

**System-Level Interactions and Floor Diaphragm Out-of-Plane Stiffness Effects on  
the Lateral Response of Light-Frame Wood Shear Walls**

by  
**Esmail Morshedi**

Supervised by  
**Dr. Ghasan Doudak**

Thesis submitted to the University of Ottawa  
in partial fulfillment of the requirements for the degree of

**DOCTOR OF PHILOSOPHY**  
in Civil Engineering

Department of Civil Engineering  
Faculty of Engineering  
University of Ottawa

© Esmail Morshedi, Ottawa, Canada, 2026

## **Abstract**

The lateral performance of light-frame wood shear walls is significantly influenced by system-level interactions that are not fully captured by conventional design approaches. In particular, the impact of floor diaphragm out-of-plane stiffness on the lateral behaviour of wood shear walls, and its role in mitigating cumulative deformations along the height of multi-storey systems, introduces complexities in the interpretation of global response parameters such as inter-storey drift, structural stiffness, and force distribution among structural elements.

This study investigates the impact of floor diaphragm out-of-plane stiffness on the behaviour of wood shear walls through a combined experimental and numerical approach. An experimental program was conducted on single-storey, single- and multi-segment shear walls subjected to monotonic loading to characterize local and global deformation mechanisms under realistic system-level boundary conditions. A total of 14 test configurations were investigated. The key experimental parameters include shear wall configuration (single- and multi-segment), floor assembly configuration (continuous joists versus blocking), openings of varying lengths, types of anchorage systems (e.g., discrete hold-downs and continuous rods), shear wall aspect ratio, and wall-to-floor boundary conditions.

A numerical modelling framework incorporating system-level effects was developed and validated using the results from the single-storey shear wall tests. The validated model was subsequently extended to multi-storey shear wall systems to evaluate the influence of system-level interactions, including diaphragm out-of-plane stiffness, on the global response of buildings. Particular attention was given to the cumulative effects of hold-down elongations along the height, global structural

stiffness, and force distribution among shear walls at both the storey level and along the building height.

In addition, the four-term deflection equation in Canadian Standard Association for Engineering Design in Wood standard (CSA O86) was examined through comparisons with experimental results from single-storey, single- and multi-segment shear wall tests and numerical predictions. Its applicability was further evaluated for multi-storey systems by comparing its predictions with results from parametric studies considering various system-level characteristics and modelling assumptions related to wood shear wall behaviour.

The experimental results demonstrated that the out-of-plane stiffness of the floor assembly and system-level effects significantly reduce hold-down force and deformation demands, with average and maximum reductions of approximately 50% and 77%, respectively, while increasing nail deformations by about 50% on average. This behaviour indicates that a floor assembly promotes a racking-dominated response by restraining rocking deformations associated with hold-down elongations. Shear walls with higher aspect ratios exhibited greater reductions in hold-down demands and larger increases in nail deformations. While shear walls equipped with Anchor Tie-down System (ATS) rods showed similar trends to those with discrete hold-downs, the uplift stiffness of the ATS system was approximately 50% lower than the theoretical design value.

Both experimental and numerical results indicated limited bending deformation in chord members, which is not consistent with the assumptions made in the four-term deflection equation in CSA O86, which is based on cantilever beam theory. Sheathing panel shear deformations were found to be significantly lower than those predicted by the CSA O86 equation and remained relatively constant as the nail connections entered the nonlinear range. The CSA O86 equation also underestimates nail deformations and does not capture their increase due to system-level effects.

Similarly, it overestimates the hold-down force and deformation demands. Part of this overestimation is attributed to the contribution of anchor bolts, which are neglected in CSA O86 but were observed to resist up to 30% of the hold-down force. In contrast, perpendicular-to-grain deformation of the bottom plate was significantly higher than predicted by the CSA O86. However, this was a localized behaviour that primarily increased the nail deformations near the base rather than contributing to rigid-body rocking.

In multi-storey systems, the CSA O86 deflection equation, which includes cumulative effects, produced results similar to analyses that neglect floor diaphragm stiffness. However, excluding cumulative effects more closely represents buildings with realistic floor systems. Incorporating floor diaphragm out-of-plane stiffness significantly influenced inter-storey drift, global structural stiffness, and force distribution among shear walls both along the height and at the storey level. Neglecting floor out-of-plane stiffness can lead to overestimation of hold-down forces and deformations, which in turn results in underestimation of global structural stiffness and, consequently, design base shear. Furthermore, inclusion of floor effects altered force distribution among shear walls, with some walls experiencing increases in force demands of up to 100–200%, potentially leading to perceived premature failure if not properly accounted for in design.

The findings of this study provide insight into the deformation mechanisms of wood shear wall systems and emphasize the need for refined modelling and interpretation methods, particularly for multi-storey applications.



## Acknowledgements

I would like to express my deepest gratitude to my supervisor, Dr. Ghasan Doudak, for his continuous guidance, unwavering support, and patience throughout the course of this research. His insight, encouragement, and high standards have profoundly shaped both this work and my personal development as a researcher. I am especially grateful for his trust in my abilities and for the invaluable discussions that strengthened my understanding and perspective.

I would also like to sincerely thank my defence examiners, Dr. Murat Saatcioglu, Dr. Martin Noël, Dr. Christian Viau, and Dr. Chun Ni, for their time, thoughtful feedback, and valuable comments, which have contributed to improving the quality and clarity of this thesis.

I am grateful to the Structures Laboratory members, particularly Dr. Muslim Majeed and Dr. Gamal Elnabelsya, for their supports, collaboration, and the constructive environment they helped create. Their assistance and shared knowledge made this journey both productive and enjoyable.

My deepest and most heartfelt appreciation goes to my family, my dear parents and my sister, whose unconditional love, encouragement, and sacrifices have been the foundation of everything I have achieved. Their belief in me, especially during the most challenging moments, has been my greatest source of strength and motivation. Words cannot fully express my gratitude for their constant support and patience throughout this journey.

I would also like to acknowledge the University of Ottawa for providing an excellent academic environment and the resources necessary to carry out this research.

I gratefully acknowledge the financial and research support provided by the Canadian Wood Council and Mitacs. Their contributions played an important role in enabling this research and the work presented in this thesis.

I would like to acknowledge Simpson Strong-Tie for their continuous support in providing the wood connection hardware used in this research. I also gratefully acknowledge Three Islands Steel for their support in supplying a portion of the nails used in this study.

Finally, I extend my sincere thanks to all those who, in one way or another, have supported and accompanied me throughout this journey. This accomplishment would not have been possible without you.

# Table of Contents

|                                                                                         |     |
|-----------------------------------------------------------------------------------------|-----|
| Abstract .....                                                                          | ii  |
| Acknowledgements .....                                                                  | v   |
| List of Figures .....                                                                   | x   |
| List of Tables.....                                                                     | xvi |
| Chapter 1 Introduction and Problem Statement .....                                      | 1   |
| 1.1 Background and motivation .....                                                     | 1   |
| 1.2 Deflection of multi-storey LFW shear walls.....                                     | 7   |
| 1.3 Research gaps.....                                                                  | 9   |
| 1.4 Research objectives .....                                                           | 11  |
| 1.5 Scope and limitations .....                                                         | 12  |
| 1.6 Thesis structure.....                                                               | 12  |
| Chapter 2 Literature review.....                                                        | 14  |
| 2.1 Introduction .....                                                                  | 14  |
| 2.2 Analytical Models .....                                                             | 14  |
| 2.3 Finite Element Models .....                                                         | 18  |
| 2.4 Mechanics-based Models .....                                                        | 21  |
| 2.4.1 Shear Wall Deflection in Multi-storey LFW buildings.....                          | 27  |
| 2.5 Experimental tests of wood shear walls .....                                        | 30  |
| 2.6 Experimental Tests on Floor Diaphragms.....                                         | 32  |
| 2.7 Experimental and numerical studies on the global behaviour of LFW shear walls ..... | 33  |
| 2.8 Concluding remarks .....                                                            | 35  |
| Chapter 3 Experimental program .....                                                    | 37  |
| 3.1 Introduction .....                                                                  | 37  |
| 3.2 Materials.....                                                                      | 37  |
| 3.3 Fabrication of test specimens .....                                                 | 42  |
| 3.4 Shear wall test specimens.....                                                      | 43  |
| 3.4.1 Configuration #1.....                                                             | 45  |
| 3.4.2 Configuration #2.....                                                             | 48  |
| 3.4.3 Configuration #3.....                                                             | 49  |
| 3.4.4 Configuration #4.....                                                             | 50  |
| 3.4.5 Configuration #5.....                                                             | 50  |
| 3.4.6 Configuration #6.....                                                             | 51  |
| 3.4.7 Configuration #7.....                                                             | 51  |

|                                                  |     |
|--------------------------------------------------|-----|
| 3.4.8 Configuration #8.....                      | 52  |
| 3.4.9 Configuration #9 .....                     | 53  |
| 3.4.10 Configuration #10, #11 and #12.....       | 53  |
| 3.4.11 Configuration #13.....                    | 55  |
| 3.4.12 Configuration #14.....                    | 56  |
| 3.5 Floor assembly (beam) test specimens .....   | 59  |
| 3.6 Material and connection test specimens ..... | 60  |
| 3.6.1 Material test.....                         | 60  |
| 3.6.2 Connection test .....                      | 61  |
| 3.7 Test setup and instrumentation .....         | 64  |
| 3.7.1 Full-scale shear wall test setup .....     | 64  |
| 3.7.2 Floor assembly bending test setup.....     | 69  |
| 3.7.3 Connection test setup.....                 | 71  |
| 3.7.4 Nail bending test.....                     | 74  |
| Chapter 4 Test results .....                     | 75  |
| 4.1 Introduction .....                           | 75  |
| 4.2 Material test results .....                  | 75  |
| 4.3 Connection test results.....                 | 82  |
| 4.4 Floor assembly tests .....                   | 91  |
| 4.5 Full-scale shear wall tests.....             | 94  |
| 4.5.1 Configuration #1.....                      | 94  |
| 4.5.2 Configuration #2.....                      | 103 |
| 4.5.3 Configuration #3.....                      | 105 |
| 4.5.4 Configuration #4.....                      | 109 |
| 4.5.5 Configuration #5.....                      | 110 |
| 4.5.6 Configuration #6.....                      | 114 |
| 4.5.7 Configuration #7.....                      | 118 |
| 4.5.8 Configuration #8.....                      | 120 |
| 4.5.9 Configuration #9.....                      | 122 |
| 4.5.10 Configuration #10.....                    | 125 |
| 4.5.11 Configuration #11.....                    | 127 |
| 4.5.12 Configuration #12.....                    | 129 |
| 4.5.13 Configuration #13.....                    | 131 |
| 4.5.14 Configuration #14.....                    | 133 |

|                                                                                                             |     |
|-------------------------------------------------------------------------------------------------------------|-----|
| 4.5.15 EEEP parameters of the full-scale shear wall tests.....                                              | 135 |
| Chapter 5 Numerical analysis and results .....                                                              | 137 |
| 5.1 Introduction .....                                                                                      | 137 |
| 5.2 Numerical modeling framework.....                                                                       | 137 |
| 5.3 Analysis method .....                                                                                   | 146 |
| 5.4 Verification of the numerical model.....                                                                | 147 |
| 5.5 Multi-storey shear wall.....                                                                            | 149 |
| Chapter 6 Discussion.....                                                                                   | 157 |
| 6.1 Effect of the floor assembly on the shear wall stiffness .....                                          | 157 |
| 6.2 Effect of the floor on shear wall resistance .....                                                      | 161 |
| 6.3 Effect of the floor assembly on nail deformations .....                                                 | 162 |
| 6.4 Effect sheathing panel AR on nail deformations.....                                                     | 167 |
| 6.5 Floor out-of-plane and system effects impact on HD force and deformation demands .....                  | 169 |
| 6.5.1 Influence of multiple sheathing panels and AB contribution on HD force demands .....                  | 175 |
| 6.6 Validity of CSA O86 four-term deflection equation .....                                                 | 178 |
| 6.6.1 Deflection due to bending of shear wall chord members .....                                           | 181 |
| 6.6.2 Deflection due to shear deformation of the sheathing panel .....                                      | 184 |
| 6.6.3 Shear wall deflection due to nail deformation .....                                                   | 186 |
| 6.6.4 Shear wall deflection due to HD elongation .....                                                      | 190 |
| 6.6.5 Shear wall deflection due to perpendicular-to-grain compression deformations .....                    | 195 |
| 6.6.6 Concluding remark on the four-term deflection equation .....                                          | 198 |
| 6.7 Multi-storey effects .....                                                                              | 200 |
| Chapter 7 Summary, conclusion and future works .....                                                        | 214 |
| Appendix A Shear Wall Configurations, Design Information, and Complete Test Results (Response Curves) ..... | 224 |
| A.1 Configuration #1.....                                                                                   | 224 |
| A.2 Configuration #2.....                                                                                   | 227 |
| A.3 Configuration #3.....                                                                                   | 229 |
| A.4 Configuration #4.....                                                                                   | 231 |
| A.5 Configuration #5.....                                                                                   | 233 |
| A.6 Configuration #6.....                                                                                   | 235 |
| A.7 Configuration #7.....                                                                                   | 237 |
| A.8 Configuration #8.....                                                                                   | 239 |
| A.9 Configuration #9.....                                                                                   | 241 |

|                                                                                       |     |
|---------------------------------------------------------------------------------------|-----|
| A.10 Configuration #10.....                                                           | 243 |
| A.11 Configuration #11.....                                                           | 245 |
| A.12 Configuration #12.....                                                           | 247 |
| A.13 Configuration #13.....                                                           | 249 |
| A.14 Configuration #14.....                                                           | 253 |
| Appendix B Discussion of material and connection test results and failure modes ..... | 257 |

## List of Figures

|                                                                                                                                                                                                                                                                                                                             |    |
|-----------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------|----|
| Figure 1.1: Light-frame wood building (WoodWorks 2025).....                                                                                                                                                                                                                                                                 | 1  |
| Figure 1.2: Wood shear wall .....                                                                                                                                                                                                                                                                                           | 3  |
| Figure 1.3: Force transfer mechanism in a wood shear wall .....                                                                                                                                                                                                                                                             | 4  |
| Figure 1.4: Realistic boundary conditions in a LFW building .....                                                                                                                                                                                                                                                           | 6  |
| Figure 1.5: Cumulative effects of deformations in LFW shear walls .....                                                                                                                                                                                                                                                     | 7  |
| Figure 1.6: Realistic representation of a multi-storey LFW shear wall.....                                                                                                                                                                                                                                                  | 8  |
| Figure 1.7: Lateral deflection of LFW shear walls: (a) High out-of-plane stiffness of the floor; (b) Flexible floor .....                                                                                                                                                                                                   | 8  |
| Figure 2.1: Deformation of the shear wall due to: (a) nail deformation; (b) panel shear deformation .....                                                                                                                                                                                                                   | 16 |
| Figure 2.2: Shear wall deflections due to (a) bending of chord members; and (b) shear deformation of the sheathing panel.....                                                                                                                                                                                               | 23 |
| Figure 2.3: Shear wall cross section: (a) actual; (b) equivalent cross section .....                                                                                                                                                                                                                                        | 24 |
| Figure 2.4: (a) comparison between nail deformation models and test data; (b) comparison between different shear wall deflection models and shear wall test results (Wang 2009) .....                                                                                                                                       | 25 |
| Figure 2.5: Shear wall deflection due to rocking motion .....                                                                                                                                                                                                                                                               | 26 |
| Figure 2.6: Cantilever beam assumption of wood shear walls.....                                                                                                                                                                                                                                                             | 27 |
| Figure 2.7: Displacement of $i$ th storey including the displacement due to the bending of the storey below (Canadian Standards Association 2019).....                                                                                                                                                                      | 28 |
| Figure 2.8: Deflection of a wood shear wall due to cumulative effects of HD elongations (Canadian Standards Association 2019) .....                                                                                                                                                                                         | 29 |
| Figure 2.9: Discontinuous rim board above shear wall (Skaggs et al. 2008). .....                                                                                                                                                                                                                                            | 33 |
| Figure 3.1: Spruce-Pine-Fir lumber.....                                                                                                                                                                                                                                                                                     | 37 |
| Figure 3.2: OSB sheathing panel.....                                                                                                                                                                                                                                                                                        | 38 |
| Figure 3.3: Laminated Veneer Lumber (LVL) joist.....                                                                                                                                                                                                                                                                        | 39 |
| Figure 3.4: Discrete HD system (Simpson Strong-Tie 2022) .....                                                                                                                                                                                                                                                              | 40 |
| Figure 3.5: Anchor Tie-down System (Simpson Strong-Tie 2022) .....                                                                                                                                                                                                                                                          | 40 |
| Figure 3.6: Bearing plates and anchor bolts .....                                                                                                                                                                                                                                                                           | 41 |
| Figure 3.7: Shear wall fabrication .....                                                                                                                                                                                                                                                                                    | 42 |
| Figure 3.8: Plan layout of the 3-storey LFW building (Morshedi et al. 2024).....                                                                                                                                                                                                                                            | 43 |
| Figure 3.9: Geometries of the shear wall test configurations: (a) A.1; (b) Configuration #2, Configuration #5, Configuration #6, Configuration #13, and Configuration #14; (c) Configuration #3; (d) Configuration #4 and Configuration #9; (e) Configuration #7 and Configuration #8; (f) Configuration #10, Configuration |    |

|                                                                                                                                                                                                            |    |
|------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------|----|
| #11 and Configuration #12; (g) Configuration #10, Configuration #11 and Configuration #12; (h) Configuration #10, Configuration #11 and Configuration #12 .....                                            | 45 |
| Figure 3.10: Shear walls in Configuration #1 .....                                                                                                                                                         | 46 |
| Figure 3.11: Shear walls in Configuration #2 .....                                                                                                                                                         | 48 |
| Figure 3.12: Inter-storey floor connection details .....                                                                                                                                                   | 51 |
| Figure 3.13: Different shear wall boundary conditions .....                                                                                                                                                | 52 |
| Figure 3.14: Blocking floor assembly configuration .....                                                                                                                                                   | 53 |
| Figure 3.15: Floor assembly configurations .....                                                                                                                                                           | 59 |
| Figure 3.16: Nail connection geometries: (a) sheathing-to-framing connection; (b) end-nail withdrawal connection; (c) end-nail lateral connection; (d) double-member lateral connection .....              | 62 |
| Figure 3.17: HD connection assembly .....                                                                                                                                                                  | 63 |
| Figure 3.18: Two-ply built-up member .....                                                                                                                                                                 | 63 |
| Figure 3.19: Test setup for full-scale shear wall test .....                                                                                                                                               | 65 |
| Figure 3.20: Elevation view of full-scale shear wall test setup: (a) back view; (b) front view .....                                                                                                       | 66 |
| Figure 3.21: Instrumentation layout: (a) front view; (b) back view .....                                                                                                                                   | 67 |
| Figure 3.22: Location of DTs in SEG1: (a) TL corner; (b) panel interface; (c) panel interface for stud uplift; (d) BR corner in SEG1 .....                                                                 | 69 |
| Figure 3.23: (a) Test setup for floor assembly bending test; (b) floor assembly on top of shear wall .....                                                                                                 | 70 |
| Figure 3.24: Tow-point loading test .....                                                                                                                                                                  | 71 |
| Figure 3.25: Connection test apparatuses: (a) end-nail lateral connection; (b) end-nail withdrawal connection; (c) frame-to-frame connection; (d) sheathing-to-framing connection; (e) HD connection ..... | 72 |
| Figure 3.26: Connection test setups: (a) sheathing-to-framing connection; (b) frame-to-frame connection; (c) HD connection; (d) ATS rod tensile test .....                                                 | 73 |
| Figure 3.27: Nail bending test setup and apparatus .....                                                                                                                                                   | 74 |
| Figure 4.1: Stud bending test .....                                                                                                                                                                        | 75 |
| Figure 4.2: Load-displacement curves from stud bending tests .....                                                                                                                                         | 76 |
| Figure 4.3: LVL bending test .....                                                                                                                                                                         | 76 |
| Figure 4.4: Load-displacement curve from LVL bending test .....                                                                                                                                            | 77 |
| Figure 4.5: Nail bending test: a) tested specimen; b) load-displacement curves of 8d nail specimens .....                                                                                                  | 78 |
| Figure 4.6: Offset method for determining the nail yield strength .....                                                                                                                                    | 79 |
| Figure 4.7: Universal Testing Machine .....                                                                                                                                                                | 80 |
| Figure 4.8: Stress-strain diagrams; a) ATS-SR4; b) ATS-SR5 .....                                                                                                                                           | 80 |
| Figure 4.9: Yield point calculation methods .....                                                                                                                                                          | 81 |

|                                                                                                                                                     |     |
|-----------------------------------------------------------------------------------------------------------------------------------------------------|-----|
| Figure 4.10: Connection specimens: a) sheathing-to-framing; b) end nail lateral; c) end nail withdrawal; d) frame-to-frame; e) hold-down .....      | 83  |
| Figure 4.11: Test setups for nail connection tests .....                                                                                            | 83  |
| Figure 4.12: (a) comparison between sheathing-to-framing connection tests; (b) EEEP idealization .....                                              | 84  |
| Figure 4.13: Sheathing-to-framing failure modes .....                                                                                               | 85  |
| Figure 4.14: Comparison between different nail slip behaviours.....                                                                                 | 86  |
| Figure 4.15: (a) End-nail lateral connection; (b) End-nail withdrawal connection; (c) frame-to-frame connection .....                               | 86  |
| Figure 4.16: Frame connection failure modes.....                                                                                                    | 87  |
| Figure 4.17: HD test setup.....                                                                                                                     | 87  |
| Figure 4.18: F–D diagrams of HD connections.....                                                                                                    | 88  |
| Figure 4.19: HD failure modes.....                                                                                                                  | 89  |
| Figure 4.20: (a) Built-up member test; (b) F–D curve .....                                                                                          | 91  |
| Figure 4.21: Representation of built-up section with its corresponding solid section.....                                                           | 91  |
| Figure 4.22: Floor assemblies: a) continuous LVL; b) discrete blocking .....                                                                        | 92  |
| Figure 4.23: Floor assembly test configurations .....                                                                                               | 93  |
| Figure 4.24: Load-displacement diagrams of floor assembly tests .....                                                                               | 93  |
| Figure 4.25: Shear wall test setup in Configuration #1 .....                                                                                        | 95  |
| Figure 4.26: Shear wall F–D curves: (a) top displacement; (b) sliding at the bottom of the shear wall .....                                         | 96  |
| Figure 4.27: Initiation and progression of the failures in Configuration #1 .....                                                                   | 97  |
| Figure 4.28: Failure modes at the top and sheathing panel interface in SEG1.....                                                                    | 97  |
| Figure 4.29: HD (HDU5) slippage in shear wall Configuration #1 .....                                                                                | 98  |
| Figure 4.30: HD force and displacement demands in Configuration #1 .....                                                                            | 98  |
| Figure 4.31: Perpendicular-to-grain compression deformations (Configuration #1) .....                                                               | 100 |
| Figure 4.32: Perpendicular-to-grain compression deformations .....                                                                                  | 101 |
| Figure 4.33: Nail deformations in Configuration #1 .....                                                                                            | 101 |
| Figure 4.34: Bending deformation in SEG2: a) compression chord; a) tension chord; c) lateral deflection diagrams of compression chord in SEG2 ..... | 102 |
| Figure 4.35: Configuration #2 .....                                                                                                                 | 103 |
| Figure 4.36: Sheathing panel tearing and nail failures in Configurations #3 .....                                                                   | 106 |
| Figure 4.37: Nail deformation in Configuration #3.....                                                                                              | 107 |
| Figure 4.38: Top plates separation and nail damages at the top of shear walls (Configuration #3).....                                               | 107 |
| Figure 4.39: (a) deformation of HD in SEG2; (b) HD force-displacement curves .....                                                                  | 108 |
| Figure 4.40: Nail failures at the top and bottom edges of the sheathing panel.....                                                                  | 109 |



|                                                                                                                                                                 |     |
|-----------------------------------------------------------------------------------------------------------------------------------------------------------------|-----|
| Figure 4.41: (a) Shear wall F–D behaviour in Configuration #5; (b) Fractured HD in SEG1 .....                                                                   | 111 |
| Figure 4.42: (a) Shear wall force–HD force diagrams; (b) HD F–D diagrams; HD force ratio.....                                                                   | 112 |
| Figure 4.43: Formation of the gap between shear wall top plates and floor assembly.....                                                                         | 113 |
| Figure 4.44: Nail failures at the bottom edge of shear walls in Configuration #5 .....                                                                          | 114 |
| Figure 4.45: (a) shear wall force-displacement curve; (b) HD at the floor level; (c) Tension HD failure                                                         | 115 |
| Figure 4.46: (a) HD force–displacement curves; (b) HD force ratio.....                                                                                          | 116 |
| Figure 4.47: (a) separation of the top plate from chord at fractured chord location ; (b) nail failures at the top of the shear wall .....                      | 117 |
| Figure 4.48: (a) HD force; (b) HD elongation; (c) initial imperfections .....                                                                                   | 118 |
| Figure 4.49: Failure modes: (a) nail failure and panel separation; (b) HD slippage; (c) chord fracture ...                                                      | 120 |
| Figure 4.50: SEG2 tension chord fracture in Configuration #8 .....                                                                                              | 122 |
| Figure 4.51: Installation of strain gages on chord members .....                                                                                                | 123 |
| Figure 4.52: HD force-displacement (Configuration #9).....                                                                                                      | 124 |
| Figure 4.53: Strain at the interface of HD and chord member .....                                                                                               | 125 |
| Figure 4.54: Bending deformation: (a) tension chord; (b) compression chord (Configuration #10) .....                                                            | 127 |
| Figure 4.55: HD forces (Configuration #11).....                                                                                                                 | 129 |
| Figure 4.56: Shear wall Configuration #12 (after test).....                                                                                                     | 130 |
| Figure 4.57: Bending deformations: (a) tension chord (SEG1); (b) compression chord (SEG2); (c) rate of change of bending deformations (Configuration #13) ..... | 132 |
| Figure 4.58: (a) nail failure at panel interface; (b) failure of the ATS rod system.....                                                                        | 133 |
| Figure 4.59: Perpendicular-to-grain compression deformations (Configuration #14) .....                                                                          | 134 |
| Figure 4.60: Bending deformations of the chord members (Configuration #14).....                                                                                 | 135 |
| Figure 4.61: Failure modes in Configuration #14 .....                                                                                                           | 135 |
| Figure 5.1 Realistic representation of a wood shear wall.....                                                                                                   | 139 |
| Figure 5.2: FE representation of the wood shear wall.....                                                                                                       | 139 |
| Figure 5.3: F-D relationships for frame end connections: (a) end-nail lateral; (b) end-nail withdrawal...                                                       | 140 |
| Figure 5.4: Sheathing-to-framing F-D relationship.....                                                                                                          | 141 |
| Figure 5.5: Bending of the chord at HD location: a) shear wall test; b) HD test .....                                                                           | 143 |
| Figure 5.6: HD bracket in numerical model.....                                                                                                                  | 143 |
| Figure 5.7: Calculation of the AB effective stiffness .....                                                                                                     | 144 |
| Figure 5.8: AB force-deformation (Configuration #12).....                                                                                                       | 145 |
| Figure 5.9: FE model of the wood shear wall in SAP2000 .....                                                                                                    | 146 |
| Figure 5.10: Validation of the numerical model against experimental shear wall test results .....                                                               | 147 |

|                                                                                                                                                    |     |
|----------------------------------------------------------------------------------------------------------------------------------------------------|-----|
| Figure 5.11: Validation of the numerical model against test results: (a)–(c) HDs demands; (d)–(e) nail deformation demands .....                   | 148 |
| Figure 5.12: Designated shear walls for multi-storey numerical simulation.....                                                                     | 150 |
| Figure 5.13: Two-segment three-storey shear wall configuration.....                                                                                | 151 |
| Figure 5.14: Three-segment three-storey shear wall configuration.....                                                                              | 151 |
| Figure 5.15: Numerical model of three-segment shear wall configuration.....                                                                        | 152 |
| Figure 5.16: The effect of the floor on the lateral behaviour of multi-storey shear walls.....                                                     | 152 |
| Figure 5.17: Shear wall force versus inter-storey drift curves .....                                                                               | 153 |
| Figure 5.18: Distribution of the shear wall forces at different storey levels .....                                                                | 154 |
| Figure 5.19: HD force demands in shear walls .....                                                                                                 | 155 |
| Figure 6.1: The effect of the floor assembly on the initial stiffness of shear walls.....                                                          | 157 |
| Figure 6.2: Comparison between the initial stiffness ( $K_{0-40}$ ) of the shear walls from EEEP analysis .....                                    | 158 |
| Figure 6.3: Comparison between shear wall initial stiffness values from CSA O86, test results and numerical simulations .....                      | 159 |
| Figure 6.4: (a) comparison between the peak and factored resistance shear walls; (b) shear wall overstrength values.....                           | 161 |
| Figure 6.5: Nail deformation in single-segment shear walls without floor .....                                                                     | 163 |
| Figure 6.6: Nail deformations: (a) Configuration #11; (b) Configuration #12.....                                                                   | 164 |
| Figure 6.7: Nail deformations of multi-segment shear walls: (a) unequal wall lengths; (b) equal wall lengths .....                                 | 165 |
| Figure 6.8: The effect of sheathing panel AR on the nail deformations .....                                                                        | 168 |
| Figure 6.9: Comparison between HD forces from CSA O86, test, and numerical simulations .....                                                       | 169 |
| Figure 6.10: Comparison between HD elongation from CSA O86, test, and numerical simulations .....                                                  | 172 |
| Figure 6.11: Comparison of theoretical and measured stiffness of the ATS rods: (a) ATS-SR4; (b) ATS-SR5 .....                                      | 174 |
| Figure 6.12: (a) CSA O86 shear wall rocking behaviour assumption; (a) Realistic configuration of a shear wall.....                                 | 175 |
| Figure 6.13: Effect of multiple panels of HD force demand .....                                                                                    | 176 |
| Figure 6.14: Measured and model prediction HD and AB forces .....                                                                                  | 177 |
| Figure 6.15: Shear wall deflection components: (a) test results; (b) numerical simulation .....                                                    | 180 |
| Figure 6.16: Lateral displacement of shear wall chord members: (a) left chord; (b) right chord.....                                                | 181 |
| Figure 6.17: Bending deformation of the chord members of shear wall Configuration #10.....                                                         | 182 |
| Figure 6.18: Lateral displacement of shear wall chord members in Configuration #10 from numerical simulation: (a) left chord; (b) right chord..... | 183 |

|                                                                                                                                                                    |     |
|--------------------------------------------------------------------------------------------------------------------------------------------------------------------|-----|
| Figure 6.19: Installation of strain gauges along the diagonal of the sheathing panel in Configuration #10 .....                                                    | 184 |
| Figure 6.20: (a) Shear wall deflection due panel shear deformations; (b) contribution of the sheathing panel shear deformation in total shear wall deflection..... | 185 |
| Figure 6.21: (a) Shear wall deflection due to nail deformations; (b) contribution of the nail deformations in total shear wall deflection.....                     | 187 |
| Figure 6.22: (a) Nail deformation mechanism in wood shear walls; (b) locations of nail deformation measurements recorded in Configuration #10.....                 | 189 |
| Figure 6.23: Ratio of corner nail deformation to nail deformation at DT locations.....                                                                             | 189 |
| Figure 6.24: Contribution of different deflection components including nail deformations at corner locations .....                                                 | 190 |
| Figure 6.25: Contribution of HD elongation in total shear wall deflection.....                                                                                     | 191 |
| Figure 6.26: Shear wall deflection due to HD elongations at different shear wall load levels.....                                                                  | 194 |
| Figure 6.27: Contribution of perpendicular-to-grain compression of the bottom plate to the total lateral deflection of the shear wall Configuration #10 .....      | 196 |
| Figure 6.28: Compression perpendicular to grain deformation of the bottom plate .....                                                                              | 197 |
| Figure 6.29: Multi-segment single-storey shear wall .....                                                                                                          | 201 |
| Figure 6.30: Representation of a wood shear wall in a multi-story system.....                                                                                      | 202 |
| Figure 6.31: Deformed shape of three-storey shear wall (M1 model) .....                                                                                            | 203 |
| Figure 6.32: Comparison between shear wall F-D curves in a multi-storey system and corresponding CSA O86 curves according to M1 model .....                        | 204 |
| Figure 6.33: Effect of cumulative displacements on the lateral displacement of shear walls .....                                                                   | 205 |
| Figure 6.34: (a)-(c) Shear wall force for; (d)-(f) HD force; (g)-(i) overturning moment.....                                                                       | 206 |
| Figure 6.35: Inter-storey drift of SEG1 in three-segment three-storey shear wall .....                                                                             | 208 |
| Figure 6.36: Variation of the bending moment in floor assembly.....                                                                                                | 211 |

## List of Tables

|                                                                               |     |
|-------------------------------------------------------------------------------|-----|
| Table 3-1: Fastener specifications.....                                       | 39  |
| Table 3-2: Anchorage devices specifications.....                              | 41  |
| Table 3-3: Shear wall design information-Configuration #1.....                | 47  |
| Table 3-4: Shear wall deflections-Configuration #1. ....                      | 47  |
| Table 3-5: Shear wall test matrix .....                                       | 57  |
| Table 3-6: Shear wall design information. ....                                | 58  |
| Table 3-7: Shear wall deflections.....                                        | 58  |
| Table 3-8: Material test specimens.....                                       | 61  |
| Table 3-9: Connection test matrix .....                                       | 64  |
| Table 4-1: MOE values of the stud and LVL members .....                       | 77  |
| Table 4-2: Bending yield strength of common nails .....                       | 79  |
| Table 4-3: ATS rod test data .....                                            | 81  |
| Table 4-4: EEEP idealization parameters.....                                  | 84  |
| Table 4-5: Summary of HD test results.....                                    | 90  |
| Table 4-6: EEEP idealization parameters of the shear walls.....               | 136 |
| Table 5-1: Wood material properties.....                                      | 138 |
| Table 5-2: HD and perpendicular to grain stiffness values .....               | 142 |
| Table 5-3: Distribution of forces between shear walls in the third level..... | 155 |
| Table 5-4: Distribution of HD forces in shear walls in the first storey ..... | 156 |

# Chapter 1

## Introduction and Problem Statement

### 1.1 Background and motivation

Light-frame wood (LFW) construction is the predominant building method for residential and low-rise commercial buildings in North America. In addition to its construction simplicity and cost effectiveness, this method of construction has several other advantages such as energy efficiency, durability, environmentally friendly, and high strength-to-weight ratio. The latter is a paramount factor in considering the structural behaviour of a building subjected to seismic loading. Figure 1.1 depicts a common layout of a multi-storey LFW building characterised by stacked walls and relatively simple regular layout. This type of layout is typical in engineered buildings with residential and commercial occupancies and is in contrast to Part 9 residential buildings with maximum height of three storeys or less, and where the layout is typically more irregular. The focus of current study is on engineered buildings although some of the findings may be extended to non-engineered buildings.

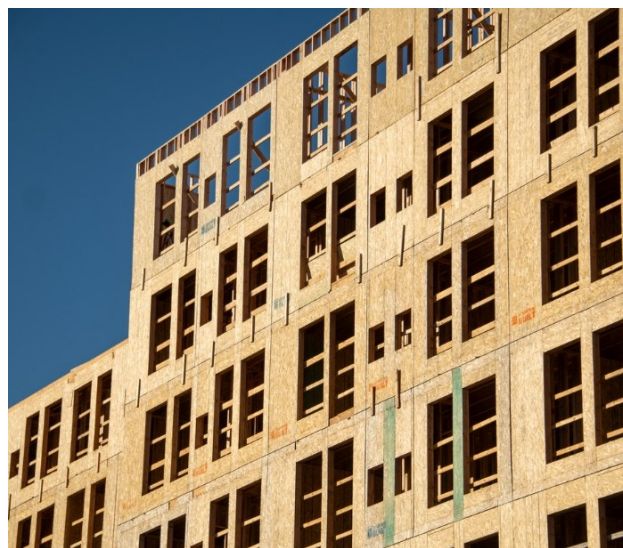
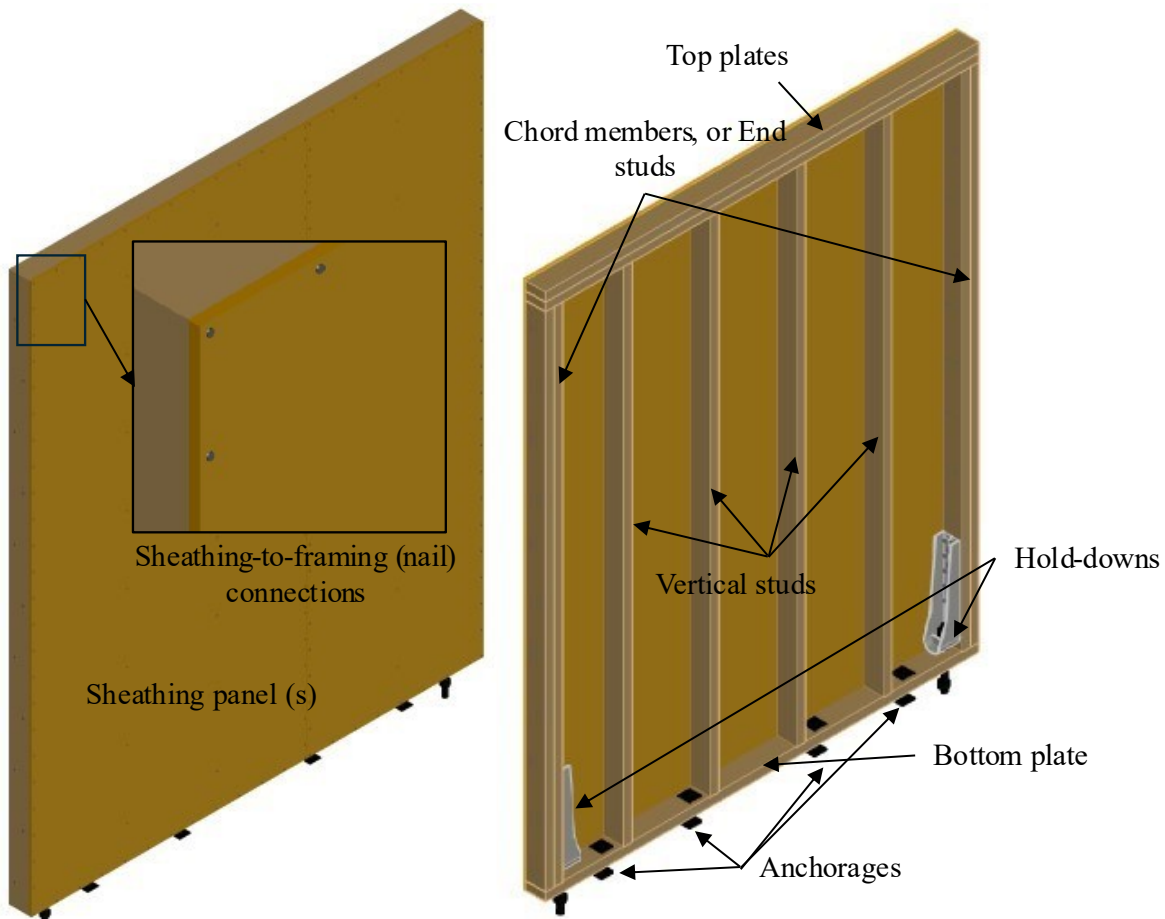


Figure 1.1: Light-frame wood building (WoodWorks 2025)

The LFW walls in this type of buildings could be loadbearing or non-loadbearing (i.e., partition wall), and they could resist vertical gravity loads, horizontal lateral load, both in and out of plane, and in many cases a combination of these loads. Walls resisting in-plane lateral forces from wind and earthquake are referred to as shear walls and such wall system could extend over multiple storeys. Combining multi-storey shear walls within a building is referred to as the lateral force resisting system (LFRS). Continuity in shear walls along the height of the building is imperative and key to the performance of such structural systems, as illustrated in Figure 1.1. These walls collect the inertial forces generated at each diaphragm level and transfer them into the foundation through a complex but well-defined load path.

Depending on architectural features, such as window or door openings, shear lines can be comprised of multiple shear wall segments, which collectively resist the lateral forces. A typical shear wall, along with its key elements, is depicted in Figure 1.2.



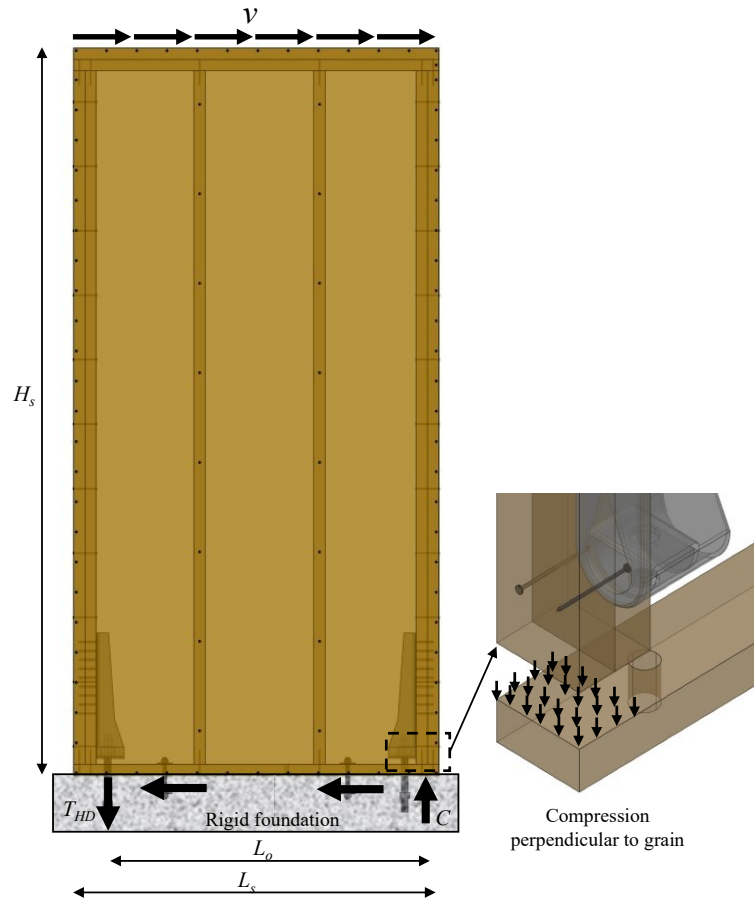
**Figure 1.2: Wood shear wall**

As illustrated in Figure 1.2, a typical shear wall comprises several interconnected elements that collectively form an integrated structural component. Dimensional lumber elements, such as  $2 \times 4$  in. ( $38 \times 89$  mm) or  $2 \times 6$  in. ( $38 \times 140$  mm) are generally used in LFW construction as framing elements (e.g., vertical studs and horizontal plates). Each vertical stud is connected to the horizontal top- and bottom plates through a pair of construction nails; the most typical fastener used for this purpose is 16d ( $89 \times 4.1$  mm) common nail. Nails of different sizes (e.g., 6d, 8d, 10d) are commonly used for sheathing-to-framing connections, while varying the spacing between nails controls the strength and stiffness of the shear wall. The primary role of the intermediate vertical studs is to support gravity loads from the storeys above as well as wind loads in the out-of-plane direction, and they are usually spaced at 16 in. (406 mm) or 24 in. (610 mm) apart. Boundary

elements at the ends of each shear wall are referred to as chord members or end studs, and they are usually constructed using several studs that are connected using construction nails.

The in-plane stability of the shear wall is ensured by attaching plywood or oriented strand board (OSB) sheathing panels to the framing elements using dowl-type fasteners such as nails, staples or screws. This results in an efficient diaphragm element capable of resisting lateral forces.

Figure 1.3 schematically illustrates the load transfer mechanism of an isolated shear wall.



**Figure 1.3: Force transfer mechanism in a wood shear wall**

The force from the floor or roof diaphragm ( $v$ ) is first collected by shear wall top plates and then distributed into various elements of the shear wall, including nails, sheathing panel(s), and the chord members (i.e., end studs) until it finally reaches the base of the shear wall. At the base of the



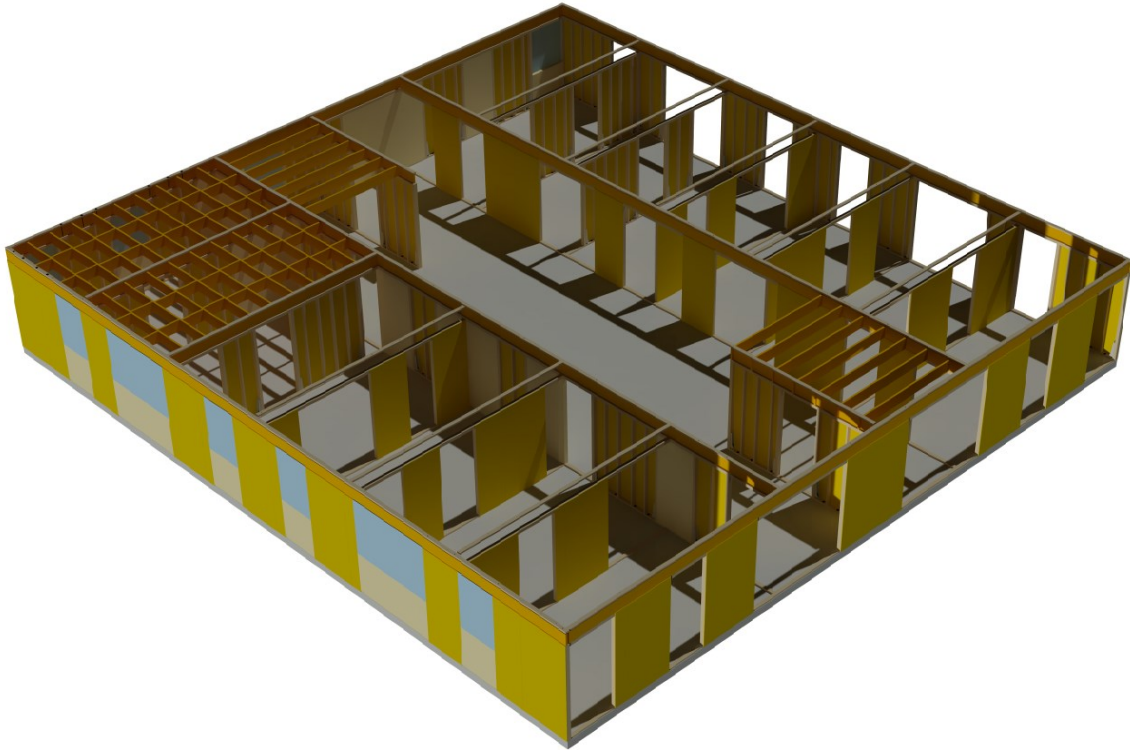
shear wall, the horizontal shear force is resisted by anchor bolts (ABs), which are designed to resist the shear forces and restrain the shear wall against sliding. In order to resist the overturning moment caused by the applied shear force at the floor level, hold-downs (HD) are attached to the end studs of the shear wall and anchored to the foundation or the storey below it.

The strength of a wood shear wall is primarily governed by nail connections, since they are responsible for transferring and mobilizing in-plane shear. The stiffness of a shear wall depends on all constituent elements as well as the shear wall configuration. The contribution of individual elements to stiffness varies with configuration; for instance, HD devices have a negligible effect on long shear walls but a noticeable influence on the stiffness of short shear walls.

The deflection of a wood shear wall is traditionally calculated according to the cantilever model and based on beam theory. In this regard, a wood shear wall is assumed to act as a cantilever beam in which the elements of the shear wall act in series. Accordingly, the total deflection of the shear wall is calculated as the summation of the deflections corresponding to different elements in the shear wall, as outlined in the CSA O86 (Canadian Standards Association 2019). These deflection components include bending of the chord members, shear deformation of the sheathing panel(s), deformation of the nail connections, crushing of the bottom plate beneath the compression chord, and elongation of the HD device.

These assumptions could be considered valid when a single-storey shear wall is isolated from the rest of the building. However, in an actual building, the distribution of forces and deformations within a shear wall depends not only on the shear wall configuration, but also on its boundary conditions and its interaction with other structural and non-structural components. Figure 1.4 shows an inside view of a typical LFW building that demonstrates the complexity of the boundary elements surrounding each shear wall segment. As such, a shear wall is not an isolated structural

component; it is rather a part of an intricate system of interconnected components, which renders simplified engineering assumptions related to the tributary area force distribution or cantilever beam model too crude and simplistic to represent the real behaviour of LFW buildings.



**Figure 1.4: Realistic boundary conditions in a LFW building**

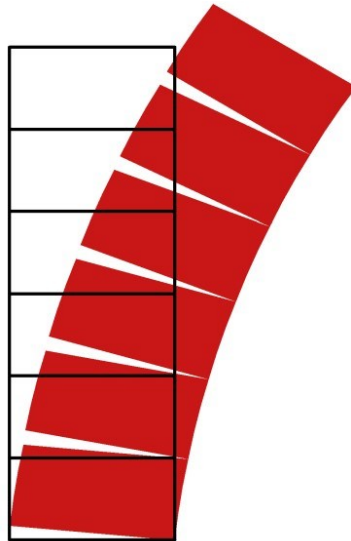
Depending on the building layout, a shear line could consist of multiple different shear walls with different lengths and configurations, each connected to the floor system and surrounded by openings of different configurations. Other complexities include intersecting with perpendicular walls on one or both ends.

Regarding the impact of the floor diaphragm, the floor system may exhibit varying level of flexural rigidity in the out-of-plane direction depending on floor joist directions (e.g., whether they run parallel or perpendicular to the shear wall). In addition, multiple plies of sawn lumber or laminated veneer lumber (LVL) joists are typically mounted at the exterior shear walls. Hence, the boundary

conditions imposed by the aforementioned elements (i.e., system-level effects) may affect the distribution of forces and deformations within the shear walls, resulting in the accuracy of current methodologies to obtain deflection and strength based on simplified assumptions being questioned.

## **1.2 Deflection of multi-storey LFW shear walls**

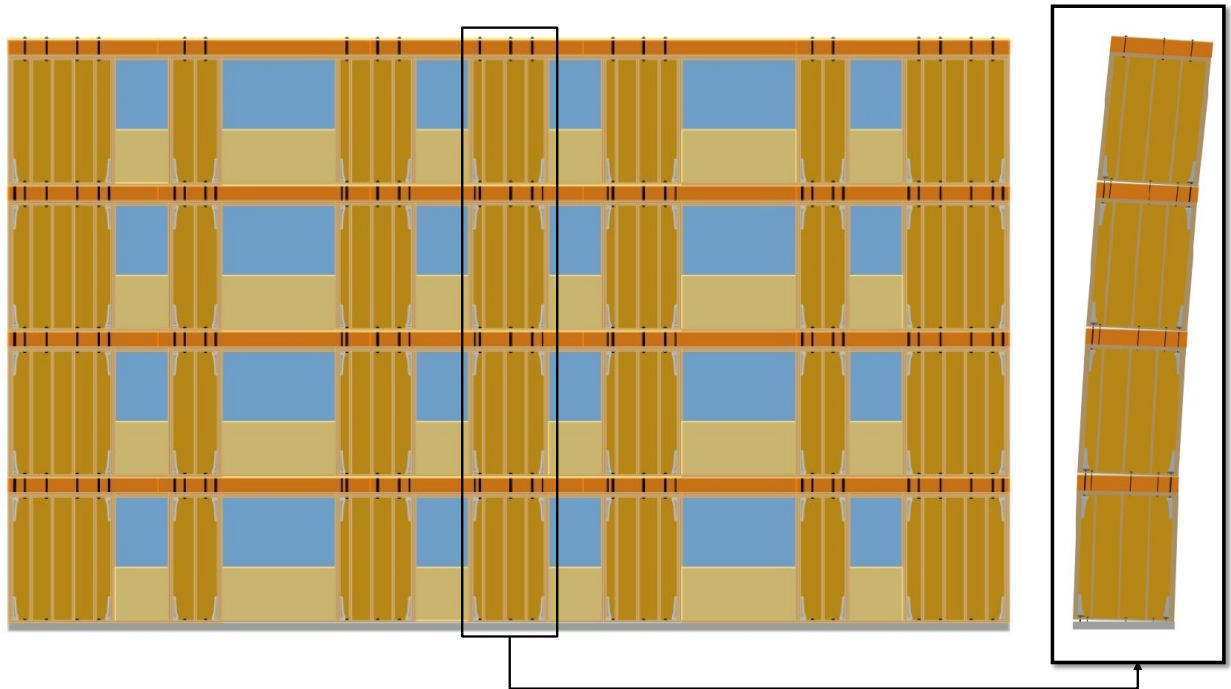
The current design provisions in the CSA O86 (Canadian Standards Association 2019) regarding inter-storey drift calculations of multi-storey LFW shear walls require the inclusion of cumulative effects due to bending deformations of the chord members and rigid body rotation due to HD elongations of lower storeys (see Figure 1.5). This is based on the assumption that individual shear walls are stacked vertically and act independently, without restraint from the floor system or interaction with other components of the building.



**Figure 1.5: Cumulative effects of deformations in LFW shear walls**

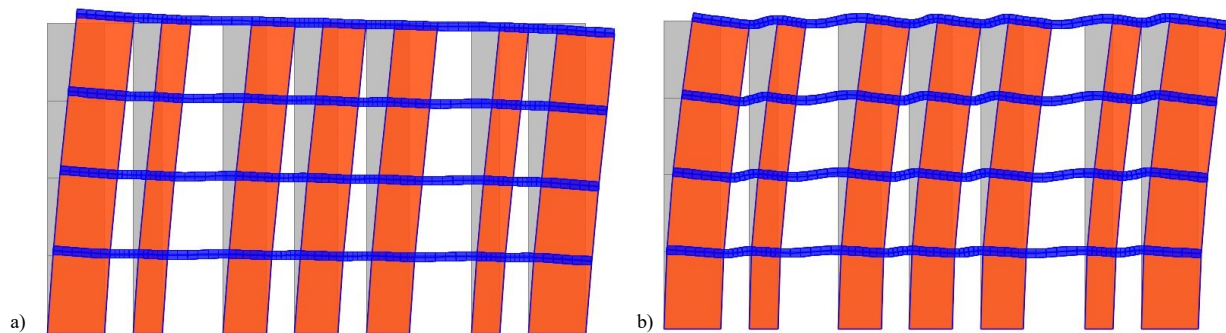
Due to the complex behaviour of LFW shear wall buildings described earlier, it can be deduced that the building deflection shown in Figure 1.5 may not be representative of the actual building deformation encountered in reality. A more realistic representation of a multi-storey shear wall

system is depicted in Figure 1.6 , where the deflection of an individual shear wall along the building height cannot be evaluated without considering system-level interactions.



**Figure 1.6: Realistic representation of a multi-storey LFW shear wall**

A major omission in the CSA O86 (Canadian Standards Association 2019) is the out-of-plane flexural stiffness of the floor system. Figure 1.7(a) and (b) present two idealized scenarios with a building containing a very high out-of-plane flexural stiffness of the floor system and one containing a completely flexible floor, respectively.



**Figure 1.7: Lateral deflection of LFW shear walls: (a) High out-of-plane stiffness of the floor; (b) Flexible floor**

Figure 1.7(a) and (b) and the preceding discussions make it clear that the lateral response of LFW shear walls in real buildings can differ markedly from that based on the simplified assumptions adopted in the CSA O86 (Canadian Standards Association 2019). The floor out-of-plane stiffness influences not only the behaviour of individual shear walls at each storey but also, and possibly more importantly, the overall shape of the lateral displacement over the entire building.

The out-of-plane flexural stiffness of the floor assembly above the shear wall may vary along a shear line, depending on its configuration (e.g., the presence of continuous members or blocking). In addition, openings of varying lengths can lead to non-uniform flexural behaviour of the floor system at different locations, resulting in distinct boundary conditions above each shear wall segment.

Earlier studies have reported that HD force distribution and elongations are often unequal at the two ends of shear walls. This lack of symmetry has been attributed to the participation of the floor system and the specific boundary conditions at the intersection of shear walls (van de Lindt et al. 2010). Pei & Van De Lindt (2011) also demonstrated numerically that neglecting realistic boundary conditions can lead to inaccurate estimation of internal forces, particularly HD demands. Therefore, a reliable understanding of the true behaviour of LFW shear walls has several practical consequences, as it supports refinement of design provisions to estimate shear wall deflections with greater confidence, enables the development of improved numerical models that better represent real buildings, and helps to decrease uncertainties in key response parameters when these buildings are evaluated under seismic loading.

### **1.3 Research gaps**

A large body of knowledge exists on the topic of LFW shear walls in isolation, where focus has been on factors influencing their response to lateral loads. However, quantitative investigations of

the role of floor out-of-plane stiffness on the lateral behaviour of multi-segment and multi-storey shear walls remain very limited. This gap in knowledge restricts the understanding of the actual lateral behaviour and response of LFW shear walls, especially multi-storey buildings.

The influence of realistic system-level boundary conditions on the behaviour of wood shear walls has not been adequately quantified. Therefore, a comprehensive understanding of the behaviour of these structural components in actual multi-storey LFW buildings requires targeted experimental studies that explicitly represent the different realistic boundary conditions similar to those in real buildings. Current analytical and experimental approaches commonly rely on isolated single-storey shear wall models to estimate stiffness and lateral deformation, neglecting inter-storey interaction, diaphragm constraints, and load redistribution mechanisms. This simplification is likely to result in biased estimates of global stiffness and, consequently, inaccurate predictions of force and deformation demand in real multi-storey structures.

Neglecting aspects like the out-of-plane stiffness of the floor can affect the prediction of the dynamic characteristics of LFW buildings, which are critical for seismic design. In particular the fundamental period ( $T_a$ ) is expected to be sensitive to the stiffness estimate of the building. An empirical formula for period calculation is provided by the National Building Code of Canada (Canadian Commission on Building and Fire Codes 2020), in order to determine the design seismic base shear force. A comparison between the NBC 2020 empirical formula for LFW shear wall systems and data obtained experimentally showed significant scatter among period values obtained during earthquakes, shake table, forced vibration, and ambient vibration tests (Hafeez, Doudak, and McClure 2019).

A framework that incorporates these influential parameters for the design of LFW buildings is still lacking. Developing such a framework would allow multi-storey LFW buildings to be modeled

and designed more consistently, with behaviours resembling those of actual buildings. In addition, a more accurate numerical representation would pave the way for further parametric and sensitivity analyses that are otherwise impractical to perform experimentally.

The current study directly addresses these critical gaps by providing experimental evidence and validated numerical models to quantify the impact of floor out-of-plane stiffness and realistic boundary conditions on the lateral deflection, strength, and stiffness of single- and multi-storey LFW buildings.

#### **1.4 Research objectives**

To address the gaps outlined in the previous section, the present study experimentally investigates the stiffness and deformation characteristics of LFW shear walls incorporating realistic boundary conditions. The key influencing parameters examined include single- and multi-segment shear walls, shear walls with different aspect ratios, opening lengths, and floor configurations in terms of out-of-plane stiffness (e.g., floor joist orientation), HD type (e.g., discrete versus continuous), and wall-to-floor boundary condition. The specific objectives of the project are to:

- Evaluate, experimentally, the applicability and accuracy of the CSA O86 four-term deflection equation for LFW shear walls.
- Investigate how floor out-of-plane stiffness and wall-to-floor interaction influence shear wall behaviour.
- Develop a numerical model, which can be validated against the experimentally tested wood shear walls and compared to the CSA O86 deflection predictions.
- Extend the validated numerical model to multi-storey shear walls, enabling assessment of how the experimentally observed parameters influence the global stiffness and deformation behaviour of multi-storey LFW buildings.

## **1.5 Scope and limitations**

This study primarily investigates the lateral behaviour of wood shear walls under realistic system-level boundary conditions. Particular attention is given to the influence of floor out-of-plane flexural stiffness on the lateral deflection of multi-segment, single-storey wood shear walls, as well as system-level effects on HD forces and deformations. Based on the experimental observations, a numerical modeling framework is developed to capture the key system-level interactions.

It should be highlighted that the experimental program is limited to single-storey shear wall tests conducted along a single wall line, without incorporating full three-dimensional floor system effects. Instead, floor influence is represented through a floor assembly consisting of continuous LVL joists or discrete blocking. The shear walls were constructed using a single OSB panel thickness (11 mm) and  $2 \times 4$  in. ( $38 \times 89$  mm) dimensional lumber throughout. A nail spacing of 100 mm was used for all specimens except one, which had a spacing of 50 mm. Only monotonic loading was applied.

## **1.6 Thesis structure**

Chapter 1 introduces LFW shear wall buildings and presents the problem statement, research gaps, and objectives of the study.

Chapter 2 provides a comprehensive review of the literature relevant to the research objectives. The review is organized into seven sections, including analytical, finite element, and mechanics-based models for predicting wood shear wall deflections. The chapter then reviews experimental studies at the wall level, followed by the limited available research on floor diaphragms. Numerical modeling framework for global analysis of LFW buildings is subsequently discussed. The chapter concludes with a summary of key findings and identified research needs.



Chapter 3 describes the experimental program, including material and connection testing, floor assembly configurations, and shear wall tests. The chapter also details the test setup and instrumentation scheme used in the experiments.

Chapter 4 presents the experimental results, covering materials, connections, floor assembly, and shear walls.

Chapter 5 outlines the development of the numerical model and presents the numerical results. This includes validation of the model against the experimental shear wall tests, followed by its extension to a parametric study of multi-storey shear walls.

Chapter 6 discusses the experimental and numerical findings, including comparisons between test results, numerical predictions, and relevant code provisions. Multi-storey effects are discussed and their implications in design of LFW shear walls are explained.

Chapter 7 summarizes the main conclusions of the study and provides recommendations for future research.

## **Chapter 2**

### **Literature review**

#### **2.1 Introduction**

Several early attempts have been made by researchers to predict the strength and stiffness of wood shear walls, and despite their simplicity, these methods have been shown to provide reasonable predictions (e.g., McCutcheon, 1985; Tuomi & McCutcheon, 1978).

A brief history and overview of these methods are provided in this chapter, and the assumptions underlying the development of each method are explained. Building upon the existing literature, the four-term deflection equation for single-storey shear walls in the CSA O86 (Canadian Standards Association 2019) is discussed in detail, and its extension to multi-storey buildings is explained. The chapter also presents published experimental studies on shear walls and diaphragms. Subsequently, experimental tests of full-scale buildings are reviewed. Finally, the chapter reviews numerical investigations concerning system effects on the demand parameters in LFW shear wall buildings.

#### **2.2 Analytical Models**

Early studies on analytical models investigating the behaviour of LFW shear walls were based on simplified assumptions of the kinematics of the walls, and using energy methods, rendering them suitable for design and preliminary evaluation purposes. These methods were mainly developed based on specific shear wall configurations, and as a result, they may satisfactorily predict the response of shear walls with similar configurations; however, their accuracy in predicting other shear wall configurations remains questionable (McCutcheon 1985; Tuomi and McCutcheon 1978). The accuracy of such methods is largely limited to the assumed simplified boundary conditions, which may differ from those in real buildings. In reality, the distribution of forces and

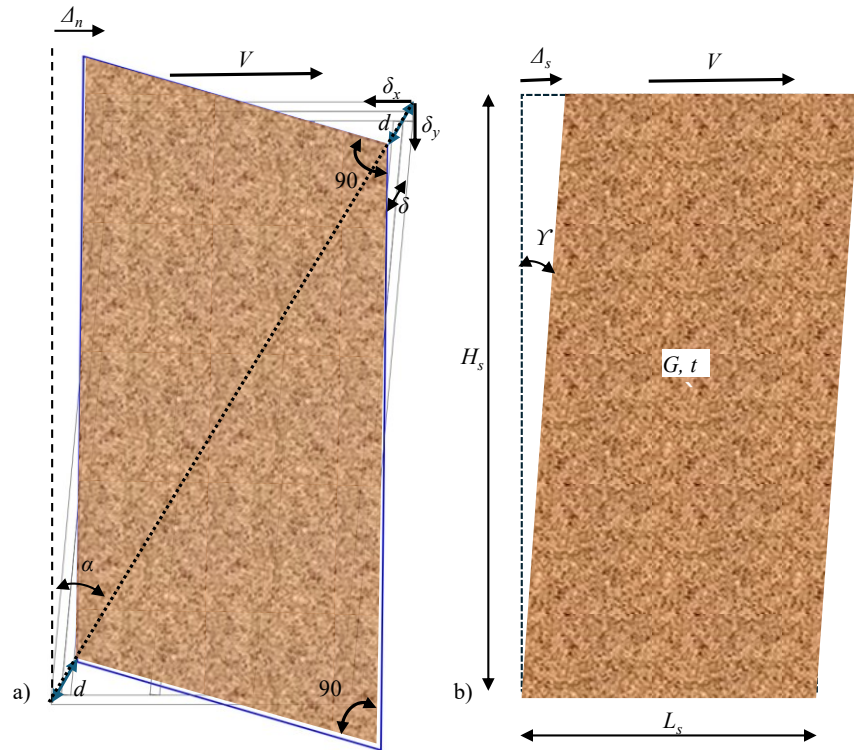
deformations within individual shear wall elements can be significantly influenced by actual boundary conditions.

Tuomi & McCutcheon (1978) established an analytical closed-form solution for predicting the racking behaviour of wood shear walls. The model was based on several assumptions, including linear behaviour of the sheathing-to-framing connections (nails), rigid sheathing panel and framing elements, uniform distribution of nails around the perimeter of the sheathing panel, and small wall deformations and distortions (i.e., small rotations). The model predictions of racking strength showed satisfactory agreement with experimental results, although it was based on few full-scale tests. However, given the highly simplified, and to some extent unrealistic assumptions, Robertson (1980) raised concerns regarding the model's inability to predict the behaviour of shear walls with multiple sheathing panels and account for realistic boundary conditions. The author confirmed that the distribution of nail forces and deformations was significantly affected by the boundary conditions of the shear wall.

Easley et al. (1982) used the concept of force and moment equilibrium to derive an analytical formulation for predicting the strength and stiffness of LFW shear walls. The formulation was based on racking deformation mode of the shear wall and assumed that the framing elements remained fully connected, with no possibility of separation. A linear formulation was proposed for the initial stiffness of wood shear walls. The formulation showed satisfactory agreement with experimental test results and finite element simulations, primarily at low levels of loading. This formulation, however, was shown to underestimate the shear wall stiffness (Gupta and Kuo 1985). Gupta & Kuo (1985, 1987) used the strain energy concept to develop relationships for the strength and stiffness of wood shear walls. In the formulation, the elastic bending behaviour of the framing elements and the shear rigidity of the sheathing panel were incorporated. It was concluded that the

bending stiffness of the studs and the shear stiffness of the sheathing panel have an insignificant impact on the shear wall strength, while they play an important role in determining the stiffness. Furthermore, accounting for stud uplift was shown to have a significant impact on shear wall stiffness.

McCutcheon (1985) introduced nonlinearity in nail behaviour into the energy-based formulation developed by Tuomi & McCutcheon (1978) in order to improve the prediction of the racking deformations. Figure 2.1(a) and (b) schematically depict a single-panel shear wall, which was used in model development, along with its corresponding deformed kinematic modes.



**Figure 2.1: Deformation of the shear wall due to: (a) nail deformation; (b) panel shear deformation**

In Figure 2.1(a),  $d$  represents the nail deformations at the top and bottom corners of the wall, measured along the wall diagonal, and  $\Delta_n$  denotes the horizontal deflection of the wall due to nail deformations. The angle  $\alpha$  is defined as the angle between the vertical edge of the shear wall and its diagonal. The horizontal deflection of the shear wall also includes the contribution of in-plane

shear deformation of the sheathing panel ( $\Delta_s$ ), as shown in Figure 2.1(b), where  $\gamma$  represents the shear distortion angle of the sheathing panel.

The horizontal shear wall deflection due to the nail deformations at opposite corners was defined by Eq. (2.1) (McCutcheon 1985).

$$\Delta_n = \frac{2d}{\sin \alpha} \quad (2.1)$$

The total nail deformation at each sheathing-to-framing connection is denoted  $\delta$  in Figure 2.1(a). A power function was assumed to represent the nonlinear load-deformation behaviour of the nail connection, as defined in Eq. (2.2).

$$\delta = \left( \frac{p}{A} \right)^{1/B} \quad (2.2)$$

where  $p$  is the force in the nail, and  $A$  and  $B$  are empirical constants obtained from the calibration of Eq. (2.2) against nail connection tests. The values of these constants depend on the sheathing panel type and thickness as well as the framing density.

Other assumptions including rigid behaviour of the framing elements and the sheathing panel were kept unchanged. Stud uplift was also excluded from the formulation. As a result, the model remained restricted to idealized boundary conditions and simplified deformation mechanisms. Comparisons between the model predictions and experimental test results showed reasonable agreement at small displacements; however, significant deviations were observed at moderate to large displacements.

Patton-Mallory & McCutcheon (1987) used different mathematical forms to fit experimental data of wood shear walls with varying lengths. Although these fitted functions improved agreement with test results for specific wall configurations, their accuracy was found to be dependent on shear wall length, limiting their general applicability.

Stewart (1987) undertook analytical and experimental studies on wood shear walls and developed several models for predicting shear wall peak strength as well as shear wall stiffness. The proposed formulations suggested that the strength and stiffness of wood shear walls were primarily governed by the strength and stiffness of the sheathing-to-framing connections. Contrary to the findings in Gupta & Kuo (1985, 1987), Stewart (1987) concluded that stud bending did not significantly influence the strength and stiffness of shear walls. In addition, the effect of separation between framing elements was found to have an insignificant impact on both the strength and stiffness of the walls.

In summary, analytical methods are distinguished by their simplicity, requiring only a limited number of input parameters, typically physical characteristics of the shear wall (e.g., wall length, sheathing thickness, nail diameter, and spacing). Their predictions are generally reasonable, particularly at small displacement levels and generally under the design strength level. However, these methods rely on simplified boundary condition assumptions and are often calibrated for specific wall configurations. In real buildings, where boundary conditions differ from the underlying assumptions, simplified formulations may not accurately capture the distribution of forces and deformations and, consequently, the true behaviour of shear walls. Therefore, although analytical methods are useful for preliminary assessment and design, their applicability becomes limited when detailed response and system-level interactions are of interest.

## **2.3 Finite Element Models**

FE-based models are inherently more complex than analytical ones; however, they offer a high level of versatility, allowing structures of arbitrary geometry to be modeled under various loading conditions. One of the main advantages of these models is their ability to provide detailed predictions of forces and deformations within individual structural elements. While this capability

represents a significant strength of the method, it also requires that all input parameters be defined with appropriate F–D relationships.

Foschi (1977) employed the FE method to model wood diaphragms, in which the sheathing panel was represented as linear elastic material with orthotropic plane-stress properties and the framing members were modeled using elastic beam elements. The sheathing-to-framing connections were modeled by nonlinear spring elements. The nonlinear behaviour of these connection springs was defined using an exponential function fitted to experimental data obtained from connection tests. The study concluded that the overall nonlinear behaviour of the diaphragm closely followed, and was primarily governed by, the nonlinear response of the sheathing-to-framing connections.

Itani & Cheung (1984) used a two-dimensional 4-node quadrilateral element with plane stress behaviour to represent the plywood sheathing, while linear elastic beam element was used for framing members. A nonlinear elastic behaviour was assumed for the connections between sheathing the framing elements. Comparison between the FE model and the experimental data showed satisfactory results; however, the level of accuracy was not consistent for all specimens. It is worth noting that the tested shear walls barely reach displacements greater than 20 mm, and therefore, the accuracy of the FE model was not verified at large displacement levels.

Gutkowski & Castillo (1988) used FE method to model wood shear walls. Empirical formulas available in the literature were adopted to represent nail slip behaviour for the sheathing-to-framing connections. The FE results were compared with experimental data, and the comparisons showed that nail slip behaviour has a significant influence on the ability of the model to predict the response observed in shear wall tests.

Falk & Itani (1989) developed a FE model for floor diaphragms. Their results indicated that the increase in diaphragm stiffness was not proportional to the increases in connection stiffness, while

nail spacing exhibited a more pronounced influence. It was reported that the high variability observed in nail connection test results did not translate into a comparable level of variability in diaphragm stiffness, due to the smoothing effect of the diaphragm action.

Similar to the procedure adopted by Foschi (1977), Dolan (1989) used the FE method to analyze wood shear walls under both static and dynamic loading. The primary difference between the models was the inclusion of out-of-plane degrees of freedom for the sheathing panel by Dolan (1989), which were introduced to capture potential buckling behaviour. The resulting model was verified against experimental shear wall test results, and the comparisons showed good agreement. White (1995) modified the model developed by Dolan (1989), noting that buckling of the sheathing panel was not observed for panels greater than certain thicknesses (e.g., 9.5 mm). It was concluded that in order to improve computational efficiency, the out-of-plane degrees of freedom of the sheathing panel could be eliminated without compromising accuracy. Similar to Dolan (1989), the effect of anchorage uplift was neglected in the formulation.

Folz and Filiatrault (2001) developed a FE model for the cyclic analysis of wood shear walls entitled Cyclic Analysis of Shear walls (CASHEW). Several simplifications were adopted, including rigid behaviour of the framing elements, rigid behaviour of the sheathing panel, and neglecting HD effect. However, the translational and rotational rigid body movement of the sheathing panel were included in the FE formulations. This model proved particularly useful for developing cyclic load-displacement response for shear wall configurations that were not tested experimentally.

Hassanzadeh Shirazi (2012) and Pang and Hassanzadeh Shirazi (2013) developed a co-rotational formulation model referred to as Matlab-Cyclic Analysis of Shear walls (M-CASHEW), which built on the CASHEW program developed by Folz and Filiatrault (2001). The distinction between



the M-CASHEW and CASHEW is that the M-CASHEW model considers the flexural stiffness of the framing elements (i.e., vertical studs, sill and top plates). It also includes the HD connections and the connections between vertical studs and sill- and top plates. Ziaei Ghehnavieh (2017) developed a 3D building model ( Timber3D), which was an extension of the 2D M-CASHEW model developed by Hassanzadeh Shirazi (2012) and (Pang and Hassanzadeh Shirazi 2013).

The uncertainty and inconsistencies between FE predictions and actual shear wall response quantities can be partially reduced by incorporating experimentally derived mechanical properties of materials and connections. Another source of inconsistency between measured data and FE results arises from assumptions embedded in FE formulations and from limitations in accurately capturing the true kinematics of shear walls. Many FE models developed to date for the analysis of LFW shear walls are based on small displacement and rotation assumptions. This modeling approach remains reliable when force and deformation levels are sufficiently low; however, wood shear walls consist of a large number of interconnected elements, and the assumption of small displacements and rotations may no longer be valid at large deformation levels, particularly when separation between individual components can occur. Such effects can significantly influence FE predictions, especially with respect to the distribution of forces and deformations among individual elements. Consequently, the development of FE framework capable of simulating the actual behaviour of wood shear walls remains an open and important research question. Contributing to this topic would not only benefit the research community but could also support engineering practice by improving the reliability of LFW building design.

## **2.4 Mechanics-based Models**

Another approach to predicting the deflection of wood shear walls is based on the principles of mechanics. In this approach, the wood shear wall shown in Figure 1.3 is idealized as a cantilever

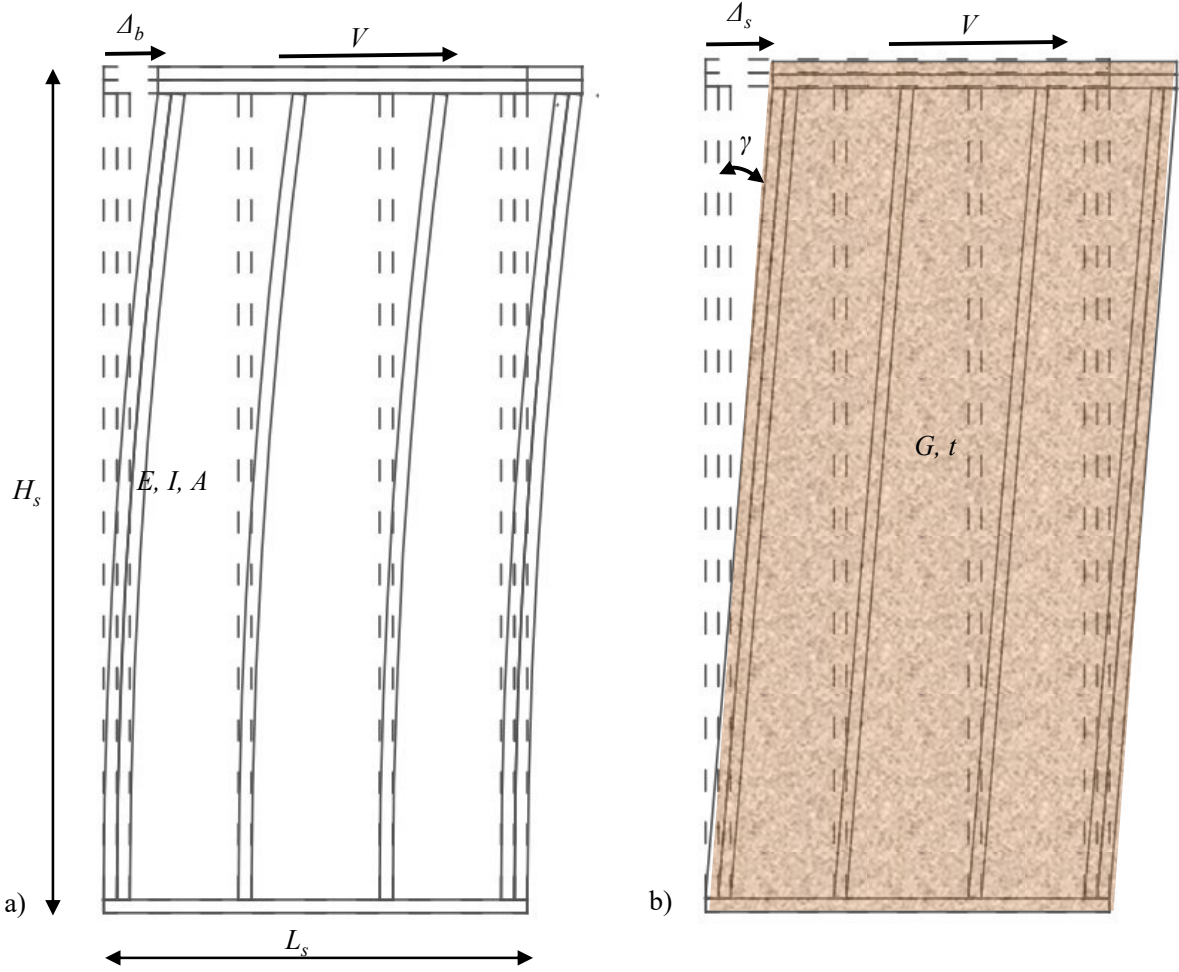
beam with fixed-free ends. The deflection of the shear wall ( $\Delta_{sw}$ ) due to the applied load at the top of the wall is expressed as the sum of deflection components associated with the individual components comprising the shear wall. The current methodology in the CSA O86 (Canadian Standards Association 2019) for calculating the deflection of single-storey wood shear walls is expressed in Eq. (2.3).

$$\Delta_{sw} = \frac{2vH_s^3}{3EAL_s} + \frac{vH_s}{B_v} + 0.0025H_se_n + \frac{H_s}{L_s}d_a \quad (2.3)$$

In Eq. (2.3), the first term represents the shear wall deflection due to bending of the chord members ( $\Delta_b$ ), the second corresponds to shear deformation of the sheathing panel ( $\Delta_s$ ), the third accounts for sheathing-to-framing (nail) deformations ( $\Delta_n$ ), and the fourth term represents deflection due to HD uplift and perpendicular-to-grain compression deformation of the bottom plate beneath the compression chord ( $\Delta_a$ ).

In Eq. (2.3)  $v$  is the uniform shear force at the top of the shear wall (N/mm),  $H_s$  is the shear wall height (mm),  $E$  is the modulus of elasticity of the chord members (N/mm<sup>2</sup>),  $A$  is the total cross-sectional area of the chord members (mm<sup>2</sup>),  $L_s$  is the length of the shear wall (mm),  $B_v$  is the shear-through-thickness rigidity of the sheathing panel (N/mm),  $e_n$  is the nail deformation (mm), and  $d_a$  is the sum of the HD elongation ( $d_{a1}$ ) and the compression deformation of the bottom plate (refer to Figure 1.3) under the compression chord ( $d_{a2}$ ). Each component of the total shear wall deflection is discussed in detail in the following sections.

In calculating the shear wall deflection due to the bending of chord members, the vertical studs are assumed to be fixed at the base and free at the top, resulting in bending deflection mode consistent with that shown in Figure 2.2(a).



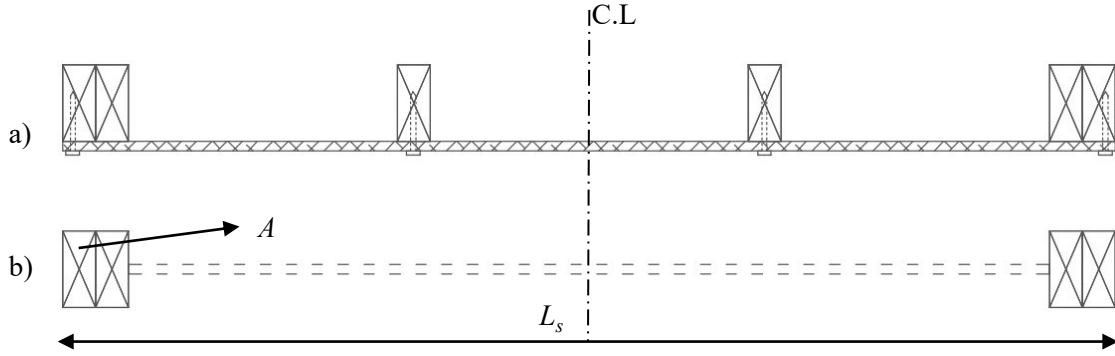
**Figure 2.2: Shear wall deflections due to (a) bending of chord members; and (b) shear deformation of the sheathing panel**

It is assumed that the flexural resistance of the boundary members (chords) governs the bending response, while the contribution of intermediate studs is neglected. Under these assumptions, the bending deflection of the shear wall due to a shear force  $V$  applied at the top is given by Eq. (2.4):

$$\Delta_b = \frac{VH_s^3}{3EI} \quad (2.4)$$

In Eq. (2.4), parameter  $I$  represents the moment inertia of an equivalent shear wall cross section, which only includes the chord members, about an axis that passes through the wall centerline and is perpendicular to the plane of the wall (Figure 2.3). The use of an equivalent cross section implies full composite action between the sheathing panel and the chord members, provided through the sheathing-to-framing connections. Figure 2.3(a) and (b) illustrate the actual shear wall cross

section and the corresponding idealized equivalent section, respectively. Based on this idealization, the moment of inertia is approximated by  $I \approx 2A(L/2)^2$ . Substitution of this expression into Eq. (2.4) results in the first term in Eq. (2.3). It is evident that the assumptions of full base fixity of the chord members and complete composite action is not fully representative of actual wood shear wall behaviour, but it facilitates the derivation of the bending contribution term.



**Figure 2.3: Shear wall cross section: (a) actual; (b) equivalent cross section**

The corresponding deformed configuration of the shear wall due to shear deformations in the sheathing panel is illustrated in Figure 2.2(b). Parameters  $V$ ,  $\gamma$ ,  $G$ , and  $t$ , shown in the figure, denote the shear force at the top of the shear wall, shear distortion (strain) angle, in-plane shear modulus, and the thickness of the sheathing panel, respectively. The shear strain is defined as shown in Eq. (2.5):

$$\gamma = \frac{\Delta_s}{H_s} \quad (2.5)$$

In Eq. (2.5), the relationship between shear strain and shear stress is expressed by Eq. (2.6):

$$\gamma = \frac{\tau}{G} = \frac{V/Lt}{G} \quad (2.6)$$

Combining Eqs. (2.5) and (2.6) yields the second term of Eq. (2.3), expressed as Eq. (2.7):

$$\frac{\Delta_s}{H_s} = \frac{V/Lt}{G} \rightarrow \Delta_s = \frac{vH_s}{Gt} = \frac{vH_s}{B_v} \quad (2.7)$$

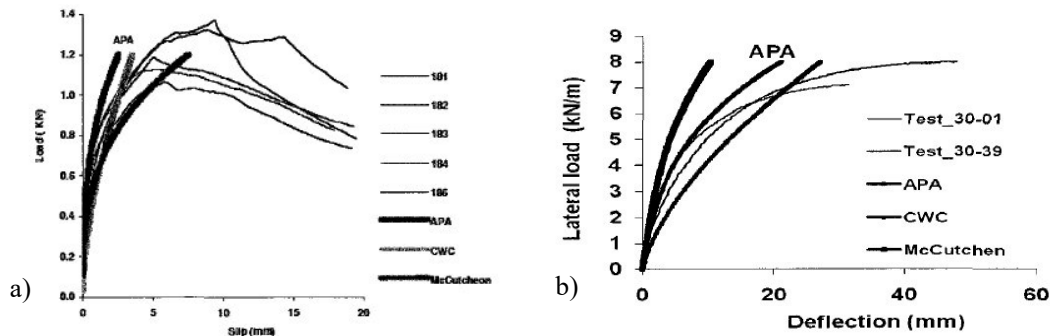
The parameters appearing in Eq. (2.7) are defined previously.

The shear wall deflection due to nail deformations is proportional to both the wall height and the nail deformation, as represented by the third term in Eq. (2.3). The parameter  $e_n$  is defined in Eq. (2.8):

$$e_n = \left( \frac{0.013vs}{d_f^2} \right)^2 \quad (2.8)$$

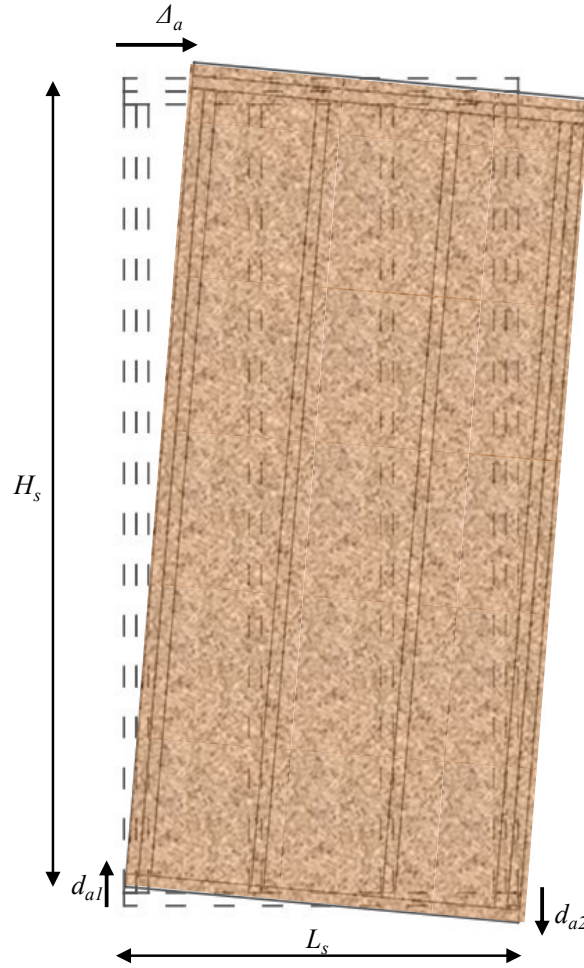
where  $s$  is the nail spacing (mm), and  $d_f$  is the nail diameter (mm). Because the equation is empirical, it indirectly incorporates other important aspects, such as sheathing panel properties and framing density; however, these effects are included implicitly and remain subject to uncertainty. In addition, the presence of the constant coefficient 0.0025 in Eq. (2.3) is an adjustment factor calibrated based on the geometry and experimental shear wall test results (McCutcheon 1985; Patton-Mallory and McCutcheon 1987).

Wang (2009) compared several nail deformation relationships proposed by McCutcheon (1985), the American Plywood Association (APA, 2001), and the Canadian Wood Council (CWC, 2005) against experimental nail test results, as shown in Figure 2.4(a). Significant discrepancies among the formulations were observed, particularly at higher load levels. In addition, the corresponding shear wall deflection models derived from these documents were compared with shear wall test results (Figure 2.4(b)). The comparisons indicate that both nail-level and wall-level models exhibit considerable variability relative to experimental observations.



**Figure 2.4: (a) comparison between nail deformation models and test data; (b) comparison between different shear wall deflection models and shear wall test results (Wang 2009)**

The last term in the deflection equation, Eq. (2.3), represents the horizontal deflection of the shear wall due to its rocking motion,  $\Delta_a$ , as illustrated in Figure 2.5.



**Figure 2.5: Shear wall deflection due to rocking motion**

The horizontal deflection associated with rocking is obtained by multiplying the shear wall aspect ratio ( $H/L$ ) by the sum of  $d_{a1}$  and  $d_{a2}$ , as expressed in Eq. (2.9):

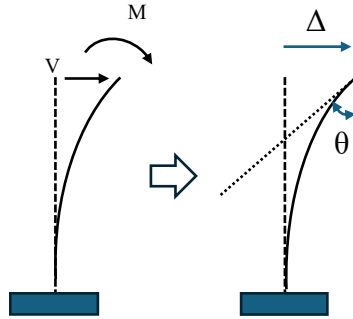
$$\Delta_a = \frac{H_s}{L_s} (d_{a1} + d_{a2}) \rightarrow \Delta_a = \frac{H_s}{L_s} d_a \quad (2.9)$$

It is worth noting that deflection equation in the CSA O86 (Canadian Standards Association 2019) does not explicitly account for the presence of multiple sheathing panels or the contribution of other anchorage systems, such as ABs. Under these assumptions, a wood shear wall is idealized as

a single rigid unit with continuous sheathing over its full length, anchored only at its ends, yet prevented from sliding.

#### 2.4.1 Shear Wall Deflection in Multi-storey LFW buildings

In addition to the deflection equation outlined in Eq. (2.3), the CSA O86 (Canadian Standards Association 2019) standard outlines consideration for the cumulative effects of lateral displacements when analysing a multi-storey LFW building. This formulation is also based on the idealization of the shear wall as a cantilever beam subjected to a force and moment applied at the free end, as depicted in Figure 2.6. It is further assumed that the moment developed at the base of a given storey is fully transferred to the storey below.



**Figure 2.6: Cantilever beam assumption of wood shear walls**

According to the CSA O86 (Canadian Standards Association 2019), the inter-storey drift,  $\Delta_i^{storey}$ , at the  $i^{th}$  level for blocked shear walls is calculated using Eq. (2.10):

$$\Delta_i^{storey} = \Delta_{b,i}^{storey} + \Delta_{s,i}^{storey} + \Delta_{n,i}^{storey} + \Delta_{a,i}^{storey} \quad (2.10)$$

In Eq. (2.10)  $\Delta_{b,i}^{storey}$ ,  $\Delta_{s,i}^{storey}$ ,  $\Delta_{n,i}^{storey}$ ,  $\Delta_{a,i}^{storey}$  are different components of the lateral deflection of the shear wall, associated with bending of chord members, shear deformations of sheathing panel(s), nail slip and overturning, respectively. The total deflection,  $\Delta_i^{total}$ , at  $i^{th}$  storey can be obtained as follows, Eq. (2.11):

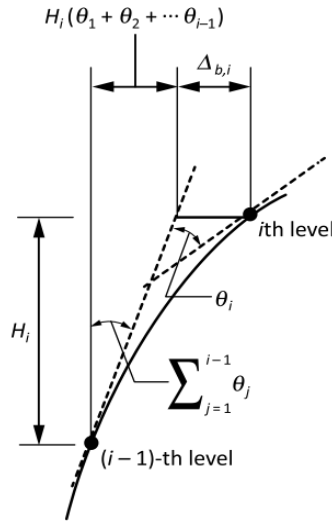
$$\Delta_i^{total} = \sum_{j=1}^i \Delta_j^{storey} \quad (2.11)$$

In calculating the inter-storey drift of individual storeys,  $\Delta_{s,i}^{storey}$  and  $\Delta_{n,i}^{storey}$  are treated the same as for single-storey shear walls, since these components contribute only to the shear behaviour and do not produce any cumulative effects.

On the other hand, the deflection due to bending deformations at the  $i^{th}$  storey,  $\Delta_{b,i}^{storey}$ , is calculated as the bending deflection at that storey plus the sum of the deflections resulting from the cumulative bending of all lower storeys, as given in Eq. (2.12):

$$\Delta_{b,i}^{storey} = \Delta_{b,i} + H_i \left( \sum_{j=1}^{i-1} \theta_j \right) \quad (2.12)$$

Where  $\theta_j$  is the change in the slope over the  $j^{th}$  storey height subjected to bending moment and horizontal shear force, as depicted in Figure 2.6, and  $\Delta_{b,i}$  is the horizontal distance from the top of the  $i^{th}$  storey to the tangent of the deflection curve at the bottom of the  $i^{th}$  storey, (see Figure 2.7).



**Figure 2.7: Displacement of  $i^{th}$  storey including the displacement due to the bending of the storey below (Canadian Standards Association 2019)**

According to the cantilever beam theory,  $\Delta_{b,i}$  and  $\theta_j$  are calculated using Eqs. (2.13) and (2.14), respectively.

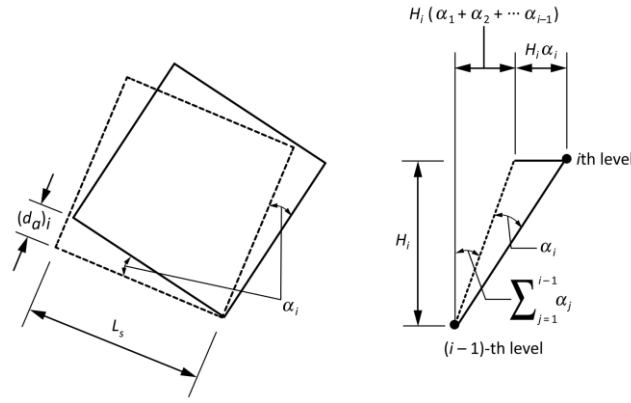


$$\Delta_{b,i} = \frac{M_i H_{s,i}^2}{2(EI)_i} + \frac{V_i H_{s,i}^3}{3(EI)_i} \quad (2.13)$$

$$\theta_i = \frac{M_i H_{s,i}}{(EI)_i} + \frac{V_i H_{s,i}^2}{2(EI)_i} \quad (2.14)$$

Here,  $M_i$  is moment at the top of  $i^{\text{th}}$  storey as a result of the overturning moments of the storeys above. Other parameters were defined previously.

The elongation of HD devices results in additional cumulative deformations over the height of the building as is illustrated in Figure 2.8.



**Figure 2.8: Deflection of a wood shear wall due to cumulative effects of HD elongations (Canadian Standards Association 2019)**

The corresponding shear wall deflection due to HD can be calculated using Eq. (2.15). The first term in Eq. (2.15) corresponds to the shear wall deflection due to HD elongation at the  $i^{\text{th}}$  storey, while the second term represents the cumulative effect of HD elongations of the storeys below.

$$\Delta_{a,i}^{\text{storey}} = H_{s,i} \left( \sum_{j=1}^i \alpha_j \right) = H_{s,i} \alpha_i + H_{s,i} \left( \sum_{j=1}^{i-1} \alpha_j \right) \quad (2.15)$$

Here,  $\alpha_i$  is the shear wall rigid body rotation angle, Eq. (2.16):

$$\alpha_i = \frac{(d_{a1} + d_{a2})_j}{L_s} \quad (2.16)$$

The cumulative effect of deformations over the height of multi-storey LFW buildings is commonly evaluated based on the assumption that a wood shear wall behaves as an isolated structural component stacked on top of each other, with no interaction with other members of the building. Accounting for cumulative deformations over the height of the building without considering

system-level interactions not only leads to an overestimation of inter-storey drifts but may also result in improper load distribution among different structural components, especially at the design stage. This, in turn, creates inconsistencies between the designed building and its actual structural behaviour.

Because the inter-storey drift of multi-storey buildings is obtained by extending the deflection formulation developed for single-storey shear walls, any conservatism inherent in the single-storey deflection equation, as discussed in the previous section, is directly propagated into the multi-storey inter-storey drift predictions.

Therefore, a more realistic representation of wood shear wall behaviour, one that accounts for system effects and realistic boundary conditions, can significantly improve design tools and lead to building designs that are more consistent with reality.

## **2.5 Experimental tests of wood shear walls**

The tests conducted by Patton-Mallory et al. (1985) indicated that for shear walls with ARs between 1 and 4, the stiffness exhibited a linear relationship to the shear wall length, while shear walls with low ARs (i.e., AR between 0.33 and 1.0) exhibited a nonlinear relationship with shear wall length.

Falk and Itani (1987) demonstrated that diaphragm stiffness is governed primarily by the sheathing-to-framing connections. Because these fasteners exhibit nonlinear behaviour, their stiffness decreases with increasing displacement, resulting in a corresponding increase in the period of shear walls and floor diaphragms.

He et al. (1999) showed that the door and window openings significantly reduced strength and stiffness, causing a shift in failure modes, especially in walls with oversize panels. The reduction

in strength and stiffness was proportional to the area or length of the opening in relation to the overall area or length of the shear wall.

Doudak et al. (2006) performed experimental testing and showed a marked reduction in both stiffness and strength of shear walls associated with large openings. Similar to He et al. (1999), the reduction was consistent with the relative length of the opening to the total length of the shear wall. The sheathing panel type and orientation were shown to have marginal influence on the shear wall strength and stiffness (Atherton, 1983; Dolan and Madsen, 1992; Durham et al., 2001; He et al., 1999; Serrette et al., 1997). The relationship between shear wall strength and stiffness in relation to the wall length was demonstrated to be proportional (Bagheri and Doudak, 2020a; Dolan, 1989; Salenikovich and Dolan, 2003b, 2003a), however, this proportionality did not hold for shear walls with large aspect ratios.

Bagheri et al. (2019) and Bagheri and Doudak, (2020b) conducted an extensive experimental investigation on wood shear walls to examine the effects of key parameters, including shear wall AR, HD type, sheathing panel type, and nail spacing. Sheathing panel shear deformation was found to be negligible for all specimens. The strength and stiffness of the shear walls exhibited an inverse relationship with wall AR. The results showed that the bending deformation of the chord members was not consistent with the corresponding deformation predicted by CSA O86 (Canadian Standards Association 2019), especially at high load levels. It was noted that the bending deformations of the chords was affected by the rigid body rotation of the sheathing panel and nail deformations. It was shown that the formulation in CSA O86 (Canadian Standards Association 2019) underestimates the shear wall deflection due to nail deformations.

Bagheri et al. (2019) and Bagheri and Doudak, (2020b) also performed half-scale two-storey stacked shear wall tests. In these tests, no floor diaphragm was provided at the interface between

storeys, and the shear walls were stacked directly on top of each other. The test results indicated that the displacement at the second storey closely followed the cumulative displacement assumption in CSA O86 (Canadian Standards Association 2019). The cumulative HD elongation was identified as the primary source of this displacement accumulation.

## **2.6 Experimental Tests on Floor Diaphragms**

Investigations of LFW floor and roof diaphragm behaviour have mainly focussed on the in-plane characteristics of these structural elements (Bott, 2005; Corda, 1982; GangaRao and Luttrell, 1979; Jewell, 1981; Tissell and Elliott, 1977). Corda (1982) and Bott (2005) showed that blocking is the most effective parameters in increasing the diaphragm shear stiffness. Bott (2005) noted that adding blocking to diaphragms reduced the damping ratio, and the presence of diaphragm chord members, although had a significant impact on the flexural stiffness, had a moderate impact on the diaphragm shear stiffness. The study showed that the presence of shear wall increased both shear and flexural stiffness of the diaphragms.

Filiatrault et al. (2002) tested a full-scale two-storey LFW building under different construction stages. Similar to previous studies, blocking was found to be one of the most significant factors in increasing the shear stiffness of the diaphragm. Further increase in diaphragm shear stiffness was attained when the subfloor adhesive was combined with the blocking. The study also noted that addition of the second storey increased the first-floor diaphragm flexural stiffness. Bott (2005) Skaggs et al. (2008) tested a series of one-storey 3D LFW shear wall buildings using monotonic loading. The intended goal was to simulate the in-plane behaviour of the floor diaphragm. To minimize the contribution of the floor out-of-plane stiffness to the shear wall resistance, discontinuous rim boards above the shear wall were utilized, as depicted in Figure 2.9.



**Figure 2.9: Discontinuous rim board above shear wall (Skaggs et al. 2008).**

Based on the reviewed literature, none of the previous studies have evaluated the effect of out-of-plane stiffness of the floor diaphragms on the response of LFW buildings.

## **2.7 Experimental and numerical studies on the global behaviour of LFW shear walls**

van de Lindt et al. (2010) experimentally studied the seismic behaviour of a six-storey LFW building. The full-scale building was tested on a shake table in Japan. Results from the test revealed that distribution of HD forces was not consistent at both ends of the shear walls. This inconsistency was thought to be as a result of the contribution of the floor system and the boundary conditions at each end of the shear wall where two shear walls intersect. Furthermore, the measured HD force and elongation demands showed a lower effective stiffnesses compared to theoretical values. The maximum inter-storey drift occurred in the upper storeys, with the cumulative effect of HD elongations contributing to the overall storey drift of these storeys.

Steiger et al. (2017) investigated the dynamic properties of a three-storey LFW building using AVT and forced vibration methods. The forced vibration tests were conducted over a wide excitation frequency range of 0.2–14 Hz. The AVT results yielded higher natural frequencies compared to those obtained from the forced vibration tests. This difference was attributed to the

presence of initial static friction in the building, which resulted in higher initial stiffness and, consequently, higher measured frequencies. Furthermore, the forced vibration tests showed that the natural frequencies decreased with increasing excitation amplitude. It was reported that higher excitation levels led to a 10–20% reduction in structural stiffness, highlighting the influence of displacement amplitude on the effective stiffness of the building.

Hafeez et al. (2014) investigated the vibration period of LFW buildings using ambient vibration testing (AVT) and numerical analysis. The AVT results indicated that the measured building periods were consistent with those calculated using the four-term deflection equation in CSA O86 (Canadian Standards Association 2019), without considering cumulative displacement effects. The numerical analyses of multi-storey buildings showed that incorporating cumulative displacement effects significantly increased the predicted building period, causing it to exceed the upper-bound period specified in NBC 2020 (Canadian Commission on Building and Fire Codes 2020).

Pei and van de Lindt (2009) numerically investigated the effect of cumulative displacements of an experimentally tested three-storey stacked shear wall. One important aspect of this study was the omission of the floor system in the stacked shear wall. Due to this omission, the results showed that the drifts of upper storeys are affected by cumulative displacements due to HD elongations of the storeys. The HD force and deformation measurements showed a lower tension stiffness compared to the theoretical value.

Pei and Van De Lindt (2011) numerically investigated the seismic performance of a full-scale six-storey LFW shear wall building. The numerical model used in the study was able to reasonably capture response quantities such as inter-storey drifts, storey accelerations, etc. However, it was noted that the accuracy of the model in predicting HD forces varied. It was noted that the boundary condition of the shear walls could be a reason why the HD force predictions differed from

measured ones. The prediction of base shear force and torsional response of the building model was not consistent with the measured responses. It is worth mentioning that the effective stiffness of the HDs in the numerical model was assumed to be 50% of the theoretical value to account for the realistic representation of the HDs as observed in previous studies (van de Lindt et al. 2010; Pei et al. 2010). Yet, even with this modification, the HD forces and elongations were not entirely consistent with the measured data.

Bagheri et al. (2019) numerically investigated the effect of out-of-plane stiffness of floor diaphragm on the lateral displacement of multi-storey stacked shear walls. The results showed a considerable reduction in shear wall deflections when considering the out-of-plane rigidity of the floor diaphragm. In other words, the cumulative effect of lateral displacements of lower storeys on the drift of the storeys above was significantly reduced by out-of-plane stiffness of the floor system.

## **2.8 Concluding remarks**

The reviewed literature demonstrates that significant progress has been made in understanding and modelling the strength and stiffness of LFW shear walls at both the component and system levels. Based on the literature reviewed in this document, analytical, FE-based, and mechanics-based models have been shown to provide reasonable predictions of shear wall response for idealized configurations and simplified boundary conditions.

However, the influence of realistic boundary conditions, such as floor diaphragm restraint, shear walls with openings, multi-segment shear walls, shear walls with different types of HDs, and wall-to-floor interactions have not yet been fully addressed. Current design frameworks do not explicitly account for the effects of floor diaphragm out-of-plane stiffness, nor do they capture how variations in boundary conditions change the distribution of internal forces and cumulative deformations in multi-storey shear wall systems.

Cumulative deformations can significantly affect the predicted period of LFW buildings. In particular, displacement accumulation associated with HD elongation suggests that cumulative effects may play an important role in multi-storey systems. However, the magnitude of this cumulative effect remains to be clarified. Therefore, reliable characterization and quantification of cumulative displacement mechanisms are essential for accurately evaluating building stiffness and predicting the vibration period.

Experimental studies on floor diaphragms have primarily focused on in-plane behaviour, while the out-of-plane stiffness of floor systems and its role in constraining shear wall kinematics has received little direct experimental investigation. Nevertheless, numerical studies indicate that floor out-of-plane stiffness can significantly influence shear wall deflections and inter-storey drift accumulation, particularly in stacked shear walls. These findings suggest that neglecting such system-level interactions may lead to inaccurate drift predictions and representations of load sharing among structural components.

Motivated by these gaps, the present research aims to experimentally investigate the effects of floor diaphragm out-of-plane stiffness, openings with different length, different types of HD, multi-segment shear walls, aspect ratio, and varying boundary conditions on the lateral behaviour of wood shear walls. The experimental findings are further employed to advise the development of a numerical modelling framework capable of capturing the essential mechanics of wood shear walls subjected to realistic boundary conditions.



## Chapter 3

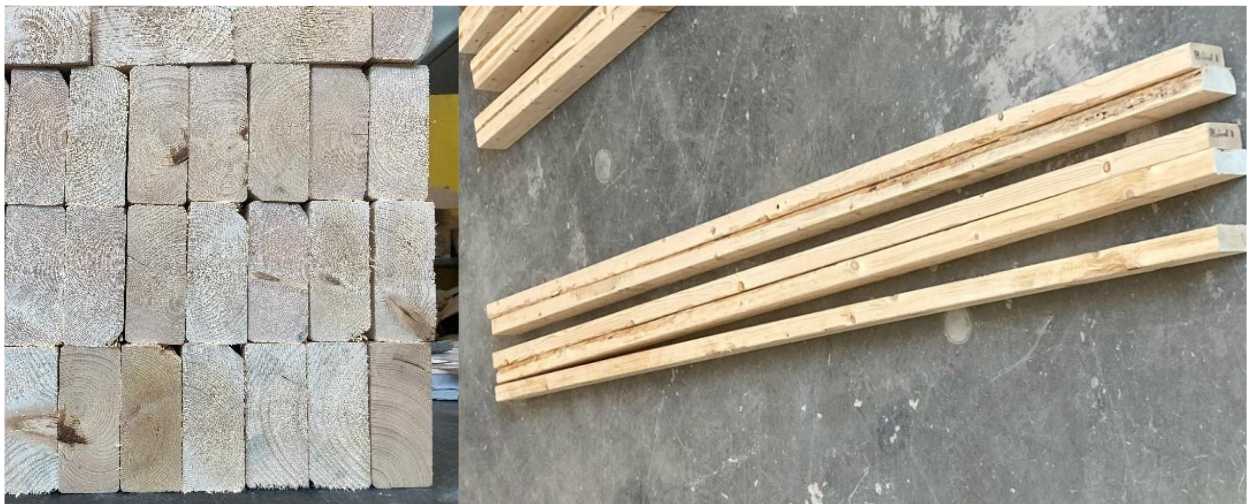
### Experimental program

#### 3.1 Introduction

In this chapter, the materials employed in the experimental program are first described. The test specimens selected for this study are then introduced, along with the rationale underlying each test configuration. Subsequently, the test setups for the full-scale shear wall tests, floor assembly tests, and connection tests are presented. The corresponding test matrix for each category of tests is provided within the respective sections. The instrumentation layout for the shear wall tests is then described, along with the characteristics of the measuring devices and their roles in recording the experimental data.

#### 3.2 Materials

Canadian visually-graded Spruce-Pine-Fir (S-P-F) lumber of No. 2 Grade was used for the framing members of the shear walls in this study (Figure 3.1).

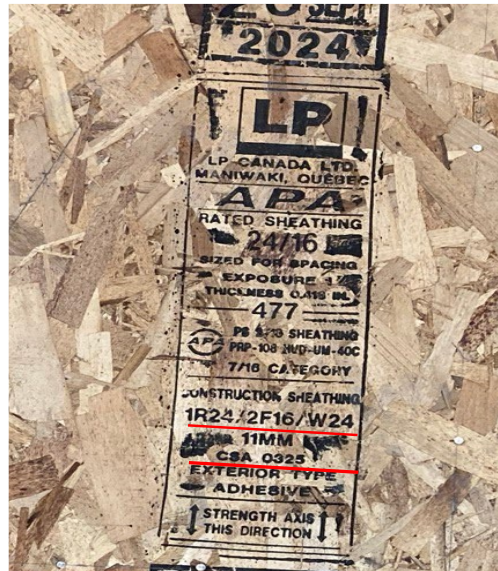


**Figure 3.1: Spruce-Pine-Fir lumber**

Although  $2 \times 6$  in. ( $38 \times 140$  mm) lumber elements are usually used in LFW wall construction, primarily due to its ability to resist axial load, when needed, and for energy consideration (deeper

space for insulation batts), 2 × 4 in. (38 × 89 mm) framing elements were used due to ease of handling and the fact that framing depth has no significant impact on the shear wall's in-plane shear performance. This layout represents the most widely used practice in the vast majority of Canadian LFW construction. All lumber elements were purchased from a local lumber yard, and nearly all shear walls were constructed using the same batch of material. All lumber was stored in the laboratory before testing, and the moisture content (MC) at the time of wall assembly was below 12%.

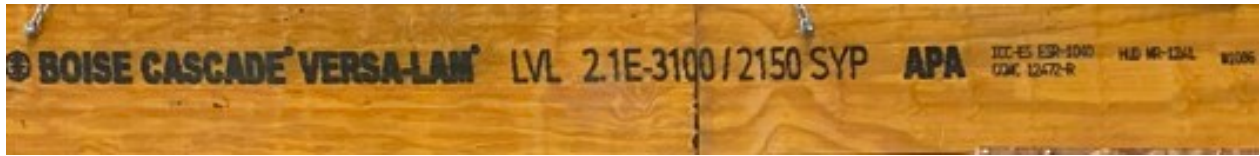
Oriented Strand Board (OSB) sheathing panels were used to cover shear walls and provide the in-plane stability (Figure 3.2). The size of the panels was a standard size of 4 × 8 ft (1220 × 2440 mm). For this study, a single panel thickness of 11 mm was used. This thickness is selected because it is widely used in practice as well as commonly adopted in existing experimental research. The sheathing panels were APA-rated and classified as grade 1R24/2F16 in accordance with the CSA O325 (Canadian Standard Association 2021).



**Figure 3.2: OSB sheathing panel**

Southern Yellow Pine (SYP) Laminated Veneer Lumber (LVL) joists (Figure 3.3) were used in this experimental program as a floor assembly in order to represent the floor's out-of-plane flexural

stiffness above the shear walls. The LVL joist had nominal dimensions of  $11\text{-}7/8 \times 1\text{-}3/4$  in. ( $301.6 \times 44.45$  mm). Usually, single or multiple LVL joists are used as rim joists on the perimeter of the building, and they connect the bottom plate from the upper storey to the double top plates from the lower storey.



**Figure 3.3: Laminated Veneer Lumber (LVL) joist**

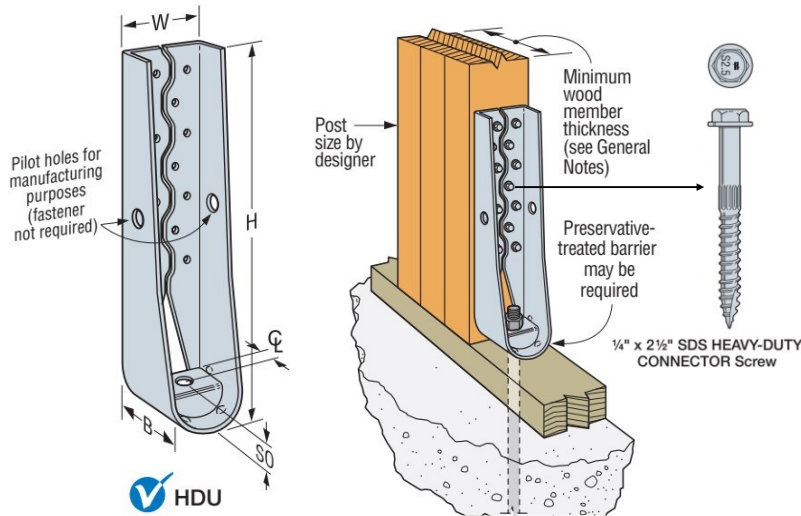
As shown in Figure 3.3, the designation 2.1E represents the modulus of elasticity of  $2.1 \times 10^6$  psi (approximately 14500 MPa). The values 3100/2150 psi denote the allowable bending stress and allowable tension stress parallel to grain, respectively, corresponding to approximately 21.3 and 14.8 MPa.

Common wire nails of different sizes were used in this study, and their specifications are based on ASTM F1617 (ASTM F1667 2022). 8d common wire nails were used to connect the sheathing panel to the framing elements, while 10d common wire nails were used to connect multiple framing members, such as shear wall chord members (end studs) and shear wall top plates. Top- and bottom plates to vertical stud connections were formed using 16d common wire nails. The specifications of the fasteners are summarized in Table 3-1.

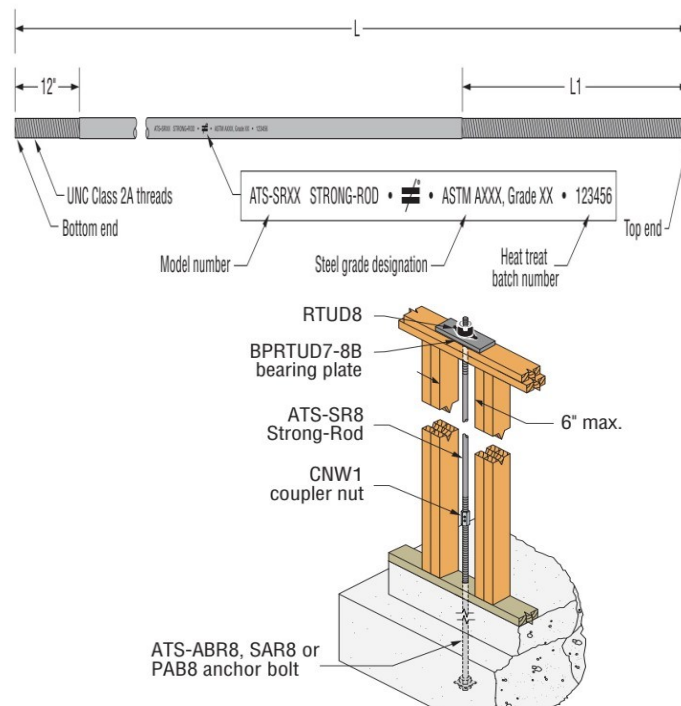
Table 3-1: Fastener specifications

| Nail size | Nail diameter (mm) | Nail length (mm) |
|-----------|--------------------|------------------|
| 8d        | 3.33               | 63.5             |
| 10d       | 3.76               | 76.2             |
| 16d       | 4.11               | 88.9             |

Two types of anchorage systems were used in this study: discrete HD devices (Figure 3.4) and Anchor Tie-down System (ATS) (Figure 3.5). The former has traditionally been used in LFW construction, especially for low-rise buildings, while the latter has been introduced as a solution to resist overturning moments primarily in mid-rise LFW buildings.



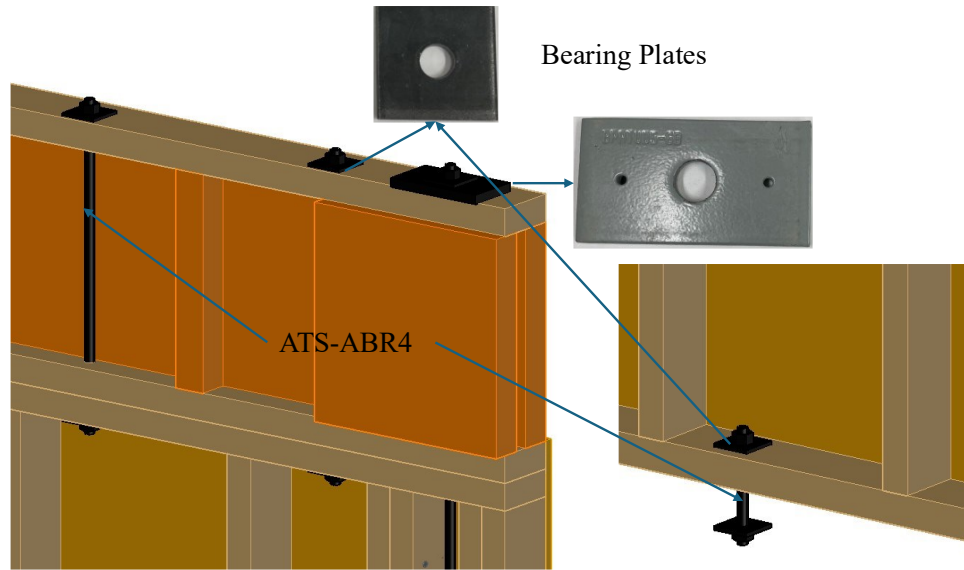
**Figure 3.4: Discrete HD system (Simpson Strong-Tie 2022)**



**Figure 3.5: Anchor Tie-down System (Simpson Strong-Tie 2022)**

For the shear wall tests, Simpson Strong-Tie (Simpson Strong-Tie 2022) HDU5 devices were used to represent traditional HD devices, while for the ATS system, continuous ATS-SR4 and ATS-SR5 rods were employed. In addition to HDU5, HDU2 and HDU4 devices were used separately in the HD connection tests to determine their mechanical properties, and to establish a better understanding of the actual expected behaviour of these HD devices. Simpson Strong-Tie ABs

(ATS-ABR4) were used at the base and floor levels of the shear walls (see Figure 3.6). It is noted that the specification of ATS-ABR4 is similar to that of ATS-SR4.



**Figure 3.6: Bearing plates and anchor bolts**

To minimize the effects of wood crushing and to facilitate load transfer over a larger area of the wood, bearing plates (BP) were used at the AB locations as well as at the top of the walls where the ATS rods terminate, as depicted in Figure 3.6.

The specifications of the anchorage devices are summarized in Table 3-2.

Table 3-2: Anchorage devices specifications

| Device Name        | Dimensions (in)  |                       |                  |                   |                   | AB diameter (in) | Fasteners          | Factored Tensile Resistance (kN) | Deflection at Factored Tensile Resistance (mm) |
|--------------------|------------------|-----------------------|------------------|-------------------|-------------------|------------------|--------------------|----------------------------------|------------------------------------------------|
|                    | <i>W</i><br>(in) | <i>H (L*)</i><br>(in) | <i>B</i><br>(in) | <i>CL</i><br>(in) | <i>SO</i><br>(in) |                  |                    |                                  |                                                |
| <b>HDU2-SDS2.5</b> | 2 7/8            | 8 11/16               | 3 1/4            | 1 3/8             | 1 3/8             | 5/8              | (6) 1/4" x 2 1/2"  | 12.9                             | 2.3                                            |
| <b>HDU4-SDS2.5</b> | 2 7/8            | 10 15/16              | 3 1/4            | 1 3/8             | 1 3/8             | 5/8              | (10) 1/4" x 2 1/2" | 21.49                            | 3.3                                            |
| <b>HDU5-SDS2.5</b> | 2 7/8            | 13 3/16               | 3 1/4            | 1 3/8             | 1 3/8             | 5/8              | (14) 1/4" x 2 1/2" | 28.43                            | 3.89                                           |
| <b>ATS-SR4</b>     | -                | 111                   | -                | -                 | -                 | 1/2              | -                  | 27.42                            |                                                |
| <b>ATS-SR5</b>     | -                | 111                   | -                | -                 | -                 | 5/8              | -                  | 43.36                            |                                                |
| <b>ATS-ABR4</b>    | -                | 6                     | -                | -                 | -                 | 1/2              | -                  | 24.59                            |                                                |
| <b>BP 1/2</b>      | 1/2              |                       | 1/2              |                   |                   |                  |                    |                                  |                                                |
| <b>WBP6-3x5.5</b>  | 3                |                       | 5 1/2            |                   |                   |                  |                    |                                  |                                                |



### 3.3 Fabrication of test specimens

All shear walls were fabricated manually in the Structural Laboratory at the University of Ottawa (see Figure 3.7).



**Figure 3.7: Shear wall fabrication**

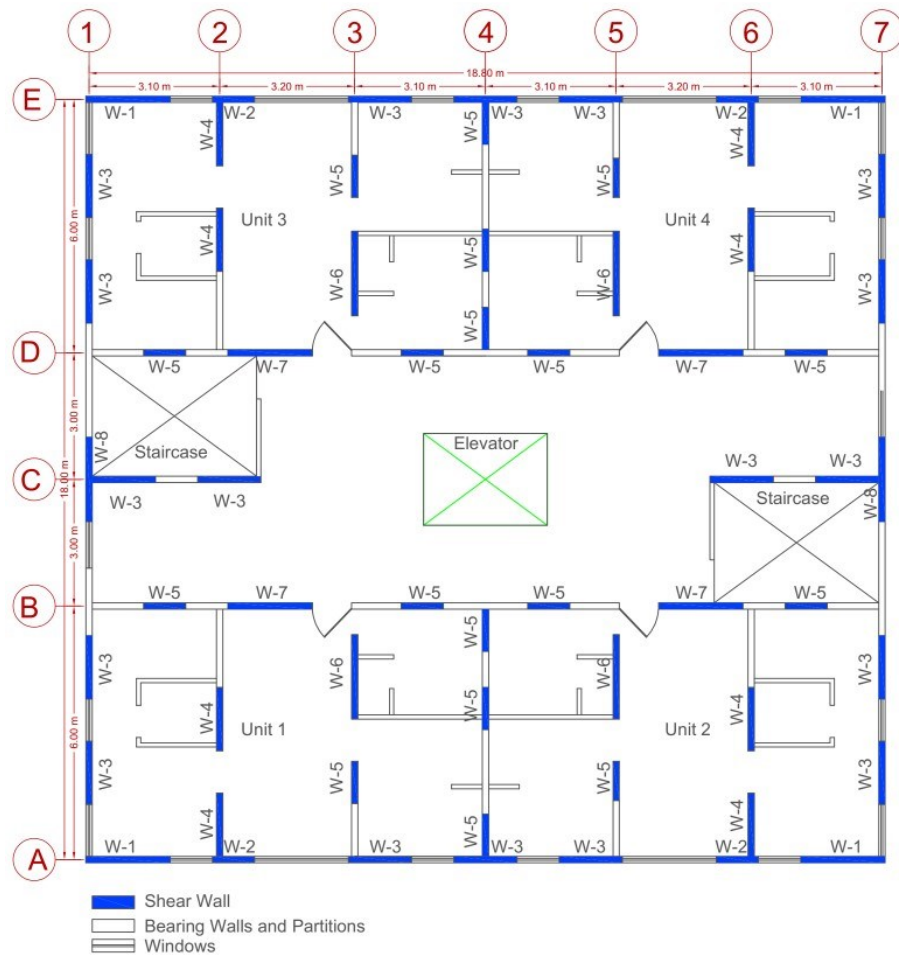
All lumber and sheathing panels were cut and assembled in situ, and nails were driven manually using a hammer (Figure 3.7(a)–(c)). For shear walls incorporating more than one sheathing panel on one side of the wall, a 2 mm gap was provided to prevent contact between adjacent panels (Figure 3.7(d)). Perimeter nails were installed with an edge distance of 9 mm, which corresponds to the minimum edge distance required by CSA O86 (Canadian Standards Association 2019).

For shear walls equipped with ATS continuous rods, as depicted in Figure 3.7(e) and (f), the nail pattern at the vertical edges of the sheathing panel at the rod locations differed from the standard configuration. Since CSA O86 (Canadian Standards Association 2019) does not specify

requirements for this condition, the nail pattern at these locations was adopted based on the recommendations provided in the Simpson Strong-Tie catalogue (Simpson Strong-Tie 2022).

### 3.4 Shear wall test specimens

This section describes the shear wall configurations investigated in the experimental program. It should be noted that the selection of the shear wall configurations was based on a realistic plan layout (Figure 3.8) of a three-storey LFW building archetype (Morshedi et al. 2024).



**Figure 3.8: Plan layout of the 3-storey LFW building (Morshedi et al. 2024)**

Since the focus of the present study was investigating factors such as out-of-plane flexural stiffness of the floor, shear wall AR, multi-segment shear walls with openings of varying lengths, and shear walls with different boundary conditions, the arrangement of shear walls within the building plan

was selected to represent a range of scenarios. These include different floor joist orientations (floor joists perpendicular to the shear wall, resulting in blocking, versus floor joists parallel to the shear wall, resulting in continuous LVL members above the wall), boundary conditions, the presence of multiple shear wall segments along a shear line, openings of varying lengths, and shear walls with different ARs.

As noted in Section 3.2, all framing members and sheathing panel thickness were consistent for all fourteen shear walls.

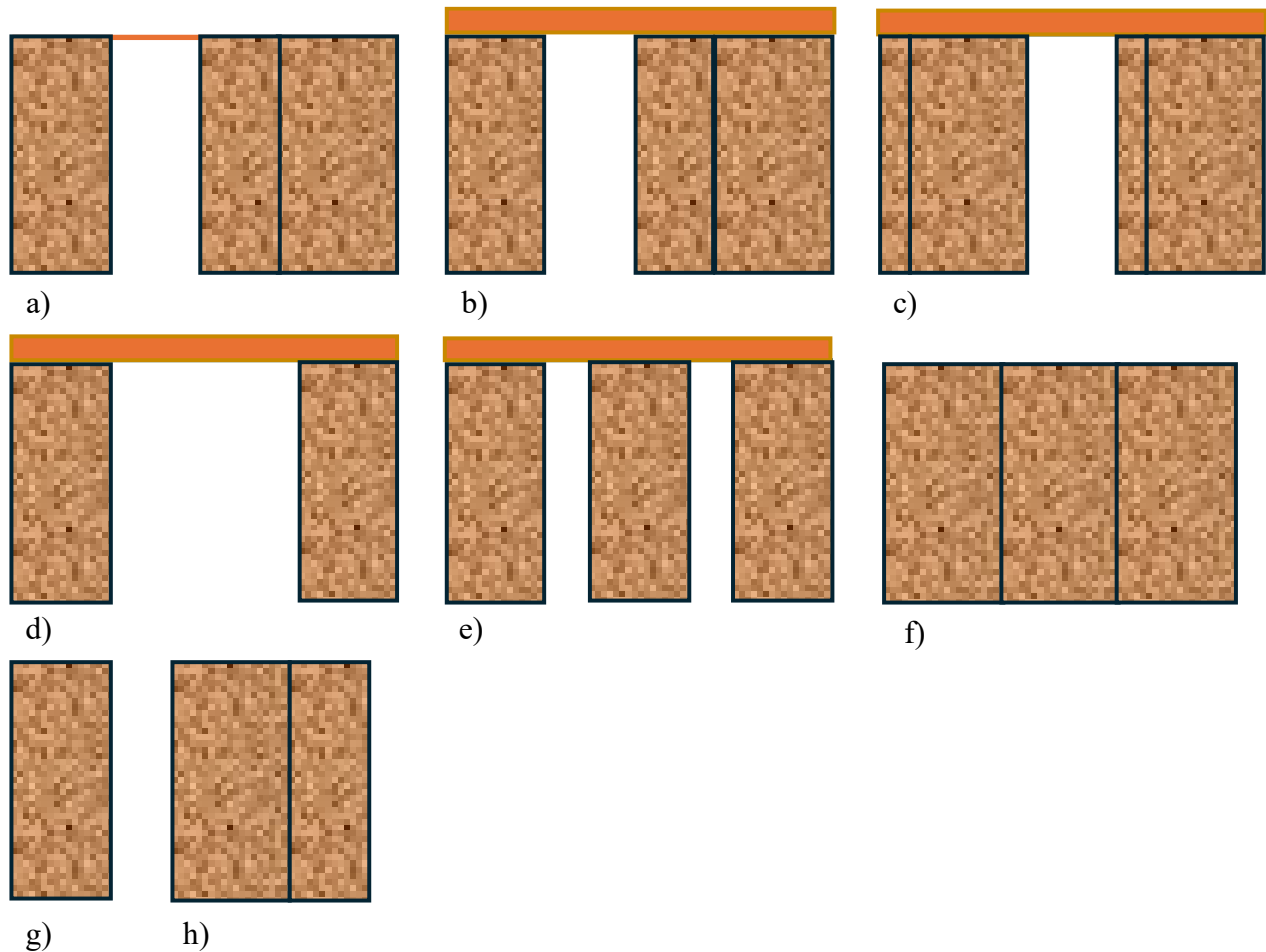
The nail spacing for all shear walls was selected to be 100 mm, which is a typical nail spacing often used in LWF shear walls, except for one shear wall, where the nail spacing was chosen to be the minimum allowed in the CSA O86 (Canadian Standards Association 2019) standard, namely 50 mm. The intermediate (field) nail spacing was 300 mm.

Floor blocking, as well as single and double LVL joists, were used above the shear walls to represent floors with varying out-of-plane flexural stiffness. Detailed explanations of each configuration are provided in later sections. All wall configurations had a height of 2.44 m. For configurations incorporating the floor assembly, an additional height floor depth of 340 mm is included. The total length of all multi-segment shear walls was 4.0 m.

The vertical (intermediate) studs were spaced at 400 mm O.C., except 1.0 m shear walls in which a single stud was used in center. The shear wall included  $2 \times 4$  in. ( $38 \times 89$  mm) studs at both ends serving as chord members, connected using two rows of 10d common nails at 220 mm O.C., staggered along the length of the chords, as shown in Figure 3.10. This nailing pattern complies with Sentence 6.5.6.4.2 of the CSA O86 standard (Canadian Standards Association 2019) for built-up columns design requirements. Each shear wall was equipped with two HDU5 devices attached to the chord members using Strong-Drive Screws (SDS).



Figure 3.9 presents the eight shear wall geometries, comprising a total of fourteen test configurations, included in the experimental program.

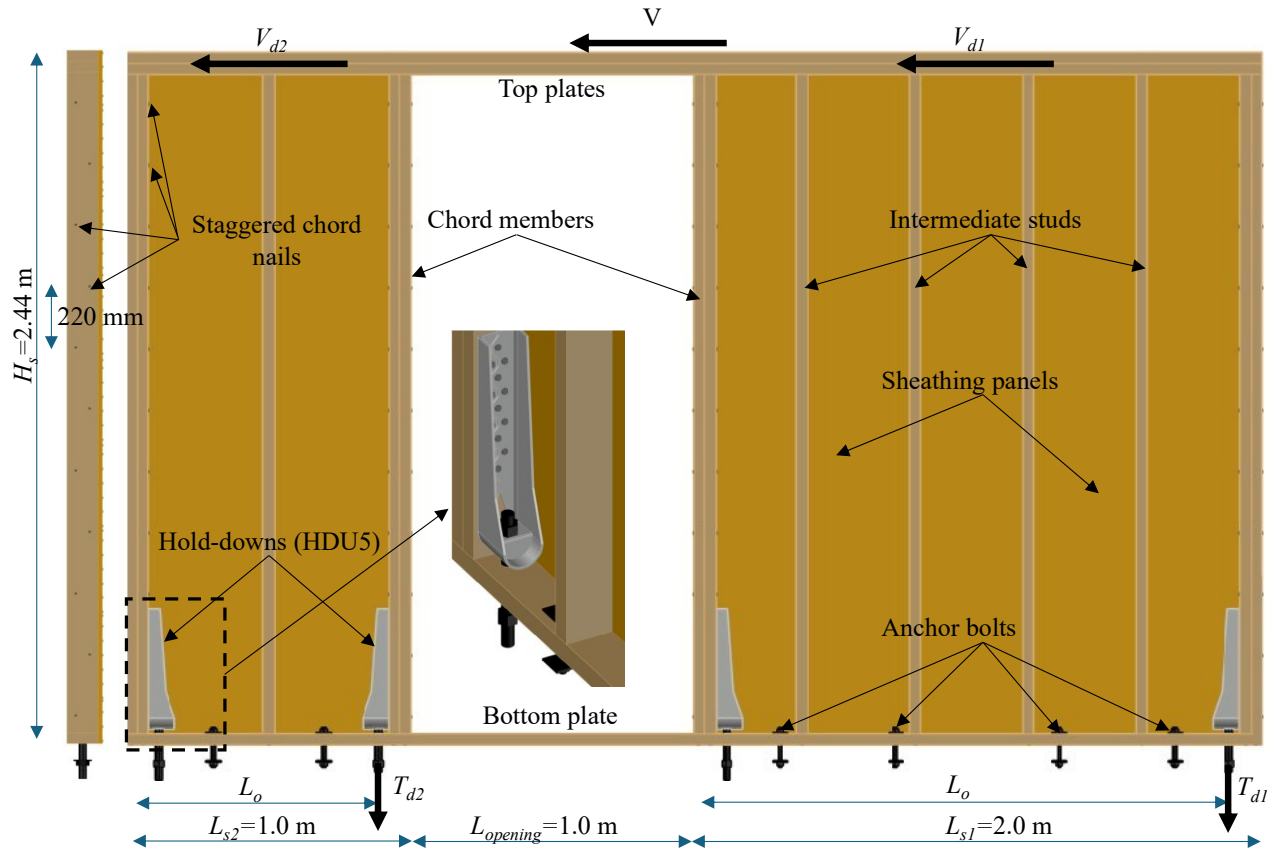


**Figure 3.9: Geometries of the shear wall test configurations: (a) Configuration #1; (b) Configuration #2, Configuration #5, Configuration #6, Configuration #13, and Configuration #14; (c) Configuration #3; (d) Configuration #4 and Configuration #9; (e) Configuration #7 and Configuration #8; (f) Configuration #10, Configuration #11 and Configuration #12; (g) Configuration #10, Configuration #11 and Configuration #12; (h) Configuration #10, Configuration #11 and Configuration #12**

### 3.4.1 Configuration #1c

This configuration (Figure 3.9(a)) comprised two shear walls, with respective lengths of 2.0 m and 1.0 m, separated by a 1.0 m opening. This configuration is considered to be a reference case, since no floor assembly is present above the walls; as such, the effect of floor out-of-plane stiffness on

the behaviour of the shear wall can be characterized and investigated in relation to other walls. A detailed depiction of Configuration #1 can be seen in Figure 3.10.



**Figure 3.10: Shear walls in Configuration #1**

The intent of this configuration was to study the force- and deformation demands where two shear walls with different ARs and with an opening in-between are connected through continuous top plates. The recorded forces and deformations of this shear wall were then considered as a basis for comparison with those configurations that include, for example, floor effects.

The factored shear resistance,  $V_{rs}$ , of the shear walls in this configuration is 7.9 kN/m, as calculated by Sentence 11.1.5.2 in the CSA O86 (Canadian Standards Association 2019). The total factored shear resistance of the configuration is 23.7 kN. The distribution of forces between individual shear walls ( $V_{di}$ ) was based on their relative stiffness to the total stiffness of the shear line. For this purpose, WoodWorks® software was used to determine the forces and deformations of the

individual shear walls. WoodWorks® evaluates the shear wall stiffness using the four-term deflection equation, specified in Chapter 2 (Eq. 2.3).

Individual HDs in each shear wall were designed based on a 20% increase in the corresponding  $V_{di}$  value of each shear wall, consistent with Sentence 11.8.2 of CSA O86 (Canadian Standards Association 2019). The individual HD design force ( $T_{di}$ ) was calculated as the product of the  $V_{di}$  value and the shear wall height ( $H_s$ ), divided by the overturning length ( $L_o$ ) of the wall, as shown in Figure 3.10. The latter is the distance between the tension HD anchor and center of the compression chord.

Based on the individual HD forces obtained from the analysis, the Simpson Strong-Tie HDU5 was deemed satisfactory. Table 3-3 summarizes the design information for the individual shear walls and their corresponding HDs in Configuration #1.

Table 3-3: Shear wall design information-Configuration #1.

| Shear Wall ID | $L_s$ (mm) | $V_{rs}$ (kN) | $V_d$ (kN) | (D/C) <sub>sw</sub> | $L_o$ (mm) | $T_r$ (kN) | $T_d$ (kN) | (D/C) <sub>HD</sub> |
|---------------|------------|---------------|------------|---------------------|------------|------------|------------|---------------------|
| 1             | 2000       | 15.8          | 17.4       | 1.1                 | 1851       | 28.4       | 27.5       | 0.97                |
| 2             | 1000       | 7.9           | 6.3        | 0.8                 | 851        | 28.4       | 21.7       | 0.76                |

$L_s$ : Shear wall length;  $V_{rs}$ : Factored shear resistance;  $V_d$ : Design shear force; (D/C)<sub>sw</sub>: Shear wall demand-to-capacity ratio;  $L_o$ : Overturning length;  $T_r$ : HD factored tensile resistance;  $T_d$ : HD design force; (D/C)<sub>HD</sub>: HD demand-to-capacity ratio

Using the information in Table 3-3 and Eq. (2.3), the deflection of each shear wall in Configuration #1 was calculated and presented in Table 3-4.

Table 3-4: Shear wall deflections-Configuration #1.

| Shear wall ID | $e_n$ (mm) | $d_{a1}$ (mm) | $d_{a2}$ (mm) | $\Delta_b$ (mm) | $\Delta_s$ (mm) | $\Delta_n$ (mm) | $\Delta_{a1}$ (mm) | $\Delta_{a2}$ (mm) | $\Delta_t$ (mm) |
|---------------|------------|---------------|---------------|-----------------|-----------------|-----------------|--------------------|--------------------|-----------------|
| 1             | 1.0        | 3.15          | 0.41          | 0.66            | 1.9             | 6.4             | 4.15               | 0.54               | 13.6            |
| 2             | 0.54       | 2.46          | 0.33          | 0.95            | 1.4             | 3.3             | 7.05               | 0.94               | 13.6            |

$e_n$ : Nail deformation;  $d_{a1}$ : HD elongation;  $d_{a2}$ : Compression perpendicular to grain deformation;  $\Delta_b$ : Shear wall deflection due to bending;  $\Delta_s$ : Shear wall deflection due to sheathing panel shear deformation;  $\Delta_n$ : Shear wall deflection due to nail deformations;  $\Delta_{a1}$ : Shear wall deflection due to HD elongation;  $\Delta_{a2}$ : Shear wall deflection due to compression perpendicular to grain deformation of the bottom plate



In multi-storey buildings, the ABs transfer shear forces from upper storeys to lower storeys. In this study, however, in order to investigate the effect of floor out-of-plane flexural stiffness on the force and deformation demands of the shear wall, these ABs were used to provide a confinement for the LVL joist to engage its flexural stiffness. This scenario is representative of cases in which floor joists run parallel to the shear walls. A similar restraint mechanism may also occur when shear walls are located at the edge of a building, where single or multiple joists typically run over the shear walls.

Since CSA O86 (Canadian Standards Association 2019) does not account for the effect of the floor in the calculation of shear wall force and deformation demands, the values corresponding to Configuration #1 presented in Table 3-3 and Table 3-4 are also applicable to this configuration.

### **3.4.3 Configuration #3**

This configuration included two identical shear walls, each measuring 1.5 m in length, with a 1.0 m opening, as shown in Figure 3.9(c). The floor assembly included a single LVL joist which was placed on top of the shear walls and fastened in place using ABs. The intent of this configuration was to investigate the influence of different boundary conditions imposed by floor out-of-plane flexural stiffness on shear walls with identical ARs ( $AR = 1.6$ ). Previous research on full-scale building tests has reported inconsistent HD force demands at the two ends of shear walls (van de Lindt et al. 2010), and therefore, the present study aimed to investigate how boundary conditions surrounding the HDs impact their force and deformation demands.

Another objective of this configuration was to investigate the application of a very narrow sheathing panel at one side of each shear wall. In practice, the use of very narrow sheathing panels is unavoidable. For example, for a wall length of 1.5 m and standard sheathing panel dimensions of  $1220 \times 2440$  mm, a narrow panel with a width of approximately 280 mm is required. However,

it remains unclear whether the efficiency of such narrow panels is similar to that of larger panels. Currently, there is no requirement in CSA O86 (Canadian Standards Association 2019) standard specifying a maximum aspect ratio for sheathing panels in walls composed of multiple panels.

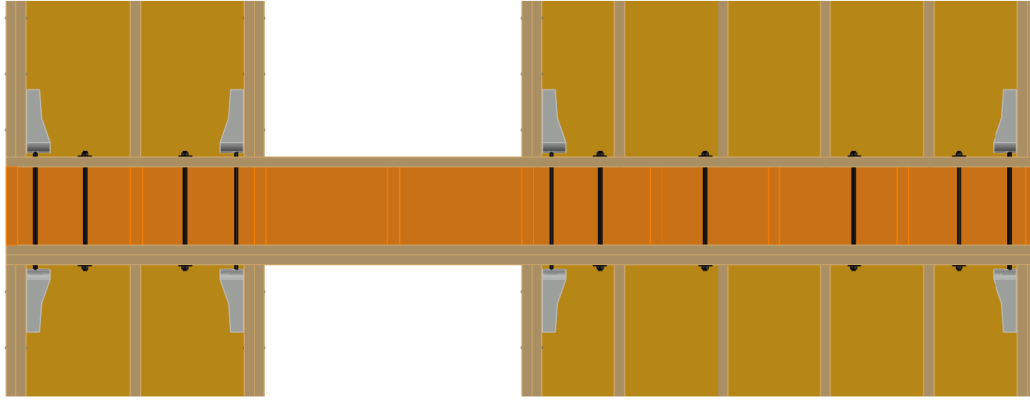
#### **3.4.4 Configuration #4**

This configuration corresponds to the geometry shown in Figure 3.9(d) and comprised two identical, high-AR shear walls, each measuring 1.0 m in length ( $AR = 2.4$ ), separated by a 2.0 m opening. The intent of this configuration is to investigate the influence of floor out-of-plane flexural stiffness on the force and deformation demands of high-AR shear walls containing a long opening. To represent the floor effect, a single LVL joist was installed at the top of the shear walls and securely fastened to the walls below using ABs.

The deflection response of shear walls with such high aspect ratios is commonly assumed to be governed by rocking behaviour resulting from HD elongation and bending deformation of the chord members. While the rocking-dominated response may have a limited impact on deflection calculations for low-rise buildings, it can significantly influence deflection estimates in mid-rise buildings, where cumulative deformations over the height of the building become more pronounced. Accordingly, this configuration was considered to assess whether a high-AR shear wall would still exhibit rocking behaviour when coupled with a floor assembly.

#### **3.4.5 Configuration #5**

The geometry of this configuration was similar to that of Configuration #2, with the only difference being the wall-to-floor connection details. In addition to the ABs, HD devices were also added at both ends (i.e., top and bottom) of the shear walls at the floor level, as depicted in Figure 3.12. This configuration represents a scenario, where some restraints is provided from shear wall at the upper storey.



**Figure 3.12: Inter-storey floor connection details**

The presence of HD connections at the floor level is expected to provide additional confinement to the floor assembly, represented here by the LVL joist, thereby potentially increasing its out-of-plane flexural rigidity. This investigation examined whether this increased rigidity violates the common assumption that a shear wall behaves as a cantilever beam with a free end.

#### **3.4.6 Configuration #6**

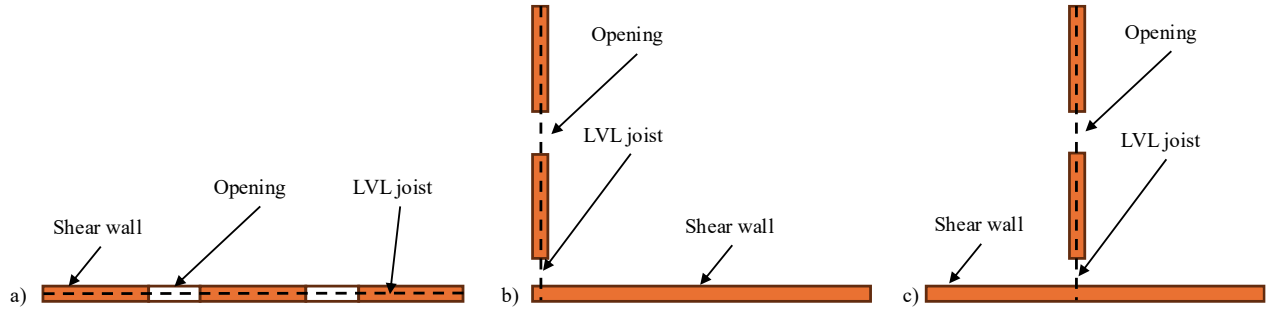
This configuration is similar to Configuration #5, except that two LVL joists were placed on top of the shear walls. The selection of this configuration is motivated by the fact that an actual floor system may consist of multiple rim-joist components, causing higher levels of out-of-plane flexural stiffness of the floor.

#### **3.4.7 Configuration #7**

This configuration comprised three identical high-AR shear walls ( $AR = 2.4$ ), each measuring 1.0 m in length, separated by two short openings of 0.5 m. A single LVL joist was used to represent the effect of the floor out-of-plane stiffness on the shear walls behaviour (Figure 3.9(e)).

The selection of this configuration was motivated by the question of whether high-AR shear walls, whose response is commonly assumed to be dominated by HD uplift (rocking behaviour) and chord member bending, behave similar to a single shear wall in isolation without any constraint from boundary conditions. Such conditions may arise when high-AR shear walls are separated by very

short openings (Figure 3.13(a)), when a shear wall is located near a building corner and connected to another wall in the perpendicular direction through the floor system (Figure 3.13(b)), or when a shear wall intersects with another shear wall through the floor system (Figure 3.13(c)).



**Figure 3.13: Different shear wall boundary conditions**

Figure 3.13(b) and (c) show that the uplift of the narrow shear wall is influenced not only by its own HD connection, but also by the restraint imposed by the perpendicular shear wall. The HD of the perpendicular shear wall provides additional restraint against uplift for the narrow shear wall. Similarly, in Figure 3.13(c), the uplift behaviour of the narrow shear wall is affected not only by its own HD connection but also by the axial stiffness of the perpendicular shear wall, which provides additional restraint against uplift.

Although the configuration examined in this study does not include perpendicular walls, it is believed that the small openings simulate conditions similar to those illustrated in Figure 3.13 (b) and (c).

### 3.4.8 Configuration #8

This configuration was similar to Configuration #7, except that the continuous LVL joist was replaced by discrete blockings. This represents a situation where the floor joists run perpendicular to the shear wall plane. In such situation, blocking is required between adjacent floor joists to transfer shear forces from the floor system to the shear walls below and to form a continuous load path. This configuration was also considered to examine how a floor assembly with blocking



should be simulated in numerical models of LFW shear walls. Figure 3.14 shows the details of the blocking floor assembly.



**Figure 3.14: Blocking floor assembly configuration**

#### **3.4.9 Configuration #9**

This configuration comprises two identical, high-AR shear walls, each measuring 1.0 m in length, separated by a 2.0 m opening. It is identical to Configuration #4, except that the continuous LVL joist is replaced by discrete blocking. The rationale for selecting this configuration follows that of Configuration #8, but with a larger opening between the shear walls in order to alter the effect of the floor stiffness on the shear wall. The floor assembly configuration and connection details are similar to those depicted in Figure 3.14.

#### **3.4.10 Configuration #10, Configuration #11 and Configuration #12**

This section describes three single, isolated shear wall configurations, with individual lengths of 1.0 m, 2.0 m, and 3.66 m, and with corresponding ARs of 2.4, 1.2, and 0.67, respectively. The selection of these configurations was based on three main objectives.

First, these configurations were intended to establish the strength and stiffness characteristics of isolated shear walls for later comparison with configurations that include multiple shear walls in a shear line. In particular, this allowed investigation of whether the strength and stiffness of multiple shear walls acting together can be approximated as the summation of individual shear walls, and how shear wall strength and stiffness vary with wall length.

Second, these configurations were used to investigate the influence of multiple sheathing panels and the presence of ABs on the distribution of forces and deformations within shear wall elements. Currently, the CSA O86 (Canadian Standards Association 2019) procedure for calculating HD forces does not distinguish between shear walls sheathed with a single panel and those composed of multiple panels. In practice, however, sheathing panels have standard dimensions, and longer shear walls inevitably require multiple panels.

In addition, the CSA O86 (Canadian Standards Association 2019) assumes that ABs resist only shear forces and do not contribute to uplift resistance; only HDs are designed to resist uplift resulting from the overturning moment. These configurations were expected to provide a necessary basis for examining the validity of these assumptions and for establishing a clearer understanding of force and deformation mechanisms in wood shear walls.

Third, these configurations aimed to improve understanding of how shear walls with different characteristics interact when arranged in a shear line. Proper understanding of the behaviour of isolated shear walls forms the cornerstone for investigating the response of multi-segment shear wall systems.

### 3.4.11 Configuration #13

The shear walls in this configuration were equipped with ATS-SR4 continuous rods as HD devices. The geometry of this configuration was similar to Configuration #2 and consisted of two shear walls, one measuring 1.0 m and the other 2.0 m in length, with 1.0 m opening in between. At the shear wall ends, the chord members consisted of multiple studs placed on both sides of the ATS rods. To develop the required tension–compression force couple at shear walls end along the height of the shear walls, the sheathing panels must be properly connected to the chord members on both sides of the ATS rods. The nail spacing along the top and bottom edges of the sheathing panels, as well as at the panel interfaces in longer shear wall, was 100 mm, while the nail spacing at the shear wall ends in the vertical direction differed. The nail spacing along the shear wall chords was 150 mm (6 in.), in accordance with manufacturer recommendations (Simpson Strong-Tie 2022).

The intent of selecting this configuration was twofold. First, similar to shear walls with discrete HDs, this configuration was used to investigate the impact of floor out-of-plane flexural stiffness on the force and deformation demands of shear walls equipped with ATS continuous rods, and to enable direct comparison with shear walls using discrete HD devices.

Second, previous studies have reported that the effective stiffness of continuous ATS system can be significantly lower than the theoretical stiffness assumed in design (van de Lindt et al. 2010). This is of particular importance, as overestimation of the stiffness of these critical structural components at the design level may inadvertently lead to excessive lateral deformations in real buildings. Moreover, overestimation of the effective stiffness of ATS system directly affects the global stiffness of the multi-storey buildings, the fundamental period, and consequently the design base shear.

Third, currently, the CSA O86 (Canadian Standards Association 2019) does not provide a specific design procedure for shear walls equipped with continuous ATS rods, particularly regarding the

nail pattern along the panel edges at the rod locations. In practice, their design is typically based on manufacturer recommendations (Simpson Strong-Tie 2022). Accordingly, this configuration was intended to improve the understanding of force and deformation distribution in such shear walls, as well as to experimentally investigate the actual stiffness of ATS system under realistic boundary conditions.

#### **3.4.12 Configuration #14**

The geometry of this configuration was similar to Configuration #13; however, several construction details were changed. The nail spacing was reduced to 50 mm. The nail spacing at the ATS rod locations, i.e., shear wall ends where the sheathing was attached to the chord members, was 100 mm according to manufacturer (Simpson Strong-Tie 2022). The nail edge distance was increased from 9 mm to 18 mm. ATS-SR5 continuous rods were used as HD devices at the ends of the shear walls. In this configuration, the ATS rods were intentionally not designed for a 20% increase in demand forces. This choice was made to examine the consequences of situations in which one or more continuous rods do not fully conform to design requirements, reflecting the inherent variability in the strength and stiffness characteristics of these structural components.

The intent of this configuration is to investigate the effect of floor out-of-plane flexural stiffness on the force and deformation demands of shear walls with tighter nail spacing. Of particular interests were the force and deformation demands imposed on the continuous ATS rods. As discussed in Configuration #13, the effective stiffness of ATS system is of critical importance, and this configuration provides an additional opportunity to assess the effective stiffness of continuous rod systems. In addition, this configuration allowed investigation of the effect of increased nail edge distance along the sheathing panel edges.

Table 3-5 presents a summary of the shear wall configurations investigated in the experimental program of this study.

Table 3-5: Shear wall test matrix

| Test configuration | Shear wall | Shear wall length (m) | Nail spacing (mm) | Opening length (m) | No. LVL joists | Anchor bolts | Hold-down |
|--------------------|------------|-----------------------|-------------------|--------------------|----------------|--------------|-----------|
| 1                  | 1          | 2.0                   | 100               | 1.0                | -              | (4) ½"       | HDU5      |
|                    | 2          | 1.0                   | 100               |                    |                | (2) ½"       | HDU5      |
| 2                  | 1          | 2.0                   | 100               | 1.0                | 1              | (4) ½"       | HDU5      |
|                    | 2          | 1.0                   | 100               |                    |                | (2) ½"       | HDU5      |
| 3                  | 1          | 1.5                   | 100               | 1.0                | 1              | (3) ½"       | HDU5      |
|                    | 2          | 1.5                   | 100               |                    |                | (3) ½"       | HDU5      |
| 4                  | 1          | 1.0                   | 100               | 2.0                | 1              | (4) ½"       | HDU5      |
|                    | 2          | 1.0                   | 100               |                    |                | (2) ½"       | HDU5      |
| 5                  | 1          | 2.0                   | 100               | 1.0                | 1              | (4) ½"       | HDU5      |
|                    | 2          | 1.0                   | 100               |                    |                | (2) ½"       | HDU5      |
| 6                  | 1          | 2.0                   | 100               | 1.0                | 2              | (4) ½"       | HDU5      |
|                    | 2          | 1.0                   | 100               |                    |                | (2) ½"       | HDU5      |
| 7                  | 1          | 1.0                   | 100               | 0.5                | 1              | (2) ½"       | HDU5      |
|                    | 2          | 1.0                   | 100               |                    |                | (2) ½"       | HDU5      |
|                    | 3          | 1.0                   | 100               |                    |                | (2) ½"       | HDU5      |
| 8                  | 1          | 1.0                   | 100               | 0.5                | Blocking       | (2) ½"       | HDU5      |
|                    | 2          | 1.0                   | 100               |                    |                | (2) ½"       | HDU5      |
|                    | 3          | 1.0                   | 100               |                    |                | (2) ½"       | HDU5      |
| 9                  | 1          | 1.0                   | 100               | 1.0                | Blocking       | (4) ½"       | HDU5      |
|                    | 2          | 1.0                   | 100               |                    |                | (2) ½"       | HDU5      |
| 10                 | 1          | 1.0                   | 100               | -                  | -              | (2) ½"       | HDU5      |
| 11                 | 1          | 2.0                   | 100               | -                  | -              | (4) ½"       | HDU5      |
| 12                 | 1          | 3.66                  | 100               | -                  | -              | (6) ½"       | HDU5      |
| 13                 | 1          | 2.0                   | 100               | 1.0                | 1              | (4) ½"       | ATS-SR4   |
|                    | 2          | 1.0                   | 100               |                    |                | (2) ½"       | ATS-SR4   |
| 14                 | 1          | 2.0                   | 50                | 1.0                | 1              | (4) ½"       | ATS-SR5   |
|                    | 2          | 1.0                   | 50                |                    |                | (2) ½"       | ATS-SR5   |

Table 3-6 summarizes the design information for the shear walls in the experimental program. These design values were used for comparison with the experimental results. Table 3-7 summarizes the calculated design deflections for all shear wall configurations. These deflections were calculated using Eq. (2.3) without incorporating system-level boundary condition effects. These

were used as reference values for comparison with the experimental and numerical results presented later in this document.

Table 3-6: Shear wall design information.

| Configuration  | Shear Wall ID | $L_s$<br>(mm) | $V_{rs}$<br>(kN) | $V_d$<br>(kN) | (D/C) <sub>sw</sub> | $L_o$<br>(mm) | $T_r$<br>(kN) | $T_d$<br>(kN) | (D/C) <sub>HD</sub> |
|----------------|---------------|---------------|------------------|---------------|---------------------|---------------|---------------|---------------|---------------------|
| #1, #2, #5, #6 | 1             | 2000          | 15.8             | 17.4          | 1.1                 | 1851          | 28.4          | 27.5          | 0.97                |
|                | 2             | 1000          | 7.9              | 6.3           | 0.8                 | 851           | 28.4          | 21.7          | 0.76                |
| #3             | 1             | 1500          | 11.85            | 11.5          | 1.0                 | 1351          | 28.4          | 25.68         | 0.9                 |
|                | 2             | 1500          | 11.85            | 11.85         | 1.0                 | 1351          | 28.4          | 25.68         | 0.9                 |
| #4, #9         | 1             | 1000          | 7.9              | 7.9           | 1.0                 | 851           | 28.4          | 27.18         | 0.96                |
|                | 2             | 1000          | 7.9              | 7.9           | 1.0                 | 851           | 28.4          | 27.18         | 0.96                |
| #7, #8         | 1             | 1000          | 7.9              | 7.9           | 1.0                 | 851           | 28.4          | 27.18         | 0.96                |
|                | 2             | 1000          | 7.9              | 7.9           | 1.0                 | 851           | 28.4          | 27.18         | 0.96                |
|                | 3             | 1000          | 7.9              | 7.9           | 1.0                 | 851           | 28.4          | 27.18         | 0.96                |
| #10            | 1             | 1000          | 7.9              | 7.9           | 1.0                 | 851           | 28.4          | 27.18         | 0.96                |
| #11            | 1             | 2000          | 15.8             | 15.8          | 1.0                 | 1851          | 28.4          | 25.0          | 0.88                |
| #12            | 1             | 3660          | 28.9             | 28.9          | 1.0                 | 3510          | 28.4          | 24.1          | 0.85                |
| #13            | 1             | 2000          | 15.8             | 17.75         | 1.1                 | 1772          | 27.42         | 29.36         | 1.07                |
|                | 2             | 1000          | 7.9              | 5.95          | 0.8                 | 772           | 27.42         | 22.56         | .82                 |
| #14            | 1             | 2000          | 26.1             | 29.8          | 1.14                | 1772          | 43.36         | 49.2          | 1.13                |
|                | 2             | 1000          | 13.05            | 9.34          | 0.71                | 772           | 43.36         | 35.42         | 0.82                |

$L_s$ : Shear wall length;  $V_{rs}$ : Factored shear resistance;  $V_d$ : Design shear force; (D/C)<sub>sw</sub>: Shear wall demand-to-capacity ratio;  $L_o$ : Overturning length;  $T_r$ : HD factored tensile resistance;  $T_d$ : HD design force; (D/C)<sub>HD</sub>: HD demand-to-capacity ratio

Table 3-7: Shear wall deflections

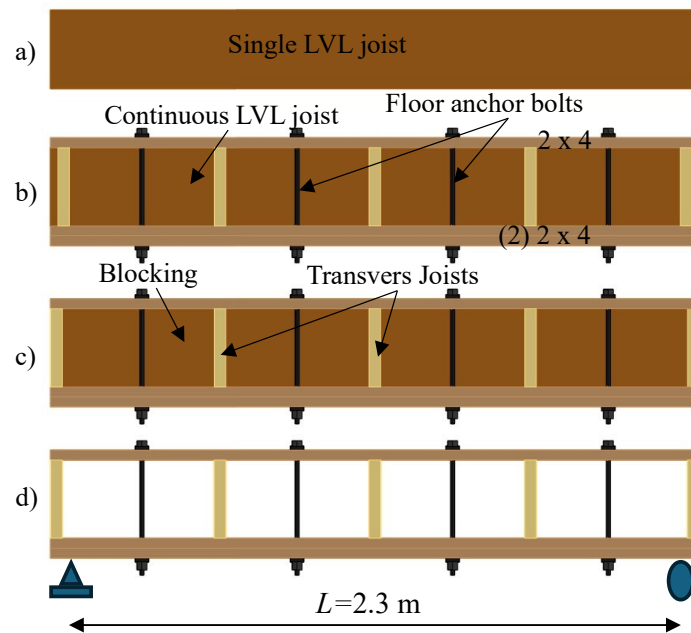
| Configuration  | Shear wall ID | $e_n$<br>(mm) | $d_{a1}$<br>(mm) | $d_{a2}$<br>(mm) | $\Delta_b$<br>(mm) | $\Delta_s$<br>(mm) | $\Delta_n$<br>(mm) | $\Delta_{a1}$<br>(mm) | $\Delta_{a2}$<br>(mm) | $\Delta_t$<br>(mm) |
|----------------|---------------|---------------|------------------|------------------|--------------------|--------------------|--------------------|-----------------------|-----------------------|--------------------|
| #1, #2, #5, #6 | 1             | 1.04          | 3.15             | 0.41             | 0.66               | 1.95               | 6.34               | 4.15                  | 0.54                  | 13.6               |
|                | 2             | 0.54          | 2.46             | 0.33             | 0.93               | 1.37               | 3.29               | 7.05                  | 0.94                  | 13.6               |
| #3             | 1             | 0.86          | 2.94             | 0.40             | 0.8                | 1.70               | 5.25               | 5.31                  | 0.72                  | 13.8               |
|                | 2             | 0.86          | 2.94             | 0.40             | 0.8                | 1.75               | 5.25               | 5.31                  | 0.72                  | 13.8               |
| #4, #9         | 1             | 0.86          | 3.11             | 0.42             | 1.19               | 1.75               | 5.25               | 8.92                  | 1.20                  | 18.3               |
|                | 2             | 0.86          | 3.11             | 0.42             | 1.19               | 1.75               | 5.25               | 8.92                  | 1.20                  | 18.3               |
| #7, #8         | 1             | 0.86          | 3.11             | 0.42             | 1.19               | 1.75               | 5.25               | 8.92                  | 1.20                  | 18.3               |
|                | 2             | 0.86          | 3.11             | 0.42             | 1.19               | 1.75               | 5.25               | 8.92                  | 1.20                  | 18.3               |
|                | 3             | 0.86          | 3.11             | 0.42             | 1.19               | 1.75               | 5.25               | 8.92                  | 1.20                  | 18.3               |
| #10            | 1             | 0.86          | 3.11             | 0.42             | 1.19               | 1.75               | 5.25               | 8.92                  | 1.20                  | 18.3               |
| #11            | 1             | 0.86          | 2.86             | 0.38             | 0.61               | 1.75               | 5.25               | 3.77                  | 0.50                  | 11.9               |
| #12            | 1             | 0.86          | 2.76             | 0.37             | 0.32               | 1.75               | 5.25               | 1.92                  | 0.26                  | 9.5                |
| #13            | 1             | 1.08          | 3.75             | 0.22             | 0.33               | 1.97               | 6.59               | 5.16                  | 0.30                  | 14.4               |
|                | 2             | 0.49          | 2.88             | 0.17             | 0.44               | 1.32               | 2.99               | 9.10                  | 0.54                  | 14.4               |
| #14            | 1             | 0.76          | 3.98             | 0.38             | 0.56               | 3.31               | 4.64               | 5.48                  | 0.52                  | 14.5               |
|                | 2             | 0.30          | 2.86             | 0.27             | 0.70               | 2.07               | 1.83               | 9.04                  | 0.85                  | 14.5               |

$e_n$ : Nail deformation;  $d_{a1}$ : HD elongation;  $d_{a2}$ : Compression perpendicular to grain deformation;  $\Delta_b$ : Shear wall deflection due to bending;  $\Delta_s$ : Shear wall deflection due to sheathing panel shear deformation;  $\Delta_n$ : Shear wall deflection due to nail deformations;  $\Delta_{a1}$ : Shear wall deflection due to HD elongation;  $\Delta_{a2}$ : Shear wall deflection due to compression perpendicular to grain deformation of the bottom plate

### 3.5 Floor assembly (beam) test specimens

The purpose of the floor assembly tests was to investigate the flexural stiffness of different floor configurations representative of those commonly used in LFW buildings. Depending on the construction detailing of the floor assembly, its flexural stiffness can vary significantly.

Four floor assembly configurations were selected for this experimental program. All assemblies had a span of 2.3 m. The bottom flange consisted of two  $2 \times 4$  in. ( $38 \times 89$  mm) dimensional lumber members nailed together using 10d common nails spaced at 220 mm along the length of the member and staggered on both sides. A  $44.5 \times 301.6$  mm LVL member was used in all configurations, either as a continuous joist or cut into discrete blocking pieces. A single  $2 \times 4$  in. ( $38 \times 89$  mm) member was placed on top of the LVL, and the whole specimen was fastened securely using floor-level ABs. Floor assembly configurations are illustrated in Figure 3.15(a)–(d).



**Figure 3.15: Floor assembly configurations**

The first configuration (Figure 3.15(a)) consisted of a single continuous LVL joist and was considered the reference case for evaluating floor flexural stiffness. The second configuration (Figure 3.15(b)) represented a condition in which the LVL joist was confined by adjacent framing

members. In this configuration, the bottom flange simulates the top plates of a lower-storey shear wall, while the top flange represents the bottom plate of an upper-storey shear wall.

The third configuration (Figure 3.15(c)) was similar to the second configuration, except that the continuous LVL joist was replaced by discrete blocking elements. The fourth configuration (Figure 3.15(d)) represented a case in which floor joists pass over the shear walls without blocking between them. This configuration represented an extreme lower-bound condition in which the influence of the floor on the shear walls is minimal.

The number of ABs in the second and third configurations (Figure 3.15(b) and (c)) was varied between 2 and 6 to examine their influence on the behaviour of the floor assembly.

The experimental investigation of these floor assemblies also provides estimates of the flexural stiffness, which are essential for quantifying realistic floor out-of-plane stiffness values to inform the modelling techniques described in Chapter 5.

### **3.6 Material and connection test specimens**

#### **3.6.1 Material test**

The purpose of these tests was to establish the mechanical properties of the materials and connections employed in the full-scale shear wall tests. The results provided the basis for characterizing the behaviour of individual components within the shear wall assembly. The measured properties were subsequently used as input parameters in numerical simulations of the shear wall tests, allowing such models to better represent the experimental data. This approach supports the development of more reliable numerical models for parametric and sensitivity studies. The material tests conducted in this study include the assessment of the modulus of elasticity (MOE) of 2 × 4 in. (38 × 89 mm) framing members, the MOE of LVL joist, the tensile properties of ATS rods, and the bending yield stress of nails.



The MOE of lumber members and LVL joist was determined in accordance with the procedure in the ASTM D198 standard (ASTM D198 2022). Five 2 × 4 in. (38 × 89 mm) stud specimens and one LVL joist specimen were investigated.

Tensile properties of the ATS rods were estimated experimentally following ASTM A370 (ASTM A370 2024) procedure. Five ATS-SR4 and ten ATS-SR5 rod specimens were selected. Since ATS-SR5 rods are partially threaded, five specimens were taken from the threaded portion of the rod and five from the smooth portion. ATS-SR4 rods are fully threaded; therefore, five specimens were considered sufficient.

The bending yield stress of nails was assessed according to the test procedure specified in ASTM F1575 (ASTM F1575 2025). Five specimens were selected for each nail type to verify that their mechanical properties are consistent with those specified in CSA O86 (Canadian Standards Association 2019).

Table 3-8 summarizes the materials tested in this study along with the number of specimens used for each test.

Table 3-8: Material test specimens

| Material        | No. of Specimens | Specimen length (mm) | Standard   |
|-----------------|------------------|----------------------|------------|
| 2 x 4 stud      | 5                | 1220                 | ASTM D198  |
| LVL joist       | 1                | 2400                 | ASTM D198  |
| ATS-SR4         | 5                | 700                  | ASTM A370  |
| ATS-SR5         | 10               | 700                  | ASTM A370  |
| 8d common nail  | 5                | 63.5                 | ASTM F1575 |
| 10d common nail | 5                | 75                   | ASTM F1575 |
| 16d common nail | 5                | 88.9                 | ASTM F1575 |

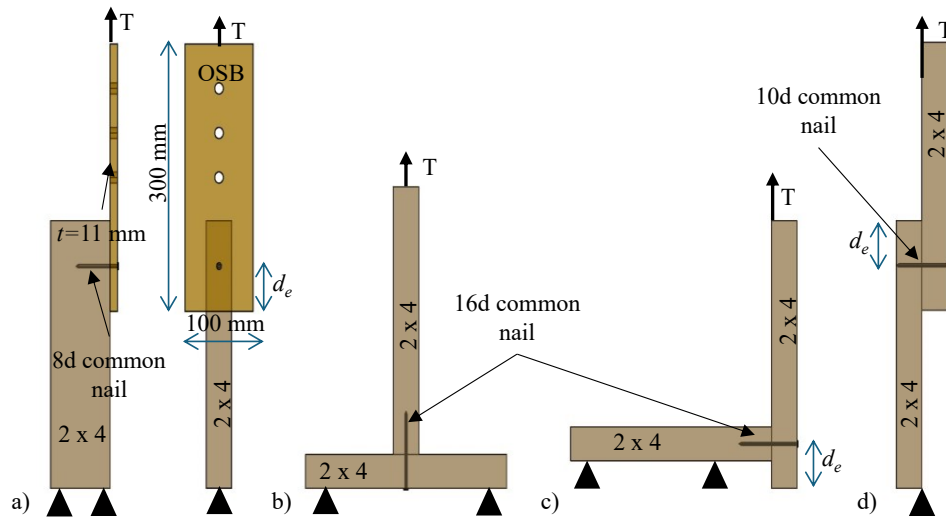
### 3.6.2 Connection test

Several connection test types were considered in this study, including sheathing-to-framing connections, frame-to-frame connections, and HD connections. All connection tests were conducted in accordance with the procedures specified in ASTM D1761 (ASTM D1761 2020).

The purpose of these tests was to determine the actual behaviour of the connections in the wood

shear walls under investigation. The results were subsequently used in the numerical modelling presented in Chapter 5.

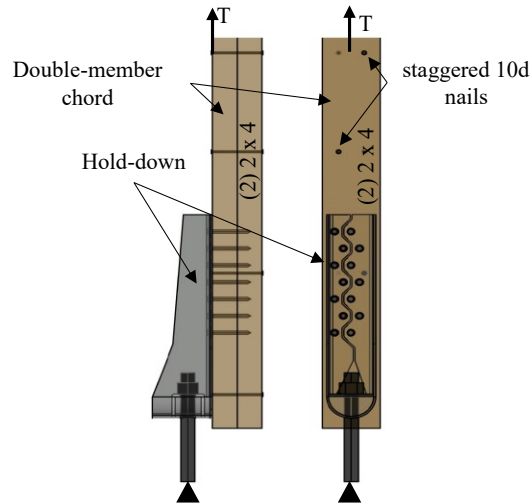
The sheathing-to-framing connection (Figure 3.16(a)) tests were conducted using a single nail size (8d), 11 mm OSB panel as side member, and  $2 \times 4$  in. ( $38 \times 89$  mm) lumber as main members, consistent with shear wall tests in this study. Four nail edge distances ( $d_e = 50, 20, 15$ , and  $10$  mm) were investigated for sheathing-to-framing connections.



**Figure 3.16: Nail connection geometries: (a) sheathing-to-framing connection; (b) end-nail withdrawal connection; (c) end-nail lateral connection; (d) double-member lateral connection**

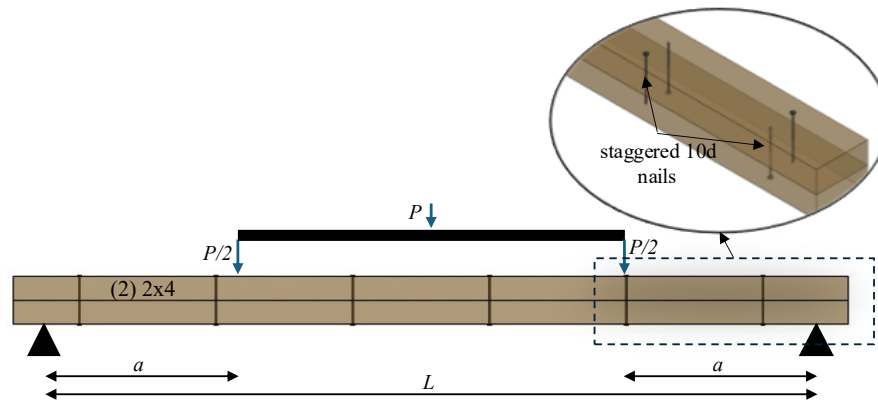
The frame-to-frame connections included end-nail withdrawal (Figure 3.16(b)), end-nail lateral (Figure 3.16(c)), and double-member lateral connections (Figure 3.16(d)). End-nail connections represent the connections between vertical studs and the top and bottom plates in the shear wall. The double-member connection represents the connection in shear wall chord members. The  $d_e$  value for all frame-to-frame connections was 50 mm (ASTM D1761 2020).

HD assemblies were tested to evaluate their tensile resistance (Figure 3.17). The HD devices examined in this study include Simpson Strong-Tie HDU2, HDU4, and HDU5 devices. Similar to nail connections, HD connection tests were conducted in accordance with ASTM D1761 (ASTM D1761 2020).



**Figure 3.17: HD connection assembly**

Multiple-ply built-up members are commonly used in LFW shear walls as chord and/or strut members. Figure 3.17 shows the two-ply built-up member used as the chord and strut members of the shear walls in this study.



**Figure 3.18: Two-ply built-up member**

These built-up members are typically fastened together with nails or screws and resist combined axial force and bending moment due to their structural role. CSA O86 (Canadian Standards Association 2019) considers the cross-sectional area and flexural stiffness of these members to be equivalent to those of solid members, thereby neglecting their composite nature. In practice, however, the effective flexural stiffness of built-up members differs from that of an equivalent member with a uniform cross section. This parameter is important for the numerical modelling and

validation of shear wall tests. Accordingly, a specimen consisting of a double  $2 \times 4$  in. ( $38 \times 89$  mm) member was selected to experimentally evaluate its effective flexural rigidity. The member had a length of 1220 mm, and the two members were connected using 10d nails spaced at 220 mm, staggered on both sides. The test was conducted following the procedure in ASTM D198 standard (ASTM D198 2022).

Table 3-9 summarizes the connection test matrix and the number of specimens for each connection type.

Table 3-9: Connection test matrix

| Connection           | No. of Specimens | Fastener                                            |
|----------------------|------------------|-----------------------------------------------------|
| Sheathing-to-framing | 30               | 8d common nail                                      |
| End-nail withdrawal  | 5                | 16d common nail                                     |
| End-nail Lateral     | 5                | 16d common nail                                     |
| Frame-to-frame       | 5                | 10d common nail                                     |
| HDU2                 | 10               | (6) $\frac{1}{4}$ " x $2 \frac{1}{2}$ " SDS screws  |
| HDU4                 | 10               | (10) $\frac{1}{4}$ " x $2 \frac{1}{2}$ " SDS screws |
| HDU5                 | 13               | (14) $\frac{1}{4}$ " x $2 \frac{1}{2}$ " SDS screws |
| Built-up member      | 1                | 10d common nails @ 220 mm                           |

### 3.7 Test setup and instrumentation

In this section, the test setups for the full-scale shear wall tests, floor assembly tests, and connection tests are described. For each test setup, the corresponding instrumentation scheme is presented, and the role of each measurement device is explained.

#### 3.7.1 Full-scale shear wall test setup

Figure 3.19 illustrates the test setup for full-scale shear wall test. The test setup consisted of several key components, including a loading frame that supports the actuator. The frame was securely fastened to the strong floor to ensure that it remains stationary during testing.

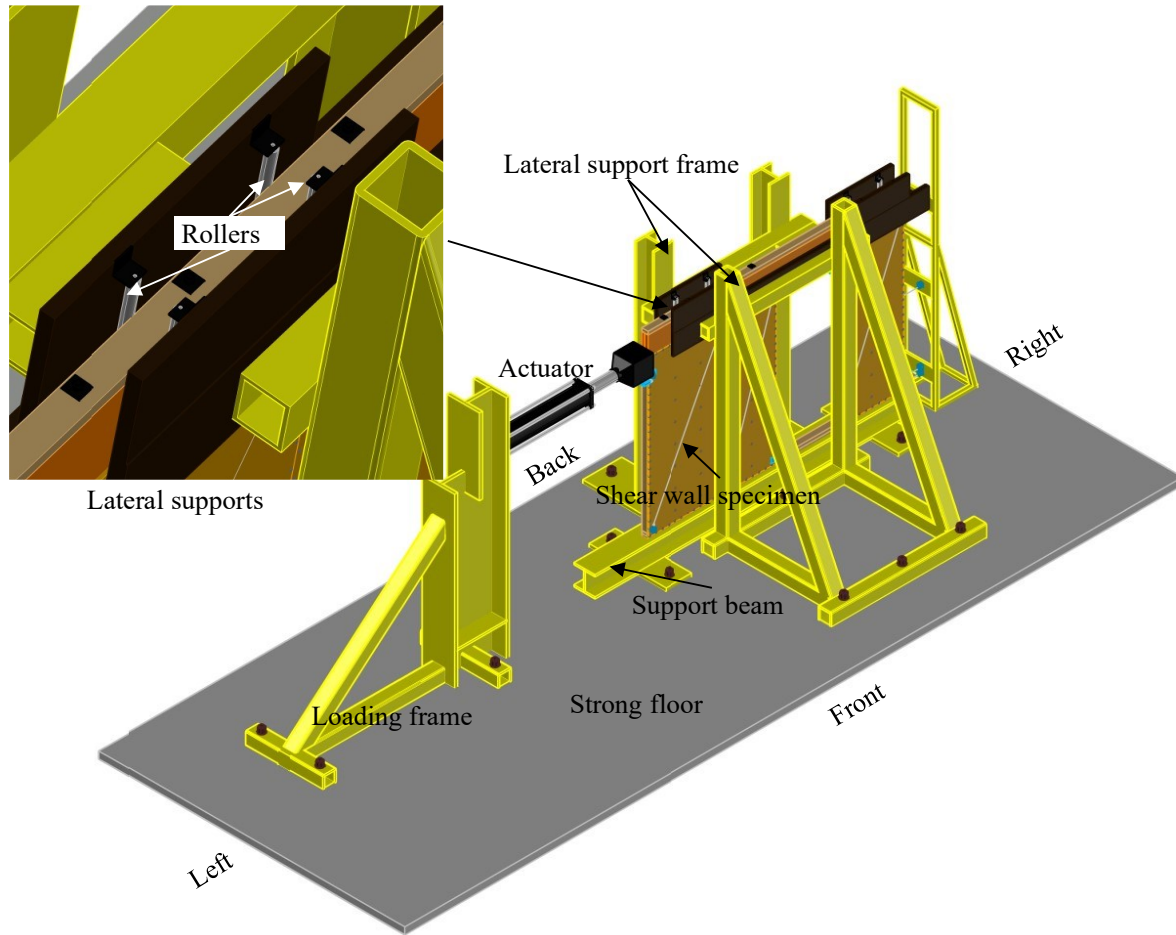
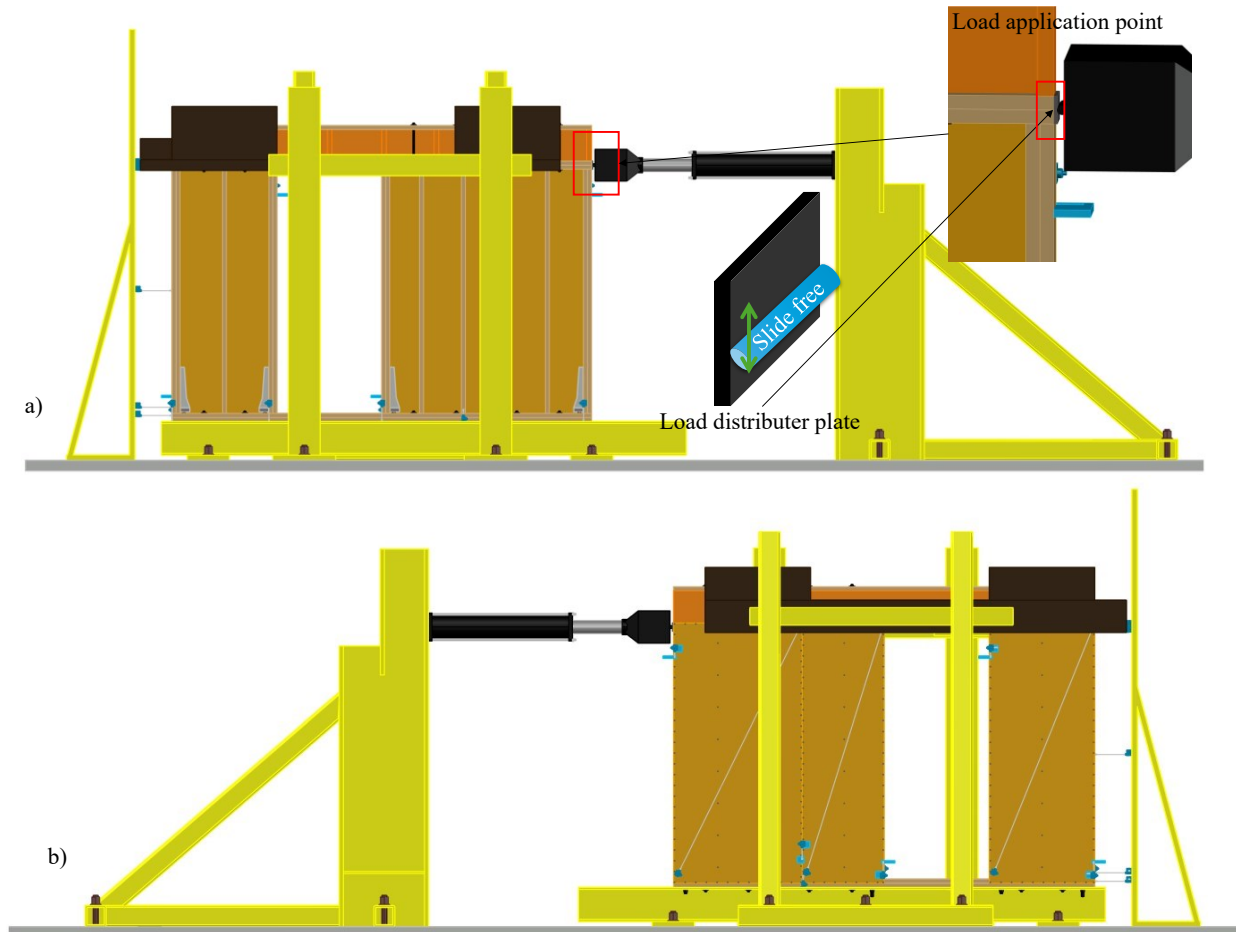


Figure 3.19: Test setup for full-scale shear wall test

The second component of the setup is an I-shaped support beam, on which the shear wall specimen was placed. The specimen was positioned on the upper flange of the beam and secured using ABs. The support beam was equipped with thick flanges to prevent local deformation under the applied shear wall forces.

Lateral stability of the shear wall specimen was ensured through the use of lateral support frames. These frames restrain out-of-plane movement of the specimen at the top of the shear wall on both sides. To allow smooth in-plane movement of the specimen during testing, roller-type lateral supports were employed. The specimens were laterally supported at two locations, with four rollers provided at each location.

It should be noted that the actuator load was applied at the top of the shear wall rather than at the floor level. This form of load application was adopted to maintain the lateral stability of the floor assembly (beam) above the shear wall, and to provide consistency in the load application location for all test specimens. Figure 3.20(a) and (b) present back and front elevation views of the test setup.

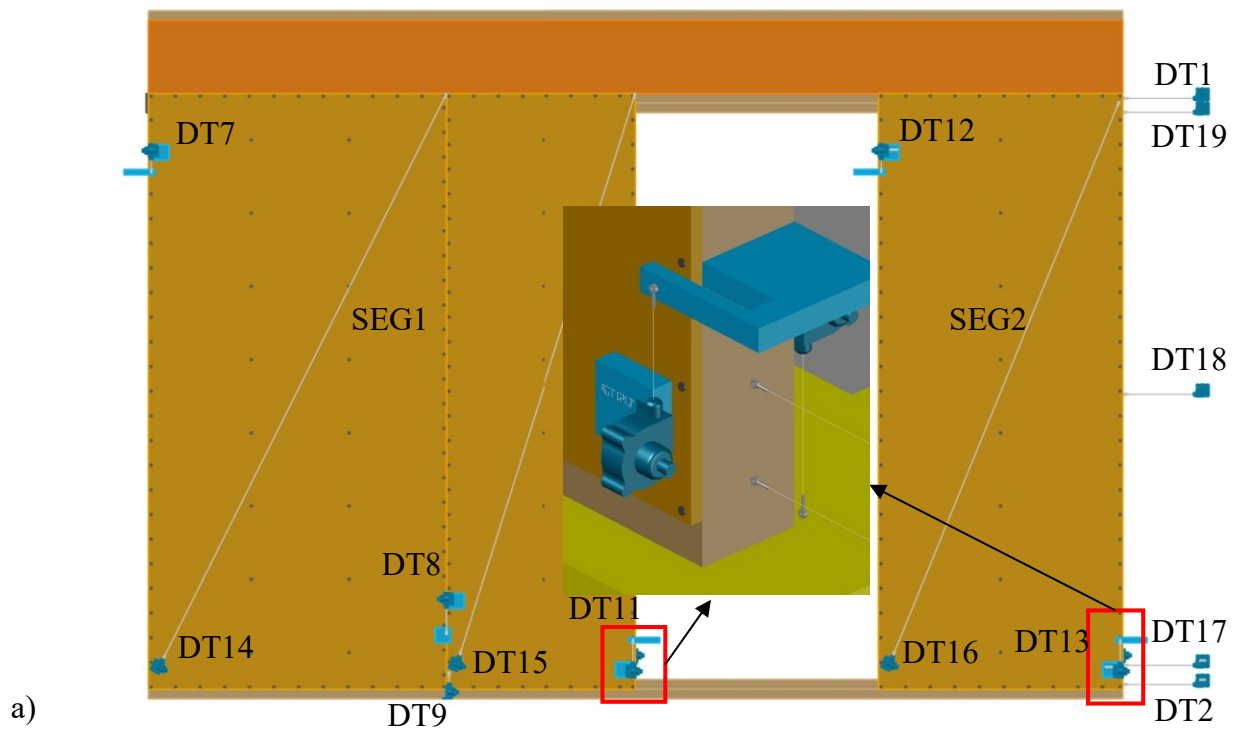


**Figure 3.20: Elevation view of full-scale shear wall test setup: (a) back view; (b) front view**

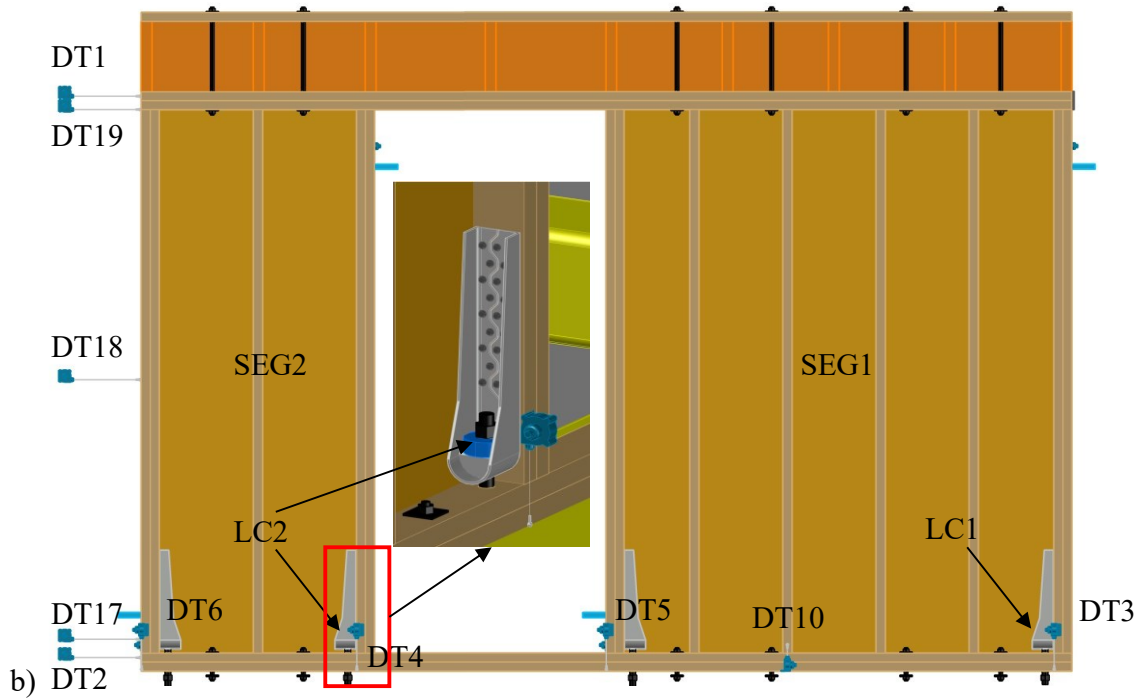
The load was applied to the shear wall specimen monotonically using a displacement-controlled actuator at a constant rate of 4 mm/min. As the focus of the study was to capture the kinematics and deformations of the wall components rather than the cyclic response characteristics, a monotonic loading regime was adopted. Measurements from all instrumentation devices were recorded using a data acquisition system at a sampling rate of 20 points per second. At the load

application point, a load distributor plate was placed between the actuator tip and the shear wall top plates to distribute the applied force and prevent local crushing due to load concentration, as shown in Figure 3.20(a). A roller mechanism at this location allowed free vertical movement, ensuring that only horizontal load was applied to the shear wall. The actuator was restrained in the horizontal direction to remain stationary during testing.

To measure the force and deformation demands during testing, several measurement devices were installed at selected locations on the shear wall specimen. Figure 3.21 illustrates the layout and distribution of the instrumentation for a representative specimen consisting of two segments, designated as SEG1 and SEG2.



**Figure 3.21: Instrumentation layout: (a) front view; (b) back view**



**Figure 3.20: Instrumentation layout: (a) front view; (b) back view (continued)**

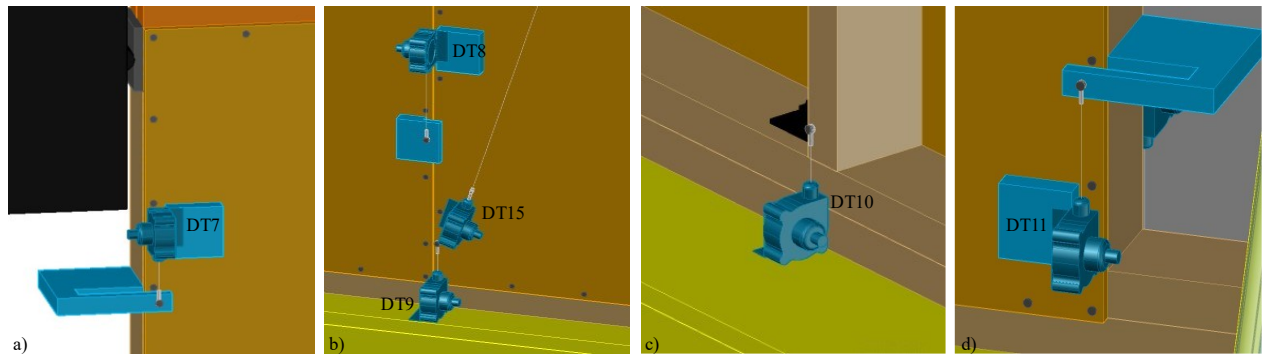
Deformations were measured using displacement transducers (DTs). DT1 and DT2 were used to record the horizontal displacements at the top and bottom of the shear wall, respectively.

The elongations of the HD devices associated with SEG1 and SEG2 were measured using DT3 and DT4, respectively. Deformation perpendicular to grain under the compression chords of SEG1 and SEG2 was captured by DT5 and DT6.

Nail deformations at selected locations in SEG1 were measured using DT7, DT8, DT9, and DT11 devices. DT7 was installed near the top-left (TL) corner of SEG1 (Figure 3.22(a)). DT8 was placed at the panel interface to measure nail deformation at the middle (MID) between the two panels (Figure 3.22(b)). DT9 was used to measure nail deformation at the bottom corner of the right-hand panel in SEG1 (Figure 3.22(b)). An additional device, DT10, was installed at the panel interface to measure potential uplift of the adjoining stud (Figure 3.22(c)). The measurements from DT9 and



DT10 were subsequently used to determine the net nail deformation at the panel interface. Finally, nail deformation at the bottom-right (BR) corner was measured using DT11 (Figure 3.22(d)).



**Figure 3.22: Location of DTs in SEG1: (a) TL corner; (b) panel interface; (c) panel interface for stud uplift; (d) BR corner in SEG1**

In a similar manner to SEG1 (Figure 3.22(a) and (d)), nail deformations at the TL and BR corners of SEG2 were measured using displacement transducers DT12 and DT13.

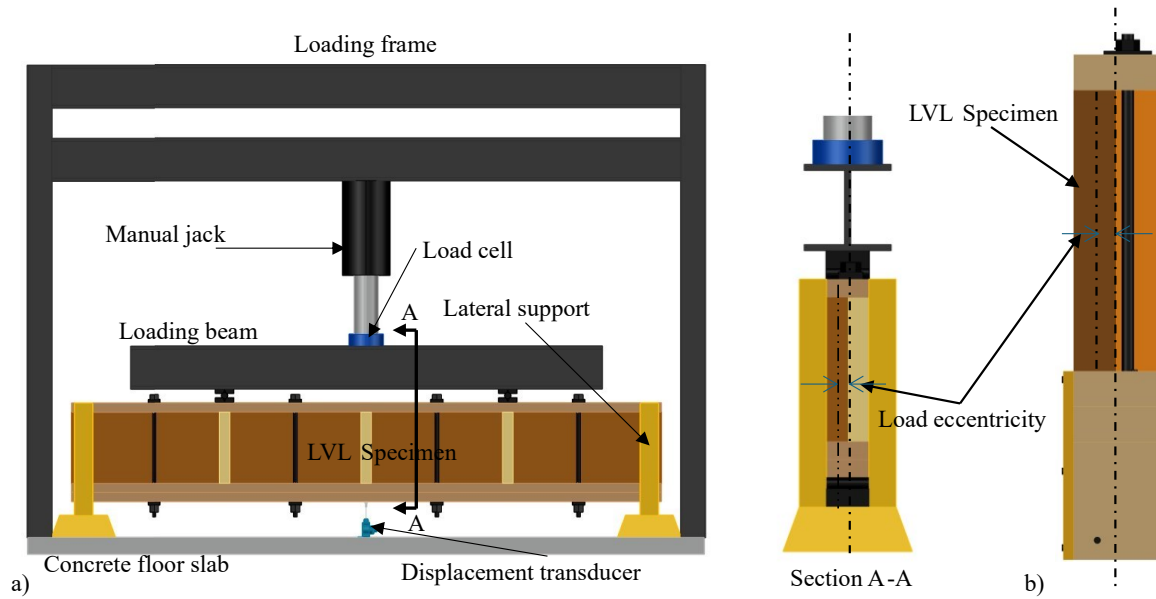
Shear deformations of the sheathing panels were measured using DT14 through DT16. To assess bending deformations of the end studs, DT17, DT18, and DT19 were installed at the bottom, mid-height, and top of the compression chord in SEG2.

The locations of the displacement transducers were kept constant for most shear wall configurations. Even when the layout of the transducers was modified depending on the geometry of the wall configuration, the underlying concept remained the same: all key deformations described above were still measured, ensuring consistency in the recorded data across all shear wall configurations.

Load transfer through the HDs was measured using two load cells (LC), LC1 and LC2, installed on the anchor bolts at the tension chords (Figure 3.21(b)). It is noted that the exact instrumentation layout may vary depending on the shear wall configuration under consideration.

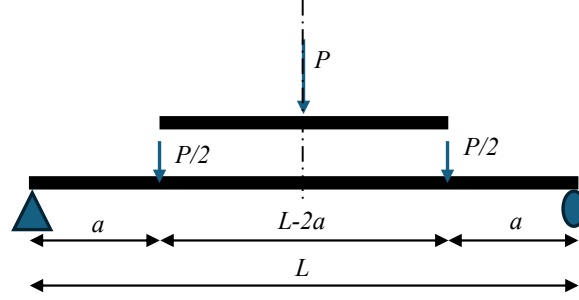
### 3.7.2 Floor assembly bending test setup

Figure 3.23(a) illustrates the test setup for the floor assembly bending test.



**Figure 3.23: (a) Test setup for floor assembly bending test; (b) floor assembly on top of shear wall**  
The setup consisted of a loading frame rigidly connected to the concrete slab. Load was applied in a quasi-static manner to the specimen through a loading beam using a loading jack. Roller-type load distributors were placed at the load application points to minimize load concentration effects. The floor assembly specimen was supported at its ends, providing lateral support at those locations. Roller supports allowed free rotation in the bending plane, ensuring that the specimen was not restrained against bending. The floor assembly was loaded in a manner that replicated the real floor configuration, including any eccentricity, as depicted in Figure 3.23(b).

The four-point loading test layup in accordance with ASTM D198 (ASTM D198 2022) was used, as schematically illustrated in Figure 3.24. The total length of all floor assembly specimens was  $L=2.3$  m. The parameter  $a$  was equal to 575 mm, corresponding to the distance between the support and the load application point.



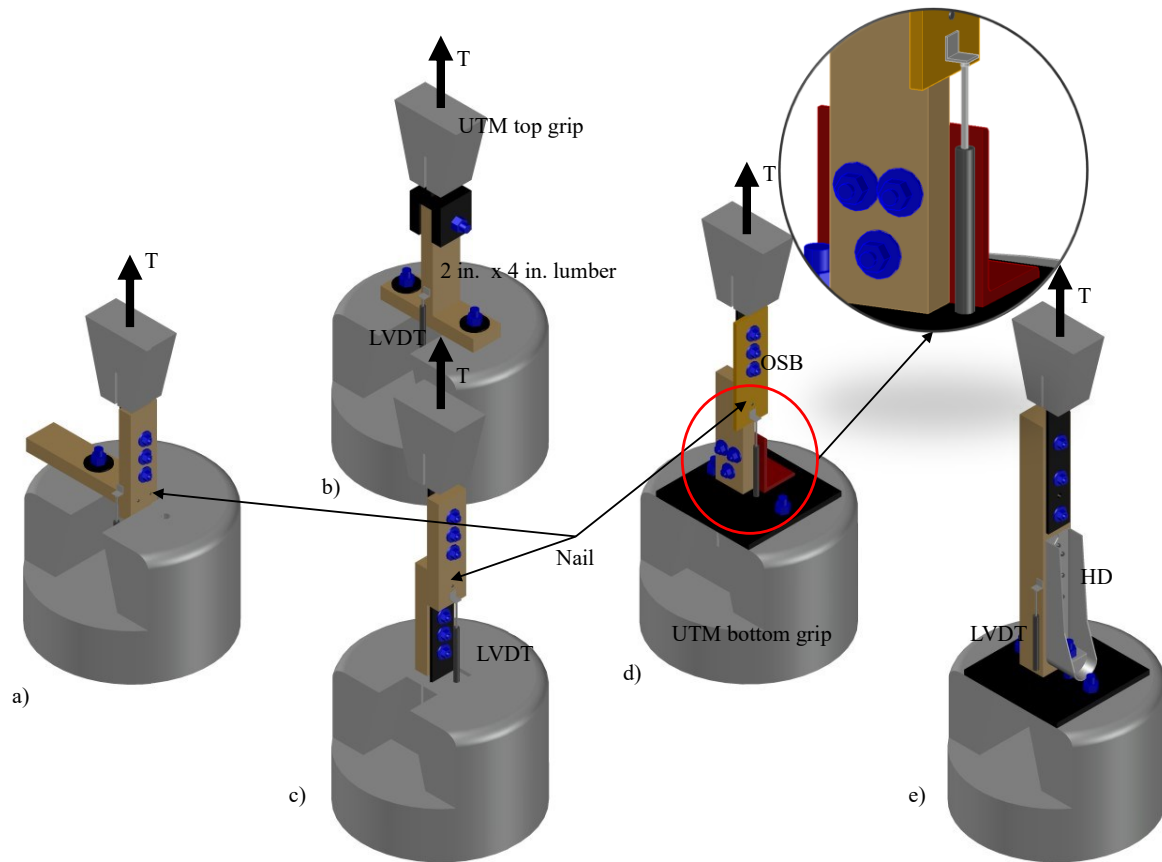
**Figure 3.24: Tow-point loading test**

In order to measure the vertical deflection of the assembly a DT was placed at midspan. Load was measured using a load cell installed between the jack and the loading beam. Data acquisition was employed to continuously record load and displacement measurements. The specimens were loaded statically at a loading rate of 6 mm/min, and the load of the hydraulic jack ( $P$ ) as well as mid-span displacement ( $\Delta_c$ ) were recorded at a sampling rate of 20 points per second.

The same setup was also used for the stud bending tests, to determine their MOE values, as well as for the built-up member test shown in Figure 3.18. However, for these tests the total specimen length was  $L=1.2$  m, and the parameter  $a$  was 400 mm.

### 3.7.3 Connection test setup

Figure 3.25 illustrates the test apparatus used for the connection tests in this study. The connection tests, as well as the tensile tests of the ATS rods, were conducted using a Universal Testing Machine (UTM).

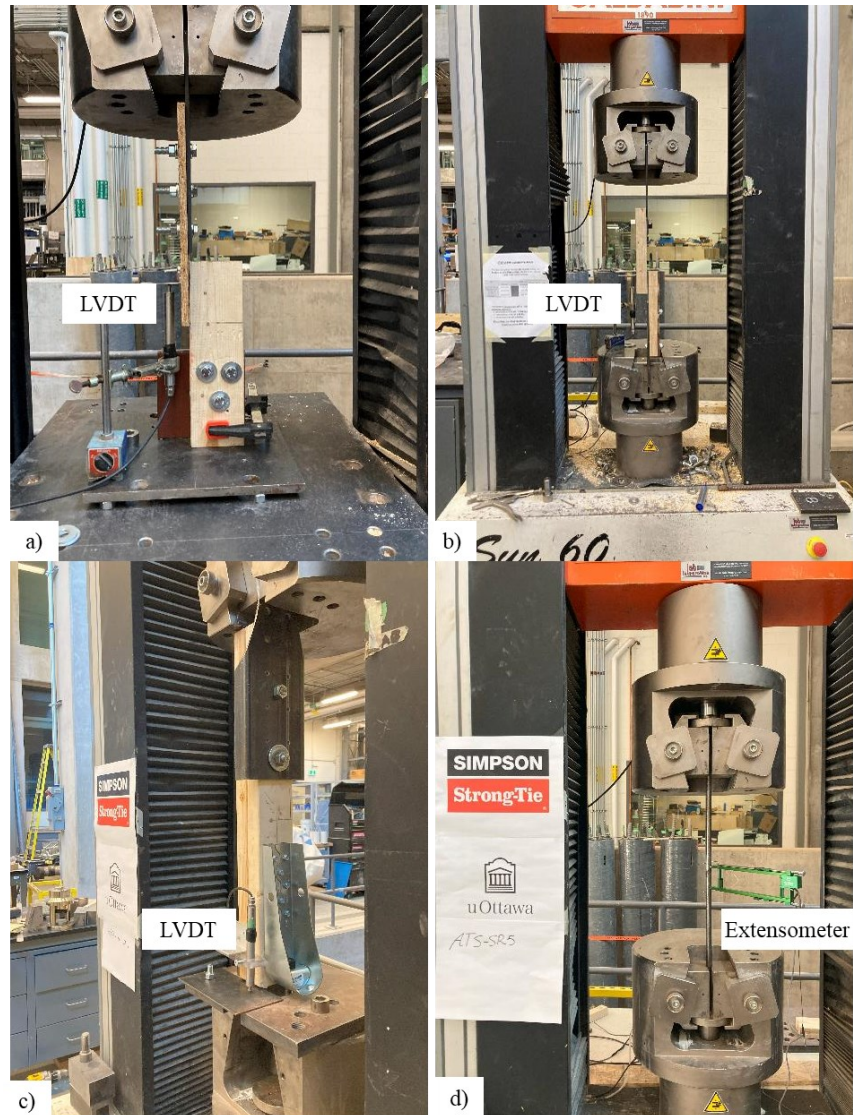


**Figure 3.25: Connection test apparatuses: (a) end-nail lateral connection; (b) end-nail withdrawal connection; (c) frame-to-frame connection; (d) sheathing-to-framing connection; (e) HD connection**

All connection tests were performed under monotonic loading at a rate of 2.54 mm/min, consistent with the procedure specified in ASTM D1761 (ASTM D1761 2020). The applied load was recorded using the UTM load cell, while displacements were measured using an LVDT. Load and displacement data were continuously recorded by the data acquisition system at a sampling rate of 20 points per second.

The test setups were designed to minimize load eccentricity with respect to the shear plane in the lateral connection tests. However, for the HD connection tests, eccentricity between the HD shear plane and the HD anchor bolt was unavoidable. The specimens were securely fastened to ensure that the recorded load–displacement response represented the behaviour of the connection itself,

rather than any slippage between the specimen and the test setup. Figure 3.26(a)–(d) show the UTM and several connection test specimens, including the ATS rod tensile test.

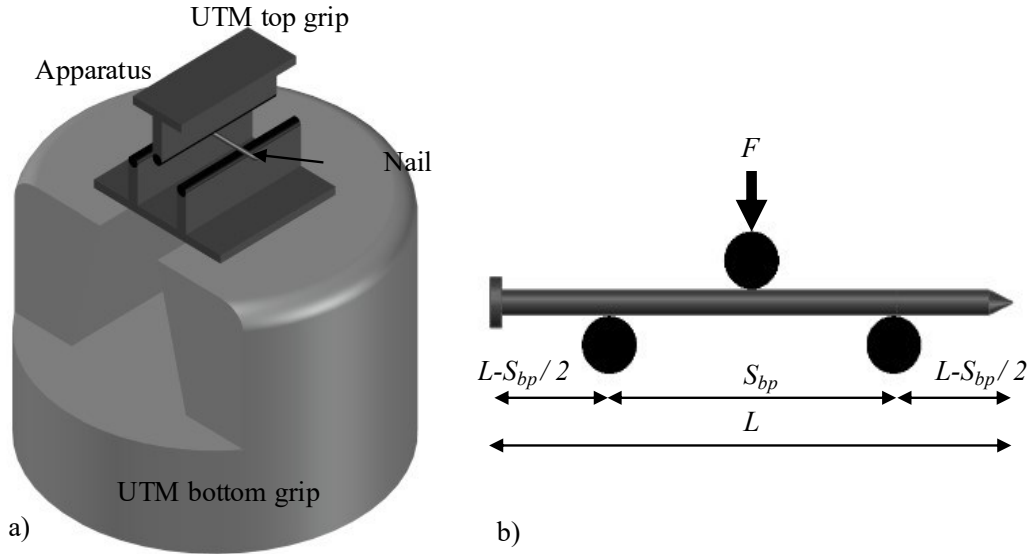


**Figure 3.26: Connection test setups: (a) sheathing-to-framing connection; (b) frame-to-frame connection; (c) HD connection; (d) ATS rod tensile test**

The tensile resistance of the ATS rods was determined using the UTM, where a clip-on axial extensometer with a resolution of  $1 \mu\epsilon$  was used to record axial strain, as depicted in Figure 3.26 (d). Tension tests were conducted at a constant loading rate of 5 mm/min in accordance with the recommendations of ASTM A370 (ASTM A370 2024). The data acquisition system recorded the load and strain data at a rate of 20 sample points per second.

### 3.7.4 Nail bending test

The nail bending tests were conducted using UTM. Figure 3.27(a) shows the test setup and apparatus used for these tests.



**Figure 3.27: Nail bending test setup and apparatus**

Figure 3.27(b) illustrates the dimensions of the setup and the positioning of the nail during testing. In this figure,  $L$  represents the nail length, and the parameter  $S_{bp}$  denotes the distance between the two supports. This distance is defined as a function of the nail diameter. Accordingly, the corresponding values of  $S_{bp}$  for 8d, 10d, and 16d nails are 38 mm, 43 mm, and 48 mm, respectively (ASTM F1575 2025).

The load was applied quasi-statically at a constant rate of 6.0 mm/min, consistent with ASTM 1575 (ASTM F1575 2025). Force and displacement were measured using the UTM load cell and an LVDT, respectively, and were recorded by the data acquisition system at a sampling rate of 20 points per second.



## Chapter 4

### Test results

#### 4.1 Introduction

This chapter presents the experimental results obtained from tests on materials, connections, floor assemblies, and full-scale shear walls. All component level tests (i.e., materials, connections, floor assemblies) are mainly aimed at establishing an accurate understanding of the behaviour of such components in order to use them as inputs into the developed numerical models. Establishing accurate material properties is in itself not the subject of interest, however, it helps reduce uncertainties in the model such that inferences and conclusions can be generalized.

#### 4.2 Material test results

The characteristics of the materials listed in Chapter 3 (Table 3-8) were investigated, and the results are provided in this section. Figure 4.1(a) and (b) illustrate the undeformed and deformed configurations, respectively, of the stud bending tests. The aim of this test was to measure the MOE of the studs used in shear wall tests and to determine the bending rigidity in the weak direction of the studs in order to establish their potential contribution to the overall deflection of the wall.

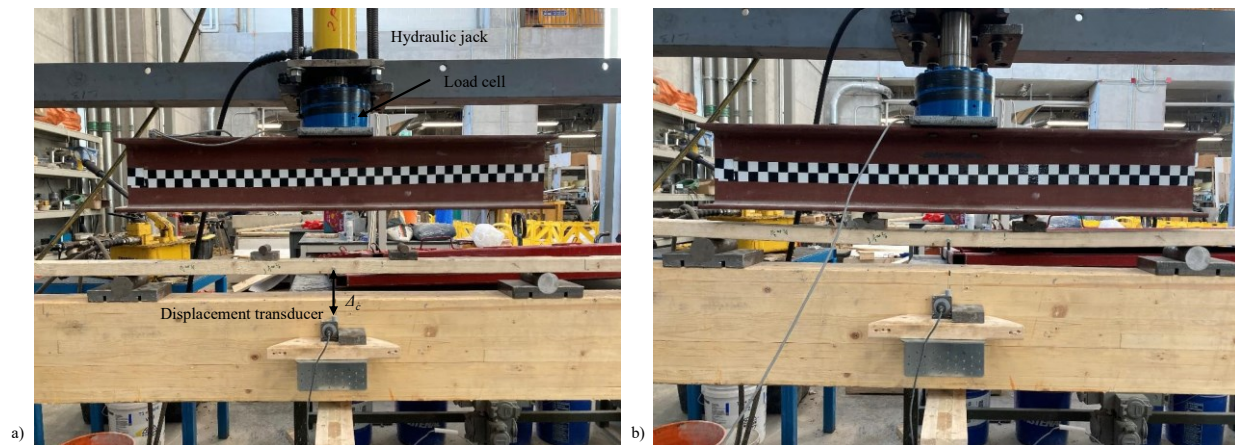
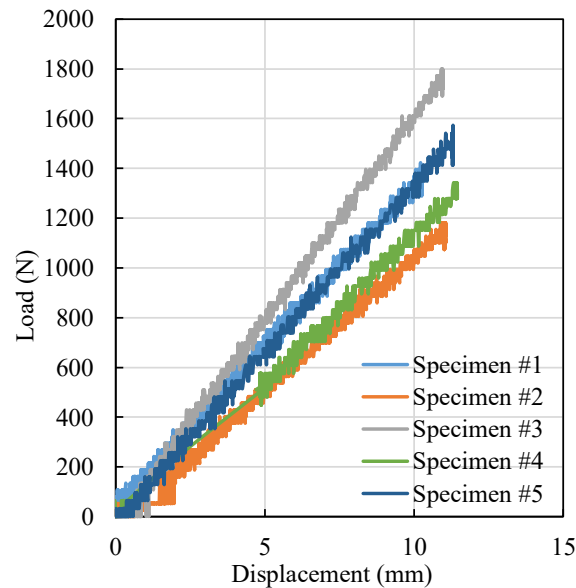


Figure 4.1: Stud bending test

The load was applied to the wide face of the stud, inducing weak axis bending. As the objective of these tests was to determine the MOE of the studs used in the experimental program, the specimens were only loaded non-destructively. Figure 4.2 depicts the experimental load–displacement curves of the stud, indicating that the specimens were only loaded in the linear-elastic region of the load–deflection curve.



**Figure 4.2: Load-displacement curves from stud bending tests**

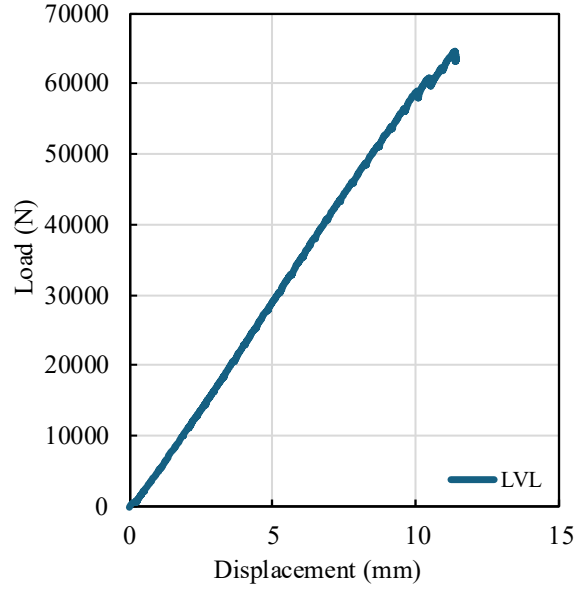
To obtain the MOE of the LVL, the same procedure similar to studs was followed, except that the load was applied to the narrow face (edge) of the LVL, inducing strong-axis bending (Figure 4.3).



**Figure 4.3: LVL bending test**

This configuration was selected because the LVL joist was loaded about their strong axis during the shear wall tests. Figure 4.4 illustrates the load–displacement curve of the LVL.





**Figure 4.4: Load-displacement curve from LVL bending test**

In order to obtain the MOE of the studs, the procedure outlined in ASTM D198 (2022) was adopted. According to this standard, the MOE ( $E$ ) of a member under 4-point bending and with simply supported boundary conditions can be calculated using Eq. (4.1).

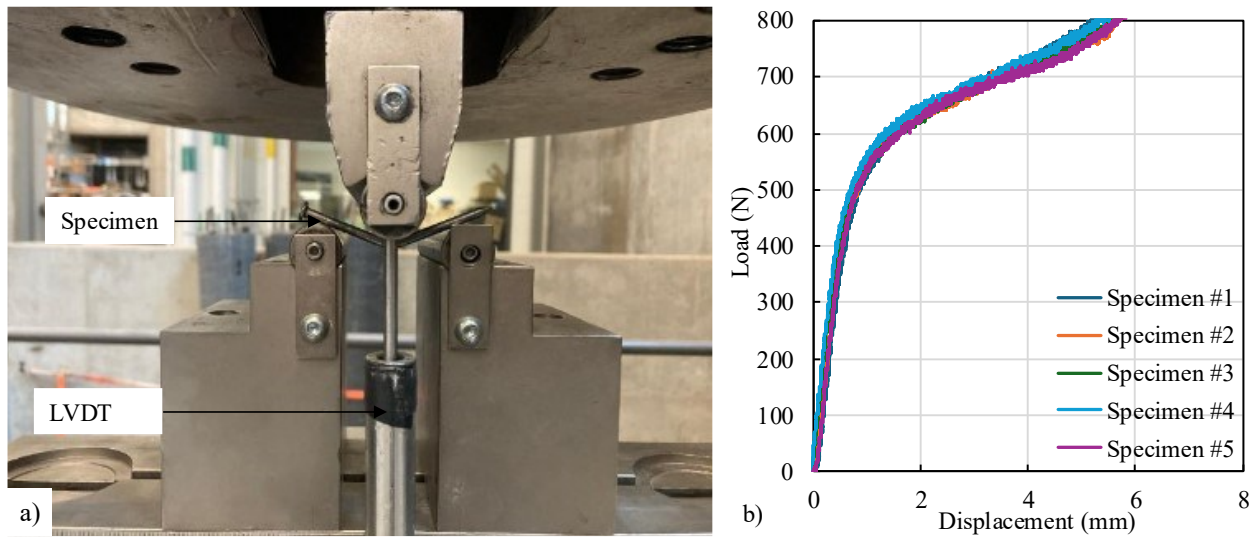
$$E = \frac{Pa}{4bd^3\Delta_c} (3L^2 - 4a^2) \quad (4.1)$$

Where  $L$  is the length of the specimen between supports, which was 1200 mm for studs and 2300 mm for LVL. The parameter  $a$  is the distance between support and load application point and was 400 mm for studs and 575 mm for LVL.  $b$  and  $d$  denote the width and depth of the members. The resulting values of  $E$  for each specimen are reported in Table 4-1.

Table 4-1: MOE values of the stud and LVL members

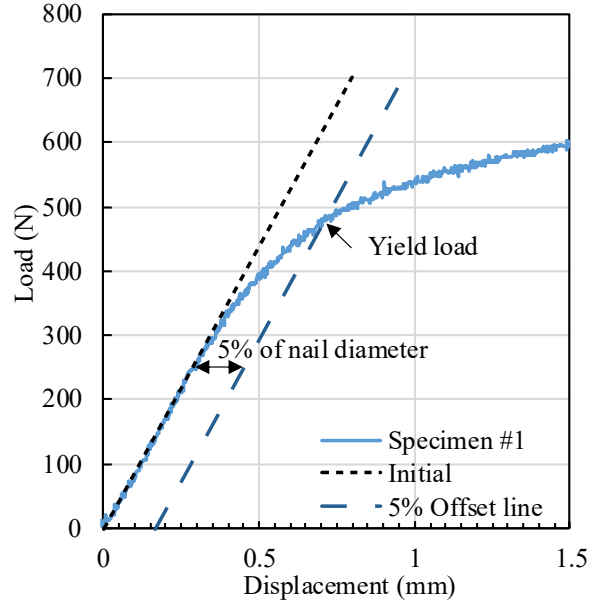
| Specimen       | $P/\Delta_c$ (N/mm) | $E$ (N/mm <sup>2</sup> ) |
|----------------|---------------------|--------------------------|
| Stud #1        | 130.12              | 9531.12                  |
| Stud #2        | 107.00              | 7837.61                  |
| Stud #3        | 165.00              | 12086.03                 |
| Stud #4        | 112.33              | 8228.02                  |
| Stud #5        | 134.10              | 9822.65                  |
| Average (stud) |                     | 9501.09                  |
| CV (stud)      |                     | 0.16                     |
| LVL #1         | 5780.7              | 9902                     |

The average value of  $E=9,501.09 \text{ N/mm}^2$  for the studs was consistent with the value of  $9,500 \text{ N/mm}^2$  specified for S-P-F studs in CSA O86 (Canadian Standards Association 2019). However, the corresponding value for LVL differed significantly from the value provided by the manufacturer ( $14,400 \text{ N/mm}^2$ ). It should be noted that only one specimen was tested for this material. Nail bending tests were conducted for 8d, 10d, and 16d nails. Figure 4.5(a) and (b) show the nail at the end of the test and the corresponding results for 8d nails as an example.



**Figure 4.5: Nail bending test: a) tested specimen; b) load-displacement curves of 8d nail specimens**

To calculate the nominal bending yield strength ( $F_{yb}$ ) of the nails, the offset method specified in ASTM 1575 (ASTM F1575 2025) was adopted, as illustrated in Figure 4.6. Accordingly, Table 4-2 summarizes the resulting nominal yield strength for 8d, 10d, and 16d nails, along with the average value for each nail size and the corresponding coefficients of variation (CV).



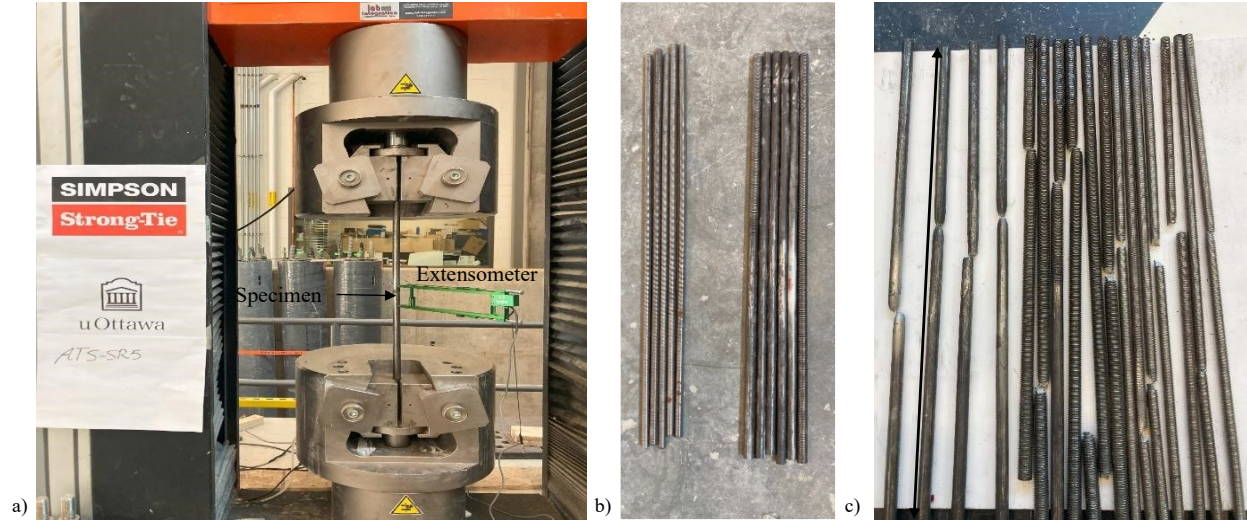
**Figure 4.6: Offset method for determining the nail yield strength**

Table 4-2: Bending yield strength of common nails

| Specimen    | $F_{yb}$ (N/mm <sup>2</sup> ) |        |        |
|-------------|-------------------------------|--------|--------|
|             | 8d                            | 10d    | 16d    |
| Specimen #1 | 747.44                        | 707.03 | 694.42 |
| Specimen #2 | 731.19                        | 765.95 | 746.69 |
| Specimen #3 | 682.45                        | 756.13 | 716.82 |
| Specimen #4 | 755.56                        | 785.59 | 672.02 |
| Specimen #5 | 731.19                        | 667.75 | 724.29 |
| Average     | 729.57                        | 736.49 | 710.85 |
| CV          | 0.03                          | 0.06   | 0.04   |

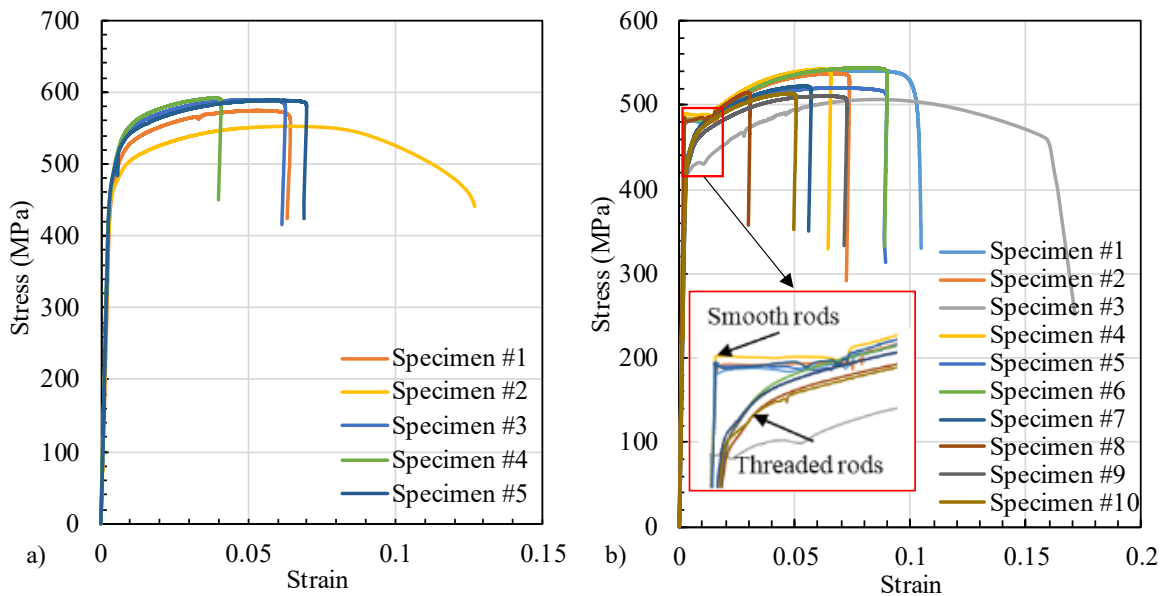
According to the CSA O86 standard (Canadian Standards Association 2019), the specified bending yield strength of nails is defined as a function of nail diameter, and for 8d, 10d, and 16d nails, the corresponding values are 633, 612, and 595 N/mm<sup>2</sup>, respectively. These values, as expected, are lower than the nominal values reported in Table 4-2.

Figure 4.7(a) shows the rod during testing, while Figure 4.7(b) and (c) illustrate the rod specimens before and after testing, respectively.



**Figure 4.7: Universal Testing Machine**

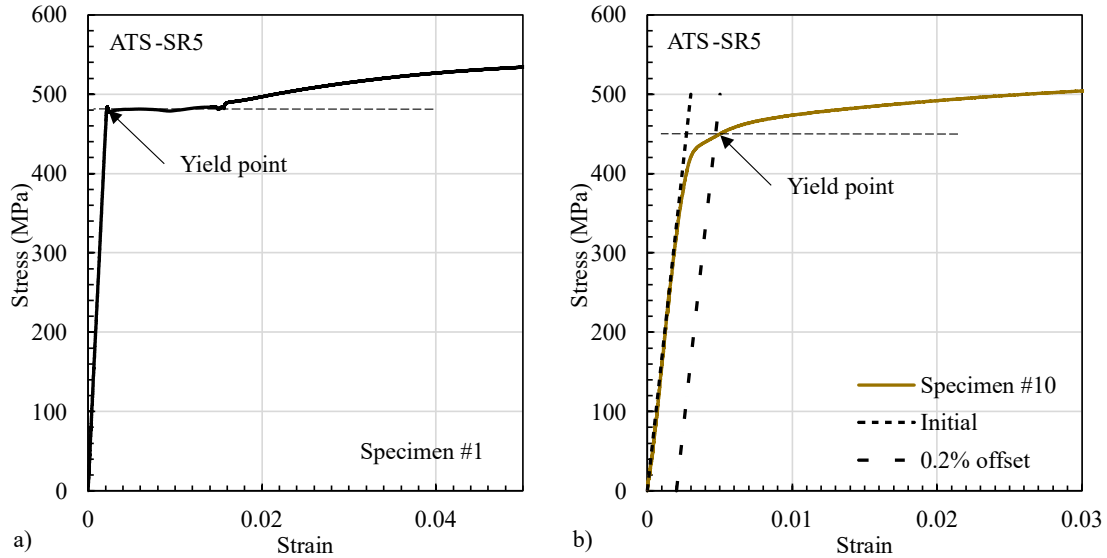
Figure 4.8(a) and (b) illustrate the stress-strain diagrams of the rods used in the ATS-SR4 and ATS-SR5 rod specimens, respectively. The stress in each specimen was calculated by dividing the tensile force ( $T$ ) recorded by the UTM by the net cross-sectional area of the corresponding rods according to manufacturer (Simpson Strong-Tie 2022).



**Figure 4.8: Stress-strain diagrams; a) ATS-SR4; b) ATS-SR5**

While the smooth ATS-SR5 bars (Figure 4.8b) exhibited distinct yield points, threaded rods, in both ATS-SR4 and ATS-SR5, did not.

To obtain the yield stress ( $\sigma_y$ ), the procedure in ASTM A370 (2024) was adopted. For rods with a distinct yield point, the yield stress corresponds to the stress level at plateau section of the stress-strain diagram (Figure 4.9(a)). For specimens without a clear yield point (Figure 4.9(b)), the standard provides several methods; in this study, the 0.2% offset method (ASTM A370 2024) was used to determine the yield stress of such specimens.



**Figure 4.9: Yield point calculation methods**

Table 4-3 summarizes the results of the ATS rod tests. In this table,  $T_y$  is the load corresponding to the yield point and  $T_u$  represents the maximum tensile load. Elongation of individual rods was calculated as the difference between the measured specimen length at fracture and the initial rod length.

Table 4-3: ATS rod test data

| ATS-SR4     |          |                                 |           |           |                 |
|-------------|----------|---------------------------------|-----------|-----------|-----------------|
| Specimen    | Rod type | $\sigma_y$ (N/mm <sup>2</sup> ) | $T_y$ (N) | $T_u$ (N) | Elongation (mm) |
| Specimen #1 | threaded | 485                             | 44431     | 56682     | 33              |
| Specimen #2 | threaded | 470                             | 43057     | 54520     | 19              |
| Specimen #3 | threaded | 495                             | 45347     | 58166     | 31              |
| Specimen #4 | threaded | 500                             | 45805     | 58384     | 28              |
| Specimen #5 | threaded | 500                             | 45805     | 58072     | 35              |
| Average     |          | 490                             | 44889     | 57165     | 29              |
| CV          |          | 0.03                            | 0.03      | 0.03      | 0.21            |

Table 4-3: ATS rod test data (continued)

| ATS-SR5      |          |                                 |           |           |                 |
|--------------|----------|---------------------------------|-----------|-----------|-----------------|
| Specimen     | Rod type | $\sigma_y$ (N/mm <sup>2</sup> ) | $T_y$ (N) | $T_u$ (N) | Elongation (mm) |
| Specimen #1  | smooth   | 480                             | 69984     | 78919     | 50              |
| Specimen #2  | smooth   | 485                             | 70713     | 78380     | 51              |
| Specimen #3  | smooth   | 483                             | 70068     | 78003     | 50              |
| Specimen #4  | smooth   | 490                             | 71442     | 79100     | 51              |
| Specimen #5  | smooth   | 483                             | 70421     | 79322     | 36              |
| Specimen #6  | threaded | 448                             | 65318     | 75893     | 52              |
| Specimen #7  | threaded | 450                             | 65610     | 76184     | 32              |
| Specimen #8  | threaded | 480                             | 69984     | 75083     | 39              |
| Specimen #9  | threaded | 440                             | 64152     | 74471     | 38              |
| Specimen #10 | threaded | 450                             | 65610     | 74918     | 38              |
| Average      |          | 469                             | 68330     | 77027     | 44              |
| CV           |          | 0.04                            | 0.04      | 0.02      | 0.18            |

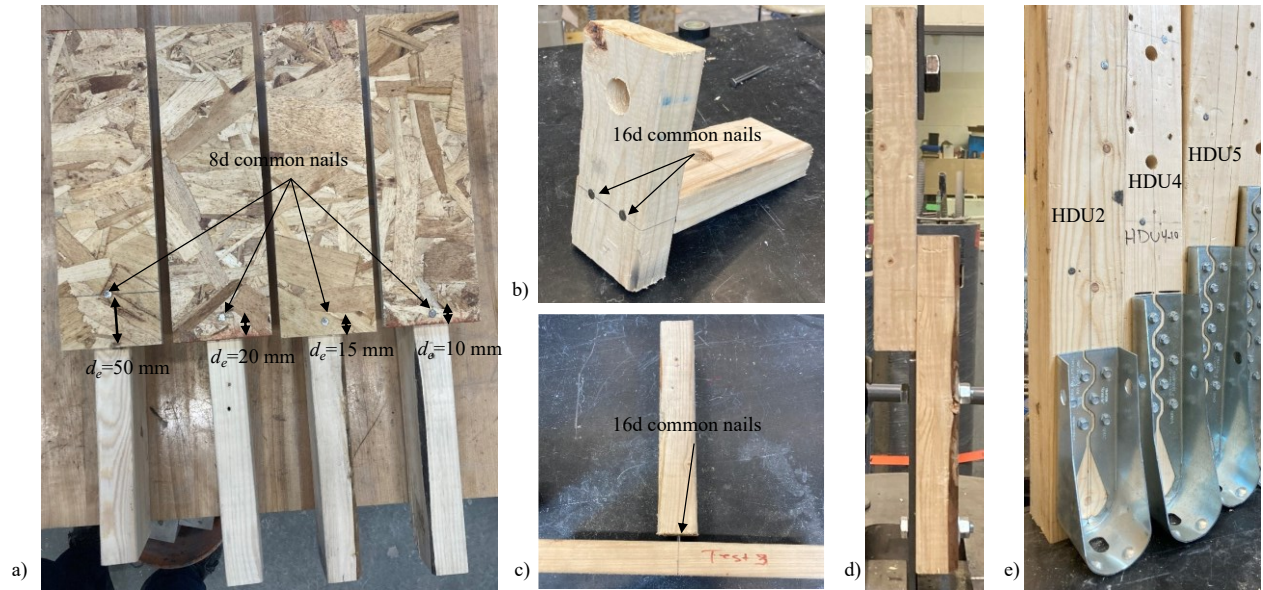
The average  $\sigma_y$  values for ATS rods was 490 MPa and 469 MPa for ATS-SR4 and ATS-SR5, respectively, both of which were higher than the manufacturer-specified 300 MPa (Simpson Strong-Tie 2022). The corresponding  $T_y$  values were 44.9 kN for ATS-SR4 and 68.3 kN for ATS-SR5, which, as expected, also exceeded their design values,  $T_{r\_AST-SR4} = 27.42$  kN and  $T_{r\_ATS-SR5} = 43.36$  kN, provided by the manufacturer (Simpson Strong-Tie 2022).

The average maximum tensile strengths ( $T_u$ ) divided by the respective design resistances yield overstrength factors of 2.1 for ATS-SR4 and 1.8 for ATS-SR5.

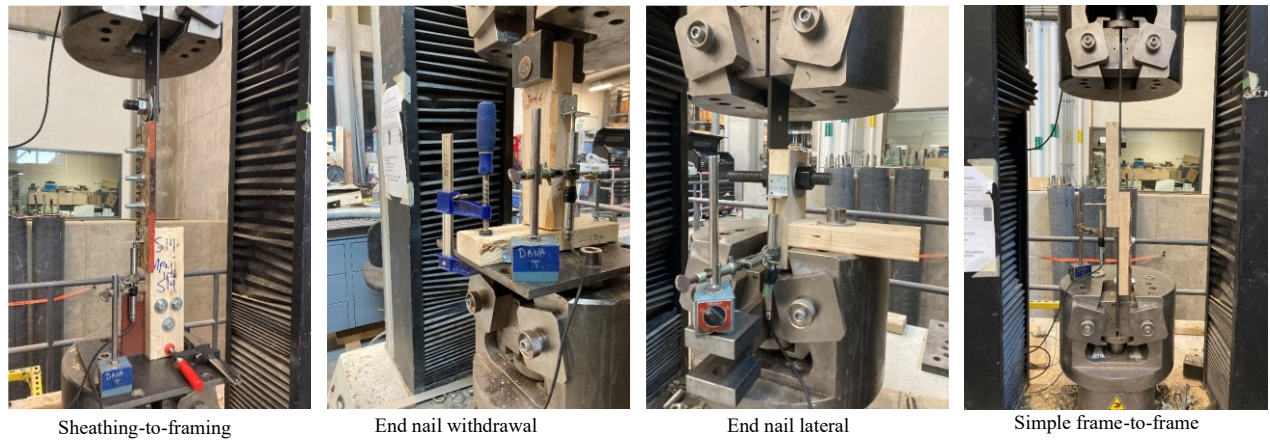
### 4.3 Connection test results

Figure 4.10(a)–(e) present all the connection types investigated in this study, including sheathing-to-framing, framing-to-framing, as well as HD connections. Figure 4.11 shows the nail connections during test.



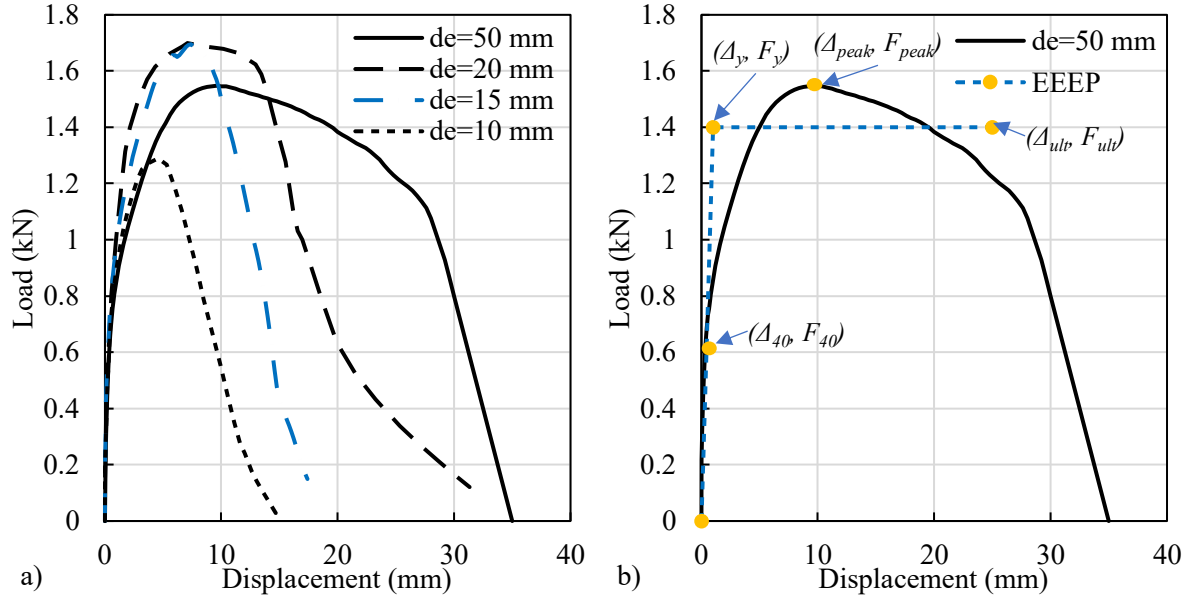


**Figure 4.10: Connection specimens: a) sheathing-to-framing; b) end nail lateral; c) end nail withdrawal; d) frame-to-frame; e) hold-down**



**Figure 4.11: Test setups for nail connection tests**

Figure 4.12(a) shows the average F–D diagrams for sheathing-to-framing connections with edge distances ( $d_e$ ) of 50, 20, 15, and 10 mm.



**Figure 4.12: (a) comparison between sheathing-to-framing connection tests; (b) EEEP idealization**

Figure 4.12(a) shows the significance of the  $d_e$  value on the sheathing-to-framing connection behaviour, particularly ductility and displacement capacity. In general, it can be observed that as the  $d_e$  value decreases, the displacement capacity, the displacement corresponding to peak resistance ( $\Delta_{peak}$ ), and the peak resistance ( $F_{peak}$ ) all decrease.

The equivalent energy elastic-plastic (EEEP) idealization (ASTM E2126 2019), shown in Figure 4.12(b), was used to quantitatively compare the sheathing-to-framing test results. The parameters  $\Delta_{40}$ ,  $\Delta_y$ ,  $\Delta_{peak}$ , and  $\Delta_{ult}$  represent displacements corresponding to 40% of the peak resistance, the yield resistance ( $F_y$ ), the peak resistance, and the ultimate resistance ( $F_{ult}$ ), respectively. The ultimate resistance was taken as  $0.8F_{peak}$ . Table 4-4 summarizes the EEEP parameters.

Table 4-4: EEEP idealization parameters

| $d_e$ | $K_{0-40}=F_{40}/\Delta_{40}$<br>(kN/mm) | $\Delta_y$<br>(mm) | $F_y$<br>(kN) | $\Delta_{peak}$<br>(mm) | $F_{peak}$<br>(kN) | $\Delta_{ult}$<br>(mm) | Ductility<br>$\mu=\Delta_{ult}/\Delta_y$ | Energy<br>(J) |
|-------|------------------------------------------|--------------------|---------------|-------------------------|--------------------|------------------------|------------------------------------------|---------------|
| 50 mm | 1.56                                     | 0.89               | 1.39          | 9.60                    | 1.55               | 24.62                  | 27.54                                    | 33.70         |
| 20 mm | 1.48                                     | 1.07               | 1.58          | 7.13                    | 1.70               | 15.00                  | 14.06                                    | 22.88         |
| 15 mm | 1.93                                     | 0.77               | 1.49          | 7.40                    | 1.69               | 10.93                  | 14.17                                    | 15.70         |
| 10 mm | 1.92                                     | 0.60               | 1.15          | 4.60                    | 1.29               | 7.07                   | 11.84                                    | 7.77          |

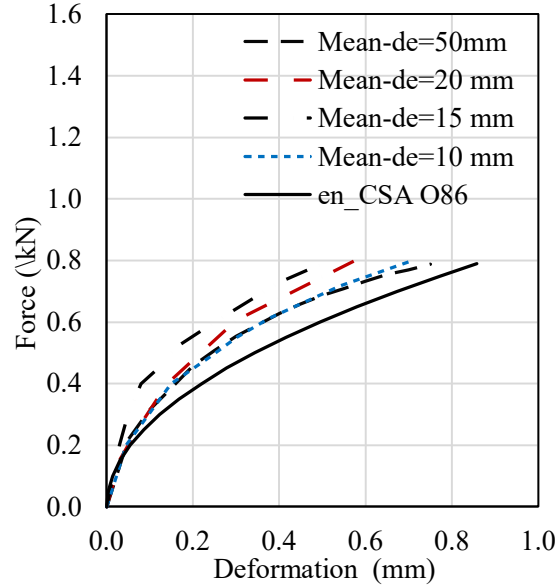


Table 4-4 shows that the ductility of the sheathing-to-framing connections decreased significantly as the  $d_e$  value decreased. The reduction in  $\Delta_{peak}$  value was approximately proportional to the decrease in  $d_e$ . It is noted that, in a wood shear wall, the failure mode of sheathing-to-framing (nail) connections depends on how the force is applied to the connection. In particular, because the nails are located close to the edge of the sheathing panel, sheathing-to-framing connections may experience different failure mechanisms. Accordingly, the selected edge distances were intended to capture a broad range of potential failure modes that can develop in a wood shear wall. Figure 4.13 shows the common failure modes observed in the sheathing-to-framing connection tests.



**Figure 4.13: Sheathing-to-framing failure modes**

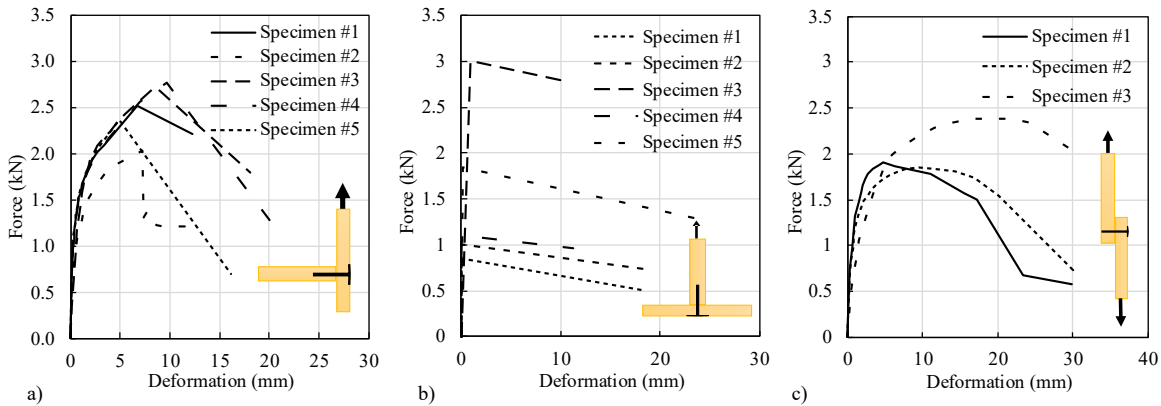
For  $d_e=50$  mm, the nail pull-through failure mode caused significant wood crushing of the framing member around the nail as well as extensive nail bending. For smaller edge distances ( $d_e=10$  and  $15$  mm), the nails were pulled out from the sheathing panel edge, resulting in less local wood crushing around the nail in framing member and less nail bending. For  $d_e=20$  mm, the specimen experienced wood crushing around the nail in the framing member and nail bending, although the nail was ultimately pulled out from the panel edge. These are the types of failure modes that were observed during the shear wall tests. A comparison was conducted between the average curves for each sheathing-to-framing connection category (i.e.,  $d_e=50$ ,  $20$ ,  $15$ ,  $15$ , and  $10$  mm) and the nail slip equation provided in CSA O86 (Canadian Standards Association 2019), as shown in Figure 4.14.



**Figure 4.14: Comparison between different nail slip behaviours**

It can be seen from Figure 4.14 that the nail slip behaviour predicted by CSA O86 (Canadian Standards Association 2019) provides a lower estimate of the sheathing-to-framing connection resistance, while simultaneously predicting higher slip values, particularly at increased nail forces, compared to the test results. However, considering the variability in the experimental data, the CSA O86 (Canadian Standards Association 2019) curve may be regarded as a conservative lower-bound estimate of nail slip behaviour.

Figure 4.15(a)–(c) show the corresponding F–D diagrams for end-nail lateral and withdrawal behaviour, as well as frame-to-frame connections.



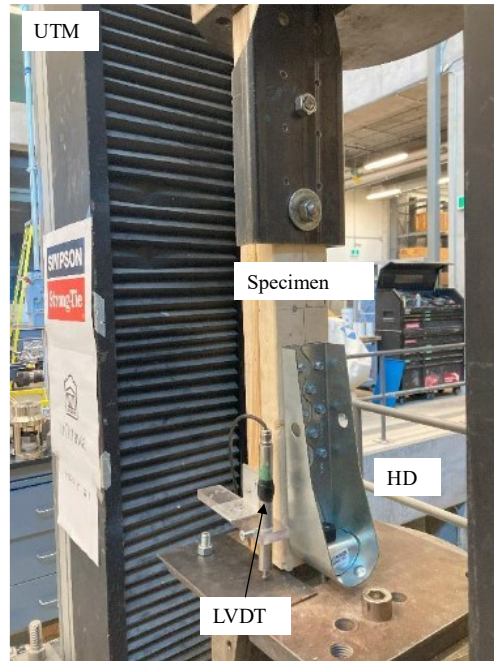
**Figure 4.15: (a) End-nail lateral connection; (b) End-nail withdrawal connection; (c) frame-to-frame connection**

The failure modes corresponding to frame connection tests are shown in Figure 4.16.



**Figure 4.16: Frame connection failure modes**

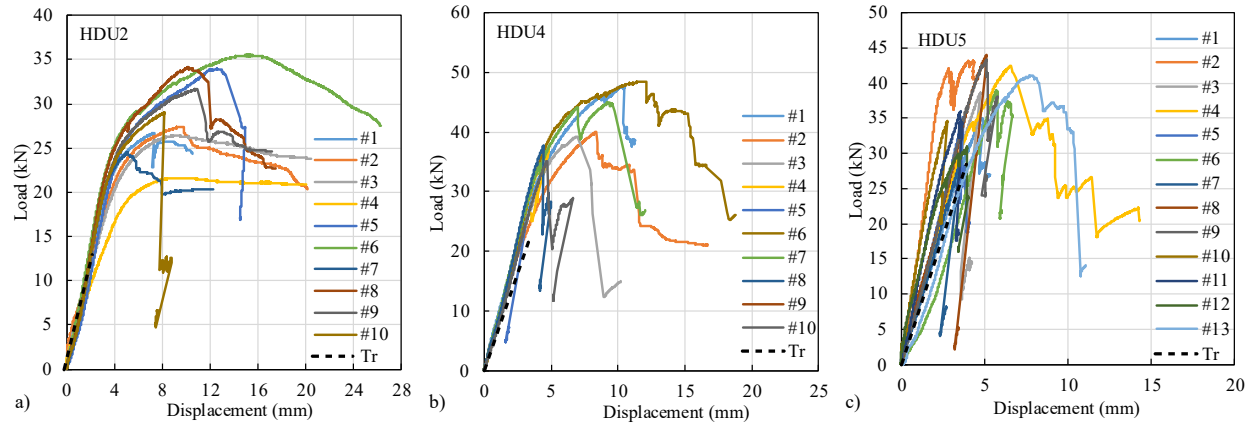
Figure 4.17 shows the HD test setup used in this study.



**Figure 4.17: HD test setup**

Figure 4.18(a)–(c) show the F–D curves for the HD connections (e.g., HDU2, HDU4, and HDU5).

The dashed lines in each figure connect the origin to the point corresponding to the  $T_r$  value of each HD device, as provided by the manufacturer (Simpson Strong-Tie 2022). These lines allow comparison between the measured behaviour and manufacturer’s curve of the HD connections.



**Figure 4.18: F–D diagrams of HD connections**

In terms of initial stiffness, the initial portion of the ascending measured F–D curves shows reasonable agreement with those provided by the manufacturer (Simpson Strong-Tie 2022).

In terms of the maximum resistance measured during testing, different HD devices exhibited varying levels of overstrength values.

While Figure 4.18(a) and (b) indicate onset of yielding in the response of HDU2 and HDU4 devices at a displacement of approximately 5 mm, Figure 4.18(c) indicates that HDU5 exhibited a comparatively more brittle response. This behaviour could be attributed to the more screws in HDU5 compared to HDU2 and HDU4, which may promote a more brittle failure mode.

The most common failure mode in HDU2 devices was significant wood crushing around the connecting screws. The HD itself deformed noticeably in some cases, while stud fracture due to bending at the top of the HD (last row of screws) was uncommon. In contrast, HDU4 and HDU5 devices mostly failed due to splitting and fracture of the chord member at the last row of HD screws. Chord fracture was more likely if wood defects were present at the end of the HD. Due to the eccentricity of the applied load and the HD anchor rod, HDU4 and HDU5 devices experienced high bending moments in addition to tension at elevated load levels. This increased the stress concentration at the top of the HD, promoting stud bending fracture.



Figure 4.19 shows the different failure modes observed in the HD tests.



**Figure 4.19: HD failure modes**

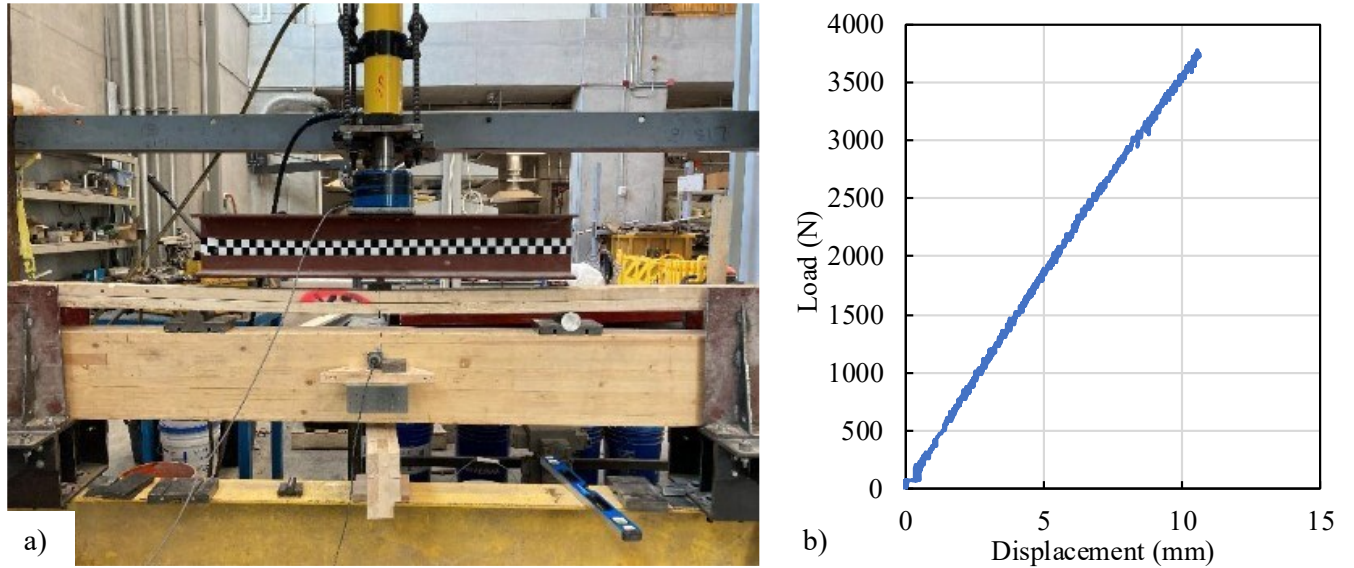
Table 4-5 summarizes the HD test data. The parameters in this table were defined the same way as in Table 4-4 for sheathing-to-framing connections.

Table 4-5: Summary of HD test results

|              | HDU2       |                 |            | HDU4       |                 |            | HDU5       |                 |            |
|--------------|------------|-----------------|------------|------------|-----------------|------------|------------|-----------------|------------|
| Specimen     | $K_{0-40}$ | $\Delta_{peak}$ | $F_{peak}$ | $K_{0-40}$ | $\Delta_{peak}$ | $F_{peak}$ | $K_{0-40}$ | $\Delta_{peak}$ | $F_{peak}$ |
|              | (kN/mm)    | (mm)            | (kN)       | (kN/mm)    | (mm)            | (kN)       | (kN/mm)    | (mm)            | (kN)       |
| Specimen #1  | 6.35       | 7.34            | 26.70      | 8.37       | 10.28           | 47.69      | 7.74       | 4.84            | 35.01      |
| Specimen #2  | 6.08       | 9.56            | 27.44      | 7.88       | 8.35            | 39.99      | 16.09      | 4.00            | 43.17      |
| Specimen #3  | 5.53       | 8.97            | 26.37      | 7.73       | 6.98            | 39.30      | 3.69       | 4.72            | 38.58      |
| Specimen #4  | 4.87       | 9.17            | 21.62      | 7.97       | 4.65            | 37.40      | 8.79       | 6.53            | 42.40      |
| Specimen #5  | 5.64       | 12.57           | 33.94      | 7.63       | 3.03            | 24.23      | 9.07       | 3.70            | 35.11      |
| Specimen #6  | 7.17       | 15.15           | 35.47      | 8.34       | 11.95           | 48.52      | 5.60       | 5.66            | 38.95      |
| Specimen #7  | 6.23       | 5.00            | 24.44      | 8.70       | 9.36            | 45.03      | 7.93       | 3.47            | 28.74      |
| Specimen #8  | 6.69       | 10.14           | 34.10      | 8.46       | 4.44            | 37.82      | 9.03       | 5.09            | 43.97      |
| Specimen #9  | 6.27       | 10.90           | 31.66      | 7.84       | 5.29            | 42.05      | 8.84       | 5.08            | 43.29      |
| Specimen #10 | 6.38       | 8.18            | 29.01      | 7.50       | 4.69            | 35.12      | 16.01      | 2.74            | 34.61      |
| Specimen #11 |            |                 |            |            |                 |            | 11.84      | 3.52            | 35.94      |
| Specimen #12 |            |                 |            |            |                 |            | 12.64      | 3.93            | 30.90      |
| Specimen #13 |            |                 |            |            |                 |            | 6.71       | 7.84            | 41.15      |
| Average      | 6.12       | 9.70            | 29.08      | 8.04       | 6.90            | 39.72      | 9.54       | 4.70            | 37.83      |
| STDV         | 0.64       | 2.80            | 4.58       | 0.40       | 2.95            | 7.03       | 3.70       | 1.39            | 4.91       |
| CV           | 0.11       | 0.29            | 0.16       | 0.05       | 0.43            | 0.18       | 0.39       | 0.30            | 0.13       |

According to Table 4-5, the average initial stiffness values ( $K_{0-40}$ ) for HDU2, HDU4, and HDU5 connections are greater than the manufacturer-provided values of 5.5, 6.5, and 7.3 kN/mm, respectively (Simpson Strong-Tie 2022). However, the variability of this parameter is pronounced, particularly for HDU5 that shows a CV value of 0.4. Comparison of the average  $F_{peak}$  values for individual HDs with their corresponding factored resistances ( $T_r = 12.9, 21.5,$  and  $28.4$  kN for HDU2, HDU4, and HDU5) showed varying levels of overstrength.

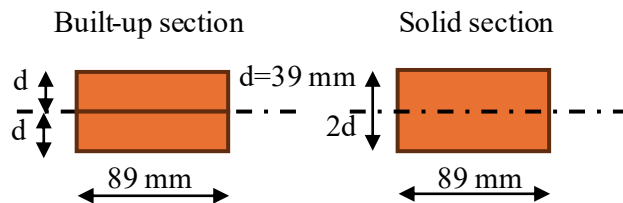
Figure 4.20(a) and (b) show the built-up member test and its load-displacement curve, respectively.



**Figure 4.20: (a) Built-up member test; (b) F–D curve**

Similar to the stud bending test calculations, the flexural rigidity of the built-up member ( $EI_{built\ up}$ ) was calculated using Eq. (4.1) and the slope of the load-displacement curve ( $P/\Delta_c=360.12\text{ N/mm}$ ) was obtained from Figure 4.20(b). The corresponding value of  $EI_{built\ up}$  was  $1.16 \times 10^{10}\text{ N}\cdot\text{mm}^2$ .

To quantify the bending rigidity ratio of the built-up member, the value of  $EI_{built\ up}$  was divided by the flexural rigidity of a solid member with the same cross-sectional area ( $EI_{solid}$ ), as illustrated in Figure 4.21.

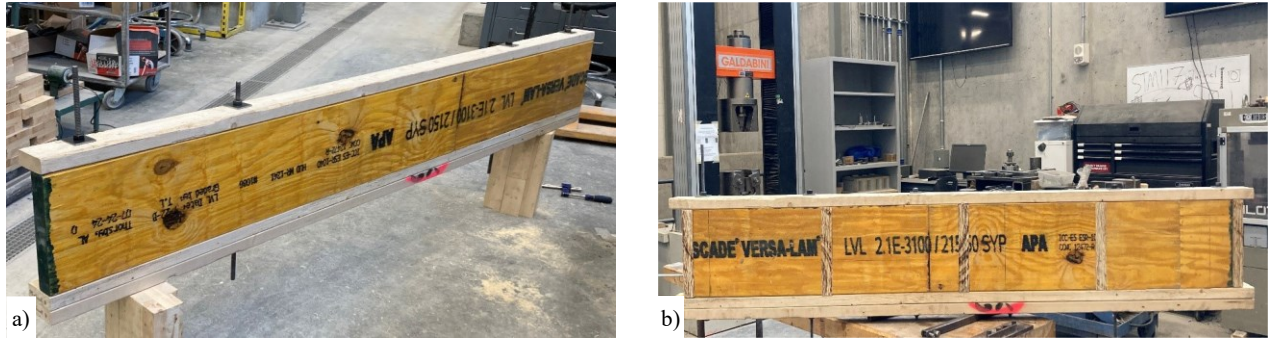


**Figure 4.21: Representation of built-up section with its corresponding solid section**

The bending rigidity ratio ( $EI_{built\ up} / EI_{solid}$ ) was 0.4. This value was used as a modification factor to the moment inertia of the shear wall chord members in the numerical analysis.

#### 4.4 Floor assembly tests

Figure 4.22(a) and (b) show the floor assembly specimens with continuous LVL joists and discrete blockings, respectively.



**Figure 4.22: Floor assemblies: a) continuous LVL; b) discrete blocking**

Floor assembly tests were not loaded to failure, as the focus of this study was on the stiffness of the assembly, which is governed by the initial linear portion of the load-displacement curve. Conducting non-destructive tests allowed investigation of the effect of the number of ABs on floor assemblies with continuous LVL joists and blockings. This approach was adopted to evaluate the sensitivity of the floor assembly stiffness to the number of ABs. To achieve this, after each test the specimen was unloaded, and the number of ABs was increased or decreased.

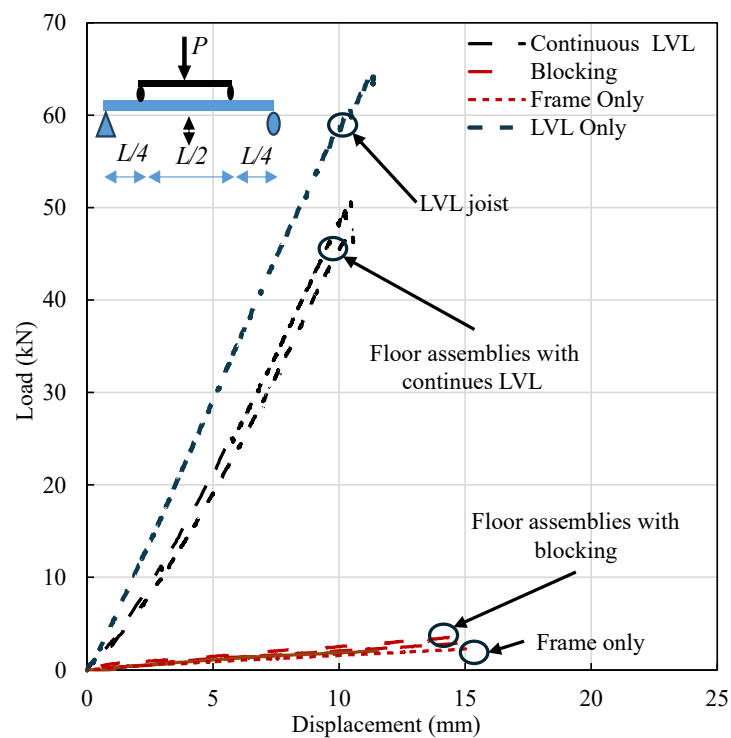
Figure 4.23 shows the different floor assembly configurations tested in this study. Specimens with six ABs showed minimal separation between the LVL joist and the top and bottom plate members, whereas specimens with only two ABs exhibited noticeable gaps. In contrast, large gap openings were observed in floor assemblies with blockings, regardless of the number of ABs. The load-displacement curves of the floor assemblies are then summarized in Figure 4.24.

Given that material and connection-level behavior are not the primary focus of this study, a detailed discussion of the materials, connections, and their associated failure modes is presented in Appendix B.





**Figure 4.23: Floor assembly test configurations**



**Figure 4.24: Load-displacement diagrams of floor assembly tests**

Figure 4.24 shows that the LVL-only specimen exhibited higher initial stiffness than the assemblies with top and bottom plates. However, as the load increased, the stiffness of the assemblies with continuous LVL and plates approached that of the LVL-only specimen. The lower initial stiffness of these assemblies was attributed to incomplete engagement of the components due to initial slippage between members. Floor assemblies with discrete blockings exhibited significantly lower stiffness than those with continuous LVL. The number of ABs had a negligible effect on the stiffness of the floor assemblies, both with continuous LVL and with blockings.

#### **4.5 Full-scale shear wall tests**

This section presents the results from the full-scale shear wall tests. It is noteworthy to mention that in configurations with multiple shear walls, individual walls are referred to as SEG1 (Segment 1), SEG2, and so on, starting from the actuator side. Measured data from the full-scale shear wall tests were compared with their corresponding values at a load level equal to the factored resistance of each shear wall configuration ( $V_r$ ). This load level, hereafter referred to as the ‘design level,’ is used consistently throughout the following discussion

In the following, Configuration #1 is described in detail, while for other configurations, specific commentary regarding the reported configuration would be highlighted. A complete set of data for all configurations can be found in Appendix A.

##### **4.5.1 Configuration #1**

Figure 4.25 shows the deformed shape of Configuration #1 after the test. In this figure, a red vertical line at SEG1 indicates the original position of the wall.



**Figure 4.25: Shear wall test setup in Configuration #1**

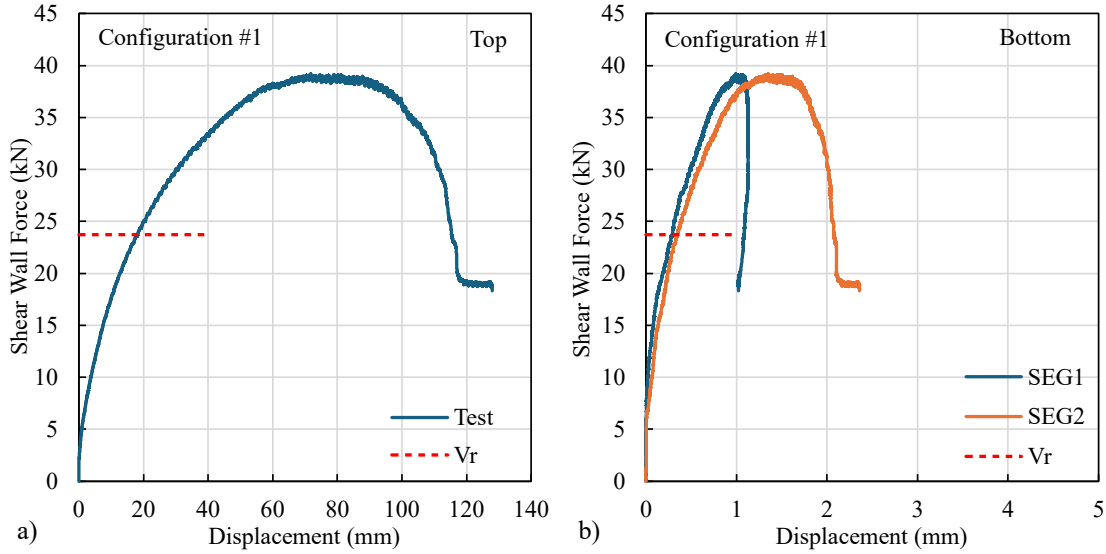
A total of 19 DTs were utilized to record key deformations of the shear walls in this configuration. These measurements included the top and bottom displacements of the shear wall; nail and sheathing panel shear deformations in SEG1 and SEG2; HD elongations and perpendicular-to-grain compression deformations in both SEG1 and SEG2; and lateral displacements of the compression chord in SEG2 along the height of the shear wall. In addition, two LCs were utilized to measure the HD forces in SEG1 and SEG2.

Prior to applying the lateral load, the tension HDs were tightened, and initial forces were recorded: 2.06 kN for SEG1, and 2.2 kN for SEG2. These values were used to calculate the actual HD forces after applying lateral loading.

During testing, the condition of the shear wall was documented at the design load, no local failure was observed when the actuator load was 23.7 kN, corresponding to the design level value,  $V_r$ , of the specimen, according to CSA O86 (Canadian Standards Association 2019). The load reached a peak load of 39.9 kN at a displacement of 71.64 mm (2.9% IDR). The test continued until a displacement of 127.83 mm (5.2% IDR) was reached, where the test was terminated. At this

displacement, the shear wall resistance was 18.5 kN, corresponding to 46% of peak load and 78% of the design level capacity.

Figure 4.26(a) and (b) show the F–D curves for the shear walls in Configuration #1.

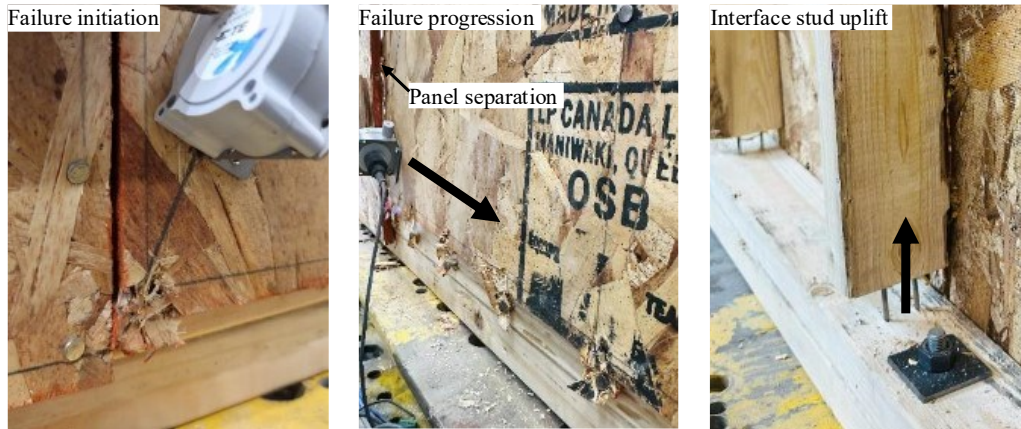


**Figure 4.26: Shear wall F–D curves: (a) top displacement; (b) sliding at the bottom of the shear wall**

Figure 4.26(a) displays the F–D curve for shear wall configuration #1. The graph depicts a typical load-displacement curve for LFW shear walls with start of nonlinear behaviour at early levels of loading, a yield region that is not easily distinguishable, and significant amount of post-yield displacement capacity. When the actuator load reached 37 kN, failure occurred at the bottom-left corner of the wider panel in SEG1, where the corner of the sheathing panel tore and nails pulled out. Subsequently, the failure propagated along the bottom edge of the sheathing, causing significant uplift at the panel interface (Figure 4.27).

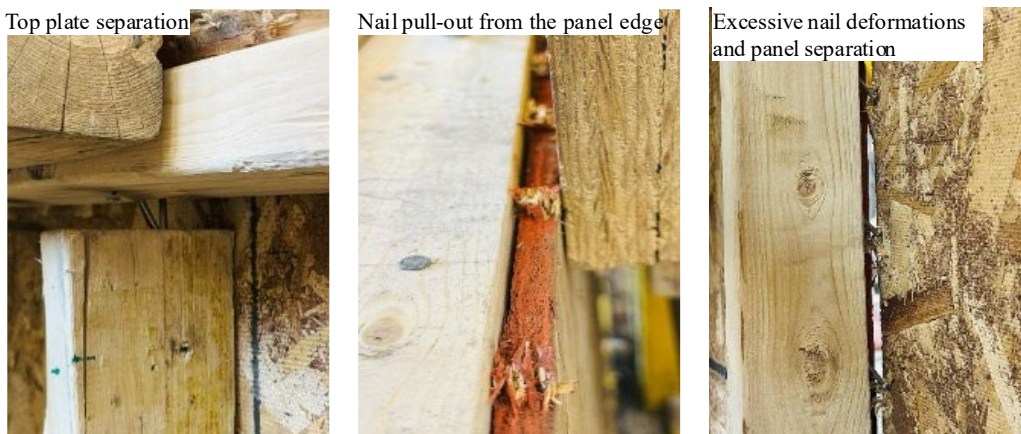
Figure 4.26b shows minimal sliding at the bottom of the shear wall with less than 0.5 mm displacement at a load equal to  $V_r$ , indicating that the designed ABs were sufficient.





**Figure 4.27: Initiation and progression of the failures in Configuration #1**

At SEG1, the uplift of the adjoining stud at the panel interface caused severe nail damage along the right edge of the narrow panel, resulting in its separation from the stud. The uplift of stud at this location also pushed the top plates, leading to separation from vertical stud. This caused nails at the top edge of the narrow sheathing panel to fail (Figure 4.28).



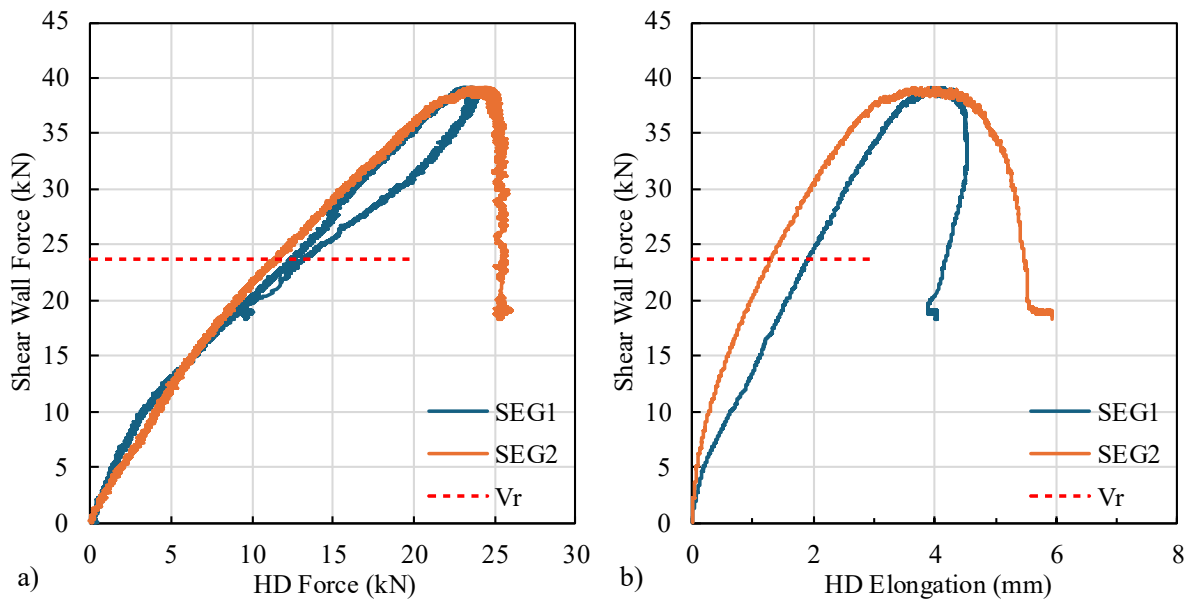
**Figure 4.28: Failure modes at the top and sheathing panel interface in SEG1**

Although limited amount of slippage occurred between the HD device and the chord member, no HD failure was observed by the end of the test, as depicted in Figure 4.29.

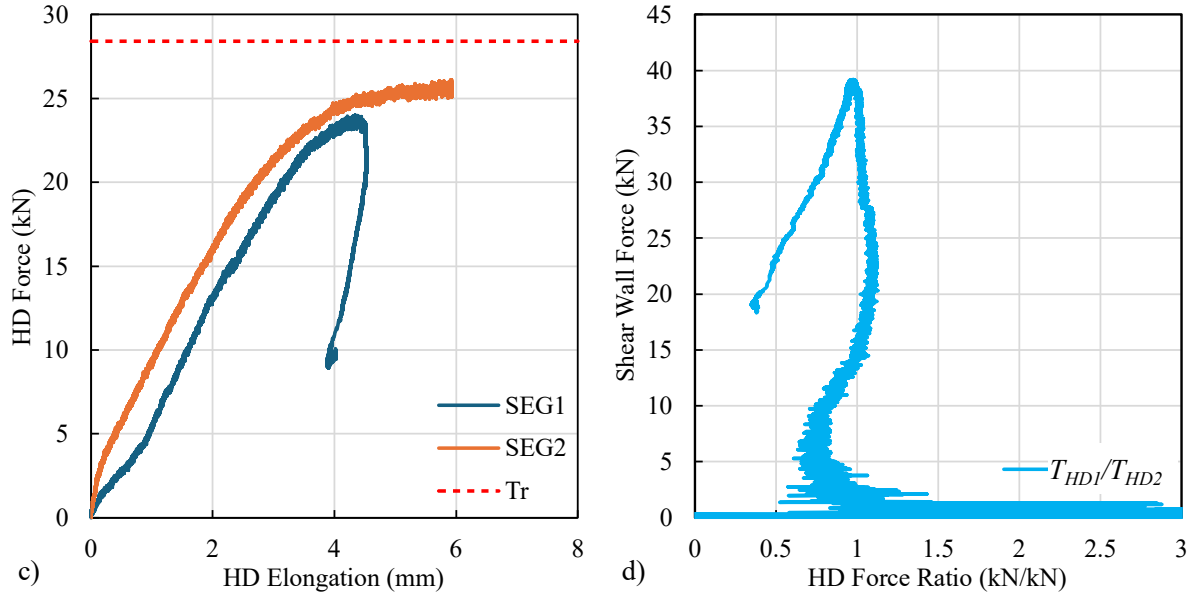


**Figure 4.29: HD (HDU5) slippage in shear wall Configuration #1**

Figure 4.30(a) and (b) present the force in the HDs and the HD elongations for SEG1 and SEG2 as function of the applied load on the shear wall. A comparison was made between HD design forces and those obtained from the test. At the shear wall design load level ( $V_r = 23.7$  kN), the experimental HD forces in SEG1 and SEG2 were 12.4 kN and 11.5 kN, which were 44% and 36% lower than the corresponding design values. The associated elongations were 2.2 and 1.6 mm, which were 30% and 35% lower than the design values.



**Figure 4.30: HD force and displacement demands in Configuration #1**



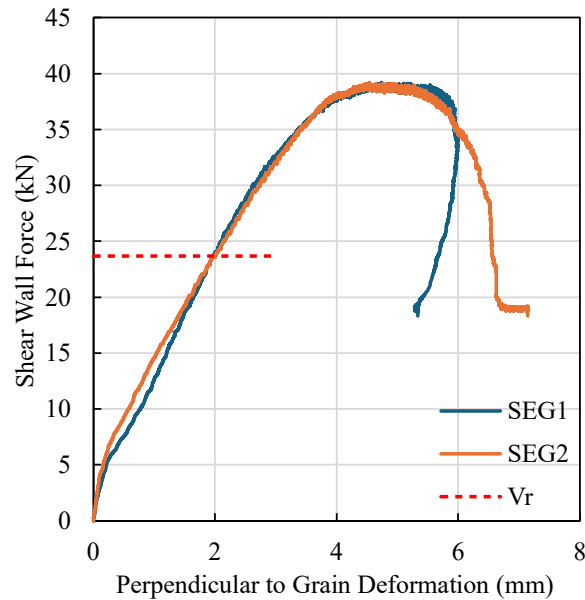
**Figure 4.30: HD force and displacement demands in Configuration #1 (continued)**

Figure 4.30(c) shows that both HDs in SEG1 and SEG2 behaved nearly linearly and did not reach their  $T_r$  value by the time the shear wall reached its peak load. After the shear wall surpassed its peak, the force in the HD of SEG1 began to decrease, reflecting that the ultimate failure in SEG1 preceded failure in SEG2. The plateau in the HD force-elongation curve of SEG2 is not indicative of failure; rather, it reflects geometric nonlinearity due to large rotations of the HD. This phenomenon is more clearly reflected in Figure 4.30(a) and (b), where the HD force in SEG2 remained nearly constant after the shear wall surpassed its peak resistance Figure 4.30(a), while the HD elongation in Figure 4.30(b) continued to increase slightly. This increase is nothing more than the result of the rotation of the chord member at the base of the shear wall.

Figure 4.30(d) illustrates the ratio of HD forces in SEG1 and SEG2 ( $T_{HD1}/T_{HD2}$ ). This figure shows that, during most of the shear wall loading, the HDs in SEG1 and SEG2 carried nearly equal forces. At low shear wall load levels, the ratio indicates that the HD in SEG1 exhibited slightly lower force than the HD in SEG2, likely due to non-uniform force distribution among shear wall elements at

the beginning of loading. As the force in the shear wall increased and all elements became engaged, the ratio stabilized and fluctuated around 1.0.

Figure 4.31 shows the perpendicular-to-grain compression deformations in SEG1 and SEG2.



**Figure 4.31: Perpendicular-to-grain compression deformations (Configuration #1)**

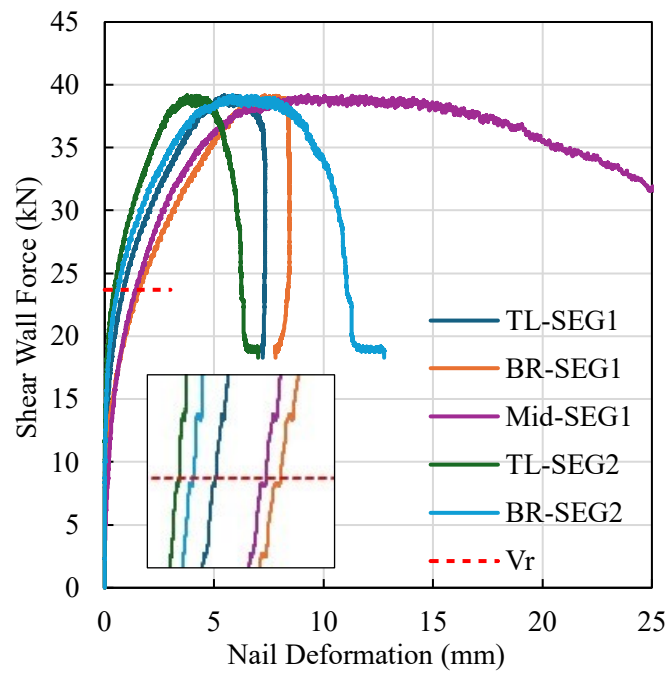
The test results in Figure 4.31 show equal deformations of 2 mm for both SEG1 and SEG2 at the shear wall design load level. These deformations are considerably higher, five and six times the design values for SEG1 and SEG2, respectively, than the design values of  $d_{a2}=0.41$  mm and  $d_{a2}=0.33$  mm listed in Table 3-8. Figure 4.32 illustrates the non-uniform perpendicular-to-grain compression deformations of the shear wall bottom plate. However, it is noted that this deformation is not uniform beneath the chord member and represents the maximum indentation into the bottom plate at the outer edge of the chord.





**Figure 4.32: Perpendicular-to-grain compression deformations**

Figure 4.33 illustrates the nail deformations of SEG1 and SEG2. The nail deformations at the shear wall design force,  $V_r$ , are magnified for clarity and to facilitate comparison between different nail deformation components.

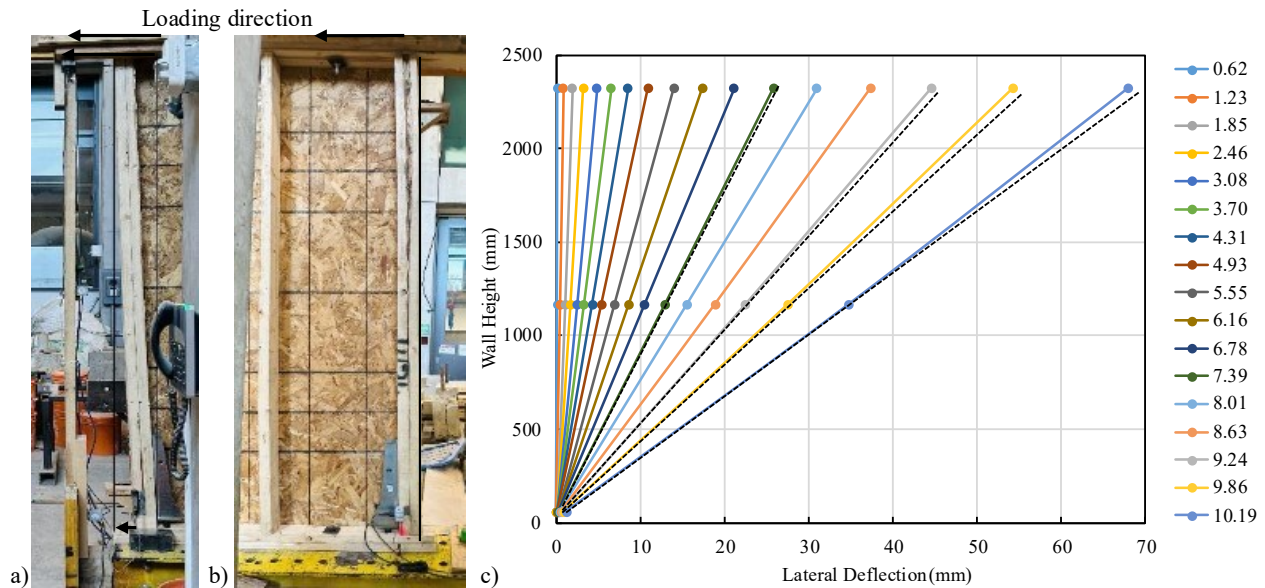


**Figure 4.33: Nail deformations in Configuration #1**

Figure 4.33 shows that nail deformations in SEG1 were greater than those in SEG2, with average values of 1.2 mm and 0.53 mm, respectively, which were almost consistent with the design values of  $e_n=1.04$  mm and  $e_n=0.54$  mm, as listed in Table 3-7. In SEG1, nail deformations along the

perimeter of the larger panel (Mid-SEG1 and BR-SEG1) were greater than in the smaller panel (TL-SEG1). The TL-SEG1 and BR-SEG1 nail deformation components remained nearly constant after the peak shear wall resistance. The continuously increasing Mid-SEG1 component indicates that nail deformations were concentrated at the interface between the panels, which is consistent with the observed failure localization at that location (Figure 4.28).

Figure 4.34(a) and (b) show the deformed shapes of the chord members in SEG2. While the chord members remained nearly straight during the test, the tension and compression chords exhibited slightly different deformed shapes. This difference was due to the boundary conditions surrounding each member. Figure 4.34(c) depicts the lateral deflection of the compression chord along the height of the shear wall at different load levels. These curves nearly remained straight with minimal bending. The straight lines were drawn tangential to the lower portion of the selected curves, and the difference between these lines and the upper portion of the curves represents the magnitude of the bending deformations.



**Figure 4.34: Bending deformation in SEG2: a) compression chord; a) tension chord; c) lateral deflection diagrams of compression chord in SEG2**

The measured shear deformations of the sheathing panels were very small and insignificant to contribute to the overall wall deformation.

#### 4.5.2 Configuration #2

Figure 4.35 shows the shear walls in Configuration #2, which is in principle similar to Configuration #1 but with an added floor assembly constructed above and connected to the shear wall.



**Figure 4.35: Configuration #2**

The instrumentation scheme was similar to that of Configuration #1, except that an additional DT was installed at the panel interface in SEG1 to capture the uplift of the adjoining stud, as this was observed during Configuration #1 testing.

The specimen reached a peak resistance of 39.6 kN at a displacement of 75.93 mm (3.1% IDR). The test continued until a final displacement of 141.35 mm (5.8% IDR), at which point the specimen had a residual capacity of 19 kN. The shear wall resistance at this final displacement corresponded to 48% of the peak load and 80% of the design load. No damage was observed until the shear wall load reached its design resistance of 23.7 kN

In general, the failure modes observed in Configuration #2 were similar to those observed in Configuration #1 (Figure 4.27). A notable difference, however, was the more uniform distribution of nail deformations across both shear wall segments. This was due to greater mobilization of the shear wall in a racking mode and increased sheathing panel rotation, resulting in a larger number of nails being engaged along the height of the shear wall. Regarding the HDs, minimal slippage between the HD devices and the chord members was observed, similar to that shown in Figure 4.29, and no signs of failure were observed when the shear wall reached its peak resistance and throughout the end of the test.

Both HDs behaved linearly and reached ultimate values that were smaller than the anticipated  $T_r$  value of 28.4 kN. The HD forces in SEG1 and SEG2 were 10.2 kN and 9.36 kN, respectively, when the shear wall reached its design force,  $V_r$ , of 23.7 kN. These values were significantly lower than the corresponding design-level forces of 22.9 kN and 18.1 kN, representing reductions of 55% and 48%, respectively. Similarly, the test results showed HD elongations of 1.2 mm and 0.93 mm for SEG1 and SEG2, respectively. The corresponding design values were 3.1 mm and 2.5 mm, showing a significant reduction of 62% in both shear walls. The HD force ratio ( $T_{HD1} / T_{HD2}$ ) was similar to Figure 4.30(d) but fluctuated around 1.1 which showed a slightly higher difference in the ratio between HD forces in SEG1 and SEG2.



The perpendicular-to-grain compression deformations under the chord members in SEG1 and SEG2 were 2 mm, equal to those observed in Configuration #1, when the shear wall load reached its design level value.

The average nail deformations for SEG1 and SEG2 at the load level equal to  $V_r$  were 1.3 mm and 0.7 mm, respectively. These values showed 25% and 30% increase in nail deformations compared to the design counterparts.

Similar to Configuration #1, the lateral deformations of the compression chord in SEG2 along the height of the shear wall showed nearly linear behaviour, which means relatively small bending deformations.

#### **4.5.3 Configuration #3**

The specimen in this configuration was loaded until it reached a maximum force of 40.4 kN at a lateral displacement of 84.1 mm, corresponding to 3.3% IDR. The test continued until the shear wall reached a lateral displacement of 140 mm (5.7% IDR), at which point it maintained a force level of 33.6 kN, which represented 80% of the maximum load. Relative to the shear wall design force of 23.7 kN, the resistance at the end of the test was approximately 40% higher.

No signs of local damage were observed until the applied load reached the specimen design shear force,  $V_r = 23.7$  kN. Near the peak shear force, damage initiated at the corners of both SEG1 and SEG2, where the nails tore the edges of the sheathing panels and were pulled out perpendicular to the panel edges as illustrated in Figure 4.36.

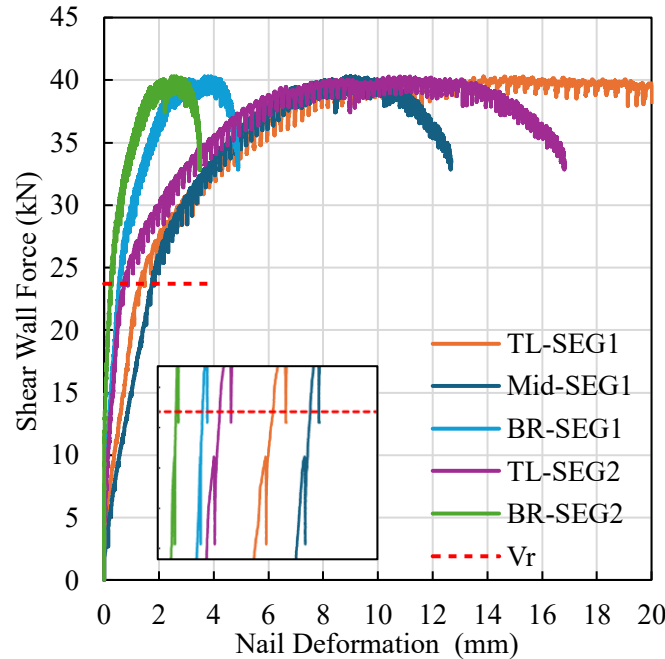


**Figure 4.36: Sheathing panel tearing and nail failures in Configurations #3**

Figure 4.37 illustrates the nail deformations across Configuration #3. Larger nail deformations were observed along the perimeter of the larger panels ( $4' \times 8'$ ) compared to those along the perimeter of the narrow panels ( $1' \times 8'$ ).

At the shear wall design load level, the average nail deformation along the panel edges of the large panel in SEG1 was 1.5 mm, whereas the corresponding deformation for the smaller panel was 0.5 mm. The design-level nail deformation was 0.86 mm. Considering the nail deformations along the edges of the larger panel, being the primary contributor to the total lateral displacement of the shear wall, the measured value represents a 75% increase relative to the design value.

Nail deformations in SEG2 were lower than those in SEG1. In SEG2, the average nail deformation along the perimeter of the larger panel was 1.1 mm, compared to 0.24 mm along the perimeter of the smaller panel. These observations highlight the influence of the sheathing panel AR on nail deformations.



**Figure 4.37: Nail deformation in Configuration #3**

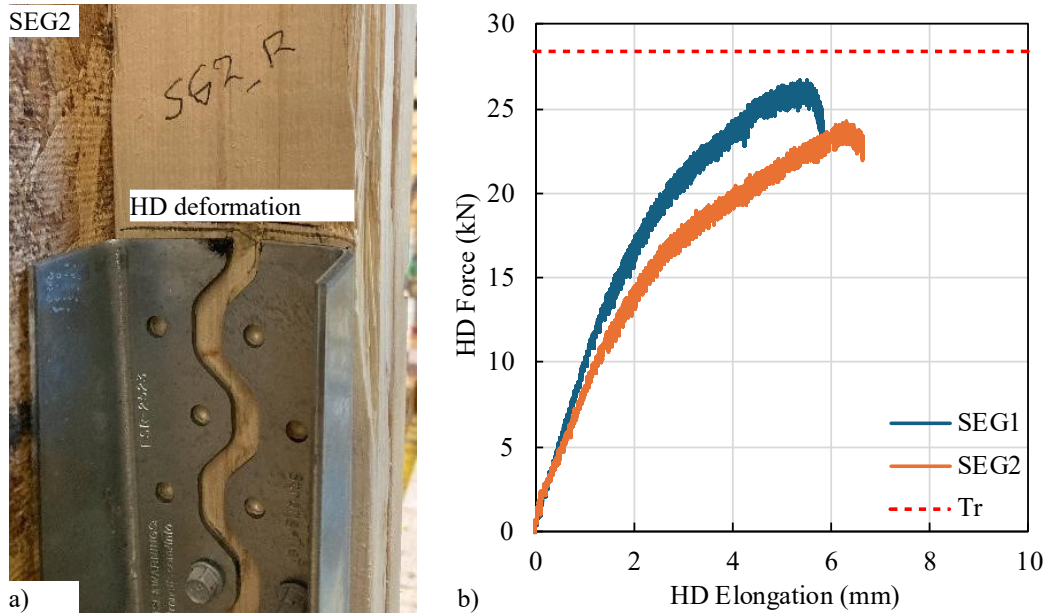
The nails along the top edge of the narrow panel in SEG1 were damaged and pulled out from the panel edge. This was caused by the separation of the top plates from vertical studs. This failure mode did not occur in SEG2 due to a different boundary condition (see Figure 4.38).



**Figure 4.38: Top plates separation and nail damages at the top of shear walls (Configuration #3)**

Contrary to Configurations #1 and #2, where significant uplift of the adjoining stud at the panel interface was observed, the interface stud did not uplift in this configuration. This behaviour resulted from the placement of the larger panel, whose clockwise rotation exerted a substantial downward force on the adjoining stud.

Figure 4.39(a) and (b) illustrate the HD deformation in SEG2 at the end of the test, along with the HD force-elongation curves for the HDs in Configuration #3.



**Figure 4.39: (a) deformation of HD in SEG2; (b) HD force-displacement curves**

The HDs exhibited moderate deformations while the force demands remained below the  $T_r$  value of 28.4 kN. At the shear wall design level, the measured HD forces were 12.8 kN and 11.3 kN for SEG1 and SEG2, respectively, which were considerably lower than the corresponding design value of 21.4 kN for both SEG1 and SEG2, a reduction of 40% for HD in SEG1 and nearly 50% for the HD in SEG2. The HD elongations at the same load level were 1.3 mm and 1.4 mm for HDs in SEG1 and SEG2, respectively. These values show nearly 55% lower than their design values 2.9 mm.

The perpendicular-to-grain compression deformations in SEG1 and SEG2 were both measured at 0.92 mm, approximately half of those observed in Configurations #1 and #2 (2 mm), yet more than twice the design value of 0.4 mm.



#### 4.5.4 Configuration #4

The specimen in this configuration reached its maximum capacity of 26.2 kN at the lateral displacement of 102.5 mm corresponding to 4.2% IDR. Loading continued until the shear wall lateral displacement reached 168.2 mm (6.9% IDR), at which the test was terminated. The corresponding shear wall resistance at this displacement level was 19.6 kN, representing 75% of the peak resistance and remaining approximately 20% higher than the design shear force of 15.8 kN. The dominant failure mode in this configuration was excessive nail yielding in both shear walls.

No local damage was observed until the specimen reached the shear wall design force,  $V_r = 15.8$  kN, at which point the displacement was 22.5 mm, exceeding the design value of 18.3 mm. Initial local damage occurred at a shear wall force of approximately 24 kN, marked by nail failure at the bottom corner of SEG1. This was followed by nail failure at the bottom corner of SEG2 as the shear wall approached its peak resistance. Subsequently, nail failure occurred at the top-right corner of SEG1 and propagated to other nails along the top and bottom edges of both shear walls (see Figure 4.40).

Nail failures at the top of SEG1 were more severe than those in SEG2, attributed to excessive gap opening between the top plates and the shear wall chord in SEG1, as highlighted in Figure 4.40.



**Figure 4.40: Nail failures at the top and bottom edges of the sheathing panel**

The average measured nail deformations in SEG1 and SEG2 were 1.8 mm and 1.4 mm, respectively, which are significantly higher than the design value of 0.86 mm. Compared with the design value, the measured deformations represent increases of 110% in SEG1 and 63% in SEG2. At peak shear wall resistance, the forces in the HDs remained below the factored tensile resistance,  $T_r = 28.4$  kN. At the shear wall design level,  $V_r = 15.8$  kN, the measured HD forces were 12.15 kN and 9.3 kN in SEG1 and SEG2, respectively. These values are significantly lower than the corresponding HD forces at the design level,  $T_d = 22.7$  kN, representing reductions of 47% and 60% in SEG1 and SEG2, respectively.

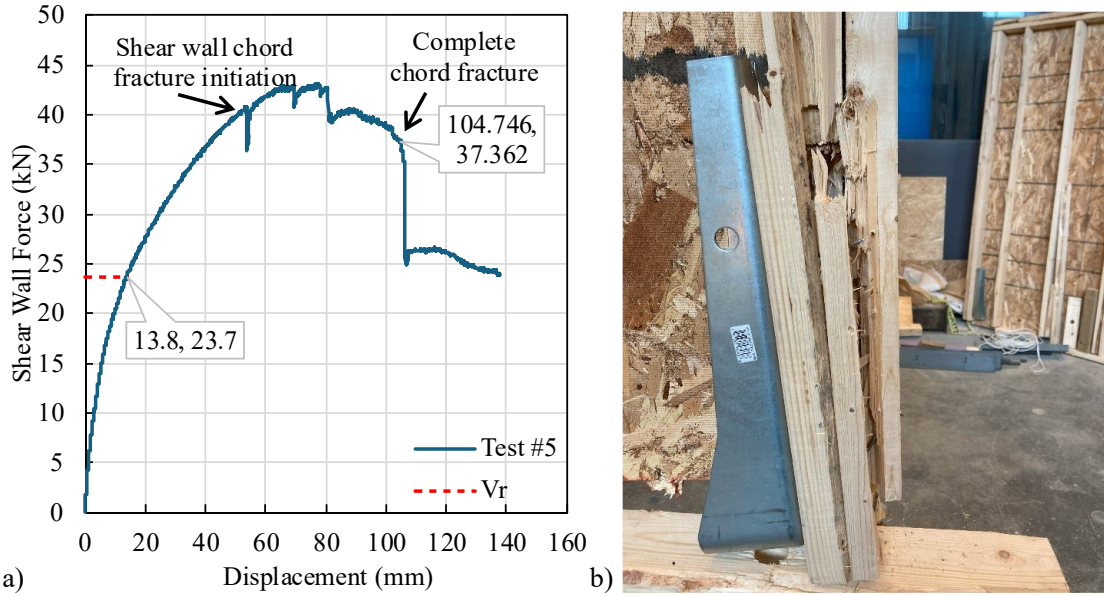
The corresponding HD elongations at the shear wall design level were 1.3 mm and 1.5 mm for SEG1 and SEG2, respectively. Compared with the HD design elongation of 3.1 mm, these values represent reductions of 58% and 50% in SEG1 and SEG2, respectively.

At a load level equal to the shear wall design load ( $V_r = 15.8$  kN), the perpendicular-to-grain compression deformations were 1.8 mm and 1.7 mm for SEG1 and SEG2, respectively. The corresponding design values were 0.42 mm for both segments.

#### **4.5.5 Configuration #5**

This configuration is the same as Configuration #2 except that in this configuration the HD connections were added at the floor level. Interestingly, the inclusion of the HD connection at the floor level shifted the failure mode from nail yielding and panel edge tearing to chord fracture at the HD location of SEG1 and significantly affected the force and deformation distribution in the specimen.

Figure 4.41(a) and (b) illustrate the shear wall F–D response of Configuration #5 and the fractured shear wall chord at the HD location in SEG1, respectively.



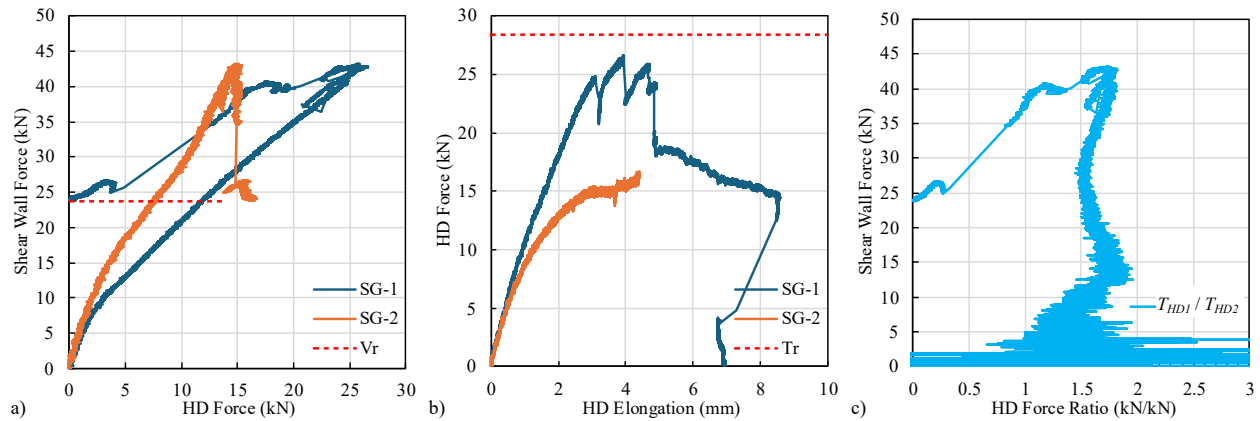
**Figure 4.41: (a) Shear wall F–D behaviour in Configuration #5; (b) Fractured HD in SEG1**

The specimen was loaded to a peak resistance of 43.16 kN at a displacement of 77.4 mm (3.2% IDR). At the design shear force,  $V_r = 23.7$  kN, the corresponding displacement was 13.8 mm, closely matching the design value of 13.6 mm. A crack formed at the last row of HD screws in the chord member of SEG1 just before reaching the peak resistance of the shear wall, as shown in Figure 4.41(a). As loading continued, the crack propagated through the inner ply of the chord member, leading to fracture of the chord attached to HD device at a displacement of 105 mm (4.3% IDR). Despite this, the shear wall continued to carry a load of approximately 25 kN until the test was terminated at a displacement of 140 mm (5.7% IDR). At this displacement level, the shear wall resistance was about 60% of the peak resistance and approximately equal to its  $V_r$  value. This behavior highlights the redundancy due to system effects, as SEG2 remained intact and its HD undamaged.

The inclusion of HD connections at the floor level significantly affected the distribution of HD forces. Figure 4.42(a) shows the shear wall force versus HD force, where, at the shear wall design load level of 23.7 kN, the HD forces in SEG1 and SEG2 are 11.7 kN and 7.4 kN, respectively,

substantially lower than the design values of 22.9 kN and 18.1 kN. The reduction in HD forces was on the order of 50%. Figure 4.42(b) illustrates linear behaviour for the HD in SEG1, while the nonlinearity in SEG2 was attributed to rigid-body rotation of the HD, as the force remains low compared to its  $T_r=28.4$  kN. The HD force ratio ( $T_{HD1} / T_{HD2}$ ) shown in Figure 4.42(c) indicates that the inclusion of the HD at the floor level resulted in higher HD forces in SEG1 than in SEG2. The ratio remained above 1.5 for most loading stages, indicating approximately 50% higher force in SEG1.

Measured HD elongations at the shear wall design load were 1.2 mm and 0.8 mm for SEG1 and SEG2, respectively. Compared with the design values of 3.1 mm and 2.5 mm, these results indicate reductions in HD elongations exceeding 63% and 66% for SEG1 and SEG2, respectively.



**Figure 4.42: (a) Shear wall force–HD force diagrams; (b) HD F–D diagrams; HD force ratio**

Nail deformations were evaluated at the shear wall design level, and the average measured values were 1.2 mm and 0.8 mm in SEG1 and SEG2, respectively. Compared with the design values of  $e_n = 1.0$  mm for SEG1 and  $e_n = 0.5$  mm for SEG2, these results represent increases of approximately 20% and 55%, respectively.

The perpendicular-to-grain compression deformations in both SEG1 and SEG2 were 1.3 mm, at the shear wall design load level. Compared with the design values of 0.4 mm and 0.3 mm, the test

results indicated deformations more than three times higher. This trend was consistent with previous shear wall configurations.

Figure 4.43 shows the gap between the floor assembly and the shear wall top plates at the opening location. The observation indicated that the shear wall top plate undergoes bending, while the floor assembly remained relatively horizontal.



**Figure 4.43: Formation of the gap between shear wall top plates and floor assembly.**

Another important observation from this test was that the HD connections at the floor level prevented separation between the chord members and the top plates (Figure 4.43). As a result, the nails at the top of the shear walls did not pull out from the panel edges.

Extensive nail failures were observed along the bottom edge of SEG1, while the nails at the bottom edge of SEG2 exhibited only moderate damage (Figure 4.44). The excessive failures in SEG1 were caused by HD fracture; with the HD no longer functioning, the panel itself was required to resist the uplift in SEG1.



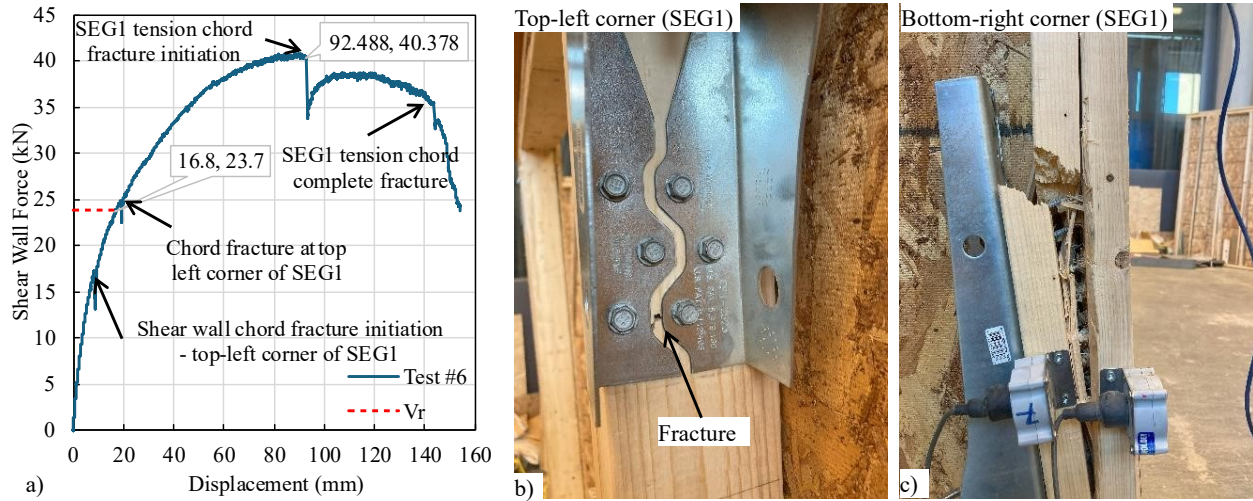


**Figure 4.44: Nail failures at the bottom edge of shear walls in Configuration #5**

#### **4.5.6 Configuration #6**

Compared to Configuration #5, this configuration only differed in number of LVL joists within the floor assembly. Two LVL joists were used. The lateral behaviour of this configuration was almost similar to that of the Configuration #5.

During the loading of this specimen, several unwanted events occurred. Figure 4.45(a)–(c) illustrate the load–displacement response of the shear wall in Configuration #6, along with the HD fracture failure modes.



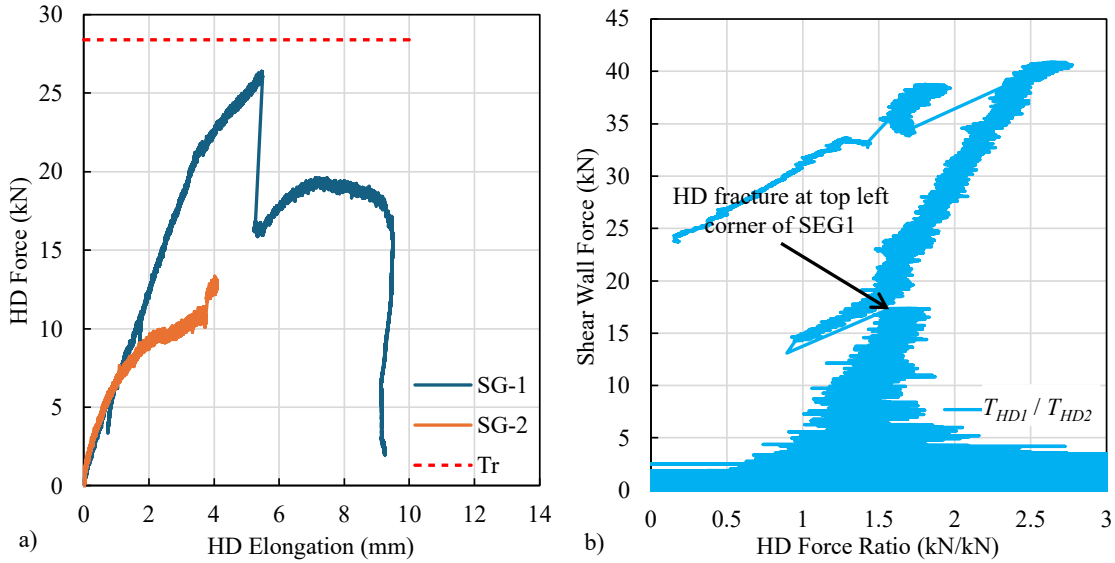
**Figure 4.45: (a) shear wall force-displacement curve; (b) HD at the floor level; (c) Tension HD failure**

When the shear wall resistance reached 17 kN, one of the plies of the compression chord in SEG1 fractured at the location of the HD at the floor level (Figure 4.45(b)). Following this fracture, the HD at the top-right corner ceased to contribute to the wall resistance due to the failure of the compression chord attached to the HD. This local failure slightly softened the specimen, resulting in a displacement of 16.8 mm at the factored resistance, higher than the corresponding value of 13.8 mm in Configuration #5 (Figure 4.45(b)).

The specimen then reached a maximum force of 40.9 kN at a displacement of 88.3 mm (3.6% IDR). Shortly after, a crack formed at the bottom of the tension chord near the top of the last HD screw and began propagating through the thickness of the chord member. While the specimen was still resisting load, the tension chord in SEG1 completely fractured at a displacement of 140 mm (5.7% IDR), as shown in Figure 4.45(c).

Due to force redistribution, the test continued until it was terminated at a displacement of 154 mm (6.3% IDR). At this stage, the specimen still maintained a resistance of 23.7 kN, corresponding to 60% of the shear wall's peak resistance and equal to its design shear force,  $V_r$ .

Figure 4.46(a) and (b) illustrate the HD force-elongation curves and the HD force ratio ( $T_{HD1} / T_{HD2}$ ) for the shear walls in Configuration #6.



**Figure 4.46: (a) HD force–displacement curves; (b) HD force ratio**

The HD in SEG1 reached a maximum force of 26.4 kN at the time of fracture, which was still below the  $T_r$  value of 28.4 kN. It should be noted that the initial HD tightening force of 2.4 kN was excluded from the data to reflect only the force due to lateral loading. When this initial tightening force is considered, the total HD force exceeds the  $T_r$  value. In contrast, the HD force in SEG2 remained well below the  $T_r$  value.

Figure 4.46(b) illustrates that the HD force ratio ( $T_{HD1} / T_{HD2}$ ) exhibited an increasing trend, highlighting a higher rate of increase in HD force in SEG1 compared to SEG2.

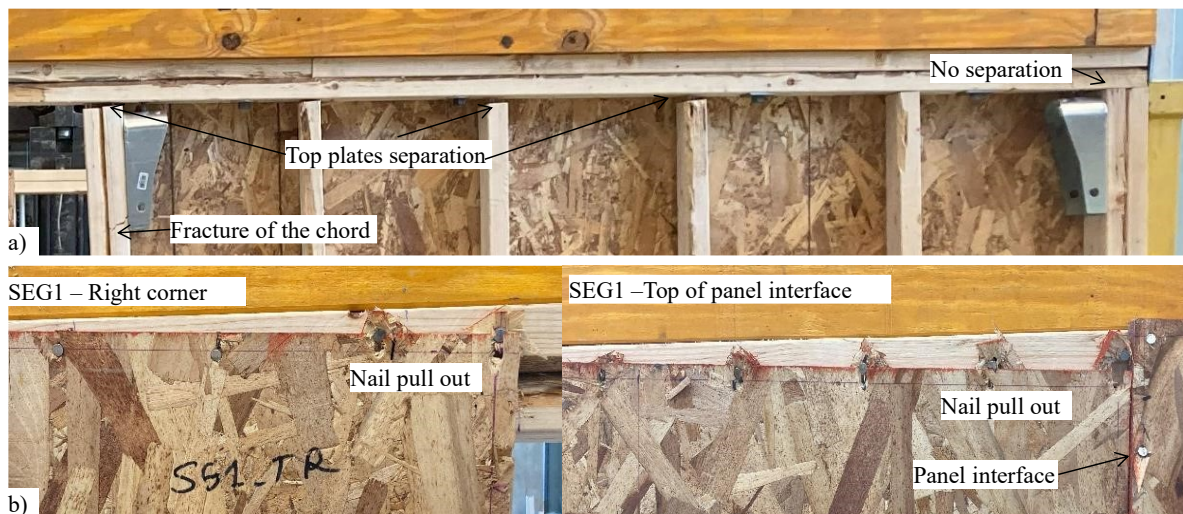
At the shear wall design load,  $V_r$ , the HD forces in SEG1 and SEG2 were 9.8 kN and 5.6 kN, respectively. Compared with the corresponding design-level forces of 22.9 kN and 18.1 kN, these values represent reductions of 56% and 70%.

Regarding the HD elongations at the shear wall design level, the measured values were 1.6 mm and 0.77 mm for SEG1 and SEG2, respectively. Compared with the design values of 3.15 mm and



2.46 mm for SEG1 and SEG2, these represent significant reductions of 50% and 70%, consistent with the reductions in HD forces.

Double LVL floor assembly also caused more uniform nail deformation across both shear walls. At the shear wall design load, the average measured nail deformations in SEG1 and SEG2 were 1.4 mm and 1.2 mm, respectively. Compared with the design values of  $e_n = 1.0$  mm and  $e_n = 0.5$  mm for SEG1 and SEG2, these values indicated 35% and 122% increases in nail deformations. The fracture of the chord member at the floor level caused the top plate to separate from vertical studs, consequently leading to the failure of several nails at the top of SEG1 (Figure 4.47(a) and (b)).



**Figure 4.47: (a) separation of the top plate from chord at fractured chord location ; (b) nail failures at the top of the shear wall**

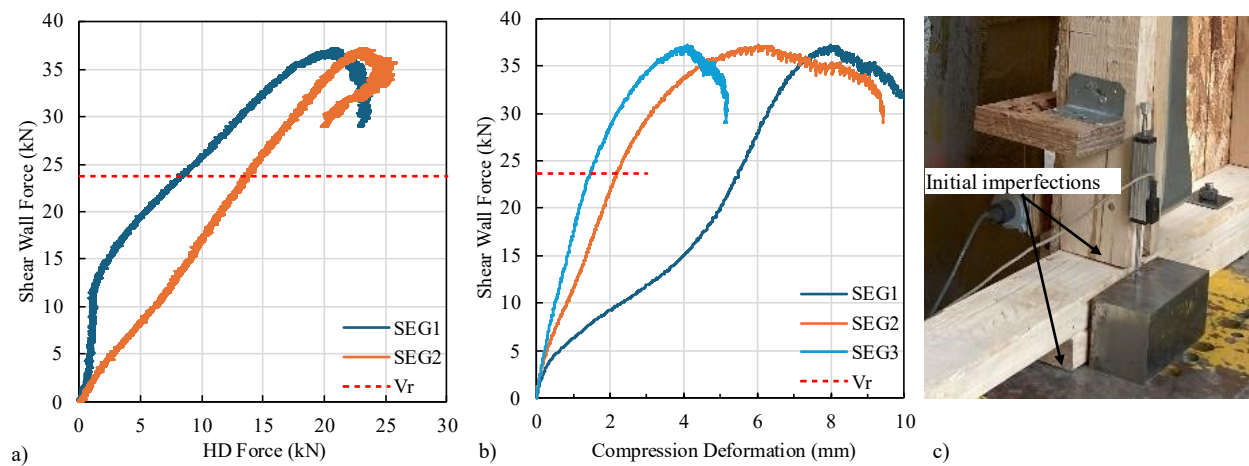
Figure 4.47(a) also shows that the undamaged HD at the floor level prevented separation between the top plates and the chord member. This restraint is believed to have induced bending in the chord. When combined with the high axial load at the HD location at the bottom of the shear wall, this bending promoted fracture of the chord at its base.

#### 4.5.7 Configuration #7

In this configuration, the shear wall reached a maximum resistance of 37.2 kN at a displacement of 90 mm (3.6% IDR). Loading continued until the test was terminated at a displacement of 168 mm (6.8% IDR), at which point the resistance had decreased to 29.2 kN (78% of the peak load) yet remained above the design load level ( $V_r = 23.7$  kN). Notably, the displacement at the design level was 27.2 mm, significantly exceeding the design value of 18.3 mm.

The maximum HD forces in SEG1 and SEG2 reached 23.8 kN and 26.0 kN, respectively, both remaining below the design value of 28.4 kN. At the shear wall design load level, the corresponding HD forces were 8.3 kN and 13.5 kN, representing reductions of 63% and 40% relative to the design values. It is noted that no LC was installed to measure the HD force in SEG3. The corresponding HD elongations were 1.1 mm, 1.7 mm, and 1.4 mm for SEG1, SEG2, and SEG3, respectively. Compared to the design value of 3.1 mm at the shear wall design level, these measurements showed reductions of 65%, 45%, and 55%, respectively.

An exceptional observation was made in SEG1 regarding the HD force and compression perpendicular-to-grain deformation, as shown in Figure 4.48(a) and (b). These figures present the HD force and elongation as functions of the shear wall force.



**Figure 4.48: (a) HD force; (b) HD elongation; (c) initial imperfections**

As shown in Figure 4.48(a), the HD in SEG1 developed force at a significantly lower rate than in SEG2 during the early stages of loading, up to a shear wall force of approximately 12 kN. Inspection revealed that imperfections, specifically an initial gap between the compression chord and the bottom plate, as well as incomplete contact between the bottom plate and the base beam, as illustrated in Figure 4.48(c), caused this behaviour.

Figure 4.48(b) further indicates that the compression perpendicular-to-grain deformation in SEG1 deviated significantly from that of the other segments until the shear wall force reached approximately 20 kN. Beyond this level, the F–D response exhibited a similar trend to SEG2 and SEG3, indicating that contact had been established between the chord and the bottom plate.

The compression perpendicular-to-grain deformation in SEG1 (5.5 mm), influenced by the initial imperfection, was not included in the comparative assessment. The corresponding values at the shear wall design load were 2.2 mm and 1.5 mm for SEG2 and SEG3, respectively. Compared to the design value of 0.42 mm, these results indicated significantly larger deformation magnitudes.

The average measured nail deformations during the test were 1.0 mm, 1.7 mm, and 2.1 mm for SEG1, SEG2, and SEG3, respectively, measured at a level corresponding to the shear wall design load level. The design value of the nail deformation was 0.86 mm for all segments. A comparison with the design values showed increases of 20%, 98%, and 144% in SEG1, SEG2, and SEG3, respectively

The main failure mode was nail failure. The first failures occurred concurrently at the top-right corner of SEG2 and the bottom corner of SEG3. For all three shear walls, as displacement increased, the nails at the top- and bottom edges at the opposite corners pulled out from the edge of the sheathing panel, as illustrated in Figure 4.47(b). Large nail deformations, particularly along

the bottom edges of SEG2 and SEG3, caused the sheathing panels to separate from the studs, as shown in Figure 4.49(a).



**Figure 4.49: Failure modes: (a) nail failure and panel separation; (b) HD slippage; (c) chord fracture**

Despite moderate slippage between the HD devices and the chord members (Figure 4.49(b)), the HDs did not fail during the test. A crack formed in the chord member of SEG2 at the HD location, as shown in Figure 4.49(c), at a load level close to the peak shear wall resistance. However, the crack did not propagate and remained stable until the end of the test.

#### **4.5.8 Configuration #8**

This configuration was similar to Configuration #7, except that the continuous LVL joist in the floor assembly was replaced with blocking. The details of the blocked floor assembly configuration are provided in Section 3.4.8.

The specimen in this configuration reached a peak resistance of 38.3 kN at a displacement of 98 mm (4% IDR), with no signs of failure up to this displacement level. The average nail deformation at the peak resistance of the shear wall was 8.0 mm for all three segments in the configuration. By the end of the test, nail deformations ranged between 9 mm and 13 mm. Additionally, some nail failures were observed at the bottom of the shear walls at the end of the test. A crack formed in

the tension chord of SEG2 at the HD location immediately after the shear wall passed its peak resistance. The chord subsequently fractured at a displacement of 112 mm (4.5% IDR). The test was terminated at this displacement, at which point the shear wall resistance was 24.5 kN, corresponding to 65% of the peak resistance and still exceeding the design load level,  $V_r = 23.7$  kN. This sustained resistance was attributed to the contribution of the other two segments, which remained operational.

At the shear wall design load level, the average measured nail deformations in SEG1, SEG2, and SEG3 were 0.7 mm, 1.6 mm, and 1.6 mm, respectively. The corresponding design nail deformation for all three segments was 0.86 mm. Comparison between measured nail deformations and design value showed that, while SEG1 exhibited a 14% lower deformation than the design value, the nail deformations in SEG2 and SEG3 were approximately 90% higher than the design values.

The maximum HD forces were 28.6 kN and 23.1 kN for SEG1 and SEG2, respectively. The HD force in SEG1 exceeded its  $T_r$  value of 28.4 kN; however, despite observed slippage between the HD device and the chord member, HD failure was not observed in SEG1. In contrast, fracture of the chord member in SEG2 initiated at an HD force of 23.1 kN, which is below its  $T_r$  value of 28.4 kN.

At the shear wall design load level, the measured HD forces were 11.1 kN for SEG1 and 11.8 kN for SEG2. Compared to the design HD force of 22.7 kN, these values indicated reductions of approximately 50%. Similarly, the measured HD elongations were about 50% lower than the corresponding design values.

The measured perpendicular-to-grain compression deformations were 4.3 mm, 1.7 mm, and 1.5 mm for SEG1, SEG2, and SEG3, respectively. Compared to the design-stage value of 0.4 mm, the measured deformations were considerably higher. Similar to Configuration #7, the large



deformation observed in SEG1 was attributed to an imperfection beneath the compression chord in SEG1. Excluding SEG1 and considering only SEG2 and SEG3, the measured perpendicular-to-grain deformations were approximately four times greater than the design values.

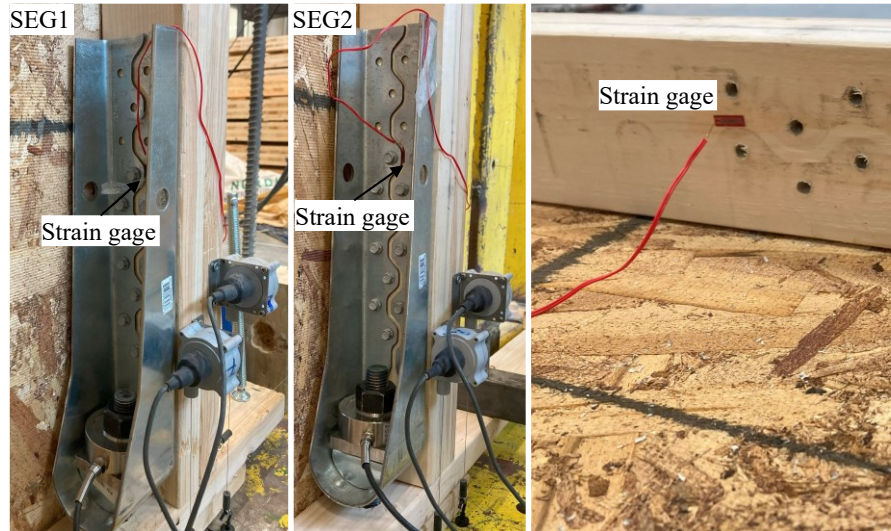
In contrast to Configuration #7, where the main failure mode was excessive nail deformations, this specimen failed due to fracture of the chord member at the HD location.



**Figure 4.50: SEG2 tension chord fracture in Configuration #8**

#### **4.5.9 Configuration #9**

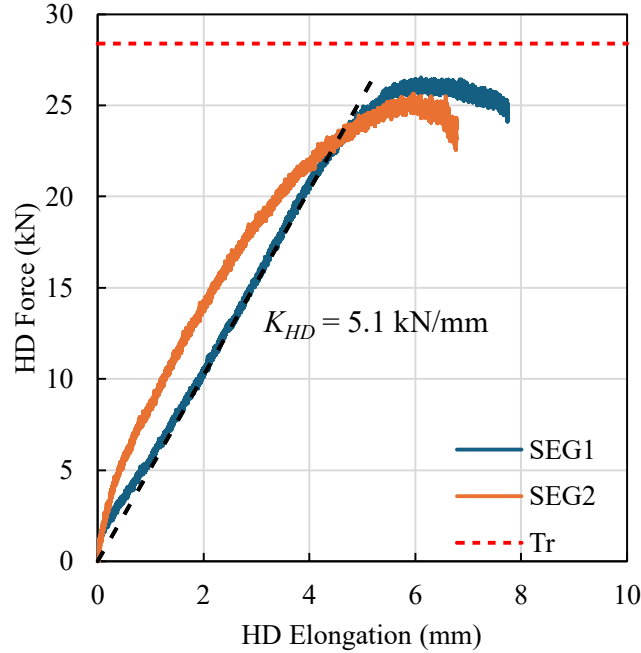
Having observed HD failures (i.e., chord fractures) in several previous configurations, two strain gauges were installed on the inner side of the tension chords in this configuration to better understand the underlying causes. These strain gauges were intended to measure the tensile strain at the interface between the HD device and the chord member, specifically at the location of the last row of HD screws where fractures initiated (Figure 4.51).



**Figure 4.51: Installation of strain gages on chord members**

The shear wall in this configuration reached a maximum resistance of 25.5 kN at a displacement of 118.4 mm (4.8% IDR). Loading continued until a displacement of 162.7 mm (6.7% IDR), at which point the test was terminated. At this displacement, the shear wall resistance was 22.6 kN, corresponding to 88% of the peak resistance and still 43% higher than the design load level ( $V_r = 15.8$  kN). The measured displacement at the design load level was 25 mm, which is 36% greater than the design value of 18.3 mm.

A closer examination of the measured HD force-displacement response, as illustrated in Figure 4.52, indicated that the measured HD stiffness ( $K_{HD}$ ) was 5.1 kN/mm, which showed approximately 30% lower than the design value of 7.3 kN/mm reported by the manufacturer (Simpson Strong-Tie 2022). This reduced stiffness may partly explain the larger shear wall displacement observed at the design load level.



**Figure 4.52: HD force-displacement (Configuration #9)**

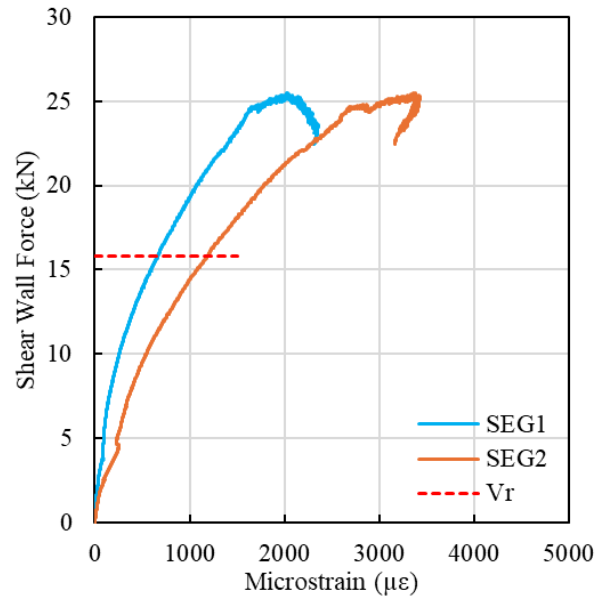
The maximum measured HD forces were 26.5 kN in SEG1 and 25.6 kN in SEG2, both remaining below the  $T_r$  value of 28.4 kN. At the shear wall design load level, the HD forces were 13.0 kN and 12.6 kN for SEG1 and SEG2, respectively. Compared to the design HD forces, these values were approximately 45% lower. The corresponding HD elongations were 2.5 mm for SEG1 and 1.7 mm for SEG2, representing reductions of 20% and 46% relative to the design values, respectively, with a greater reduction observed in SEG2.

The perpendicular-to-grain compression deformations were 1.9 mm and 2.2 mm for SEG1 and SEG2, respectively. These values are approximately five times the design value of 0.42 mm. The significant discrepancy between the measured and design values is consistent with observations from previous tests.

Average nail deformations measured during testing were 1.5 mm for SEG1 and 1.0 mm for SEG2. These values correspond to the shear wall design load level. Compared to the design nail deformation of 0.86 mm, the average measured values indicated increases of 74% and 16% for SEG1 and SEG2, respectively.



Figure 4.53 presents the measured strains at the interface between the HD device and the chord members of SEG1 and SEG2 as a function of shear wall force. Although this shear wall did not fail due to chord fracture at the HD location, the stress distribution at the fracture-prone region of the chord member provides insight into possible causes of previous HD failures.



**Figure 4.53: Strain at the interface of HD and chord member**

At the shear wall design load level ( $V_r = 15.8$  kN), the strain gauges recorded  $660 \mu\epsilon$  at the HD-chord interface in SEG1 and  $1180 \mu\epsilon$  in SEG2. At this load level, the HD forces were approximately 13 kN in both segments. The higher strain observed in SEG2 may suggest localized effects of the boundary conditions at the chord member interface, which could be one of the contributing factors to the previously observed HD failures.

The main failure mode was excessive nail deformation, similar to that shown in Figure 4.47 and Figure 4.49(a).

#### 4.5.10 Configuration #10

This configuration consisted of a single-segment shear wall without a floor assembly. A LC was used to quantify the contribution of the AB in resisting uplift induced by the overturning moment at the base of the wall.

The shear wall reached a peak resistance of 13.4 kN at a displacement of 104.5 mm (4.3% IDR). The specimen was then pushed well beyond peak capacity, and the test was terminated at a displacement of 185 mm (7.5% IDR). At this stage, the load had decreased to 2.8 kN, corresponding to 20% of the peak resistance and 35% of the shear wall design load ( $V_r = 7.9$  kN). At the shear wall design load level, the measured displacement of the shear wall was 22.8 mm, which is 24% higher than the design-stage prediction of 18.3 mm.

The nail deformations at the TL and BR edges of the shear wall were 0.83 mm and 0.98 mm, respectively. Considering the average deformation of 0.9 mm, it was comparable to the design value of  $e_n = 0.86$  mm.

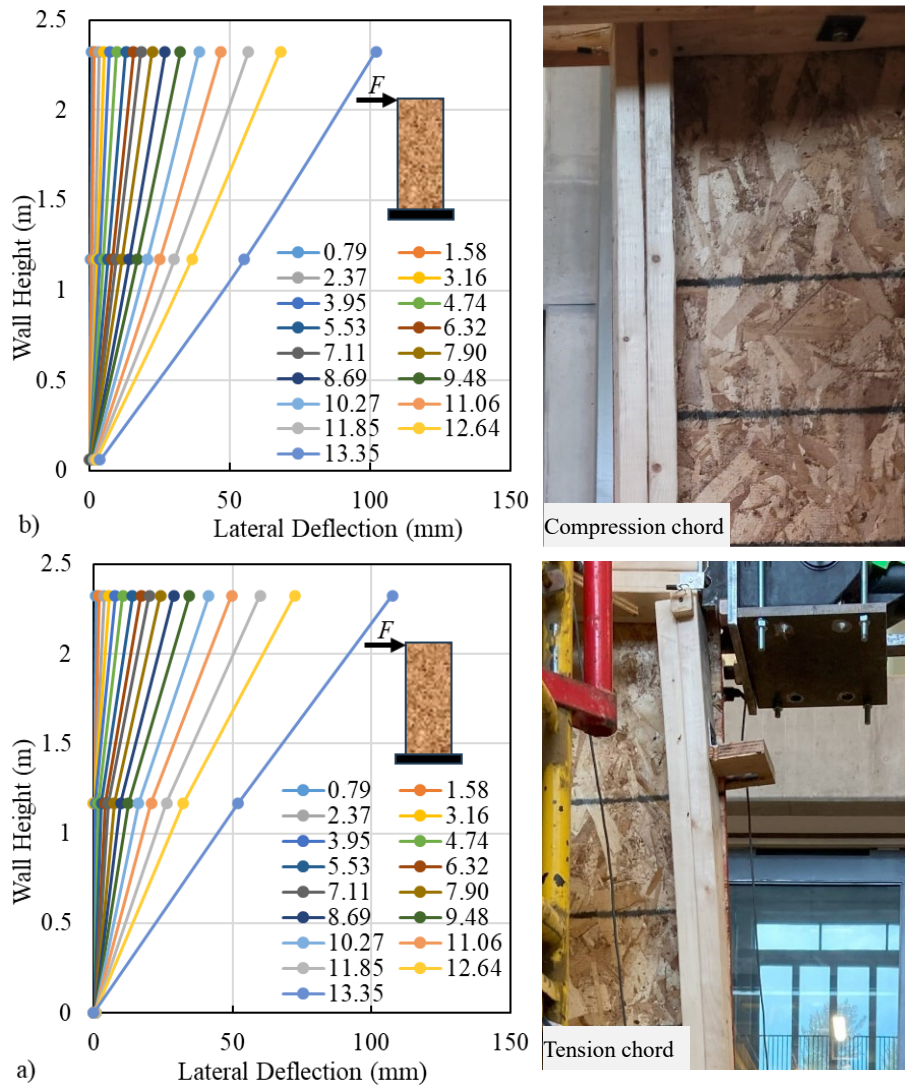
The maximum HD force was measured at 28.2 kN, which was close to the  $T_r$  value of 28.4 kN. Although moderate slippage was observed between the HD device and the chord member, the HD did not fail when the shear wall reached its peak resistance. At the shear wall design load, the measured HD force and elongation were 14.9 kN and 1.5 mm, respectively. Compared to the design values, the measured force corresponded to 66% of the design value (22.65 kN), while the measured elongation was 50% of the design value (3.1 mm).

The measured force in the AB was 1.4 kN, indicating a contribution of approximately 6% relative to the HD design force.

The compression perpendicular-to-grain deformation was 1.7 mm, which was approximately four times the corresponding design value of 0.42 mm.

The measured lateral displacements of the tension and compression chords along the height of the shear wall exhibited opposite bending modes, as depicted in Figure 4.54(a) and (b). The tension chord bent in the direction of the wall displacement (loading direction), whereas the compression

chord bent opposite to the loading direction. Bending deformation was more pronounced near the actuator tip due to localized stud bending at that location.



**Figure 4.54: Bending deformation: (a) tension chord; (b) compression chord (Configuration #10)**

#### 4.5.11 Configuration #11

This configuration consisted of a two-meter, single-segment shear wall without the floor assembly.

A LC was used to measure the contribution of the HD at the compression chord.

The shear wall reached a peak resistance of 27.23 kN at a displacement of 61 mm (2.5% IDR). The peak shear wall resistance sustained until the displacement reached 80 mm (3.2% IDR). Beyond this point, the shear wall resistance started to diminish. The test was continued until a displacement

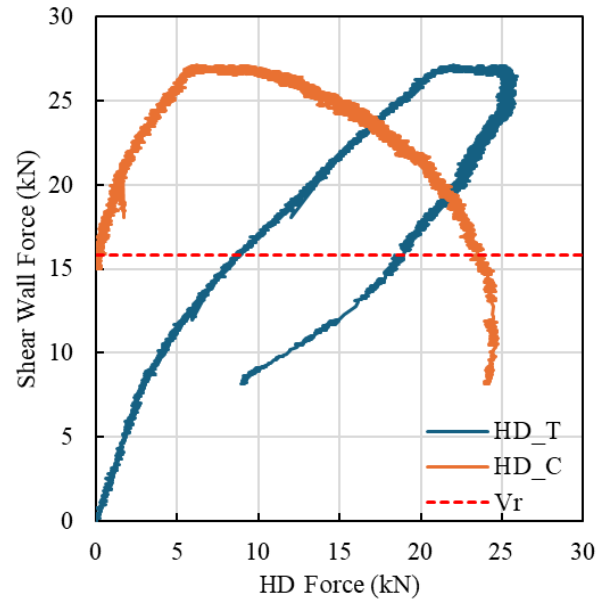
of 172 mm (7% IDR), at which point it was terminated. At this displacement, the shear wall resistance was 8.28 kN, corresponding to 30% of the peak resistance and 50% of the shear wall design load level ( $V_r=15.8$  kN).

The average nail deformation across the shear wall was 0.91 mm, which is comparable to the design value of  $e_n = 0.86$

The maximum measured HD force was 26 kN, which was lower than the  $T_r$  value of 28.4 kN. Although slippage was observed between the HD device and the chord member, the HD did not fail throughout the test. At the shear wall design load level, the measured HD force was 8.65 kN, which was approximately 50% lower than the design value of 20.8 kN. The measured HD elongation was 1.45 mm, also about 50% lower than the design value of 2.8 mm.

The compression perpendicular-to-grain deformation was measured as 0.65 mm, representing a 75% increase compared to the design value of 0.37 mm.

Figure 4.55 plots the HD force against the shear wall force. The LC at the compression chord did not record any force until the shear wall load reached approximately 15 kN. The force in HD at the compression side became more pronounced at displacement levels beyond the peak. It is noted that the force developed in the HD at the compression side was primarily due to the rotation of the compression chord at large shear wall displacements.



**Figure 4.55: HD forces (Configuration #11)**

The bending deformations of the chord members were similar to those observed during Configuration #10 testing, as shown in Figure 4.54(a) and (b).

#### 4.5.12 Configuration #12

This configuration consisted of a 3.66 m long, single-segment shear wall without a floor assembly. A LC was used to measure the contribution of the AB in resisting uplift due to the overturning moment at the base of the shear wall.

The shear wall reached a maximum resistance of 53.5 kN at a displacement of 50 mm (2% IDR), and this resistance was sustained until a displacement of 60 mm (2.5% IDR). The test was terminated at a displacement of 142.6 mm (5.8% IDR), at which point the load had decreased to 6.4 kN, corresponding to 12% of the peak resistance and 22% of the shear wall design load ( $V_r = 28.9$  kN).

The maximum force in the HD reached 26.5 kN, which was lower than the  $T_r$  value of 28.4 kN. At the shear wall design load level, the measured HD force was 8 kN, corresponding to 40% of the

HD design load of 20 kN. The HD elongation at this load level was 1.4 mm, which was approximately 50% lower than the design value of 2.74 mm.

The maximum load in the AB was 5.6 kN, and at the shear wall design load level it was 2.45 kN. At this load level, the AB force was approximately 30% of the HD force. This shows a significant contribution of the ABs in resisting the overturning moment at the base of the shear wall.

The perpendicular-to-grain compression deformation under the compression chord was 1.5 mm at the shear wall design load level, which was nearly four times greater than the design value of 0.37 mm.

Bending deformation was similar to those of Configurations #10 and #11.

The final failure mode was governed by nail failure. Despite moderate slippage between the HD device and the chord member, the HD did not fail during the test. Figure 4.56 shows the specimen after the test.



**Figure 4.56: Shear wall Configuration #12 (after test)**

#### 4.5.13 Configuration #13

This configuration utilized the ATS rod system as HD. The shear wall did not exhibit any signs of failure at the shear wall design load level of 23.7 kN. At this load level, the measured shear wall displacement was 21.3 mm, which was higher than the design value of 14.4 mm.

The maximum shear wall resistance in this configuration was 37.9 kN at a displacement of 68.3 mm, corresponding to 2.8% IDR. The specimen was then loaded further to a displacement of 130 mm (5.3% IDR), at which point the test was terminated. At this displacement, the shear wall resistance was 16.8 kN, corresponding to 44% of the peak resistance and 70% of the shear wall design load.

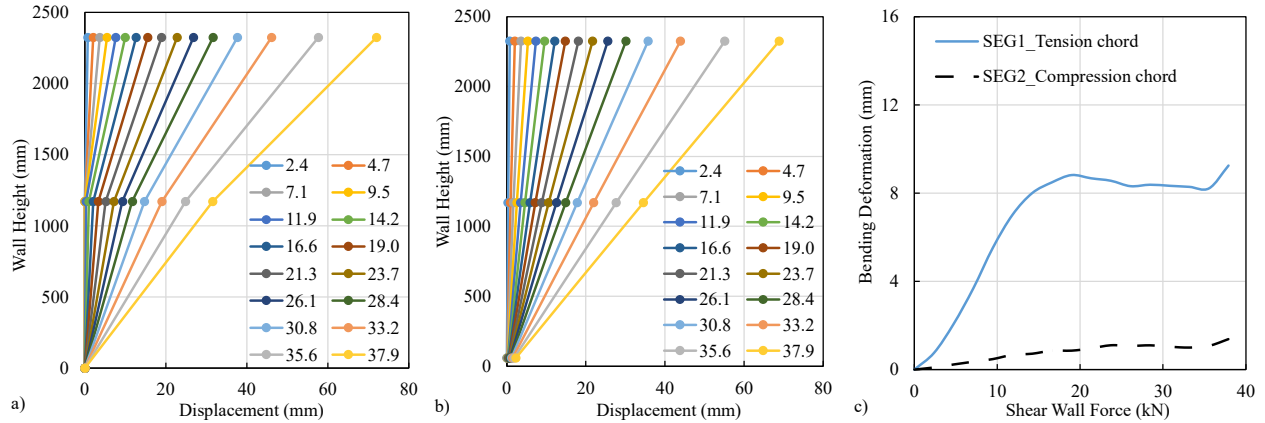
The maximum forces developed in the HDs of SEG1 and SEG2 were 26.4 kN and 19.8 kN, respectively, both lower than the  $T_r$  value of 27.4 kN. The HD forces in SEG1 and SEG2 were 12.2 kN and 7.1 kN, corresponding to reductions of approximately 50% and 63% relative to the design values of 24.5 kN and 18.8 kN, respectively.

The perpendicular-to-grain compression deformations were measured as 1.1 mm for SEG1 and 1.2 mm for SEG2. These values were approximately six times greater than the corresponding design values of 0.22 mm and 0.17 mm.

The average measured nail deformations across SEG1 and SEG2 at the shear wall design load level were 1.8 mm and 1.2 mm, respectively. Compared with the design values of  $e_n = 1.1$  mm for SEG1 and  $e_n = 0.5$  mm for SEG2, the measured deformations were 63% and 120% higher, respectively.

The measured lateral displacements of the chord members along the height of the shear wall exhibited different levels of bending deformation in the tension and compression chords. Figure 4.57(a) and (b) show the deflected shapes of the tension chord in SEG1 and the compression chord

in SEG2, respectively. Figure 4.57(c) illustrates how the bending deformation evolved as the shear wall load increased.



**Figure 4.57: Bending deformations: (a) tension chord (SEG1); (b) compression chord (SEG2); (c) rate of change of bending deformations (Configuration #13)**

As can be seen from Figure 4.57(c), the tension chord in SEG1 exhibited higher bending deformations than the compression chord in SEG2. The figure clearly indicates that the bending deformations of the chord member in SEG2 were significantly lower than those in SEG1. It is noted that, similar to Configurations #10 – #12, the bending deformations of the tension chord in SEG1 were influenced by the localization effect of the actuator.

The main failure mode was nail yielding and nail pull-out from the edges of the sheathing panels, similar to previous tests. Nails at the panel interface in SEG1 exhibited extensive deformations (Figure 4.58(a)). At the end of the test, the top plate of the floor assembly above SEG1, where the ATS rod plate was seated, was severely damaged (Figure 4.58(b)).





**Figure 4.58: (a) nail failure at panel interface; (b) failure of the ATS rod system**

#### **4.5.14 Configuration #14**

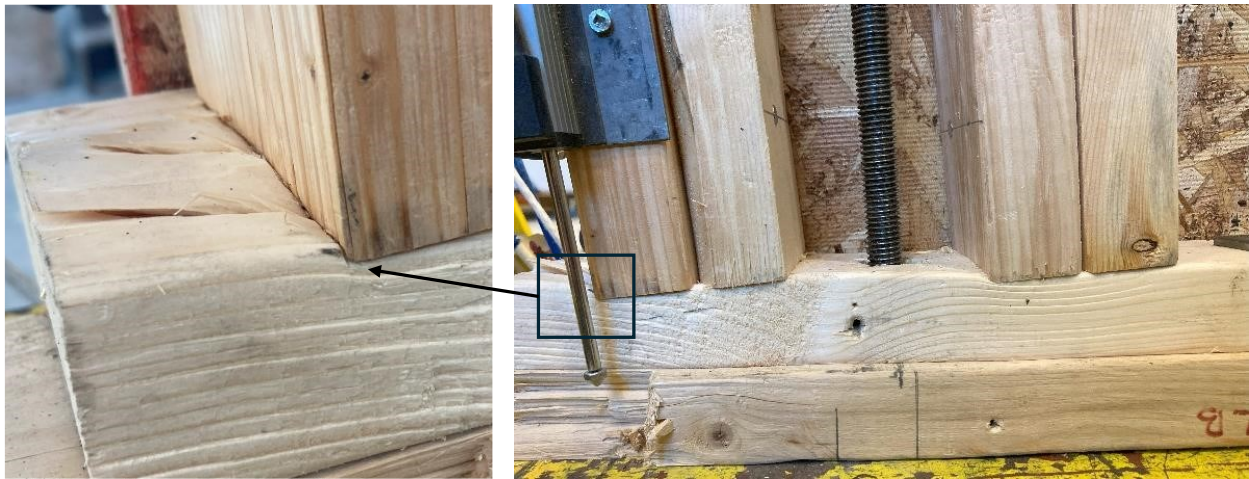
The shear wall in this configuration reached its design load level ( $V_r = 39.16$  kN) at a displacement of 25.1 mm, which is considerably higher than the design value of 14.5 mm.

The shear wall attained a maximum resistance of 75.36 kN at a displacement of 117 mm (4.8% IDR). The test was terminated at a displacement of 173.8 mm (7.15% IDR), where the shear wall resistance was 43.8 kN, corresponding to approximately 60% of the peak resistance. At the end of the test, the resistance remained about 12% higher than the shear wall design load level.

The maximum forces in the ATS rods were 50.2 kN and 31 kN for SEG1 and SEG2, respectively. The rod force in SEG1 exceeded the  $T_r$  value of 43.36 kN, while the force in SEG2 remained below this value. At the shear wall design load level, the measured rod forces were 20.4 kN for SEG1 and 6.7 kN for SEG2. Compared to the design rod forces of 41.5 kN and 29.5 kN, the measured values correspond to approximately 50% and 22% of the design values, indicating significant reductions.

The design rod elongations were 3.98 mm for SEG1 and 2.86 mm for SEG2, while the corresponding measured values were 3.7 mm and 0.55 mm, representing reductions of approximately 8% and 80%, respectively.

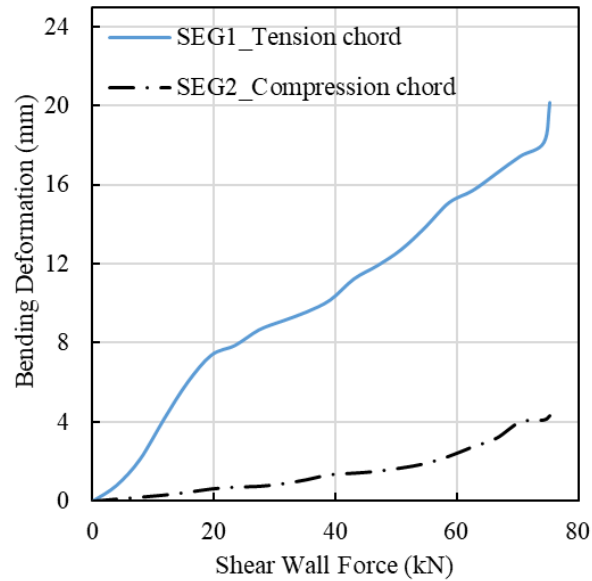
The measured perpendicular-to-grain compression deformations were 3.0 mm in SEG1 and 3.8 mm in SEG2. Compared to the design values of 0.38 mm and 0.22 mm, these results indicate substantially higher deformation in this component. Figure 4.59 shows the crushing of the bottom plate beneath the compression chord members in SEG2.



**Figure 4.59: Perpendicular-to-grain compression deformations (Configuration #14)**

The average measured nail deformations across SEG1 and SEG2 at the shear wall design load level were 1.18 mm and 0.5 mm, respectively. The corresponding design values were 0.76 mm for SEG1 and 0.3 mm for SEG2. Comparison of the measured and design values indicated that the measured deformations were approximately 55% and 67% higher in SEG1 and SEG2, respectively.

The bending deformations of the chord members were similar to those observed in Configuration #13, with the deformed shapes of the chords in SEG1 and SEG2 resembling those shown in Figure 4.57(a) and (b). Figure 4.60 illustrates the rate of change of bending deformations in the chord members of SEG1 and SEG2 of Configuration #14 as a function of shear wall force.



**Figure 4.60: Bending deformations of the chord members (Configuration #14)**

The primary failure modes were excessive nail deformations, particularly at the bottom of the sheathing panels, and separation of the panels from the studs. Due to the increased edge nail distance in this configuration, no tearing of the sheathing panel edges was observed.



**Figure 4.61: Failure modes in Configuration #14**

#### 4.5.15 EEEP parameters of the full-scale shear wall tests

Table 4-6 summarizes the EEEP parameters obtained from the shear wall tests conducted in this study. These parameters were calculated using the same procedure described for the nail

connection tests in Section 4.3, ensuring consistency in the evaluation of the load–deformation response across different test types.

Table 4-6: EEEP idealization parameters of the shear walls

| Shear wall configuration                                                                                                                                                                                                                                                                                                                                                                                                                                                                                 | $V_r$ (kN) | $K_{0-40}$ (kN/mm) | $\Delta_y$ (mm) | $F_y$ (kN) | $\Delta_{peak}$ (mm) | $F_{peak}$ (kN) | $\Delta_{ult}$ (mm) | $F_{ult}$ (kN) | $\Delta_f$ (mm) | $F_f$ (kN) | $F_f / F_{peak}$ | $\Omega$ | $\mu$ | $F_{peak}$ (kN/m) |
|----------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------|------------|--------------------|-----------------|------------|----------------------|-----------------|---------------------|----------------|-----------------|------------|------------------|----------|-------|-------------------|
| 1                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                        | 23.70      | 2.01               | 17.38           | 34.96      | 71.64                | 39.39           | 110.20              | 31.51          | 127.83          | 18.51      | 0.47             | 1.66     | 6.34  | 13.13             |
| 2                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                        | 23.70      | 2.22               | 15.77           | 35.04      | 75.93                | 39.60           | 98.97               | 31.68          | 141.35          | 19.06      | 0.48             | 1.67     | 6.27  | 13.20             |
| 3                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                        | 23.70      | 2.14               | 17.45           | 37.32      | 82.09                | 40.38           | 139.20              | 32.30          | 139.20          | 33.66      | 0.83             | 1.70     | 7.98  | 13.46             |
| 4                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                        | 15.80      | 0.92               | 25.94           | 23.77      | 102.47               | 26.18           | 164.42              | 20.95          | 168.17          | 19.63      | 0.75             | 1.66     | 6.34  | 13.09             |
| 5                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                        | 23.70      | 2.44               | 15.39           | 37.55      | 77.36                | 43.16           | 106.13              | 34.52          | 137.66          | 24.04      | 0.56             | 1.82     | 6.90  | 14.39             |
| 6                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                        | 23.70      | 2.17               | 16.61           | 35.97      | 88.33                | 40.87           | 144.10              | 32.70          | 154.01          | 23.73      | 0.58             | 1.72     | 8.68  | 13.62             |
| 7                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                        | 23.70      | 1.08               | 30.91           | 33.52      | 89.93                | 37.21           | 168.87              | 29.77          | 168.77          | 29.22      | 0.79             | 1.57     | 5.46  | 12.40             |
| 8                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                        | 23.70      | 1.29               | 26.05           | 33.73      | 97.83                | 38.25           | 112.75              | 30.60          | 112.80          | 28.47      | 0.74             | 1.61     | 4.33  | 12.75             |
| 9                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                        | 15.80      | 0.87               | 26.49           | 23.07      | 118.41               | 25.53           | 162.69              | 22.61          | 162.69          | 22.61      | 0.89             | 1.62     | 6.14  | 12.77             |
| 10                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                       | 7.90       | 0.44               | 27.29           | 11.87      | 104.59               | 13.37           | 161.33              | 10.70          | 185.63          | 2.80       | 0.21             | 1.69     | 5.91  | 13.37             |
| 11                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                       | 15.80      | 1.74               | 13.96           | 24.29      | 61.37                | 27.23           | 132.11              | 21.79          | 172.23          | 8.28       | 0.30             | 1.72     | 9.47  | 13.62             |
| 12                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                       | 28.92      | 4.09               | 11.66           | 47.69      | 50.04                | 53.54           | 95.22               | 42.84          | 142.62          | 6.38       | 0.12             | 1.85     | 8.17  | 14.63             |
| 13                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                       | 23.70      | 1.48               | 22.94           | 33.99      | 68.26                | 37.93           | 115.37              | 30.35          | 129.06          | 16.82      | 0.44             | 1.60     | 5.03  | 12.64             |
| 14                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                       | 39.15      | 1.78               | 37.18           | 66.32      | 117.02               | 75.36           | 136.37              | 60.29          | 173.84          | 43.79      | 0.58             | 1.92     | 3.67  | 25.12             |
| Average                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                  |            |                    |                 |            |                      |                 |                     |                |                 |            |                  | 1.70     | 6.48  | 14.16             |
| STDV                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                     |            |                    |                 |            |                      |                 |                     |                |                 |            |                  | 0.10     | 1.65  | 3.22              |
| CV                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                       |            |                    |                 |            |                      |                 |                     |                |                 |            |                  | 0.06     | 0.25  | 0.23              |
| $V_r$ : factored resistance of the shear wall (design level); $K_{0-40}$ : initial stiffness; $\Delta_y$ : yield displacement; $F_y$ : yield resistance; $\Delta_{peak}$ : peak displacement; $F_{peak}$ : peak resistance; $\Delta_{ult}$ : ultimate displacement; $F_{ult}$ : ultimate resistance corresponding to $0.8 F_{peak}$ ; $\Delta_f$ : displacement at test termination; $F_f$ : shear wall resistance at test termination; $\Omega$ : shear wall overstrength; $\mu$ : shear wall ductility |            |                    |                 |            |                      |                 |                     |                |                 |            |                  |          |       |                   |

## Chapter 5

### Numerical analysis and results

#### 5.1 Introduction

Numerical simulation has long been used as a complementary tool to experimental investigations, since it helps to verify the results obtained and enables reliable interpretation of the observed behaviour. Once validated through experimental result, numerical models can be used to expand the investigation to include cases or parameters that are not feasible to investigate experimentally. By systematically varying specific parameters, numerical models can also provide valuable information on the sensitivity of system to individual variables. The following sections describe the numerical model used in this study and details key proposed approaches to achieve robust modelling techniques.

#### 5.2 Numerical modeling framework

For the modeling of wood shear walls, the commercially available software SAP2000® (Computers and Structures 2015) was used. SAP2000 includes a library of elements that can be employed to model different structural components, such as framing elements (e.g., studs), sheathing panels, as well as different types of fasteners, including nails, ABs, and HDs. The studs were modeled using *frame* elements, while *shell* elements with membrane behaviour were used to model the sheathing panels, since only in-of-plane behaviour of the panels and only translational degrees of freedom (DOF) are of interest.

The materials used for modeling the framing members and sheathing panels were assumed to exhibit linear-elastic isotropic behaviour. Despite the fact that wood is an orthotropic material, the condition analyzed in a shear wall does not necessitate the use of such orthotropic material properties, since prismatic frame members fundamentally behave as a 1D element, where stiffness

is governed by properties along its axis, not in multiple in-plane directions like a shell. There was no attempt to capture failures like local cross-grain crushing, since this behaviour was captured elsewhere, such as the connection input, which was based on test data that already account for wood crushing. Similarly, the sheathing acts primarily as a shear diaphragm, not a bending plate, where the global wall stiffness is governed only by shear deformation in the sheathing.

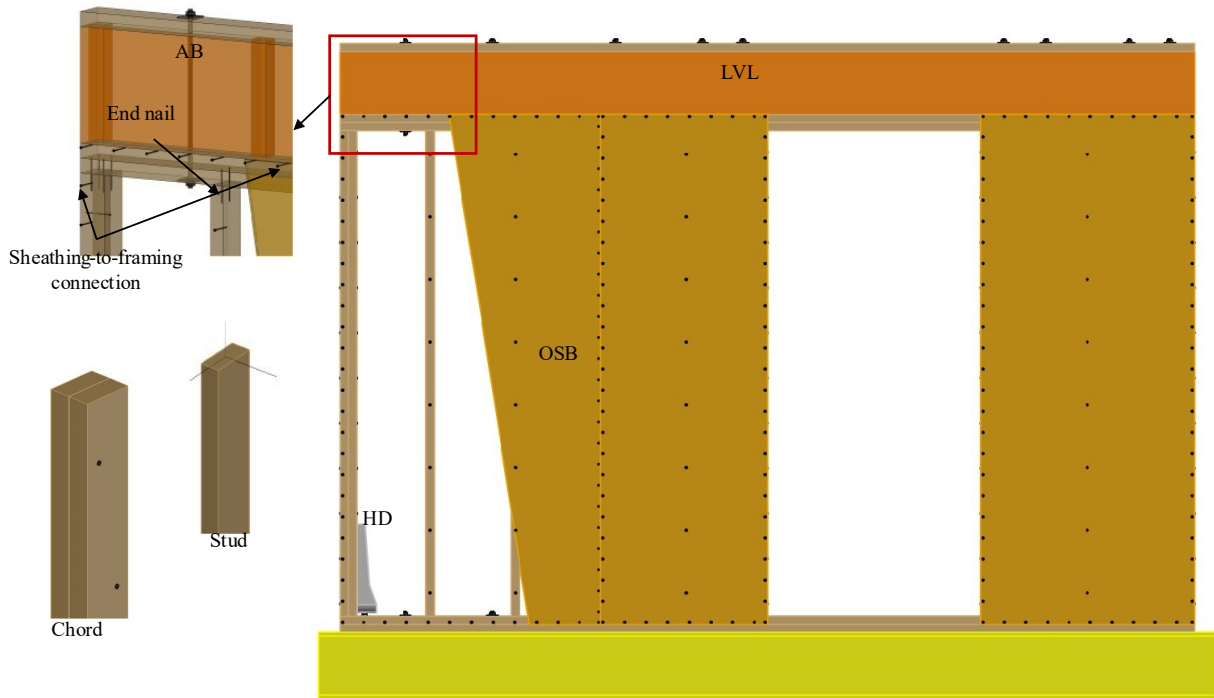
The material properties, including the MOE ( $E$ ), shear modulus ( $G$ ), and Poisson's ratio ( $\nu$ ), for both the framing and sheathing panels are provided in Table 5-1. The  $MOE$  value for the framing members was based on the average results of the stud bending tests conducted in this research. For the sheathing panels,  $E$  was derived from the shear-through-thickness rigidity ( $B_v=Gt$ ) value for OSB panels as specified in the CSA O86 (Canadian Standards Association 2019) standard. Since both the framing and the sheathing were modeled as isotropic materials, the choice of Poisson's ratio becomes a modeling parameter, not a literal physical property of wood. The goal was numerical stability and correct in-plane shear stiffness. A value of 0.35 was adopted for the Poisson's ratio of the studs (Forest Products Laboratory - USDA 2021). A corresponding value of 0.23 was adopted for sheathing panel (Grandmont et al. 2010), while the LVL material properties were based on experimental data of similar products (Younis et al. 2025).

Table 5-1: Wood material properties

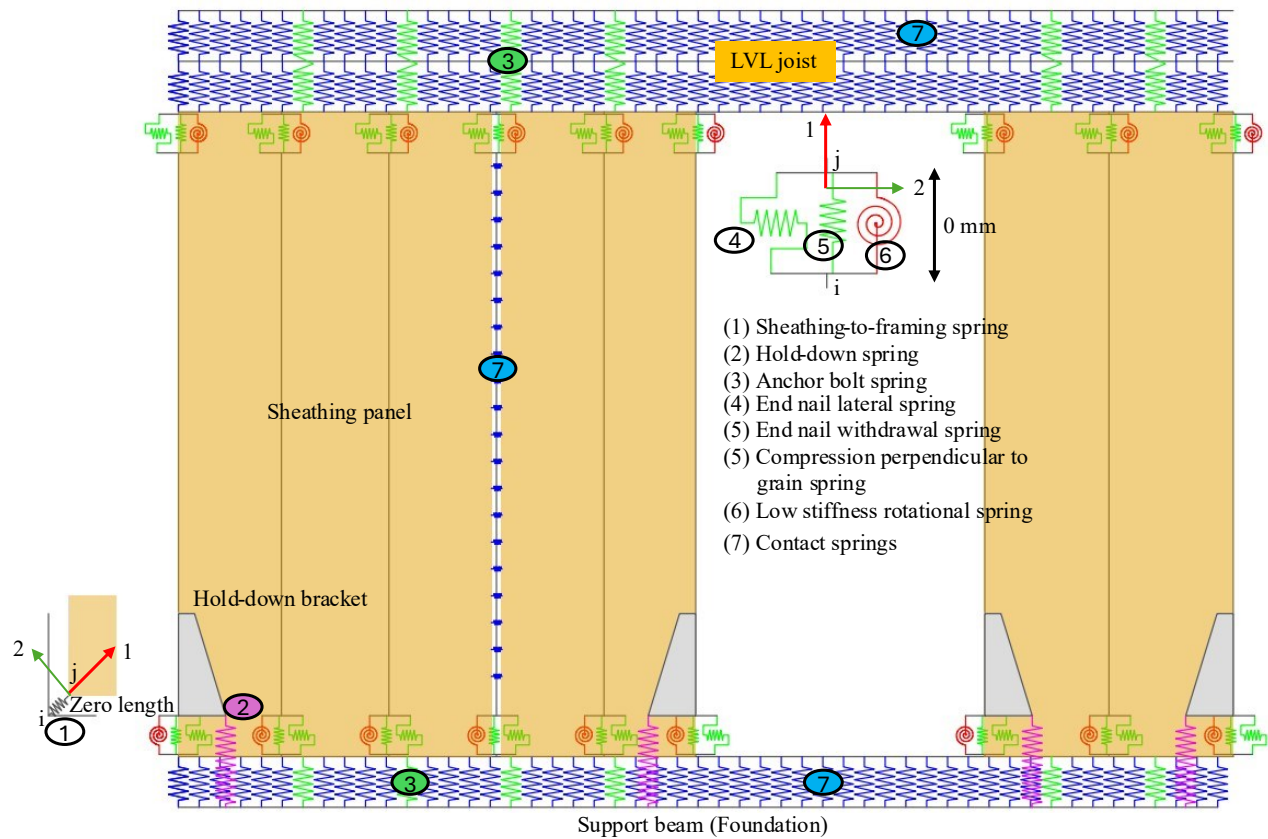
| <b>Material</b>     | $E$ (N/mm <sup>2</sup> ) | $G$ (N/mm <sup>2</sup> ) | $\nu$ |
|---------------------|--------------------------|--------------------------|-------|
| Framing stud        | 9500                     | 760                      | 0.35  |
| LVL joist           | 14000                    | 960                      | 0.40  |
| OSB Sheathing panel | 2460                     | 1000                     | 0.23  |

Figure 5.1 provides a realistic representation of a two-segment wood shear wall and its key components. Building on this, Figure 5.2 presents the corresponding FE or numerical model.



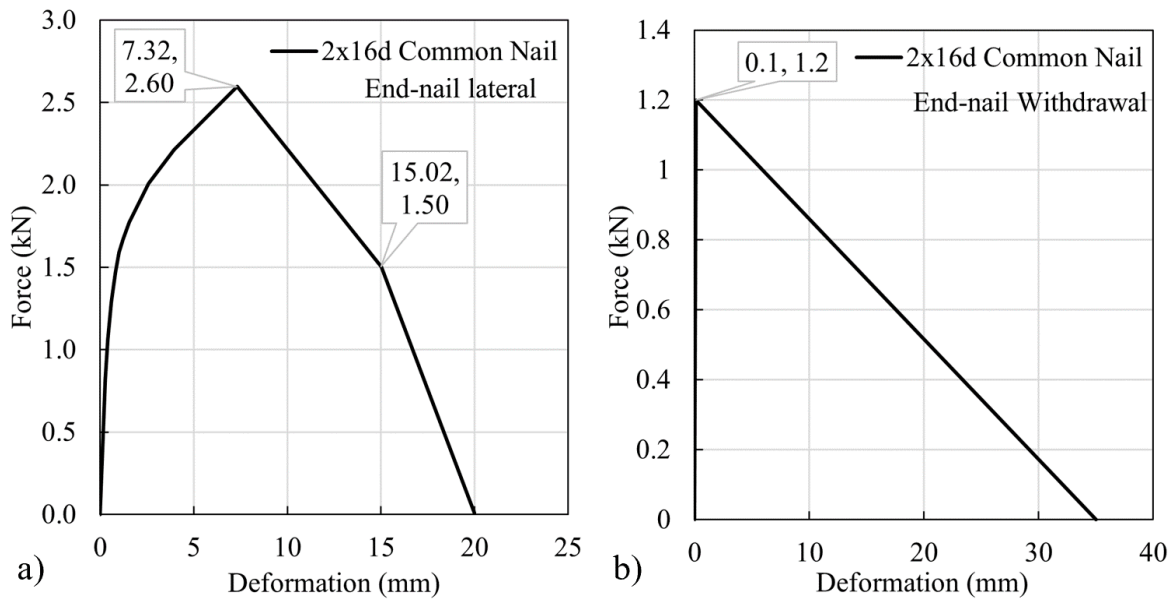


**Figure 5.1 Realistic representation of a wood shear wall**



**Figure 5.2: FE representation of the wood shear wall**

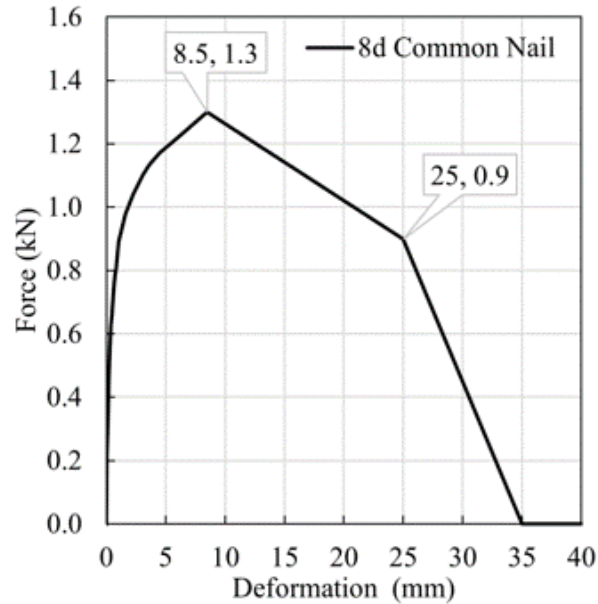
Link (spring) elements were used to model connections between the various components of the wood shear wall. Depending on the connection type, different F–D relationships were adopted. The vertical framing elements (studs) were connected to the top and bottom plates using bidirectional nonlinear link elements. The lateral behaviour of these connections was modeled using an F–D relationship developed based on the end–nail lateral connection tests in Section 4.2. Figure 5.3(a) shows an idealized curve based on end-nail lateral test data. The withdrawal behaviour was modeled using a tension-only link element, with an idealized F–D relationship derived from the end-nail withdrawal connection tests, as shown in Figure 5.3(b).



**Figure 5.3: F-D relationships for frame end connections: (a) end-nail lateral; (b) end-nail withdrawal**

To capture the behaviour of the sheathing-to-framing connections, the in-plane shear direction of the nail joint behaviour was modelled. Single-spring modeling approach was adopted. This technique avoids the overestimation of shear wall strength and stiffness that is inherent in double-spring models (Doudak and Smith 2009; Folz and Filiatrault 2001). The nonlinear behavior of these connections was defined using an idealized F–D relationship extracted from the sheathing-to-framing connection tests provided in Section 4.2 (Figure 5.4).





**Figure 5.4: Sheathing-to-framing F-D relationship**

In shear walls with multiple sheathing panels, contact between adjacent panel edges was modeled using *Gap* elements (compression-only link). The normal stiffness of these *Gap* elements was set to 40 kN/mm. This value was established through a sensitivity analysis, where values below this range led to noticeable artificial penetration and underestimation of wall stiffness after panel contact, whereas significantly higher values produced no meaningful change in global response but caused numerical ill-conditioning of the stiffness matrix. The adopted value therefore provides a numerically stable approximation of hard contact without distorting the structural response.

A linear elastic response was assumed for the HDs. This assumption is justified for two reasons. First, both the full-scale shear wall tests and the isolated hold-down connection tests showed that the force and deformation demand in the HDs remained well within their elastic range. No yielding, slip, or nonlinear uplift deformation was observed in the instrumentation records, indicating that the HDs were not a source of nonlinearity in the global system response. Second, the analysis considered monotonic loading, under which HDs typically exhibit a predominantly elastic F–D relationship up to capacities far exceeding the demands observed in LFW shear walls. Because

cyclic degradation, pinching, and low-cycle fatigue effects are not activated under monotonic loading, there is no need to incorporate stiffness degradation behaviour in the HD model.

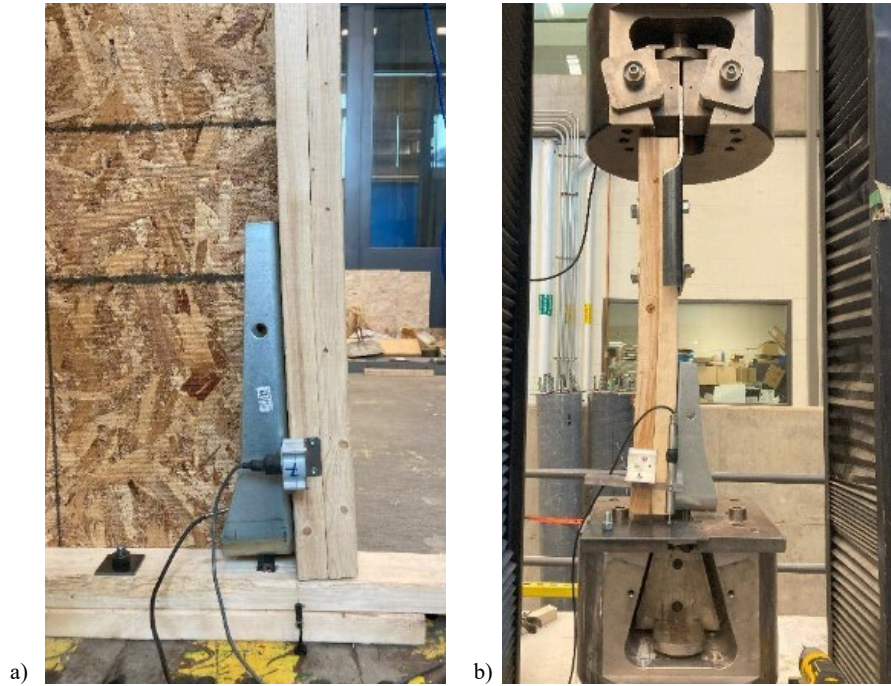
The stiffness of the HDs was derived based on the HD force and elongations demand curves recorded during the full-scale shear wall tests. To model the compression perpendicular-to-grain behaviour between the vertical studs and the top and bottom plates, *Gap* element was defined at frame intersection joints. A linear F–D relationship for this gap element was derived based on the measured perpendicular-to-grain compression deformations.

Table 5-1 summarizes the stiffness values assigned to the HD ( $K_{HD}$ ) and compression perpendicular-to-grain ( $K_c$ ) link elements.

Table 5-2: HD and perpendicular to grain stiffness values

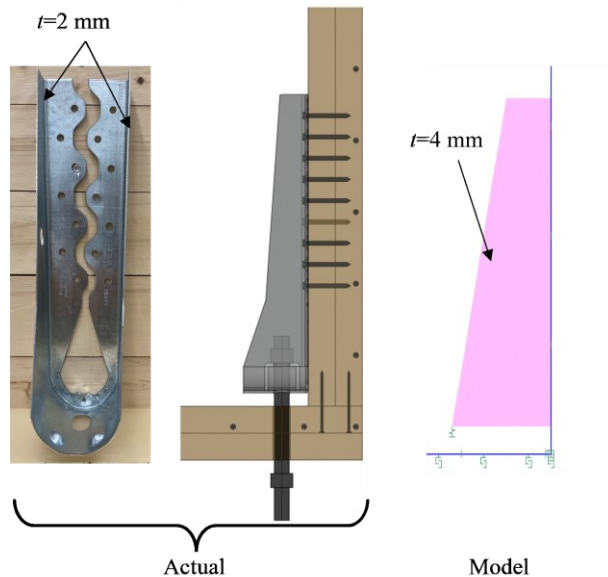
|         | $K_{HD}$ (kN/mm) | $K_c$ (kN/mm) |
|---------|------------------|---------------|
| Average | 6.53             | 6.94          |
| STDV    | 1.23             | 2.70          |
| CV      | 0.19             | 0.39          |

Observations from both the full-scale shear wall tests as well as isolated HD connection tests showed that attaching the HD device to the chord members produced a localized increase in flexural stiffness at the HD location. This stiffening effect caused the chord to develop a distinct change in curvature (i.e., a visible “kink”) near the upper end of the HD device, where the stiffness transitioned back to that of the unreinforced chord (see Figure 5.5).



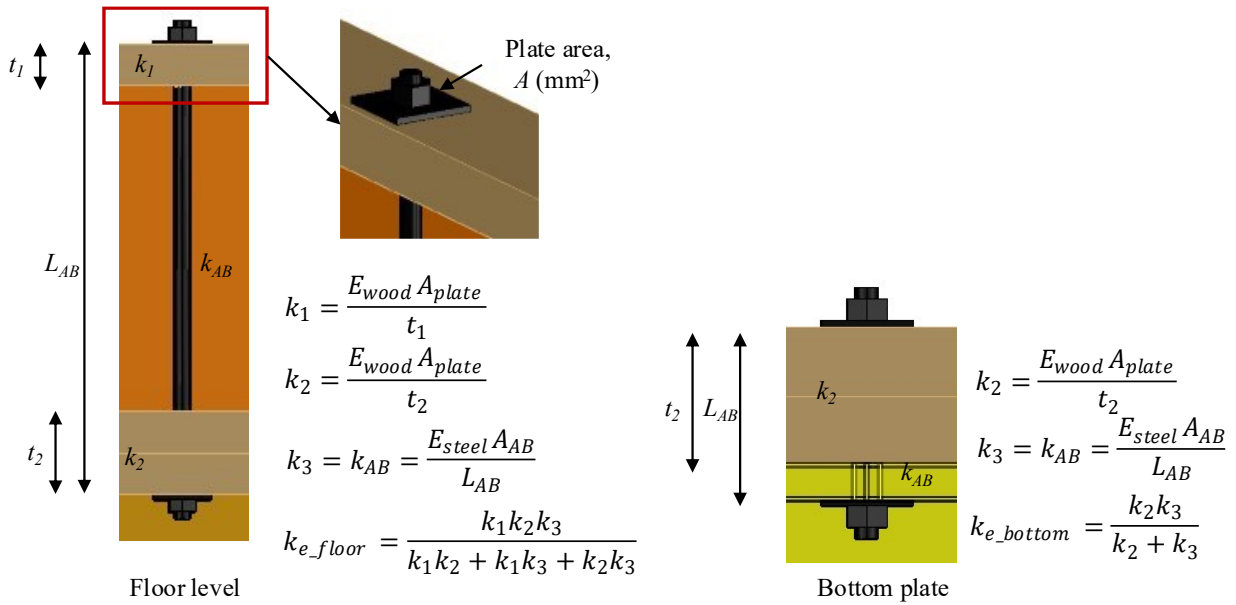
**Figure 5.5: Bending of the chord at HD location: a) shear wall test; b) HD test**

To capture this behavior in the numerical model, the HD device was represented as a steel bracket rigidly connected to the base of the chord members, providing the same localized stiffening observed experimentally. The bracket thickness was set to  $t = 4$  mm, corresponding to the combined thickness of the physical HD components used in the tests (Figure 5.6).



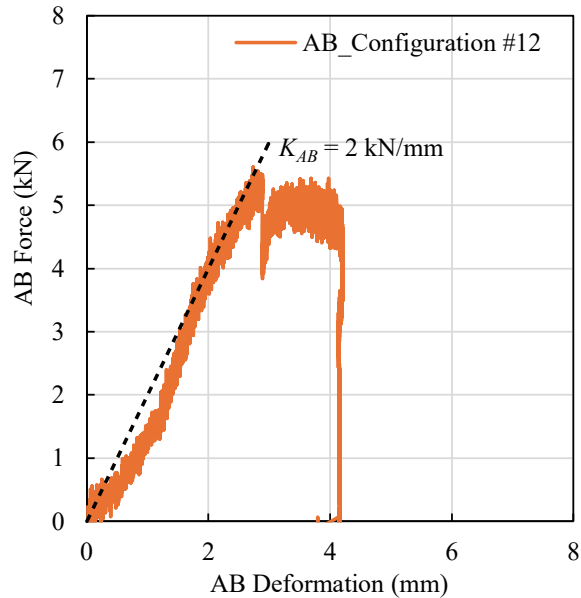
**Figure 5.6: HD bracket in numerical model**

The AB was assumed to be fixed in the horizontal direction, as shear wall sliding was negligible in all shear wall tests. The axial behaviour of the ABs was modeled using linear tension-only link elements. Since the AB connects multiple elements of the shear wall at the floor or bottom level, its effective stiffness must account for the contribution of all connected components. In this regard, the elements at the AB location act together as series spring system, as illustrated in Figure 5.7. The effective stiffness of the AB can then be determined from the material properties of the components using fundamental principles of mechanics.



**Figure 5.7: Calculation of the AB effective stiffness**

Based on Figure 5.7 and the material properties of the AB and stud, the stiffness values  $k_{e\_floor}$  and  $k_{e\_bottom}$  were determined as 5.8 kN/mm and 8.9 kN/mm, respectively. However, the measured AB force–deformation response in Configuration #12 showed a significantly lower stiffness compared to the mechanics-based value of 8.9 kN/mm. Figure 5.8 illustrates the AB force–deformation curve for Configuration #12, along with a linear idealization representing the effective stiffness of the AB.

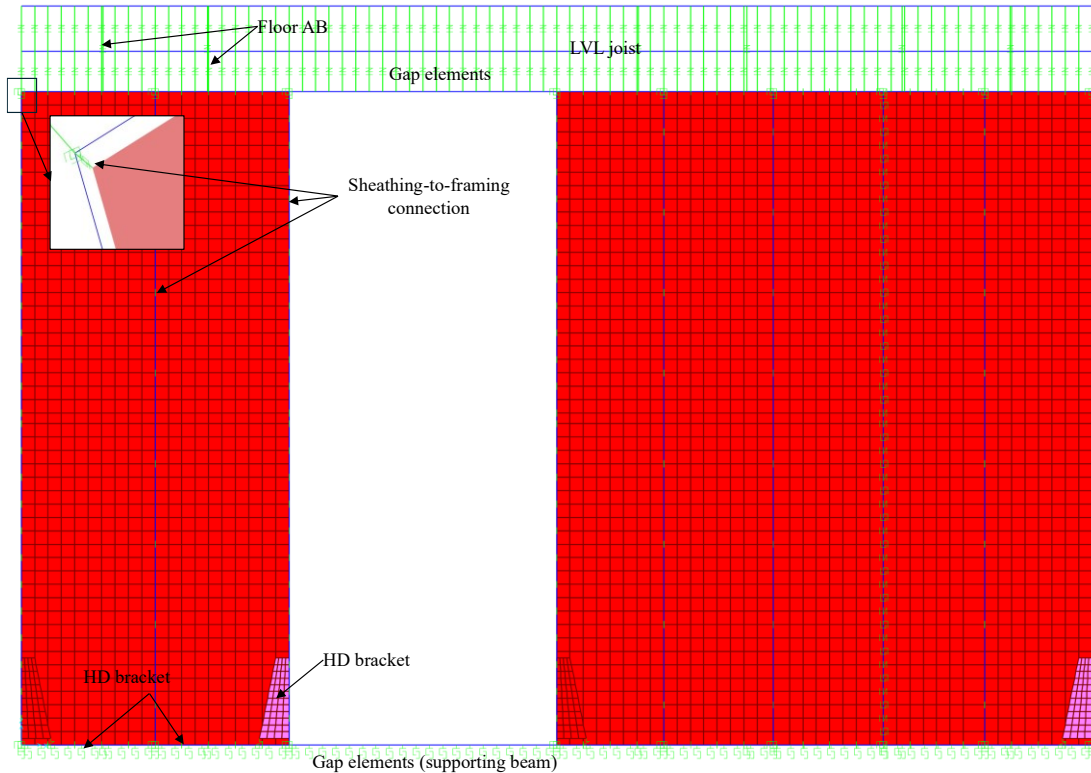


**Figure 5.8: AB force-deformation (Configuration #12)**

A stiffness value of 2 kN/mm allowed the model to reproduce AB forces consistent with the experimental results. Since there was no recording of the AB force-deformation at the floor level, the same stiffness value was applied to ABs at the floor level.

The contact between the shear wall bottom plate and the supporting steel beam, as well as the contact between the LVL joists and the surrounding plates at the floor level, was modeled using *Gap* elements. The stiffness of these gap elements was assumed to be 40 kN/mm, a value selected to ensure numerical stability and to prevent issues associated with an ill-conditioned stiffness matrix.

Figure 5.9 illustrates the FE model of the wood shear wall in SAP2000.



**Figure 5.9: FE model of the wood shear wall in SAP2000**

### 5.3 Analysis method

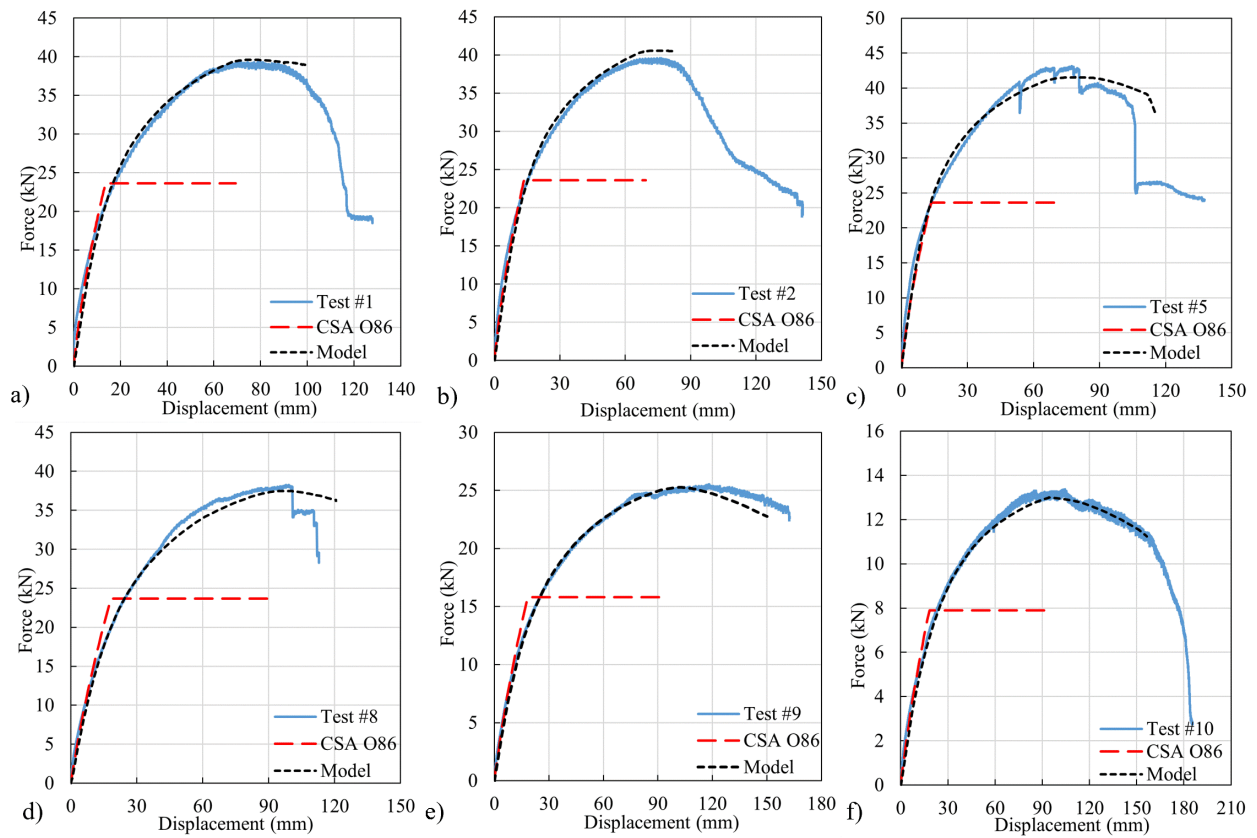
Nonlinear static analysis was performed to simulate the behaviour of wood shear walls, explicitly accounting for both material and geometric nonlinearities, including large-displacement effects. In this procedure, at each load increment, the global tangent stiffness matrix of the shear wall is updated to reflect its current displacement state. In particular, the stiffness of the sheathing-to-framing connections (nails) and other nonlinear components is evaluated based on their instantaneous force-deformation response. The equilibrium equations are then solved to determine the corresponding internal force distribution and nodal displacements.

By continuously updating the global stiffness of the shear wall, the analysis captures the nonlinear behaviour of the connections, including stiffness degradation and nail slip accumulation. The inclusion of geometric nonlinearity ensures that second-order effects associated with large displacements are incorporated, allowing the numerical model to explicitly represent potential gap

openings between framing elements, a phenomenon that becomes critical at large displacement levels. As a result, the redistribution of internal forces within the shear wall is represented in a consistent and mechanically rigorous manner throughout the loading history.

#### 5.4 Verification of the numerical model

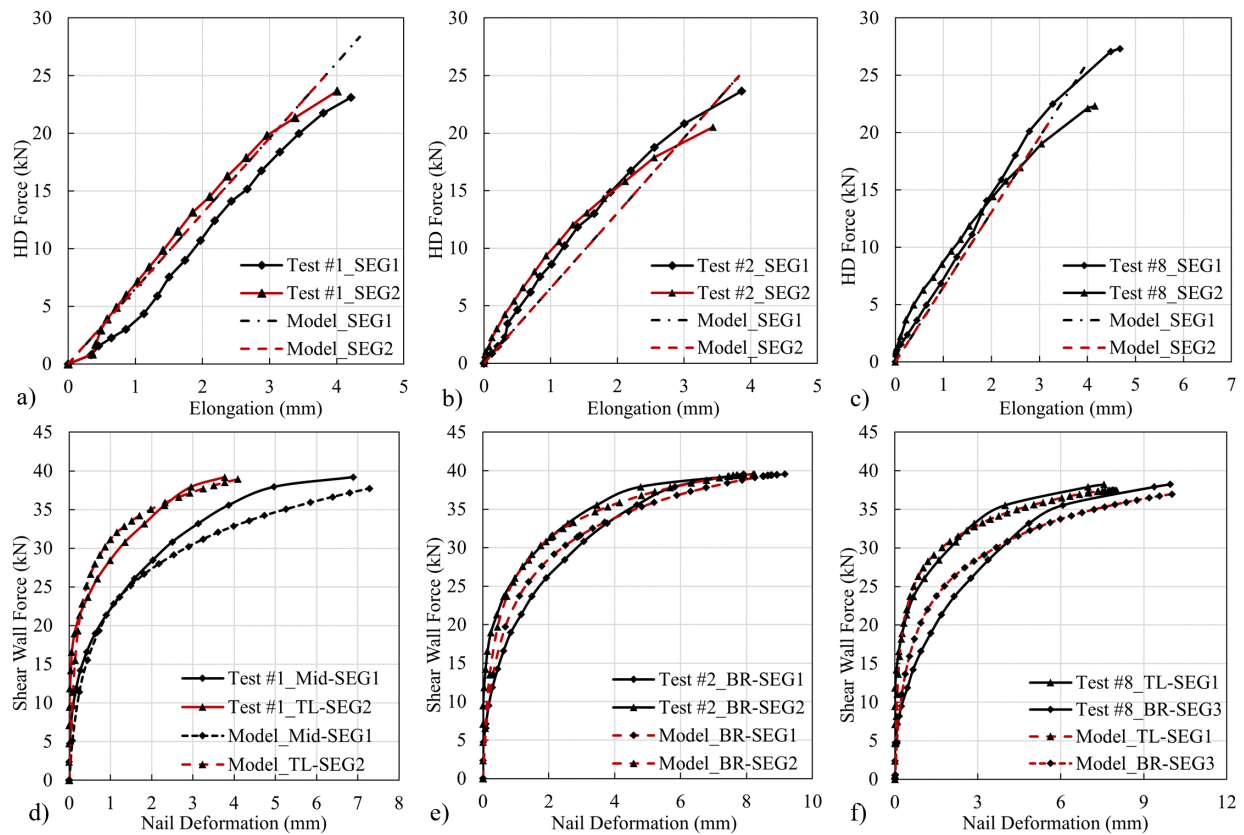
The developed numerical modelling framework was validated against the full-scale shear wall test results, reported in Section 4.5. The validation considered both the global response of the shear walls and the responses of individual components. For a comprehensive evaluation, the shear wall force-displacement curves according to CSA O86 (Canadian Standards Association 2019) were also included, allowing a comparison between the experimental results, numerical simulation, and design-level force and displacement demands, as illustrated in Figure 5.10(a)–(f).



**Figure 5.10: Validation of the numerical model against experimental shear wall test results**

As evident from the comparisons in Figure 5.10, the numerical model demonstrates a reasonable level of accuracy in predicting the response of the shear walls. Considering the inherent variability and uncertainty in wood as a material and the behaviour of connections joints, these results highlight that a careful combination of meticulously selected F–D relationships for the various connections, precise modeling of the shear wall kinematics, and a robust analysis procedure can capture the response of the shear walls with excellent agreement.

In addition to validating the numerical model against the overall shear wall response, comparisons were also made at the level of individual shear wall components. Of particular interest, the force and deformation demand of the HDs and the nail deformations, measured at selected locations during testing, were compared for a number of shear walls, as shown in Figure 5.11.



**Figure 5.11: Validation of the numerical model against test results: (a)–(c) HDs demands; (d)–(e) nail deformation demands**



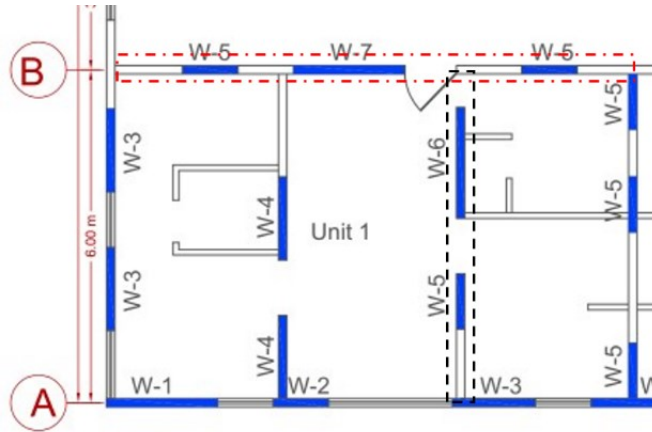
Figure 5.11(a)–(c) show the HD force and deformation demands measured during tests of shear walls in Configurations #1, #2, and #8. The numerical results demonstrate reasonable agreement with the experimental data. Some differences between the model predictions and the test results were expected, as the actual HD behaviour may not be perfectly linear.

The same considerations apply to the nail deformations shown in Figure 5.11(d)–(f). Due to the fact that only a single average value representing the F–D relationship was used for all nails, the model did not capture the inherent variability of the nail connections. As a result, some inconsistencies between the numerical predictions and the experimental measurements were expected. Nevertheless, when considering the average response, the numerical model closely reproduces the behaviour observed in the tests.

### **5.5 Multi-storey shear wall**

Building upon the validated numerical framework in previous sections, the model was extended to a multi-storey shear wall system in order to investigate the influence of floor out-of-plane stiffness on the displacement demands along the height of the walls. In this extended model, all previously defined components, including studs, sheathing panels, HDs, nails, ABs, and gap elements, were incorporated with the same parameter values as described in the models used for the validation in Section 5.1. This approach allows the assessment of vertical displacement profiles and the distribution of forces in different components along the height of the shear walls.

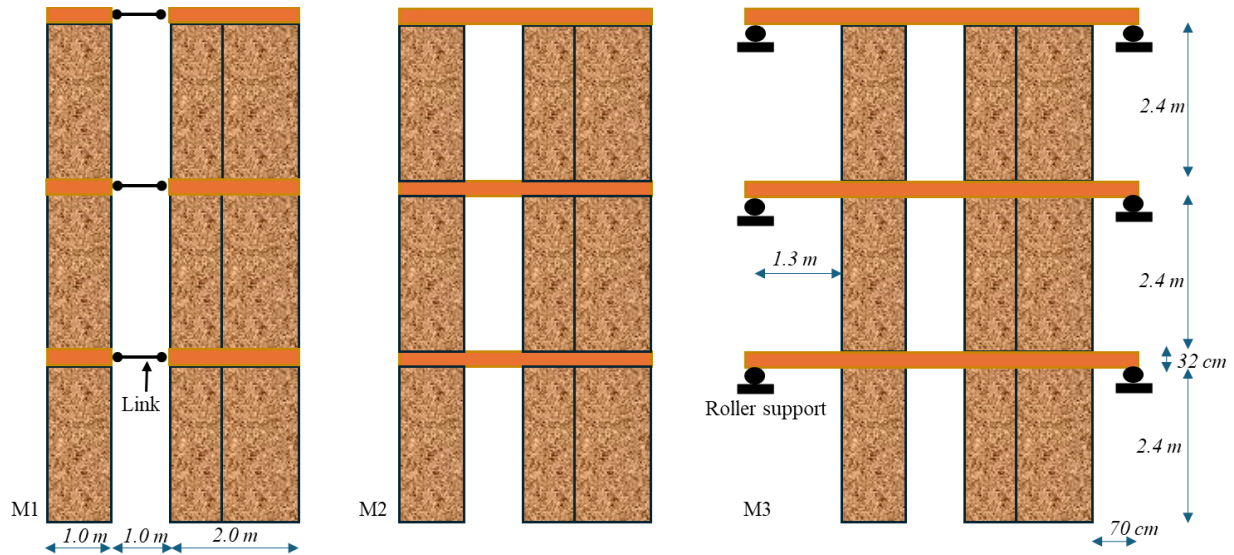
Figure 5.12 shows a portion of the plan layout presented in Figure 3.8, highlighting the shear walls selected for the multi-storey investigation.



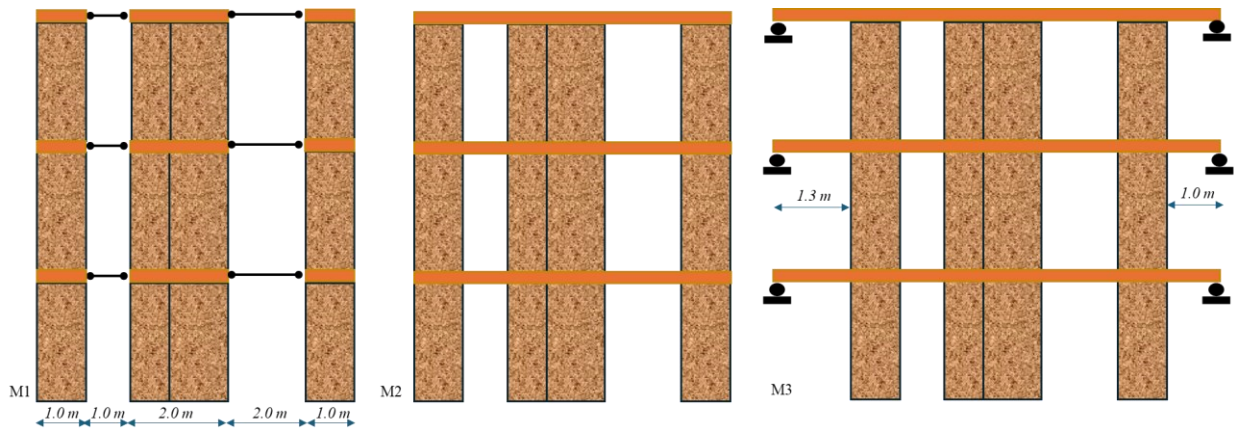
**Figure 5.12: Designated shear walls for multi-storey numerical simulation**

For this investigation, two three-storey configurations, consisting of three and two shear wall segments were considered. For each configuration, three distinct modelling scenarios were adopted to systematically examine the impact of the floor on the global response of the shear wall system. In the first scenario, referred to as M1, the effect of the floor was neglected, and the shear walls were connected through axial links incapable of resisting bending moments such that they acted independently. This represents the current assumption in the CSA O86 (Canadian Standards Association 2019). In the second scenario (M2), a floor assembly was included but was limited in extent to only be on top of the shear walls, providing some uplift restraint while limiting interaction with adjacent structural components (e.g., perpendicular walls). In the third scenario (M3), the floor was extended beyond the shear walls and was anchored to perpendicular shear walls, as shown in Figure 5.12. This represents the most realistic scenario in terms of boundary conditions for the floor system, while still maintaining the simplicity of investigating a 2D frame.

These scenarios were deliberately selected to represent varying degrees of floor participation that may exist in real LFW buildings, and to isolate the contribution of floor out-of-plane stiffness to the global displacement response of multi-storey shear wall systems. Figure 5.13 and Figure 5.14 present the elevation views of the two- and three-segment three-storey shear wall configurations, respectively.



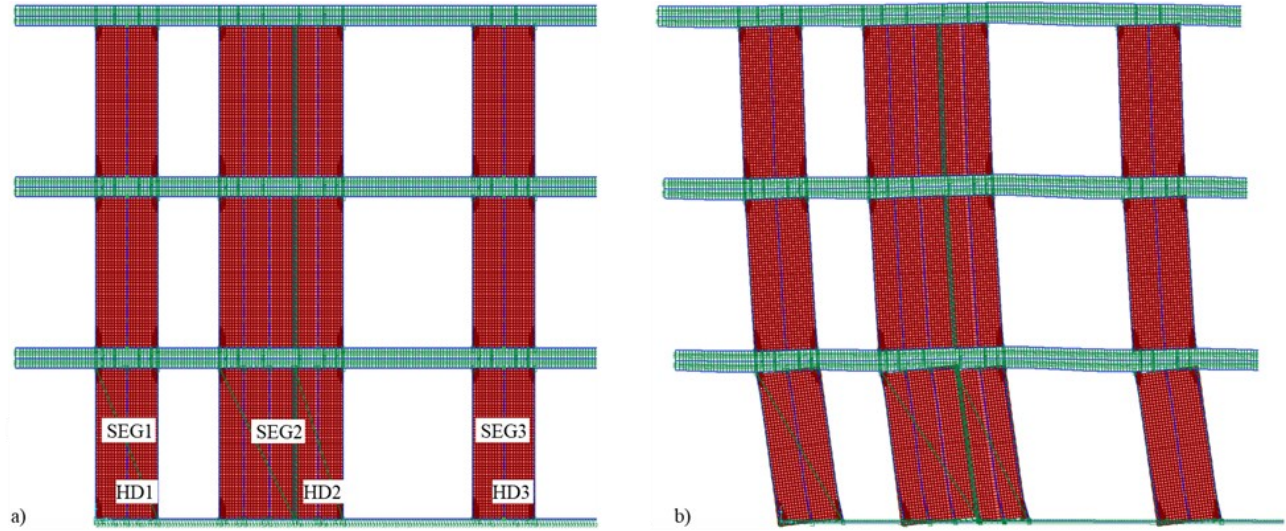
**Figure 5.13: Two-segment three-storey shear wall configuration**



**Figure 5.14: Three-segment three-storey shear wall configuration**

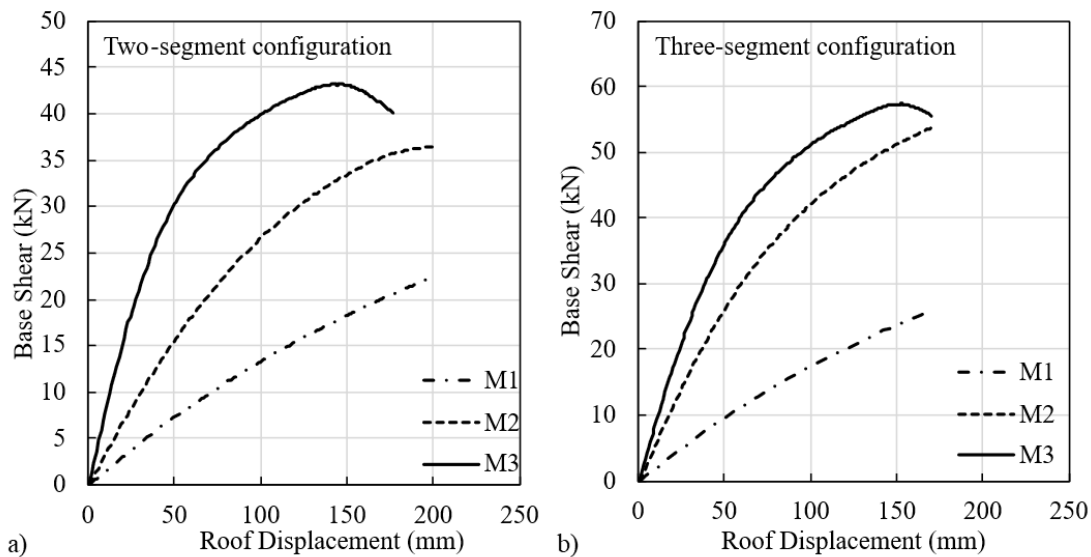
All shear walls were constructed using 8d nails, with a uniform nail spacing of 100 mm applied at all three storeys. HDs were selected based on storey level: HDU8 was used at the first and second storeys, while HDU2 was assigned to the third storey. The shear wall configurations were first analysed and designed using WoodWorks® software, from which the base shear forces were determined to be 9.4 kN and 12.7 kN for the two- and three-segment configurations, respectively. Subsequently, the shear walls were modelled in SAP2000®, in a similar manner to approach described in Section 5.1, and nonlinear static analyses were performed. The lateral load patterns applied over the height of the shear walls were defined to be consistent with the distribution of

lateral forces obtained from the WoodWorks® analysis results for both configurations. For the two-segment configuration, the storey forces were 5.7 kN, 3.2 kN, and 0.5 kN at storeys 3, 2, and 1, respectively; the corresponding values for the three-segment configuration were 7.3 kN, 3.8 kN, and 1.6 kN. Figure 5.15(a) and (b) illustrate the three-segment shear wall configuration with scenario M3 before and after analysis.



**Figure 5.15: Numerical model of three-segment shear wall configuration**

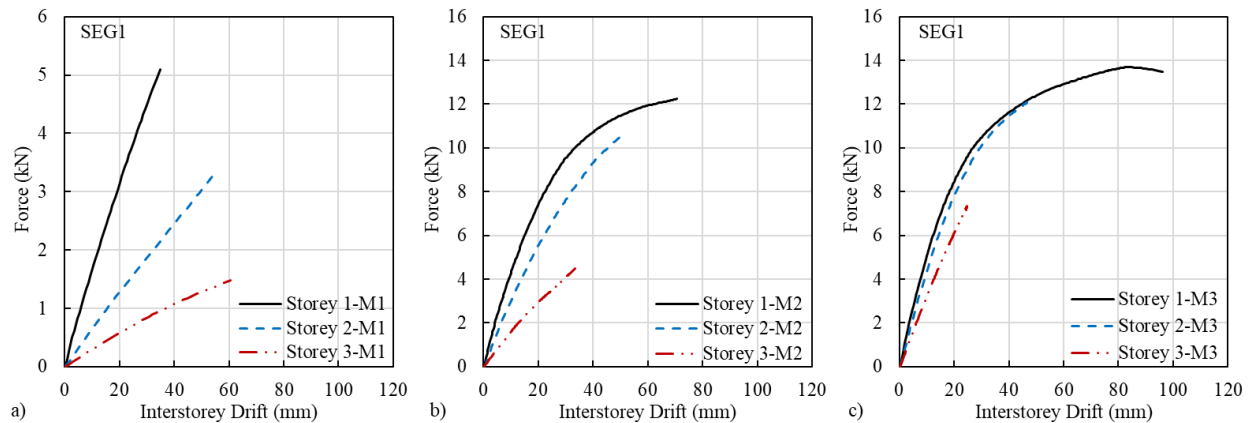
The base shear-roof displacement responses obtained from the nonlinear static analyses of two- and three-segment shear walls are presented in Figure 5.16(a) and (b).



**Figure 5.16: The effect of the floor on the lateral behaviour of multi-storey shear walls**

It is evident from Figure 5.16 that the inclusion of floor out-of-plane stiffness considerably reduced the lateral displacement of both shear walls configurations. This clearly demonstrates that the global lateral stiffness of multi-storey shear walls is strongly influenced by the out-of-plane stiffness of the floor system. Furthermore, these figures show that considering realistic boundary conditions for the floor system, as represented by M3 modelling assumption, resulted in further increase in the global stiffness of the multi-storey shear walls. The analysis showed that the lateral stiffness of the two-segment model based on the M3 modelling assumption was 5.5 times higher than that obtained using the M1 assumption. For the three-segment model, the corresponding increase was a factor of 4. At a roof displacement of 150 mm, the predicted capacity of both models was nearly doubled. It should again be emphasized that although increasing the stiffness and capacity only produces positive effects when considering the performance of shear walls resisting wind loading, in the case of seismic design the increase in stiffness would lead to increase in base shear forces that needs to be considered in the evaluation.

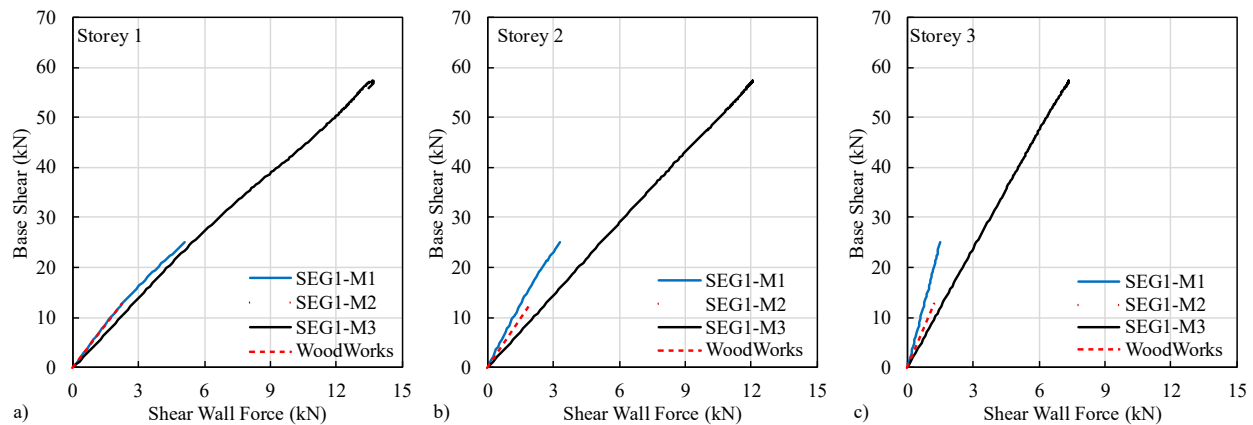
Figure 5.17(a)–(c) present the force versus inter-storey drift responses of SEG1 in the three-segment shear wall configuration at different storey levels. The comparisons presented hereafter are based on the three-segment model; however, similar comparisons were conducted for the two-segment model, and the results were consistent.



**Figure 5.17: Shear wall force versus inter-storey drift curves**

These results showed that, at a given force level, the same shear wall exhibited different deformation demands under different modelling scenarios. This observation indicates that the stiffness of a shear wall within a system is governed not only by its own properties but also by the characteristics of the system. In particular, assumptions regarding floor out-of-plane rigidity had a significant influence on the effective stiffness of individual shear walls within the system.

The distribution of forces along the height of the shear walls was also influenced by the characteristics of the structural system. Figure 5.18(a)–(c) present the relationship between shear wall force and base shear for SEG1 in the three-segment configuration at different storey levels.



**Figure 5.18: Distribution of the shear wall forces at different storey levels**

These figures indicated that floor out-of-plane rigidity, as well as the interaction between shear walls and surrounding structural components, resulted in different force demands on a shear wall at a given storey. For instance, the forces in SEG1 at the first storey remained nearly identical across the different modelling assumptions, whereas variations were observed at the second and third storeys as the modelling assumptions changed. Also shown in these figures are the shear wall forces extracted from WoodWorks software, which assumed no floor constraint in the force distribution. This force distribution was consistent with the current mechanics-based shear wall assumptions adopted in CSA O86 (Canadian Standards Association 2019).

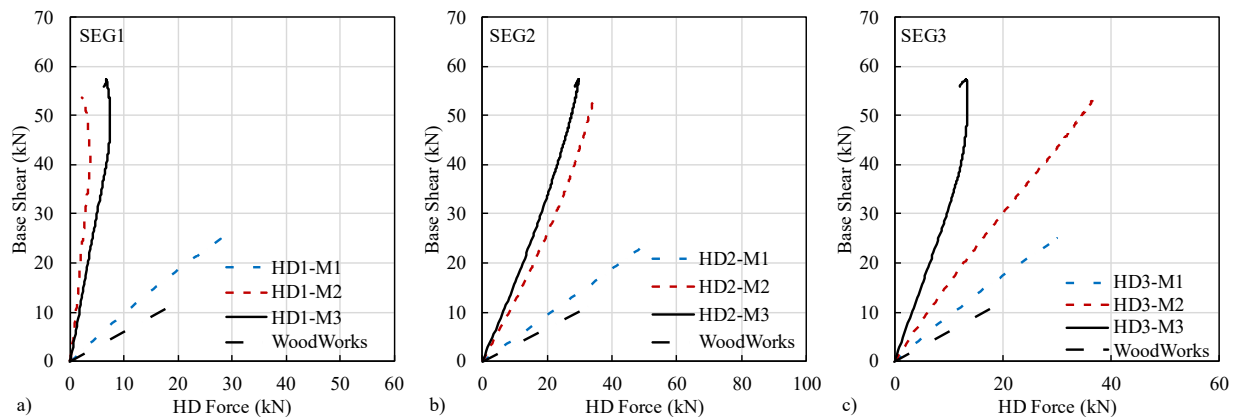
System-level characteristics can also affect the distribution of forces among shear walls at a given storey. Table 5-3 illustrate the influence of different modelling assumptions on the force distribution between shear walls at the third-storey level of the three-segment shear wall configuration.

Table 5-3: Distribution of forces between shear walls in the third level

|           | SEG1 (kN) | SEG2 (kN) | SEG3 (kN) |
|-----------|-----------|-----------|-----------|
| <b>M1</b> | 1.5       | 11.5      | 1.5       |
| <b>M2</b> | 2.1       | 9.4       | 3.0       |
| <b>M3</b> | 3.1       | 6.8       | 4.6       |

As shown in Table 5-3, compared to the M1 modelling assumption, the M3 modelling assumption resulted in higher force demands in SEG1 and SEG3, while the force in SEG2 was significantly reduced. Although SEG1 and SEG3 had the same length (1.0 m), the increase in force demand was more pronounced in SEG3, indicating a non-uniform force distribution.

Figure 5.19(a)–(c) present the HD force demands at the bottom storey of the three-segment shear wall configuration as a function of base shear.



**Figure 5.19: HD force demands in shear walls**

Significant reductions in HD force demands were observed under the M2 and M3 modelling assumptions. These figures also show that the HD forces obtained from WoodWorks software were close to those from the model without floor effects (M1). Table 5-4 compares the HD forces at the



first storey of the three-segment shear wall model. These forces were extracted at a base shear force of 12.65 kN, consistent with the value used in WoodWorks.

Table 5-4: Distribution of HD forces in shear walls in the first storey

|                  | SEG1 (kN) | SEG2 (kN) | SEG3 (kN) |
|------------------|-----------|-----------|-----------|
| <b>WoodWorks</b> | 21.1      | 37.3      | 21.1      |
| <b>M1</b>        | 13.1      | 27.1      | 14.0      |
| <b>M2</b>        | 1.3       | 10.2      | 7.9       |
| <b>M3</b>        | 2.1       | 8.0       | 4.0       |

The differences between the WoodWorks results and the M1 model were primarily due to the contribution of the AB, which is not considered in WoodWorks. In addition, as discussed earlier, the distribution of shear wall forces over the height was influenced by the modelling assumptions. Consequently, the cumulative HD forces were also affected by these assumptions.

Overall, the results presented in this section demonstrate that the modelling assumptions adopted for floor out-of-plane stiffness have a significant influence on the global and local responses of multi-storey shear wall systems. Differences are observed in lateral displacement demands, effective stiffness, force distribution along the height, force sharing between shear wall segments in the storey level, and HD force demands. The comparative results for the M1, M2, and M3 scenarios provided a consistent basis for evaluating the role of floor participation in multi-storey LFW shear walls.

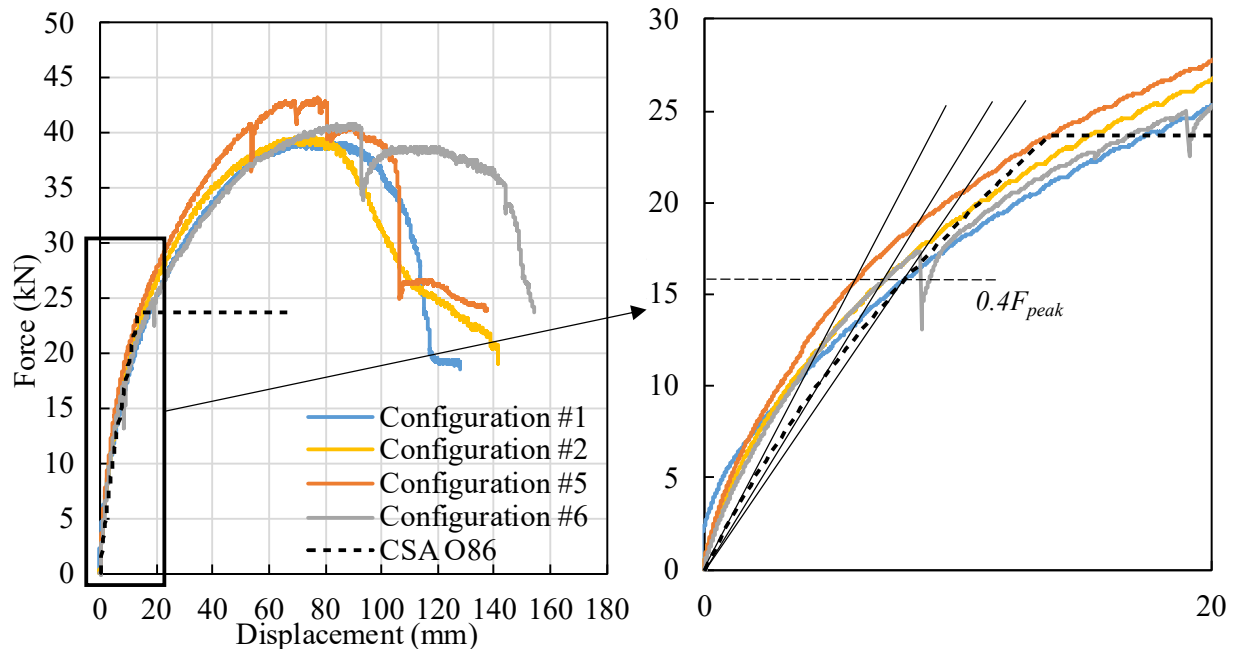


## Chapter 6

### Discussion

#### 6.1 Effect of the floor assembly on the shear wall stiffness

Figure 6.1 compares the F–D curves for Configurations #1, #2, #5, and #6 recorded during testing. These configurations were geometrically identical, each consisting of a 1.0 m and a 2.0 m shear wall separated by a 1.0 m opening, but different in terms of the presence of the floor assembly (Configurations #2, #5 (one LVL beam), and #6 (two LVL beams)) and wall-to-floor HD connections (Configurations #5 and #6) relative to the reference case (Configurations #1).

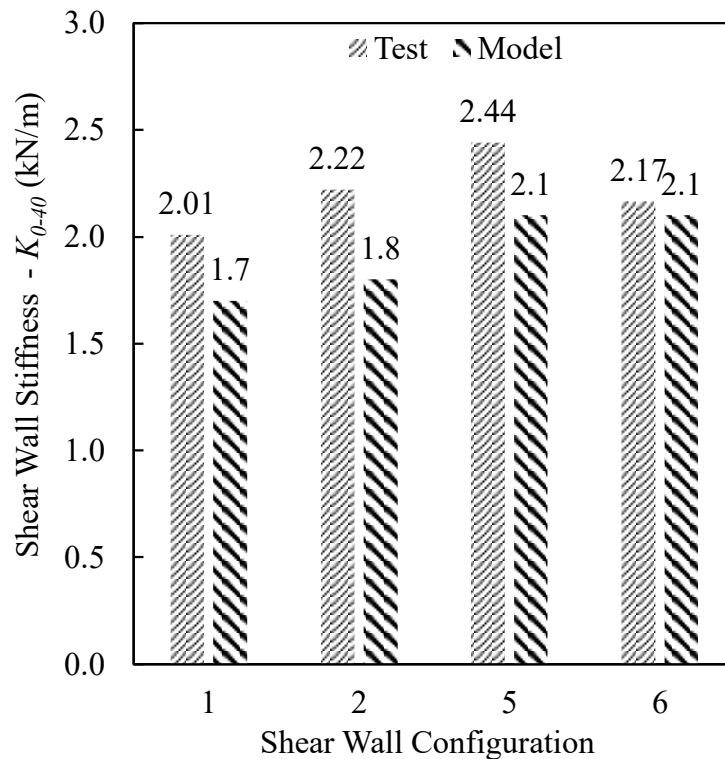


**Figure 6.1: The effect of the floor assembly on the initial stiffness of shear walls**

Based on visual observation, the curves indicate that the presence of the floor assembly had only a marginal influence on the overall initial ascending portion of the response. Also shown in Figure 6.1 is the F–D design curve obtained from CSA O86 (Canadian Standards Association 2019) based on the four-term deflection equations with inputs consistent with those from the tests. The comparison demonstrates reasonable overall agreement between the test results and the design

values up to the factored resistance of the shear wall ( $V_r = 23.7$  kN). In fact, less nonlinearity seems to be exhibited by the design curve, and it is observed to fit an average behaviour, where it appears to underestimate the stiffness at low load levels, while for higher load levels it tends to overestimate the stiffness.

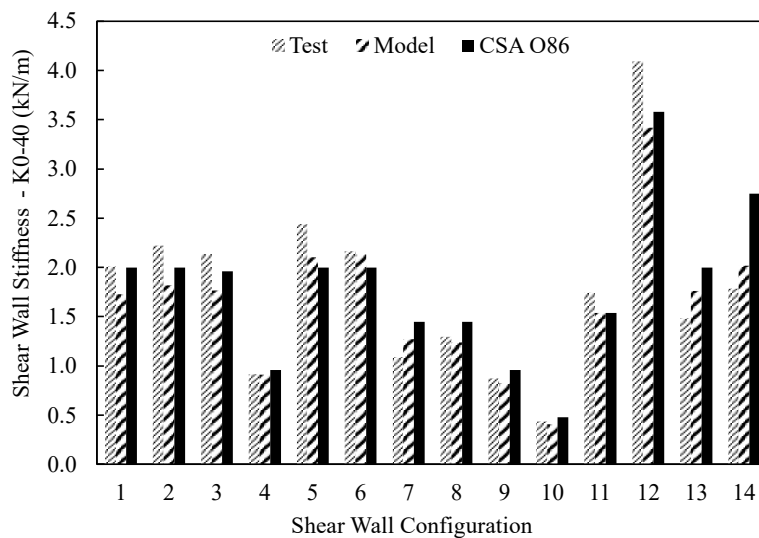
To quantitatively compare the shear wall stiffnesses, Figure 6.2 presents the  $K_{0-40}$  values for Configurations #1, #2, #5, and #6 obtained from EEEP analysis for these configurations (ASTM E2126 2019). For comparison, the values obtained from the model developed in Chapter 5 are also presented in the figure.



**Figure 6.2: Comparison between the initial stiffness ( $K_{0-40}$ ) of the shear walls from EEEP analysis**

It can be observed that there is a marginal increase in initial stiffness, with Configuration #5 exhibiting the highest increase (20%), where the floor-to-wall connection included HDs at the floor level. In general, the inclusion of the floor assembly and wall-to-floor connections seem to cause an increase in the shear wall stiffness by an average of 14%. The model predictions show

reasonable agreement with the corresponding experimental data in terms of general trends, while the values of stiffness are consistently underestimated. This is expected since the tested shear wall specimens involve other mechanisms such as friction, self weight of wall (providing a small counter moment), and other physical compatibilities due to the test setup. It is interesting to note that even in the model there was no significant difference between Configurations #5 and #6, indicating that introducing one or two LVL joists had insignificant impact on the performance. This also provides an explanation on the observation where Configuration #6 did not yield higher stiffness value compared to Configuration #5, leading to the hypothesis that the difference observed is likely due to variability as well as the fact that the stiffness of these two shear walls was primarily governed by the sheathing-to-framing connections. Since the shear wall stiffness was primarily governed by the sheathing-to-framing connections, the relative out-of-plane stiffness of the floor assembly was effective only up to a certain level, beyond which further increases in floor out-of-plane stiffness did not result in a noticeable increase in overall shear wall stiffness. Figure 6.3 compares the initial stiffness values of all shear wall configurations obtained from CSA O86 (Canadian Standards Association 2019), test, and numerical simulation results.



**Figure 6.3: Comparison between shear wall initial stiffness values from CSA O86, test results and numerical simulations**

As illustrated in Figure 6.3, although the presence of the floor assembly led to an increase in the initial stiffness for some configurations, as discussed previously, several specimens (i.e., Configurations #4, #7, #8, #9, #13, and #14) consistently exhibit lower initial stiffness compared to CSA O86 (Canadian Standards Association 2019). It should be noted that no corresponding tests without a floor assembly were conducted for these configurations, preventing a direct comparison. Nevertheless, CSA O86 (Canadian Standards Association 2019) generally provides a close prediction of the initial stiffness for most configurations, except for shear walls with ATS continuous rods (Configurations #13 and #14), for which it consistently overestimates the initial stiffness.

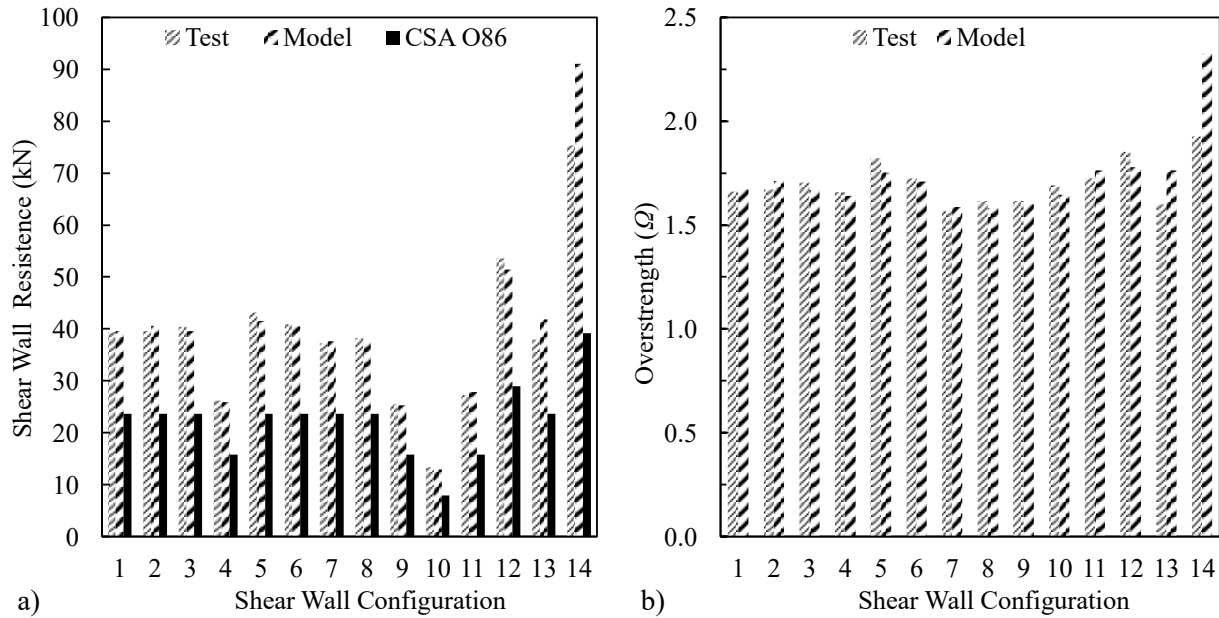
Shear walls with ATS rods showed a comparatively different behaviour. Configuration #13 exhibited a lower stiffness ( $K_{0-40}=1.48$  kN/m) than Configuration #2 with discrete HDs ( $K_{0-40}=2.22$  kN/m). This reduction could be attributed to the effective axial stiffness of the ATS rods, which is discussed later in Section 6.5.1. Given the lower overall stiffness observed in Configurations #13 and #14 compared to their CSA O86 (Canadian Standards Association 2019) values, further experimental studies are needed to better quantify the initial stiffness of shear walls incorporating ATS rod systems.

The comparison of initial stiffness across the shear wall configurations in this study did not indicate a consistent increase, and no clear trend is observed. This conclusion is subject to several limitations, including the limited number of test specimens as well as the definition of initial stiffness based on the  $K_{0-40}$  method, which could be sensitive to the shape of the F–D curve. Furthermore, variability in the experimental data may contribute to inconsistencies, such as those observed between Configurations #7 and #8. The nail connection tests exhibited considerable

variability, particularly in stiffness and displacement at the nail design load. Although not necessarily proportional, this variability may have propagated into the overall shear wall response.

## 6.2 Effect of the floor on shear wall resistance

The peak resistance of the shear walls obtained from testing and numerical simulations was compared with the corresponding factored resistance calculated in accordance with CSA O86 (Canadian Standards Association 2019), as shown in Figure 6.4(a). In addition, the overstrength factor,  $\Omega$ , was calculated for each configuration based on both test and numerical results and compared across the individual specimens, as illustrated in Figure 6.4(b).



**Figure 6.4: (a) comparison between the peak and factored resistance shear walls; (b) shear wall overstrength values**

As expected, the comparisons in Figure 6.4(a) show that the factored resistance values are consistently lower than the corresponding peak resistances, which is in line with the underlying design philosophy. A comparison of the overstrength ( $\Omega$ ) values for single-segment walls without floor influence (Configurations #10, #11, and #12) and multi-segment walls with floor suggests that the presence of the floor had a negligible effect on shear wall strength.

The average overstrength relative to the factored design resistance was  $\Omega = 1.7$  with a coefficient of variation (CV) of 0.06, indicating low variability in the test data. A value of overstrength equal to 1.86 is reported in the commentary to Canadian Wood Council Wood Design Manual (Canadian Wood Council 2020), however, this value is based on specified strength. The equivalent value from the test would be  $1.7 * \phi$  (where  $\phi = 0.8$ ), yielding a factor of 1.4, which seems to be significantly lower than reported in the Wood Design Manual (Canadian Wood Council 2020). The higher overstrength factor in the commentary likely reflects a broader experimental database, including variations in sheathing thickness, nail diameter, nail spacing, and sheathing type (e.g., plywood versus OSB).

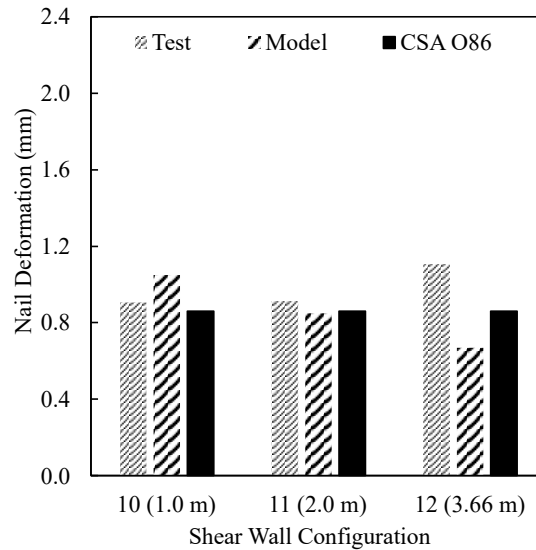
### **6.3 Effect of the floor assembly on nail deformations**

In order to better understand how the floor assembly influenced the nail deformations, the behaviour of single-segment walls without floor interaction was first examined. Figure 6.5 compares the nail deformations obtained from testing of Configurations #10, #11, and #12 with the corresponding values calculated according to CSA O86 (Canadian Standards Association 2019) at the design stage. Since nail deformations were recorded at multiple locations during testing, the average measured value was used for comparison with the CSA O86 (Canadian Standards Association 2019) values.

An important consideration is that the nail deformations calculated in accordance with CSA O86 (Canadian Standards Association 2019) is expected to represent the maximum nail deformations at the shear wall corners. However, due to instrumentation constraints, nail deformations could not be recorded exactly at the corner locations. Consequently, the actual corner deformations are expected to be greater than the measured values reported herein. A detailed discussion of this topic is presented in the subsequent sections. The nail deformations from the numerical simulations were

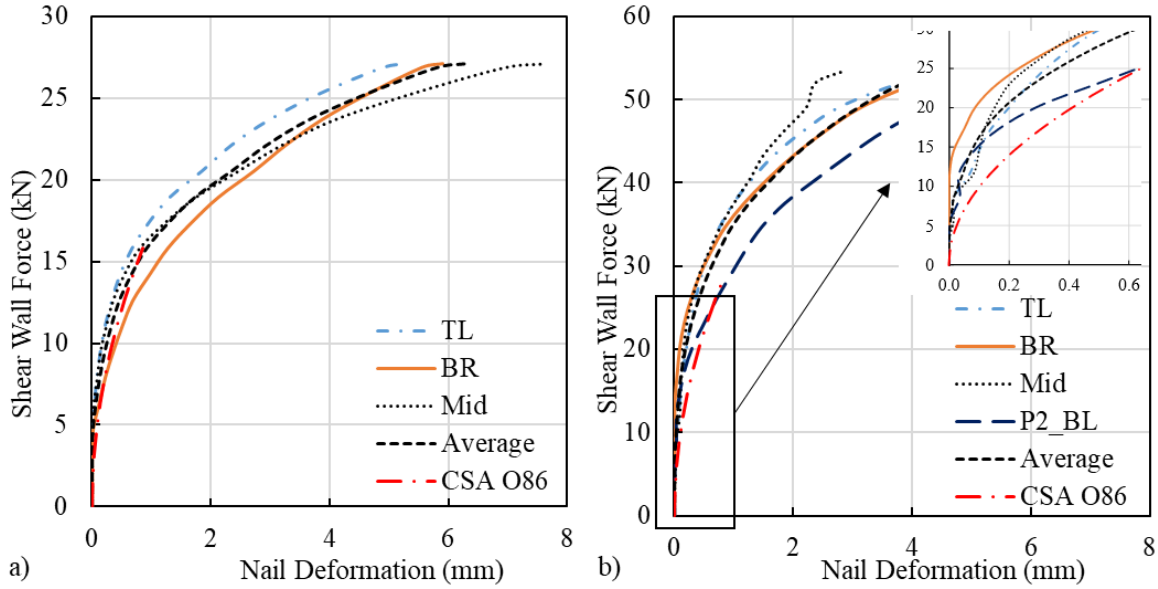
extracted at locations corresponding to the DT positions used in the experiments to provide direct comparison with the test results. It should be noted that these comparisons were conducted at a load level corresponding to the factored resistance of each shear wall configuration ( $V_r$ ), obtained in accordance with the provisions of the CSA O86 (Canadian Standards Association 2019).

Figure 6.5 presents the nail deformations for shear wall Configurations #10–#12.



**Figure 6.5: Nail deformation in single-segment shear walls without floor**

As shown in Figure 6.5, the nail deformations recorded during testing are in reasonable agreement with those calculated in accordance with CSA O86 (Canadian Standards Association 2019). As the measured and numerical values were averaged among recordings at different locations, the average values can be influenced by extreme local deformations. This is to say, if the variability between measured values is high, the average value may have had tendency toward very large or very small values depending on the frequency of data. To illustrate the influence of extreme values on the computed average, Figure 6.6(a) and (b) present the distribution of nail deformations recorded at various locations in Configurations #11 and #12, respectively.



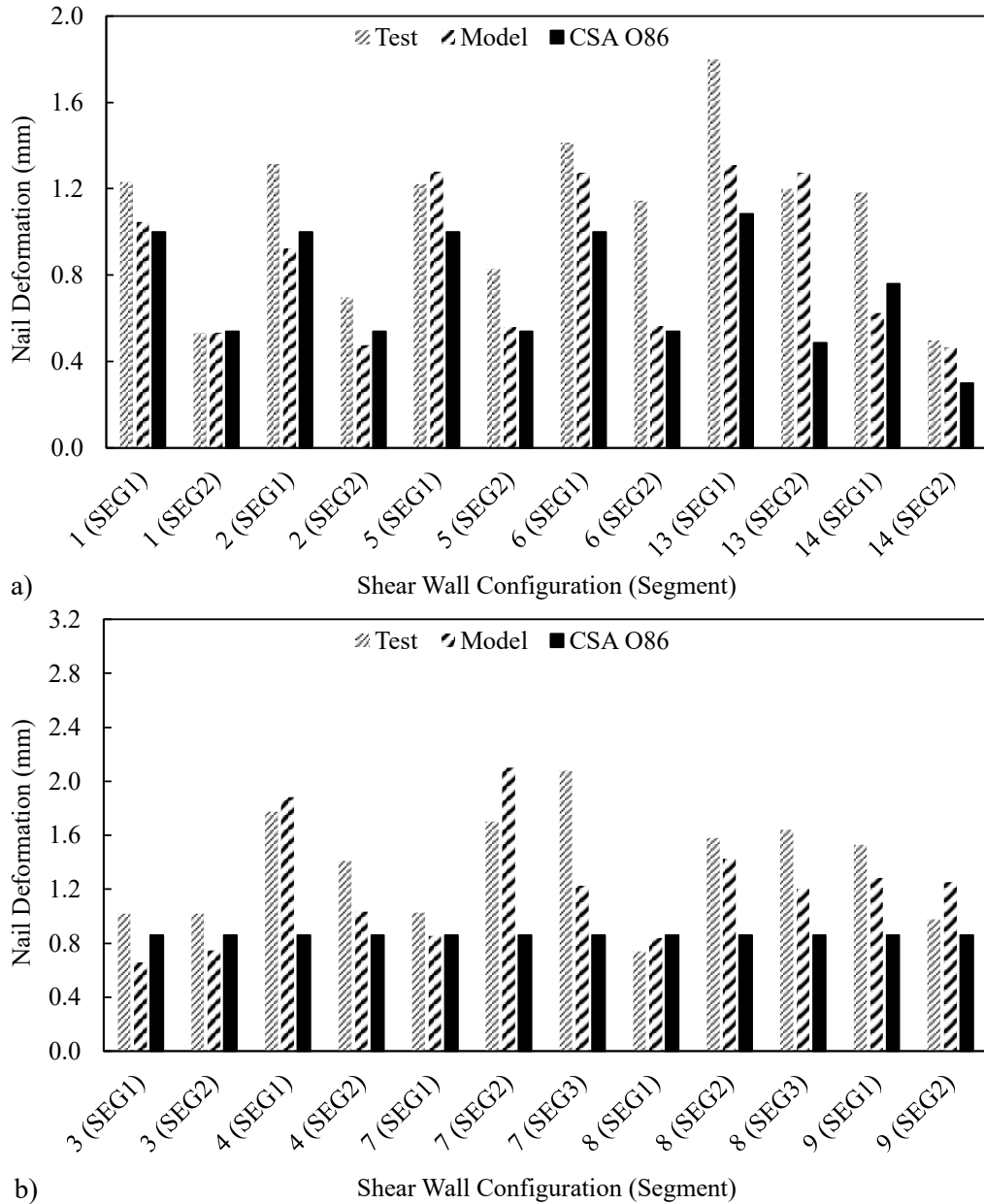
**Figure 6.6: Nail deformations: (a) Configuration #11; (b) Configuration #12**

As observed in these figures, the average curve for Configuration #11 closely follows the CSA O86 (Canadian Standards Association 2019) curve (Figure 6.6(a)), whereas the difference between the average curve for Configuration #12 and the corresponding CSA O86 (Canadian Standards Association 2019) curve (Figure 6.6(a)) was significant for a wide range of shear wall loads. It is observed that the CSA O86 (Canadian Standards Association 2019) curve is relatively close to the P2\_BL curve, particularly at load levels approaching the shear wall factored resistance.

One possible explanation for the discrepancy is that nail deformations were measured at only a limited number of discrete locations; given the length of the wall, the nail deformations at non-instrumented locations may have either exceeded or been lower than the values predicted by CSA O86 (Canadian Standards Association 2019). Another contributing factor may be the loading mechanism, where the actuator applied force at only one end of the wall. This could lead to a non-uniform load distribution along the wall, an effect that is less pronounced in shorter specimens. Evidence of this behaviour can be seen in Figure 6.6(b), where the BR curve shows no measurable response until the shear wall load reached approximately 13 kN (less than 25% of the peak load).



Similar to single-segment walls, comparisons were made to assess how floor presence affects nail deformations in multi-segment shear walls. Figure 6.7 illustrates the nail deformations of the multi-segment shear walls. Figure 6.7(a) corresponds to configurations with wall segments of unequal lengths, while Figure 6.7(b) presents those with equal wall lengths. These comparisons were conducted at each shear wall's design load level.



**Figure 6.7: Nail deformations of multi-segment shear walls: (a) unequal wall lengths; (b) equal wall lengths**

Configuration #1 in Figure 6.7(a) represents the specimen without a floor assembly and serves as the reference case. For this configuration, the test and numerical results were close to the CSA O86 (Canadian Standards Association 2019) values, indicating good agreement under baseline conditions.

Configuration #2 showed approximately 30% increase in nail deformations in both SEG1 and SEG2. When HD connections were introduced at the floor level (Configuration #5), the increase was 22% in SEG1 and 53% in SEG2. Incorporating double LVL joists in the floor assembly even led to more significant increase in nail deformations, reaching 41% in SEG1 and 112% in SEG2. The latter clearly demonstrates the role of high out-of-plane floor stiffness in promoting racking (over rocking) behaviour in shear walls.

The effect was even more pronounced in shear walls with ATS rods. Configuration #13 exhibited increases of 66% and 147% in SEG1 and SEG2, respectively. Configuration #14, with 50 mm nail spacing, showed increases of 56% in SEG1 and 66% in SEG2.

Referring to Figure 6.7(b), shear walls with higher ARs experienced different levels of increase in nail deformation due to the floor. In general, as the AR increased (Configurations #4, #7, and #8), the floor-induced increase in nail deformations also became more significant. This could possibly be explained by the important role HD plays in walls with high AR ( $AR = 2.44$ ).

The lower nail deformations observed in SEG1 of Configurations #7 and #8 were attributed to a local and unexpected issue with the supporting beam. These segments initially underwent a rigid-body movement caused by the settlement of the compression chords, resulting from an unnoticed gap between the shear wall bottom plate and the supporting beam.

A comparison between Configurations #4 and #9 in Figure 6.7(b) indicates that a continuous LVL joist caused a greater increase in nail deformation compared to floor assembly with blocking. In

Configuration #4, SEG1 and SEG2 showed increases of 107% and 64%, respectively, whereas the corresponding increases in Configuration #9 were 78% and 14%. A similar trend was observed when comparing Configurations #7 and #8, where Configuration #7 exhibited approximately 20% greater increase in nail deformation than Configuration #8.

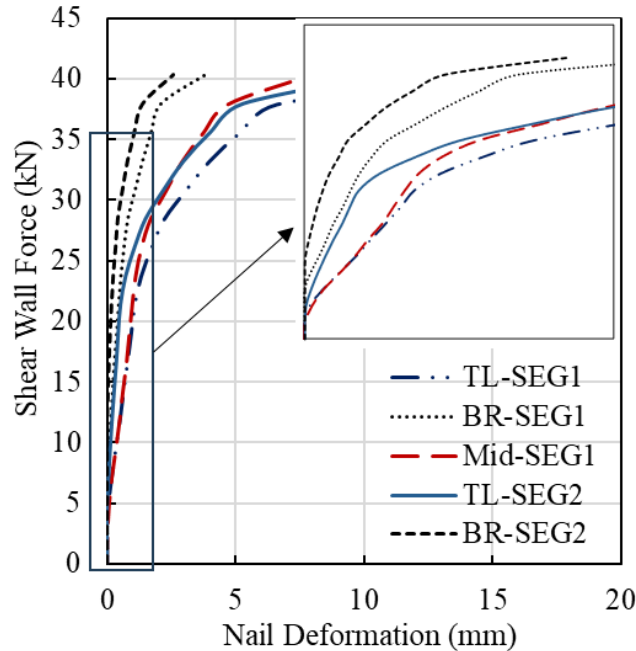
Overall, the presence of the floor assembly increased nail deformation by an average of 50%, with minimum and maximum increases of 14% and 147%, respectively, indicating substantial variability. This increase is more pronounced in shear walls with higher AR (2.4) compared to those with lower AR (1.2) (Figure 6.7(a)). This trend is consistently observed in both the experimental results and numerical simulations. This indicates that, when floor assembly and system-level effects are present, CSA O86 (Canadian Standards Association 2019) significantly underestimate shear wall deflections associated with nail deformations.

It should be noted that the observed increase in nail deformations due to the presence of the floor assembly occurs concurrently with a reduction in HD deformation, as discussed in detail in Section 6.5, representing a shift in demands among shear wall components. The inclusion of the floor assembly restrained HD uplift deformations, leading to a redistribution of force and deformation demands. As a result, a greater portion of the deformation was mobilized through the racking mode of the shear wall, resulting in increased nail deformations.

#### **6.4 Effect sheathing panel AR on nail deformations**

The current CSA O86 (Canadian Standards Association 2019) design provisions do not specify a maximum allowable AR for individual sheathing panels. Instead, the code limits the overall AR of the wood shear wall or shear wall segments to 3.5:1, irrespective of the number or geometry of the sheathing panels within the wall.

The results from Configuration #3 indicate that the nail deformations around the narrow 280 mm wide sheathing panel were considerably lower than those recorded along the perimeter of the larger panels (i.e., 1200 x 2400 mm) within the same wall (see Figure 6.8). This observation suggests that, although the global wall AR satisfies the design standard limit, significant variation in panel AR within a single wall can lead to non-uniform nail deformation demands.



**Figure 6.8: The effect of sheathing panel AR on the nail deformations**

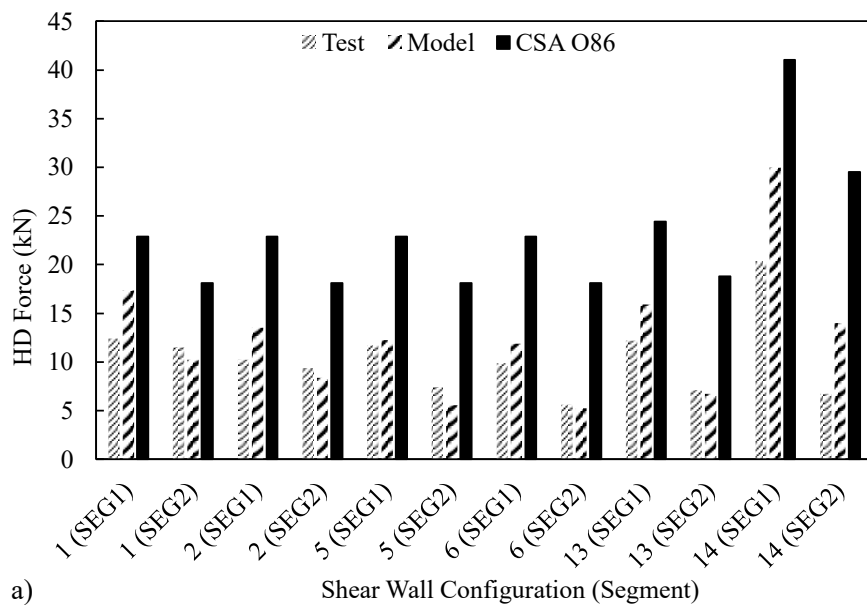
BR-SEG1 and BR-SEG2 in Figure 6.8 represent the nail deformations along the edges of the narrow panel located at the bottom right corners of individual segments of Configuration #3. The pronounced difference in deformation between this narrow panel and the adjacent larger panels highlights the nonuniform nail deformation within the same shear wall.

Currently, CSA O86 (Canadian Standards Association 2019) limits the global shear wall AR to less than 3.5; however, it does not explicitly address the maximum panel AR. In contrast, the Eurocode 5 (EC 5 1995) requires that the AR of the smallest panel be less than 4.0. It should therefore be considered that the AR of individual sheathing panels within a shear wall does not vary significantly. Beyond nail deformation considerations, the presence of a very narrow panel

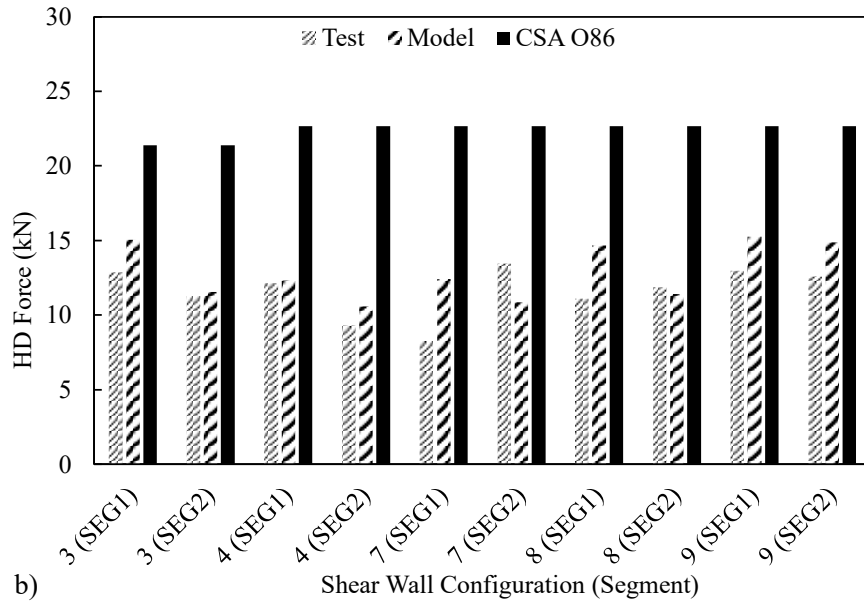
adjacent to a larger panel may also introduce force imbalance at the adjoining stud, potentially leading to premature localized failures, as explained in Section 4.5.3.

### 6.5 Floor out-of-plane and system effects impact on HD force and deformation demands

Figure 6.9 presents the comparison of HD forces obtained from CSA O86 (Canadian Standards Association 2019), test results, and numerical simulations. For consistency, the HD forces were evaluated at the factored resistance ( $V_r$ ) of each shear wall configuration. Figure 6.9(a) includes shear wall assemblies composed of two segments with lengths of 2.0 m (SEG1) and 1.0 m (SEG2), while Figure 6.9(b) corresponds to multi-segment shear walls, where the shear walls in Configuration #3 are 1.5 m in length, and those in Configurations #4, #7, #8, and #9 are all 1.0 m long.



**Figure 6.9: Comparison between HD forces from CSA O86, test, and numerical simulations**



**Figure 6.9: Comparison between HD forces from CSA O86, test, and numerical simulations (continued)**

Consistently across all configurations, the measured HD forces were lower than those design values calculated in accordance with CSA O86 (Canadian Standards Association 2019).

Comparison between Configurations #1, #2, #5, and #6 illustrates that the inclusion of the floor assembly significantly reduced the HD forces. It is also noted that Configuration #1 exhibited lower HD forces relative to CSA O86 (Canadian Standards Association 2019) even in the absence of a floor assembly. This reduction could be attributed to factors such as the presence of multiple sheathing panels in the shear wall and the contribution of ABs, which are not explicitly accounted for in CSA O86 (Canadian Standards Association 2019).

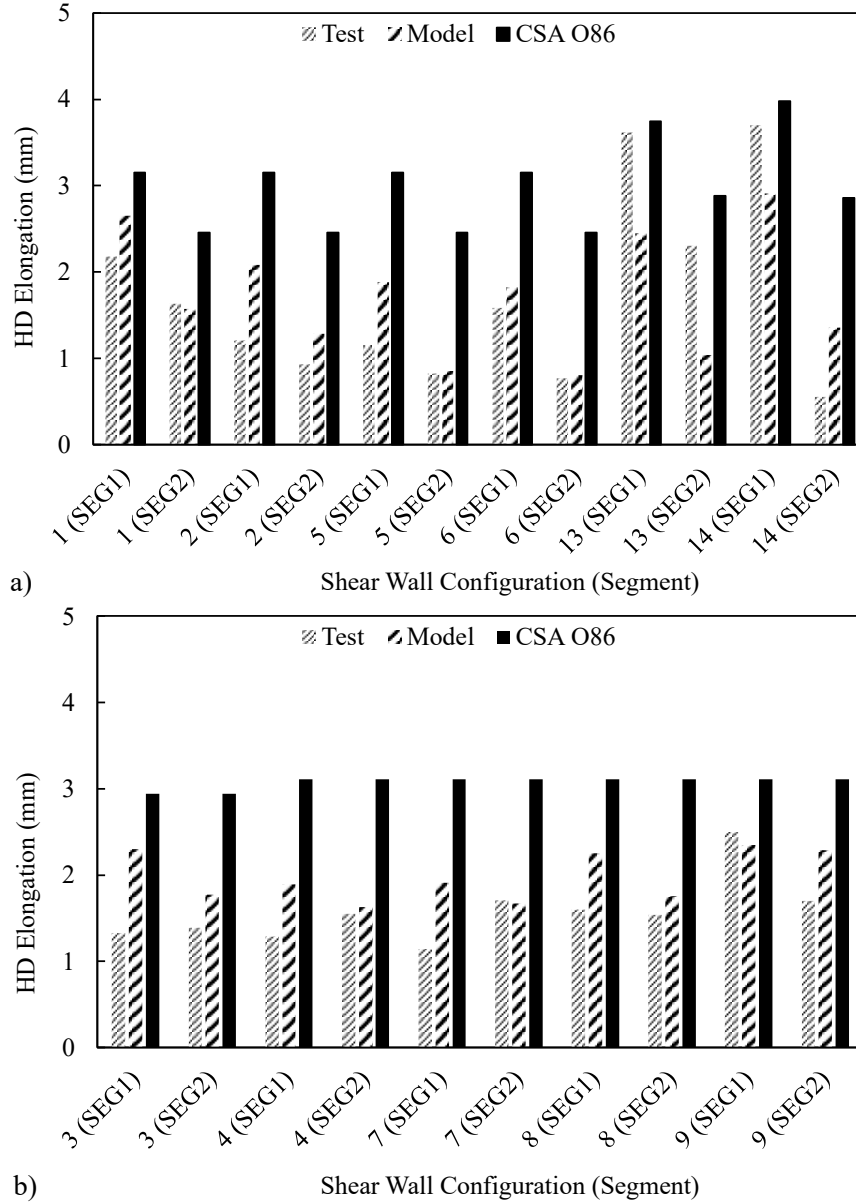
The addition of the floor assembly in Configuration #2 resulted in reductions of 55% and 48% in HD forces for SEG1 and SEG2, respectively, compared to CSA O86 (Canadian Standards Association 2019) values. When HD connections were added at the floor level and two LVL joists were used (Configuration #6), the reductions increased to 57% and 69% for SEG1 and SEG2, respectively.

Similarly, in shear wall configurations with ATS rods, the presence of the floor assembly led to significant reductions in rod forces. Configuration #13 showed reductions of 50% and 62% for SEG1 and SEG2, respectively, while Configuration #14 exhibited reductions of 50% and 77%.

According to Figure 6.9(b), the HD force reduction in multi-segment shear walls with equal ARs is, on average, approximately 50%. A comparison between Configurations #4 and #9 indicates that the use of a continuous LVL joist as the floor assembly resulted in a 25% greater reduction in HD forces in SEG2 compared to a floor assembly with discrete blocking, while the reduction in SEG1 was nearly identical for both configurations. This suggests that the bending stiffness of the continuous LVL, particularly over a larger opening, had a more pronounced effect in reducing the HD forces in SEG2 compared to the shear wall with a blocking floor assembly.

Similarly, a comparison between Configurations #7 and #8, representing configurations with continuous LVL and blocking floor assemblies, shows a comparable trend to that observed for Configurations #4 and #9, but with a lower level of reduction. This difference may be attributed to the relatively short opening (0.5 m), where the bending stiffness of the LVL is less effective than in configurations with larger openings.

Similar to HD force comparisons, Figure 6.10 presents the corresponding comparisons of HD elongations from CSA O86 (Canadian Standards Association 2019), test results, and numerical simulations.



**Figure 6.10: Comparison between HD elongation from CSA O86, test, and numerical simulations**

Consistently with the reductions observed in HD forces (Figure 6.9), the HD elongations obtained from the test results and numerical simulations were generally significantly lower than those calculated in accordance with CSA O86 (Canadian Standards Association 2019) design values.

A comparison of Configurations #2, #5, and #6 with Configuration #1 (no floor assembly) showed a substantial reduction in HD elongations, with SEG2 (AR = 2.44) exhibiting generally more reduction than SEG1 (AR = 1.2). The average reduction was approximately 55%, with the



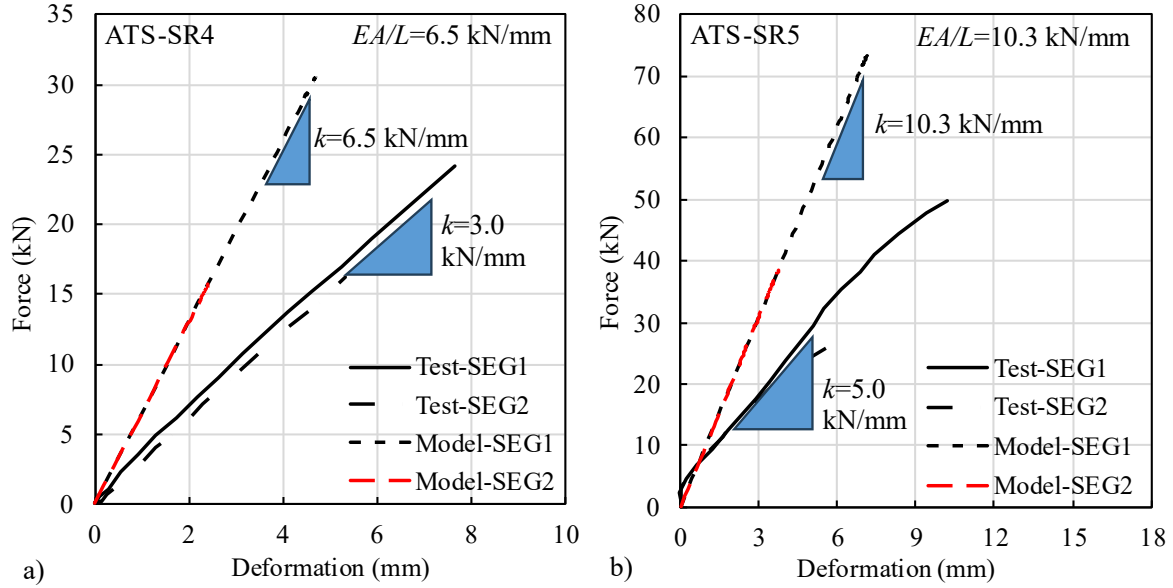
maximum reduction (about 70%) observed in Configuration #6, where HD connections were provided at the floor level.

For configurations consisting of shear walls with equal ARs, the average reduction in HD elongation was about 50%. However, the magnitude of reduction varies with the floor assembly type: Configuration #9 (blocking floor assembly) showed a lower reduction of approximately 20%, whereas Configuration #4 (continuous LVL floor assembly) exhibited a significantly higher reduction of about 60%.

A maximum reduction of 80%, relative to the CSA O86 (Canadian Standards Association 2019) design value, was observed in the HD elongation of SEG2 in Configuration #14 (shear wall with ATS rods and 50 mm nail spacing).

An important observation was that the measured stiffness values of the ATS rod system in configurations #13 and #14 were significantly lower than the corresponding theoretical values,  $EA/L$ , where  $E$  is the modulus of elasticity of the rod material,  $A$  is the net cross-sectional area of the rod, and  $L$  is the rod length, taken as the height of the shear wall plus the thickness of the floor assembly.

Figure 6.11(a) and (b) present the F–D curves of the ATS rods in Configurations #13 and #14, respectively, highlighting this discrepancy between the measured and theoretical stiffness values.



**Figure 6.11: Comparison of theoretical and measured stiffness of the ATS rods: (a) ATS-SR4; (b) ATS-SR5**

As shown in Figure 6.11, the effective stiffness of the ATS system is approximately 50% of the theoretical values, consistent with observations reported by other researchers (Pei and van de Lindt 2009; Pei and Van de Lindt 2011). The numerical model predicted ATS system stiffness close to the theoretical values, highlighting that the model, in its current state, cannot capture the actual behaviour of shear wall systems with ATS rods. This topic is a subject of future investigation by the author.

This finding is significant, as it likely contributes to the reduced stiffness observed in shear walls equipped with ATS rods, as discussed in Section 6.1. It also underscores the need for additional experimental and numerical studies, because overestimating the stiffness of these anchorage systems at the design stage may lead to excessive actual drift demands of multi-storey shear walls, especially mid-rise LFW shear wall buildings.

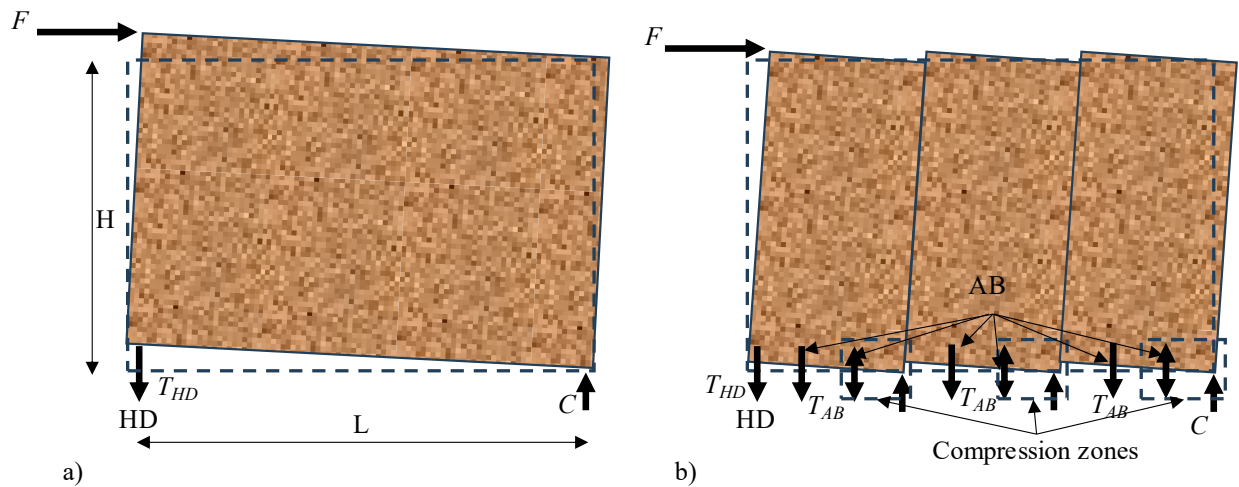
In conclusion, the presence of the floor had a significant impact on HD force and elongation demands. This effect was more pronounced in high-AR shear walls. The configuration of the floor

assembly, whether a continuous LVL joist or discrete blocking, affected the HD demands differently depending on the opening length.

### 6.5.1 Influence of multiple sheathing panels and AB contribution on HD force demands

The current approach adopted in CSA O86 (Canadian Standards Association 2019) for determining the HD force is based on the assumption that the entire shear wall behaves as a single, uniformly sheathed panel (Figure 6.12(a)). The contribution of ABs in resisting the overturning moment at the base of the shear wall is also neglected.

In practice, however, sheathing panels are manufactured in standard sizes, and depending on the wall length, a shear wall is typically covered with multiple panels, as illustrated in Figure 6.12(b).

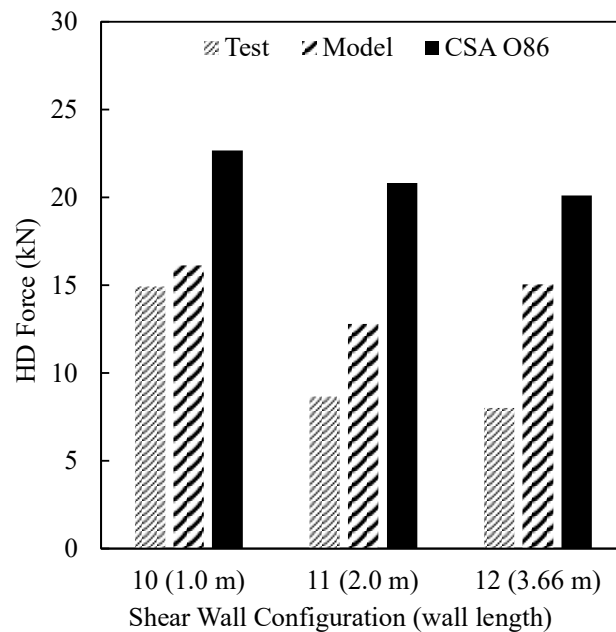


**Figure 6.12: (a) CSA O86 shear wall rocking behaviour assumption; (b) Realistic configuration of a shear wall**

As illustrated in Figure 6.12(a), the entire shear wall is assumed to rotate about a single point, with the compression zone concentrated at that location. In contrast, Figure 6.12(b) suggests a different deformation mechanism. As the shear wall undergoes lateral displacement, the sheathing panels would rotate individually, depending on flexibility of the vertical nail connections at the panel interfaces.

Since each sheathing panel can rotate independently, the uplift from one side of the panel applies an uplift force to the shear wall bottom plate while compressing the opposite side. Additionally, because the ABs are distributed along the shear wall, they resist a portion of the uplift from each panel depending on their locations. As such, assuming that the overturning moment at the base of the shear wall is resisted solely by the HDs may not be accurate; rather, it is resisted by the combined action of the HDs and the ABs.

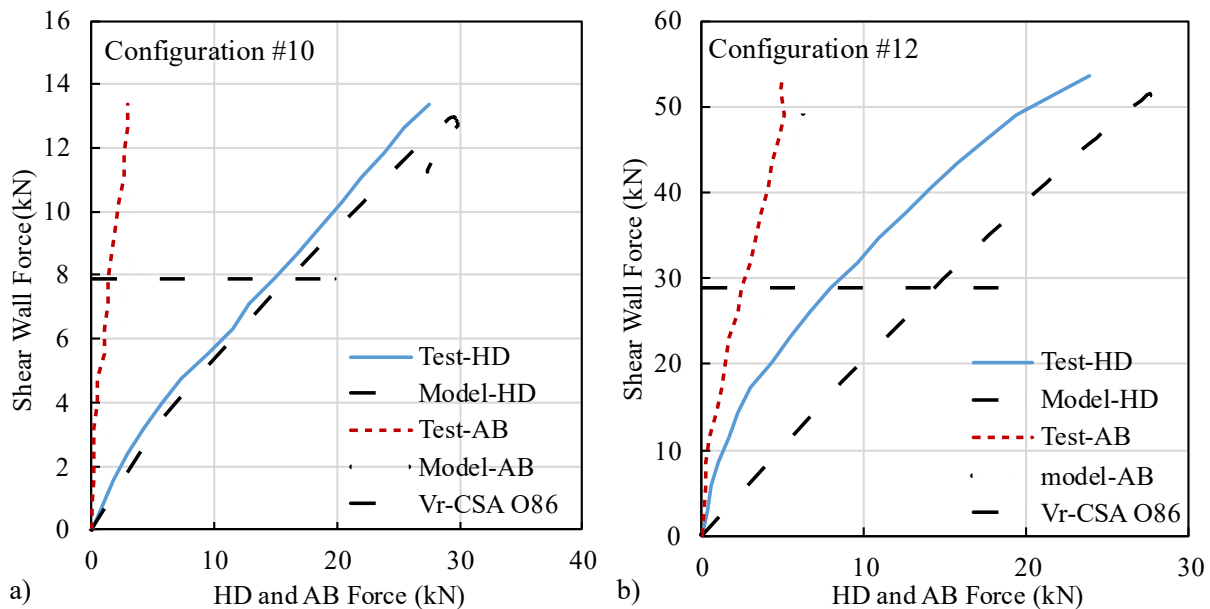
Figure 6.13 illustrates the resulting HD forces for shear wall Configurations #10, #11, and #12, which consist of one, two, and three sheathing panels, respectively. For consistency with the HD force demands from CSA O86 (Canadian Standards Association 2019), both the experimental and numerical results are reported at the factored resistance level of each configuration.



**Figure 6.13: Effect of multiple panels of HD force demand**

Figure 6.13 shows that the experimental and numerical HD forces are lower than the corresponding values calculated according to the CSA O86 (Canadian Standards Association, 2019), with the numerical model consistently overestimating the HD force.

Although the HD design forces are nearly identical, the measured HD forces in Configurations #11 and #12 are lower than those in Configuration #10. This difference can be explained by the load transfer mechanism. Since the HD is connected only to the shear wall chord, its force is primarily influenced by the sheathing panel attached to the chord member. In shear walls with multiple panels, since the boundary conditions around each panel differ, it can lead to non-uniform load sharing among panels. As a result, in walls with lengths of 2.0 m (two panels) and 3.66 m (three panels), the load may be distributed among multiple panels differently, leading to lower measured HD forces compared to the design values as well as Configuration #10 where only one panel exist. The discrepancy between the measured and numerical predictions can in part be attributed to the local behaviour of the HD and AB connections, particularly at the early stages of loading. Figure 6.14(a)–(b) plot the variation of HD and AB forces against shear wall force for Configurations #10 and #12, respectively.



**Figure 6.14: Measured and model prediction HD and AB forces**

As shown in Figure 6.14(a), the measured HD force and the model prediction are in reasonable agreement. Although a similar overall trend is observed for the AB force, local conditions at the early stages of loading introduce discrepancies between the model predictions and the test results. In Figure 6.14(b), both the HD and AB forces in Configuration #12 exhibit significantly different behaviour at the early stages of loading due to local effects, which led to noticeable discrepancies between the model predictions and the experimental results. Since the numerical model assumed linear behaviour, consistent with the general response of the HD and AB, it was not capable of capturing these localized initial effects.

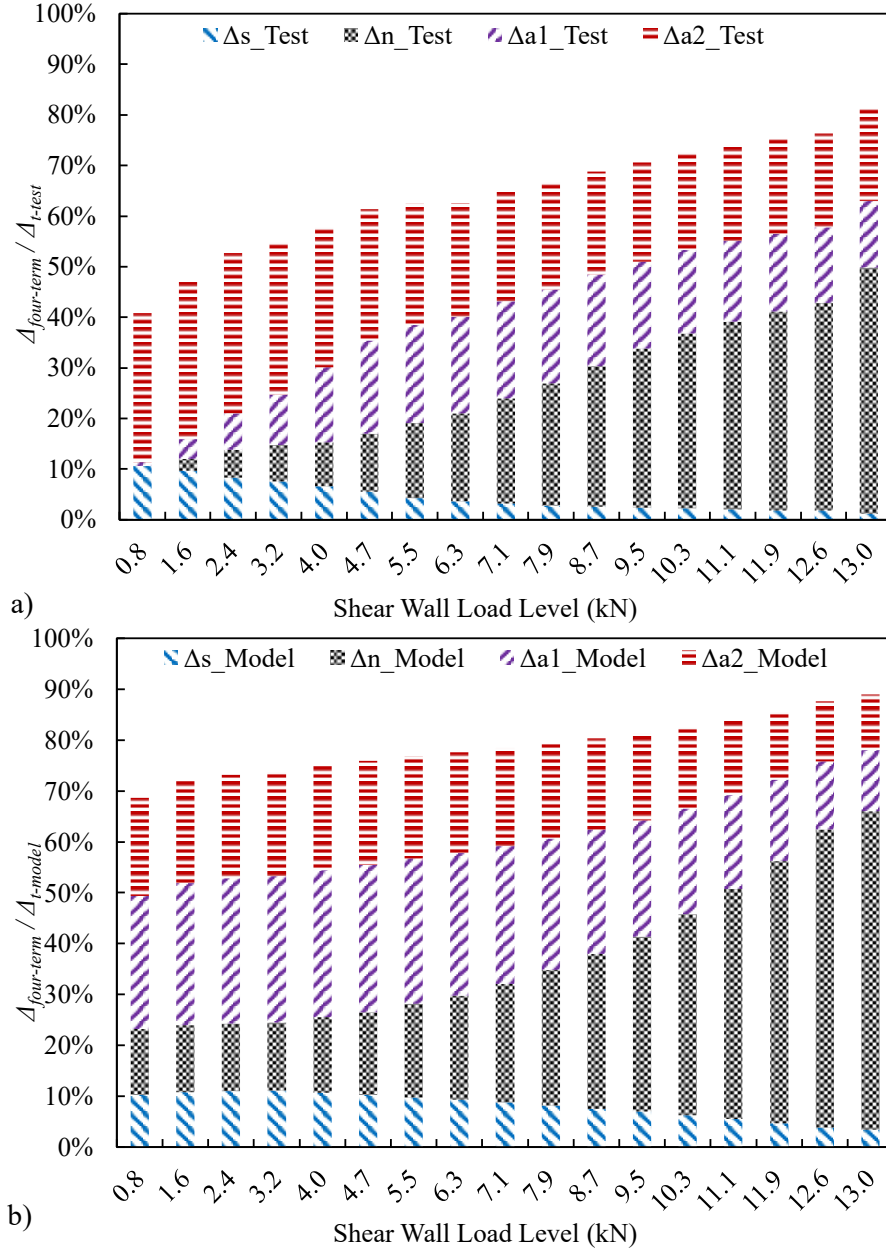
Figure 6.14(a) and (b) indicate that the AB forces in Configurations #10 and #12 represent approximately 10% and 30% of the resisted HD forces, respectively. The findings of this section suggests that the CSA O86 (Canadian Standards Association 2019) approach for estimating HD forces provides an upper bound prediction at the design stage, which does not fully reflect the actual distribution of forces within the shear wall.

The overestimation of HD forces by the CSA O86 (Canadian Standards Association 2019) has a direct impact on the calculation of storey drifts. In the worst-case scenario, the unrealistically high uplift forces and associated deformation demands accumulate over the height of the building, amplifying the total drift due to the cumulative effects considered in CSA O86 (Canadian Standards Association 2019).

## **6.6 Validity of CSA O86 four-term deflection equation**

The validity of the four-term deflection equation in CSA O86 (Canadian Standards Association 2019) was evaluated by examining the summation of the individual shear wall deflection components from test results and numerical predictions. It is worth noting that the CSA O86 (Canadian Standards Association 2019) values represent the design values provided in Table 3-7.

Contributions of the different deflection components (i.e., bending, shear, nail, bottom plate crushing, and HD deformations) were extracted at different load levels and summed to obtain the total shear wall deflection at that load level. The relative contribution of each deflection component from four-term equation with inputs from the test results, was expressed as a percentage of the total shear wall deflection from the test at the corresponding load level, allowing a direct assessment of their significance as a function of increasing load. It is noted that the CSA O86 (Canadian Standards Association 2019) values and their significance were evaluated by comparison with the total deflection at the design stage (Table 3-7.). The load levels considered in this evaluation ranged from zero load up to the peak resistance of the shear wall. Figure 6.15(a) and (b) illustrate how the individual deflection components, obtained from both test results and numerical simulations, are combined to illustrate the proportion of each calculated component relative to the total deflection of the shear wall in Configuration #10, considered as a representative sample shear wall. The reason for this selection is that this single-segment shear wall is not affected by system-level interactions or boundary conditions, ensuring a consistent comparison with CSA O86 (Canadian Standards Association 2019) values. The individual deflection components calculated based on test data and numerical models were summed using the four-term deflection equation ( $\Delta_{four-term}$ ) from CSA O86 (Canadian Standards Association 2019) and were then compared with the total deflection from the test ( $\Delta_{t-test}$ ) and the model ( $\Delta_{t-model}$ ). The vertical axis in Figure 6.15 represents the ratio of each deflection term in the four-term equation to the corresponding total deflection ( $\Delta_{t-model}$  or  $\Delta_{t-test}$ ).



**Figure 6.15: Shear wall deflection components: (a) test results; (b) numerical simulation**

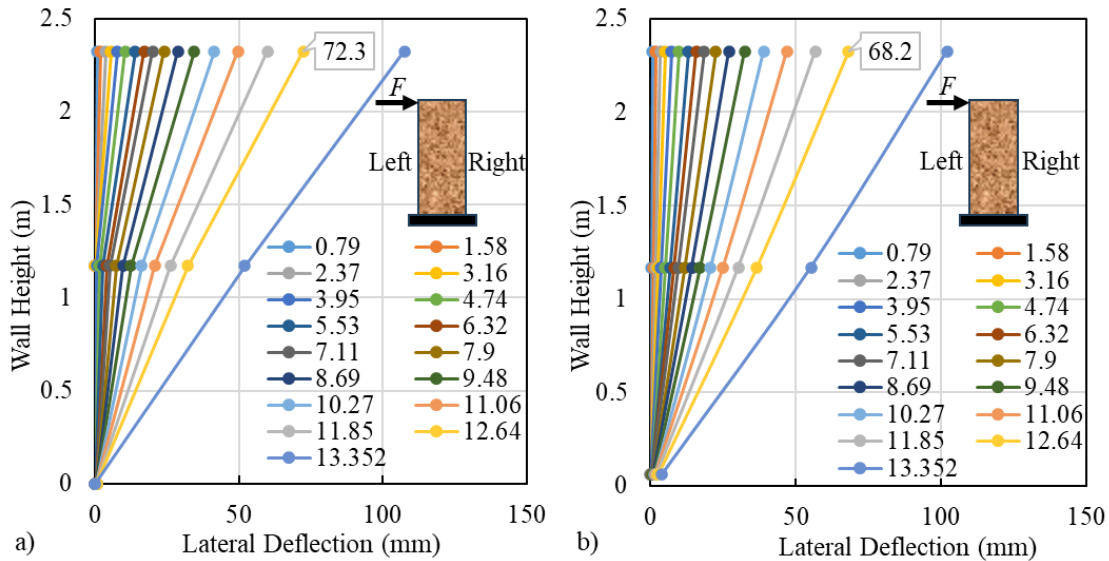
Figure 6.15(a) clearly shows that the summation of the measured individual deflection components testing does not fully match the measured total deflection of the shear wall. Similarly, for the numerical model, Figure 6.15(b) indicates that although the sum of the deflection components closely approximates the total shear wall deflection, it still remains lower than total deflection from model, particularly at low load levels.



One source of discrepancy between  $\Delta_{four-term}$  and  $\Delta_t$  can be attributed to limitations in capturing corner nail deformations during testing. In addition, the contribution of chord member bending could not be quantified precisely in either the test measurements or the numerical simulations. Another significant factor, particularly evident in the measured deflections, is the recording of nail and HD deformations at the early stages of loading. Considering these observations, each deflection component is discussed in detail in the subsequent sections.

### 6.6.1 Deflection due to bending of shear wall chord members

To assess the contribution of the shear wall chord members, Figure 6.16 presents the lateral displacement profiles of the left and right chord members of Configuration #10 at different load levels. Configuration #10 was selected because it is the shortest wall, where bending deflection is expected to be more pronounced, and is not influenced by boundary condition effects associated with system-level behavior.

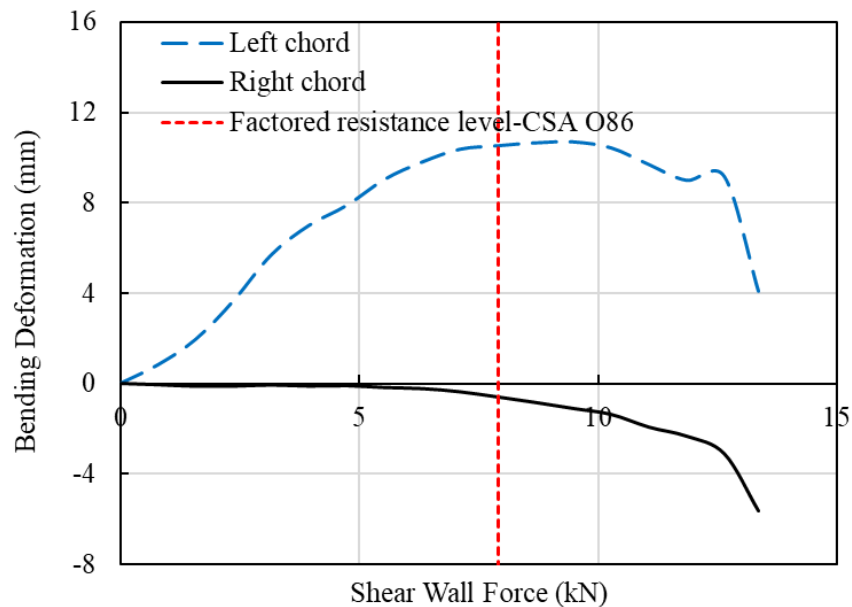


**Figure 6.16: Lateral displacement of shear wall chord members: (a) left chord; (b) right chord**

As shown in Figure 6.16, the two chords did not deform identically. The left chord exhibited a bending mode in the same direction as the global shear wall displacement, whereas the right chord bent in the opposite direction. For example, at a load level of 12.64 kN, the top displacement of

the left chord was 72.3 mm, while that of the right chord was 68.2 mm. Since these values represent total lateral displacements, the difference between the two sides indicates that the observed behaviour is localized rather than a global bending mode of the shear wall. Test observations further confirmed that the left chord was influenced by the proximity of the actuator load, which likely contributed to this asymmetric response.

To quantify the bending deformation, the difference between the top and mid-height displacements of each chord was calculated. Figure 6.17 presents the resulting bending deformation of the left and right chords in Configuration #10 as a function of shear wall force up to peak resistance.

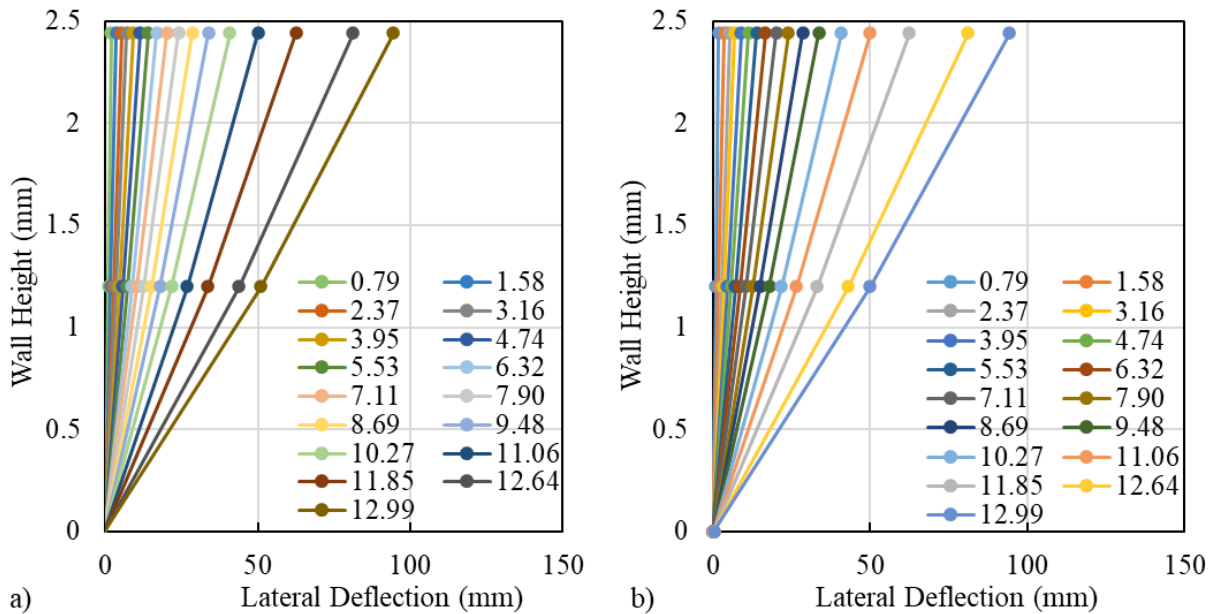


**Figure 6.17: Bending deformation of the chord members of shear wall Configuration #10**

Figure 6.17 shows that the rate of bending deformation changes as the shear wall force increases. The left chord exhibited positive bending, reaching nearly 11 mm around the factored shear wall resistance ( $V_r = 7.9$  kN), while the right chord showed only a small negative bending of -0.6 mm at the same load. Comparing the 11 mm bending deflection of the left chord with the total shear wall displacement of 22 mm at the shear wall design load level suggests that bending accounts for roughly 50% of the total displacement. However, this proportion is not physically consistent with

the overall deformation mechanism, highlighting that this component is highly localized rather than a true global bending contribution.

The numerical simulation results for Configuration #10 are shown in Figure 6.18(a) and (b) for left and right chords, respectively.



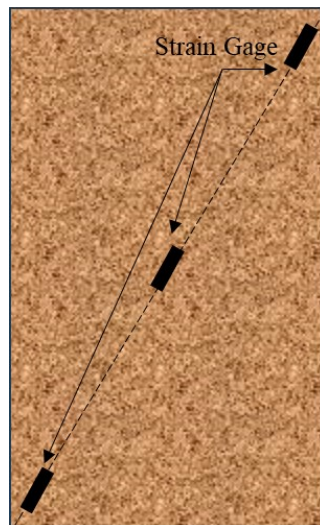
**Figure 6.18: Lateral displacement of shear wall chord members in Configuration #10 from numerical simulation: (a) left chord; (b) right chord**

Figure 6.18 shows that, in the numerical model, both chord members bent in the direction opposite to the shear wall deflection. This behavior contradicts the bending assumption in CSA O86 (Canadian Standards Association 2019), where chord members are considered fixed at the base and deform as cantilever beams toward the direction of shear wall deflection. The discrepancy between the left chord in the model (Figure 6.18(a)) and the test (Figure 6.16(a)) may be attributed to the actuator's load application point. Additionally, the actual physical dimensions of the framing elements and their interactions with other components in the shear wall likely contribute to the differences between the model and test results. The numerical model represents elements as line members connected at nodes along their centerline axes, which does not capture the true physical contacts and interactions present in the test specimen.

Overall, the experimental and numerical results indicate that chord bending is a localized and relatively small component of the total shear wall deflection. While the flexibility of the chord members could contribute to the overall shear wall displacement, it cannot be quantified explicitly. Moreover, this bending appears to primarily induce rotation of the sheathing panels, which is eventually transferred into the racking mode of shear wall deformation rather than representing a global bending of the wall.

### 6.6.2 Deflection due to shear deformation of the sheathing panel

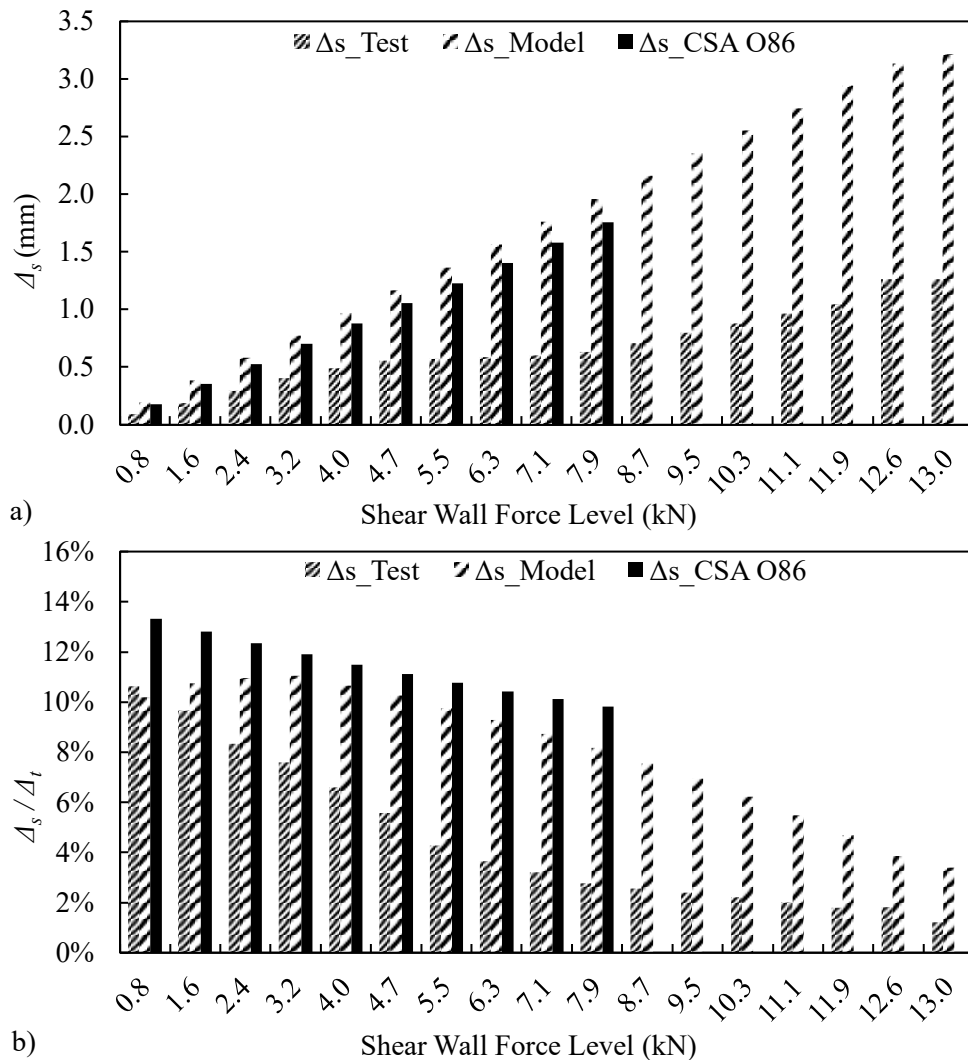
None of the DTs recorded shear deformation of the sheathing panels during the shear wall tests. To clarify whether this observation was due to the absence of measurable shear deformation or the limited sensitivity of the DTs, strain gauges were installed in selected configurations to capture any possible extension along the diagonal of the sheathing panel, as shown in Figure 6.19.



**Figure 6.19: Installation of strain gauges along the diagonal of the sheathing panel in Configuration #10**

Strain gauge data were integrated along the diagonal of the sheathing panel to calculate its diagonal elongation. The resulting shear deflection component of the shear wall in Configuration #10 was then compared with the corresponding values from CSA O86 (Canadian Standards Association 2019) and the numerical simulation at different load levels, from the early stages of loading up to

the peak resistance of the shear wall, as shown in Figure 6.20(a). In addition, Figure 6.20(b) compares the deflection due to panel shear deformation from CSA O86 (Canadian Standards Association 2019), test results, and numerical simulations with their respective total shear wall deflections, highlighting its relative significance as the applied load increases.



**Figure 6.20: (a) Shear wall deflection due panel shear deformations; (b) contribution of the sheathing panel shear deformation in total shear wall deflection**

Figure 6.20(a) illustrates that the CSA O86 (Canadian Standards Association 2019) values and the numerical simulation results are reasonably close, up to the load level corresponding to the factored resistance of the shear wall ( $V_r = 7.9$  kN); beyond this level, CSA O86 (Canadian Standards Association 2019) does not provide predictions. This agreement was expected, as identical material

properties for the sheathing panel were adopted in both the CSA O86 (Canadian Standards Association 2019) formulation and the numerical model. The measured deflection, however, shows significantly lower values.

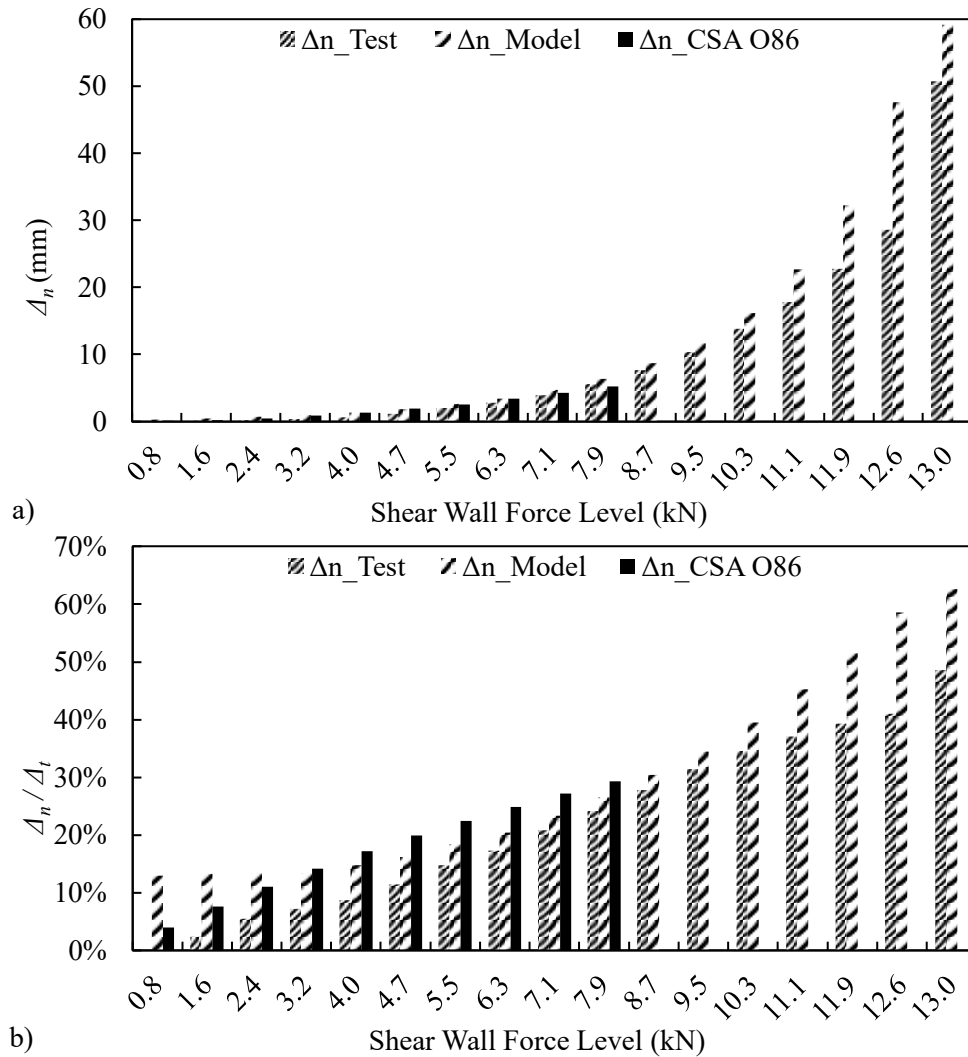
Figure 6.20(b) shows that both CSA O86 (Canadian Standards Association 2019) and the numerical results indicates that the relative contribution of shear deformation to the total shear wall deflection decreased as the load increased, so that it reached its lowest proportion near peak resistance. In contrast, the test results indicated a much smaller contribution of shear deformation, especially at higher load levels. Unlike the gradual and nearly uniform reduction predicted by CSA O86 (Canadian Standards Association 2019) and the numerical model, the experimental results exhibited a sharp reduction up to the factored resistance load level, followed by a milder rate of decrease toward shear wall peak resistance.

Two main factors may explain this behaviour. First, the actual relative density of the sheathing panel was 0.62 as measured in the lab, which is higher than the value of 0.49 assumed in CSA O86 (Canadian Standards Association 2019), leading to a stiffer panel response and consequently lower shear deformation. Second, early localized wood crushing around the sheathing-to-framing connections likely redistributed stresses and reduced the effective shear demand in the panel. This localized mechanism was not possible to capture accurately in numerical modelling and is not explicitly considered in the CSA O86 (Canadian Standards Association 2019) formulation.

From a design perspective, overestimation of the shear component of wall deflection at the factored resistance level, and its subsequent amplification by the  $R_d R_o$  factors to estimate inelastic deformation, may lead to an inaccurate prediction of shear-related deflection of the shear wall at higher load levels, as evidenced in Figure 6.20.

### **6.6.3 Shear wall deflection due to nail deformation**

Figure 6.21(a) compares the shear wall deflection due to nail deformations from CSA O86 (Canadian Standards Association 2019) design values, test results, and numerical simulations at different load levels up to the peak resistance for shear wall configuration #10. Furthermore, to highlight the relative significance of nail-induced shear wall deflection, Figure 6.21(b) compares the deflection due to nail deformation from CSA O86 (Canadian Standards Association 2019) design values, test results, and numerical simulations with their respective total shear wall deflections as the load increases.

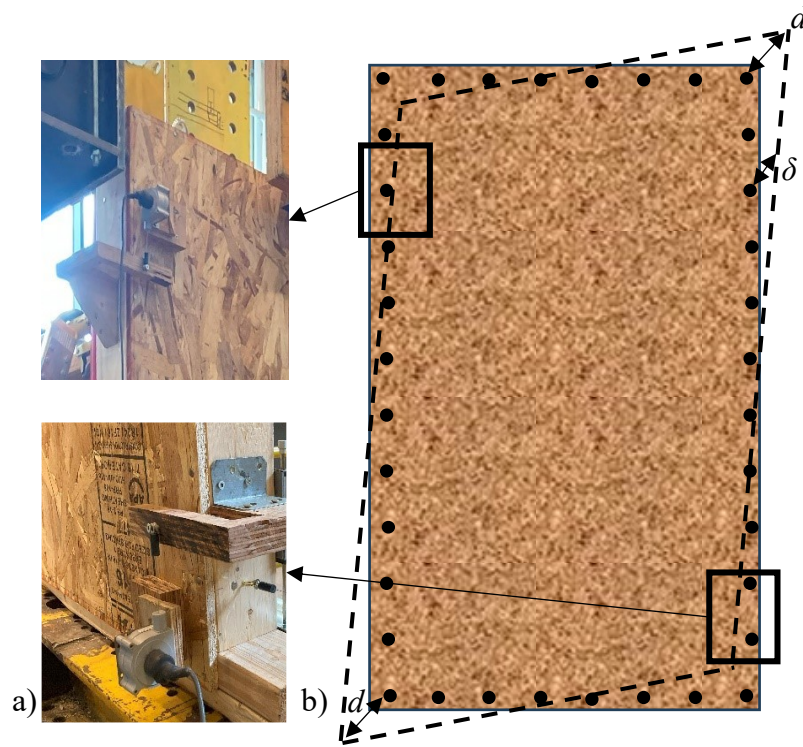


**Figure 6.21: (a) Shear wall deflection due to nail deformations; (b) contribution of the nail deformations in total shear wall deflection**

Figure 6.21(a) illustrates the increase in shear wall deflection with applied load, with the rate of increase becoming exponential beyond the factored shear resistance ( $V_r = 7.9$  kN). Figure 6.21(b) shows that the contribution of nail deformations to the total shear wall deflection increases progressively with increasing load level. At low load levels, the test results indicated significantly lower values compared to CSA O86 (Canadian Standards Association 2019) and with the discrepancy being more pronounced in the early stages of loading. In contrast, the numerical results predicted higher contributions than both the test results and CSA O86 (Canadian Standards Association 2019) values.

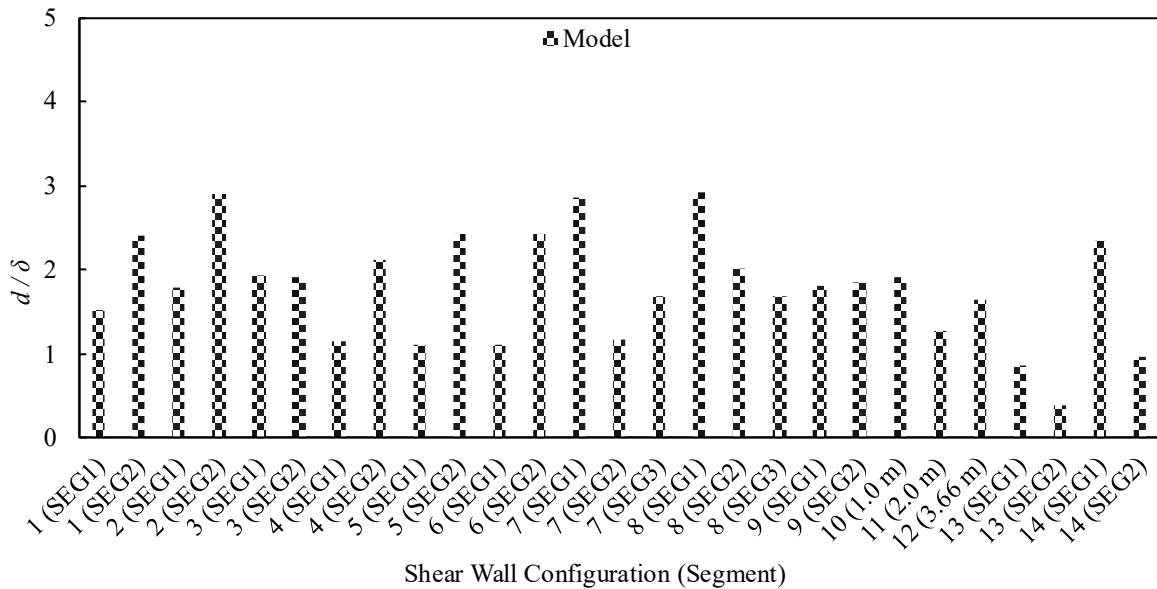
The lower values obtained from the tests may be attributed to measurement limitations, including the location and sensitivity of the instrumentation at very low deformation levels. From a kinematic standpoint, the shear wall deflection component due to nail slip should include the deformation at the wall corners, denoted as  $d$  in Figure 6.22(b). However, due to practical limitations, it was not feasible to install DTs exactly at the shear wall corners during testing. As shown in Figure 6.22(a), the DTs were installed at locations, where the recorded deformation corresponded to  $\delta$  in Figure 6.22(b). Because of the wall geometry and deformation mechanism,  $\delta$  does not fully represent the corner deformation,  $d$ , which governs the global nail-slip contribution to wall deflection.





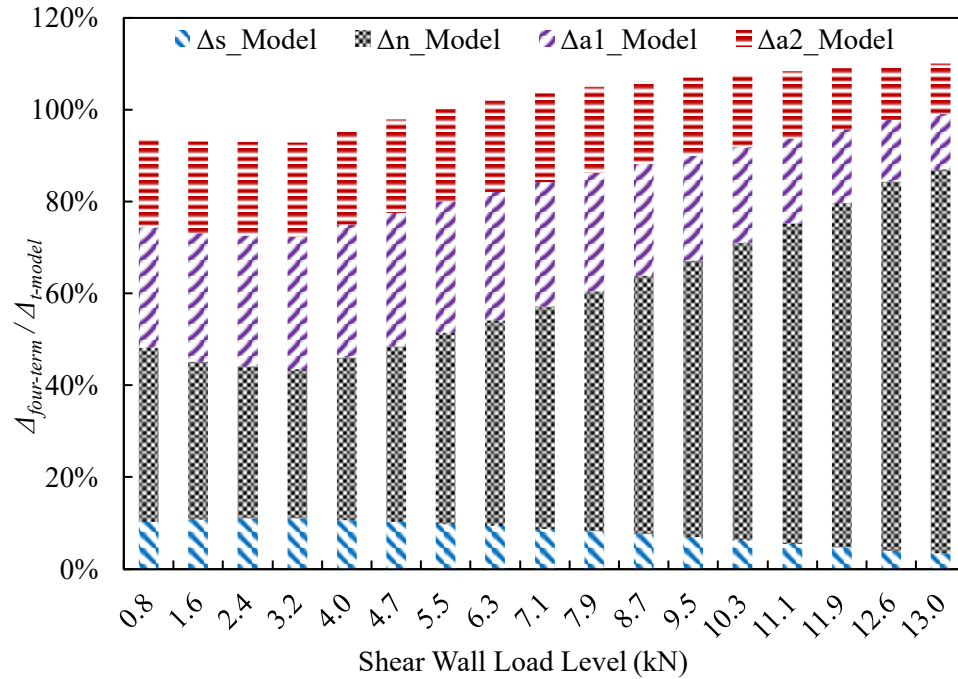
**Figure 6.22: (a) Nail deformation mechanism in wood shear walls; (b) locations of nail deformation measurements recorded in Configuration #10**

Given the prior validation of the numerical model for nail deformations at the DT locations, the ratio of corner nail deformations to those at the DT locations ( $d/\delta$ ) was obtained from the numerical model. This ratio is illustrated in Figure 6.23 for all shear walls, with an average value of  $d/\delta = 1.8$ .



**Figure 6.23: Ratio of corner nail deformation to nail deformation at DT locations**

According to Figure 6.23 and the average value of  $d/\delta = 1.8$ , the shear wall deflection in Figure 6.15(b) was recalculated by incorporating the corner nail deformations from the numerical model into the four-term deflection equation. Figure 6.24 presents the ratio of the shear wall deflection obtained from the four-term equation to the total shear wall deflection extracted from the numerical model.

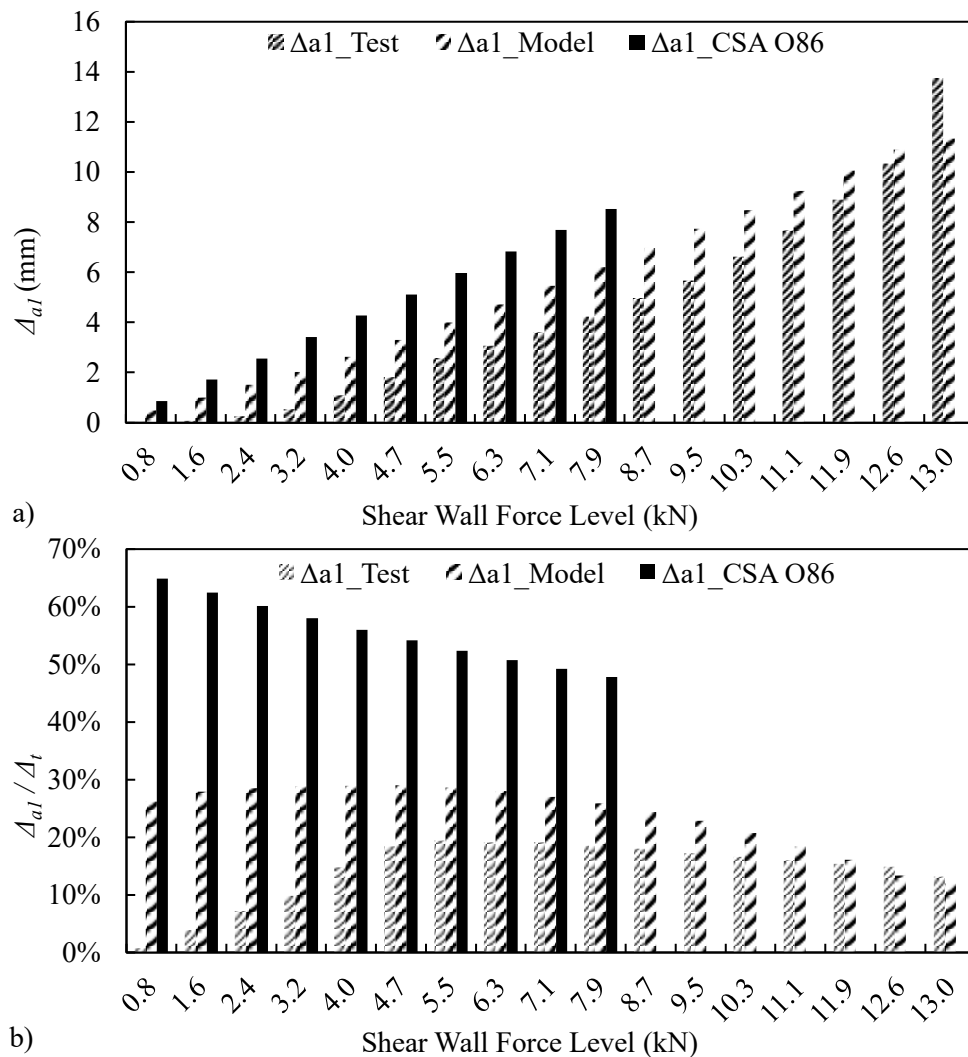


**Figure 6.24: Contribution of different deflection components including nail deformations at corner locations**

Figure 6.24 demonstrates that the shear wall deflection using four-term equation, incorporating corner nail deformations, reasonably matches the total shear wall deflection across a wide loading range. Although the built-up deflection slightly underestimates the total deflection at low load levels, the overall prediction is reasonable for a wide range of loading. The discrepancy at lower load levels may be attributed to stud bending, a behaviour supported by Figure 6.17, which shows a higher rate of flexural deformation at lower loads. Additionally, Figure 6.24 indicates that at peak resistance, nail deformations account for approximately 80% of the total shear wall deflection.

#### 6.6.4 Shear wall deflection due to HD elongation

Figure 6.25(a) presents the shear wall deflection of Configuration #10 due to HD elongation ( $\Delta_{al}$ ). The  $\Delta_{al}$  values from CSA O86 (Canadian Standards Association 2019), test measurements, and numerical predictions are compared across load levels up to the peak resistance. This comparison illustrates the development of HD forces at different stages of loading and their variation with increasing shear wall force. In addition, to demonstrate the relative contribution of uplift-induced deflection, Figure 6.25(b) compares HD elongation from CSA O86 (Canadian Standards Association 2019), test results, and numerical simulations with their respective total shear wall deflections as the load increases.



**Figure 6.25: Contribution of HD elongation in total shear wall deflection**

Figure 6.25(a) shows that CSA O86 (Canadian Standards Association 2019) design values significantly overestimate  $\Delta_{al}$  across all load levels up to the factored resistance ( $V_r = 7.9$  kN). The figure also indicates that the rate of shear wall deflection due to HD elongation increases almost linearly and remains consistent across all three cases, despite differences in magnitude. However, compared to the total deflection of the shear wall, the contribution of the  $\Delta_{al}$  in total shear wall deflection decreases with increasing load (Figure 6.25(b)). Figure 6.25(b) signifies that the highest contribution of the HD elongation in shear wall deflection is in the initial loading stages with lowest contribution when the shear wall reached its peak resistance.

In contrast, referring to Figure 6.25(b), the measured HD contribution to shear wall deflection is minimal at the early stages of loading and increases with shear wall force, reaching a maximum at approximately 5.5 kN, corresponding to a deflection of 13.25 mm (0.5% IDR). Beyond this point, the contribution remains nearly constant, with a slight decrease up to peak resistance.

Furthermore, such a behaviour at low load levels was likely due to the partial engagement of the HD and the non-uniform force distribution among shear wall components at small displacements, where not all elements were fully activated. This explanation is consistent with the inherent behaviour of such structural systems, which consist of multiple interconnected components with spatial variability.

The numerical results captured the test behaviour reasonably well at moderate to high load levels. However, noticeable discrepancies still existed at the early stages of loading. In the numerical model, the HD contribution remained nearly constant up to the factored resistance of the shear wall and then decreased at a steady rate, generally following the overall trend observed in the tests.

The discrepancies between the test and numerical results can be attributed to the local behaviour of the HD, particularly at low load levels where partial engagement and non-uniform force

distribution are more pronounced. These sources of inconsistency are discussed in detail in Section 6.5.1 (Figure 6.14). Figure 6.26 illustrates the shear wall deflection due to HD elongation at different load levels for Configurations #4, #6, and #7 as representative examples. These examples demonstrate that the actual rate of change in HD elongation is not necessarily linear with increasing shear wall load, contrary to the trends predicted by CSA O86 (Canadian Standards Association 2019) and numerical models, particularly at early stages of loading where an elastic behaviour of the shear wall is expected.

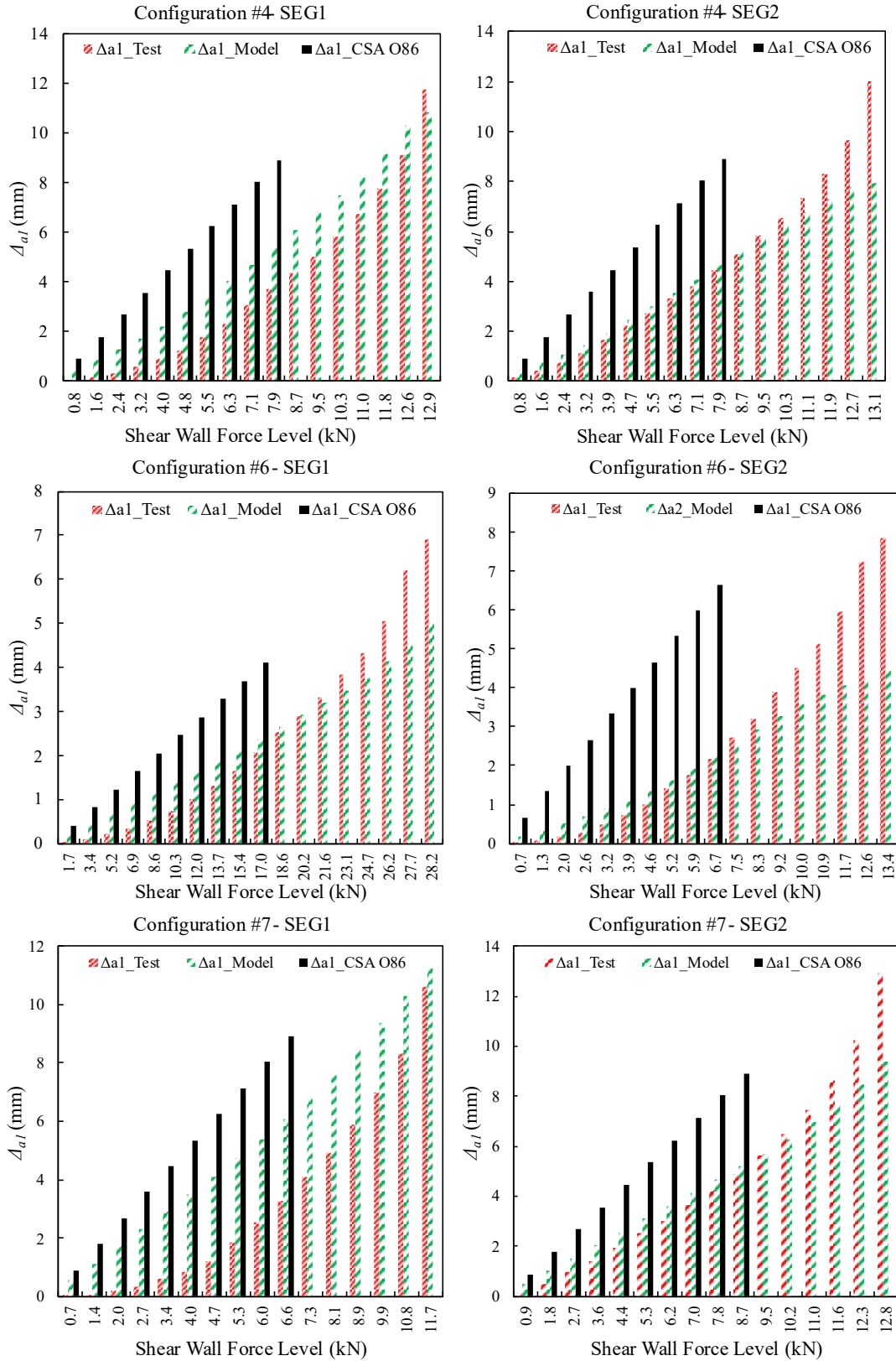
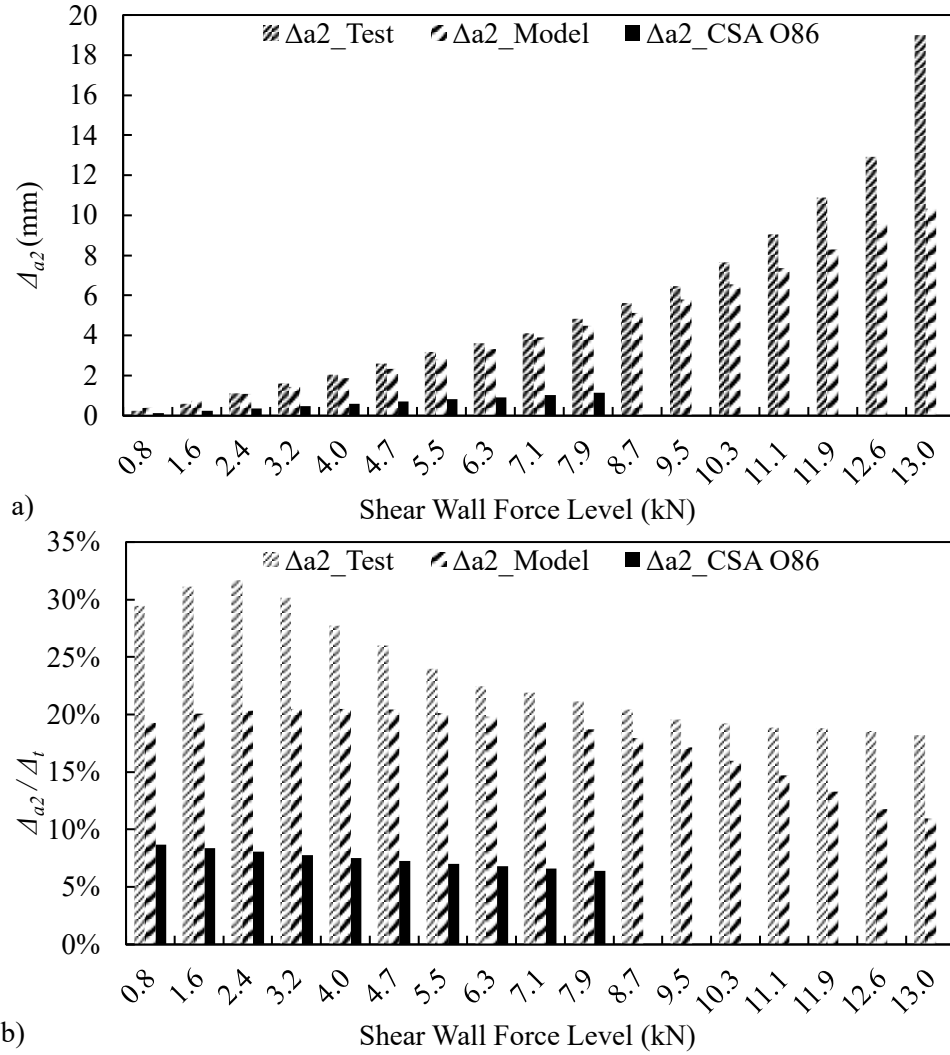


Figure 6.26: Shear wall deflection due to HD elongations at different shear wall load levels

These observed HD behaviours highlight the influence of local effects on force distribution within structural components composed of multiple discrete, interconnected elements, which in turn contributes to the observed discrepancies of the shear wall response at the beginning of the loading. These observations are particularly important in the context of multi-storey buildings, where cumulative displacement effects are often assumed to be most significant within the elastic range of shear wall response. Although HD elongation is expected to accumulate over the height of the structure, the reduced contribution at low load levels suggests that its overall impact may be less pronounced than commonly assumed. In this regard, local behavioural effects play a key role in limiting the contribution of HD elongation to global structural response.

#### **6.6.5 Shear wall deflection due to perpendicular-to-grain compression deformations**

Figure 6.27(a) presents the contribution of perpendicular-to-grain compression deformation of the bottom plate beneath the compression chord  $\Delta_{a2}$  relative to the total deflection of the shear wall. It is noted that the perpendicular-to-grain compression deformation ( $d_{a2}$ ) was multiplied  $H_s/L_s$ . Additionally, to illustrate the relative contribution of  $\Delta_{a2}$  to the total shear wall deflection, Figure 6.27(b) compares  $\Delta_{a2}$  from CSA O86 (Canadian Standards Association 2019), test results, and numerical simulations with their corresponding total shear wall deflections as the load increases.



**Figure 6.27: Contribution of perpendicular-to-grain compression of the bottom plate to the total lateral deflection of the shear wall Configuration #10**

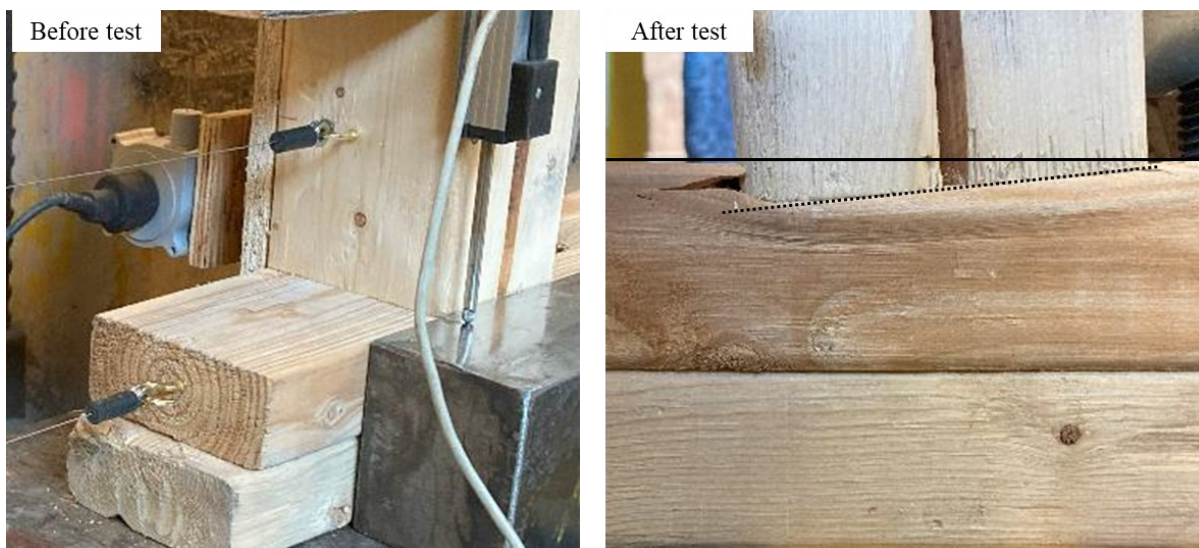
As shown in Figure 6.27(a), CSA O86 (Canadian Standards Association 2019) significantly underestimates this deformation component of the shear wall compared to measured and model prediction data. According to Figure 6.27(b), the test results indicate that more than 20% of the total lateral deflection of Configuration #10 is consistently associated with crushing of the bottom plate under the compression chord across all load levels, whereas CSA O86 (Canadian Standards Association 2019) predicts a contribution of less than 10%. The higher values observed at the early stages of loading may be attributed to initial gaps between the chord and the bottom plate,



representing construction imperfections, during which little to no resistance is mobilized (refer to Figure 6.28).

The numerical simulation shows reasonable agreement with the experimental results; however, the relative contribution of this component to the total lateral deflection is slightly lower than that observed in the tests.

The primary source of discrepancy in CSA O86 (Canadian Standards Association 2019) lies in its assumption of uniform compression beneath the compression chord. Experimental observations revealed a non-uniform stress distribution, due to rotation of the compression chord about its outer edge, where stress concentrated at that location lead to localized crushing and amplified perpendicular-to-grain deformation, as depicted in Figure 6.28.



**Figure 6.28: Compression perpendicular to grain deformation of the bottom plate**

The damage pattern clearly confirmed that compression was not uniformly distributed, contrary to the CSA O86 (Canadian Standards Association 2019) assumption. Instead, stress concentration at the outer edge caused localized punching of the bottom plate. It is noted that the magnitude of this deformation mode was strongly influenced by the material properties of wood, particularly the modulus of elasticity in perpendicular to the grain direction.

Another important aspect to note is that in LFW shear walls, the compression perpendicular to grain that develops at the loaded corner is often idealized as contributing to a global rigid-body rotation of the wall, leading to simplified expressions that relate this deformation directly to  $H_s/L_s$  \*  $d_a$ . However, experimental evidence, including observations made in this study, shows that this assumption does not hold in practice. The observed compression deformation is highly localized, occurring over a small region near the corner stud, and does not propagate as a uniform rotation across the full wall length. The flexibility of the sheathing-to-framing system, the distributed slip along the panel edges, and the restraining effects of angle brackets and other connectors prevent the wall from behaving as a rigid body. As a result, it is highly questionable that the localized crushing can be directly linked to a global rotation mechanism, and the traditional geometric relationship between horizontal displacement and perpendicular-to-grain compression may not be valid for real wall behaviour. Furthermore, this deflection mode may not be classified as a rigid-body mode, since a true rigid-body mode requires zero relative deformations within the shear wall. In reality, however, crushing of the chord member generates additional nail deformations at the bottom edge of the sheathing panel, which in turn directly affect the lateral deflection of the shear wall.

#### **6.6.6 Concluding remark on the four-term deflection equation**

Although still limited in scope, the current study provided some key observations and recommendations that should be considered in the development of the design standard:

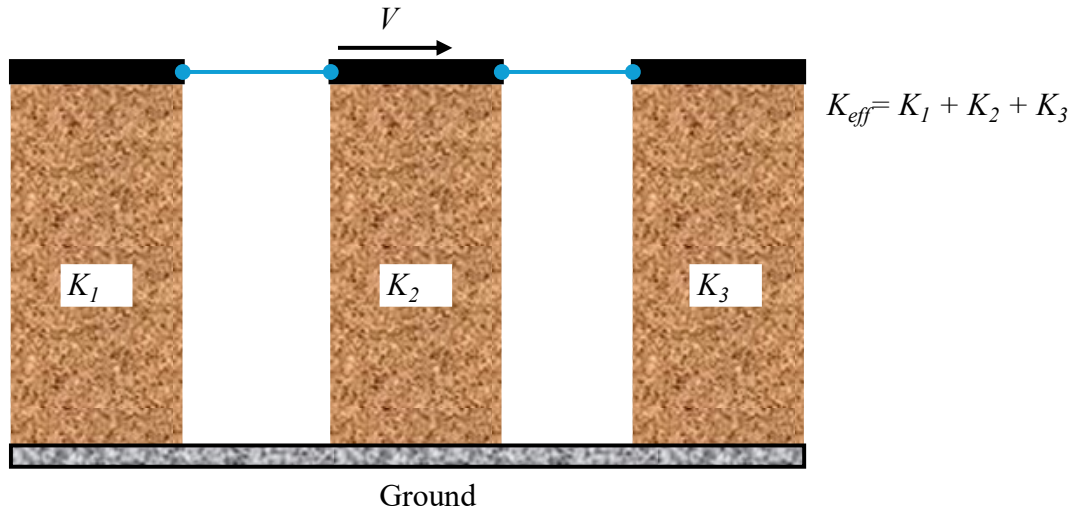
- The experimental and numerical results showed the bending component to only have local effects, with negligible contribution to the global lateral deflection of the shear wall. This is also consistent with the approach taken in the Eurocode (EC 5 1995).

- The shear deformation of the sheathing panel exhibited limited contribution at the early stages of loading and decreased rapidly as the load increased. This conclusion was derived from strain gauge measurements; however, it should be interpreted with caution because strain data are notoriously unreliable and affected by the local conditions, which in the case of OSB panels could be very significant (i.e., locally fibres could be oriented in random directions). The actual reading of the DTs of zero diagonal extension should also be taken into considerations. Until additional experimental evidence becomes available, this component may be retained in the formulation, particularly since the numerical model indicated a non-zero contribution. However, it is conceivable that due to the flexible boundary conditions of the panels (nail connections), their deformation contribution may be insignificant to consider in the deflection equation.
- The deflection due to nail deformations appeared to be underestimated by CSA O86 (Canadian Standards Association 2019). This discrepancy was attributed to the non-uniform distribution of nail deformations along the panel edges. Depending on the floor-wall interaction and boundary conditions, the measured nail deformations ranged from 1.3 to 2.5 times the CSA O86 (Canadian Standards Association 2019) values. These observations highlight the complexity of nail behaviour in LFW shear walls and merit further investigation to attain reliable estimates of nail-related deflection that involves accurate modeling of boundary conditions and system effects.
- The CSA O86 (Canadian Standards Association 2019) overestimated the shear wall deflection associated with HD elongation. On average, the predicted HD elongation by CSA O86 was approximately 50% higher than the experimental and numerical results.

- The CSA O86 (Canadian Standards Association 2019) significantly underestimates the deflection component associated with perpendicular-to-grain crushing of the bottom plate beneath the compression chord. The test results indicate values that are, on average, approximately four times greater than those calculated using CSA O86 (Canadian Standards Association 2019). This finding is particularly important because this deformation component is amplified by the shear wall aspect ratio; therefore, any underestimation becomes more pronounced in walls with higher ARs. At the same time, given the highly discrete nature of wood shear wall systems and the localized character of this deformation mode, it remains unclear whether this behavior contributes to a global rocking response or primarily contributes to the racking deformation of the shear wall.

## **6.7 Multi-storey effects**

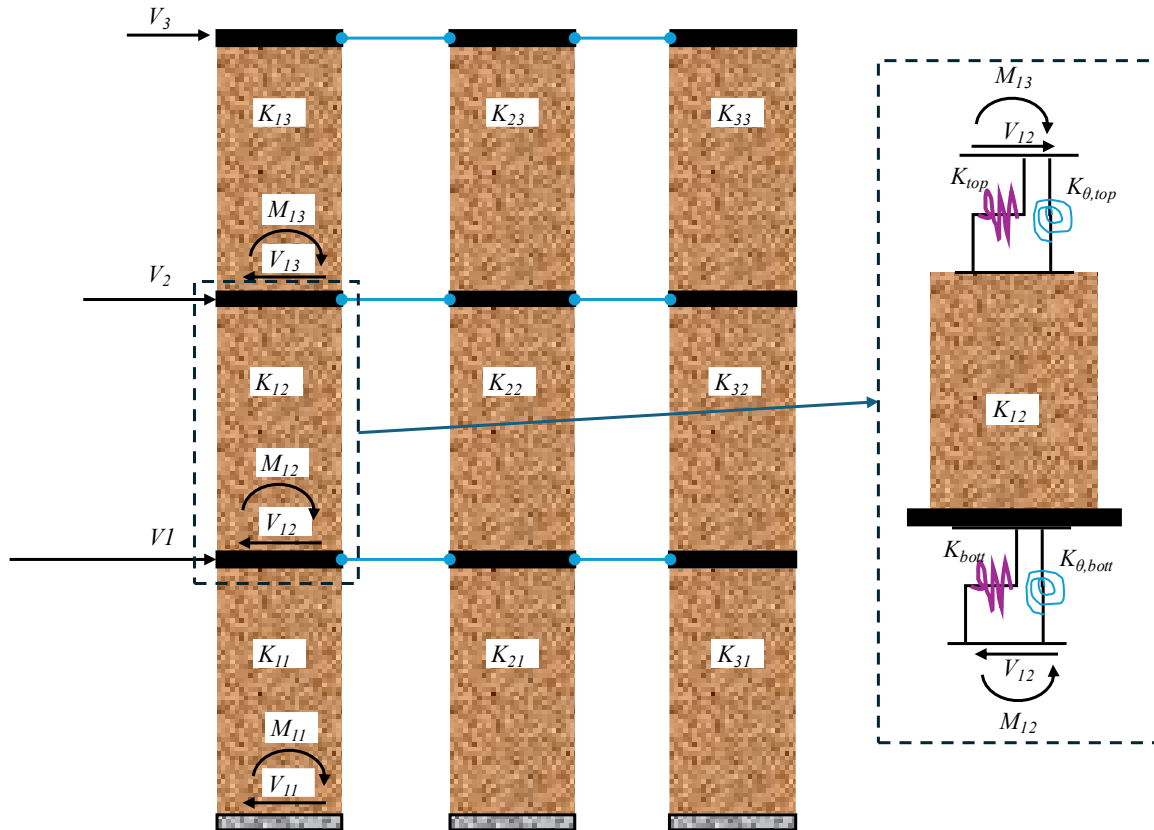
The results in Section 5.5 illustrated that the lateral stiffness of a multi-storey shear wall was strongly influenced by the out-of-plane rigidity of the floor diaphragm as well as by the interaction of the shear walls with surrounding structural components. It was also confirmed that the distribution of forces, both at the storey level and along the height of the shear walls, depended on the relative stiffness of each wall within the overall system. To better understand the factors influencing the stiffness of shear walls in a multi-storey system, it is first vital to revisit the fundamental assumptions governing single-storey shear wall behaviour. Figure 6.29 illustrates a representative multi-segment single-storey shear wall system.



**Figure 6.29: Multi-segment single-storey shear wall**

For the system in Figure 6.29, if equal lateral displacement at the floor level is assumed, the effective stiffness of the system could be taken as the sum of the stiffnesses of the individual wall segments. This assumption is valid provided that the base of the shear walls is sufficiently rigid relative to the wall stiffness, in order to prevent differential rotations or settlements between wall segments. This condition is generally satisfied for single-storey shear walls and the shear walls in the first storey of a multi-storey building.

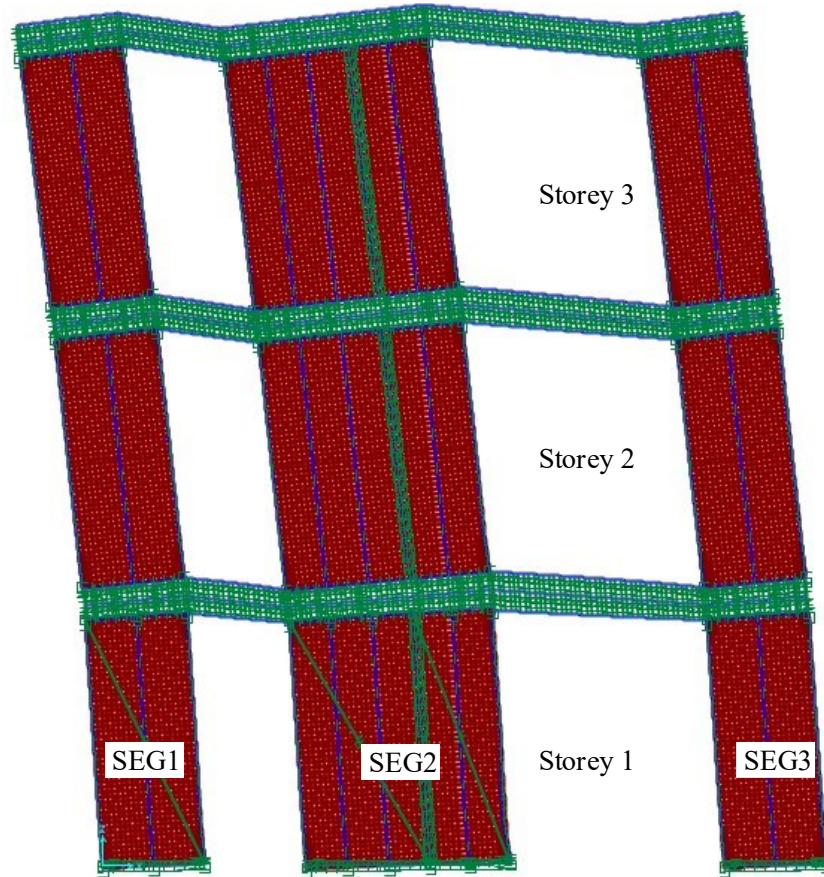
However, the stiffness of shear walls within multi-storey system studied in Section 5.5 did not depend solely on their individual properties (e.g., diaphragm stiffness, relative wall stiffness, overturning compatibility). System-level interactions altered their effective stiffness and the force distribution between them. Figure 6.30 presents a representative multi-storey shear wall system consistent with the M1 model, previously discussed in Section 5.5, highlighting these interactions.



**Figure 6.30: Representation of a wood shear wall in a multi-storey system**

These interdependencies are important because they directly affect (i) the distribution of forces among the shear walls within a storey and (ii) the variation of force demands over the height of the multi-storey system.

The multi-storey representation shown in Figure 6.30 is consistent with the current assumption in CSA O86 (Canadian Standards Association 2019), in which the impact of the out-of-plane stiffness of the floor diaphragm on the lateral deflection of the shear walls and distribution of the forces at the storey level and over the height of the shear wall system is neglected. The corresponding deformed shape of the three-storey shear wall in accordance with CSA O86 (Canadian Standards Association 2019) assumption (M1 model) is presented in Figure 6.31.

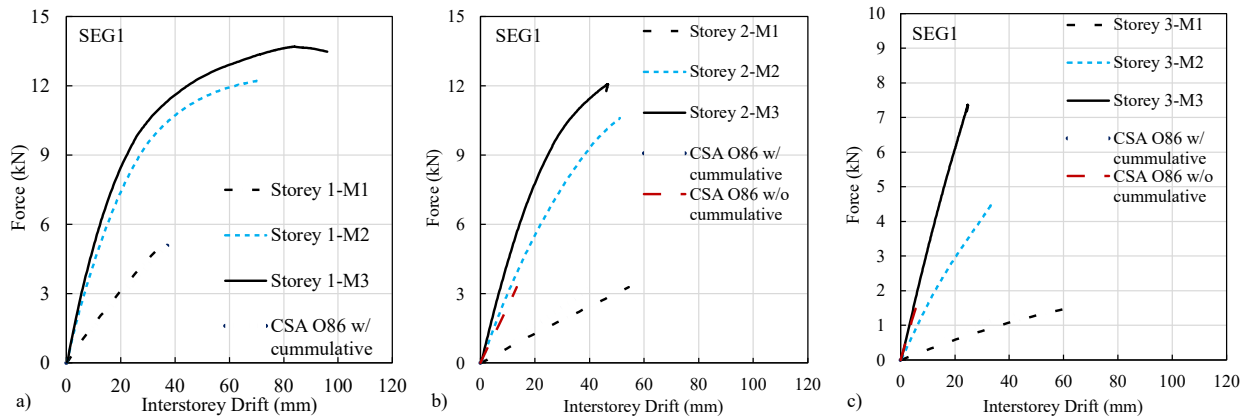


**Figure 6.31: Deformed shape of three-storey shear wall (M1 model)**

Figure 6.31 clearly highlights the cumulative nature of storey displacements, demonstrating how lateral deformations accumulate over the height of the wall. However, as also discussed in Section 5.4, the actual floor system exhibits an out-of-plane flexural stiffness, as represented by the M2 model. In addition, the shear walls interact with surrounding structural components through floor continuity at each storey level, as incorporated in the M3 model.

Figure 6.32 presents the F–D curves of SEG1 (identified in Figure 6.31) at different storeys, extracted from the M1, M2, and M3 modelling assumptions. For comparison, the corresponding F–D relationships of the individual shear walls, calculated in accordance with CSA O86 (Canadian Standards Association 2019) using the four-term equation with and without cumulative effects are also included in these figures. It is important to note that the inputs to the CSA O86 (Canadian Standards Association 2019) equation, including force distribution and mechanical properties,

were consistent with those used in the M1 numerical model, since it is the model that most closely represent the design equation.

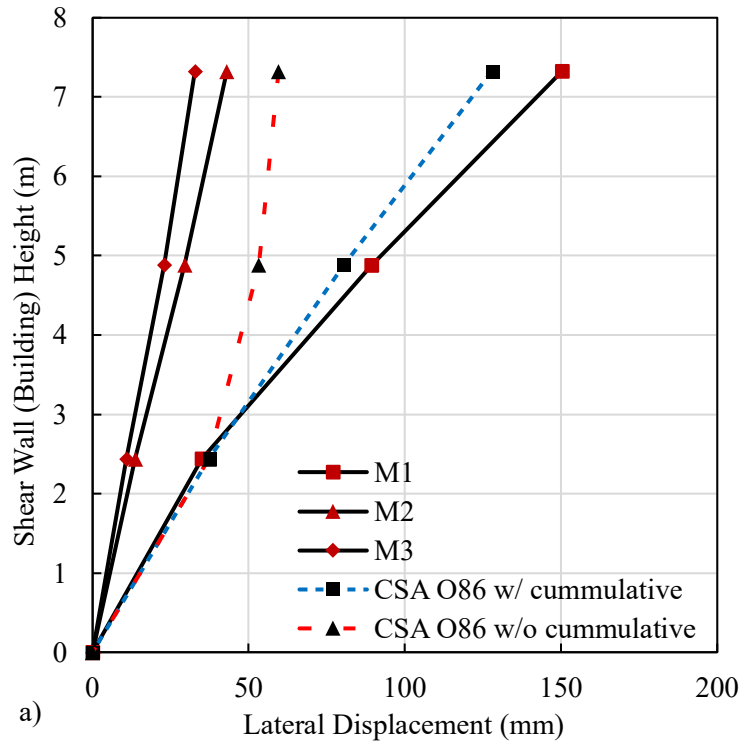


**Figure 6.32: Comparison between shear wall F-D curves in a multi-storey system and corresponding CSA O86 curves according to M1 model**

As expected, the CSA O86 (Canadian Standards Association 2019) F–D curves, which include the cumulative effects, are closest to the numerical results obtained based on the M1 modeling assumption. It is interesting to note, however, that excluding the cumulative effects shifts the curves toward those from the M2 and M3 models. This is not trivial but can have a logical explanation. There is nothing inherent in the four-term deflection equation without cumulative effect that simulates the effect of floor out of plane stiffness or interaction between walls; this effect is included implicitly since the presence of the floor shifts the rocking motion into racking motion (i.e., reducing HD elongations and increasing nail deformations), as demonstrated in Sections 6.4 and 6.5, thereby limiting the accumulation of the rigid body rotation component, and hence aligning mode with Models M2 and M3.

Figure 6.33 provides a different representation of Figure 6.32 by plotting the lateral displacement profile of SEG1 in Figure 6.31 along the building height for different modelling assumptions (M1, M2, and M3), together with the lateral deflection calculated using the CSA O86 (Canadian Standards Association 2019) four-term equation.



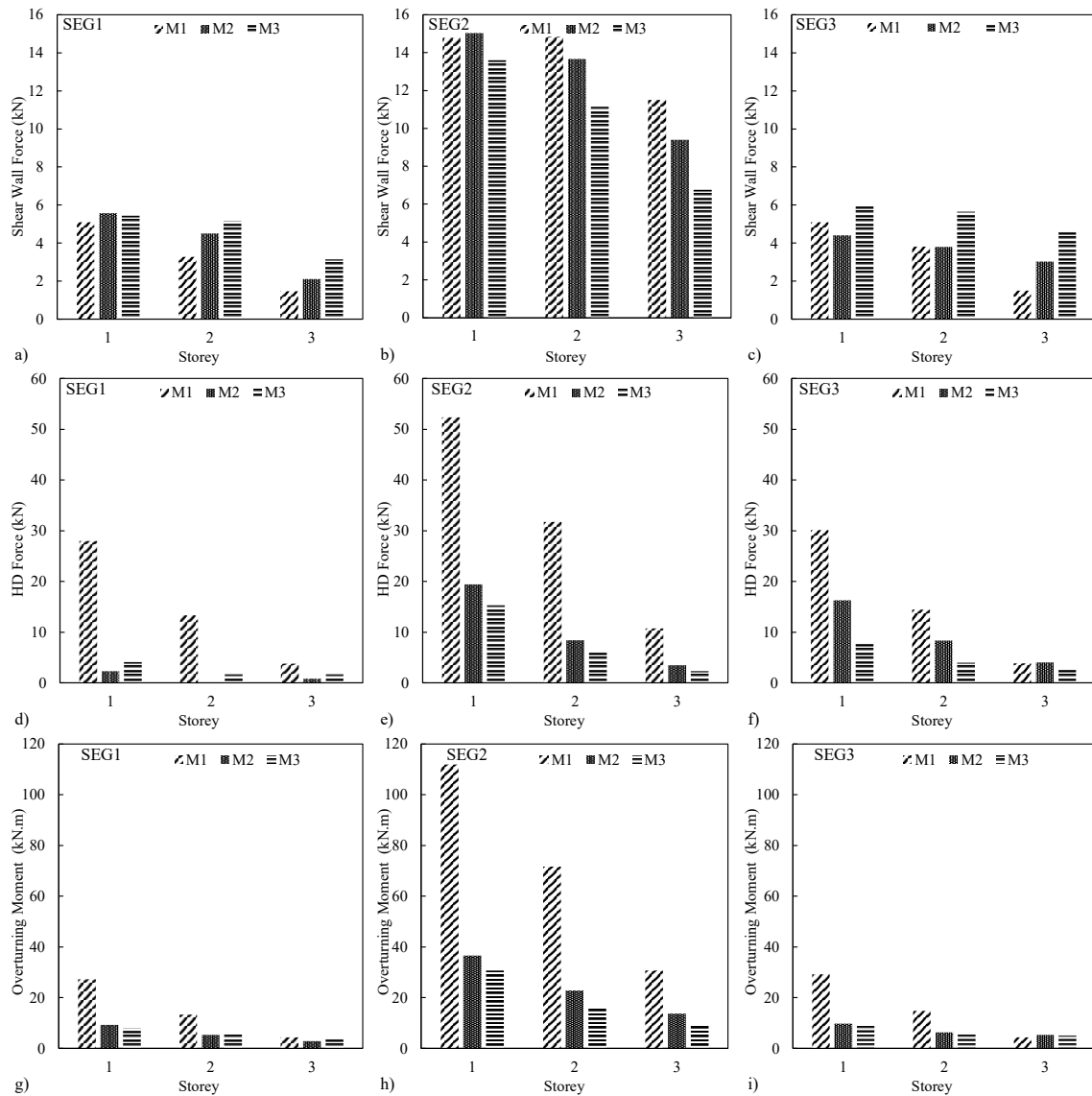


**Figure 6.33: Effect of cumulative displacements on the lateral displacement of shear walls**

Figure 6.33 shows, once again, that including the cumulative displacement effects in the CSA O86 (Canadian Standards Association 2019) four-term equation results in a reasonable match with the displacement profile of the shear wall obtained from the M1 model. Although still overestimating the lateral displacement, the curve without cumulative effects deviates significantly from the M1 model, especially for higher storeys, and aligns more closely with the M2 and M3 models. The remaining discrepancy between the CSA O86 (Canadian Standards Association 2019) prediction without cumulative effects and the M2 and M3 results can be attributed to system-level influences on force distribution and deformation demands that are not captured by design standard equation (Canadian Standards Association 2019).

Figure 6.34(a)–(i) present the distribution of shear wall force, HD force, and overturning moment for SEG1, SEG2, and SEG3 at different storeys. Although the three buildings reached different

base shear force, for consistency, comparisons among the different modelling assumptions were performed at an equal base shear level of 25 kN.



**Figure 6.34: (a)–(c) Shear wall force for; (d)–(f) HD force; (g)–(i) overturning moment**

Figure 6.34(a)–(c) show that the shear wall forces for SEG1, SEG2, and SEG3 in the first storey remained almost unchanged regardless of the modelling assumption. One plausible explanation for this behaviour could be obtained from the test results of Configurations #1, #2, #5, and #6, where the presence of the floor did not significantly influence the overall response of the shear wall when only a single storey was investigated, as discussed in Section 6.1. By considering the support beam

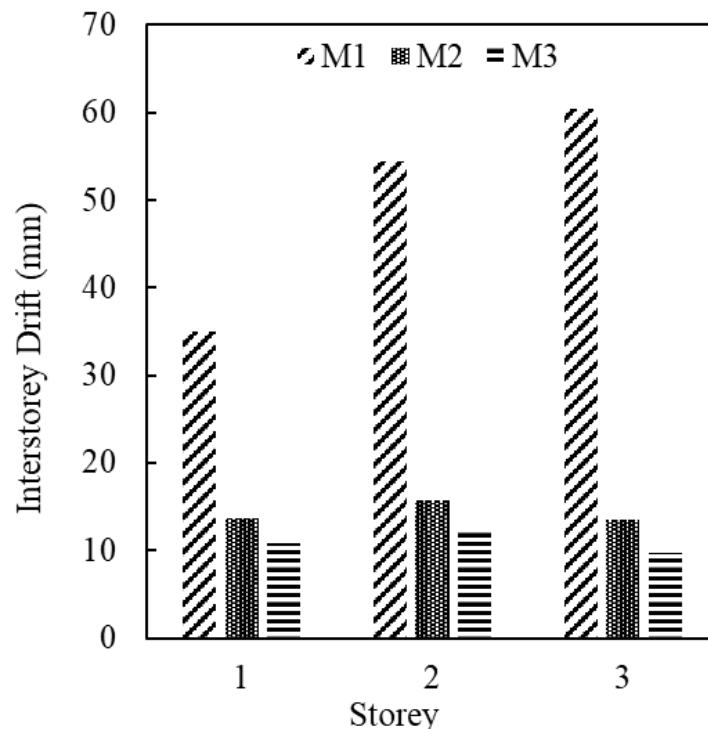
in those experiments as a rigid foundation, the first storey can effectively mimic the behaviour of a single-storey shear wall. However, it should be noted that cumulative HD forces and overturning moments from upper storeys may still influence the displacement response of the first-storey wall. In contrast, Storeys 2 and 3 in Figure 6.34(a)–(c) exhibits noticeable differences in shear wall force demand depending on whether M1, M2, or M3 was adopted. Relative to the forces in SEG1 and SEG3 obtained from M1 Model, the M3 modelling assumption increases the forces in these shear walls by approximately 50% at Storey 2. This increase becomes more pronounced at Storey 3, where the forces in SEG1 and SEG3 rise by about 100% and 200%, respectively, compared to those from M1 Mode.

This discrepancy can likely be attributed to the dependence of upper-storey shear wall behaviour on base flexibility as a result of cumulative effects. When the floor's restraining effect is not considered, cumulative effects can influence the relative stiffness of the shear walls along the building height, leading to a redistribution of forces.

From a design perspective, the significant increase in forces in SEG1 and SEG3 at Storeys 2 and 3 is particularly important. Since current design practice, as per CSA O86 (Canadian Standards Association 2019), is based on the M1 modelling assumption, in a design scenario where shear walls are optimized, system-level effects captured by the M2 and M3 assumptions may alter the force distribution and potentially lead to premature failure of certain walls. Therefore, it is essential that design-stage force estimations adequately reflect realistic force distributions within the system. Figure 6.34(d)–(f) further demonstrates the influence of modelling assumptions on the HD force distribution. Under the M1 assumption, the cumulative effect of HD forces resulted in the maximum demands of 28 kN and 30 kN for SEG1 and SEG3, respectively, at the first storey, with a significant reduction toward the upper storeys. When floor out-of-plane stiffness was introduced

(M2 and M3), this effect was significantly mitigated, leading to a considerable reduction in HD demands. This reduction in HD demands largely explains the deviation of the CSA O86 without cumulative effects curves and the curves from M2 and M3 modelling assumptions shown in Figure 6.33. Figure 6.34(d) and (f) highlight how system effects and differing boundary conditions surrounding two identical shear walls (e.g., SEG1 and SEG3) influence the distribution of HD forces. A similar reduction pattern due to system effects is also observed in the HD forces of SEG2, as illustrated in Figure 6.34(e).

As discussed previously, the impact of floor out-of-plane stiffness, and more generally system-level effects, on the reduction of HD demand and the distribution of forces among shear walls is clearly reflected in the inter-storey drift values. Figure 6.35 illustrates the inter-storey drift of SEG1 in the three-segment, three-storey shear wall under different modelling assumptions.



**Figure 6.35: Inter-storey drift of SEG1 in three-segment three-storey shear wall**

As can be seen from Figure 6.35, the M1 modelling assumption produced the highest inter-storey drifts in all three storeys. In contrast, the M2 and M3 assumptions resulted in significantly lower

inter-storey drift values and almost equal inter-storey drifts. The source of this difference can be traced back to the force distributions shown in Figure 6.34(a) and (d) where the M1 model produced considerably higher HD forces, and consequently higher HD elongations (Figure 6.34(b)) despite relatively lower shear forces at the storey levels for SEG1 (Figure 6.34(a)). Since HD elongations in the M1 modelling assumption directly contribute to total storey displacements, they lead to amplified inter-storey drifts of the shear walls along the height of the building. When realistic floor stiffness and boundary conditions were incorporated (M2 and M3), HD forces were reduced, and the resulting inter-storey drifts became not only smaller but also nearly uniform across the three storeys.

An important takeaway from the comparison in Figure 6.35 is the compliance of the calculated inter-storey drifts with the allowable drift limits specified in the NBC. These drift limits are intended to ensure a certain level of safety by relating drift to the expected damage state of structural elements. However, the high inter-storey drift values shown in Figure 6.35 do not necessarily correspond to an equivalent level of damage.

This interpretation is supported by the force levels predicted by the M1 model for SEG1 at Storeys 1, 2, and 3, which are 5.1 kN, 3.3 kN, and 1.5 kN, respectively (Figure 6.34(a)). These forces are well below the peak resistance of the shear wall (approximately 13 kN), suggesting that the associated internal deformations, and thus damage, are likely limited.

This suggests that the inter-storey drifts presented in Figure 6.35 include rigid-body components, primarily arising from cumulative HD elongations in the lower storeys, which do not directly contribute to material damage within the shear wall. As a result, drift alone, as calculated here, may not be a fully reliable indicator of damage.

At the same time, if HD elongations do accumulate over the height of the building, their influence on the global response cannot be disregarded. Such effects should be accounted for in the stiffness modelling of multi-storey shear wall systems, as they may alter the force distribution among shear walls. Therefore, a careful balance is required in design: while system-level effects should be adequately captured to ensure a realistic representation of structural behaviour, drift limits should be interpreted with consideration of their underlying assumptions.

The inter-storey drift of multi-storey LFW shear walls is also influenced by the cumulative overturning moments at the base of the shear wall, generated by storey forces from the levels above. In CSA O86 (Canadian Standards Association 2019), the calculation of these overturning moments is based on the M1 modelling assumption.

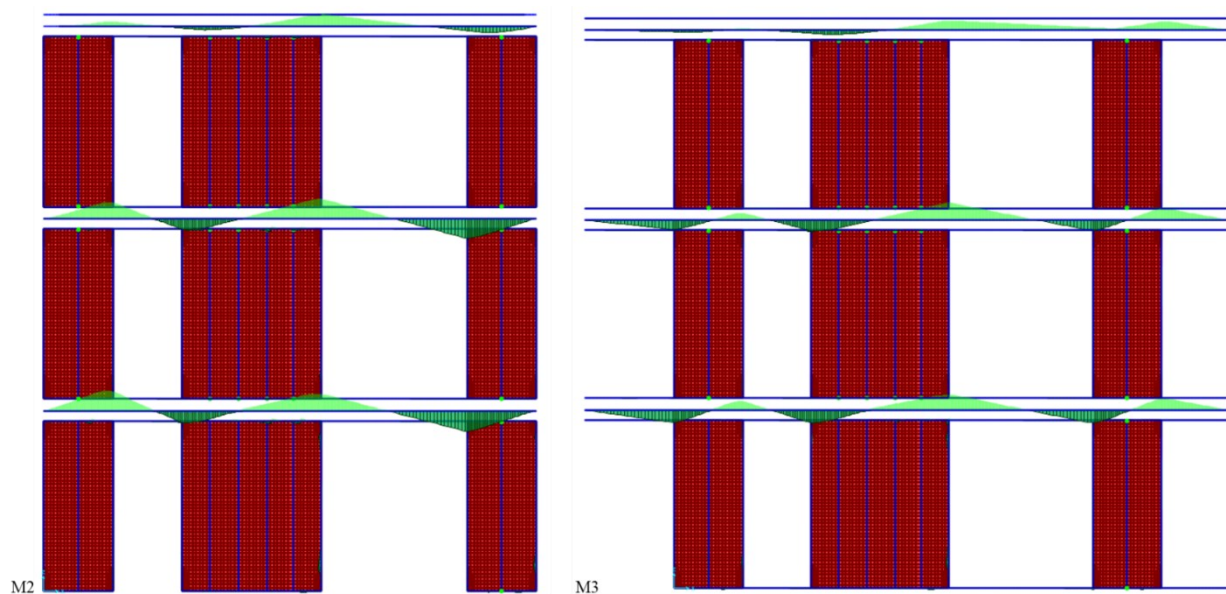
According to CSA O86 (Canadian Standards Association 2019), the contribution of overturning moments to inter-storey drift calculations is twofold. First, they are directly used to determine the lateral deflection of the shear wall. Second, they are used to evaluate the rotation angle ( $\theta$ ) of the chord member at the top of the wall, which becomes particularly important when cumulative bending effects should be included in drift calculations. Both components are based on the assumption that a wood shear wall behaves as a cantilever structural component.

Figure 6.34(g)–(i) illustrate that the overturning moments at the base of the shear walls in each storey are strongly affected by floor flexibility and the adopted modelling approach. At Storeys 1 and 2, incorporating floor stiffness (M2 and M3) results in reductions of approximately 70% or more compared to the M1 model. In contrast, at Storey 3, the overturning moments are nearly identical across all modelling assumptions.

The reduction in overturning moment due to system-level effects is particularly important, as it directly influences both the rotation angle ( $\theta$ ) and the lateral deflection of the shear wall at the floor

level. By neglecting the out-of-plane stiffness of the floor diaphragm and system-level interaction, CSA O86 (Canadian Standards Association 2019) can significantly overestimate the lateral deflection of multi-storey shear walls (Figure 6.5), particularly when cumulative effects along the height of mid-rise buildings are considered.

Figure 6.36 further illustrates the bending moment developed within the floor assembly. Accounting for these moments, which arise from the floor's out-of-plane stiffness, highlights the importance of maintaining overturning moment equilibrium and compatibility at the base of each shear wall. Additionally, the shear forces induced by these bending moments must be accounted for in the force equilibrium around each wall, as both the bending moments and resulting shear forces ultimately govern the internal force distribution in multi-storey shear wall systems.



**Figure 6.36: Variation of the bending moment in floor assembly**

This internal floor action, together with the boundary conditions at each storey, significantly influenced the global response of the multi-storey shear wall system and the distribution of forces among its components. The representation in Figure 6.36 therefore contrasts with the current assumption in CSA O86 (Canadian Standards Association 2019), which treats a multi-storey shear

wall as a series of stacked walls within a shear line without explicitly accounting for floor flexural interaction and system-level coupling effects.

For taller buildings, where cumulative effects along the height become more pronounced, if M1 modelling assumption is used at the design stage, the discrepancies between real and designed force and deformation demands can be further amplified. In other words, treating each action in the shear wall independently and designing based on its maximum demand, whether in terms of force or deformation, does not necessarily lead to a safer system. As demonstrated, different modelling assumptions can increase demands in some shear walls while reducing demands in others, thereby altering the force distribution. Consequently, shear walls that experience higher demands than those assumed at the design stage may be more susceptible to unfavorable or premature failure. Therefore, the adoption of an appropriate and realistic modelling assumption at the design stage is essential to accurately capture distribution of force and deformation demands in multi-storey wood shear wall systems.

The discussion above can be summarized in the following key remarks.

- Across all comparisons, the modelling results consistently show that the out-of-plane flexural stiffness of the floor diaphragm plays a dominant role in shaping the global deformation mode of multi-storey shear walls.
- Realistic floor stiffness suppresses rocking, shifts the deformation mode to shear, and dramatically reduces HD and overturning demands.
- Cumulative effects are most pronounced in upper storeys only when rocking is permitted; once floor stiffness is included, these effects diminish.



- Stiffer systems (e.g., M2 / M3) attract more shear force but at the same force level seem to deflect less than more flexible assumption in (e.g., M1), adequately meeting drift limits.
- Design methods that ignore floor flexural stiffness risk misrepresenting both force and deformation demands, especially in taller buildings where cumulative effects are more significant.
- Accurate prediction of multi-storey shear wall behaviour requires modelling that includes floor flexural stiffness and system-level interactions. Neglecting these effects leads to deformations that are too conservative, base shear forces that are unconservative, while also developing unrealistic estimates of uplift, overturning, and drift.

## Chapter 7

### Summary, conclusion and future works

An experimental and numerical investigation was conducted to assess the role of floor out-of-plane stiffness and associated system-level interactions on the behaviour of multi-storey, multi-segment light frame wood (LFW) shear walls. A particular emphasis was put on the suitability of the CSA O86 four-term deflection equation for estimating lateral deflections to simulate multi-storey deflection with and without considering the cumulative effects outlined in the Canadian timber design standard. A finite element (FE) modelling approach incorporating realistic boundary conditions was developed and verified against experimental observations. The validated model was then extended to multi-storey systems to facilitate parametric analyses of how the system-level interactions affect the global deformation characteristics of LFW structures. The key conclusions that can be drawn from this study are as follows:

1. Floor out-of-plane stiffness and system-level effects did not influence the initial stiffness of single-storey wood shear walls. While a tendency for increased stiffness was observed when wall-to-floor connections incorporated HD elements at the floor level, this observation could not be adequately substantiated due to the small number of test specimens. The presence of the floor also had a negligible effect on shear wall strength.
2. On average, the floor assembly resulted in an increase in nail deformations of about 50%. The range was between 14% to 147%, reflecting considerable variability. The impact of the floor was more pronounced in shear walls with a higher aspect ratio.
3. Sheathing panel aspect ratio significantly influences the distribution of nail deformations in shear walls. Nails around narrow panels experienced considerably smaller deformations

than those around larger panels. Very narrow panels adjacent to larger panels also resulted in excessive nail failures at panels' interface. Maintaining panels of similar AR promoted more uniform nail deformation across the wall and enhanced energy dissipation.

4. Floor out-of-plane and system-level effects substantially reduced HD forces and elongations, with average reduction of about 50% (ranging from 36% to 77%); with the largest reductions occurring in shear walls with ATS rods. Reductions were generally more pronounced in shear walls with higher aspect ratios. The impact of blocking floor assemblies was minimal for long openings and negligible differences were observed between continuous LVL and blocking for very short openings. Part of the reduction in HD demands was also attributed to ABs, which contributed 6–30% of the HD force in resisting overturning moments.
5. The effective stiffness of the ATS system was considerably lower than the theoretical design value, measuring only about 50% of the expected stiffness in the two tested configurations. This discrepancy can lead to significant differences between the calculated inter-storey drift during design and the actual response of the shear walls.
6. Experimental and numerical results indicated that the bending deformation of shear wall chord members does not fully conform to the assumptions of the CSA O86 four-term deflection equation. The experimental results also indicated that the shear deformation in the panels remained nearly constant as sheathing-to-framing connections enter into the nonlinear range. This deformation mode was observed to diminish to near zero at peak shear wall resistance.
7. The CSA O86 four-term deflection equation underestimated the nail deformation under realistic boundary conditions. This underestimation may lead to reduced prediction of

energy dissipation. The inability to capture a realistic nail deformation could result in premature localized failures and diminished overall system performance.

8. The CSA O86 four-term deflection equation significantly overestimated the HD contribution to overall shear wall deflection. This overestimation is partly due to neglecting the contribution of ABs and the assumption of the shear wall as a rigid body when determining HD demands.
9. The CSA O86 assumption of uniform crushing beneath the compression chord seemed not to reflect the experimental observations, which showed localized indentation in the shear wall bottom plate at the outer edge of the compression chord, resulting in significantly higher deformations than predicted by CSA O86. However, this was a localized behaviour that primarily increased the nail deformations at the base of the shear wall rather than contributing to rigid-body rocking behaviour of the shear wall.
10. In multi-storey systems, the CSA O86 deflection equation, which includes cumulative effects, produced results similar to analyses that neglect floor diaphragm stiffness. However, excluding cumulative effects more closely represents buildings with realistic floor systems. The cumulative effects tend to significantly overestimate the rocking deformation, where it was clearly observed that the floor out of plane stiffness shifted the rocking behaviour (dominated by HD elongation) to shear racking behaviour (dominated by nail deformation).
11. Neglecting floor out-of-plane stiffness and system-level effects substantially underestimated shear wall force demands, with increases of 100–200% when these effects were considered. Distribution of forces along the height of the multi-storey shear wall was

governed by changes in relative stiffness among shear walls, which were influenced by cumulative effects and floor restraint conditions.

12. Inter-storey drift calculation was strongly influenced by rigid-body components arising from cumulative HD elongations. Therefore, treating inter-storey drift as a direct indicator of shear wall damage may overestimate the actual damage of the wall leading to perceived premature failure.

13. Overturning moments in multi-storey shear walls are highly sensitive to floor stiffness, with substantial reductions observed when system-level effects are included.

Based on the observations from this study, several key directions warrant further investigation:

- This study considered a limited number of shear wall configurations. Future research should explore additional configurations to allow for generalization of the findings.
- The experimental part of this study focused on single-storey shear walls. Although the floor system was included in the experimental program, the analysis of multi-storey behaviour was limited to numerical investigations. Future experimental investigations should include work on multistorey (e.g., two-storey) shear walls, incorporating floor out-of-plane stiffness to examine cumulative displacement effects.
- This study employed a monotonic loading regime. Given the increased nail deformations due to system effects, it is recommended that cyclic tests be conducted to assess how floor diaphragms influence the energy dissipation of shear walls, particularly in multistorey buildings.
- Shear walls with ATS rods require further investigation to quantify the effective stiffness of the ATS system.

## **Bibliography**

- ASTM A370. (2024). ASTM A370 – 24a, Standard Test Methods and Definitions for Mechanical Testing of Steel Products.
- ASTM D198. (2022). ASTM D198 -22a, Standard Test Methods of Static Tests of Lumber in Structural Sizes.
- ASTM D1761. (2020). ASTM D1761 – 20, Standard Test Methods for Mechanical Fasteners in Wood and Wood-Based Materials.
- ASTM E2126. (2019). ASTM E2126 - 19, Standard Test Methods for Cyclic (Reversed) Load Test for Shear Resistance of Vertical Elements of the Lateral Force Resisting Systems for Buildings.
- ASTM F1575. (2025). ASTM F1575/F1575M - 21, Standard Test Method for Determining Bending Yield Moment of Nails.
- ASTM F1667. (2022). F1667/F1667M – 21a, Standard Specification for Driven Fasteners: Nails, Spikes, and Staples.
- Atherton, G. H. (1983). ULTIMATE STRENGTH OF STRUCTURAL PARTICLEBOARD DIAPHRAGMS. *Forest Products Journal*, 33(5).
- Bagheri, M. M., & Doudak, G. (2020a). Structural characteristics of light-frame wood shear walls with various construction detailing. *Engineering Structures*, 205. <https://doi.org/10.1016/j.engstruct.2019.110093>
- Bagheri, M. M., & Doudak, G. (2020b). Structural characteristics of light-frame wood shear walls with various construction detailing. *Engineering Structures*, 205. <https://doi.org/10.1016/j.engstruct.2019.110093>
- Bagheri, M. M., Doudak, G., & Gong, M. (2019). Impact of Out-of-Plane Stiffness of the Diaphragm on Deflection of Wood Light-Frame Shear Walls. *International Journal of Civil and Environmental Engineering*, 13(July).
- Bott, J. W. (2005). Horizontal stiffness of wood diaphragms. Virginia Tech.
- Canadian Commission on Building and Fire Codes. (2020). National Building Code of Canada. In National Research Council Canada, Ottawa, Canada. National Research Council of Canada.
- Canadian Standard Association. (2021). Construction Sheathing. CSA O325-21. In Components (Number June).

- Canadian Standards Association. (2019). Engineering design in wood. CSA O86-19. CSA O86.
- Canadian Wood Council. (2020). Wood Design Manual. Canadian Wood Council.
- Computers and Structures, Inc. (2015). SAP2000 (18). Computers and Structures, Inc.
- Corda, D. N. (1982). The in-plane shear response of timber diaphragms. West Virginia University.
- Dolan, J. D. (1989). The dynamic response of timber shear walls. In Thesis Doctor of Philosophy (Number October).
- Dolan, J. D., & Madsen, B. (1992). Monotonic and cyclic tests of timber shear walls. *Canadian Journal of Civil Engineering*, 19(3). <https://doi.org/10.1139/192-050>
- Doudak, G., & Smith, I. (2009). Capacities of OSB-Sheathed Light-Frame Shear-Wall Panels with or without Perforations. *Journal of Structural Engineering*, 135(3). [https://doi.org/10.1061/\(asce\)0733-9445\(2009\)135:3\(326\)](https://doi.org/10.1061/(asce)0733-9445(2009)135:3(326))
- Doudak, G., Smith, I., McClure, G., Mohammad, M., & Lepper, P. (2006). Tests and finite element models of wood light-frame shear walls with openings. *Progress in Structural Engineering and Materials*, 8(4). <https://doi.org/10.1002/pse.223>
- Durham, J., Lam, F., & Prion, H. G. L. (2001). Seismic Resistance of Wood Shear Walls with Large OSB Panels. *Journal of Structural Engineering*, 127(12). [https://doi.org/10.1061/\(asce\)0733-9445\(2001\)127:12\(1460\)](https://doi.org/10.1061/(asce)0733-9445(2001)127:12(1460))
- Easley, J. T., Foomani, M., & Dodds, R. H. (1982). FORMULAS FOR WOOD SHEAR WALLS. *ASCE J STRUCT DIV*, V 108(N ST11). <https://doi.org/10.1061/jsdeag.0006075>
- EC 5. (1995). Eurocode 5: Design of timber structures — Part 1-2: General — Structural fire design.
- Falk, R. H., & Itani, R. Y. (1987). Dynamic Characteristics of Wood and Gypsum Diaphragms. *Journal of Structural Engineering*, 113(6). [https://doi.org/10.1061/\(asce\)0733-9445\(1987\)113:6\(1357\)](https://doi.org/10.1061/(asce)0733-9445(1987)113:6(1357))
- Falk, R. H., & Itani, R. Y. (1989). Finite Element Modeling of Wood Diaphragms. *Journal of Structural Engineering*, 115(3). [https://doi.org/10.1061/\(asce\)0733-9445\(1989\)115:3\(543\)](https://doi.org/10.1061/(asce)0733-9445(1989)115:3(543))
- Filiatrault, A., Fischer, D., Folz, B., & Uang, C. M. (2002). Experimental parametric study on the in-plane stiffness of wood diaphragms. *Canadian Journal of Civil Engineering*, 29(4). <https://doi.org/10.1139/102-036>

- Folz, B., & Filiatrault, A. (2001). Cyclic Analysis of Wood Shear Walls. *Journal of Structural Engineering*, 127(4). [https://doi.org/10.1061/\(asce\)0733-9445\(2001\)127:4\(433\)](https://doi.org/10.1061/(asce)0733-9445(2001)127:4(433))
- Forest Products Laboratory - USDA. (2021). Wood Handbook: Wood as an Engineering Material. In USDA - General Technical Report.
- Foschi, R. O. (1977). ANALYSIS OF WOOD DIAPHRAGMS AND TRUSSES - 1. DIAPHRAGMS. *Canadian Journal of Civil Engineering*, 4(3). <https://doi.org/10.1139/l77-043>
- GangaRao, H. V. S., & Luttrell, L. D. (1979). Preliminary Investigation into the Response of Timber Diaphragms. Proceedings of a Workshop on Design of Horizontal Wood Diaphragms, Applied Technology Council, Publication ATC-7-1.
- Grandmont, J. F., Cloutier, A., Gendron, G., & Desjardins, R. (2010). Effect of density on the properties of oriented strandboard web stock used in wood i-joists. In *Forest Products Journal* (Vol. 60, Numbers 7–8). <https://doi.org/10.13073/0015-7473-60.7.592>
- Gupta, A. K., & Kuo, G. P. (1985). Behavior of Wood-Framed Shear Walls. *Journal of Structural Engineering*, 111(8). [https://doi.org/10.1061/\(asce\)0733-9445\(1985\)111:8\(1722\)](https://doi.org/10.1061/(asce)0733-9445(1985)111:8(1722))
- Gupta, A. K., & Kuo, G. P. (1987). Wood-Framed Shear Walls with Uplifting. *Journal of Structural Engineering*, 113(2). [https://doi.org/10.1061/\(asce\)0733-9445\(1987\)113:2\(241\)](https://doi.org/10.1061/(asce)0733-9445(1987)113:2(241))
- Gutkowski, R. M., & Castillo, A. L. (1988). Single- and Double-Sheathed Wood Shear Wall Study. *Journal of Structural Engineering*, 114(6). [https://doi.org/10.1061/\(asce\)0733-9445\(1988\)114:6\(1268\)](https://doi.org/10.1061/(asce)0733-9445(1988)114:6(1268))
- Hafeez, G., Doudak, G., & McClure, G. (2019). Dynamic characteristics of light-frame wood buildings. *Canadian Journal of Civil Engineering*, 46(1). <https://doi.org/10.1139/cjce-2017-0266>
- Hafeez, G., Mustafa, A., Doudak, G., & McClure, G. (2014). Predicting the Fundamental Period of Light-Frame Wood Buildings. *Journal of Performance of Constructed Facilities*, 28(6). [https://doi.org/10.1061/\(asce\)cf.1943-5509.0000519](https://doi.org/10.1061/(asce)cf.1943-5509.0000519)
- Hassanzadeh Shirazi, S. M. (2012). Propagation of uncertainty in light-frame wood buildings. Clemson University.
- He, M., Magnusson, H., Lam, F., & Prion, H. G. L. (1999). Cyclic Performance of Perforated Wood Shear Walls with Oversize OSB Panels. *Journal of Structural Engineering*, 125(1). [https://doi.org/10.1061/\(asce\)0733-9445\(1999\)125:1\(10\)](https://doi.org/10.1061/(asce)0733-9445(1999)125:1(10))



- Itani, R. Y., & Cheung, C. K. (1984). Nonlinear Analysis of Sheathed Wood Diaphragms. *Journal of Structural Engineering*, 110(9). [https://doi.org/10.1061/\(asce\)0733-9445\(1984\)110:9\(2137\)](https://doi.org/10.1061/(asce)0733-9445(1984)110:9(2137))
- Jewell, R. B. (1981). The static and dynamic experimental analysis of wooden diaphragms. West Virginia University.
- McCutcheon, W. J. (1985). Racking Deformations in Wood Shear Walls. *Journal of Structural Engineering*, 111(2). [https://doi.org/10.1061/\(asce\)0733-9445\(1985\)111:2\(257\)](https://doi.org/10.1061/(asce)0733-9445(1985)111:2(257))
- Morshedi, E., Doudak, G., Fazileh, F., & Fathi-Fazl, R. (2024). Seismic Performance Assessment of Wood Light-Frame Shearwalls Using the Performance-Based Unified Procedure. *Journal of Structural Engineering*, 150(11). <https://doi.org/10.1061/jsendh.steng-13742>
- Pang, W., & Hassanzadeh Shirazi, S. M. (2013). Corotational Model for Cyclic Analysis of Light-Frame Wood Shear Walls and Diaphragms. *Journal of Structural Engineering*, 139(8). [https://doi.org/10.1061/\(asce\)st.1943-541x.0000595](https://doi.org/10.1061/(asce)st.1943-541x.0000595)
- Patton-Mallory, M., & McCutcheon, W. J. (1987). PREDICTING RACKING PERFORMANCE OF WALLS SHEATHED ON BOTH SIDES. *Forest Products Journal*, 37(9).
- Patton-Mallory, M., Wolfe, R. W., Soltis, L. A., & Gutkowski, R. M. (1985). Light-Frame Shear Wall Length and Opening Effects. *Journal of Structural Engineering*, 111(10). [https://doi.org/10.1061/\(asce\)0733-9445\(1985\)111:10\(2227\)](https://doi.org/10.1061/(asce)0733-9445(1985)111:10(2227))
- Pei, S., & van de Lindt, J. W. (2009). Coupled shear-bending formulation for seismic analysis of stacked wood shear wall systems. *Earthquake Engineering and Structural Dynamics*, 38(14). <https://doi.org/10.1002/eqe.926>
- Pei, S., & Van de Lindt, J. W. (2011). Seismic numerical modeling of a six-story light-frame wood building: Comparison with experiments. *Journal of Earthquake Engineering*, 15(6). <https://doi.org/10.1080/13632469.2010.544840>
- Pei, S., van de Lindt, J. W., Ni, C., & Pryor, S. E. (2010). Experimental seismic behavior of a five-storey double-midply wood shear wall in a full-scale building. *Canadian Journal of Civil Engineering*, 37(9). <https://doi.org/10.1139/L10-058>
- Polensek, A. (1972). Static and dynamic analysis of wood-joint floors by the finite element method.
- Robertson, R. A. (1980). Discussion of “Racking Strength of Light-Frame Nailed Walls.” *Journal of the Structural Division*, 106(9). <https://doi.org/10.1061/jsdeag.0005523>

- Salenikovich, A. J., & Dolan, J. D. (2003a). The racking performance of shear walls with various aspect ratios. Part 2. Cyclic tests of fully anchored walls. *Forest Products Journal*, 53(11–12).
- Salenikovich, A. J., & Dolan, J. D. (2003b). The racking performance of shear walls with various aspect ratios. Part I. Monotonic tests of fully anchored walls. *Forest Products Journal*, 53(10).
- Serrette, R. L., Encalada, J., Juadines, M., & Nguyen, H. (1997). Static Racking Behavior of Plywood, OSB, Gypsum, and FiberBond Walls with Metal Framing. *Journal of Structural Engineering*, 123(8). [https://doi.org/10.1061/\(asce\)0733-9445\(1997\)123:8\(1079\)](https://doi.org/10.1061/(asce)0733-9445(1997)123:8(1079))
- Simpson Strong-Tie. (2022). Strong-Rod Systems: Seismic and Wind Anchor Tiedown System Guide. In Simpson Strong-Tie. “Simpson Strong-Tie. <https://www.strongtie.com>.”
- Skaggs, T. D., Martin, Z. A., Keith, E. L., & Yeh, B. (2008). Full-Scale 3D Wall Bracing Tests. *Structures Congress 2008: Crossing Borders*, 1–10.
- Steiger, R., Feltrin, G., Weber, F., Nerbano, S., & Motavalli, M. (2017). Experimental modal analysis of a multi-storey light-frame timber building. *Bulletin of Earthquake Engineering*, 15(8). <https://doi.org/10.1007/s10518-015-9828-9>
- Stewart, W. (1987). The seismic design of plywood sheathed shear walls. University of Canterbury.
- Tissell, J. R., & Elliott, J. R. (1977). Plywood diaphragms.
- Tuomi, R. L., & McCutcheon, W. J. (1978). RACKING STRENGTH OF LIGHT-FRAME NAILED WALLS. *ASCE J Struct Div*, 104(7). <https://doi.org/10.1061/jsdeag.0004955>
- van de Lindt, J. W., Pei, S., Pryor, S. E., Shimizu, H., & Isoda, H. (2010). Experimental Seismic Response of a Full-Scale Six-Story Light-Frame Wood Building. *Journal of Structural Engineering*, 136(10). [https://doi.org/10.1061/\(asce\)st.1943-541x.0000222](https://doi.org/10.1061/(asce)st.1943-541x.0000222)
- Wang, Q. (2009). Relationship between fastening properties and load-deflection response of wood shear walls [MSc Thesis]. University of New Brunswick.
- White, M. W. (1995). Parametric study of timber shear walls. Virginia Polytechnic Institute and State University.
- WoodWorks. (2025). WoodWorks – Wood Products Council.

- Younis, H., Abadi, H. Al, Patel, V. I., & Abuel-Naga, H. (2025). Experiment and numerical analysis of laminated veneer lumber beams with carbon fiber reinforced polymer reinforcement. *Structures*, 82. <https://doi.org/10.1016/j.istruc.2025.110662>
- Ziaei Ghehnavieh, E. (2017). Seismic analysis of light-frame wood building with a soft-story deficiency. Clemson University.

## Appendix A

### Shear Wall Configurations, Design Information, and Complete Test Results (Response Curves)

In this appendix, the geometry of each shear wall configuration in the experimental program, along with the corresponding design information, is presented. In addition, the results from the tests and numerical models are presented for the response quantities recorded during testing and their corresponding numerical simulation results.

#### A.1 Configuration #1

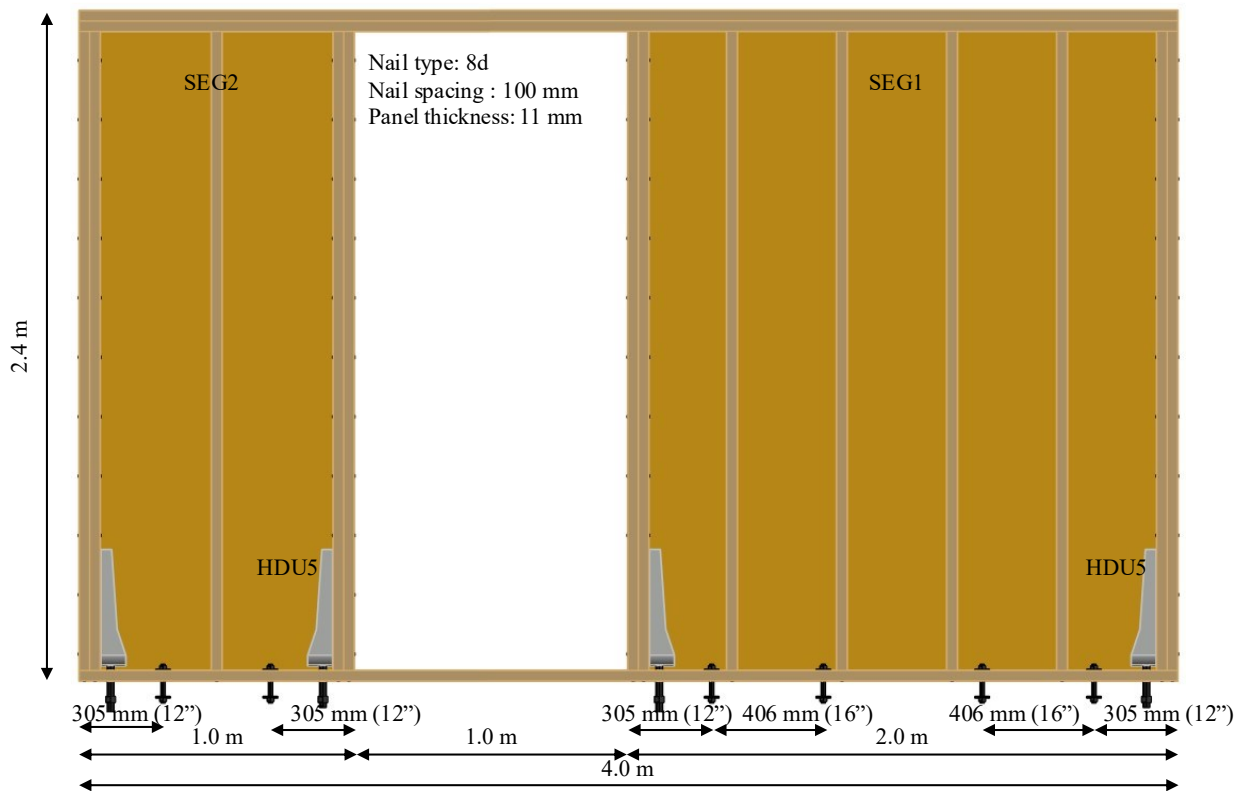


Figure A.1: Configuration #1 geometry

Table A-1: Shear wall design information (Configuration #1)

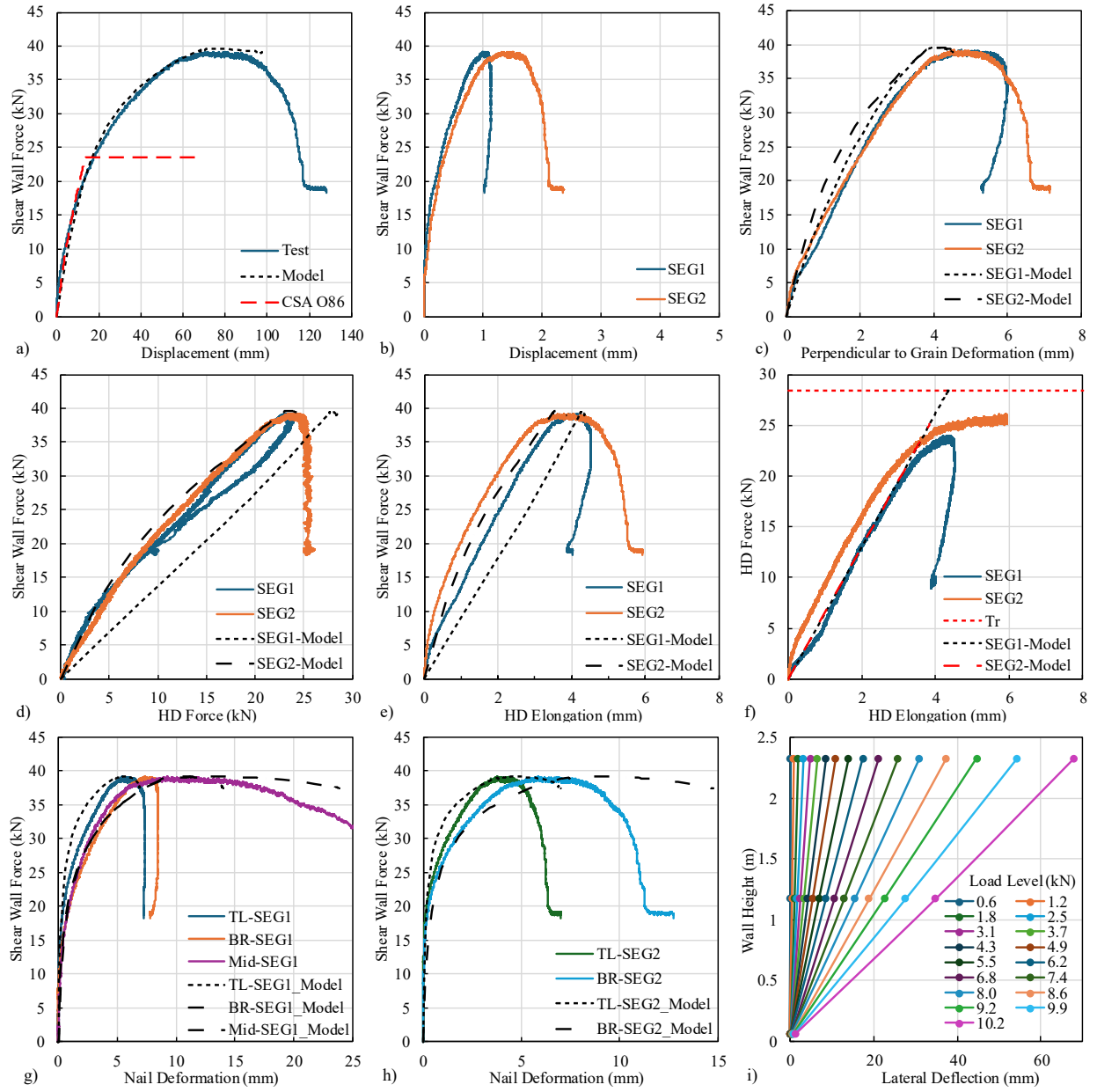
| Configuration  | Shear Wall ID | $L_s$ (mm) | $V_{rs}$ (kN) | $V_d$ (kN) | (D/C) <sub>sw</sub> | $L_o$ (mm) | $T_r$ (kN) | $T_d$ (kN) | (D/C) <sub>HD</sub> |
|----------------|---------------|------------|---------------|------------|---------------------|------------|------------|------------|---------------------|
| #1, #2, #5, #6 | 1             | 2000       | 15.8          | 17.4       | 1.1                 | 1851       | 28.4       | 27.5       | 0.97                |
|                | 2             | 1000       | 7.9           | 6.3        | 0.8                 | 851        | 28.4       | 21.7       | 0.76                |

$L_s$ : Shear wall length;  $V_{rs}$ : Factored shear resistance;  $V_d$ : Design shear force; (D/C)<sub>sw</sub>: Shear wall demand-to-capacity ratio;  $L_o$ : Overturning length;  $T_r$ : HD factored tensile resistance;  $T_d$ : HD design force; (D/C)<sub>HD</sub>: HD demand-to-capacity ratio

Table A-2: Shear wall design deflections (Configuration #1)

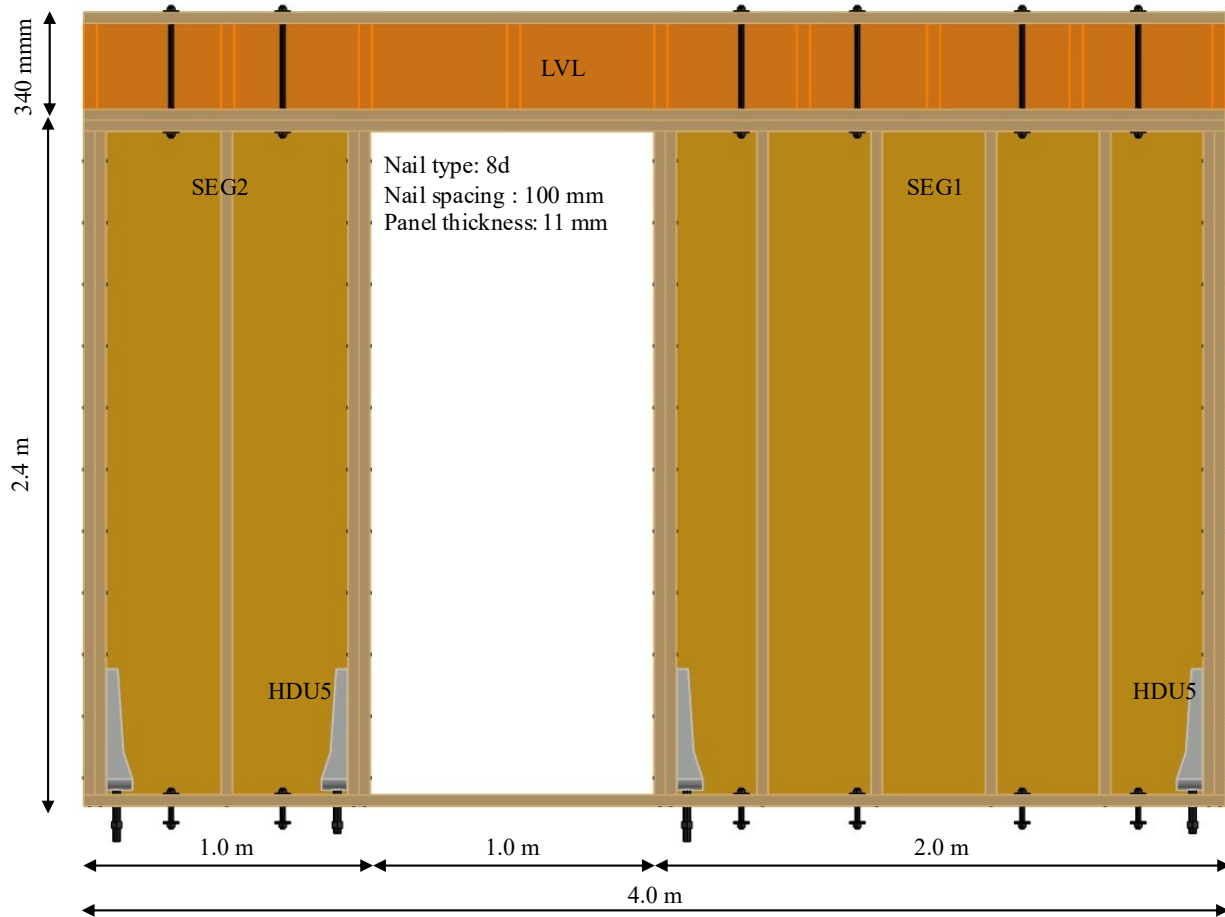
| Configuration  | Shear wall ID | $e_n$ (mm) | $d_{a1}$ (mm) | $d_{a2}$ (mm) | $\Delta_b$ (mm) | $\Delta_s$ (mm) | $\Delta_n$ (mm) | $\Delta_{a1}$ (mm) | $\Delta_{a2}$ (mm) | $\Delta_t$ (mm) |
|----------------|---------------|------------|---------------|---------------|-----------------|-----------------|-----------------|--------------------|--------------------|-----------------|
| #1, #2, #5, #6 | 1             | 1.04       | 3.15          | 0.41          | 0.66            | 1.95            | 6.34            | 4.15               | 0.54               | 13.6            |
|                | 2             | 0.54       | 2.46          | 0.33          | 0.93            | 1.37            | 3.29            | 7.05               | 0.94               | 13.6            |

$e_n$ : Nail deformation;  $d_{a1}$ : HD elongation;  $d_{a2}$ : Compression perpendicular to grain deformation;  $\Delta_b$ : Shear wall deflection due to bending;  $\Delta_s$ : Shear wall deflection due to sheathing panel shear deformation;  $\Delta_n$ : Shear wall deflection due to nail deformations;  $\Delta_{a1}$ : Shear wall deflection due to HD elongation;  $\Delta_{a2}$ : Shear wall deflection due to compression perpendicular to grain deformation of the bottom plate



**Figure A.2: Response quantities in Configuration #1: (a) shear wall F–D; (b) shear wall bottom plate slide; (c) perpendicular-to-grain compression deformations; (d) shear-wall force vs. hold-down force; (e) shear-wall force vs. hold-down elongation; (f) hold-down F–D; (g) and (h) nail deformations in SEG1 and SEG2, respectively; (i) chord lateral displacement profile**

## A.2 Configuration #2



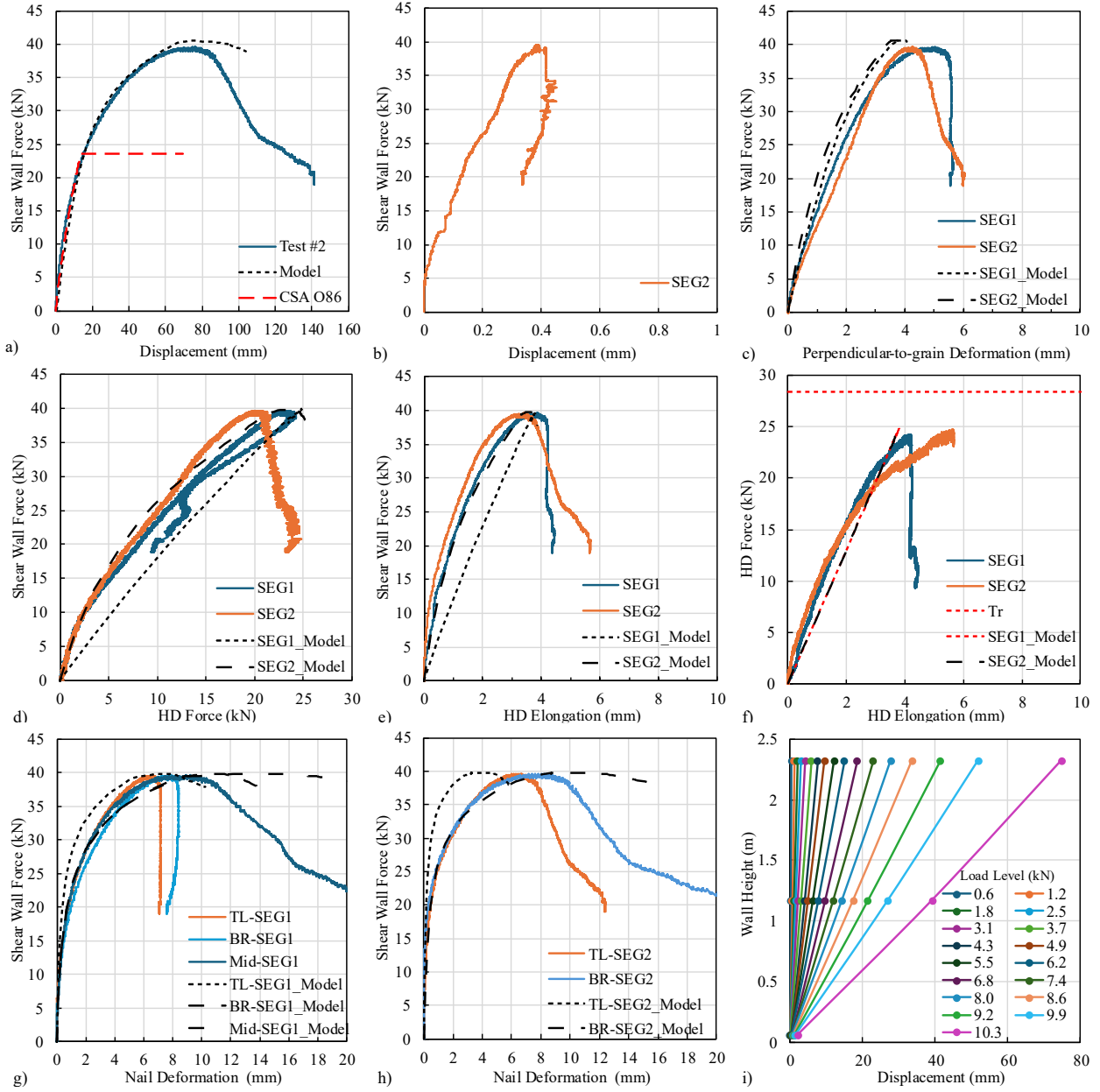
**Figure A.3: Configuration #2 geometry**

Table A-3: Shear wall design information (Configuration #2)

| Configuration | Shear Wall ID | $L_s$<br>(mm) | $V_{rs}$<br>(kN) | $V_d$<br>(kN) | $(D/C)_{sw}$ | $L_o$<br>(mm) | $T_r$<br>(kN) | $T_d$<br>(kN) | $(D/C)_{HD}$ |
|---------------|---------------|---------------|------------------|---------------|--------------|---------------|---------------|---------------|--------------|
| #2            | 1             | 2000          | 15.8             | 17.4          | 1.1          | 1851          | 28.4          | 27.5          | 0.97         |
|               | 2             | 1000          | 7.9              | 6.3           | 0.8          | 851           | 28.4          | 21.7          | 0.76         |

Table A-4: Shear wall design deflections(Configuration #2)

| Configuration | Shear wall ID | $e_n$ (mm) | $d_{a1}$ (mm) | $d_{a2}$ (mm) | $\Delta_b$ (mm) | $\Delta_s$ (mm) | $\Delta_n$ (mm) | $\Delta_{a1}$ (mm) | $\Delta_{a2}$ (mm) | $\Delta_t$ (mm) |
|---------------|---------------|------------|---------------|---------------|-----------------|-----------------|-----------------|--------------------|--------------------|-----------------|
| #2            | 1             | 1.04       | 3.15          | 0.41          | 0.66            | 1.95            | 6.34            | 4.15               | 0.54               | 13.6            |
|               | 2             | 0.54       | 2.46          | 0.33          | 0.93            | 1.37            | 3.29            | 7.05               | 0.94               | 13.6            |



**Figure A.4: Response quantities in Configuration #2: (a) shear wall F–D; (b) shear wall bottom plate slide; (c) perpendicular-to-grain compression deformations; (d) shear-wall force vs. hold-down force; (e) shear-wall force vs. hold-down elongation; (f) hold-down F–D; (g) and (h) nail deformations in SEG1 and SEG2, respectively; (i) chord lateral displacement profile**



### A.3 Configuration #3

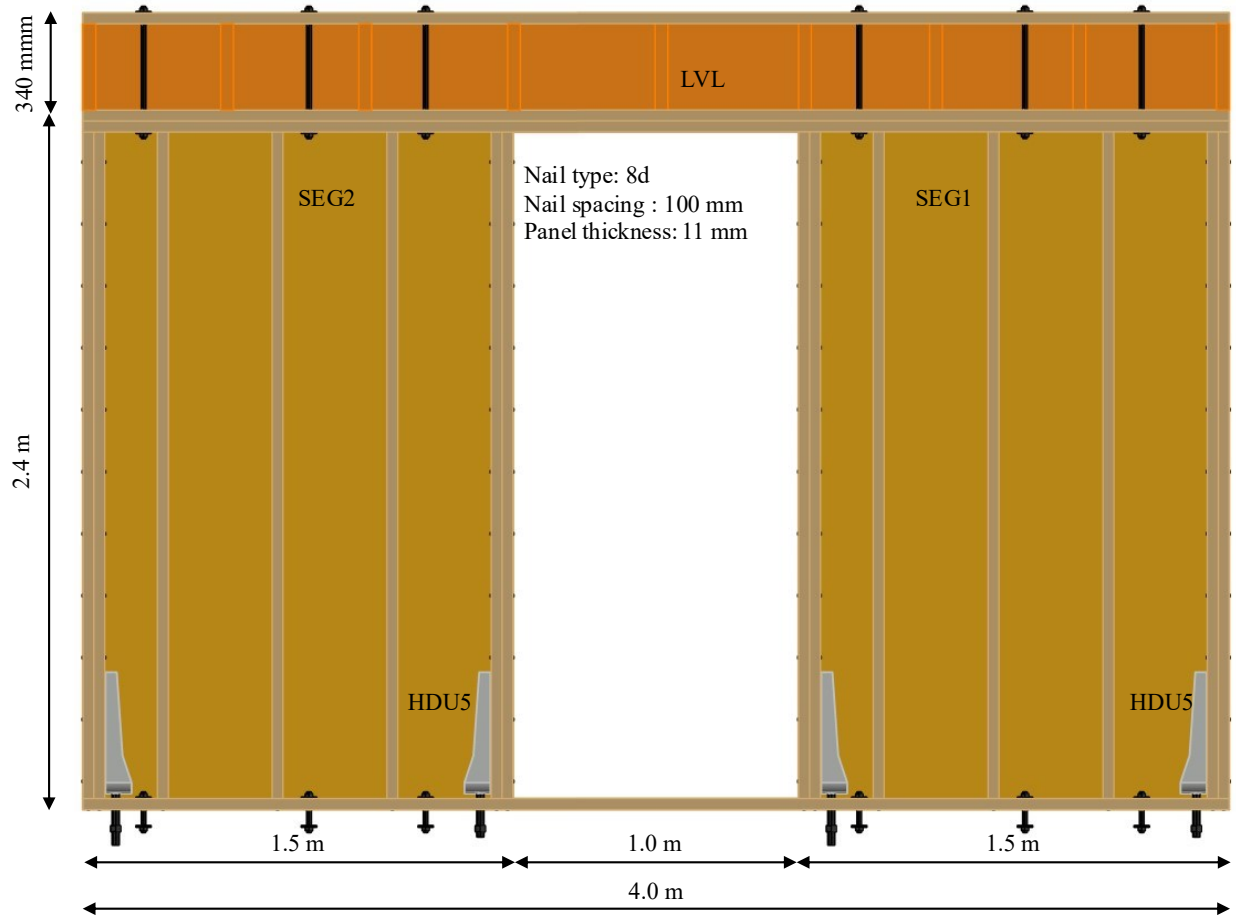


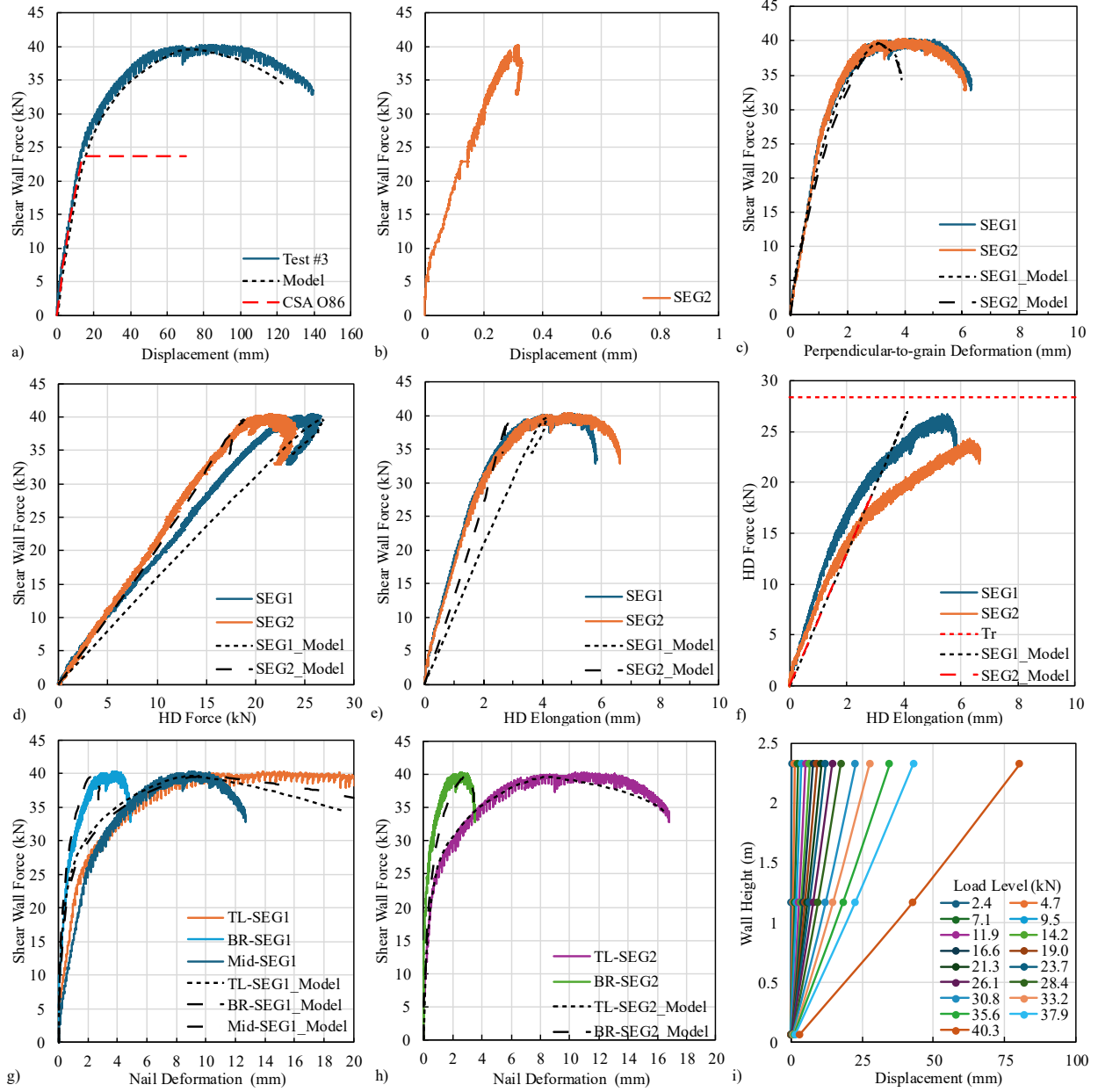
Figure A.5: Configuration #3 geometry

Table A-5: Shear wall design information (Configuration #3)

| Configuration | Shear Wall ID | $L_s$ (mm) | $V_{rs}$ (kN) | $V_d$ (kN) | (D/C) <sub>sw</sub> | $L_o$ (mm) | $T_r$ (kN) | $T_d$ (kN) | (D/C) <sub>HD</sub> |
|---------------|---------------|------------|---------------|------------|---------------------|------------|------------|------------|---------------------|
| #3            | 1             | 1500       | 11.85         | 11.5       | 1.0                 | 1351       | 28.4       | 25.68      | 0.9                 |
|               | 2             | 1500       | 11.85         | 11.85      | 1.0                 | 1351       | 28.4       | 25.68      | 0.9                 |

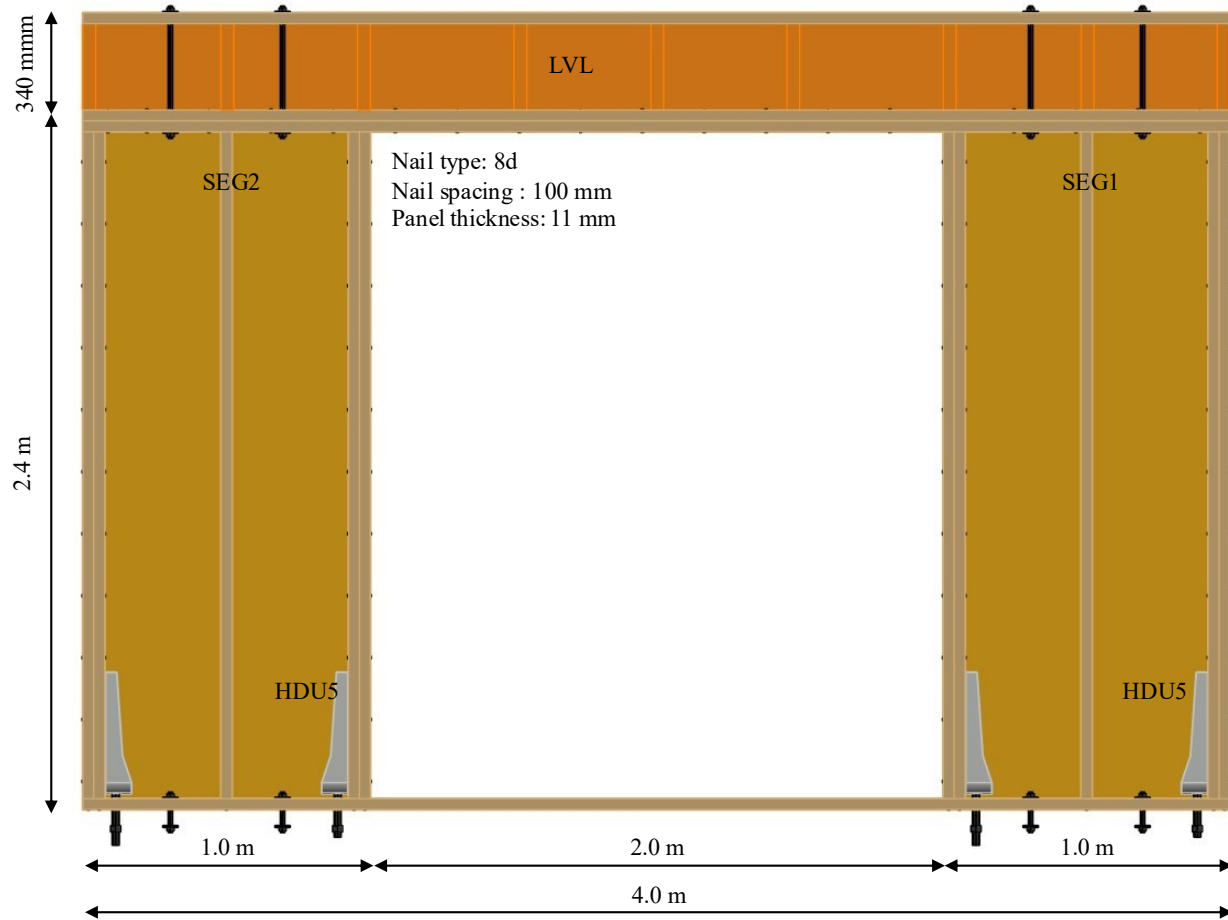
Table A-6: Shear wall design deflections (Configuration #3)

| Configuration | Shear wall ID | $e_n$ (mm) | $d_{a1}$ (mm) | $d_{a2}$ (mm) | $\Delta_b$ (mm) | $\Delta_s$ (mm) | $\Delta_n$ (mm) | $\Delta_{a1}$ (mm) | $\Delta_{a2}$ (mm) | $\Delta_t$ (mm) |
|---------------|---------------|------------|---------------|---------------|-----------------|-----------------|-----------------|--------------------|--------------------|-----------------|
| #3            | 1             | 0.86       | 2.94          | 0.40          | 0.8             | 1.70            | 5.25            | 5.31               | 0.72               | 13.8            |
|               | 2             | 0.86       | 2.94          | 0.40          | 0.8             | 1.75            | 5.25            | 5.31               | 0.72               | 13.8            |



**Figure A.6: Response quantities in Configuration #3: (a) shear wall F–D; (b) shear wall bottom plate slide; (c) perpendicular-to-grain compression deformations; (d) shear-wall force vs. hold-down force; (e) shear-wall force vs. hold-down elongation; (f) hold-down F–D; (g) and (h) nail deformations in SEG1 and SEG2, respectively; (i) chord lateral displacement profile**

#### A.4 Configuration #4



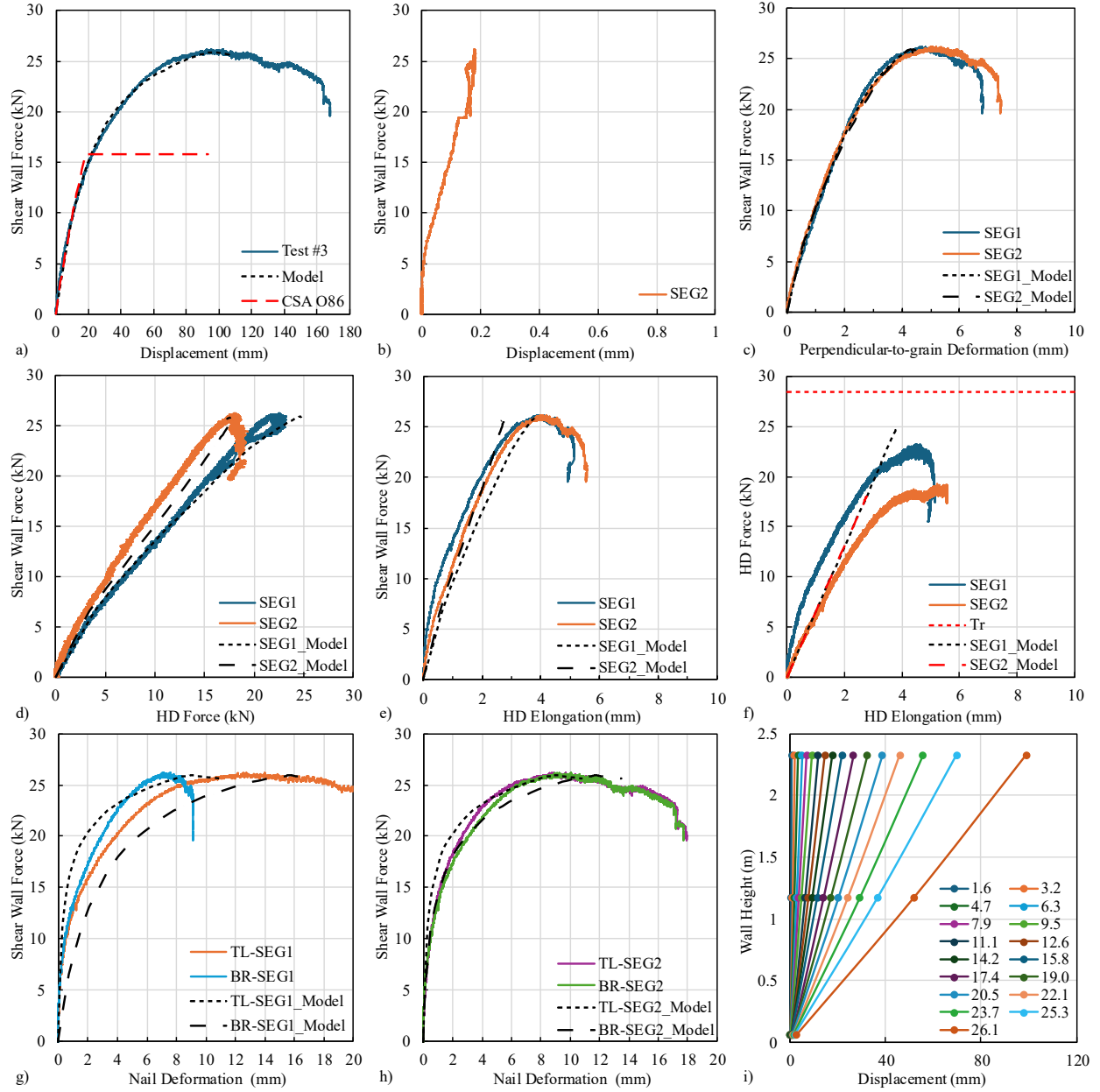
**Figure A.7: Configuration #4 geometry**

Table A-7: Shear wall design information (Configuration #4)

| Configuration | Shear Wall ID | $L_s$ (mm) | $V_{rs}$ (kN) | $V_d$ (kN) | (D/C) <sub>sw</sub> | $L_o$ (mm) | $T_r$ (kN) | $T_d$ (kN) | (D/C) <sub>HD</sub> |
|---------------|---------------|------------|---------------|------------|---------------------|------------|------------|------------|---------------------|
| #4            | 1             | 1000       | 7.9           | 7.9        | 1.0                 | 851        | 28.4       | 27.18      | 0.96                |
|               | 2             | 1000       | 7.9           | 7.9        | 1.0                 | 851        | 28.4       | 27.18      | 0.96                |

Table A-8: Shear wall design deflections (Configuration #4)

| Configuration | Shear wall ID | $e_n$ (mm) | $d_{a1}$ (mm) | $d_{a2}$ (mm) | $\Delta_b$ (mm) | $\Delta_s$ (mm) | $\Delta_n$ (mm) | $\Delta_{a1}$ (mm) | $\Delta_{a2}$ (mm) | $\Delta_t$ (mm) |
|---------------|---------------|------------|---------------|---------------|-----------------|-----------------|-----------------|--------------------|--------------------|-----------------|
| #4            | 1             | 0.86       | 3.11          | 0.42          | 1.19            | 1.75            | 5.25            | 8.92               | 1.20               | 18.3            |
|               | 2             | 0.86       | 3.11          | 0.42          | 1.19            | 1.75            | 5.25            | 8.92               | 1.20               | 18.3            |



**Figure A.8: Response quantities in Configuration #4: (a) shear wall F–D; (b) shear wall bottom plate slide; (c) perpendicular-to-grain compression deformations; (d) shear-wall force vs. hold-down force; (e) shear-wall force vs. hold-down elongation; (f) hold-down F–D; (g) and (h) nail deformations in SEG1 and SEG2, respectively; (i) chord lateral displacement profile**

### A.5 Configuration #5

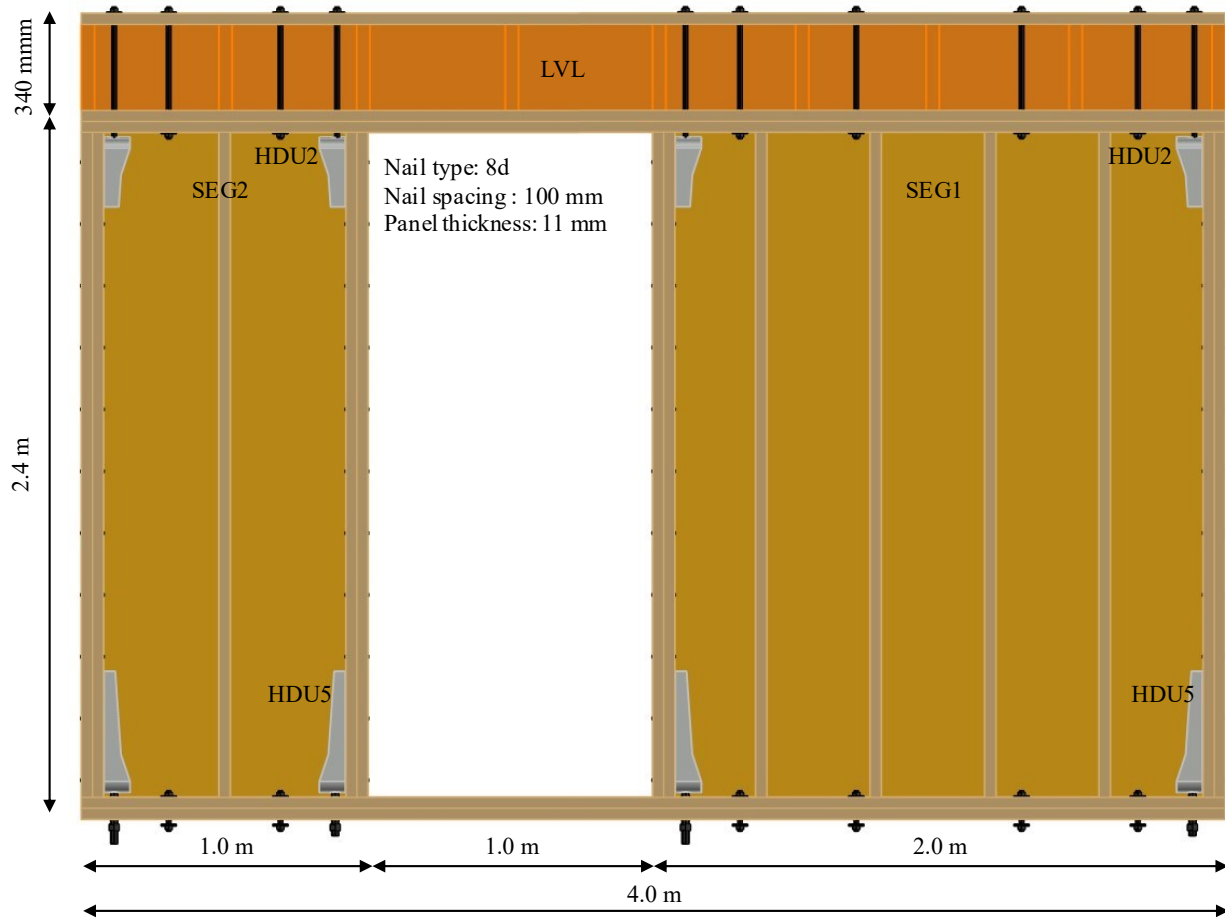


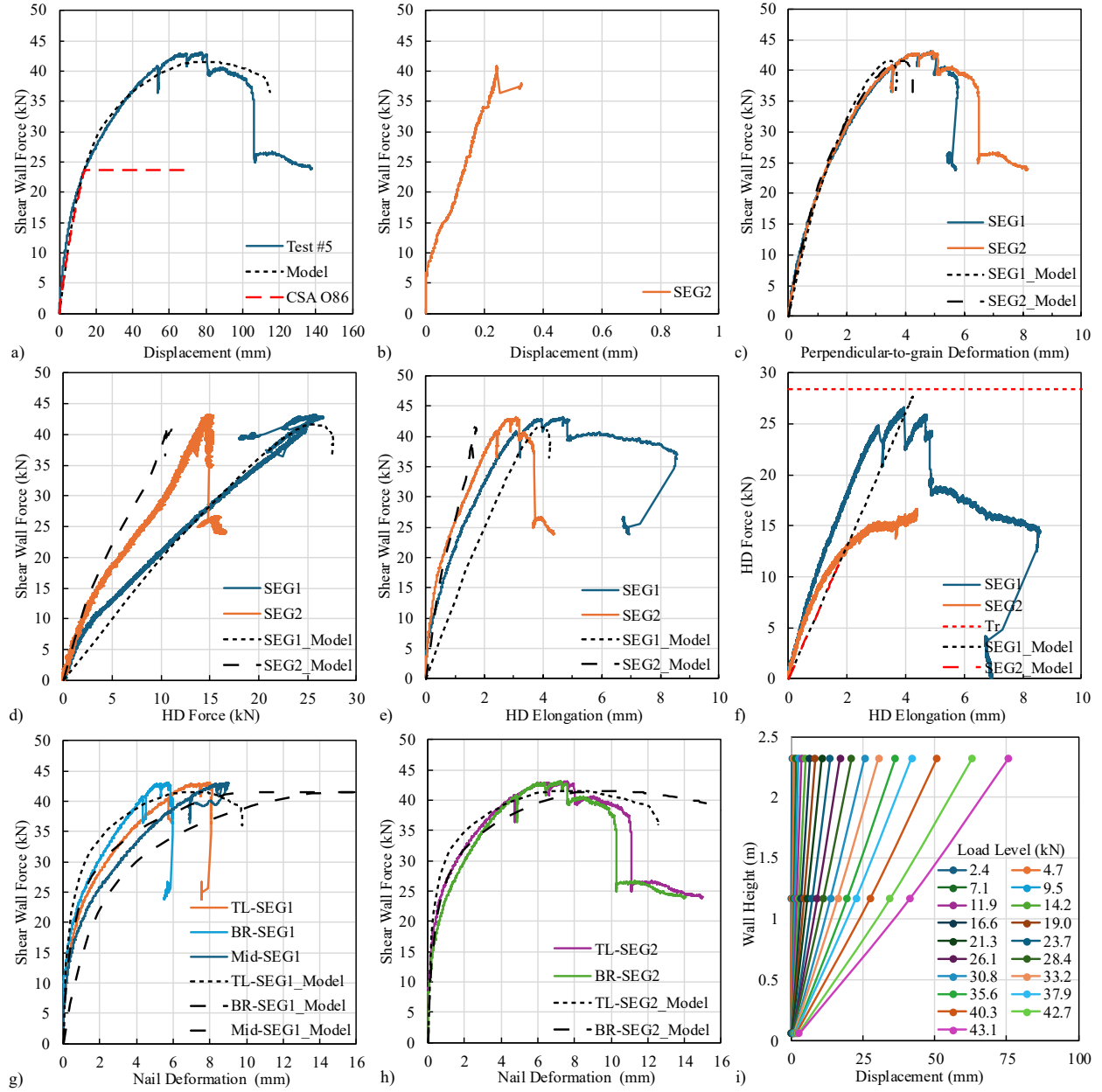
Figure A.9: Configuration #5 geometry

Table A-9: Shear wall design information (Configuration #5)

| Configuration | Shear Wall ID | $L_s$ (mm) | $V_{rs}$ (kN) | $V_d$ (kN) | (D/C) <sub>sw</sub> | $L_o$ (mm) | $T_r$ (kN) | $T_d$ (kN) | (D/C) <sub>HD</sub> |
|---------------|---------------|------------|---------------|------------|---------------------|------------|------------|------------|---------------------|
| #5            | 1             | 2000       | 15.8          | 17.4       | 1.1                 | 1851       | 28.4       | 27.5       | 0.97                |
|               | 2             | 1000       | 7.9           | 6.3        | 0.8                 | 851        | 28.4       | 21.7       | 0.76                |

Table A-10: Shear wall design deflections (Configuration #5)

| Configuration | Shear wall ID | $e_n$ (mm) | $d_{a1}$ (mm) | $d_{a2}$ (mm) | $\Delta_b$ (mm) | $\Delta_s$ (mm) | $\Delta_n$ (mm) | $\Delta_{a1}$ (mm) | $\Delta_{a2}$ (mm) | $\Delta_t$ (mm) |
|---------------|---------------|------------|---------------|---------------|-----------------|-----------------|-----------------|--------------------|--------------------|-----------------|
| #5            | 1             | 1.04       | 3.15          | 0.41          | 0.66            | 1.95            | 6.34            | 4.15               | 0.54               | 13.6            |
|               | 2             | 0.54       | 2.46          | 0.33          | 0.93            | 1.37            | 3.29            | 7.05               | 0.94               | 13.6            |



**Figure A.10: Response quantities in Configuration #5: (a) shear wall F–D; (b) shear wall bottom plate slide; (c) perpendicular-to-grain compression deformations; (d) shear-wall force vs. hold-down force; (e) shear-wall force vs. hold-down elongation; (f) hold-down F–D; (g) and (h) nail deformations in SEG1 and SEG2, respectively; (i) chord lateral displacement profile**

## A.6 Configuration #6

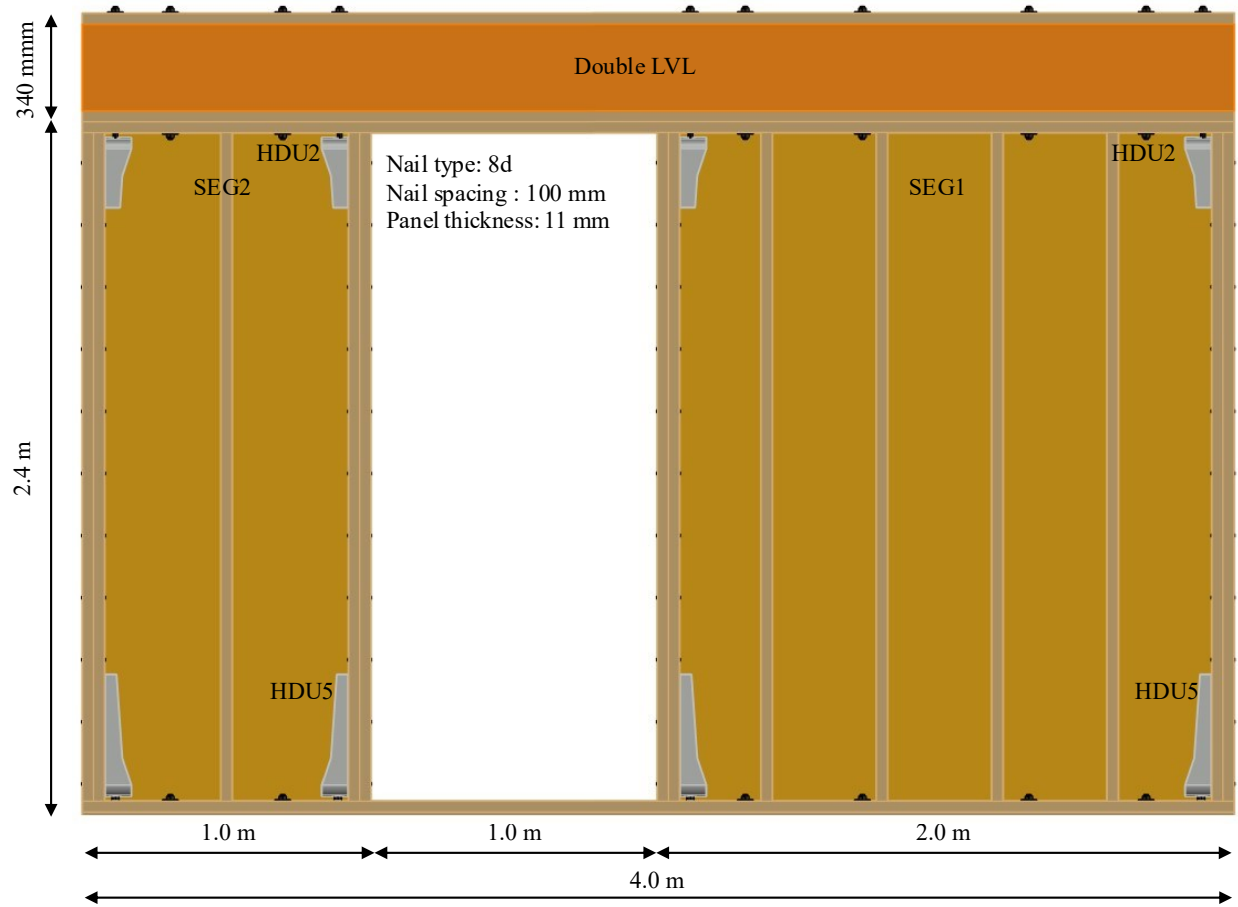


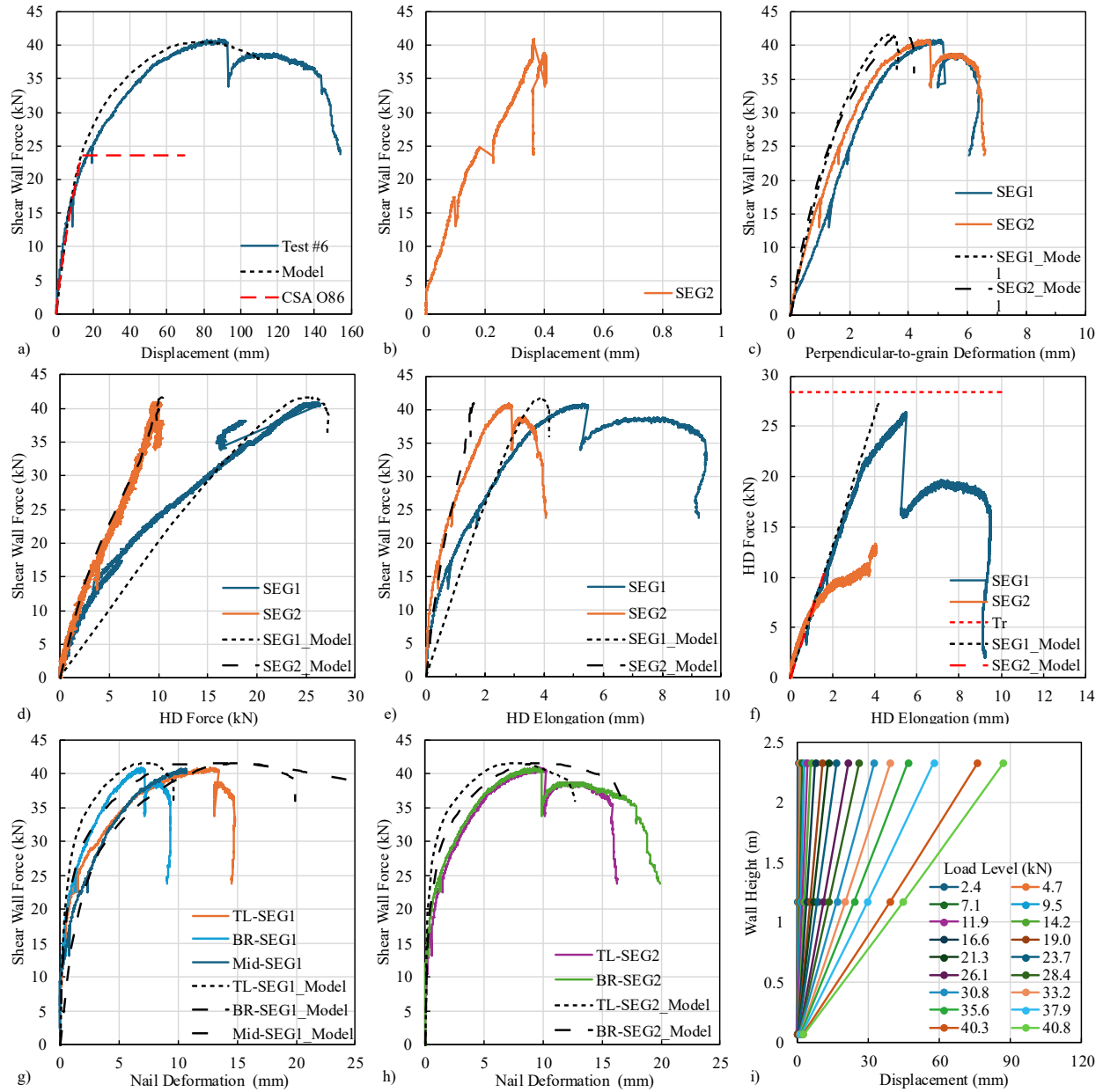
Figure A.11: Configuration #6 geometry

Table A-11: Shear wall design information (Configuration #6)

| Configuration | Shear Wall ID | $L_s$ (mm) | $V_{rs}$ (kN) | $V_d$ (kN) | (D/C) <sub>sw</sub> | $L_o$ (mm) | $T_r$ (kN) | $T_d$ (kN) | (D/C) <sub>HD</sub> |
|---------------|---------------|------------|---------------|------------|---------------------|------------|------------|------------|---------------------|
| #6            | 1             | 2000       | 15.8          | 17.4       | 1.1                 | 1851       | 28.4       | 27.5       | 0.97                |
|               | 2             | 1000       | 7.9           | 6.3        | 0.8                 | 851        | 28.4       | 21.7       | 0.76                |

Table A-12: Shear wall design deflections (Configuration #6)

| Configuration | Shear wall ID | $e_n$ (mm) | $d_{a1}$ (mm) | $d_{a2}$ (mm) | $\Delta_b$ (mm) | $\Delta_s$ (mm) | $\Delta_n$ (mm) | $\Delta_{a1}$ (mm) | $\Delta_{a2}$ (mm) | $\Delta_t$ (mm) |
|---------------|---------------|------------|---------------|---------------|-----------------|-----------------|-----------------|--------------------|--------------------|-----------------|
| #6            | 1             | 1.04       | 3.15          | 0.41          | 0.66            | 1.95            | 6.34            | 4.15               | 0.54               | 13.6            |
|               | 2             | 0.54       | 2.46          | 0.33          | 0.93            | 1.37            | 3.29            | 7.05               | 0.94               | 13.6            |



**Figure A.12: Response quantities in Configuration #6: (a) shear wall F–D; (b) shear wall bottom plate slide; (c) perpendicular-to-grain compression deformations; (d) shear-wall force vs. hold-down force; (e) shear-wall force vs. hold-down elongation; (f) hold-down F–D; (g) and (h) nail deformations in SEG1 and SEG2, respectively; (i) chord lateral displacement profile**



### A.7 Configuration #7

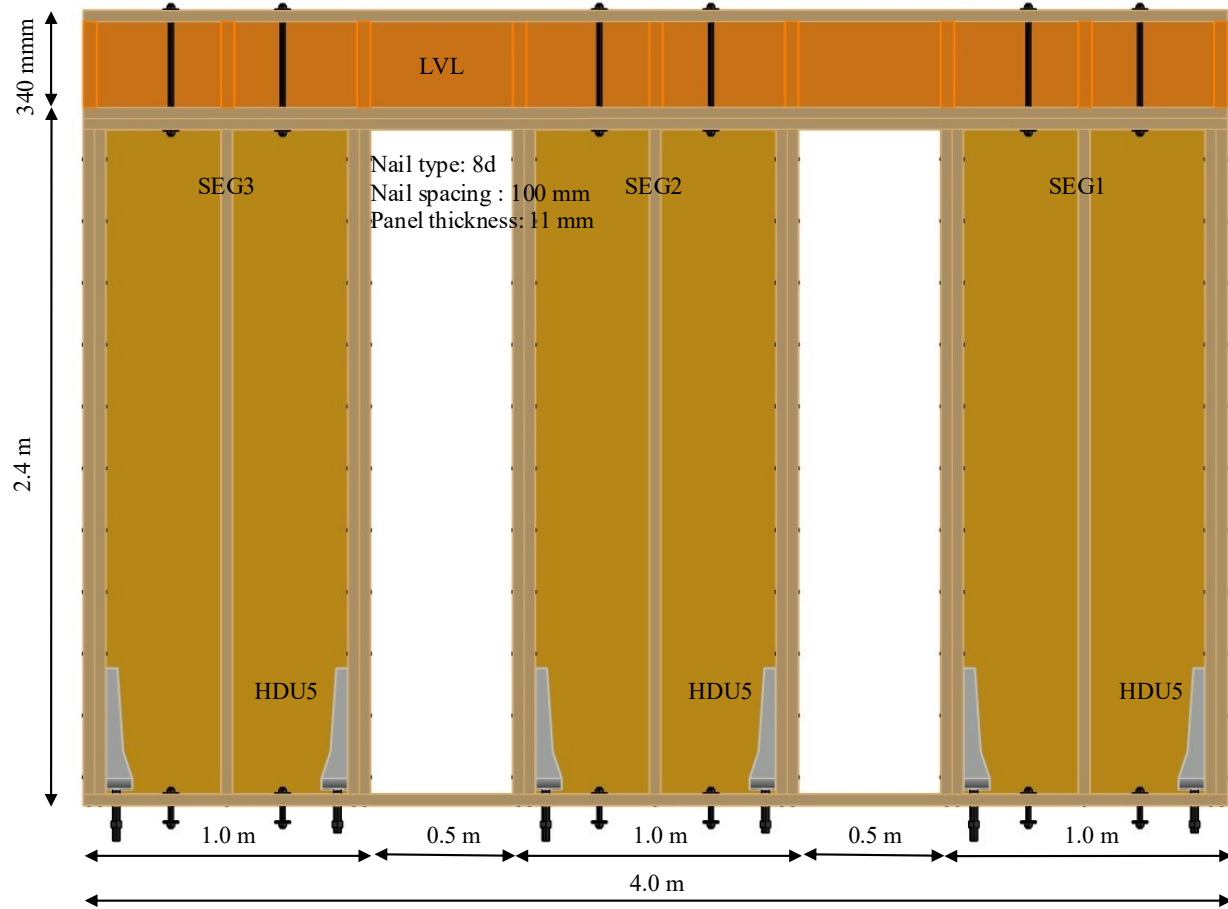


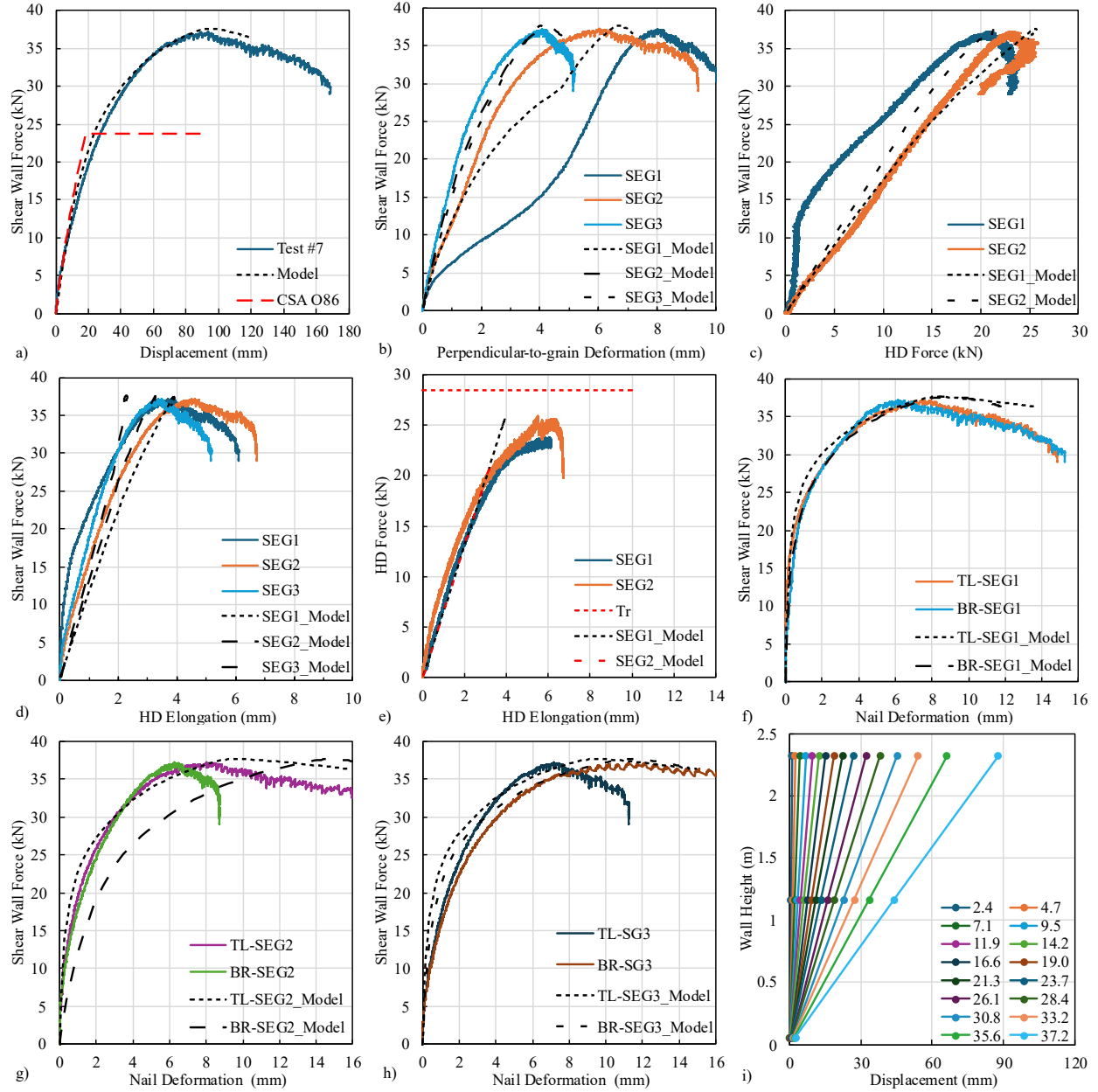
Figure A.13: Configuration #7 geometry

Table A-13: Shear wall design information (Configuration #7)

| Configuration | Shear Wall ID | $L_s$ (mm) | $V_{rs}$ (kN) | $V_d$ (kN) | (D/C) <sub>sw</sub> | $L_o$ (mm) | $T_r$ (kN) | $T_d$ (kN) | (D/C) <sub>HD</sub> |
|---------------|---------------|------------|---------------|------------|---------------------|------------|------------|------------|---------------------|
| #7            | 1             | 1000       | 7.9           | 7.9        | 1.0                 | 851        | 28.4       | 27.18      | 0.96                |
|               | 2             | 1000       | 7.9           | 7.9        | 1.0                 | 851        | 28.4       | 27.18      | 0.96                |
|               | 3             | 1000       | 7.9           | 7.9        | 1.0                 | 851        | 28.4       | 27.18      | 0.96                |

Table A-14: Shear wall design deflections (Configuration #7)

| Configuration | Shear wall ID | $e_n$ (mm) | $d_{a1}$ (mm) | $d_{a2}$ (mm) | $\Delta_b$ (mm) | $\Delta_s$ (mm) | $\Delta_n$ (mm) | $\Delta_{a1}$ (mm) | $\Delta_{a2}$ (mm) | $\Delta_t$ (mm) |
|---------------|---------------|------------|---------------|---------------|-----------------|-----------------|-----------------|--------------------|--------------------|-----------------|
| #7            | 1             | 0.86       | 3.11          | 0.42          | 1.19            | 1.75            | 5.25            | 8.92               | 1.20               | 18.3            |
|               | 2             | 0.86       | 3.11          | 0.42          | 1.19            | 1.75            | 5.25            | 8.92               | 1.20               | 18.3            |
|               | 3             | 0.86       | 3.11          | 0.42          | 1.19            | 1.75            | 5.25            | 8.92               | 1.20               | 18.3            |



**Figure A.14: Response quantities in Configuration #7: (a) shear wall F–D; (b) perpendicular-to-grain compression deformations; (c) shear-wall force vs. hold-down force; (d) shear-wall force vs. hold-down elongation; (e) hold-down F–D; (f) and (h) nail deformations in SEG1, SEG2, and SEG3, respectively; (i) chord lateral displacement profile**

## A.8 Configuration #8

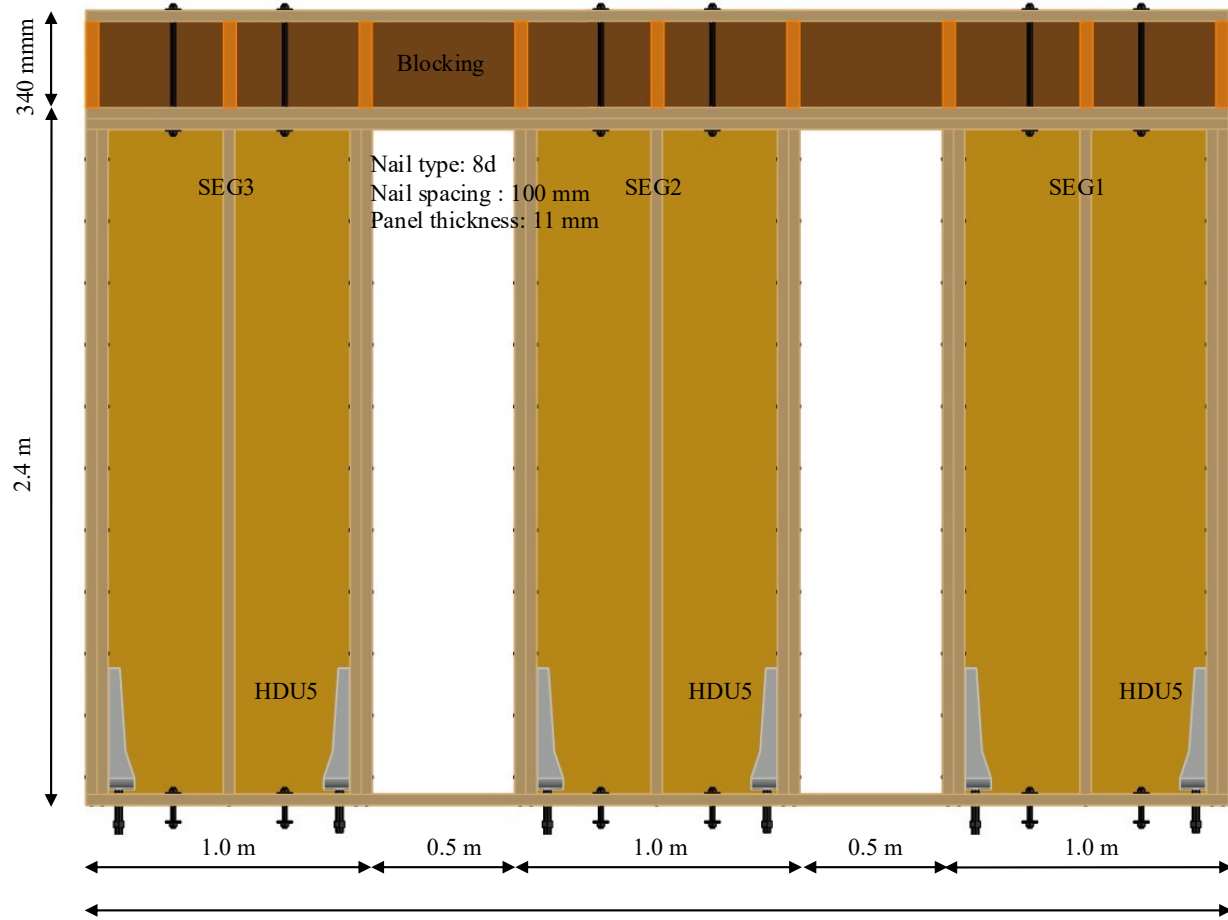


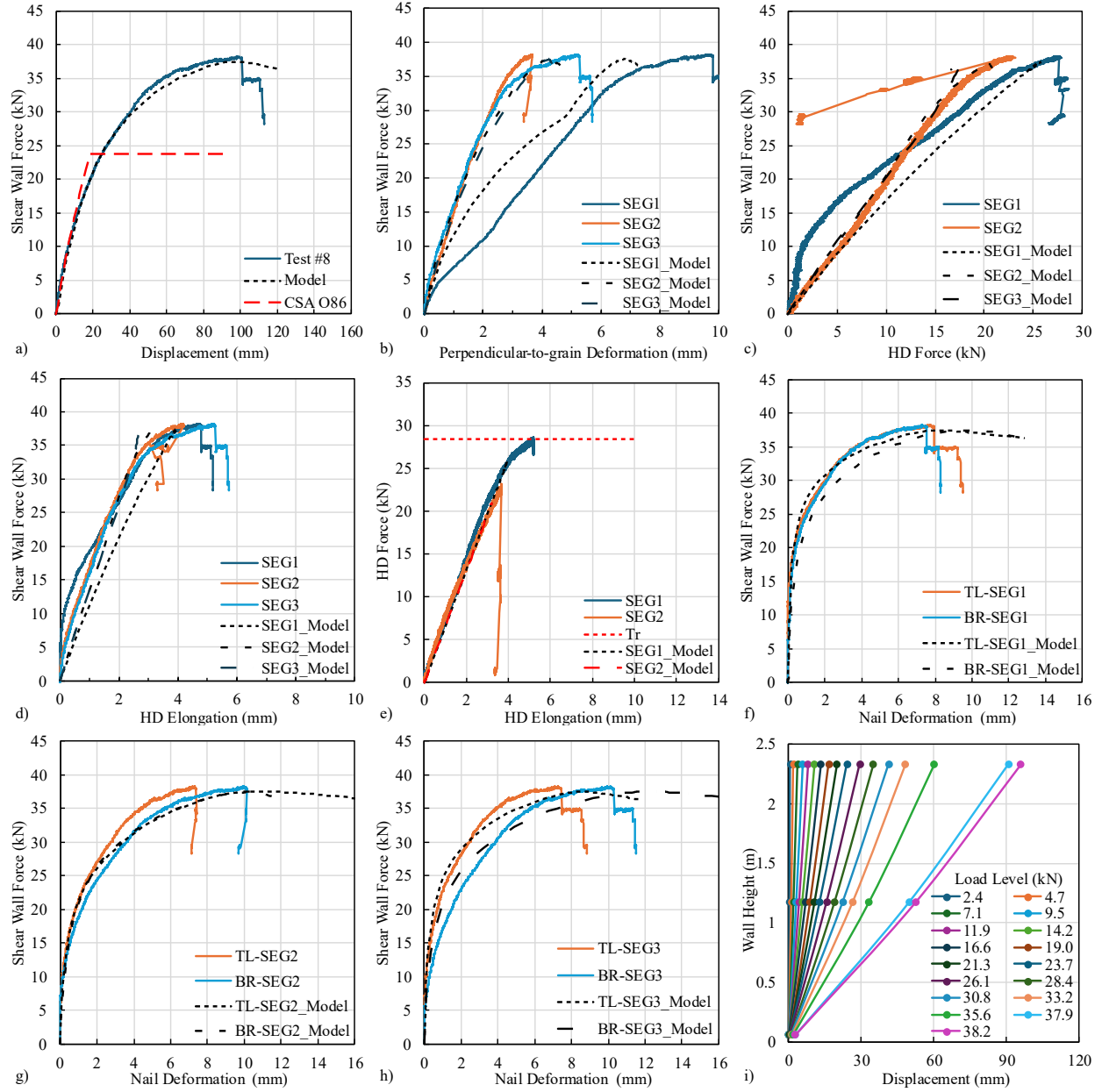
Figure A.15: Configuration #8 geometry

Table A-15: Shear wall design information (Configuration #8)

| Configuration | Shear Wall ID | $L_s$ (mm) | $V_{rs}$ (kN) | $V_d$ (kN) | (D/C) <sub>sw</sub> | $L_o$ (mm) | $T_r$ (kN) | $T_d$ (kN) | (D/C) <sub>HD</sub> |
|---------------|---------------|------------|---------------|------------|---------------------|------------|------------|------------|---------------------|
| #8            | 1             | 1000       | 7.9           | 7.9        | 1.0                 | 851        | 28.4       | 27.18      | 0.96                |
|               | 2             | 1000       | 7.9           | 7.9        | 1.0                 | 851        | 28.4       | 27.18      | 0.96                |
|               | 3             | 1000       | 7.9           | 7.9        | 1.0                 | 851        | 28.4       | 27.18      | 0.96                |

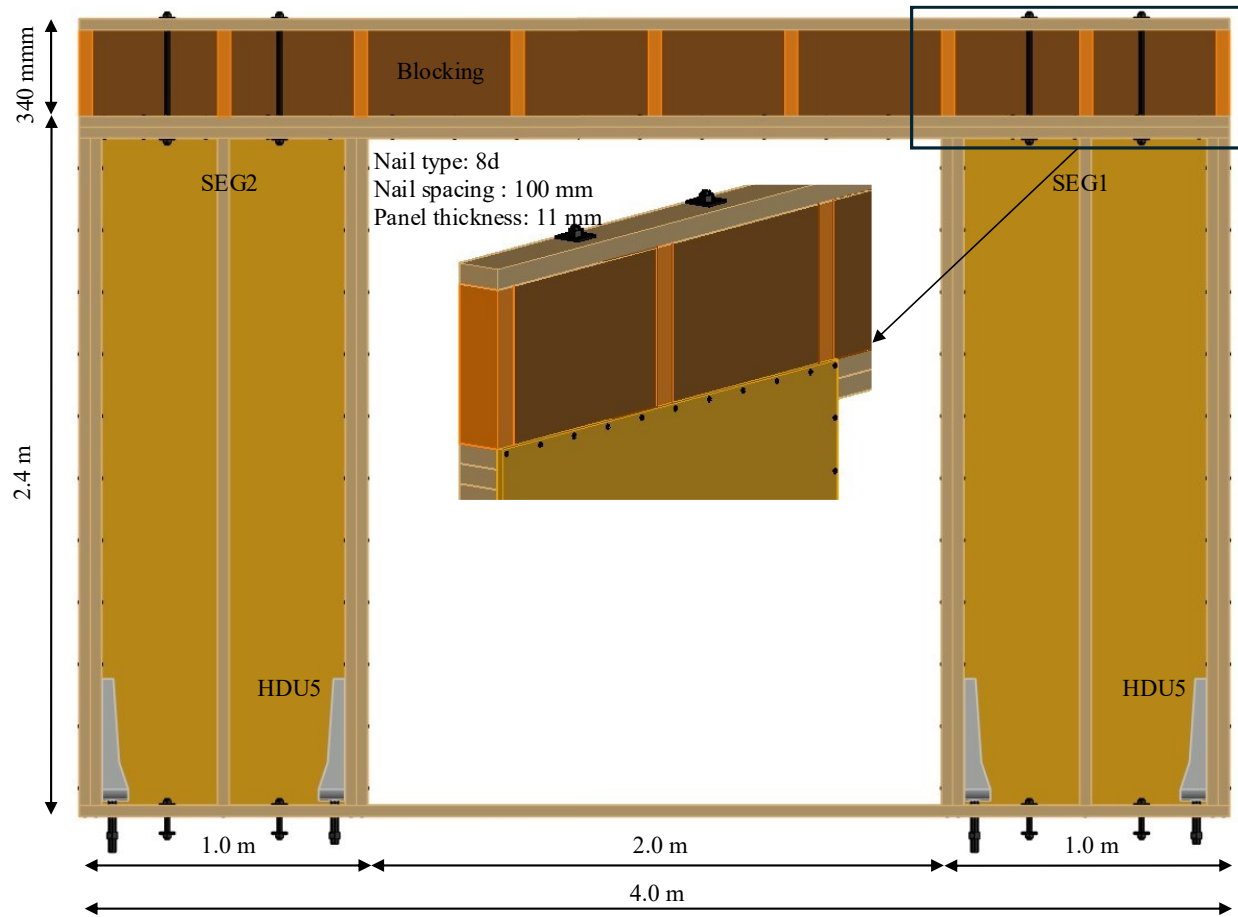
Table A-16: Shear wall design deflections (Configuration #8)

| Configuration | Shear wall ID | $e_n$ (mm) | $d_{a1}$ (mm) | $d_{a2}$ (mm) | $\Delta_b$ (mm) | $\Delta_s$ (mm) | $\Delta_n$ (mm) | $\Delta_{a1}$ (mm) | $\Delta_{a2}$ (mm) | $\Delta_t$ (mm) |
|---------------|---------------|------------|---------------|---------------|-----------------|-----------------|-----------------|--------------------|--------------------|-----------------|
| #8            | 1             | 0.86       | 3.11          | 0.42          | 1.19            | 1.75            | 5.25            | 8.92               | 1.20               | 18.3            |
|               | 2             | 0.86       | 3.11          | 0.42          | 1.19            | 1.75            | 5.25            | 8.92               | 1.20               | 18.3            |
|               | 3             | 0.86       | 3.11          | 0.42          | 1.19            | 1.75            | 5.25            | 8.92               | 1.20               | 18.3            |



**Figure A.16: Response quantities in Configuration #8: (a) shear wall F–D; (b) perpendicular-to-grain compression deformations; (c) shear-wall force vs. hold-down force; (d) shear-wall force vs. hold-down elongation; (e) hold-down F–D; (f) and (h) nail deformations in SEG1, SEG2, and SEG3, respectively; (i) chord lateral displacement profile**

### A.9 Configuration #9



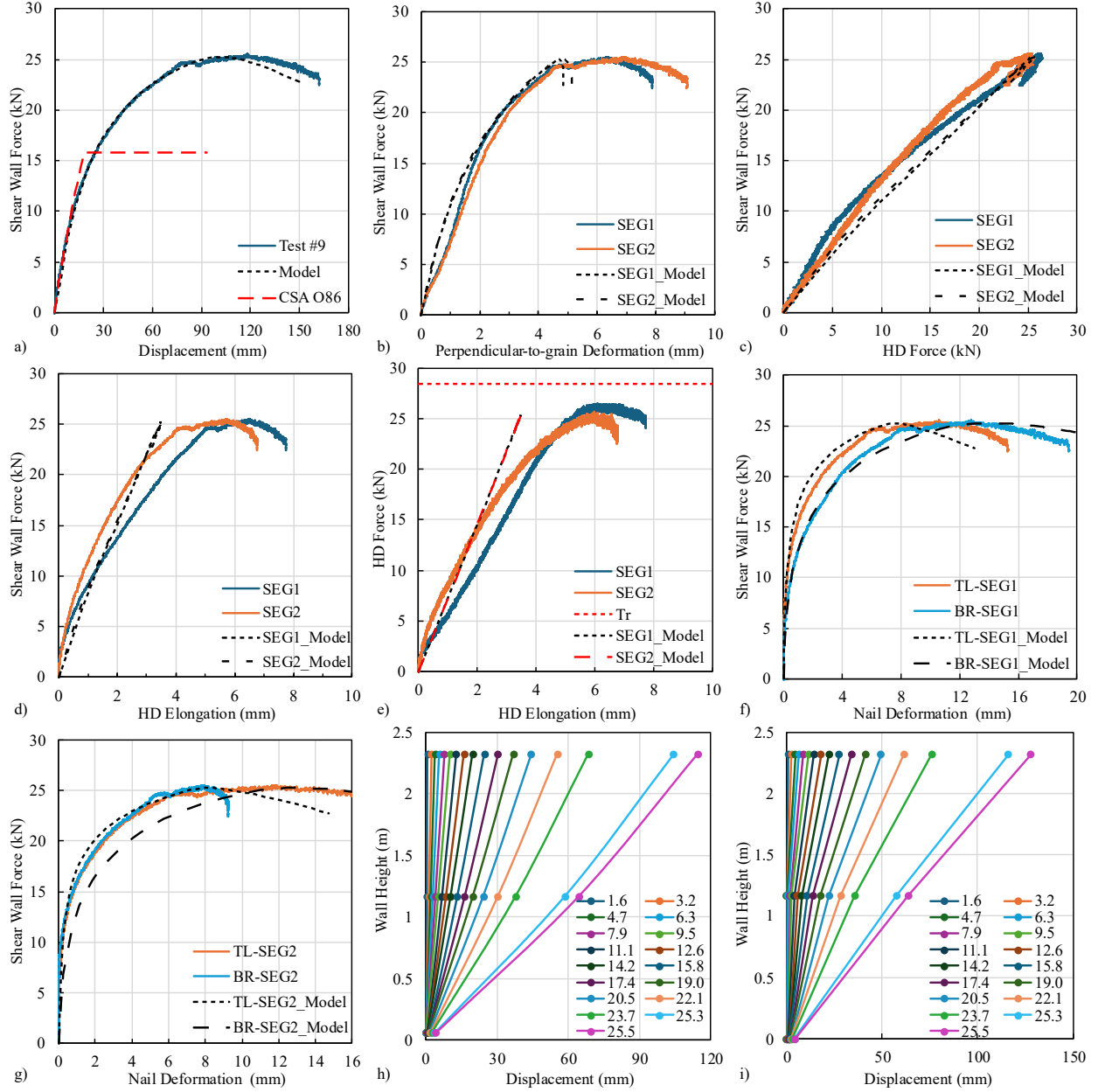
**Figure A.17: Configuration #9 geometry**

**Table A-17: Shear wall design information (Configuration #9)**

| Configuration | Shear Wall ID | $L_s$ (mm) | $V_{rs}$ (kN) | $V_d$ (kN) | (D/C) <sub>sw</sub> | $L_o$ (mm) | $T_r$ (kN) | $T_d$ (kN) | (D/C) <sub>HD</sub> |
|---------------|---------------|------------|---------------|------------|---------------------|------------|------------|------------|---------------------|
| #9            | 1             | 1000       | 7.9           | 7.9        | 1.0                 | 851        | 28.4       | 27.18      | 0.96                |
|               | 2             | 1000       | 7.9           | 7.9        | 1.0                 | 851        | 28.4       | 27.18      | 0.96                |

**Table A-18: Shear wall design deflections (Configuration #9)**

| Configuration | Shear wall ID | $e_n$ (mm) | $d_{a1}$ (mm) | $d_{a2}$ (mm) | $\Delta_b$ (mm) | $\Delta_s$ (mm) | $\Delta_n$ (mm) | $\Delta_{a1}$ (mm) | $\Delta_{a2}$ (mm) | $\Delta_t$ (mm) |
|---------------|---------------|------------|---------------|---------------|-----------------|-----------------|-----------------|--------------------|--------------------|-----------------|
| #9            | 1             | 0.86       | 3.11          | 0.42          | 1.19            | 1.75            | 5.25            | 8.92               | 1.20               | 18.3            |
|               | 2             | 0.86       | 3.11          | 0.42          | 1.19            | 1.75            | 5.25            | 8.92               | 1.20               | 18.3            |



**Figure A.18: Response quantities in Configuration #8: (a) shear wall F–D; (b) perpendicular-to-grain compression deformations; (c) shear-wall force vs. hold-down force; (d) shear-wall force vs. hold-down elongation; (e) hold-down F–D; (f) and (g) nail deformations in SEG1 and SEG2, respectively; (h) and (i) chord lateral displacement profile in SEG1 (compression chord) and SEG2 (tension chord), respectively**

### A.10 Configuration #10

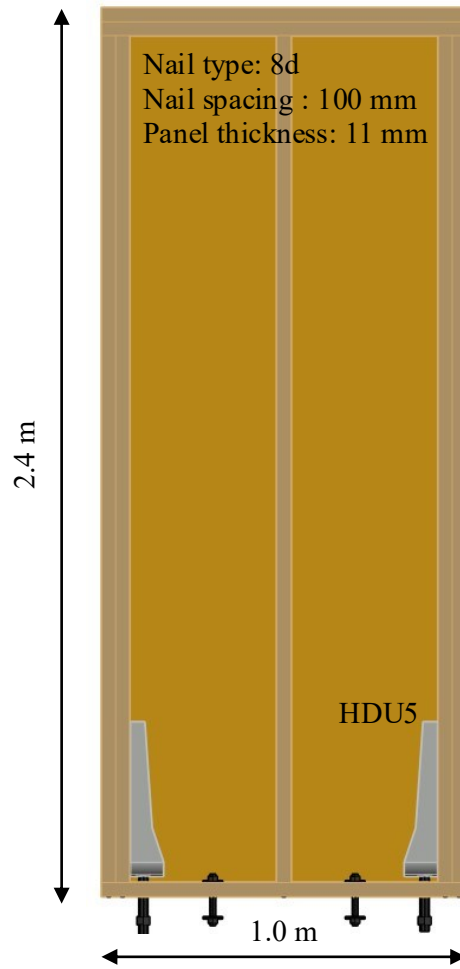


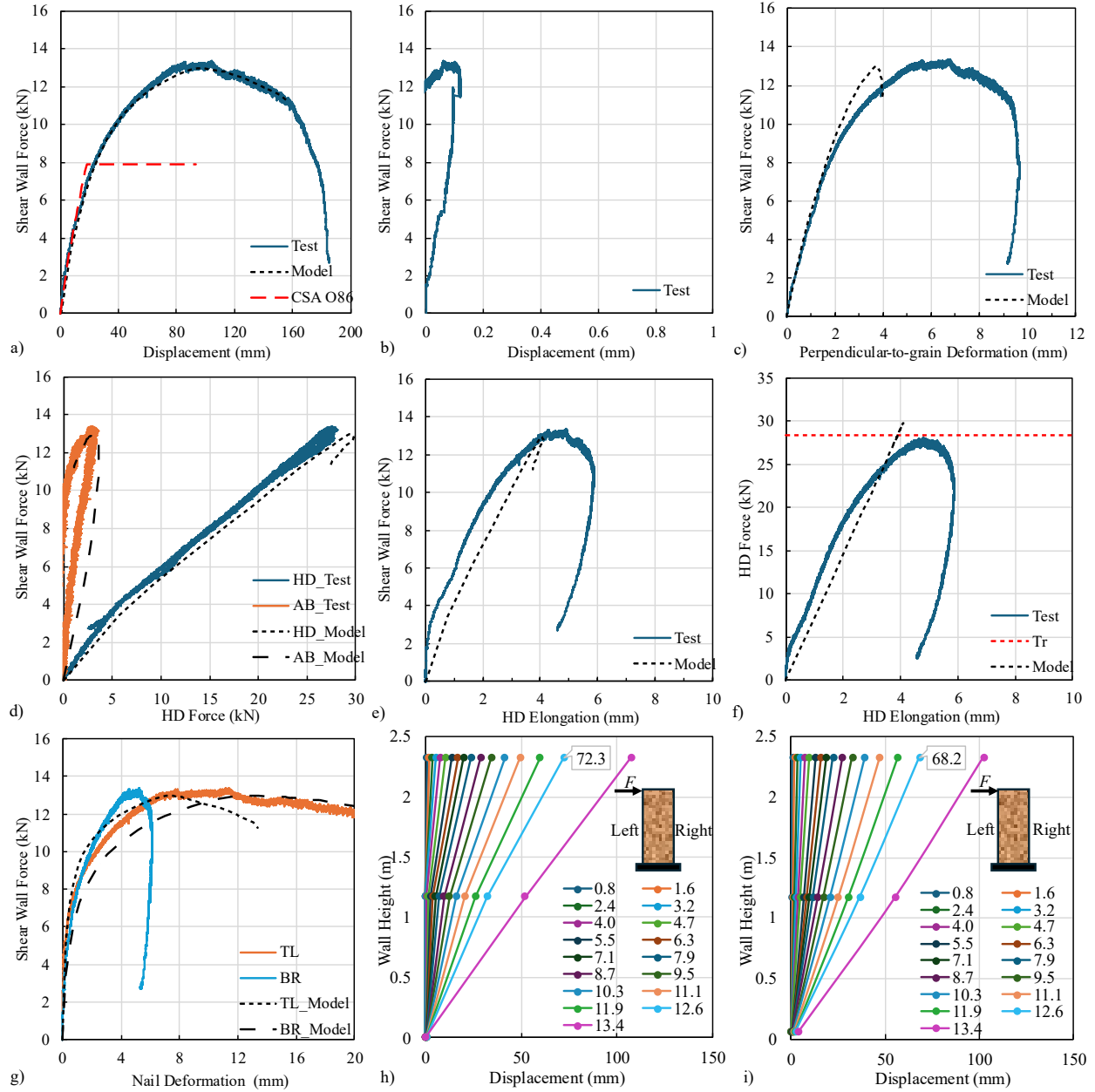
Figure A.19: Configuration #10 geometry

Table A-19: Shear wall design information (Configuration #10)

| Configuration | Shear Wall ID | $L_s$ (mm) | $V_{rs}$ (kN) | $V_d$ (kN) | (D/C) <sub>sw</sub> | $L_o$ (mm) | $T_r$ (kN) | $T_d$ (kN) | (D/C) <sub>HD</sub> |
|---------------|---------------|------------|---------------|------------|---------------------|------------|------------|------------|---------------------|
| #10           | 1             | 1000       | 7.9           | 7.9        | 1.0                 | 851        | 28.4       | 27.18      | 0.96                |

Table A-20: Shear wall design deflections (Configuration #10)

| Configuration | Shear wall ID | $e_n$ (mm) | $d_{a1}$ (mm) | $d_{a2}$ (mm) | $\Delta_b$ (mm) | $\Delta_s$ (mm) | $\Delta_n$ (mm) | $\Delta_{a1}$ (mm) | $\Delta_{a2}$ (mm) | $\Delta_t$ (mm) |
|---------------|---------------|------------|---------------|---------------|-----------------|-----------------|-----------------|--------------------|--------------------|-----------------|
| #10           | 1             | 0.86       | 3.11          | 0.42          | 1.19            | 1.75            | 5.25            | 8.92               | 1.20               | 18.3            |



**Figure A.20: Response quantities in Configuration #10: (a) shear wall F–D; (b) shear wall bottom plate slide; (c) perpendicular-to-grain compression deformations; (d) shear-wall force vs. hold-down and anchor bolt force; (e) shear-wall force vs. hold-down elongation; (f) hold-down F–D; (g) nail deformations; (h) and (i) tension and compression chords lateral displacements, respectively**



### A.11 Configuration #11

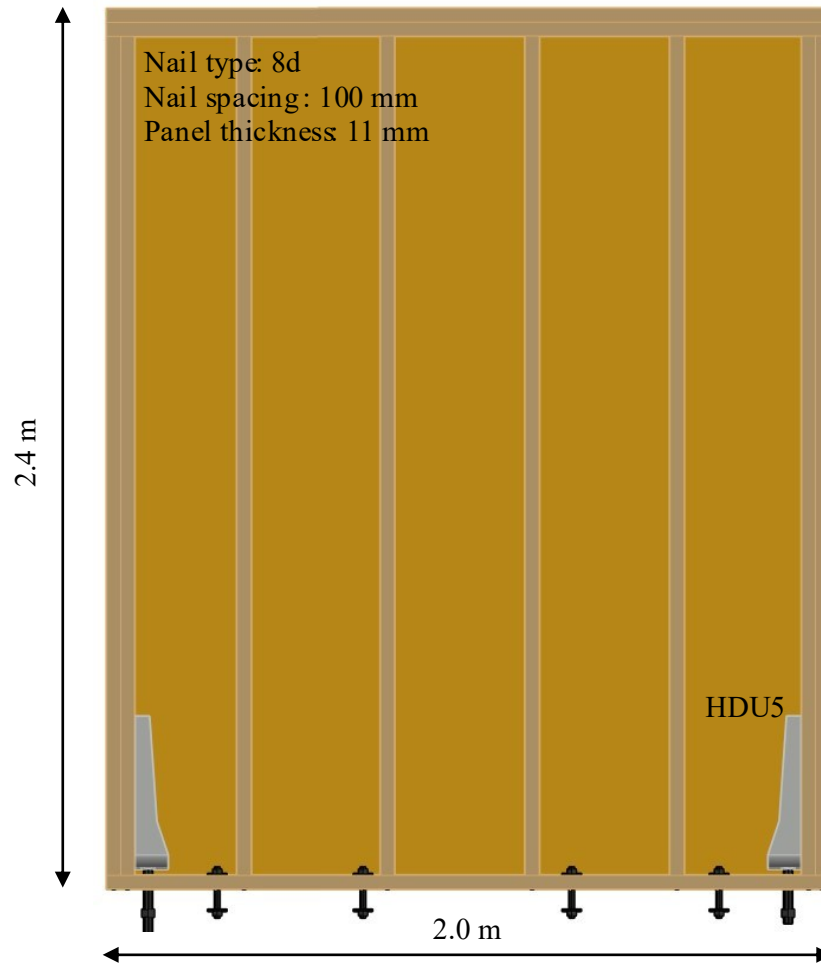


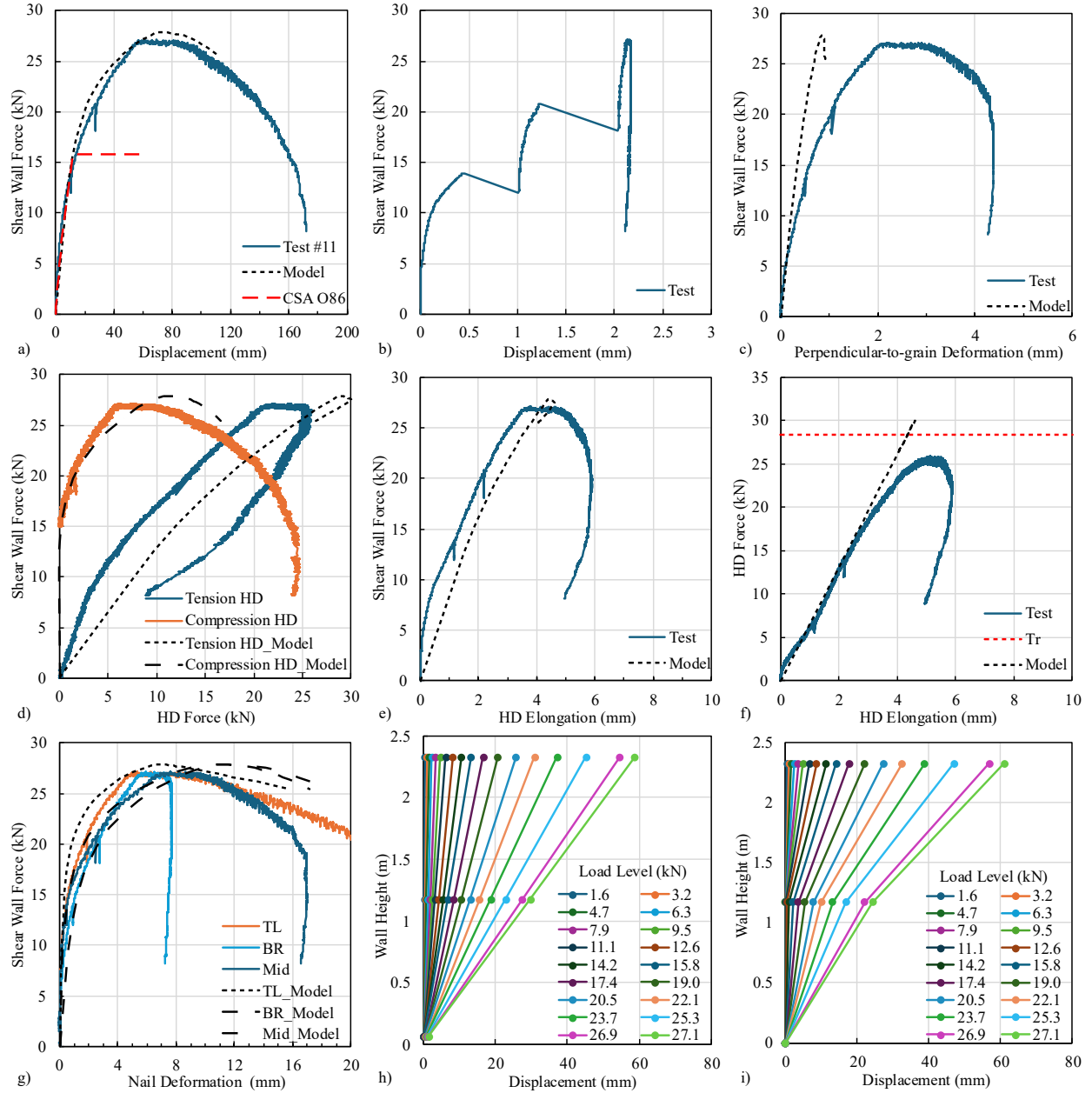
Figure A.21: Configuration #11 geometry

Table A-21: Shear wall design information (Configuration #11)

| Configuration | Shear Wall ID | $L_s$ (mm) | $V_{rs}$ (kN) | $V_d$ (kN) | (D/C) <sub>sw</sub> | $L_o$ (mm) | $T_r$ (kN) | $T_d$ (kN) | (D/C) <sub>HD</sub> |
|---------------|---------------|------------|---------------|------------|---------------------|------------|------------|------------|---------------------|
| #11           | 1             | 2000       | 15.8          | 15.8       | 1.0                 | 1851       | 28.4       | 25.0       | 0.88                |

Table A-22: Shear wall design deflections (Configuration #11)

| Configuration | Shear wall ID | $e_n$ (mm) | $d_{a1}$ (mm) | $d_{a2}$ (mm) | $\Delta_b$ (mm) | $\Delta_s$ (mm) | $\Delta_n$ (mm) | $\Delta_{a1}$ (mm) | $\Delta_{a2}$ (mm) | $\Delta_t$ (mm) |
|---------------|---------------|------------|---------------|---------------|-----------------|-----------------|-----------------|--------------------|--------------------|-----------------|
| #11           | 1             | 0.86       | 2.86          | 0.38          | 0.61            | 1.75            | 5.25            | 3.77               | 0.50               | 11.9            |



**Figure A.22: Response quantities in Configuration #11: (a) shear wall F–D; (b) shear wall bottom plate slide; (c) perpendicular-to-grain compression deformations; (d) shear-wall force vs. tension and compression hold-down force; (e) shear-wall force vs. hold-down elongation; (f) hold-down F–D; (g) nail deformations; (h) and (i) compression and tension chords lateral displacements, respectively**

### A.12 Configuration #12

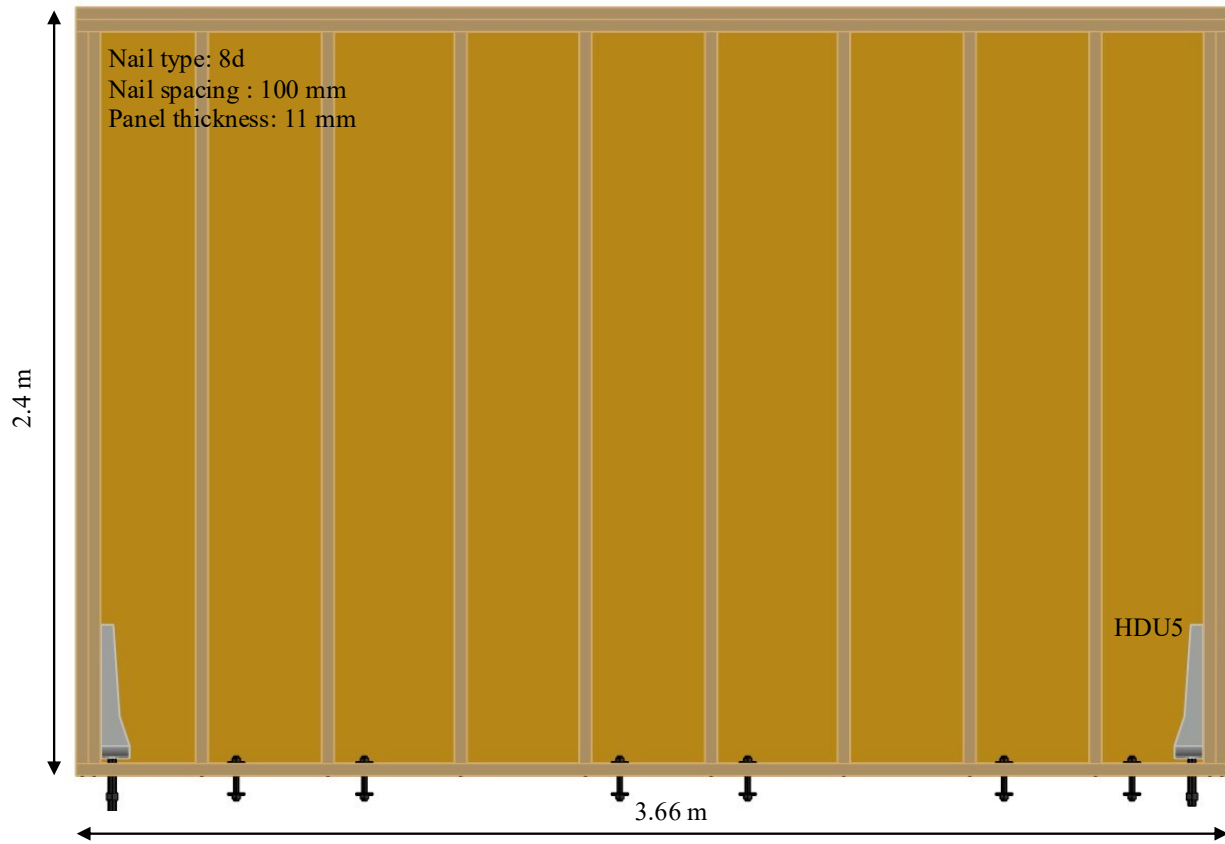


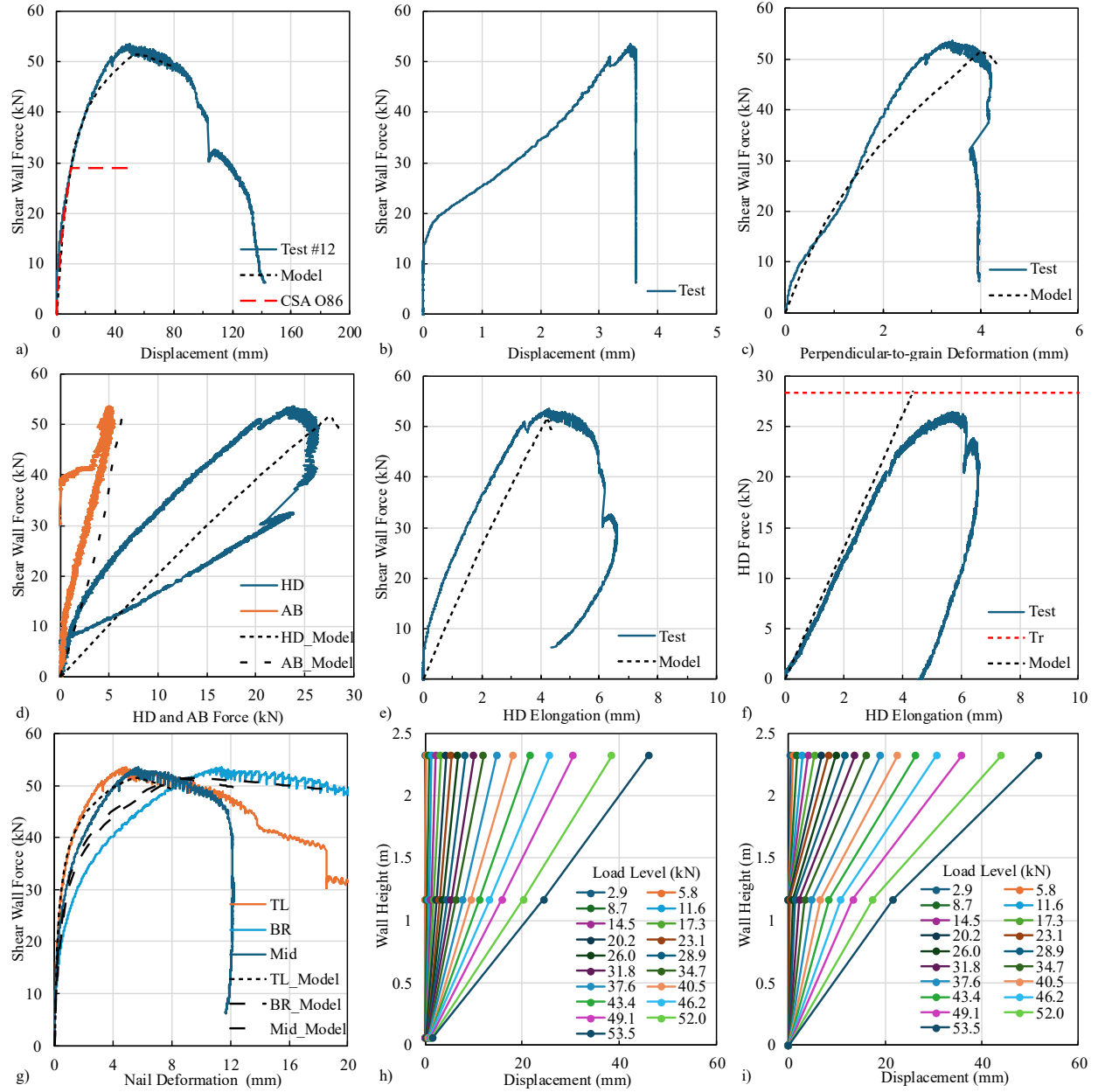
Figure A.23: Configuration #12 geometry

Table A-23: Shear wall design information (Configuration #12)

| Configuration | Shear Wall ID | $L_s$ (mm) | $V_{rs}$ (kN) | $V_d$ (kN) | (D/C) <sub>sw</sub> | $L_o$ (mm) | $T_r$ (kN) | $T_d$ (kN) | (D/C) <sub>HD</sub> |
|---------------|---------------|------------|---------------|------------|---------------------|------------|------------|------------|---------------------|
| #12           | 1             | 3660       | 28.9          | 28.9       | 1.0                 | 3510       | 28.4       | 24.1       | 0.85                |

Table A-24: Shear wall design deflections (Configuration #12)

| Configuration | Shear wall ID | $e_n$ (mm) | $d_{a1}$ (mm) | $d_{a2}$ (mm) | $\Delta_b$ (mm) | $\Delta_s$ (mm) | $\Delta_n$ (mm) | $\Delta_{a1}$ (mm) | $\Delta_{a2}$ (mm) | $\Delta_t$ (mm) |
|---------------|---------------|------------|---------------|---------------|-----------------|-----------------|-----------------|--------------------|--------------------|-----------------|
| #12           | 1             | 0.86       | 2.76          | 0.37          | 0.32            | 1.75            | 5.25            | 1.92               | 0.26               | 9.5             |



**Figure A.24: Response quantities in Configuration #12: (a) shear wall F–D; (b) shear wall bottom plate slide; (c) perpendicular-to-grain compression deformations; (d) shear-wall force vs. hold-down and anchor bolt force; (e) shear-wall force vs. hold-down elongation; (f) hold-down F–D; (g) nail deformations; (h) and (i) tension and compression chords lateral displacements, respectively**

A.13 Configuration #13

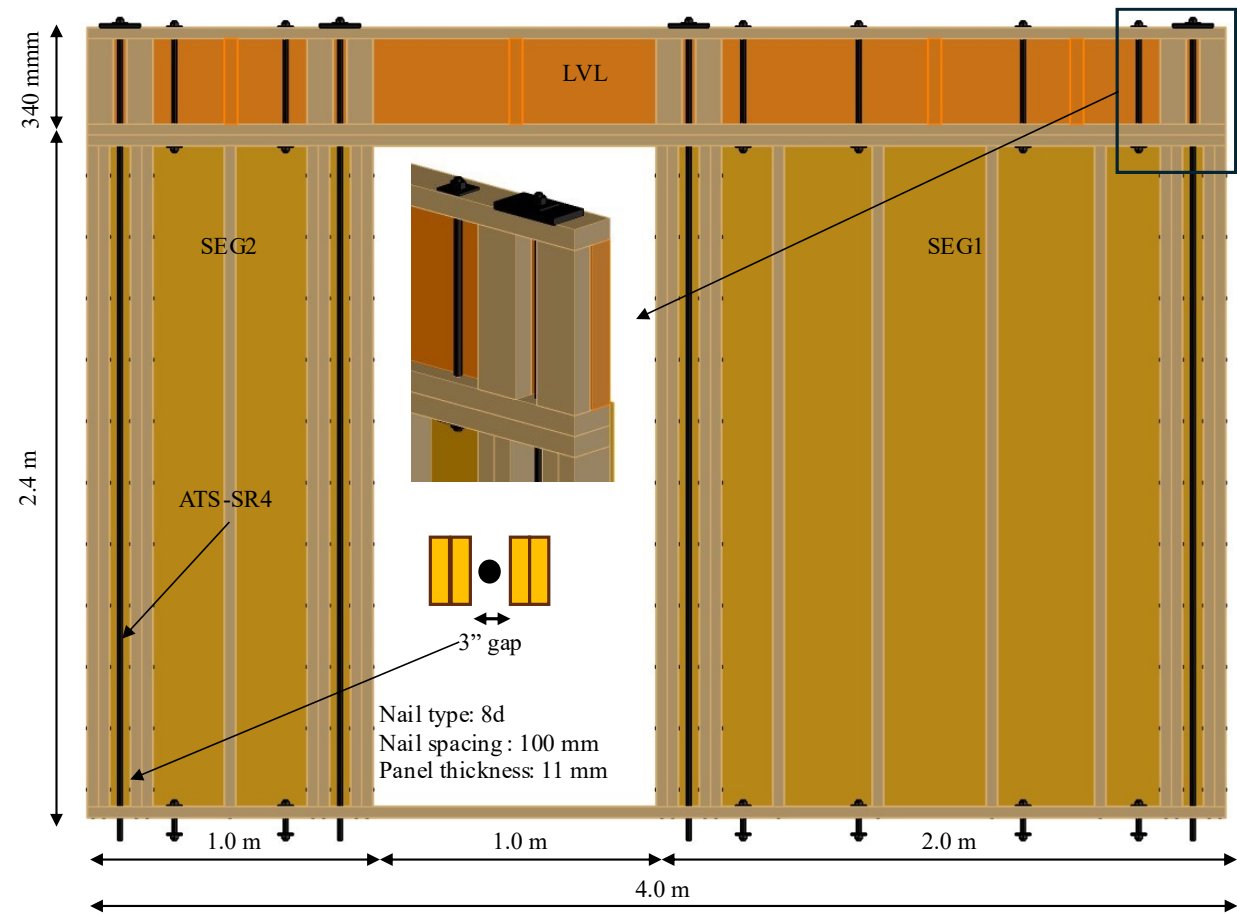


Figure A.25: Configuration #13 geometry

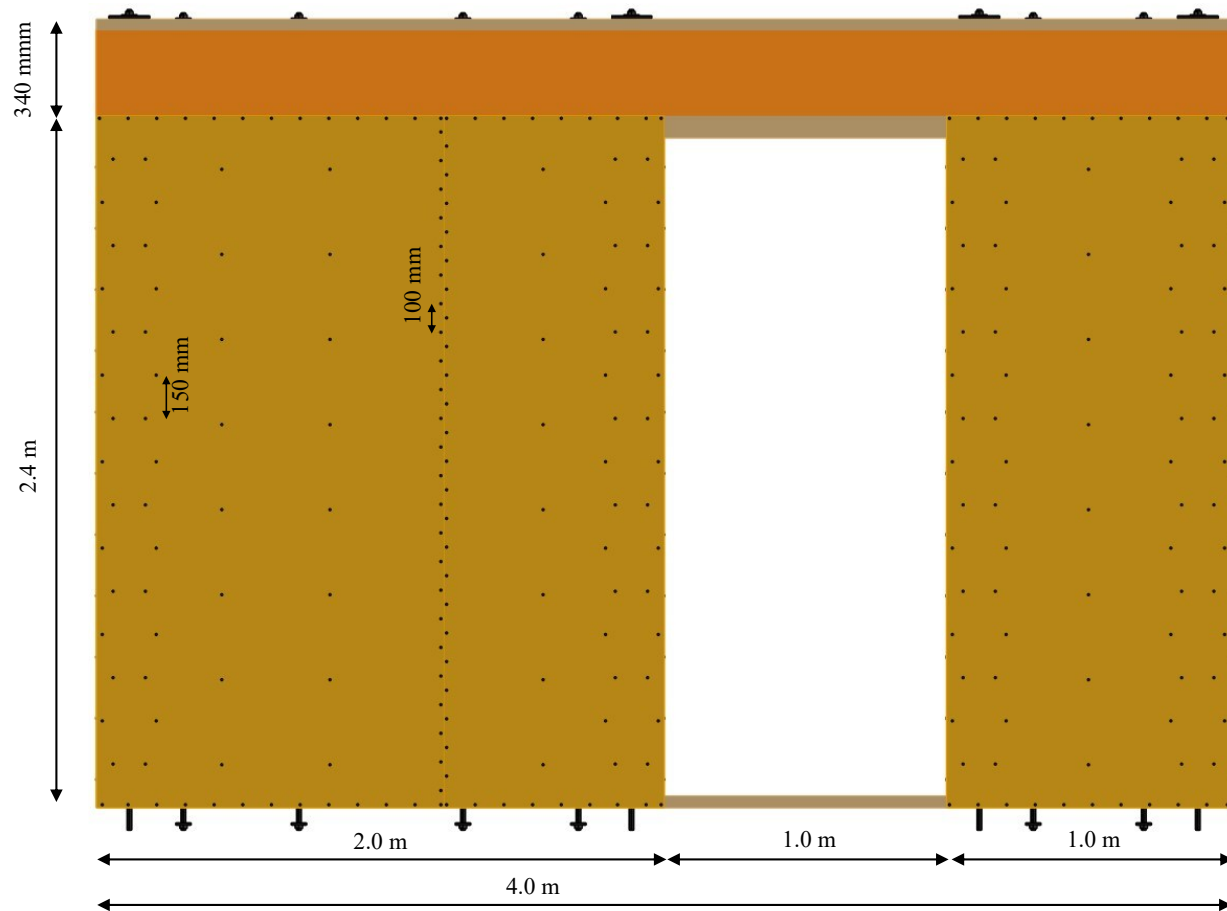


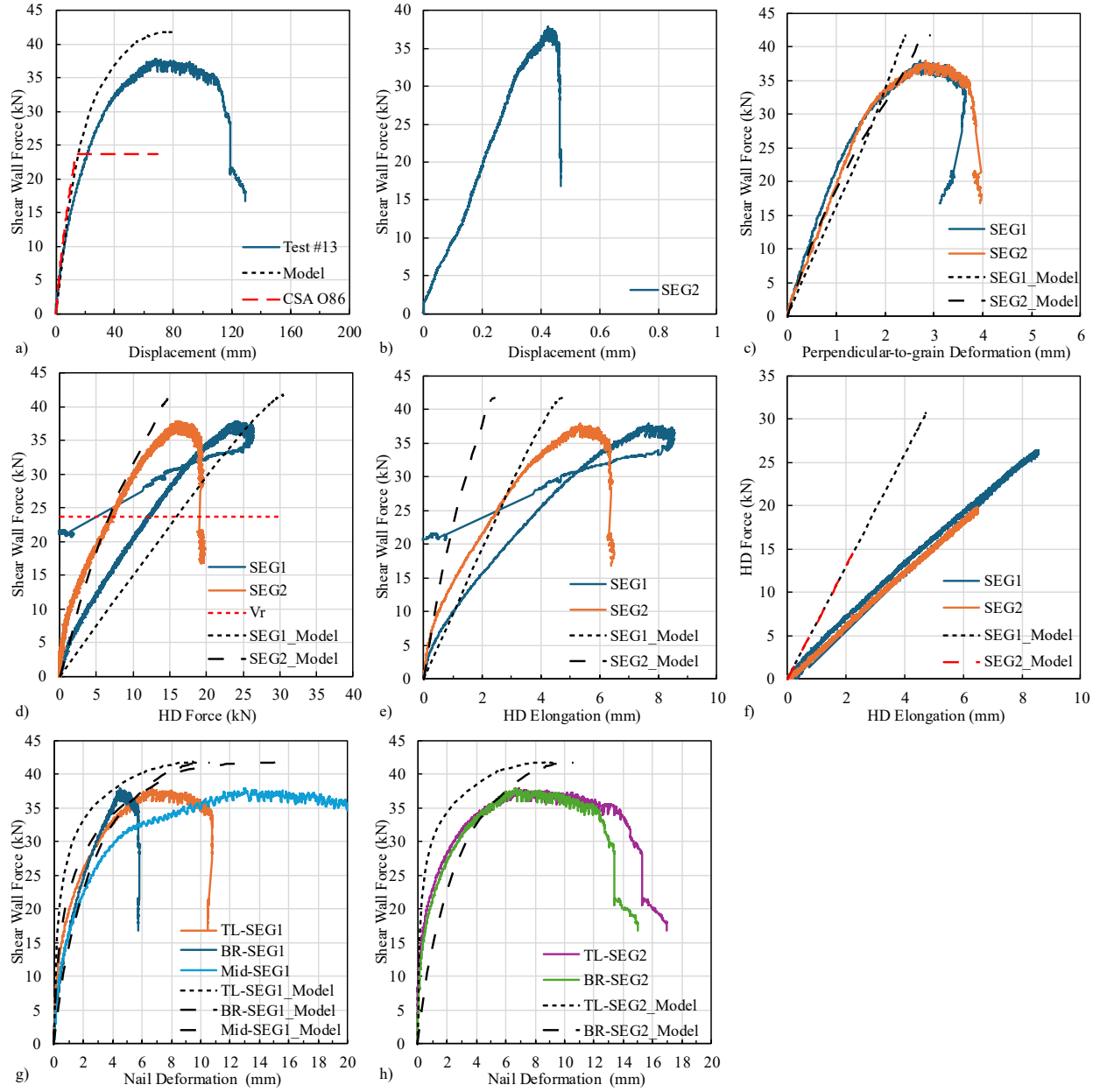
Figure A.25: Configuration #13 geometry (continued)

Table A-25: Shear wall design information (Configuration #13)

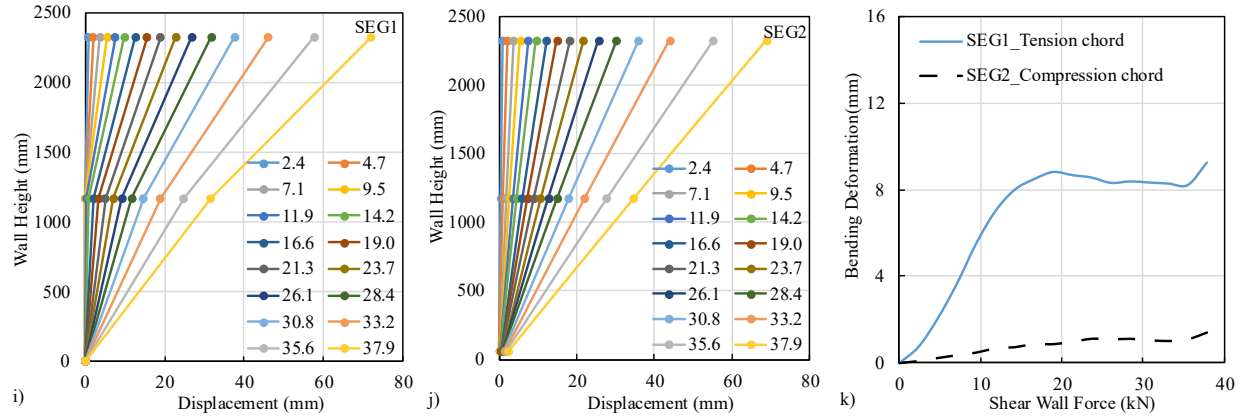
| Configuration | Shear Wall ID | $L_s$ (mm) | $V_{rs}$ (kN) | $V_d$ (kN) | (D/C) <sub>sw</sub> | $L_o$ (mm) | $T_r$ (kN) | $T_d$ (kN) | (D/C) <sub>HD</sub> |
|---------------|---------------|------------|---------------|------------|---------------------|------------|------------|------------|---------------------|
| #13           | 1             | 2000       | 15.8          | 17.75      | 1.1                 | 1772       | 27.42      | 29.36      | 1.07                |
|               | 2             | 1000       | 7.9           | 5.95       | 0.8                 | 772        | 27.42      | 22.56      | .82                 |

Table A-26: Shear wall design deflections (Configuration #13)

| Configuration | Shear wall ID | $e_n$ (mm) | $d_{a1}$ (mm) | $d_{a2}$ (mm) | $\Delta_b$ (mm) | $\Delta_s$ (mm) | $\Delta_n$ (mm) | $\Delta_{a1}$ (mm) | $\Delta_{a2}$ (mm) | $\Delta_t$ (mm) |
|---------------|---------------|------------|---------------|---------------|-----------------|-----------------|-----------------|--------------------|--------------------|-----------------|
| #13           | 1             | 1.08       | 3.75          | 0.22          | 0.33            | 1.97            | 6.59            | 5.16               | 0.30               | 14.4            |
|               | 2             | 0.49       | 2.88          | 0.17          | 0.44            | 1.32            | 2.99            | 9.10               | 0.54               | 14.4            |



**Figure A.26: Response quantities in Configuration #1: (a) shear wall F–D; (b) shear wall bottom plate slide; (c) perpendicular-to-grain compression deformations; (d) shear-wall force vs. hold-down force; (e) shear-wall force vs. hold-down elongation; (f) hold-down F–D; (g) and (h) nail deformations in SEG1 and SEG2, respectively; (i) chord lateral displacement profile; (j) and (k) tension and compression chords lateral displacements, respectively; (l) chord bending deformation**



**Figure A.26: Response quantities in Configuration #1: (a) shear wall F–D; (b) shear wall bottom plate slide; (c) perpendicular-to-grain compression deformations; (d) shear-wall force vs. hold-down force; (e) shear-wall force vs. hold-down elongation; (f) hold-down F–D; (g) and (h) nail deformations in SEG1 and SEG2, respectively; (i) chord lateral displacement profile; (i) and (j) tension and compression chords lateral displacements, respectively; (k) chord bending deformation (continued)**



#### A.14 Configuration #14

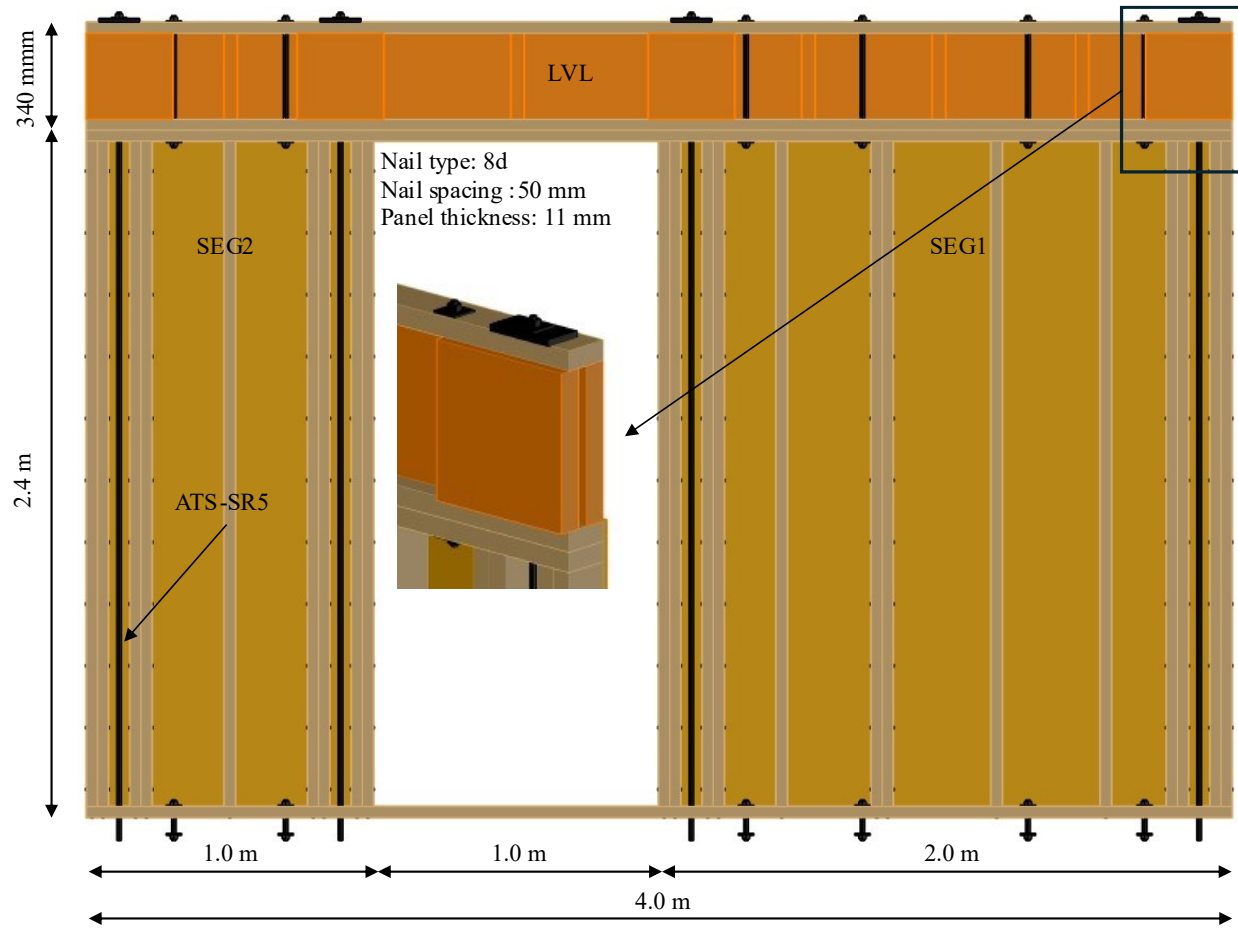


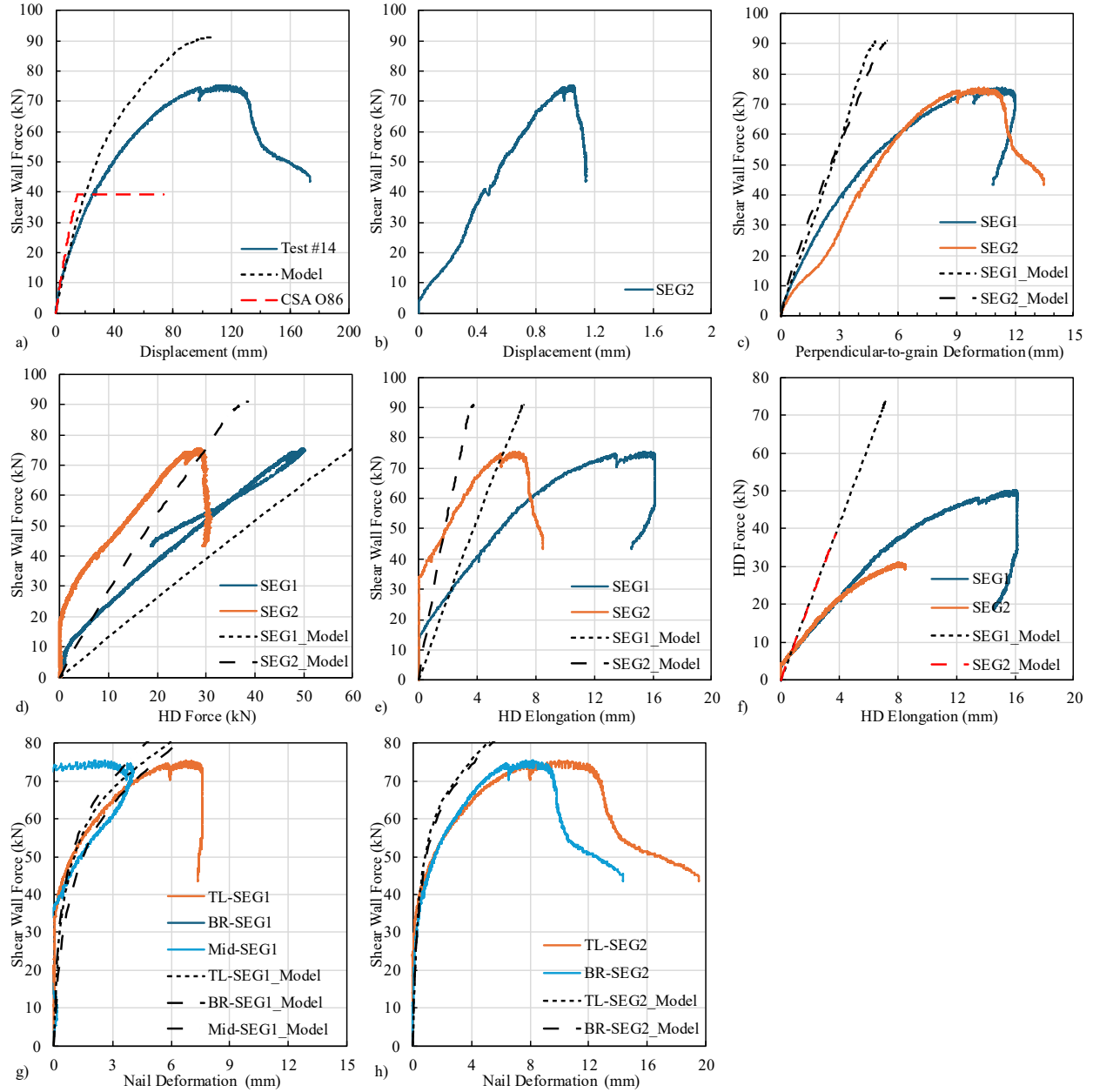
Figure A.27: Configuration #14 geometry

Table A-27: Shear wall design information (Configuration #14)

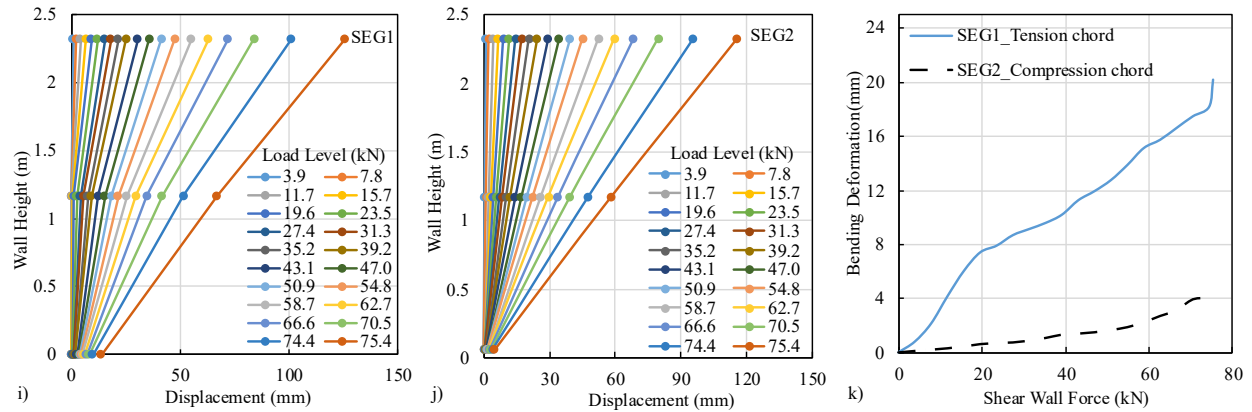
| Configuration | Shear Wall ID | $L_s$ (mm) | $V_{rs}$ (kN) | $V_d$ (kN) | (D/C) <sub>sw</sub> | $L_o$ (mm) | $T_r$ (kN) | $T_d$ (kN) | (D/C) <sub>HD</sub> |
|---------------|---------------|------------|---------------|------------|---------------------|------------|------------|------------|---------------------|
| #14           | 1             | 2000       | 26.1          | 29.8       | 1.14                | 1772       | 43.36      | 49.2       | 1.13                |
|               | 2             | 1000       | 13.05         | 9.34       | 0.71                | 772        | 43.36      | 35.42      | 0.82                |

Table A-28: Shear wall design deflections (Configuration #14)

| Configuration | Shear wall ID | $e_n$ (mm) | $d_{a1}$ (mm) | $d_{a2}$ (mm) | $\Delta_b$ (mm) | $\Delta_s$ (mm) | $\Delta_n$ (mm) | $\Delta_{a1}$ (mm) | $\Delta_{a2}$ (mm) | $\Delta_t$ (mm) |
|---------------|---------------|------------|---------------|---------------|-----------------|-----------------|-----------------|--------------------|--------------------|-----------------|
| #14           | 1             | 0.76       | 3.98          | 0.38          | 0.56            | 3.31            | 4.64            | 5.48               | 0.52               | 14.5            |
|               | 2             | 0.30       | 2.86          | 0.27          | 0.70            | 2.07            | 1.83            | 9.04               | 0.85               | 14.5            |



**Figure A.28: Response quantities in Configuration #14: (a) shear wall F–D; (b) shear wall bottom plate slide; (c) perpendicular-to-grain compression deformations; (d) shear-wall force vs. hold-down force; (e) shear-wall force vs. hold-down elongation; (f) hold-down F–D; (g) and (h) nail deformations in SEG1 and SEG2, respectively; (i) chord lateral displacement profile; (i) and (j) tension and compression chords lateral displacements, respectively; (k) chord bending deformation**



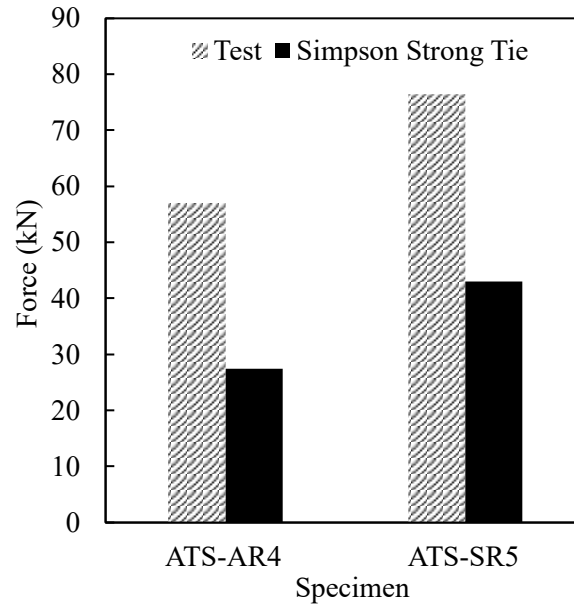
**Figure A.28: Response quantities in Configuration #14: (a) shear wall F–D; (b) shear wall bottom plate slide; (c) perpendicular-to-grain compression deformations; (d) shear-wall force vs. hold-down force; (e) shear-wall force vs. hold-down elongation; (f) hold-down F–D; (g) and (h) nail deformations in SEG1 and SEG2, respectively; (i) chord lateral displacement profile; (i) and (j) tension and compression chords lateral displacements, respectively; (k) chord bending deformation**



## Appendix B

### Discussion of material and connection test results and failure modes

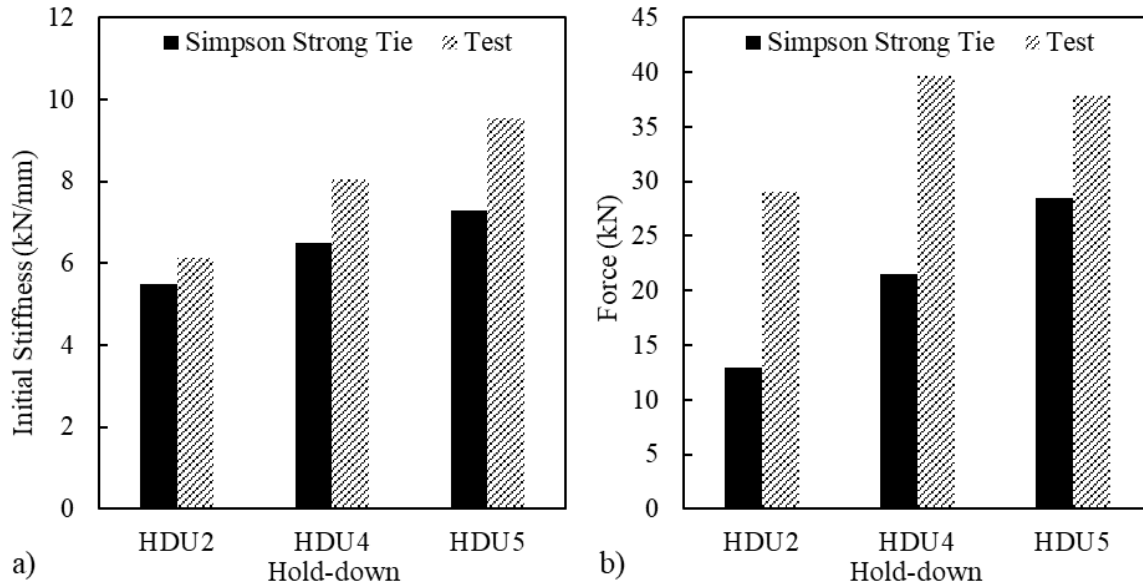
Figure B.1 compares the  $T_r$  values of the ATS rods as provided by Simpson Strong-Tie (Simpson Strong-Tie 2022) with the maximum tensile forces recorded during the tests.



**Figure B.1: Tensile resistance of ATS rods**

The comparison in Figure B.1 indicates that both ATS-SR4 and ATS-SR5 rods possessed a considerably high reserve strength beyond the specified resistance. Therefore, even in highly optimized design scenarios where the tensile demand approaches the  $T_r$  value, a margin of strength remains available in the rod.

Regarding the discrete HDs, comparisons were made between the average initial stiffness and the manufacturers' values (Simpson Strong-Tie 2022) for HDU2, HDU4, and HDU5. Also, the corresponding  $T_r$  values were compared with the manufacturer and the corresponding maximum tensile values obtained from the HD connection tests, as shown in Figure B.2(a) and (b).

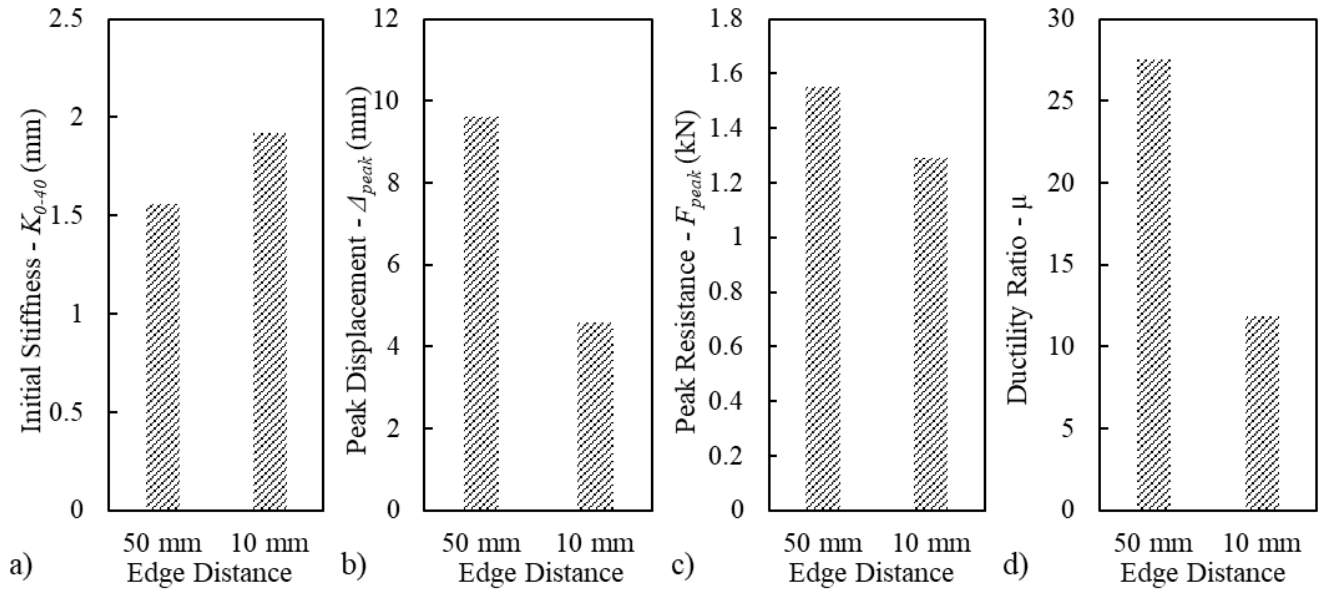


**Figure B.2: (a) Initial stiffness of discrete HDs; (b) tensile resistance of discrete HDs**

Figure B.2(a) illustrates that the average initial stiffness values obtained from the tests were higher than those reported by the manufacturer (Simpson Strong-Tie 2022) for the individual HDs. It is noted, however, that the manufacturer's reported values may reflect a broader variability in product performance. The CV of the initial stiffness from the tests were 0.11, 0.05, and 0.39 for HDU2, HDU4, and HDU5, respectively.

The failure modes observed in the HD connection tests varied by device type. HDU2 and HDU4 predominantly failed through wood crushing at the screw locations and exhibited a relatively gradual response near the ultimate load. In contrast, HDU5 connections primarily failed by brittle wood splitting, resulting in a sudden and abrupt failure.

Sheathing-to-framing connection tests exhibited markedly different behaviours as the edge distance of the nail connection varied. Figure B.3(a)–(d) compares the initial stiffness, peak load, peak displacement, and ductility of the connections for two extreme edge-distance cases: 10 mm and 50 mm.



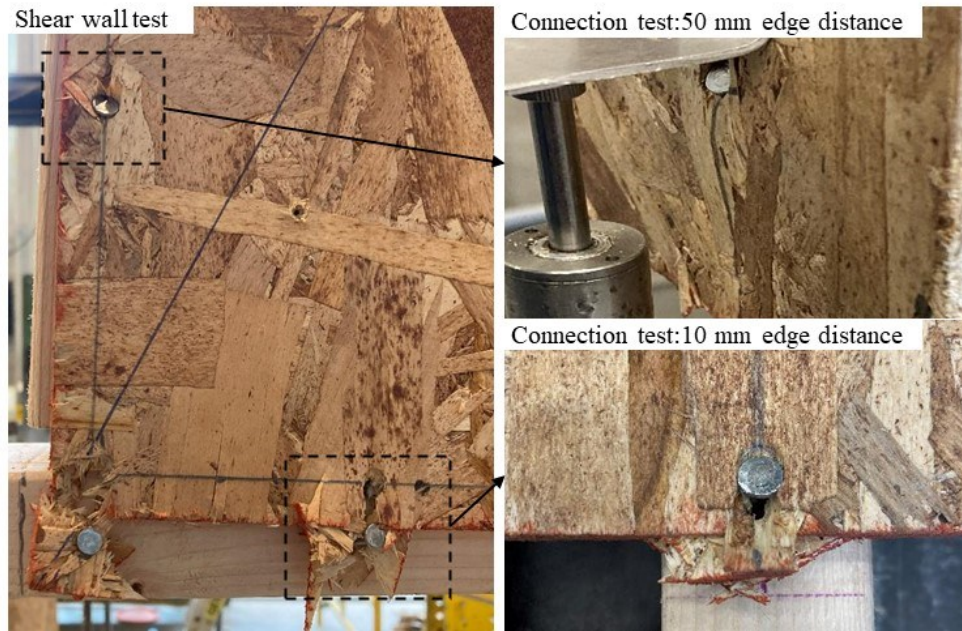
**Figure B.3: Sheathing-to-framing connection properties: (a) initial stiffness; (b) peak displacement; (c) peak resistance; (d) ductility ratio**

The results indicate that edge distance significantly influenced strength, deformation capacity and ductility of the connection, highlighting the sensitivity of local connection response to geometric detailing.

As can be inferred from Figure B.3(a) and (c), the initial stiffness and peak resistance were only slightly affected by the change in edge distance from 10 mm to 50 mm. In contrast, the deformation capacity was strongly influenced by this parameter. Connections with a 10 mm edge distance exhibited a peak displacement approximately 50% lower than those with a 50 mm edge distance (Figure B.3(b)). Consequently, the ductility of the specimens with 10 mm edge distance was reduced by nearly three times compared to that with 50 mm edge distance.

The observed failure modes were consistent with these differences in deformation capacity. Connections with a 10 mm edge distance primarily failed by inward-directed nail loading toward the panel edge, a mode similar to the corner nail failures observed during the shear wall tests at the onset of damage. Conversely, in opposite shear wall corners where the nails were loaded outward from the panel edge, the failure mode resembled that of the 50 mm edge-distance connection tests.

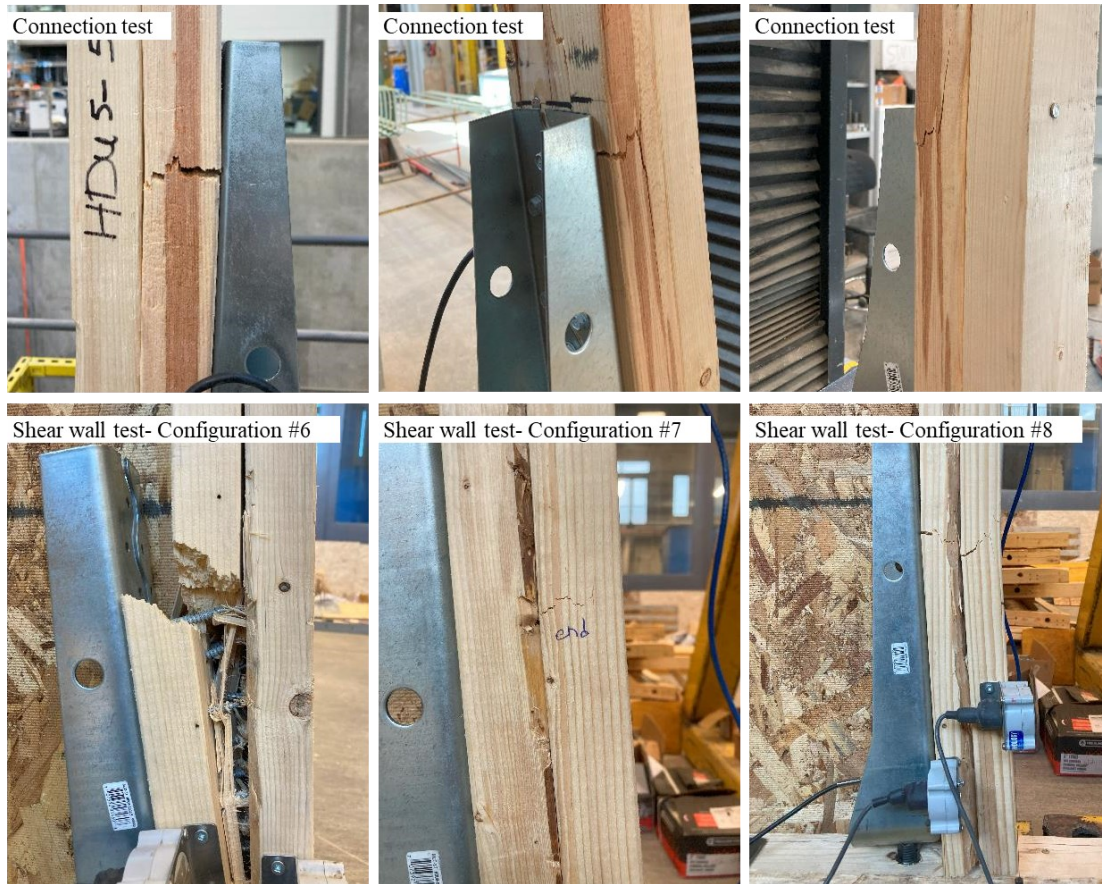
Figure B.4 presents these corresponding failure modes from both the shear wall and connection tests, confirming the influence of edge distance on local connection behaviour.



**Figure B.4: Common nail failure modes in shear wall and connection tests**

As seen in Figure B.4, depending on the kinematic of the shear wall and the sheathing panel, two adjacent nails experience different failure modes. The nail failure modes at the bottom side of the panel where the shear wall uplifted was similar to that of connection tests with 10 mm edge distance. This failure mode was believed to control the peak resistance of the shear wall as it was observed during the shear wall test the onset of the shear wall strength degradation was the time that corner showed failure initiation. The current minimum edge distance in CSA O86 is 9 mm and it is recommended that for a better shear wall performance in terms of the peak resistance and delaying the strength degradation, the minimum distance criterion to be increased to at 20 mm. This would also cause an increase in shear wall ductility as per nail connection test results. Fracture of the tension chord at the location of the HD in Configurations #6, #7, and #8 was observed during full-scale shear wall tests. A similar failure mechanism was also recorded in the HD connection tests. These failure modes are illustrated in Figure B.5.





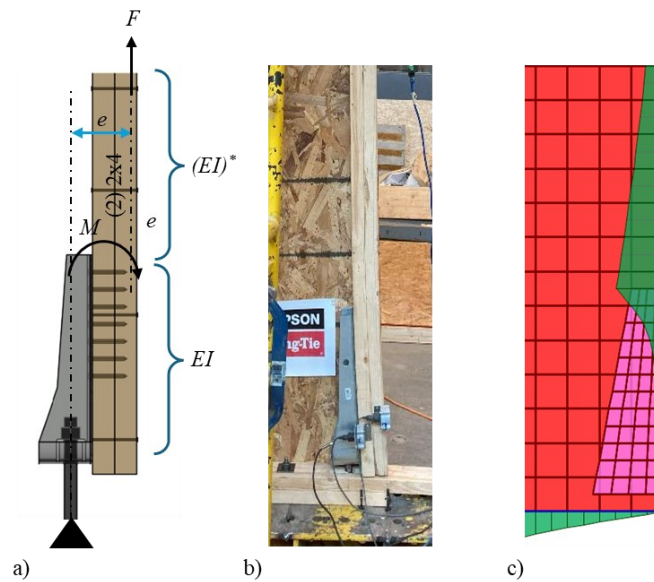
**Figure B.5: HD fracture failure modes**

All fractures of the shear wall chord at the HD locations initiated at the last row of screws connecting the HD device to the chord member. This failure mode was also observed in some HD connection tests, particularly for HDU5 specimens. The primary cause of this failure was excessive bending demand at the chord where HD ends. These failures occurred in configurations where the addition of HD connections at the floor level imposed bending on the chord members, as the floor clamping maintained a right angle between the floor and the chord, inducing local bending at the chord ends.

Furthermore, the high local flexural rigidity of the chord member, which was amplified by the presence of the HD device at the bottom of the shear wall, and its difference with the bending

rigidity of the chord just above the HD, created a sudden stiffness discontinuity at the last row of screws, leading to a kinking mode.

Figure B.6(a)–(c) illustrate the HD force transfer mechanism, the deformed HD during testing, and the bending moment distribution at the top of the HD from the numerical simulation.



**Figure B.6: (a) Load path around HD in wood shear wall; (b) deformed shape of the chord member at the HD location (kinking); (c) bending moment distribution from numerical analysis**

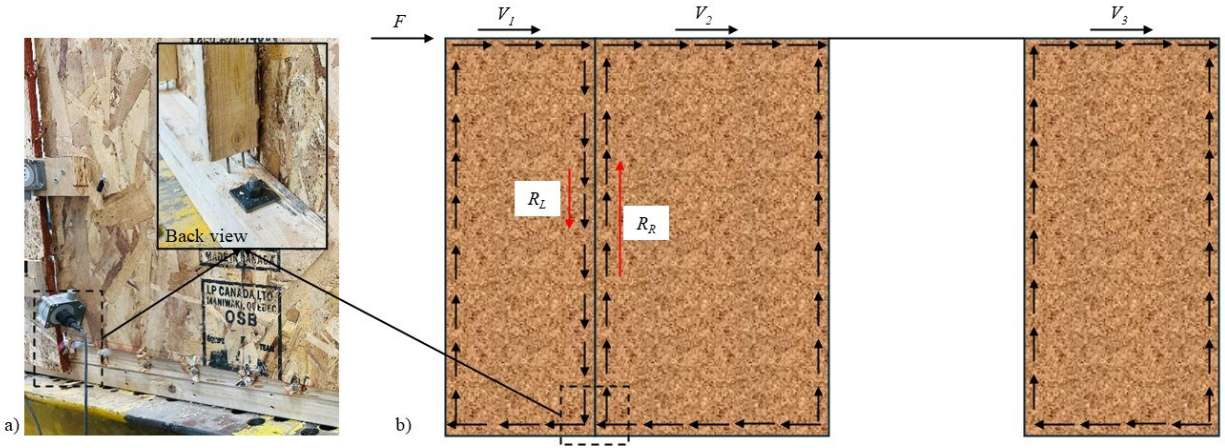
Figure B.6(a) illustrates the load path at the HD location of a wood shear wall. The eccentricity between the HD anchor and the line of action of the force, aligned with the nail line at the edge of the panel, induces a bending moment at the top of the HD device. Additionally, the geometry of the HD device adds extra flexural stiffness to the bottom of the chord member, causing an abrupt change in bending rigidity around the HD, as shown in the figure. As a result of this force transfer mechanism and sudden change in chord rigidity, Figure B.6(b) shows the deformed chord during the shear wall test. Since direct measurement of the bending moment magnitude and distribution at this location was not possible, Figure B.6(c) presents the numerical representation of the bending moment distribution at the HD.

For HDs with higher capacity, such as HDU5, which use multiple screws to attach the device to the chord, microcracks or initial splitting around screws are likely. Existing wood defects, such as knots, further increase the susceptibility to fracture. Therefore, it is recommended that chord members around HD locations be free of defects. Moreover, as the HD capacity and stiffness increase, the number of plies in the chord member should also be increased to prevent abrupt changes in bending rigidity.

Another failure mode observed in Configurations #1 and #2 was the early uplift of the adjoining stud at the panel interface. This failure occurred due to the presence of sheathing panels with different ARs in SEG1 of the shear wall. Figure B.7(a) shows the observed failure in Configuration #1, while Figure B.7(b) presents a schematic of the shear wall with an idealized force distribution along the panel edges.

As illustrated, if the resultant forces on the left ( $R_L$ ) and right ( $R_R$ ) sides of the adjoining stud are unequal or significantly different, the stud becomes susceptible to uplift, as occurred in Configuration #1. This was caused by the placement of a narrower panel at the beginning of the wall segment, as shown in Figure B.7(b). Consequently, the excessive uplift of the adjoining studs caused nails at the bottom of the larger panel to fail first. Also, the uplift of the stud at the panel interface resulted in excessive nail deformations along the right edge of the narrower panel, and eventually separation of the panel from the stud.

This failure mode is expected to be particularly critical under cyclic loading. After initial failure at the larger panel, the shear wall response is expected to differ significantly in the opposite loading direction. To mitigate this failure, it is recommended that sheathing panels within a wall segment have similar or nearly equal ARs, which can delay or prevent the uplift of adjoining studs.



**Figure B.7: (a) Failure mode due to uplift of the adjoining stud in a shear wall with sheathing panels of different ARs; (b) idealized force distribution around the panel edges in Configuration #1**

In summary, these results demonstrate that panel layout, connection detailing, and local geometry critically affect shear wall resistance, ductility, and failure progression, highlighting the need for careful design and consistent detailing.