

AN ABSTRACT OF THE THESIS OF

Arijit Sinha for the degree of Master of Science in Civil Engineering and Wood Science
presented on February 23, 2007.

Title: Strain Distribution in OSB and GWB in Wood Frame Shear Walls.

Abstract approved: _____

Rakesh Gupta

The overall goal of this study was to gain an insight into the load sharing aspect between oriented strand board (OSB) and gypsum wall board (GWB) in shear wall assembly during racking load. More specifically the objectives of the study were to: (1) evaluate qualitatively the load sharing between OSB and GWB in a wood frame shear wall assembly, (2) analyze the failure progression of GWB and OSB, (3) study the strain profile around fastener on GWB and OSB sides of shear wall, and (4) study the effect of GWB on shear wall behavior.

Monotonic tests were conducted on 2440 x 2440 mm walls with 38 x 89 mm Douglas-fir studs 610 mm on center. Two 1220x2440x11.1 mm OSB panels were installed and fastened vertically to the frame with Stanley Sheather plus ring shank nails 102 mm and 305 mm on center along panel edges and intermediate studs, respectively. Two 12.7 mm GWB panels were installed oriented vertically on the face opposite the OSB using standard dry wall screws on some walls. Anchorage to the walls was provided by two 12.7 mm A307 anchor bolts installed 305 mm inward on the sill plate from each end of the wall. In addition to these anchor bolts, walls included hold-downs installed at the end studs of the wall and were attached to the foundation with 15.9 mm Grade 5 anchor bolts making the walls fully anchored. The loading was monotonic and based on ASTM E564-00. Sixteen walls were tested in total, out of which 11 (Type A) were sheathed on both sides with OSB and GWB, while 5 walls were tested without GWB (Type B).

Optical measurement equipment based on the principle of Digital Image Correlation (DIC) was used for data acquisition and analysis. DIC is a full-field, non-contact technique for measurement of displacements and strains. The set up consist of a pair of cameras arranged at an angle to take stereoscopic images of the specimen. The

system returns full field 3D displacement and strain data measured over the visible specimen surfaces.

The tests revealed that load is shared by both OSB and GWB initially in a shear wall assembly. GWB fails locally prior to OSB and load shifts to OSB as GWB starts to fail. Beyond this point, load continues to increase and walls finally fail in OSB.

The tests also revealed that load path in wall type A and B is different. Failure in wall type A starts at the uplift corner in GWB and then moves to the uplift corner in OSB. Finally the walls fail at middle of top plate for GWB and OSB both. In wall type B the failure is initiated at the uplift corner in OSB followed by middle region at sill level and ends up at middle section of wall where two panels meet. The uplift corner fasteners are of prime importance in both types of wall and panels.

Comparing the strain profiles created using DIC, strains only near fasteners are observed and no detectable strain is observed in the field of the panel. There is a steady built up of strain in wall type B from start to failure and there is no abrupt change in strain during entire loading indicating a ductile failure. Wall type B shows more ductile behavior than wall type A because of the lack of ability of GWB to deform at higher load in wall type A whereas OSB in wall type B continues to deform at higher load. Also OSB panel in wall type B experiences higher strains than the OSB panel for wall type A for a given load. In wall type A, there is higher strain around the fasteners in GWB than in OSB in the initial part of loading. GWB is stiffer than OSB, it attracts load and in turn deformation is higher than OSB. But being brittle, GWB fails at around 60% of the ultimate wall capacity and load shifts to OSB. This is indicated by large change in strain in OSB. OSB continues to attract load but the strain in OSB increases at a faster rate till failure indicating a much less ductile behavior than that of wall type B.

Contribution of GWB towards strength of the wall is marginal (0.8%) while an increase of 50% was observed in overall stiffness of the walls. Since GWB is stiffer than OSB, it contributes more to the overall stiffness of the wall. Ductility factor of the system increases by 20% and the ductility of the system increases by 13% while energy dissipated by the wall decreases when GWB is included in the shear wall assembly. GWB being brittle reduces the ability to deform before failing and hence a decrease in peak, failure and yield displacements is observed in magnitude of 18%, 13% and 27%, respectively

Overall, these tests suggest that initially during loading of a wall the load is shared between OSB and GWB. However, the proportion of load sharing is not known. As GWB fails first the load shifts to the OSB panel which resists it till the failure of the wall. This aspect of load sharing between structural sheathing and gypsum wall board is not incorporated in current design practices. It is recommended that more tests especially with cyclic and dynamic loading be conducted to better understand and quantify the aspect of load sharing.

STRAIN DISTRIBUTION IN OSB AND GWB IN WOOD FRAME SHEAR WALLS

by
Arijit Sinha

A THESIS

submitted to

Oregon State University

in partial fulfillment of
the requirements for the
degree of

Master of Science

Presented February 23, 2007
Commencement June 2007

Master of Science thesis of Arijit Sinha presented on February 23, 2007

APPROVED:

Major Professor Representing Wood Science and Civil Engineering

Chair of the Department of Civil, Construction and Environmental Engineering

Head of the Department of Wood Science and Engineering

Dean of the Graduate School

I understand that my thesis will become part of the permanent collection of Oregon State University libraries. My signature below authorizes release of my thesis to any reader upon request.

Arijit Sinha, Author

ACKNOWLEDGMENTS

I would like to thank the following people for their support in helping me complete this project:

- Dr. Rakesh Gupta – For his guidance, support, and providing me the opportunity to take part in this research project at Oregon State University.
- Milo Clauson – For his help guidance and support. I have really enjoyed working with Milo, particularly because of his extremely unique and diverse knowledge, sense of humor, and his unflappable optimism
- Dr. Lech Muszynski – For his support, encouragement and guidance throughout the project.
- Heather Redler, Alex Mench, Yi Wang and Henrik Larsen – For their help in the lab.
- All of my family and friends.
- Louisiana Pacific Company for donating OSB used in the project.
- Pabco Gypsum Company for donating gypsum wall board used in this project
- Sun Stud Company – For donating the dimension lumber used in the project.

TABLE OF CONTENTS

	PAGE
INTRODUCTION	1
MATERIALS AND METHODS	4
WALL SPECIMENS.....	4
TEST SETUP	4
MONOTONIC TESTING	5
DATA ACQUISITION.....	5
DATA ANALYSIS	6
EEEEP CURVE	7
TEST MATRIX.....	7
RESULTS AND DISCUSSION.....	15
DISPLACEMENTS	15
LOAD SHARING	15
OSB AND GWB (Wall type A)	15
OSB (Wall type A and B)	16
STRAIN PROFILE	17
WALL TYPE A	17
WALL TYPE B	18
FAILURE PROGRESSION	18
GWB.....	18
OSB.....	19
CONTRIBUTION OF GWB.....	21
CONCLUSIONS	33
BIBLIOGRAPHY	35
APPENDICES	37

LIST OF FIGURES

<u>Figure</u>	<u>Page</u>
1. Schematic of shear wall test specimen	8
2. Test Set up in the Lab	9
3. DIC set up for the shear wall test	10
4. Equivalent Energy Elastic Plastic curve	11
5. Nailing schedule for OSB and GWB (solid dots) showing imaged areas (roman numerals)...	12
6. Load vs. local displacement curve for uplift corner in GWB	24
7. Global Load vs. Local principle strain for uplift corner (Fastener S9 and 9)	25
8. Global Load vs. local principle strain (in OSB) plot for fastener 9	26
9. Global load vs. local principle strain in OSB plot for fastener 5	27
10. Strain profile in GWB at various stages of loading	28
11. Strain profile in OSB (Wall type A) at various stages of loading	29
12. Strain profile in OSB (Wall type B) at various stages of loading	30
13. Typical Load Deflection curve for Type A & B Walls	31

LIST OF TABLES

<u>Table</u>	<u>Page</u>
1. Modulus of Elasticity of various sheathing materials.....	3
2. EEEP Parameters	13
3. Test Matrix.....	14
4. EEEP curve parameters	32

LIST OF APPENDICES

<u>APPENDIX</u>	<u>PAGE</u>
A - WALL CONSTRUCTION	38
B – DIGITAL IMAGE CORRELATION.	47
C – DATA ANALYSIS.....	51
D - LOAD VS. LOCAL DISPLACEMENT	60
E - LOAD STRAIN DIAGRAMS.....	74
F - STRAIN PROFILE.....	91
G – FAILURE MAP	126
I – DETAILED LITERATURE REVIEW	135

LIST OF APPENDIX FIGURES

<u>Figure</u>	<u>Page</u>
A1. Wood frame shear wall construction.....	39
A2. Flexure Test set up and equations.....	39
A3. Wall Layout.....	45
B1. Load Displacement Diagram from preliminary test.....	50
B2. Load strain Diagram from preliminary test.....	50
C1. Load Deflection Diagram for all the walls depicted by two generic curve types.....	57
C2. Load Deflection Diagram for all the walls.....	58
C3. Areas of interest (aoi) at the uplift corner.....	59
D1. Load Displacement diagram at S2 (Zone I).....	60
D2. Load Displacement diagram at S5 (Zone III).....	61
D3. Load Displacement diagram at S6 (Zone III).....	61
D4. Load Displacement diagram at S7 (Zone III).....	62
D5. Load Displacement diagram at S8 (Zone III).....	62
D6. Load Displacement diagram at S9 (Zone II).....	63
D7. Load Displacement diagram at S11 (Zone IV).....	63
D8. Load Displacement diagram at S12 (Zone IV).....	64
D9. Load Displacement diagram at S13 (Zone V).....	64
D10. Load Displacement diagram of Zone VI.....	65
D11. Load Displacement diagram for Zone I.....	66
D12. Load Displacement diagram for Zone II.....	66
D13. Load Displacement diagram for Zone III (Nail 5).....	67
D14. Load Displacement diagram for Zone III (Nail 7).....	67
D15. Load Displacement diagram for Zone IV (Nail 11).....	68
D16. Load Displacement diagram for Zone IV (Nail 12).....	68
D17. Load Displacement diagram for Zone V (Nail 13).....	69
D18. Load Displacement diagram for Zone VI.....	69
D19. Load Displacement diagram for Zone I.....	70
D20. Load Displacement diagram for Zone II.....	70
D21. Load Displacement diagram for Zone III (Nail 5).....	71
D22. Load Displacement diagram for Zone III (Nail 7).....	71
D23. Load Displacement diagram for Zone IV (Nail 11).....	72
D24. Load Displacement diagram for Zone IV (Nail 12).....	72
D25. Load Displacement diagram for Zone V (Nail 13).....	73
E1. Load vs. Strain Diagram for S2.....	74

LIST OF APPENDIX FIGURES (Continued)

<u>Figure</u>	<u>Page</u>
E2. Load vs. Strain Diagram for S9	75
E3. Load vs. Strain Diagram for S8	76
E4. Load vs. Strain Diagram for S7	76
E5. Load vs. Strain Diagram for S6	77
E6. Load vs. Strain Diagram for S5	77
E7. Load vs. Strain Diagram for S11	78
E8. Load vs. Strain Diagram for S12	78
E9. Load vs. Strain Diagram for S13	79
E10. Load vs. Strain Diagram for Zone VI	79
E11. Load vs. Strain Diagram for Nail 2	80
E12. Load vs. Strain Diagram for Nail 4	80
E13. Load vs. Strain Diagram for nail 5	81
E14. Load vs. Strain Diagram Zone VI	81
E15. Load vs. Strain Diagram zone VI	82
E16. Load vs. Strain Diagram for Nail 9	82
E17. Load vs. Strain Diagram for Nail 10	83
E18. Load vs. Strain Diagram for other fastener in Zone II	83
E19. Load vs. Strain Diagram for Nail 12	84
E20. Load vs. Strain Diagram for Nail 13	84
E21. Load vs. Strain Diagram for Nail 1	85
E22. Load vs. Strain Diagram for Nail 2	85
E23. Load vs. Strain Diagram for Nail 4	86
E24. Load vs. Strain Diagram for Nail 5	86
E25. Load vs. Strain Diagram for Nail 6	87
E26. Load vs. Strain Diagram for Nail 8	87
E27. Load vs. Strain Diagram for Nail 9	88
E28. Load vs. Strain Diagram for Nail 10	88
E29. Load vs. Strain Diagram for Nail 11	89
E30. Load vs. Strain Diagram for Nail 12	89
E31. Load vs. Strain Diagram for Nail 13	90
S1. Scale of reference for strain profile	91
F1. Strain Profile at middle of the sill region (Zone III) for wall 1	92
F2. Strain Profile at middle of the sill region (Zone III) for wall 2	93
F3. Strain profile at the uplift corner (Zone II) for wall 5	94

LIST OF APPENDIX FIGURES (Continued)

<u>Figure</u>	<u>Page</u>
F4. Strain profile (e_{xx}) at uplift corner (Zone II) for wall 5	95
F5. Strain profile (e_{yy}) at uplift corner (Zone II) for wall 5	96
F6. Strain profile (e_{xy}) at uplift corner (Zone II) for wall 5	97
F7. Strain profile at the uplift corner (Zone II) for wall 6	98
F8. Strain profile at the uplift corner (Zone I) for wall 7	99
F9. Strain profile at the uplift corner (Zone I) for wall 8	100
F10. Strain profile at the intermediate stud near to uplift corner (Zone VI) for wall 9.....	101
F11. Strain profile at the intermediate stud near to comp. corner (Zone VI) for wall 10	102
F12. Strain profile at the mid wall region (Zone IV) for wall 11	103
F13. Strain profile at the mid wall region (Zone IV) for wall 12	104
F14. Strain profile at the middle of the top plate (Zone V) for wall 13	105
F15. Strain profile at the middle of the sill plate (Zone III) for wall 1	106
F16. Strain profile at the middle of the sill plate (Zone III) for wall 2	107
F17. Strain profile at the uplift corner (Zone II) for wall 5	108
F18. Strain profile (e_{xx}) at uplift corner (Zone II) for wall 5	109
F19. Strain profile (e_{yy}) at uplift corner (Zone II) for wall 5	110
F20. Strain profile (e_{xy}) at uplift corner (Zone II) for wall 5	111
F21. Strain profile at the uplift corner (Zone II) for wall 6	112
F22. Strain profile at the compression corner (Zone I) for wall 7	113
F23. Strain profile at the compression corner (Zone I) for wall 8	114
F24. Strain profile at intermediate stud near uplift corner (Zone VI) for wall 9.....	115
F25. Strain profile at intermediate stud near comp. corner (Zone VI) for wall 10	116
F26. Strain profile at middle of the wall (Zone IV) for wall 11	117
F27. Strain profile at middle of the wall (Zone IV) for wall 12.....	118
F28. Strain profile at middle of the wall (Zone IV) for wall 13.....	119
F29. Strain profile at middle of the sill plate (Zone II) for wall 3	120
F30. Strain profile at middle of the sill plate (Zone II) for wall 4	121
F31. Strain profile at middle of wall (Zone IV) for wall 14.....	122
F32. Strain profile at middle of top plate (Zone VI) for wall 14	123
F33. Strain profile at uplift corner (Zone II) for wall 15	124
F34. Strain profile at compression corner (Zone I) for wall 16	125
G1. Wall 1 failure map	126
G2. Wall 2 failure map	127

LIST OF APPENDIX FIGURES (Continued)

<u>Figure</u>	<u>Page</u>
G3. Wall 3 failure map	127
G4. Wall 4 failure map	128
G5. Wall 5 failure map	128
G6. Wall 6 failure map	129
G7. Wall 7 failure map	129
G8. Wall 8 failure map	130
G9. Wall 9 failure map	130
G10. Wall 10 failure map	131
G11. Wall 11 failure map	131
G12. Wall 12 failure map	132
G13. Wall 13 failure map	132
G14. Wall 14 failure map	133
G15. Wall 15 failure map	133
G16. Wall 16 failure map	134

LIST OF APPENDIX TABLES

<u>Table</u>	<u>Page</u>
A1. MOE values 2x4(S-series)	40
A2 MOE values for 2x4 (T series)	42
A3. Wall member layout	46
C1. List of Wall Test	55
C2. Summary of Test and EEEP Parameters	56
G1. Abbreviation key for failure map	126

STRAIN DISTRIBUTION IN OSB AND GWB IN WOOD FRAME SHEAR WALLS

INTRODUCTION

Majority of the buildings built in United States are wood structures. Traditionally wood structures have performed well during seismic events because wood can withstand very high load if applied for a short duration of time. The main lateral force resisting system (LFRS) of the wood frame structure is the shear wall and how well a wood structure performs during an earthquake is largely dependent on the competency of the shear wall. Shear walls are generally made from nominal 2x4 or 2x6 framing lumber with wood structural panel as sheathing attached to it on one side. It is attached to the framing with the help of dowel type fasteners (nails, screws, staples etc.) spaced as per strength and stiffness requirements. Sheathing is generally of oriented strand board (OSB) or plywood. Gypsum wall board (GWB) is attached on the other side of the wall for exterior walls. GWB is considered a non structural element in LFRS. However, damage assessment after 1994 Northridge earthquake suggested that the most of the shear wall failure was due to cracking and tearing of GWB. Pulling out of nails in OSB and plywood also contributed to failures (Schierle, 2002a). The total estimated damage was worth \$40 billion and more than half this amount and 60 fatalities were attributed to the damages in wood frame structures. 48000 housing units were rendered uninhabitable (Schierle, 2002b). The question that such a huge human and economic loss raised was how to improve existing code provisions and retrofit the existing structures to resist earthquake damages in future. A better understanding of the behavior of OSB and GWB in a wood frame shear wall assembly is a logical step in providing for a potential solution.

Propelled by the enormity of damages during the 1994 Northridge Earthquake, Consortium of Universities for Research in Earthquake Engineering (CUREE) conducted rigorous testing as a part of an exhaustive study to account for the damages during Northridge earthquake. One of the findings of this study suggested incorporating the more complicated behavior of finish material effects on shear wall assemblies while considering damage-limitation performance (Cobeen et al, 2004). The contribution of GWB is not included in current design standards (AFPA 2001) but GWB is slightly stiffer than OSB (Table 1) or other sheathing material, but at the same time, it is brittle. Since stiffness attracts load, it is highly probable that major proportion of the initial load is transferred

through the GWB during any seismic event. Being brittle it cracks and subsequently can not withstand the load after failure. In most modern design this aspect is completely overlooked.

As wood shear walls are the major lateral force resisting system in most buildings, they have been the subject of various studies and research (Filiatrault, 2002). However, few studies describe the load sharing between GWB and OSB in a wood frame shear wall assembly. Similarly, little research on the contribution of GWB to strength and stiffness during a seismic event is available.

Wolfe (1983) tested 30 walls to study the contribution of GWB to the racking resistance of light-frame walls and determined that the contribution can be explained by the law of superposition, i.e., racking resistance of walls with GWB and structural wood panels appeared to equal the sum of contributions of the elements tested independently. Walls tested with panels oriented horizontally were more than 40% stronger and stiffer than those with panels oriented vertically. Finally, Wolfe concluded that GWB could provide significant contribution to the racking resistance when subjected to monotonic loading.

Karacabeyli and Ceccotti (1996) tested 2.44 x 4.88 m (8' x 16') walls with GWB on one side and OSB on other and concluded that peak load increased but ductility decreased due to brittle nature of GWB, when compared to only OSB as sheathing. For monotonic tests, they verified the law of superposition proposed by Wolfe (1983) up to a drift of approximately 50 mm. Johnson (1997) concluded that GWB helps resist shear in the low to moderate loading, but plywood resists most of the shear near capacity under monotonic loading. Uang and Gatto (2003) studied the effect of GWB on peak strength, initial stiffness, absorbed energy and deformation capacity. They observed 12% increase in shear wall strength and 31% decrease in shear wall deformation capacity. Initial stiffness increased by 60% as expected because GWB is stiffer than OSB and attracts more load in the beginning. Toothman (2003) tested 2.44m x 1.2m walls and found similar results as Uang and Gatto (2003) but concluded that the principle of superposition is not valid. While observing failure patterns for the walls sheathed on both sides using nails, GWB panels were always first to fail. This is because of the relative ease with which nail could tear the sheathing and also GWB being stiffer than OSB attracts more load. Toothman concluded that by adding GWB in the structure there is an increase in overall strength, elastic stiffness and energy dissipation before failure of the wall. He also concluded that GWB provides a substantial amount of shear resistance.

To include GWB in the shear wall assembly design process, it is imperative to investigate the amount or proportion of the load experienced by GWB in a shear wall during a seismic event. A better understanding of the role of GWB in the mechanism of shear wall assembly and the extent of distribution of load during a seismic event are required to increase design efficiency. This study addresses this aspect by testing shear walls under monotonic loading, hence seeks to analyze the load sharing between OSB and GWB in a wood frame shear wall assembly.

This project was divided into two parts. The first part addressed the issue of load sharing between OSB and GWB in a wood frame shear wall assembly. The second part provided insight into differences in performance between walls with and without GWB. Hence these two parts allowed us to investigate load sharing between OSB and GWB. Specifically the objectives of this project were:

1. To evaluate qualitatively the load sharing between OSB and GWB in a wood frame shear wall assembly,
2. To study the strain profile around fasteners in GWB and OSB,
3. To analyze the failure progression of GWB and OSB and
4. To study the effects of GWB on shear wall behavior.

Table 1. Modulus of Elasticity of various sheathing materials

Material	Modulus of Elasticity	Source
Plywood	7-13 GPa	Youngquist, 2000
OSB	5-8 GPa	Youngquist, 2000
GWB	5-12 GPa	Deng & Furono, 2001

MATERIALS AND METHODS

WALL SPECIMENS

Shear wall test specimens were designed and constructed in accordance with the 2000 International Residential Code prescribed braced panel construction. All tests were conducted on identical 2440x2440 mm walls constructed using stud grade 38x89 mm kiln dried Douglas-fir framing as shown in Figure 1. Framing studs were spaced at 610 mm on center, connected to the sill plate and first top plate using two 16d (3.33x82.6 mm) nails per connection, driven through the plates and into the end grain of the stud. A second top plate was connected to the first top plate using 16d nails at 610 mm on center. The walls were sheathed using two 1220x2440x11.1 mm OSB panels that were attached vertically to the wall frame. The 24/16 APA rated OSB panels were connected to the wall frame using 8d (2.87 x 63.5 mm) ring shank sheathing nails (Sheather Plus, Stanley) spaced 102 mm on center along the panel edges and 305 mm along the intermediate studs (field nailing). The walls were additionally sheathed with two 1220x2440x12.7 mm GWB panels installed vertically on the face opposite to the OSB structural panels. The GWB panels were attached to the framing with bugle head coarse wallboard screws (2.31x41.3 mm) spaced 305 mm on center along the panel edges and intermediate studs. Sheathing to framing connections was staggered (not shown in figure) on the end post and top plate. Double end studs were required because walls were anchored with hold-downs, and were connected together using 16d (3.33x82.6 mm) framing nails at 305 mm on center. Framing nails were full round head, strip cartridge, and smooth shank SENCO® nails that were driven using a SENCO® SN 65 pneumatically driven nail gun. Sheathing nails were Stanley Sheather plus nails driven pneumatically as well.

TEST SETUP

The test set up is shown in Figure 2. Specimens were bolted to a fabricated steel beam firmly attached to the strong floor to simulate a fixed foundation. Specimens were loaded using a 490 kN (110 kip) servo controlled hydraulic actuator with a 254 mm total stroke, and controlled by an MTS 406 servo controller. The hydraulic actuator was attached to the strong wall and supported by a 102 mm hydraulic cylinder. This allows the actuator to raise and lower freely during the test without creating additional vertical loading on the wall. A 111.2 kN (25 kip) load cell attached to the piston provided force measurements. A steel C-channel, laterally braced to the strong wall, was attached to the load cell and

hydraulic actuator. The C-channel was connected to the top plate of the wall using four evenly spaced 12.7 mm (0.5 in.) A307 bolts installed through both top plate members. To insure a tight non-slip bolted connection, 12.7 mm (0.5 in) holes were drilled in the top plates after the walls were positioned.

The data acquisition system connected to the test frame consisted of 8 channels of position and load readings. The data from all 8 channels were recorded with a computer using LabView 6i program. Load readings are obtained from a load cell attached to the hydraulic actuator (channel 1), while deflection at the top of the wall is transferred by the actuator's internal position sensor (channel 2). The remaining six channels were not used.

MONOTONIC TESTING

Monotonic tests were based on the ASTM E564-00 (ASTM 2000) test protocol. This protocol requires ultimate load to be reached in no less than 5 minutes. All walls were tested at 0.76 mm/sec. This corresponded to a time to failure of approximately 7 minutes.

DATA ACQUISITION

An optical measurement instrument based on digital image correlation (DIC) was used to capture and analyze data. DIC is a full-field, non-contact technique for measurement of displacements and strains. The set up consisted of a pair of cameras arranged at an angle to take stereoscopic images of the area of interest as shown in Figure 3. The cameras were externally triggered and connected to a computer where data was recorded. Image files of undeformed and deformed specimen obtained with the DIC set up were analyzed using proprietary software named Vic 3D (Correlated Solutions Inc.). To calculate displacement at any point, a small subset of pixels was used. This subset has a unique light intensity pattern and the DIC software searches the best matching subset in the image of deformed specimen, using mathematical correlation of intensity patterns, from undeformed specimen image. Once the correlation is finished the system returns full field 3D displacement data measured over the visible specimen surfaces and then calculates strains. Surface topography, displacement maps and strain profiles are obtained from the software. Numerical data for any selected point or area in the image could be extracted from the output files so that some other program (e.g., excel, etc.) may be used to analyze the data for that area.

Previous research has analyzed displacement fields (Ambu et al. 2005) or crack propagation (Samarasinghe et al. 1996) and others have validated the system by

conventional methods or mathematically with a model (Sadeq 2002, Choi and Shah 1997). There is a lack of literature which uses DIC for larger sample sizes. As of now the application of DIC is limited to small samples with a viewing area of 100 x 75 mm for concrete samples (Choi and Shah 1997) or 4 x 5 cm (Samarasinghe and Kulasiri, 2000). This study is an attempt to use it for 250 x 250 mm areas in a 2440 x 2440 mm shear wall thereby concentrating on strain near fasteners.

DATA ANALYSIS

The theory of digital image correlation has been described in detail by several researchers and a detailed treatment of the subject can be found in Sutton et al. (1983). The underlying principle of DIC is that the points on the undeformed surface can be tracked to new positions on the image of deformed surface using a least square error minimization technique. It allows measurement of large deformations and strains, far beyond elastic limits of materials. So failure initiating events and at failure strain development may be observed and analyzed. However once the failure occurs, the specimen undergoes large deformation in little amount of time. Tracking the random event would require immediate re-setting of the image acquisition rate to the maximum 5 frames per second, which proved practically impossible. Consequently, the failure events occur between frames acquired by the cameras, and post failure strains, as returned by Vic3d, appear erratic.

Strains are determined by calculating gradients of displacements, u , v and w by correlating the position of speckles in a Cartesian coordinate system. Values of various displacement gradients ($\delta u/\delta x$, $\delta v/\delta y$, $\delta w/\delta z$, $\delta u/\delta y$, $\delta v/\delta x$ etc.) which are used to derive strains are subsequently calculated. The output strain tensor components denoted e_{xx} , e_{yy} , e_{xy} , e_1 and e_2 correspond to strain in x , y directions, shear strain and major and minor principle strains, respectively. In this study local strains in major principle directions (e_1) are considered as they represent the maximum normal strain at a plane and are dependent on the strain in global x and y directions and also on the shear strains at that point.

An area of interest for numerical data analysis was chosen. The selection of area of interest was based on the magnitude of local principle strain in that area hence area which encountered the maximum principle strain on corresponding sides, i.e. on OSB and GWB was selected as area of interest to extract after preliminary analysis. The numerical data underlying in that area and analyze it using other data analysis tools (excel). The area of

interest was rectangular in shape, included 40 data points and had a physical area of 80 – 100 mm². A detailed description and illustration can be found in Sinha (2007).

EEEEP CURVE

An analysis of load deflection curve and Equivalent Energy Elastic Plastic (EEEEP) curve provides useful tools to calculate various parameters of the walls. An EEEP curve is a perfectly elastic plastic representation of the actual response of the specimen. The curve is plotted such that it equals the area under the load deflection curve until failure. This allows a direct comparison of wall performance on energy basis. Figure 4 shows the various points of interests used to derive the EEEP curve. The parameters derived from EEEP curve are listed in Table 2.

TEST MATRIX

A total number of 16 walls were tested monotonically for the project as shown in the test matrix in Table 3. Eleven walls were sheathed on one side with OSB and the other side with GWB (Type A). Five walls were tested with OSB on one side and no sheathing on other side (Type B).

Figure 5 shows the nailing schedule for OSB and GWB and the areas imaged during the wall tests are marked accordingly. The marked area and corresponding roman numerals are the zones that were imaged. The number of walls tested for each zone is listed in Table 3. Figure 5 also shows the fastener configuration and numbering scheme on the OSB side. The fasteners on the GWB side are referred to as the same number as that for the OSB side, but preceded by a prefix S (for screws) and are shown as fasteners filled with black.

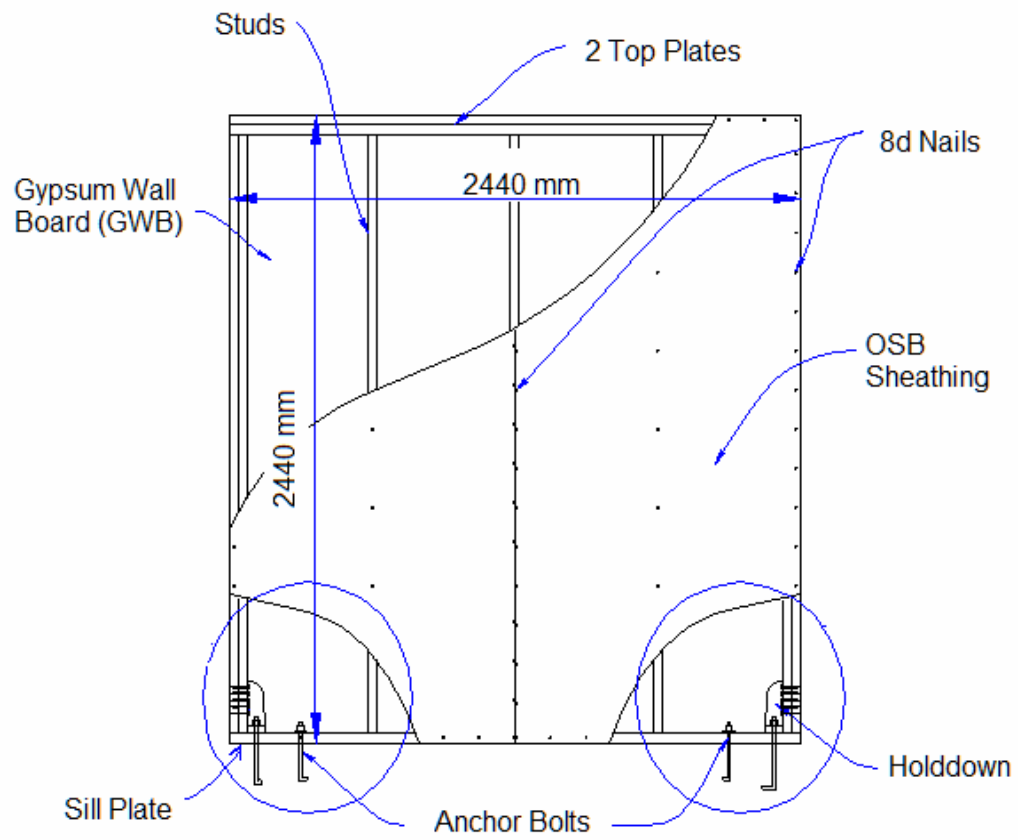


Figure 1. Schematic of shear wall test specimen

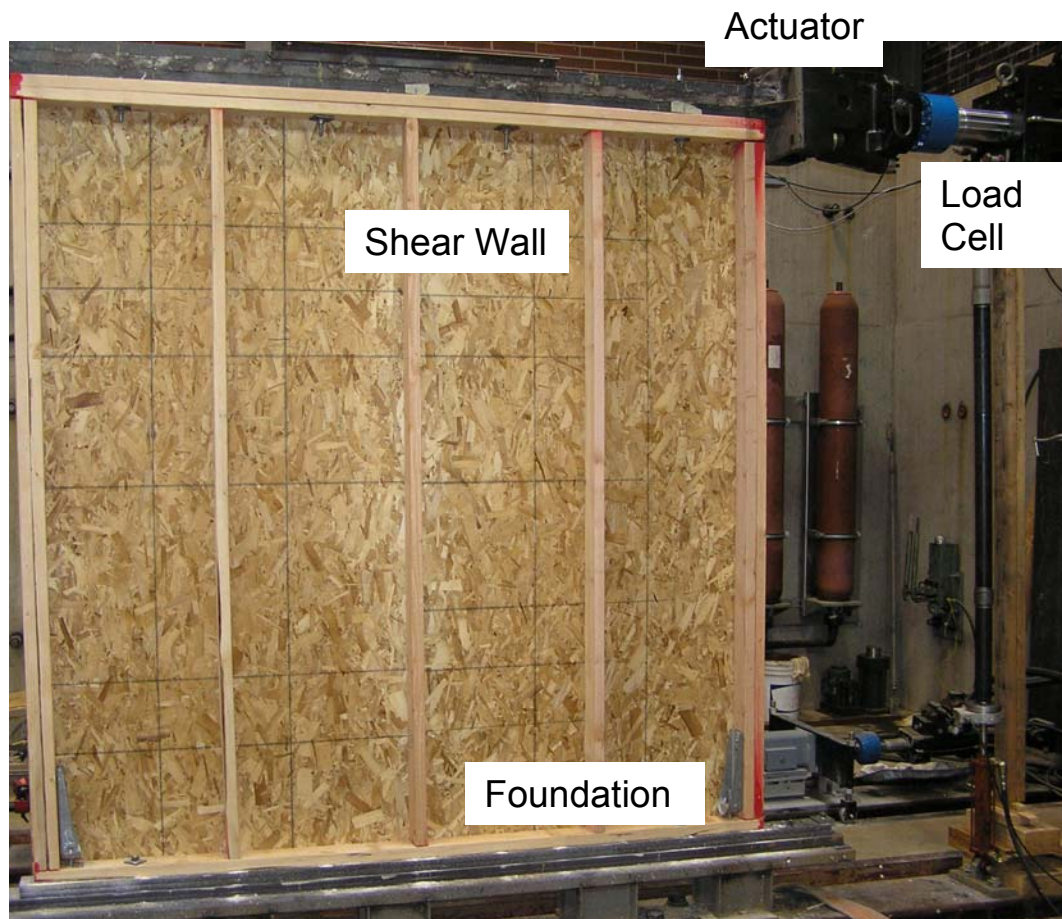
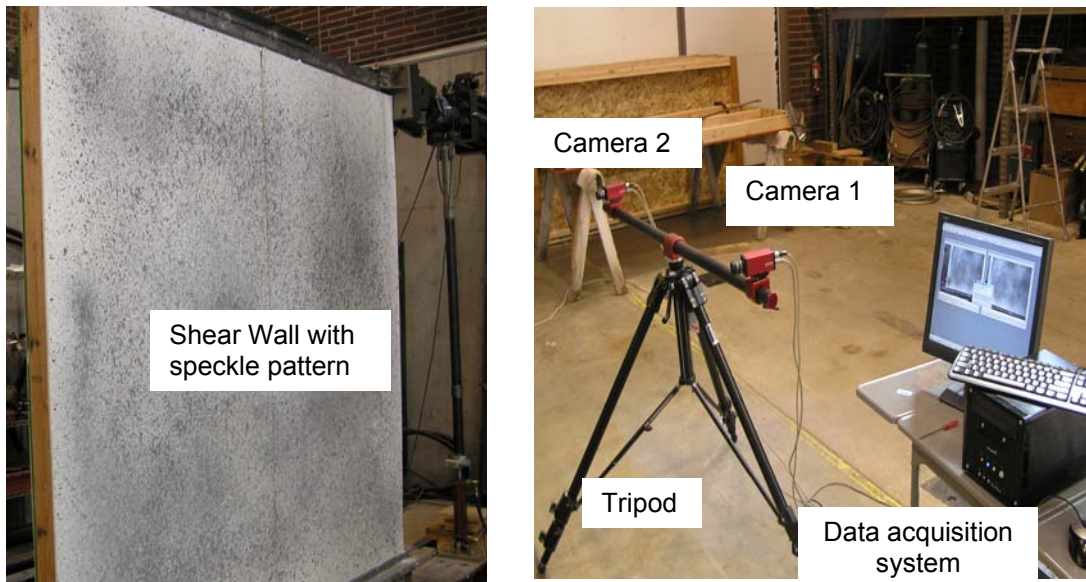


Figure 2. Test Set up in the Lab



(a) Shear Wall with applied speckle pattern

(b) DIC data acquisition system

Figure 3. DIC set up for the shear wall test

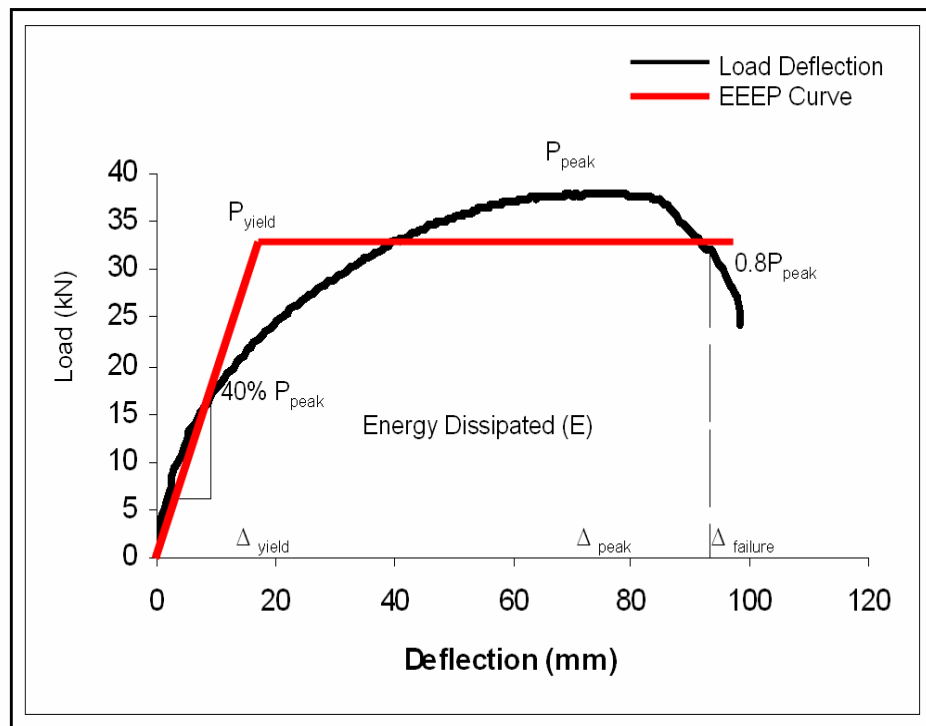


Figure 4. Equivalent Energy Elastic Plastic curve

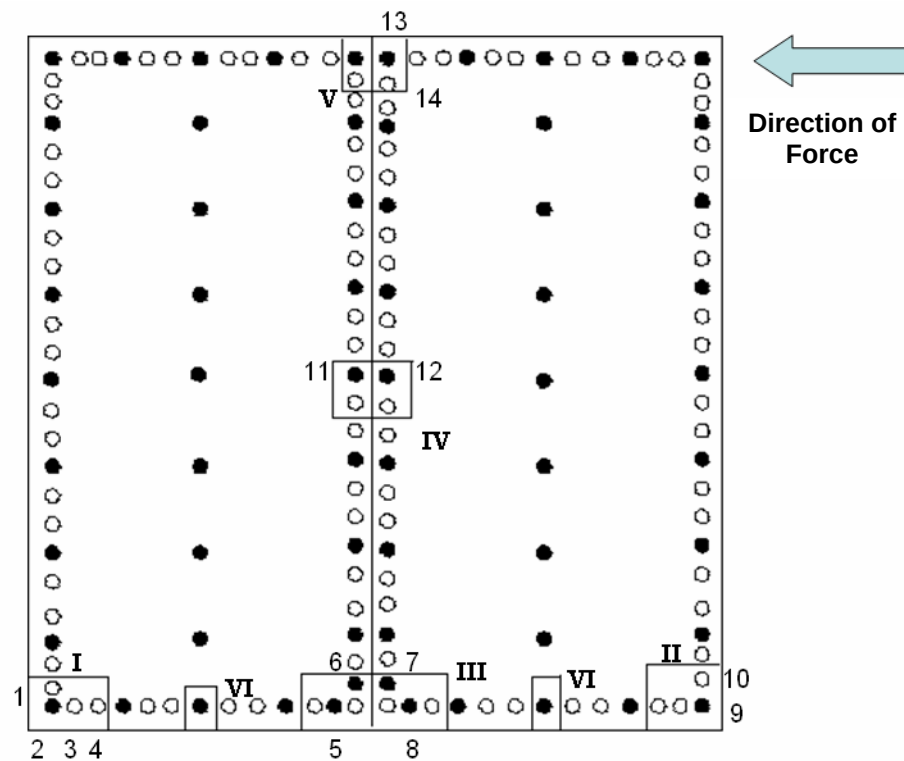


Figure 5. Nailing schedule for OSB and GWB (solid dots) showing imaged areas (roman numerals).

Table 2. EEEP Parameters

Parameters	Units	Description
P_{peak}	kN	Measured peak load
Δ_{peak}	mm	Measured displacement at peak load
Δ_{yield}	mm	Calculated yield displacement from EEEP curve (P_{yield}/K_e)
$\Delta_{failure}$	mm	Measured post peak displacement at 80% peak load
K_e	kN/mm	Calculated elastic shear stiffness ($0.4 P_{peak}/\Delta_{0.4P_{peak}}$)
E	J	Calculated energy under the curve to failure
μ		Calculated ductility factor $\Delta_{failure}/\Delta_{yield}$
D		Calculated ductility ratio $\Delta_{peak}/\Delta_{yield}$

Table 3. Test Matrix

Area no	Description of areas	Number of Walls		
		Type A	Type B	
I	Compression Corner	2	1	
II	Uplift corner	2	1	
III	Center of Sill	2	2	
IV	Mid Wall	2	1*	
V	Middle of top plate	1	1*	
VI	Intermediate studs	2	0	
Note * Two areas were imaged on one wall simultaneously.				

RESULTS AND DISCUSSION

DISPLACEMENTS

Basic data obtained after correlation of images is the displacement in global x, y, and z directions termed as u, v, and w, respectively. Figure 6 shows a typical load displacement diagram for the uplift corner of GWB in wall type A. As seen in the graph the vertical displacement (v) is greater than the horizontal displacement (u). Being the uplift corner the vertical displacement here should be more than the horizontal displacement. As seen in figure 6 the out of plane displacement (w) is negligible as compared to u and v, which is expected as the panel does not deform much out of plane, hence a low value of w. Displacement graphs for other locations on the wall are given in Sinha (2007) (Appendix D) and they all show expected trends.

LOAD SHARING

OSB AND GWB (Wall type A)

Figure 7 represents comparative global load vs. local strain diagram for GWB and OSB panel near the uplift corner (fasteners 9 and S9). As shown in the Figure, OSB experiences lower strains than GWB near the uplift corner throughout the entire period of loading indicating that GWB undergoes more deformation than OSB for a given load.

As the shear wall assembly is loaded GWB, which is stiffer than OSB (Table 1), attracts considerable load, hence undergoes more deformation which results in higher strains till it fails locally. The start of failure of GWB near connection S9 is observed as scattering of data points circled in Figure 7 which is clearly observed at around 25 kN. An apparent change of slope is observed in the curve for OSB at 25 kN. The reason for the change in slope is attributed to load shift towards OSB from GWB as GWB around fastener S9 starts to fail. As the load reaches 30 kN the connection on the GWB side (S9) fails resulting in very high values of optically measured strains. Once the GWB fails, the paper cover tears apart causing the ruptured material within it fall out hence resulting in very high values of optically measured strains.

From Figure 7, consider a ratio of principle strains on corresponding areas in GWB and OSB. Up to 25 kN, which is the linear range of the load strain curve, the strain near fastener S9 is 4 times higher than that of the corresponding OSB side fastener 9, hence the load in GWB is higher than that carried by OSB. Beyond 25 kN, strain in the OSB

increases at a faster rate. This continues up to complete failure of GWB at around 30 kN. Therefore it can be concluded that GWB transfers load during initial low loading, but at higher loads OSB transfer most of the load. This is in accordance to what Johnson (1997) concluded that GWB helps resist shear in the low to moderate loading, but plywood (structural sheathing) resists most of the shear near capacity under monotonic loading

The ratio of strain distribution is different for different regions of the wall because it is based on two factors namely load path and connection stiffness of both panels. However for most of the fasteners the strain in GWB is higher than that of OSB for initial loading period and can be found in Sinha (2007).

OSB (Wall type A and B)

Figures 8 and 9 are global load vs. local strain plots for corresponding areas in wall types A and B around fastener 9 and 5, respectively. The strains in OSB of type B walls are much higher as compared to type A walls at any given load, as observed from Figure 8 and 9. The strain in OSB panels of type A walls are low for initial period of loading and at around 25 kN starts increasing at a faster rate until failure of the wall. The onset of strain in OSB of wall type A is delayed because of the presence of GWB which attracts load initially. In both the graphs (for wall type A) a change of slope is observed at 25 kN which signifies a load shift from GWB to OSB as GWB starts to fail around this load for wall type A.

As observed previously (Fig. 7) the connection at GWB side in wall type A starts to fail around 25 kN. At a similar stage of loading a change of slope is observed in the OSB side for type A walls signifying shifting of load from GWB to OSB as the connections shows signs of failure. Fastener 9 on both types of walls fails early (Fig 7 and 8), close to 30 kN for GWB in wall type A and at around 23 kN in OSB for wall type B, as compared to rest of the fasteners as after that high and erratic value of strains are returned by the optical system. Onset of strains was the earliest near fastener 9 and failure is also initiated in this region rendering this fastener the most critical fastener out of all the fasteners tested. However test of all fasteners around the panel is required to generalize this result.

The strain in wall type B increases steadily from start to failure with no abrupt change in strain during entire loading indicating a ductile failure. Wall type B shows more ductile behavior than wall type A because of the lack of ability of GWB to deform at higher load in wall type A where as OSB in wall type B continues to deform at higher load. Also OSB

panel in wall type B experiences higher strains than the OSB panel for wall type A for a given load. In wall type A, there is higher strain in GWB than in OSB in the initial part of loading. GWB is stiffer than OSB, it attracts load and in turn deformation is higher than OSB. But being brittle, GWB fails at around 60% of the ultimate wall capacity and load shifts to OSB. This is indicated by large change in strain in OSB (Figure 8 and 9). OSB continues to attract load but the strain in OSB increases at a faster rate until failure indicating a much less ductile behavior than that of wall type B.

STRAIN PROFILE

WALL TYPE A

Each picture in figure 10 and 11 are the composite plots showing distribution of principle strain of areas imaged on different walls using DIC superimposed on a wall image and representing their relative positions in GWB and OSB side of wall type A, respectively. Images obtained from seven walls were used to compile the plots, whereas contour plots for the rest of the four walls can be found in Sinha (2007) (Appendix F). The arrow indicates the direction of loading. The loading arrows are reversed because one side (e.g. GWB) is on the back side of the other side (e.g. OSB). All the profiles discussed in this section are the contour profile of principle strain (ϵ_1) in the material. The numbering scheme for the fasteners is shown in figure 5.

Figure 10 (A1 through A4) shows the progressive distribution of strain around fasteners for the GWB up to failure. The dark green color shows that the strain in this area is below the smallest contour scale step (± 0.0075). All the various color contours show the tensile and compressive strain in accordance with contour scale shown in Fig. 10 (± 0.06). Similar profiles for the OSB side are shown in Figure 11 (B1 to B4) which has the same scale of reference as Figure 10. Shades of green indicate no strain and while red and purple indicate compressive and tensile strains, respectively. As seen in both the figures, no significant strain level could be detected in the field of the panel, and large strains are concentrated around the fasteners on both sides of the wall.

At 10 kN (Fig. 10 & 11) there is hardly any noticeable strain in either GWB or OSB. Most of the panel is colored in shades of green, hence almost no detectable strain in that area. As load is increased from 10 kN to 20 kN the OSB side (Fig. 11 B2) does not experience any strain except fastener 9 in zone II, around which a slight increase of strain is observed. However, on the GWB side, the strain starts to appear near fasteners S8, S9 and S6 (Fig. 10 A2). The onset of strain near the fasteners in GWB implies some load is

being transferred through GWB at this initial stage of loading. As GWB is stiffer than OSB, it attracts load and hence higher strains are observed in the GWB panel.

As the load increases to 30 kN, the connections at S9 (zone II) and S6 (zone III) have already failed because numerically high and erratic values of strain are calculated by the optical DIC system. Also the panel corner near S6 is significantly deformed (Fig. 10 A3). Strain near other fasteners such as S8, S7, S11 and S13 are also building up. Strains in the uplift corner (zone II) on the OSB side are clearly visible and are highly concentrated over fastener 9 (Fig. 11 B3). Considerable build up of strain in the panel localized to fastener can be observed along the panel edge at middle part of wall while no strain is observed in the field of the panel. At failure, high strain concentrations around most of the fasteners are observed in GWB (Fig. 10 A4).

WALL TYPE B

Figure 12 shows the strain profile in OSB for wall type B. Data from four walls were used to generate the plot. The plot is generated in a manner similar to that of Figures 10 and 11 and uses the same scale of reference as shown in Figure 12. At 10 kN (Fig. 12 C1) there is hardly any strain in the OSB panel as everything is green. As the load increases to 20 kN, strain has started to build up near fasteners 4, 5, 8, 9 and 13.

Up to 30 kN there is a steady strain build up in the type B walls (Fig. 12 C3) but all localized to the fastener and in the field of the panel the strains are below the detectable range. Nails 5 and 13 have considerable amount of strain and are at the verge of failure as optically recorded deformations are high. Strain is being concentrated around the fasteners which are at the joint of the two panels, making zone III and IV the critical zones for wall type B. At failure most of the nails have strain around them signifying failure of the connections while there is no strain in the field of the panel.

FAILURE PROGRESSION

GWB

Strains are observed around fasteners S9, S8 and S6 at an early stage of loading (Fig. 10 A2). But in terms of magnitude much higher strains are generated near fastener S9 (1%) as compared to S8 (0.1%) and S6 (0.2%). At 30 kN (Figure 10 - A3), the area around S9 has already failed. A look at the load strain curve for that region (Fig. 7) indicates that S9 started failing around 23 kN, i.e. around 60 % of wall capacity and has completely failed around 28 kN.

At 30 kN, strains around other fasteners near the sill plate are high and as the load is increased, the areas around these fasteners also begin to fail. At the ultimate wall capacity (Fig 10-A4), area around S9 has completely failed where as there is sign of failure around other fasteners near sill plate. It is clear from figure 10 that failure of the wall is initiated at S9, i.e. in GWB near the uplift corner (zone II). Therefore S9 in zone II is the critical fastener on the GWB side

The panel at the uplift corner undergoes enormous deformation at failure of the panel, as high strains are recorded in that region. As the GWB fails and the stress increases, the paper cover of GWB tear opens causing the material within it to fall out. Brittle failure of GWB leads to large instantaneous displacement of that part of the panel. Most of the strain is recorded either near the sill and the top plate, while the strains in central portion (zone IV) of the wall in GWB are lower than the top plate (zone V) and the sill (zone II and III). As most of the strain in GWB is around the fasteners at the sill, GWB predominantly transfers load at the sill level.

OSB

During the initial part of loading from 0-10 kN OSB both, type A and B walls, have low strains in the panels (Fig. 11 B1 and 12 C1). As the load increases and reaches 20 kN, type A walls still have strains in the undetectable range (Fig. 11 B2) while type B walls (Fig. 12 C2) start to experience some strain around fasteners. Localized strain fields can be observed around fasteners 4, 5, 8, 9 and 13 in wall type B. At 30 kN steady build up of strain is observed in wall type B. In type A walls (Fig. 11 B3) more built up of strain is observed around the fastener in the uplift corner (Zone III) and over the joint of the panels (Zone IV and V), while in type B walls nails 5 and 13 are on the verge of failure as the deformations recorded optically are high, and strain is being concentrated all around the fasteners which are at the joint of the two panels (Zone III, IV and V). At failure, in figure 12 C4, strains around all the fasteners are observed, while for type A walls (Fig. 11 B4) no strain concentrations are observed near some fasteners in Zone I, III and VI. However, fasteners along the middle of the wall (type A), where the two panel edges meet and the fasteners in the uplift region, have high strain concentrations around them at failure while not much of the strain is concentrated near the fasteners at the sill plate. While OSB transfers load all around but more strain is observed in the middle of the wall where the long edges of two OSB panels meet. As the middle post comprises of a single stud, there is inadequate edge distance for the fasteners in that region hence decreasing the connection stiffness in that region and as a result more strain is observed.

At failure most fasteners exhibit high strains around them, but the failure progression is different in type A and B walls. For OSB in type A walls, the failure starts at the uplift corner near nail 9 at approximately 30 kN (Fig. 11 B3), followed by the nails in the middle of the walls (11-12) where the panels meet, and then fails in zone V. The other fasteners, such as 2, 4, 6, and 7, all show some strain around them at failure but not as much as nail 9, 11 and 12. As onset of strain at the uplift corner nails is earlier than the other nails, making the uplift corner a critical zone for the shear wall assembly. However more tests are needed to confirm this as not all the fasteners were imaged in this study. As more fasteners are imaged in future tests, onset of strain around some other fastener is possible making that the critical fastener. This phenomenon can be due to the fact gypsum fails at that corner first and the load shifts to the OSB panel. Also these walls being fully anchored hence sheathing transfer overturning forces into the wall end studs, and subsequently into the foundation through the hold-downs, which makes the fasteners in the vicinity of hold down critical.

As in case of OSB panels in type A walls, strain near fastener 9 in type B walls also starts to increase in the initial stage of loading and then, as it is loaded further, fails at around 23 kN (Fig. 8), hence is the initiation of failure. For type B walls, the critical zone is the central region of the wall, where the edges of two panels meet as failure occurs there next. Hence for both types of wall nail 9 is of prime importance as the failure is initiated from that region. Also, nails in zones III, IV and V, which are in the middle stud of the wall, experience high amount of strains.

Comparing the failure pattern of OSB in wall type A and B, it is observed that wall type A mostly fails in the middle and some in the bottom near the sill, whereas wall type B fails near the sill and also in the middle of the panels. Analyzing failure progression in type A walls, the shear wall fails first at the uplift corner (zone II) of the gypsum side. Failure continues on to the uplift corner (zone II) of the OSB side. Finally zone V of the OSB side fails leading to failure of wall. The failure in wall type B is uniform over all the fasteners imaged, which is preferred kind of failure as all the fasteners are contributing towards transfer of load to foundation and indicates more efficient design. The presence of GWB in wall type A prevents the OSB in wall type A to fail in a similar manner to that of wall type B. GWB is stiffer than OSB, it attracts load and in turn deformation is higher than OSB. But being brittle, GWB fails at around 60% of the ultimate wall capacity predominantly near the sill plate signifying that it carries more load in that area until its failure than OSB,

and then the load shifts to OSB hence sharing the load with OSB and in turn preventing failure of OSB near the sill plate.

Different load paths ensure different failure progression for type A and B walls. For wall type A, failure is initiated in GWB near fastener S9 (zone II). As the test progresses fastener 9 on OSB sides fails subsequently and then zone V in GWB and OSB fails. For wall type B, failure is initiated in the uplift corner fastener 9 goes to zone III. At failure although all zones show high strain in them but it is zones IV and V which exhibit more damage.

CONTRIBUTION OF GWB

A summary of results from all the 16 monotonic tests are presented in Table 4. Typical load deflection curves for both wall types are shown in Figure 13. As shown in Table 4 and Figure 13, the contribution of GWB towards strength of wall is marginal (0.8%), whereas elastic shear stiffness increased by 50%. As GWB is stiffer than OSB, it contributes to the overall stiffness of the wall but not towards the strength. As shown in Table 4 GWB in the shear wall system reduces the yield, peak and failure displacements (deformation capacity) by 27%, 18% and 13%, respectively. As a result the walls sheathed only with OSB dissipate more energy on its way to failure than a wall with both OSB and GWB. This is observed through Figure 13 as the wall type B curve has greater area under it than wall type A. This is because GWB being brittle fails early and does not provide any resistance after its failure and restricts the overall displacement of the wall, whereas OSB continues to provide resistance and deflects a lot more before completely failing. Uang and Gatto (2003) found 12% increase in shear wall strength, 60% increase in initial stiffness of the wall and a decrease in deformation capacity of 31% by adding GWB. The differences are probably due to loading conditions and types of fasteners used.

Toothman (2003) and Karacabeyli & Ceccotti (1996) also found an increase in strength of walls when GWB is added. Toothman also found a decrease in deformation capacity of the wall by 10% which is similar to the current study. Toothman concluded that energy dissipated by both types of walls were approximately equal. The results of this study are different than Toothman (2003) and Karacabeyli & Ceccotti (1996) due to variation in size of walls and different fasteners used for attaching OSB and GWB to the frame. This study uses ring shank nails and standard dry wall screws as opposed to smooth shank nails, for both OSB and GWB, used by Toothman (2003) and Karacabeyli and Ceccotti

(1996). Ring shank nails have greater withdrawal values so a higher amount of energy is needed for failure.

As shown in table 4, the yield displacement decreases on addition of GWB by an average value of 6.5 mm. Also the peak displacement and failure displacement decreases by an average of 13.4 mm and 12 mm, respectively on addition of GWB in shear wall assembly. When GWB is included the ductility factor increased by a substantial amount (20%) and ductility is increased by 13%. This can be attributed to the increase in elastic stiffness, which decreased yield displacement. Ductility values alone do not provide much insight into the performance of the walls and is a function of elastic stiffness, yield displacement, and failure displacement. Elastic stiffness can vary with the amount of initial load, which affects the yield point and in turn ductility. As Ductility factor is a ratio of failure displacement to yield displacement, small decrease in yield displacement tend to have a major effect on ductility values. In this study the yield displacement decreases by 27% which in turn increases the ductility parameters. Also seen in Table 4, ductility of wall type A is greater than that of B while the energy dissipated to failure is less. This is due to standard way of calculation of ductility parameters which is dependent on yield displacement, failure displacement and peak displacements which in turn also have standard methods for calculation. Numerical parameters should always be looked in conjunction with graphical tools available to determine the true characteristics of the walls; hence all the parameters should be looked into in conjunction and not in isolation.

For wall types A and B the load path is different as presence of GWB in the assembly alters the way load is carried by the shear wall system. For the sake of redundancy in the system wall type A, with dual load paths, one through GWB and another through OSB are preferred. Shear wall is designed assuming that transfer of load is through OSB only, presence of GWB ensures redundancy but as GWB transfers bulk of the loading initially, it defeats the purpose of the design.

Inferred from figures 8 and 9 is the fact that there is a steady build up of strain in OSB panel of type B walls till failure. The onset of strains is delayed when GWB is present in shear wall assembly. Hence after the onset of strain, the deformation of panel is high in a short duration of time while a steady build up in wall type B ensures more ductile behavior of the panel. The magnitude of strain is also greater for wall type B than that of type A walls for a given load.

For the assumption in design that OSB carries all the load during a seismic event to be true, the load strain curve for corresponding areas for OSB in wall type A and B has to be similar and should have the same generic shape. The differences in the two curves indicate an aspect of load sharing between structural sheathing and GWB which is not incorporated in the design process.

The failures in the OSB panels were all around the fasteners and no strains were observed in the field. The high concentration of strain around the fastener in OSB indicates stress concentration near the fasteners while there is none in the field of panel. Hence the whole panel is not being utilized for the purpose of transferring shear and only the area in the vicinity of fasteners is being used. The stress needs to be carried by the whole panel to justify an efficient design and this could be done by designing a panel which addresses this issue, or changing the nailing pattern which ensures adequate strength is developed in the wall and also that a majority of the panel area is being utilized in carrying the load.

Although GWB does not increase the load carrying capacity of the wall, does reduce the capacity to dissipate energy and alters the way OSB carries load, it can not be done away with in practice because of aesthetics and fire rating of the structure. However arrangements can be made to ensure that the load is transferred only through OSB and GWB is structurally isolated. This can be achieved by designing some innovative connections which will not rigidly attach GWB to the wooden frame behind and allowing it to move as the wall moves without deforming. Another option might be to sheath both sides with OSB and then on one side attach the GWB on the top of OSB but not attaching it to the frame structurally so that stiffness of either side of wall is approximately same and hence ensure equal sharing of load. Further research in this field is required to develop a more accurate and efficient design procedure.

The walls tested although might not be exact replica of the walls constructed in actual practice, but are standard walls. Based on these walls the current code values are determined for design. By testing code compliant standard walls, uniformity in design is ensured and it gives a reference for the data to be compared across all the walls. Practically, it is impossible to test the entire different wall configurations existing in the field. However the walls in practice will show similar trends as the standard walls and shall provide more than satisfactory estimate of the shear wall behavior.

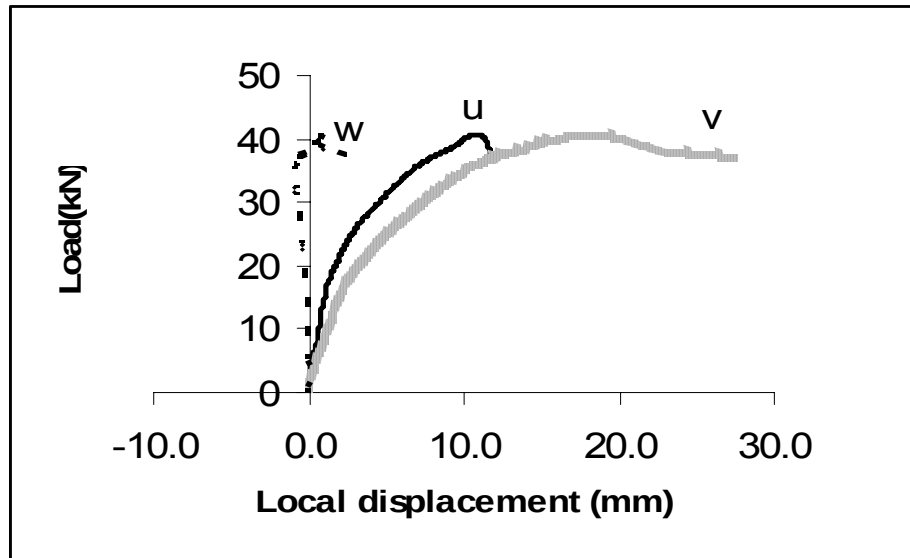


Figure 6. Load vs. local displacement curve for uplift corner in GWB

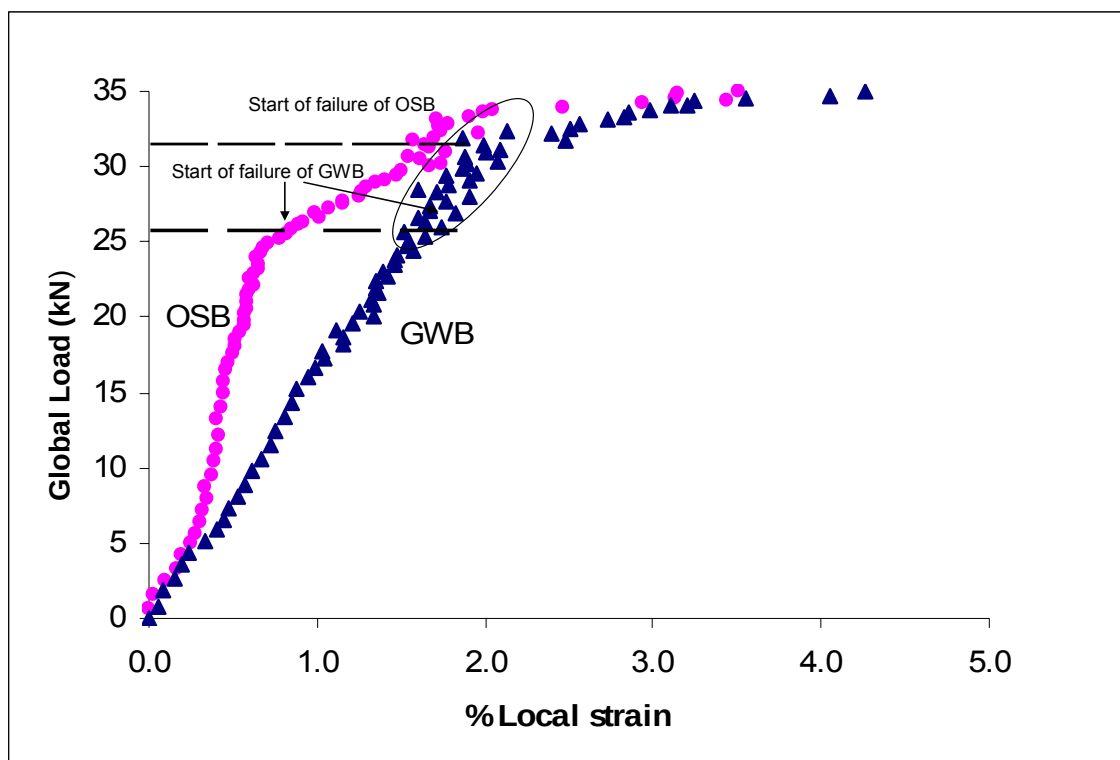


Figure 7. Global Load vs. Local principle strain for uplift corner (Fastener S9 and 9)

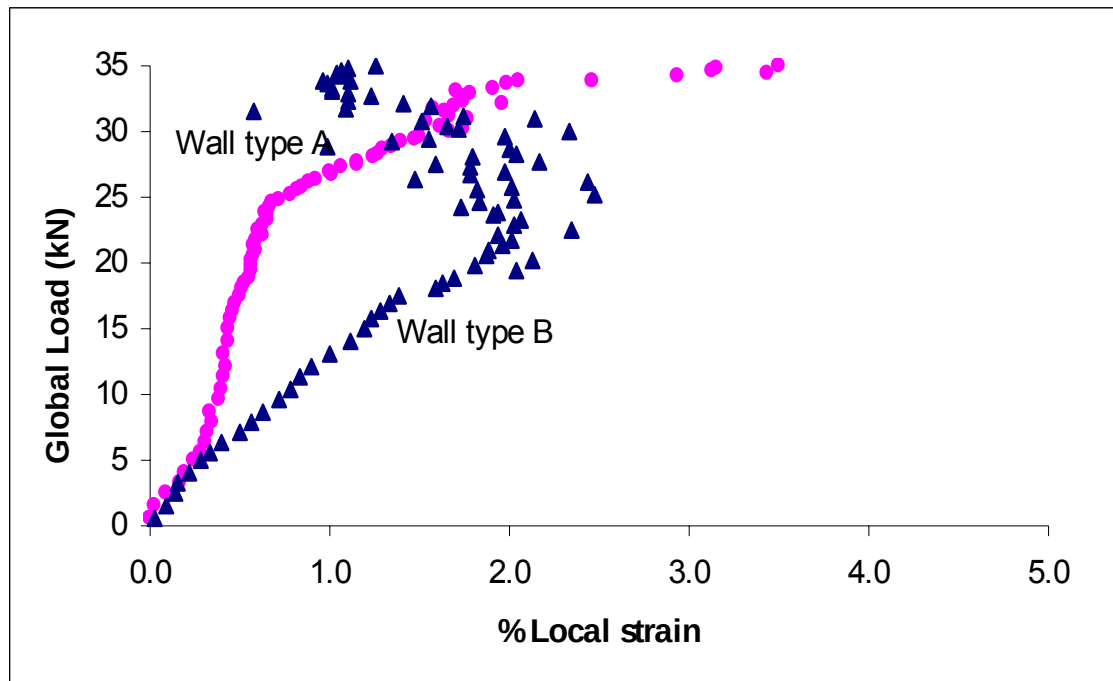


Figure 8. Global Load vs. local principle strain (in OSB) plot for fastener 9

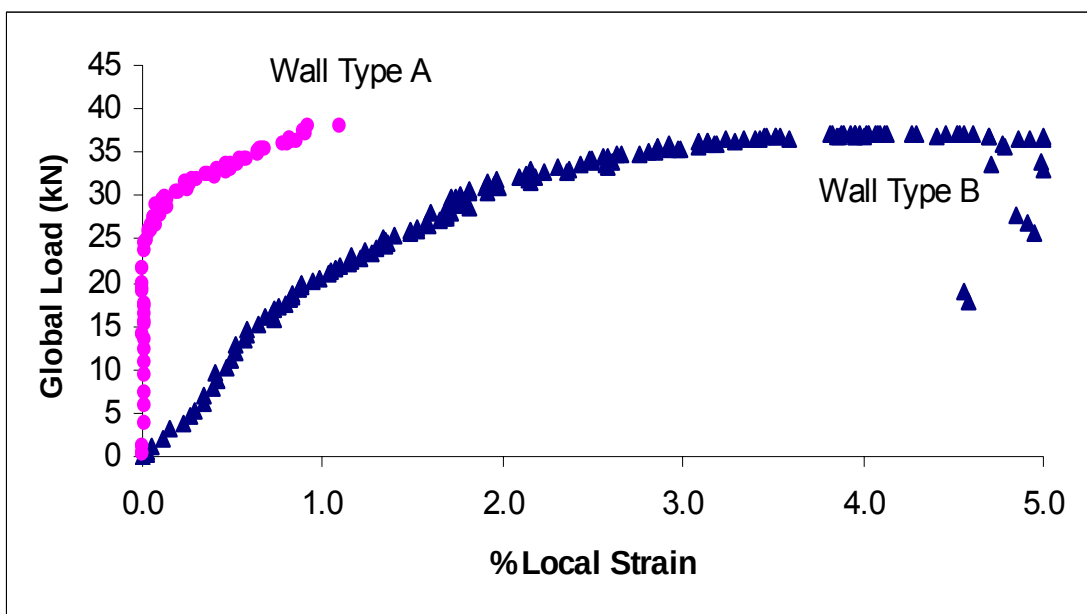


Figure 9. Global load vs. local principle strain in OSB plot for fastener 5

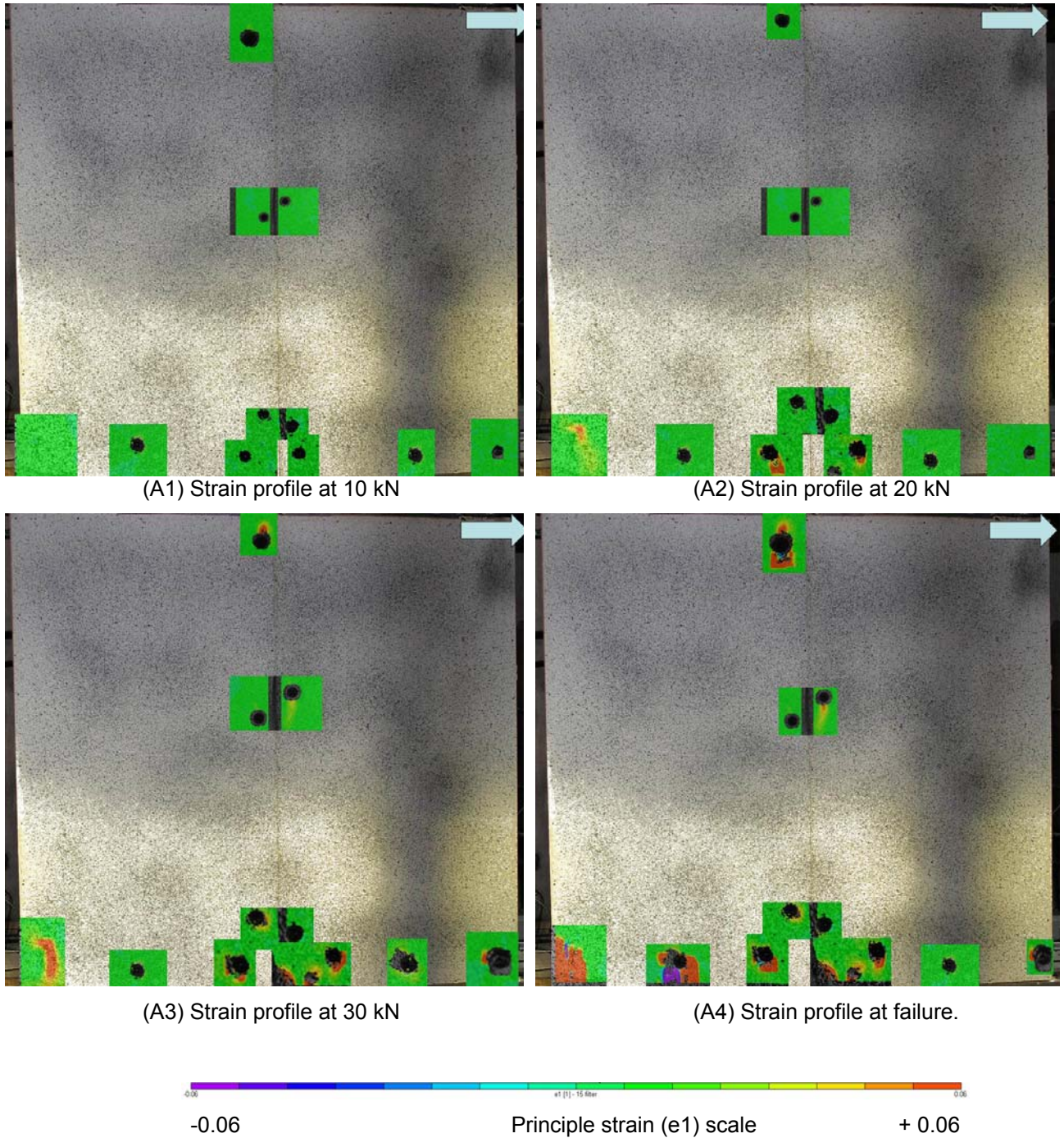
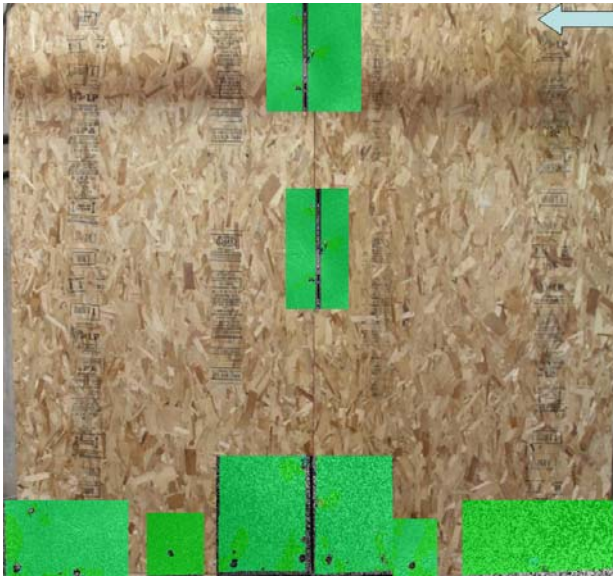


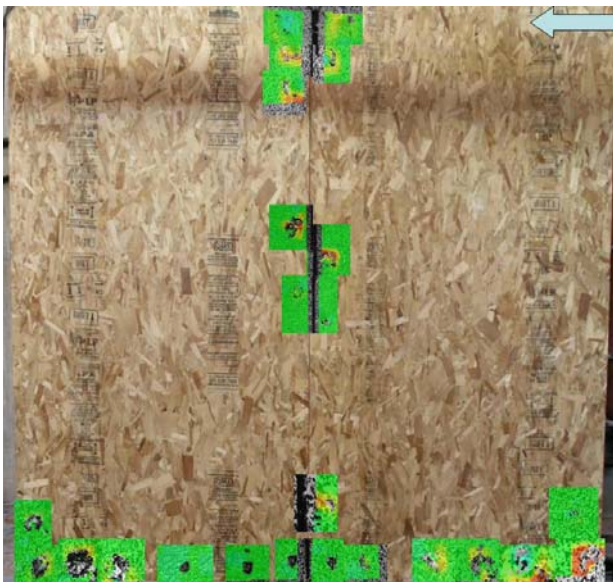
Figure 10. Strain profile in GWB at various stages of loading



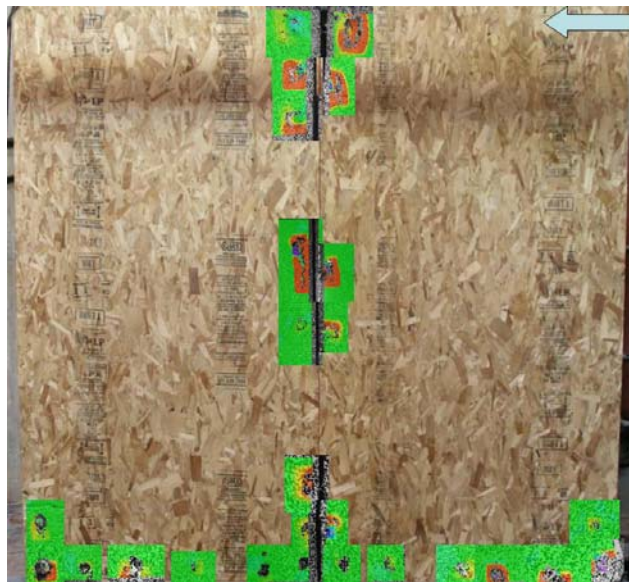
(B1) Strain profile at 10 kN



(B2) Strain profile at 20 kN



(B3) Strain profile at 30 kN



(B4) Strain profile at failure

Figure 11. Strain profile in OSB (Wall type A) at various stages of loading

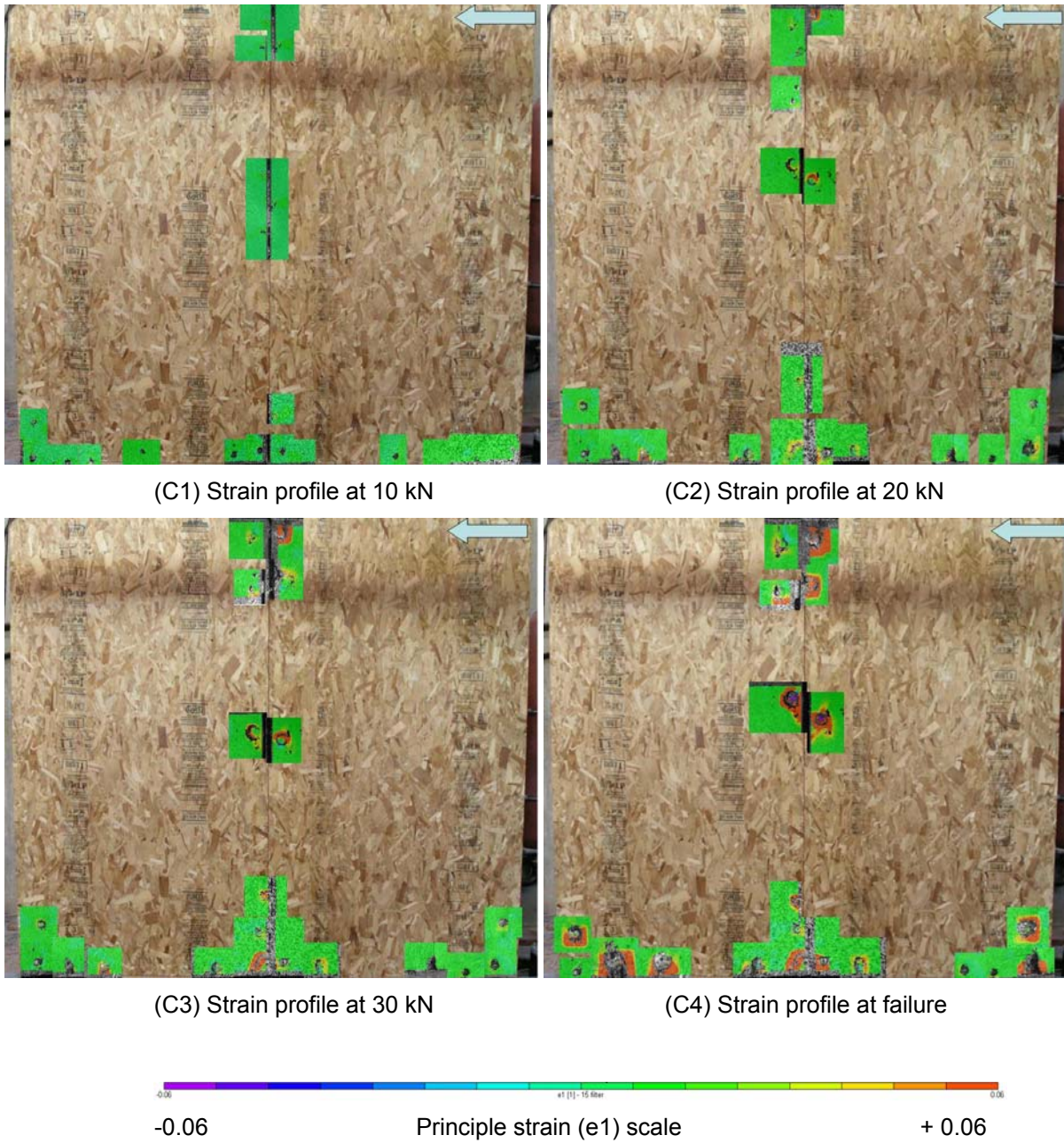


Figure 12. Strain profile in OSB (Wall type B) at various stages of loading

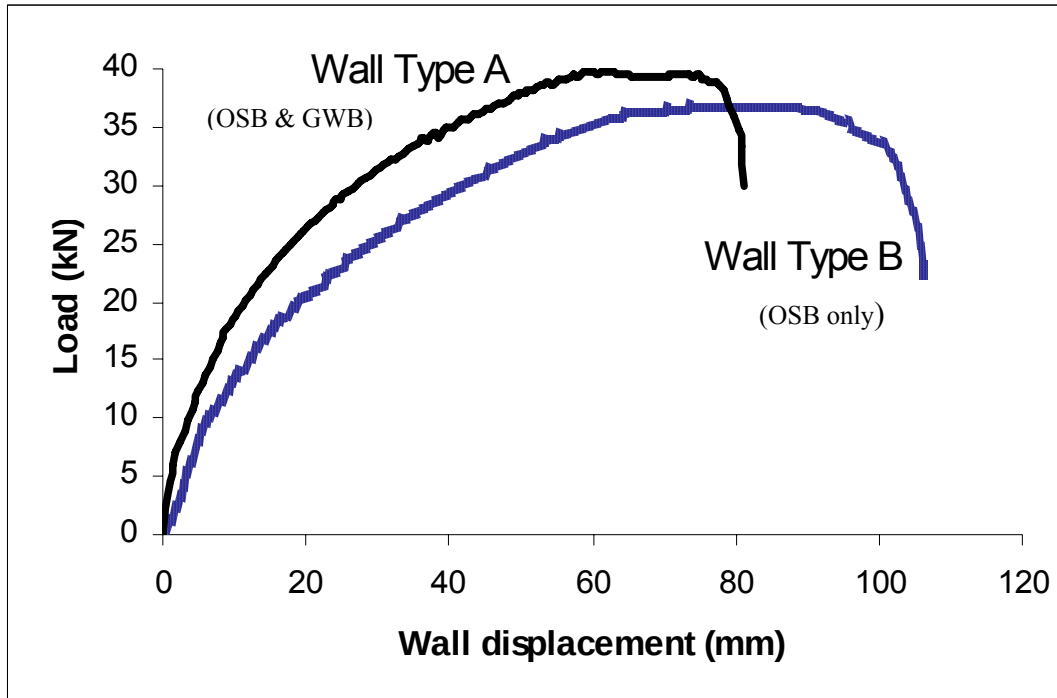


Figure 13. Typical Load Deflection curve for Type A & B Walls.

Table 4. EEEP curve parameters

Parameters (Average Value)	Walls with OSB only (Type B)	Walls with OSB and GWB (Type A)	% Increase (+) Decrease (-)	COV %	
				Type A	Type B
P_{peak} (kN)	38.4	38.7	0.8	6.8	2.6
K_e (kN/mm)	1.43	2.14	49.7	13.4	13.9
μ	4	4.81	20.3	18.5	18.4
D	3.25	3.68	13.2	16.2	17.5
Δ_{peak} (mm)	75.9	62.5	-17.7	11.2	7.4
$\Delta f_{failure}$ (mm)	93.4	81.5	-12.7	12.3	7.2
Δ_{yield} (mm)	23.8	17.3	-27.3	17.3	14
Energy dissipated (J)	2742	2466	-10.1	18.3	11.7

CONCLUSIONS

Conclusions based on results of this study include:

1. Load is shared by both OSB and GWB initially in a shear wall assembly. GWB fails at around 60% of the actual wall strength and once GWB fails load shifts to OSB.
2. There is a steady build up of strain in wall type B from start to failure and there is no abrupt change in strain during entire loading indicating a ductile failure. Wall type B shows more ductile behavior than wall type A because of the lack of ability of GWB to deform at higher load in wall type A where as OSB in wall type B continues to deform at higher load.
3. Higher strains are observed in GWB during initial part of loading. GWB is stiffer than OSB, hence attracts more load and in turn deformation is higher than OSB. OSB in walls with GWB (type A) experiences lower strains than the walls with OSB only (type B) throughout the loading. The strain in OSB in wall A increases at a higher rate after the failure of GWB.
4. Strains in OSB and GWB both are concentrated around the fasteners. Strains in the field of the panel were below the detection limit.
5. The load path for both wall types is different. Failure in wall type A starts at the uplift corner in GWB and then moves to the uplift corner in OSB. Finally the walls fail at middle of top plate for GWB and OSB both. In wall type B the failure is initiated at the uplift corner in OSB followed by middle region at sill level and ends up at middle section of wall where two panels meet. The uplift corner fasteners are of prime importance in both types of wall and panels.
6. Gypsum wall board (GWB) does not contribute towards overall strength of the shear wall, but it increases the stiffness of the wall by 50%. GWB is stiffer than OSB, and hence considerably contributes to stiffness.
7. Ductility factor (μ) of the system increases by 20% and the ductility of the system increases by 13% while energy dissipated by the wall decreases when GWB is included in the shear wall assembly. GWB being brittle reduces the ability to deform before failing and hence 18%, 13% and 27% decrease is observed in peak, failure and yield displacements, respectively.

Recommendations based on the results of this study include:

1. Further tests and supporting results are needed to generalize the failure progression pattern for both types of walls.
2. All the walls had ringed shank nails which eliminate withdrawal as a mode of failure. Further testing is required with conventional nails for a conclusion that could be generalized.
3. Cyclic and dynamic tests of walls using DIC should be done to provide a complete picture of shear wall behavior during a seismic event.
4. Study should be conducted on all the other fasteners which are omitted in this study. Study on other fasteners could reveal more about failure progression and identify new critical zones.
5. Effect of GWB in shear wall system needs to be considered, either by incorporating it in the design for damage limitation or by structurally detaching GWB from the shear wall frame by means of innovative connections.
6. The aspect of load sharing needs to be quantified to develop efficient design procedure.
7. More efficient connection or panel or both designs are needed to utilize the whole panel for shear transfer.

BIBLIOGRAPHY

- Ambu, R., Aymerich, F., and Bertolino, F. (2005). "Investigation of the effect of damage on the strength of notched composite laminates by digital image correlation" *Journal of Strain Analysis*, 40(5), 451-461.
- American Forest and Paper Association (AFPA). (2001). "National design specification® for wood construction." Washington, D.C.
- American Society of Testing & Materials (ASTM). (2000). "Standard method of static load test for shear resistance of framed walls for buildings." *ASTM E 564-00*, West Conshohocken, PA.
- Choi, S. and Shah, S. P. (1997). "Measurement of deformations on concrete subjected to compression using image correlation." *Experimental Mechanics*, 37(3), 307-313.
- Cobeen, K., Russell, J., and Dolan, D.J. (2004). "Recommendations for Earthquake Resistance in the Design and Construction of Woodframe Buildings." *CUREE Publication No. W-30b*, Richmond, CA.
- Correlated Solutions Inc. *Vic 3D User Manual*. West Columbia, SC.
- Deng, Y.H. and Furuno, T. (2001). "Properties of gypsum particleboard reinforced with polypropylene fibers." *Journal of Wood Science*, 47(6), 445-450.
- Filiatrault, A. (2002). "Woodframe project testing and analysis: Literature review." *CUREE Publication No. W-03*, Richmond, CA.
- Johnson, A.C. (1997). "Monotonic and cyclic performance of long shear walls with openings." MS Thesis, Virginia Polytechnic Institute and State University, Blacksburg, VA.
- Karacabeyli, E. and Ceccotti, A. (1996). "Test results on Lateral Resistance of Nailed Shear Walls." *Proc. 1996 International Wood Engineering Conference*, New Orleans, LA.
- Sadeq, S. (2002). "Soil structure interaction in transparent synthetic soils using digital image correlation." PhD thesis, Polytechnic Univ., Brooklyn, NY.
- Samarasinghe, S., Kulasiri, G. D. and Nicolle, K. (1996). "Study of Mode-I and Mixed-Mode Fracture in Wood using Digital Image Correlation Method." *Proceedings of 1996 International Wood Engineering Conference*, New Orleans, LA.
- Samarasinghe, S. and Kulasiri, G. D. (2000). "Displacement fields of wood in tension based on image processing: Part 1." *Silva Fennica*, 34(3), 251-259.
- Schierle, G.G. (2002a). "Woodframe Project: Case studies." *CUREE Publication No. W-04*, Richmond, CA.
- Schierle, G.G. (2002b). "Northridge Earthquake Field Investigations: Statistical Analysis of Woodframe Damage." *CUREE Publication No. W-09*, Richmond, CA.

- Sinha, A. (2007). "Strain Distribution in OSB and GWB in Wood Frame Shear Walls." MS thesis, Oregon State University, Corvallis, OR.
- Sutton, M.A., Wolters, W.J., Peters, W.H., Rawson, W.F., and McNeill, S.R. (1983). "Determination of displacements using an improved digital image correlation method." *Image and Vision Computing*, 1(3), 133-139,
- Toothman, A.J. (2003). "Monotonic and Cyclic performance of light- frame shear walls with various sheathing materials." MS Thesis, Virginia Polytechnic Institute and State University, Blacksburg, VA.
- Uang, C.M., and Gatto, K. (2003). "Effects of Finish Materials and Dynamic Loading on theCyclic Response of Woodframe Shearwalls." *Journal of Structural Engineering*, 129(10), 1394-1402.
- Wolfe, R.W. (1983). "Contribution of Gypsum Wallboard to the Racking Resistance of Light-Frame Walls." *FPL – 439*, U. S. Department of Agriculture, Forest Product laboratory, Madison, WI.
- Younquist, J.A. (2000). "Wood based composites and panel products" Wood as an engineering material." *General Technical Report 113*, U.S. Department of Agriculture, Forest Service, Forest Products Laboratory, Madison, WI.

APPENDICES

Appendix A - Wall Construction

A total of sixteen 2440 x 2440 mm walls were constructed to be tested monotonically. All 2x4 framing members were kiln-dried Douglas Fir-Larch Stud grade. Two 2x4 studs were used at each end of all walls except for the sill plate which was made of only one 2x 4. The walls also used one intermediate 2x4 stud at the center of the wall and at center of each panel. Framing studs were spaced at 610 mm on center, and were connected to the sill plate and first top plate using two 16d (3.33x82.6 mm) nails per connection, driven through the plates and into the end grain of the stud. A second top plate was connected to the first top plate using 16d nails at 610 mm on center. The walls were sheathed using two 1220x2440x11.1 mm oriented strand board (OSB) panels that were attached vertically to the wall frame while spaced 3.2 mm apart. The 24/16 APA rated OSB panels were connected to the wall frame using ring shank sheather plus 8d nails spaced 102 mm on center along the panel edges and 305 mm along the intermediate studs. The walls were additionally sheathed with two 1220x2440x12.7 mm GWB (GWB) panels installed vertically on the face opposite to the OSB structural panels. The gypsum panels were attached to the framing with bugle head coarse wallboard screws (2.31x41.3 mm) spaced 305 mm on center along the panel edges and intermediate studs. Sheathing to framing connections was staggered. Double end studs were required as the walls were with hold-downs, and were connected together using 16d nails at 305 mm on center. Framing nails were full round head, strip cartridge, and smooth shank SENCO® nails that were driven using a SENCO® SN 65 pneumatically driven nail gun. While framing nail were full round head, strip cartridge, ring shank Stanley sheather plus nails.

The modulus of elasticity for each 2x4 calculated and the studs were numbered and drawn for construction randomly. The modulus of elasticity of the lumber was determined by a simple flexure test. Each specimen of lumber was simply supported at 1.2 m (4 ft) on center with two point loads applied to the specimen at 0.4 m (16") apart pneumatically. The set up weighing 222.5 N (50 lbs) is allowed to rest on the specimen and the deflection is recorded. The load was then increased to around 450 N and the deflection recorded again. The modulus of elasticity (E) was determined by computing the stiffness of the piece from the results of the flexure test (Fig. A2).

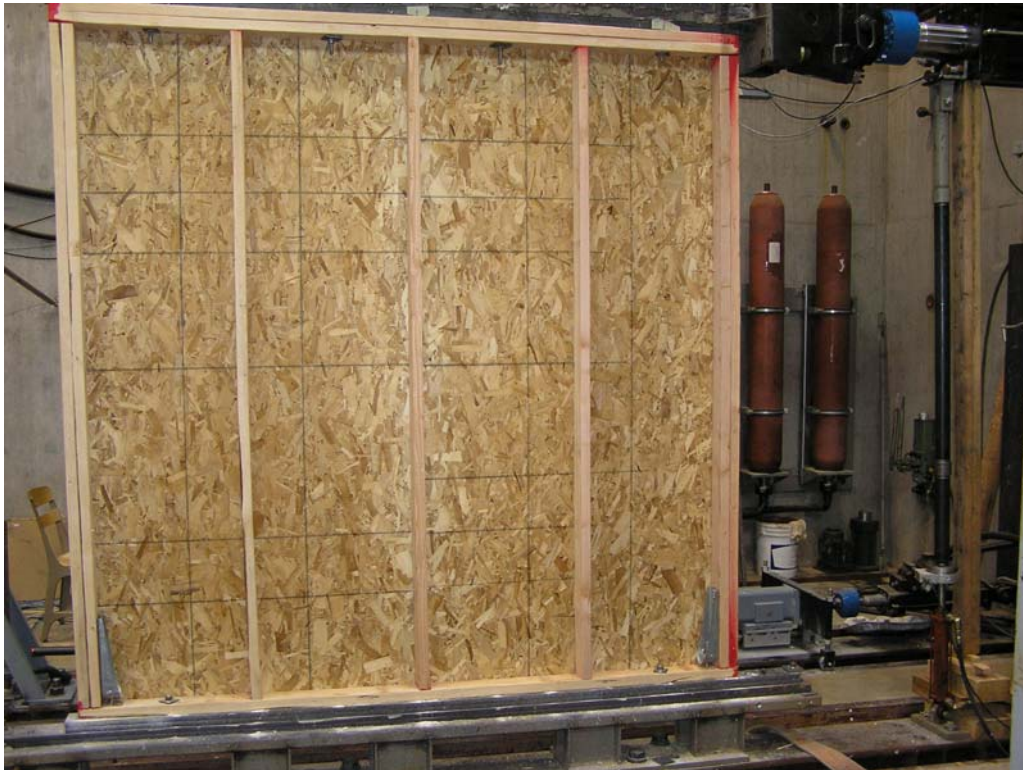
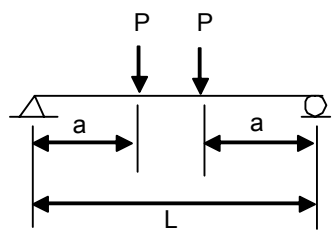


Figure A1. Wood frame shear wall construction.



$$\Delta = \frac{Pa}{24EI}(3L^2 - 4a^2)$$

$$k = \frac{P}{\Delta}$$

$$E = \frac{ka}{24I}(3L^2 - 4a^2)$$

Figure A2. Flexure Test set up and equations

Where the variables are defined in the diagram are;

Δ = deflection at $L/2$ $a = L/3$

E = modulus of elasticity

k = stiffness

I = moment of inertia.

The dimension lumbars procured were in two lengths 2440 mm (96") and 2337 mm (92") and are marked with a prefix S and T respectively while numbering. The MOE values are listed in Table A1 and A2 for S and T series, respectively. All pieces of lumber were randomly used to construct the test walls. The 2x4 pieces which were 2337 mm were cut 12.7 mm (0.5") off of one end to make it ready to use. Due to the small sample size, the random assignment of members imposes a greater probability of experimental error. Figure A3 and Table A3 show the configuration of the constructed walls along with the pieces used for construction of each wall, respectively.

Table A1. MOE values 2x4(S-series)

Sample no.	Deflection (in)	E x 10⁶(psi)	E (MPa)
S1	0.122	1.634	11268
S2	0.167	1.194	8232
S3	0.103	1.936	13347
S4	0.092	2.167	14942
S5	0.149	1.338	9226
S6	0.155	1.286	8869
S7	0.118	1.690	11650
S8	0.111	1.796	12385
S9	0.127	1.570	10824
S10	0.148	1.347	9288
S11	0.109	1.829	12612
S12	0.144	1.385	9546
S13	0.089	2.240	15446
S14	0.119	1.675	11552
S15	0.103	1.936	13347
S16	0.146	1.366	9416
S17	0.136	1.466	10108
S18	0.116	1.719	11851
S19	0.160	1.246	8592
S20	0.090	2.215	15274
S21	0.101	1.974	13611
S22	0.162	1.231	8486
S23	0.129	1.546	10657
S24	0.133	1.499	10336
S25	0.092	2.167	14942
S26	0.125	1.595	10998
S27	0.116	1.719	11851
S28	0.132	1.510	10414
S29	0.095	2.099	14470

Table A1(continued). MOE values 2x4 (S-series)

Sample no.	Deflection		E (MPa)
	(in)	E x 106(psi)	
S30	0.108	1.846	12729
S31	0.102	1.955	13477
S32	0.092	2.167	14942
S33	0.088	2.266	15621
S34	0.108	1.846	12729
S35	0.102	1.955	13477
S36	0.092	2.167	14942
S37	0.088	2.266	15621
S38	0.108	1.846	12729
S39	0.142	1.404	9681
S40	0.086	2.318	15985
S41	0.150	1.329	9165
S42	0.122	1.634	11268
S43	0.126	1.582	10910
S44	0.114	1.749	12059
S45	0.128	1.558	10740
S46	0.089	2.240	15446
S47	0.144	1.385	9546
S48	0.099	2.014	13886
S49	0.108	1.846	12729
S50	0.105	1.899	13092
S51	0.118	1.690	11650
S80	0.167	1.194	8232
S82	0.131	1.522	10494
S83	0.105	1.899	13092
S84	0.129	1.546	10657
S85	0.121	1.648	11361

Table A2 MOE values for 2x4 (T series)

Sample no.	Deflection (in)	E x 10 ⁶ (psi)	E (MPa)
T1	0.109	1.829	12612
T2	0.123	1.621	11176
T3	0.118	1.690	11650
T4	0.097	2.055	14172
T5	0.110	1.813	12497
T6	0.171	1.166	8039
T7	0.149	1.338	9226
T8	0.173	1.152	7946
T9	0.113	1.764	12165
T10	0.162	1.231	8486
T11	0.109	1.829	12612
T12	0.124	1.608	11086
T13	0.118	1.690	11650
T14	0.092	2.167	14942
T15	0.137	1.455	10034
T16	0.166	1.201	8281
T17	0.131	1.522	10494
T18	0.155	1.286	8869
T19	0.119	1.675	11552
T20	0.132	1.510	10414
T21	0.155	1.286	8869
T22	0.129	1.546	10657
T23	0.104	1.917	13218
T24	0.140	1.424	9819
T25	0.111	1.796	12385
T26	0.139	1.434	9890
T27	0.128	1.558	10740
T28	0.149	1.338	9226
T29	0.156	1.278	8812
T30	0.158	1.262	8701
T31	0.140	1.424	9819
T32	0.127	1.570	10824
T33	0.103	1.936	13347
T34	0.172	1.159	7992
T35	0.123	1.621	11176
T36	0.100	1.994	13747
T37	0.141	1.414	9750
T38	0.099	2.014	13886
T39	0.124	1.608	11086
T40	0.111	1.796	12385

Table A2 (continued). MOE values for 2x4 (T series)

Sample no.	Deflection (in)	E x 10 ⁶ (psi)	E (MPa)
T41	0.150	1.329	9165
T42	0.109	1.829	12612
T43	0.122	1.634	11268
T44	0.097	2.055	14172
T45	0.171	1.166	8039
T46	0.115	1.734	11954
T47	0.167	1.194	8232
T48	0.132	1.510	10414
T49	0.166	1.201	8281
T50	0.150	1.329	9165
T51	0.089	2.240	15446
T52	0.156	1.278	8812
T53	0.157	1.270	8756
T54	0.152	1.312	9044
T55	0.110	1.813	12497
T56	0.132	1.510	10414
T57	0.106	1.881	12969
T58	0.091	2.191	15106
T59	0.172	1.159	7992
T60	0.155	1.286	8869
T61	0.121	1.648	11361
T62	0.129	1.546	10657
T63	0.118	1.690	11650
T64	0.113	1.764	12165
T65	0.160	1.246	8592
T66	0.205	0.973	6706
T67	0.120	1.662	11456
T68	0.141	1.414	9750
T69	0.158	1.262	8701
T70	0.125	1.595	10998
T71	0.103	1.936	13347
T72	0.108	1.846	12729
T73	0.111	1.796	12385
T74	0.130	1.534	10575
T75	0.152	1.312	9044
T76	0.123	1.621	11176
T77	0.136	1.466	10108
T78	0.157	1.270	8756
T79	0.080	2.492	17184
T80	0.143	1.394	9613

Table A2 (continued). MOE values for 2x4 (T series)

Sample no.	Deflection (in)	E x 10 ⁶ (psi)	E (MPa)
T81	0.118	1.690	11650
T82	0.206	0.968	6673
T83	0.128	1.558	10740
T84	0.137	1.455	10034
T85	0.121	1.648	11361
T86	0.158	1.262	8701
T87	0.084	2.374	16365
T88	0.125	1.595	10998
T89	0.155	1.286	8869
T90	0.126	1.582	10910
T91	0.160	1.246	8592
T92	0.129	1.546	10657
T93	0.106	1.881	12969
T94	0.158	1.262	8701
T95	0.179	1.114	7680
T96	0.166	1.201	8281
T97	0.098	2.035	14027
T98	0.105	1.899	13092
T99	0.136	1.466	10108
T100	0.152	1.312	9044
T101	0.118	1.690	11650
T102	0.131	1.522	10494
T103	0.109	1.829	12612

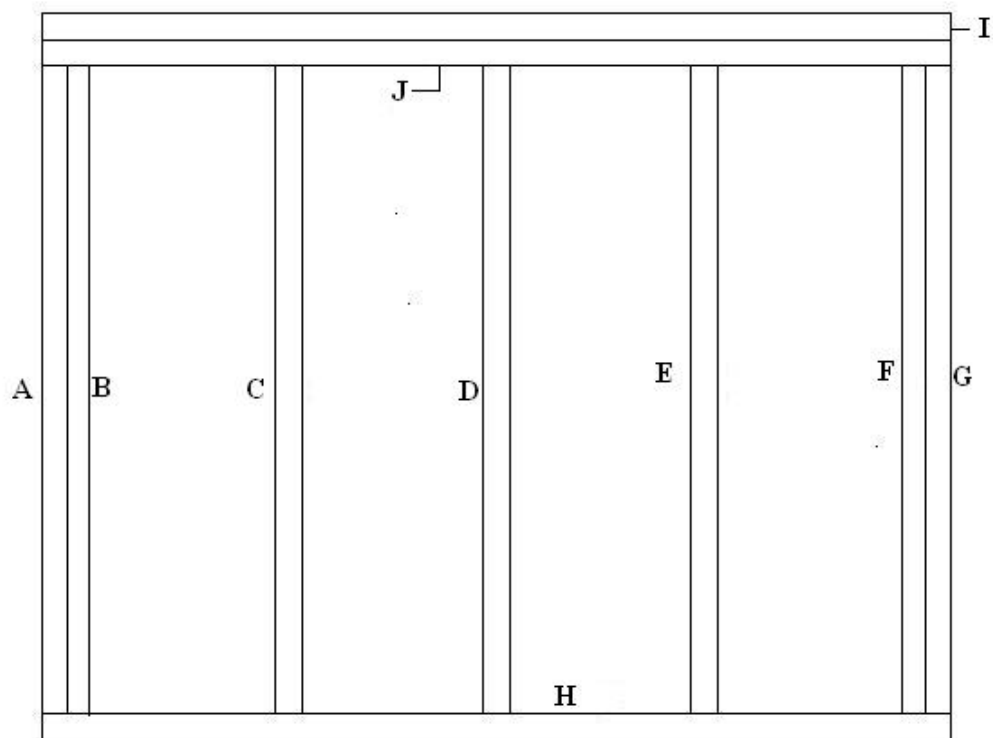


Figure A3. Wall Layout

Table A3. Wall member layout

Wall Number	Wall Member									
	A	B	C	D	E	F	G	H	I	J
1	T38	T39	S43	T54	T 07	T43	T91	S49	S11	S3
2	T3	T24	T70	T92	T 90	S36	S31	S24	S30	S46
3	T32	T42	T80	T89	T 99	T57	T59	S40	S2	S28
4	T5	T52	T88	T77	T96	T47	T8	S50	S21	S6
5	T31	T17	T35	T69	T60	S32	T1	S20	S5	S8
6	T44	T36	T97	T86	T40	T11	T30	S18	S14	S15
7	T22	T14	T78	T29	T45	T25	T23	S51	S25	S22
8	T19	T79	T10	S29	T50	T65	T46	S12	S48	S23
9	T58	T15	T28	T98	T20	T81	S38	S1	S27	S42
10	T87	T49	T75	T92	T74	T17	T62	S84	S16	S44
11	T82	T83	T73	T95	T68	T76	T56	S80	S41	S33
12	T61	T27	T21	S13	T41	T64	T37	S17	S7	S39
13	T48	T84	T53	T63	T6	T66	T71	S83	S9	S35
14	T9	T67	T94	T100	T103	T25	T12	S34	S37	S4
15	T13	T33	T16	T34	T72	T25	T51	S82	S26	S10
16	S45	T4	T101	T102	T26	T55	T93	S85	S19	S47

Appendix B – Digital Image Correlation.

Digital Image Correlation (DIC) is a full-field, non-contact technique for measurement of displacements and strains. The set up consist of a pair of cameras arranged at an angle to take stereoscopic images of the scene (Fig 3). The Image files obtained with the DIC set up are essentially tagged image file format (tiff) files. These are analyzed using a software program called Vic 3D (Correlated solutions Inc.). The cameras are focused on the shear wall and triggered by an external signal. To calculate displacement at any point, a small subset of pixels is used. This subset has a unique light intensity pattern and the DIC software searches the best matching subset in deformed image using mathematical correlation of intensity patterns. The system returns full field 3D displacement and strain data measured over a region or area of interest (aoi) on the visible specimen surfaces.

The theory of digital image correlation has been described in detail by several researchers and a detail treatment of the subject can be found in Sutton et al. (1983). Therefore only a brief description is given here. The underlying principle of DIC is that points on the undeformed surface can be tracked to new positions on the deformed image using a least square error minimization technique. It allows measurement of large deformations and strains, far beyond elastic limits of materials, so failure initiating events and at failure strain development may be observed and analyzed. To achieve this, the object surface must have a good random light intensity pattern that makes a small area surrounding a point unique and able to be tracked by the system. Therefore, specimens are usually speckled with paint to obtain a random speckle pattern on the surface. Surface is illuminated with a white light source and the intensity distribution of light reflected by the surface is captured by a pair of digital cameras and stored as a two dimensional array of grey intensity values on a computer. Typically, light intensity signals are discretely sampled by an array of sensors (1024x1024) of the CCD camera. The gray-scale image can be expressed numerically as intensity function $I(x,y)$ at each pixel location. Thus, if we have two images for a moving body, the cross-correlation function can be calculated for the two images. The peak location of the cross correlation function will indicate the magnitude and direction of displacement of the body. For two large images, the technique is usually applied by dividing both images into sub images or interrogation windows. The cross-correlation is calculated for each two corresponding interrogation windows. The peak values at various interrogated windows represent the complete picture of movements of different parts of the image. The correlation function is

sensitive to changes in the amplitude intensity of the functions to be correlated (Sadeq et al. 2003). Therefore, correlation functions are usually normalized using the mean intensity values of both images. Two common methods for normalization, the normalized cross-correlation (NCC) function and the zero normalized cross-correlation function (ZNCC). The Vic 3D software gives liberty to the users to use any one of those correlation techniques. All the results discussed here are with NCC function.

Digitized images captured before and after deformation are then compared by a digital image correlation routine to obtain displacements and strains. Before correlation, discrete grey intensity level array is reconstructed using bilinear interpolation to obtain a continuous intensity distribution over the whole image. This is because, a point in the undeformed image can map into a gap between the pixels in the deformed image (Samarasinghe et al. 1997). To obtain the displacements and gradients, a mathematical relationship between the actual displacement of a point and the light intensity of a small area surrounding the point needs to be established. Values of interest for surface measurements are displacements in x and y directions (u and v), normal strains ($\partial u/\partial x$, $\partial v/\partial y$) and components of shear strain ($\partial u/\partial y$, $\partial v/\partial x$). It is assumed that the light intensity of points do not change as a result of object motion hence subset in the undeformed image can be mapped to a subset of similar intensity in the deformed image.

Once the correlation is finished, surface topography and strain profiles are obtained from the software. Also numerical analysis of any selected area in the image can be extracted with the software and some other program (e.g. excel etc.) is used to analyze those. Numerical extraction for the area analyzed by DIC calculates strains by calculating displacements, u , v and w by correlating the position of speckles in a Cartesian coordinate system. Subsequently, calculated are the values of various displacement gradients ($\delta u/\delta x$, $\delta v/\delta y$, $\delta w/\delta z$, $\delta u/\delta y$, $\delta v/\delta x$ etc.) which are used to derive strains. Strains are named as e_x , e_y , e_{xy} , e_1 and e_2 which corresponds to strain in x, y directions, shear strain, major and minor principle strain, respectively.

VERIFICATION

To verify the system preliminary tests were conducted. The primary objective of these tests were was familiarization with the set up, to determine at what distance and geometry it works the best for the current study and to verify the system.

The camera was successively set up at increasing distances from the object. The distances ranged from 1m to 10 m at random intervals. Each time the DIC system was calibrated and then five images were captured of the unloaded sample. These images were then analyzed using the software Vic3d. As the samples were not loaded all the strains and displacement returned by the system must be zero. Our primary focus was to find the optimum distance for a good correlation result with minimum standard deviation and also to infer any trend in the efficiency of the set up as the distance increases or decreases. Although in all the cases the strain value was zero, the highest correlation was obtained at 2m from the test specimen. Correlation is also dependent on type of lenses used, included angle of the cameras and the speckle pattern, light intensity, contrast at the surface of the specimen, focus of the cameras and aperture of the camera.

In a preliminary wall test and data was logged using both DIC system and conventional load cell. Figure B1 shows the load displacement diagram obtained from the load cell and DIC. As seen in the figure the two curves are identical to each other. The loading was halted for a few minutes at the 15 kN mark and then restarted again from the very same position and this fact is also observed in both the curves. Calculations of strains by the software are dependent on the displacement values of various points on the specimen. If the displacement values returned by the system are accurate then a safe assumption about the veracity of strain values can be made.

A small wall (60 x 60 cm) was constructed and tested, to verify the DIC system, with a strain gauge attached at near one of the fastener at the uplift corner. The strain gauge was aligned in the vertical direction to record the values of strain in y direction ϵ_y . Figure B2 shows the load vs. strain curve at that point as recorded by strain gauge and also calculated by DIC (ϵ_{yy}). The differences are within experimental limits (10%). The differences might be due to the fact that DIC could not extract the data over the area strain gauge is adhered to wall surface, but an area adjacent to it.

These two tests, former verifying the displacement data and the latter verifying the strain data provided the confidence in the DIC system for test of full scale shear walls. Various trial and error methods provided an idea about the pixel resolution to use, outer limits of included angle between the cameras, lighting conditions and adequacy of the speckle pattern. All these parameters are subjective or qualitative in nature and most of them are interrelated, hence a generalized relationship could not be established.

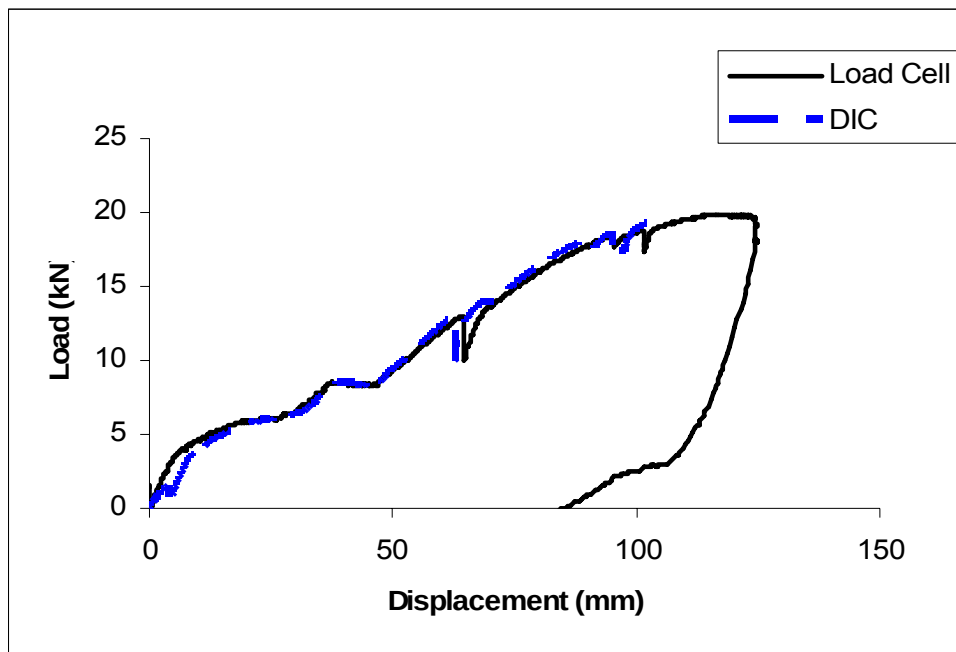


Figure B1. Load Displacement Diagram from preliminary test.

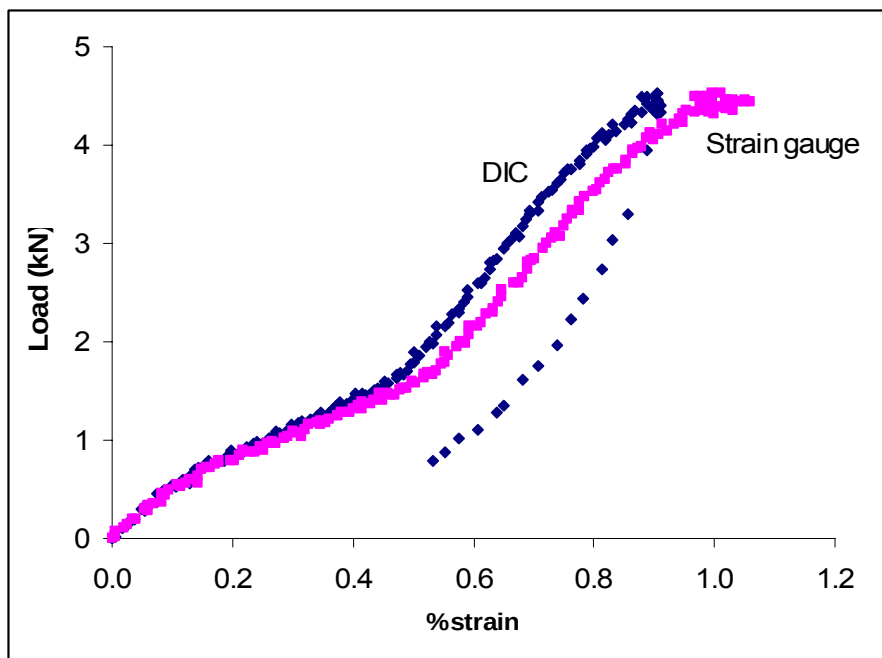


Figure B2. Load strain Diagram from preliminary test.

Appendix C – Data Analysis

A total of sixteen walls of size 2440 x2400 mm were tested for this study monotonically with hold downs. Eleven walls were tested with sheathing on both sides (OSB and GWB) while five walls had only OSB on one side. For every test, a load-displacement curve was produced from the data obtained by load cell and actuator. Nearly every parameter of shear walls can be obtained from this graph. The displacement used to generate the graph is the inter story drift, which is the displacement of the top of the wall relative to sill plate. The load-displacement graph for monotonic tests as illustrated in Figure 4 is always positive and produces a curved line characteristic of its one directional loading.

WALL CAPACITY

Wall capacity refers to the ultimate load (P_{peak}) the wall can with stand during loading.

WALL FAILURE

The walls tested were considered to be failed at 80% of ultimate load at the descending part of load displacement diagram. Failure load is denoted by $0.8 P_{peak}$. For light-frame shear walls the failure is seldom sudden, but instead a gradual decline mirroring its increase in load. This value of $0.8 P_{peak}$ is an arbitrarily defined value hence variation could arise while comparing the parameters based on this value.

The failure data is used to measure ductility of the structure. The more a structure can deflect on its way to failure and the more load it can resist at failure are important to the reliability of the structure. It is crucial that a shear wall be able to deflect by a significant amount to withstand the ground motions produced by a seismic event.

ENERGY DISSIPATED

A shear wall must be able to undergo large deformations and hence dissipate large amounts of energy during an earthquake. Experimental testing gives the most accurate and realistic means of predicting the hysteretic behavior of a shear wall. The amount of energy dissipated by a structure can be calculated directly from the load-displacement curve as it is simply the area under the curve measured from the initial displacement until the failure displacement of the wall.

EEEE Parameters

Wood structures have an entirely different load displacement behavior as compared to steel or concrete. Light-frame wood construction does not have a distinct yield load, and the proportional limit cannot be definitely identified. Several definitions have been proposed for the yield load in the past. To determine the yield load in this study, the use of an equivalent energy elastic plastic (EEEE) curve is incorporated as illustrated in Figure 6. An EEEP curve is a perfectly elastic plastic representation of the actual response of the specimen. The curve is plotted such that it equals the area under the load deflection curve until failure. This allows a direct comparison of wall performance on energy basis.

The EEEP curve is a function of the yield load and displacement, the failure displacement, area under the observed load-displacement graph, and the elastic stiffness. The EEEP curves consist of an elastic region that proceeds at a constant slope until yielding occurs, that is followed by a horizontal plastic region maintained until failure. The elastic portion pass through the origin and point of 40% peak load, and at a slope equivalent to the elastic shear stiffness. The intersection of elastic and plastic portion of the curve gives the yield point. Assuming that P_{yield} is a function of the elastic stiffness, the area under the load-displacement graph, and the failure displacement, it can be calculated as follows:

$$P_{yield} = \frac{-\Delta_{failure} \pm (\Delta_{failure}^2 - 2A/K_e)^{0.5}}{-1/K_e}$$

Where P_{yield} = Yield Load (kN).

A = Area under the load deflection curve till failure (kN mm).

K_e = Elastic Stiffness (kN/mm).

$$\text{Yield Displacement} = \Delta_{yield} = P_{yield} / K_e$$

ELASTIC STIFFNESS

The elastic stiffness, K_e , is defined by the slope of the secant passing through the origin and the point on the load-displacement curve that is equal to 40% of the peak load, P_{peak} .

$$\text{Elastic Stiffness} = K_e = \frac{0.4 P_{peak}}{\Delta 0.4 P_{peak}}$$

The definition of elastic stiffness is based on the ASTM standard for cyclic tests of mechanical connections. The elastic stiffness is a good representation of the stiffness that a wall would exhibit when subjected to low to moderate displacements (Toothman 2003).

DUCTILITY

Ductility is an important feature of a structural system, which enables it to undergo plastic deformation without failure. The ability of walls to deform but not break is crucial when subjected to the sudden and powerful earthquakes. In design, more ductile performance is credited with lower seismic forces to resist, as deformation in the inelastic region provides significantly more energy dissipation. Several methods have been proposed to express the ductility of a structure. One accepted measurement of ductility is the ratio of the peak displacement to the yield displacement:

$$\text{Ductility} = D = \Delta_{\text{peak}} / \Delta_{\text{yield}}$$

This definition only considers the structure's ability to yield until reaching its maximum load. The most commonly accepted definition is the ASTM E2126 definition, which defines the ductility factor, μ , as the ratio of the failure displacement and the yield displacement.

$$\text{Ductility Factor} = \mu = \Delta_{\text{failure}} / \Delta_{\text{yield}}$$

This value represents the amount of displacement that a structure can undergo from yielding until failure and assumes that most ductile structures, such as light-frame shear walls, are able to resist loads far beyond Δ_{peak} . When the structural component has reached its capacity, it transfers additional load onto other components.

The ductility factor introduced above is the ratio of two displacements and is therefore not a measure of the structure's ability to withstand large deformations without failing. If a structure undergoes large deformations before failing but has a large yield displacement, the structure is not necessarily a ductile system. The reverse is also true, so ductility should always be considered together with other performance indicators.

Another ratio used to define characteristic of wall is the toughness index, calculated as the measure of displacement capacity remaining after reaching the peak capacity.

$$\text{Toughness Index} = D_f = \Delta_{\text{failure}} / \Delta_{\text{peak}}$$

DIC DATA ANALYSIS

Strains are determined by calculating gradients of displacements, u , v and w by correlating the position of speckles in a Cartesian coordinate system. Values of various displacement gradients ($\delta u/\delta x$, $\delta v/\delta y$, $\delta w/\delta z$, $\delta u/\delta y$, $\delta v/\delta x$ etc.) which are used to derive strains are subsequently calculated. The output strain tensor components denoted e_{xx} , e_{yy} , e_{xy} , e_1 and e_2 correspond to strain in x , y directions, shear strain and major and minor principle strains, respectively. In this study local strains in major principle directions (e_1) are considered as they represent the maximum normal strain at a plane and are dependent on the strain in global x and y directions and also on the shear strains at that point.

An area of interest for numerical data analysis was chosen. The selection of area of interest was based on the magnitude of local principle strain in that area hence an area which encountered the maximum strain on corresponding sides, i.e. on OSB and GWB was selected as area of interest to extract after preliminary analysis. An attempt was made to keep consistency in the number of data points selected over which the averaging is done. The physical area could vary due to scaling and pixel resolution of the set up. Sometimes, the area of interest chosen might have some uncorrelated speckles which is not considered in the analysis and was automatically deleted from the area of interest by the software. A larger physical area chosen might provide less data points that could be analyzed than a relatively smaller area of interest. Hence consistency in selecting the number of data point was paramount. The numerical data underlying in that area and analyze it using other data analysis tools (excel). The area of interest was rectangular in shape, included 40 data points and had a physical area of approximately $80 - 100 \text{ mm}^2$. The figure C3 shows the area of interest used in one of the walls in GWB side.

Table C1. List of Wall Test

Wall no.	Area Imaged	Zone	Sheathing	
			OSB	GWB
1	Middle of Sill	III	yes	yes
2	Middle of Sill	III	yes	yes
3	Middle of Sill	III	yes	No
4	Middle of Sill	III	yes	No
5	Uplift Corner	II	yes	yes
6	Uplift Corner	II	yes	yes
7	Compression Corner	I	yes	yes
8	Compression Corner	I	yes	yes
9	Intermediate Stud	VI	yes	yes
10	Intermediate Stud	VI	yes	yes
11	Middle of the wall	IV	yes	yes
12	Middle of the wall	IV	yes	yes
13	Top Middle of Wall	V	yes	yes
14	Top Middle of Wall	V	yes	No
15	Uplift Corner	II	yes	No
16	Compression Corner	I	yes	No

Table C2. Summary of Test and EEEP Parameters

Wall Number	P _{peak} kN	0.8 P _{peak} kN	0.4 P _{peak} kN	Δ _{peak} mm	Δ _{failure} mm	Δ0.4P _{peak} mm	Stiffness kN/mm	Energy Dissipated E (kNmm)	Δ _{yield} mm	Ductility D	μ
1	34.5	27.6	13.8	53.1	72.9	8.4	1.64	1905	18.2	2.91	4.00
2	38.3	30.7	15.3	49.4	46.4	5.2	2.97	1406	11.7	4.23	3.97
5	40.2	32.2	16.1	61.2	74.8	9.9	1.63	2209	21.1	2.90	3.54
6	38.1	30.5	15.2	75.0	94.8	7.9	1.92	2903	17.6	4.27	5.39
7	39.9	31.9	15.9	68.5	82.2	7.5	2.12	2560	16.3	4.21	5.05
8	42.1	33.7	16.8	46.9	0.0	6.3	2.66	1411	0.0	0.00	0.00
9	34.6	27.6	13.8	56.9	84.3	5.6	2.48	2442	12.6	4.50	6.66
10	35.9	28.7	14.3	56.8	71.2	6.6	2.18	2025	14.5	3.91	4.90
11	40.1	32.1	16.0	60.3	81.2	7.6	2.10	2563	16.8	3.60	4.84
12	40.3	32.3	16.1	61.6	71.6	7.6	2.13	2228	16.5	3.73	4.34
13	41.5	33.2	16.6	69.0	100.5	9.7	1.71	3360	22.0	3.14	4.58
3	39.3	31.4	15.7	78.1	97.4	10.3	1.53	2850	21.4	3.64	4.54
4	37.6	30.0	15.0	71.7	89.5	9.2	1.63	2603	20.1	3.56	4.45
14	39.1	31.3	15.7	84.3	103.4	10.0	1.57	3231	22.3	3.77	4.63
15	38.9	31.1	15.5	75.4	90.7	12.9	1.21	2648	28.7	2.63	3.16
16	37.1	29.6	14.8	70.1	86.2	12.1	1.23	2378	26.6	2.63	3.24

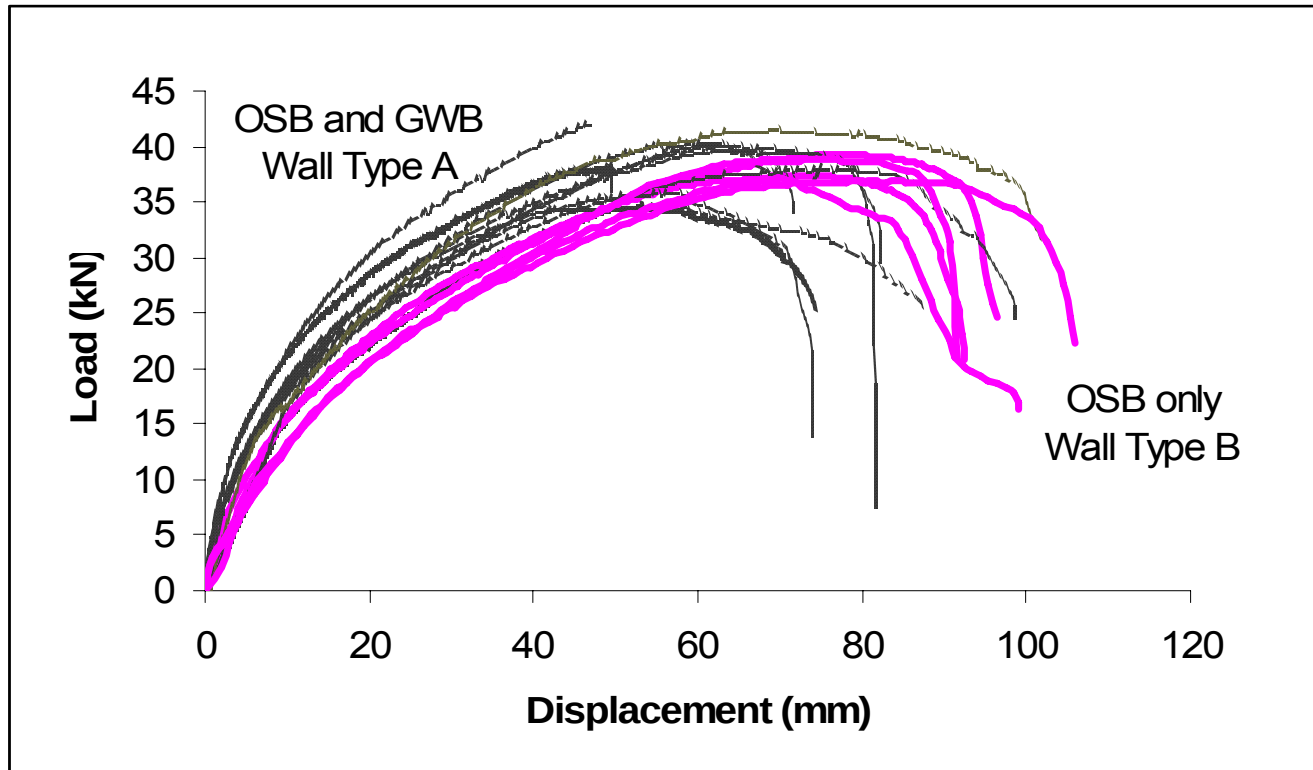


Figure C1. Load Deflection Diagram for all the walls depicted by two generic curve types

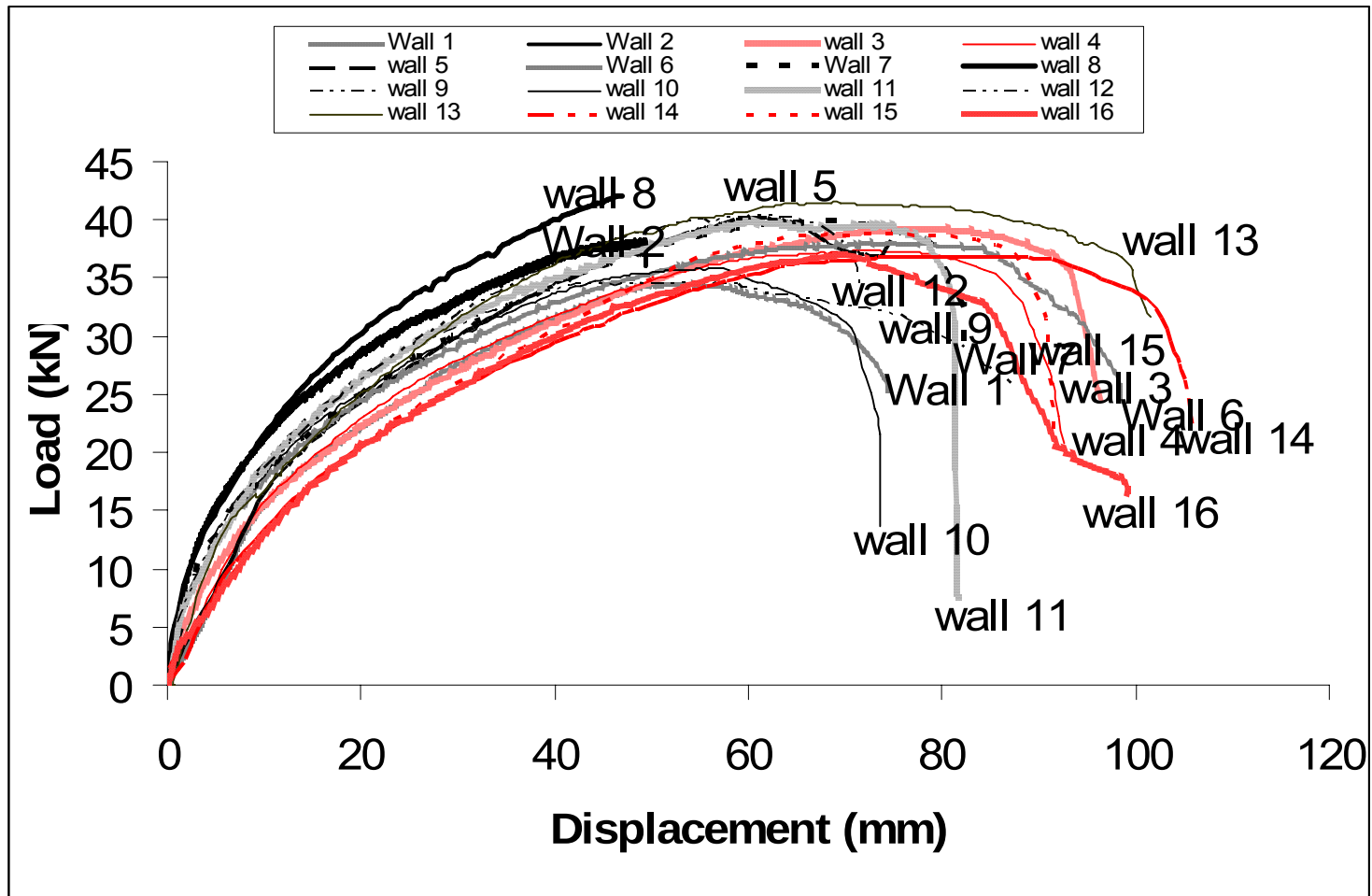
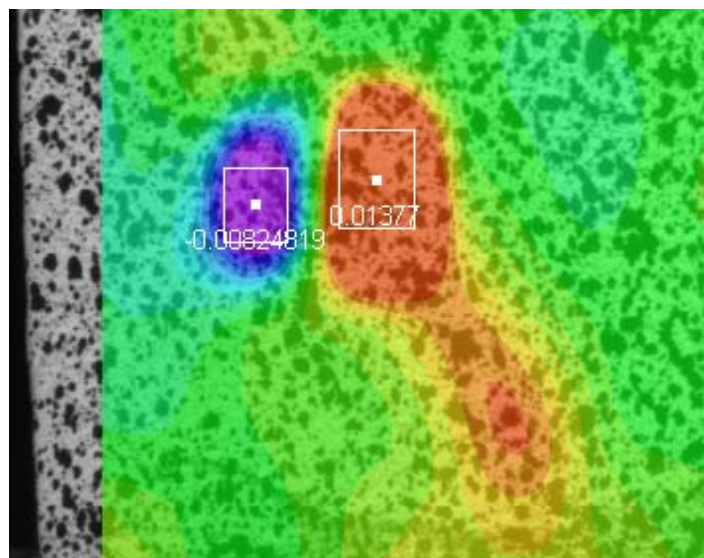
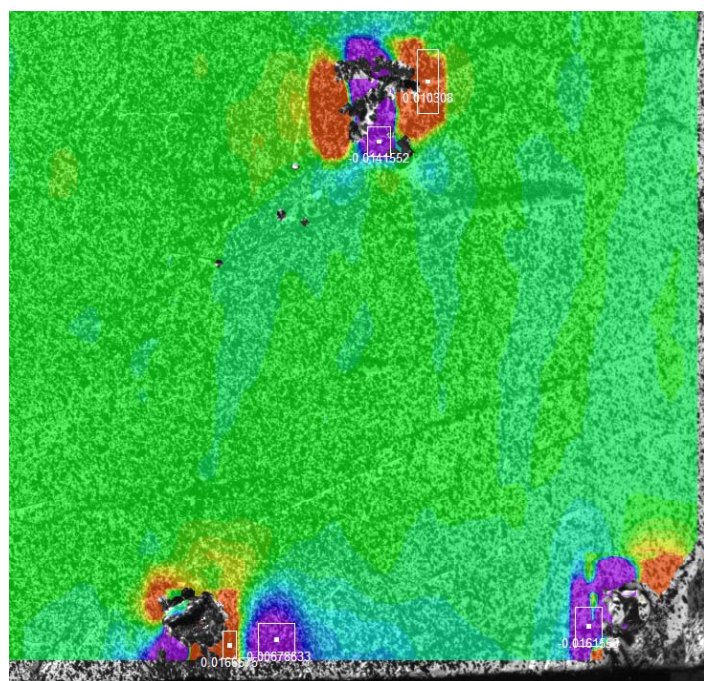


Figure C2. Load Deflection Diagram for all the walls



(A) GWB side



(B) OSB side

Figure C3. Areas of interest (aoi) at the uplift corner.

Appendix D - Load vs. Local Displacement

GWB Side

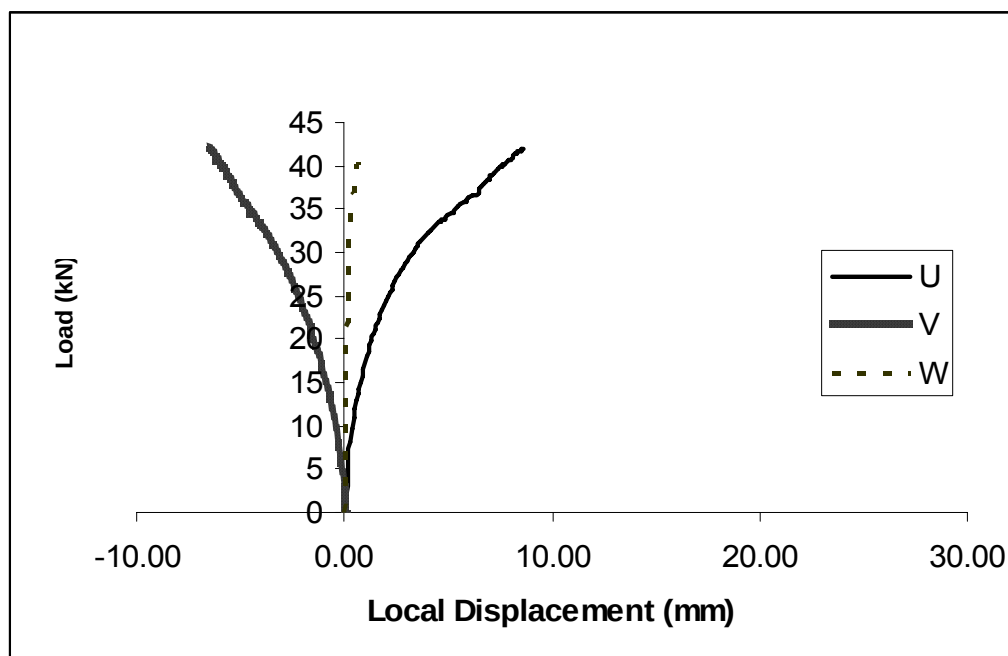


Figure D1. Load Displacement diagram at S2 (Zone I)

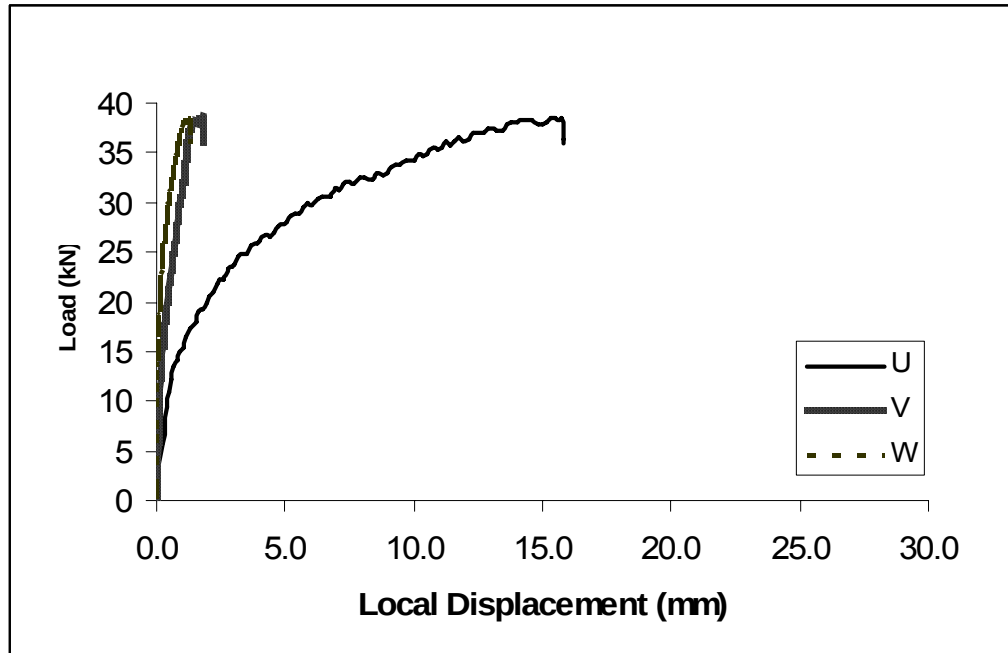


Figure D2. Load Displacement diagram at S5 (Zone III)

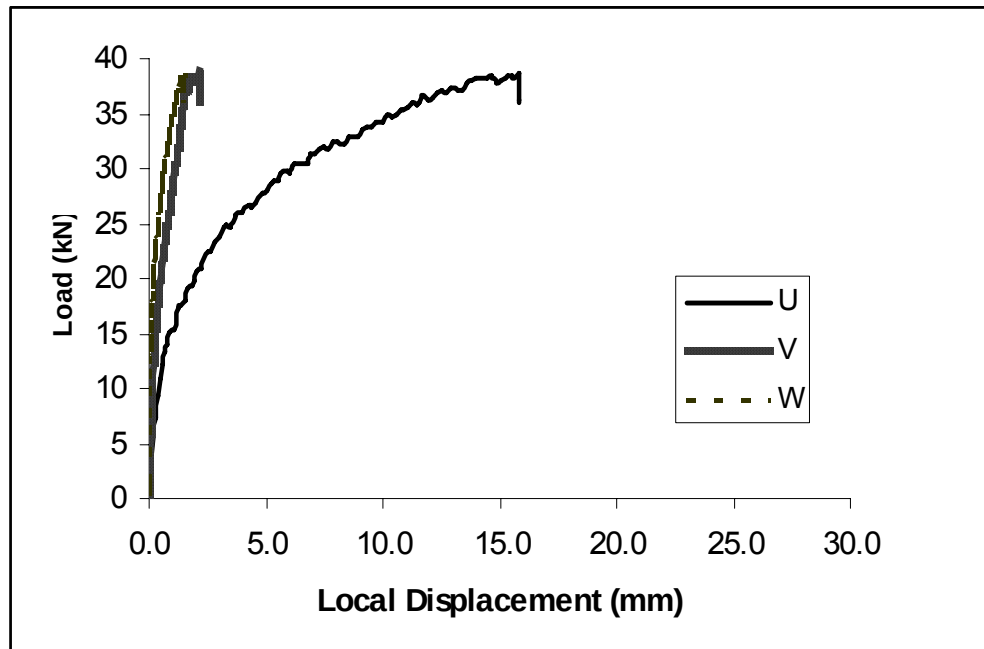


Figure D3. Load Displacement diagram at S6 (Zone III)

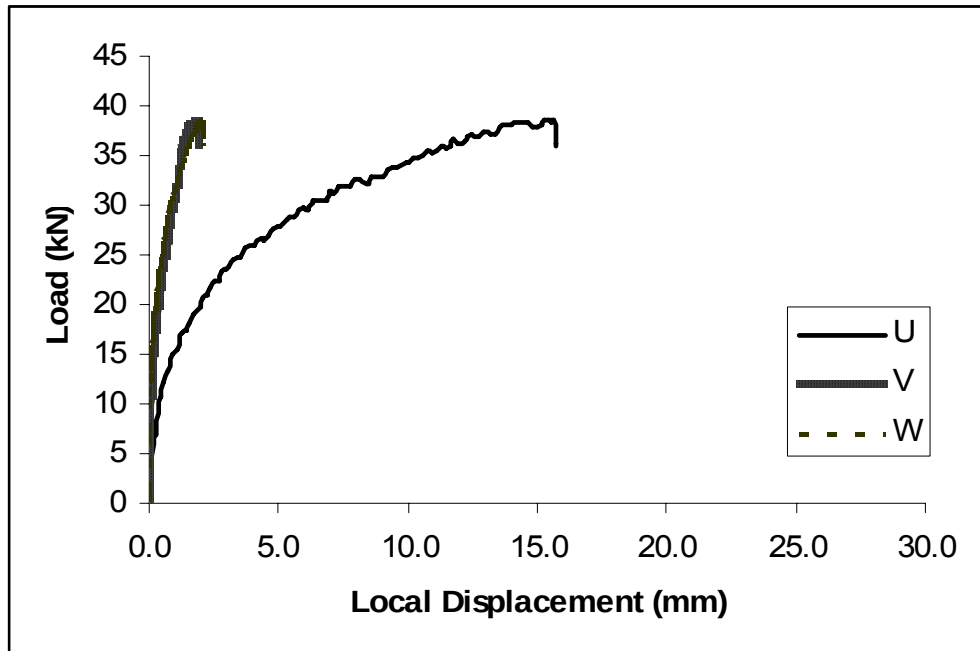


Figure D4. Load Displacement diagram at S7 (Zone III)

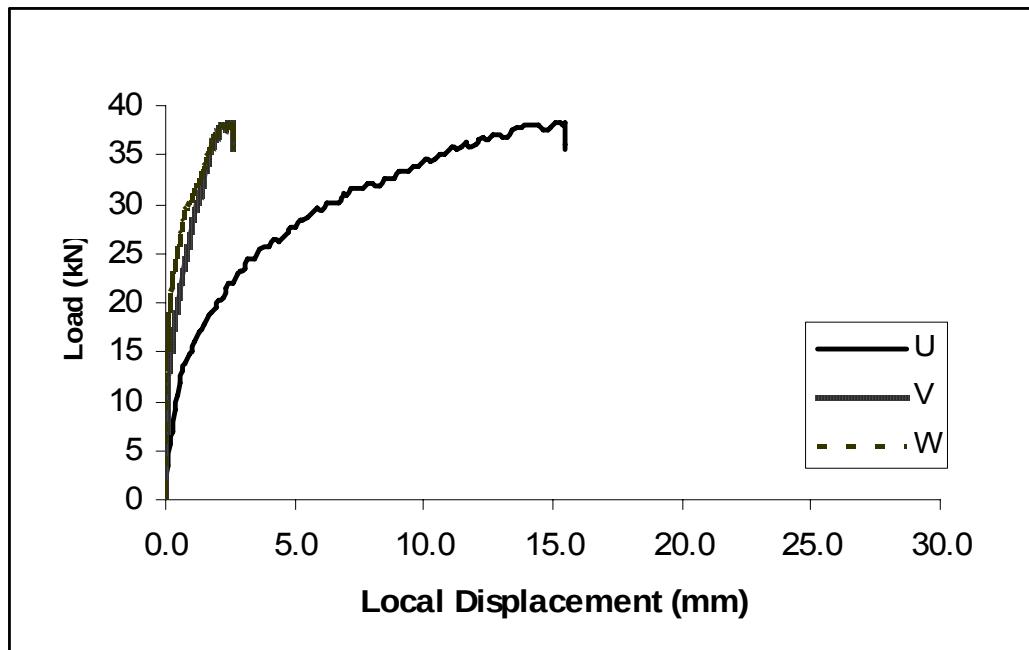


Figure D5. Load Displacement diagram at S8 (Zone III)

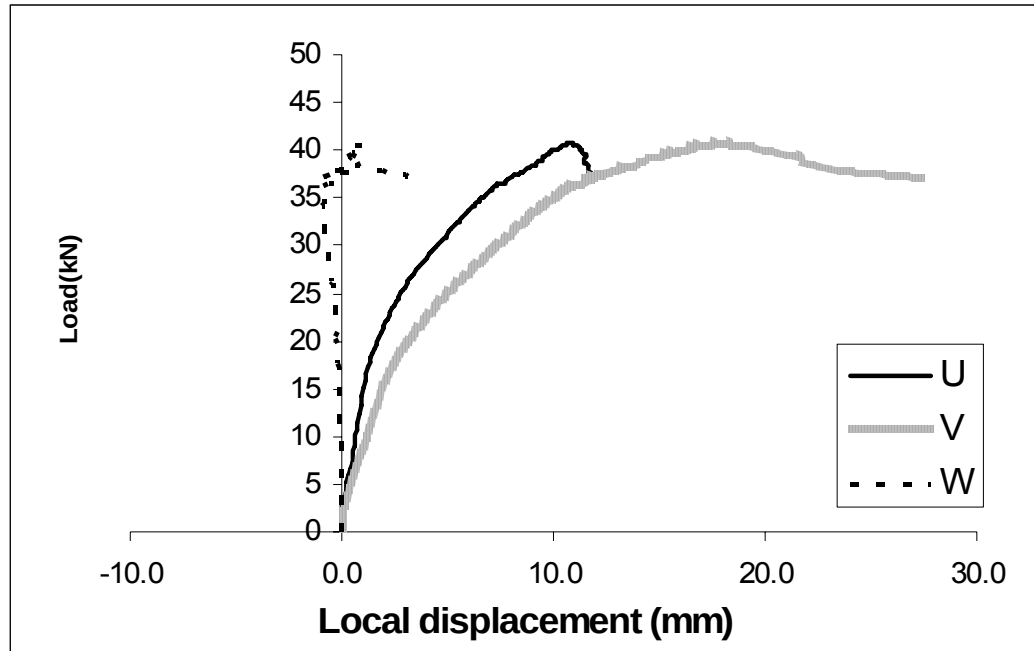


Figure D6. Load Displacement diagram at S9 (Zone II)

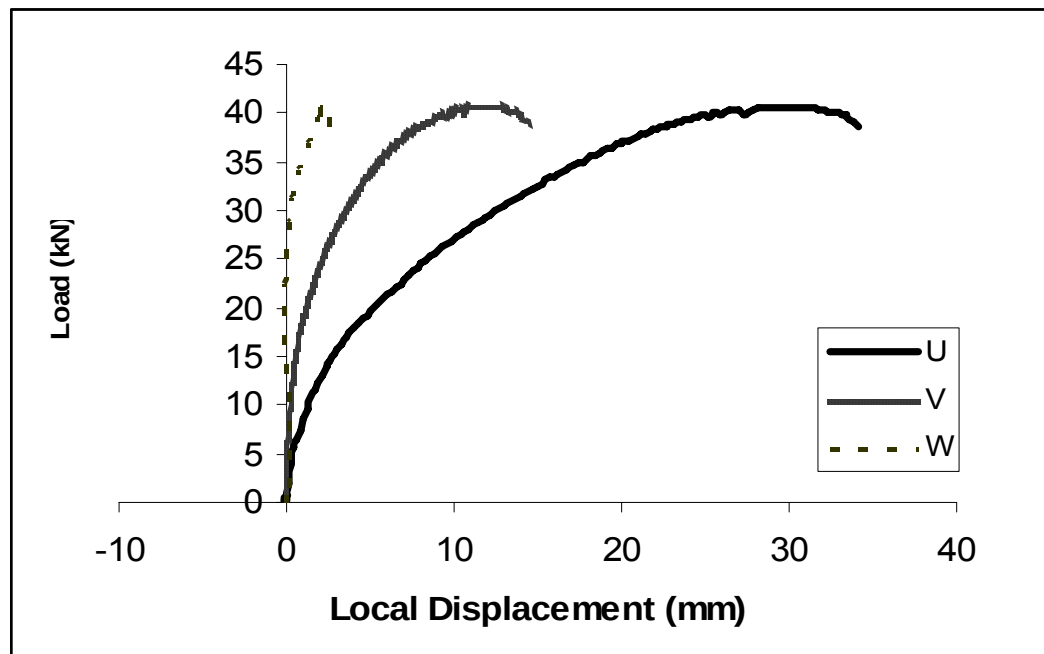


Figure D7. Load Displacement diagram at S11 (Zone IV)

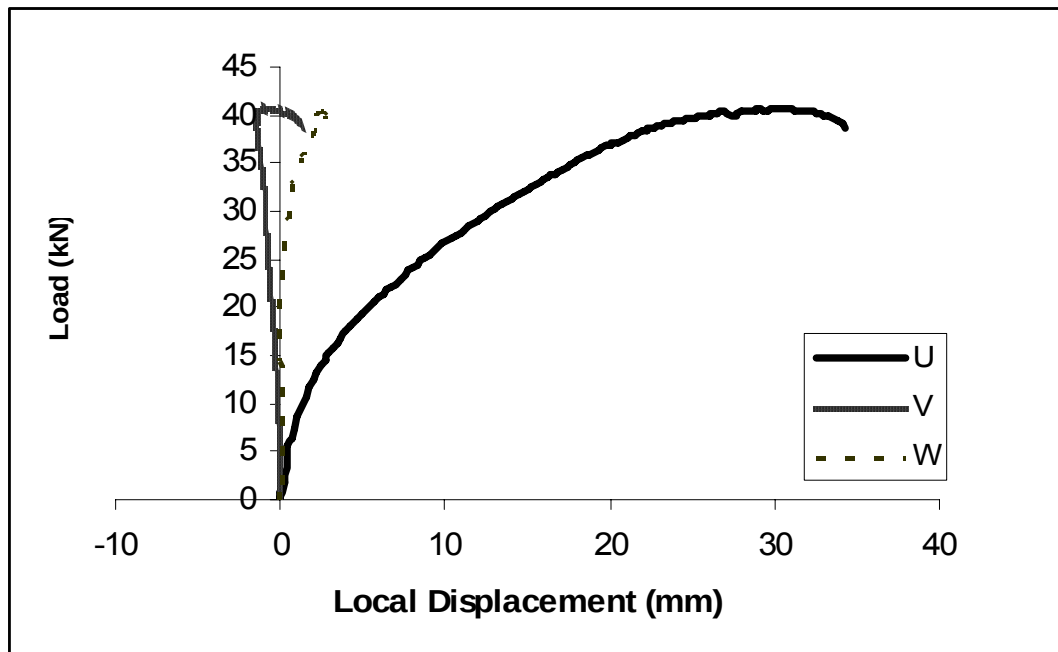


Figure D8. Load Displacement diagram at S12 (Zone IV)

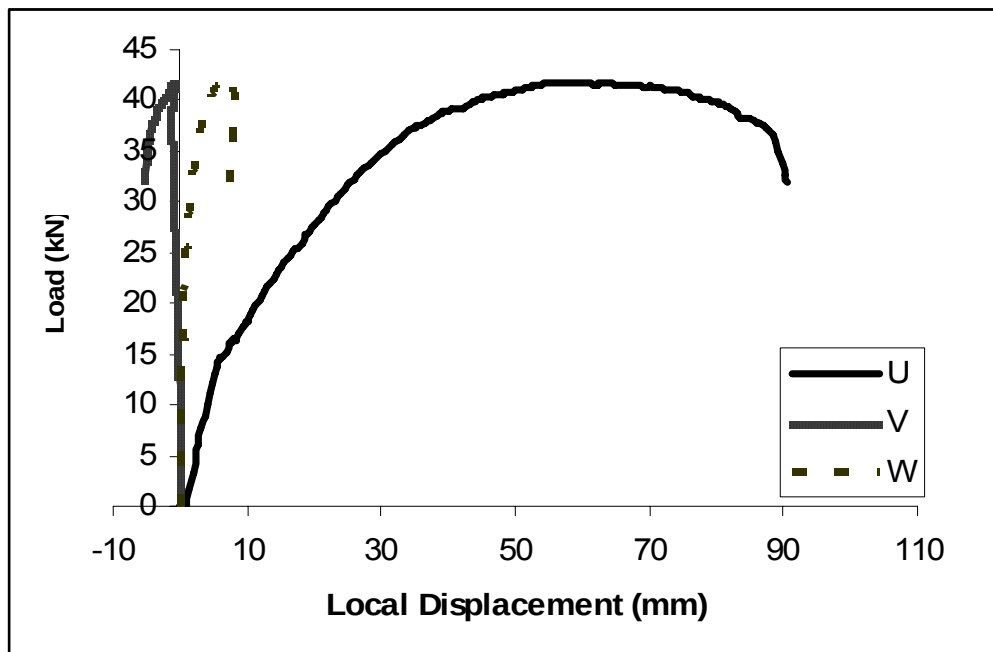


Figure D9. Load Displacement diagram at S13 (Zone V)

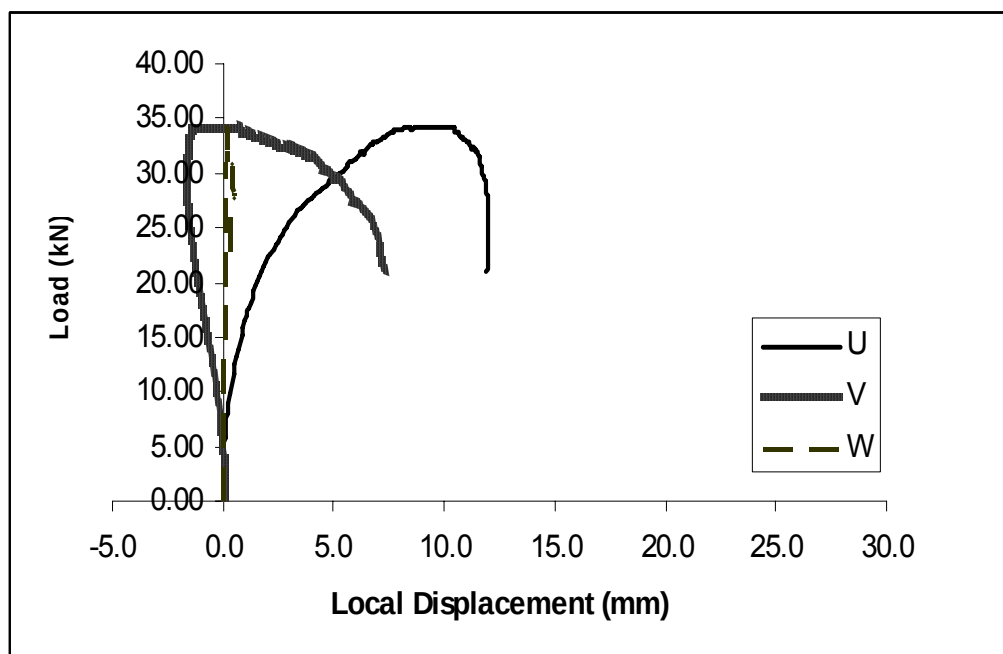


Figure D10. Load Displacement diagram of Zone VI

Type A Walls

OSB side

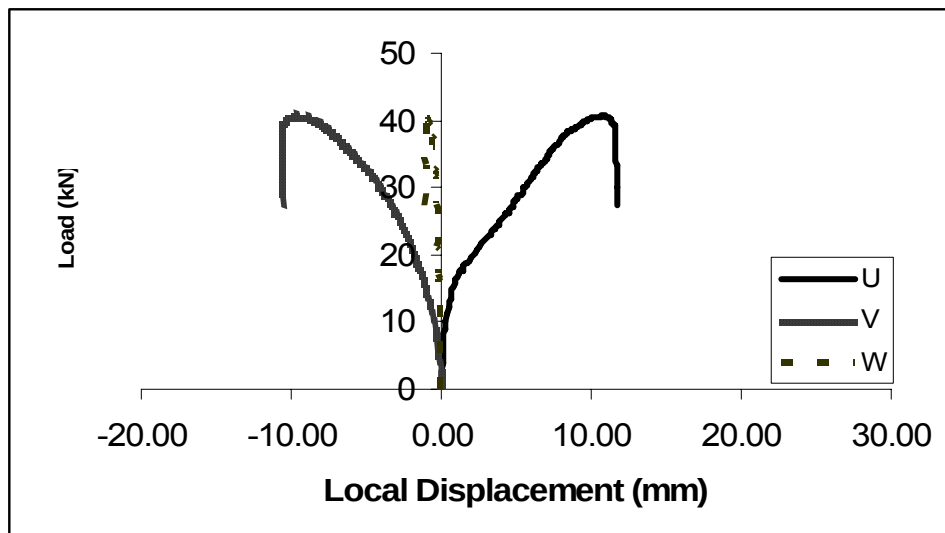


Figure D11. Load Displacement diagram for Zone I

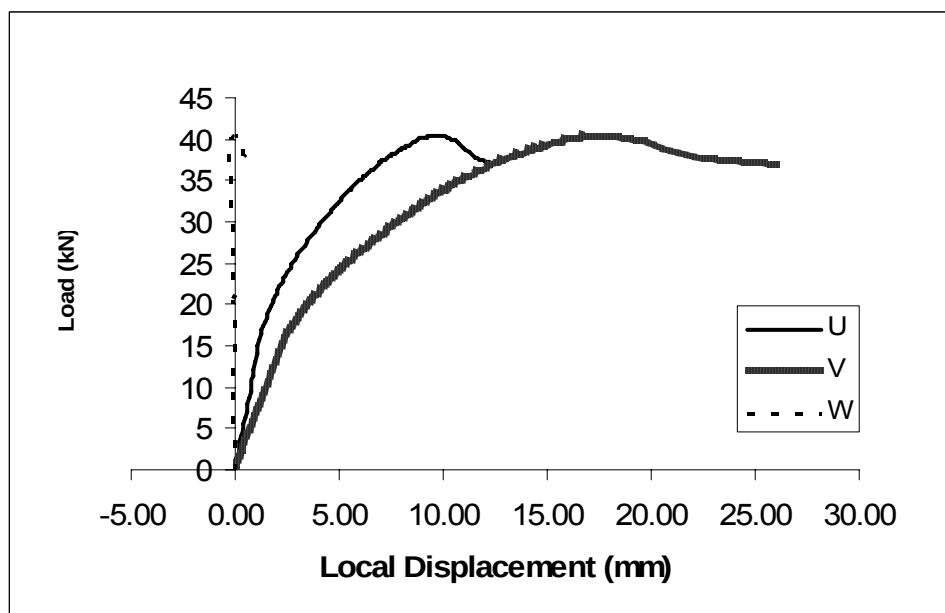


Figure D12. Load Displacement diagram for Zone II

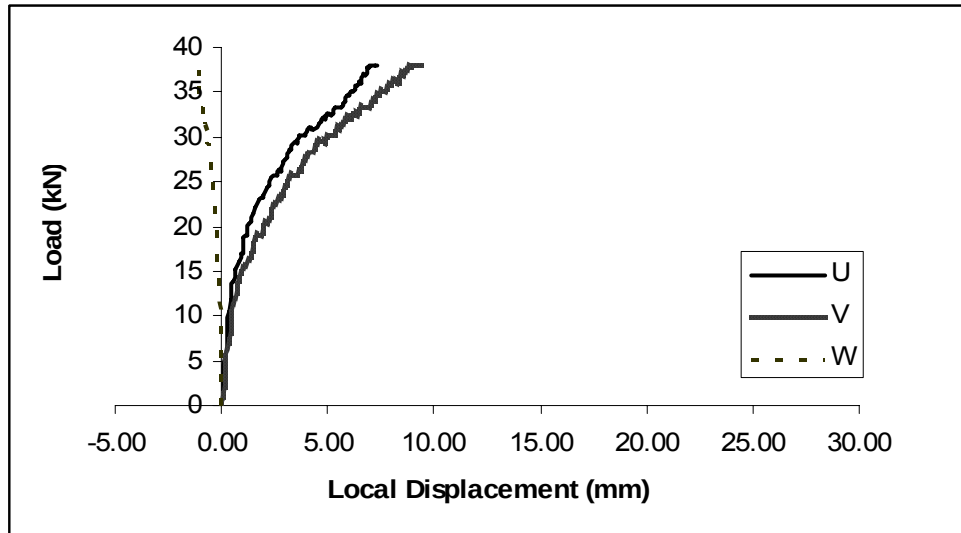


Figure D13. Load Displacement diagram for Zone III (Nail 5)

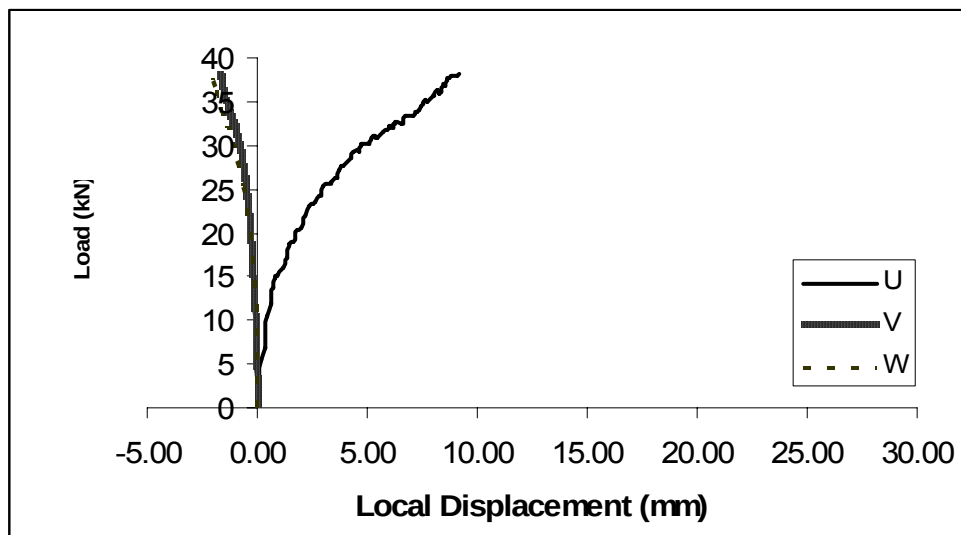


Figure D14. Load Displacement diagram for Zone III (Nail 7)

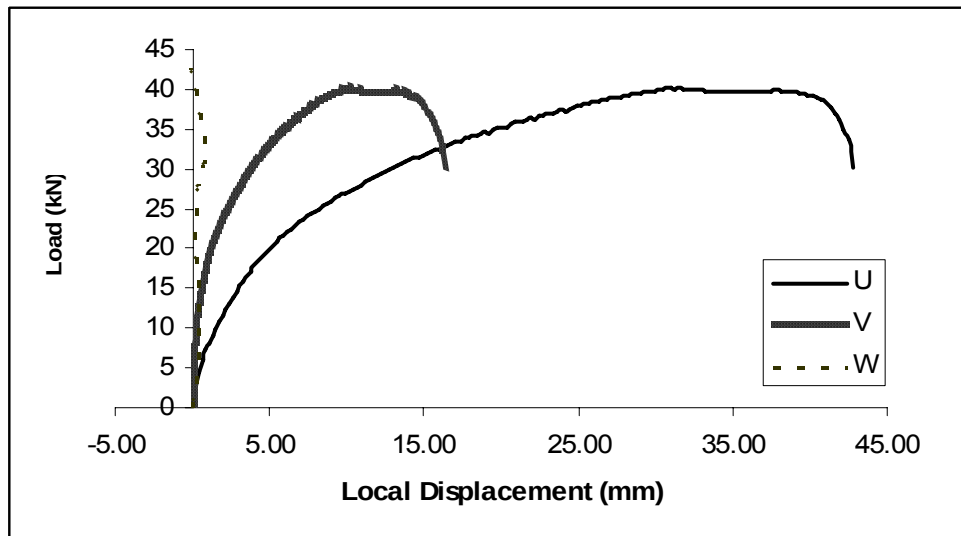


Figure D15. Load Displacement diagram for Zone IV (Nail 11)

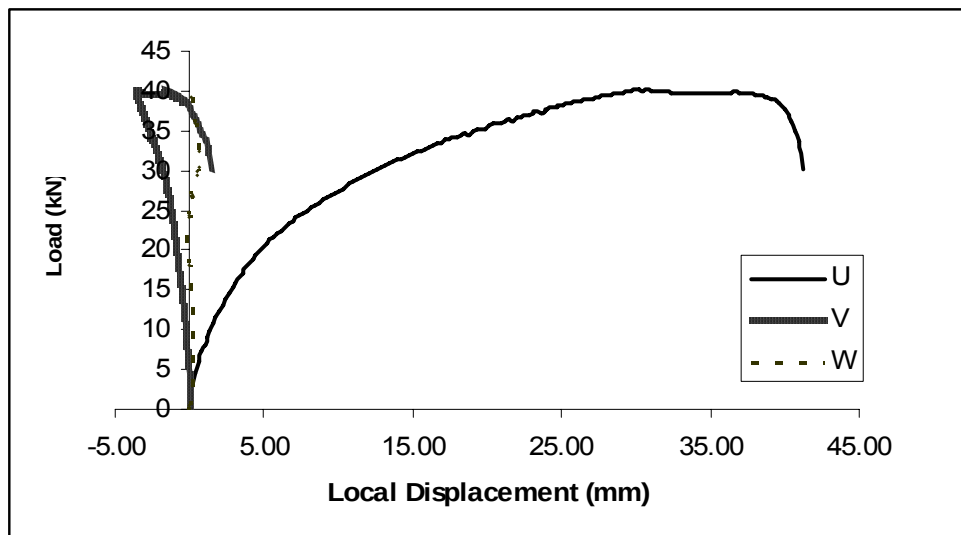


Figure D16. Load Displacement diagram for Zone IV (Nail 12)

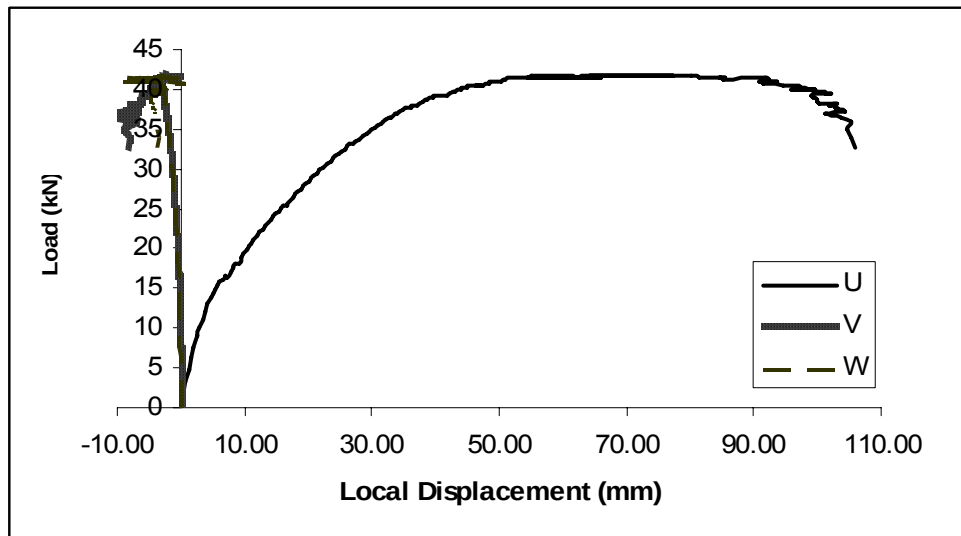


Figure D17. Load Displacement diagram for Zone V (Nail 13)

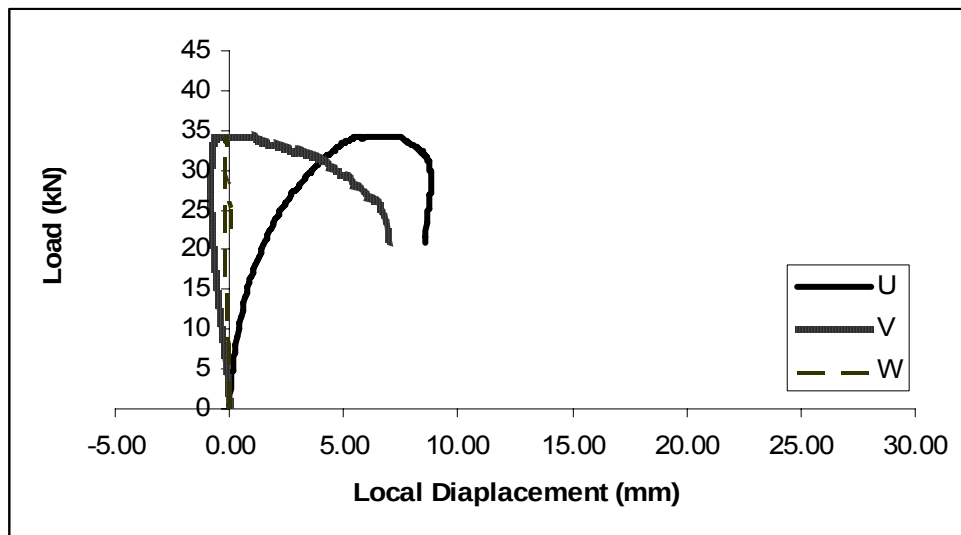


Figure D18. Load Displacement diagram for Zone VI

Type B Walls

OSB side

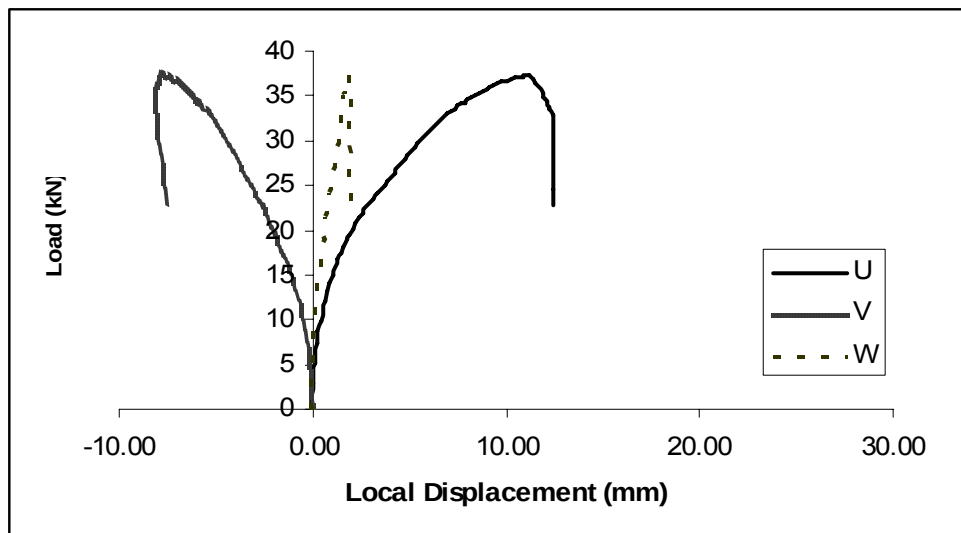


Figure D19. Load Displacement diagram for Zone I

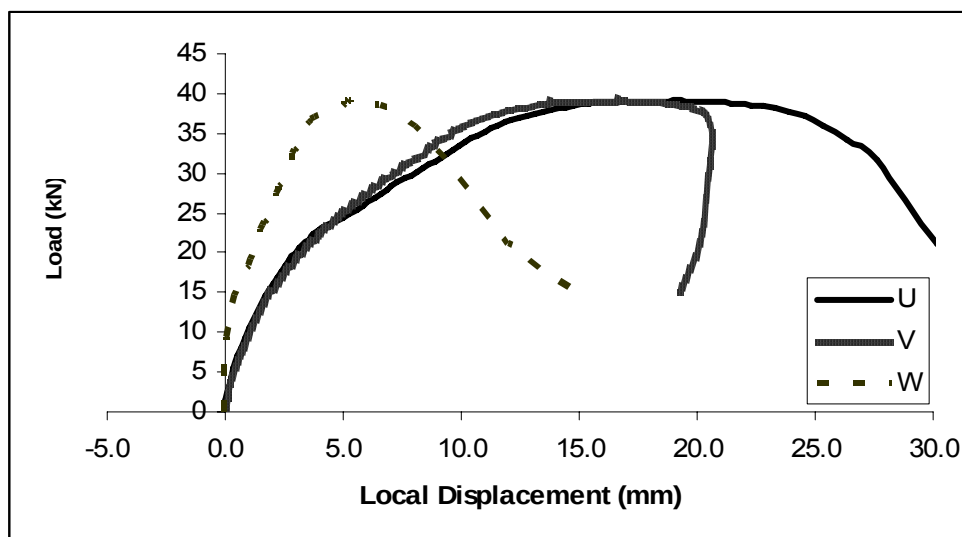


Figure D20. Load Displacement diagram for Zone II

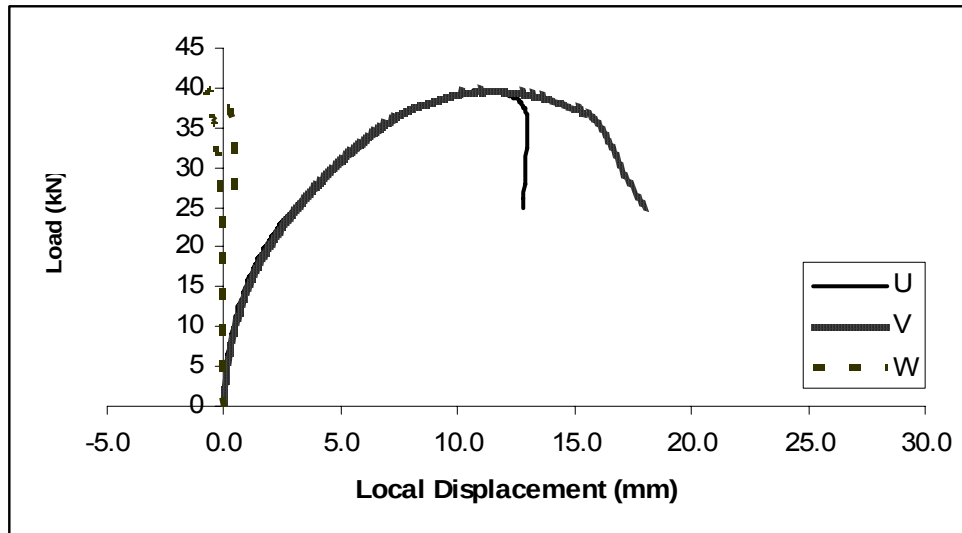


Figure D21. Load Displacement diagram for Zone III (Nail 5)

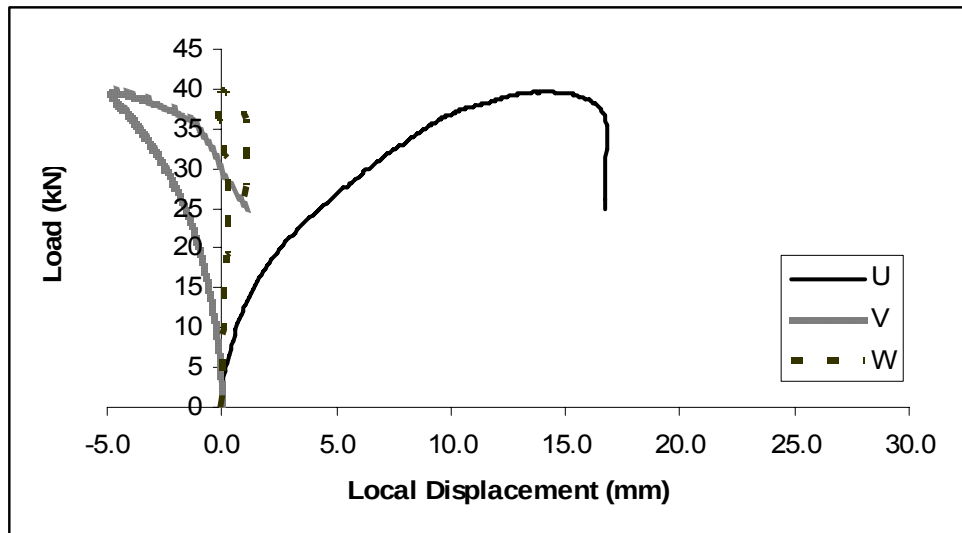


Figure D22. Load Displacement diagram for Zone III (Nail 7)

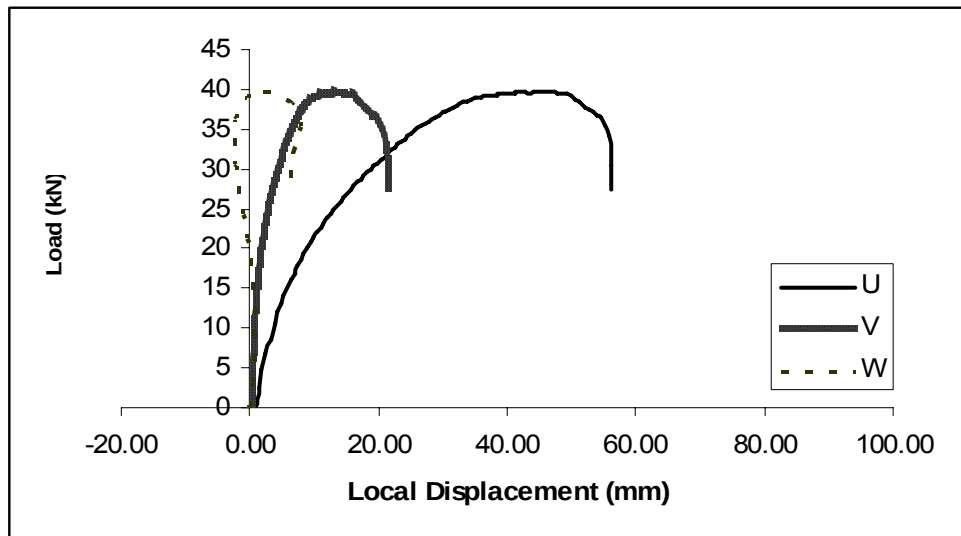


Figure D23. Load Displacement diagram for Zone IV (Nail 11)

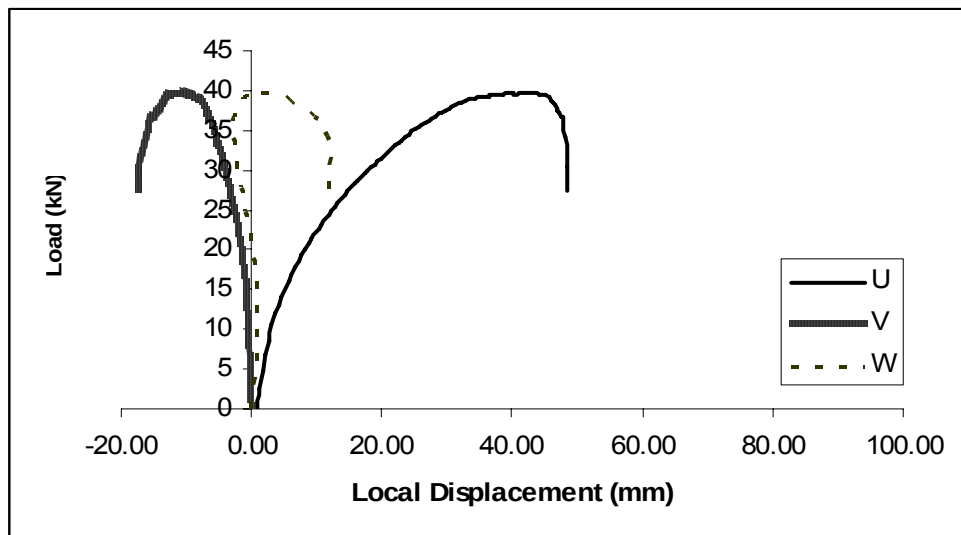


Figure D24. Load Displacement diagram for Zone IV (Nail 12)

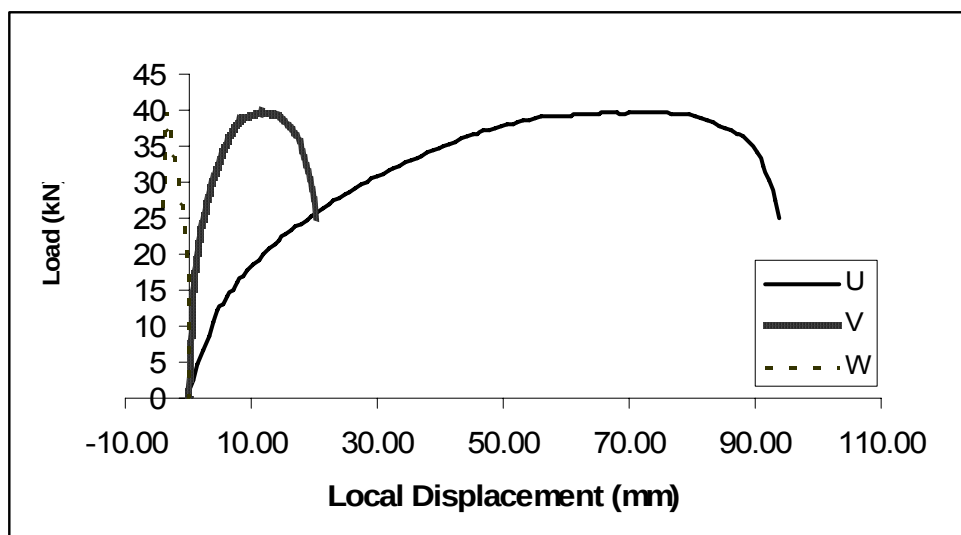


Figure D25. Load Displacement diagram for Zone V (Nail 13)

Appendix E - Load Strain Diagrams

Strains are determined by calculating gradients of displacements, u , v and w by correlating the position of speckles in a Cartesian coordinate system. Values of various displacement gradients ($\delta u/\delta x$, $\delta v/\delta y$, $\delta w/\delta z$, $\delta u/\delta y$, $\delta v/\delta x$ etc.) which are used to derive strains are subsequently calculated. The output strain tensor components denoted e_{xx} , e_{yy} , e_{xy} , e_1 and e_2 correspond to strain in x , y directions, shear strain and major and minor principal strains, respectively. In this study local strains in major principle directions (e_1) are considered as they represent the maximum normal strain at a plane and are dependent on the strain in global x and y directions and also on the shear strains at that point. Second principal strain (e_2) always have a value numerically lesser than that of e_1 . All the principal strains are plotted for the fastener in uplift corner of GWB (Fig. E2). For the rest only the strains in x , y , shear strain and first major principal strain are plotted.

WALL TYPE A

GWB

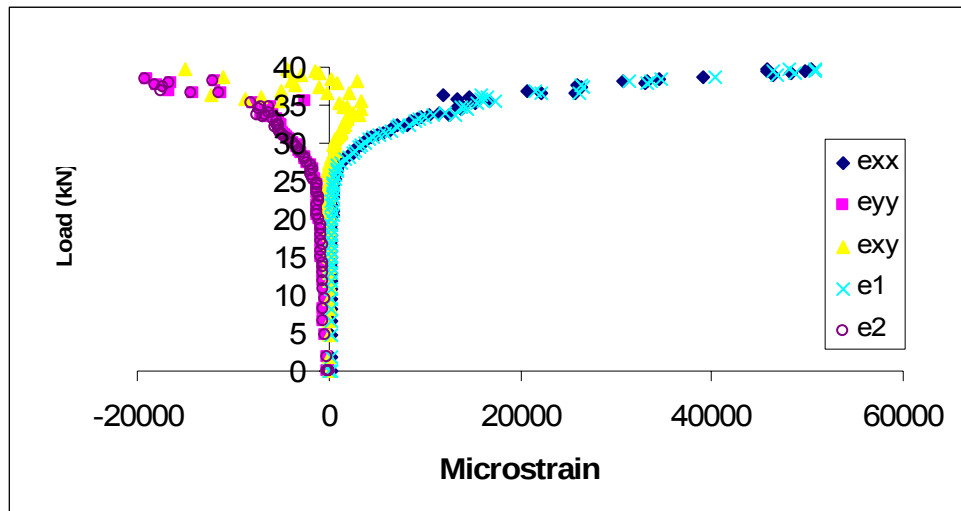


Figure E1. Load vs. Strain Diagram for S2

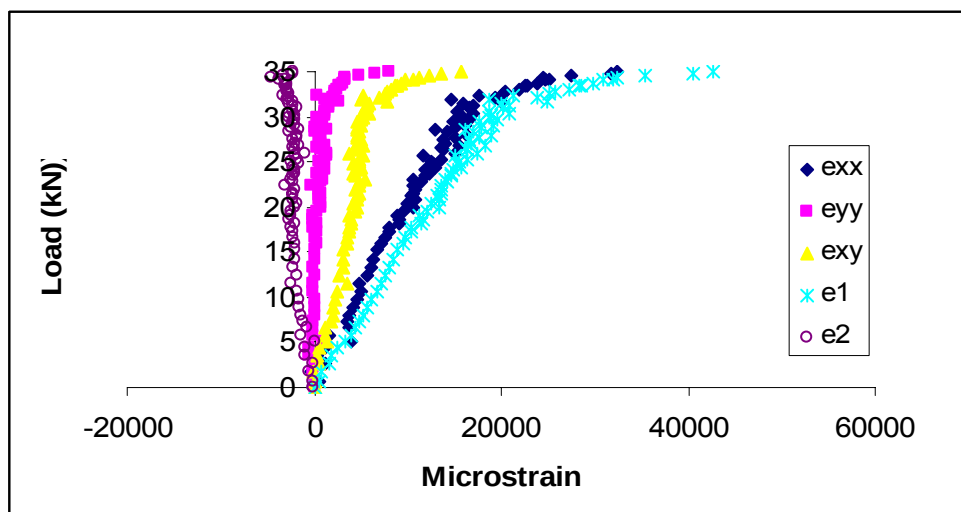


Figure E2. Load vs. Strain Diagram for S9

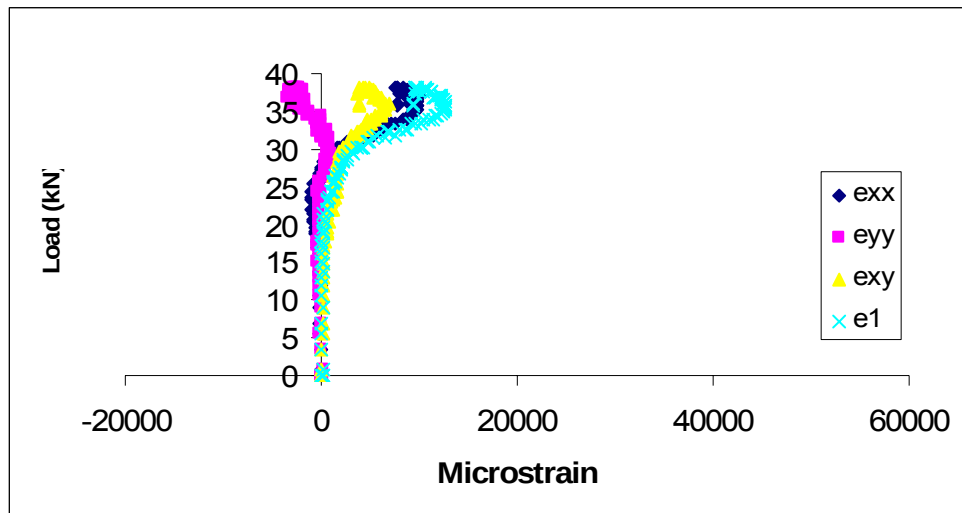


Figure E3. Load vs. Strain Diagram for S8

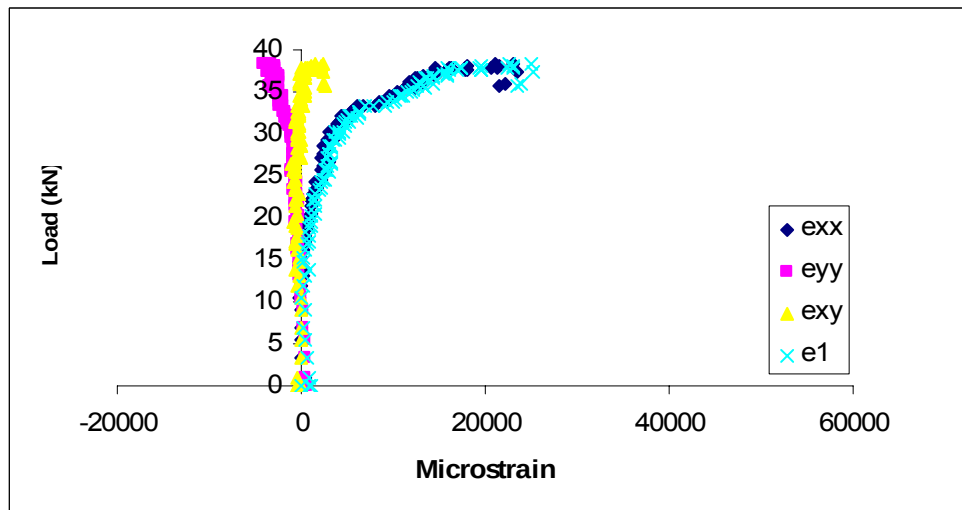


Figure E4. Load vs. Strain Diagram for S7

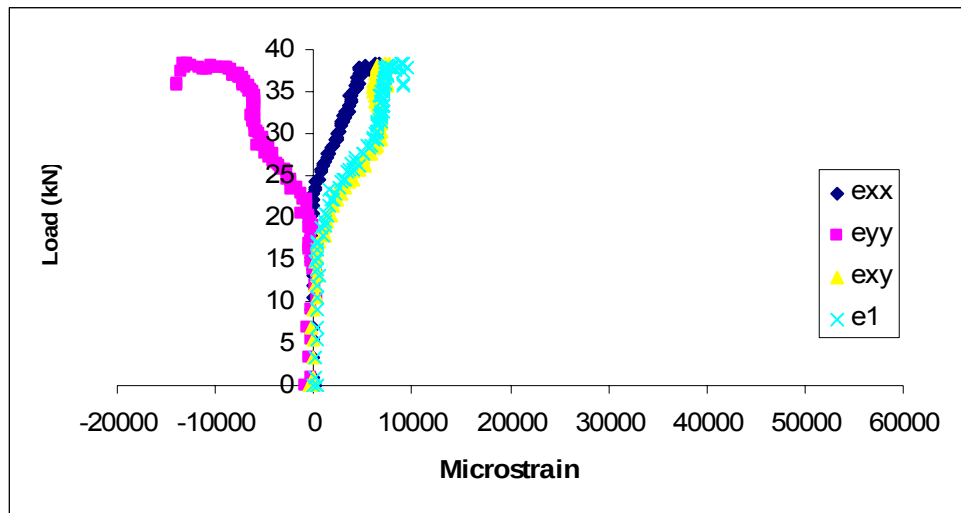


Figure E5. Load vs. Strain Diagram for S6

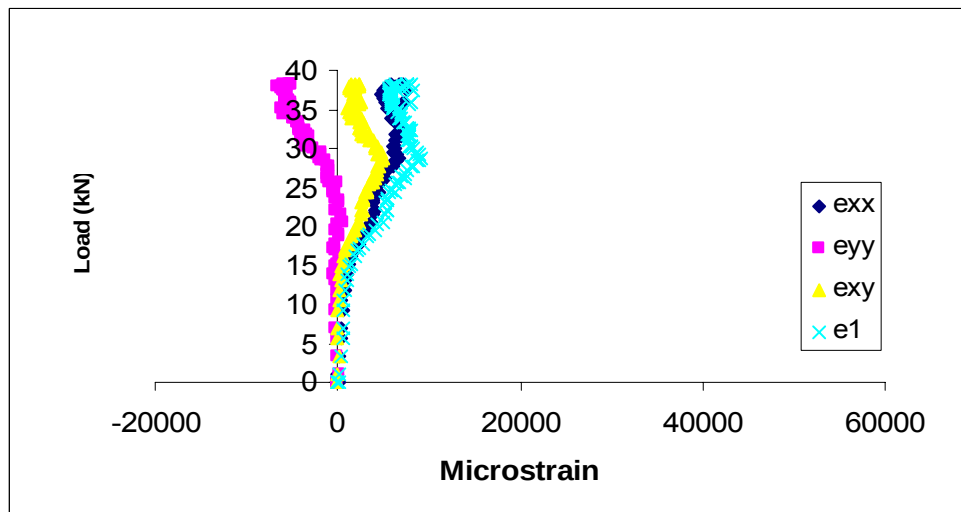


Figure E6. Load vs. Strain Diagram for S5

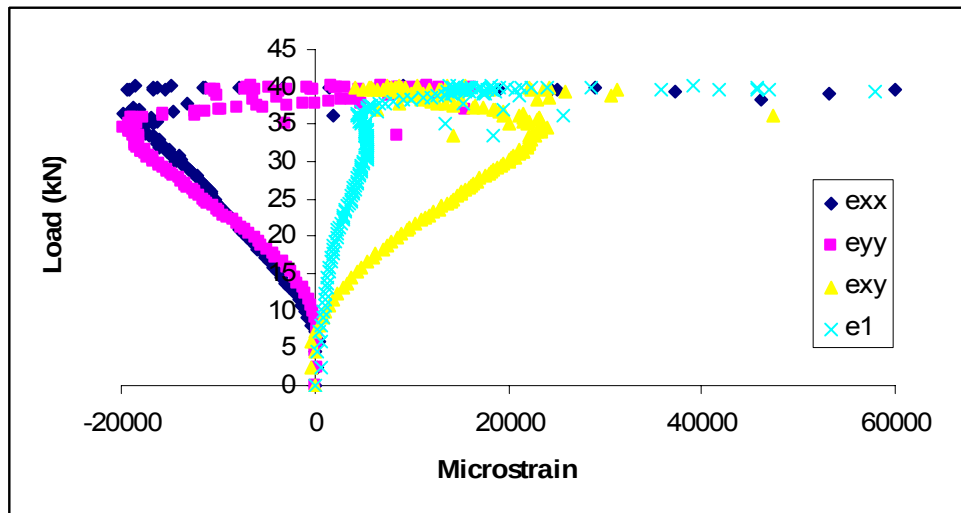


Figure E7. Load vs. Strain Diagram for S11

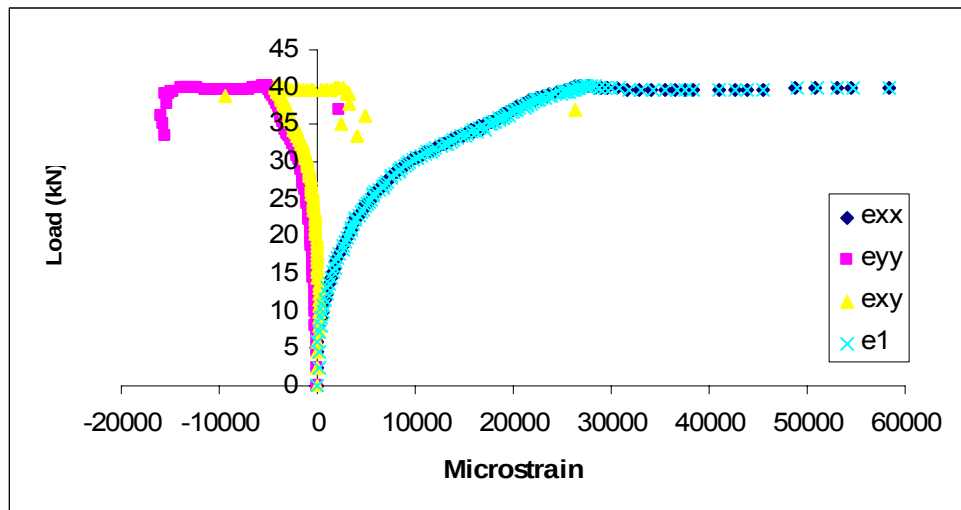


Figure E8. Load vs. Strain Diagram for S12

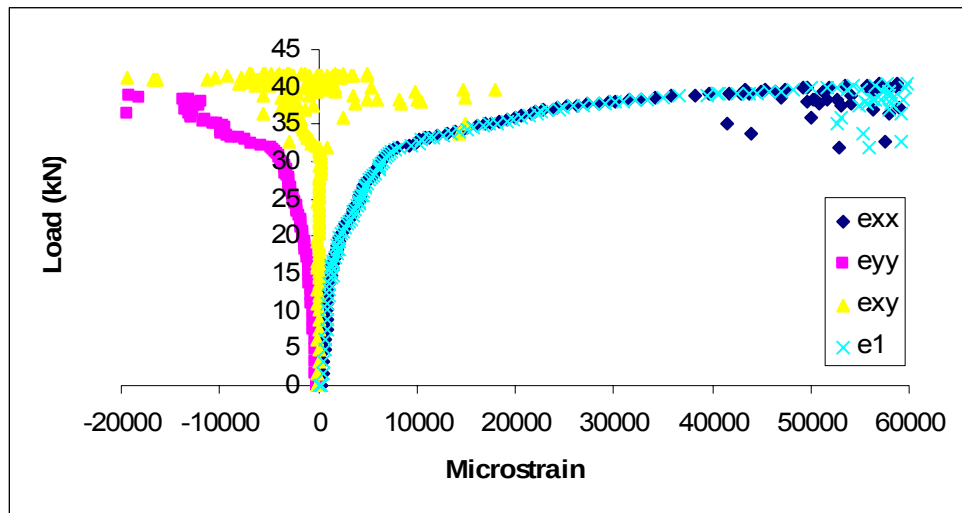


Figure E9. Load vs. Strain Diagram for S13

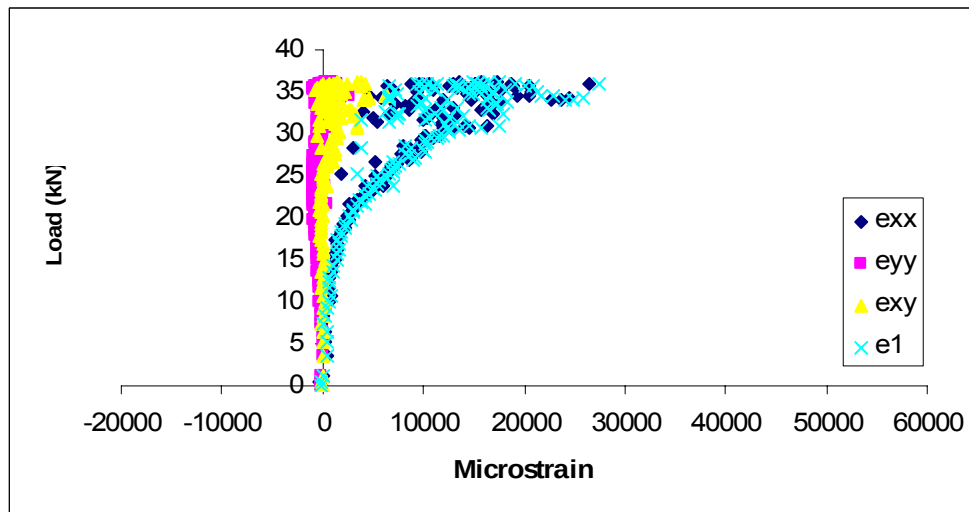


Figure E10. Load vs. Strain Diagram for Zone VI

OSB

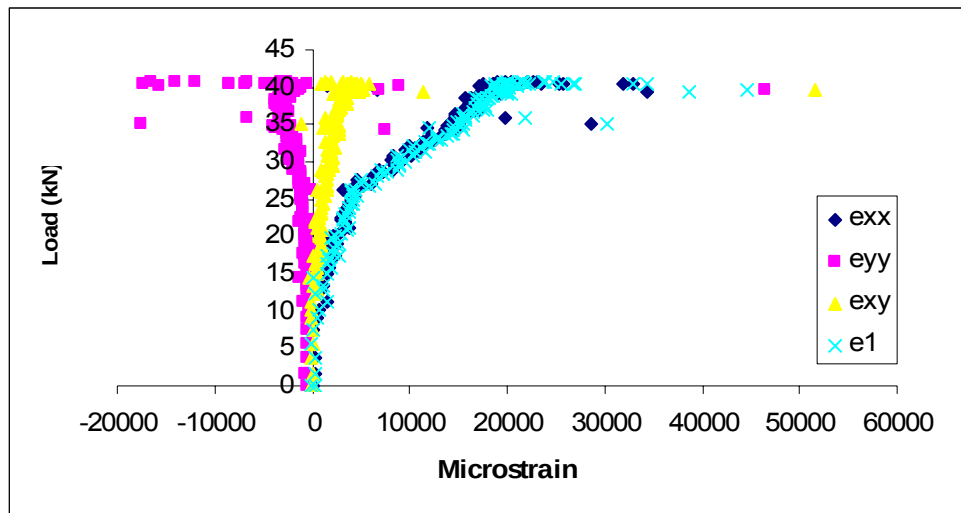


Figure E11. Load vs. Strain Diagram for Nail 2

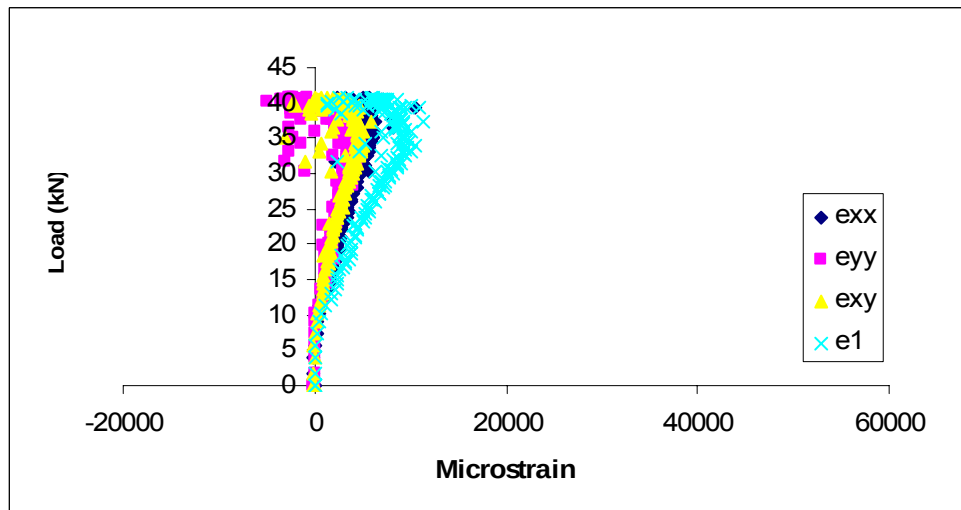


Figure E12. Load vs. Strain Diagram for Nail 4

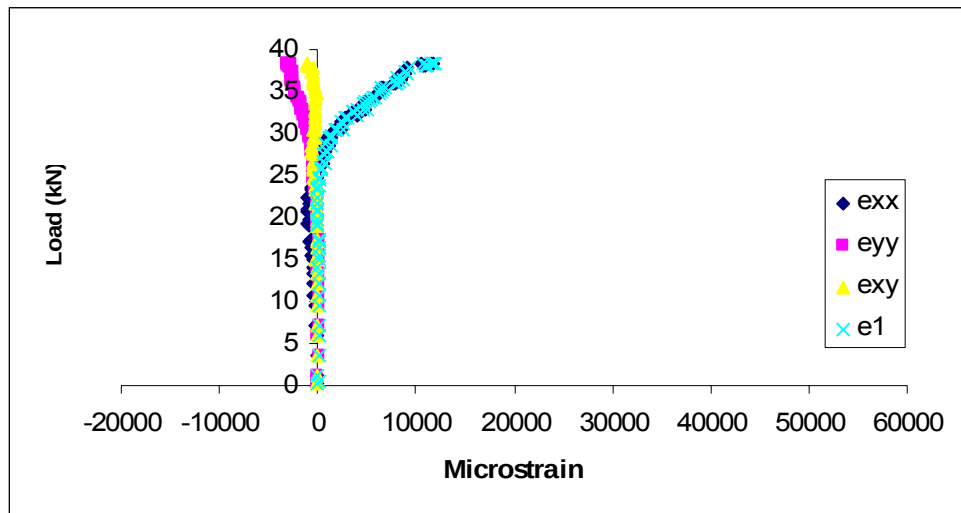


Figure E13. Load vs. Strain Diagram for nail 5

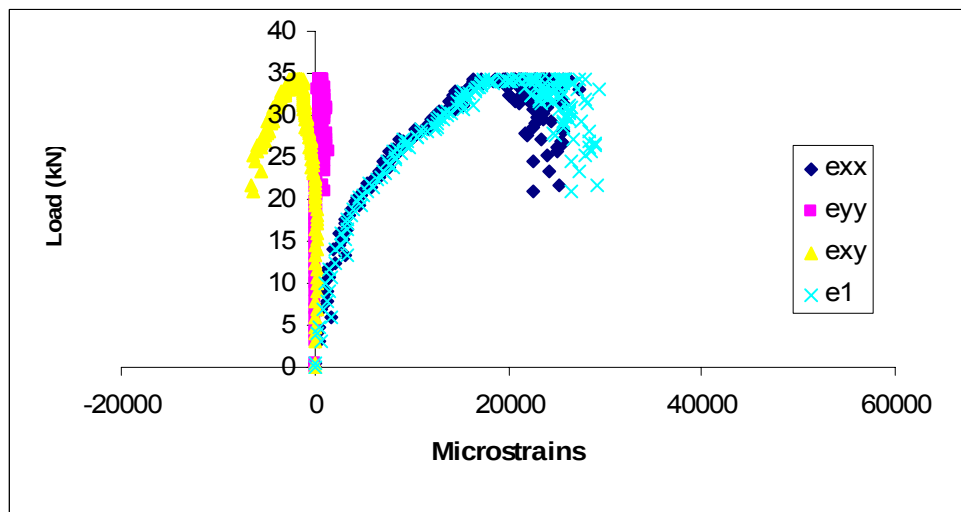


Figure E14. Load vs. Strain Diagram Zone VI

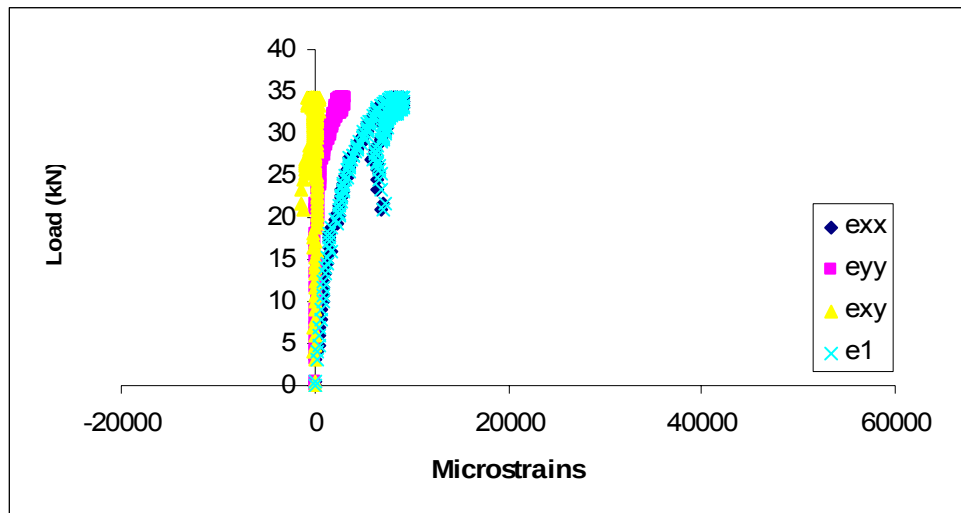


Figure E15. Load vs. Strain Diagram zone VI

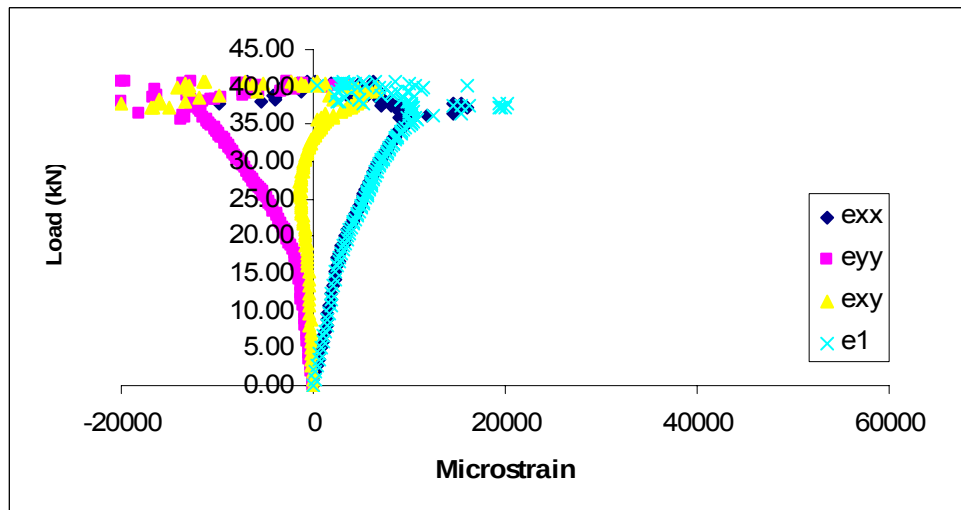


Figure E16. Load vs. Strain Diagram for Nail 9

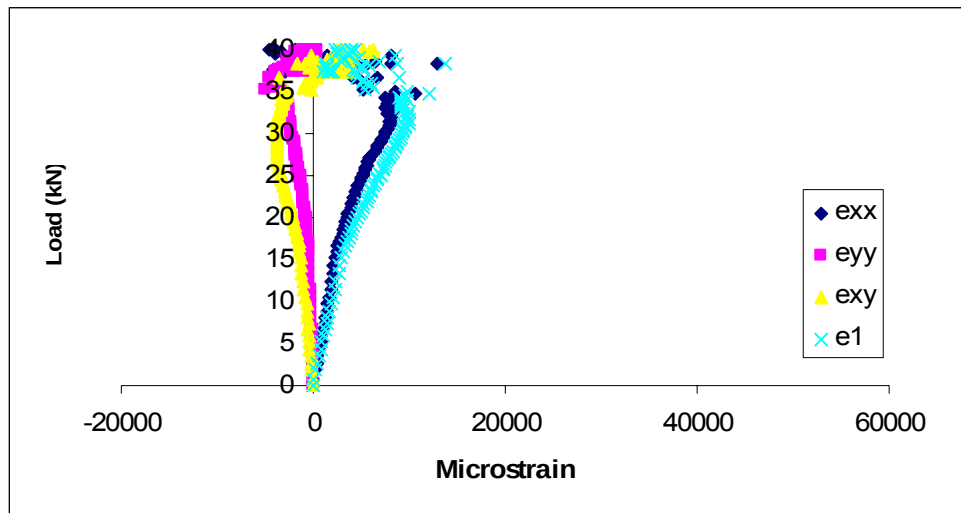


Figure E17. Load vs. Strain Diagram for Nail 10

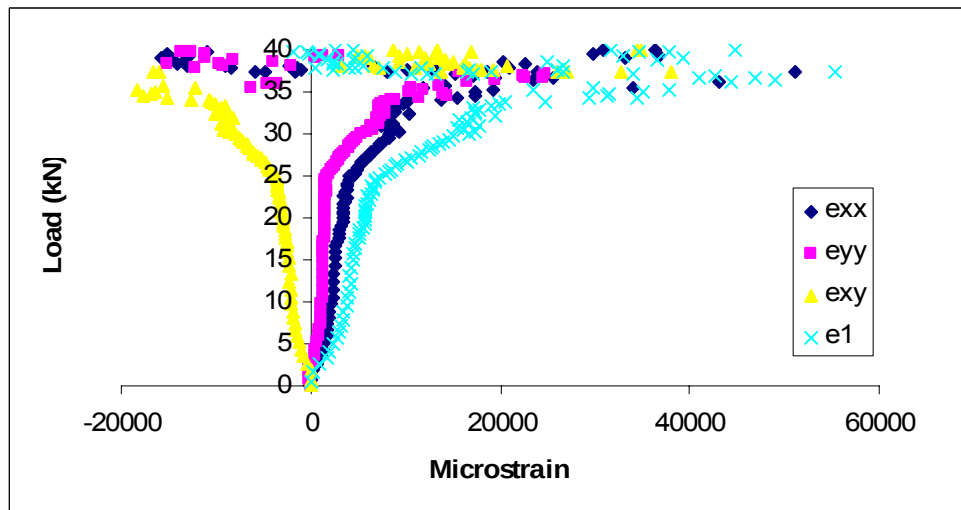


Figure E18. Load vs. Strain Diagram for other fastener in Zone II

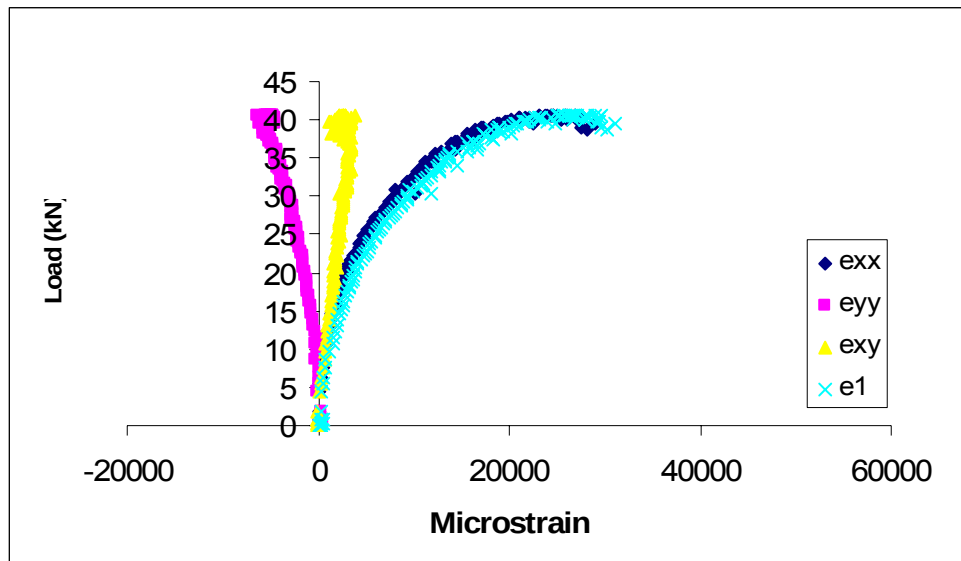


Figure E19. Load vs. Strain Diagram for Nail 12

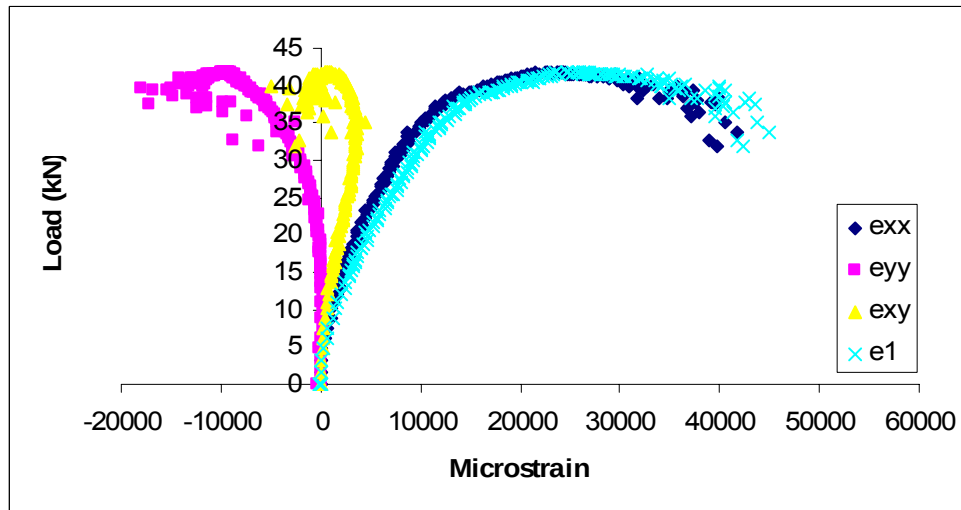


Figure E20. Load vs. Strain Diagram for Nail 13

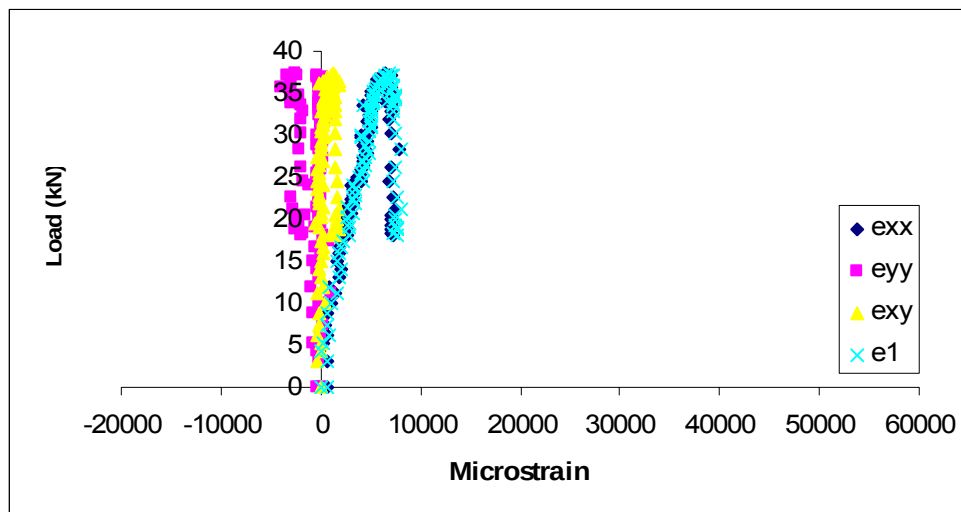
WALL TYPE B**OSB**

Figure E21. Load vs. Strain Diagram for Nail 1

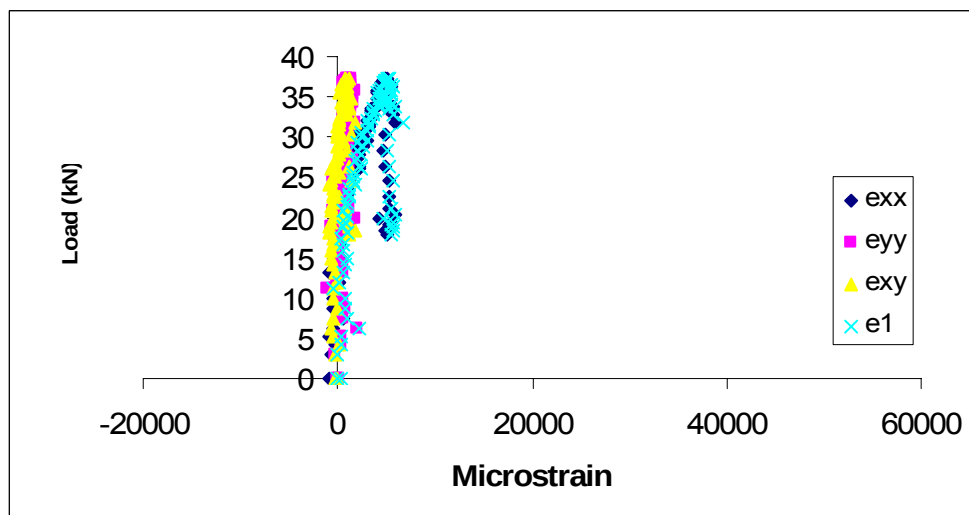


Figure E22. Load vs. Strain Diagram for Nail 2

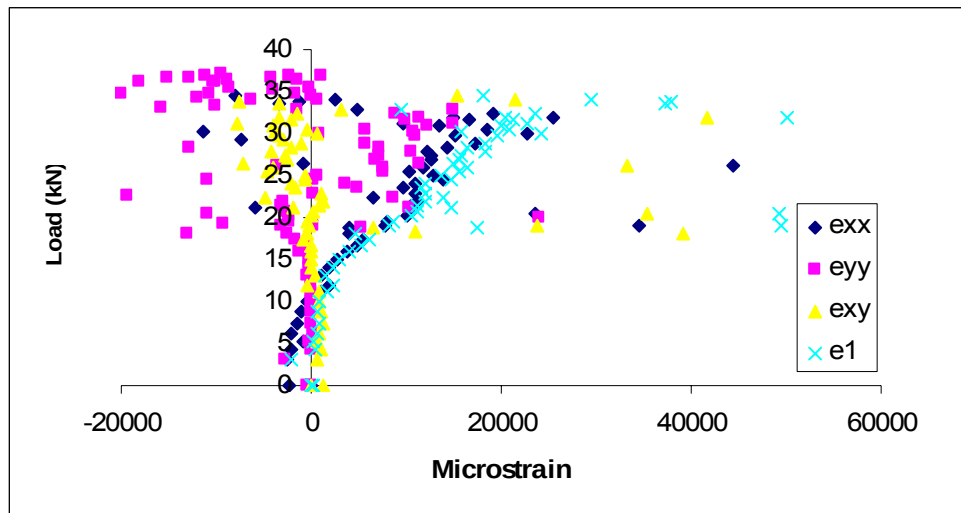


Figure E23. Load vs. Strain Diagram for Nail 4

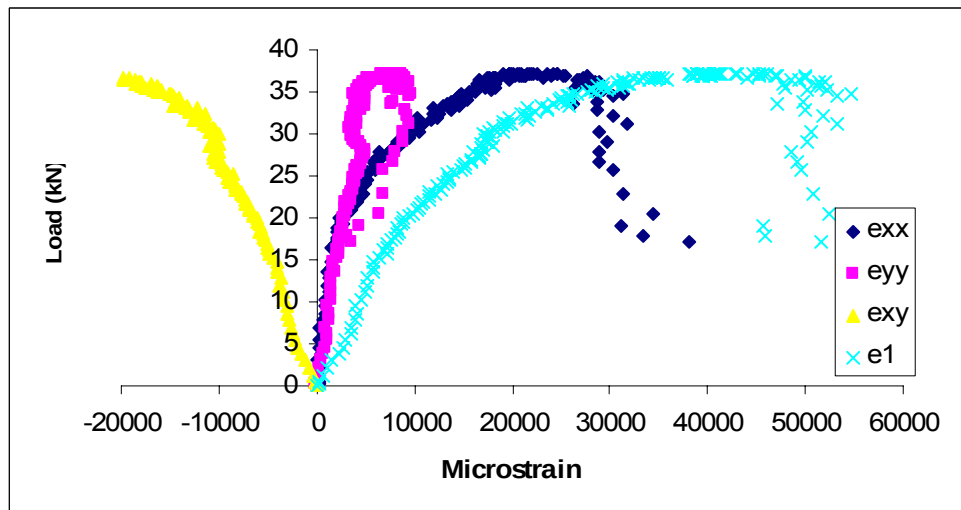


Figure E24. Load vs. Strain Diagram for Nail 5

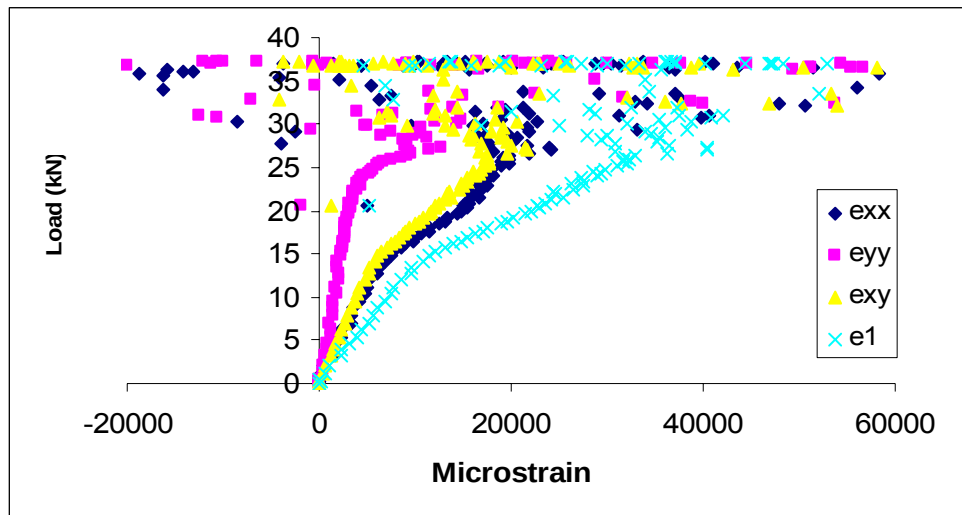


Figure E25. Load vs. Strain Diagram for Nail 6

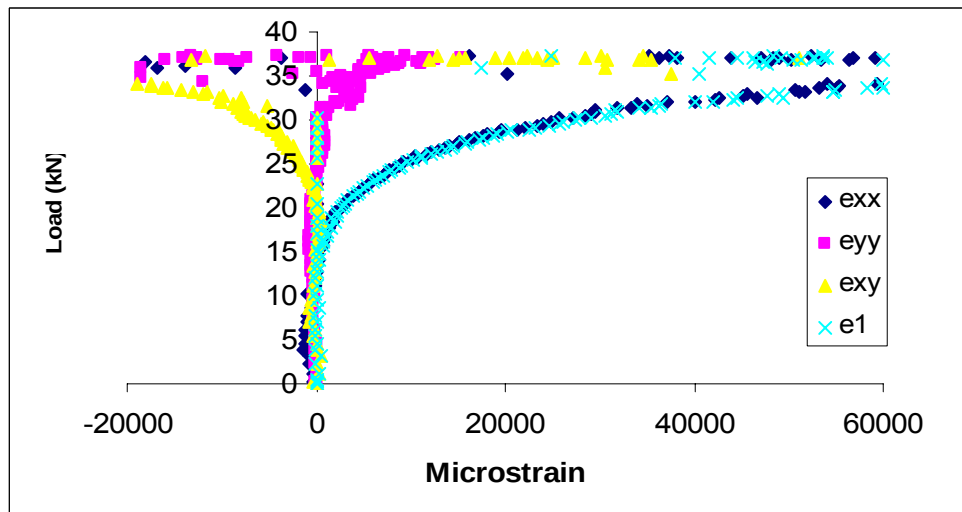


Figure E26. Load vs. Strain Diagram for Nail 8

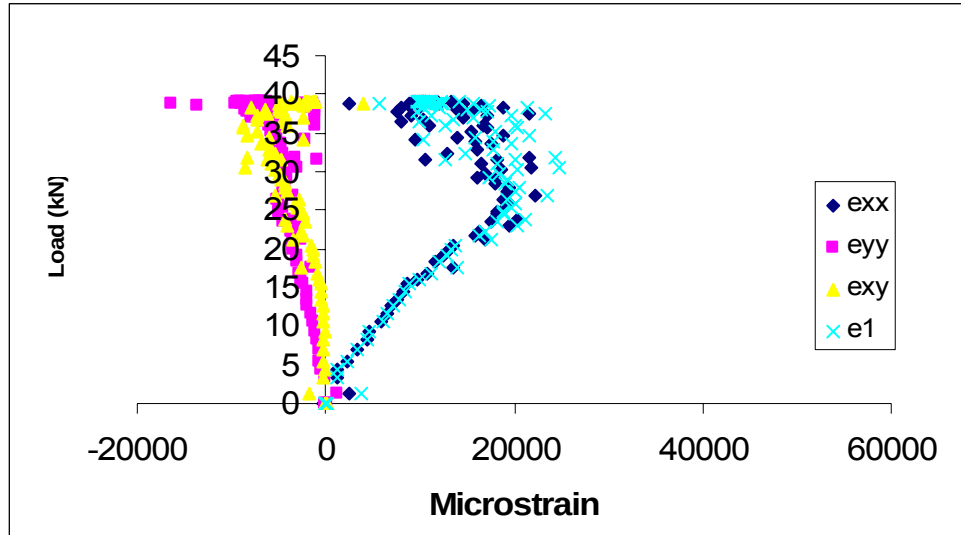


Figure E27. Load vs. Strain Diagram for Nail 9

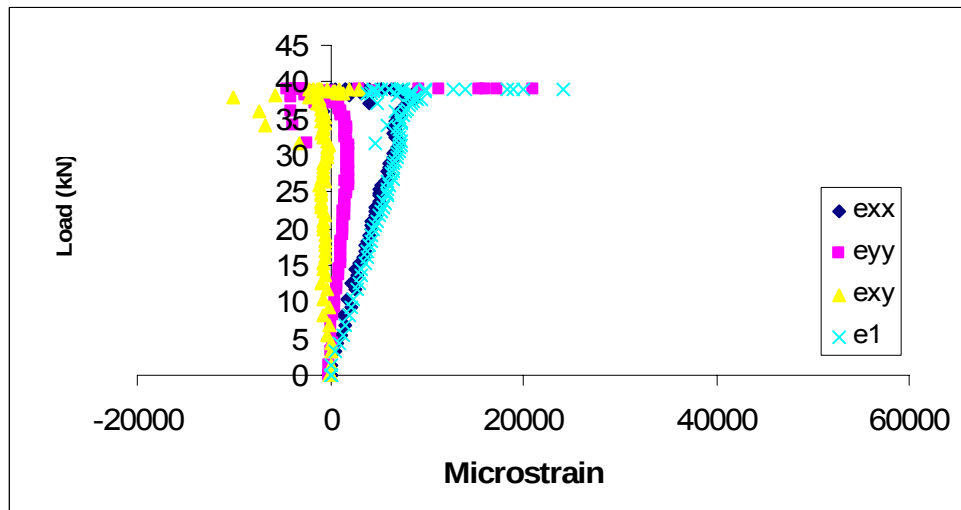


Figure E28. Load vs. Strain Diagram for Nail 10

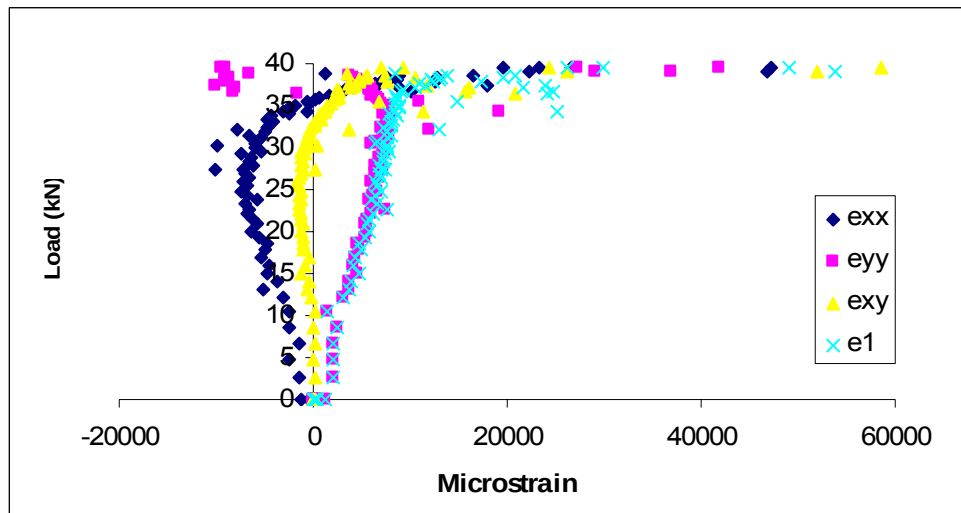


Figure E29. Load vs. Strain Diagram for Nail 11

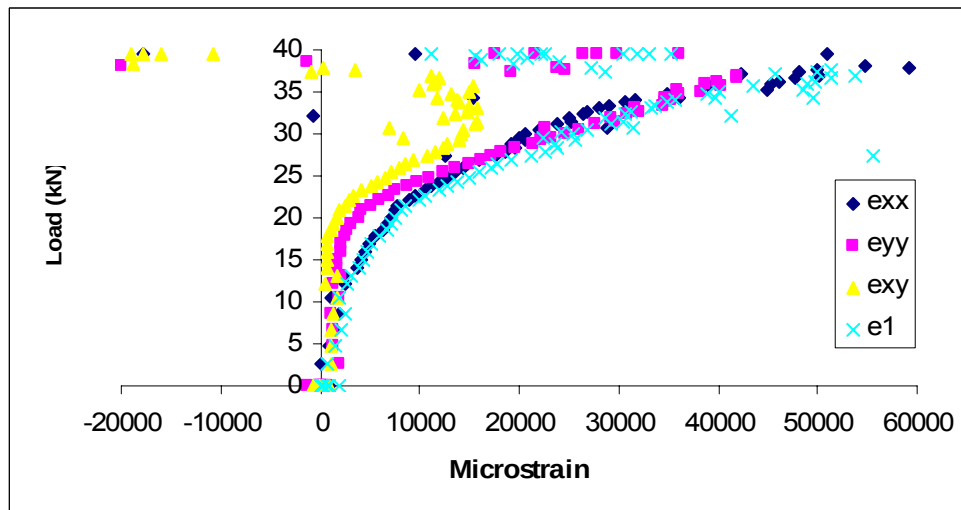


Figure E30. Load vs. Strain Diagram for Nail 12

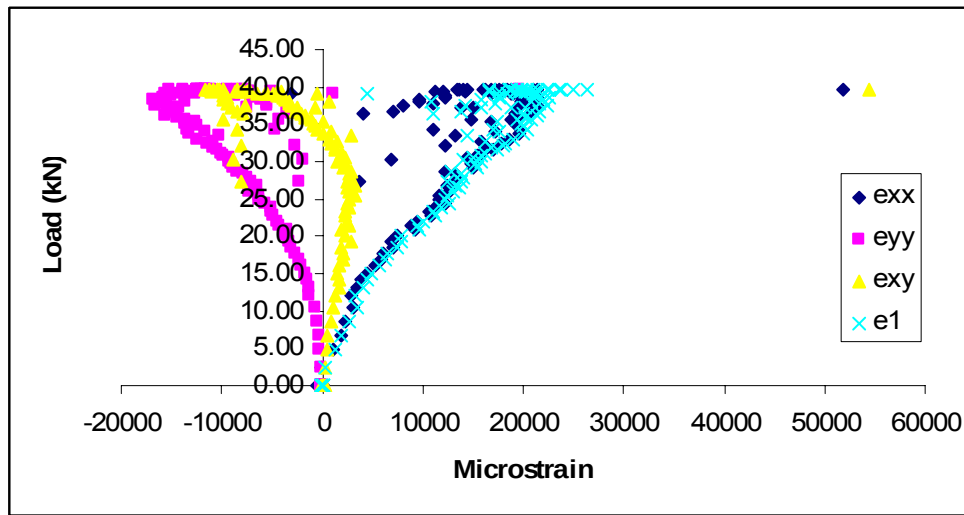


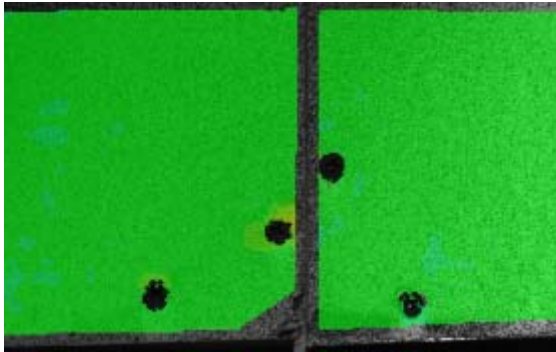
Figure E31. Load vs. Strain Diagram for Nail 13

-0.06 Principle strain (e1) scale + 0.06

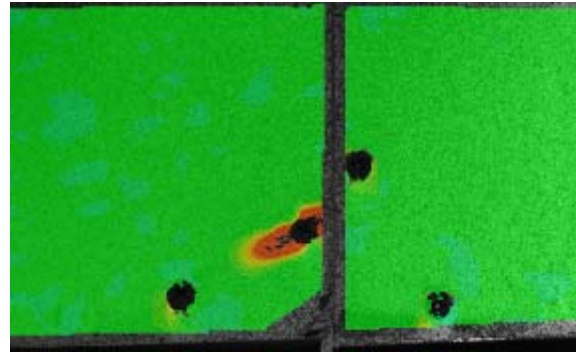
Figure S1. Scale of reference for strain profile.

GWB

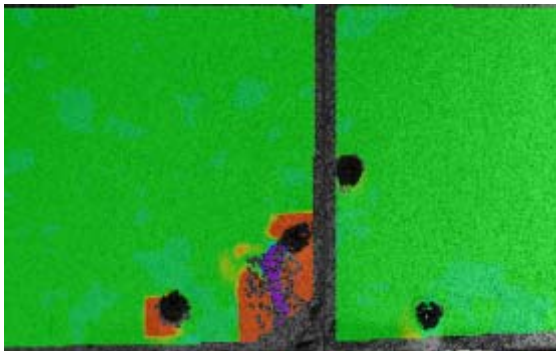
Wall 1 (GWB)



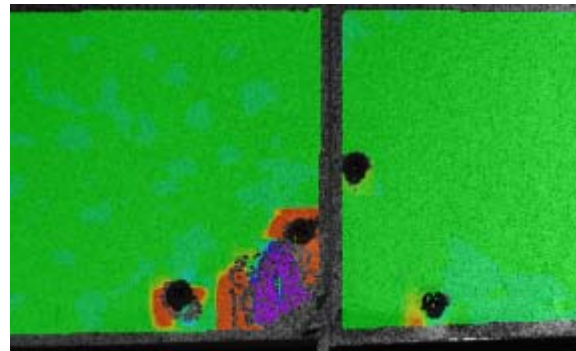
A. 10 kN



B. 20 kN



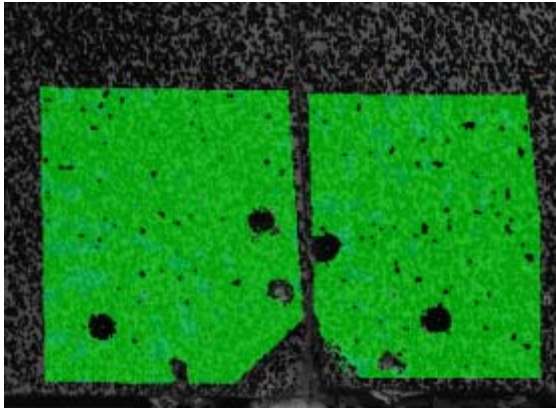
C. 30 kN



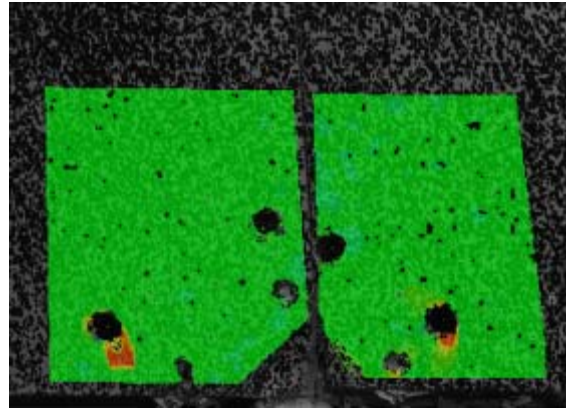
D. failure

Figure F1. Strain Profile at middle of the sill region (Zone III) for wall 1

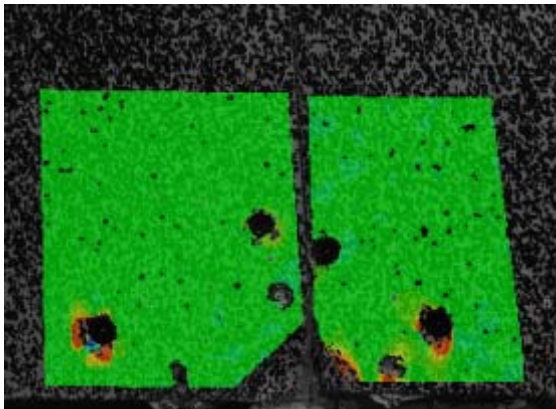
Wall 2 (GWB)



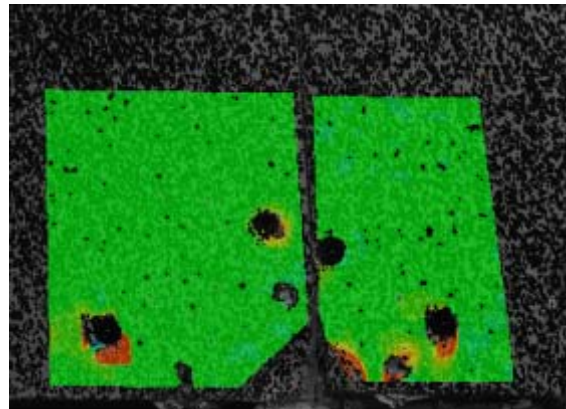
A. 10 kN



B. 20 kN



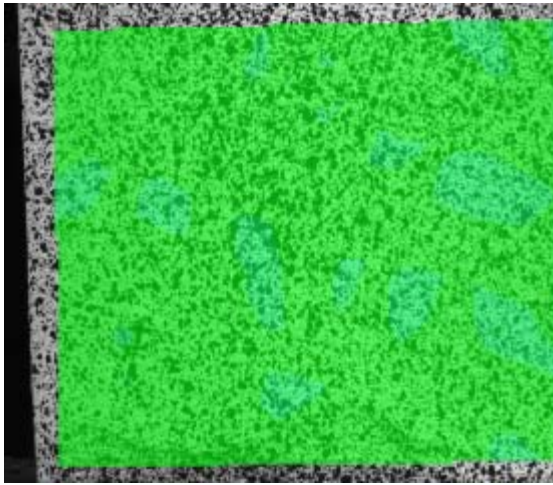
C. 30 kN



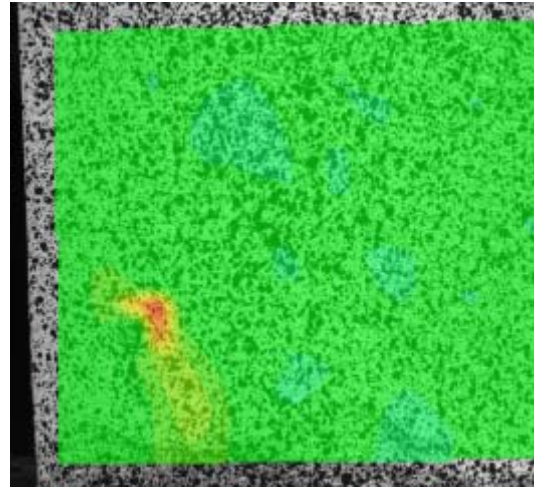
D. failure

Figure F2. Strain Profile at middle of the sill region (Zone III) for wall 2

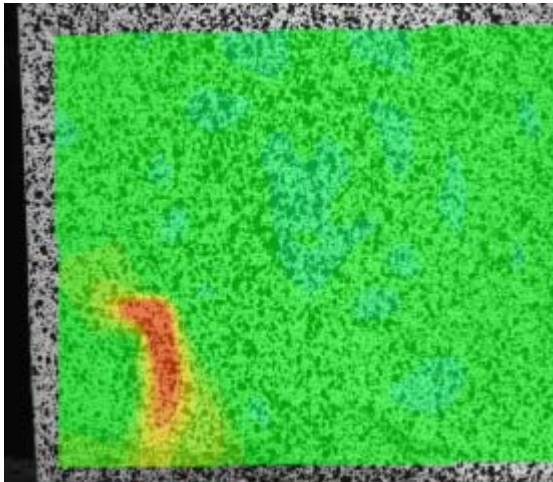
Wall 5 (GWB)



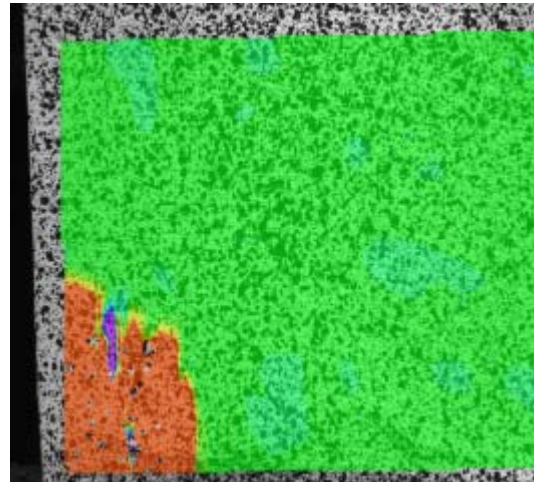
A. 10 kN



B. 20 kN



C. 30 kN

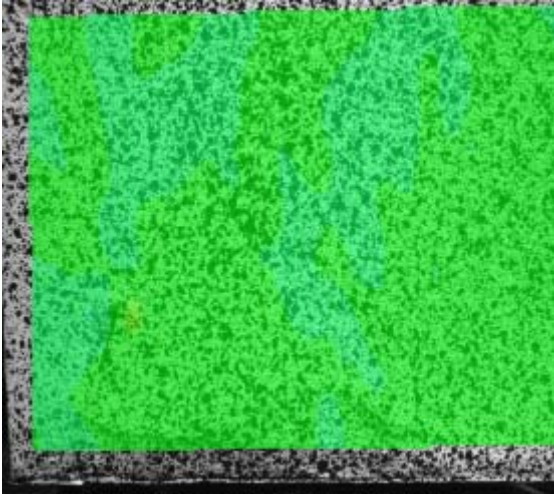


D. failure.

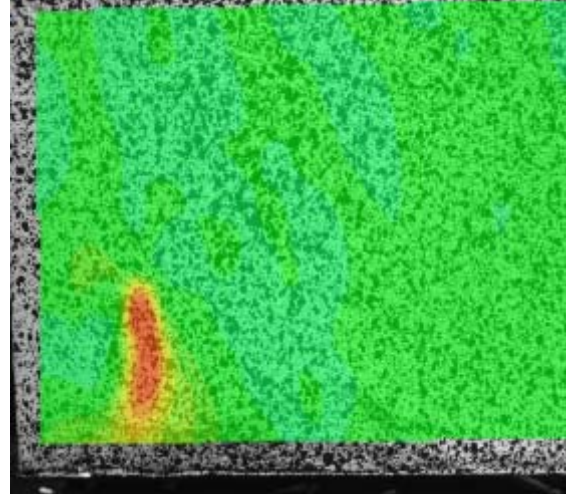
Figure F3. Strain profile at the uplift corner (Zone II) for wall 5

Wall 5 (GWB)

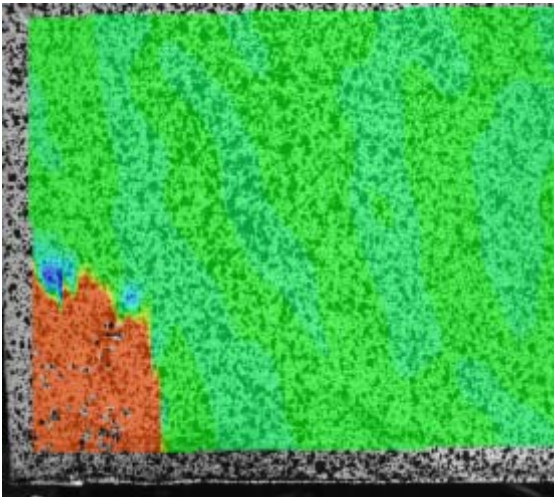
e_{xx}



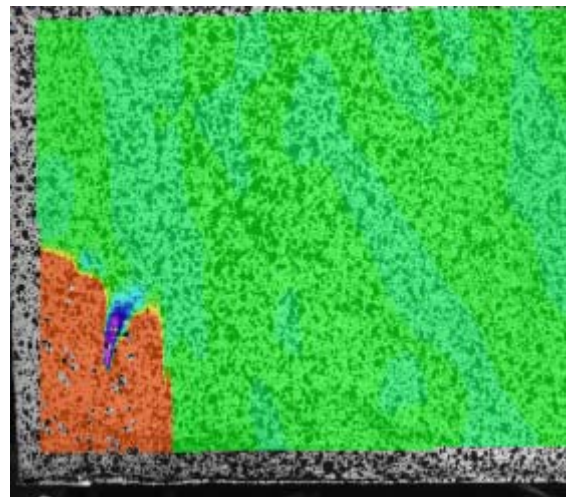
A. 10 kN



B. 20 kN

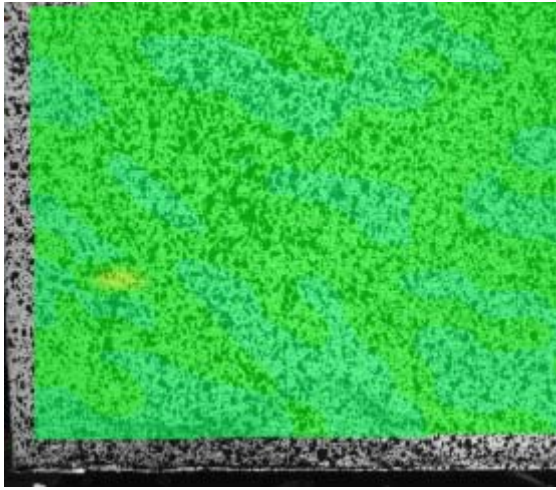


C. 30 kN

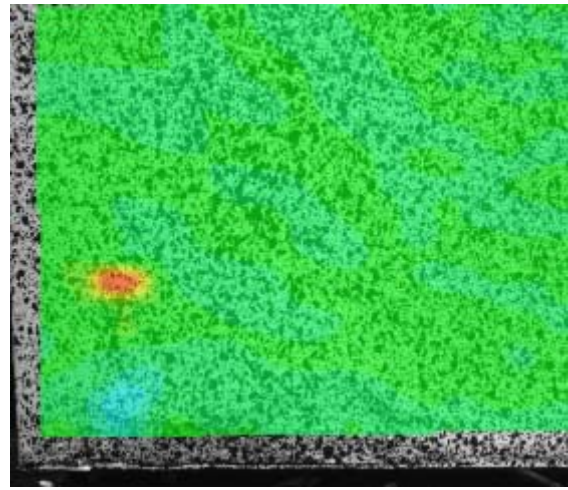


D. failure

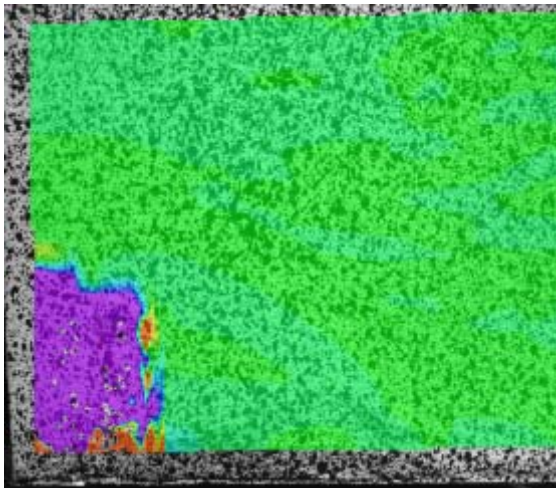
Figure F4. Strain profile (e_{xx}) at uplift corner (Zone II) for wall 5

ϵ_{yy} 

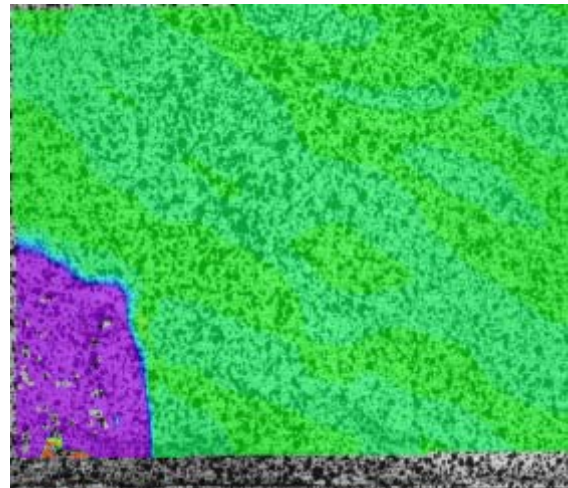
A. 10 kN



B. 20 kN

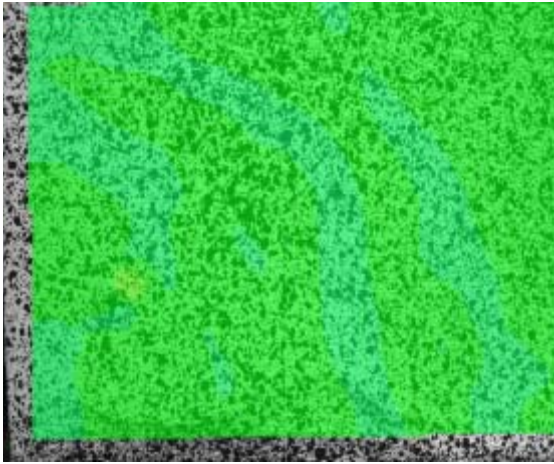


C. 30 kN

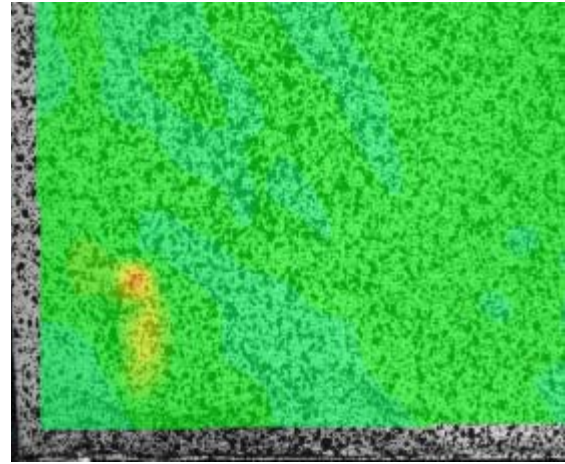


D. failure

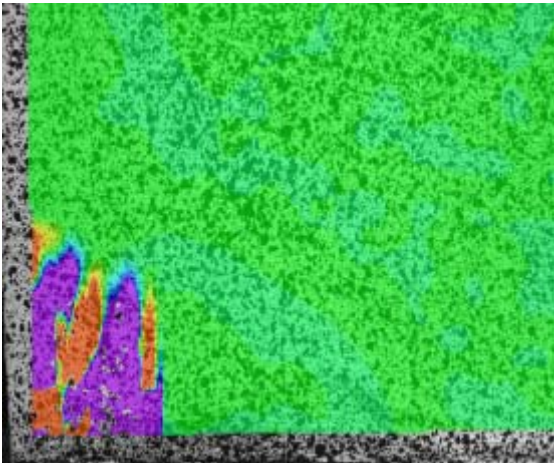
Figure F5. Strain profile (ϵ_{yy}) at uplift corner (Zone II) for wall 5

ϵ_{xy} 

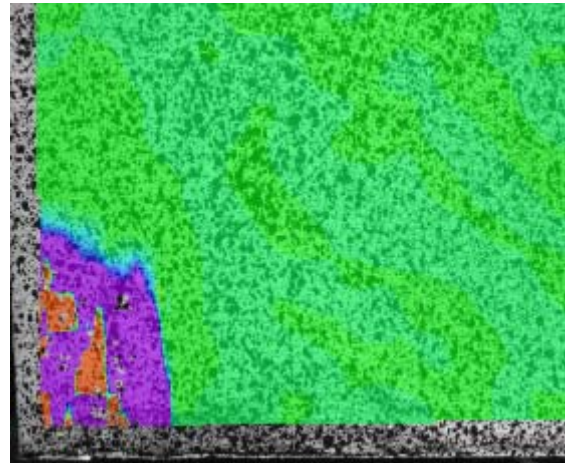
A. 10 kN



B. 20 kN



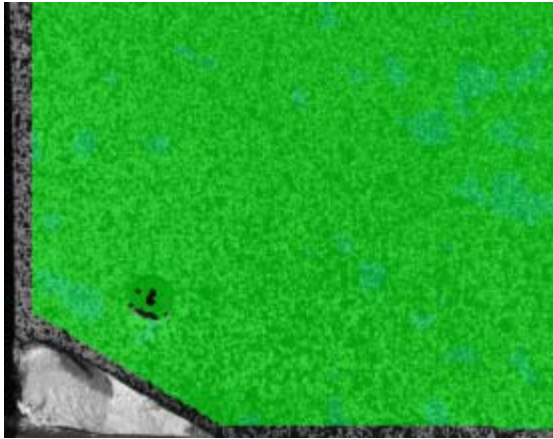
C. 30 kN



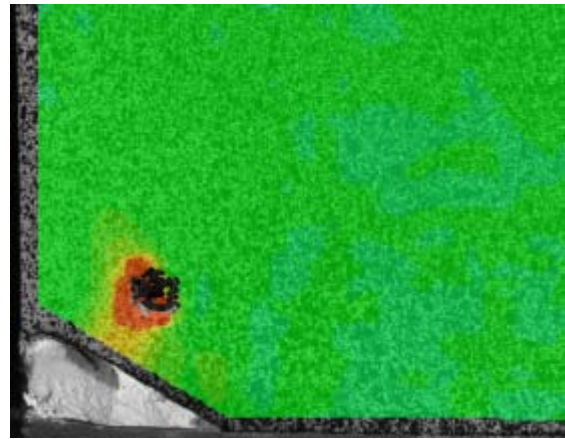
D. failure

Figure F6. Strain profile (ϵ_{xy}) at uplift corner (Zone II) for wall 5

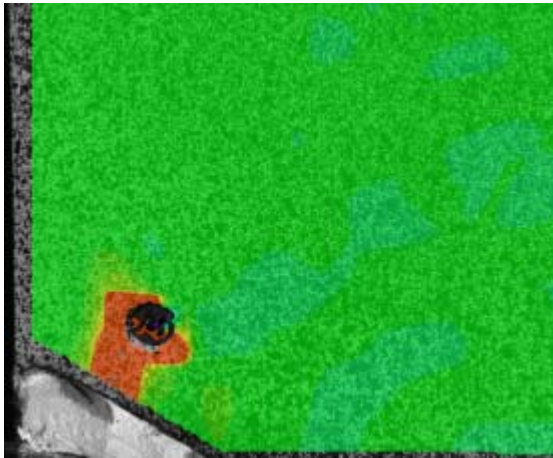
Wall 6 (GWB)



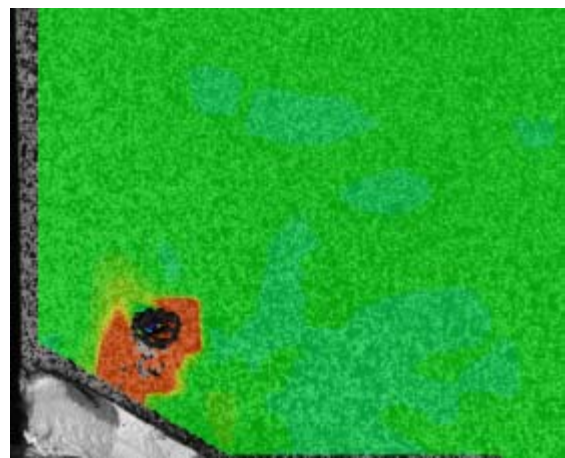
A. 10 kN



B. 20 kN



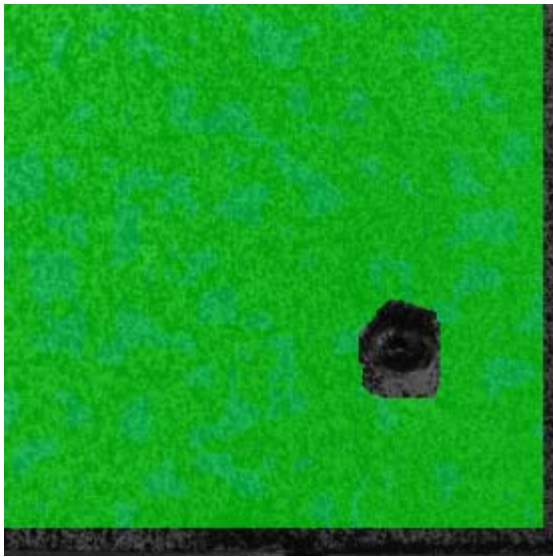
C. 30 kN



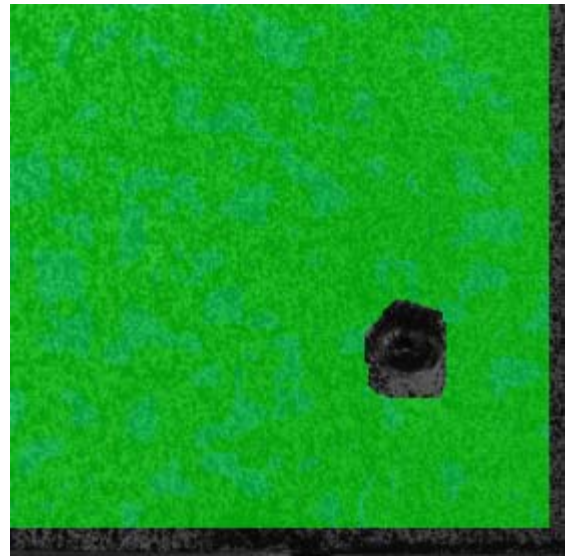
D. failure

Figure F7. Strain profile at the uplift corner (Zone II) for wall 6

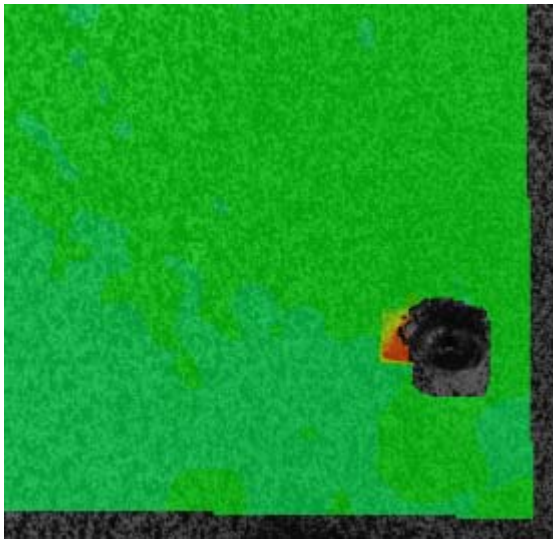
Wall 7 (GWB)



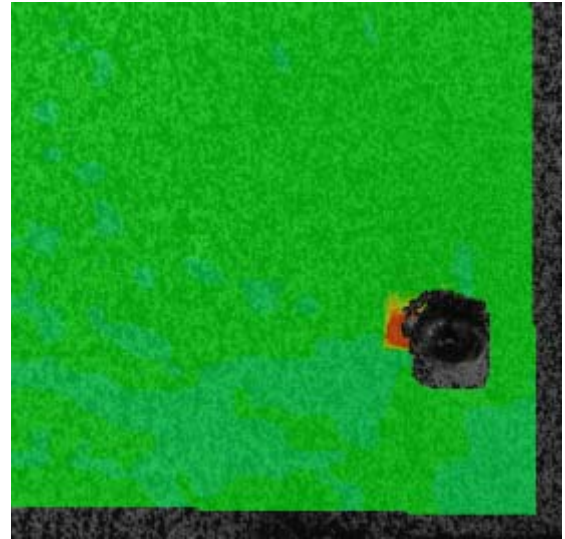
A. 10 kN



B. 20 kN



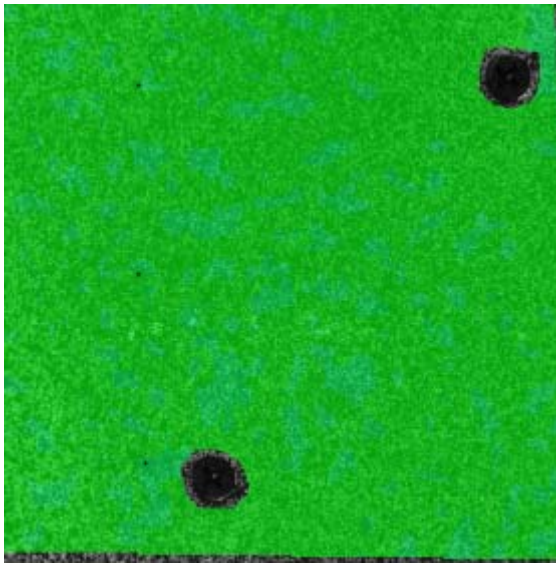
C. 30 kN



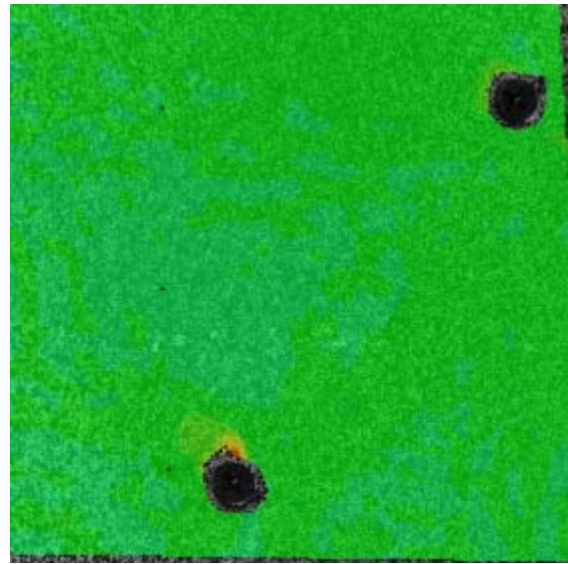
D. failure

Figure F8. Strain profile at the uplift corner (Zone I) for wall 7

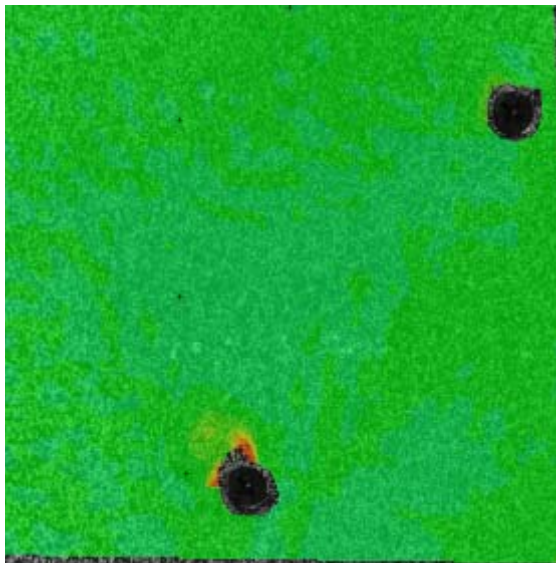
Wall 8 (GWB)



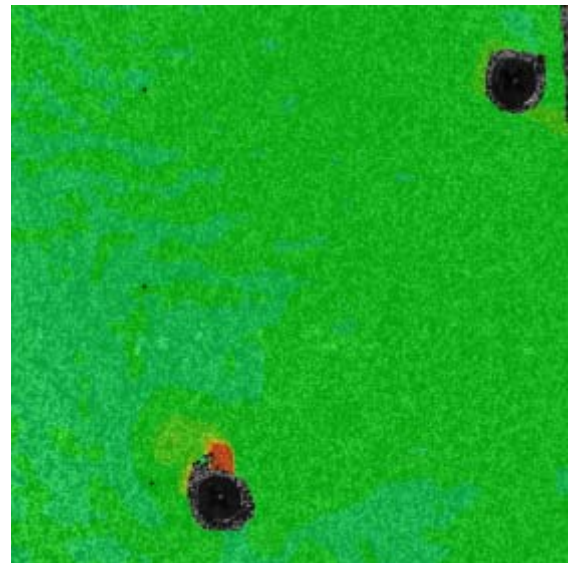
A. 10 kN



B. 20 kN



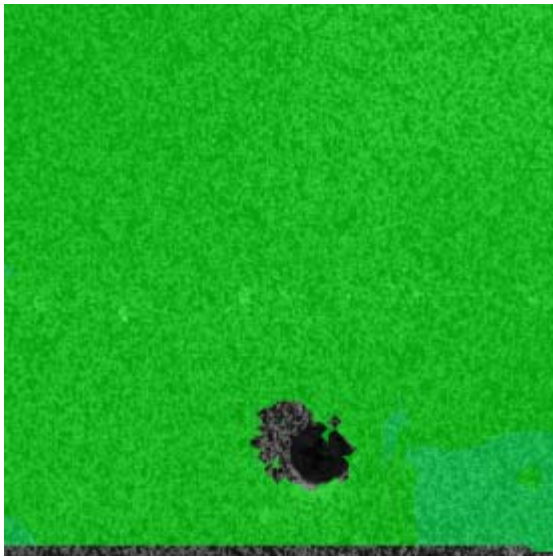
C. 30 kN



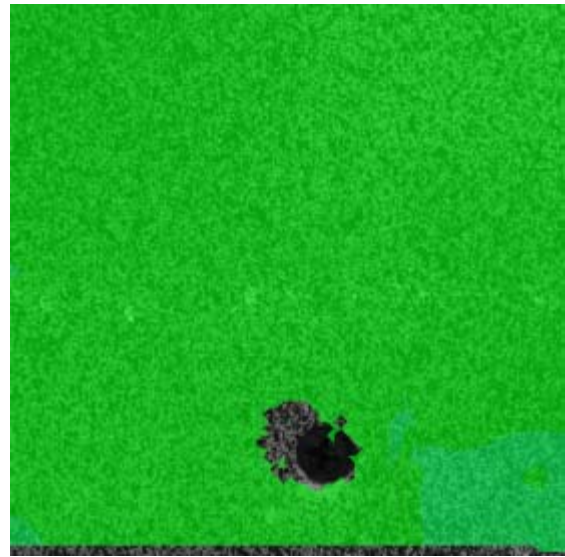
D. failure

Figure F9. Strain profile at the uplift corner (Zone I) for wall 8

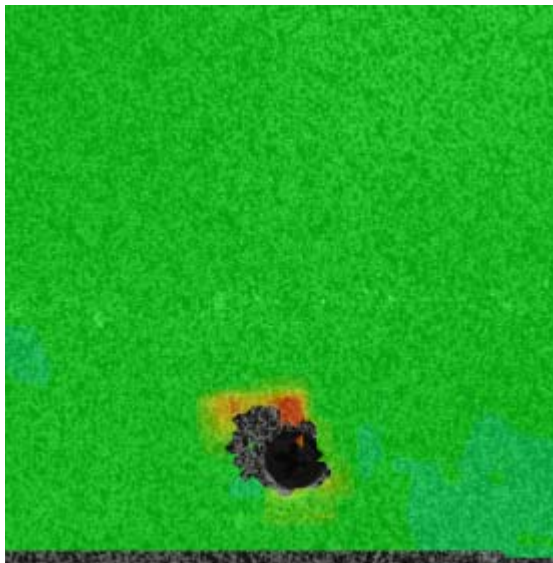
Wall 9 (GWB)



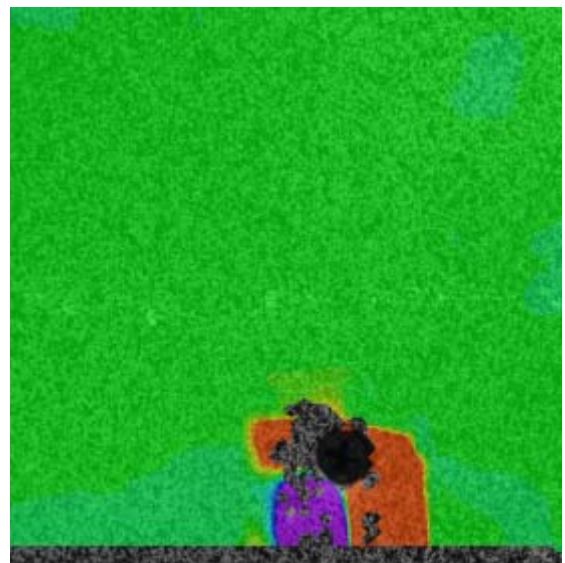
A. 10 kN



B. 20 kN



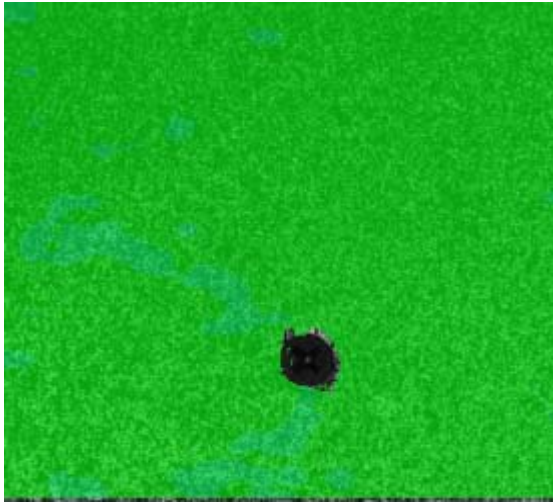
C 30 kN



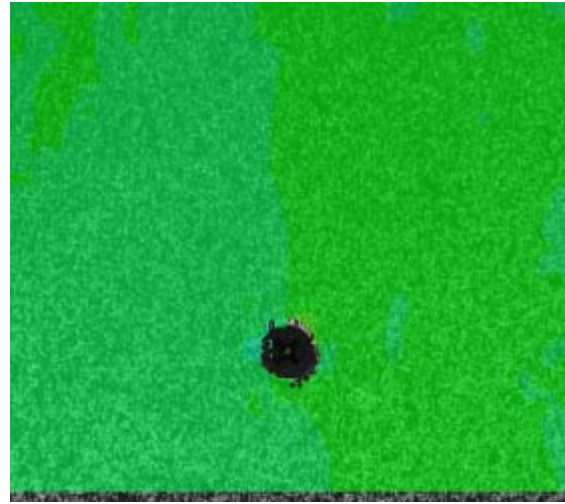
D. failure

Figure F10. Strain profile at the intermediate stud near to uplift corner (Zone VI) for wall 9

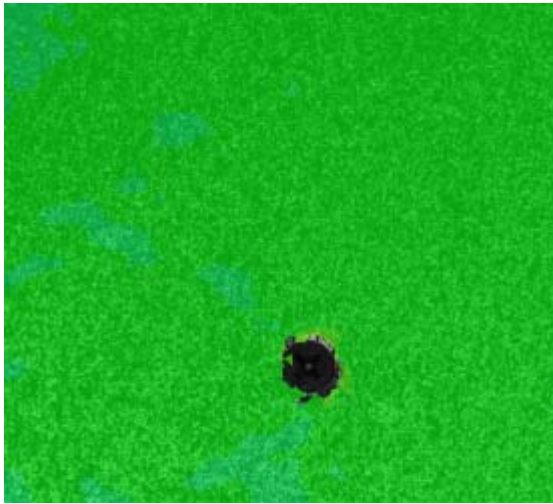
Wall 10 (GWB)



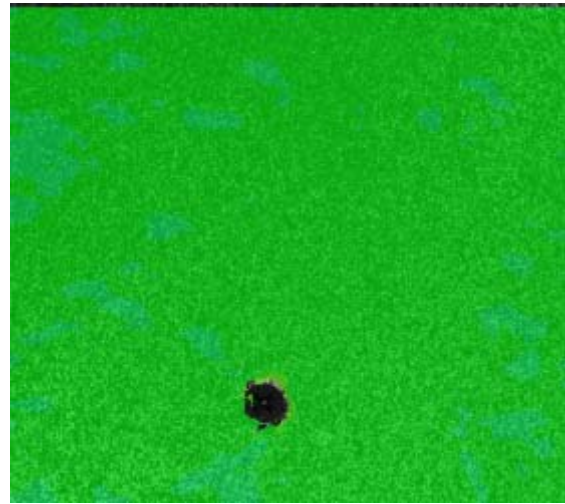
A. 10 kN



B. 20 kN



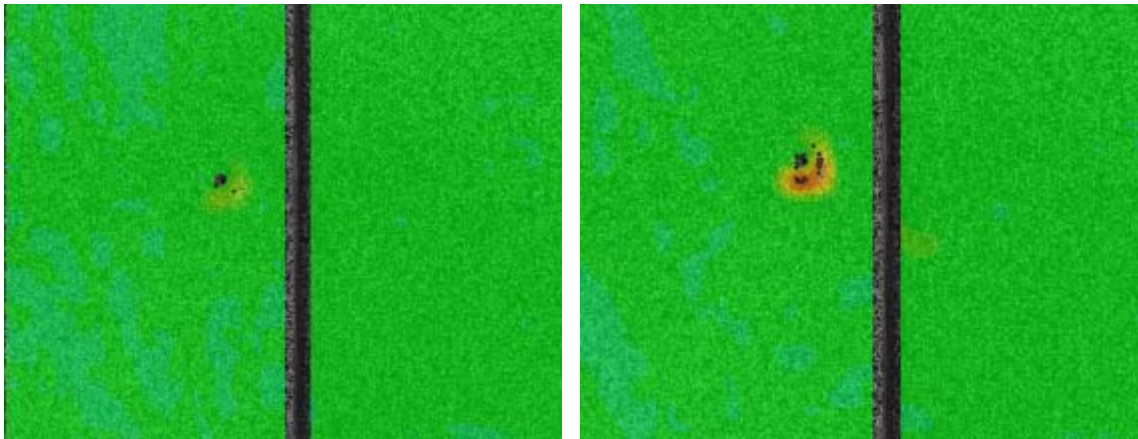
C. 30 kN



D. failure

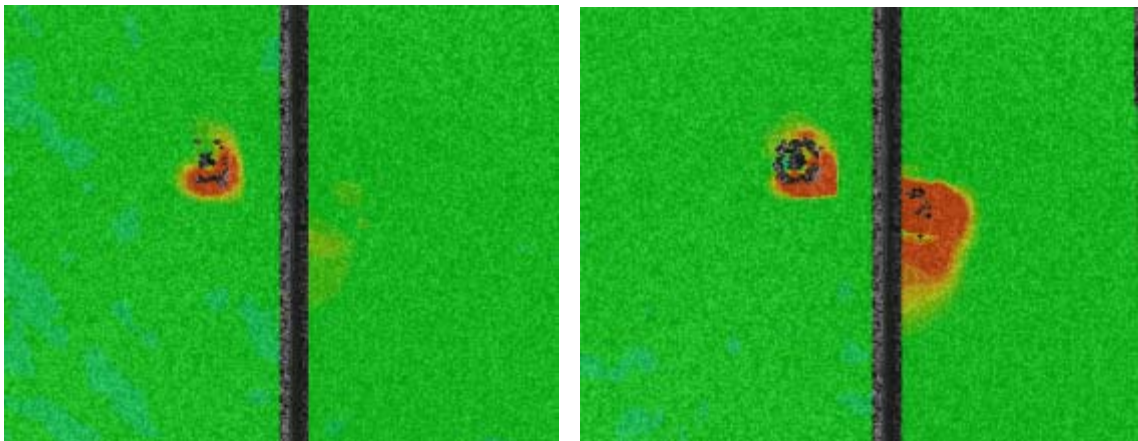
Figure F11. Strain profile at the intermediate stud near to comp. corner (Zone VI) for wall 10

Wall 11 (GWB)



A. 10 kN

B. 20 kN

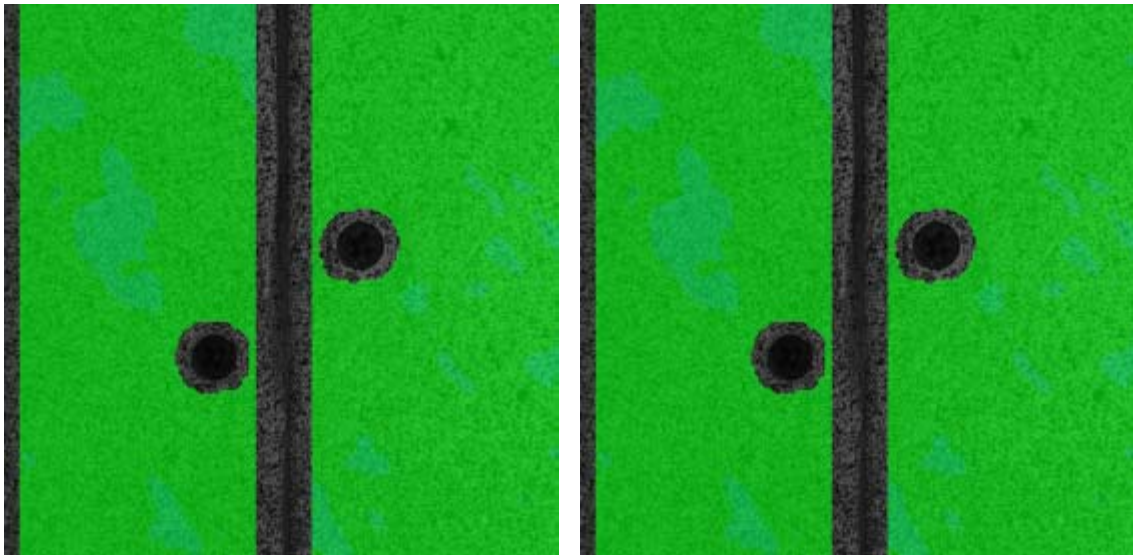


C. 30 kN

D. failure

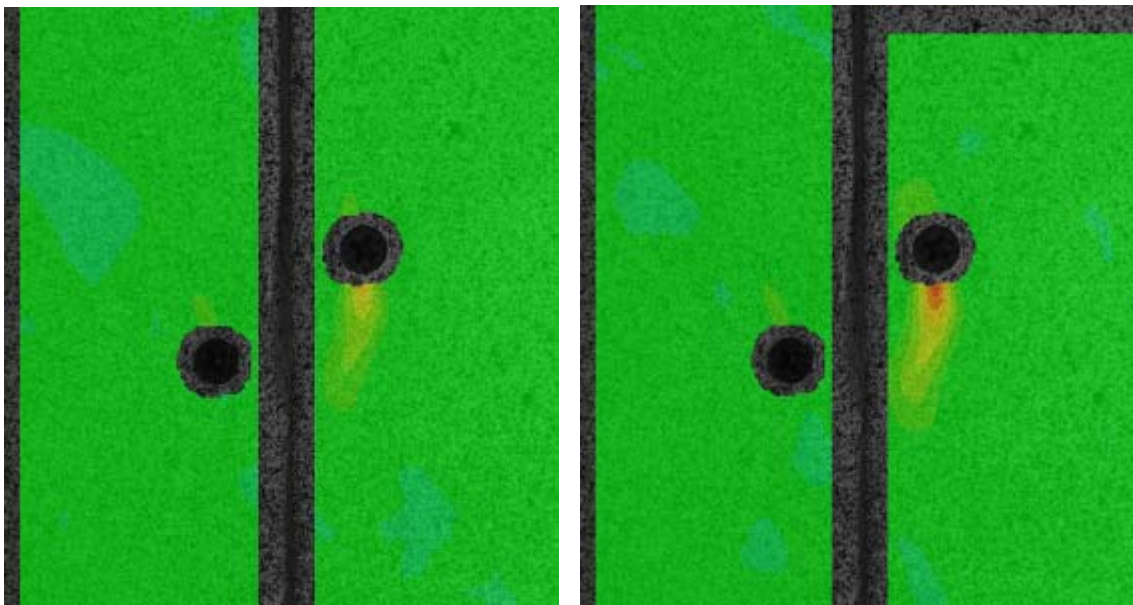
Figure F12. Strain profile at the mid wall region (Zone IV) for wall 11

Wall 12 (GWB)



A. 10 kN

B. 20 kN

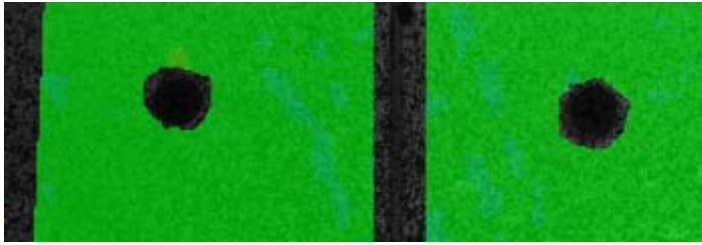


C. 30 kN

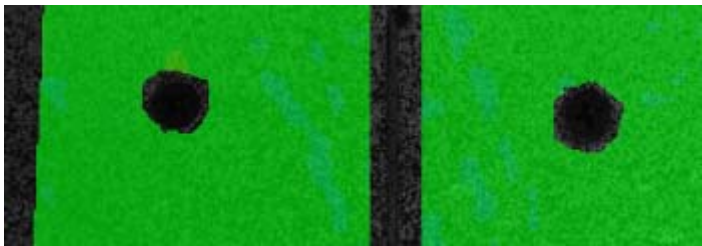
D. failure

Figure F13. Strain profile at the mid wall region (Zone IV) for wall 12

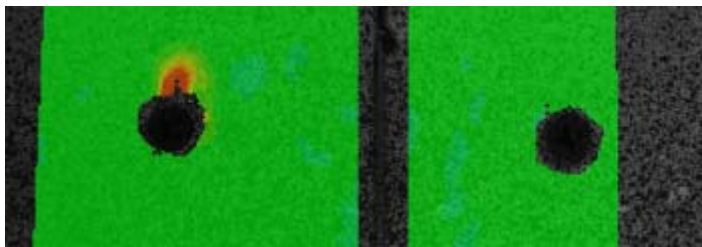
Wall 13 (GWB)



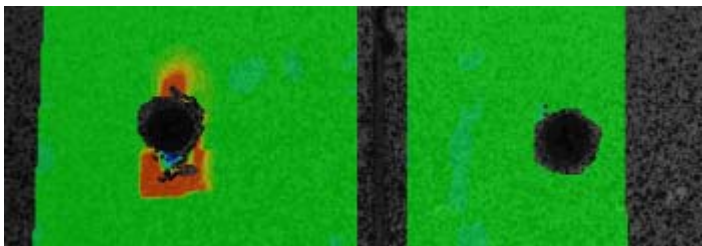
A. 10 kN



B. 20 kN



C. 30 kN

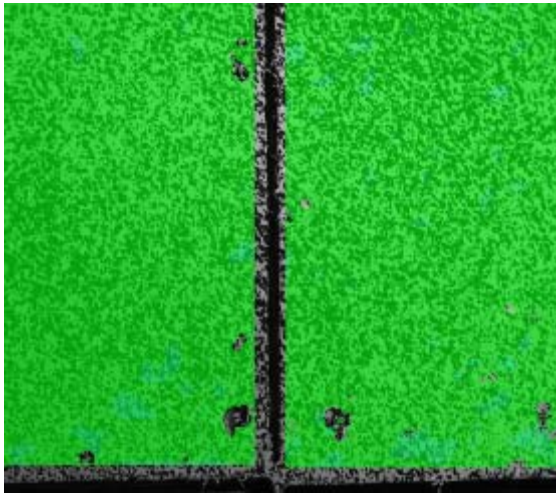


D. Failure

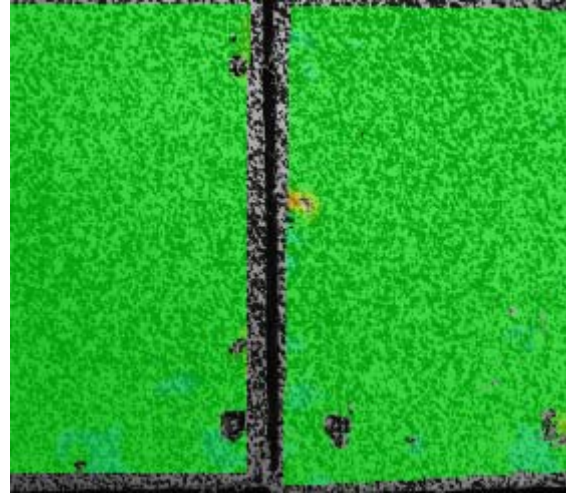
Figure F14. Strain profile at the middle of the top plate (Zone V) for wall 13

OSB

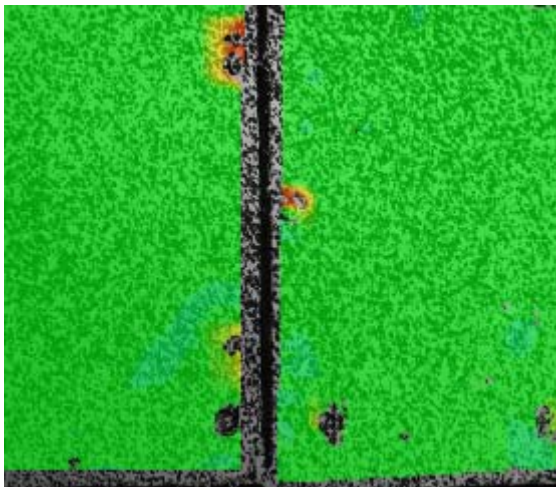
Wall 1 (OSB)



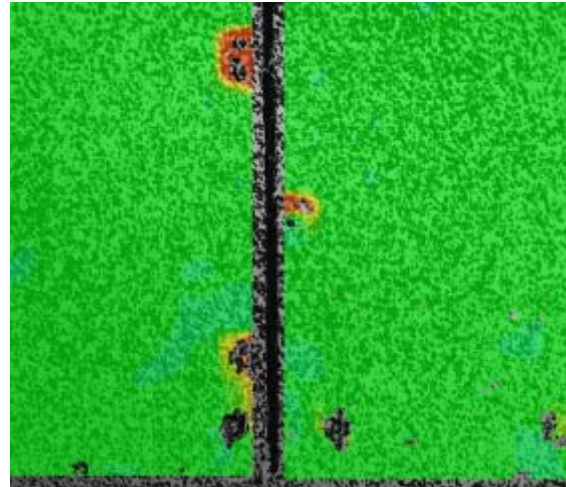
A. 10 kN



B. 20 kN



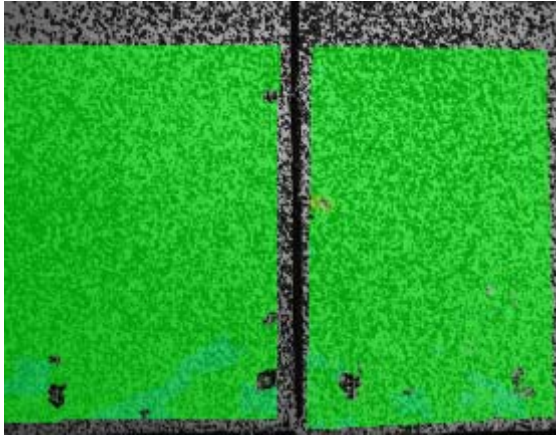
C. 30 kN



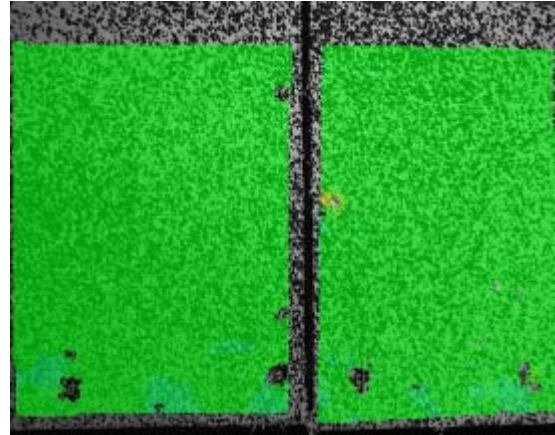
D. failure

Figure F15. Strain profile at the middle of the sill plate (Zone III) for wall 1

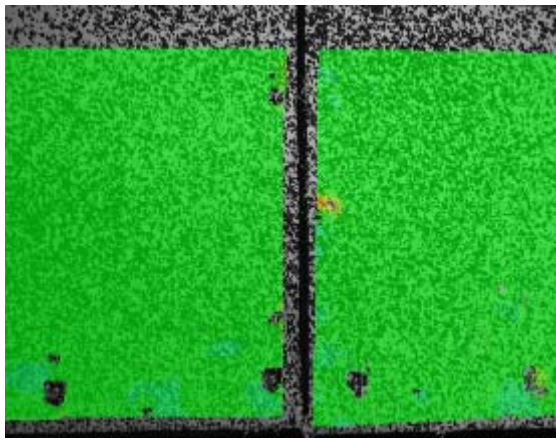
Wall 2 (OSB)



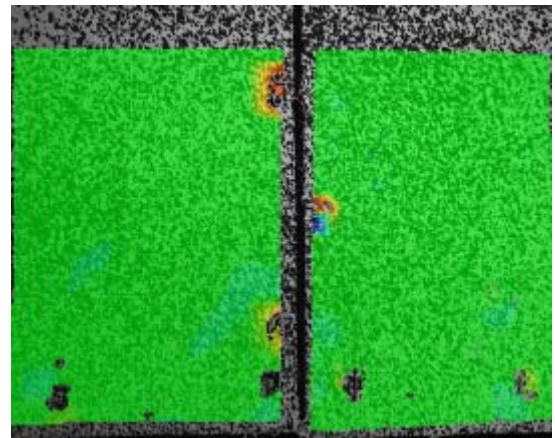
A. 10 kN



B. 20 kN



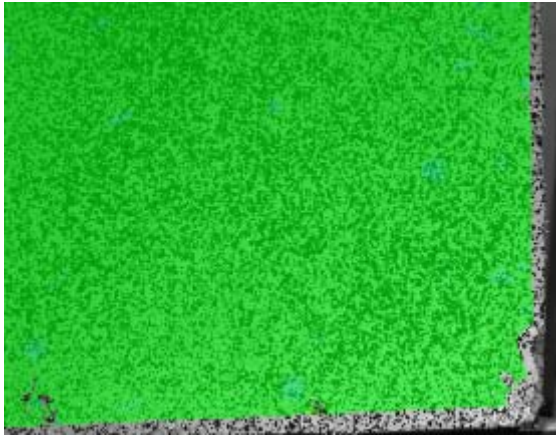
C. 30 kN



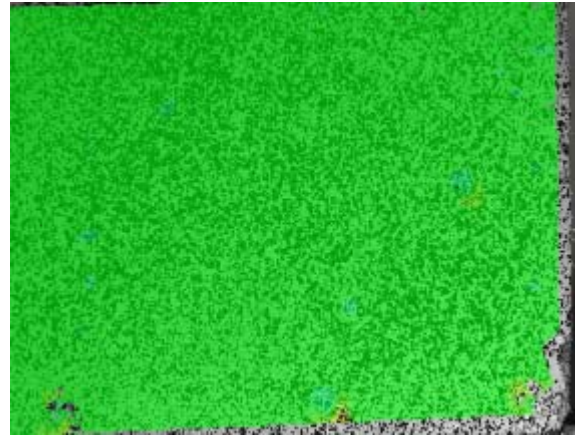
D. failure

Figure F16. Strain profile at the middle of the sill plate (Zone III) for wall 2

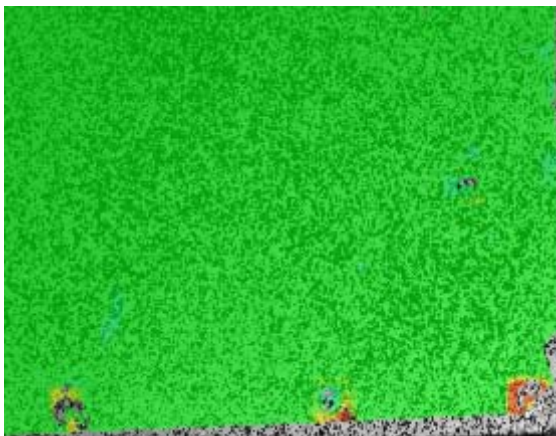
Wall 5 (OSB)



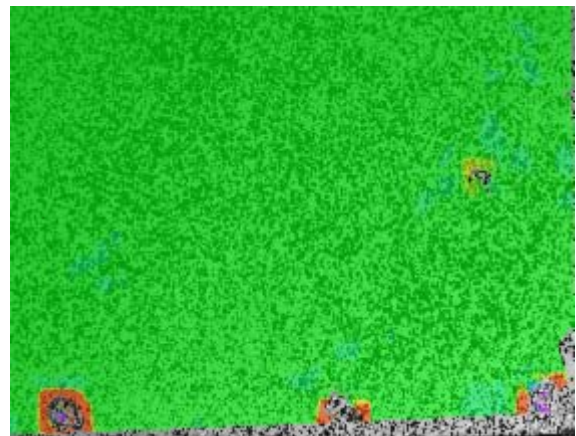
A. 10 kN



B. 20 kN



C. 30 kN

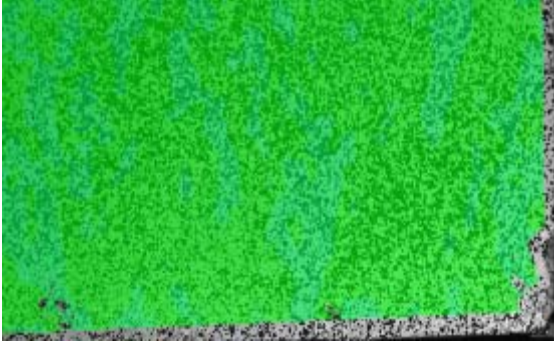


D. Failure

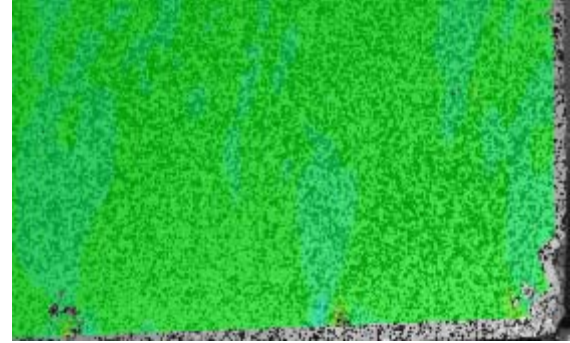
Figure F17. Strain profile at the uplift corner (Zone II) for wall 5

Wall 5 (OSB)

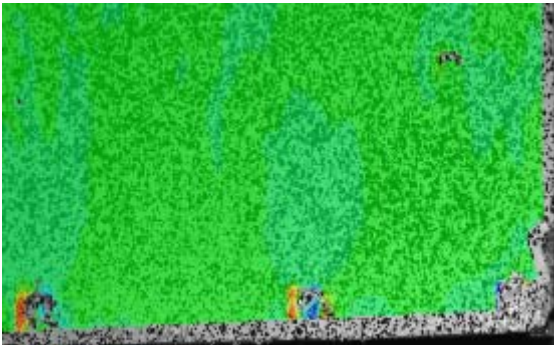
e_{xx}



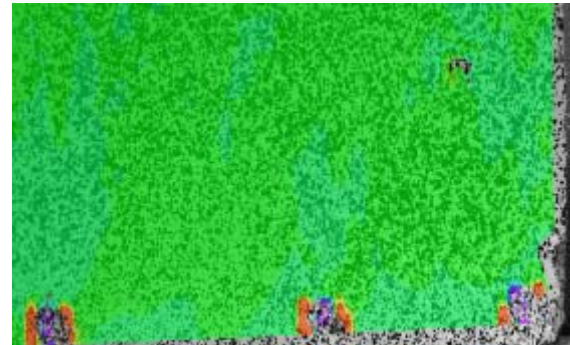
A. 10 kN



B. 20 kN

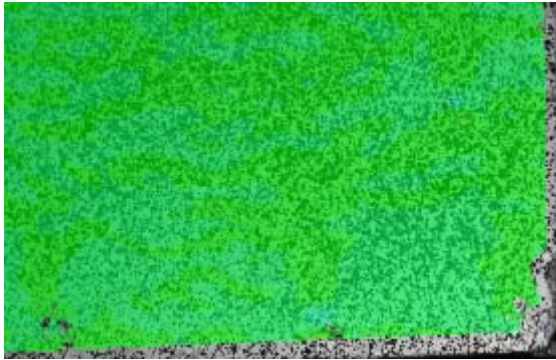


C. 30 kN

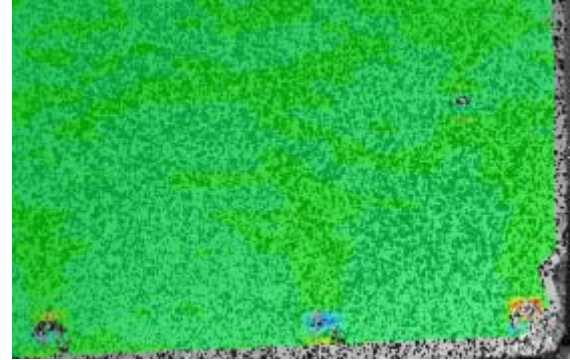


D. failure

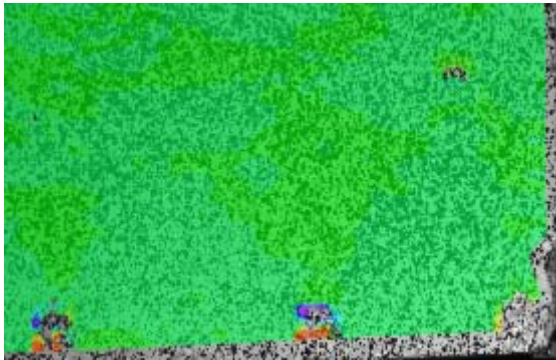
Figure F18. Strain profile (e_{xx}) at uplift corner (Zone II) for wall 5

e_{yy} 

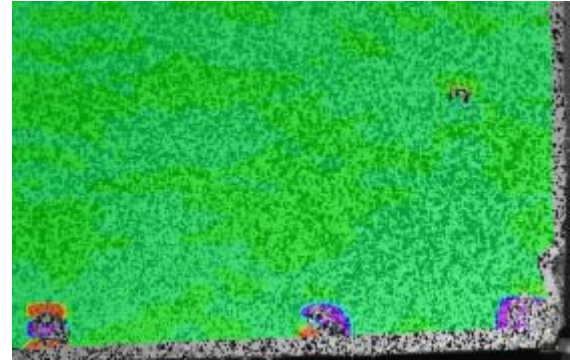
A. 10 kN



B. 20 kN

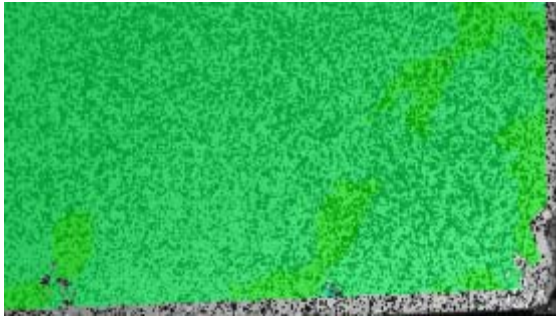


C. 30 kN

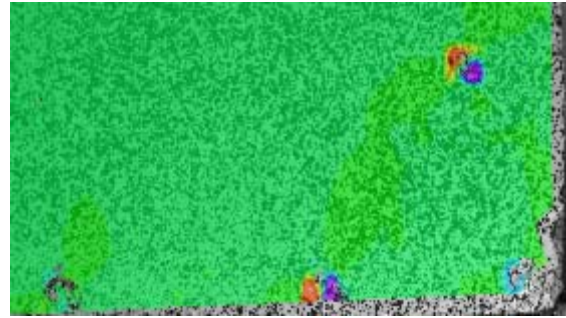


D. failure

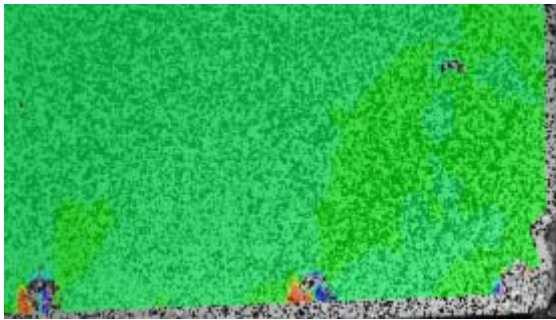
Figure F19. Strain profile (e_{yy}) at uplift corner (Zone II) for wall 5

e_{xy} 

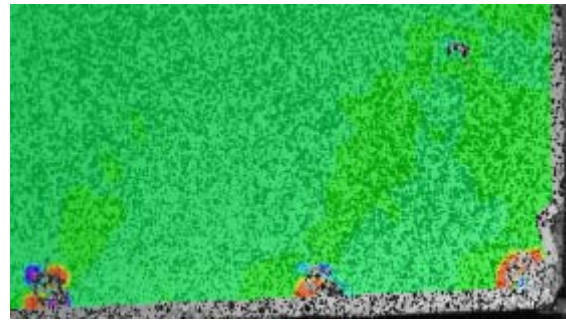
A. 10 kN



B. 20 kN



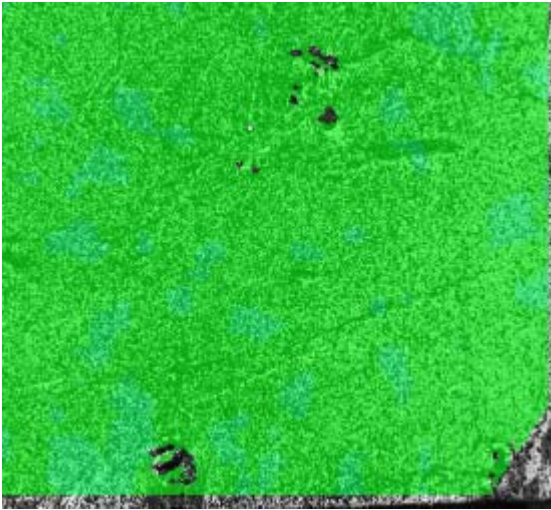
C. 30 kN



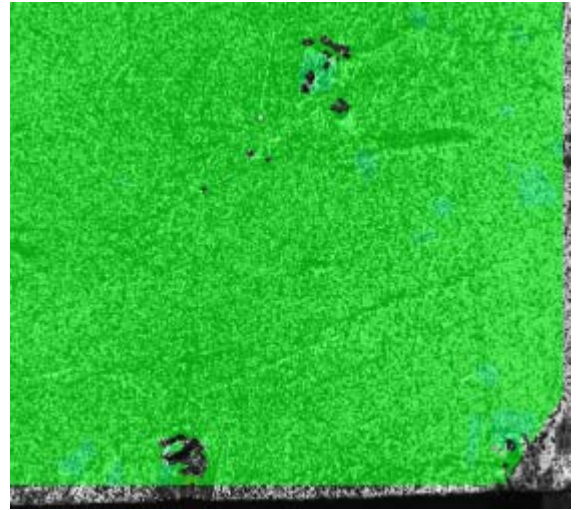
D. failure

Figure F20. Strain profile (e_{xy}) at uplift corner (Zone II) for wall 5

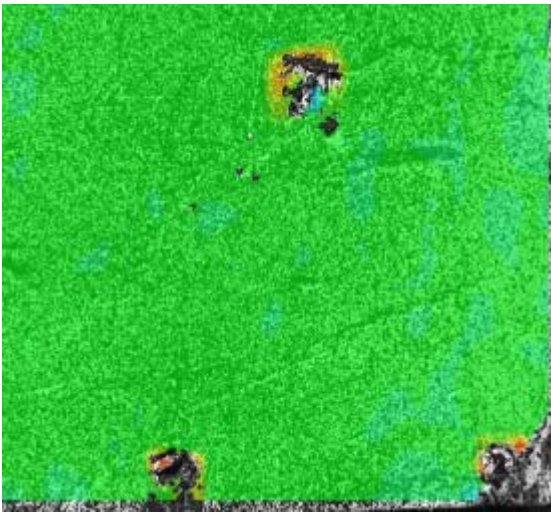
Wall 6 (OSB)



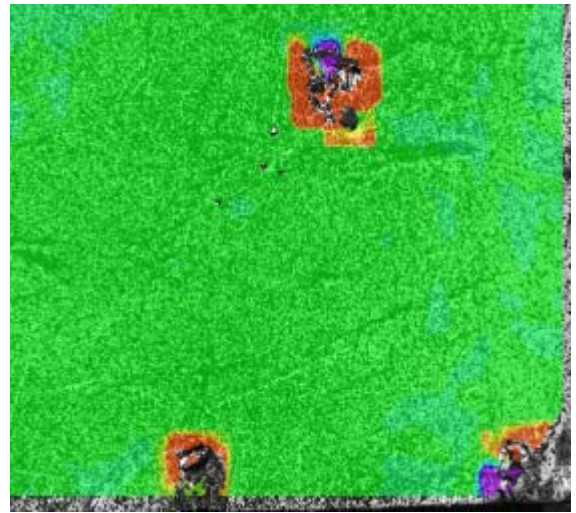
A. 10 kN



B. 20 kN



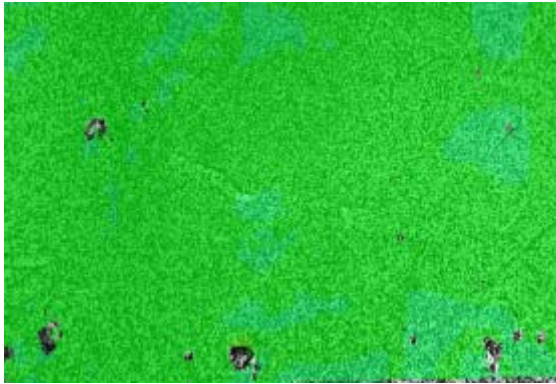
C. 30 kN



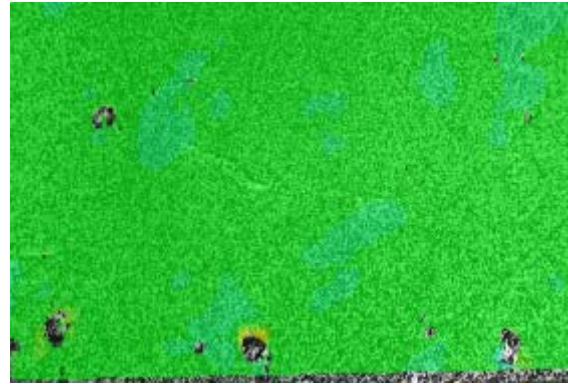
D. failure

Figure F21. Strain profile at the uplift corner (Zone II) for wall 6

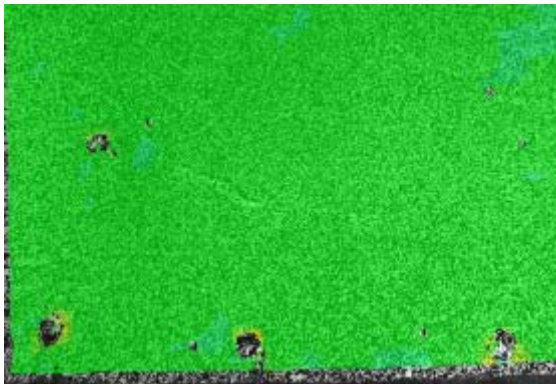
Wall 7 (OSB)



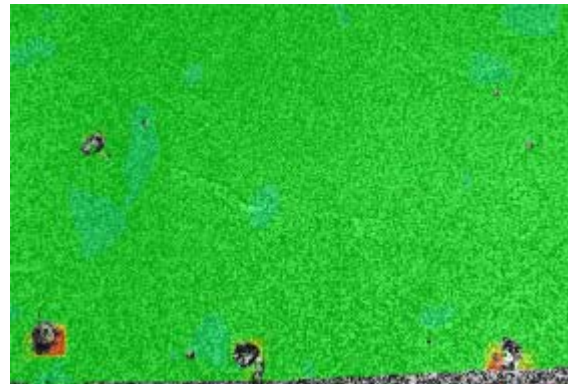
A. 10 kN



B. 20 kN



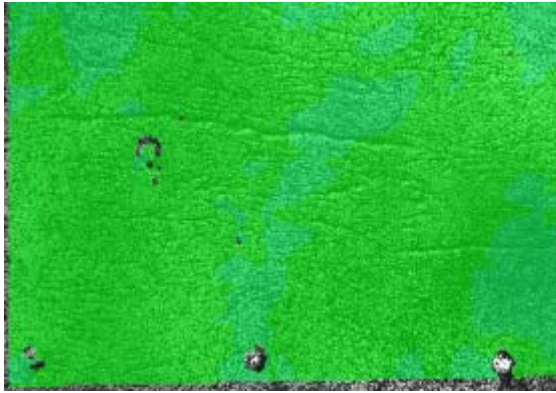
C. 30 kN



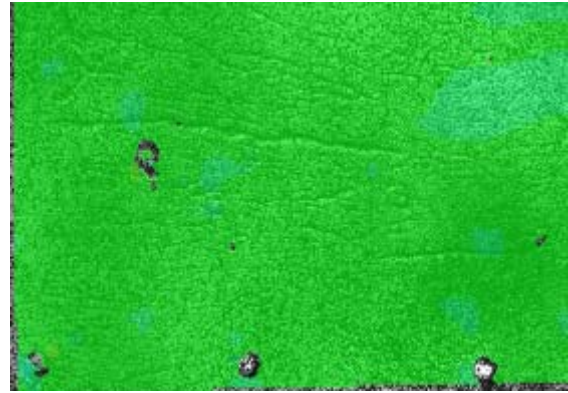
D. failure

Figure F22. Strain profile at the compression corner (Zone I) for wall 7

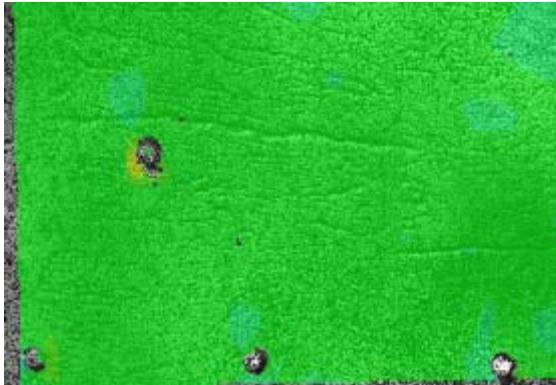
Wall 8 (OSB)



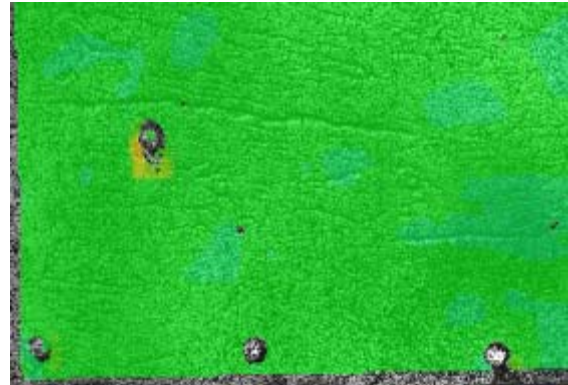
A. 10 kN



B. 20 kN



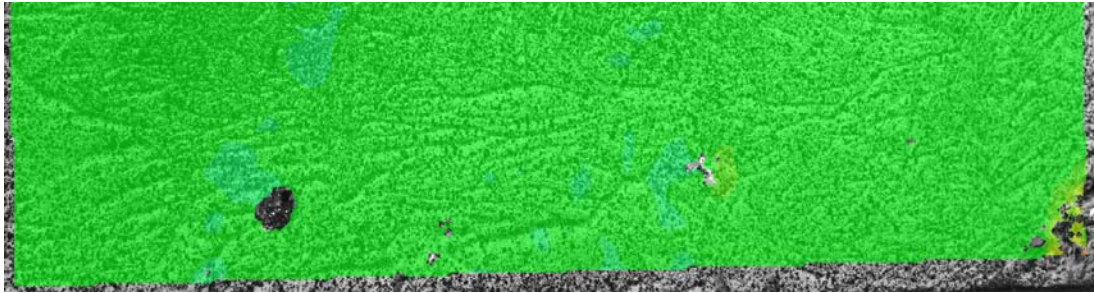
C. 30 kN



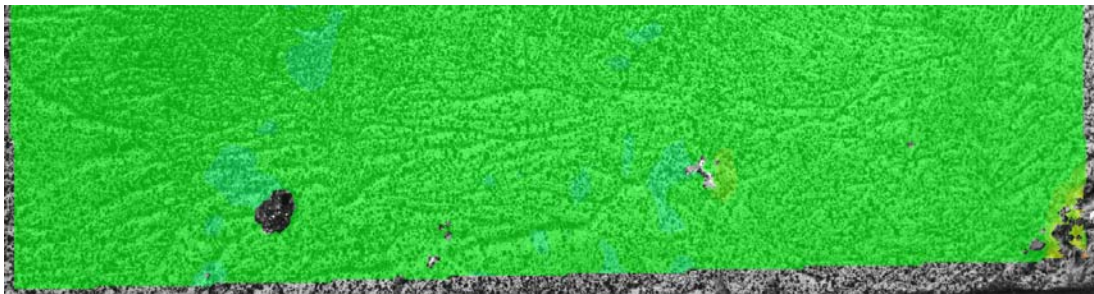
D. failure

Figure F23. Strain profile at the compression corner (Zone I) for wall 8

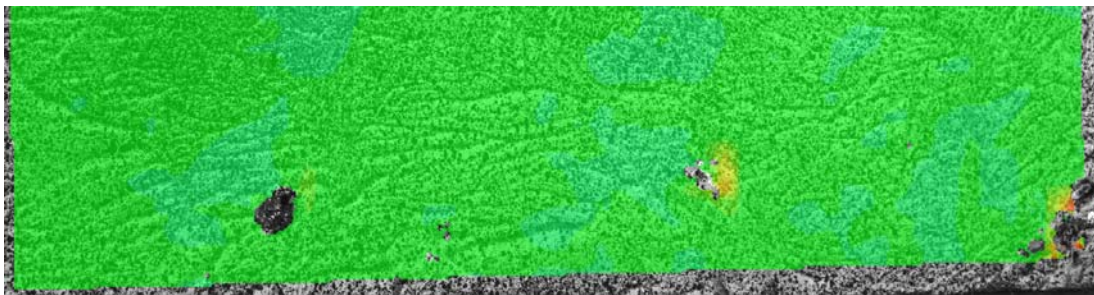
Wall 9 (OSB)



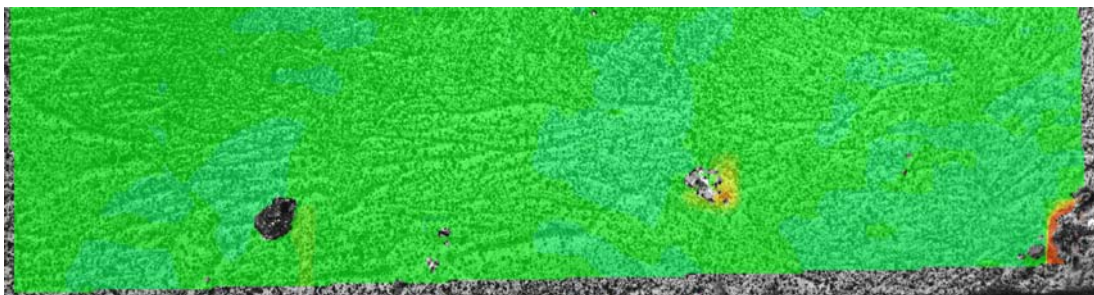
A. 10 kN



B. 20 kN



C. 30 kN



D. failure

Figure F24. Strain profile at intermediate stud near uplift corner (Zone VI) for wall 9

Wall 10 (OSB)



A. 10 kN



B. 20 kN



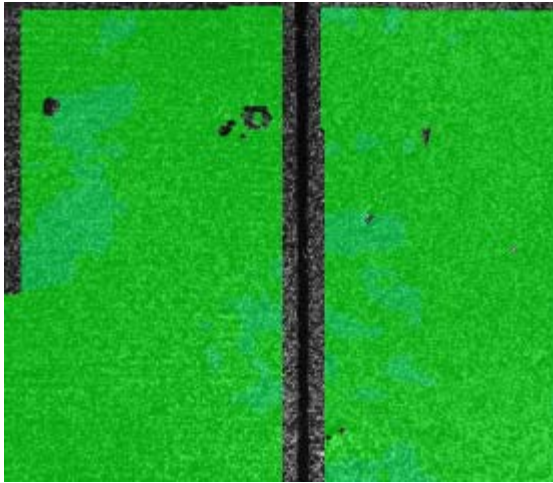
C. 30 kN



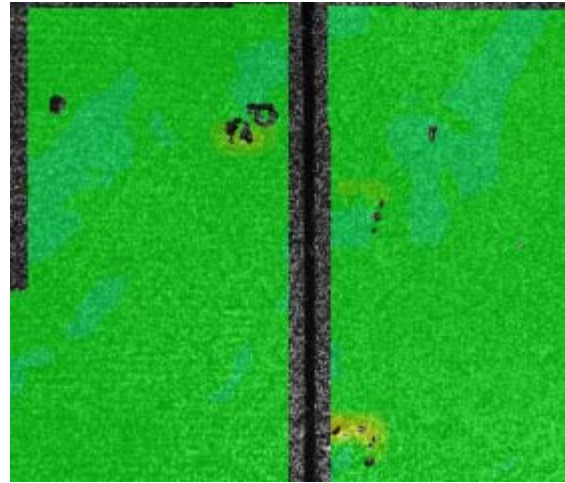
D. failure

Figure F25. Strain profile at intermediate stud near comp. corner (Zone VI) for wall 10

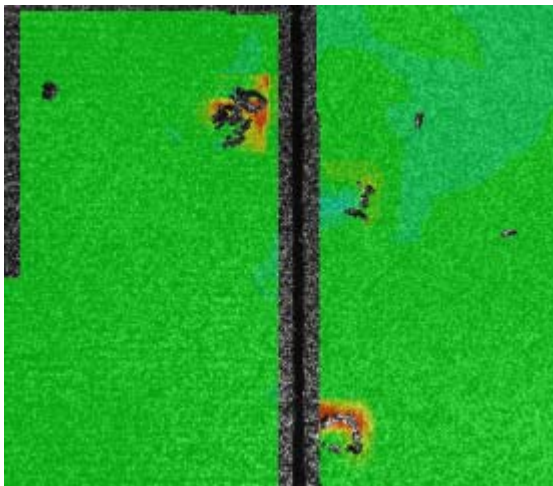
Wall 11 (OSB)



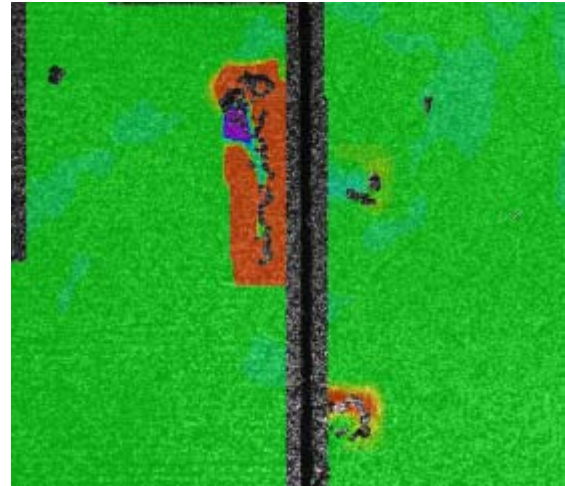
A 10 kN



B. 20 kN



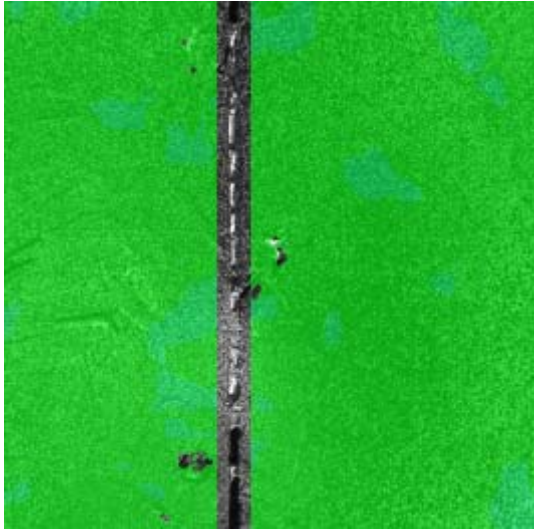
C. 30 kN



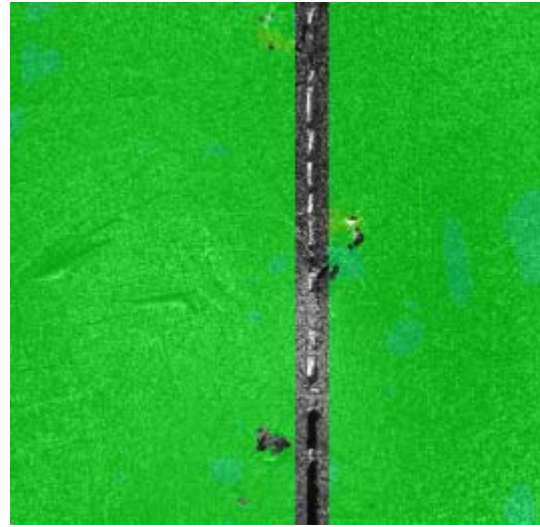
D. failure

Figure F26. Strain profile at middle of the wall (Zone IV) for wall 11

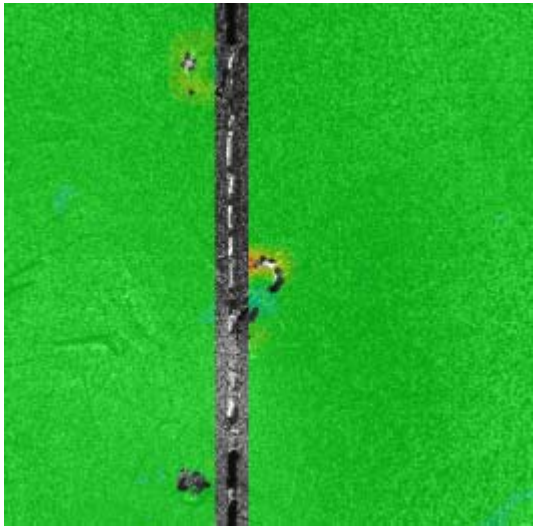
Wall 12 (OSB)



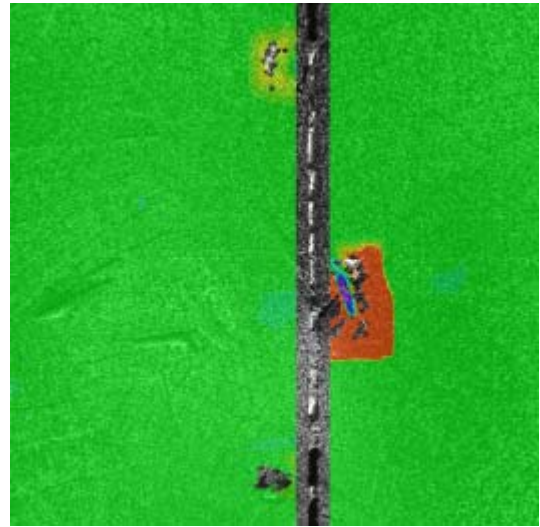
A. 10 kN



B. 20 kN



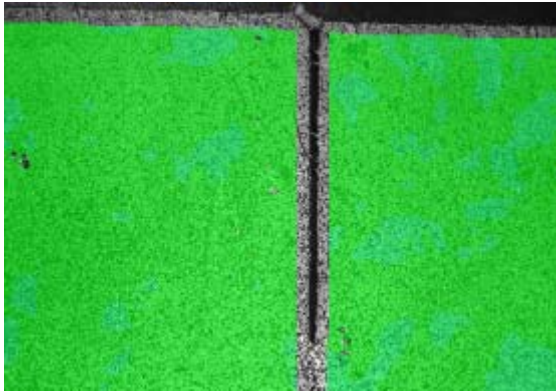
C. 30 kN



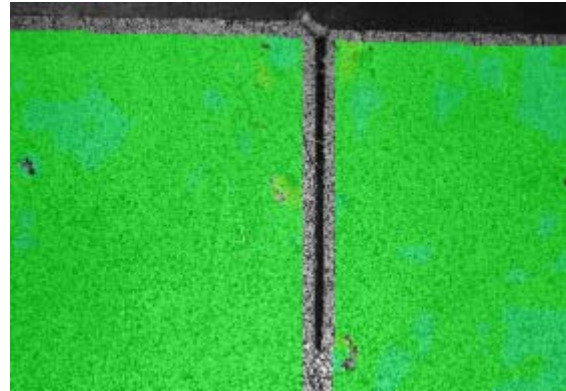
D. failure

Figure F27. Strain profile at middle of the wall (Zone IV) for wall 12

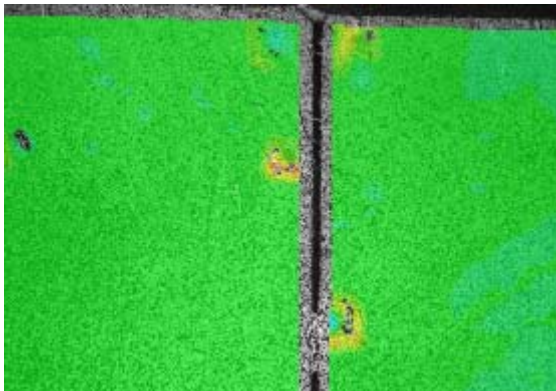
Wall 13 (OSB)



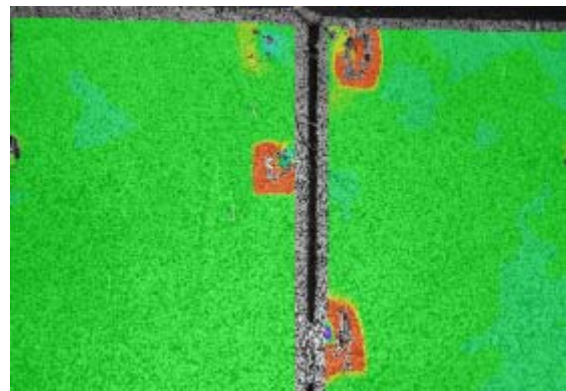
A. 10 kN



B. 20 kN



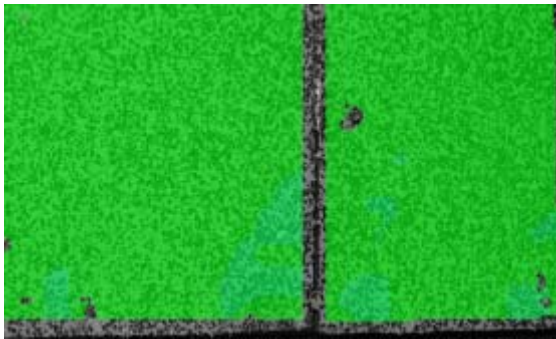
C. 30 kN



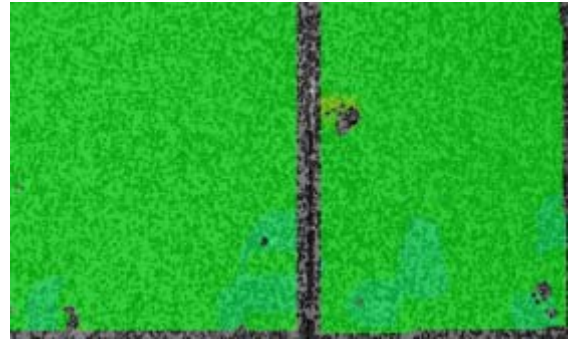
D. Failure

Figure F28. Strain profile at middle of the wall (Zone IV) for wall 13

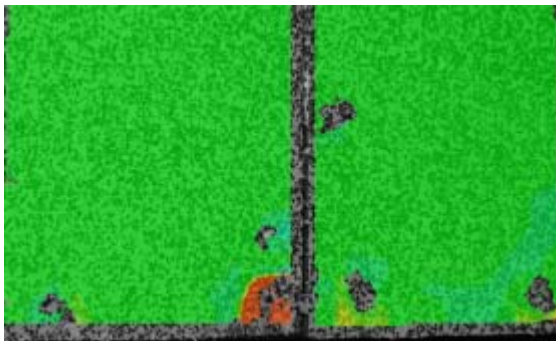
Wall 3 (OSB)



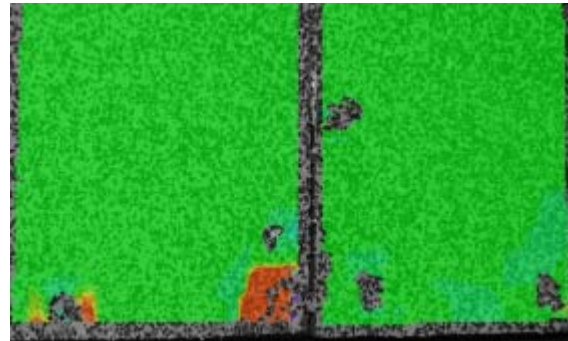
A. 10 kN



B. 20 kN



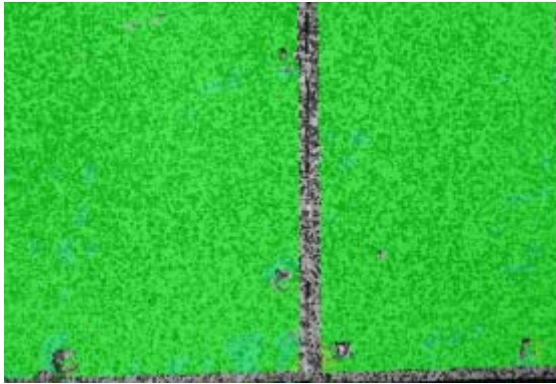
C. 30 kN



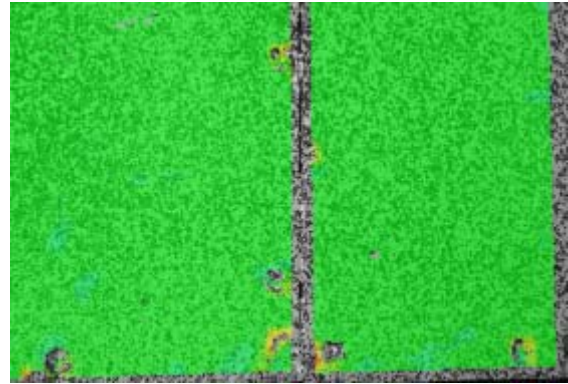
D. failure

Figure F29. Strain profile at middle of the sill plate (Zone II) for wall 3

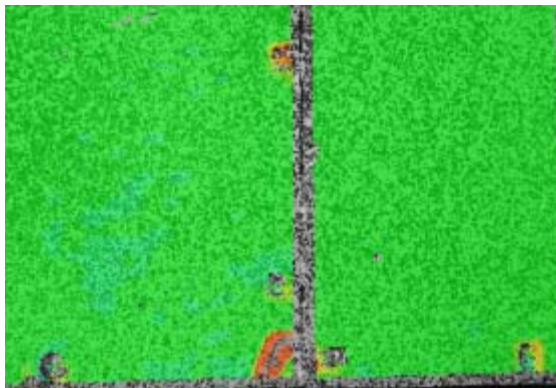
Wall 4 (OSB)



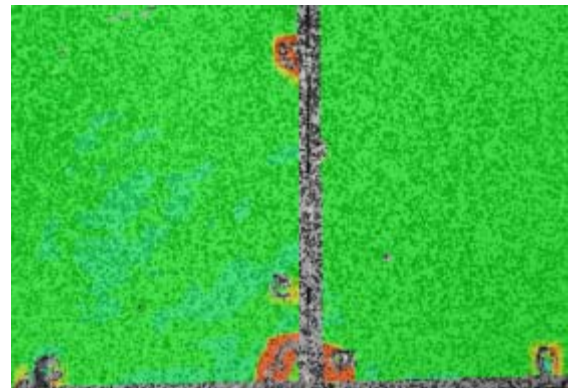
A. 10 kN



B. 20 kN



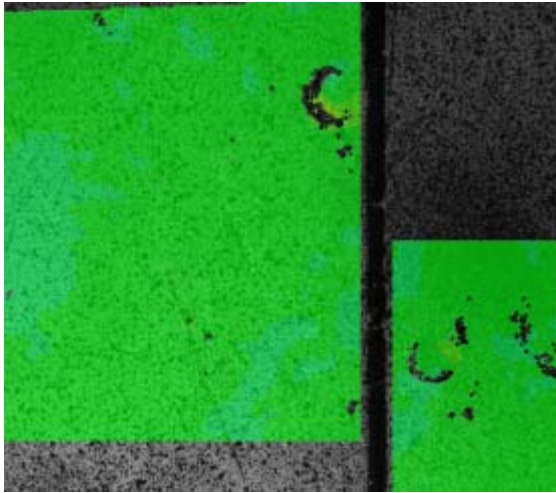
C. 30 kN



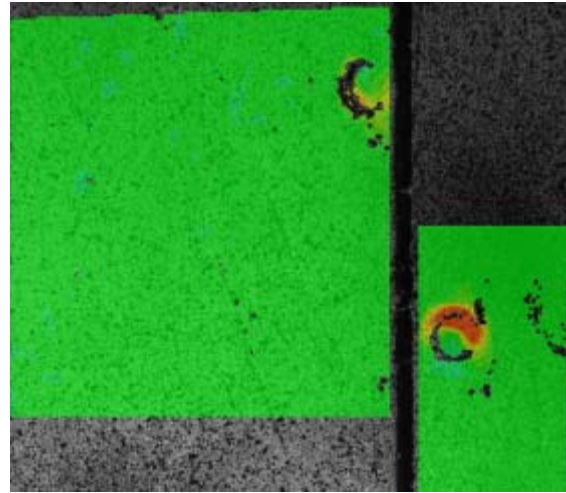
D. failure

Figure F30. Strain profile at middle of the sill plate (Zone II) for wall 4

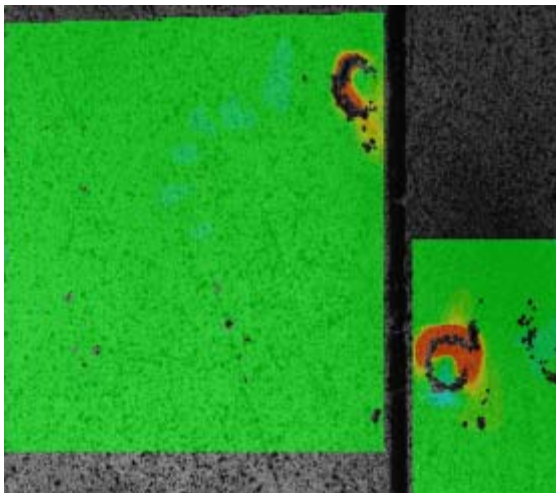
Wall 14 (OSB)



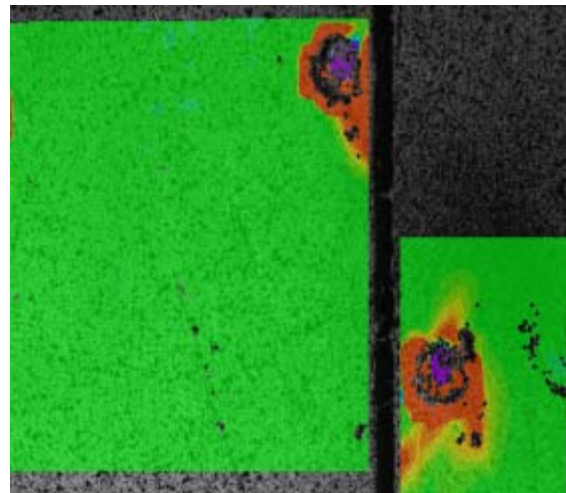
A. 10 kN



B. 20 kN



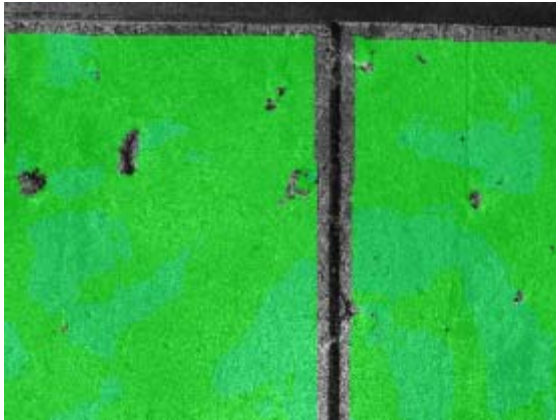
C. 30 kN



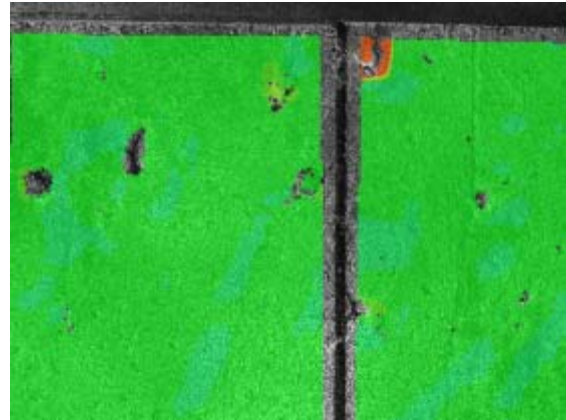
D. failure

Figure F31. Strain profile at middle of wall (Zone IV) for wall 14

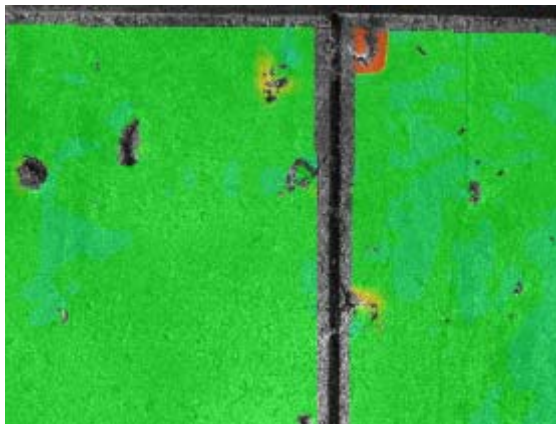
Wall 14 (OSB)



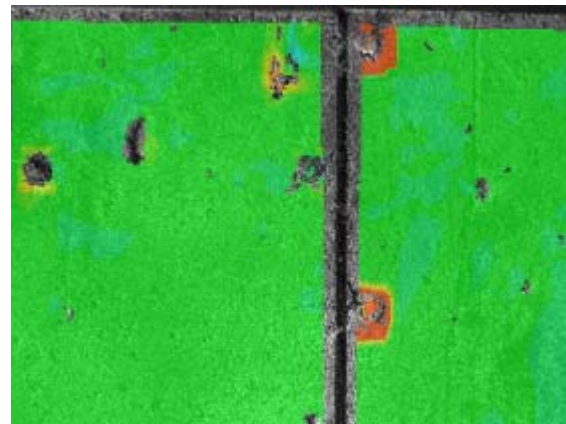
A. 10 kN



B. 20 kN



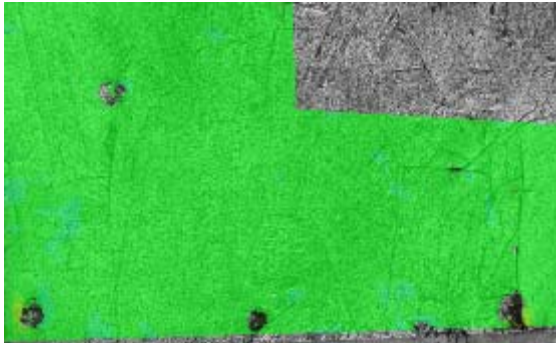
C. 30 kN



D. Failure

Figure F32. Strain profile at middle of top plate (Zone VI) for wall 14

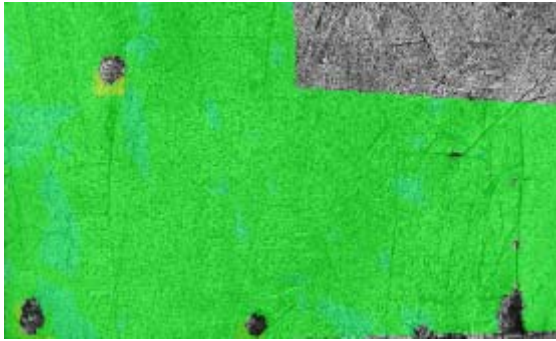
Wall 15 (OSB)



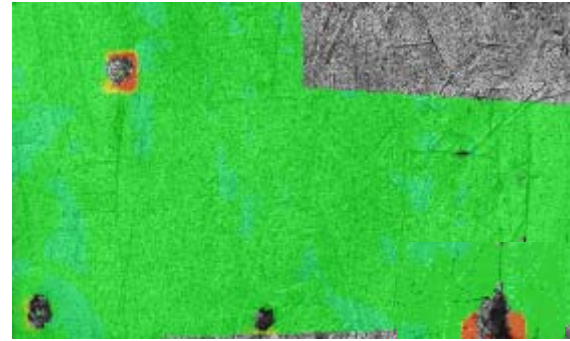
A. 10 kN



B. 20 kN



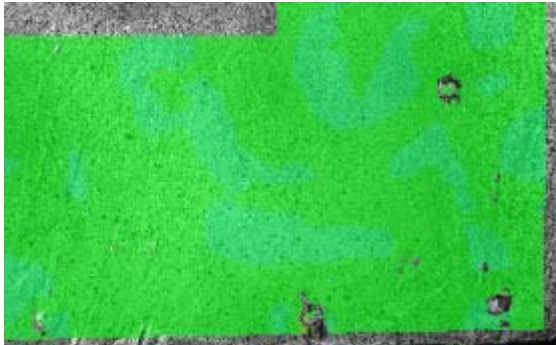
C. 30 kN



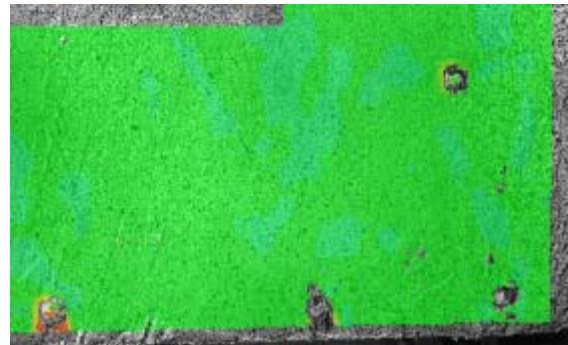
D. failure

Figure F33. Strain profile at uplift corner (Zone II) for wall 15

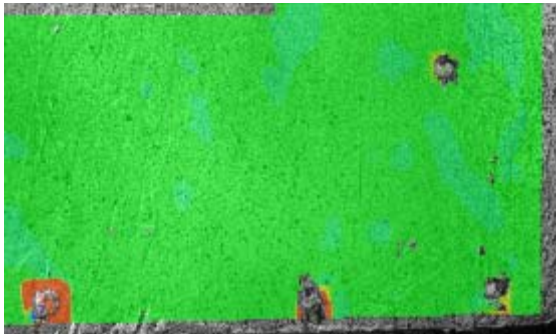
Wall 16 (OSB)



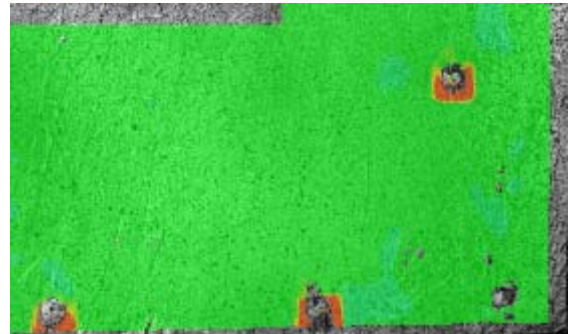
A. 10 kN



B. 20 kN



C. 30 kN



D. failure

Figure F34. Strain profile at compression corner (Zone I) for wall 16

Appendix G – Failure Map

Following figures are schematic showing the type of failure of each nail on OSB side. All the walls are loaded from right to left as shown in Figure G1. The abbreviation key is listed in Table G1. No GWB failure map was recorded.

Table G1. Abbreviation key for failure map

Symbol	Type of Failure
P	Pull Through
T	Edge tear out
W	Withdrawal
N	No failure or can not be ascertained



T	T	P	P	N	N	N	N	N	P	P	T	T	T	T	P	N	N	N	N	N	P	P	P
T											T	T										P	P
P											T	T										P	P
P					N						T	T				N						P	P
P											T	T										P	P
P					N						T	T				N						N	N
N											T	T										N	N
N					N						T	T										N	N
N											T	T										P	P
N					N						T	T				N						P	P
N											T	T										P	W
N					N						T	T										W	W
N											T	T				P						W	W
N					N						T	T										W	W
N											T	T				P						T	T
P											T	T										T	T
P					P						T	T										T	T
P											T	T										T	T
P	P	P	N	N	T	N	N	N	N	T	T	T	T	T	P	P	P	P	W	W	W	T	T

Figure G1. Wall 1 failure map

T	T	P	P	N	N	N	N	N	P	P	T	P	P	P	N	N	N	N	N	N	N	N	N	N	N	N	N
T													N	N													N
P													N	N													N
P					N								N	N				N									N
P													N	N													N
P					N								N	N				N									N
N													N	N													N
N					N								N	N				N									N
N													N	N													N
N					N								N	N				N									N
N													N	N													N
N					N								N	N				N									N
N													N	N													N
N					N								N	N				N									N
N													N	N													N
N					N								N	N				N									N
N													T	T													T
N					N								T	T				N									T
P													T	T													T
P													T	T													T
P	P	P	N	N	N	N	N	N	N	N	T	T	T	T	P	P	N	N	N	N	N	N	T	T	T	T	T

Figure G2. Wall 2 failure map

P	P	P	N	N	N	N	N	N	N	P	P	T	T	T	T	N	N	N	N	N	P	P	P	P	P	P	P
P													T	T													P
P													P	P				N									P
P					N								P	P													P
P													P	P													P
P					N								P	P				N									P
P													P	P													P
P					N								P	P				N									N
N													P	P													N
N					N								P	P				N									N
N													T	T													N
N													T	T				N									T
N					N								T	T													T
N													T	T													T
N													T	T				N									T
P					N								T	T													T
P													T	T				N									T
P					N								T	T													T
P													T	T													T
P													T	T													T
T	T	P	P	P	N	N	N	T	T	T	T	T	T	T	T	T	T	T	T	T	T	T	T	T	T	T	T

Figure G3. Wall 3 failure map

P	P	P	N	N	N	N	N	N	N	T	T	T	T	T	N	N	N	N	N	N	T	T	T
P												T	T	T									T
N												T	T	T									P
N						N						T	T	T			N						P
N												T	T	T									P
N						N						P	P	P			N						P
N												P	P	P									P
N						N						P	P	P			N						P
N												P	P	P									P
N						N						T	P	P			N						P
N												T	P	P									P
N						N						T	P	P			P						P
N												T	P	P									P
N						N						T	P	P			P						P
N												T	P	P									P
N						N						T	P	P			P						P
N												T	P	P									P
N						P						T	T	T			P						P
P												T	T	T									P
P												T	T	T									T
P	P	P	P	P	P	P	P	P	P	T	T	T	T	T	T	T	T	T	T	T	T	T	T

Figure G6. Wall 6 failure map

T	T	T	P	N	N	N	N	N	N	N	P	P	P	P	N	N	N	N	N	N	N	P	P	P	P
T												P	P	P										P	
T												P	P	P										P	
P					N							P	P	P			N							P	
P												P	P	P										P	
P												P	P	P										N	
N					N							T	T	T			N							N	
N												T	T	T										N	
N												T	T	T										N	
N					N							T	T	T			N							N	
N												T	T	T										N	
N												T	T	T										N	
N												T	T	T										N	
N					N							T	T	T			N							N	
N												T	T	T										N	
N												T	T	T										N	
N												T	T	T										N	
N					N							T	T	T										N	
N												T	T	T										N	
N												T	T	T										N	
N												T	T	T										N	
N												T	T	T										N	
N												T	T	T										N	
N												T	T	T										N	
N												T	T	T										N	
N												T	T	T										N	
N												T	T	T										N	
N												T	T	T										N	
N												T	T	T										N	
N												T	T	T										N	
N												T	T	T										N	
N												T	T	T										N	
N												T	T	T										N	
N												T	T	T										N	
N												T	T	T										N	
N												T	T	T										N	
N												T	T	T										N	
N												T	T	T										N	
N												T	T	T										N	
N												T	T	T										N	
N												T	T	T										N	
N												T	T	T										N	
N												T	T	T										N	
N												T	T	T										N	
N												T	T	T										N	
N												T	T	T										N	
N												T	T	T										N	
N												T	T	T										N	
N												T	T	T										N	
N												T	T	T										N	
N												T	T	T										N	
N						</																			

Figure G7. Wall 7 failure map

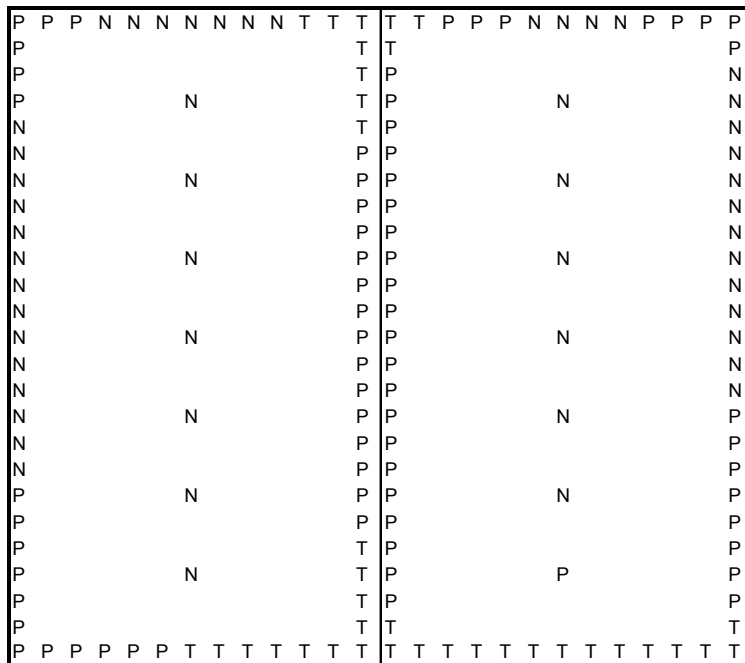


Figure G8. Wall 8 failure map

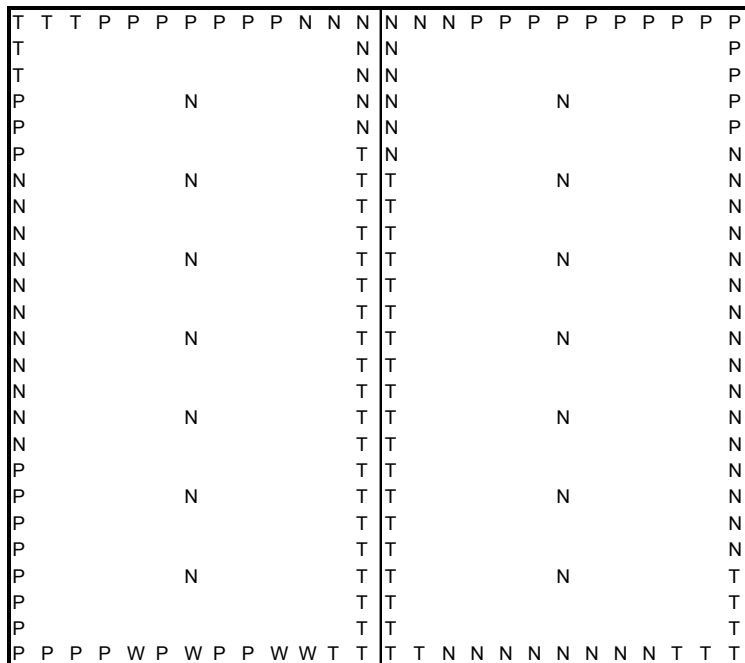


Figure G9. Wall 9 failure map

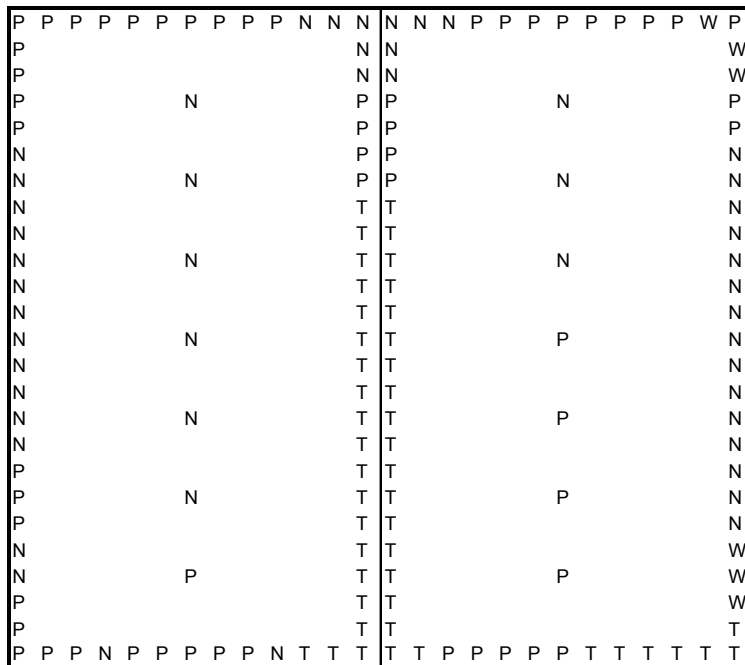


Figure G10. Wall 10 failure map

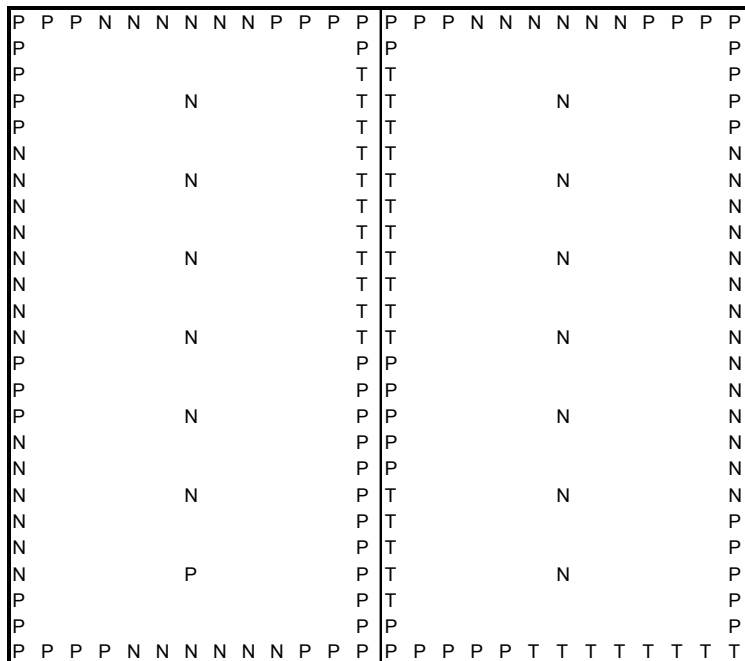


Figure G11. Wall 11 failure map

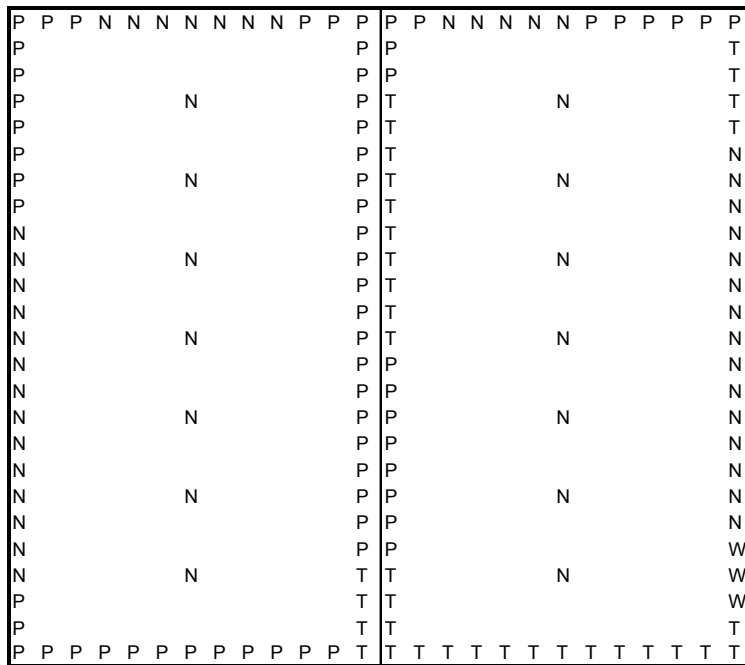


Figure G12. Wall 12 failure map

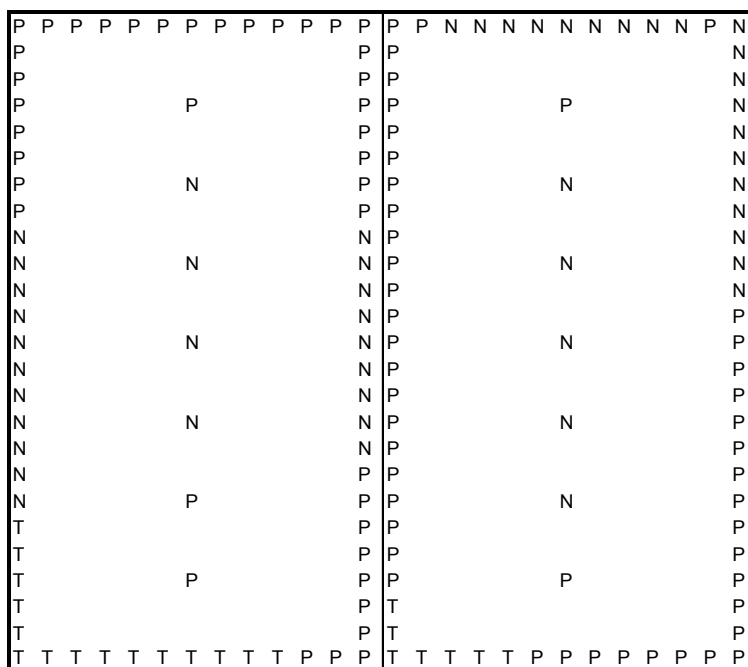


Figure G13. Wall 13 failure map

T	T	T	P	P	P	P	P	P	T	T	T	T	T	T	N	N	N	N	P	P	P	P	P	P	P	P
T															T											P
P															T											P
P															T					N						P
P															T											P
P															T					N						N
N															T											N
N															T					N						N
N															T											N
N															T											N
N															T					P						N
N															T											N
N															T					P						N
N															T											N
N															T											N
N															T					P						N
N															T											N
N															T											T
N															T					P						T
P															T											T
P															T											T
P	P	P	P	P	P	P	P	P	P	T	T	T	T	T	T	T	P	P	P	P	P	T	T	T	T	T

Figure G14. Wall 14 failure map

T	T	T	P	P	P	P	P	P	P	T	T	T	T	T	T	N	N	N	N	P	P	P	P	P	T	T	T
T																T											T
P																T					N						T
P																T					T						P
P																T											P
P																T					N						P
P																T											P
P																T					N						P
P																T											P
N																T											P
N																T					N						P
N																T											P
N																T											P
N																T					N						P
N																T											P
N																T											P
N																T											P
N																T					P						P
N																T											P
N																T											P
N																T					P						T
N																T											T
N																T					P						T
N																T											T
P																T					P						T
P																T					P						T
P																T					P						T
P	P	P	P	P	P	P	P	P	P	P	P	P	P	P	P	P	P	P	P	P	T	T	T	T	T	T	T

Figure G15. Wall 15 failure map

T	T	T	T	T	T	T	T	T	T	T	T	T	P	P	P	P	P	P	P	P	P	P	P
T												T	T									P	P
P												T	P									P	P
P			N									T	P			N						P	P
P												T	P									P	P
P												T	P									P	P
P			N									T	T			N						P	P
P												T	T									P	P
N												T	T									P	P
N			N									T	T			N						P	P
N												T	T									P	P
N												T	T									P	P
N			P									T	T			P						P	P
N												T	T									P	P
N												T	T									P	P
N			P									T	T			P						P	P
N												T	T									P	P
N												T	T									P	P
N			P									T	T			P						T	T
N												T	T									T	T
N												T	T			P						T	T
P												T	T									T	T
P												T	T									T	T
P	P	P	P	T	T	T	T	T	T	T	T	T	T	T	T	T	T	T	T	T	T	T	T

Figure G16. Wall 16 failure map

Appendix I – Detailed literature review

As wood shear wall is the major lateral force resisting system in majority of the buildings, it has been the subject of various studies and research. The relatively high allowable strength and the ease with which the panels are installed make wood structural panels as economical choice for shear walls (Breyer et al. 1999). Wood shear walls typically resist in plane lateral forces induced by seismic event or wind, and they resist the vertical loads and transverse wind loads as well (Lam et al. 1997).

The prior testing done were mainly using monotonic and sequential phased displacement (SPD) test protocols. These methods were sufficient for most regions, but in seismic prone areas, the behavior under actual earthquake loading needed to be studied. Lately cyclic protocols are being used to simulate earthquake loading. Prior tests have considered many different aspects of shear walls, such as strength and stiffness contribution of various sheathing material on the shear walls (Toothman, 2003, CUREE 1999).

Monotonic loading was the only method of testing shear walls for many years because it provided a good indication of the behavior under unidirectional load, but it was more analogous to wind load rather than earthquake load. Many studies and research has evaluated the behavior of shear wall under monotonic loading. Wolfe (1983) tested 13 different types of wall, 30 walls in total, under Monotonic loading using ASTM E 564 standard. Tests were conducted with varying wall lengths, GWB orientation and wind bracings. All GWB panels were 0.5in thick and no other sheathing material was used. All walls were made up of construction grade spruce- pine- fir (SPF) 2 by 4 studs spaced 24in on center, end nailed to single top and bottom plates by using two 16d common nails at each connection. The control walls were 8'X 8' wood frame with GWB diaphragm. Two 4'X8' GWB panels were applied parallel to height of the wall. Three different wall lengths (8', 16', and 24') were used. 22 walls had .5 in GWB attached to one side only and remaining 8 had only diagonal bracing, but no wall board sheathing. After testing the author concluded that taped wallboard joints, load displacement characteristic and wall failure mechanism was independent of type and nature of construction. Taped joints were found to be competent in transferring load as there was no significant weakness along the wallboard joint. The one failure mechanism consistent with all types of wall was the nail tearing and bending through paper surface. GWB had fastener failure at the bottom plate. All walls displayed increase in stiffness for the second load application. There was less tendency of rotation amongst the GWB panels. The effect of GWB panel orientation showed a 50% increase in strength for 24' long walls.

Wolfe concluded that GWB has a significant contribution in to racking resistance of the wall, which varies with panel orientation and wall length. Racking resistance of the walls with GWB and Wind bracing were found to be the linear sum of resistance provided by each element. Horizontal orientation of panels was found to provide 40% greater strength and stiffness than the vertical oriented panels. The relationship between wall length ultimate shear strength was approximately linear, but wall stiffness was found to be a power function of length.

Johnson (1997) performed monotonic testing on different type of walls with different opening ratios. On one side sheathing was of plywood and other side GWB. Tie down anchors were used with all the walls. The apparatus set up calculated the drift at every load. Observation about failure of GWB revealed that as drift increases cracking of tape joints around the opening starts. At large displacements nail starts to pull out on the edges and tear the edges. Field nails also encounter some pull and lateral displacement. Johnson (1997) concluded that GWB helps resist shear in the low to moderate loading, but plywood resists most of the shear near capacity under monotonic loading.

Toothman (2003) tested shear walls under monotonic loading, with structural sheathing on one side and GWB on other and concluded that the contribution of Gypsum is not additive. Based on monotonic testing, OSB panels were the strongest material based on ultimate strength, able to resist 11.16 kN. Hardboard had an average strength of 9.26 kN and fiberboard's strength was 6.75 kN. Gypsum was weakest material with an average max strength of 4.45 kN. The failure mode of the walls typically involved the sheathing nails either pulling out of the framing or tearing through the sheathing along the bottom plate. Most of the walls showed this failure mode but some walls under monotonic testing reached the peak load and then maintained a load after failure through substantial displacement. No inference on this behavior is drawn. The general mode of failure for OSB walls was sheathing nails pulling out of framing and tearing through the sheathing along the bottom plate, which resulted in separation of end stud from top plate. For GWB sheathed walls the failure of nails started along the bottom plate and continued around the perimeter of the wall. The ductility of the walls sheathed on one side with GWB and on other side material such as OSB or plywood, increased by a substantial amount as compared to the sheathing material alone. When failure pattern was observed for walls sheathed on both sides, gypsum panels were always first to fail. This is because of the relative ease with which nail could tear the sheathing. Toothman concluded that by adding gypsum panel in the structure there is an

increase in overall strength, elastic stiffness and energy dissipation before failure of the structure also GWB provides a substantial amount of shear resistance.

Monotonic tests are unidirectional, but cyclic tests composed of fully reversible load cycles. The choice of test protocol also affects the test results (Uang and Gatto, 2003). They tested walls with different structural sheathing (OSB, plywood, gypsum) using different protocols namely Monotonic, CUREE, ISO and Sequential Phased displacement protocols and quantified the differences in parameters such as peak strength, initial stiffness etc. They also analyzed the effect of gypsum wall board on peak strength, initial stiffness, absorbed energy and deformation capacity. 12% increase in shear wall strength and 31% decrease in shear wall deformation capacity was observed. As GWB is stiff so it was expected that it increased the initial stiffness by 60%. One kip average strength increase corresponded well with one kip peak strength of GWB which was one kip justified the superposition of GWB and the structural sheathing as reasonable. But Toothman 2003, found similar results but he concluded that the principle of superposition is not valid. The results were not additive.

Study of failure pattern of GWB + OSB revealed that failure occurred due to nail tear out at top of the OSB panels, also top plate separation occurred and led to sheathing separation which led to failure when subjected to CUREE loading protocol and pull through of the screws when subjected to monotonic loading.

Karacabeyli and Ceccotti (1996) tested contribution of GWB on shear wall capacity of 8x16 feet walls. The tests concluded that GWB on one side and OSB on other increased the peak load but decreased ductility when compared to only OSB as sheathing. Also observed was that till small deflection of about 1" the law of superposition is valid to determine the lateral resistance but after that the relationship becomes complex.

McMullin and Merrick, (2001) tested a total of seventeen specimens constructed of different configuration. All walls had one 2 ft 10 in. wide by 6 ft 10-1/2 in. tall rough opening in the same location for all walls. Wall variables included fastener type and spacing, edge fastening, top plate restraint, addition of a 3 ft by 4 ft rough window opening, wallboard panel orientation, various repair methodologies, innovative construction techniques, and the addition of a door frame, door trim and baseboard. To simulate ceiling and corner returns, additional wooden members were added to accommodate this condition. All walls were sheathed on both sides with Gypsum wall board of 1/2 in thickness. The 4ft by 8 ft panels were attached using varying fasteners and fastener spacing scenarios. The gypsum wallboard was

installed with the long dimension oriented horizontally, and varying the location of butt joints with respect to wall openings.

Three different loading protocols were used two Monotonic and one Cyclic (CUREE) Protocols. For the first monotonic protocol, the wall was loaded to a maximum drift of 4%. At this point, the loading was reversed to a peak drift of 4% in the opposite direction. This 4% limit on the frame was chosen to represent a level of distortion that would be beyond repair and likely in a collapse state. After testing it was observed that very limited additional damage occurred when the drift exceeds 3%. In addition the maximum drift was set as a precaution of damaging the wood framing excessively. This allowed for the majority of wood framing to be used for several tests. The second monotonic loading protocol was developed during discussion of the design of the test program. Except for a few unloading and reloading steps, the protocol was identical to the original monotonic protocol. The cyclic protocol was standard CUREE Caltech Protocol.

Various damages were observed during the testing. Hairline cracks starting at the opening corners were the most prevalent form of initial damage. Cracking of the finish over the fastener head, cracking of wallboard joints, and the crushing of wallboard at wall boundaries all occurred at larger sustained drifts. Global buckling of large portions of the panels and the loss of portions or even whole panel sections was noted at large displacement levels. Two distinct failure modes were observed during the testing. The first failure mode involved the fasteners pulling out of the wallboard along the wall perimeter allowing displacement of the wallboard relative to the framing. It appeared that the upper half of the framing remained essentially vertical, and all the lateral movement occurred by bending of the studs in the lower half of the wall. For this mode, all wallboard joints remained in good condition and free of cracking. The second failure mode consisted of wallboard joint failure, allowing relative rotation of individual wallboard panels. Cracking at wall panel openings commonly occurred at drifts of close to 0.25%, with the cracks widening and lengthening at larger displacement levels. Wall fastener popping was also noticed at wall drift levels of 0.25%-0.75%, usually initiating at wall boundaries, particularly at the bottom plate. Maximum wall strength was achieved around 1% drift on average. All walls, independent of fastener type and spacing, had comparable initial stiffness, however the walls having tighter fastener spacing were observed to have less deformation capacity once the peak strength was developed.

Probably the most important parameter having an effect on the ultimate wall strength was the vertical flexibility in the middle of the wall. The walls in which no anchors were installed to resist the vertical movement of the middle portions of the wall pier exhibited lower ultimate

strengths than the restrained cases. The restrained walls developed 310 lbs/ft resistance for gypsum wallboard with screws at 16 in. on center while the identical walls which allowed vertical movement at the middle portion of the wall pier developed only 194 lbs/ft lateral resistance.

The addition of a window opening did reduce the ultimate strength of the wall.

A common assumption that the ultimate strength of the walls is a linear function of the individual wall pier segments is somehow inaccurate but it may not be unreasonable. The innovative systems implemented for some of the tests showed that they indeed reduce crack lengths and widths at equivalent drift levels for the identical walls.

McMullin and Merrick concluded that both screws and nails achieve acceptable performance levels for gypsum-sheathed walls. The increased density of wall screws significantly influenced wall strength, however resulted in less deformation capacity after ultimate load results. Monotonic loading reasonably predicts the cyclic behavior of gypsum-sheathed walls and damage states are also comparable, although the monotonic loading seems to place an upper bound on attainable ultimate drift. The ability of the wall to move vertically in the middle did show significant influence on both the strength of the wall and the damage developed. The ability of the pier to “roll” as opposed to “rack” appeared to have more effect on the behavior of the walls than any other parameter studied, except the addition of more wall openings. Improvement in the performance of the wallboard was obtained by making alterations to the installation. Using wallboard of tougher material, fasteners with larger heads, and reinforcing the re-entrant corners of openings all appeared to improve the performance of the walls. Minimal repair methods tested for this project resulted in walls that resisted between 0.803 to 1.235 times the load of the walls before the original damage.

Toothman, 2003, on Cyclic testing (indigenous protocol based on SPD) found hardboard to resist more load than the others with Gypsum again being the weakest of OSB, hardboard and fiberboard with an average strength of 3.7 KN. Different failure pattern were observed for different sheathing materials. In OSB nails typically pulled out of the framing or tore through the sheathing on the bottom plate. In some cases nails also pulled out along the top plate. Nail pullout allowed the end stud to separate from the top plate which forced the wall to fail. But in hardboard, the nails pulled out of frame but did not tear the sheathing and in fiberboard nails tore the whole sheathing along the perimeter of the panel. In case of GWB the nails completely failed and tore off the panel also the panels fell off the frame before the protocol finished.

Effect of Gypsum wallboard was explicitly investigated and found that gypsum provided an average additional strength of 3.2KN hence Gypsum should be considered to supply a substantial amount of shear resistance when subjected to monotonic loading, but is not linearly additive. In cyclic testing the contribution was 2.2 kN/m (on a unit length basis). But the area of concern is that the failure displacements of the specimens decreased when gypsum was present, hence a loss in ductility. After reaching the maximum load there is no contribution of Gypsum in any aspect such as stiffness, strength etc and also the contribution of Gypsum is none when used without hold downs. When walls were analyzed with or without hold downs Toothman concluded that the average peak load the wall can withstand increases three times with hold downs. The contribution of Gypsum in providing strength is negligible for walls without hold downs. Same trend can be observed in case of elastic stiffness. Also ductility decreased when OSB/GWB sheathings were used without hold downs. For walls without hold downs it was observed that gypsum does not increase the hysteretic energy when combined with a dissimilar sheathing material.

Uang and Gatto (2003) investigated the effect of a modified dynamic loading on wood frame walls in order to understand the effect of wall finish material and combined effect on the lateral resistance of the wall system. Their primary concern was the influence of wall finish material on performance parameter related to design, namely strength, stiffness and deformation capacity.

A total of eighteen 2.4m square specimen using different sheathing configurations were tested. All the sheathing was either OSB (10mm) or Plywood (12mm) of the size 1.2m x 2.4m attached to the framing with 8d box nails applied at 102 mm on center at panel perimeter and 305 mm on panel interior. All exterior cladding was 22 mm, three-coat, Portland cement stucco, and all interior finish was 13 mm gypsum wallboard (GWB) fastened to the wood framing using 32 mm long wallboard screws spaced at 406 mm on center. Adequate ties were used for hold down. The initial wall stiffness for all specimens was calculated and compared using both ASTM E 564 and FEMA 273 (1997) methods. The ASTM method takes the measure at 33% of the ultimate strength and FEMA uses 80% of the ultimate strength.

When comparing the walls with finish and without finish it was found that both strength and stiffness are increased due to the addition of wall finish materials. However, due to the increase in strength, a more brittle failure is observed and the deformation capacity is reduced, as the failure patterns shift from the sheathing connections to the structural framing members. It can also be seen that the dynamic effects are not nearly as pronounced as the addition of finish materials.

The general failure modes of wall panels without finish were dominated by nail failure. Nails pulling through the sheathing, nails pulling out of the structural framing, and even nail fracture were observed. The nail failure resulted in increased panel rotations, which in turn led to wall failure. For the specimens having sheathing on one side only, at failure the corner studs often twisted significantly due to eccentric effects resulting in torsion in the walls. Once the finish materials were added, the stud twisting was significantly reduced.

From the test results it is inferred that with the addition of wall finish materials it significantly affect the wall response. The finish materials increase strength and stiffness, however deformation capacity is reduced. The addition of GWB seems consequential since it results in a 12% increase in strength and a 31% reduction in deformation capacity. 34% increase in strength accompanies the use of stucco using the specified attachment and about 31% reduction in deformation capacity is seen.

