

STUDY OF DEFLECTION OF SINGLE AND MULTI-STOREY LIGHT FRAME WOOD SHEAR WALLS

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ABSTRACT

The behavior of wood shear walls has been the focus of researchers and engineers for many years due to their availability in the North American construction landscape. A review of the established literature showed that most of the research have focused on the shear wall behavior as a whole with no investigation specifically targeting the individual components of its deflection. Also, little to no attention has been given to the investigation of the cumulative effects especially when the out-of-plane diaphragm stiffness is considered. The current study aims at investigating the effects of construction details variation on the behavior of the shear walls and evaluating whether the current deflection equation, as per wood design standard (CSA 2014) can adequately predict the overall wall stiffness.

A total of 27 full-scale single-storey walls, with different construction details and aspect ratios, were tested under either static or monotonic (as both are the same) loading. The parameters that were varied in the testing were the stud size and spacing, nail diameter and spacing, sheathing panel type and thickness and hold-down anchoring system/type. For the two-storey walls, two different loading cases were considered, namely where the load was applied at the top or bottom storey only. The results showed that the strength and stiffness correlated almost directly to the inverse of the wall aspect ratio. There was no clear trend when considering the effect of the walls' aspect ratios on ductility. Unexpectedly, walls with aspect ratios not permitted according to the wood design standard (4:1 and 6:1) followed similar strength and stiffness trends and had sufficient ductility ratios as those with smaller aspect ratios. This observation explains in part some of the discrepancies found between engineering calculations and behavior of actual

building with light frame wood shear walls. Significant discrepancies were found when comparing the various deflection constituent with those estimated using the design expression. Adding more end studs and changing the size of the studs had no significant effect on the overall wall capacity and little effect on its stiffness. Reducing the stud spacing had, as expected, no effect on the wall capacity; however, the results showed that the bending stiffness was affected by the overall number of studs in the wall and not solely by the end studs. Shear walls sheathed with plywood panels exhibits slightly higher peak load and initial stiffness than those with OSB, which was mainly attributed to the greater panel thickness, and possibly density, of the plywood. Both sheathing types provided similar levels of ductility, as expected. Thicker sheathing increased the capacity and stiffness of the wall with no significant change observed in ductility ratio. The wall strength was significantly affected by the nail diameter and nail spacing, but no difference was observed when the nail edge/end distance was increased. The results also showed that discrete hold-down system behaved in a non-linear manner with a significantly greater initial stiffness than that assumed in design. The study also showed that having continuous hold-down connections has a positive effect on the capacity, stiffness and ductility of the wall when compared with discrete hold-downs. Having no hold-down adversely affects the wall capacity and stiffness, but did not affect the ductility of the wall. For the two-storey walls, the deflection estimated based on the cumulative effect assumption showed slight differences when compared with that observed in the experimental study. It was observed that the majority of the cumulative effect stems from the rigid body rotation due to deformation in the hold-down devices.

A Computer shear wall model (through SAP2000) was developed using linear “frame” and “membrane” elements for the framing and sheathing members, respectively, whereas the sheathing to framing nails and hold-down were modeled using nonlinear springs. It was found

that the model was capable of predicting the peak load, ultimate deflection and yield loads with reasonable accuracy, but overestimated the initial stiffness and ductility of the walls. In general, when the force-displacement curves were compared it was evident that the model was capable of predicting the wall behaviour with reasonable accuracy. When investigating the cumulative effects using the model, the results clearly showed that the assumption of cumulative effects due to rigid body rotation is valid for stacked shearwalls with no consideration for the floor diaphragm. The effect of the diaphragm on the behavior of the shear walls, in particular its out-of-plane rigidity was simulated by modeling the floors as beam. The out of plane stiffness of the shear walls was investigated for idealized (infinitely stiff or flexible) as well as “realistic”. The results showed reductions in the shearwall deflection in the magnitude of approximately 80% considering the out of plane rigidity of the diaphragm. It was also concluded that considering conservative estimates of out of plane stiffness might lead to a very significant reduction in deflection and that assuming the floor diaphragm to be infinitely rigid out of plan seems reasonable. For diaphragms supported on multiple panels further reduction in the deflection was observed. More work, particularly at the experimental level, is needed to verify the finding obtained in the numerical investigation related to the effect of out of plane diaphragm stiffness.

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1. INTRODUCTION

1.1 Light Frame Wood Construction

Using wood as a building material has been accompanied by several advantages, including cost-effectiveness, durability, ease of construction and environmental attributes. Light frame wood construction has therefore dominated the North American landscape for centuries. Prior to the first edition of the National Building Code of Canada (NBCC 1941) medium to high-rise wooden buildings were part of the North American landscape and no specific restrictions prevented their construction. However, through the development of modern codes and due to concerns about risk of fire, the first edition of National Building Code of Canada (NBCC 1941) reduced the height limit for wood buildings to 7 storeys; a limit that later was further decreased to 4 storeys in 1953 (Figure 1.1). Due to numerous research effort in the area of structural and fire safety, the province of British Columbia introduced legislations to allow for 5- and 6-storeys light wood frame buildings (CWC 2009). Other provinces followed suit in 2015 when the national building code (NBCC 2015) permitted wood construction for residential, business and office service occupancies with a maximum height of 6 storeys. The tendency seems to point in the direction of seeking changes to prescriptive code requirements to allow for even taller buildings involving wood and wood-hybrid materials. Achieving this goal is not possible, unless the lateral behavior of the structure is well understood to ensure that adequate strength and stiffness is provided to resist the applied loads.

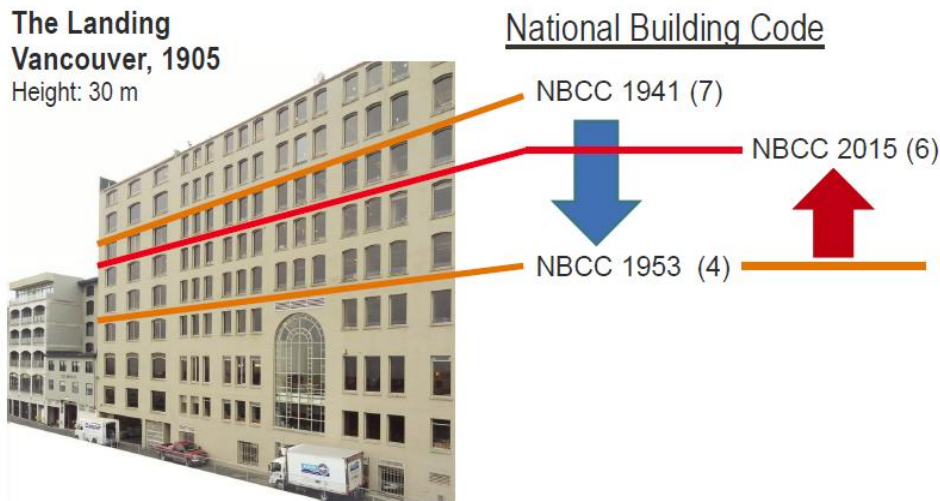


Figure 1.1: Maximum Height of Buildings Allowed by NBCC (FPinnovations 2015)

1.2 General Overview on the Performance of Light Frame Construction under Wind and Seismic Events

Investigating the performance of wood structures under high wind and earthquake events provide an insight into their strength and weaknesses. The damage observed in light frame wood structures in high wind events has pointed to gable end roof-to-wall connections and sheathing attachment as critical areas that required special attention in design (e.g. Douglas 1992, Wolfe et. al. 1993). This had led to code development and enhancement of provisions including more stringent requirements for roof-to-wall connections.

Wood frame buildings have behaved reasonably well in past earthquake events especially when it comes to life safety (NRC 2000, Rainer and Karacabeyli 1999). One of the most serious deficiencies was attributed to weak- or soft-storeys, which were typically found in buildings with large and often continuous openings for parking.

In 1949, the Federal Housing Administration introduced the first edition of modern days light frame shear wall sheathed with plywood, which presented an alternative to diagonally braced walls. A standard for testing shear walls to resist racking was also introduced (Federal Housing Administration 1949).

In 1966, the American plywood Association (APA 1966) published the results of 39 shear wall tests with different construction detailing. The study showed that wood shear walls were capable of resisting significant amount of lateral load before failure. The following year, the Uniform Building Code (UBC 1967) provided, for the first time, allowable load tables for wood shear walls.

Even with the introduction of design provisions for light frame shearwalls, reports from subsequent earthquakes still showed that more research is required to attain a better understanding of the response of wood buildings to seismic loads (Lam et. al 2002). Particularly, the 1994 Northridge earthquake demonstrated deficiencies in the ability of wood shear walls to resist lateral loading and to control the structural and non-structural damage where poor construction practice also contributed to damage. Examples of deficiencies in construction were highlighted and included missing hold-downs, misplaced bolts, improper nailing, soft storey, and irregular configurations (Lam et. al 2002), as shown in Figure 1.2.



Figure 1.2: Meadows Apartment Complex, 1994 Northridge Earthquake (Lam et. al 2002)

The 1995 Kobe earthquake in Japan also highlighted some poor performance of the lateral load resisting systems in light frame wood buildings (Figure 1.3). It was for example reported that inadequate shear wall length (i.e. inadequate strength and stiffness) was the major reason of collapse of such buildings (Lam et. al 2002).



Figure 1.3: Collapse of Typical Building, 1995 Kobe Earthquake (Lam et. al 2002)

1.3 Wood Shear Walls as Lateral Force Resisting Systems

1.3.1 Wood Shear Wall Components

Typical wood shear walls consist of studs, sheathing and fasteners. Visually graded lumber members are used as studs and wall plates and have a rectangular cross-section with typical sizes of 38 x 140 mm (CSA 2014). Oriented strand board (OSB) and Plywood (Fig. 1.4) are commonly used as sheathing material and are to be properly fastened to the stud wall framing.



a) OSB



b) Plywood

Figure 1.4: Shear Wall sheathing material

The fasteners used in wood shear wall construction may consist of nails, staples or screws. However, nails are most commonly used compared to other types of fasteners. The dimensions and mechanical properties of common nails used in wood frame construction are provided in the wood design standard (CSA 2014).

The ability of wood-frame construction to resist wind and earthquake loads is attributed primarily to the numerous small connections (typically nails), where an overloading of one connection causes the force to flow into less loaded adjacent connectors. Structural panels (plywood or OSB) acting in combination with studs and joists, create very effective lateral-force

resisting building assemblies. Additional nails or special framing anchors are typically required to connect the diaphragms and shear walls. Hold-down connections are typically used at the corners of the walls or wall panels and additional anchor bolts are required to connect the shear wall to the floor below or to the foundation (Figure 1.5).

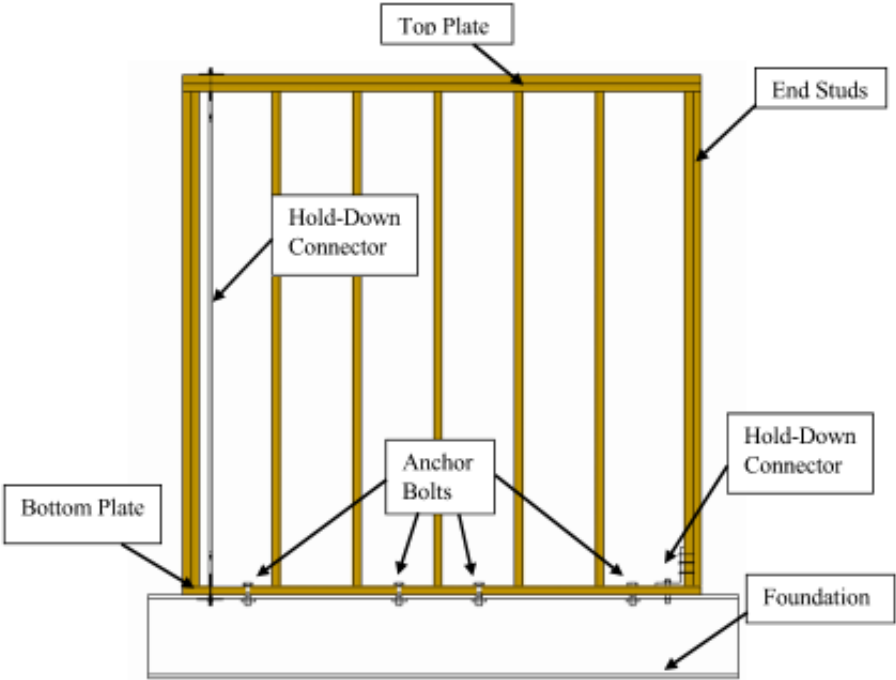


Figure 1.5: Sketch of Wood Shear Wall

1.3.2 Load Path in Typical Light-Frame Building

In light-frame buildings, lateral force resisting system (LFRS) consists of shear walls, diaphragms and connectors to provide a continuous system that is capable of transferring lateral loads as well as overturning moments to the foundation. Figure 1.6 shows the load path in typical light-frame building (Salinkovich 2000). As can be seen in Figure 1.6, the horizontal diaphragm transfers the lateral forces to the shear walls, where overturning causes tension and compression

at the end studs of the shear walls. Hold-downs are therefore provided to transfer such forces to the foundation.

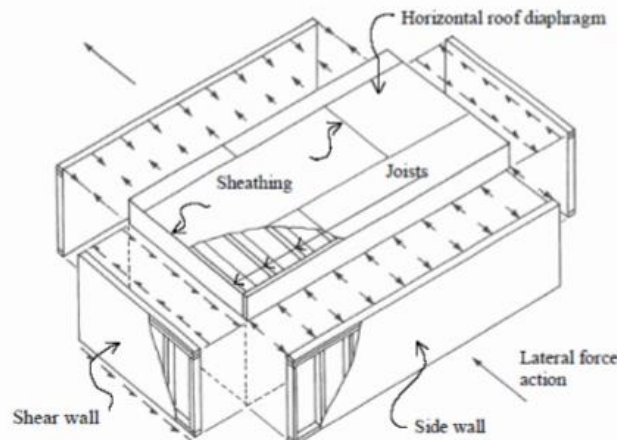


Figure 1.6: Load Path in Typical Light-Frame Building (Salenikovich 2000)

1.3.3 Light-frame Wood Shear Walls Stiffness and Resistance

One of the most important seismic design concepts is to determine the building period, which in turn is based on the structure's mass and stiffness. The mass can be estimated with relative ease but the stiffness, especially one that is representative of the "actual" behavior of the system, is much more difficult to determine. Thus, a number of mechanics-based models have been developed to predict the capacity and deflection of the light frame wood shear walls (APA 1952, Burgess 1976, McCutcheon 1978). The wood design standard (CSA 2014) provides a 4-term deflection equation, which includes the contribution of stud bending, panel shear, nail slip and anchorage system elongation. In this equation, the shear wall is considered as a continuous cantilever beam; however the equation was originally limited in scope to a single-story wall. The 2009 edition of the CSA-O86 standard incorporated cumulative effects due to rotation of the bottom level (bending and hold-down elongation) which is expected in multi-storey walls and an

approach based on the cantilever model was adopted (CSA 2009). However, the accuracy of cantilever theory has not been evaluated experimentally. These issues will be discussed more in-depth in Chapter 2.

Wood shear walls are designed to provide adequate stiffness and strength to resist lateral and uplift forces. The wall capacity is provided primarily by the strength of the fastener between the sheathing and the studs. Such failure (sheathing-to-framing connection failure) provides the most ductile behaviour to the building since it involves yielding of the nails and some limited crushing in the wood. The joint capacity is in turn a function of the nail size and spacing, sheathing thickness and the density of the stud member.

A mechanics-based approach was established to calculate the factored shear resistance of the shear walls, which considers the minimum of the joint capacity between the sheathing and framing or the buckling of the sheathing panel (Ni et. al. 2012).

1.3.4 Research contribution

Focusing on the behavior of wood shear walls has been an interest to researchers for many years. However, the focus has mainly been on establishing the capacity of the walls provided different construction detailing. For this purpose, parameters such as aspect ratio, stud spacing and size, panel thickness, nail diameter and spacing were changed and their effect on the wall behavior as a whole were investigated. Where research has lacked is in investigating the individual contribution from bending, shear, nail slip and overturning.

Furthermore, no experimental study has been undertaken, which focuses on multi-storey walls to evaluate the accuracy of contemporary design approaches in predicting the deflection expression provided in the standard.

The current study attempts to provide a better understanding of the behaviour of single and multi-storey light frame shear walls through an approach involving experimental studies at different levels of the structural elements (i.e. components, connections and subsystems levels) as well as numerically for multi-level wall systems.

1.4 Research Objectives

The overall objective of the current research study is to investigate the validity of the assumptions in the four-term deflection equation included in contemporary North American timber design standards for light frame wood shear walls. More specifically the aim is to:

- 1) Investigate the effects of construction details variation on the behavior of the shear walls,
- 2) Evaluate whether the current deflection equation can adequately predict the overall wall stiffness,
- 3) Investigate the deflection equation by separating its constituents and studying their accuracy individually,
- 4) Investigate whether the cantilever assumption applies to single and multi-storey wood shear walls, through an assessment of the current design model,
- 5) Provide guidance on the use of the design approach based on construction detailing.

1.5 Research Methodology

The methodology employed in this research is to conduct racking tests on single and two-storey walls and to evaluate the wall behavior in terms of overall stiffness, capacity and ductility. The focus will be on evaluating the stiffness of the wall (especially the initial stiffness), however since the walls will be tested to destruction, their strength and ductility will be available as test outputs and would therefore be reported and discussed as well. The first step in the research is to

investigate 27 full-scale single-storey walls where the displacement representing each component of the four term deflection equation is evaluated separately. The experimental campaign includes varying levels of wall aspect ratios and construction details. Also component tests, including stud bending tests, nail joint tests and steel rod tension tests are evaluated to establish material properties and provide input for the numerical modeling component of the research.

The second phase of the research deals with multi-level shear walls. In order to investigate the cantilever and cumulative assumptions, two half-scale two-storey shear walls are investigated. Suitable numerical model are identified and validated using the experimental results. These models are then used to investigate the cumulative effects for a mid-rise building with and without the effects of out-of-plane stiffness of the floor diaphragm.

1.6 Thesis Organization

Chapter 1 presents background information on light frame wood buildings and introduces the research needs and objectives.

Chapter 2 presents detailed review of previous relevant research studies, where the behavior of full scale shear walls is discussed experimentally and theoretically. The review is focused on research related to shear wall deflection.

Chapter 3 describes the experimental program. All the details regarding to test materials, test specimens, test matrices, setup and component tests are presented in this chapter.

Chapter 4 contains the experimental results for all tests performed, including the component tests as well as full-scale tests on single-storey and two-storey walls.

Chapter 5 discusses the experimental results. Comparison between walls with different aspect ratios and construction details are made. Also, the results obtained for the four separate components of the shear wall deflection equation are discussed.

Chapter 6 describes the numerical modeling procedures and results. It also provides comparison and discussion between the numerical and experimental results.

Chapter 7 presents comparisons between the experimental results and the code equation in order to identify discrepancies and suggest improvements.

Chapter 8 provides conclusions and recommendations for future research

2. LITERATURE REVIEW

2.1. Introduction

Light frame shear walls are considered to be efficient lateral load resisting systems. Several research studies have investigated the performance of such walls, but the focus has mainly been on the ultimate resistance and ductility of the system. This chapter provides a summary of relevant research that has been conducted on light frame wood shear walls. Section 2.2 focuses on analytical/numerical studies while Section 2.3 introduces standard test procedures that have been followed by researchers in conducting experimental studies. Section 2.4 presents experimental studies on single storey walls. Finally, section 2.5 presents background on deflection of multi-storey walls together with the relevant research.

2.2. Analytical Modeling

Generally, there are two main analytical models to predict the deflection of light frame wood shear wall, namely energy-based models and mechanics-based models.

2.2.1. Energy-Based Models:

Tuomi and McCutcheon (1976) introduced a new model based on energy method to predict the racking deflection of shear walls considering linear load-slip relationship for the nail joints. However, since it is well-established that the nail slip behavior is highly nonlinear, a subsequent study by McCutcheon (1985) improved the model by incorporating the nonlinear nail load-slip relationship. The study assumed that the deflection of the wall was only a function of the nail slip and sheathing panel shear deformation. The external energy due to external force was calculated and the horizontal deflection caused by nail joints was found by equating the external energy to

the internal energy due to the nails' distortion. Comparison with the experimental results showed that the model was capable of estimating the racking deflection at low load level, while it underestimated the deflection at higher load levels. The reason given was that the model assumed the resistance is increasing indefinitely with increasing deflection.

2.2.2. Mechanics-Based Models

The first version of the deflection equation was derived in 1952 based on diaphragm deflection (Countryman 1952). In the development of the deflection equation, six quarter-scale and four full-scale specimens were tested. The original equation developed was similar to that published in the current standard (CSA 2014) with the exception of the omission of the hold-down elongation contribution. The shear wall was considered as cantilevered I-beam with a point load applied at the free end. Other assumptions are that the wall is blocked and uniformly nailed. The development of the deflection equation in the format it appears in the design standard today is provided below.

The general format of the 4-terms deflection equation is presented in equation 2.1.

$$\Delta_t = \Delta_b + \Delta_s + \Delta_n + \Delta_a \quad (2.1)$$

Where, Δ_t is the total horizontal deflection of the wall, Δ_b is bending of the end studs, Δ_s is panel shear deformation, Δ_n is horizontal deflection caused by nail slip and Δ_a is the horizontal deflection caused by hold-down elongation.

For a cantilever wall with a concentrated load applied at the free end (see Figure 2.1), the bending deflection could be expressed as equation (2.2).

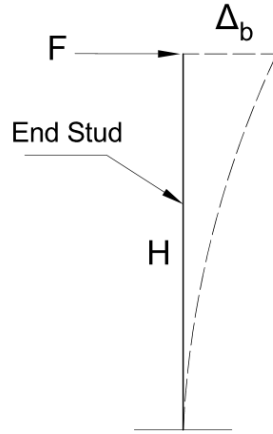


Figure 2.1: Bending Term

$$\Delta_b = \frac{FH^3}{3EI} \quad (2.2)$$

Where, F is the concentrated load at the free end, H is height of the wall, E is the wall modulus of Elasticity and I is the moment of inertia of the shear wall (Eq. 2.3).

$$I = 2A\left(\frac{L}{2}\right)^2 = \frac{AL^2}{2} \quad (2.3)$$

Where, L is length of the wall and A is the cross sectional area of end studs.

The total force on the wall can be expressed as the uniform diaphragm force multiplied by the wall length as shown in Equation (2.4).

$$F = v * L \quad (2.4)$$

Therefore, bending deflection can be expressed as shown in equation 2.5.

$$\Delta_b = \frac{FH^3}{3EI} = \frac{v*L*H^3}{3E\left(\frac{AL^2}{2}\right)} = \frac{2vH^3}{3EAL} \quad (2.5)$$

It should be noted that since equation 2.5 is obtained based on cantilever beam assumption, its accuracy would highly depend on how the panel is attached to the studs which could influence the deflected shape of the end studs and thereby the contribution to the wall deflection.

The shear deformation due to panel deformation is illustrated in Figure 2.2.

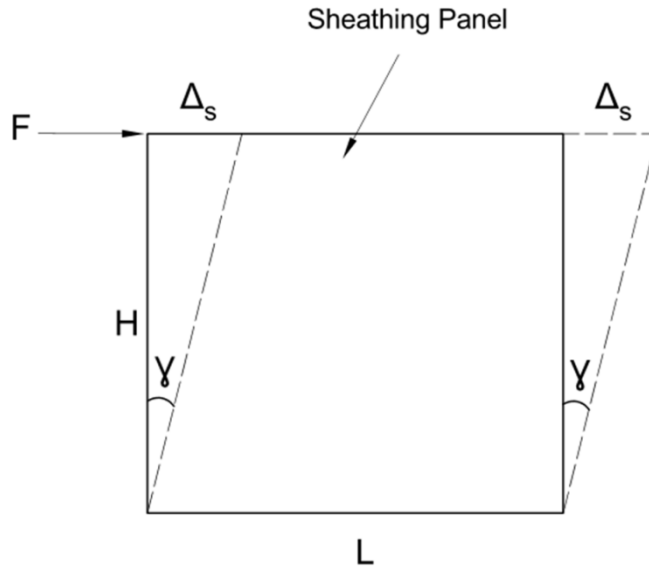


Figure 2.2: Shear Term

The shear strain in the panel can be written as shown in equation 2.6:

$$\gamma = \frac{\Delta_s}{H} \quad (2.6)$$

Where, Δ_s is the horizontal deflection in the panel and H is the height of the wall.

The relationship between shear stress and strain can also be written as shown in equation 2.7.

$$\gamma = \frac{\tau}{G} \quad (2.7)$$

Where, τ is shear stress and G is modulus of rigidity.

By equating 2.6 and 2.7, one obtains:

$$\frac{\Delta_s}{H} = \frac{\tau}{G} \quad (2.8)$$

$$\Delta_s = \frac{\tau \cdot H}{G} \quad (2.9)$$

Defining the shear stress as the force applied on the panel divided by the shear area (length x thickness), one obtains:

$$\tau = \frac{F}{L \cdot t} \quad (2.10)$$

The shear per unit length is equal to force divided by the length of the wall.

$$v = \frac{F}{L} = \tau \cdot t \quad (2.11)$$

Where, L is the wall length, t is the panel thickness.

The sheathing panel shear-through thickness rigidity (B_v) can be expressed as:

$$B_v = G \cdot t \quad (2.12)$$

The value can be obtained from Canadian wood design standard (CSA 2014).

The deflection due to panel shear deformation can be written as shown in equation 2.13:

$$\Delta_s = \frac{\tau \cdot H}{G} = \frac{v \cdot H}{\frac{B_v}{t}} = \frac{v \cdot H}{B_v} \quad (2.13)$$

Although the nail slip equation was originally derived for diaphragms, calculating the nail slip for a shear wall can be done by selecting the appropriate assumptions. In this derivation it is assumed that the nails at the panel edges are equally strained even if it is well established that the nails near the corners experience higher forces and therefore exhibited increased deflection. This assumption is considered adequate to represent the average estimate of the nail slip component.

Once the nail slip in the horizontal and vertical directions are known (e_n), the total nail slip at panel corner can be determined as $2\sqrt{2}e_n$ since for an 8'x8' panel, the elongation is at 45 degree angle with the panel edge (Figure 2.3). For other wall aspect ratios, the total deflection parallel with the panel diagonal (e'_n) can be calculated using the following equation:

$$e'_n = \sqrt{2}e_n \cos(45 - \beta) \quad (2-14)$$

Where, e_n is nail slip in both horizontal and vertical directions and β is the angel between e'_n direction and vertical.

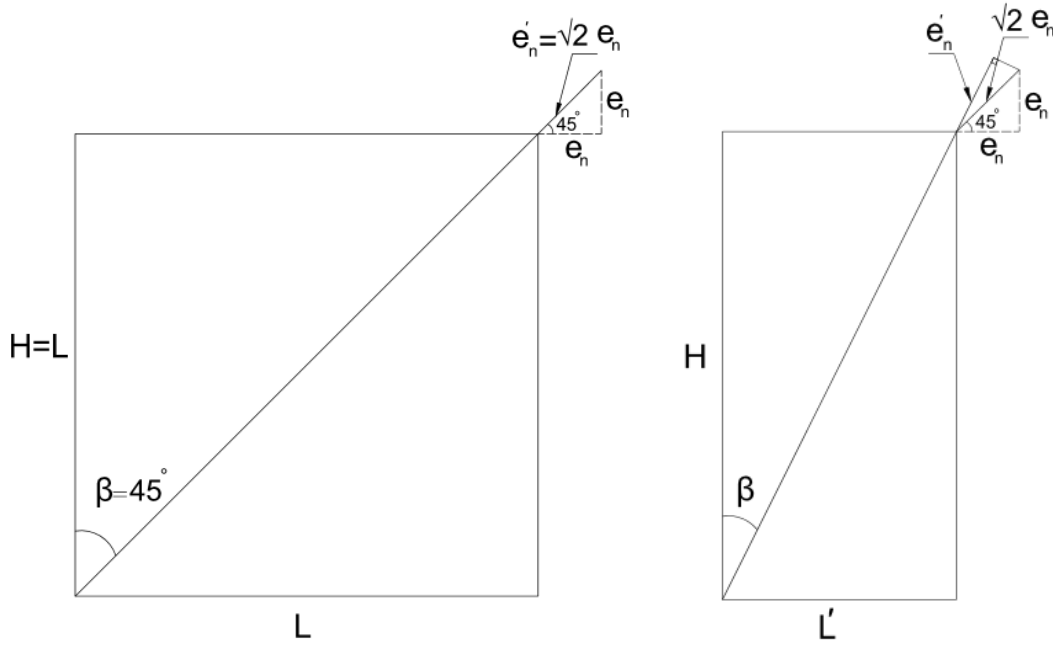


Figure 2.3: Nail Slip at Panel Corner for Walls with Different Aspect Ratios

Since the horizontal panel shear deformation is proportional to diagonal elongation of the panel, the shear wall's horizontal deflection caused by nails distortion may similarly be taken as proportional to nail deflection parallel with the panel diagonal (Countryman 1952). The relation is presented in equation 2.15.

$$\frac{\Delta_n}{e'_n} = \frac{\Delta_s}{\delta_s} \quad (2.15)$$

Where, δ_s is the diagonal extension or compression of the panel due to shear stress.

Following the above assumptions, Equation 2.16 could be derived to calculate the horizontal deflection caused by nails distortion:

$$\Delta_n = 0.0025He_n \quad (2.16)$$

The Canadian wood design standard (CSA 2014) contains an empirical equation to estimate the nail deformation (e_n) as expressed in equation 2.17.

$$e_n = \left(\frac{0.013 v_s}{d_f^2} \right)^2 \quad (2.17)$$

Where, v_s is load per nail in N and d_f is the nail diameter in mm.

The nail slip expression (equation 2.16) has been verified experimentally by other researchers (e.g. Wang 2009). However, the study showed that the nail slip expression was accurate only at low load levels.

The last term in the deflection equation considers the wall's rigid-body rotation, primarily due to hold-down slip. The horizontal wall deflection due to overturning is a function of the vertical displacement due to rigid body rotation multiplied by the wall's aspect ratio, as shown in equation 2.17:

$$\Delta_a = \frac{H d_a}{L} \quad (2.17)$$

When a hold-down is not provided, the uplift component is taken by some of the panel to framing joints and an expression is provided in the Canadian wood design standard (CSA 2014) to approximate d_a (equation 2.18):

$$d_a = 2.5 d_F K_m \left[\frac{(v H_s - P_{ij}) \frac{S_n}{L_s}}{n_u} \right]^{1.7} \quad (2.18)$$

where, d_F is nail diameter, K_m is creep factor, v is maximum shear force per unit length due to specified load, H_s is height of the wall, P_{ij} is the specified uplift restraint force, S_n is nail spacing around panel edge, L_s is length of the wall and n_u is unit lateral nail resistance.

When hold-down is provided, d_a can be obtained from the manufacturer since it highly depends on the hold-down system used.

The total shear wall deflection can be expressed as shown in equation 2.19 (CSA 2014):

$$\Delta_t = \frac{vH^3}{3EI} + \frac{vH}{B_v} + 0.0025He_n + \frac{Hd_a}{L} \quad (2.19)$$

A similar expression can be found in the US design standard (NDS 2012) and other design guidelines (APA 2007). Although the background for the expression is the same as that used in the Canadian design standard, different methods are used to calculate the nail slip (e_n) component.

In addition to the rigid body rotation contribution caused by the hold-down slip, compression perpendicular to grain deformation, d_c , may also contribute to the horizontal wall deformation, as can be seen in Figure 2.4 (Salenikovich 2000).

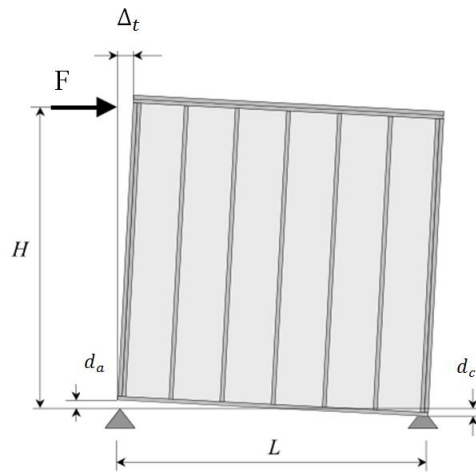


Figure 2.4: Wall Rigid Body Rotation Contributors (Salenikovich 2000)

To incorporate this effect in the deflection equation (Eq. 2.19), the last term needs to be replaced by the following term (Salenikovich 2000):

$$\frac{H(d_a + d_c)}{L} \quad (2.20)$$

Attempts have also been made to simplify the deflection equation. For example, Martin (2002) proposed a nonlinear equation and combined the three first terms of 4-term equation as shown in equation 2.21.

$$\Delta = \left(\frac{vh}{1000C} \right)^{1.9} + \frac{hd_a}{b} \quad (2.21)$$

Where, h is the wall height, C is a constant calibrated to the 4-term equation, d_a is vertical elongation of hold-down system and b is the length of the wall.

The author of the study concluded that the proposed model fits the 4-term equation very well, however, in the case of dense nail spacing, it under-predicts the deflection at low loads (Martin 2002). The same study also proposed a simplified linear equation for estimating the wall deflection:

$$\Delta = \left(\frac{vh}{1000G'} \right) + \frac{hd_a}{b} \quad (2.22)$$

Where, G' is the scant stiffness based on the load-displacement predicted by the 4-term equation. For both the nonlinear and linear equations, the calibrated constants (C and G') did not change for walls with different aspect ratios. The author concluded that the hold-down slip (which is not considered in the calibration) is sensitive to aspect ratio (Figure 2.5).

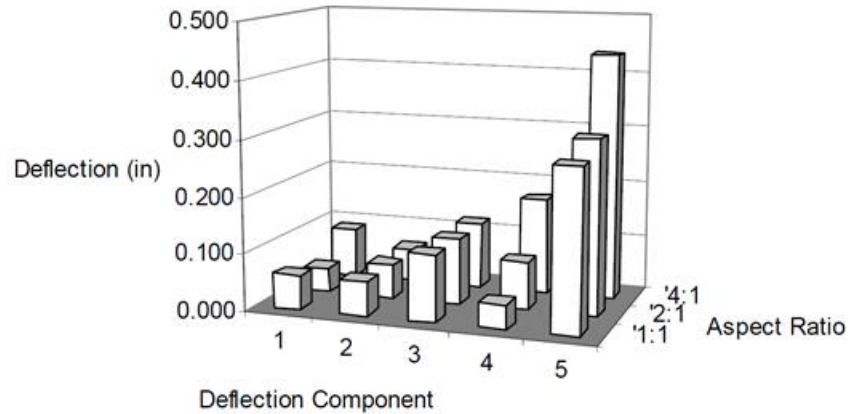


Figure 2.5: Shear Wall Deflection by Component of the 4- Term Deflection Equation:

1: Bending Term 2: Shear Term 3: Nail Slip 4: Hold-Down 5: Summation of 1 to 4

(Martin 2002)

2.3. Test Standards

In North America, the American Society of Testing and Materials (ASTM) provides guidance to researchers on standardized methods to conduct experimental studies that are consistent and harmonized amongst different labs. The main documents that are used for the purpose of wood shear wall testing procedures are: ASTM E564-06 (2012), ASTM E72 (2014) and ASTM E2126 (2011).

ASTM E564-06 (2012) focusses on testing wall panels only under monotonic loads to determine shear stiffness and strength of the wall assembly. The standard does not provide rigid framework regarding the test configurations and wall size (e.g. aspect ratio) may vary depending on the scope of the investigation. Walls are required to be attached to their base using anchorage connections and gravity load may be applied to resist rigid body rotation and reflect actual construction conditions. Preloading of 10% of estimated ultimate load is applied and loads are to

be applied at a constant rate of displacement to reach the target limit in a time frame that is not less than 5 minutes.

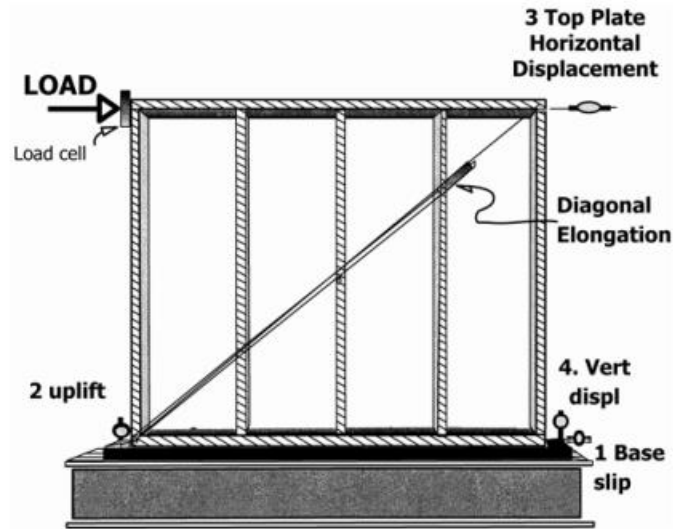


Figure 2.6: Shear Wall Configuration in ASTM E564-06 (2012)

ASTM E72 (2014) is similar in principle to the procedure provided in the ASTM E564 standard outlined in the previous paragraph. The standard includes testing procedures for walls, floors and roofs subjected to different load conditions (e.g. concentrated, impact, and racking loads etc.) in order to determine the structural properties of those components. For wood shear walls, the size of the specimen is set to 8 by 8 ft. (2.4 by 2.4 m) and the wall framing are constructed as detailed in Figure 2.7.

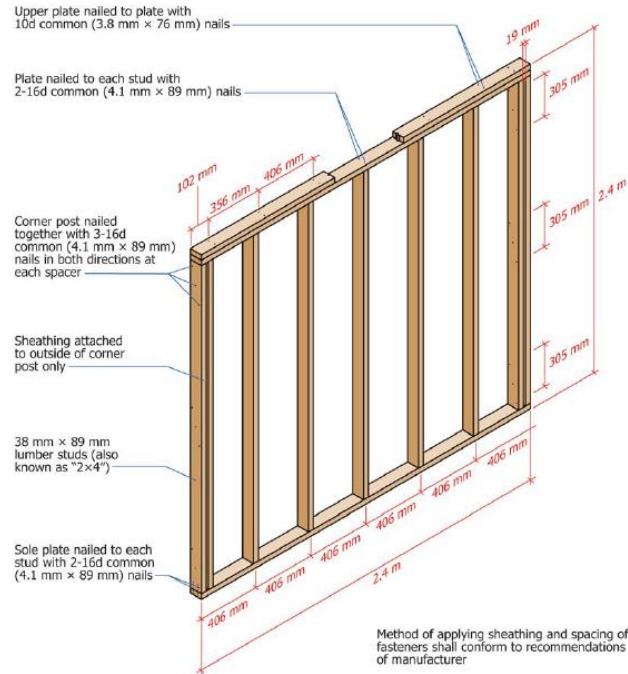


Figure 2.7: Standard Wood Frame (ASTM 72-14)

In this standard, the test assembly is attached to the rigid base using bolts and hold-downs to resist shear and overturning (Figure 2.8). As a result the walls tested according to this standard has the tendency to show higher stiffness compared to those tested using the ASTM E564-06 (2012) (Suzuki et. al 1978, Griffiths 1984).

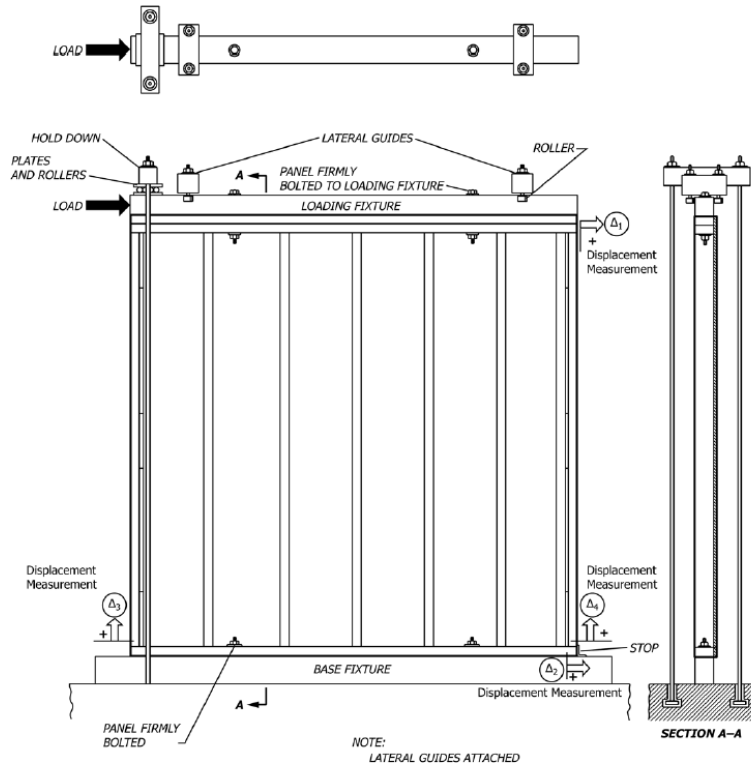


Figure 2.8: Shear Wall Test Setup (ASTM 72-14)

The ASTM E2126 (2011) standard focuses on the resistance of shear walls under cyclic loading and includes three different loading protocols. In this standard, there is no requirement limiting the wall size. Also, the wall assembly is required to be attached to the base through bolts and hold-downs to overcome the uplift at the tension side of the wall (Figure 2.9).

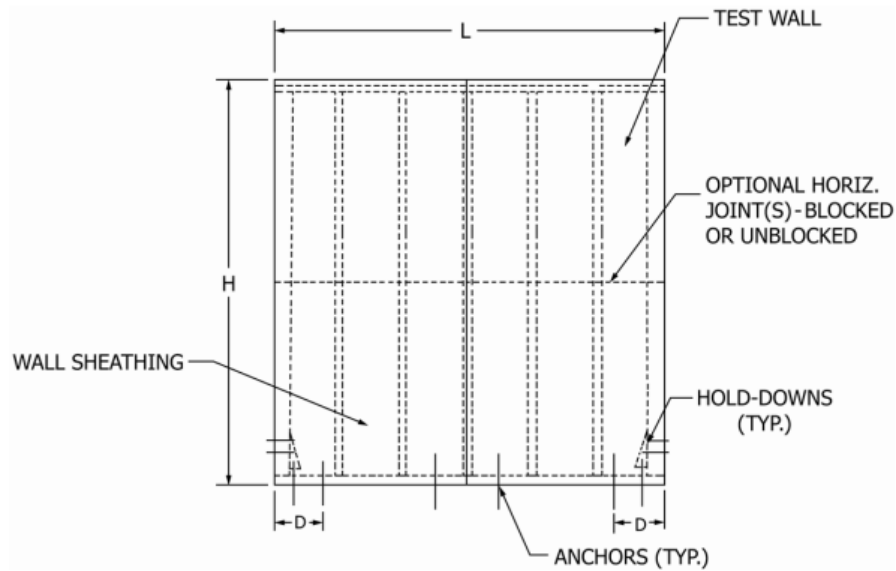


Figure 2.9: Shear Wall Configuration (ASTM E2126 2011)

2.4. Experimental Studies

This section describes research with predominantly experimental work related to single-storey light-frame wood shear walls.

Dean et. al. (1986) studied the hysteretic behavior of wood shear walls by testing 11 full-scale walls subjected to static cyclic as well as shake table loading. Both panel thicknesses and nail spacing were varied in this study. The research mainly focused on the dynamic response of nails, framing connections (behavior of the joints between the framing members) and foundation connections (hold-down). The authors concluded that the nail slip was the main contributor to the wall deflection and to the ductility of the wall when adequate sheathing panel thickness was provided.

Mallory et. al. (1984) investigated the racking behavior of 20 small-scale shear walls sheathed with plywood and GWB. The height of all the walls was 2 feet (609.6mm), while different wall length of 2, 4, 6 and 8 feet were used (resulting in aspect ratios of 1:1, 1:2, 1:3 and 1:4). The aim

of the study was to establish a relationship between the wall length and capacity. The study also aimed at investigating whether a two-sided shear wall performance can be obtained by simply summing the behaviour of the individual single-sided walls. The study concluded that the racking resistance of the walls sheathed with plywood was directly proportional to the length of the wall. However racking resistance of gypsum-sheathed walls was not directly proportional to the wall length but could be estimated by linear relationship. It was also concluded that the performance of double-sided walls could be predicted by the sum of individual single-sided wall racking behavior.

Dolan and Madsen (1991) tested a total of 11 full-scale shear walls to investigate the hold-down influence on the total behavior of the wall. Their results showed that installing hold-down increased the capacity of the wall and reduced the rigid body rotation. Moreover, when hold-down was used, the gravity load had insignificant effect on the racking performance of the wall.

Lam (1997) studied the lateral resistance of 11 2.4m x 7.3m wood shear walls built of oversized sheathing panels under monotonic and cyclic loading. Eight of the shear walls were built using one large 2.4m x 7.3m sheathing panel instead of using the standard 2.4m x 1.2m OSB panels. Nail spacing was also changed to investigate its influence on the behavior of the wall. Reduced nail spacing (used as wall reinforcement) was also applied only at the wall corners. The monotonic test results showed that decreasing nail spacing increased the wall's stiffness and strength. Reducing nail spacing at the four corners showed no tangible improvement in both stiffness and load carrying capacity. The wall constructed with one large sheathing panel with 76mm nail spacing was also compared with the wall made of standard OSB panel with 152mm nail spacing. It was concluded that, although the number of nails used to build the wall with large

panel was less than the conventional wall, the wall's loading capacity and stiffness more than doubled.

Salenikovich et al (2000) conducted monotonic and cyclic testing on 56 shear walls with different aspect ratios (1:1, 2:1, 4:1, 2:3) and anchorage conditions (full anchorage, intermediate anchorage and no anchorage). The height of all the walls was 2.4m (8ft.) and OSB panels were fastened to the wall framing through 8d (2.84mm diameter) common nails. The influence of nail edge distance on the wall behavior was also investigated. The study found that the loading regime (monotonic vs. cyclic) did not affect the initial stiffness and the strength of the wall. Narrow walls (4:1) showed lower stiffness as their deflections were magnified in proportion to the aspect ratio; however shear strength of the narrow walls did not degrade at higher deflections due to small displacement demand on sheathing-to-framing connections. It was also found that the deformation at peak load and failure was highly influenced by edge distance of the nails for unrestrained walls (walls without hold-down). The author concluded that the minimum edge-distance requirement by the Building Seismic Safety (BSSC 1998) guidelines (10 mm) might not be sufficient in providing ductile resistance of the shear wall and should be increased. The author also concluded that the bending and shear deformation were linearly proportional to the racking force. However, nail slip and hold-down elongation showed nonlinear behavior and were key controlling factors in wall performance.

Sang (2000) tested 16 wood shear walls with different aspect ratios (i.e. 1:1 to 6:1) and various openings under reversed cyclic loading. The effect of sheathing material, stud spacing and hold-down were also investigated. The study concluded that walls with OSB and plywood panels had similar behavior. Also, it was concluded that initial stiffness increased as stud spacing decreased. The results showed that walls with higher aspect ratio had lower racking resistance and that

using hold-down increased the racking resistance. The study also found that the ultimate strength of the shear wall was linearly proportional to the shear wall length.

Ni and Karacabeyli (2002) investigated the behavior of shear walls with and without hold-downs. The study also investigated the effect of wall length and gravity load on the behavior of the walls without hold-downs. It was observed that the amount of vertical load and aspect ratio influence the capacity of the walls. Based on the results, it was noted that when sufficient vertical load is applied, walls without hold-down perform similar to walls with hold-downs. Without the presence of vertical load, walls without hold-down achieved 50% of the capacity of fully restrained walls.

Martin et. al. (2002) tested 89 shear walls with different configuration and aspect ratios. The aim of the study was to investigate the influence of the loading protocol (monotonic vs. cyclic) on the behavior of the walls. The study also aimed at comparing the measured deflection obtained from the tests to that calculated using the 4-term deflection equation (equation 2.19). It was found that the 4-term equation under-predicted the deflection in general, which was more pronounced for narrow walls (aspect ratio of 4:1) where crushing of the bottom plate led to higher deflection. Differences were found between cyclic and monotonic loading in this study. However, the authors noted that differences in boundary conditions could have been the source of inconsistency. It is also noteworthy to mention here that several studies (e.g. Dolan 1989, Karacabeyli et. al 1996, Dolan 1997, Lam 1997, He et. al 1998, Gatto and Uang 2001) have shown that the initial stiffness as well as the general behavior is the same when comparing backbone curve from cyclic testing of shear walls to monotonic tests. In fact, loading protocol has little impact on initial stiffness.

Karim (2003) conducted 4 full-scale shear wall tests subjected to cyclic loads, where sheathing material was the only parameter changed. The main focus was on the energy dissipation capacity of the walls using different sheathing material. The study concluded that the performance of walls constructed with plywood performed better than those constructed with OSB. It was reported that walls with plywood dissipated more energy and experienced higher load level compared to OSB walls.

Seaders (2009) studied the behavior of 20 wood shear walls under different load protocols with and without hold-down. The results of the study showed that using hold-down had a significant impact on the load carrying capacity, deflection and energy dissipation capacity. The results showed that the load carrying capacity and deflection exhibited an increase by a factor of two, while energy dissipation increased by factor of 9 when hold-down was used. The study also found that using hold-down changed the failure mode (by changing load path), where partially anchored walls (walls without hold-down) exhibited only failure at sheathing to sill plate nail connections without any visible damage in other nails. In contrast, fully anchored walls showed more distributed damage throughout the wall, where failure was mainly observed in the nail joints.

Wang et al (2009) suggested an empirical relationship to predict the nail slip behavior based on testing 135 nail-joint specimens. The proposed nail-slip relationship considered lumber density, sheathing thickness, and nail diameter. To investigate the accuracy of the model in improving the wall deflection, 13 shear walls with different configurations were tested. Two different sheathing thickness (9.5mm and 18.5mm) and two different nail diameters (2.5mm and 4.1mm) were considered. Also, the impact of nail spacing at panel edges on the wall behavior was investigated, where two nail spacing (50mm and 150mm) were considered in the test program.

Two methods of uplift prevention including hold-down and tie-rod were used to compare their impact on the wall performance. The author concluded that the 4-term equation modified by the proposed nail slip term matched the wall deflection more closely near ultimate load when compared to APA model (APA 2007). However, the model still under-predicted the wall deflection as shown in Figure 2.10.

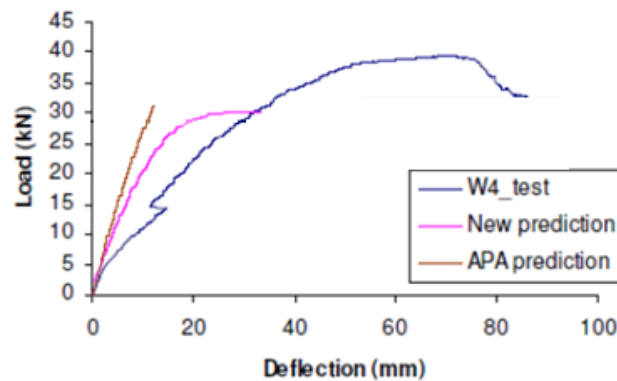


Figure 2.10: Comparison of Wang (2009) Model vs. APA (2007) Model (Wang 2009)

The study also found that walls with tie-rods experience higher ultimate load as well as higher deflection at yield point compared to walls with hold-down. Nail spacing at panel edges influenced the initial stiffness and ultimate load where, decreasing the nail spacing at panel edges from 150mm to 50 mm increased the initial stiffness and ultimate load by the factor of 1.1 and 3.0, respectively. The results indicated that the wall strength is proportional to the number of the nail spacing at the panel edge. However, the results differed for larger nails where, decreasing the nail spacing at panel edges from 150mm to 50 mm increased the initial stiffness and ultimate load by the factor of 1.2 and 1.5, respectively.

Sartori et. al. (2012) tested a total of 22 walls monotonically and cyclically. The sheathing panel, vertical load, hold-down and nail spacing were varied. The results showed that applying 10 and

20 kN/m vertical load increased the wall capacity by 20 and 28 %, respectively, compared to the case with no vertical load. It was also concluded that decreasing nail spacing increased wall stiffness significantly. The ultimate loading capacity of the wall was found to be influenced by nail spacing. To investigate the influence of sheathing panel, three cases were considered, where the wall was sheathed with OSB on both sides, with Gypsum Fiber Board (GFB) on both sides, or with OSB on one side and GFB on the other side. The results showed that using GFB reduced the ductility of the wall. The study also showed that hold-down influenced the capacity and the stiffness when it became active.

Casagerande et. al. (2016) investigated the capacity, stiffness and ductility of timber-to-timber and timber-to-steel connections fastened through nails and screws. Two full-scale walls of Light-Timber Frame (LTF) and four Cross-Laminated Timber (CLT) walls were tested. Sheathing-to-framing connection failure was the main source of LTF walls failure while, hold-down failure was the predominant failure mode of CLT walls. The author concluded that since sheathing-to-framing connection is more ductile compared to hold-down connection, a higher ductility of LTF walls could be achieved compared to CLT walls.

2.5. Deflection of Multi-Storey Wood Shear Walls

The cantilever beam approach (section 2.2.3) could be extended to include stacked, multi-storey wood shear walls by incorporating the rotation at lower levels, which accounts for the cumulative deflection. Multi-storey expressions have been developed and incorporated in the Canadian wood design standard (CSA 2014), as shown in Equation (2.23):

$$\Delta_i^{Total} = \sum_{j=1}^i \Delta_j^{Storey} \quad (2.23)$$

In which, Δ_i^{Total} is total deflection at the i-th level and Δ_i^{Storey} is interstorey drift at the i-th level, as expressed in Equation 2.24.

$$\Delta_i^{Storey} = \Delta_{b,i}^{Storey} + \Delta_{s,i}^{Storey} + \Delta_{n,i}^{Storey} + \Delta_{a,i}^{Storey} \quad (2.24)$$

Where $\Delta_{b,i}^{Storey}$ is interstorey drift at the i-th level due to bending, $\Delta_{s,i}^{Storey}$ is interstorey drift at the i-th level due to panel shear, $\Delta_{n,i}^{Storey}$ is the interstorey drift at i-th level due to nail slip and $\Delta_{a,i}^{Storey}$ is interstorey drift at the i-th level due to vertical elongation of the wall anchorage system.

2.5.1. Deflection Due to Bending

Deflection due to bending at i-th level consists of the stud bending based on cantilever assumptions and the cumulative bending due to the rotation of the bottom levels:

$$\Delta_{b,i}^{Storey} = \Delta_{b,i} + H_i (\sum_{j=1}^{i-1} \theta_j) = \Delta_{b,i} + H_i (\theta_1 + \theta_2 + \dots + \theta_{i-1}) \quad (2.25)$$

Where, $\Delta_{b,i}$ 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, H_i is the height of the wall at i-th storey and θ_j is the change in slope over the j-th storey height subjected to bending.

The definition of θ_j and $\Delta_{b,i}$ is shown in Figure 2.11 and is calculated as shown in equation 2.26 and equation 2.27 respectively.

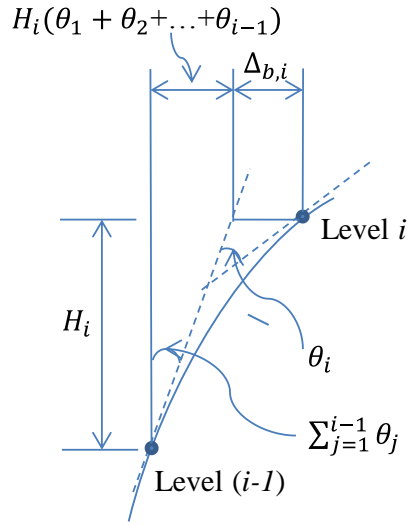


Figure 2.11: $\Delta_{b,i}$ and θ_j definition

$$\theta_j = \frac{M_j H_j}{(EI)_j} + \frac{V_j H_j^2}{2(EI)_j} \quad (2.26)$$

Where, M_j is overturning moment at level j , $(EI)_j$ is bending stiffness of the shear wall at the j -th storey and V_j is shear force in shear wall at the j -th storey.

$$\Delta_{b,i} = \frac{M_i H_i^2}{2(EI)_i} + \frac{V_i H_i^3}{3(EI)_i} \quad (2.27)$$

Figure 2.12 shows the deflected shape of the multi-storey wall assemblies due to bending.

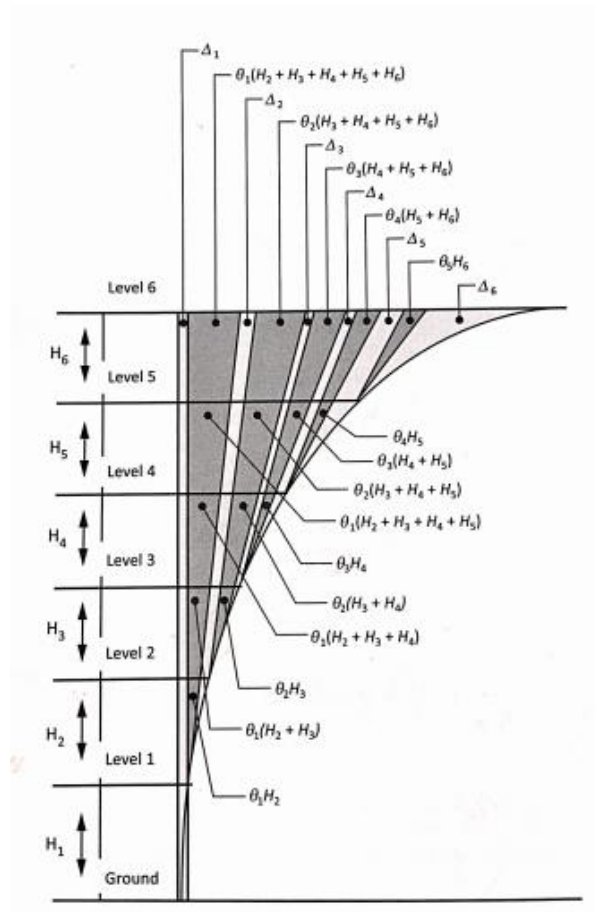


Figure 2.12: Deflection Due to Bending

2.5.2. Deflection Due to Panel Shear

There is no cumulative effect in multi-storey walls when deflection due to panel shear is calculated:

$$\Delta_{s,i}^{storey} = \Delta_{s,i} \quad (2.28)$$

Where,

$$\Delta_{s,i} = \frac{V_{f,i}H_i}{B_{v,i}} \quad (2.29)$$

$V_{f,i}$ is the maximum shear per unit length due to specified loads at the top of the wall at the i-th storey in N/mm and $B_{v,i}$ is shear-through-thickness rigidity of the sheathing at the i-th storey in N/mm.

2.5.3. Deflection Due to Nail Slip

There is no cumulative effect in multi-storey walls when deflection due to nail slip is calculated:

$$\Delta_{n,i}^{Storey} = \Delta_{n,i} \quad (2.30)$$

Where,

$$\Delta_{n,i} = 0.0025H_i e_{n,i} \quad (2.31)$$

Where, $e_{n,i}$ is the nail deformation at the i-th storey in mm.

2.5.4. Deflection Due to Elongation in Wall Anchorage System

Similar to bending, the rotation at bottom levels due to anchorage system elongation causes cumulative deflection at top levels as follows:

$$\Delta_{a,i}^{Storey} = H_i (\sum_{j=1}^i \alpha_j) = H_i \alpha_i + H_i (\sum_{j=1}^{i-1} \alpha_j) \quad (2.32)$$

Where,

$$\alpha_j = \frac{(d_a)_j}{L_s} \quad (2.33)$$

Where,

$(d_a)_j$ is the total vertical elongation of the wall anchorage system at the j-th storey.

Figure 2.13 and Figure 2.14 show the deflection due to vertical elongation of the wall anchorage system.

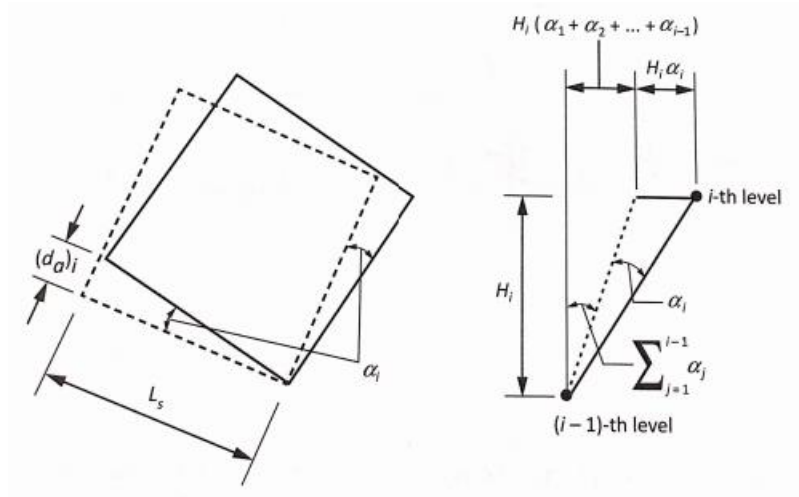


Figure 2.13: Deflection Due to Vertical Elongation of the Wall Anchorage System at the i-th Storey (CSA 2014)

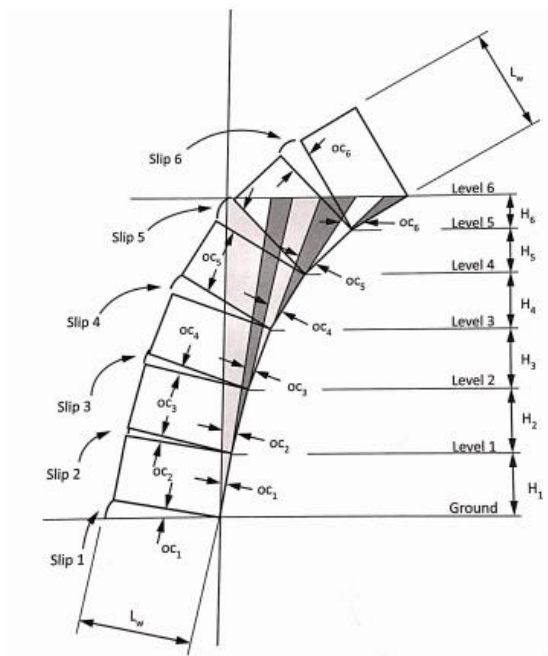


Figure 2.14: Deflection Due to Vertical Elongation of the Wall Anchorage System (CSA 2014)

The Canadian wood design standard (CSA 2014) recognizes that the multi-storey approach represents one option that is yet to be validated by experimental results. The standard also notes that this approach is conservative for determining the deflections and inter-storey drifts as it likely underestimates the shear wall stiffness. This further emphasizes the need for reliable estimates of the shearwall stiffness, especially for multi-storey shearwalls.

A key study focusing on the deflection of multi-storey walls is that by Pei et. al. (2009) who developed a coupled shear-bending model to predict the dynamic response of multi-storey shear walls. The main focus of the study was on taking into account the bending deformation associated with rotation of the diaphragm due to rod elongation. Six rigid-body degrees of freedom (DOF) were assigned to each storey diaphragm and all stiffness components were represented as springs as shown in Figure 2.15.

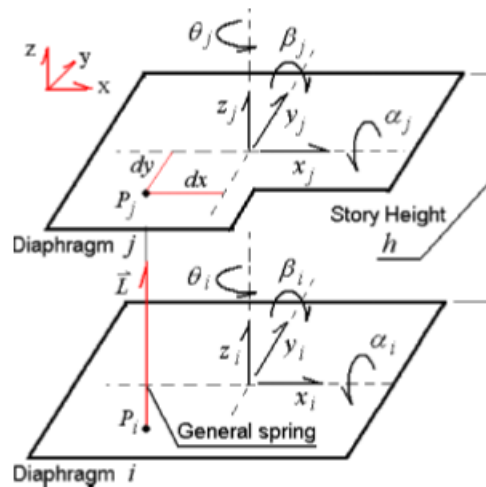


Figure 2.15: Global DOF and Spring Connectivity (S. Pei et. al. 2009)

Two springs represented the wall performance, one was the evolutionary parameter hysteric model (EPHM) to simulate the general hysteric behavior of the wall and the other one was double-linear spring to model the vertical stiffness of the hold-down system (Figure 2.16).

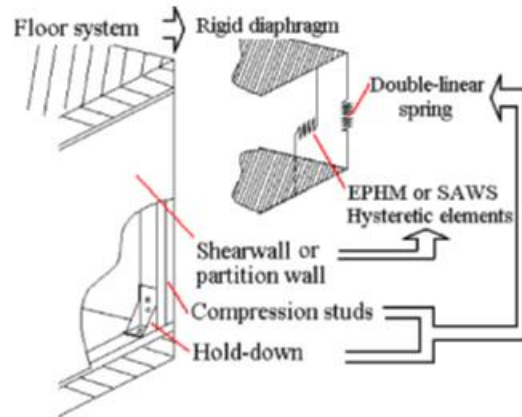


Figure 2.16: Shear Wall Components in SAPWood V2.0 (S. Pei et. al. 2009)

The authors also validated their model by testing a three-storey wood shear wall assembly with steel rods as continuous hold-down devices on the uniaxial shake table (Pei et. al. 2009). The setup configuration is shown in Figure 2.17. The diaphragms were allowed to rotate freely during the test to allow uplift accumulation in the system. Although the experimental setup physically captures the presence of the floor joists, allowing the floor diaphragm to undergo rigid body rotation does not represent the out of plane stiffness of the floor adequately.

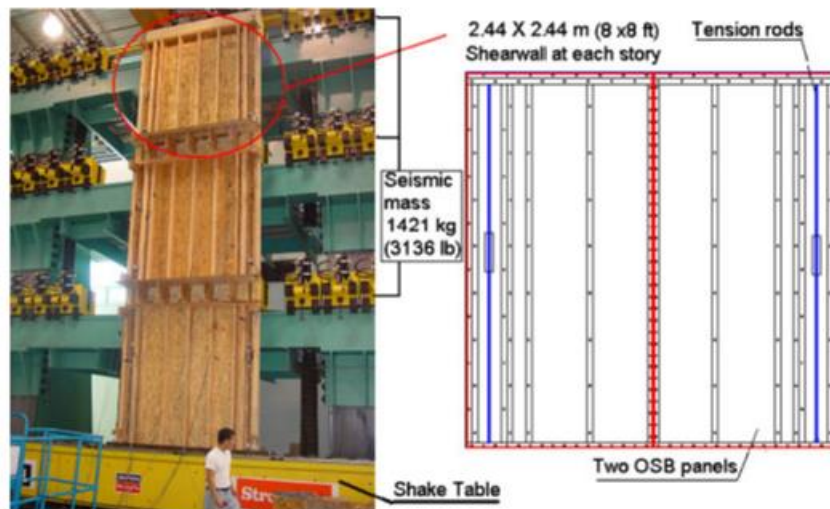


Figure 2.17: Three-Storey Shear Wall Test Configuration (S. Pei et. al. 2009)

Comparing the time history responses between the test results and the model showed that the coupled shear-bending formulation was able to capture the behavior of high aspect ratio wall assemblies.

2.6. Summary

This chapter reviewed key research studies on the subject of light-frame wood walls behavior under the effect of lateral loads. Parameters that affect the wall behavior include sheathing panel thickness, nails diameter and spacing, aspect ratio and whether hold-down is used in the construction of the wall. In general it is shown that current models found in the design standards tend to under-predict the wall deflection (e.g. Martin 2002, Wang 2009). Also, most of the studies have focused on the wall behavior as a whole with no investigation targeting specifically the individual components of the deflection (bending, panel shear, nail slip and overturning).

The current study aims to build on existing knowledge by focusing on the contribution of the individual components in the deflection equation. The study also aims at investigating the validity of the cantilever assumption for multi-storey shearwalls. The outcome of this research will contribute to the knowledge base and help designers better evaluate the behavior of single and multi-storey wood light frame shear walls.

3 EXPERIMENTAL TESTING PROGRAM

3.1 Materials

This section describes all the materials involved in the experimental testing program including sheathing panels, framing members, fasteners and hold-down connectors.

3.1.1 Sheathing Panels

The sheathing panel types used in this study consisted of 1220 mm x 2440 mm (4' x 8') Oriented Strand Boards (OSB) and Plywood due to their wide use in light frame wood construction. Two different thicknesses of OSB, namely 11 mm and 15.5 mm and 12.5 mm Plywood panels were investigated. The selection of thickness was based on the material's commercial availability.

3.1.2 Framing Lumber

Spruce-Pine-Fir (S-P-F), No.2 grade was used as framing members to construct the shear walls in this study. This was primarily due to their wide availability and use in light-frame walls especially in eastern Canada. Two different sizes consisting of 2" x 4" (38 x 89mm) and 2" x 6" (38 x 140mm) were investigated.

3.1.3 Fasteners

Nails used to connect the sheathing to the framing and the framing members together consisted of common wire nails with smooth-shank. Table 3.1 shows all nail sizes used in the current study.

Table 3.1: Nail Sizes Used in This Study

Nail Size	Nail Diameter in mm (inch)	Nail Length in mm (inch)
6d	2.84 (0.113)	50.8 (2)
8d	3.32 (0.131)	60.32 (2 3/8)
12d	3.76 (0.148)	82.55 (3 1/4)

3.1.4 Hold-Down

The hold-down system used in the full-scale tests included HDU2 and HDU4 Simpson Strong-Tie hold-downs as well as Simpson Strong-Tie anchor tie-down system (ATS). A typical HDU hold-down with dimensional characteristics together and type of fasteners used are presented in Figure 3.1 and Table 3.2. All the variables in Table 3.2 are defined in Figure 3.1.

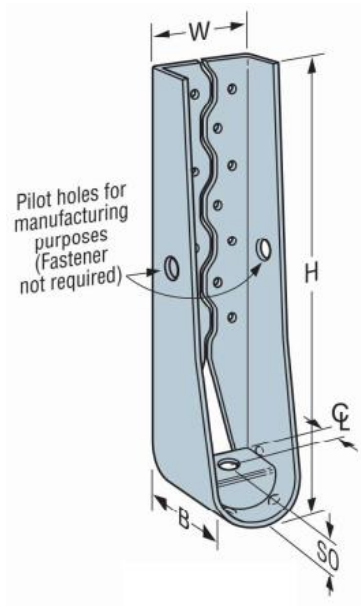


Figure 3.1: Typical HDU Hold-Down (Simpson Strong-Tie 2017)

Table 3.2: Hold-Downs Characteristics Used in This Study (Simpson Strong-Tie 2017)

Model NO.	Dimensions (in)					Fasteners	
	W	H	B	CL	SO	Anchor Bolt Dia. (in)	End Stud Fasteners
HDU2	3	$8\frac{11}{16}$	$3\frac{1}{4}$	$1\frac{5}{16}$	$1\frac{3}{8}$	5/8	(6) $\frac{1}{4}$ "x 2 $\frac{1}{2}$
HDU4	3	$10\frac{15}{16}$	$3\frac{1}{4}$	$1\frac{5}{16}$	$1\frac{3}{8}$	5/8	(10) $\frac{1}{4}$ "x 2 $\frac{1}{2}$

As seen in the Figure 3.2, the hold-down is attached to the foundation through 5/8 inch anchor bolts while also fastened to the end studs using 1/4" self-tapping screws.



Figure 3.2: HDU2 Hold-Down Installation

The tie-rod system used in this study consisted of 5/8" diameter steel bar with bearing plates and coupler nuts (Figure 3.3). The particular system used in this study had an allowable tension capacity (design level) of 29.7 kN (Simpson Strong-Tie 2017).



a) Threaded Rod b) Bearing Plate c) Coupler Nut

Figure 3.3: ATS System Components (Simpson Strong-Tie 2017)

The tie-rod was installed in the stud cavity near the end stud (Figure 3.4(a)) where one end was connected to the top plate of the wall through anchor bolt and bearing plate (Figure 3.4(b)), and the other end was anchored to the foundation beam through an anchor bolt. The coupler nut was placed on the threaded rod and rotated clockwise until the threads were visible through the witness hole on the coupler nut. Figure 3.5 shows the principle of the ATS system installed on a two-storey wall.



a) Tie-Rod Installation



b) Bearing Plate Installation

Figure 3.4: ATS System Installation

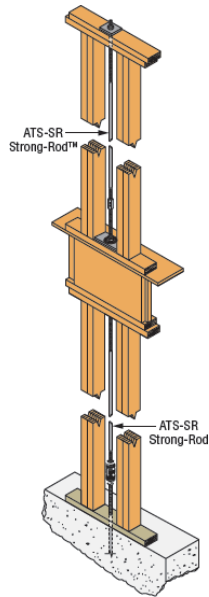
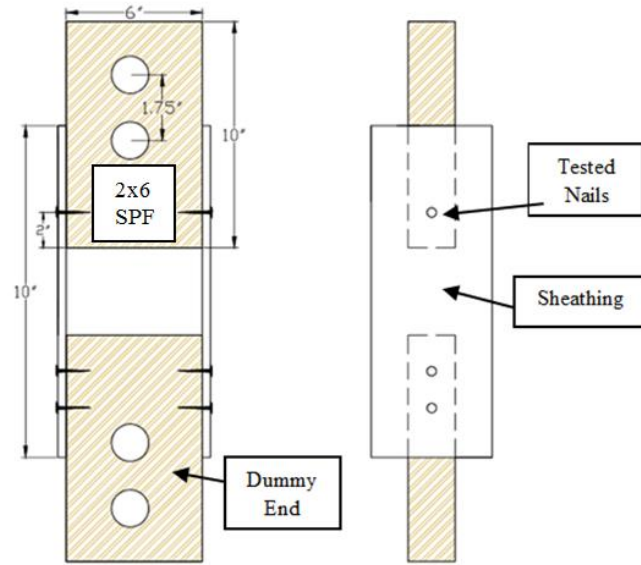


Figure 3.5: ATS System Installation (Simpson Strong-Tie 2017)

3.2 Specimen Fabrication

3.2.1 Nail Joint Specimens Fabrication

The test specimens consisted of two 2"x 6" (38 x 140mm) wood pieces with length of 10" (254mm) and with either 4"x10" (101.6 x 254mm) OSB or plywood pieces fastened on both sides to the framing member using nails (Figure 3.6). Two 3/4" (19.1mm) holes were pre-drilled in each framing piece. One end was considered as test joint while the other was the dummy end. The specimen had a nail placed on each side of the lumber section at the tested end. To minimize slip at the dummy end, four 12d nails with the length of 3 1/4" (82.55mm) were used. During the test, the relative displacement between the sheathing panel and lumber piece at the test end was measured which means that any possible slippage at the dummy end would not affect the test results.



a) Nail Joint Test Specimen Sketch



b) Typical Joint Test Specimen

Figure 3.6: Nail Joint Specimen Fabrication

3.2.2 Fabrication of Full-Scale Wall Specimens

27 single-storey shear walls with different construction details and aspect ratios were tested under static racking loading. The height of all the walls was 2.4m (8') while various wall lengths of 0.4 m, 0.6 m, 0.8 m, 1.2 m and 2.4 m were used. This resulted in aspect ratios of 6:1, 4:1, 3:1,

2:1 and 1:1. The walls consisted of two top plates, one bottom plate and end studs in conformance with the ASTM 72 standard (2014).

Stud spacing of 12" (304.8mm), 16" (406.4mm) and 24" (609.6mm) were investigated. The studs were connected to the top and bottom plates using 3 1/4" (12d) smooth-shank common nails. The walls were sheathed with 4'x 8' (1.2 x 2.4 m) OSB or plywood panels on one side only. Nail spacing of 75mm or 150mm were used. The wall hold-down system consisted of either HDU hold-down or tie-rod at the tension side of the wall.

Two different cases of half-scale two-storey walls were also tested. The length and height of the walls were 1.2m (4') and they consisted of two top plates, one bottom plate and studs spaced at 16" (406.4mm) on center. The walls were sheathed with 4' x 4' (1.2 x 1.2 m) OSB panels on one side only. Nail spacing of 150mm were used to connect the sheathing to the framing members. HDU hold-downs were used at the tension side of the wall.

3.3 Test Setup and Procedure

3.3.1 Nail Joint Test Procedure

The universal testing machine (UTM) was used for the component testing using displacement controlled loading with a rate of 2.5 mm/minute. Two linear variable differential transducers (LVDT) were used on both sides of the specimen to measure the relative slip between lumber and sheathing. Figure 3.7 shows the nail joint test setup.

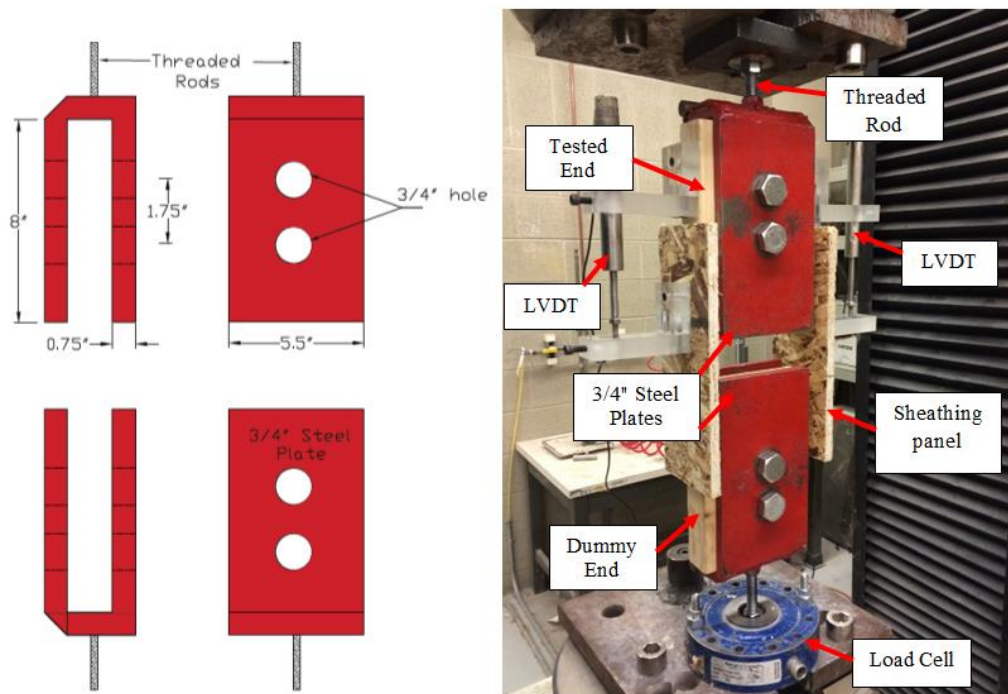


Figure 3.7: Nail Joint Test Setup

As shown in Table 3.3, the nail joint testing program included OSB and plywood panels while different nail sizes (based on market availability) were used to fasten the sheathing to the lumber.

Table 3.3: Nail Joint Test matrix

NO	Sheathing	Nail Length in mm (in)	Nail Diameter in mm	Repeat
JS-1	11mm OSB	50.8 (2)	2.84	4
JS-2	11mm OSB	60.32 (2 3/8)	3.32	2
JS-3	11mm OSB	82.55 (3 1/4)	3.76	2
JS-4	12.5mm Plywood	50.8 (2)	2.84	2
JS-5	15.5mm OSB	50.8 (2)	2.84	2

3.3.2 Stud Bending Test

The aim of the stud bending test was to determine the modulus of elasticity of the studs in the weak direction (consistent with their behaviour during the full-scale wall test). The test method followed the ASTM D198-67 Standard Test Methods of Static Test of Lumber in Structural Size

(2015), where four-point loading on simply supported stud elements was chosen in this study, as illustrated in Figure 3.8. A total of five repeats were undertaken for the stud bending tests. Figure 3.9 shows the setup details.

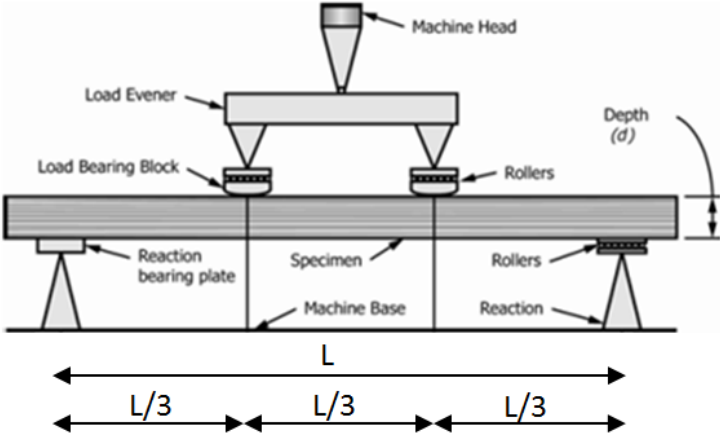


Figure 3.8: 4-Point Loading Test Method of Structural Size of Lumber (ASTM D198-67 2015)

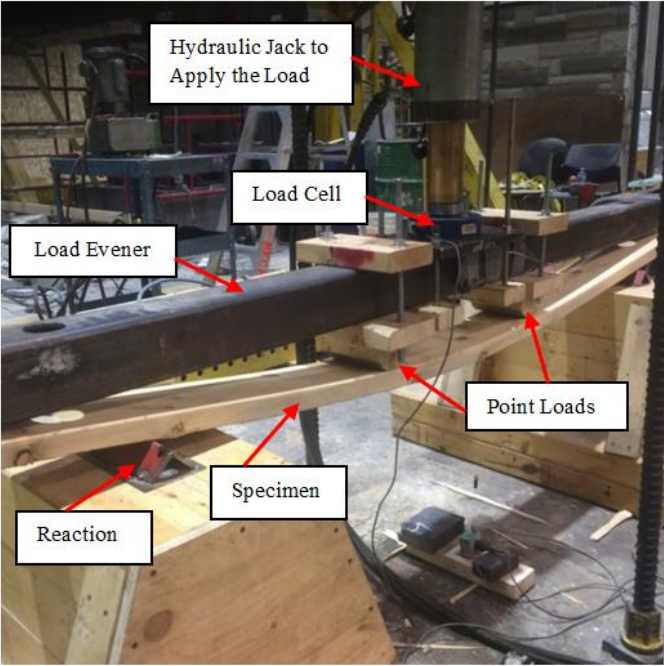


Figure 3.9: Stud Bending Test Setup

3.3.3 Tie-Rod Tension Test

ASTM A370 (2017) covers testing procedures for steel products to determine the material specifications such as tension and bending properties. To establish the stress-strain relationship for the tie-rod used in the full-scale tests, tension tests were conducted using the Universal Testing Machine (UTM). A total of two repeats were done using 5/8" diameter bar with length of 300mm. The loading rate was set to 2.5mm/min. An extensometer was used to measure the change in the length of the bar. Using the results from extensometer, the strain in the bar was obtained.

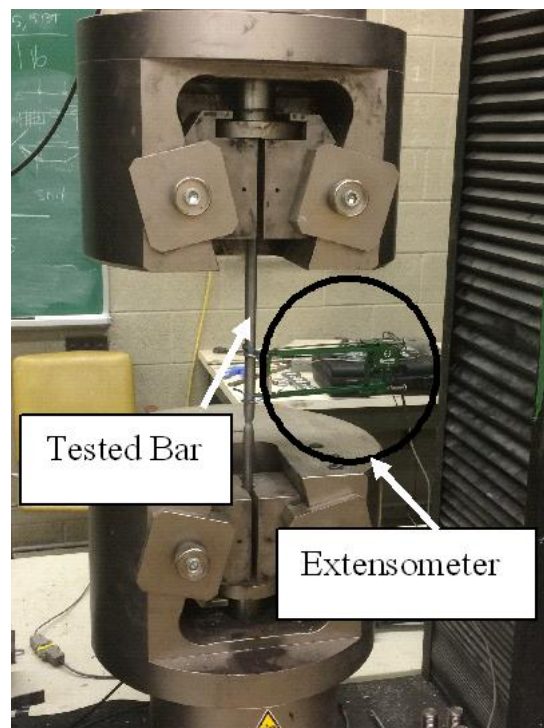


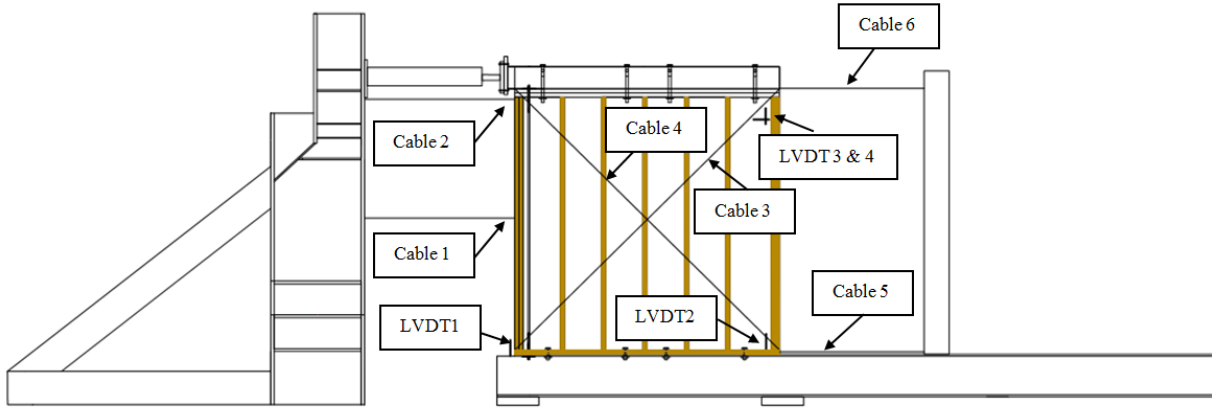
Figure 3.10: Tie-Rod Tension Test Setup

3.3.4 Full-Scale Wall Specimens Test Setup and Procedure

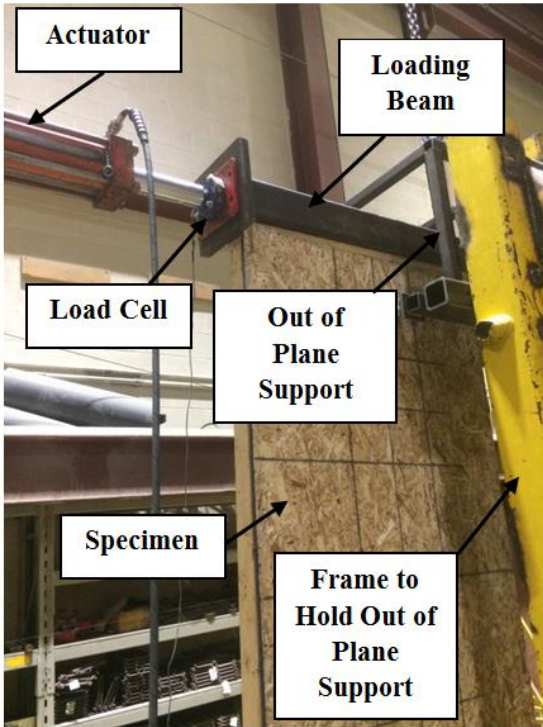
ASTM 72-2014 was followed for the full-scale shear wall tests. For single storey walls, the two top plates were connected to a steel HSS load transfer beam using 1/2" bolts spaced at 600mm on

center. The load was applied to the load transfer beam through an actuator and measured using a load cell located between the actuator and loading beam. The wall was placed on two 2"x4" (38x89mm) bottom plates that were attached to the supporting steel beam using 1/2" bolts spaced at 600 mm on center. Hold-down anchors were attached to the end studs using 1/4" screws, and 1/2" bolts were used to connect the hold-down to the supporting steel beam. When using tie-rod as a hold-down system, the rod was attached directly to the supporting steel beam through the hole made in the beam.

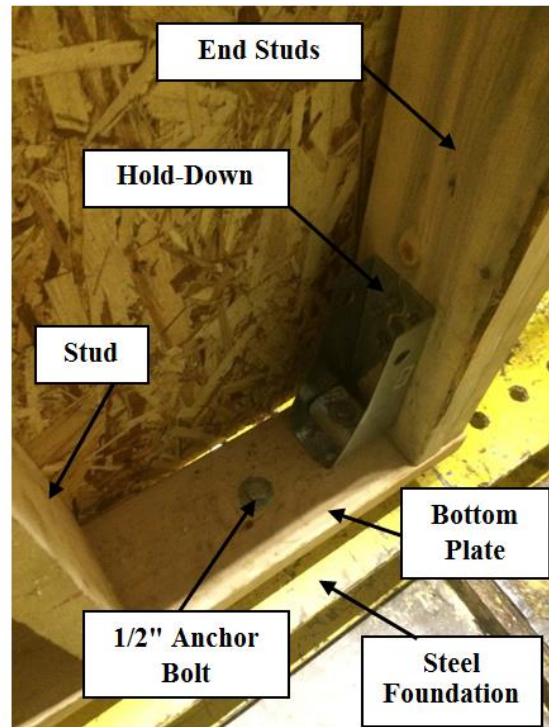
In total, 4 LVDTs and 6 draw-wire displacement transducers were used to measure the deflection of various components in the wall. LVDT 1 measured the hold-down system elongation while LVDT 2 measured the deflection perpendicular to the grain at the compression side. LVDT 3 and 4 were used to measure the nail slip in the horizontal and vertical directions. Cable 1 and 2 measured the bending of the studs while cable 3 and 4 were used to measure the shear deformation of the panel. Cable 5 measured the relative slip between the bottom plate of the wall and the support and cable 6 was used to measure the total horizontal deflection of the wall. Also, the strain in tie-rod was measured using a strain gauge. A schematic of the instrumentation is shown in Figure 3.11(a). Figure 3.11(b) and Figure 3.11(c) show key details from the test setup.



a) Single Storey Full-Scale Shear Wall Test Setup Schematic



b) Wall Test Setup



c) Hold-Down Installation

Figure 3.11: Single Storey Full-Scale Shear Wall Test Setup

All the walls were subjected to a monotonic loading following ASTM E72 (2014). The parameters that were varied in the testing were the stud size and spacing, nail diameter and spacing, sheathing panel type and thickness and hold-down type.

For the two -storey walls, two different loading cases were considered, namely where the load was applied at the top or bottom storey only. A bearing plate was used when the load was applied at the bottom storey at the level of the top plates to avoid localized crushing in the wood. The hold-down details were similar to those for the single storey walls, where they were attached to the end studs and foundation, using 1/4" screws and 1/2" bolt, respectively. The bottom wall was attached to the top wall through 1/2" bolts spaced at 600 mm on center connecting the wall plates. Out of plane supports were provided at the top of each wall to simulate diaphragm action at those locations. Figure 3.12 shows the test setup.

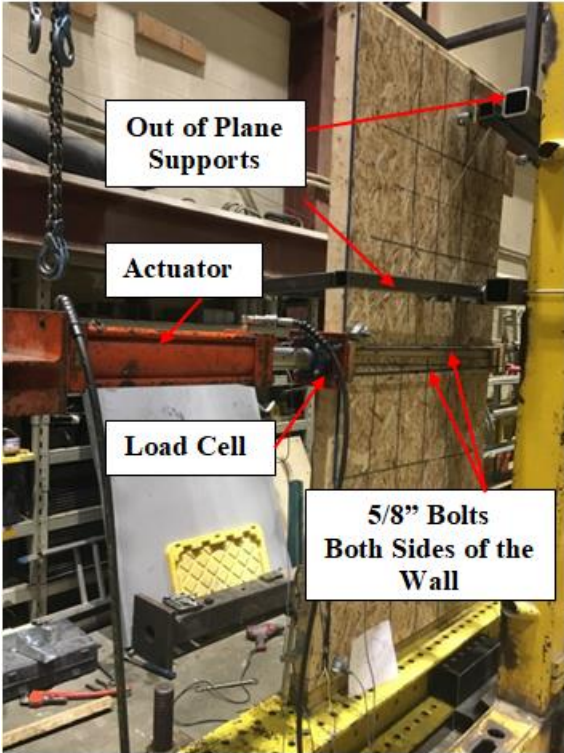


Figure 3.12: Double Storey Half-Scale Shear Wall Test Setup (Pushing Bottom Wall)

In total, 3 LVDTs and 6 cable transducers were used to measure the various deflection contributors. LVDT 1 and 2 measured the hold-down system elongation in bottom and top storey while LVDT 3 measured the deflection perpendicular to the grains in compression side of the bottom wall. Cable 1 and 2 were used to measure the bending deflection of the bottom wall end studs. Cable 2 also measured the total deflection of the bottom wall. Cable 3 and 4 measured the diagonal elongation of the bottom and top panels respectively. Cable 5 and 6 were used to measure the stud bending of top wall. Cable 6 also measured the total deflection of the top wall.

A schematic of the test setup is shown in Figure 3.13.

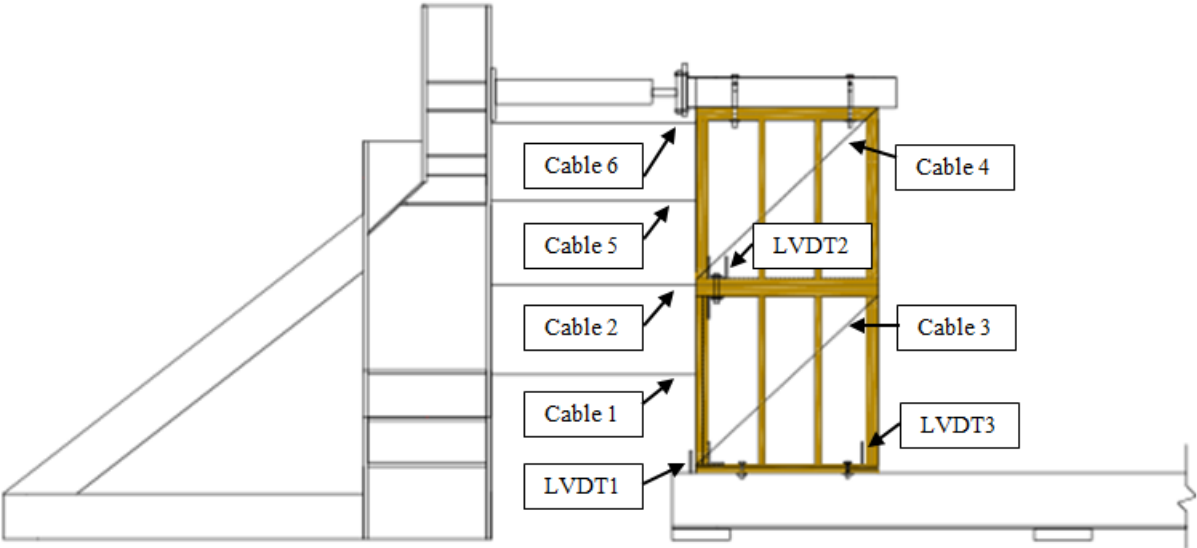


Figure 3.13: Half-Scale Double Storey Shear Wall Test Setup Schematic (Instrumentation)

Table 3.4 and Table 3.5 list the single and double storey shear wall test matrices, respectively.

The assumed stud spacing in the text matrix is 16” unless otherwise mentioned.

Table 3.4: Single-Storey Shear Wall Test Matrix

NO	ID	STUD	ASPECT RATIO	NO OF END STUDSS	NAIL DIAMETER(mm)	PANEL THICKNESS(mm)	UPLIFT PREVENTION	REPEAT	MILESTONE
1	SW-1A	2"x4"	1:1	2	2.84	11	HOLD-DOWN	3	ASPECT RATIO
	SW-1B	2"x4"	2:1	2	2.84	11	HOLD-DOWN	1	
	SW-1C	2"x4"	3:1	2	2.84	11	HOLD-DOWN	1	
	SW-1D	2"x4"	4:1	2	2.84	11	HOLD-DOWN	2	
	SW-1E	2"x4"	6:1	2	2.84	11	HOLD-DOWN	1	
2	SW-2A	2"x4"	1:1	1	2.84	11	HOLD-DOWN	1	EFFECT OF NUMBER OF STUDS
	SW-2B	2"x4"	1:1	3	2.84	11	HOLD-DOWN	1	
	SW-2C	2"x4"	2:1	1	2.84	11	HOLD-DOWN	1	
	SW-2D	2"x4"	2:1	3	2.84	11	HOLD-DOWN	1	EFFECT OF STUD SIZE
	SW-2E	2"x6"	2:1	1	2.84	11	HOLD-DOWN	1	
	SW-2F	2"x6"	1:1	2	2.84	11	HOLD-DOWN	1	
	SW-2G	2"x6"	2:1	3	2.84	11	HOLD-DOWN	1	EFFECT OF STUD SPACING
	SW-2H	2"x4"	1:1	2, S=12"	2.84	11	HOLD-DOWN	1	
	SW-2I	2"x4"	1:1	2, S=24"	2.84	11	HOLD-DOWN	1	
3	SW-3A	2"x4"	2:1	2	2.84	15.5	HOLD-DOWN	1	OSB V.S PLYWOOD
	SW-3B	2"x4"	1:1	2	2.84	11, PLYWOOD	HOLD-DOWN	1	
4	SW-4A	2"x4"	2:1	2	3.76	11	HOLD-DOWN	1	NAIL SIZE EFFECT
	SW-4B	2"x4"	2:1	2	3.32	11	HOLD-DOWN	1	
	SW-4C	2"x4"	2:1	2	2.84, S=75mm	11	HOLD-DOWN	1	
	SW-4D	2"x4"	2:1	2	2.84, End/Edge Distance =9.5mm	11	HOLD-DOWN	1	
5	SW-5A	2"x4"	1:1	2	2.84	11	TIE-ROD	1	HOLD-DOWN VS. TIE-ROD
	SW-5B	2"x4"	2:1	2	2.84	11	TIE-ROD	2	
	SW-5C	2"x4"	1:1	2	2.84	11	-	1	

Table 3.5: Double-Storey Shear Wall Test Matrix

NO	ID	Length (m)	Aspect Ratio	Nail Dia. (mm)	STUDS	Panel Thickness (mm)	Uplift Prevention	Loading Protocol (Monotonic)	Milestone
1	SW-6A	1.2	1:1	2.84	2"x4"	11	HOLD-DOWN	Pushing bottom wall in elastic range	Cantilever assumptions + cumulative effects
	SW-6B	1.2	1:1	2.84	2"x4"	11	HOLD-DOWN	Pushing bottom wall to failure	Cantilever assumptions + cumulative effects
2	SW-7A	1.2	1:1	2.84	2"x4"	11	HOLD-DOWN	Pushing top wall in elastic range	Cantilever assumptions + cumulative effects
	SW-7B	1.2	1:1	2.84	2"x4"	11	HOLD-DOWN	Pushing top wall to failure	Cantilever assumptions + cumulative effects

4 EXPERIMENTAL RESULTS

4.1 Nail Joint Component Test

4.1.1 Typical Nail Joint Load-Displacement Response

An average force-displacement curve was developed for the nail joint tests based on the measurements obtained from the two LVDTs that recorded the nail slip on each side of the test specimen. Since two nails were used at the tested end, the measured load was divided by two to provide the load per nail. Figure 4.1 shows a typical response of JS-1 group. The results of all nail component specimens can be seen in Appendix 9.1.

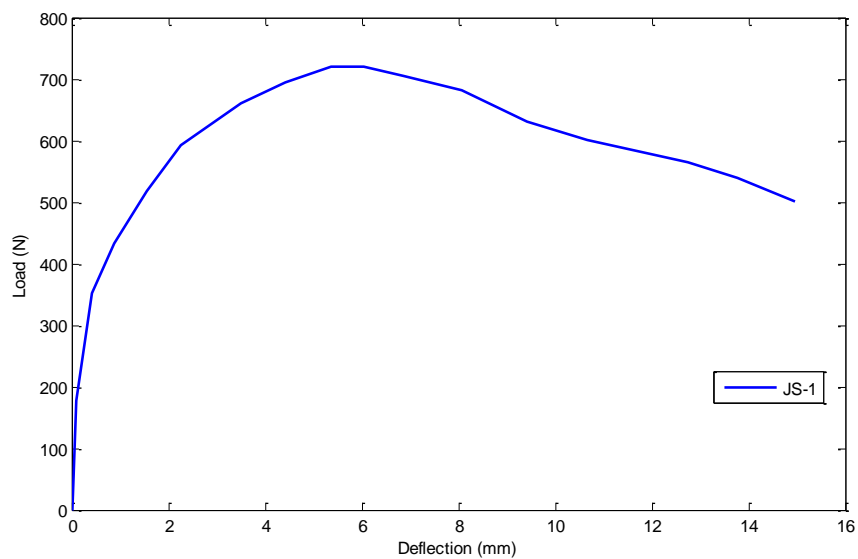


Figure 4.1: Typical JS-1 Group Load-Deflection Record (Nail diameter 2.84mm and OSB Sheathing Thickness 11mm)

4.1.2 EEEP Analysis Results

Equivalent Energy Elastic Plastic (EEEP) curves are commonly used to represent the behavior of joint specimens as well as full scale shear wall tests. ASTM E2126 (2011) contains the details of

this method, where experimentally obtained load-displacement curves can be idealized with perfect elastic-plastic curves (Figure 4.2), circumscribing an area equal to the area enclosed by the envelope curve. The initial stiffness is defined as the slope of a line connecting the origin of the coordinate system with a point taken as the ratio of 40% of the peak load. The elastic part of the bi-linear curve is determined by the initial stiffness, while the plastic part is a horizontal line at the yield point and extends until the ultimate displacement, which is defined as the displacement corresponding to 80% of the peak load (post peak), as shown in Figure 4.2. The yield point is calculated by equating the area under the envelope curve and EEEP curve.

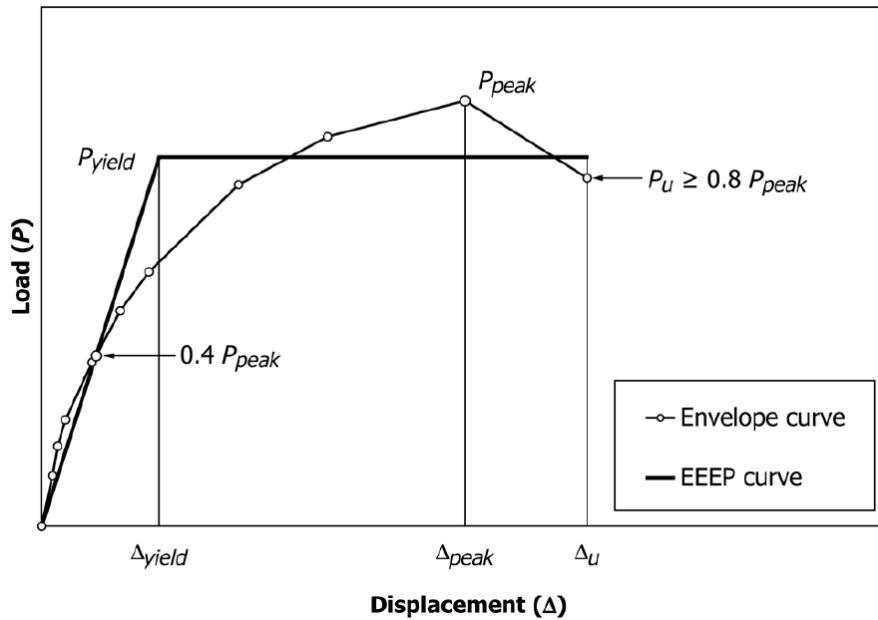


Figure 4.2: EEEP Model (ASTM E2126, 2011)

The EEEP analysis was performed for each test group and the average and coefficient of variation (COV) are provided in Table 4.1.

Table 4.1: Nail Joint Tests EEEP Results

JS Group	Peak Load (N)	Deflection At Peak Load(mm)	Initial Stiffness (N/mm)	Ultimate Deflection (mm)	Yield Load (N)	Ductility Ratio (Δ_u/Δ_{yield})
JS-1	854 (10.6%)	7.70 (46%)	929.4 (18.9%)	12.70 (16.9%)	753.1 (10.3%)	15.71 (23.5%)
JS-2	890 (1.0%)	4.30 (1.64%)	1279.5 (14%)	11.90 (32.8%)	668.4 (0.95%)	23.2 (44.85%)
JS-3	1055 (3.9%)	9.50 (12%)	812.4 (58.7%)	14.60 (11.5%)	916.9 (4.1%)	14.1 (42.7%)
JS-4	1014 (0.84%)	7.70 (25.2%)	1110.6 (22.7%)	8.05 (33.6%)	811.9 (5.8%)	13.3 (5.6%)
JS-5	1298 (6.0%)	4.0 (31.5%)	1793.5 (18.9%)	10.16 (42.5%)	1132.7 (10.3%)	16.3 (50.6%)

4.2 Stud Bending Test Results

The results from the stud bending tests provided estimates on the modulus of elasticity for the studs in the weak direction. ASTM D198 (2015) describes the procedure to obtain the modulus of elasticity of the studs based on the chosen loading method. For the 4-point loading method used in this study, Equation 4.1 (ASTM D198-15) estimates the modulus of elasticity of the studs as follows:

$$E = \frac{23PL^3}{108bd^3\Delta} \quad (4.1)$$

Where, E is the modulus of elasticity of the stud, P is applied load on the specimen, L is the span, b is the specimen width, d is the depth of the specimen and Δ is the deflection at the neutral axis measured at mid-span.

Figure 4.3 shows a typical flexural performance of the stud and Table 4.2 summarizes the modulus of elasticity (E) values obtained using equation 4.1. The results of all tested studs can be seen in Appendix 9.2.

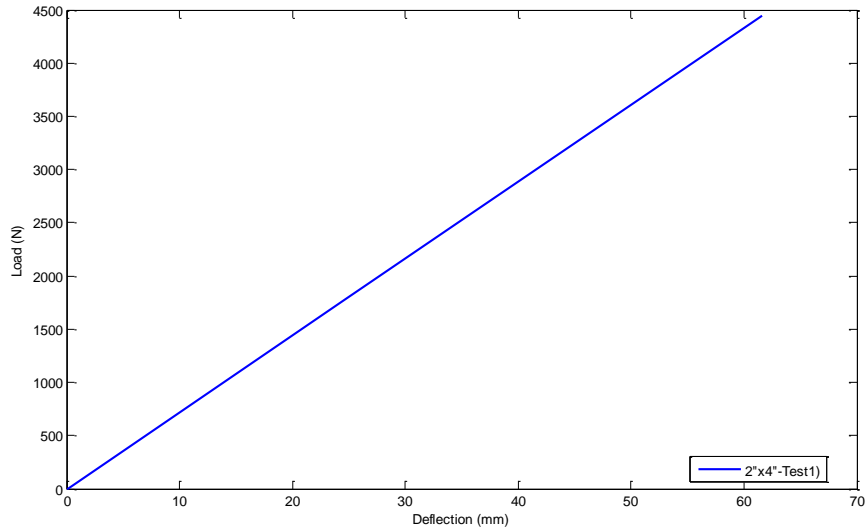


Figure 4.3: 2x4 Stud Typical Bending Performance

Table 4.2: Stud Bending Tests Results

Stud Size	No	$P_{0.4Peak}$ (N)	$\Delta_{0.4Peak}$ (mm)	E (N/mm^2)
2x4"	1	1780.00	24.65	12898
	2	1707.50	27.90	10931
	3	1666.40	27.20	10943
	4	1557.82	32.14	8658
	5	1560	25.80	10800
2x6"	1	2480	22.57	12478
	2	2420	21.22	12950
	3	2560	22.49	12925
	4	2040.8	25.70	9017
	5	1880	20.84	10243

The average modulus of elasticity for the 2x4" and 2x6" studs was $10846.18 N/mm^2$ (COV=13.84%) and $11522.58 N/mm^2$ (COV=15.54%), respectively. The modulus of elasticity obtained from the wood design standard (CSA-O86 2014) for the same species and grade (S-P-F No.2) is $9500 N/mm^2$.

4.3 Tie-Rod Tension Test

Tension tests were performed on the tie-rod forming part of the hold-down system in accordance with the ASTM A370 standard (2017). Establishing the behavior of the tie-rod (yield point and tensile strength) in isolation provides a better understanding of the behavior of the hold-down system and also serve as material input in the numerical modeling analysis undertaken in this project. Figure 4.4 shows a typical tie-rod stress-strain relationship.

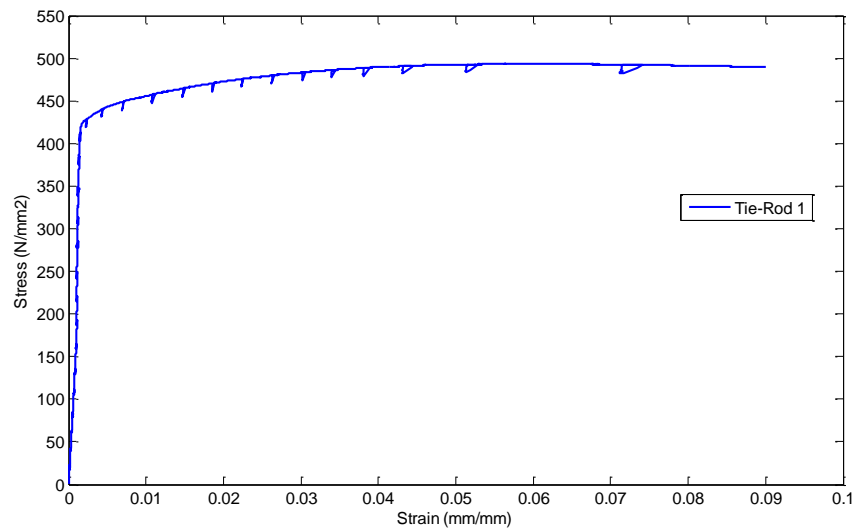


Figure 4.4: Typical Tie-Rod Stress-Strain Relationship

Table 4.3 summarizes the results obtained from the tested bars. It should be noted that based on manufacture's design manual, the tensile strength of the rod used in this study is 400 MPa.

Table 4.3: Bar Tension Test Results

No	Bar diameter in mm (in)	Yield Stress (MPa)	E (N/mm ²)	Tensile Strength (MPa)
1	15.9 (5/8)	442.8	1.92 x 10 ⁵	493.1
2	15.9 (5/8)	440.3	1.54 x 10 ⁵	492.8

4.4 Single Storey Full Scale Shear Wall Tests

4.4.1 General Observations

4.4.1.1 Group 1

Group 1 of the test matrix consisted of shear walls with different aspect ratios. Yielding in the nails was the main failure mode for the SW-1A Group (aspect ratio of 1:1). However, nails pull-through and nails withdrawal at top plate was evidence at ultimate failure, as can be seen in Figure 4.5.



**a) SW-1A-R2
Nails Yielding, Nails Pull-Through**

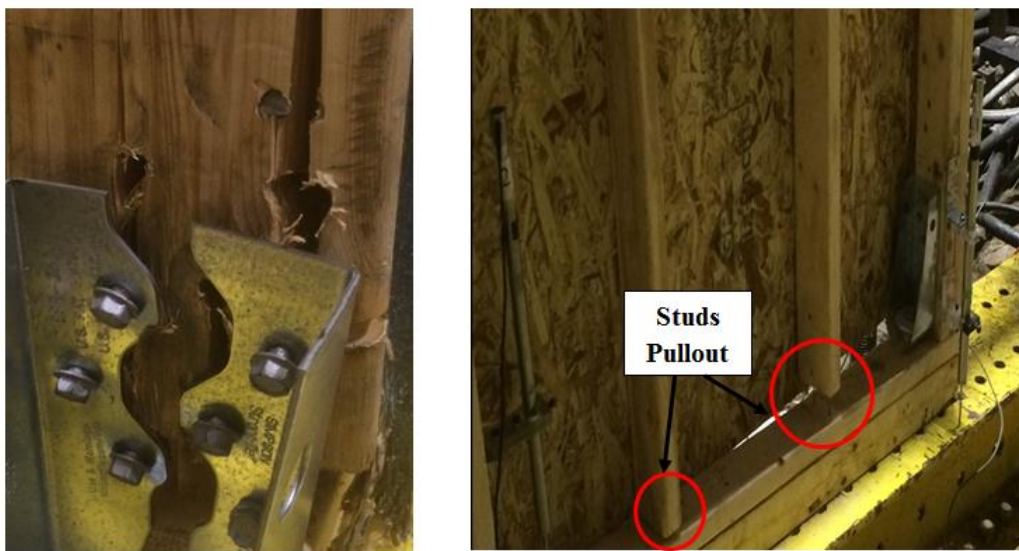


**b) SW-1A-R2
Nails Withdrawal at Top Plate**

Figure 4.5: SW-1A Group Failure Modes

Stud pullout was also noticed at very high levels of loading and deformation, which meant that this failure did not affect the wall behaviour in a significant manner.

For wall SW-1B, hold-down damage (Figure 4.6(a)) and separation between studs and bottom plate (Figure 4.6(b)) was observed. This could explain the sudden drop in capacity prior to reaching the ultimate strength of the wall. Consequently the ductility of wall SW-1B was observed to be less than wall SW-1B-R1.



a) SW-1B Hold-Down Screws Bending and Group Tear-out

b) SW-1B-R1 Studs Pullout from the Bottom Plate at the End of the Test after Yielding the Nails

Figure 4.6: SW-1B and SW-1B-R1 Failure Modes

4.4.1.2 Group 2

In group SW-2, the stud size and spacing were investigated. The common failure mode of this test group was yielding of the nails followed by nails pull-through (Figure 4.7).

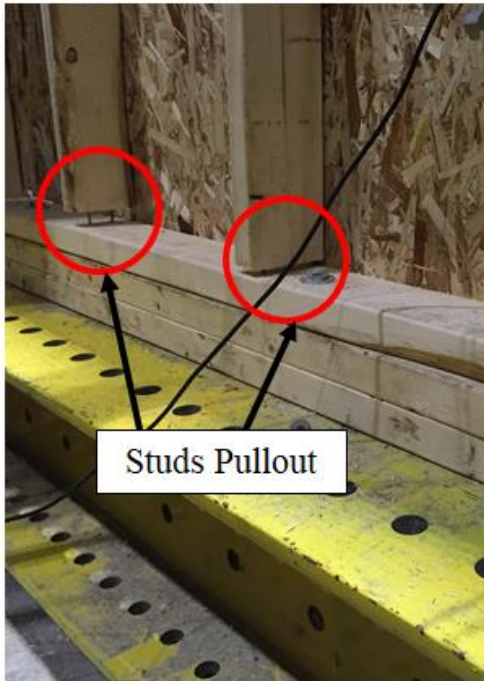


a) SW-2B Nails Yielding, Nails Pull-Through

b) Close-up View of (a)

Figure 4.7: Typical Failure of Group 2

As shown in Figure 4.8, separation between the studs and bottom plate through withdrawal in the nail was also evident. However, this separation was observed towards the end of the test and at high levels of loading. Yielding the nails was also the main failure mode for SW-2E followed by nails pull-through in the middle of the wall where the two sheathing panels meet (Figure 4.9). This resulted in a drop in recorded load and consequently less ductility compared to SW-2A.



**a) SW-2A (One 2"x4" End Stud)
Studs Pullout**



**b) SW-2E (One 2"x6" End Stud)
Nails Pull-Through Where Two Panels Join**

Figure 4.8: SW-2A and SW-2E Failure Modes

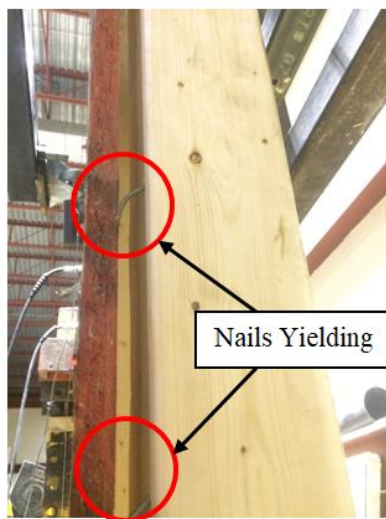
Figure 4.9 shows the deformed shape of wall SW-2I at the end of the test, where significant bending in the studs was noticeable.



Figure 4.9: SW-2I Studs Bending

4.4.1.3 Group 3

Walls with different panel thickness and type were investigated in this group. Yielding in the nails was the predominant failure mode in SW-3A (Figure 4.10 (a)), however damage to the hold-down screws (Figure 4.10 (b)) was also observed at the end of the test, which also caused the studs to separate from the bottom plate (Figure 4.10(c)).



a) Nails Yielding



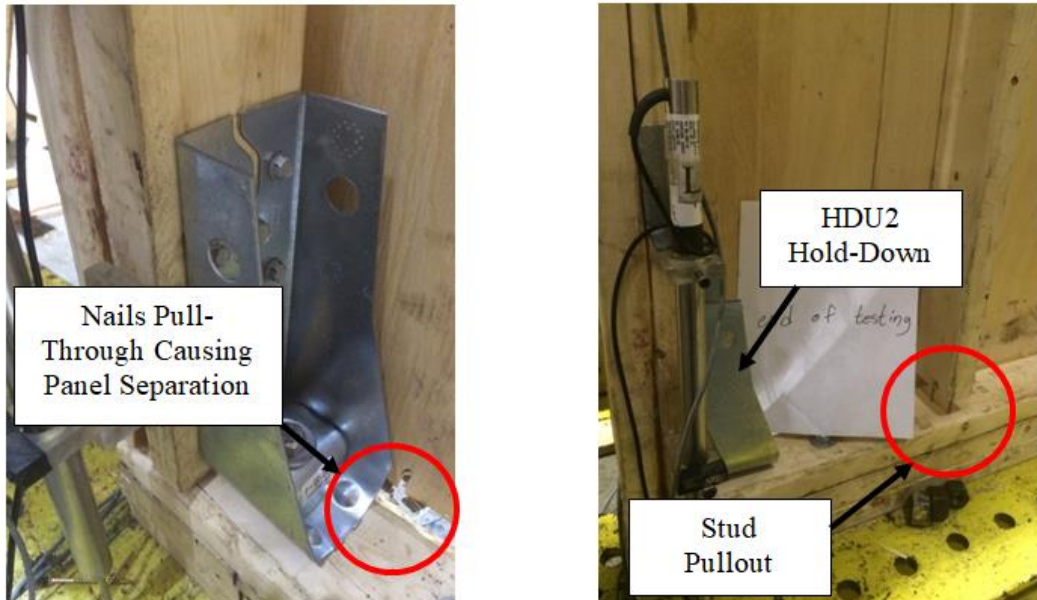
b) Hold-Down Connection Failure



c) Stud Pullout at Tension Side

Figure 4.10: SW-3A (Sheathed with 15.5mm OSB Panel) Failure Modes

Wall SW-3B also experience significant nail yielding followed by panel separation and stud pullout at the end of the test, as seen in Figure 4.11.



a) Nails Yielding, Nails Pull-Through b) Stud Pullout at the End of the Test

Figure 4.11: SW-3B (Sheathed with 12.5mm Plywood) Failure Modes

4.4.1.4 Group 4

Group 4 consisted of walls built of different nails sizes, nail spacing and end/edge distances. Typical failure mode of yielding the nails following by nails pull-through at the end of the test was observed generally for this group (Figure 4.12 (a)). Reducing the nails end/edge distance in SW-4D from 15mm to 9.5mm did not change the wall predominant failure mode. However, nails pull-through was more noticeable towards the end of the test (Figure 4.12 (b)).



a) SW-4A Nails Yielding, Nails Pull-Through



b) SW-4D Nails Pull-Through

Figure 4.12: Group 4 Failure Modes

4.4.1.5 Group 5

Group 5 investigated the behavior of the shear walls with different types of hold-down systems. Also, wall with no hold-down (SW-5C) was tested. Significant rotation of the panels was noticed for walls SW-5A (ATS and aspect ratio of 1:1) and SW-5B (ATS and aspect ratio of 2:1). Also, yielding of nails was uniformly distributed through the entire wall for walls with continuous hold-down compared to those with discrete hold-down.



Figure 4.13: SW-5B Uniformly Distributed Yielding of Nails through the Entire Wall Followed by Nails Pull-Through Towards End of the Test

As expected for wall SW-5C where there was no hold-down system installed, the rotation of the wall caused significant separation between the end studs and the bottom plate.

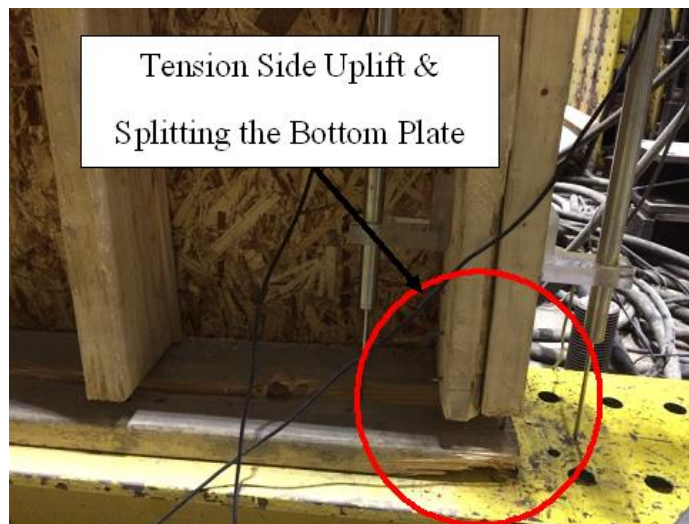


Figure 4.14: SW-5C Rigid Body Rotation of the Wall Due to Lack of Anchorage

4.4.2 Shear Wall Load-Deflection Response

For each wall, the measured displacement at the top of the wall was plotted against the load measured by the load cell. Figure 4.15 shows a typical example of the results obtained from the full scale shear wall tests. The behaviour of all tested walls can be seen in Appendix 9.3.

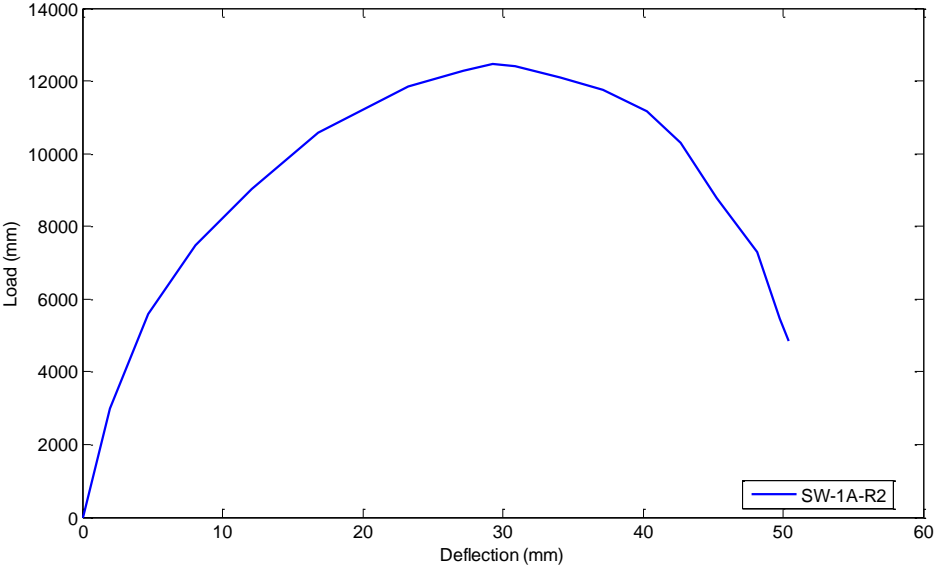
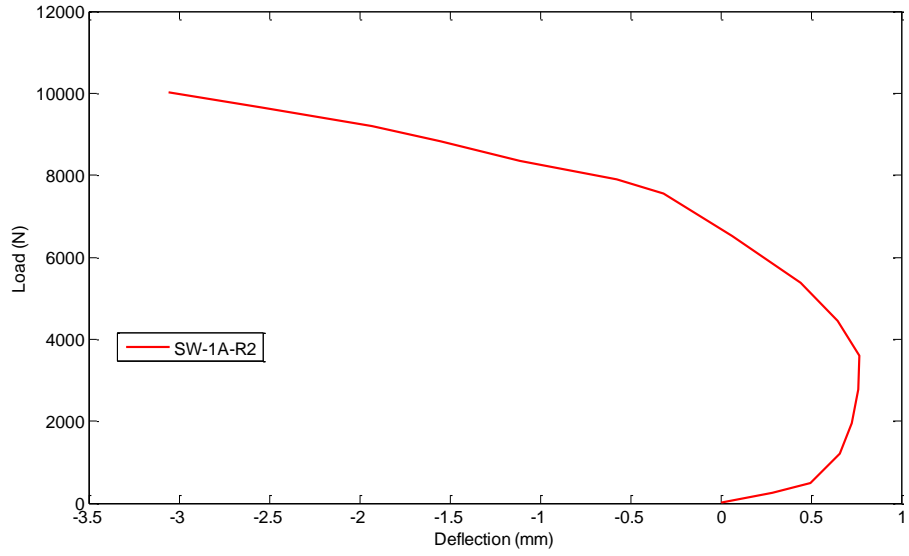
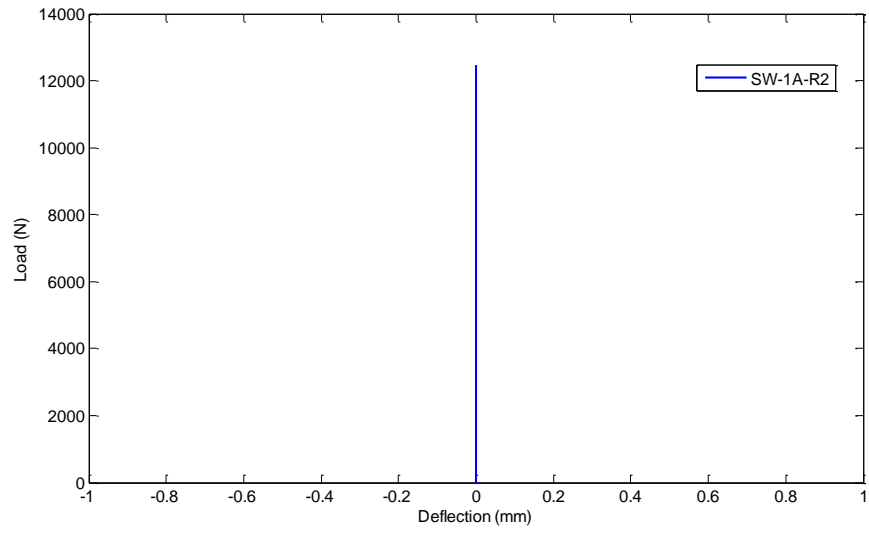


Figure 4.15: SW-1A-R2 Load-Deflection Response

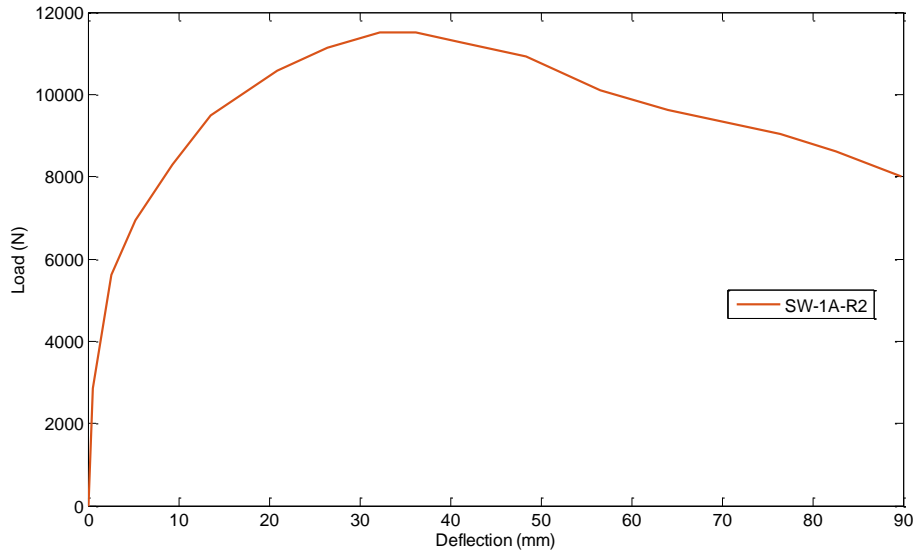
Figure 4.16 presents typical results (for SW-1A-R2) recorded for the four components of the shear wall deformation.



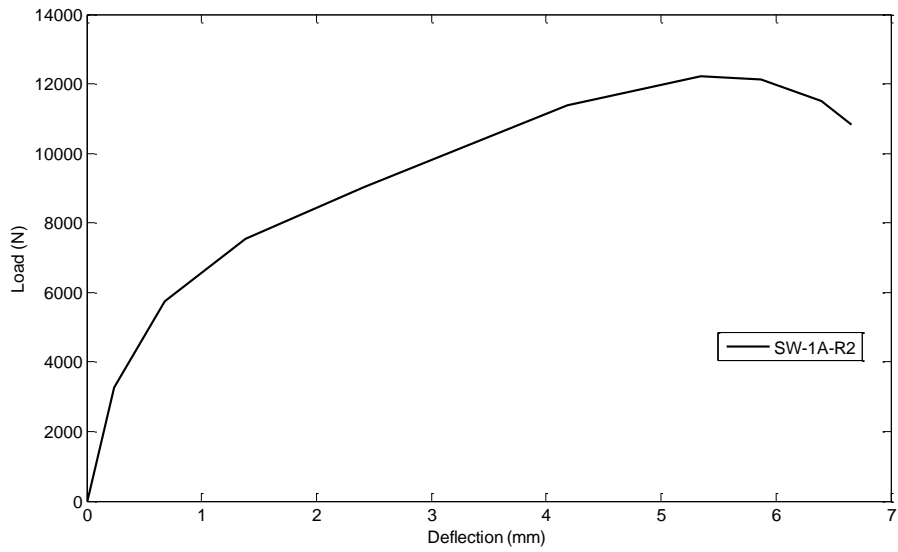
a) Stud Bending Deflection



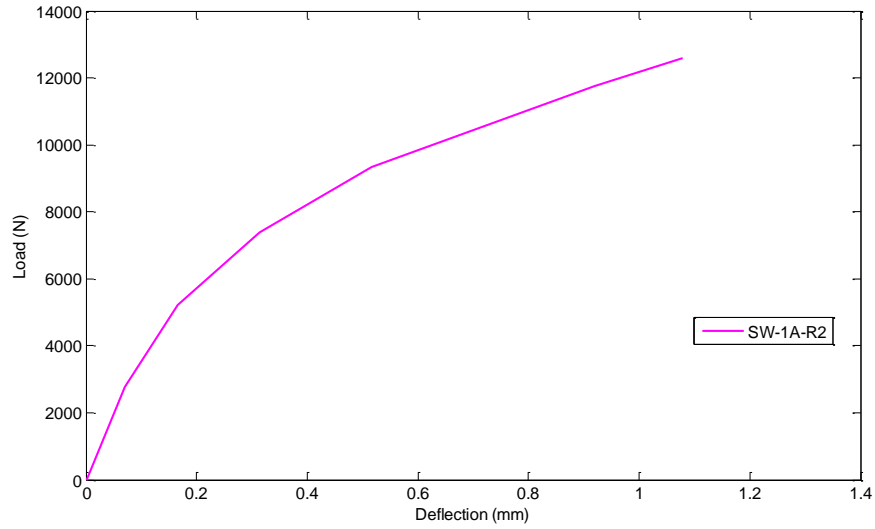
b) Horizontal Deflection Caused by Sheathing Panel Shear Deformation



c) Horizontal Deflection Caused by Nail Slip



d) Horizontal Deflection Caused by Hold-Down System Elongation



e) Horizontal Deflection Caused by Compression Perpendicular to the Grain

Figure 4.16: SW-1A-R1 Deflection Contributors

Figure 4.17 shows the summation of deflection contributors for SW-1A-R2.

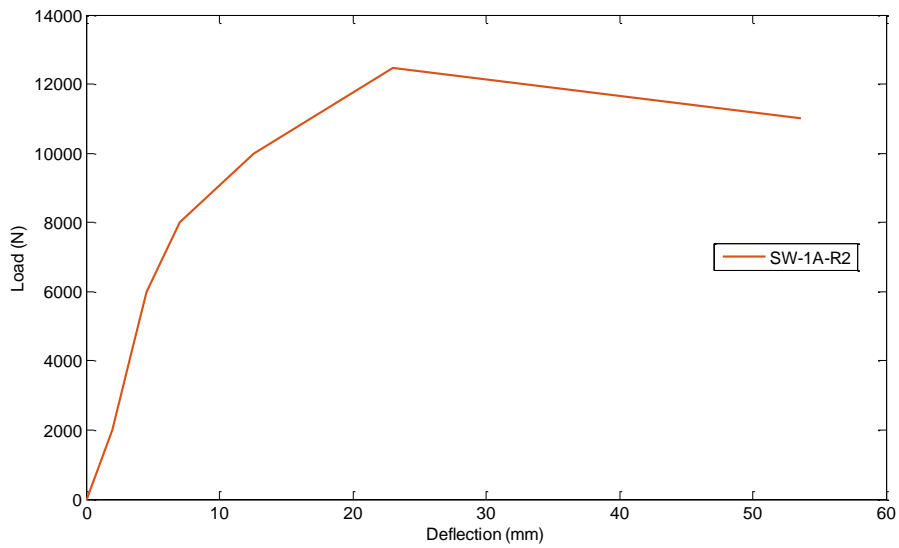


Figure 4.17: SW-1A-R2 Summation of Deflection Contributors

4.4.3 EEEP Analysis Results

The EEEP method was explained in the joint specimen section. Table 4.4 summarizes the analysis results for the full scale shear walls.

Table 4.4: Single Storey Full Scale Shear Walls EEEP Analysis Results

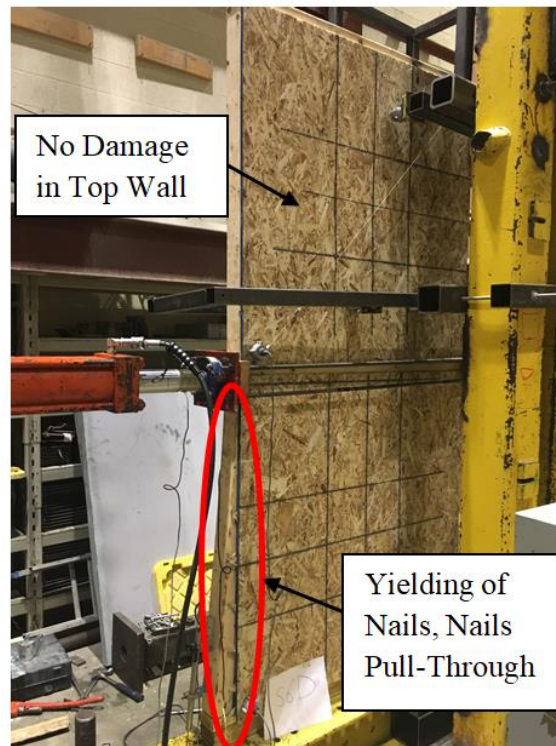
ID	Peak Load (N)	Deflection At Peak Load(mm)	Initial Stiffness (N/mm)	Ultimate Deflection (mm)	Yield Load (N)	Ductility Ratio (Δ_u/Δ_{yield})
SW-1A	11140	31.91	360	71.35	10230	2.53
SW-1A-R1	14870	57.04	800	85.46	13381.33	5.12
SW-1A-R2	12480	29.24	1220	42.69	11000	4.66
SW-1B	8340	53.55	480	68.70	7496.3	4.37
SW-1B-R1	6760	67	480	92.98	5822.2	7.60
SW-1C	4900	75.17	270	79.49	4057.4	5.31
SW-1D	3360	79.7	98	97.82	2980	3.36
SW-1E	1550	56.00	80	83.56	1257.8	5.04
SW-2A	13250	49.90	1020	75.96	11644	6.65
SW-2B	13010	28.02	1750	38.75	11595	5.86
SW-2C	7440	70.44	480	96.60	7192.1	6.47
SW-2D	8700	63.68	550	98.00	7840.5	6.85
SW-2E	9230	73.90	320	82.30	7776	3.41
SW-2F	13000	46.05	1140	42.90	11730	4.17
SW-2G	7200	33.64	430	57.94	6517	3.82
SW-2H	12928	48.21	977	87.35	13785	6.1
SW-2I	12460	40.66	926	74.14	11834	5.76
SW-3A	10280	94.10	620	180.94	9514.4	11.75
SW-3B	16400	43.62	1120	81.40	14787	6.14
SW-4A	9190	70.73	440	115.50	8314.9	5.28
SW-4B	7650	77.83	500	108.52	6986	9.17
SW-4C	11930	52	650	91.65	11010	5.37
SW-4D	7040.2	42.20	573	82.05	6765.5	6.83
SW-5A	15100	44.13	1500	67.50	14363.6	7.06
SW-5B	10170	59.29	490	96.85	8989.6	5.31
SW-5B-R1	10060	57.23	440	85.60	8902.6	4.18
SW-5C	5350	25.01	760	35.92	4500	6.08

4.5 Two-Storey Half-Scale Shear Wall Tests

4.5.1 General Observations

4.5.1.1 Group 1

Group 1 of the two-storey test matrix considered the behavior of the walls when bottom wall was loaded. Yielding the nails followed by nails pull-through towards the end of the test in bottom storey was the main failure mode. As expected, no damage was observed in the upper storey wall (Figure 4.18).



**Figure 4.18: Failure Mode of Group 1 of Two Storey Test Matrix
(Pushing Bottom Wall)**

4.5.1.2 Group 2

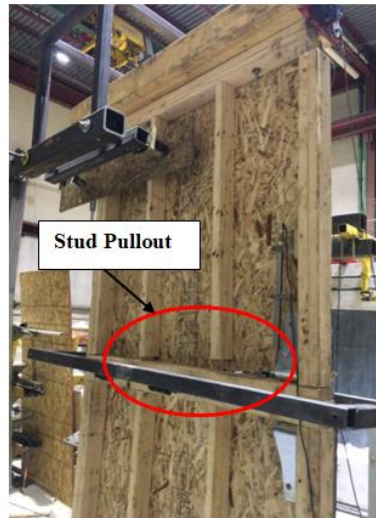
For this group, the load was applied to the upper-storey wall. Yielding of nails and nails pull-through was noticed at both bottom and top walls. Also, separation between the studs and bottom plate nail withdrawal at the edge of top plate was observed towards the end of the test (Figure 4.19).



**a) Yielding of Nails,
Nails Pull-Through**



b) Nails Withdrawal at Top Plate



c) Top Storey Studs Pullout

**Figure 4.19: Failure Modes of Group 2 of Two Storey Test Matrix
(Pushing Top Wall)**

4.5.2 Shear Wall Load-Deflection Response

Similar to the single-storey walls, various deflections components of the wall were measured using LVDTs and cable transducers. Figure 4.20 and Figure 4.21 show a typical load-deflection response of SW-7B when the top wall was loaded to failure. Figure 4.22 shows the deflection contributors for SW-7B and Figure 4.23 represents the summation of deflection contributors for wall SW-7B (bottom storey).

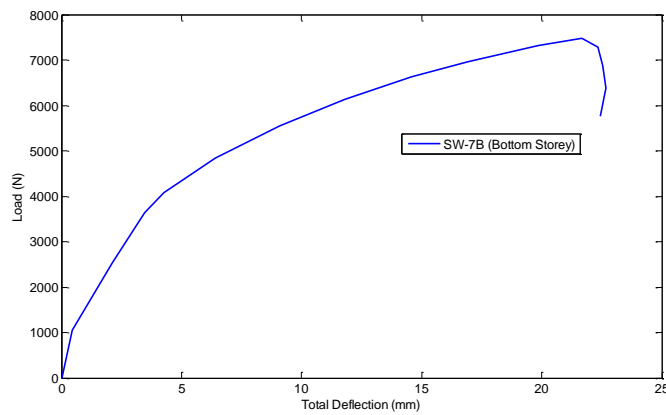


Figure 4.20: SW-7B Load-Deflection Response (Bottom Storey)

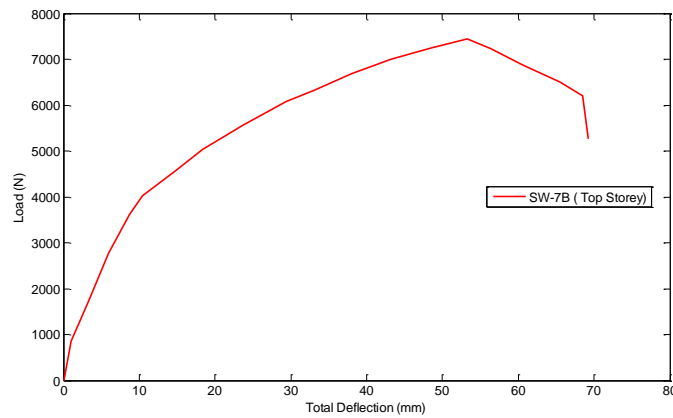
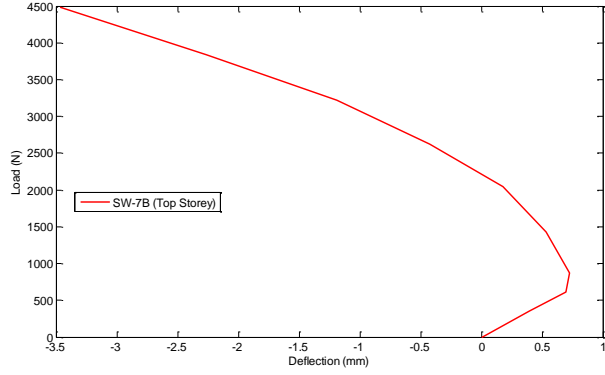
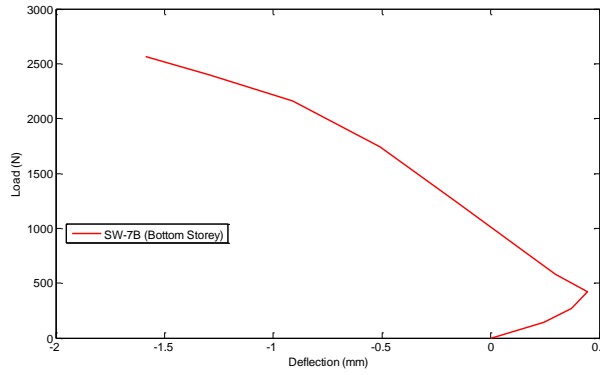
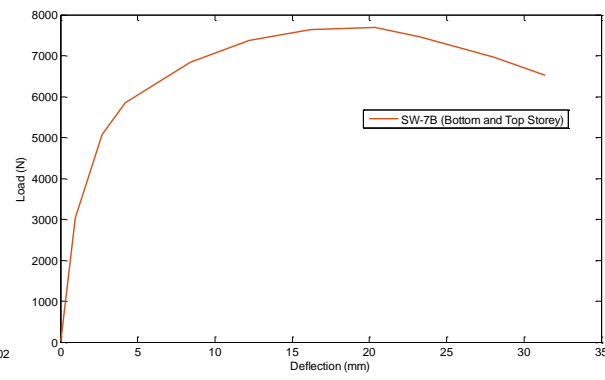
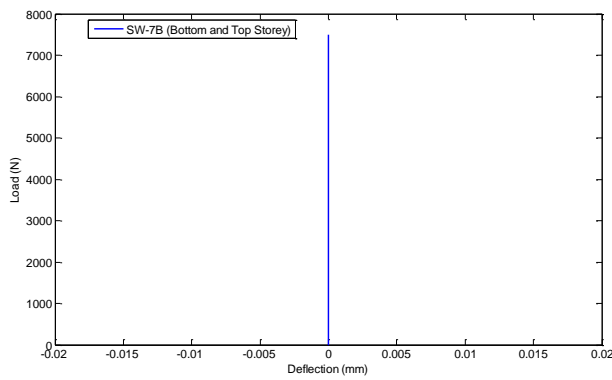


Figure 4.21: SW-7B Load-Deflection Response (Two Storey Total Deflection Measured at Top Storey)

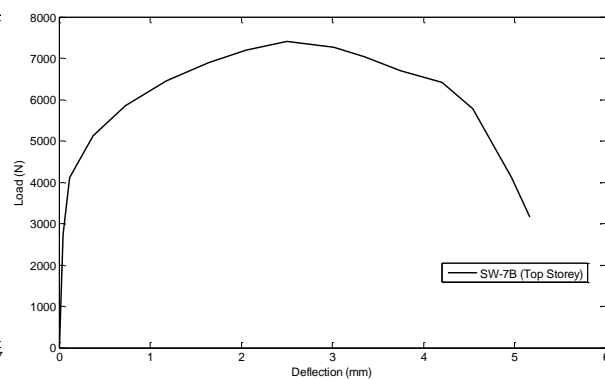
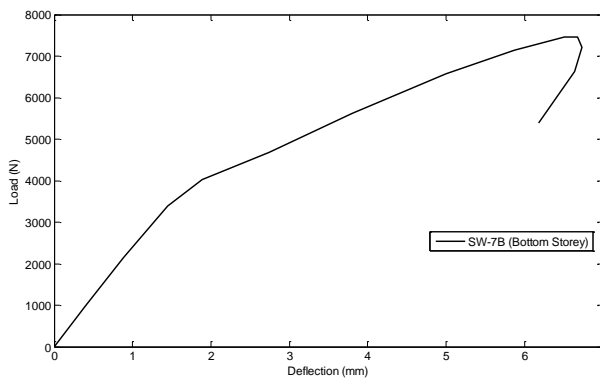


a) Stud Bending Deflection

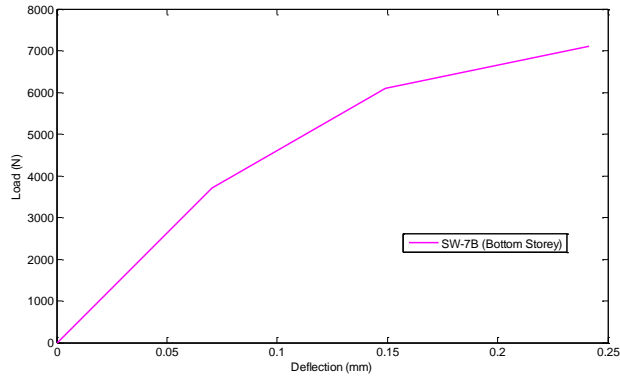


b) Horizontal Deflection Caused by Sheathing Panel Shear Deformation

c) Horizontal Deflection Caused by Nail Slip



d) Horizontal Deflection Caused by Hold-Down System Elongation



e) Horizontal Deflection Caused by Compression Perpendicular to the Grains (Bottom Storey)

Figure 4.22: SW-7B Deflection Contributors

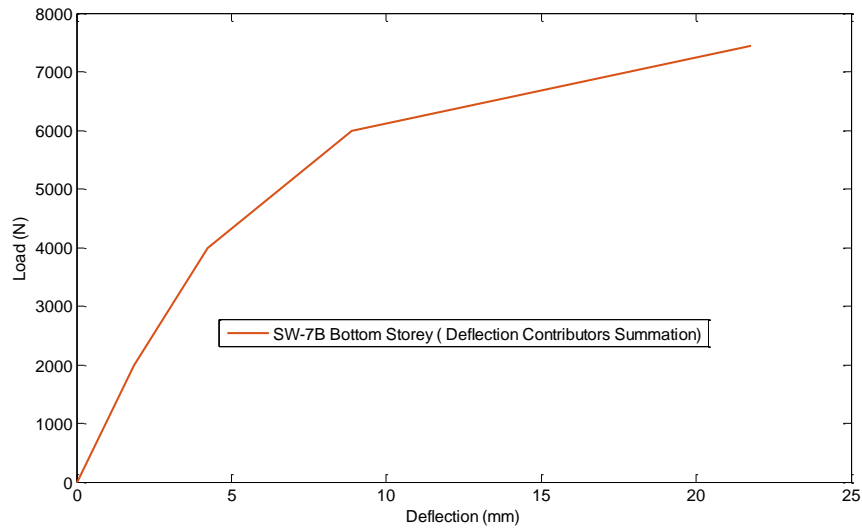


Figure 4.23: SW-7B Summation of Deflection Contributors (Bottom Storey)

4.5.3 EEEP Analysis Results

The results of EEEP analysis for the two-storey walls are summarized in Table 4.5.

Table 4.5: Two-Storey Half -Scale Shear Walls EEEP Analysis Results

ID	Storey Number	Peak Load (N)	Deflection At Peak Load(mm)	Initial Stiffness (N/mm)	Ultimate Deflection (mm)	Yield Load (N)	Ductility Ratio (Δ_u/Δ_{yield})
SW-6B	1	8420.8	23.35	1200	40.3	7560	6.4
SW-6B	2	-	33.4	890	49.42	-	5.89
SW-7B	1	7476.5	21.66	1050	22.44	6380	3.68
	2	7476.5	26.21	1190	45.87	6070	8.99

5 EXPERIMENTAL DISCUSSION

5.1 Nail Joint Component Test - Fastener Size Comparison

Three different nail sizes were used in this study including 2.84mm (JS-1), 3.32mm (JS-2) and 3.76mm (JS-3) diameter nails. The average peak load, initial stiffness and ductility ratio were compared in Figure 5.1.

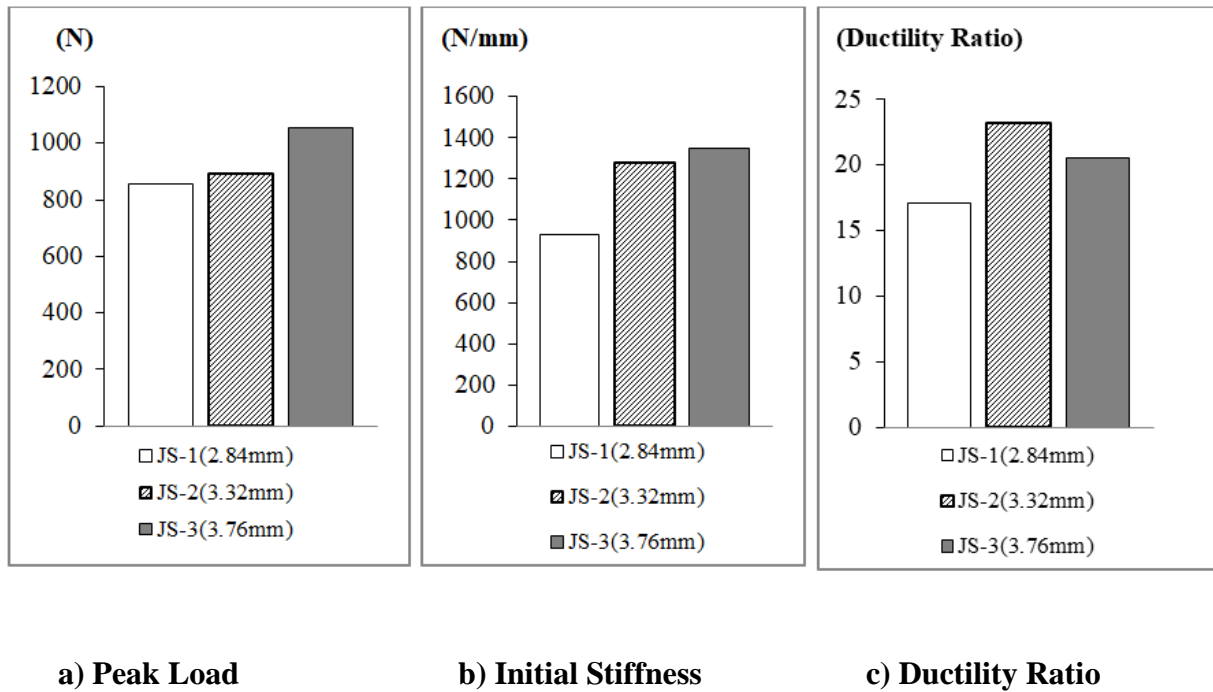


Figure 5.1: Parameter Comparison of Nail Size Variation

As can be seen from Figure 5.1, there was an increasing trend for peak load and initial stiffness when the nail size increased. However the trend for ductility is not consistent. An increase in nail diameter is not expected to change the ductility but possibly shift the bearing failure from predominantly yielding in the nail (in combination with wood crushing) to a failure dominated by wood crushing. In all cases, significant ductility was attained with a minimum ductility ratio above 15 for the nail with 2.84mm diameter.

5.2 Nail Joint Component Test - Sheathing Panel Type and Thickness Comparison

Two different sheathing panels, OSB and plywood, with two different thicknesses, 11mm and 12.5mm, respectively, were used in the nail joint component tests to evaluate the possible impact on the nail slip. The results are compared in Figure 5.2, where JS-1 is the OSB specimen while JS-4 represents the plywood specimen. Both JS-1 and JS-4 specimens used 2.84mm diameter nails.

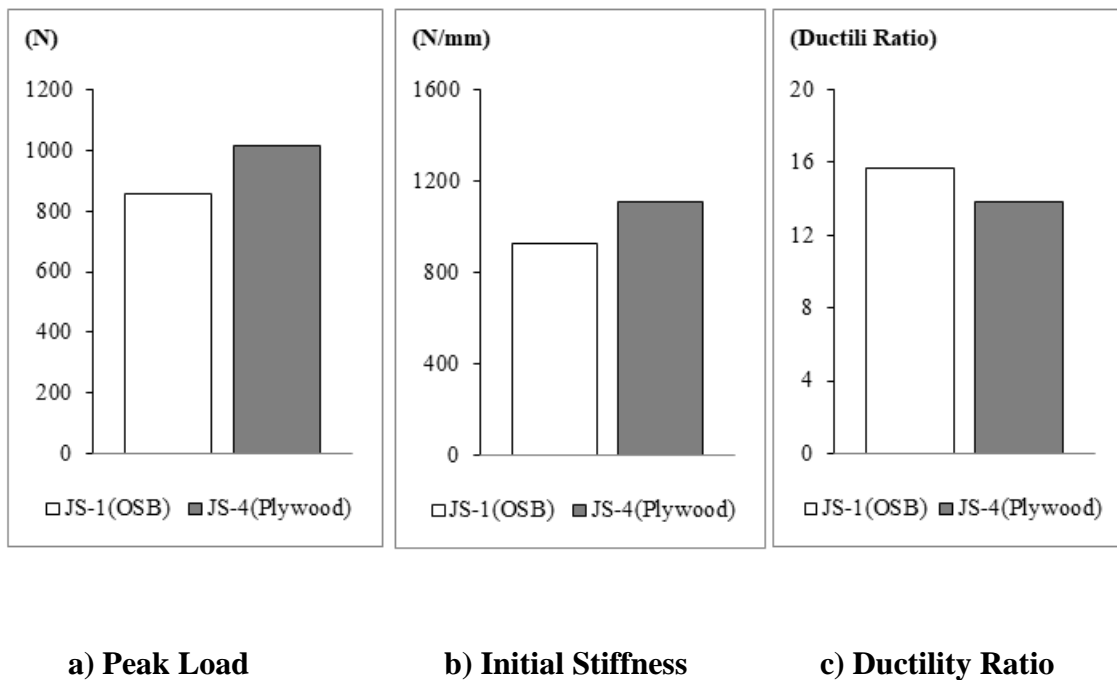


Figure 5.2: Parameter Comparison of Sheathing Panel Type Variation

From Figure 5.2, it can be seen that JS-4 exhibited around 20% higher peak load and initial stiffness. This increase can be attributed, in part, to the increase in thickness of the sheathing panel. Although not directly measured, the density of plywood is anticipated to be higher which again could contribute positively to the increased stiffness of the joint. The two specimens showed similar levels of ductility, as expected. The difference may be attributed to material

variability. The slight decline in ductility for the specimen with plywood sheathing could possibly be explained by the higher density of plywood, where less wood crushing can occur.

In order to investigate the effect of panel thickness on the results a comparison is made between two panels of the same type (OSB) but with different thicknesses. In Figure 5.3, specimen JS-5 had a panel thickness of 15.5 mm while JS-1 had a panel thickness of 11mm. Both specimens were fastened to the lumber pieces by means of 2.84mm diameter nails.

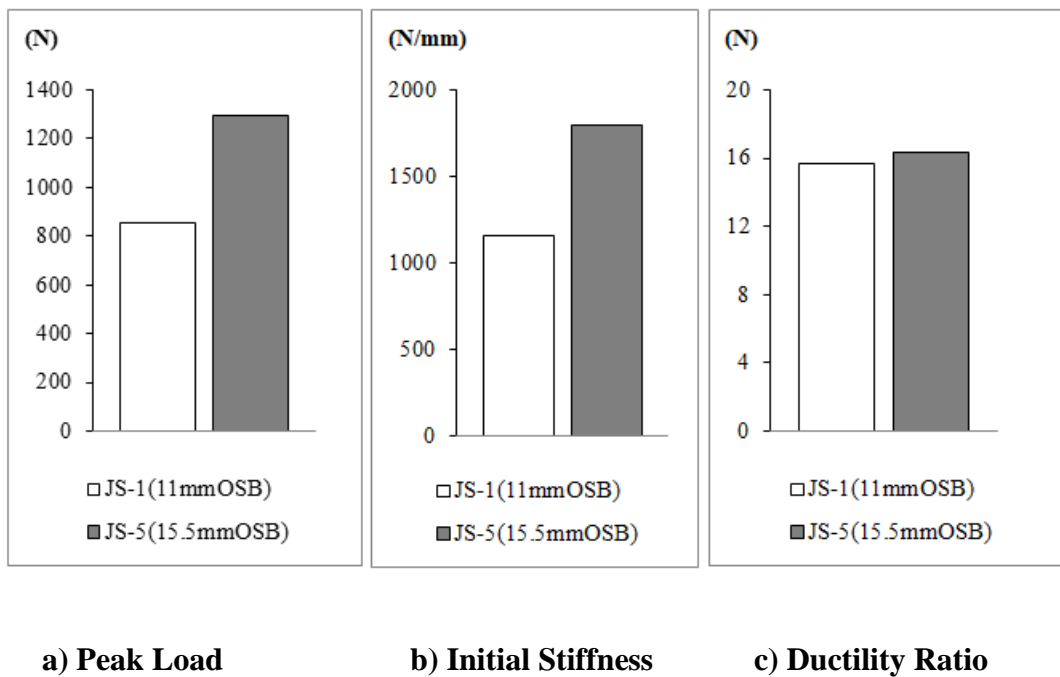


Figure 5.3: Parameter Comparison of Sheathing Panel Thickness Variation

Figure 5.3 emphasizes the positive role that sheathing thickness plays with regards to peak load and stiffness. For example, Figure 5.3(a) showed a 52% increase in peak load when JS-5 is compared to JS-1. The stiffness (Fig. 5.3(b)) also increased with approximately the same amount (55%). This observation is consistent with that made in Figure 5.2. This increase in peak load and initial stiffness seems to be dominated by the side plate thickness rather than the panel type.

As expected, no change was observed in the joint ductility (Fig. 5.3c) since the deformation and failure mechanism is the same.

5.3 Single Storey Full-Scale Shear Wall Analysis

The analyses of the full scale wall tests were presented in Chapter 4. Comparison between the tested walls based on the various parameters investigated is presented in the following sections.

5.3.1 Effect of Aspect Ratio

The aspect ratio of a shear wall is known to have a significant effect on its behavior. For example the Canadian timber design standard (CSA 2014) requires that a maximum aspect ratio of 3.5:1 be considered in the strength design of light frame wood shear walls. As such all walls respecting this aspect ratio and having the same construction detailing are expected to have the same capacity per unit length. Similarly, the deflection equation is a function of the force per nail which is expected to be directly affected by the aspect ratio. In the rigid body rotation component of the deflection equation, the deflection is directly proportional to aspect ratio. The motivation behind this investigation is to study the effect of aspect ratio, as the only variable, on the strength, stiffness and ductility of the walls. Five different aspect ratios were investigated in this study, ranging from 1:1 aspect ratio (possibly the most commonly tested size) to 6:1 ratio, which is deemed too high to provide any capacity or stiffness to the structure (CSA 2014). Figure 5.4 shows the behavior of the walls with different aspect ratios. The comparison between the walls is also made in terms of peak load, initial stiffness and ductility ratio (Figure 5.5).

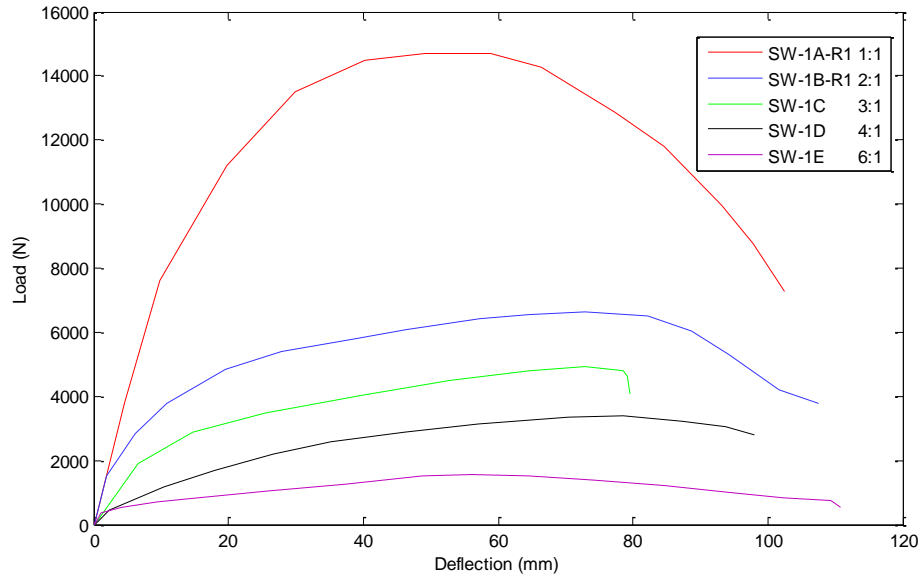
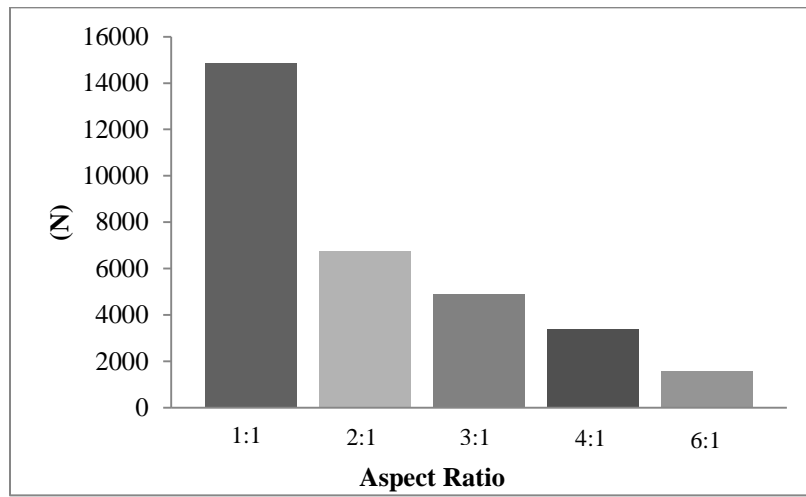
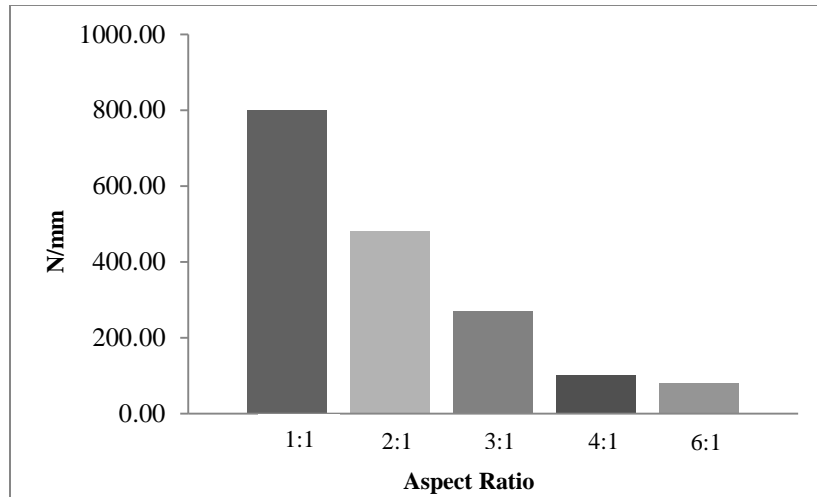


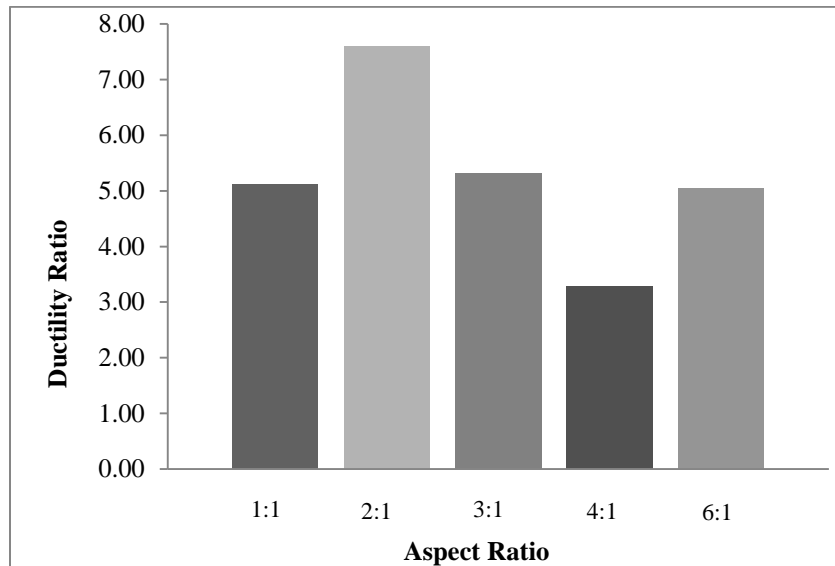
Figure 5.4: Effect of Aspect Ratio on Total Behavior of the Wall



a) Peak Load



b) Initial Stiffness



c) Ductility Ratio

Figure 5.5: Parameter Comparison of Aspect Ratio Variation

As can be seen from Figure 5.5(a) and Figure 5.5(b), the strength and stiffness dropped when the length of the wall decreased. The decrease in wall strength seems to be almost exactly proportional to the decrease in wall length (or increase in aspect ratio). Relative to wall SW-1A (aspect ratio 1:1), walls with aspect ratios of 2:1, 3:1, 4:1 and 6:1 had reductions in peak loads in the magnitude of 0.5, 0.3, 0.2 and 0.1, respectively. These drops in strength correlate almost

directly to the inverse of the wall aspect ratio, as shown in Table 5.1. Figure 5.5 (b) clearly shows that the stiffness also decreases with increased wall aspect ratio. The rate of decrease is also almost proportional to the aspect ratio.

Table 5.1: Peak Load and Stiffness Comparison Ratio

Calculated Ratio	Length Ratio	Peak Load Ratio	Stiffness Ratio
SW-1B-R1/ SW-1A-R1	0.50	0.45	0.60
SW-1C/ SW-1A-R1	0.33	0.33	0.33
SW-1D/ SW-1A-R1	0.25	0.22	0.13
SW-1E/ SW-1A-R1	0.16	0.10	0.10

Figure 5.5(c) shows the walls' ductility as function of aspect ratio. The minimum wall ductility of about 3.0 is obtained for aspect ratio 4:1, whereas in general the ductility is above a ratio of 4.0. There does not seem to be a clear trend when considering the walls' aspect ratios' effect on ductility. It can be noted that wall having aspect ratio 6:1 seems to have a ductility ratio similar to the walls with aspect ratios of 1:1 and 3:1. It is common to attribute such difference with no clear trends to the variability in the wood material. However, when it comes to defining ductility the issue is more complex than simply variability in the material. The issue relates to the definition of ductility ratio as the ratio of the ultimate deflection relative to the yield deflection. This definition works for idealized curves depicting linear elastic-plastic behavior. It is also based on the premise that a distinct yield point and ultimate failure can be defined with certain amount of accuracy. By simply considering the graphs in Figure 5.4, it can easily be concluded that wood shear walls do not behave in such an idealized manner. The yield point is very difficult to define and although ultimate failure is clear, such point should be related to a certain acceptable level of strength. In the current study the Equivalent Energy Elastic Plastic (EEEP) curve was used to determine the yield point and thereby the ductility ratio. This is an approach that may be considered rational and consistent for different configurations, however the

procedure itself may not be adequate to describe the behaviour of wood shearwalls especially regarding ductility values. The large margin of error and inconsistency in the ductility calculation due to variation in determining the yield point has also been discussed by Toothman (2003). In addition, as observed in the results from the current study, variations in determining the ultimate displacement could also contribute to the source of error when estimating the ductility.

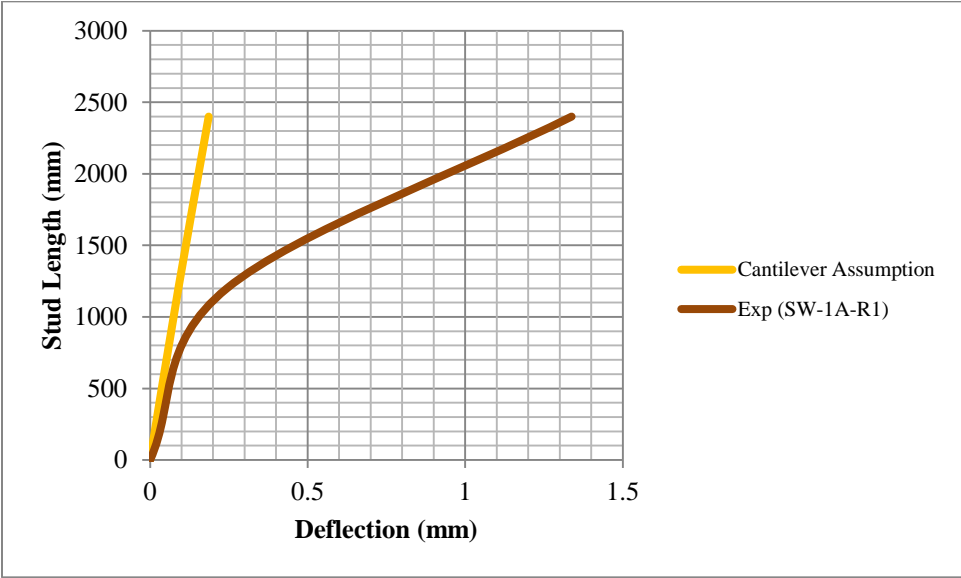
In general, it can be concluded that all the walls, including those with aspect ratios not permitted according to the wood design standard (CSA 2014) followed similar strength and stiffness trends (see Table 5.1) and had sufficient ductility ratios as stipulated for this type of wall systems. This observation indicates that the limit for aspect ratio imposed by the design standard (CSA 2014) seems arbitrary and reliance on narrow segments to provide strength and stiffness might be merited. Ignoring panels with high aspect ratios is prudent and conservative and as such the author is not recommending that any changes be made to the design standard. However, the observation may in part explain some of the discrepancies found between engineering calculations and behavior of actual building with light frame wood shear walls.

5.3.2 Wall Bending Stiffness

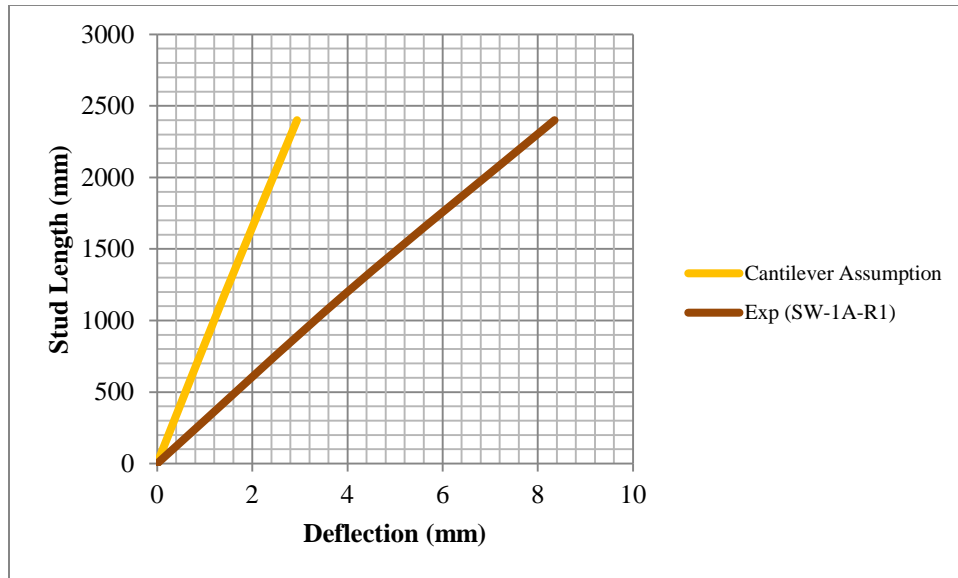
The wood design standard (CSA 2014) assumes the deflection of a shear wall to follow the assumption of a cantilever beam fixed at the base and free on top. Although this assumption has been accepted and verified by other research studies (Martin et. al. 2002, Wang et. al. 2009), the behavior of the individual components in the four term equation has not been verified through experimental investigations. The provisions in the European code 5 (2009) assume the wall bending component of the deflection equation to be negligible and therefore assigns no value to bending. The reason is generally associated with the fact that all codes try to suppress the bending component of a light frame wall shear wall since a desired mode of deformation is in

shear or racking (hence the upper limit of 3.5 on aspect ratio). Racking deformation would cause the sheathing panels to rotate and thereby engage the sheathing to framing connectors, which are the main contributors to the strength, stiffness and ductility in the system.

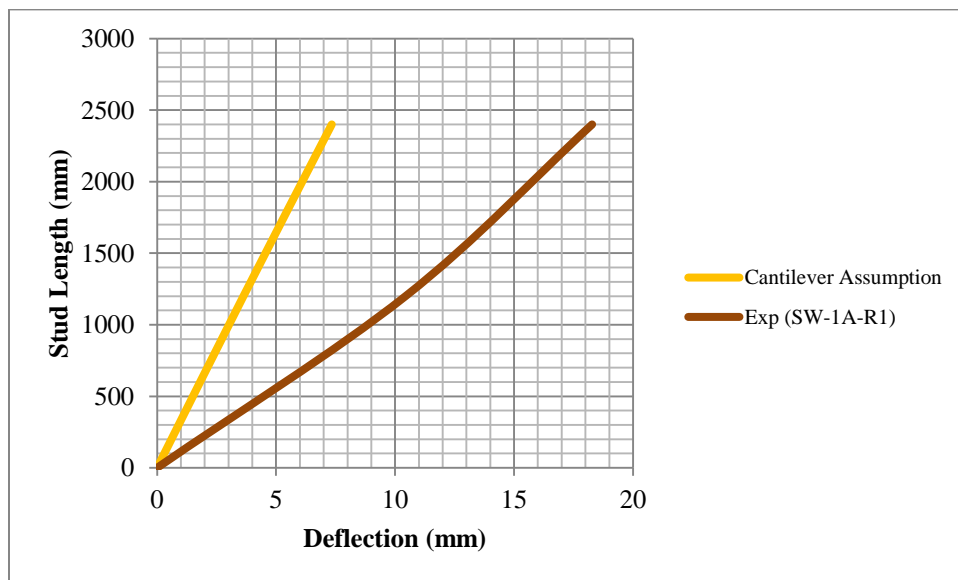
In the current study, the deformed shape of the end stud was monitored by measuring the wall's horizontal deflection at its mid-height and top. The results from the experimental test were compared to the expression provided by the wood design standard (CSA 2014) assuming a cantilevered beam using the cubic function of the wall height. This comparison is demonstrated in Figure 5.6 using wall SW-1A-R1 as an example.



a) Stud Shape at 1000N



b) Stud Shape at 6000N



c) Stud Shape at 10000N

Figure 5.6: SW-1A-R1 End Studs Bending Shape Comparison at Different Load Levels

Due to the non-linear behavior of shear walls, it is important to evaluate the wall deflection at different levels of loading. Figure 5.6 provides comparison at three load levels 1 kN, 6 kN and 10 kN. The 1 kN load (Fig. 5.6a) would reflect the “initial” stiffness of the wall. This level of

loading may be appropriate for estimating the stiffness when the behavior of the wall at low levels of loading is of interest. This could, for example, relate to evaluating the wall or structure's behavior to compare with ambient vibration level measurements or with service level wind loading. Intermediate level loading (Fig. 5.6b) could be of interest to evaluate the wall stiffness when, for example, estimating the building period for seismic design. Figure 5.6c clearly shows that using the cantilever assumption is not applicable to wood shear walls. The rotation of the sheathing counters the stud bending and could lead to a significant overestimation of the stud bending. This behavior was also corroborated by photos of the end stud taken throughout the test (see Chapter 4, Figure 4.9).

To further investigate the bending component of the end stud, comparison between testing and deflection expression in the standard is made in Figure 5.7. The stud deflection component from the experimental study was obtained by subtracting twice the measured horizontal deflection at the middle of the end stud from the stud deflection at the top. The intersection of the curve in Figure 5.8 with the Y-axis indicates the exact point when the stud was straight and no bending deformation could be observed. Negative values in Figure 5.7 indicate that although the stud had positive deflection at its top and mid-height, the top of the stud deflected less than twice the deflection at the mid-height.

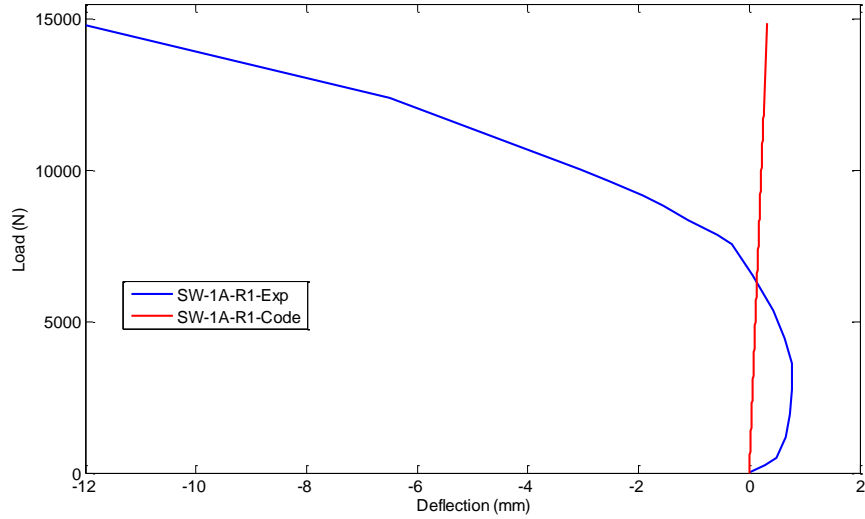


Figure 5.7: SW-1A-R1 Bending (Test vs. Code)

As seen in Figure 5.7, the bending component, albeit small in magnitude, is significant and higher than that predicted by the design expression (equation 2.5). The bending component seems affected by the rotation of the sheathing panels and nail slip. This interaction raises a question about the validity of the four terms in the deflection equations being independent from each other.

To investigate the importance of considering the bending deflection component at different levels of loading, the percentage contribution of the bending is presented in Table 5.2 and Table 5.3 for test results and the code, respectively.

Table 5.2: Bending Contribution in Total Deflection at Different Load Levels for SW-1A-R1(Test)

Load (N)	Percentage to the Loading Capacity	Bending Deflection (mm)	Total Deflection (mm)	Bending Percentage
3720	25 %	0.78	4.77	16.3 %
6000	40 %	0.34	7.75	4.4 %
7439	50 %	0	10.02	0 %
11160	75 %	0	20.2	0 %
14878	100 %	0	55.74	0 %

Table 5.3: Bending Contribution in Total Deflection at Different Load Levels for SW-1A-R1(Code)

Load (N)	Bending Deflection (mm)	Total Deflection (mm)	Bending Percentage
3720	0.08	2.15	3.70 %
6000	0.13	4.30	3.00 %
7439	0.16	6.00	2.70%
11160	0.24	11.50	2.00%
14878	0.32	18.70	1.70 %

The tables show that whereas the contribution of the bending component estimated by the design expression ranges between 2-4%, the actual contribution is very significant at low load levels (around 16%), and becomes insignificant at higher load levels. This again emphasizes the importance of loading level and underscores the need to identify the purpose of using the deflection equation. Depending on its use, the bending component could be significant and therefore should be considered or insignificant and can be ignored.

In following sections the parameters influencing the bending of the wall and their impact on total behavior will be discussed.

5.3.2.1 Effect of Number and Size of Studs

In an effort to investigate the effect of stud size and whether adding more intermediate studs influence the wall behavior were investigated. Figure 5.8 presents the results from varying the number of studs for walls with aspect ratio of 1:1.

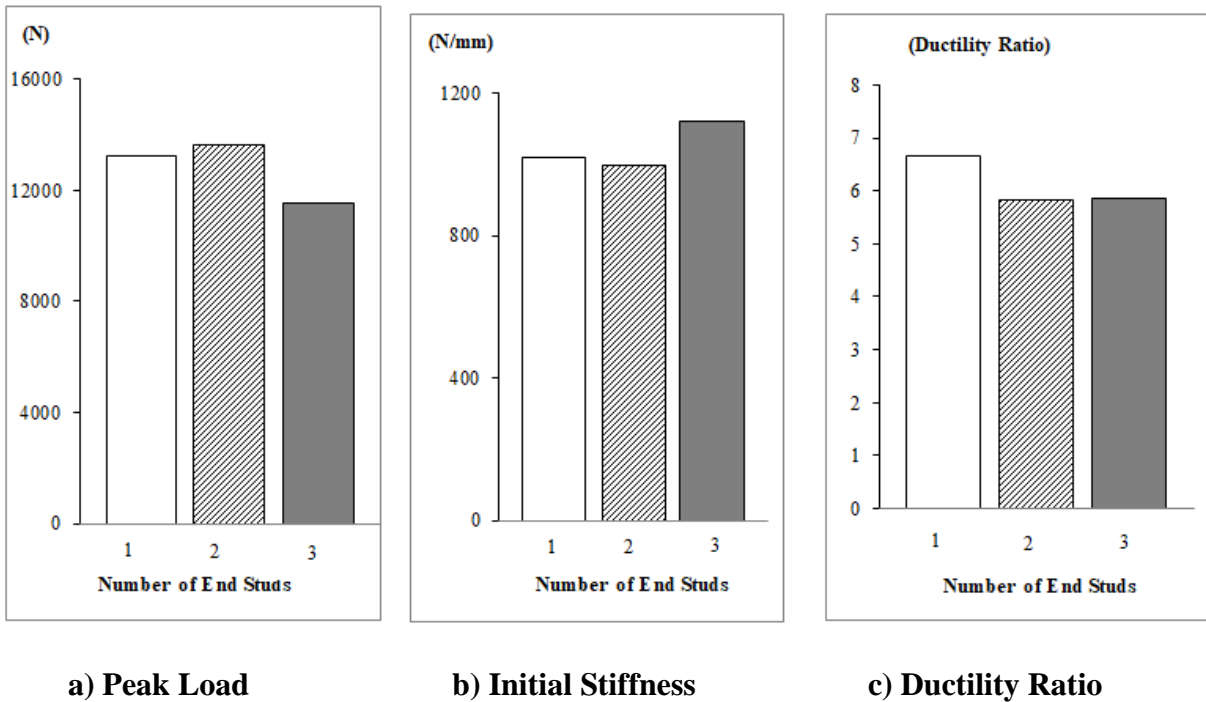
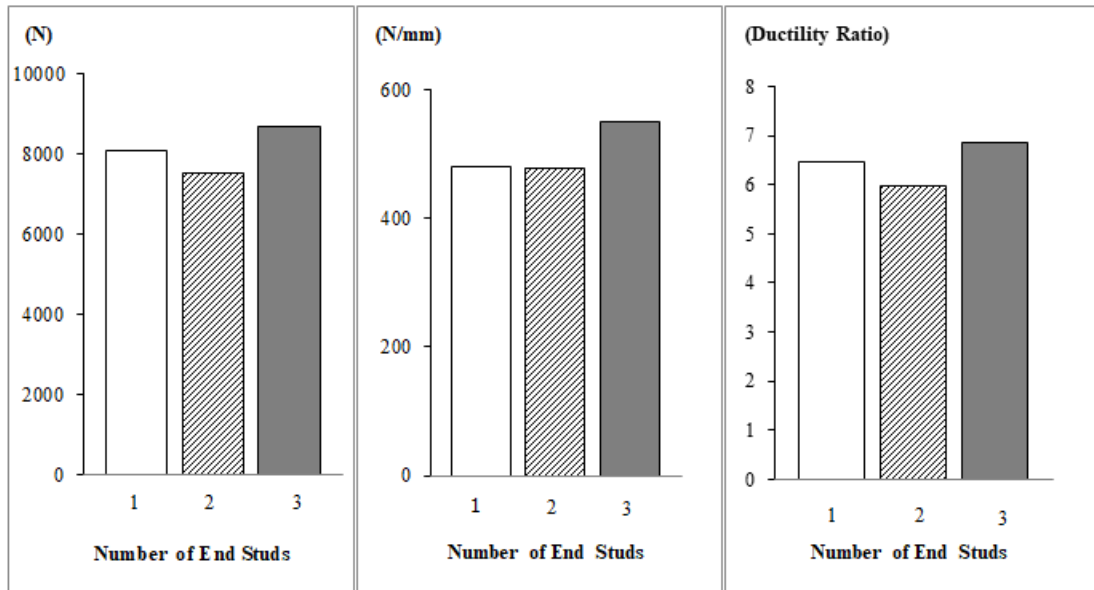


Figure 5.8: Parameter Comparison of Walls with Different Number of End Studs (2"x4" Studs, Walls Aspect Ratio: 1:1)

Figure 5.8 shows that, as anticipated, the wall strength and ductility, which are mainly governed by the behavior of nails connecting the sheathing to the studs, are not affected by changing the number of studs. It was expected that the wall stiffness would increase by approximately 2.6% for every end stud added. The results show that there is a tendency for a slight increase in stiffness when three end studs are used. In all cases, the impact of changing the number of end studs on the structural characteristics of the wall was less than 20%, which can be considered to be within the variability expected in light frame wall systems.

To confirm the validity of this observation, comparisons were also made for the walls with the aspect ratio of 2:1 (Figure 5.9).



a) Peak Load

b) Initial Stiffness

c) Ductility Ratio

Figure 5.9: Parameter Comparison of Walls with Different Number of End Studs (2"x4" Studs, Walls Aspect Ratio: 2:1)

Similar to the walls with the aspect ratio of 1:1, it was found that the strength and stiffness of the wall do not seem to be significantly affected by the number of studs. Also consistent with the observations made in the case for aspect ratio 1:1, the stiffness for aspect ratio 2:1 seem to have a more pronounced increase only when three end studs are used in the wall.

Common stud sizes used in the construction in North America are 2x4 and 2x6 nominal size. The use of 2x4 studs has dominated the existing building stock for several decades, however, recently, the use of 2x6 has become more common due to insulation requirements. The bending shape of the stud for two walls with aspect ratio of 2:1 and consisting of 2x4 and 2x6 studs are compared in Figure 5.10. It can be seen that up to 5 kN, the end studs follow an almost identical path. At higher deflection (where the end stud has an S-shape), the 2x6 studs seem to have slightly less deformation than the 2x4 studs.

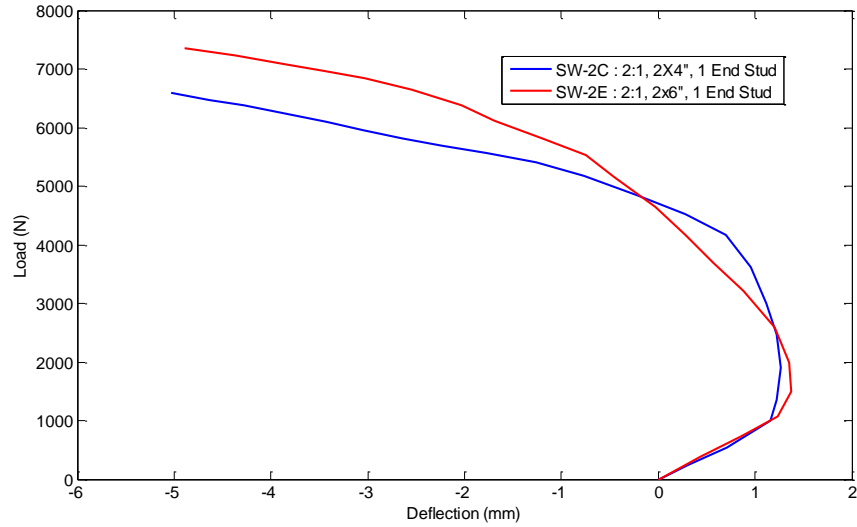


Figure 5.10: SW-2C and SW-2E End Stud Bending

5.3.2.2 Effect of Stud Spacing – Intermediate Studs

Based on the design expression for deflection, the stiffness of the wall is not expected to be affected by intermediate studs. In order to investigate the effect of intermediate studs on the bending component of the wall deflection, walls of the same size and aspect ratios but with two different stud spacing, namely 12” and 24” were investigated. Figure 5.11 shows the deflected shapes of walls.

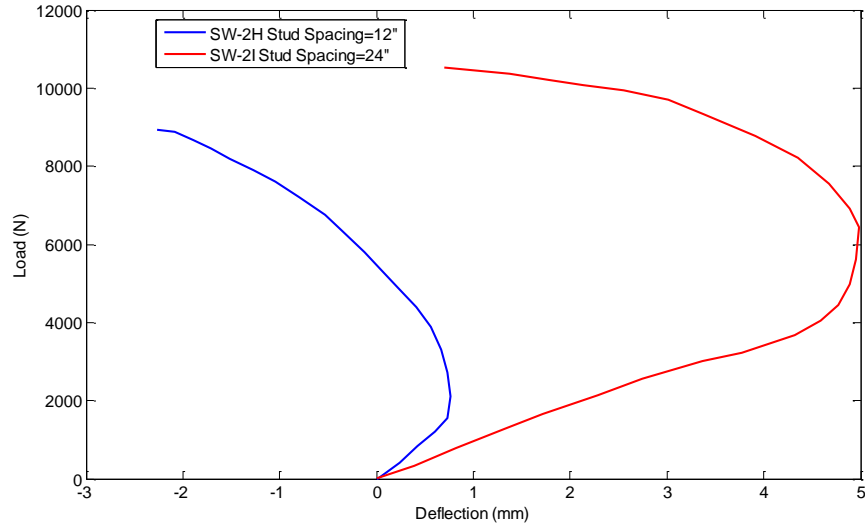


Figure 5.11: Stud Bending of Walls with 12” and 24” Stud Spacing

The results show that the deflection of the end stud is affected by the overall number of studs in the wall and not solely by the end studs.

5.3.3 Wall Shear Stiffness

As discussed in Chapter 2, the lateral force applied on the wall is expected to cause shear strain in the sheathing panels which would result in additional horizontal deflection in the wall. To investigate this behavior, two cable transducers were used to measure the diagonal elongation of the panel during the test. The results from the experimental testing program on the 27 full scale walls show no panel deformation of any significance. This observation is consistent with the assumptions made by McCutcheon (1985), where the contribution from the sheathing panel was suggested to be ignored due to the fact that they contributed little to no additional deformation to the wall relative to the other components. According to the deflection equation in the Canadian design standard (2014), the panel shear deformation is expected to be small yet it is significantly greater than what was observed in the experimental study, as shown in Figure 5.13. Although every effort was made to check the instrumentations before every test, it is not inconceivable that

there were issues with the signal or instrumentation since near zero deformation was not expected. Putting this in perspective however, the maximum expected deformation based on the deflection equation from the wood design standard was about 1.5 mm (Figure 5.12) at maximum load. Furthermore, since the effect is not considered cumulative in design, the contribution seems almost insignificant.

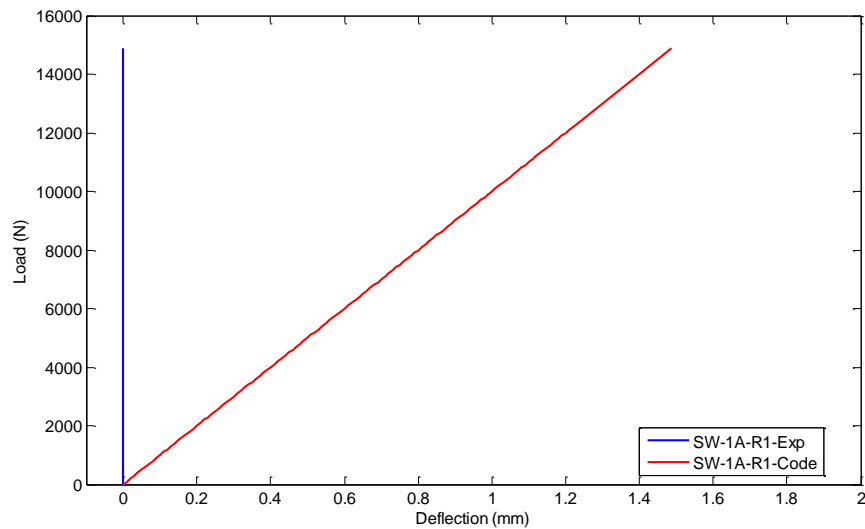


Figure 5.12: SW-1A-R1 Shear Deflection (Test vs. Code)

5.3.3.1 Effect of Sheathing Material Type and Thickness

Consistent with the approach taken at the component level, full scale walls with two different sheathing panels, OSB and plywood, and with two different thicknesses, 11mm and 12.5mm, respectively, were investigated. The results of the comparison between the two walls are shown in Figure 5.13.

From the figure, it can be seen that the wall with plywood (12.5 mm) exhibits slightly higher peak load and initial stiffness than that with OSB (11mm). This increase can be explained, as in the case for component tests, by the increase in thickness and possibly density in the plywood

sheathing panel. The two walls show similar levels of ductility, as expected, with a slight tendency for the plywood sheathing to provide less ductility.

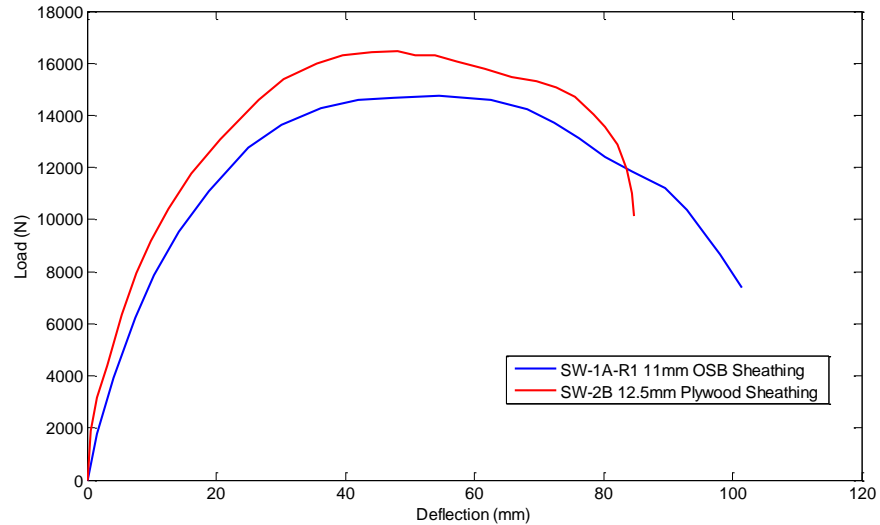


Figure 5.13: OSB Wall vs. Plywood Wall

In order to investigate the impact of sheathing panel thickness on the wall behavior, wall specimen with OSB panel thickness of 15.5mm was compared to the results obtained from that with panel thickness of 11mm.

In general, the results showed a 36% increase in peak load. A significant increase and with similar magnitude, compared to the peak load, was also observed in the stiffness. This increase is slightly less than was observed at the component level tests, but the general trends are similar. No significant change in ductility ratio was observed since the failure mechanism was the same. However, thicker panel delayed the nail and panel separation after peak load was attained, which allowed the wall to undergo higher level of deflection.

5.3.4 Nail Slippage

In order to evaluate the accuracy of the deflection equation in predicting the deflection caused by the nail slip component, comparison between the deflection expression and the experimental results is presented in Figure 5.14.

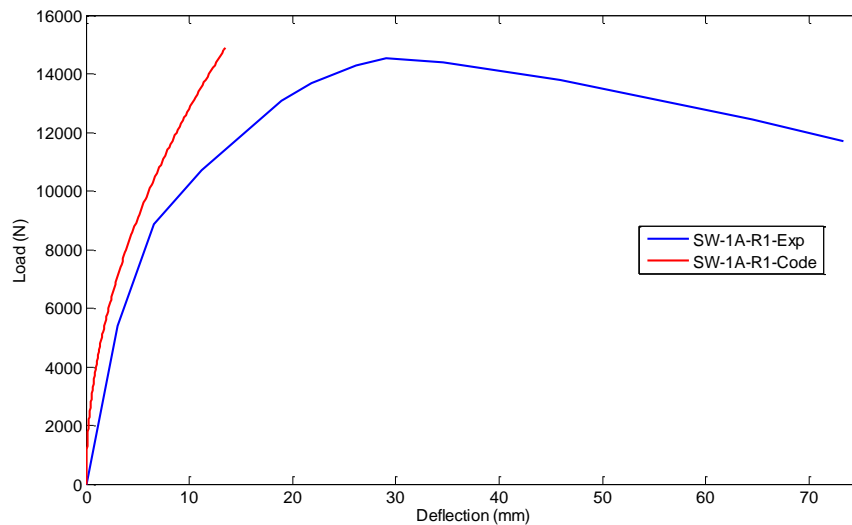


Figure 5.14: SW-1A-R1 Deflection Caused by Nail Slip (Test vs. Code)

As can be seen from Figure 5.14, the deflection expression in the wood design standard (CSA 2014) has a tendency to under-estimate the deflection at all load levels with the difference increasing for higher load levels. It is important to note that, the deflection expression only considers nail diameter when calculating the nail slip (equation 2.17). Other parameters, such as lumber density and sheathing thickness could also influence the nail slip and researchers have tried to modify the model considering such these effects (Wang et al 2009). The study by Wang et. al. (2009) considered the effect of lumber density, sheathing thickness and nail diameter to improve the code accuracy. However, as explained in Chapter 2, the authors noted that their proposed model also under-predicted the deflection.

In the following section the effect of nail diameter, nail spacing as well as end/edge nail spacing on total behavior of the wall is evaluated and discussed.

5.3.4.1 Effect of Nail Size

The effect of varying the nail size is compared by analyzing the results of the wall groups with 2.84mm, 3.32mm and 3.76mm nail diameter nails. The walls were constructed using the same sheathing panel and stud configurations.

Figure 5.15 presents the behavior of the walls, where it can be observed that, the wall strength is significantly affected by the nail diameter. The initial stiffness seems largely unaffected by the nail size, which could be attributed to the variability in the density of framing members. All three walls exhibited similar levels of ductility.

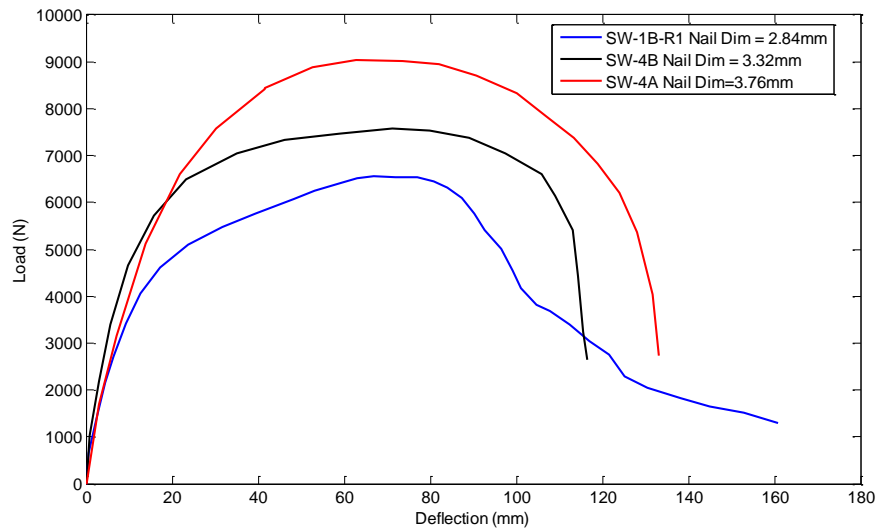
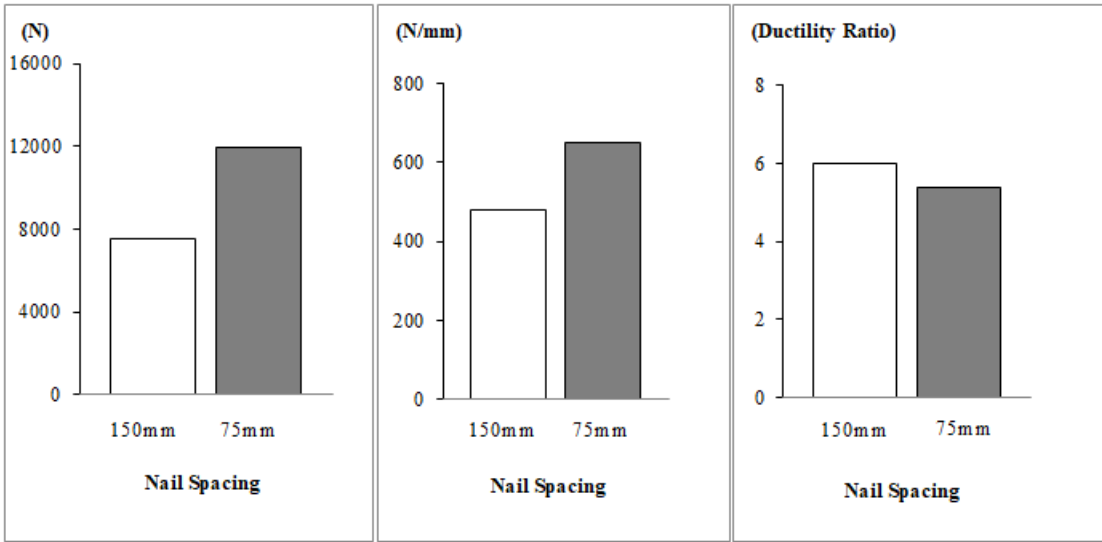


Figure 5.15: Total Behavior of Walls with Different Nail Size

5.3.4.2 Effect of Nail Spacing and End/Edge Distance

Nail spacing is expected to affect the strength proportionally. The stiffness is also expected to be affected in significant way. The effect of nail spacing was investigated by comparing two wall

specimens constructed using the same parameters, but with different nails spacing of 150mm and 75 mm for wall group SW-1B and SW-4C, respectively. The wall behavior is depicted in Figure 5.16 using the EEEP results.



a) Peak Load

b) Initial Stiffness

c) Ductility Ratio

Figure 5.16: Parameter Comparisons of Walls with Different Nail Spacing (Walls Aspect ratio: 2:1)

The results obtained for the peak load show that doubling the number of nails led to only a 58% increase in the peak load. Although according to the wood design standard, both nail spacing should yield similar behavior and provide the same ductility, the wall with 75 mm nail spacing showed a more brittle failure and did not achieve its full capacity.

A significant increase of 35.4% was observed when comparing wall SW-4C with 75mm nail spacing to Wall SW-1B.

It is worthy to mention that unlike ultimate load, initial stiffness may be more sensitive to friction between wall components and fabrications which is difficult to control.

Little difference was observed when the nail edge/end distance was increased from the minimum design requirements of 9.5 mm (CSA 2014) to that of 15 mm. The wall with the reduced end/edge distance showed failure mechanism characterized by nails pulling through the edge of the OSB panel, as shown in Figure 5.17. However, this shift in failure mode occurred towards the end of the testing, and therefore it had little to no effect on the strength, stiffness and ductility. This finding corroborates the use of 9.5 mm as the minimum edge/end distance for light frame wood shear walls.



Figure 5.17: Nails Pull-Through (Nail End/Edge Distance: 9.5mm)

5.3.5 Hold-Down System

Canadian timber design standard (CSA 2104) refers to manufacturer's information to calculate the total vertical elongation of the wall anchorage system including fastener slip, device elongation, anchor or rod elongation, etc. Simpson Strong-Tie anchorage system was used in this study and the total vertical elongation (d_a) was calculated based on the manufacturer's design manual. For the model used (HDU-SDS2.5) in this study, the design load level was 9852.811 N (2215 lb) with the corresponding deflection of 2.23 mm (0.088 inch). Figure 5.18 shows a

comparison between the expected deformation based on the design standard and that observed during the experimental testing.

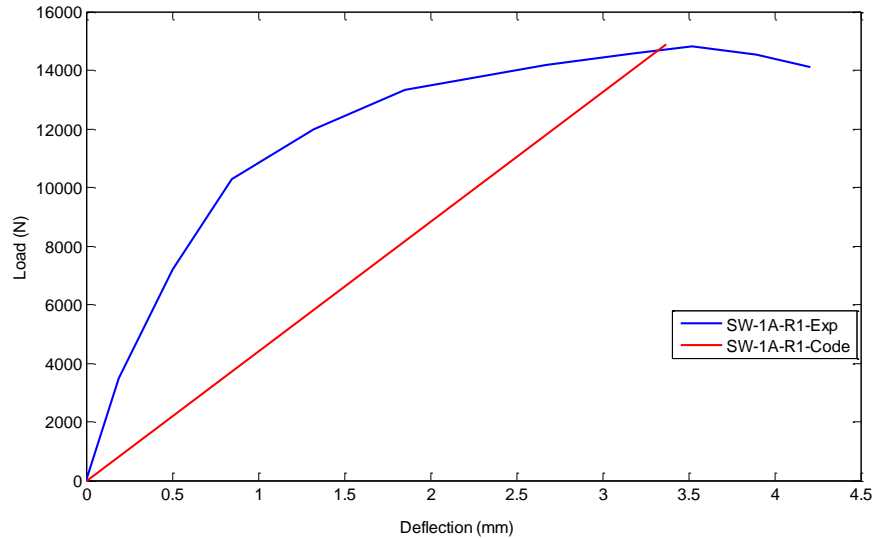


Figure 5.18: SW-1A-R1 Deflection Caused by Hold-Down Elongation (Test vs. Code)

Figure 5.18 shows that the code significantly over-estimates the horizontal deflection caused by hold-down system elongation. The deflection equation expresses that the hold-down behave in a linear elastic manner, which is widely assumed by designers. However, the test results clearly illustrate that the hold-down system used in this study behaves in a non-linear manner with a significantly higher initial stiffness that that assumed in design. This observation was limited to discrete hold-down where significant deformation was observed in the hold-down system. Figure 5.19 shows the hold-down at the end of the test. The wood design standard (CSA 2014) requires that the hold-down system is designed for 20% more load than the design level load in the wall. The design of the hold-downs in the current study was consistent with this requirement, however, at higher load and drift levels, the assumption of linear behavior in the hold-down does not seem to be correct. It is worth noting that the deflection equation in the standard is not intended to capture the behaviour at high load or deformation levels. However, as can be observed in Figure

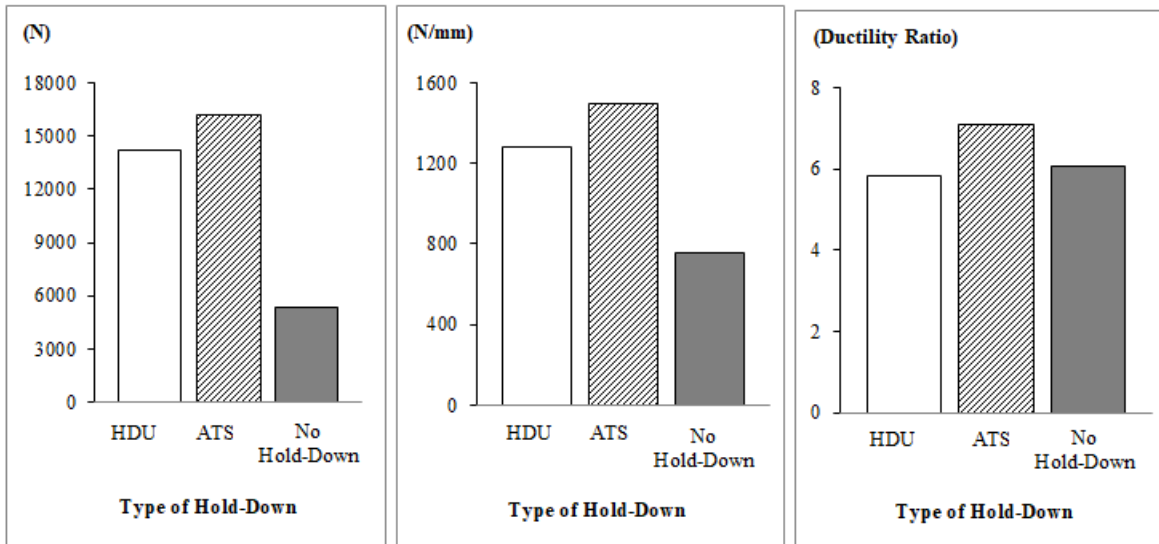
5.18, even at very low load levels, the discrepancy between the predicted and observed behaviour is significant. More work is needed in this area to establish a clearer picture of the deformation behaviour and contributions of hold-down systems to the deflection behaviour of the wall.



Figure 5.19: Screws Group Tear-Out Failure

5.3.5.1 Effect of Hold-Down Anchorage

To investigate the type of hold-down system, three different configurations were considered, namely the ATS (anchor tie-down) system, HDU hold-down (discrete hold-down), as well as the option of no hold-down. Figure 5.20 shows the comparison between the three hold-down options and their effect on the wall behavior.



a) Peak Load

b) Initial Stiffness

c) Ductility Ratio

Figure 5.20: Parameter Comparisons of Walls with/without Hold-Down Anchorage (Walls Aspect Ratio: 1:1)

Having continuous hold-down connections has a positive effect on the capacity, stiffness and ductility of the wall when compared with discrete hold-downs. The impact is not very significant, as expected, because overall the behavior of the wall, when any hold-down is present, is generally governed by the panel to framing connections. The reason for the improvement in the behavior relates to the fact that having a stronger and stiffer hold-down is going to contribute to the overall strength and stiffness of the wall and ensure that the wall does not fail prematurely while also limiting the rigid body rotation, which in turns yields less horizontal deflection of the wall. Clearly, having no hold-down adversely affects the wall capacity and stiffness. The wall with no hold-down had almost one third of the capacity of the wall with continuous hold-down. This value is lower than that predicted by the wood design standard, where a value of J_{HD} equal to 0.48 is obtained. The reduction in stiffness for the wall

with no hold-down was about 50%. This value is consistent with the one obtained using the wood design standard (CSA 2014), where a reduction of 46% is obtained.

Although ductility is more difficult to evaluate, as mentioned in section 5.3.1, the slight increase in ductility when using continuous hold-down could be attributed to shifting more displacement demand on the panel to framing connections. Not having a hold-down at all means that the force is more evenly distributed to the panel to framing nails, which also provide ductility.

5.3.6 Comparison of Observed Behavior and Summation of Deflection Contributors

The previous sections discussed the individual deflection components and compared the documented behavior with that anticipated using the four term deflection equation and other expressions available in the design standard. Although some discrepancies were found, the notion that the total deflection of the wall consists of four terms including stud bending, panel shear, nails slip and rigid body rotation has not been verified. Based on the measured values from the full scale wall tests a curve based on the four components was developed and compared with that representing the applied force as a function of the total horizontal displacement at the top right corner of the shear wall. The comparison (Figure 5.21) shows that in general summing the deflection contributions from the four individual components presents a similar behavior to the overall horizontal deflection of the wall up to peak load. Post peak the two curves have similar magnitude but the trend is different. At this stage of testing, significant nail deformation and panel buckling and distortion are observed, and therefore the measured in plane deformation in the wall is not very reliable.

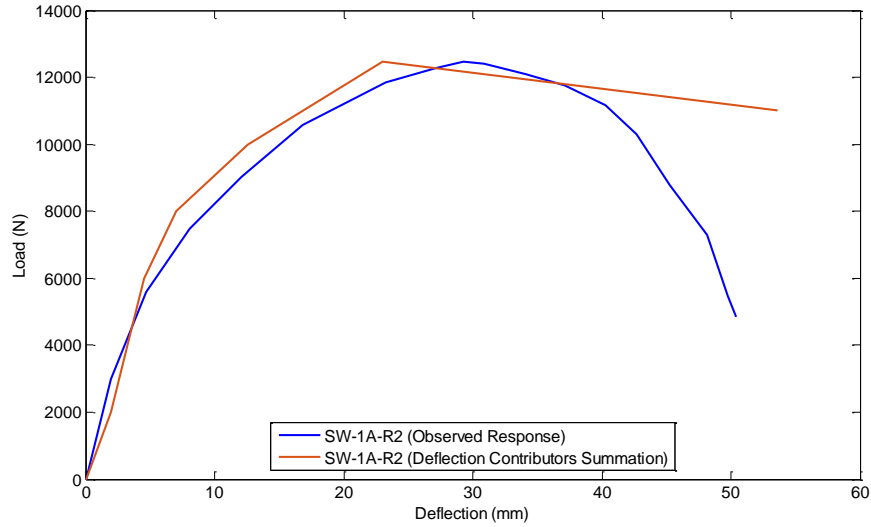


Figure 5.21: Comparison of Observed Response and Deflection Contributors Summation

5.3.7 Comparison between Total Observed Deflection and the 4-Term Design Equation

As shown in previous sections, there are significant discrepancies when comparing the various deflection constituent with those estimated using the design expression. The code expression significantly under-estimates the nail slip especially at high load levels (Figure 5.14). Figure 5.22 shows that the same pattern is also observed when the total deflection is compared to the design expression. This is primarily because the nails slip component is one of the main contributors to the wall deflection.

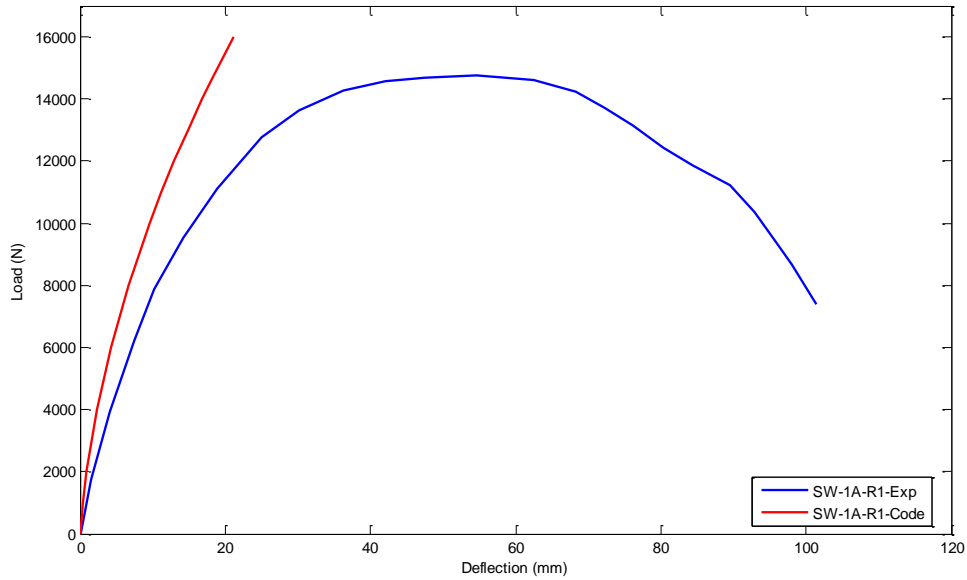


Figure 5.22: SW-1A-R1 Total Deflection (Test vs. Code)

5.3.8 Simplified 2-Term Deflection Equation

As discussed in section 5.3.2, the stud bending deflection is dependent on sheathing panel rotation and nail slip. However, the impact the bending contribution on the total deflection seems insignificant. It was also shown that there was no panel shear deformation for the wall configurations considered during the test. It is therefore proposed that the 4-term equation be simplified to include only the nail slip and hold-down terms. To evaluate this suggestion, the total observed deflection corresponding to only the nail slip and hold-down deflection is compared with that obtained from 4-terms equation for wall SW-1A-R2.

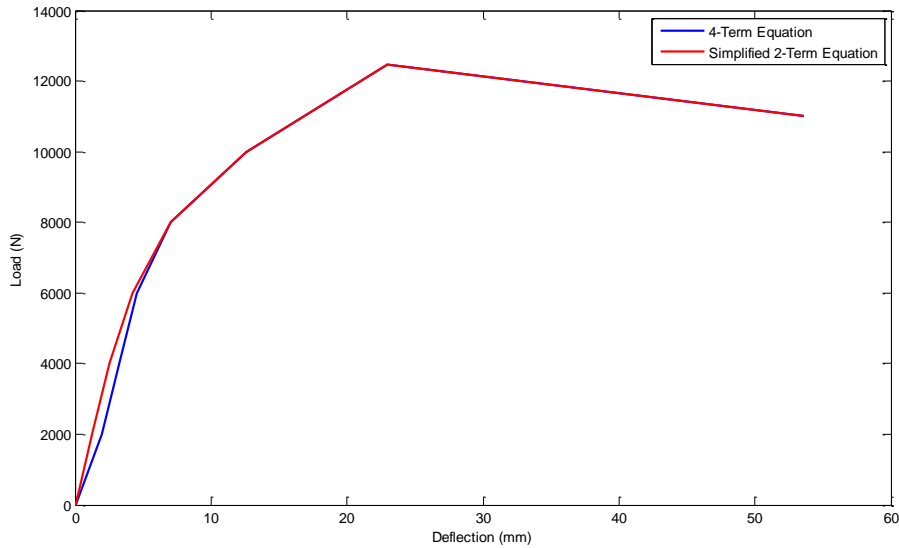


Figure 5.23: SW-1A-R2 4-Term and Simplified 2-Term Equations Comparison

Table 5.4 shows the difference between 4-term and simplified 2-term equation at different load levels.

Table 5.4: Comparison between 4-Term and Simplified 2-Term Equations at Different Load Levels

Load Level (N)	Wall Deflection Considering 4-Term Equation (mm)	Wall Deflection Considering Simplified 2-Term Equation (mm)	Under-Estimation by 2-Term Compared to 4-Term (%)
2000	1.98	1.28	35%
4000	3.26	2.51	23%
6000	4.54	4.24	7%
8000	7.00	7.00	0%

It can be seen that in general the difference between the two deflection expressions is very small due to the fact that the bending and panel shear contribution was observed to be insignificant especially at design levels.

5.4 Two-Storey Half-Scale Shear Walls Analysis

5.4.1 General

The motivation behind investigating two-storey stacked walls was to evaluate the accuracy of the cantilever assumption for multi-storey shear walls, and to validate the cumulative effect model available in the Canadian wood design standard (CSA 2014). The following sections discuss the experimental results and the issues related to design provisions.

The contribution from all individual components in the four-term deflection equation were measured separately in the experimental investigation. The deflected shape for the two-storey system showed similar trends to those found for the single-storey, where the behaviour of the individual components generally did not follow the corresponding expressions in the deflection expression, especially for end stud bending and hold-down deformation. Figure 5.2^ξ shows a comparison between the graph representing the measured horizontal deflection of the wall and the summation of the four components (bending, shear, nail-slip and rigid body rotation) measured experimentally. The graph clearly shows that estimating the total deflection of the wall based on the deflection of its constituents, as outlined in the wood design standard (CSA, 2014), seems valid.

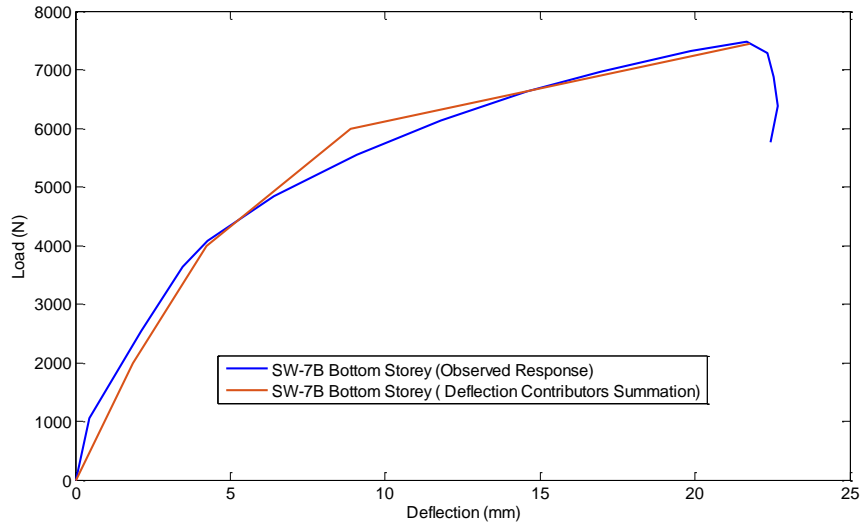


Figure 5.24: Comparison of Observed Response and Deflection Contributors Summation

5.4.2 Cantilever Assumption and Cumulative Effects

To recall from Chapters 3 and 4, during this part of the study, the upper- or lower-storey walls were loaded individually in order to isolate the behaviour and limit the number of variables.

It is possible to use the obtained data from the test directly to investigate the cumulative effects.

The approach used is predicated on the notion that when the bottom wall (SW-6B) is loaded, it will behave similar to the single storey case. Since no shear deformation or nail slip is expected in the upper wall, the only additional deformation obtained would be due to cumulative effects caused by deformation in the bottom wall only. These additional deformations can be compared with those anticipated from the design expression. To eliminate any uncertainties stemming from the other components (e.g. shear or nail slip) from the bottom storey, the actual data obtained from the experimental testing was used. The addition of the cumulative effect however, was done through the design expression and then the total deflection and deflected shape were compared with those obtained experimentally for the top storey wall. Table 5.9 compares the expected

deflection, as a function of increasing load, at the upper-storey wall considering cumulative effects to the total observed deflection for wall SW-6B.

Table 5.5: Expected Deflection Considering Cumulative Effects vs. Observed Deflection (SW-6B-Top Storey Total Deflection)

Load Level (N)	Bottom Storey Deflection (mm)	Cumulative Effects Due to Bending (mm)	Cumulative Effects Due to Rigid Body Rotation (mm)	Expected Total Deflection Considering Cumulative Effects (mm)	Total Observed Deflection (mm)	Ratio of Calculated to Observed
1000	0.88	0.01	0.20	1.09	1.17	0.93
2000	1.76	0.03	0.33	2.12	2.35	0.90
4000	3.51	0.07	0.65	4.23	4.69	0.90
6000	7.40	0.10	1.76	9.26	10.17	0.91
8000	21.43	0.13	4.60	26.16	27.82	0.94

Table 5.5 shows that the total observed deflection is slightly higher (about 10%) than the estimated deflection based on the cumulative effect assumption. It can also be observed that the majority of the cumulative effect stems from the rigid body rotation due to deformation in the hold-down devices.

The aim of applying the load at the upper-storey level (SW-7B) was to investigate a situation where the bottom-storey shearwall receives the same load and have the same shear deformation since the nail size and pattern is identical to the upper-storey shearwall. From the deformed shape of the wall it would be feasible to assess whether the cantilever assumption is reasonable and whether the cumulative effect due to bending is appropriate. Table 5.6 compares the expected total deflection at top storey to the observed total deflection.

Table 5.6: Expected Deflection Considering Cumulative Effects vs. Observed Deflection (SW-7B-Top Storey Total Deflection)

Load Level (N)	Bottom Storey Deflection (mm)	Cumulative Effects Due to Bending (mm)	Cumulative Effects Due to Rigid Body Rotation (mm)	Expected Total Deflection Considering Cumulative Effects (mm)	Total Observed Deflection (mm)	Ratio of Calculated to Observed
1000	1.45	0.05	0.43	2.95	2.00	1.47
2000	2.38	0.10	0.84	4.86	4.38	1.10
4000	6.47	0.20	1.84	13.28	10.56	1.26
6000	15.51	0.30	4.26	32.3	29.29	1.10

The results show that the deflection expression including the cumulative effects from bending and rigid body rotation overestimates the wall deflection. This is the opposite trend to that observed when the load was only applied at the bottom-storey. The results seem to indicate that the cumulative effects due to bending is not as pronounced in reality as the cantilever assumption provides. This could be explained by the fact that in reality, studs are not continuous even when walls are not separated by floor diaphragms (i.e. configuration tested).

Analyzing the results of wall SW-7B shows that the approach of considering cumulative effects is applicable in multi-storey walls where the rigid body rotation of bottom storeys due to hold-down system elongation plays a main role. However, the rotation of bottom storey is not fully transferred to the top storey.

It should be noted that the test setup does not exactly mimic reality in the sense that it omits the contribution of the out of plane stiffness of the diaphragm. Since including a diaphragm in the experimental testing was onerous, the issue of floor out of plane rigidity was address through the numerical model presented and discussed in Chapter 6. Also, the impact of the cumulative effect becomes much clearer when the number of storeys increases, and given the limitations of the

current study of not being able to test wall assemblies that are more than two storeys, extending the investigation to taller buildings was also addressed in the numerical model in Chapter 6.

6 ANALYTICAL RESULTS AND DISCUSSION

6.1 Background

Numerical modeling is usually considered complementary to experimental testing and essential to proper interpretation of experimental findings. Once validated, models of components, substructures, and entire structural systems are used to expand the knowledge beyond what may be possible experimentally.

Several attempts have been made in the past decades to model the behavior of light frame wood shear walls (e.g. Foschi 1977, Itani and Cheung 1984, Falk and Itani 1989, Dolan 1989, Van de Lindt 2005, Kasal 2005, Doudak 2005, Asiz 2009, Casagrande et. al 2015). Existing models consist of various levels of detailing, while attempting to represent the physical behaviour of the shearwall system. The following sections describe the detailed modeling procedure employed in the current study, which models the walls at the levels of nails. Comparisons between the models and the experimental results described in Chapter 4 and discussed in Chapter 5 are undertaken. The validated model is then used to explore the behaviour of multi-storey light frame wood buildings, with special emphasis on the accumulation of deflection along the height of the structure.

6.2 Modeling Procedure

The shear wall model was developed using the commercially available software SAP 2000 (Computers and Structures, Inc. 2017). All wall components including studs, sheathing and fasteners were modeled by using the tools available in the software. Framing elements such as studs and bottom and top plates, linear “frame” elements were used, while “membrane” elements were used to model the sheathing panels. Releases were provided at the end of all framing

members to simulate pin-ended conditions. Elastic orthotropic material properties were assigned to all wood elements, with properties such as modulus of elasticity, shear modulus and Poisson's ratio defined in the three orthogonal directions. Table 6.1 and Table 6.2 provide the material properties used in the model. It should be noted that the modulus of elasticity of the studs and the nail slip curves were obtained from component tests conducted in this study. Other properties were obtained from published literature such as the engineering wood design standard (CSA 2014) and the wood handbook (FPL 1999).

Table 6.1: Stud Properties

Stud Size	Modulus of Elasticity (E) (N/mm²)	Poisson's Ratio
2"x4" Studs	10800.00	0.30
2"x6" Studs	11522.00	0.30

Table 6.2: Sheathing Material Properties

ID	Shear-Through-Thickness Rigidity (B_v) (N/mm)	Poisson's Ratio
11mm OSB	11000.00	0.30
12.5mm Plywood	5700.00	0.30

Figure 6.1 shows a typical shear wall model, where the framing, sheathing panels, panel to framing connections, and hold-down are highlighted.

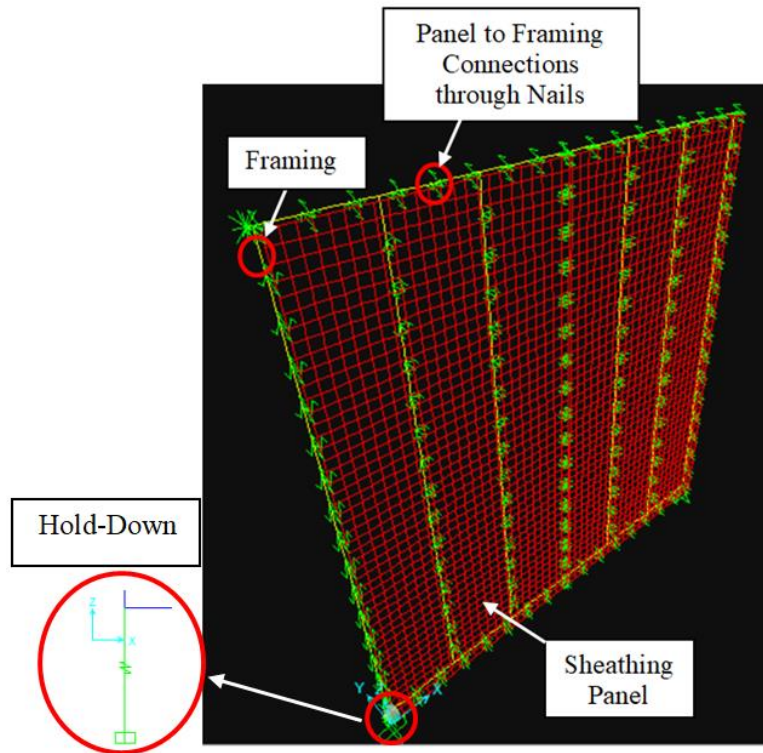


Figure 6.1: Shear Wall Model in SAP 2000

The mesh size in the model was selected to be 50mm based on a study by More et. al (2015).

The sheathing to framing nails were modeled using nonlinear springs (links) with properties in the horizontal and vertical directions. The nailed connections were represented through force-slip curves obtained from component tests (see Chapter 3 and 4). The non-linear behaviour of the nails, including the strength and stiffness degradation, was incorporated into the model using a multi-linear load-deformation function fitted to experimental results (Figure 6.2). Similarly, the hold-down devices were modeled using spring elements obtained from experimental component of the current study (Figure 6.3).

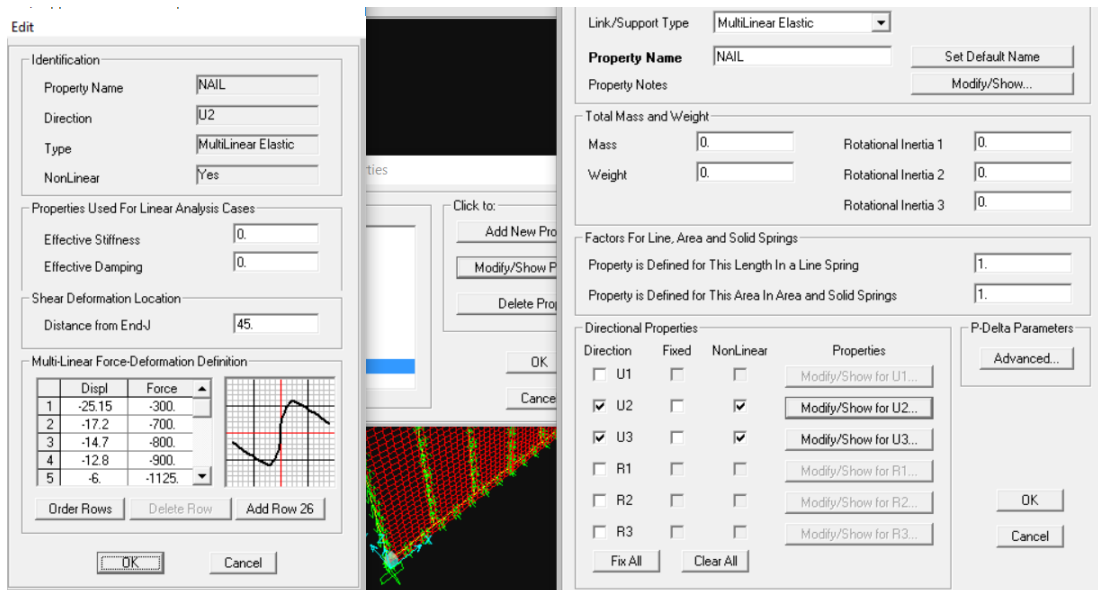


Figure 6.2: Modeling the Nails as Nonlinear Links (SAP 2000)

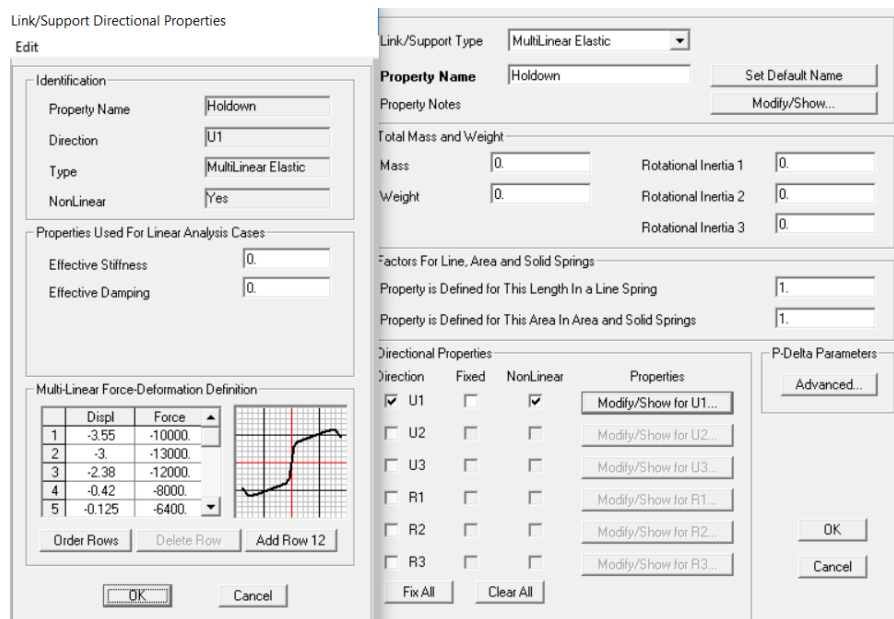


Figure 6.3: Modeling the Hold-Down as Nonlinear Spring (SAP 2000)

A schematic of the modeling approach is illustrated in details in Figure 6.4. The figure shows the all the components contributing to the deflection of the shearwall.

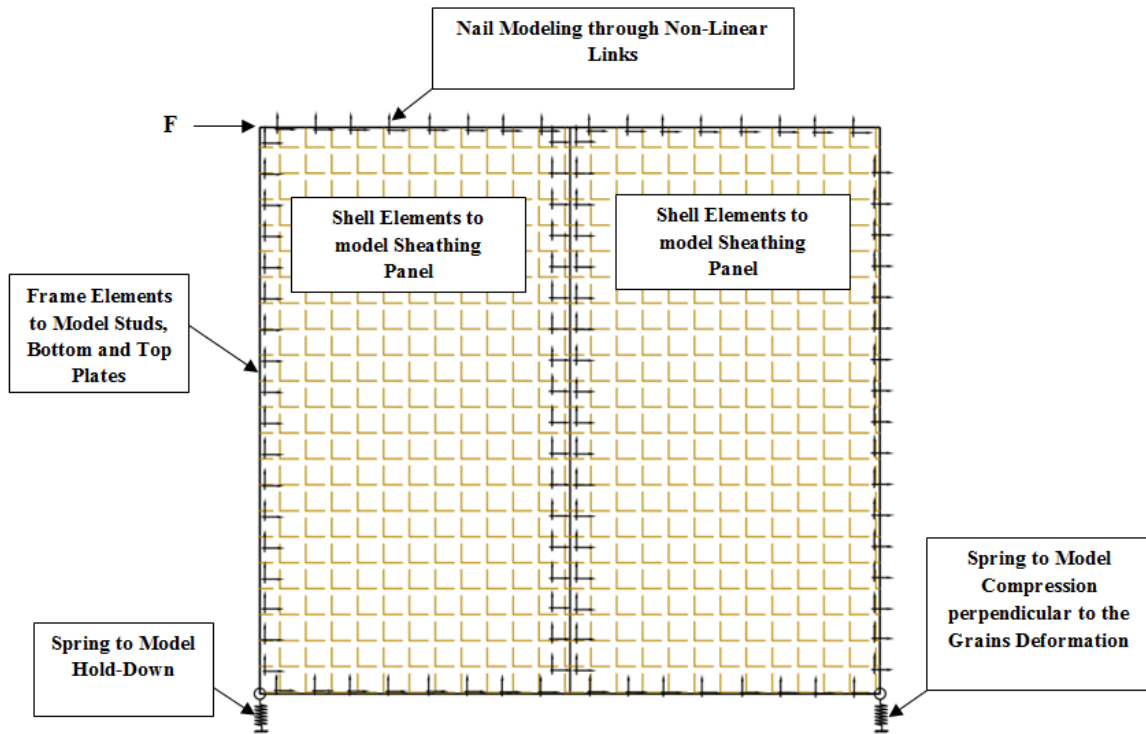


Figure 6.4: Detailed Full-Scale Shear wall Model (SAP 2000)

6.3 Results from Numerical Models

Figure 6.5 shows a typical shearwall deflection response for wall SW-1A. In order to analyze the results in a consistent manner, EEEP analysis, similar to that used in the experimental study, was used here to extract information on the ultimate wall strength, initial stiffness and wall ductility obtained from the numerical model. A summary of the results for the single-storey and two-storey wall models can be seen in Tables 6.3 and 6.4, respectively.

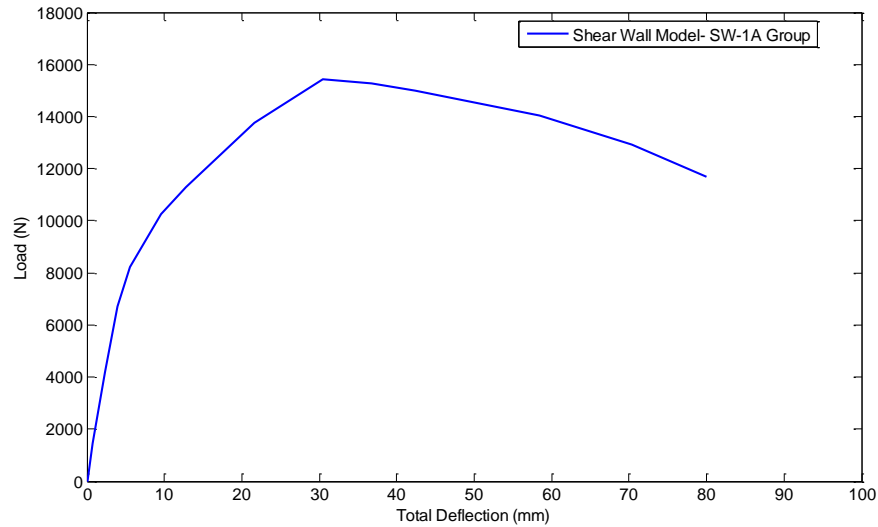


Figure 6.5: Typical Result of Single-Storey Shear Wall Model (SAP 2000)

Table 6.3: Shear Wall Model EEEP Analysis Results

ID	Peak Load (N)	Deflection At Peak Load(mm)	Initial Stiffness (N/mm)	Ultimate Deflection (mm)	Yield Load (N)	Ductility Ratio (Δ_u/Δ_{yield})
SW-1A	15426	30.4	1670	70.4	15700	7.5
SW-1B	7800	39.6	880	72	6690	9.5
SW-1C	4284.4	54.4	270	80	3950	5.40
SW-1D	2823.1	63	150	100	2610	5.8
SW-1E	1502.7	49	70	100	1360	4.9
SW-2A	15368	33.6	1610	77.6	13560	9.2
SW-2B	15443	29.6	1790	75.2	13770	9.8
SW-2C	7804.7	36	870	81.6	6860	10.32
SW-2D	7984.8	31.2	970	75.6	7030	10.38
SW-2E	7697	44.4	610	80.4	6730	7.3
SW-2F	15402	30.4	1830	71.2	13620	9.6
SW-2G	7874.2	30.4	820	76.8	7020	8.97
SW-2H	12768	35.2	1570	60.8	11490	8.31
SW-2I	12482	34.4	1460	38.4	10790	6.22
SW-3A	11405	44	1200	140	10150	16.54
SW-3B	15842	42.4	1720	80	10920	12.60
SW-4A	11185	60.2	630	96	9660	6.2
SW-4B	8422.6	31.2	690	72	7580	6.52
SW-4C	9761.8	31.9	710	81.4	8840	6.54
SW-4D	9761.8	31.9	710	81.4	8840	6.54
SW-5A	15346	30.4	1790	73.6	13680	9.63
SW-5B	7577	36.8	570	76	6770	6.42
SW-5C	8863.8	21.6	1540	44.8	7920	8.71

Table 6.4: Two Storey Shear Wall Model EEEP Analysis Results

ID	Storey Number	Peak Load (N)	Deflection At Peak Load(mm)	Initial Stiffness (N/mm)	Ultimate Deflection (mm)	Yield Load (N)	Ductility Ratio (Δ_u/Δ_{yield})
SW-6B	1	8370.60	27.50	1253	40	7380	6.79
	2	-	34.24	987.7	44.03	7260	5.99
SW-7B	1	8123.10	29.24	933.96	34.10	7040	4.52
	2	8123.10	25.96	0.99	40.10	6080	6.53

6.4 Discussion

6.4.1 Single-Storey Full-Scale Walls

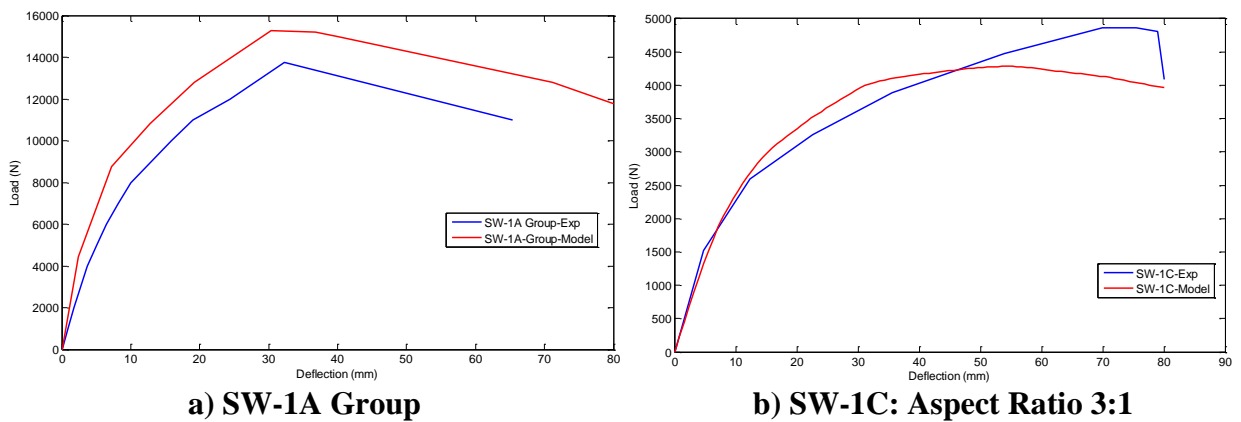
Table 6.5 provides ratios of the respective values from the model to those obtained from the shear wall tests. Also provided in the table are the average and the coefficient of variation (COV) for each variable.

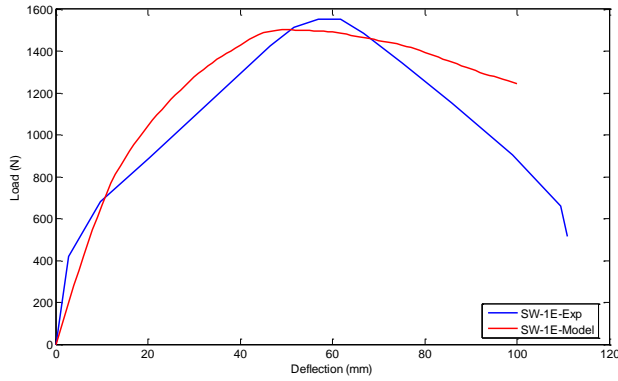
Table 6.5: Ratios of Values Obtained from the Model to Tested Values(Single Storey)

ID	Peak Load	Initial Stiffness	Ultimate Deflection	Yield Load	Ductility Ratio
SW-1A	1.13	1.67	1.61	1.32	1.15
SW-1B	1.15	1.85	0.77	1.15	1.25
SW-1C	0.87	0.99	1.01	0.97	1.02
SW-1D	0.84	1.51	1.02	0.88	1.76
SW-1E	0.97	0.88	1.2	1.08	0.97
SW-2A	1.16	1.58	1.02	1.16	1.39
SW-2B	1.19	1.02	1.94	1.19	1.67
SW-2C	0.96	1.81	0.84	0.95	1.6
SW-2D	0.92	1.76	0.77	0.9	1.52
SW-2E	0.81	1.89	0.98	0.87	2.13
SW-2F	1.18	1.61	1.66	1.16	2.30
SW-2G	1.09	1.91	1.33	1.08	2.35
SW-2H	0.83	0.99	0.7	0.83	0.82
SW-2I	0.97	1.47	0.52	0.91	0.84
SW-3A	1.11	1.94	0.77	1.07	1.41
SW-3B	0.97	1.54	0.98	0.74	2.05
SW-4A	1.22	1.64	0.83	1.16	1.18
SW-4B	1.1	1.37	0.66	1.07	0.85
SW-4C	0.82	1.10	0.89	0.8	1.22
SW-4D	1.13	0.94	1.61	1.32	1.15
SW-5A	0.87	0.99	1.0	0.97	1.02
SW-5B	0.97	0.88	1.2	1.08	0.97
SW-5C	1.15	1.85	0.77	1.15	1.25
Ave(COV)	1.02(13.45%)	1.44(26.38%)	1.05(34.76%)	1.04(15.32%)	1.40(64%)

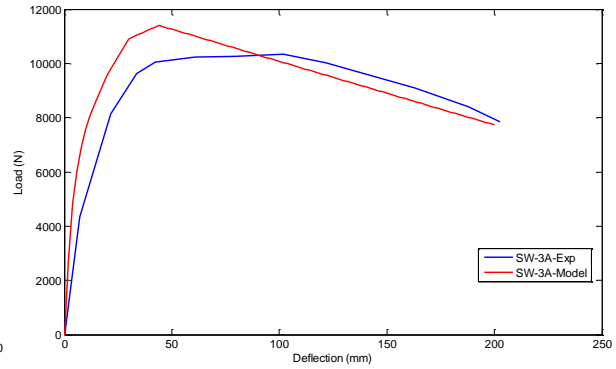
As seen in Table 6.5, the model’s prediction of the peak load seems reasonably accurate with an average ratio (of model to test) of 1.02 with the coefficient of variation (COV) of 13.5%. The initial stiffness is notoriously difficult to predict and is typically associated with higher variability (COV = 26.38%). The model in the current study seems to over-predict the initial stiffness of the walls; on average by 44%. This result was unexpected since models typically lack the ability to simulate conditions such as friction and tightening of connections associated with testing. The ultimate deflection and yield load were captured reasonably well by the model with a slight over-estimation for both values by an average of 5% and 4%, however the variability in the values representing ultimate deflection seemed higher than expected (34.8%). The model over-predicted the ductility of the walls by a ratio of 1.4. This is not unusual since the model assumes that the failure mode is restricted to yielding in the panel to framing nails, where nail pull-through failure found at high load levels during testing is not considered in the model. The over prediction of the ductility could possibly also be linked to the overestimation of the stiffness.

A comparison between the wall behaviour generated by the model and that obtained experimentally can be found in Figure 6.6.

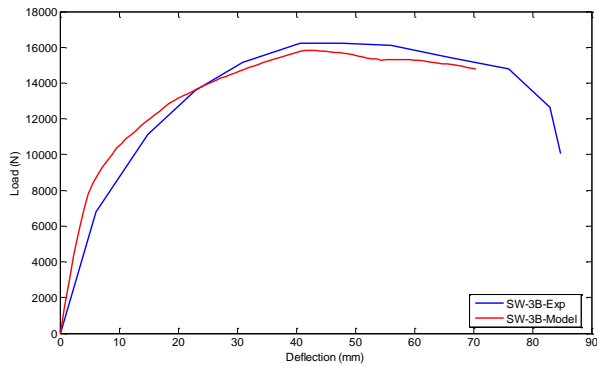




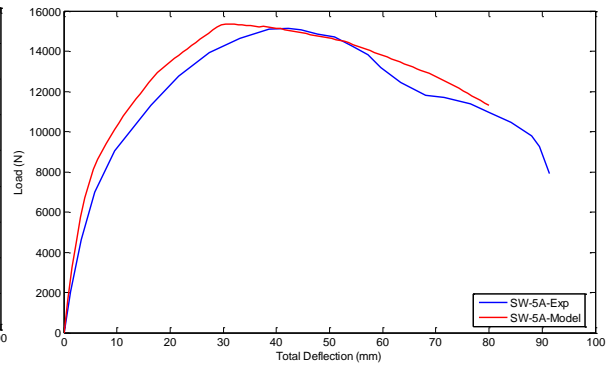
c) SW-1E: Aspect Ratio 6:1



d) SW-3A: (15.5mm OSB)



e) SW-3B: (12.5mm Plywood)



f) SW-5A: Tie-Rod Hold-Down

Figure 6.6: Model and Test Total Behavior Comparison for Walls with Different Aspect Ratio and Construction Detailing

In general it can be concluded that the model is capable of predicting the single-storey wall behaviour, as illustrated in Figure 6.6, with reasonable accuracy.

6.4.2 Two-Storey Half-Scale Walls

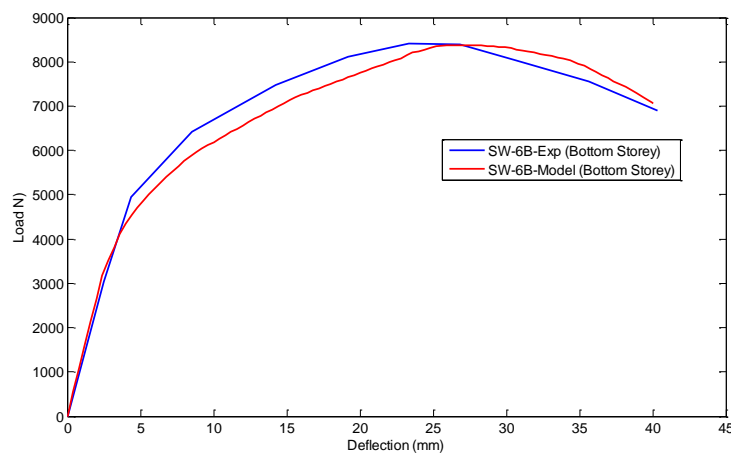
Table 6.6 compares the EEEP analysis values obtained from the model to those obtained from the shear wall tests.

Table 6.6: Ratios of Values Obtained from the Model to Tested Values(Two Storey)

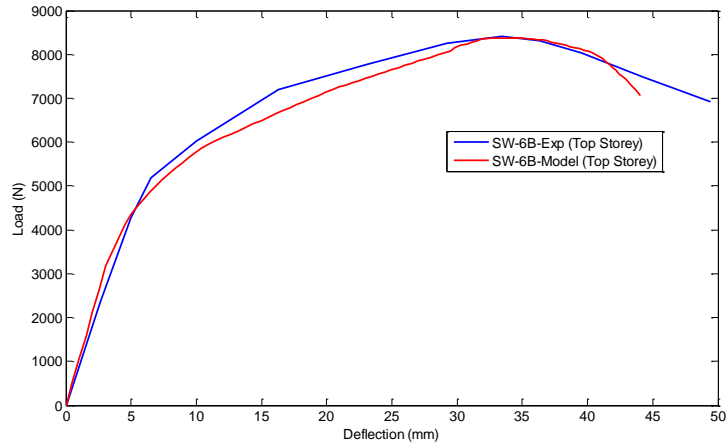
ID	Storey	Peak Load	Initial Stiffness	Ultimate Deflection	Yield Load	Ductility Ratio
SW-6B	1	0.99	1.04	0.99	0.97	1.06
	2	-	1.10	0.89	0.97	1.01
SW-7B	1	1.09	0.89	1.50	1.10	1.23
	2	1.09	0.83	0.87	1.00	0.72
Ave		1.04	0.97	1.07	1.01	1.00
(COV)		-	(6.80%)	(13%)	(28%)	(6.1%) (21.1%)

In general, it can be seen from Table 6.6 that the model is capable of predicting the wall behaviour with reasonable accuracy. Similar to the case of single storey model, the most variability is observed for the ultimate deflection and ductility estimates. Although the average values are around unity, it is not practical to obtain reliable statistical analysis given the sample size.

Figure 6.7 and Figure 6.8 compare the total behavior of SW-6B and SW-7B obtained from the test and those obtained from the model.

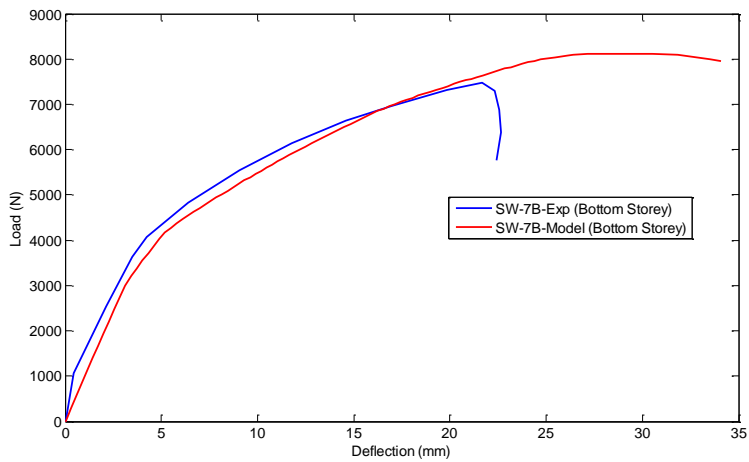


a) Bottom Storey

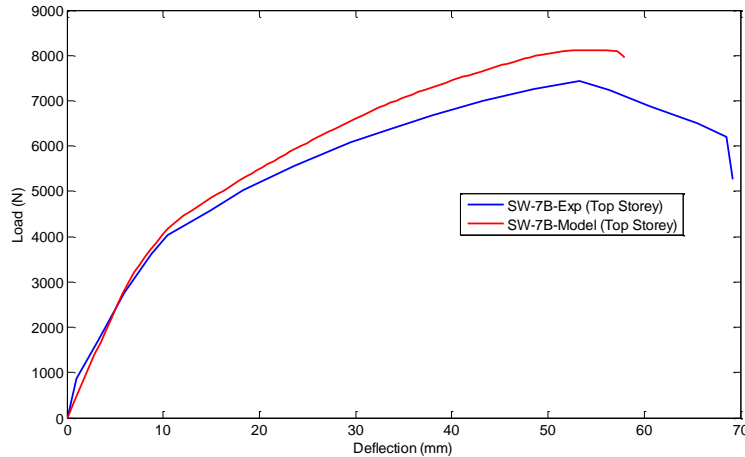


b) Top Storey

Figure 6.7: SW-6B Model and Test Total Behavior Comparison



a) Bottom Storey



b) Top Storey

Figure 6.8: SW-7B Model and Test Total Behavior Comparison

From Figure 6.7 and 6.8, it can be seen that overall the model fit is reasonable. The exception is the comparison shown in Figure 6.8 (a), where the wall failed slightly prematurely due to nail pull-through. As mentioned earlier, the model is obviously not equipped to capture such failure.

6.4.3 Cumulative Effects

The approach used to investigate the cumulative effects is analogous to that described in Chapter 5 for the experimental phase of the study. The analysis consists of obtaining the deflection from the bottom storey, and adding the cumulative effects using the design expression (CSA O86) and then comparing the results with those obtained from the model at the top-storey wall. Table 6.7 presents the results from such analysis for wall SW-6B at different load levels. The table also provides a ratio of calculated deflections to those obtained from the model.

Table 6.7: Expected Deflection Considering Cumulative Effects vs. Observed Deflection (SW-6B-Top Storey Total Deflection)

Load Level (N)	Bottom Storey Deflection (mm)	Cumulative Effects Due to Bending (mm)	Cumulative Effects Due to Rigid Body Rotation (mm)	Expected Total Deflection Considering Cumulative Effects (mm)	Total Model Deflection (mm)	Ratio of Calculated to Model
1000	0.80	0.01	0.23	1.04	1.05	0.99
2000	1.60	0.03	0.37	2.00	2.00	1.00
4000	3.60	0.07	0.75	4.42	4.41	1.00
6000	9.20	0.10	1.84	11.14	11.16	1.00
8000	22.39	0.13	6.26	28.78	29.01	0.99

The results clearly show that the assumption of cumulative effects due to rigid body rotation is valid for stacked shearwalls with no consideration for the floor diaphragm. This is consistent with the finding reported in Chapter 5 (Table 5.4), where data from the experimental study was used. The small error obtained when using the model is expected since the input information in the model and the design expression are exactly the same, whereas these values could differ slightly during the experimental testing phase.

For the case where the load was only applied at the top of the wall (wall SW-7B), two components are expected to cause cumulative effects, namely: moment at bottom storey due to bending of the top storey end stud and rigid body rotation of bottom storey. Table 6.8 presents the analysis for wall SW-7B.

Table 6.8: Expected Deflection Considering Cumulative Effects vs. Observed Deflection (SW-7B-Top Storey Total Deflection)

Load Level (N)	Bottom Storey Deflection (mm)	Cumulative Effects Due to Bending (mm)	Cumulative Effects Due to Rigid Body Rotation (mm)	Expected Total Deflection Considering Cumulative Effects (mm)	Total Model Deflection (mm)	Ratio of Calculated to Model
1000	1.35	0.05	0.43	2.75	2.80	0.98
2000	2.40	0.10	0.84	4.90	4.90	1.00
4000	5.69	0.20	1.84	11.58	11.90	0.97
6000	13.65	0.30	4.26	27.60	30.00	0.92

Although for higher loads the results differ slightly, in general, the ratio of estimated deflection from the design expression to that obtained from the model seems to indicate that the expressions for cumulative effect are valid in two-storey walls where the main contributor to the accumulated deflection is the rigid body rotation. Since the bending contribution was observed to be almost insignificant in the case of two-storey building, an attempt to highlight this effect was made by considering model for six-storey building (Figure 6.9).



Figure 6.9: 3-D View of 6-Storey Shear Wall Assembly

The wall had an aspect ratio of 1:1 and same construction characteristics as single storey wall modeled in this study. The level of applied load is not of importance in this case, since the relative behaviour is of interest, thus a 1 kN lateral load was applied on each storey level. The expected cumulative effect was calculated based on the deflection of the level below and compared with the total deflection obtained from the model. Figure 6.10 shows the deflected shape of the wall obtained from the numerical model and Table 6.9 compares the expected deflection considering cumulative effects to the total deflection from the model.

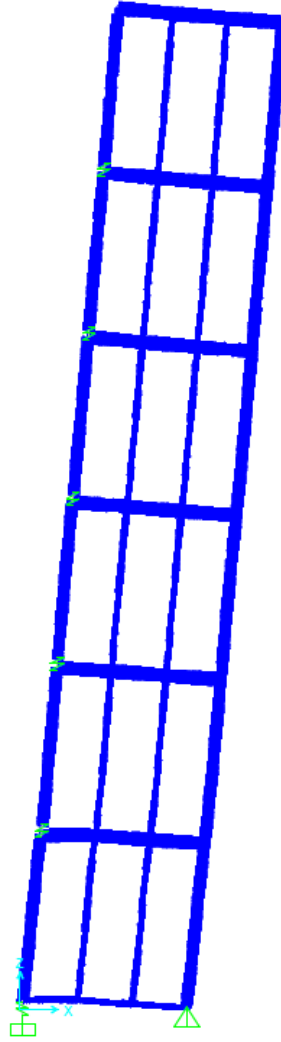


Figure 6.10: Deflected Shape of 6-Storey Walls

Table 6.9: Expected Deflection Considering Cumulative Effects vs. Model Deflection

Storey Level (N)	Bottom Storey Total Deflection (mm)	Cumulative Effects Due to Bending (mm)	Cumulative Effects Due to Rigid Body Rotation (mm)	Expected Total Deflection Considering Cumulative Effects (mm)	Total Model Deflection (mm)	Ratio of Calculated to Model
1	-	-	-	12.97	12.97	1.00
2	12.97	0.59	5.03	26.81	23.98	1.11
3	26.81	0.40	7.32	39.00	34.83	1.12
4	39.00	0.25	8.35	50.03	45.79	1.09
5	50.03	0.14	8.80	60.31	56.5	1.07
6	60.31	0.06	8.94	69.89	67.04	1.04

The results of analyzing the 6-storey model shows a good agreement between the expected deflection and observed deflection in the model, with the design expression having the tendency to slightly over-estimate the deflection especially at higher storey levels (Figure 6.11).

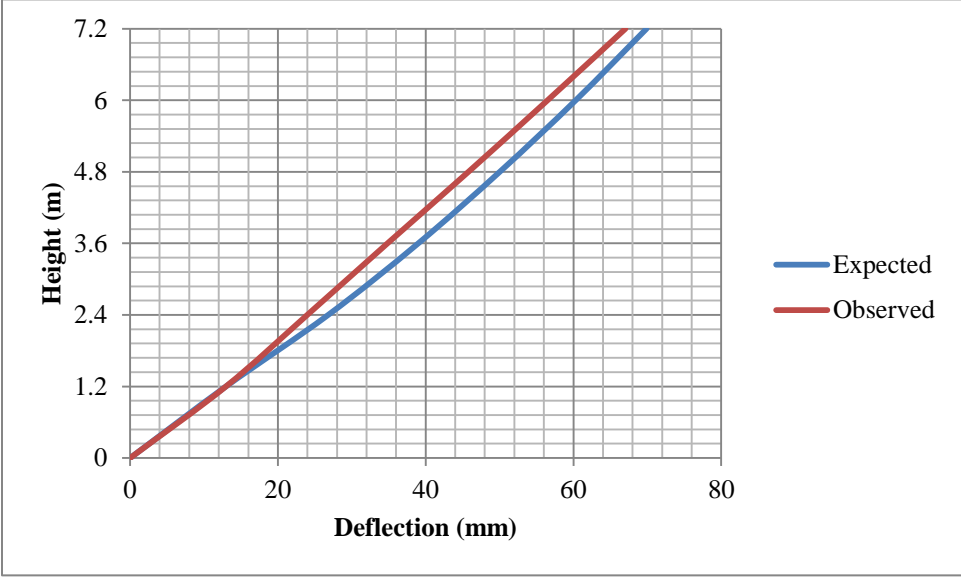


Figure 6.11: Expected Deflection at Storey Levels Considering Cumulative effects vs. Observed Deflection

The discussion above considered the shearwalls as continuous stacked walls with no regards for the floor diaphragm that exist in platform light frame wood buildings. The in-plane rigidity of light frame diaphragms has been investigated (e.g. Filiatrault 2002, Pathak 2008, Al Harash et. al 2010, Huang 2013) due to the important of this subsystem regarding lateral force distribution between the lateral force resisting system (LFRS). Where research has lacked is in evaluating the impact of out of plane rigidity of the diaphragm on the deflection of shearwalls. The out of plane rigidity of the diaphragm is assumed not to contribute to the wall deflection in the deflection expression found in the wood engineering standard (CSA O86) as well as in the two-storey model developed in this study to mimic the experimental setup. The commentary to the wood

design standard state that “the mechanics based approach is intended to provide a conservative estimate of lateral deflections and inter-storey drifts. There are other factors such as out of plane diaphragm stiffness, that could potentially reduce building deflections but its effect is not currently well understood and therefore not addressed in the standard (CWC 2017). Although over-estimating the deflection is conservative from the point of view of evaluating the building’s inter-storey drift, when the expression in the standard is used to evaluate the wall stiffness for the purpose of calculating the building period, the implication of over-predicting the deflection has non-conservative consequences, where lower stiffness would lead to lower base-shear forces on the building.

The effect of the diaphragm’s out-of-plane rigidity on the behaviour of the shearwalls is simulated by modeling the floors as beam. The stiffness of the beam is changed between zero and “infinity” and the effect on the walls is recorded. The beams are connected at each end to columns through a connection that allows full rotation (pin-ended). Two 2”x6” stud elements were used to simulate the boundary conditions of diaphragm joists bearing on an end wall. Figure 6.12 shows the 6-storey shearwall model with simulated floor diaphragm.

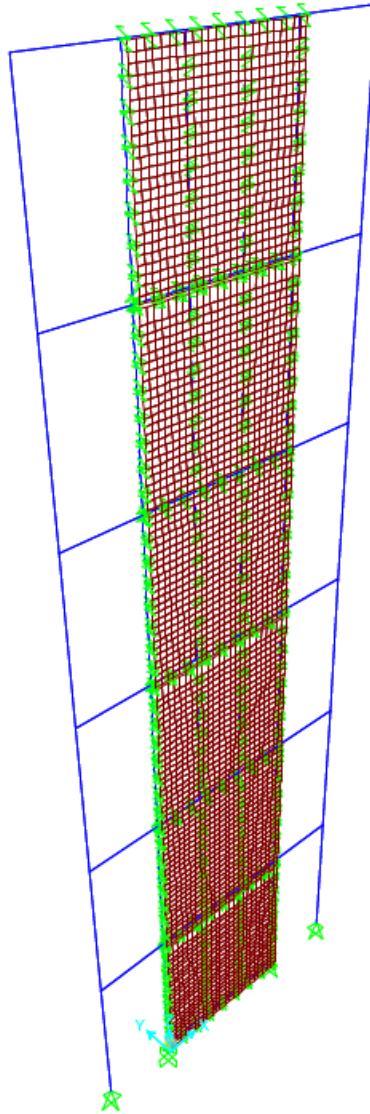


Figure 6.12: 3-D view of 6 Storey Wall with the Beam at Each Storey Level Representing the Diaphragm

The approach employed here would seem reasonable, as long as the beam stiffness is known. However, obtaining realistic values for the floor stiffness is not feasible due to the complexity of the system and variability of possible combinations. For this reason, idealized conditions (infinitely stiff or flexible) were considered. If the diaphragm is infinitely flexible out of plane, it can freely transfer the cumulative effects (particularly rigid body rotation) to upper floors. In

contrast, infinitely rigid out of plane diaphragm represent the situation where cumulative effects cannot be transferred to the top floors.

As mentioned earlier, estimating the out of plane stiffness of the floor diaphragm with any level of accuracy is not possible. In an attempt to provide an estimate for this value, a situation is considered, where an exterior shearwall supports floors joists bearing perpendicular to the wall. The shear transfer from the upper storey is assumed to occur through and rim joist consisting of a multi-ply build-up LVL beam. The specifications for the LVL beam are obtained from the technical guide report (NRC 2015), where SPF grade with an MOE of 15000 N/mm^2 is used. Typically a two-ply beam is used at the end of the joisted floor, however, in this study the effect of one, two and three joists is investigated. It should be noted that in reality, the floor system is expected to have a significantly higher stiffness than that provided immediately above the shearwalls, and as such the author believe that what is presented here could possibly be considered as a lower bound of what the floor contributes to the shearwall system.

Five cases were developed in the current study, as shown in Table 6.10, by varying the out of plane stiffness of the floor diaphragm, representing the two idealized conditions (flexible and rigid), in addition to three “realistic” cases involving rim joists. In all cases the end columns are assumed to consist of two 2”x6”.

Table 6.10: Different Diaphragm Conditions

Case No	Beam out of Plane Stiffness
1	0
2	Infinite
3	3 built-up 38” x 305” LVL
4	2 built-up 38” x 305” LVL
5	Single 38” x 305” LVL

It should be noted that case 1 in Table 6.10 represents the model with no diaphragm, which was used to compare with the experimental results and design deflection expression concerning the cumulative effects.

Table 6.11 shows the normalized deflection of the 6 storey walls for the cases presented in Table 6.10.

Table 6.11: Normalized Deflection of Each Storey for Different Cases

Storey Level	Case1 Deflection	Case2 Deflection	Case3 Deflection	Case4 Deflection	Case5 Deflection
1	0.19	0.11	0.11	0.11	0.11
2	0.36	0.15	0.15	0.16	0.16
3	0.52	0.18	0.19	0.19	0.20
4	0.68	0.20	0.21	0.22	0.23
5	0.84	0.22	0.23	0.24	0.25
6	1.00	0.23	0.24	0.25	0.27

The results clearly highlight the importance of the floor out of plane rigidity on the behaviour of the shearwalls. Comparing the deflection results considering rigid diaphragm (case 2) to the flexible diaphragm (case 1) highlights that the rigid diaphragm reduces the deflection by 76.85% on average. Simply assuming three 38”x305” built-up beam as diaphragm resulted in deflections that are very close to the case of rigid diaphragm. The table also shows that decreasing the size of the beam from three to two- or one ply 38”x305” has no significant effect on the results. It can therefore be concluded that considering even conservative estimates of out of plane stiffness would lead to a very significant reduction in deflection and that assuming the floor diaphragm to be infinitely rigid out of plan seems reasonable. This is demonstrated graphically in Figure 6.13.

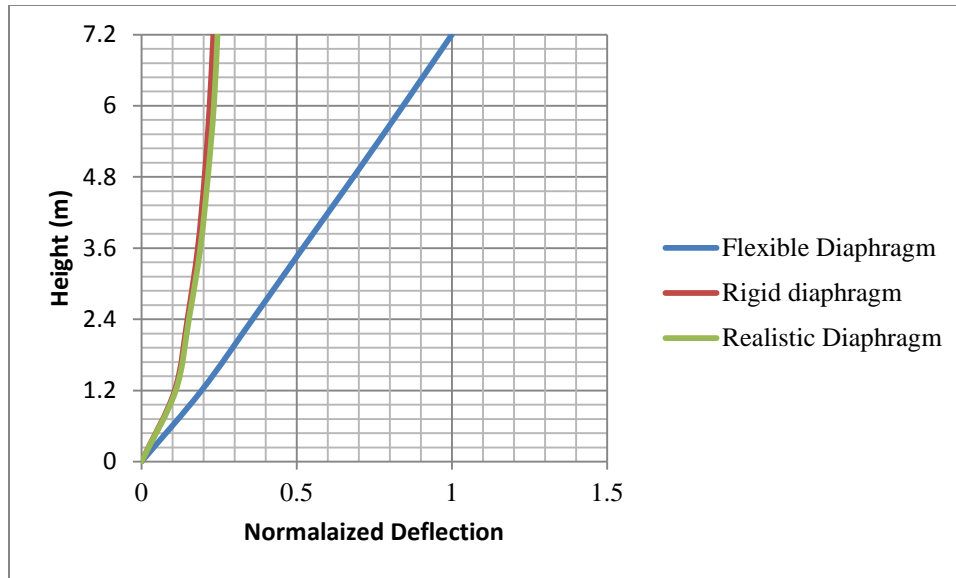


Figure 6.13: Impact of Diaphragm Assumption on Deflection of the Walls

It is rare that a single shearwall panel is used in a shear line. Typically a shearline would consist of multiple panels with similar configurations and construction details. The in-plane stiffness of the diaphragm usually ensures that the deflections in the panels is the same. Since it was observed in the single panel example earlier that the floor diaphragm has to bend in order to accommodate the rigid body rotation of the wall, having multiple panels would impose more demand on the floor diaphragm. This issue is investigated next for two and three shearwalls in the same shear line. The wall configurations are similar to the case for single panel with 3-ply LVL beam. A unit load is applied on each wall panel to obtain comparable behaviour to that of a single wall case. Figure 6.14 illustrates an example of a shear line with three shearwall panels and Table 6.12 presents the cases investigated in this study.

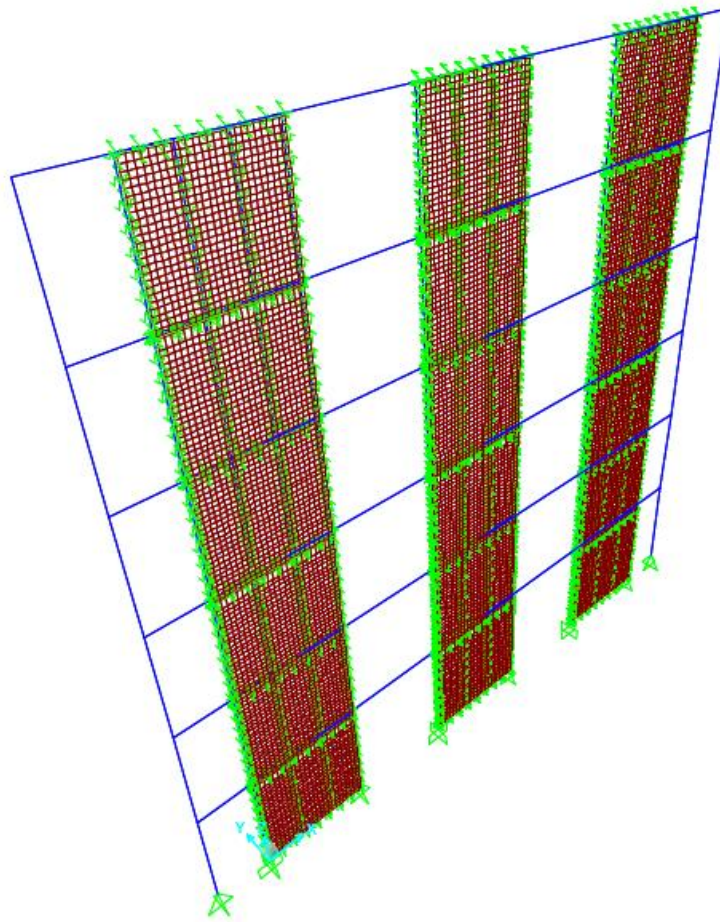


Figure 6.14: 3-D view of 6 Storey Multiple Walls with the Beam at Each Storey Level Representing the Diaphragm

Table 6.12: Multiple Shear Walls Cases in a Shear Line

Case	Number of Panels	Beam out of Plane Stiffness
A	2	3 built-up LVL joists
B	3	3 built-up LVL joists

The results, normalized to the deflection obtained at the 6-storey level for a two-panel shear line, are presented in Table 6.13.

Table 6.13: Deflection of walls with Multiple Panels

Storey Level	Case A Deflection	Case B Deflection	Ratio of Case B to Case A
1	0.31	0.27	0.87
2	0.53	0.46	0.88
3	0.70	0.62	0.88
4	0.84	0.73	0.87
5	0.94	0.82	0.87
6	1.00	0.88	0.88

The results clearly show that having multiple walls would further reduce the deflection of the shearwalls because the diaphragm has to accommodate the deformed shape of all wall segments in the shearline.

The above findings are limited to numerical modeling and should as such be considered preliminary until verified by experimental testing. The finding can potentially have a very significant impact on the way a designer can approach deflection of light frame wood shearwalls. The general approach has been that assuming light frame wood shearwalls behave as cantilevers, and assuming no out of plane stiffness in the floor diaphragm, then a mechanics-based approach describing the wall deflection, similar to that found in the wood design standard (CSA 2014) can be used. The current study does not argue against the mathematical formulations past the adoption of the mechanical model but rather puts into question the validity of the fundamental assumptions themselves. It is proposed that assumption related to the cumulative effects are questionable and may lead to non-conservative designs. More work is required to provide better assumptions and estimates of multi-storey shearwall deflections. The results from the current study seem to point in the direction of ignoring the cumulative effects for the purpose of base shear calculations. These effects may be considered for drift calculations although it is believed that this would be too conservative. Estimates of out of plane floor diaphragm stiffness would be

useful to investigate experimentally as well, although simple modeling approach from the current study seems to indicate that almost any diaphragm would have sufficient out of plane stiffness to be considered fully rigid.

7 SUMMARY, CONCLUSIONS AND FUTURE RECOMMENDATIONS

The behavior of wood shear walls was investigated experimentally and numerically in order to determine the effects of construction details' variation on the behavior of the shear walls and evaluate whether the current deflection equation can adequately predict the overall wall stiffness. Full-scale single-storey and half scale two-storey walls, with different construction details and aspect ratios, were investigated. The conclusions that can be drawn from the study are:

1. The deflected shape of the end stud was found to be non-linear with regard to the loading level. Using the cantilever assumption seems to be too conservative in estimating the end stud bending contribution especially at higher load levels. The bending component seems affected by the rotation of the sheathing panels and nail slip. This interaction raises a question about the validity of the four terms in the deflection equations being independent from each other.
2. The results from the full scale experimental testing program showed no panel deformation of any significance.
3. The deflection expression in the wood design standard has a tendency to under-estimate the nail slip with the difference increasing for higher load levels.
4. The results showed that the hold-down system used in this study behaves in a non-linear manner with a significantly higher initial stiffness that assumed in design. This observation was limited to discrete hold-down, where significant deformation was observed in the hold-down system.
5. The strength and stiffness correlated almost directly to the inverse of the wall aspect ratio. There did not seem to be a clear trend when considering the effect of walls' aspect ratios on ductility. Surprisingly, walls with aspect ratios not permitted according to the

wood design standard (4:1 and 6:1) followed similar strength and stiffness trends and had sufficient ductility ratios as those with lower aspect ratios. This observation explains in part some of the discrepancies found between engineering calculations and behavior of actual building with light frame wood shear walls.

6. Significant discrepancies were found when comparing the various deflection constituent with those estimated using the design expression. This was especially the case for the bending and panel shear components. In both cases however the deflection was very small in magnitude.
7. Adding more end studs and changing the size of the studs had no significant effect on the overall wall capacity and little effect on its stiffness. Reducing the stud spacing had, as expected, no effect on the wall capacity, however, the results showed that the deflection was affected by the overall number of studs in the wall and not solely by the end studs.
8. Shearwalls sheathed with plywood panels exhibits slightly higher peak load and initial stiffness than those with OSB. This increase was mainly attributed to the higher panel thickness and possibly density in the plywood sheathing panel. The two walls showed similar levels of ductility, as anticipated. Thicker sheathing increased the capacity and stiffness of the wall with no significant change observed in ductility ratio.
9. The wall strength was significantly affected by the nail diameter and nail spacing. However, doubling the number of nails only led to 58% increase in the peak load and reducing the nail spacing showed a more brittle failure.
10. Little difference was observed when the nail edge/end distance was increased from the minimum design requirements of 9.5 mm to that of 15 mm. The wall with the reduced end/edge distance showed failure mechanism characterized by nails pulling through the

edge of the OSB panel, however, this shift in failure mode occurred towards the end of the testing, and therefore it had little to no effect on the strength, stiffness and ductility. This finding corroborates the use of 9.5 mm as the minimum edge/end distance for light frame wood shear walls.

11. The study shows that having continuous hold-down connections has a positive effect on the capacity, stiffness and ductility of the wall when compared with discrete hold-downs. Having no hold-down adversely affected the wall capacity and stiffness. The wall with no hold-down had almost one third of the capacity of the wall with continuous hold-down, which was lower than that predicted by the wood design standard. The reduction in stiffness for the wall with no hold-down was consistent with the one obtained using the wood design standard. A slight increase in ductility when using continuous hold-down was observed, which could be attributed to shifting more displacement demand on the panel to framing connections. Not having a hold-down meant that the force was more evenly distributed to the panel to framing nails, which also provide ductility.
12. For the two-storey walls, the deflected shape showed similar trends to those found for the single-storey, where the behaviour of the individual components generally did not follow the corresponding expressions in the deflection expression, especially for end stud bending and hold-down deformation. The deflection estimated based on the cumulative effect assumption showed slight differences when compared with that observed in the experimental study. It was observed that the majority of the cumulative effect stems from the rigid body rotation due to deformation in the hold-down devices.
13. A shear wall model was developed and the results from the model were compared with those obtained experimentally. It was found that the model was capable of predicting the

peak load, ultimate deflection and yield loads with reasonable accuracy, but overestimated the initial stiffness and ductility of the walls. In general when the force displacement curves were compared it was evident that the model was capable of predicting the wall behaviour with reasonable accuracy.

14. When investigating the cumulative effects using the model, the results clearly showed that the assumption of cumulative effects due to rigid body rotation is valid for stacked shearwalls with no consideration for the floor diaphragm's out of plane stiffness. This results was corroborated when a simulated six-storey wall was investigated, with the design expression having the tendency to slightly over-estimate the deflection especially at higher storey levels.

15. The results of investigating the effect of the diaphragm's out-of-plane rigidity showed that a reduction in the shearwall deflection in the magnitude of approximately 80% was observed when the deflection results considering rigid diaphragm compared to the flexible diaphragm. The results also show that simply assuming a built-up beam consisting of one or multiple rim joists resulted in deflection values that were very close to the case of rigid diaphragm. It can therefore be concluded that considering even conservative estimates of out of plane stiffness would lead to a very significant reduction in deflection and that assuming the floor diaphragm to be infinitely rigid out of plane

16.

17. seems reasonable. Diaphragm supported on multiple panels was found to further reduce the deflection of the shearwalls.

Based on the findings from the current study, the following future studies are recommended:

- The findings from the current study related to the multi-storey cumulative effects were limited in numbers and configurations. More two- or even three-storey testing of several configurations would provide validation as well as expand the knowledge base on the behaviour of multi-storey stacked wood shearwalls.
- The effect of diaphragm out of plane stiffness was investigated numerically and as such can only be considered preliminary until verified by experimental testing. Experimental tests representing two- and three dimensional effects would be of great interest. Configurations mimicking different floor behaviours would help in the understanding of the contribution of the floor to the wall stiffness and the overall behaviour of the building.
- The two-storey walls were loaded at each level separately in order to establish the cumulative effects with little interference from other variables. It is recommended that additional testing involving loads at all levels be undertaken. This will present more realistic loading effects and further validate the numerical approach presented in the current study.
- The experimental testing in the current study was limited to monotonic tests due to the fact that studies have found that loading protocol had no effect on the wall strength. Investigating whether the loading type has any effect on the stiffness of the load (stiffness degradation) would be recommended. The protocol, number of cycles and the duration of testing could be some of the factors that could be considered in such study.

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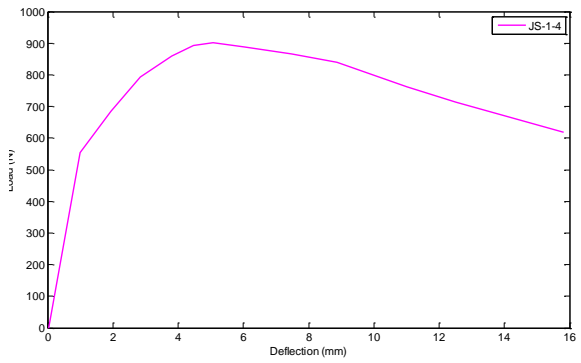
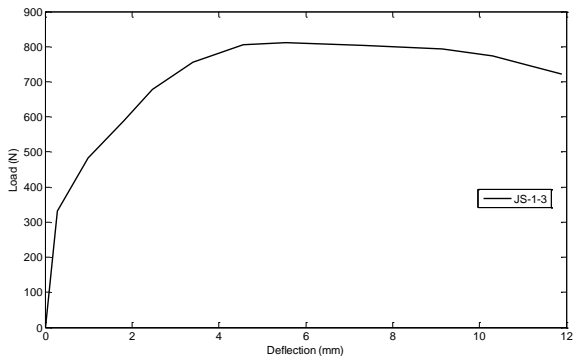
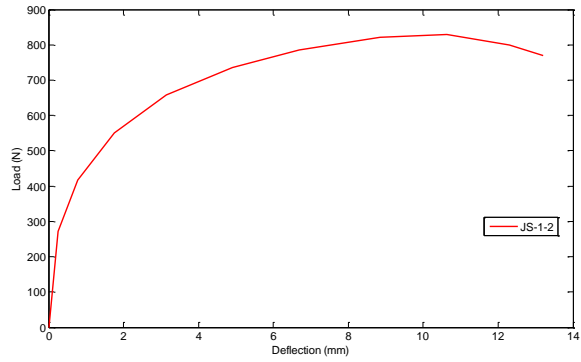
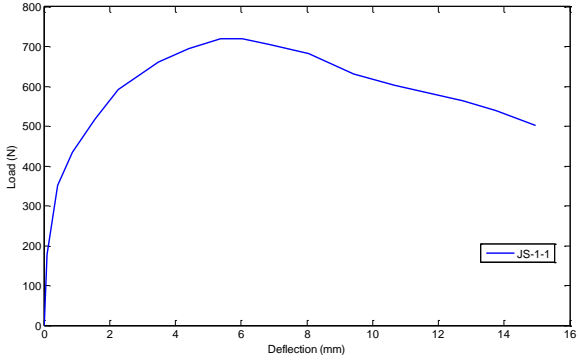
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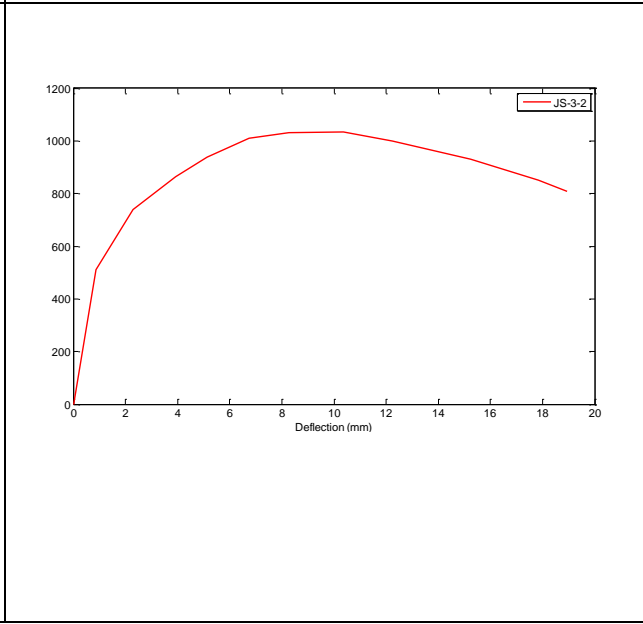
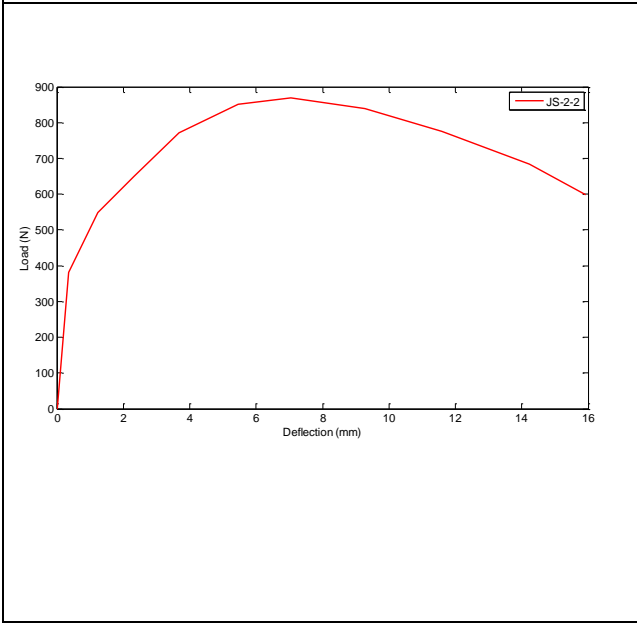
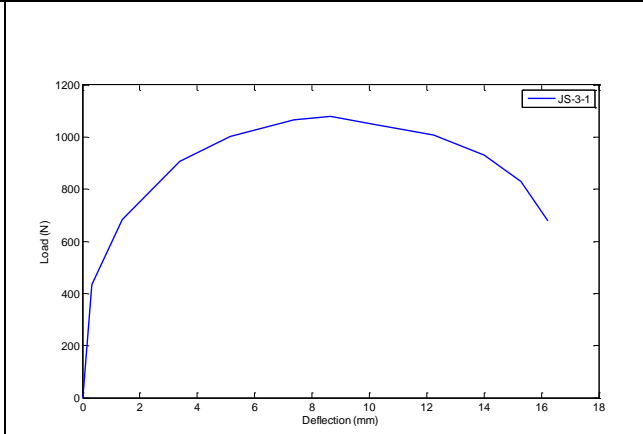
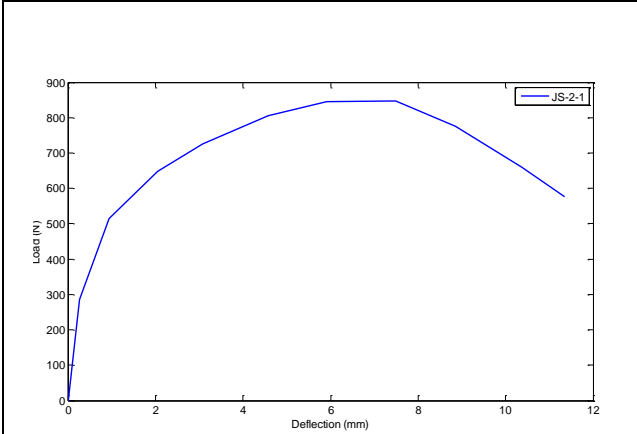
9 APPENDIX

9.1 Experimental Results of Nail Joint Component Test

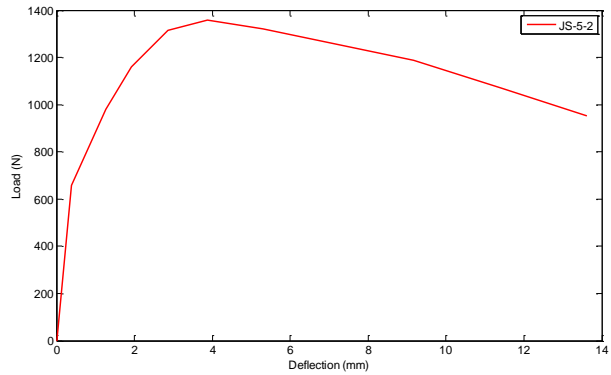
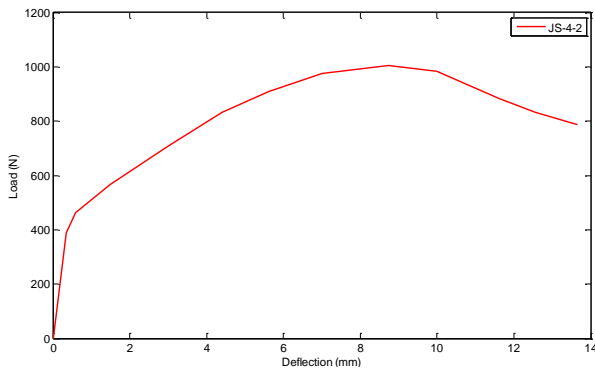
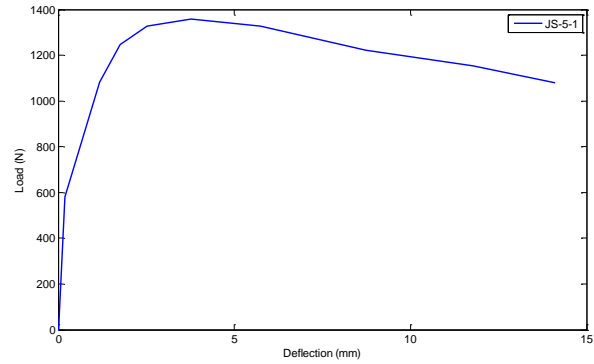
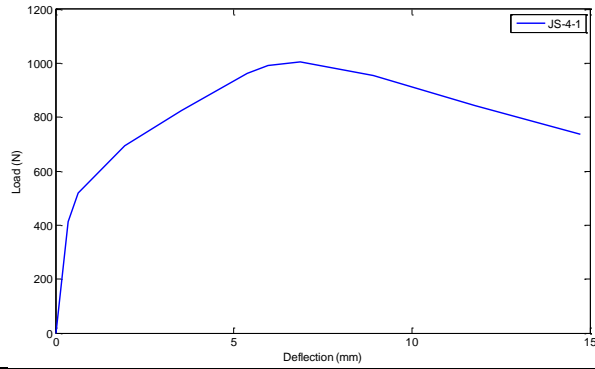
Group Number	JS-1
Sheathing Thickness (OSB)	11 mm
Nail Diameter	2.84 mm



Group Number	JS-2	Group Number	JS-3
Sheathing Thickness (OSB)	11 mm	Sheathing Thickness (OSB)	11 mm
Nail Diameter	3.32 mm	Nail Diameter	3.76 mm

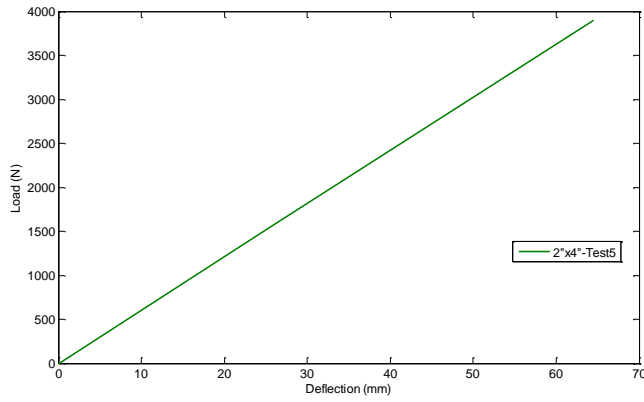
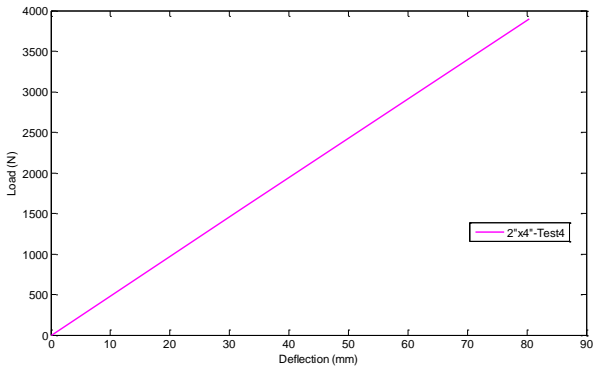
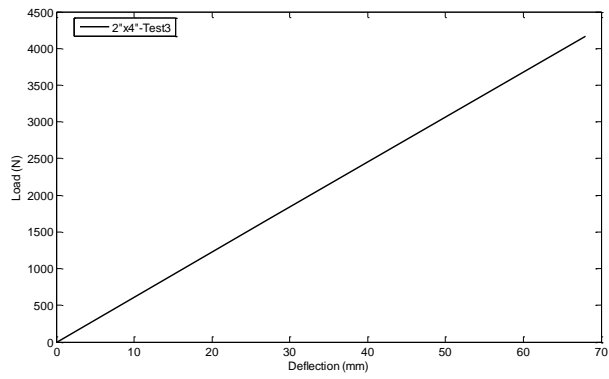
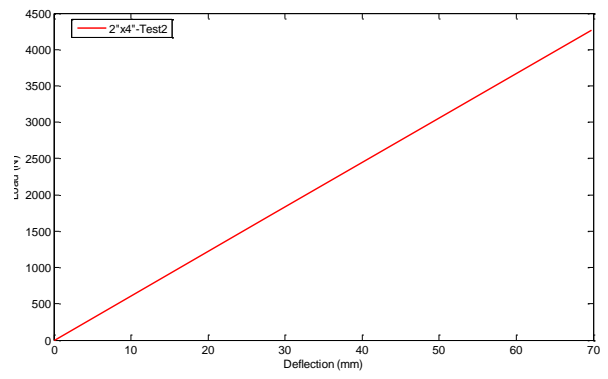
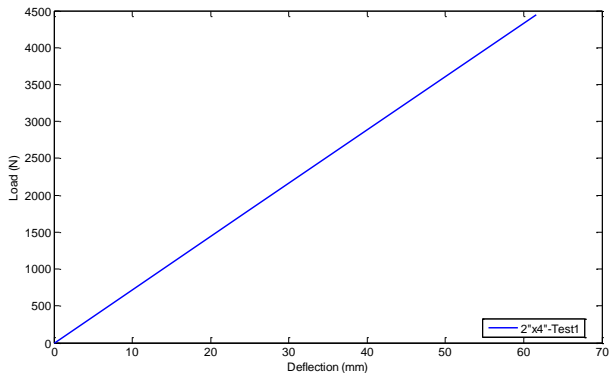


Group Number	JS-4	Group Number	JS-5
Sheathing Thickness (Plywood)	12.5 mm	Sheathing Thickness (OSB)	15.5 mm
Nail Diameter	2.84 mm	Nail Diameter	2.84 mm

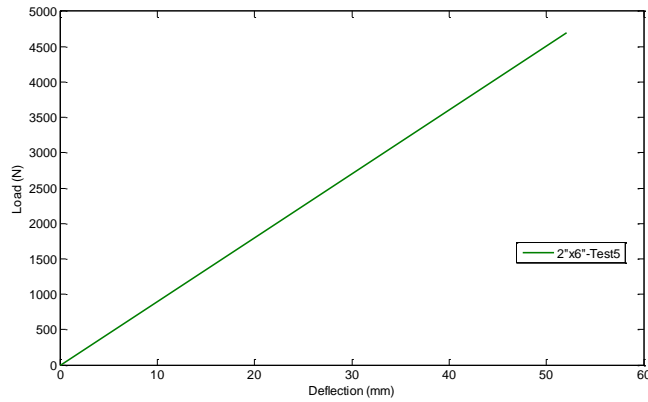
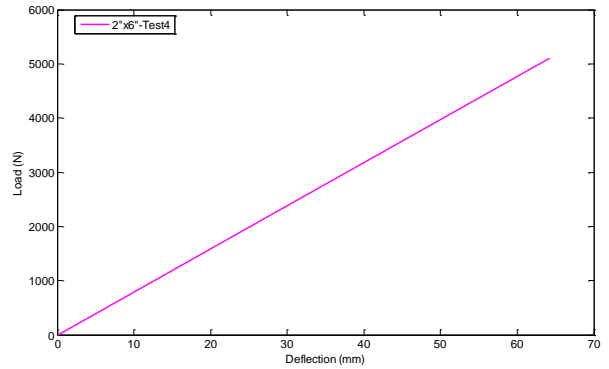
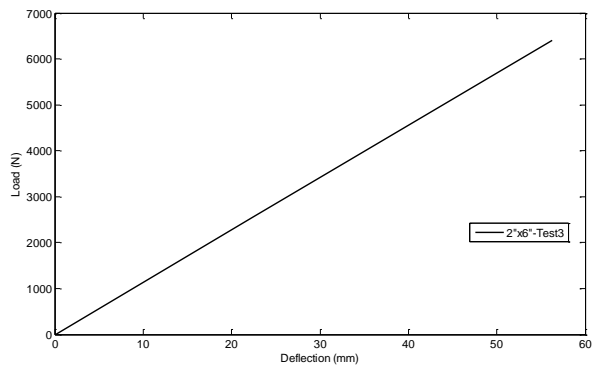
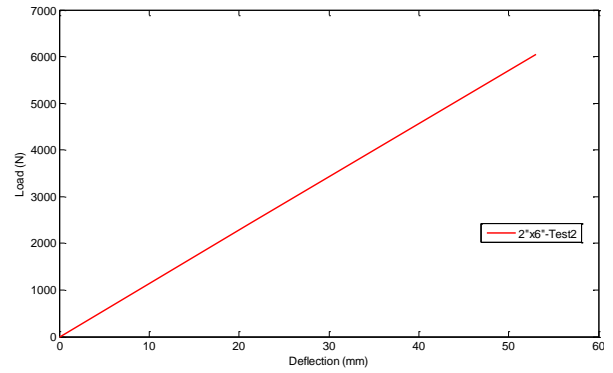
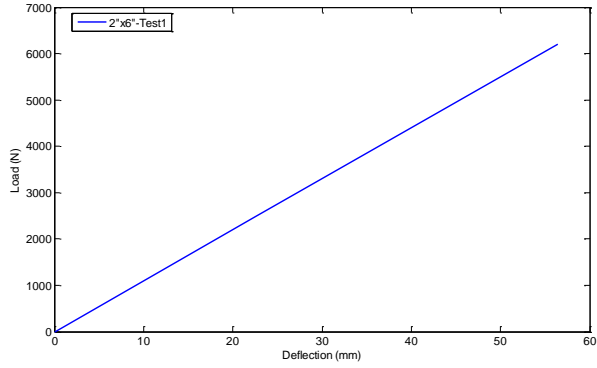


9.2 Stud Bending Component Test

Stud Size: 2"x4"



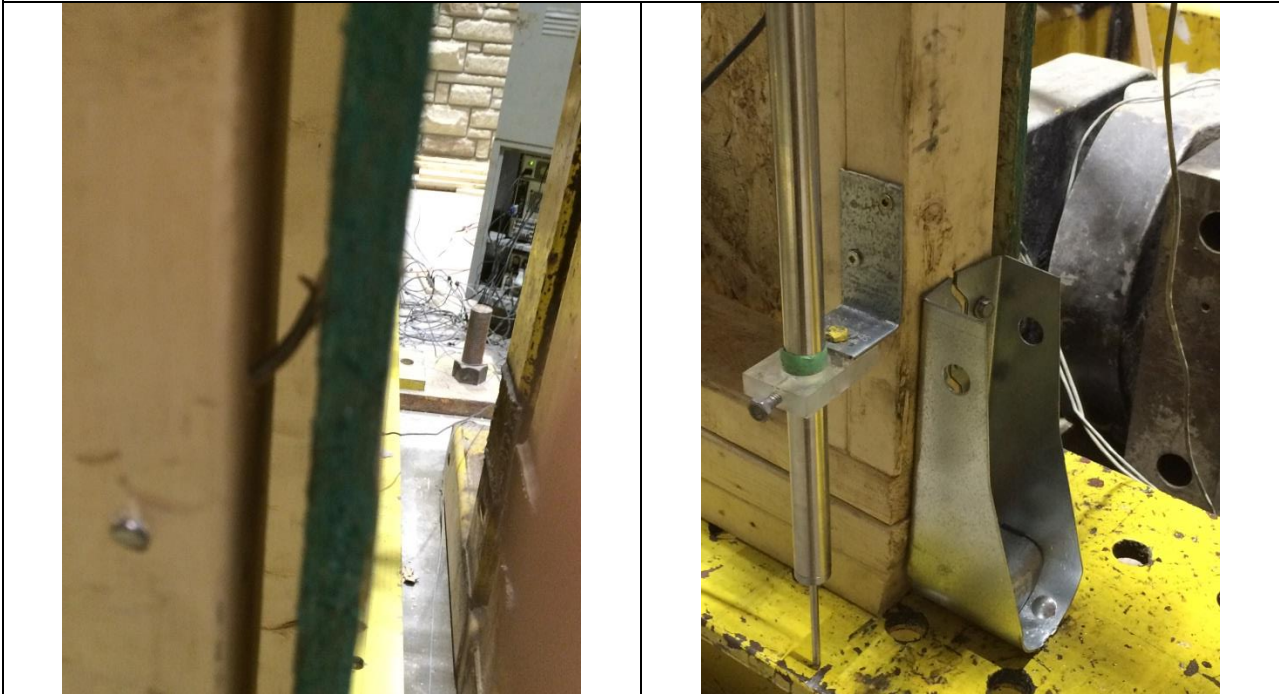
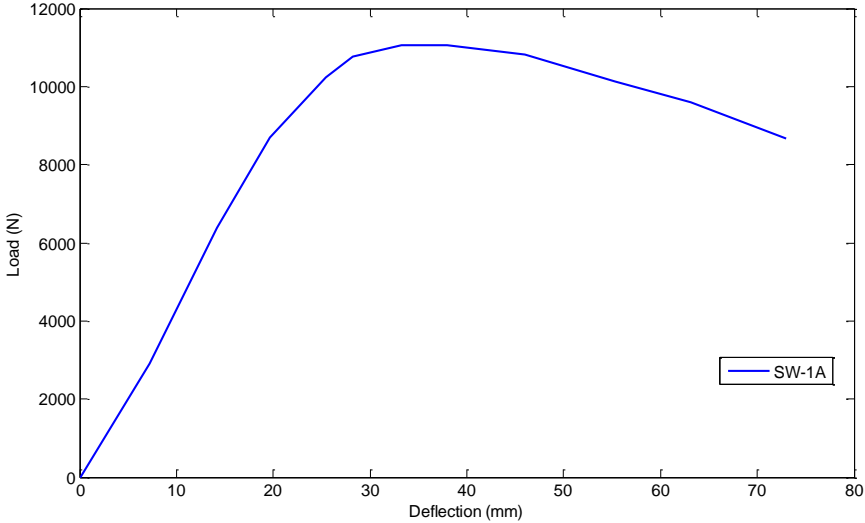
Stud Size: 2"x6"



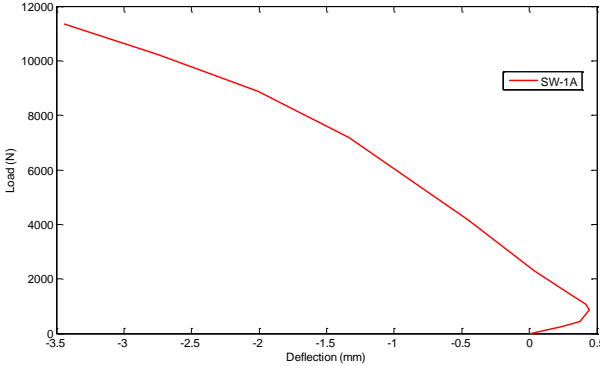
9.3 Experimental Results of Full-Scale Shear Wall Specimens

Specimen Number	SW-1A	EEEE Results	
Sheathing Thickness (OSB)	11mm	Peak Load (N)	11140.00
Nail Diameter	2.84 mm	Displacement at Peak (mm)	31.91
Stud Size	38 x89 mm	Initial Stiffness (N/mm)	360.00
Number of End Studs	2	Yield Load (N)	10230.00
Hold-Down Type	HDU2	Ductility Ratio	2.53
Aspect Ratio	1:1		

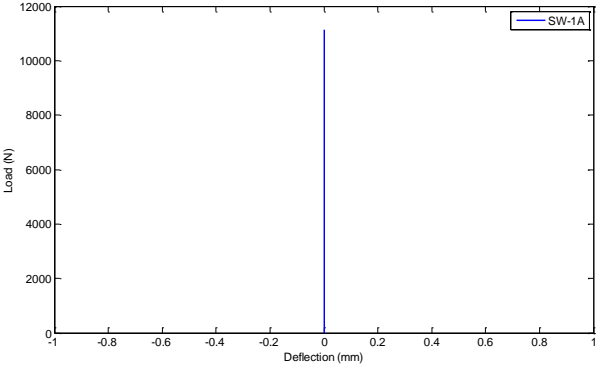
Total Behavior



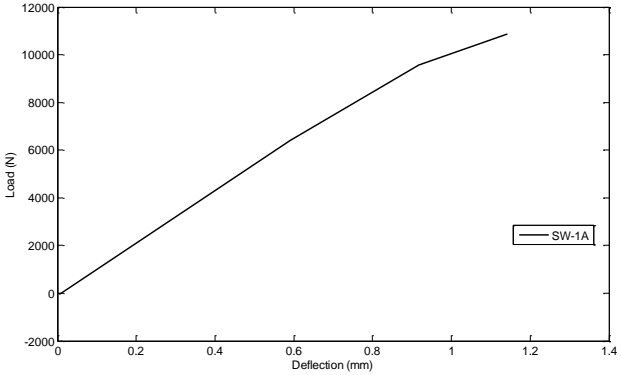
Specimen Number	SW-1A	Deflection Contributors	
Sheathing Thickness (OSB)	11 mm	a)	Bending
Nail Diameter	2.84 mm	b)	Shear
Stud Size	38 x 89 mm	c)	Hold-Down
Number of End Studs	2		
Hold-Down Type	HDU2		



a)



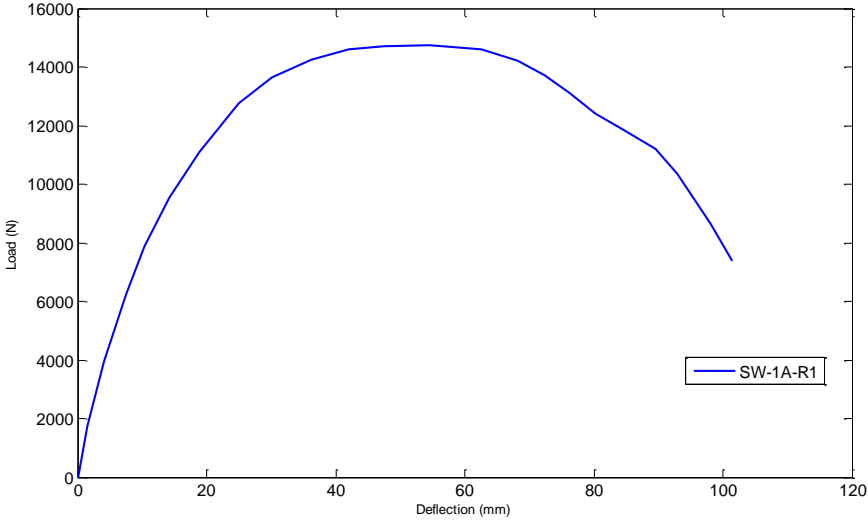
b)



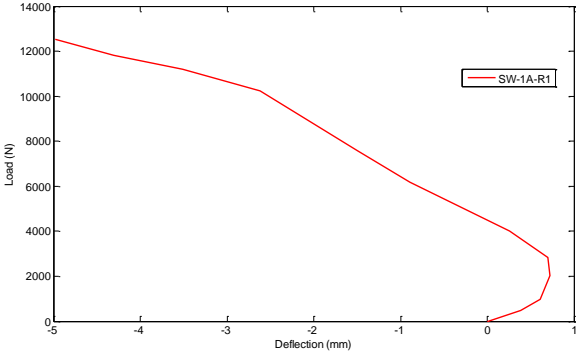
c)

Specimen Number	SW-1A-R1	EEEE Results	
Sheathing Thickness (OSB)	11mm	Peak Load (N)	14870.00
Nail Diameter	2.84 mm	Displacement at Peak (mm)	57.04
Stud Size	38 x89 mm	Initial Stiffness (N/mm)	800.00
Number of End Studs	2	Yield Load (N)	13381.33
Hold-Down Type	HDU2	Ductility Ratio	5.12
Aspect Ratio	1:1		

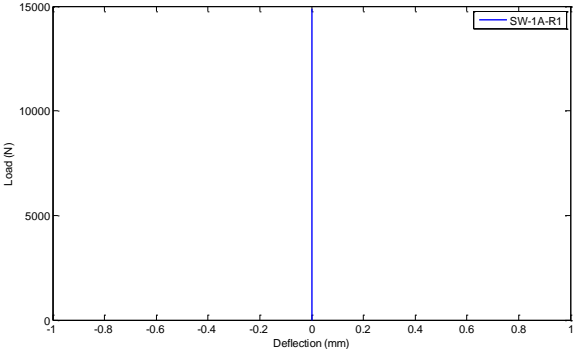
Total Behavior



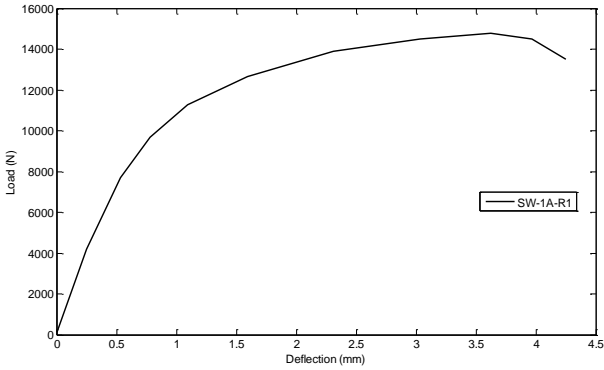
Specimen Number	SW-1A-R1	Deflection Contributors	
Sheathing Thickness (OSB)	11 mm	a)	Bending
Nail Diameter	2.84 mm	b)	Shear
Stud Size	38 x 89 mm	c)	Hold-Down
Number of End Studs	2		
Hold-Down Type	HDU2		



a)



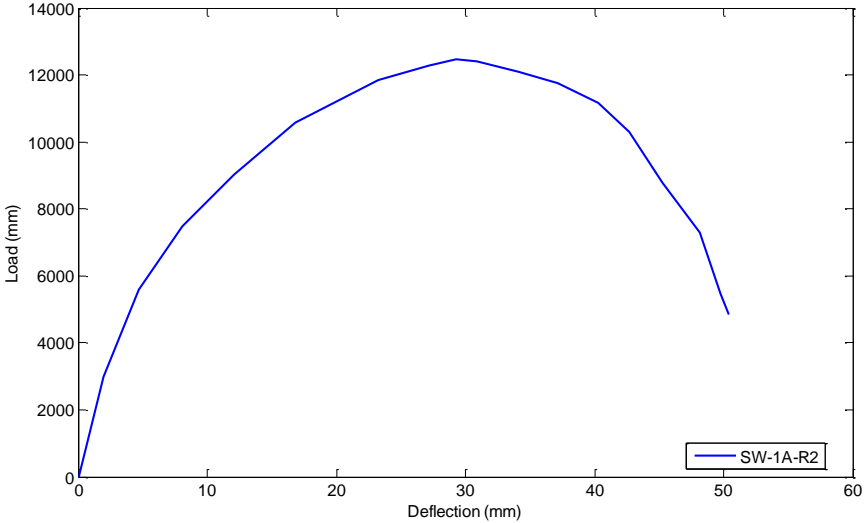
b)



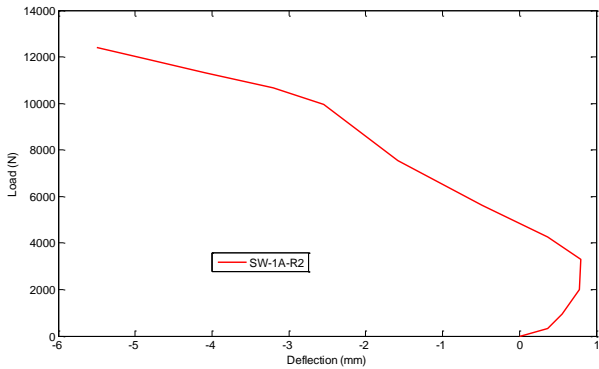
c)

Specimen Number	SW-1A-R2	EEEE Results	
Sheathing Thickness (OSB)	11mm	Peak Load (N)	12480.00
Nail Diameter	2.84 mm	Displacement at Peak (mm)	29.24
Stud Size	38 x89 mm	Initial Stiffness (N/mm)	1220.00
Number of End Studs	2	Yield Load (N)	11000
Hold-Down Type	HDU2	Ductility Ratio	4.66
Aspect Ratio	1:1		

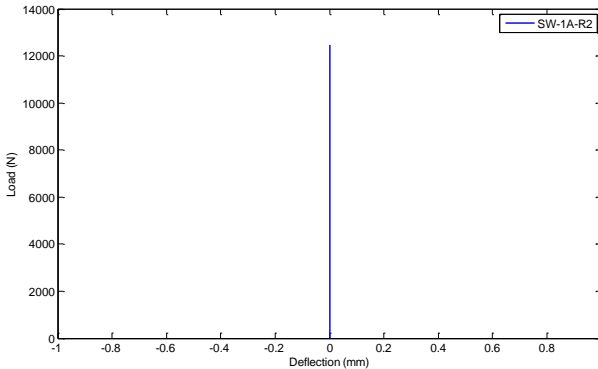
Total Behavior



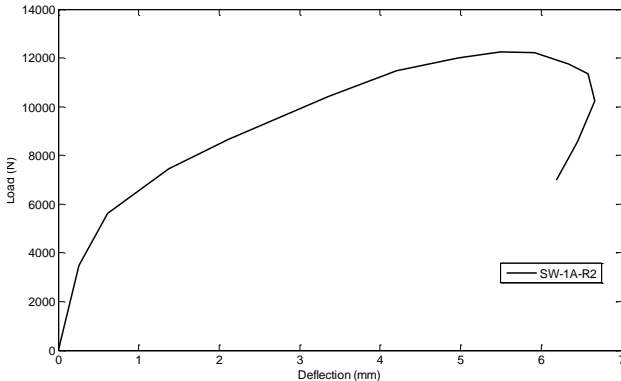
Specimen Number	SW-1A-R2	Deflection Contributors	
Sheathing Thickness (OSB)	11 mm	a)	Bending
Nail Diameter	2.84 mm	b)	Shear
Stud Size	38 x 89 mm	c)	Hold-Down
Number of End Studs	2		
Hold-Down Type	HDU2		



a)



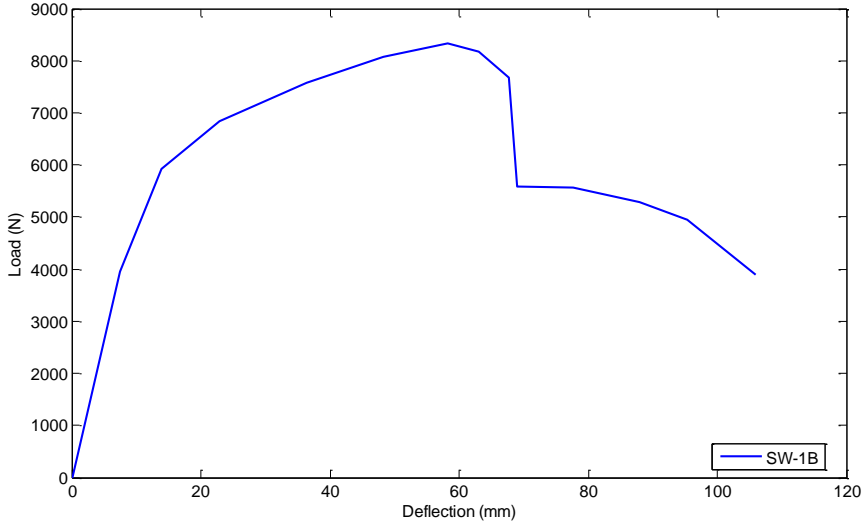
b)



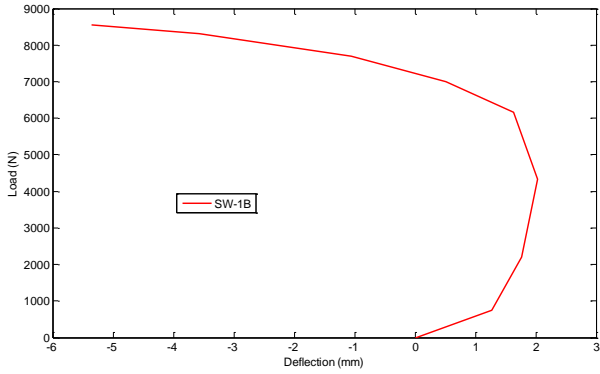
c)

Specimen Number	SW-1B	EEEE Results	
Sheathing Thickness (OSB)	11mm	Peak Load (N)	8340.00
Nail Diameter	2.84 mm	Displacement at Peak (mm)	53.55
Stud Size	38 x89 mm	Initial Stiffness (N/mm)	480.00
Number of End Studs	2	Yield Load (N)	7496.27
Hold-Down Type	HDU2	Ductility Ratio	4.37
Aspect Ratio	2:1		

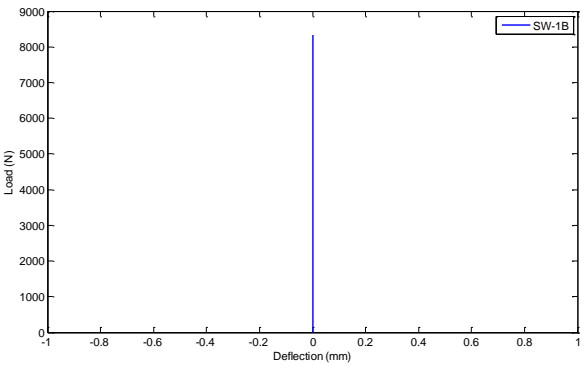
Total Behavior



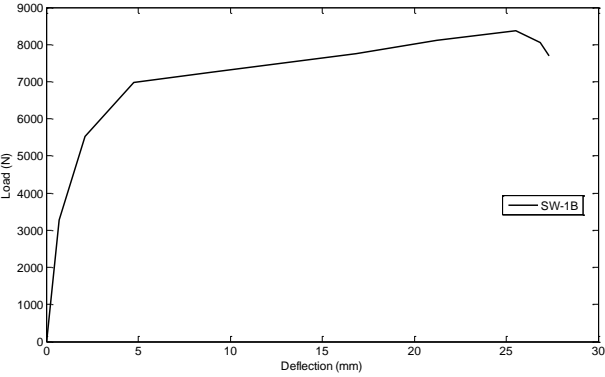
Specimen Number	SW-1B	Deflection Contributors	
Sheathing Thickness (OSB)	11 mm	a)	Bending
Nail Diameter	2.84 mm	b)	Shear
Stud Size	38 x 89 mm	c)	Hold-Down
Number of End Studs	2		
Hold-Down Type	HDU2		



a)



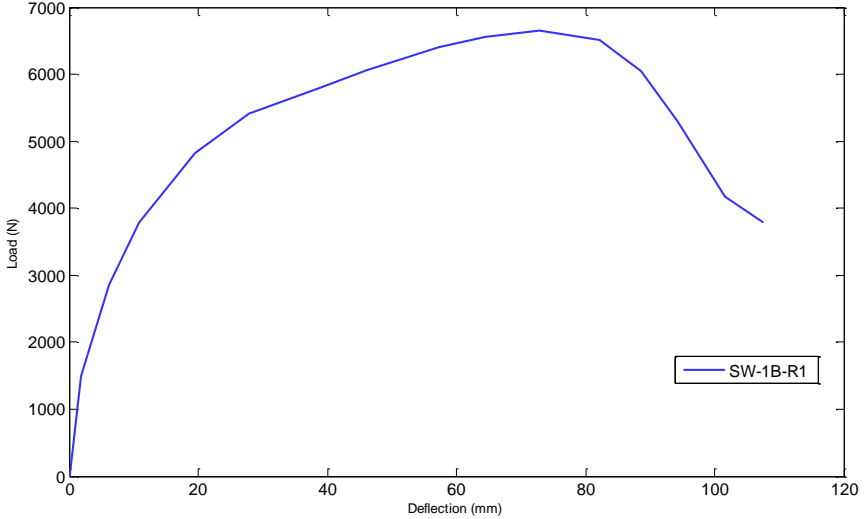
b)



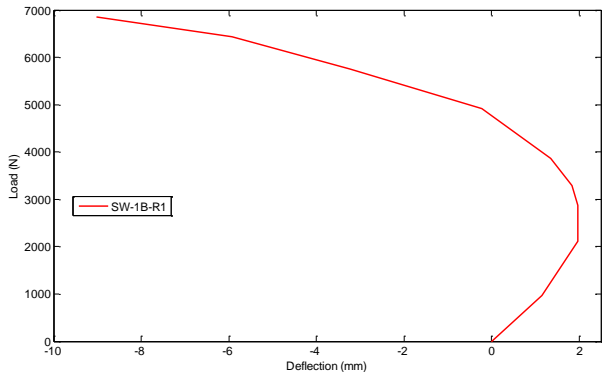
c)

Specimen Number	SW-1B-R1	EEEE Results
Sheathing Thickness (OSB)	11mm	Peak Load (N) 6760.00
Nail Diameter	2.84 mm	Displacement at Peak (mm) 66.98
Stud Size	38 x89 mm	Initial Stiffness (N/mm) 480.00
Number of End Studs	2	Yield Load (N) 5822.19
Hold-Down Type	HDU2	Ductility Ratio 7.60
Aspect Ratio	2:1	

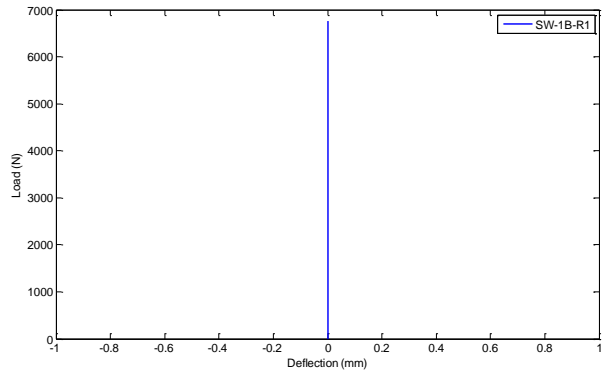
Total Behavior



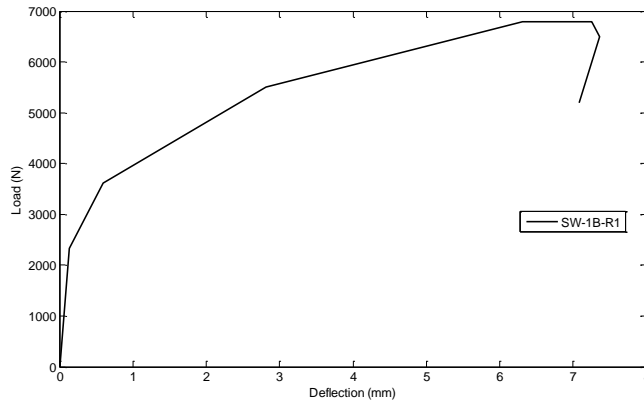
Specimen Number	SW-1B-R1	Deflection Contributors	
Sheathing Thickness (OSB)	11 mm	a)	Bending
Nail Diameter	2.84 mm	b)	Shear
Stud Size	38 x 89 mm	c)	Hold-Down
Number of End Studs	2		
Hold-Down Type	HDU2		



a)



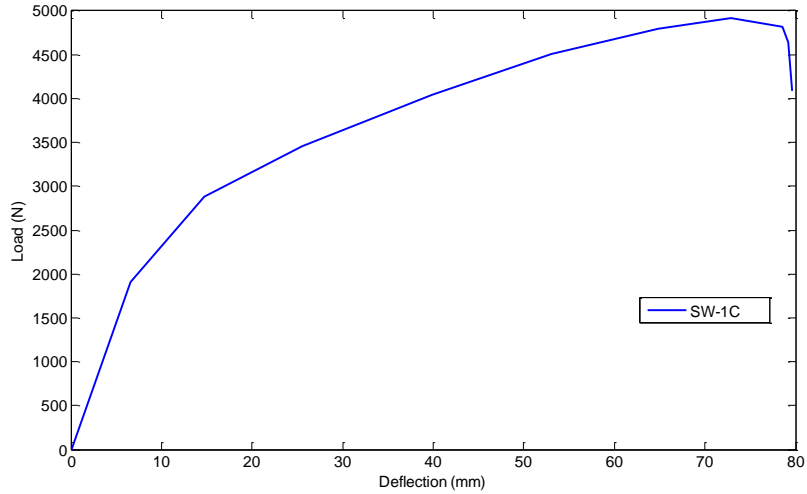
b)



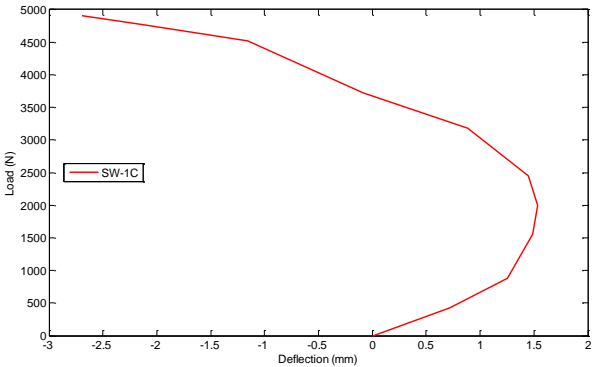
c)

Specimen Number	SW-1C	EEEE Results	
Sheathing Thickness (OSB)	11mm	Peak Load (N)	4900.00
Nail Diameter	2.84 mm	Displacement at Peak (mm)	75.17
Stud Size	38 x89 mm	Initial Stiffness (N/mm)	270.00
Number of End Studs	2	Yield Load (N)	4057.41
Hold-Down Type	HDU2	Ductility Ratio	5.31
Aspect Ratio	3:1		

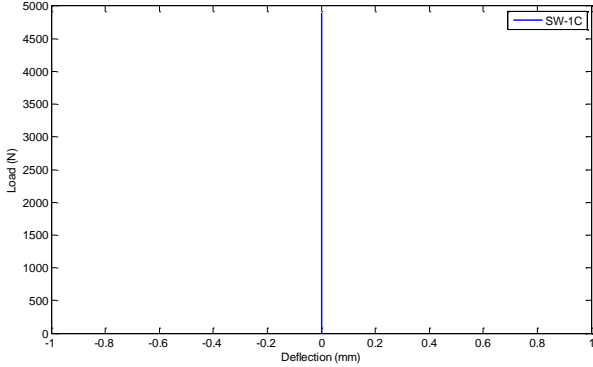
Total Behavior



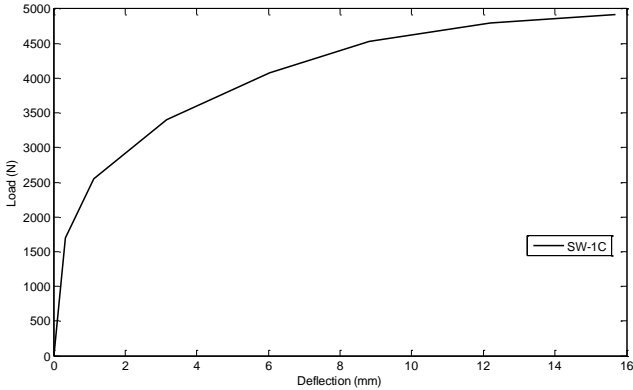
Specimen Number	SW-1C	Deflection Contributors	
Sheathing Thickness (OSB)	11 mm	a)	Bending
Nail Diameter	2.84 mm	b)	Shear
Stud Size	38 x 89 mm	c)	Hold-Down
Number of End Studs	2		
Hold-Down Type	HDU2		



a)



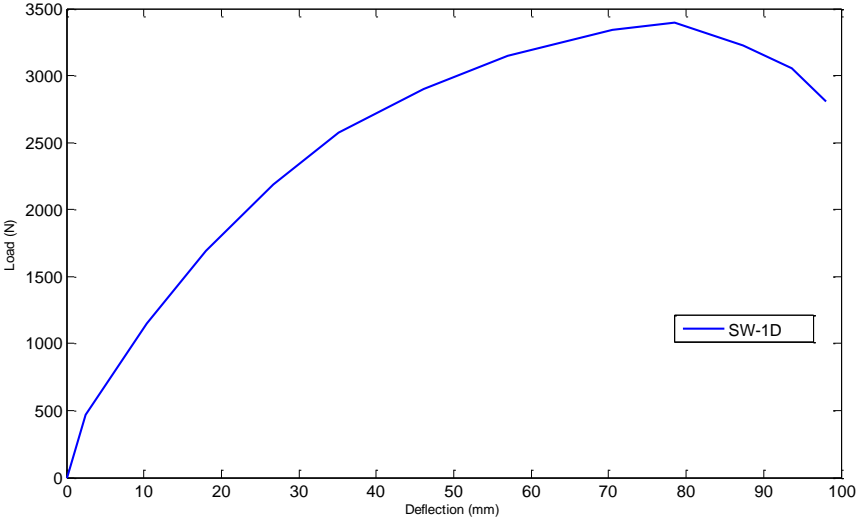
b)



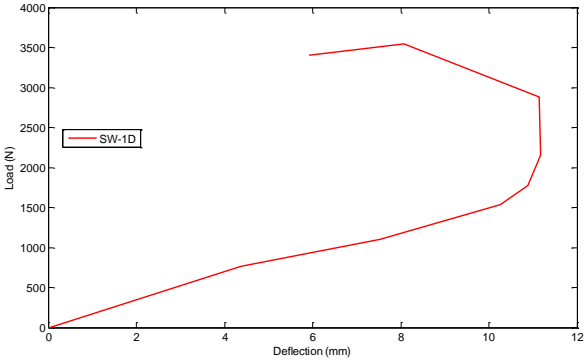
c)

Specimen Number	SW-1D	EEEE Results	
Sheathing Thickness (OSB)	11mm	Peak Load (N)	3360.00
Nail Diameter	2.84 mm	Displacement at Peak (mm)	79.70
Stud Size	38 x89 mm	Initial Stiffness (N/mm)	97.95
Number of End Studs	2	Yield Load (N)	2980.00
Hold-Down Type	HDU2	Ductility Ratio	3.36
Aspect Ratio	4:1		

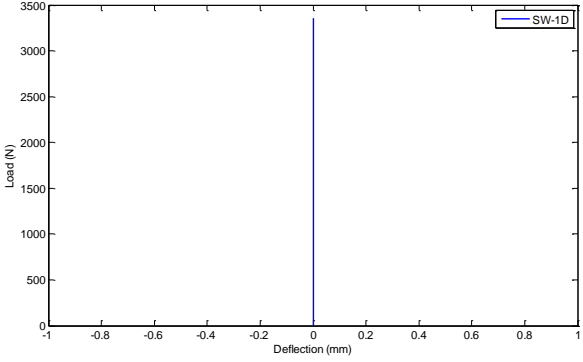
Total Behavior



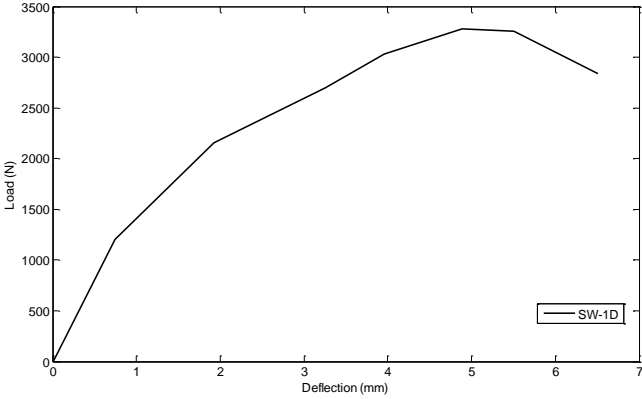
Specimen Number	SW-1D	Deflection Contributors	
Sheathing Thickness (OSB)	11 mm	a)	Bending
Nail Diameter	2.84 mm	b)	Shear
Stud Size	38 x 89 mm	c)	Hold-Down
Number of End Studs	2		
Hold-Down Type	HDU2		



a)



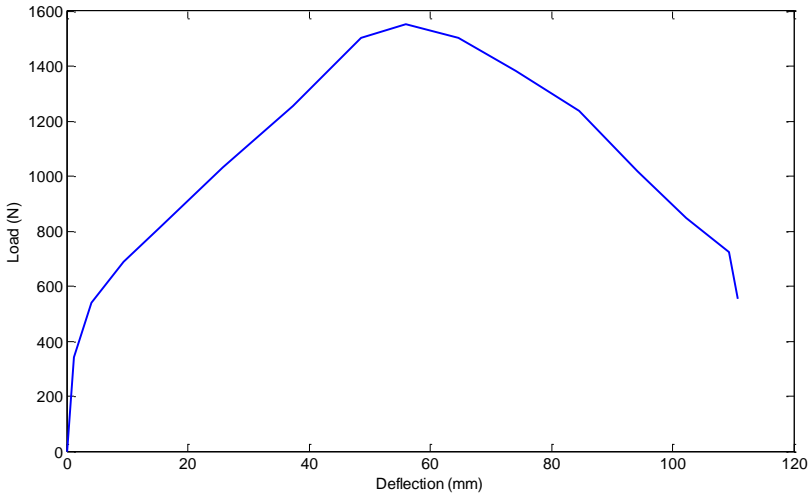
b)



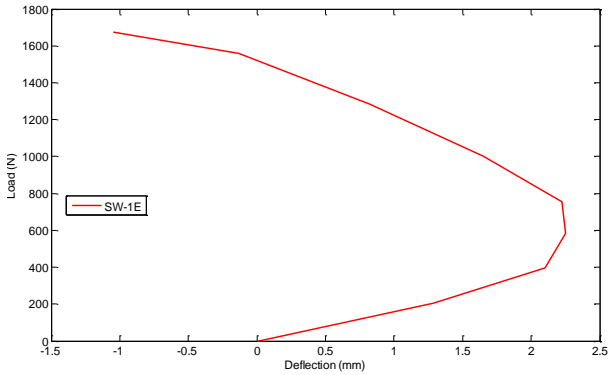
c)

Specimen Number	SW-1E	EEEE Results	
Sheathing Thickness (OSB)	11mm	Peak Load (N)	1550.00
Nail Diameter	2.84 mm	Displacement at Peak (mm)	56.00
Stud Size	38 x89 mm	Initial Stiffness (N/mm)	80.00
Number of End Studs	2	Yield Load (N)	1257.82
Hold-Down Type	HDU2	Ductility Ratio	5.04
Aspect Ratio	6:1		

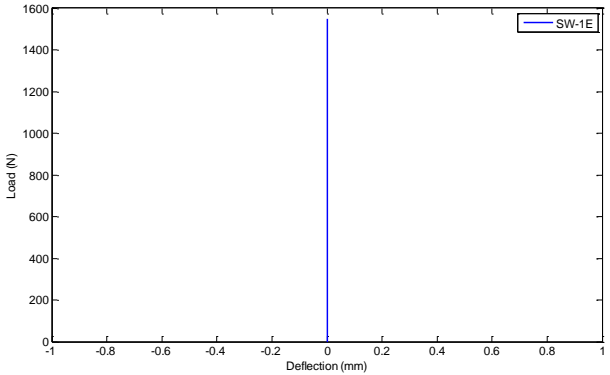
Total Behavior



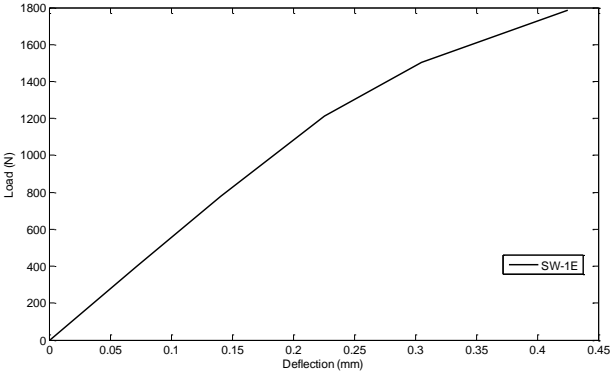
Specimen Number	SW-1E	Deflection Contributors	
Sheathing Thickness (OSB)	11 mm	a)	Bending
Nail Diameter	2.84 mm	b)	Shear
Stud Size	38 x 89 mm	c)	Hold-Down
Number of End Studs	2		
Hold-Down Type	HDU2		



a)



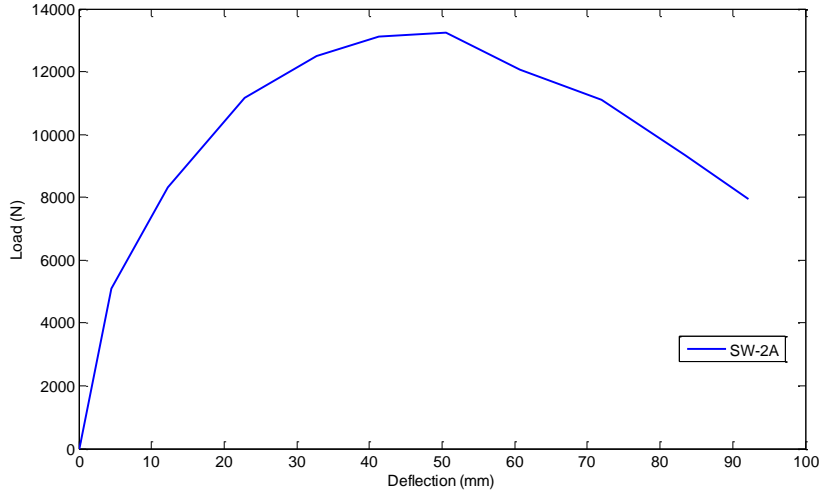
b)



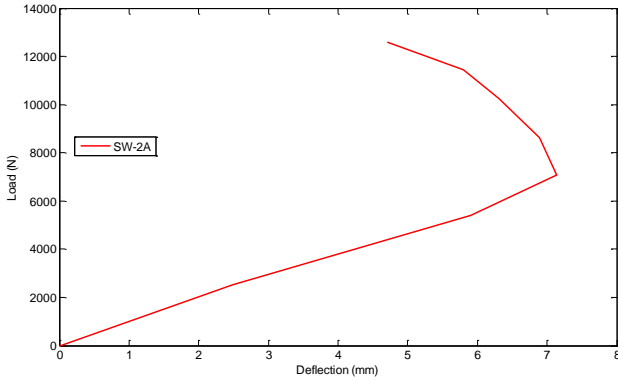
c)

Specimen Number	SW-2A	EEEE Results	
Sheathing Thickness (OSB)	11mm	Peak Load (N)	13250.00
Nail Diameter	2.84 mm	Displacement at Peak (mm)	49.90
Stud Size	38 x89 mm	Initial Stiffness (N/mm)	1020.00
Number of End Studs	1	Yield Load (N)	11643.75
Hold-Down Type	H DU2	Ductility Ratio	6.65
Aspect Ratio	1:1		

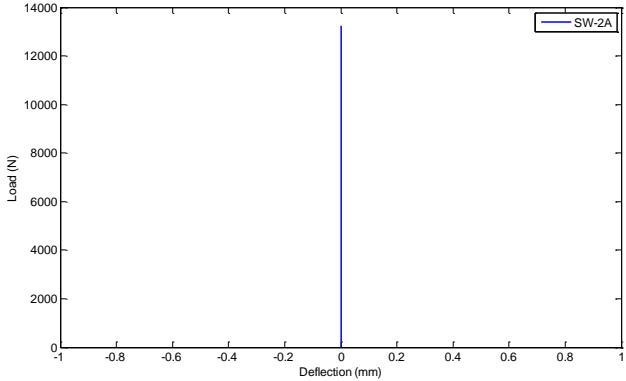
Total Behavior



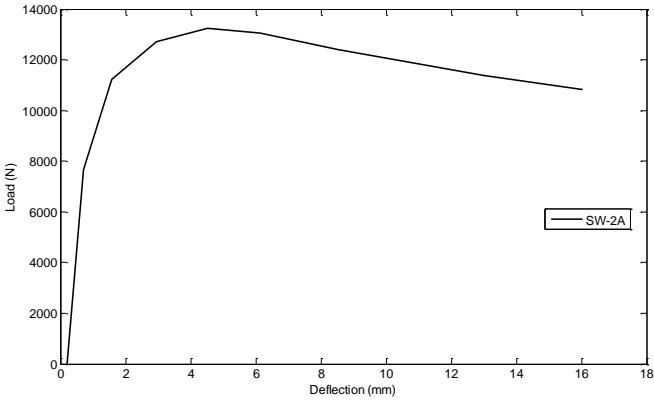
Specimen Number	SW-2A	Deflection Contributors	
Sheathing Thickness (OSB)	11 mm	a)	Bending
Nail Diameter	2.84 mm	b)	Shear
Stud Size	38 x 89 mm	c)	Hold-Down
Number of End Studs	1		
Hold-Down Type	HDU2		



a)



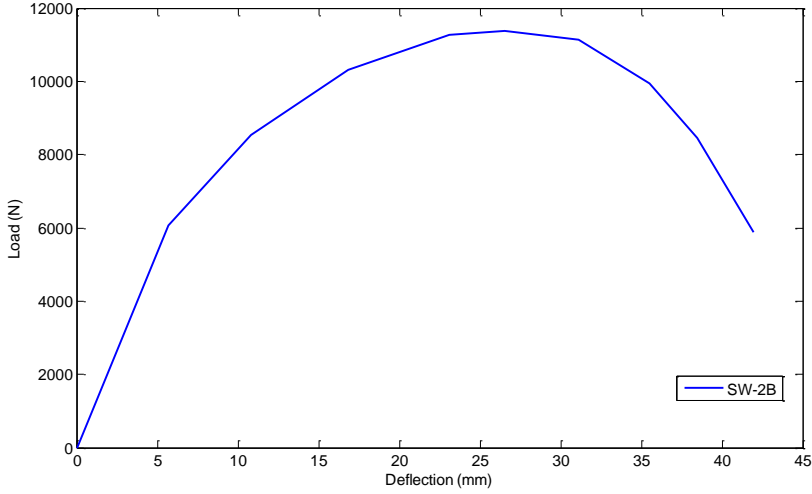
b)



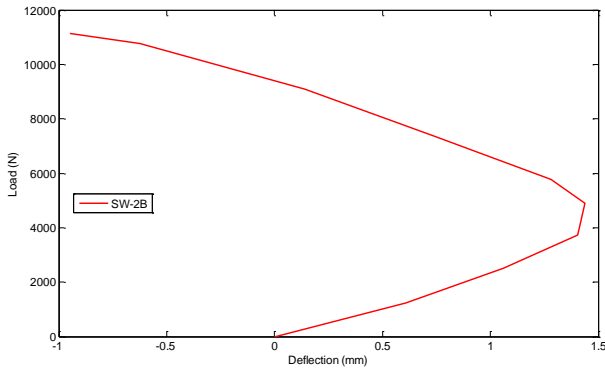
c)

Specimen Number	SW-2B	EEEE Results	
Sheathing Thickness (OSB)	11mm	Peak Load (N)	13010.00
Nail Diameter	2.84 mm	Displacement at Peak (mm)	28.02
Stud Size	38 x89 mm	Initial Stiffness (N/mm)	1750.00
Number of End Studs	3	Yield Load (N)	11595.08
Hold-Down Type	HDU2	Ductility Ratio	5.86
Aspect Ratio	1:1		

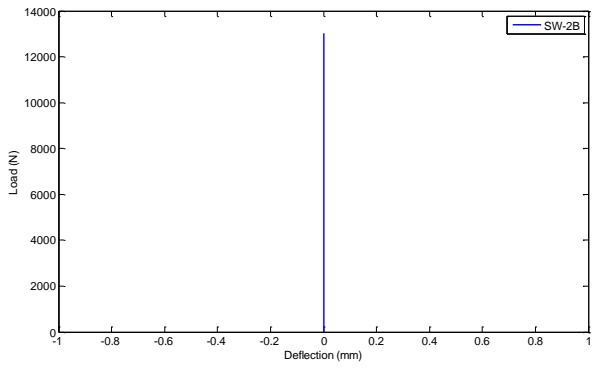
Total Behavior



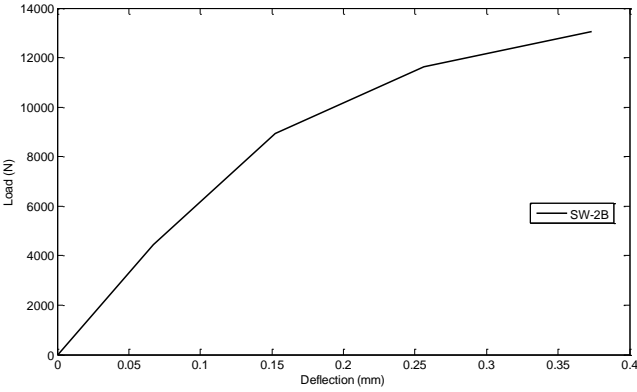
Specimen Number	SW-2B	Deflection Contributors	
Sheathing Thickness (OSB)	11 mm	a)	Bending
Nail Diameter	2.84 mm	b)	Shear
Stud Size	38 x 89 mm	c)	Hold-Down
Number of End Studs	3		
Hold-Down Type	HDU2		



a)



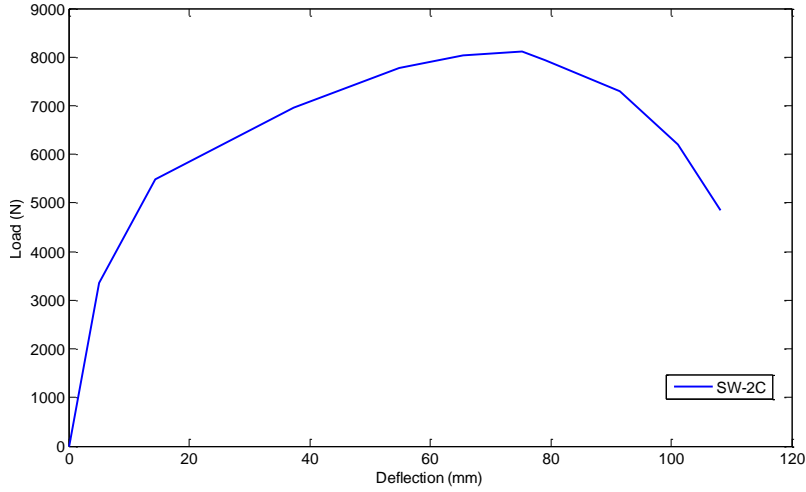
b)



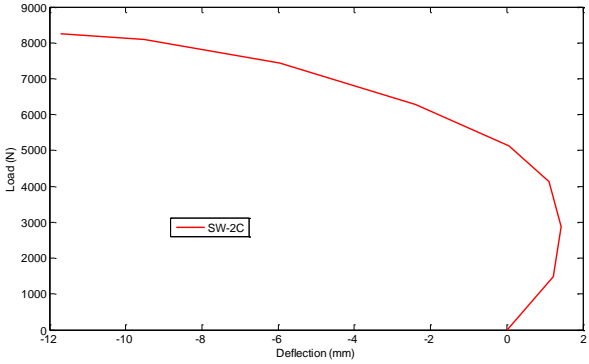
c)

Specimen Number	SW-2C	EEEE Results	
Sheathing Thickness (OSB)	11mm	Peak Load (N)	7440.00
Nail Diameter	2.84 mm	Displacement at Peak (mm)	70.44
Stud Size	38 x89 mm	Initial Stiffness (N/mm)	480.00
Number of End Studs	1	Yield Load (N)	7192.11
Hold-Down Type	HDU2	Ductility Ratio	6.47
Aspect Ratio	2:1		

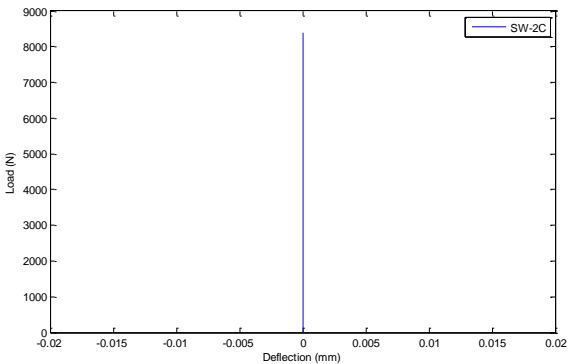
Total Behavior



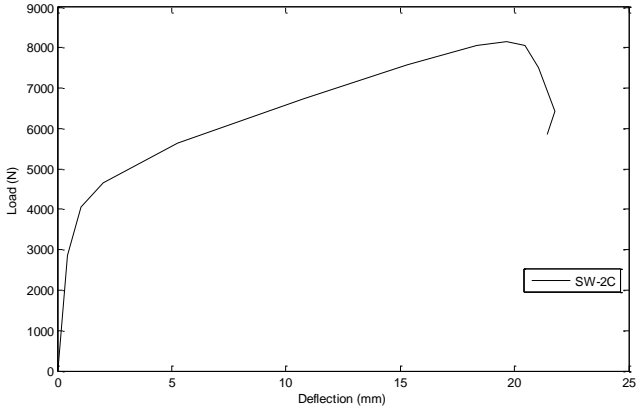
Specimen Number	SW-2C	Deflection Contributors	
Sheathing Thickness (OSB)	11 mm	a)	Bending
Nail Diameter	2.84 mm	b)	Shear
Stud Size	38 x 89 mm	c)	Hold-Down
Number of End Studs	1		
Hold-Down Type	HDU2		



a)



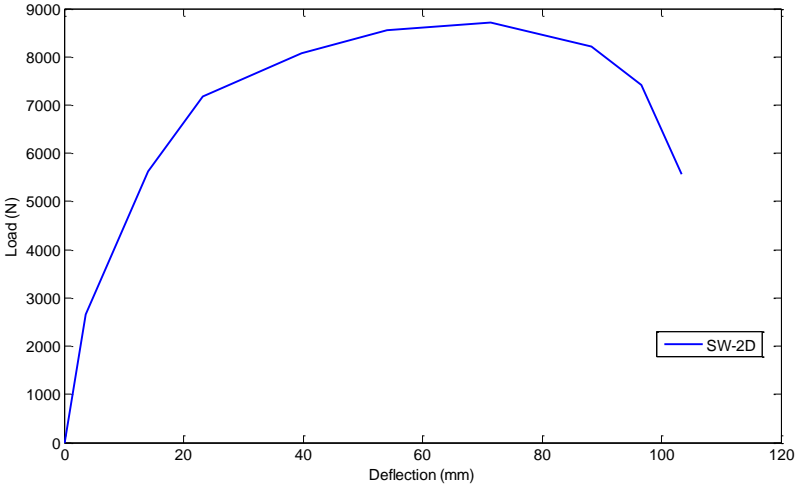
b)



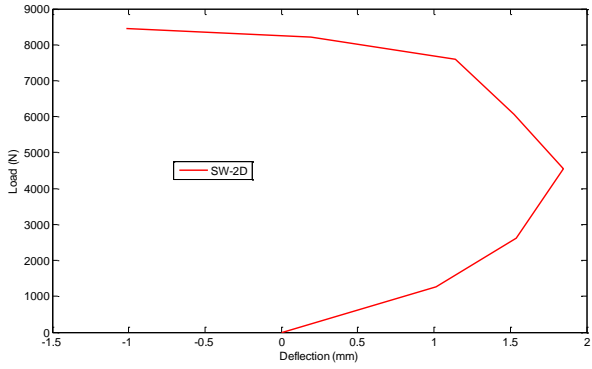
c)

Specimen Number	SW-2D	EEEE Results	
Sheathing Thickness (OSB)	11mm	Peak Load (N)	8700.00
Nail Diameter	2.84 mm	Displacement at Peak (mm)	63.68
Stud Size	38 x89 mm	Initial Stiffness (N/mm)	550.00
Number of End Studs	3	Yield Load (N)	7840.50
Hold-Down Type	HDU2	Ductility Ratio	6.85
Aspect Ratio	2:1		

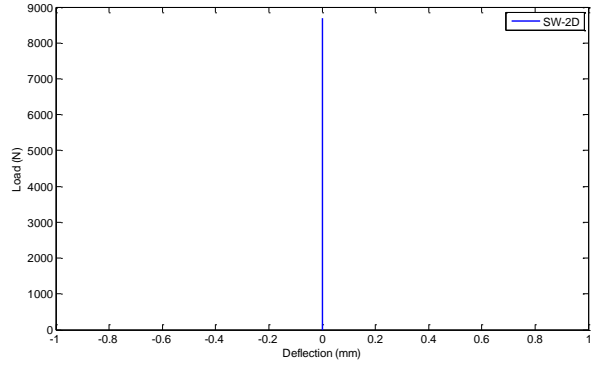
Total Behavior



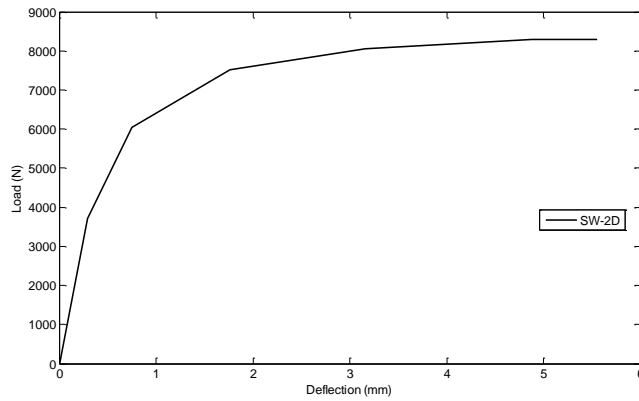
Specimen Number	SW-2D	Deflection Contributors	
Sheathing Thickness (OSB)	11 mm	a)	Bending
Nail Diameter	2.84 mm	b)	Shear
Stud Size	38 x 89 mm	c)	Hold-Down
Number of End Studs	3		
Hold-Down Type	HDU2		



a)



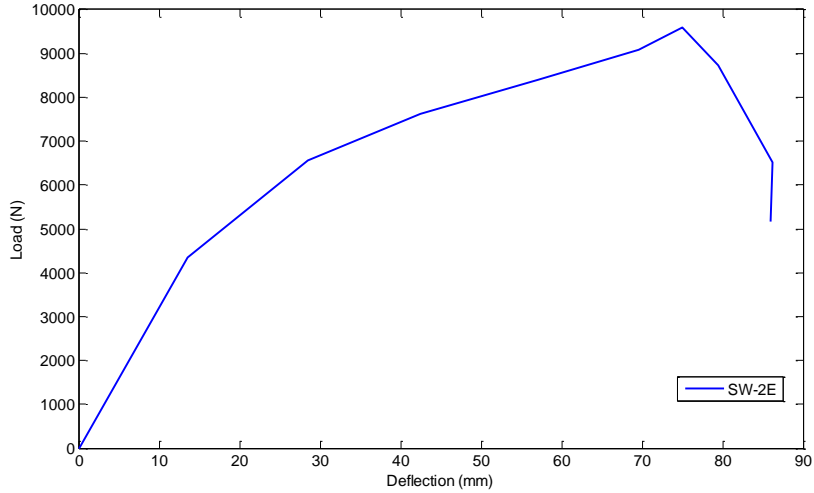
b)



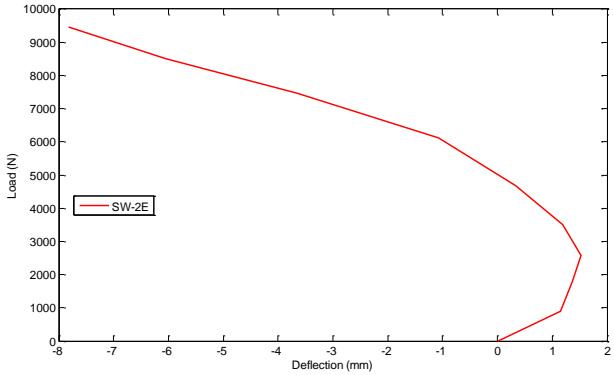
c)

Specimen Number	SW-2E	EEEE Results	
Sheathing Thickness (OSB)	11mm	Peak Load (N)	9230.00
Nail Diameter	2.84 mm	Displacement at Peak (mm)	73.90
Stud Size	38 x140 mm	Initial Stiffness (N/mm)	320.00
Number of End Studs	1	Yield Load (N)	7775.94
Hold-Down Type	HDU2	Ductility Ratio	3.41
Aspect Ratio	2:1		

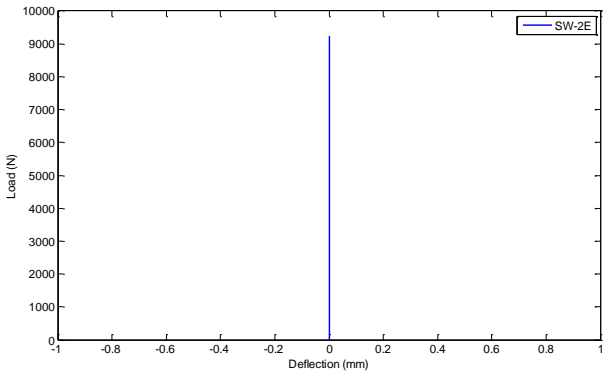
Total Behavior



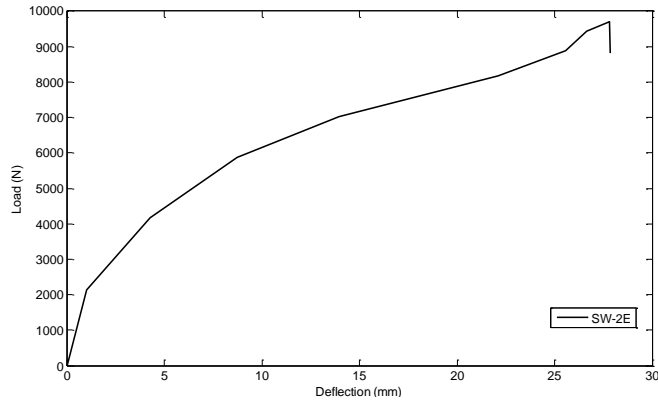
Specimen Number	SW-2E	Deflection Contributors	
Sheathing Thickness (OSB)	11 mm	a)	Bending
Nail Diameter	2.84 mm	b)	Shear
Stud Size	38 x 140 mm	c)	Hold-Down
Number of End Studs	1		
Hold-Down Type	HDU2		



a)



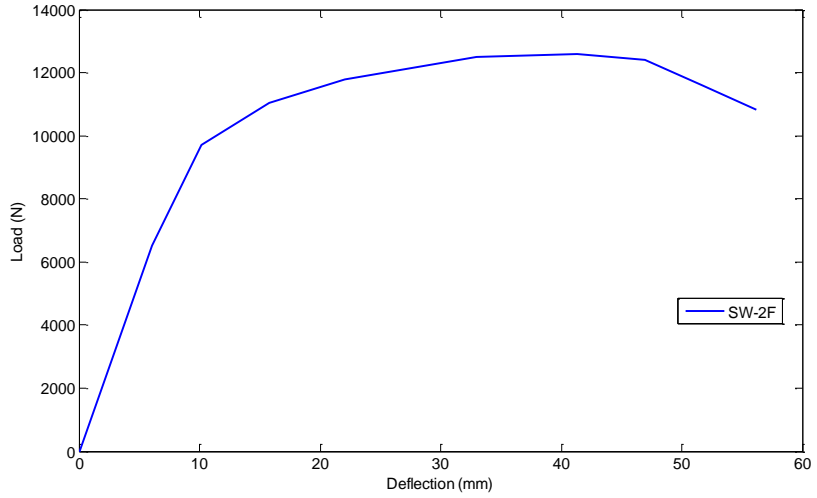
b)



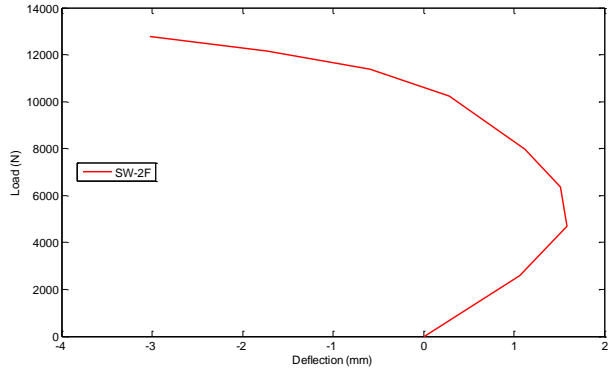
c)

Specimen Number	SW-2F	EEEE Results	
Sheathing Thickness (OSB)	11mm	Peak Load (N)	13000.00
Nail Diameter	2.84 mm	Displacement at Peak (mm)	46.05
Stud Size	38 x140 mm	Initial Stiffness (N/mm)	1140
Number of End Studs	2	Yield Load (N)	11729.50
Hold-Down Type	HDU2	Ductility Ratio	4.17
Aspect Ratio	1:1		

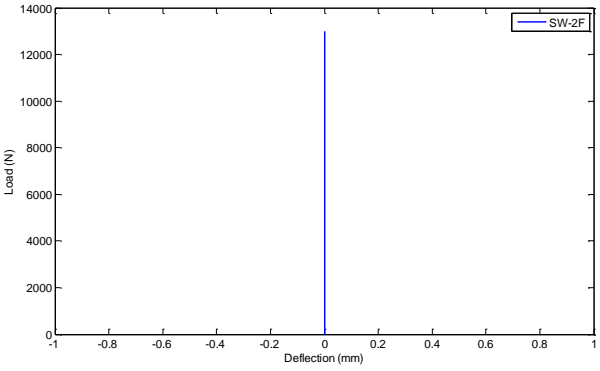
Total Behavior



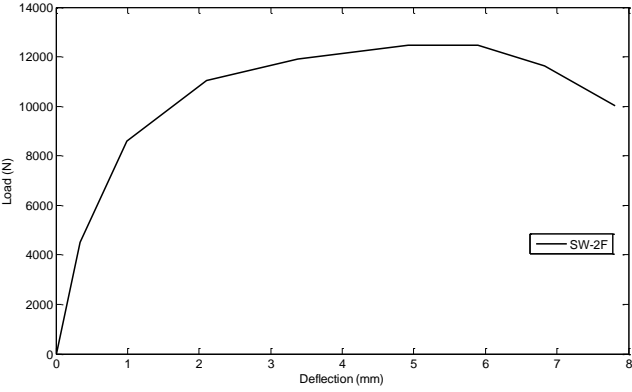
Specimen Number	SW-2F	Deflection Contributors
Sheathing Thickness (OSB)	11 mm	a) Bending
Nail Diameter	2.84 mm	b) Shear
Stud Size	38 x 89 mm	c) Hold-Down
Number of End Studs	2	
Hold-Down Type	HDU2	



a)



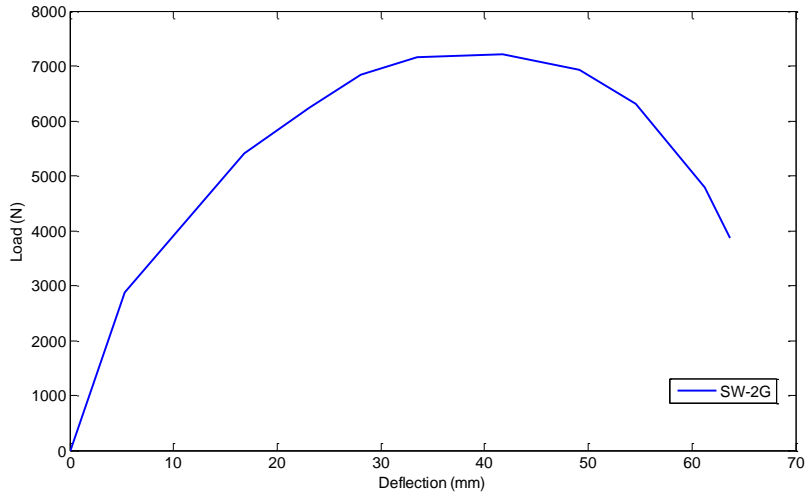
b)



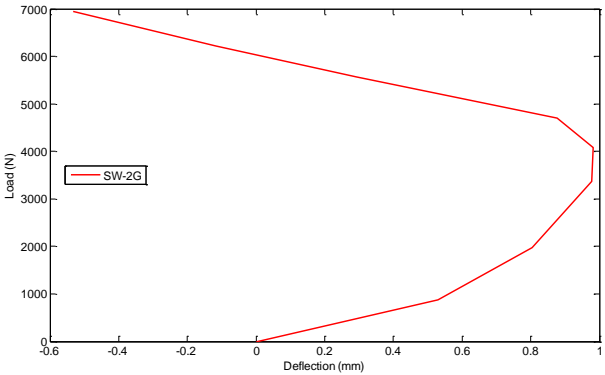
c)

Specimen Number	SW-2G	EEEE Results	
Sheathing Thickness (OSB)	11mm	Peak Load (N)	7200.00
Nail Diameter	2.84 mm	Displacement at Peak (mm)	33.64
Stud Size	38 x140 mm	Initial Stiffness (N/mm)	430.00
Number of End Studs	3	Yield Load (N)	6517.24
Hold-Down Type	HDU2	Ductility Ratio	3.82
Aspect Ratio	2:1		

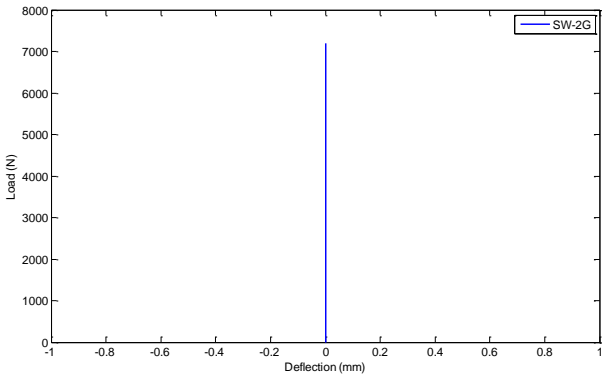
Total Behavior



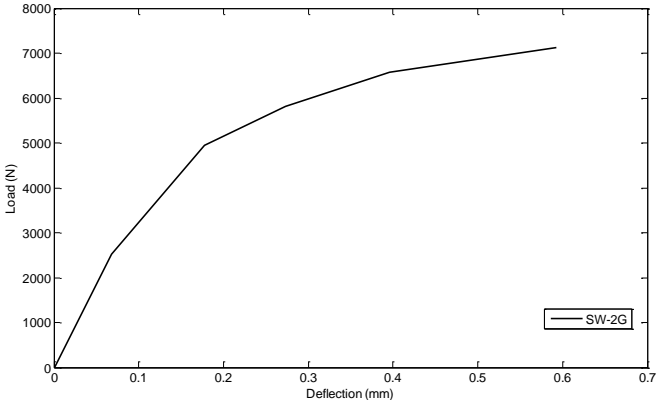
Specimen Number	SW-2G	Deflection Contributors	
Sheathing Thickness (OSB)	11 mm	a)	Bending
Nail Diameter	2.84 mm	b)	Shear
Stud Size	38 x 140 mm	c)	Hold-Down
Number of End Studs	3		
Hold-Down Type	HDU2		



a)



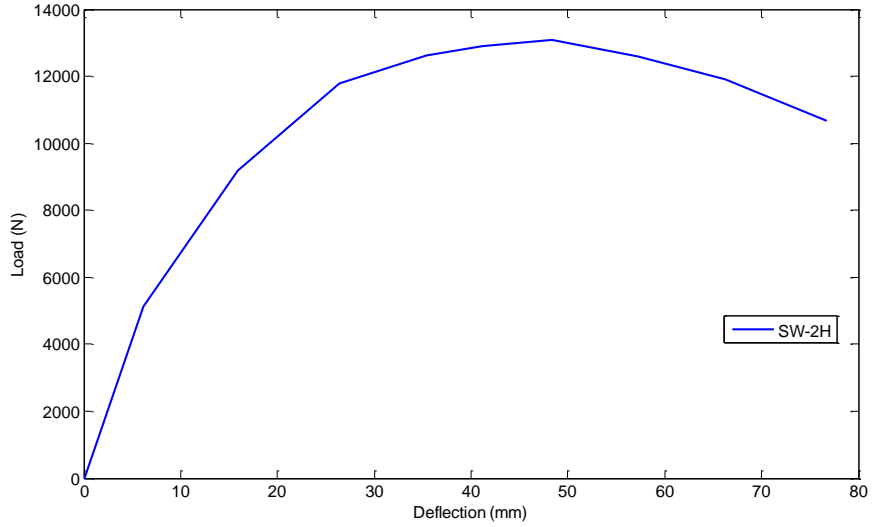
b)



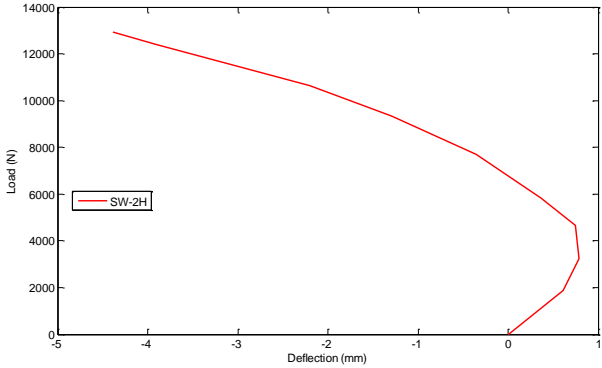
c)

Specimen Number	SW-2H	EEEE Results	
Sheathing Thickness (OSB)	11mm	Peak Load (N)	12928.00
Nail Diameter	2.84 mm	Displacement at Peak (mm)	48.21
Stud Size and Spacing	38 x89, 304.80mm	Initial Stiffness (N/mm)	977.00
Number of End Studs	2	Yield Load (N)	13785.47
Hold-Down Type	HDU2	Ductility Ratio	6.10
Aspect Ratio	1:1		

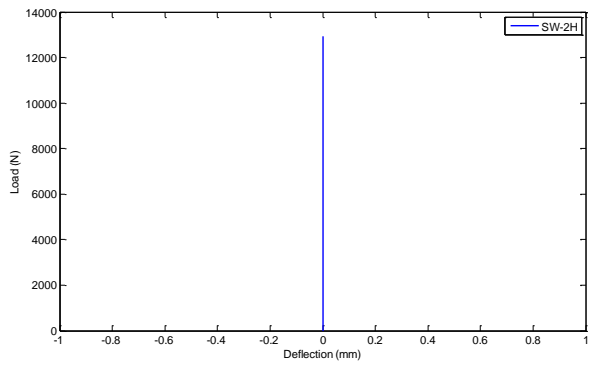
Total Behavior



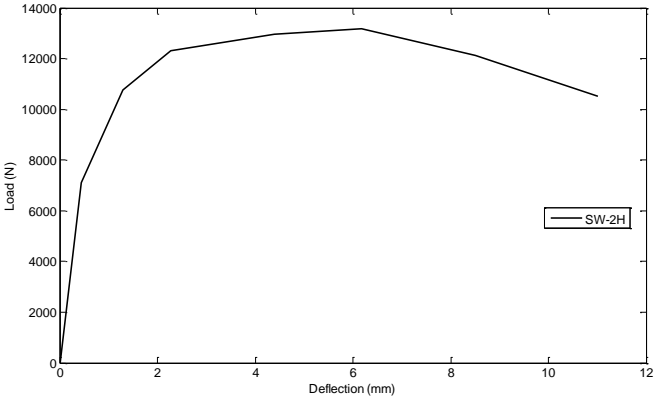
Specimen Number	SW-2H	Deflection Contributors
Sheathing Thickness (OSB)	11 mm	a) Bending
Nail Diameter	2.84 mm	b) Shear
Stud Size and Spacing	38 x89, 304.80mm	c) Hold-Down
Number of End Studs	2	
Hold-Down Type	HDU2	



a)



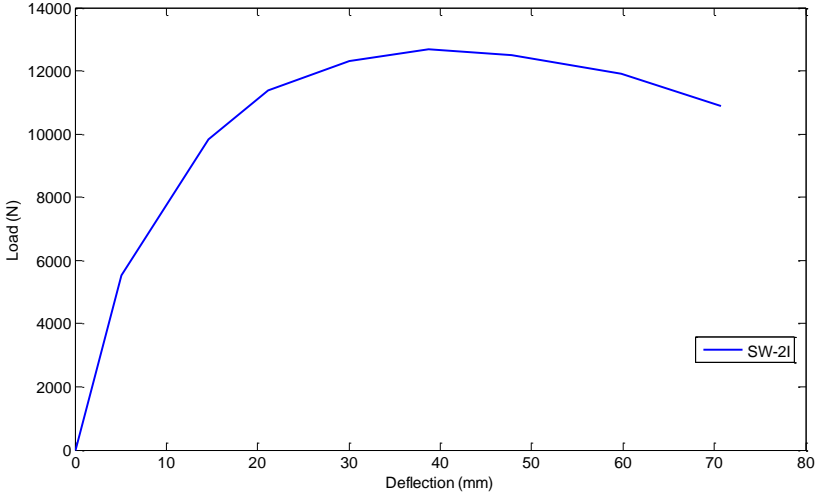
b)



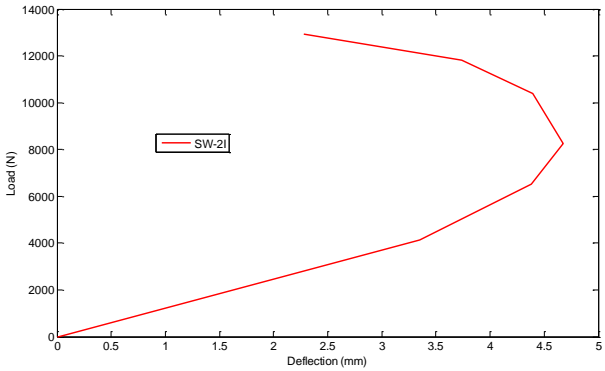
c)

Specimen Number	SW-2I	EEEE Results	
Sheathing Thickness (OSB)	11mm	Peak Load (N)	12460.00
Nail Diameter	2.84 mm	Displacement at Peak (mm)	40.66
Stud Size and Spacing	38 x89 mm, 609.60mm	Initial Stiffness (N/mm)	926.00
Number of End Studs	2	Yield Load (N)	11834.04
Hold-Down Type	HDU2	Ductility Ratio	5.76
Aspect Ratio	1:1		

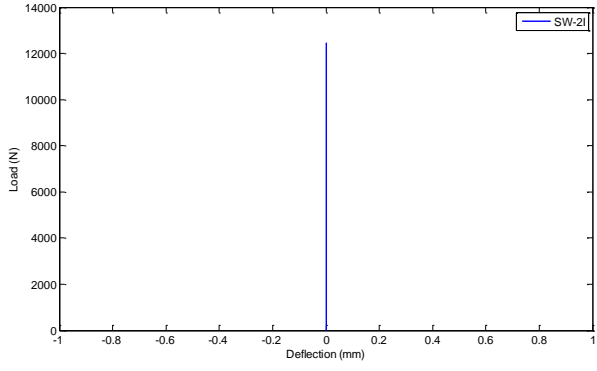
Total Behavior



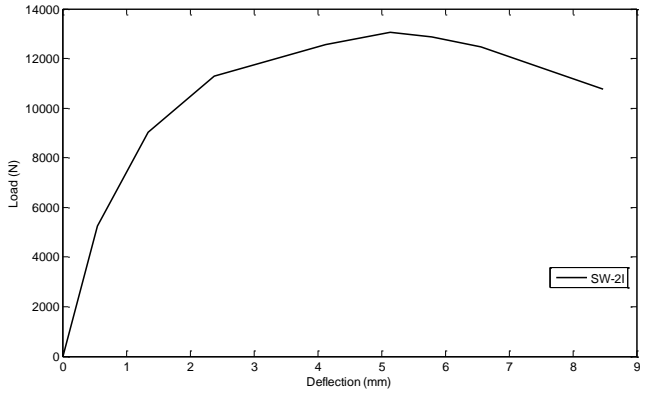
Specimen Number	SW-2I	Deflection Contributors
Sheathing Thickness (OSB)	11 m	a) Bending
Nail Diameter	2.84 mm	b) Shear
Stud Size and Spacing	38 x89 mm, 609.60mm	c) Hold-Down
Number of End Studs	2	
Hold-Down Type	HDU2	



a)



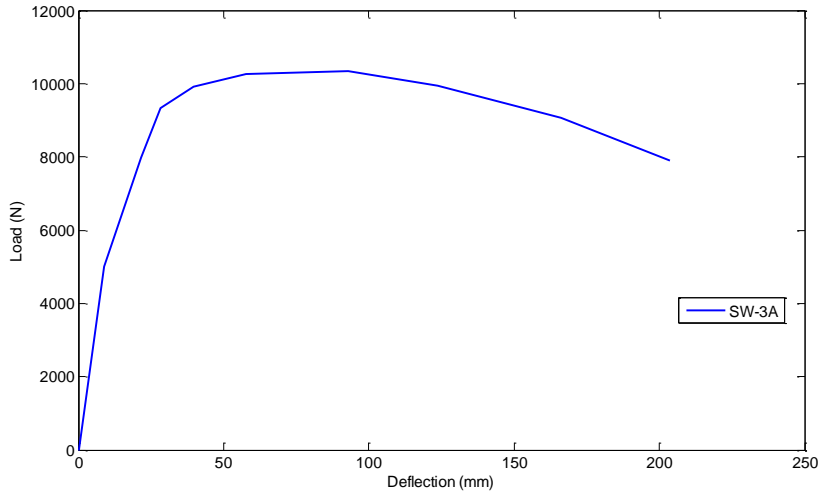
b)



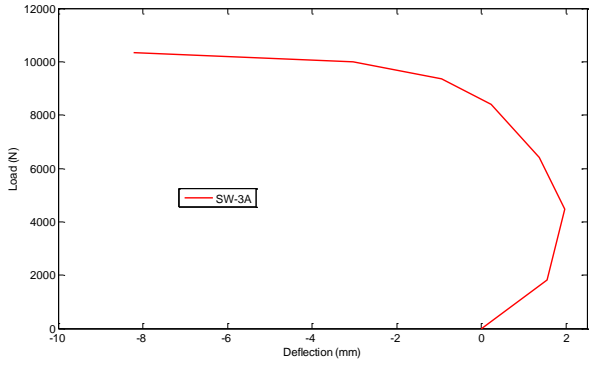
c)

Specimen Number	SW-3A	EEEE Results	
Sheathing Thickness (OSB)	15.5mm	Peak Load (N)	10280.00
Nail Diameter	2.84 mm	Displacement at Peak (mm)	94.10
Stud Size	38 x89 mm	Initial Stiffness (N/mm)	620.00
Number of End Studs	2	Yield Load (N)	9514.42
Hold-Down Type	HDU2	Ductility Ratio	11.75
Aspect Ratio	2:1		

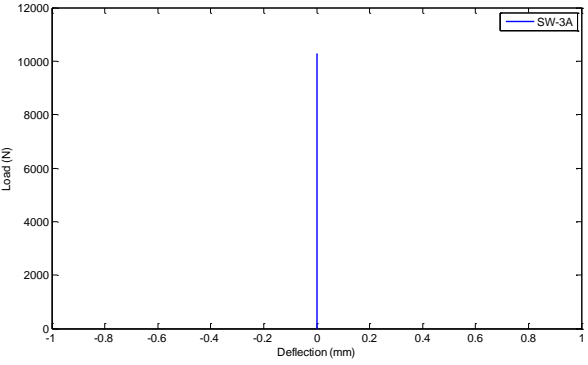
Total Behavior



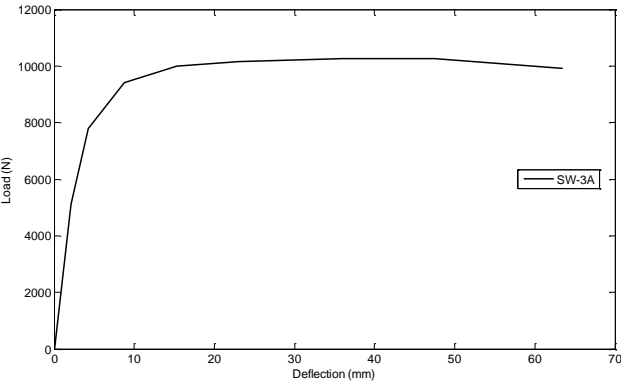
Specimen Number	SW-3A	Deflection Contributors	
Sheathing Thickness (OSB)	15.5 mm	a)	Bending
Nail Diameter	2.84 mm	b)	Shear
Stud Size	38 x 89 mm	c)	Hold-Down
Number of End Studs	2		
Hold-Down Type	HDU2		



a)



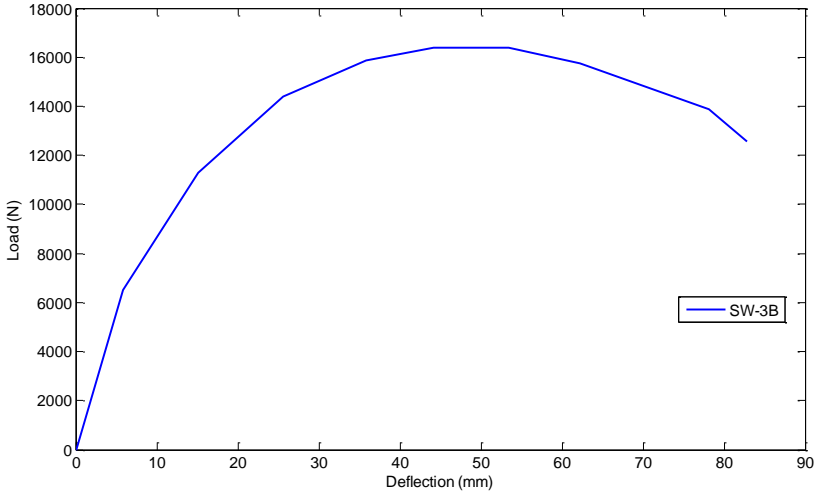
b)



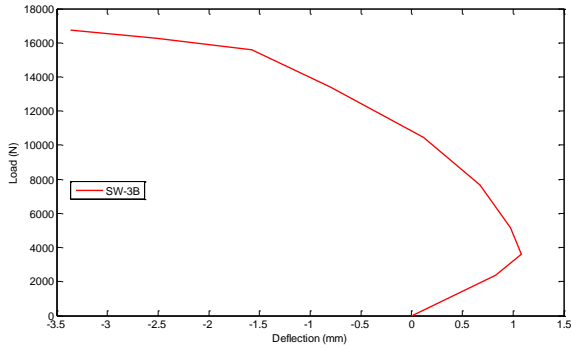
c)

Specimen Number	SW-3B	EEEE Results	
Sheathing Thickness (Plywood)	12.5mm	Peak Load (N)	16400.00
Nail Diameter	2.84 mm	Displacement at Peak (mm)	43.62
Stud Size	38 x89 mm	Initial Stiffness (N/mm)	1120.00
Number of End Studs	2	Yield Load (N)	14786.85
Hold-Down Type	HDU2	Ductility Ratio	6.14
Aspect Ratio	1:1		

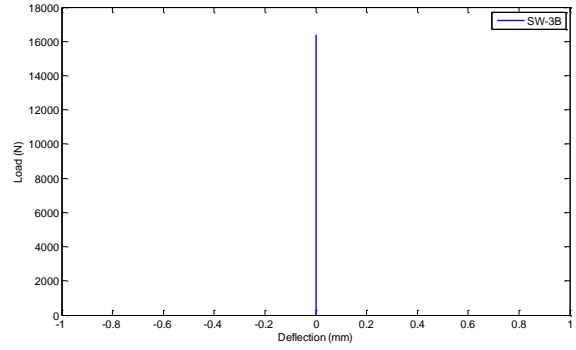
Total Behavior



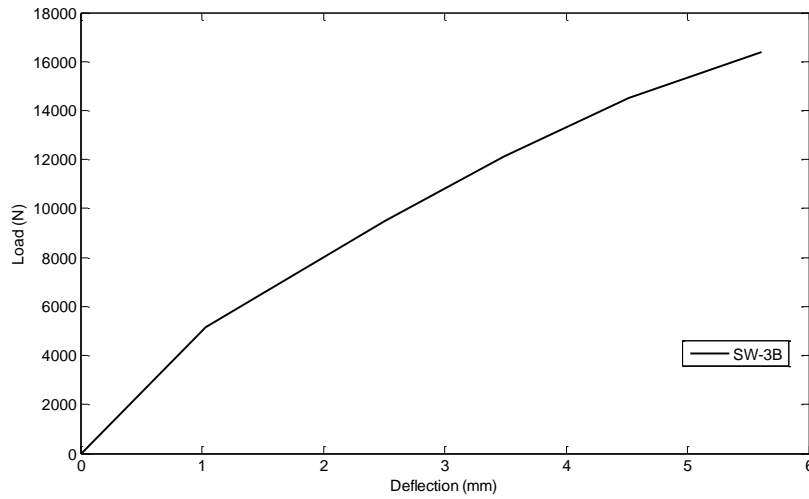
Specimen Number	SW-3B	Deflection Contributors	
Sheathing Thickness (OSB)	11 mm	a)	Bending
Nail Diameter	2.84 mm	b)	Shear
Stud Size	38 x 89 mm	c)	Hold-Down
Number of End Studs	2		
Hold-Down Type	HDU2		



a)



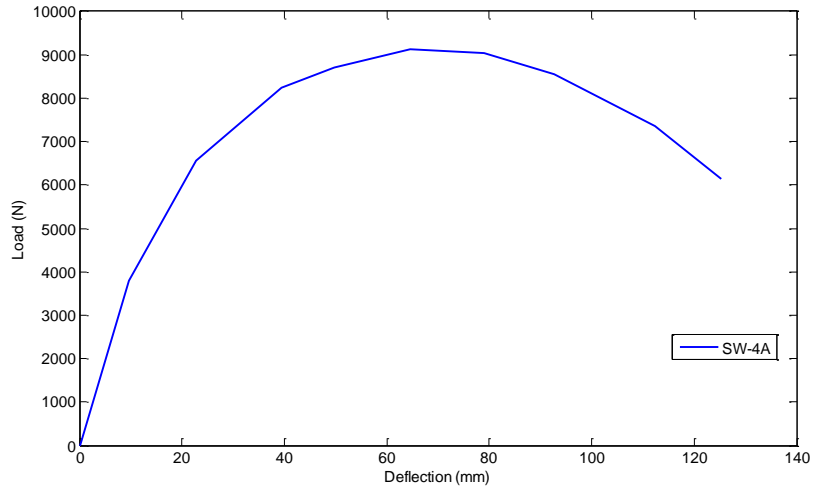
b)



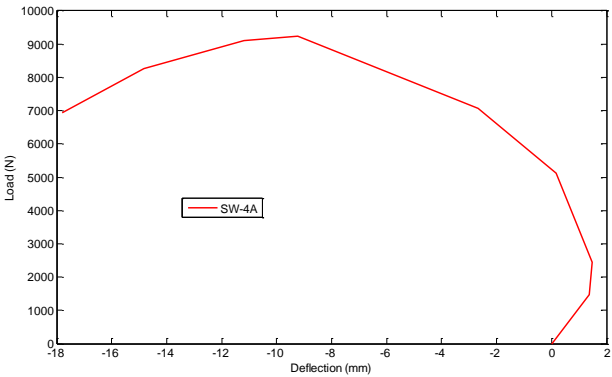
c)

Specimen Number	SW-4A	EEEE Results
Sheathing Thickness (OSB)	11mm	Peak Load (N) 9190.00
Nail Diameter	3.76mm	Displacement at Peak (mm) 70.73
Stud Size	38 x89 mm	Initial Stiffness (N/mm) 440.00
Number of End Studs	2	Yield Load (N) 8314.89
Hold-Down Type	HDU2	Ductility Ratio 5.28
Aspect Ratio	2:1	

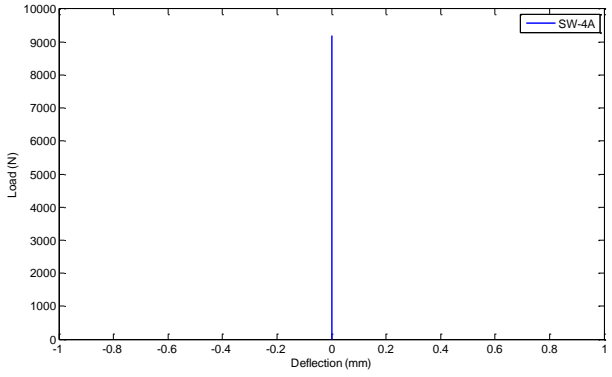
Total Behavior



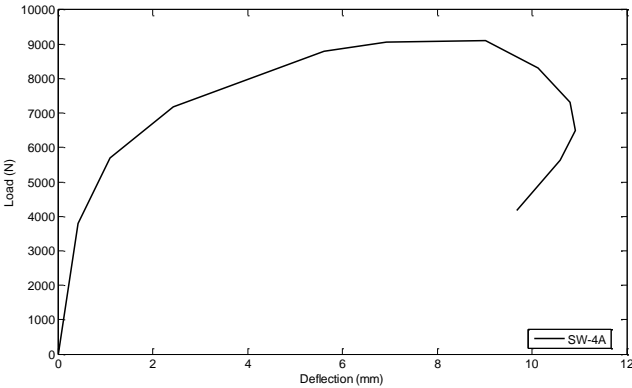
Specimen Number	SW-4A	Deflection Contributors
Sheathing Thickness (OSB)	11 mm	a) Bending
Nail Diameter	3.76 mm	b) Shear
Stud Size	38 x 89 mm	c) Hold-Down
Number of End Studs	2	
Hold-Down Type	HDU2	



a)



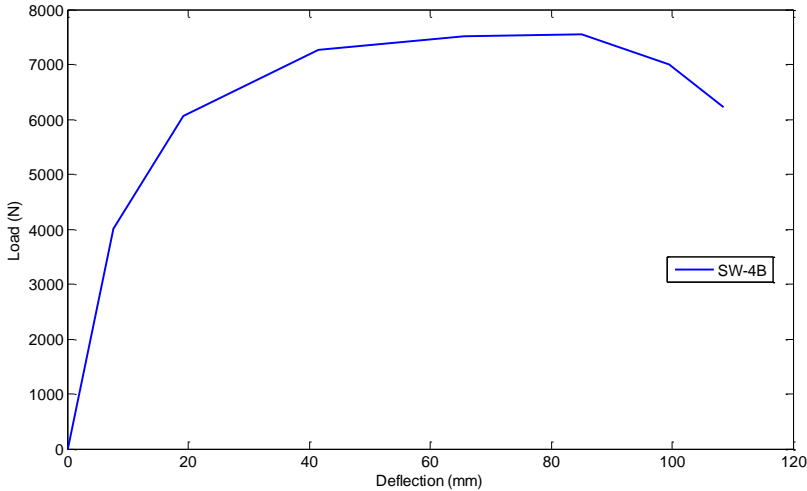
b)



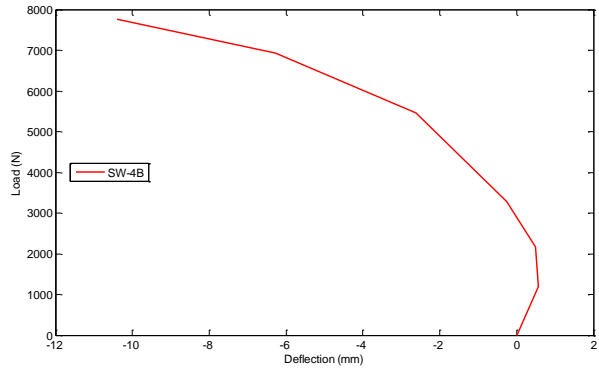
c)

Specimen Number	SW-4B	EEEE Results	
Sheathing Thickness (OSB)	11mm	Peak Load (N)	7650.00
Nail Diameter	3.32 mm	Displacement at Peak (mm)	77.83
Stud Size	38 x89 mm	Initial Stiffness (N/mm)	500.00
Number of End Studs	2	Yield Load (N)	6986.00
Hold-Down Type	HDU2	Ductility Ratio	9.17

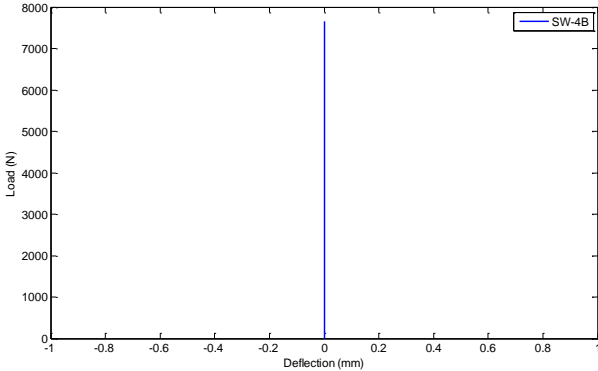
Total Behavior



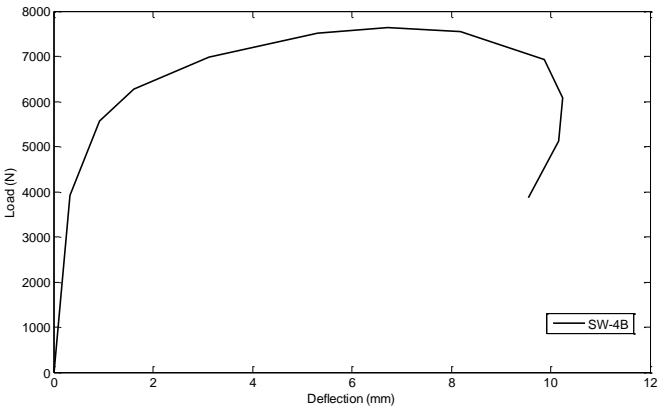
Specimen Number	SW-4B	Deflection Contributors	
Sheathing Thickness (OSB)	11 mm	a)	Bending
Nail Diameter	3.32 mm	b)	Shear
Stud Size	38 x 89 mm	c)	Hold-Down
Number of End Studs	2		
Hold-Down Type	HDU2		



a)



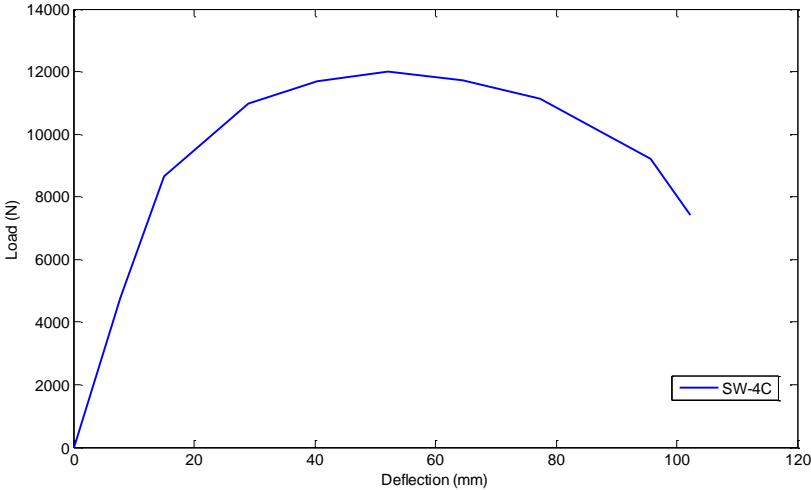
b)



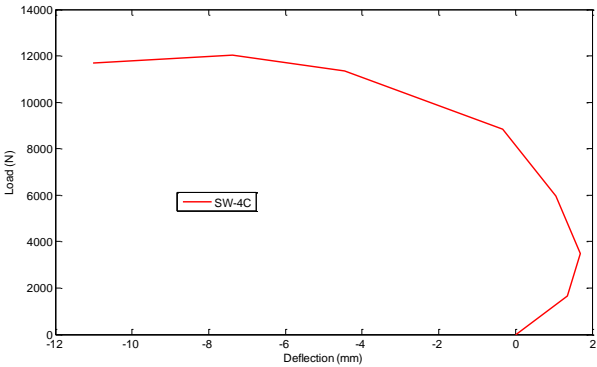
c)

Specimen Number	SW-4C	EEEE Results	
Sheathing Thickness (OSB)	11mm	Peak Load (N)	11930.00
Nail Diameter and Spacing	2.84 mm, 75mm	Displacement at Peak (mm)	52.01
Stud Size	38 x89 mm	Initial Stiffness (N/mm)	650
Number of End Studs	2	Yield Load (N)	11009.77
Hold-Down Type	HDU2	Ductility Ratio	5.37
Aspect Ratio	2:1		

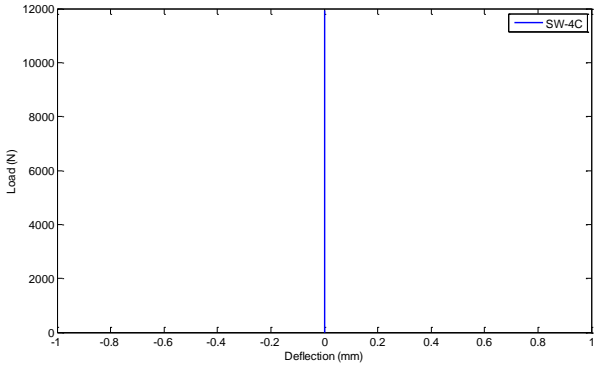
Total Behavior



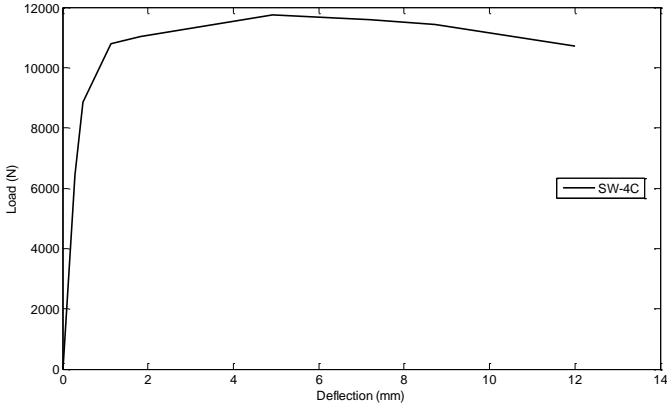
Specimen Number	SW-4C	Deflection Contributors	
Sheathing Thickness (OSB)	11 mm	a)	Bending
Nail Diameter and Spacing	2.84 mm, 75mm	b)	Shear
Stud Size	38 x 89 mm	c)	Hold-Down
Number of End Studs	2		
Hold-Down Type	HDU2		



a)



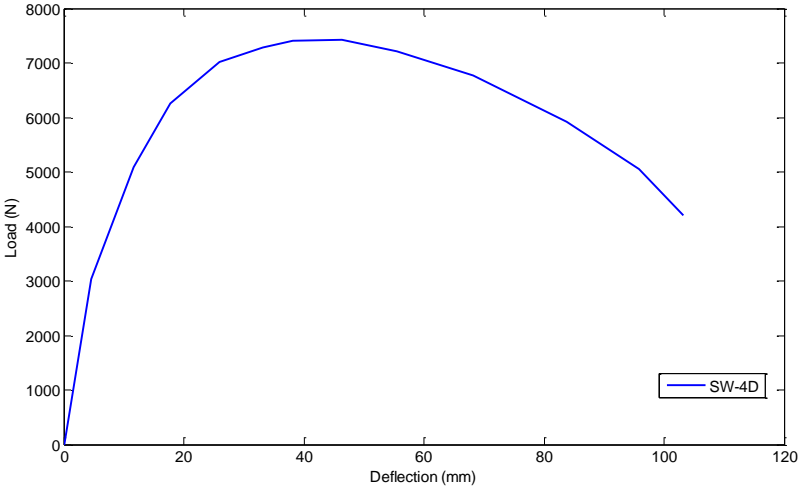
b)



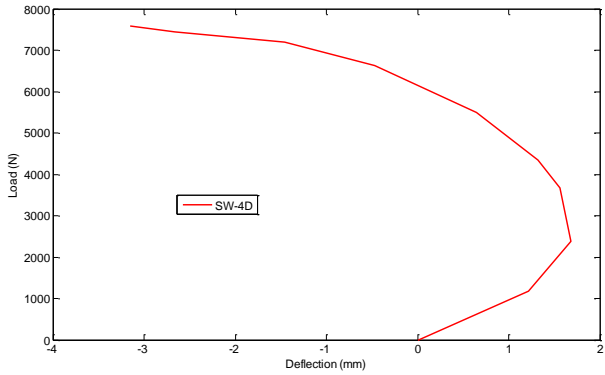
c)

Specimen Number	SW-4D	EEEE Results	
Sheathing Thickness (OSB)	11mm	Peak Load (N)	7040.20
Nail Diameter	2.84 mm	Displacement at Peak (mm)	42.20
Stud Size, End/Edge Distance	38 x89 mm, 9.5mm	Initial Stiffness (N/mm)	573.00
Number of End Studs	2	Yield Load (N)	6765.52
Hold-Down Type	HDU2	Ductility Ratio	6.83
Aspect Ratio	2:1		

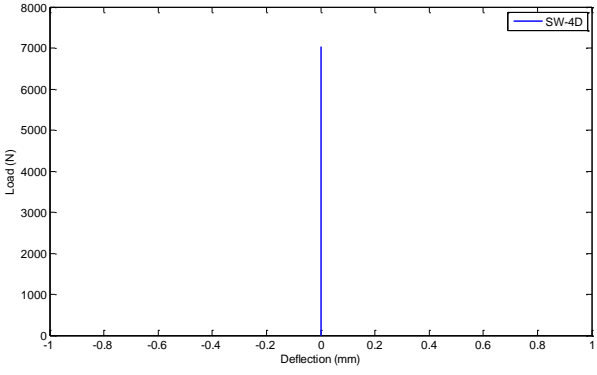
Total Behavior



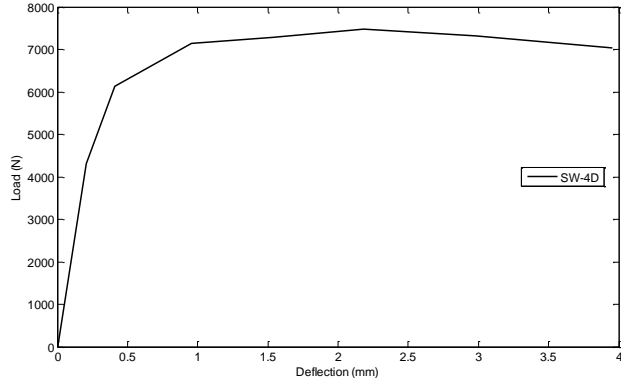
Specimen Number	SW-4D	Deflection Contributors	
Sheathing Thickness (OSB)	11 mm	a)	Bending
Stud Size, End/Edge Distance	38 x 89 mm, 9.5mm	b)	Shear
Stud Size	38 x 89 mm	c)	Hold-Down
Number of End Studs	2		
Hold-Down Type	HDU2		



a)



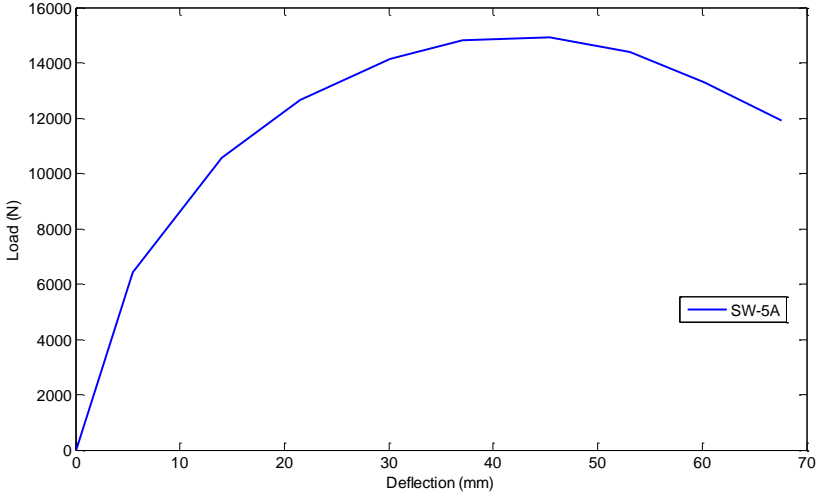
b)



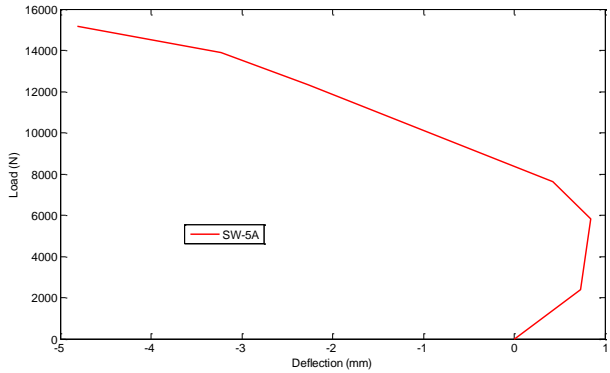
c)

Specimen Number	SW-5A	EEEE Results	
Sheathing Thickness (OSB)	11mm	Peak Load (N)	16200.00
Nail Diameter	2.84 mm	Displacement at Peak (mm)	44.13
Stud Size	38 x89 mm	Initial Stiffness (N/mm)	1500.00
Number of End Studs	2	Yield Load (N)	14363.61
Hold-Down Type	ATS	Ductility Ratio	7.06
Aspect Ratio	1:1		

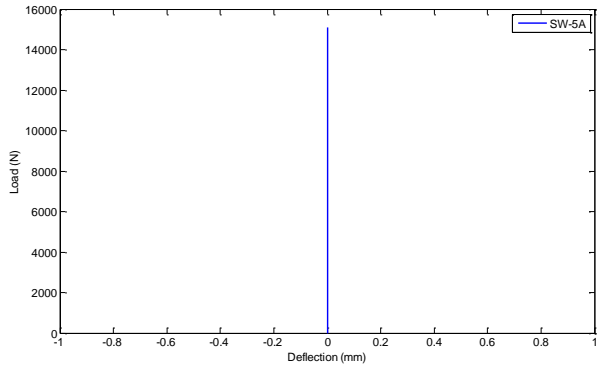
Total Behavior



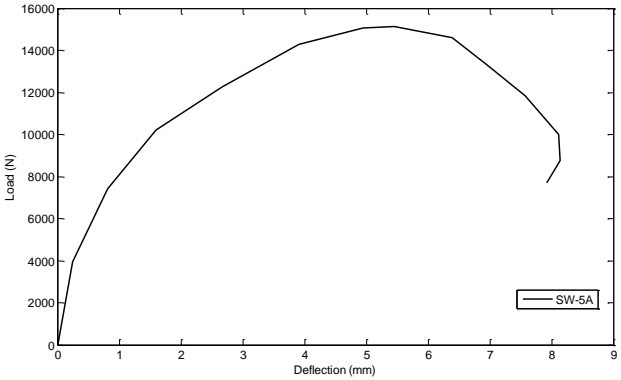
Specimen Number	SW-5A	Deflection Contributors
Sheathing Thickness (OSB)	11 mm	a) Bending
Nail Diameter	2.84 mm	b) Shear
Stud Size	38 x 89 mm	c) Hold-Down
Number of End Studs	2	
Hold-Down Type	ATS	



a)



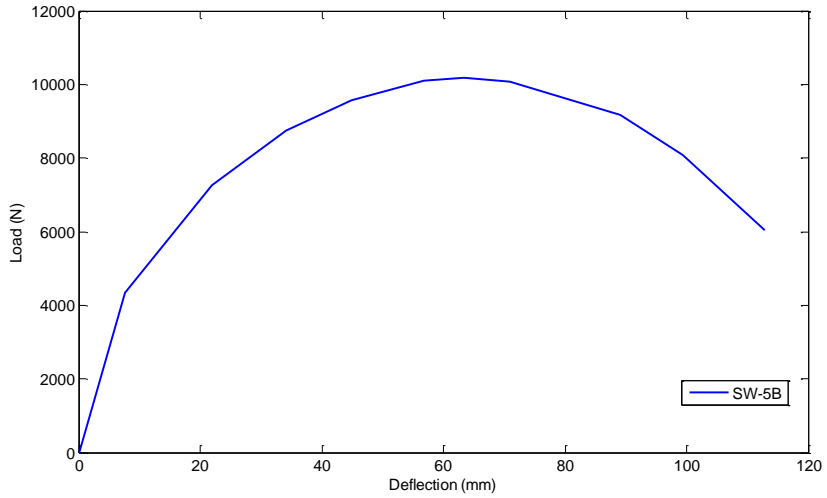
b)



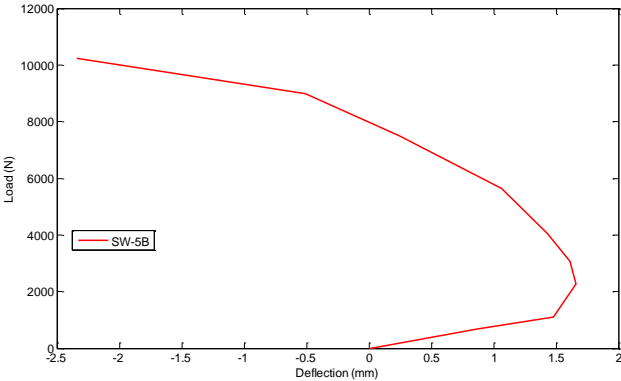
c)

Specimen Number	SW-5B	EEEE Results	
Sheathing Thickness (OSB)	11mm	Peak Load (N)	10170.00
Nail Diameter	2.84 mm	Displacement at Peak (mm)	59.29
Stud Size	38 x89 mm	Initial Stiffness (N/mm)	490.00
Number of End Studs	2	Yield Load (N)	8989.57
Hold-Down Type	ATS	Ductility Ratio	5.31
Aspect Ratio	2:1		

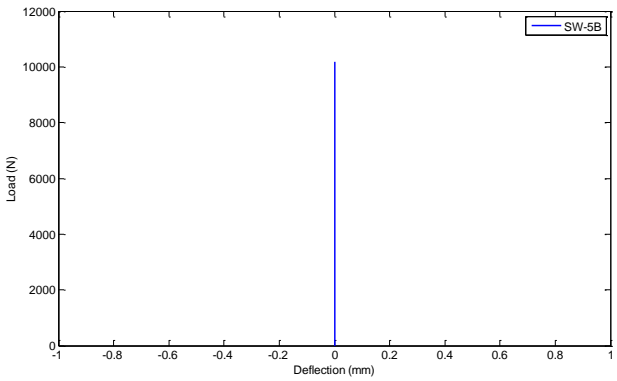
Total Behavior



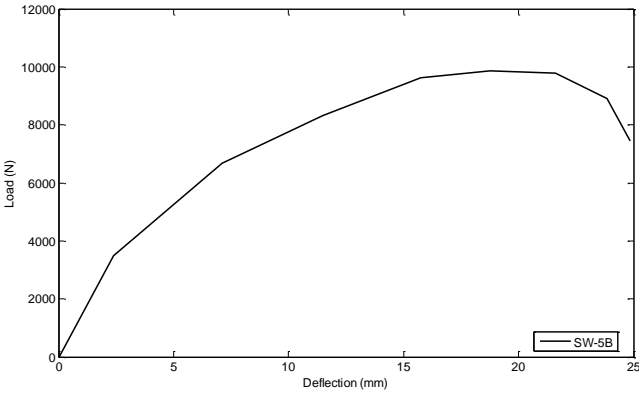
Specimen Number	SW-5B	Deflection Contributors
Sheathing Thickness (OSB)	11 mm	a) Bending
Nail Diameter	2.84 mm	b) Shear
Stud Size	38 x 89 mm	c) Hold-Down
Number of End Studs	2	
Hold-Down Type	ATS	



a)



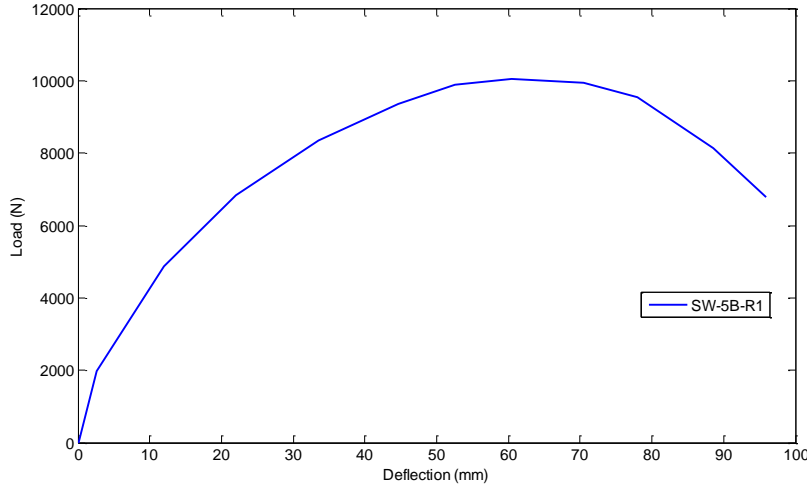
b)



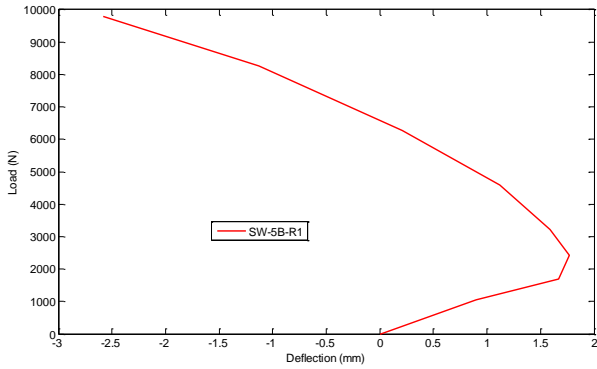
c)

Specimen Number	SW-5B-R1	EEEE Results	
Sheathing Thickness (OSB)	11mm	Peak Load (N)	10060.00
Nail Diameter	2.84 mm	Displacement at Peak (mm)	57.23
Stud Size	38 x89 mm	Initial Stiffness (N/mm)	440.00
Number of End Studs	2	Yield Load (N)	8902.55
Hold-Down Type	HDU2	Ductility Ratio	4.18
Aspect Ratio	2:1		

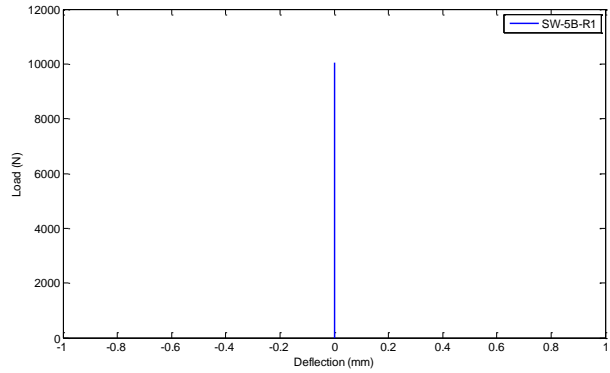
Total Behavior



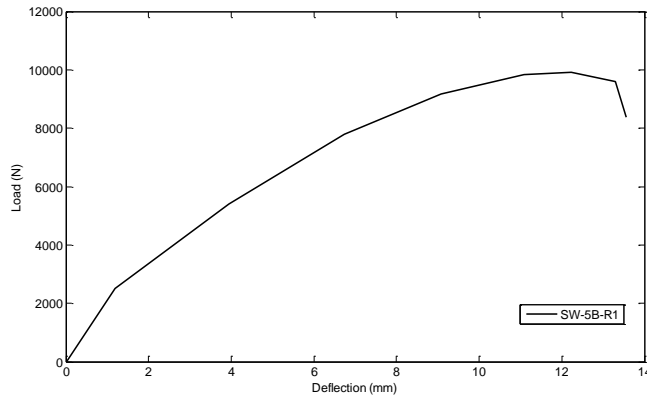
Specimen Number	SW-5B-R1	Deflection Contributors	
Sheathing Thickness (OSB)	11 mm	a)	Bending
Nail Diameter	2.84 mm	b)	Shear
Stud Size	38 x 89 mm	c)	Hold-Down
Number of End Studs	2		
Hold-Down Type	ATS		



a)



b)



c)