TESTING AND ANALYSIS OF MIDPLY™ SHEAR WALL SYSTEM

by

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Date March 29, 2000
Abstract

The thesis details a six-month period of experimental studies of the structural properties of the MIDPLY™ Wall System, which has being developed through a three-year testing program.

The MIDPLY™ shear wall was invented by Dr. Erol Varoglu, formerly with Forintek Canada Corp. and Prof. S.F. Stiemer (University of British Columbia), US Patent US5782054: Wood wall structure. The system provides greater lateral resistance to earthquakes and high winds loads. The name MIDPLY™ refers to its configuration: the plywood or OSB panel is placed between the studs. The improved performance is due to this special rearrangement of wall framing components and sheathing used in standard shear walls. The new fabrication system uses typical building methods and standard building materials, but a prefabricated production may be desirable to ensure the quality of the product.

The objective of the project is to establish the proof-of-concept for the MIDPLY™ Wall System, design and construction method. The structural properties are being developed through testing of full-scale specimens subjected to monotonic (ramp), cyclic and dynamic displacement schedules. Several anchoring techniques and hold-down connectors have been developed and tested. The tests were performed mainly on 2.44m x 2.44m and 1.22m x 2.44m MIDPLY™ shear walls.
Abstract

In the investigations covered by this thesis, were designed and tested 30 walls and were primarily studied:

- Alternate hold-down connections that prevent the fracture of the end studs of the MIDPLY™ wall by using MSR lumber or inverted-triangle hold-downs with different bolt spacing, bolt diameters, hold-down length and placement along the stud.

- Alternate hold-down connectors: steel rods and double shear hold-downs.

- The performance of MIDPLY™ walls under pseudo-dynamic tests.

- The performance of MIDPLY™ walls used as insert segments in standard shear walls.

- The performance of MIDPLY™ walls with openings. Special attention was given to the connection details between header and MIDPLY™ walls to achieve desired pin- or moment-connection.

- The performance of MIDPLY™ walls affected by shrinkage.

- The performance of repaired MIDPLY™ walls.

One can conclude that, there are many real-life applications of the system, using the superior structural performance of the MIDPLY™ walls. Thus, future studies will conduct to a final package of design values needed for structural engineers, architects and designers.
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*  

"The refining pot is for silver and the furnace for gold, but the Lord tests the hearts."  
(Proverbs Chapter 17 verses 1-3)  

*
1.1. **Shear Wall Overview**

1.1.1. **Definition**

Shear walls are vertical elements of an horizontal force resisting system. They are typically wood frame stud walls covered with a structural sheathing material like plywood. The components of a shear wall are framing members, sheathing, nails and hold-downs. When designed and constructed properly, shear walls provide the strength to resist horizontal earthquake and wind forces. They will transfer these horizontal forces to the next element in the load path below them, such as other shear walls, floors, foundation walls, slabs or footings. Shear walls also provide lateral stiffness to prevent the roof or floor above from excessive side-sway, consequently from nonstructural damage.
1.1.2. History

In the late 50's and early 60's, plywood walls were proposed simply as alternates to diagonally braced wall sections. Their acceptance for this purpose was based on their ability to meet certain load/deflection criteria. Wood frame wall construction without diagonal corner bracing demonstrated that the proposed construction provides resistance to racking stress at least equal to that provided by walls with horizontal wood board sheathing and let-in corner bracing.

Unlike the single material graph, this system depends on the strength of the material and the holding power of the fasteners. Based on this and other testing data and by incorporating knowledge gained from plywood diaphragm testing, the 1967 Uniform Building Code for the first time provided allowable load tables for plywood shear walls. Therefore, plywood diaphragm research preceded plywood shear wall research, many of the formulas used to determine shear wall strength were adapted from the earlier research.

Before the acceptance of performance-rated panels, plywood was the sheet product of choice for constructing wood shear walls. The panel thickness, panel grade, nail type and nail spacing could be combined in different ways to achieve a wall with the right design strength. With the advent of performance-rated products (like Oriented Strand Board) another variable was added: the strength of the panel. Manufacturers have the ability to adjust the proportion, size and type of wood fibers and the amount of bonding material to...
suit particular job needs. Gypboard and Gypsum Sheathing are also types of materials listed as shear-resisting materials, but their use is discouraged in earthquake regions. Gypboard and similar products are not so flexible as plywood or OSB. The shear forces are resisted, but the result of repetitive cycles of alternate direction loading is gypboard with larger and larger nail holes.

1.1.3. Experience

Hans Rainer and Erol Karacabeyli in "Performance of Wood-frame Building Construction in Earthquake" (1999) presented a detailed survey of wood-frame construction in a number of recent earthquakes. Much of the information in this section revisits the same material used by the authors. Evidence from structural failure in past earthquakes shows that shear walls offer excellent protection to buildings in seismic regions. Analysis of the performance of wood-frame construction, also reviews of the dominant factors that affect the seismic behavior of buildings have been done following a number of recent earthquakes: Alaska, 1964; San Fernando, California 1971; Edgecumbe, New Zealand 1987; Saguenay, Quebec 1988; Loma Prieta, California 1989; Northridge, California 1994; and Kobe, Japan 1995. Wherever possible, the behavior of buildings was related to the measured peak horizontal ground accelerations.

The survey shows that despite some specific deficiencies and consequent failures, a majority of wood-frame buildings of different ages, when subjected to peak ground accelerations of 0.6 g and greater, have survived the earthquakes without serious
Chapter 1 Introduction

structural damage or collapse and with very few resulting injuries and deaths. Thus, the
life-safety parameter that is implicit in building codes is sufficiently satisfied. In addition,
many modern wood-frame buildings survived the shaking without visible damage. The
few observed failures and collapses could be illustrated primarily as specific situations of
lack of lateral bracing, weak first storey, inadequate connection to foundations and the
fact that the observed ground motions exceeded the existing design requirements. Studies
on joints, shear walls, mathematical modeling and development of codes and standards
are subject to present and further research. Finally, for existing buildings that do not meet
current seismic standards, guidelines for evaluation and upgrading can be found in the
USA and Canada were wood-frame construction is widely represented. As a result of
generally positive performance in major earthquakes and particularly when compared to
other forms of house construction, the wood-frame construction is gaining acceptance in
other parts of the world, mainly in Japan.

Emphasizing the successes, we should not ignore in the same time the weaknesses that
have been observed. The recent survey of performance of wood-frame construction in
recent earthquakes in Alaska, California, Quebec, New Zealand and Japan shows that the
weak first story situation needs to be addressed in order to reduce potentially future
collapses, casualties and damages. The hope of good performances depends on
appropriate design, quality of work and good maintenance. Favorable past performance
should also not be taken to suggest that further improvements are no longer needed! In
order to ensure acceptable performance in the future, the design codes and construction practices need to be improved to reflect the new theoretical and applied research.

Improvement of seismic performance of wood-frame construction has been done consistently in the past years. Constructions with large openings in the exterior walls, large floor spans, heavy roofs or weak story became regular practice. In these situations, the regular shear walls can hardly withstand large earthquake forces and high winds. "Z-wall", "Strong-wall" and "Midply™ shear wall" are the latest innovations that try to cover the increase demand on the lateral load resistance of wood frame shear walls. The thesis tries to cover a section of the research that has been done in the project of so called "MIDPLY™ Wall System" at Forintek Canada Corp.

1.2. MIDPLY™ Wall System

Research on seismic behavior of wood-frame construction is an activity that includes, among others, issues of strength, durability, mathematical modeling and calculation of seismic response, field observations in earthquakes and laboratory testing. Investigations on these topics are carried out at a number of industrial, government and university laboratories in many parts of the world, including at Forintek Canada Corp. in Vancouver, British Columbia. Since the shear walls form the basis for the seismic resistance of both single and multistory buildings, and the demand of limited wall lengths for resisting seismic or wind forces increases, a new high-strength wall system,
MIDPLY™ Wall System, has been developed in collaboration with the University of British Columbia. The new wall system consists of wood shear wall components arranged in a way that the lateral load resistance of the system significantly exceeds that in the current standard wall arrangements.

MIDPLY™ wall system uses standard building methods as well as standard building materials. The superior lateral resistance is based on two main changes in the configuration of the standard shear wall:

- The panel is used at the centre of the wall to increase the lateral load carrying capacity without increasing the width of the wall. Nails fastening this panel to the studs work in double shear (or in triple shear with the addition of exterior sheathing), providing the increased lateral load capacity for the wall.

- The studs in the MIDPLY™ wall system are rotated at a 90° position relative to the longitudinal axis in standard stud walls. Thus, the sheathing material is fastened to the wide faces of studs (instead of the narrow face as in the standard walls). This feature increases the lateral load capacity of the MIDPLY™ wall by providing additional edge distance for fasteners on studs, panels placed at the mid-plane and also at the exterior face of the wall. This reduces the wood edge chip-out type failures. The heads of the nails are kept away from the surface of the mid-panel so that nail pull-through failure mode at the mid-ply is physically prevented.
Chapter 1  Introduction

More information and details in the construction and configuration of MIDPLY™ wall are given in Chapter 2.

1.3.  Objectives

1.3.1.  Project Objectives

The overall objectives of this three-year research project are to establish the proof-of-concept for the MIDPLY™ wall system design and construction method. The specific objectives are:

- To establish the most appropriate design and construction method and connection details for the MIDPLY™ wall system.
- To evaluate the earthquake performance of the MIDPLY™ wall system, and compare its performance to that of standard shear walls.
- To develop design guidelines for each application of the MIDPLY™ wall system.
- To develop guidelines for a quality assurance system for the manufacturing and construction of the MIDPLY™ wall system.
Chapter 1 Introduction

1.3.2. Thesis Objectives

The following studies were conducted in the period covered by this thesis:

- Alternate end studs lumber grades in MIDPLY™ shear walls: stud grade, 1650f MSR and 2400f MSR lumber.
- Alternate hold-down configurations: using “inverted-triangle” hold-downs with different bolt spacing, bolt diameters, hold-down length and placement along the stud.
- Alternate hold-down connections that prevent the fracture of the MIDPLY™ end studs: “double shear hold-downs” and steel rods.
- The performance of MIDPLY™ walls under pseudo-dynamic tests and comparison with the results obtained under shake table tests.
- The performance of MIDPLY™ walls used as insert segments in standard shear walls.
- The performance of MIDPLY™ walls with openings. Special attention was given to the details between header and MIDPLY™ walls to achieve desired pin- or moment-connections.
- The performance of MIDPLY™ walls affected by shrinkage.
- The performance of repaired MIDPLY™ walls.
1.4. *Methodology*

The objectives of the study covered in this thesis were achieved using the experience and test results from the previous two-year work in this project. The results were used to analyze new configurations of the MIDPLY™ walls and its constitutive elements and also to establish further tests that were performed and are part of this thesis. The test matrixes are largely explained in the next chapters. Monotonic and/or cyclic static tests were performed on each type of wall. Experimental results were then analyzed and compared.

1.1. *Thesis Outline*

The thesis has been organized into the following chapters:

Chapter 1: Introductory information about shear walls and project objectives.

Chapter 2: Background information of MIDPLY™ wall system and precursory work.

Chapter 3: Information about testing equipment, loading protocols and computer analysis

Chapter 4: Details about hold-down connections and fastenings.

Chapter 5: Behavior of MIDPLY™ walls under pseudo-dynamic tests.

Chapter 6: Behavior of MIDPLY™ walls used as insert segments.

Chapter 7: Behavior of MIDPLY™ walls with openings.
Chapter 8: Behavior of MIDPLY™ walls affected by shrinkage also the repaired MIDPLY™ walls.

Chapter 9: Summary and conclusions after the test results and computer analysis

Chapter 10: Bibliography

Chapter 11: Appendixes
CHAPTER 2

Background

The previous chapter confirmed the opportunity of MIDPLY™ Wall System and showed an overview of the geometry of the wall. For better understanding, the beginning of this chapter deals with the materials used for MIDPLY™ Wall System and with the construction of this innovative wood-based shear wall.

2.1. Materials used for MIDPLY™ Shear Wall

The framing members use different dimensions and grades of construction lumber. The dimension lumber is defined as surfaced softwood lumber of thickness from 38mm to 102mm. The MIDPLY™ Wall System uses 38mm x 64mm (2 x 3 inch nominal) or 38mm x 89mm (2 x 4 inch nominal) lumber. The Spruce-Pine-Fir is the species group making up the largest portion of dimension lumber in Canada. The species included in combination are Spruce (all species except Coast Sitka spruce), Jack pine, Lodgepole...
Chapter 2  Background

pine, Balsam fir and Alpine fir. Among the construction characteristics: works easily and holds nails well. Stud grade is frequently used for framing members and also Construction or Standard grades shipped as Standard and Better. Machine Stress Rating defines higher design properties and increased reliability. We take advantages of higher specified strength values and use them for the end studs. The most common grades of MSR lumber are 1650f-1.5E, 1800f-1.6E, 2100f-1.8E and 2400f-2.0E.

Plywood panels are made from sheets of softwood glued together with a waterproof adhesive. Typically the grain direction and thickness is balanced. Douglas Fir Plywood (DFP) and Canadian Softwood Plywood (CSP) are the most common softwood plywoods. Nominal panel size is 1220mm x 2440mm but sizes up to 1525mm x 3050mm are available. The most used nominal thicknesses are 7.5mm and 9.5mm.

Oriented Strand Board (OSB) is an engineered wood product made of thin strands or wafers, bonded with exterior-type binder under high temperature and pressure. The OSB is generally manufactured for engineered design applications in a range of dimensions from 1220mm x 2440mm which is the most common to 2440mm x 7320mm on special orders. The thickness varies also between 6mm and 28.5mm. The CSA Standard O452 contains certification and grade-marking requirement for three types of design-rated OSB panels: "Standard", "Plus" and "Proprietary".
Nails are manufactured in many different dimensions and types. The Wood Design Manual offer information only for common nails and spiral nails generally used in construction. Nails are defined commonly by the type and length in inches. The lengths are from 1/2 to 6 inches. The most used nails in our tests are the 3.05mm diameter power nails and 3.83mm common nails. The characteristics of these nails are found through the experimental tests.

2.2. Configuration of MIDPLY™ Shear Wall

Figure 1 illustrates the configuration of the MIDPLY™ Wall System in comparison to a standard shear wall.

![Diagram of standard and MIDPLY™ shear wall]

*Figure 2.1  Comparison of MIDPLY™ shear wall with standard shear wall*
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MIDPLY™ shear wall uses standard construction methods as well as standard construction materials as a regular shear wall: frame members, sheathing, fasteners and connectors.

The frame members are typically 38mm x 64mm (2 x 3 nominal) or 38mm x 89mm (2 x 4 nominal) using Spruce-Pine-Fir (SPF) lumber of different grades. Standard-and-better grade is used for top and bottom plates, stud grade lumber for interior and buckling studs, and stud grade, 1650f MSR or 2400f MSR lumber for the end studs depending of the type of hold-downs that is used. All the framing members are placed at a 90° rotated position relative to the longitudinal axis of the wall. The timber studs are typically spaced approximately 406mm (16 inch) or 610mm (24 inch) on center. The stiffness of the frame is relatively small compared to the stiffness of the sheathing.

The middle-sheathing panel (12.5mm) consists generally of 1.2m x 2.4m plywood or oriented strand board (OSB). The panel is used at the center of the wall to increase the lateral load capacity and to keep the same width of the wall. The exterior panel for double sheathed MIDPLY™ Wall System uses Canadian Softwood Plywood (CSP) of 9.5mm thickness. Also, gypsum board or drywall can be connected on the exterior side of the wall.

The panel and the frame are connected with (L = 82mm or 89mm, D = 3.05mm) power nails at 100mm on center. The nails work in double shear or in triple shear when an
additional sheathing is placed on the exterior of the wall. This special configuration of MIDPLY™ Shear Wall provides the increased lateral load capacity for this wall compared with the regular shear wall. When the wall is subjected to lateral in-plane loads, the fasteners work together with the rigid panels and the relatively flexible frame. The deformation of the walls from a rectangular shape to a parallelogram is known as "raking". The fasteners are very important components determining the shear wall stiffness and strength. As a result of the way the panel is fastened (to the wide faces of studs) the MIDPLY™ Shear Wall provides additional edge distance for the nails on the studs, as well as at the panel perimeter. This special configuration reduces the wood edge chip-out type failures and completely eliminates the nail pull-through failure. All wood frame shear walls are highly redundant systems. They are very simple constructions but complex to model and analyze. The reason is the nonlinear load-slip characteristic of the fasteners. The nails redistribute the loads and dissipate the energy as they deform contributing to the shear wall resistance to earthquake ground motion.

The end studs of the wall must be anchored to the foundation to resist uplift forces. The most used connectors were the invert-triangle hold-down studied in the second year of the project and the steel-rod hold-down. The behavior of the MIDPLY™ Shear Wall using steel rods as hold-downs was studied in the recently tests and the results are largely presented in this thesis. There are advantages and disadvantages for both type of hold-downs but the steel rod seems to work better in preventing the brittle failure of the end studs and increasing considerably the load carrying capacity of the wall.
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The bottom plate must also be anchored to the foundation with anchor bolts to resist the shear force along the length of the wall.

2.3.  Dimensional Properties

The overall dimensions of the shear wall influence its performance. The length of the wall is proportional with the stiffness and the load carrying capacity. Also double exterior sheathing approximately doubles the capacity of the wall. The vertical load usually helps by reducing the overturning moment and increasing the ultimate load capacity. This effect is visible mostly at short shear walls and decreases as the length of the wall enlarges. The length of the wall also influences the value of the hold-down force that opposes to racking.

The openings also reduce the capacity of a shear wall leaving room for localized failures. Present tests at Forintek Canada Corp. have been performed and are part of this thesis that explain the behavior of shear walls with openings. The stud spacing also affect the capacity of the wall but has not much contribution for the stiffness of the wall (De Klerk, 1985). The tests were done on the regular shear walls and also on the special configured MIDPLY™ walls at Forintek. The influence of the above parameters was observed in both types of shear walls. Attention to all of these variables makes the modeling of shear walls very difficult and necessitates complex analysis methods and more data from full scale walls tests.
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2.4. Precursory Work in the MIDPLY™ Wall System Project

An important volume of research has been done over the first two years in the MIDPLY™ Wall System. A summary of precursory tests, wall configurations and tests' results of MIDPLY™ walls is shown in the following paragraphs. The test program can be divided in three major sections: shake table tests, testing of MIDPLY™ walls made with Finger-Joined Lumber and MIDPLY™ walls with pre-loading history.

2.4.1. Shake Table Tests of MIDPLY™ Walls

Shake table tests of MIDPLY™ walls were performed at the Earthquake Engineering and Structural Dynamics Laboratory at the Civil Engineering Department of the University of British Columbia. For MIDPLY™ tests, the earthquake excitation was applied only in the horizontal plane. Because the frame accommodating the MIDPLY™ wall was pinned on all four supports, the wall connected to the frame provided all the racking resistance against the inertial forces due to the acceleration of the system. Three MIDPLY™ wall configurations were tested on the UBC shake table. For each wall configuration, two identical walls were built; one tested using the Kobe earthquake record, and the other using the Landers earthquake record. Table 2.1 lists detailed information of MIDPLY™ walls under shake table tests.
### Table 2.1 MIDPLY™ walls’ configuration for the shake table tests.

<table>
<thead>
<tr>
<th>Wall No.</th>
<th>Stud Spacing (mm)</th>
<th>Stud and Plate Size (mm)</th>
<th>Exterior Sheathing (mm)</th>
<th>Vertical Loads</th>
<th>Earthquake Record and scaling factors</th>
</tr>
</thead>
<tbody>
<tr>
<td>M48-01</td>
<td>610</td>
<td>38 x 64</td>
<td>-</td>
<td>-</td>
<td>Kobe earthquake, Kobe Observatory</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>N-S. 0.35g, 0.54g, 0.67g</td>
</tr>
<tr>
<td>M48-02</td>
<td>610</td>
<td>38 x 64</td>
<td>-</td>
<td>-</td>
<td>Landers Earthquake, Joshua Tree, CA, E-W. 0.35g, 0.52g, 0.54g</td>
</tr>
<tr>
<td>M49-01</td>
<td>406</td>
<td>38 x 89</td>
<td>9.5 D.Fir vertical</td>
<td>-</td>
<td>Kobe earthquake, Kobe Observatory</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>N-S. 0.35g, 0.54g, 0.67g</td>
</tr>
<tr>
<td>M49-02</td>
<td>406</td>
<td>38 x 89</td>
<td>9.5 D.Fir vertical</td>
<td>-</td>
<td>Landers Earthquake, Joshua Tree, CA, E-W. 0.35g, 0.52g, 0.54g</td>
</tr>
<tr>
<td>M50-01</td>
<td>406</td>
<td>38 x 89</td>
<td>-</td>
<td>27 kN</td>
<td>Kobe earthquake, Kobe Observatory</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>N-S. 0.35g, 0.54g, 0.67g</td>
</tr>
<tr>
<td>M50-02</td>
<td>406</td>
<td>38 x 89</td>
<td>-</td>
<td>27 kN</td>
<td>Landers Earthquake, Joshua Tree, CA, E-W. 0.35g, 0.52g, 0.54g</td>
</tr>
</tbody>
</table>

Note:

All the walls, 2.44m in length and height, had inverted-triangle hold-down connections (Figure 2.2) on the end-studs and "through-plate" anchor bolt connections on the top and bottom plates.

![Inverted-triangle hold-down](image)

**Figure 2.2 Inverted-triangle hold-down connection.**
The Figure 2.2 was taken from the Second Year Report of MIDPLY™ Wall System.

Table 2.2 gives a summary of the most important parameters obtained in the shake table tests.

**Table 2.2 Results of the MIDPLY™ walls under shake table tests.**

<table>
<thead>
<tr>
<th>Wall No.</th>
<th>Earthquake Record &amp; PGA scaling</th>
<th>Max Load (kN/m) (+ / -)</th>
<th>Max Displ. (mm) (+ / -)</th>
<th>Energy Dissipated (kN/m x mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>M48-01</td>
<td>Kobe 0.35 g</td>
<td>20.3 / -19.3</td>
<td>23 / -21</td>
<td>2459</td>
</tr>
<tr>
<td>M48-01</td>
<td>Kobe 0.52 g</td>
<td>26.1 / -21.4</td>
<td>61 / -37</td>
<td>2225</td>
</tr>
<tr>
<td>M48-02</td>
<td>Landers 0.35 g</td>
<td>12.1 / -12.9</td>
<td>11 / -12</td>
<td>2488</td>
</tr>
<tr>
<td>M48-02</td>
<td>Landers 0.52 g</td>
<td>21.5 / -21.5</td>
<td>44 / -45</td>
<td>8443</td>
</tr>
<tr>
<td>M48-02</td>
<td>Landers 0.54 g</td>
<td>22.8 / -22.0</td>
<td>99 / -58</td>
<td>6643</td>
</tr>
<tr>
<td>M49-01</td>
<td>Kobe 0.35 g</td>
<td>21.2 / -20.5</td>
<td>14 / -17</td>
<td>2139</td>
</tr>
<tr>
<td>M49-01</td>
<td>Kobe 0.52 g</td>
<td>31.3 / -26.8</td>
<td>29 / -24</td>
<td>5951</td>
</tr>
<tr>
<td>M49-01</td>
<td>Kobe 0.67 g</td>
<td>34.0 / -31.2</td>
<td>97 / -63</td>
<td>7500</td>
</tr>
<tr>
<td>M49-02</td>
<td>Landers 0.35 g</td>
<td>14.4 / -18.9</td>
<td>9 / -11</td>
<td>3598</td>
</tr>
<tr>
<td>M49-02</td>
<td>Landers 0.52 g</td>
<td>19.1 / -20.7</td>
<td>13 / -16</td>
<td>6254</td>
</tr>
<tr>
<td>M49-02</td>
<td>Landers 0.54 g</td>
<td>19.8 / -23.8</td>
<td>15 / -19</td>
<td>6426</td>
</tr>
<tr>
<td>M49-02</td>
<td>Kobe 0.67 g</td>
<td>38.0 / -33.0</td>
<td>53 / -77</td>
<td>9918</td>
</tr>
<tr>
<td>M50-01</td>
<td>Kobe 0.35 g</td>
<td>16.2 / -15.5</td>
<td>21 / -18</td>
<td>2231</td>
</tr>
<tr>
<td>M50-01</td>
<td>Kobe 0.52 g</td>
<td>27.9 / -26.7</td>
<td>54 / -56</td>
<td>8889</td>
</tr>
<tr>
<td>M50-01</td>
<td>Kobe 0.67 g (1st)</td>
<td>31.9 / -28.8</td>
<td>105 / -99</td>
<td>11016</td>
</tr>
<tr>
<td>M50-01</td>
<td>Kobe 0.67 g (2nd)</td>
<td>22.7 / -22.3</td>
<td>146 / -110</td>
<td>7754</td>
</tr>
<tr>
<td>M50-02</td>
<td>Landers 0.35 g</td>
<td>15.8 / -15.5</td>
<td>9 / -13</td>
<td>3665</td>
</tr>
<tr>
<td>M50-02</td>
<td>Landers 0.52 g</td>
<td>18.8 / -18.1</td>
<td>17 / -18</td>
<td>5783</td>
</tr>
<tr>
<td>M50-02</td>
<td>Landers 0.54 g</td>
<td>21.8 / -17.6</td>
<td>21 / -18</td>
<td>5918</td>
</tr>
<tr>
<td>M50-02</td>
<td>Kobe 0.67 g</td>
<td>30.8 / -23.1</td>
<td>105 / -49</td>
<td>7189</td>
</tr>
</tbody>
</table>
The wall M48, representing the weakest wall within the three wall configurations, showed no sign of damage at peak ground acceleration (PGA) 0.35g under both Kobe and Landers earthquakes, but failed at 0.52g of Kobe and 0.54g of Landers earthquakes, respectively. The walls M49 and M50, representing the medium and the strongest walls within the three wall configurations, showed no visual damage at PGA 0.35g, and also at the following 0.52g under both Kobe and Landers earthquakes. The two specimens of M49 failed at 0.67g of Kobe earthquake. The two specimens of M50 survived 0.67g of Kobe and 0.54g of Landers earthquakes, the largest amplitudes that the shake table could apply.

As a conclusion, in the most of the tests the walls failed in tension at the top bolt of the end-stud showing that the hold-down or the grade of the lumber of the end studs needed to be improved. Also, audible cracks of OSB rupturing at the corners of the wall were noticed in the 0.52g tests. Due to the application of vertical loads, the walls experienced excellent ductility and well-distributed failure of nails along the perimeter of the panels.

2.4.2. MIDPLY™ Walls Made by Finger-Joined Studs

Finger-joined studs have been used in the standard shear walls. Because the MIDPLY™ walls have over two times the unit load capacity of standard shear walls, the applicability of using finger-joined studs needed to be verified. A summary of the configuration of the walls with finger-joined lumber that have been tested is shown in Table 2.3.
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Table 2.3  MIDPLY™ walls with finger-joined lumber.

<table>
<thead>
<tr>
<th>Wall number</th>
<th>Load protocol</th>
<th>Studs, top and bottom plates</th>
<th>Stud spacing</th>
<th>End stud F.J. Grade</th>
<th>Vertical Loads</th>
</tr>
</thead>
<tbody>
<tr>
<td>M41-01</td>
<td>Monotonic</td>
<td>38mm x 89mm</td>
<td>610 mm</td>
<td>SPS 3</td>
<td>2 x 22.24 kN</td>
</tr>
<tr>
<td>M42-01</td>
<td>Monotonic</td>
<td>38mm x 64mm</td>
<td>406 mm</td>
<td>SPS 3</td>
<td></td>
</tr>
<tr>
<td>M42-02</td>
<td>ISO 97</td>
<td>38mm x 64mm</td>
<td>406 mm</td>
<td>SPS 3</td>
<td></td>
</tr>
<tr>
<td>M43-01b</td>
<td>Monotonic</td>
<td>38mm x 64mm</td>
<td>610 mm</td>
<td>SPS 3</td>
<td></td>
</tr>
<tr>
<td>M44-01</td>
<td>ISO 97</td>
<td>38mm x 64mm</td>
<td>610 mm</td>
<td>SPS 1</td>
<td></td>
</tr>
<tr>
<td>M45-01</td>
<td>ISO 97</td>
<td>38mm x 89mm</td>
<td>610 mm</td>
<td>SPS 1</td>
<td>2 x 22.24 kN</td>
</tr>
</tbody>
</table>

The test results of MIDPLY™ walls made of finger-joined lumber are summarized in Table 2.4. The test results of MIDPLY™ walls M40-01b and M14-01 were also included in order to compare the performance of the walls made of finger-joined and regular lumber. The two types of finger-joined material used for these walls are: SPS 1, which is a structural lumber, and SPS 3, which is a lumber used just vertically for studs only.

Table 2.4  Performance of MIDPLY™ shear walls made with finger-joined studs

<table>
<thead>
<tr>
<th>Wall Number</th>
<th>Load Protocol</th>
<th>End Stud Type</th>
<th>$P_{\text{max}}^{(1)}$ (kN/m)</th>
<th>$\delta_u^{(2)}$ (mm)</th>
<th>$K_i^{(3)}$ (kN/m/mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>M41-01</td>
<td>ramp</td>
<td>SPS 3</td>
<td>29.5</td>
<td>125</td>
<td>1.82</td>
</tr>
<tr>
<td>M42-01</td>
<td>ramp</td>
<td>SPS 3</td>
<td>24.7</td>
<td>87.2</td>
<td>1.41</td>
</tr>
<tr>
<td>M42-02</td>
<td>Cyclic - ISO 97</td>
<td>SPS 3</td>
<td>25.4</td>
<td>70.2</td>
<td>1.66</td>
</tr>
<tr>
<td>M43-01b</td>
<td>ramp</td>
<td>SPS 3</td>
<td>26.1</td>
<td>108.6</td>
<td>1.61</td>
</tr>
<tr>
<td>M44-01</td>
<td>Cyclic - ISO 97</td>
<td>SPS 1</td>
<td>26.0</td>
<td>61.0</td>
<td>1.63</td>
</tr>
<tr>
<td>M45-01</td>
<td>Cyclic - ISO 97</td>
<td>SPS 1</td>
<td>31.7</td>
<td>57.0</td>
<td>2.17</td>
</tr>
<tr>
<td>M14-01</td>
<td>Cyclic - ISO 97</td>
<td>standard</td>
<td>28.9</td>
<td>70.3</td>
<td>2.02</td>
</tr>
<tr>
<td>M40-01b</td>
<td>ramp</td>
<td>standard</td>
<td>33.2</td>
<td>111</td>
<td>1.63</td>
</tr>
</tbody>
</table>

1 $P_{\text{max}}$ for the cyclic tests refers to the maximum load of the envelope curve.
2 $\delta_u$ refers to the displacement of the wall at 80% of maximum load capacity in the descending part of the load-displacement curve (where applicable).
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3  $K_i$ refers to the secant stiffness of the wall and is measured between displacements at 10% and 40% of maximum load.

It was found that the values of secant stiffness were similar for different finger-joined wall configurations and similar to walls made with standard lumber. This indicates that there is no significant difference between the stiffness of walls made with finger-joined lumber SPS 1 and SPS 3, grade lumber and regular lumber. In general, the maximum lateral load capacity was slightly reduced in most cases when finger-joined lumber was used for the end-studs of MIDPLY™ walls. This was due to the fact that finger-joined lumber is not fabricated to withstand the large bending moments and tension forces at the end studs of the MIDPLY™ walls. The finger-joined lumber behaved well as plates and interior studs.

2.4.3.  MIDPLY™ Walls with Pre-Loading History

Structures may be subjected to several moderate earthquakes before a major earthquake strikes them, therefore it is important to study the performance of MIDPLY™ walls with pre-loading history. For this purpose, the walls M46-01 and M47-02 were subjected to five consecutive ISO98 load protocols at different maximum displacement levels.

The detailed information of the tested MIDPLY™ walls was listed in Table 2.5.
### Table 2.5 MIDPLY™ walls with pre-loading history

<table>
<thead>
<tr>
<th>Wall number</th>
<th>Load protocol</th>
<th>Vertical load (kN)</th>
<th>Plate</th>
<th>Stud</th>
<th>Stud spacing (mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>M46-01</td>
<td>ISO98</td>
<td>-</td>
<td>38 x 89</td>
<td>38 x 89</td>
<td>406</td>
</tr>
<tr>
<td>M47-02</td>
<td>ISO98</td>
<td>-</td>
<td>38 x 64</td>
<td>38 x 64</td>
<td>610</td>
</tr>
<tr>
<td>M48-03</td>
<td>Kobe -0.35g, 0.52g, ramp</td>
<td>-</td>
<td>38 x 64</td>
<td>38 x 64</td>
<td>610</td>
</tr>
<tr>
<td>M50-03</td>
<td>Kobe -0.35g, 0.52g, 0.67g, ramp</td>
<td>27 kN</td>
<td>38 x 89</td>
<td>38 x 89</td>
<td>406</td>
</tr>
</tbody>
</table>

1. Five consecutive ISO98 load protocols at maximum displacements of 6.4mm, 12.7mm, 25.4mm, 50.8 mm and 125mm.

2. Displacement response of the wall M48-01 under ground acceleration of Kobe earthquake, N-S.

3. Displacement response of the wall M51-01 under ground acceleration of Kobe earthquake, N-S.

All walls used inverted-triangle hold-downs.

The test results of MIDPLY™ walls with pre-loading history were summarized in Table 2.6. Test results of respective wall configurations under monotonic tests (M32-01 and M40-01b) and cyclic tests (M37-01 and M44-01) were also included for comparison.

### Table 2.6 Test results of MIDPLY™ walls with pre-loading history

<table>
<thead>
<tr>
<th>Wall Number</th>
<th>Load protocol</th>
<th>Vertical load (kN)</th>
<th>Plate</th>
<th>Stud</th>
<th>Stud spacing (mm)</th>
<th>P&lt;sub&gt;max&lt;/sub&gt; (kN/m)</th>
<th>δ&lt;sub&gt;u&lt;/sub&gt; (mm)</th>
<th>K (kN/m/mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>M32-01</td>
<td>Ramp</td>
<td>44.48</td>
<td>38 x 89</td>
<td>38 x 89</td>
<td>406</td>
<td>36.3</td>
<td>125</td>
<td>1.58</td>
</tr>
<tr>
<td>M37-01</td>
<td>ISO97</td>
<td>44.48</td>
<td>38 x 64</td>
<td>38 x 64</td>
<td>406</td>
<td>32.1</td>
<td>98</td>
<td>2.35</td>
</tr>
<tr>
<td>M46-01&lt;sup&gt;3&lt;/sup&gt;</td>
<td>ISO98</td>
<td>-</td>
<td>38 x 89</td>
<td>38 x 89</td>
<td>406</td>
<td>28.0</td>
<td>101</td>
<td>0.62</td>
</tr>
<tr>
<td>M40-01b</td>
<td>Ramp</td>
<td>44.48</td>
<td>38 x 64</td>
<td>38 x 64</td>
<td>610</td>
<td>33.2</td>
<td>125</td>
<td>3.98</td>
</tr>
<tr>
<td>M44-01</td>
<td>ISO97</td>
<td>-</td>
<td>38 x 64&lt;sup&gt;1&lt;/sup&gt;</td>
<td>38 x 64&lt;sup&gt;2&lt;/sup&gt;</td>
<td>610</td>
<td>26.0</td>
<td>61</td>
<td>3.97</td>
</tr>
<tr>
<td>M47-02&lt;sup&gt;3&lt;/sup&gt;</td>
<td>ISO98</td>
<td>-</td>
<td>38 x 64</td>
<td>38 x 64</td>
<td>610</td>
<td>25.6</td>
<td>80</td>
<td>0.51</td>
</tr>
</tbody>
</table>

<sup>1</sup> Five consecutive ISO98 load protocols at maximum displacements of 6.4mm, 12.7mm, 25.4mm, 50.8 mm and 125mm.

<sup>2</sup> Displacement response of the wall M48-01 under ground acceleration of Kobe earthquake, N-S.

<sup>3</sup> Displacement response of the wall M51-01 under ground acceleration of Kobe earthquake, N-S.

All walls used inverted-triangle hold-downs.
Chapter 2 Background

1. SPS 1 finger-joined lumber
2. SPS 3 finger-joined lumber
3. Response of ISO98 with maximum displacement of 125 mm after experienced four previous testing

From the tests, it was observed that walls with previous loading history still provided comparable ductility and ultimate lateral load capacities with walls subjected to only one cyclic displacement schedule.

As a general conclusion after the tests performed in the first two years it was noticed that the inverted-triangle hold-downs could not prevent end stud failure at extremely high load conditions. Further studies on hold-downs were needed to eliminate the fracture of the end studs of the MIDPLY™ walls. Possible solutions include using higher grade MSR lumber for end studs, lift hold-down to further reduce moment effect, or new hold-downs such as steel rods. These new configurations were studied in the third year and are object of this thesis.

The tests performed on the third year on MIDPLY™ Wall System project and further described can be divided in four categories:

- MIDPLY™ Wall System in pseudo-dynamic tests
- MIDPLY™ walls as insert segments
- MIDPLY™ walls with openings
- Behavior of MIDPLY™ walls affected by shrinkage and repaired MIDPLY™ walls
CHAPTER 3

Monotonic and Static-Cyclic Testing

3.1. Testing Equipment

The section describes the setup used for monotonic and static-cyclic testing. The schematic diagram of the wall test setup is shown in Figure 3.1.

![Figure 3.1 Test set-up for MIDPLY™ shear wall (a)]
Chapter 3  Monotonic and Static-Cyclic Testing

The test apparatus was configured such that the fixed edge of the wall is at the bottom while the lateral load-spreader is located at the top. The shear wall was mounted vertically between the base beam (I section steel) and the spreader bar. The wall was bolted to each beam with 12.7mm diameter class 5 steel bolts at a spacing of approximately 400mm (the bolts were positioned and staggered between the studs). The base I section beam was also bolted to the laboratory reinforced concrete floor, preventing the translation of the bottom of the shear wall. The load distribution beam can be replaced with other different length beams to accommodate walls of different lengths. The out of plane movement of the top bar is controlled by two pairs of supports. Figure 3.2 shows a photo of a 1.22m MIDPLY™ shear wall in the test apparatus and Figure 3.3 the detail A of transducer T4.

The monotonic (ramp) and cyclic displacements are applied along the top plate of the wall through the load spreader bar that is bolted to the top plates and attached to the servo-controlled hydraulic actuator. A concentrated horizontal load is applied at the right end of the spreader bar through the actuator.

Figure 3.2  Test set-up for MIDPLY™ shear wall (b)
Chapter 3  

Monotonic and Static-Cyclic Testing

The load spreader avoids the concentrated load and simulates the transfer of the loads from a floor diaphragm to the shear wall. Reversed loads may also be applied with the load spreader. The load spreader bar also distributes evenly the vertical load applied to the wall through four other hydraulic jacks, simulating the vertical dead loads.

The lateral load is applied through the displacement controlled actuator at a rate that is function of the monotonic (ramp) or static-cyclic load protocols. Information from the test is collected by a personal computer with six channels data acquisition software. The stroke transducer in the actuator measures the load and the horizontal displacement of the top plate, and other five displacement transducers (Figure 3.3) located at the wall perimeter record the following data:

- The horizontal displacement of the top end of the wall opposite to the actuator
- The uplift displacement at the bottom corners of the wall

It was found that the displacements from the stroke transducer end from the left top of the wall are very close in values to each other.

Figure 3.3  Det A – Displacement transducer
3.2. Loading Protocols

The section will describe the load protocols used in the series of tests detailed in this thesis. The loading protocols performed on the MIDPLY™ Wall System tests and further illustrated can be divided in the following categories: loading protocols for pseudo-dynamic tests, monotonic (ramp) and static-cyclic tests.

3.2.1. Loading Protocols for Pseudo-Dynamic Tests

Shake table tests can simulate the response of the shear walls under seismic loads reproducing earthquake ground motions. Inertial forces are applied to walls under shake table tests. The response of the walls depends on the dynamic properties of the walls and the characteristics of ground motions. Because the high cost in conducting such tests, reversed cyclic or pseudo-dynamic tests are often used as an alternative to obtain the dynamic properties of the walls.

The Figure 3.4 displays the Kobe acceleration time history. This earthquake record was used because of its destructiveness on light wood frame buildings. The Kobe record, as can be seen, has a few high peaks of acceleration spaced closely together. The lowest two acceleration amplitudes of 0.35g and 0.52g were chosen based on comparison purposes. The 0.67g acceleration amplitude for the Kobe earthquake was chosen based on displacement limits of the shake table facility.
Chapter 3  Monotonic and Static-Cyclic Testing

Figure 3.4  The Kobe acceleration time history

The earthquake record was used by Chun Ni, researcher at Forintek Canada Corp., to develop the load protocols used in the pseudo-dynamic tests. A constant displacement rate of 20mm/sec (0.8 inch/sec) was used in all pseudo-dynamic tests. The same rate was used for fast tension and compression tests. Fast ramp tests were conducted following pseudo-dynamic tests to evaluate the residual strength of the MIDPLY™ walls. The Figures 3.5 and Figure 3.6 display these load protocols.
Figure 3.5  Load protocols for pseudo-dynamic testing of MIDPLY™ wall M48-03.

Figure 3.6  Load protocols for pseudo-dynamic testing of MIDPLY™ wall M50-03.
3.2.2. Loading Protocols for Monotonic and Static-Cyclic Tests

The monotonic pushover load protocols were used in wood shear walls testing to determine the raking resistance values and to measure the static shear capacity of the walls. The most commonly used protocols were published by the American Society of Testing Materials Standards ASTM 1991a and ASTM 91b. The loads are being applied at a moderate rate so the material strain rates do not affect the test results. Among the advantages of these tests, we can mention: easier to perform full size specimens, and the failures can be easily observed. Monotonic tests have been performed mainly to study the behavior of hold-downs, anchor bolts and fasteners. For our monotonic tests was used a displacement at a constant rate of 7.62mm/min. A disadvantage of these tests is that they do not provide sufficient information for earthquake applications.

The cyclic tests are preferred to evaluate the behavior of the shear walls. Generally there is not an internationally agreement for the displacement history or for analyzing the test results. During the two year period of MIDPLY™ Wall System tests were used the ISO 97 and ISO 98 reversed-cyclic loading protocols. The ISO 98 protocol was proposed in the working draft of the ISO Standard “Timber structures – Joints made with mechanism fasteners – Quasi-static reversed-cyclic test method” and it was chosen in the tests presented in this thesis. The ISO 98 load protocol is shown in Figure 3.7. The displacement rate was 20mm/sec.
3.3. Parameters

This section deals with the definition of the parameters used in the test results tables in the next chapters. These parameters were used to better describe the test results and to make easier the interpretation of the test data. They were obtained through the computer calculations using the output data from the testing apparatus where the transducers have given us the primary values.

3.3.1. Energy dissipation ($E_d$)

Under monotonic or cyclic loading, the energy dissipation is function of internal friction, yielding failure of the nails and permanent deformation of the shear wall assembling. The energy dissipation was obtained by calculating approximately the area of the hysteresis loops from the load-displacement curves for each load cycle. Next is shown the

![Graph showing load protocol ISO 98](image)
procedure of computing the energy dissipation in the MIDPLY™ Wall System reports and consequently in this thesis, using a MathCad program:

**Energy Dissipation Computation**

From N.K. Allotey  
Modified by Dan Lungu

**Trapezoidal method**

**Procedure**

a) Develop a "*.txt" file from the "*.asc" file. The "*.asc" file is the primary output file coming from the test. It contains the load and the displacement measured during the test by transducers. The file must have displacement on first column and load on the second column.

b) Have the "*.txt" file and the MathCad file in the same directory.

c) Read and compute the energy for one file.

```
Energy := for k ∈ 1
            data ← READPRN("*.txt")
            Δ ← data<1>
            F ← data<2>
            n ← last(Δ)
            energydissp ← \frac{1}{2}\sum_{i=1}^{n-1} [\sum_{i=1}^{n-1} \left(\Delta_{i+1} - \Delta_i\right) \left(F_{i+1} + F_i\right) ]
            Dissip_k ← energydissp
            Dissip
```

d) View the Results

```
Energy =
```
Chapter 3  Monotonic and Static-Cyclic Testing

The value of the energy dissipation is collected directly in "kN x mm"

3.3.2. Stiffness (K)

The stiffness is defined as the ratio between the load and the deformation. In the case of wood based shear walls the stiffness is variable due to the continuing failure of the nails or unrecoverable deformation of the frame or sheathing during a static or dynamic motion. In our case, K represents the secant stiffness of the wall measured in a range of displacements corresponding to 10% and 40% of the maximum load. The values of the displacements were obtained using the program "m3Dis" written at Forintek Canada Corp. The program selects the displacements at various load levels for monotonic and cyclic tests. The values from all the tests are shown in the Appendix A. Those values were used to compute the stiffness K from the test results tables. See example Figure 3.8.

![Figure 3.8  Secant stiffness calculation](image)
Chapter 3  Monotonic and Static-Cyclic Testing

For the cyclic tests, were used the average values of loads and displacements from the tension side and compression side of the graphs. Always were used the values from the first cycle envelope.

3.3.3. Ultimate Displacement ($\delta_u$)

In the test results tables shown in the next chapters, we named the ultimate displacement ($\delta_u$) as the displacement of the wall at 80% of the maximum load capacity in the negative slope of the load-displacement graph. We considered that as the maximum allowable value of the load still carried by the wood shear wall system. The ($\delta_u$) values are also computed using the "m3Dis" program and are collected from the Appendix A.

Sometime during the final part of the tests the 80% of the maximum load in the descending portion of the envelope curve were not reached. This is due to the limitation of the test apparatus that is capable of maximum of 125mm displacement. This fact is specified in a note under each test result table on the next chapters.
CHAPTER 4

Hold-Down Connections

4.1. Overview

The end studs in a shear wall must be designed to resist the factored compressive and tensile axial forces. The axial forces in the end studs are generated by the horizontal forces applied at the top of the walls through the diaphragms (earthquakes, winds) and also by the vertical forces (dead loads and live loads). The axial force in the end stud can be calculated as follows (see Figure 4.1):

\[ T_f = V + \frac{v_f L w_H}{h} \]

where:
- \( T_f \) = maximum factored tensile force (kN)
- \( V \) = factored vertical load resultant (kN)
- \( v_f \) = factored shear force per unit length (kN/m)
Chapter 4  Hold-Down Connections

\[ L_w = \text{length of shear wall parallel to shear forces} \quad (\text{m}) \]

\[ H = \text{height of the wall} \quad (\text{m}) \]

\[ h = \text{distance center to center between the end studs} \quad (\text{m}) \]

**Figure 4.1  In plane forces acting on the shear wall**

The end studs members have to resist to axial compression forces without buckling or major baring failures. Also, the end studs of the shear walls have to be anchored to the foundation to resist uplift forces. This is done using the hold-downs.

### 4.2.  Precursory Research

During the research that has been done at Forintek Canada Corp. on the MIDPLY™ Wall System, several types of hold-downs have been studied.
4.2.1. Regular Hold-Down

A regular hold-down was used in the first 4.88 and 2.44m MIDPLY™ wall tests as shown in Figure 4.2.

![Figure 4.2 Regular hold-down used in the MIDPLY™ Wall System](image)

**Figure 4.2 Regular hold-down used in the MIDPLY™ Wall System**

The reports from the first year of MIDPLY™ Wall System project show constant end stud failures at the hold-down connection "particularly when no vertical loads were applied. In addition to the very high tensile forces generated due to high capacity of the MIDPLY™ Wall System, a severe bending moment, which adversely affected the performance of the end studs, was also generated on the end studs around relatively stiff hold-down connections".

4.2.2. Strap-tie Connection

The strap-tie hold-down can be installed on the end face or side face of the end stud. Among the disadvantages, were found:
4.2.3. Inverted-Triangle Hold-Down

The inverted-triangle hold-down (Figure 4.4) allows the end stud to rotate reducing the bending moment in the stud due to the deformation of the wall. The ultimate load capacity of the MIDPLY\textsuperscript{TM} wall was increased using this type of hold-down, also the ductility was better. This type of hold-down was used in the entire period of the second year of tests. Despite the improvements made in the parameters of the MIDPLY\textsuperscript{TM} Wall System, the new in that time hold-down has continued the produce the failure of the end studs.
4.3. **Alternative Configurations for Inverted-Triangle Hold-Down**

In the third year of the project were analyzed and redesigned different configurations for the inverted-triangle hold-down. The hold-down was redesigned for:

- different bolt spacing
- different bolt diameters
- different number of bolts used to connect the hold-down
- different grades used for the end studs

The hold-down was redesigned using the CSA 086.1-94 design code. A spreadsheet in MathCad was elaborate to design the hold-down connection. The computation is shown below and it was used to develop the comparison tables. The given example presents the...
inverted-triangle hold-down connection designed in the initial configuration with five bolts and Spruce-Pine-Fir stud grade lumber for the end studs. The results were used as reference. The general purpose was to obtain an optimal configuration for a maximum lateral strength resistance of bolted connection. Because the high tension forces combined with bending moment in the end studs of the MIDPLY\textsuperscript{TM} wall have always produced the end stud failure, our goal was to obtain a connection suitable for a factored lateral resistance of about 44 kN.

The following Mathcad spreadsheet was used to compute the factored resistance to combined bending and axial load - $T_f$ for 38mm X 89mm end studs made from Spruce-Pine-Fir stud grade lumber:

\textbf{a) Factored Tensile Resistance Parallel to Grain - $T_r$}

$$
\phi := 0.9 \quad \text{(resistance factor)}
$$

$$
f_t := 6.2 \quad \text{(specified strength in tension II to grain, MPa)}
$$

$$
K_D := 1.15 \quad \text{(load duration factor)}
$$

$$
K_{St} := 1.0 \quad \text{(service condition factor)}
$$

$$
K_T := 1.0 \quad \text{(treatment factor)}
$$

$$
K_H := 1.0 \quad \text{(system factor)}
$$

$$
F_t := f_t \cdot K_D \cdot K_H \cdot K_{St} \cdot K_T 
$$

$$
F_t = 7.13 \quad \text{MPa}
$$
Chapter 4 Hold-Down Connections

\[ K_{zt} := 1.5 \] (size factor)
\[ d := 9.525 \] (b Bolt diameter, mm)
\[ A_n := 89 \cdot (38 - d) \] (net area of cross-section, mm²)
\[ A_n = 2.534 \cdot 10^3 \text{ mm}^2 \]
\[ T_r := \phi \cdot F_t \cdot A_n \cdot K_{zt} \]
\[ T_r = 2.439 \cdot 10^4 \text{ N} \]

b) Factored Bending Moment Resistance - \( M_r \)

\[ f_b := 15.3 \] (specified strength in bending)
\[ K_{Zb} := 1.7 \] (size factor in bending)
\[ K_L := 1 \] (lateral stability factor)
\[ K_{Sb} := 1 \] (service condition factor)

\[ S := \frac{(38 - d) \cdot 89^2}{6} \]
\[ S = 3.759 \cdot 10^4 \text{ mm}^3 \]
\[ F_b := f_b \cdot (K_D \cdot K_H \cdot K_{Sb} \cdot K_T) \]
\[ F_b = 17.595 \text{ N} \]
\[ M_r := \phi \cdot F_b \cdot S \cdot K_{Zb} \cdot K_L \]
\[ M_r = 1.012 \cdot 10^6 \text{ N mm} \]

c) Resistance to Combined Bending and Axial Load - \( T_f \)

\[ ex := 44.5 \] (distance between the hold-down force and end stud axial force, mm)
\[ T_f := \frac{1}{\left( \frac{1}{T_r} + \frac{ex}{M_r} \right)} \]
\[ T_f = 1.177 \cdot 10^4 \text{ N} \]
Chapter 4  Hold-Down Connections

The next Mathcad spreadsheet was used to compute the factored lateral resistance - $P_f$ for the bolted connection of the inverted triangle hold-down on the 38mm X 89mm Spruce-Pine-Fir stud grade lumber. The hold-down (Figure 4.4) uses 250 MPa steel and the bolts are made of 300 MPa steel.

a) The Unit Lateral Strength Resistance for II to Grain Loading - $p_u$

\[
\begin{align*}
\phi &= 0.7 \quad \text{(resistance factor)} \\
K_D &= 1.15 \quad \text{(load duration factor)} \\
K_{SF} &= 1.0 \quad \text{(service condition factor)} \\
K_T &= 1.0 \quad \text{(treatment factor)} \\
n_s &= 1 \quad n_s = \text{number of shear planes} \\
n_F &= 5 \quad n_F = \text{number of bolts in a row} \\
G &= 0.42 \quad \text{(mean relative density)} \\
d &= 9.525 \quad \text{(bolt diameter)} \\
f_2 &= 63G(1 - 0.01d) \quad \text{(embedding strength of main-wood member, MPa)} \\
f_1 &= 574 \quad \text{(for ASTM A36 Steel, MPa)} \\
l_1 &= 6.35 \quad \text{(side member thickness, mm)} \\
l_2 &= 89 \quad \text{(main member thickness, mm)} \\
f_y &= 300 \quad \text{(bolt yield strength, MPa)} \\
F_1 &= 0.8f_1 \; \text{MPa} \quad f_1 = 574 \; \text{MPa} \quad F_1 = 459.2 \; \text{MPa}
\end{align*}
\]

"$p_u$" is the smallest value determined from formulae "$p_{ua}$" to "$p_{ug}$".
Chapter 4  Hold-Down Connections

\[ P_{ua} := F \frac{1 \cdot d^2 \cdot l_1}{d} \quad \text{p}_{ua} = 2.777 \times 10^4 \]

\[ P_{ub} := F \frac{1 \cdot d^2 \cdot f_2 \cdot l_2}{f_1 \cdot d} \quad \text{p}_{ub} = 1.624 \times 10^4 \]

\[ P_{ud} := F ____ \frac{1}{6} \left( \frac{f_2}{f_1 + f_2} \cdot \frac{f_y}{f_1} + \frac{1}{5} \cdot l_1 \right) \quad \text{p}_{ud} = 8.015 \times 10^3 \]

\[ P_{ue} := F ____ \frac{1}{6} \left( \frac{f_2}{f_1 + f_2} \cdot \frac{f_y}{f_1} + \frac{1}{5} \cdot l_2 \right) \quad \text{p}_{ue} = 8.032 \times 10^4 \]

\[ P_{uf} := F ____ \frac{1}{5} \left( \frac{l_1}{d} + \frac{f_2 \cdot l_2}{f_1 \cdot d} \right) \quad \text{p}_{uf} = 8.802 \times 10^3 \]

\[ P_{ug} := F ____ \frac{2}{3} \left( \frac{f_2}{f_1 + f_2} \cdot \frac{f_y}{f_1} \right) \quad \text{p}_{ug} = 4.921 \times 10^3 \]

\[ P_u := \min (\left[ P_{ua}, P_{ub}, P_{ud}, P_{ue}, P_{uf}, P_{ug} \right]) \quad \text{p}_u = 4.921 \times 10^3 \text{ N} \]

b) The Lateral Strength Resistance for II to Grain Loading - \( P_u \)

\[ P_u := \text{p}_u (K_D \cdot K_{SF} \cdot K_T) \quad \text{P}_{u} = 5.659 \times 10^3 \]

c) The Factored Lateral Strength Resistance for II to Grain Loading - \( P_r \)

\( l := l_2 \quad \text{(member thickness, mm)} \)
s := 50.8 (bolt spacing in the row)

N := n_F (number of bolts in a row)

\[
J_G := 0.33 \left( \frac{1}{d} \right)^{0.5} \left( \frac{s}{d} \right)^{0.2} \cdot N^{-0.3} \quad J_L := \min \left( [ J_G, 1.0 ] \right) \quad J_G = 0.87
\]

\[
J_L := 1.0
\]

\[
J_R := 1.0
\]

\[
J_F := \left( J_G \cdot J_L \cdot J_R \right) \quad J_F = 0.87
\]

where: 
- \( J_G \) = factor for 2 to 12 bolts in a row
- \( J_L \) = factor for loaded end distance
- \( J_R \) = factor for number of rows

\[
P_r := \phi \cdot P_{un} \cdot n_s \cdot n_F \cdot J_F \quad P_r = 1.723 \times 10^4
\]

As a conclusion, the factored lateral resistance - \( P_r \) of the bolted connection using the inverted triangle hold-down is just 18.7 kN. This value was obtained in the most used configuration with 5 (\( d = 9.53\text{mm} \)) bolts, equally spaced at 50.8mm and Spruce-Pine-Fir stud grade lumber for the end stud. The factored tensile resistance of the end stud is 24.4 kN but the factored resistance to combined bending and axial load of the 38mm X 89mm Spruce-Pine-Fir stud grade lumber is just 8.6 kN. That means that the influence of the bending moment in the end stud due to the excentricity of the hold-down connection is significant. These results are taken as reference in the following comparison table, where
the configuration of the hold-down connection has been changed. The imposed limits of these configurations are:

- the resistance to combined bending and axial load of the end stud greater than the factored lateral strength resistance of the bolted connection
- bolt spacing no greater than 76.2mm (3 inch)
- bolt diameter no greater than 11.1mm (7/16 inch)
- number of bolts no greater than 7

The code or a reasonable length of the hold-down connection imposes these limits.

<table>
<thead>
<tr>
<th>Wall Config</th>
<th>End Stud Grade</th>
<th>Bolt Spacing (mm)</th>
<th>Bolt Diameter (mm)</th>
<th>No. of Bolts</th>
<th>Tr (kN)</th>
<th>Tf (kN)</th>
<th>Pr (kN)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>stud grade</td>
<td>50.8</td>
<td>9.5</td>
<td>5</td>
<td>24.4</td>
<td>12.7</td>
<td>17.2</td>
</tr>
<tr>
<td>2</td>
<td>2400f MSR</td>
<td>50.8</td>
<td>9.5</td>
<td>5</td>
<td>56.7</td>
<td>21.8</td>
<td>18.7</td>
</tr>
<tr>
<td>3</td>
<td>2400f MSR</td>
<td>76.2</td>
<td>9.5</td>
<td>5</td>
<td>56.7</td>
<td>21.8</td>
<td>20.3</td>
</tr>
<tr>
<td>4</td>
<td>2400f MSR</td>
<td>76.2</td>
<td>11.1</td>
<td>5</td>
<td>53.5</td>
<td>20.6</td>
<td>24.6</td>
</tr>
<tr>
<td>5</td>
<td>2400f MSR</td>
<td>76.2</td>
<td>11.1</td>
<td>6</td>
<td>53.5</td>
<td>20.6</td>
<td>27.9</td>
</tr>
<tr>
<td>6</td>
<td>2400f MSR</td>
<td>76.2</td>
<td>11.1</td>
<td>7</td>
<td>53.5</td>
<td>20.6</td>
<td>31.1</td>
</tr>
</tbody>
</table>

Conclusions:

- Using 2400f MSR lumber the behavior of the hold-down connection has been improved. The MIDPLY™ wall tests that have been performed using the higher-grade lumber for the end studs proved a better capacity of the wall to sustain lateral
loads. The improved capacity can be observed in the results of the shake table tests done in the second year and summarized in Chapter 2 of this thesis. The configuration of the MIDPLY™ walls in the shake table tests used for the first time 1650f MSR lumber for the end studs, but further the end-stud failure could not be prevented. However, the improved behavior of the end studs in those tests, has established as a rule the usage of 2400f MSR in all the tests in the third year were the inverted-triangle hold-down has been used.

- The configurations 4, 5 and 6 have never been used because the factored resistance to combined bending and axial load of the end stud is lower than the factored lateral strength resistance of the bolted connection. As a result, it is presumed that the end stud will fail before the bolted connection will reach its capacity.

Up to this time, just the configurations 1 and 2 of the inverted-triangle hold-down have been tested.

4.4. **Double-Shear Hold-Down**

4.4.1. Design

It has been observed from the previous MIDPLY™ wall tests that the end studs finally failed in the case of no vertical loads applied. This certainly shows that the combined effect of the tension forces and bending moment affected the performance of the end studs even using the inverted-triangle hold-downs. One of the possible solutions to prevent the end stud failure is to further reduce the bending moment and to use the full
Chapter 4  Hold-Down Connections

capacity of the tensile resistance of the end stud. As a result of the increasing demand of a different hold-down connection, a new hold-down has been designed. The objectives of designing an alternate hold-down connection were:

- to minimize the bending of the end studs
- to minimize the bending and the movement in the vertical plane of the bolts connecting the hold-down to the end studs which were found that have been the cause of the stud splitting
- to investigate different solutions to connect the hold-down at the top of the end stud with the hold-down from the next floor in a multistory building.

The objectives were achieved by designing a totally new hold-down. My conception, the new hold-down called "double-shear hold-down" was specifically designed for the MIDPLY™ Wall System. A design drawing is shown in Figure 4.5.

The double-shear hold-down is made of the same 250 MPa steel and regular structural bolts of 300 MPa steel. Table 4.2 compares the design performances of the new connection versus the only two configurations (1 and 2) of the inverted-triangle hold-down which have been tested and whose configurations were shown in Table 4.1.
Table 4.2  Inverted-triangle vs. double-shear hold-down

<table>
<thead>
<tr>
<th>Hold-Down Type</th>
<th>Hold-Down Config.</th>
<th>End Stud Grade</th>
<th>Bolt Spacing (mm)</th>
<th>Bolt Diameter (mm)</th>
<th>No. of Bolts</th>
<th>$T_r$ (kN)</th>
<th>$T_f$ (kN)</th>
<th>$P_r$ (kN)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Inverted-triangle</td>
<td>1</td>
<td>stud grade</td>
<td>50.8</td>
<td>9.5</td>
<td>5</td>
<td>24.4</td>
<td>12.7</td>
<td>17.2</td>
</tr>
<tr>
<td></td>
<td>2</td>
<td>2400f MSR</td>
<td>50.8</td>
<td>9.5</td>
<td>5</td>
<td>56.7</td>
<td>21.8</td>
<td>18.7</td>
</tr>
<tr>
<td>Double-shear</td>
<td>1</td>
<td>2400f MSR</td>
<td>50.8</td>
<td>9.5</td>
<td>5</td>
<td>56.7</td>
<td>56.7</td>
<td>37.5</td>
</tr>
<tr>
<td></td>
<td>2</td>
<td>2400f MSR</td>
<td>50.8</td>
<td>11.1</td>
<td>5</td>
<td>53.5</td>
<td>53.5</td>
<td>45.3</td>
</tr>
<tr>
<td></td>
<td>3</td>
<td>2400f MSR</td>
<td>76.2</td>
<td>9.5</td>
<td>4</td>
<td>56.7</td>
<td>56.7</td>
<td>34.4</td>
</tr>
</tbody>
</table>

The third configuration of the double-shear hold-down is shown in Figure 4.5, below.

![Figure 4.5 Double-shear hold-down (design)](image-url)
The same Mathcad spreadsheet was used to compute the factored lateral resistance \( P_r \) for the bolted connection of the double-shear hold-down. The results are shown in Table 02 for the three configurations. In the all three examples, the factored resistance to combined bending and axial load of the end stud is greater than the factored lateral strength resistance of the bolted connection. The behavior of the end stud was improved significantly due to the new design that practically eliminates the bending moment of the end stud.

4.4.2. Benefits

Unlikely, due to the very loaded schedule of the MIDPLY™ Wall System testing, the new double-shear hold-down connection has not been tested yet. The hopes are that this
year the new hold-down to be included in some of the tests. The design values are improved over 200%, therefore some of the important benefits of using the double-shear hold-down are summarized next:

- The factored lateral strength resistance of the bolted connection is almost twice as much as the presently used inverted-triangle hold-down. The increased resistance is a result of using the bolts in double shear.
- The factored tensile resistance of the end stud is more than double compared with that of the inverted-triangle hold-down. This improvement is a result of eliminating the bending moment of the end stud, the hold-down force and the tensile force of the end stud acting on the same direction with no eccentricity.
- The bearing failure of the bottom plate of the MIDPLY™ wall is eliminated, the end stud resting in compression on one side of the rectangular shape of the structural tub. In the inverted-triangle hold-down configuration, the end stud rests on the bottom plate of the wall.
- The new hold-down is very easy to be slid into position on both the upper part and the lower part of the end stud. Moreover, the double-shear hold-down can come already mounted on the wall if the wall is industrial manufactured and shipped as a package on the site.
- The connection between the top hold-down in one floor and the bottom hold-down of the next floor in a multistory building is easy to carry out.
Chapter 4  Hold-Down Connections

- The necessity of two hold-downs with to bolt connection per end of the wall is eliminated, the new hold-down using just one central bolt to take the tension force of the end studs.

4.5.  Steel Rods as Hold-Downs

From shake table and pseudo-dynamic tests, it was observed that the inverted-triangle hold-downs could not regularly prevent end stud failure at severely high load conditions. The fracture of the end studs forced the wall to fail in a brittle manner. As a result, the ductility of the wall was reduced. In order to prevent this undesirable failure mode, the steel rod acting as hold-downs connectors were studied and applied to the MIDPLY⁷⁷ walls.

![Figure 4.7 MIDPLY™ wall with steel rods as hold-downs](image)

Testing and Analysis of Midply⁷⁷ Shear Wall System  - 52 -
4.5.1. Wall Configuration

MIDPLY™ wall M52-12 was a 1.22m insert installed in a standard shear wall frame. The wall was built with Spruce-Pine-Fire stud grade studs (38mm x 64mm for interior studs and 38mm x 89mm for the end studs) and 38mm x 64mm standard and better grade for the top and bottom plates. The 11mm (7/16") OSB was used as sandwich sheathing and the 9.5mm (3/8") Canadian Softwood Plywood (CSP) was for the exterior sheathing. 3.05 mm diameter and 89 mm length power nails connected both OSB and CSP panels at 100 mm on center. Instead of inverted-triangle hold-down connections, two 15.9mm (5/8") steel rods were used at each end as hold-down connections, as shown in Figure 4.7.

4.5.2. Loading Protocols

The reversed cyclic displacement schedule ISO 98 was used in the test. The protocol is detailed in section 3.2.2, Chapter 3.

4.5.3. Test Results of MIDPLY™ Wall Using the Steel Rods as Hold-Downs

The test results of wall M52-12 are listed in Table 4.3. With the steel rods installed at both ends of the wall, the M52-12 wall showed excellent performance. The graphic response can be seen in Figure 4.8. Since the steel rod takes the uplift force, no failure was observed at the end studs. The wall was able to resist 127mm (5") horizontal deflection without significant loss of lateral load resistance. The load carrying capacity at 127mm deflection was still above 80% of its maximum load.
Chapter 4  Hold-Down Connections

Upon inspection of the wall, it was found that the nails fractured between the sandwich OSB and stud framing along the perimeter of the panel. Nail pull-through and nail withdrawal occurred along the perimeter of the exterior panel. The nail withdrawal occurred only on those fractured nails. Bearing failure was also observed on both top and bottom plates underneath the end studs. Different than the stud fracture under the uplift force, which is a brittle failure, the bearing failure does not considerably affect the performance of the wall.

Table 4.3  Test results of MIDPLY™ wall using steel-rods

<table>
<thead>
<tr>
<th>Wall number</th>
<th>Load protocol</th>
<th>Vertical load</th>
<th>Length (m)</th>
<th>Plates &amp; interior studs</th>
<th>End studs (mm)</th>
<th>Stud spacing (mm)</th>
<th>P_max (kN)</th>
<th>δ_u (mm)</th>
<th>K (kN/mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>M52-12</td>
<td>ISO98</td>
<td>1st 3rd</td>
<td>none</td>
<td>38 x 64</td>
<td>38 x 89</td>
<td>406</td>
<td>52.4</td>
<td>125*</td>
<td>2.11</td>
</tr>
</tbody>
</table>

P_max - maximum load of the envelope curve.

δ_u - displacement of the wall at 80% of maximum load capacity in the descending part of the load-displacement curve.

K - secant stiffness of the wall measured between points corresponding to the upper portion of the loads at 10% and 40% of the maximum load.

* - displacement at 80% of maximum load in the descending portion of the envelope curve was not achieved (i.e. specimen did not fail or reach the near collapse point). These values are the maximum displacements recorded at the test.
4.5.4. Benefits

Because a MIDPLY™ wall with exactly the same configuration as M52-12 but using regular hold-down or inverted-triangle hold-down has not been tested yet, a direct comparison could not be made. However, the benefits of using the steel rods as hold-downs were visible and remarkable not just in the process of building the wall but also in the technical properties of the wall, observed during and after the test. Some of these benefits based on the wall's behavior, failure mode and previous described test results are detailed next:

- The tensile force was taken by the steel rods so no failure of the end studs was observed. As a result, the MSR lumber, which is more expensive, was replaced by stud grade as material for the end studs.
- The brittle fracture of the end stud due to the high tensile forces was replaced by the bearing failure on the top and bottom plates. This kind of failure was considered as acceptable and allows the wall to reach much better performances regarding the maximum load, energy dissipation and ductility. The wall was able to resist to maximum of 127mm horizontal deflection and a load of 52.4 kN without dropping its load capacity under 80% of the maximum in the descending part of the load-displacement curve.
- The cost of the materials used for the connection (end studs + hardware) also the time consumed to built the wall was well bellow compared with the case of the inverted-triangle hold-down connection.
Because no other visible failures were noticed after the test and the wall was still standing, some other tests have been done (that are presented in the next chapters of this thesis) by simply repairing these walls. Simply adding an equal number of nails along the perimeter without taking the wall down, were achieved about 80% of the initial lateral resistance. This fast and not expensive retrofit applicable to MIDPLY™ Wall System mostly using steel rods as hold-downs, can be a big advantage after some unwanted events as earthquakes and hurricanes.

Based on the improved performances of the MIDPLY™ walls using steel rods, the majority of the tests done in the third year and described in this thesis have used the configuration with steel rods.

![Figure 4.8 Response of MIDPLY™ wall M52-12](image)

Figure 4.8 Response of MIDPLY™ wall M52-12
4.5.5. Disadvantages

Some of the disadvantages of using the steel rods as hold-downs can be:

- The difficulty of mounting the steel rods on the site in the multi-storey building.
- The difficulty of accessing the steel rods connectors to adjust the length. This adjustment is necessary due to the shrinkage of the building.
CHAPTER 5

MIDPLY™ Walls Under Pseudo-Dynamic Tests

This chapter compares the performance of MIDPLY™ walls under shake table and pseudo-dynamic tests. The results of the shake table tests were summarized in Chapter 2 of this thesis and were taken from the "Second Year Report of MIDPLY™ Wall System". The pseudo-dynamic tests were performed at the beginning of the third year and are detailed in this chapter.

5.1. Overview

Shake table tests can simulate the response of MIDPLY™ walls under seismic events by accurately reproducing earthquake ground motions. Inertial forces are applied to walls under shake table tests. The response of MIDPLY™ wall depends on the dynamic and static properties of the wall (initial lateral stiffness, natural period of vibration) and the characteristics of ground motions. However, because of the high cost in conducting such
tests, reversed cyclic or pseudo-dynamic tests are often used as an alternative to obtain the dynamic properties of the walls. From the deformation response \( u(t) \) of the wall obtained from a given ground motion \( \ddot{u}_g(t) \), the natural vibration period \( T_n \) can be computed. For a system with no damping,

\[
\omega_n^2 u(t) = -\ddot{u}_g(t)
\]

where:

\[
T_n^2 = \frac{(2\pi)^2}{\omega_n^2}
\]

The pseudo-acceleration response \( A(t) \) of the wall can be also computed from the deformation response \( u(t) \).

\[
A(t) = \omega_n^2 u(t)
\]

As a result, the dynamic properties of the wall and subsequently the behavior of the wall under a real earthquake motion may be predicted from the response of the wall under pseudo-dynamic load protocols. The question is whether the reversed cyclic or pseudo-dynamic tests can reasonably predict the behavior under shake table tests, knowing that some other parameters of the wall (variable stiffness during the test) lead to an inelastic behavior.

To answer this question, pseudo-dynamic tests were conducted to compare the performance of the walls under shake table and pseudo-dynamic tests. In the pseudo-dynamic tests, the walls M48-03 and M50-03 were tested using the displacement responses of MIDPLY™ walls M48-01 and respectively M50-01 under shake table tests.
5.2. Walls Configuration

The configuration of the walls M48-03 and M50-03 were identical to M48-01 and M50-01. The detailed information of the configuration of tested MIDPLY™ walls are listed in Table 5.1.

Table 5.1. Pseudo-dynamic tests of MIDPLY™ walls.

<table>
<thead>
<tr>
<th>Pseudo-Dynamic Tests</th>
<th>Shake Table Tests</th>
<th>Stud Spacing (mm)</th>
<th>Stud and Plate Size (mm)</th>
<th>Vertical Loads (kN)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Wall no.</td>
<td>Wall no.</td>
<td>Load Protocol</td>
<td>Load Protocol</td>
<td></td>
</tr>
<tr>
<td>M48-03</td>
<td>M48-01</td>
<td>Kobe 0.35g</td>
<td>Kobe 0.35g</td>
<td>610</td>
</tr>
<tr>
<td>M48-03a</td>
<td>M48-01a</td>
<td>Kobe 0.52g</td>
<td>Kobe 0.52g</td>
<td>610</td>
</tr>
<tr>
<td>M48-03b</td>
<td>-</td>
<td>Ramp-compression</td>
<td>-</td>
<td>610</td>
</tr>
<tr>
<td>M48-03c</td>
<td>-</td>
<td>Ramp-tension</td>
<td>-</td>
<td>610</td>
</tr>
<tr>
<td>M50-03</td>
<td>M50-01</td>
<td>Kobe 0.35g</td>
<td>Kobe 0.35g</td>
<td>406</td>
</tr>
<tr>
<td>M50-03a</td>
<td>M50-01a</td>
<td>Kobe 0.52g</td>
<td>Kobe 0.52g</td>
<td>406</td>
</tr>
<tr>
<td>M50-03b</td>
<td>M50-01b</td>
<td>Kobe 0.67g</td>
<td>Kobe 0.67g</td>
<td>406</td>
</tr>
<tr>
<td>M50-03c</td>
<td>-</td>
<td>Ramp-compression</td>
<td>-</td>
<td>406</td>
</tr>
</tbody>
</table>

1 Loading protocols using the displacement response of walls under shake table tests.

The wall M48-03 was built using 38mm × 63mm (2 inch × 3 inch nominal) Spruce-Pine-Fire for interior studs, top and bottom plates, and 38mm × 89 mm (2 inch × 4 inch nominal) 1650f MSR for the end studs. 12.5mm thickness Oriented Strand Board (OSB) was used as middle sheathing panel connected with power nails (D = 3.05mm, L =
82mm) at 100mm on center spacing. The studs were spaced at 610mm on center. No vertical loads were applied.

The wall M50-03 used 38mm × 89mm (2 inch × 4 inch nominal) SPF for interior studs, top and bottom plates, and 38mm × 89mm (2 inch × 4 inch nominal) 1650f MSR for end studs. OSB of the same 12.5mm thickness was used as middle sheathing panel. The frame and the panel were connected with the same type of power nails (D = 3.05mm, L = 82mm) at 100mm on center. The studs in the wall M50-03 were spaced at 406mm on center and it was applied a vertical load of 27 kN on top of the load-spreader bar.

5.3. Loading Protocols

The loading protocols used for M48-03 and M50-03 pseudo-dynamic tests were detailed in the section 3.2.1, Chapter 3. A constant displacement rate of 20 mm/sec (0.8 inch/sec) was used in all pseudo-dynamic tests. The same rate was used for the fast ramp tension and compression tests. The fast ramp tests were conducted following pseudo-dynamic tests to evaluate the residual strength of the MIDPLY™ walls.

5.4. Test results of MIDPLY™ Walls Under Pseudo-Dynamic Tests

Table 5.2 summarizes the maximum loads and displacements in the tension and compression zones recorded under shake table and pseudo-dynamic tests.
Table 5.2. Results of MIDPLY™ walls under shake and pseudo-dynamic tests (1)

<table>
<thead>
<tr>
<th>Wall No.</th>
<th>Max Displ. (mm)</th>
<th>Max Load (kN)</th>
<th>Wall No.</th>
<th>Max Displ. (mm)</th>
<th>Max Load (kN)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>(+)</td>
<td>(-)</td>
<td>(+)</td>
<td>(-)</td>
<td>(+)</td>
</tr>
<tr>
<td>M48-01</td>
<td>23</td>
<td>21</td>
<td>49.5</td>
<td>47.0</td>
<td>M48-03</td>
</tr>
<tr>
<td>M48-01a</td>
<td>61</td>
<td>37</td>
<td>63.7</td>
<td>52.2</td>
<td>M48-03a</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>M48-03b</td>
</tr>
<tr>
<td>M50-01</td>
<td>21</td>
<td>18</td>
<td>39.5</td>
<td>37.8</td>
<td>M50-03</td>
</tr>
<tr>
<td>M50-01a</td>
<td>54</td>
<td>56</td>
<td>68.0</td>
<td>65.2</td>
<td>M50-03a</td>
</tr>
<tr>
<td>M50-01b</td>
<td>105</td>
<td>99</td>
<td>77.8</td>
<td>70.2</td>
<td>M50-03b</td>
</tr>
<tr>
<td>M50-03c</td>
<td>-</td>
<td>120</td>
<td>-</td>
<td>66.4</td>
<td></td>
</tr>
</tbody>
</table>

Table 5.3 summarizes the energy dissipation recorded under shake table and pseudo-dynamic tests.

Table 5.3. Results of MIDPLY™ walls under shake and pseudo-dynamic tests (2)

<table>
<thead>
<tr>
<th>Wall No.</th>
<th>Energy Dissipated (kN·mm)</th>
<th>Wall No.</th>
<th>Energy Dissipated (kN·mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>M48-01</td>
<td>6001</td>
<td>M48-03</td>
<td>3037</td>
</tr>
<tr>
<td>M48-01a</td>
<td>5430</td>
<td>M48-03a</td>
<td>6715</td>
</tr>
<tr>
<td></td>
<td></td>
<td>M48-03b</td>
<td>2512</td>
</tr>
<tr>
<td></td>
<td></td>
<td>M48-03c</td>
<td>1627</td>
</tr>
<tr>
<td>M50-01</td>
<td>5444</td>
<td>M50-03</td>
<td>2547</td>
</tr>
<tr>
<td>M50-01a</td>
<td>21690</td>
<td>M50-03a</td>
<td>12950</td>
</tr>
<tr>
<td>M50-01b</td>
<td>26880</td>
<td>M50-03b</td>
<td>16590</td>
</tr>
<tr>
<td></td>
<td></td>
<td>M50-03c</td>
<td>4344</td>
</tr>
</tbody>
</table>
5.5. **Load-Displacement Graphs and Discussions**

The load-displacement graphs, shown in Figures 5.1 to 5.5, compare also the performances of the MIDPLY™ walls under shake table and pseudo-dynamic tests.

Test responses of the MIDPLY™ walls M32-01 and M40-01b under monotonic tests were also included. They were compared with the responses of the walls M48-03b, M48-03c and M50-03c tested now under fast ramp compression and tension. The configurations of M32-01 and M40-01b walls and the results were taken from the Second Year Annual Progress Report submitted to Science Council of BC by Forintek Canada Corp. The results of the two tests are also summarized in Table 3 and Table 5, Chapter 2 in this thesis.

As noticed from Figure 5.1, the responses are similar for M48-01 and M48-03 at maximum ground accelerations 0.35g and 0.52g. The envelope curves of both walls followed the load-slip curve obtained under monotonic test. No visual damages were observed for both walls at ground accelerations 0.35g.
At ground accelerations 0.52g (Figure 5.2), no visual damages were observed for wall M48-03, however, one of the end studs was broken in wall M48-01. This explained why the load in the test M48-01 (see Figure 5.2) was significantly reduced before it reached its maximum displacement.
The end studs in wall M48-03 were broken during fast tension and compression ramp tests (Figure 5.3).

The load capacities were significantly reduced after the studs had failed, as can be seen in Figures 5.4 and 5.5.
Figure 5.4  Comparison of M48-03b and M40-01b under fast compression ramp test

Figure 5.5  Comparison of M48-03c and M40-01b under fast compression ramp test
As for the walls M48-01 and M48-03, the responses for M50-01 and M50-03 are also similar at maximum ground accelerations 0.35g, 0.52g and 0.67g. The graphs can be seen in Figures 5.6 to 5.8. The envelope curves of both walls followed the load-slip curve obtained under monotonic test. No visual damages were observed for both walls at ground accelerations 0.35g and 0.52g. The walls M50-01 and M50-03 showed greater load capacity (120%) and ductility compared to walls M48-01 and M48-03. The difference is due to the fact that the walls M50-01 and M50-03 have a configuration with 38mm × 89mm studs and smaller (406mm) stud spacing. However, one of the end studs in wall M50-03 failed in tension at 0.67g ground acceleration.

![Figure 5.6](image)

**Figure 5.6**  *Comparison of hysteresis loops of M50-01 and M50-03 under shake table and pseudo-dynamic tests (ground acceleration 0.35g)*
Figure 5.7  Comparison of hysteresis loops of M50-01 and M50-03 under shake table and pseudo-dynamic tests (ground acceleration 0.52g)

Figure 5.8  Comparison of hysteresis loops of M50-01 and M50-03 under shake table and pseudo-dynamic tests (ground acceleration 0.67g)
The results show that the inverted-triangle hold-downs still could not prevent the end stud failure at extremely high load conditions. Further studies on hold-downs were needed to eliminate the fracture of the end studs of the MIDPLY™ wall. The possible solutions including a higher-grade MSR lumber for the end studs, new hold-downs such as steel rods or innovative alternative hold-downs as "double-shear hold-down" were already discussed in Chapter 4. Further tests on the MIDPLY™ Wall System have been done since then and some of the solutions (higher-grade MSR lumber and steel rod hold-downs) have already been included in the test matrixes. The MIDPLY™ wall with openings and the MIDPLY™ wall as insert segment are a few of the applications that
will be described in the next chapters. Most of the tests have been done using the improved solution with steel rods. However, the double-shear hold-down, which promises a lot, has not been tested yet.
6.1. Overview

The MIDPLY™ Wall System can be used as insert segment in a standard shear wall system. As a result of the special arrangement of the frame components, the MIDPLY™ wall system fits perfectly to the most commonly used nominal 4 inch thick standard shear wall system. Therefore, this configuration is an ideal component that does not disturb the appearance of the building. It can be used for the rehabilitation of an existing wood frame building in order to improve its earthquake or wind resistance. The behavior of the hybrid wall system is detailed in the next sections of this chapter.

A package of twelve tests was done to study the performance of the MIDPLY™ wall segments inserted in standard shear walls.
6.2. Wall Configuration

A MIDPLY™ wall segment, 1.22m x 2.36m in dimension, was inserted in the middle part of a 4.88m standard shear wall (Figure 6.1 and 6.2). The 1.22m MIDPLY™ segment was intended to utilize its high lateral resistance capacity to improve the performance of the regular shear wall.

![Figure 6.1 Schematic diagram of MIDPLY™ wall as insert segment in standard shear wall system.](image)

The MIDPLY™ insert segment was constructed with Spruce-Pine-Fir (SPF) framing members. The 38mm x 63mm (2 inch x 3 inch nominal) stud-grade was used for interior studs and standard-and-better grade for the top and bottom plates. The studs were spaced at 406mm on center. When the steel rods were used as hold-downs, the end and buckling studs were stud-grade 38mm x 89mm (2 inch x 4 inch nominal). In the cases of using the inverted-triangle hold-downs, the end studs were built from the high grade 2400f MSR.
The middle sheathing panels (12.5mm OSB) were connected by power nails at 100 mm on center.

The standard shear wall used double 38mm × 89mm (2 inch × 4 inch nominal) SPF for the top plate and the end studs, and single 38mm × 89 mm (2 inch × 4 inch nominal) SPF for bottom plate and interior studs. The studs were spaced at 406 mm on center. Canadian Softwood Plywood (CSP) of 9.5 mm thickness was sheathed to the frame using power nails (D = 3.05 mm, L = 65 mm) at 150mm on center spacing. The Simpson HD2A hold-downs were used at the end studs.

![Figure 6.2 MIDPLY™ wall segment inserted in a standard shear wall system.](image)

The detailed information of the configuration of tested MIDPLY™ walls are listed in Table 6.1.
Table 6.1 Test matrix of MIDPLY™ walls used as insert in standard shear walls

<table>
<thead>
<tr>
<th>Wall No.</th>
<th>Load protocol</th>
<th>MIDPLY Segment</th>
<th>Standard Shear Wall</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>Length (m)</td>
<td>Ext. panels</td>
</tr>
<tr>
<td>M52-01</td>
<td>monotonic</td>
<td>1.22m</td>
<td>-</td>
</tr>
<tr>
<td>M52-02(^1)</td>
<td>monotonic</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>M52-03</td>
<td>ISO 98</td>
<td>1.22m</td>
<td>-</td>
</tr>
<tr>
<td>M52-04(^1)</td>
<td>ISO 98</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>M52-05</td>
<td>monotonic</td>
<td>1.22m</td>
<td>-</td>
</tr>
<tr>
<td>M52-06(^1)</td>
<td>monotonic</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>M52-07</td>
<td>ISO 98</td>
<td>1.22m</td>
<td>-</td>
</tr>
<tr>
<td>M52-08(^1)</td>
<td>ISO 98</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>M52-09</td>
<td>monotonic</td>
<td>1.22m</td>
<td>1 side</td>
</tr>
<tr>
<td>M52-10</td>
<td>ISO 98</td>
<td>1.22m</td>
<td>1 side</td>
</tr>
<tr>
<td>M52-11</td>
<td>ISO 98</td>
<td>1.22m</td>
<td>2 sides</td>
</tr>
<tr>
<td>M52-12(^5)</td>
<td>ISO 98</td>
<td>1.22m</td>
<td>1 side</td>
</tr>
</tbody>
</table>

\(^1\) The 16ft frame from the previous test was resheathed on the two 1.83m segments at the ends of the wall. The 1.22m MIDPLY wall was removed as shown in Figure 6.3.

\(^2\) Power nails (D = 3.05mm, L = 82mm) were used to connect the middle panel.

\(^3\) Power nails (D = 3.05mm, L = 89mm) were used to connect the middle panel and one exterior 9.5mm CSP panel.

\(^4\) Power nails (D = 3.05mm, L = 89mm) were used to connect the middle panel and one exterior 9.5mm CSP panel, and common nails (L = 64mm) for the other exterior 9.5mm CSP panel.
The frame from wall M52-11 was reused after removing the exterior panel and 1.22m MIDPLY wall was installed in the middle of the frame (Figure 6.4).

Figure 6.3 Wall configuration for M52-02, M52-04, M52-06 and M52-08

Figure 6.4 Wall configuration for M52-12
6.3. **Loading Protocols**

The monotonic and reversed cyclic loading protocols were used in the tests. The loading rate for the monotonic tests was approximately 7.62mm/min. For the cyclic tests was used the ISO 98 protocol. The monotonic and static-cyclic loading protocols for the M52-01 to M52-12 tests (MIDPLAY™ walls as inserts) were detailed in the section 3.2.2, Chapter 3.

6.4. **Test Results of MIDPLY™ Walls Used as Insert Segments**

Table 6.2 gives a summary of the tests’ results of the MIDPLY™ walls used as inserts in standard shear walls.