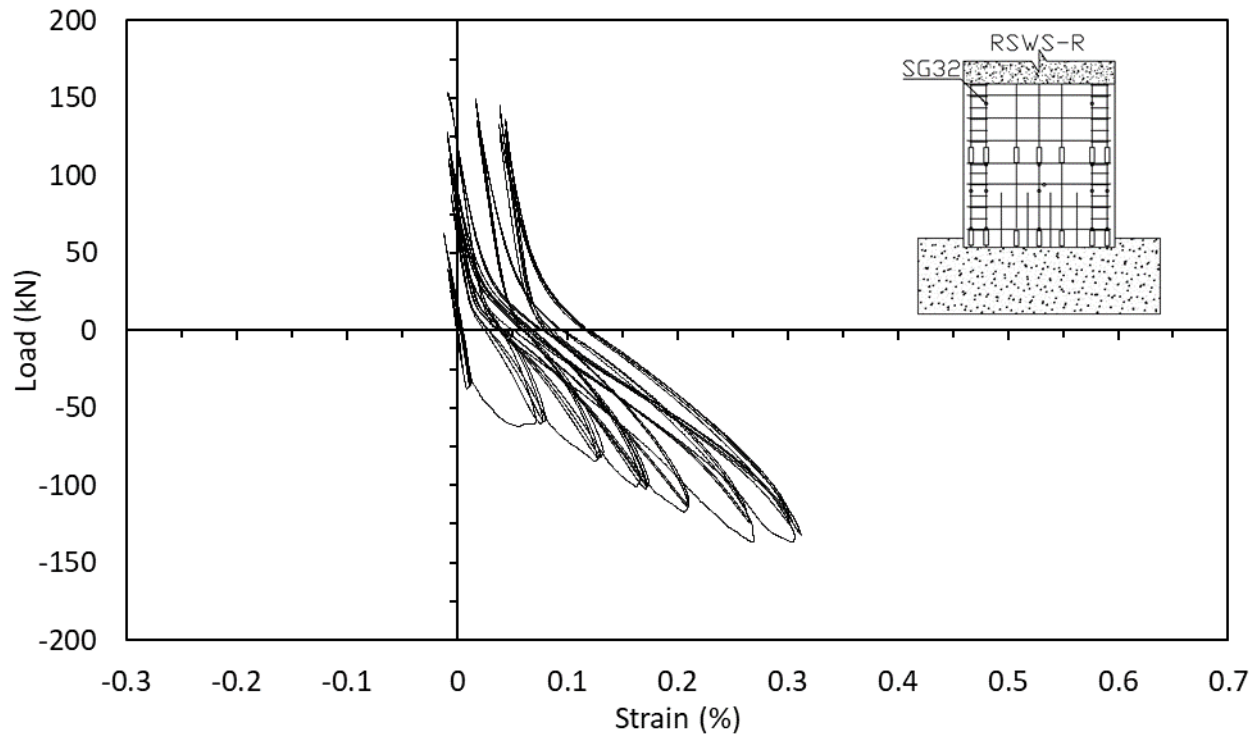


Figure B.29 SG32 Response for wall RWS-R

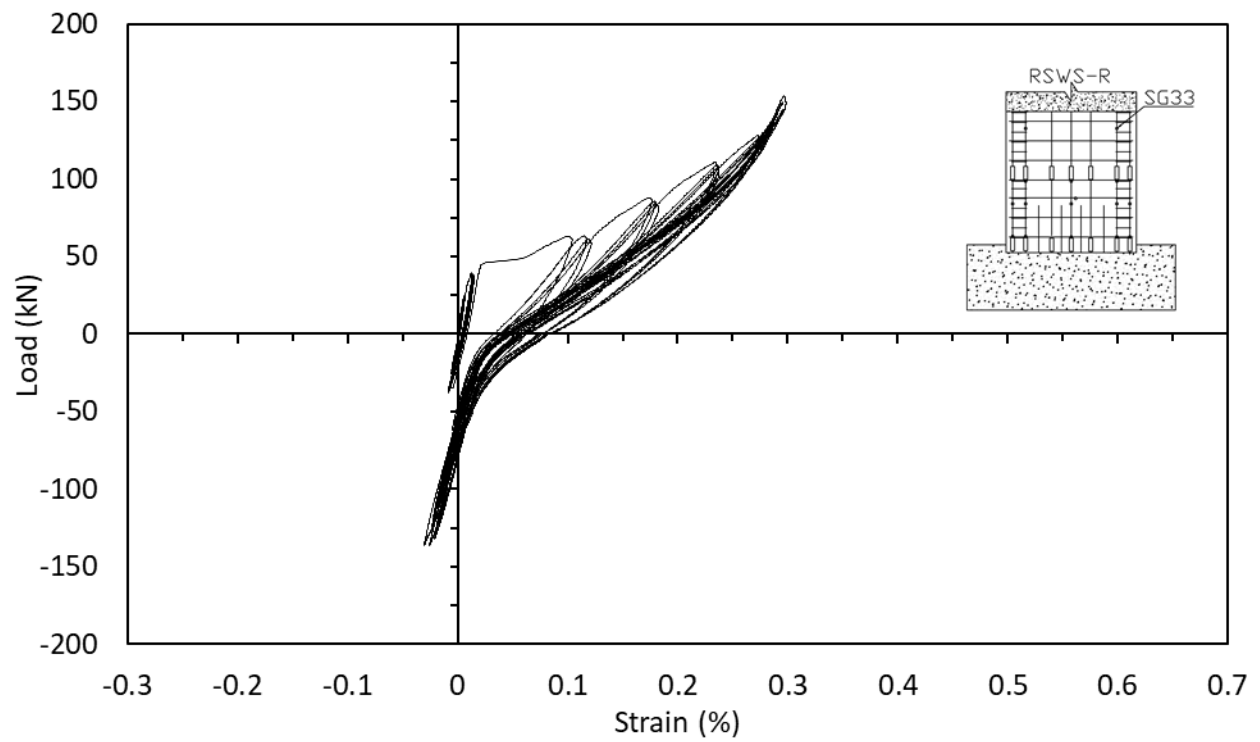


Figure B.30 SG33 Response for wall RWS-R

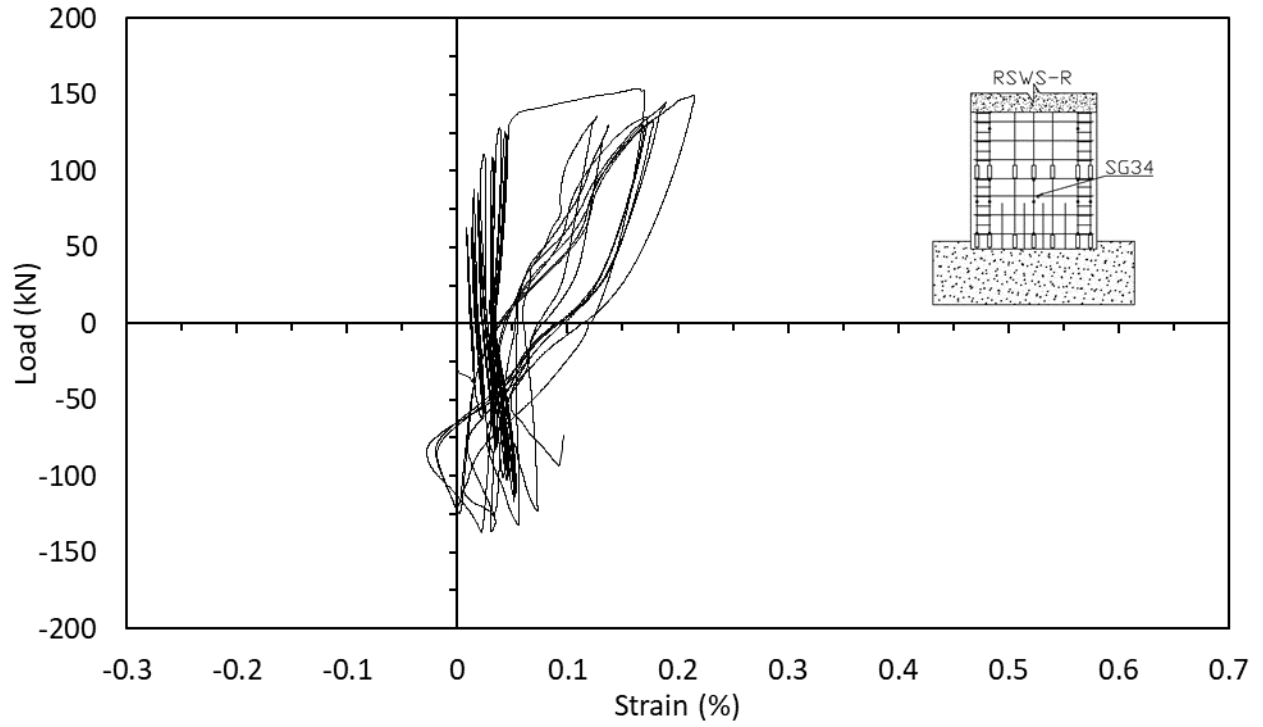


Figure B.31 SG34 Response for wall RSWS-R

B.2 Strain Gauges Used in Wall RSWN

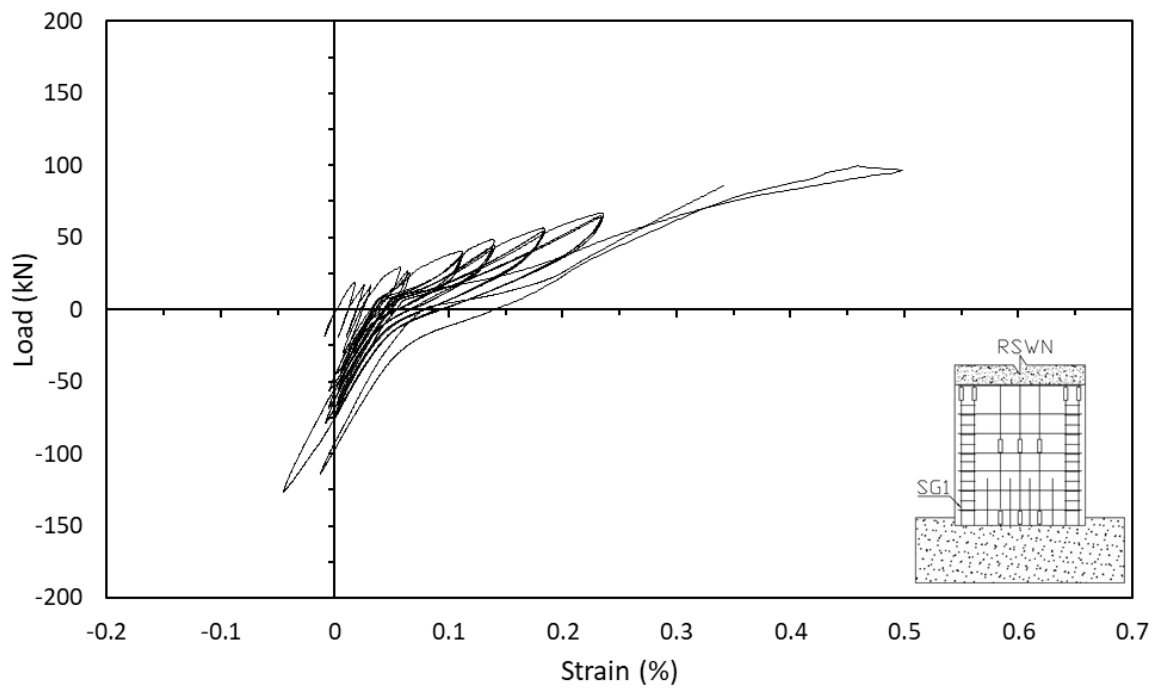


Figure B.32 SG1 Response for wall RSWN

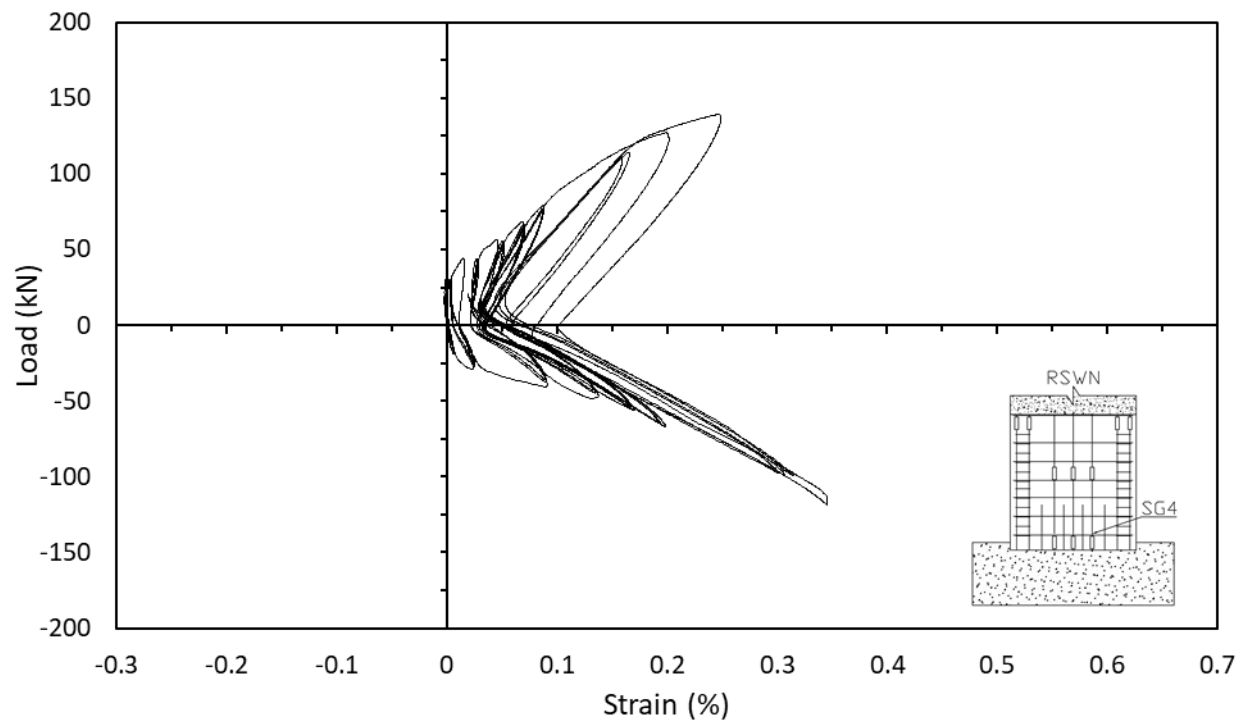


Figure B.33 SG4 Response for wall RSWN

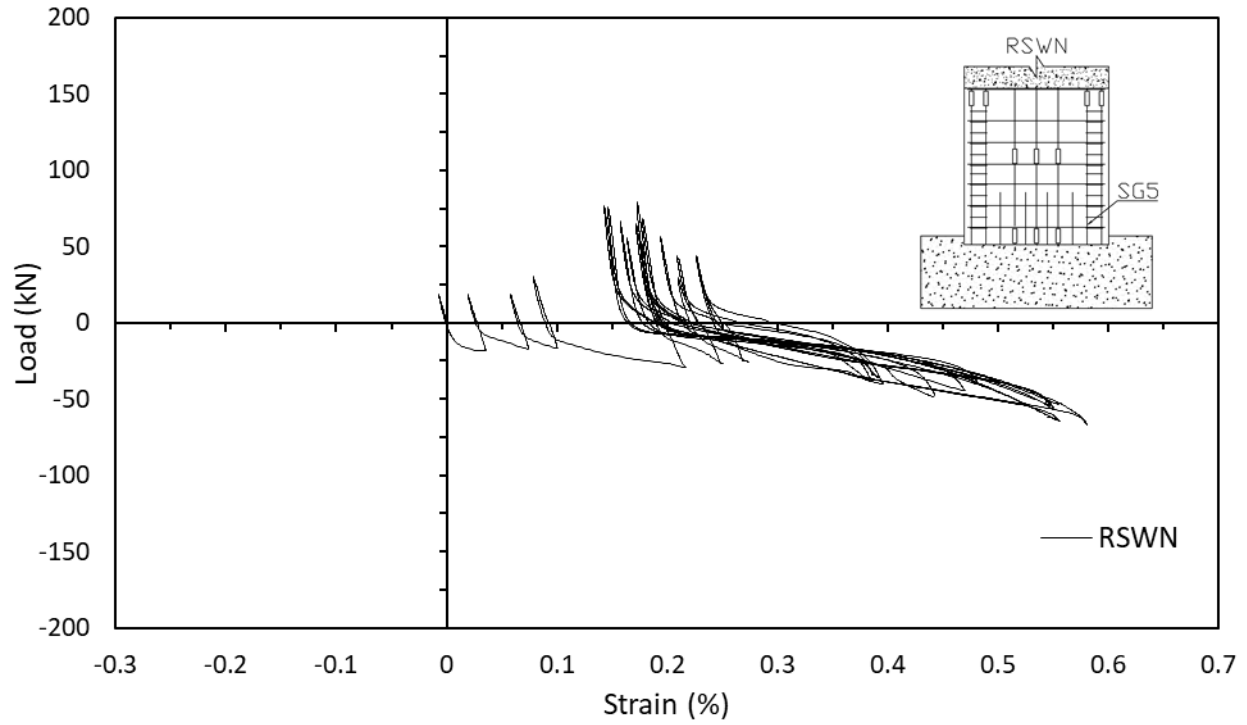


Figure B.34 SG5 Response for wall RSWN

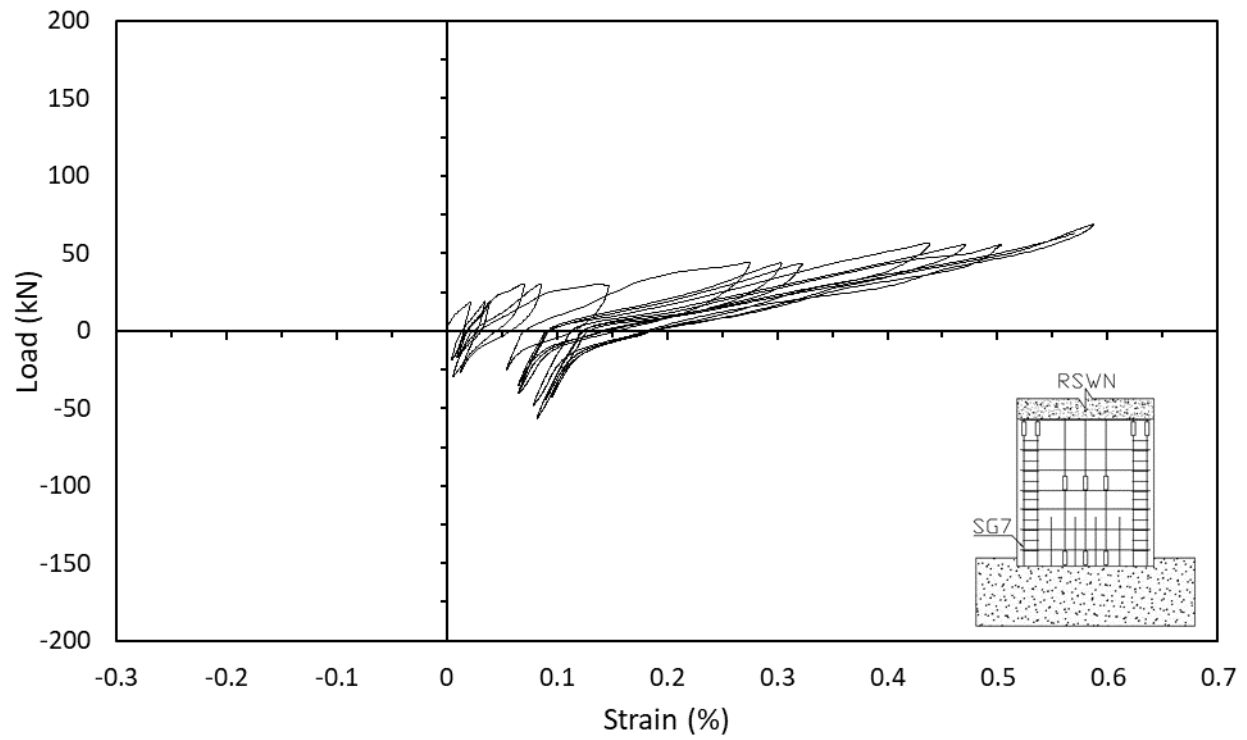


Figure B.35 SG7 Response for wall RSWN

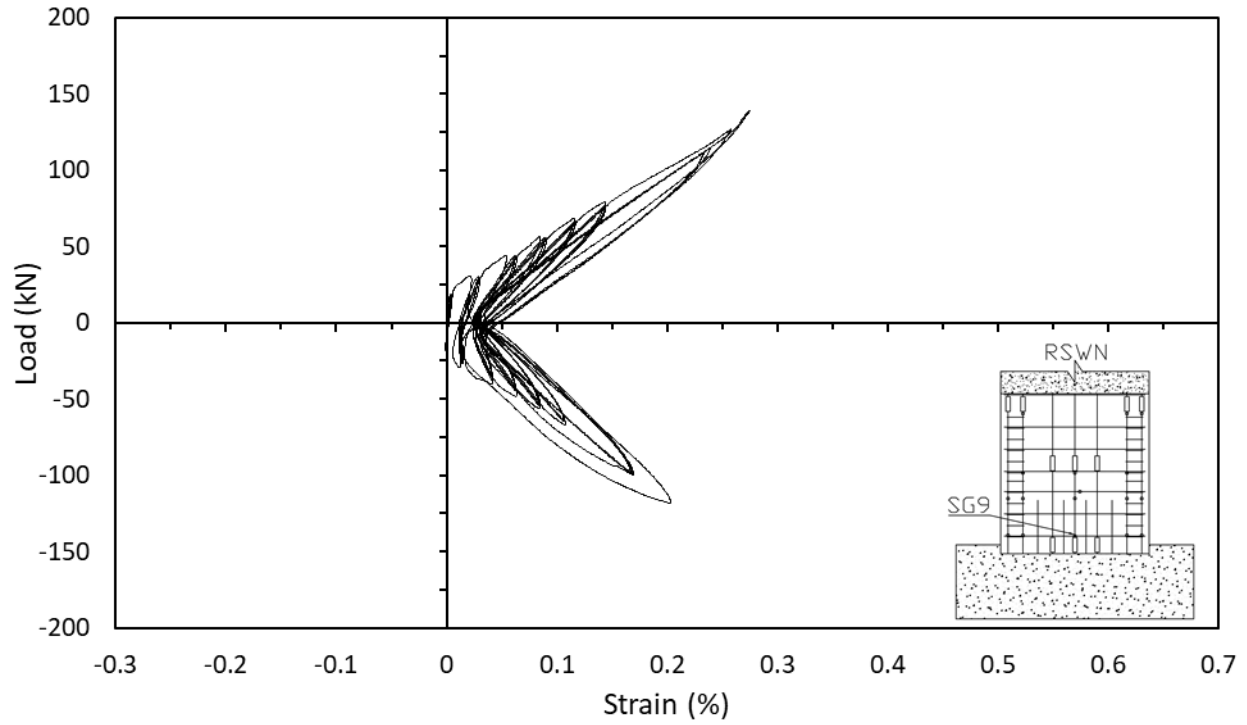


Figure B.36 SG9 Response for wall RSWN

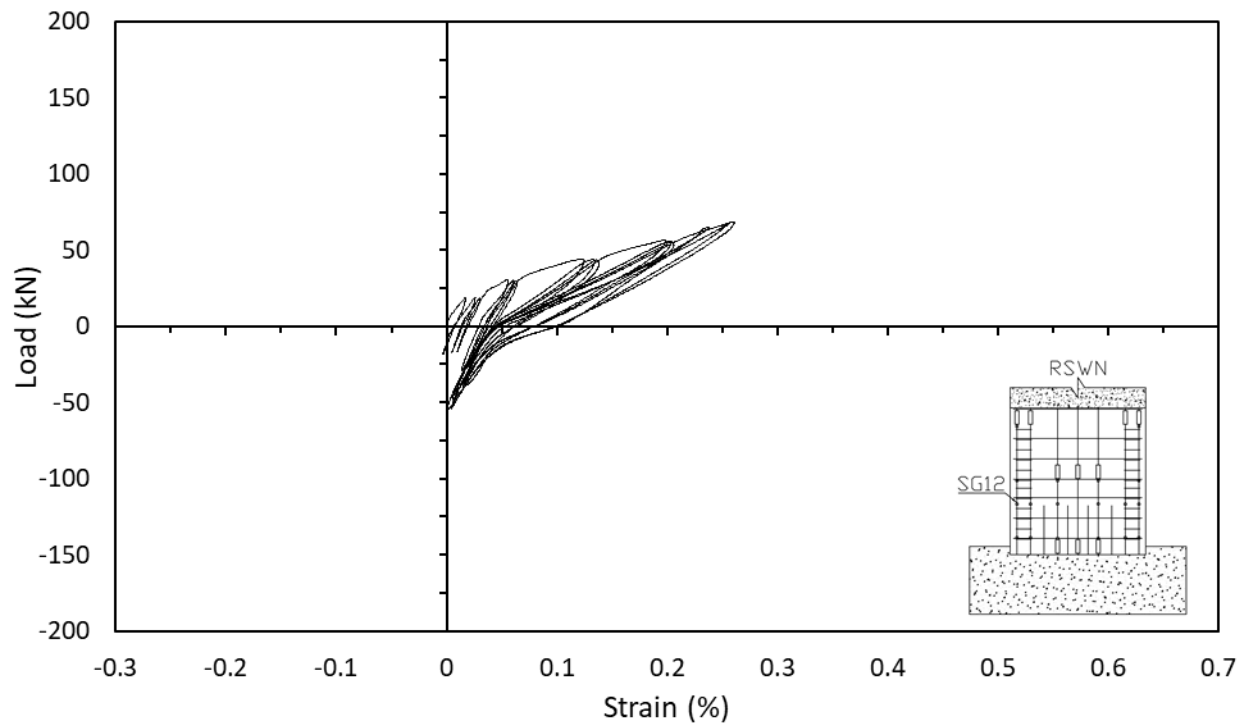


Figure B.37 SG12 Response for wall RSWN

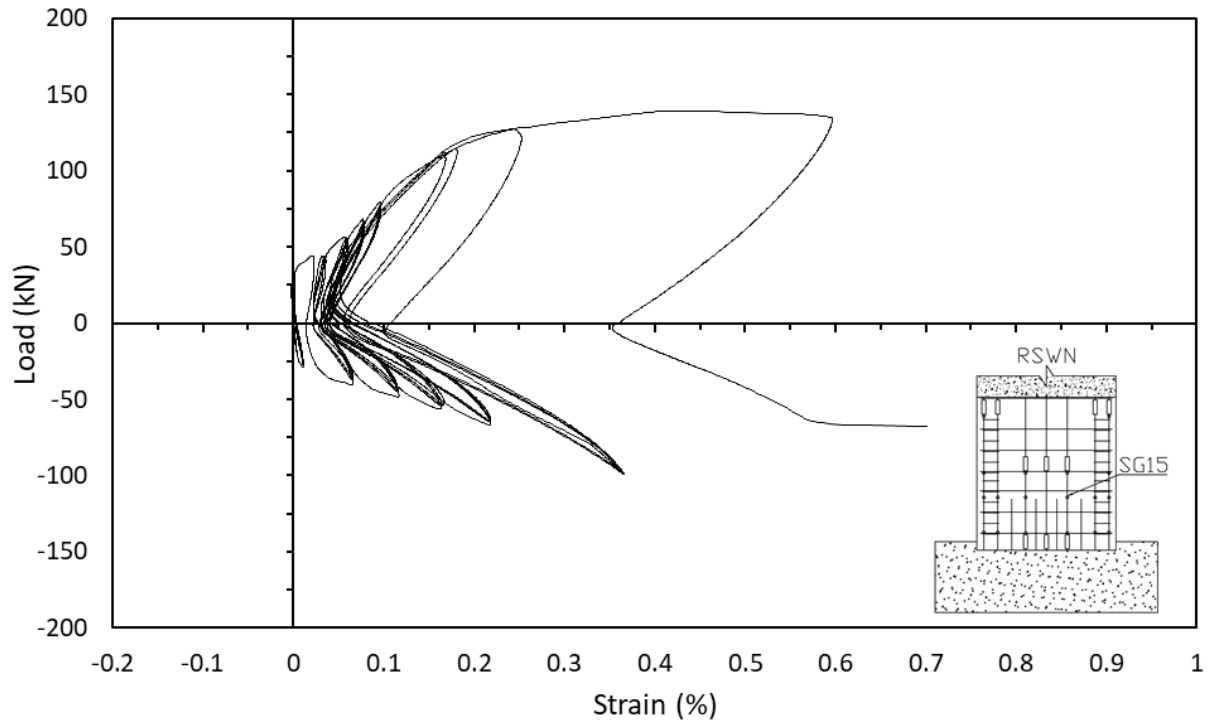


Figure B.38 SG15 Response for wall RSWN

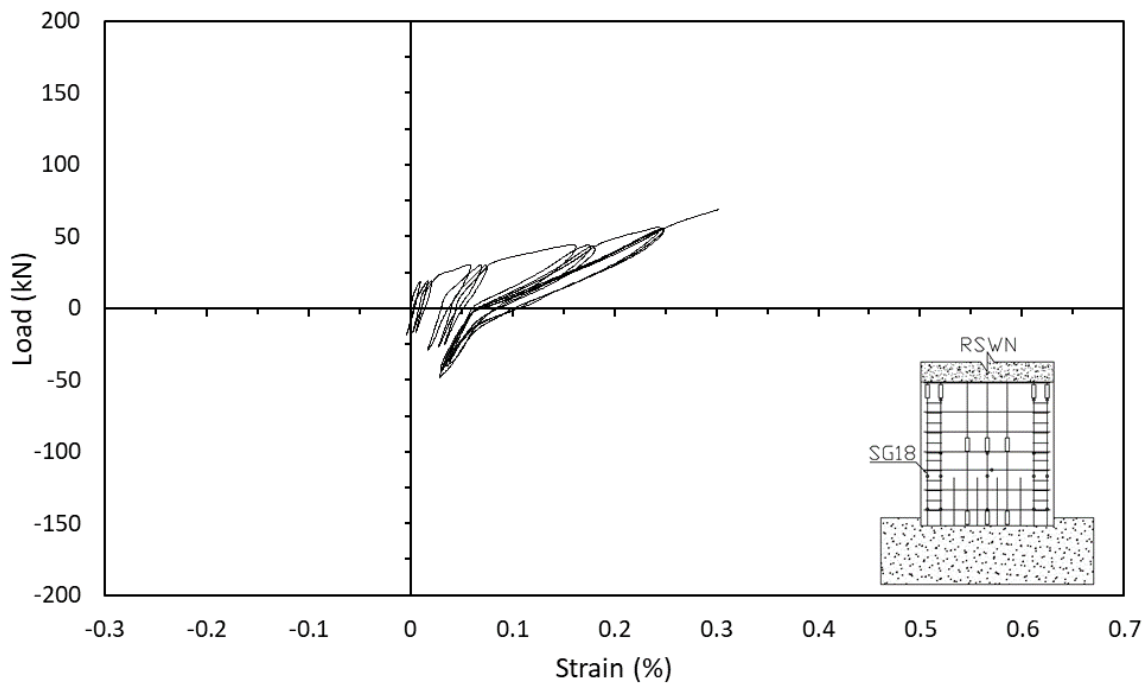


Figure B.39 SG18 Response for wall RSWN

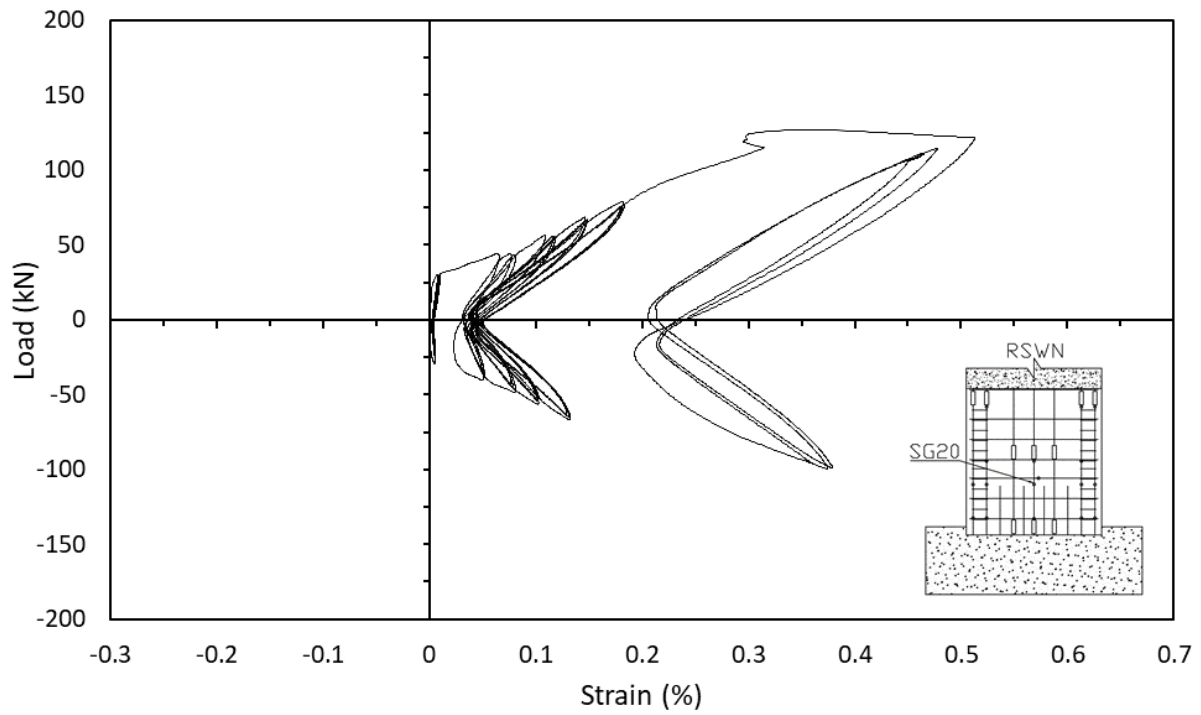


Figure B.40 SG20 Response for wall RSWN

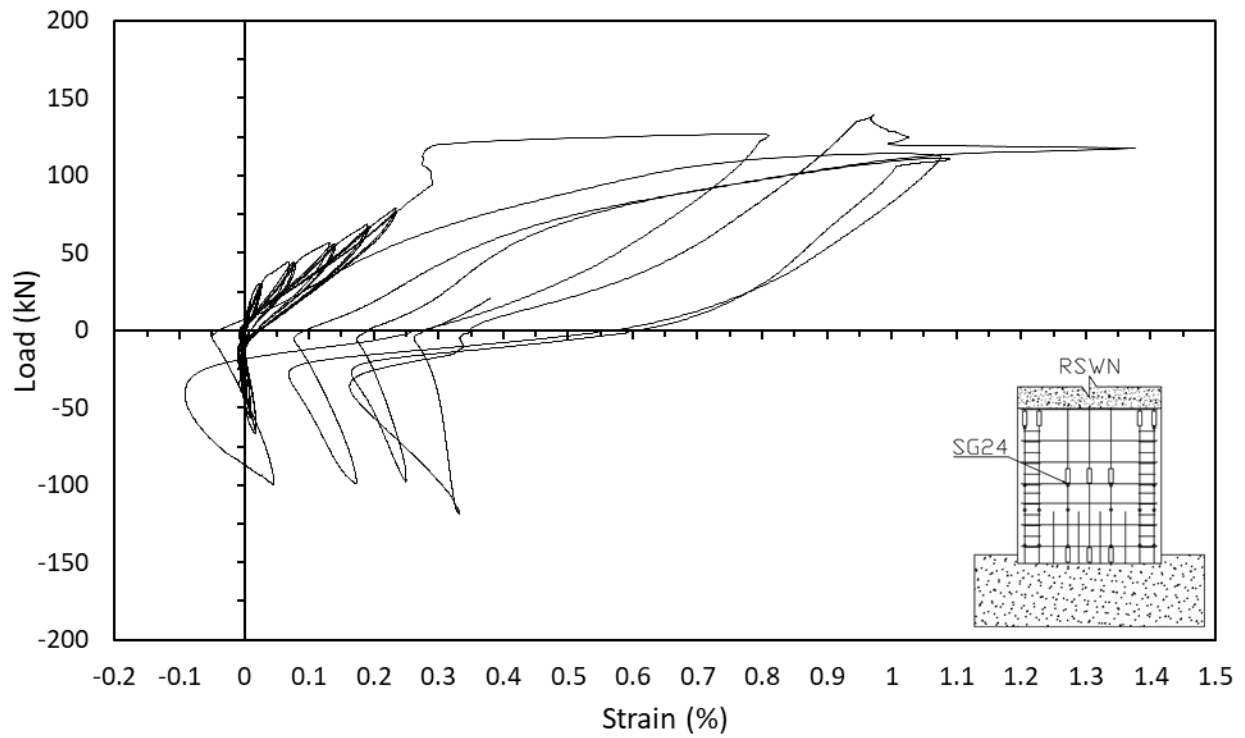


Figure B.41 SG24 Response for wall RSWN

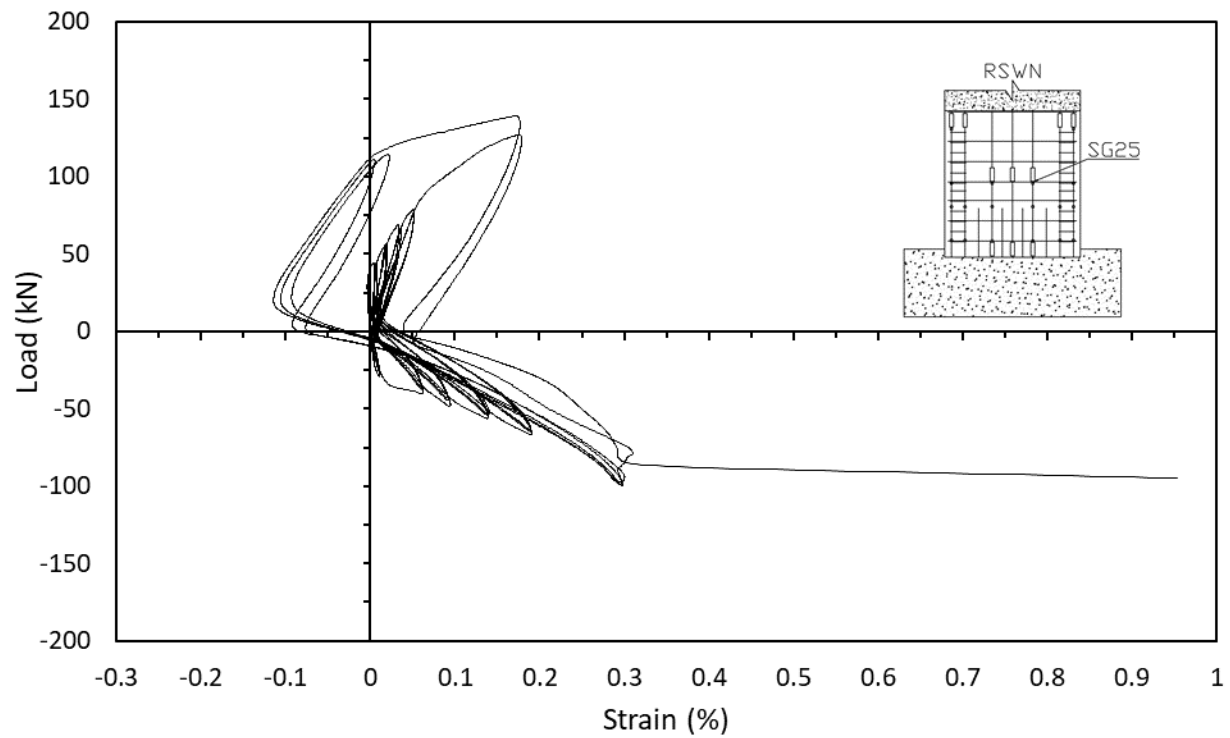


Figure B.42 SG24 Response for wall RSWN

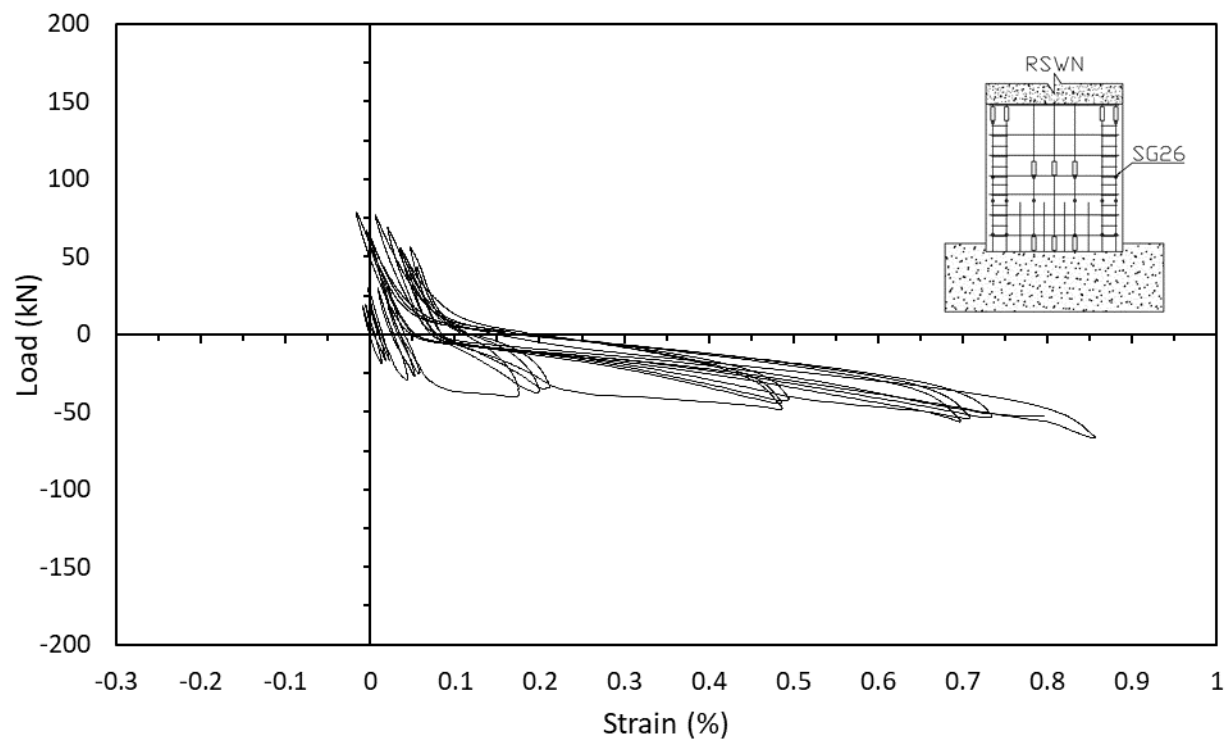


Figure B.43 SG26 Response for wall RSWN

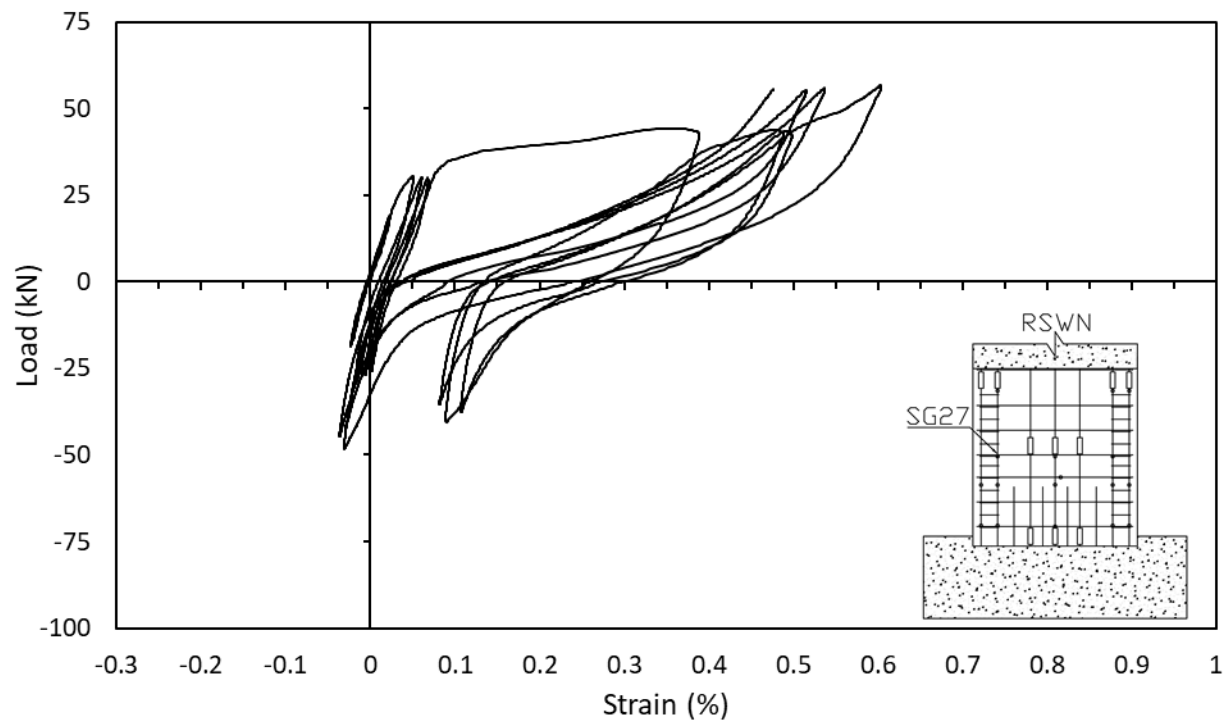


Figure B.44 SG27 Response for wall RSWN

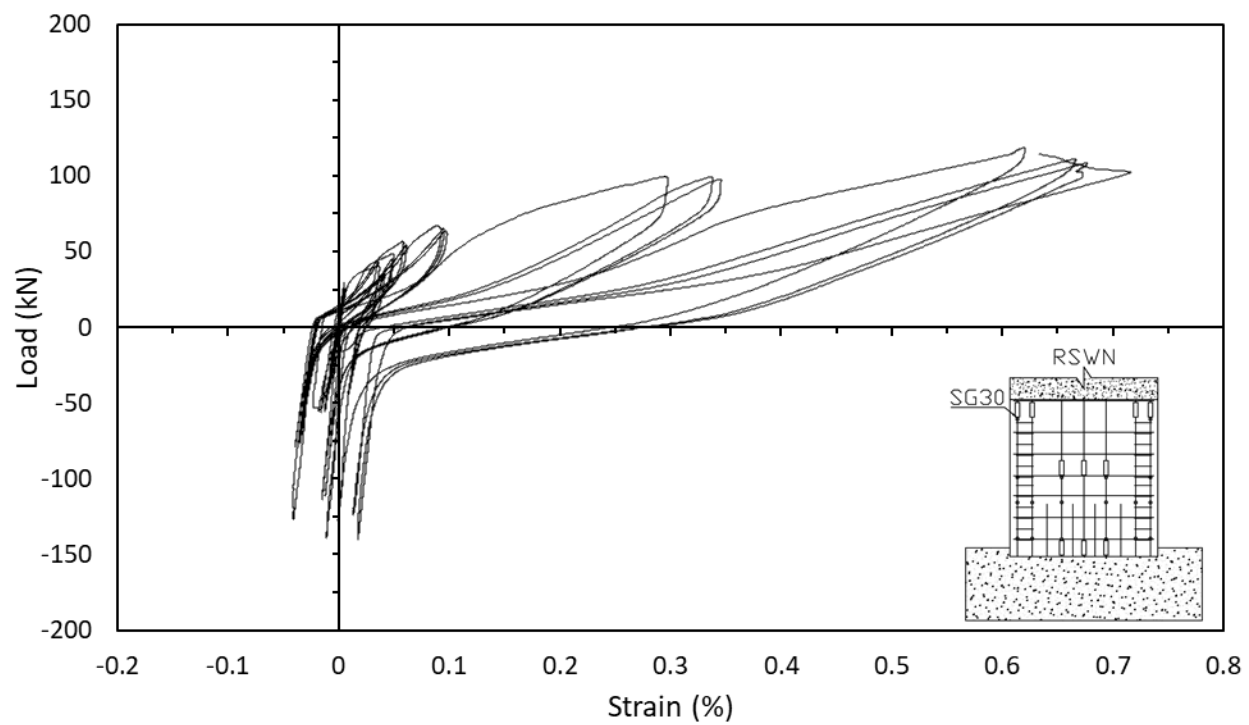


Figure B.45 SG30 Response for wall RSWN

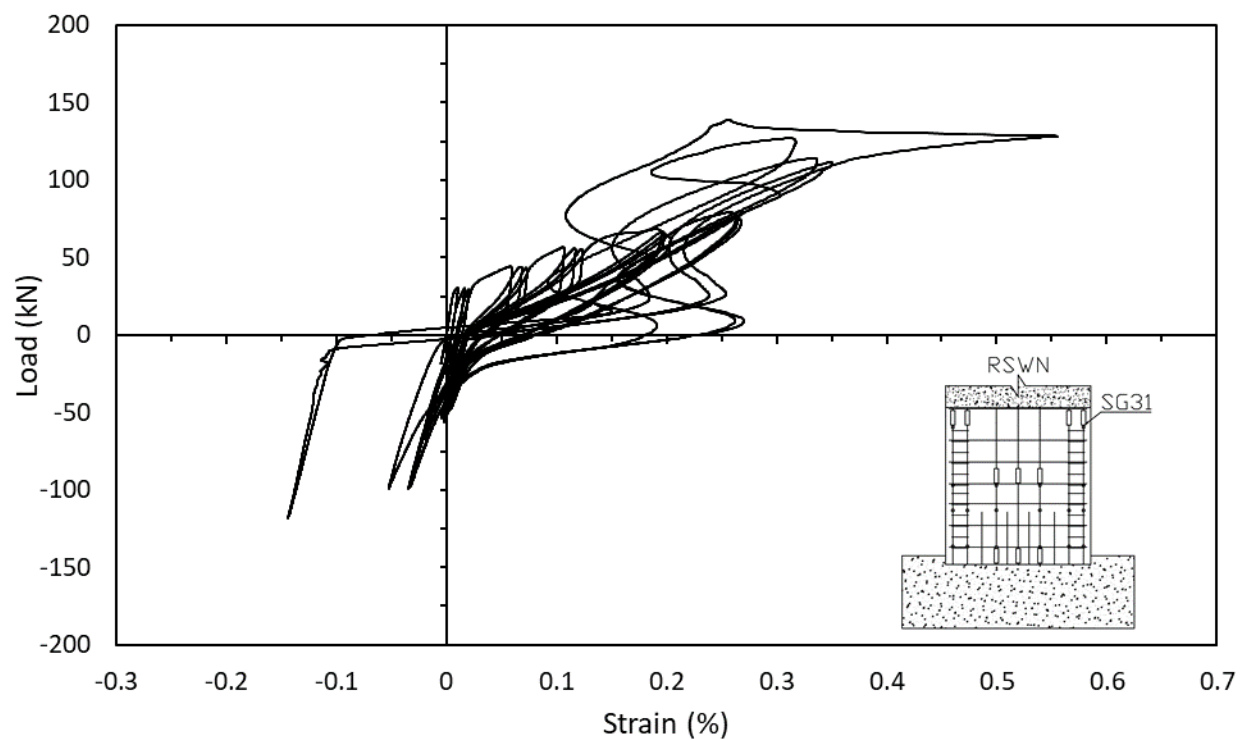


Figure B.46 SG31 Response for wall RSWN

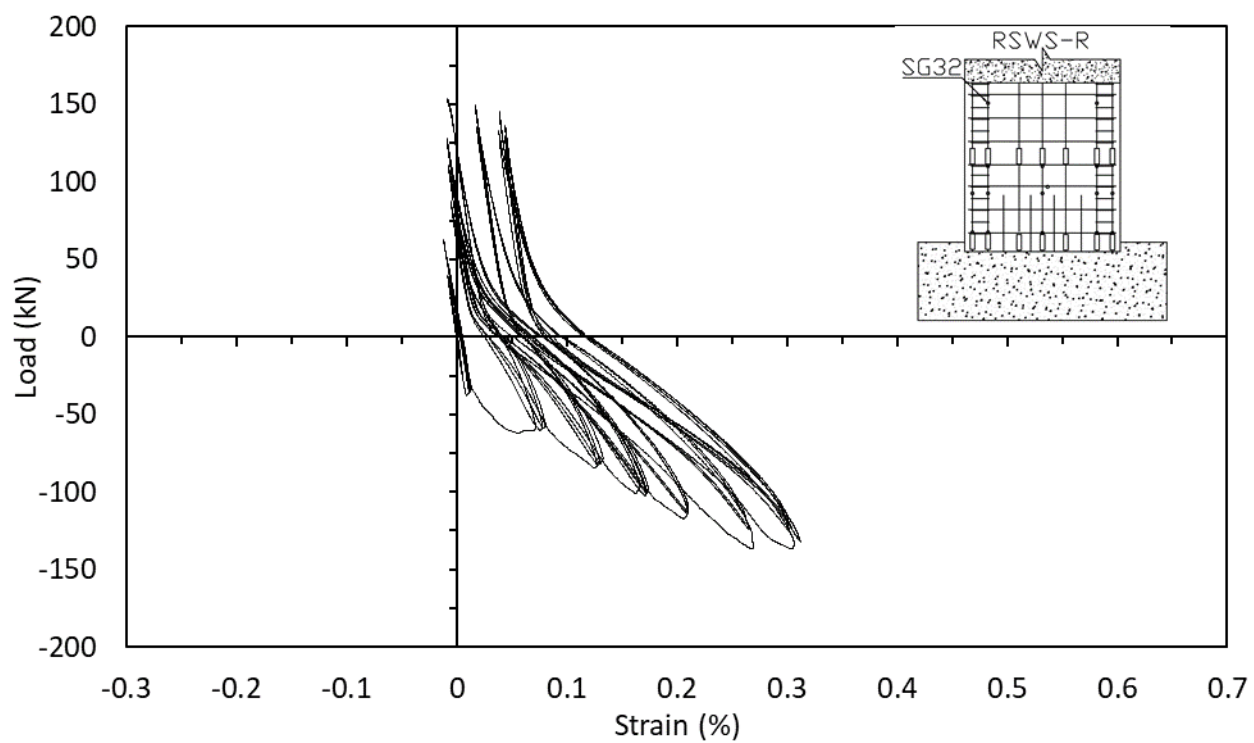


Figure B.47 SG20 Response for wall RSWN

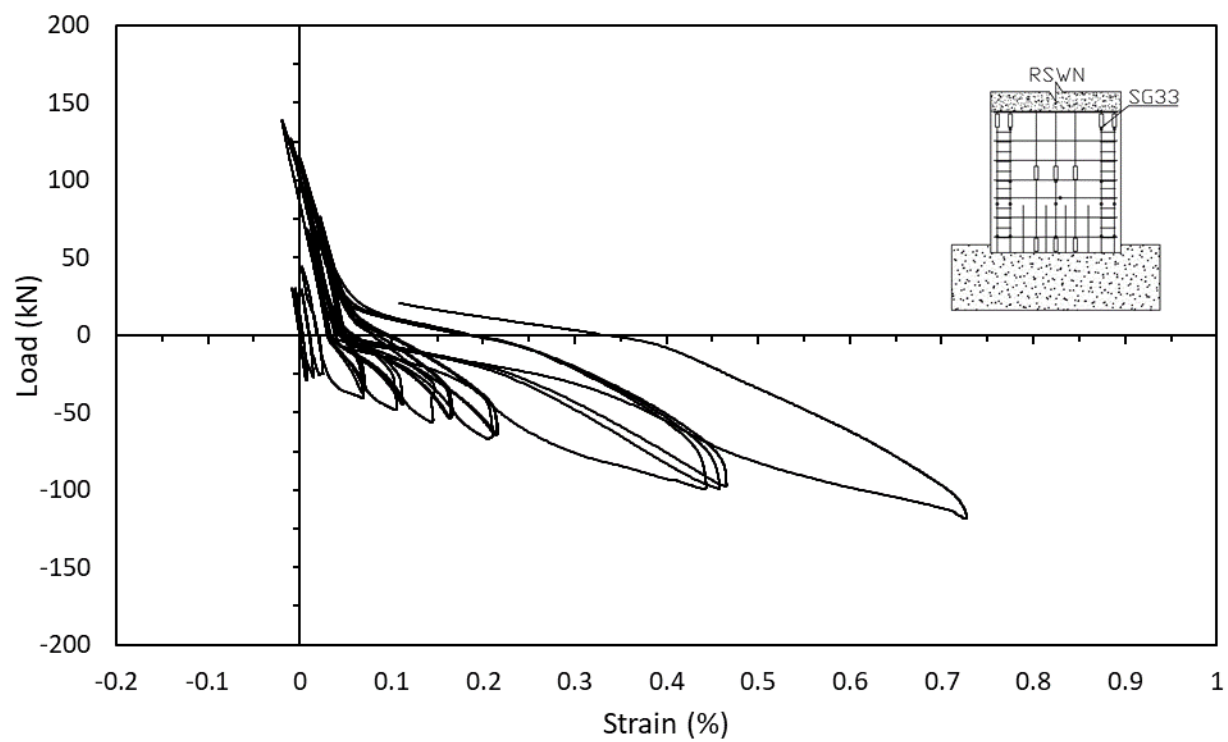


Figure B.48 SG33 Response for wall RSWN

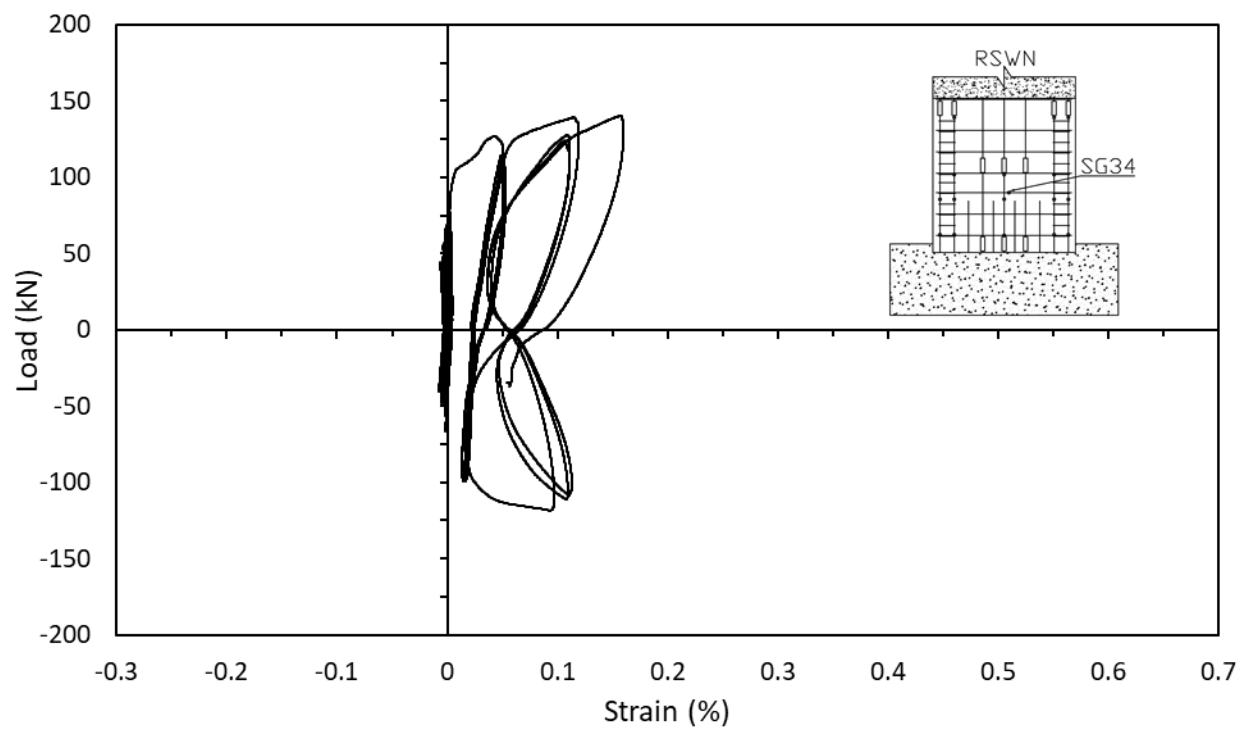


Figure B.49 SG34 Response for wall RSWN