STATIC AND DYNAMIC TESTING OF THE MIDPLY™ SHEAR WALL SYSTEM

by

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We accept this thesis as conforming to the required standard:

University of British Columbia
Vancouver, Canada
October, 2001

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The work documented in this thesis constitutes the second year of the three-year MIDPLY™ project. The MIDPLY™ shear wall system is a new invention for strengthening new and existing light frame timber buildings against earthquakes and high winds. The system needed to be tested before its implementation in the field (US Patent US5782054: Wood Wall Structure).

As part of an ongoing research project undertaken by Forintek Canada Corp. and the University of British Columbia, the MIDPLY™ shear wall system was designed, tested, and developed in order to procure an improved product over the conventional shear wall presently used in construction.

The objective of this project was three-fold:

1. Improve the behaviour of timber shear walls by utilizing conventional timber products combined with new technology - the MIDPLY™ wall.
2. Quantify the improvements though full-scale static and dynamic testing of the MIDPLY™ shear walls.
3. Determine failure modes and load-displacement characteristics of the MIDPLY™ walls.
Abstract

The work in this thesis comprises of static testing, dynamic testing, structural modeling, and shear wall connection design and implementation. Several configurations of 2.44m x 2.44m walls were tested statically at Forintek Canada Corp., where from three configurations were chosen to be tested dynamically at the Earthquake Engineering Laboratory at UBC. The configurations of MIDPLY™ walls were tested under two different earthquake records.

In total, 40 static tests, which include monotonic and reversed cyclic tests, and 6 dynamic tests were performed as part of the scope of this thesis. Throughout the testing, the parameters that were varied were lumber size, stud spacing, lumber type, loading protocol, hold down connection type, and vertical loading.

The results of the testing clearly showed the strengths and weaknesses of the MIDPLY™ shear wall system. Two strengths were that the MIDPLY™ wall could withstand higher loads and displacements than the conventional light frame timber shear wall used in most buildings in North America. Also, some common failure modes from other walls were eliminated. The weakness was that the wall sometimes failed in a brittle manner through end-stud failure.

The future plans of the MIDPLY™ project include new connection designs to further improve racking performance.
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“I will show you what he is like who comes to me and hears my words and puts them into practice. He is like a man building a house, who dug down deep and laid the foundation on rock. When a flood came, the torrent struck that house but could not shake it, because the house was well built.”

Luke 6:47,48 - NIV
This Thesis is dedicated to my parents, Christiaan and Johanna,

who always told me the Dutch saying:

"Eerst zaken, dan vermaken."

Their encouragement made this Thesis a reality.
Chapter 1: Introduction

1.1 Background

Among the world's natural disasters that cause countless cases of deaths, injuries, monetary loss, and homelessness each year are earthquakes and hurricanes. Although only recorded in the last 3000 years, the history of these disasters is as old as the earth itself, and will continue to plague mankind as long as the earth lasts. Who should be prepared against these impending disasters? Everyone living in a seismically active area or hurricane prone area, which covers most of the globe. Since the content of this thesis deals with the structural safety against earthquakes and high winds, the contents herein concern a vast populace.

Insurance against disasters is usually available in developed countries, but what better insurance is there than preparation – the structural safety of one's home. Earthquakes have been known to cause significant devastation through structural damage, fires, damage to transportation routes, and utilities damage. On the average, 20 earthquakes of moment magnitude 7.0 or greater occur in the world each year (USGS), and 10,000 people die each year due to earthquakes (Naeim, 1989). Many earthquakes have caused tens of thousands of deaths in one event. For example, the 1976 Tangshan earthquake in China officially caused 255,000 deaths, with an estimated total of 655,000
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deads due to indirect causes. Furthermore, the monetary costs of recovery are staggering, especially in developed countries such as the United States and Japan. In the 1994 Northridge, California earthquake, damage estimates exceeded 20 billion dollars U.S. (http://www.neic.cr.usgs.gov/), (Foliente, 1994).

From the statistics of the top 30 deadliest hurricanes in the U.S., an average of 522 people die due to the effects of one hurricane (USGS), and the average monetary losses are 13.1 billion dollars US for each hurricane. In the last 40 years, and average of 6 hurricanes have stricken the U.S. per year, and more than 2 hurricanes per year of the magnitude described above (http://www.aoml.noaa.gov/).

More shocking statistics than these are observed in third-world countries where the residential building quality is relatively poor. "Most casualties were directly caused by the collapse of weak houses and buildings. (The Guatemala, Tangshan, and Friuli earthquakes all occurred at night when people were in their structurally hazardous homes.) (Bolt, 1988). In seismically active areas, there is a positive correlation between a region's building quality and its number of deaths per capita due to earthquakes. During the writing of this chapter, two major earthquakes struck that killed over 100,000 people and costed billions of dollars U.S. in damage. The first one occurred in San Salvador, El Salvador with a magnitude of 7.6 on the Richter scale, killing over 650 people, and the second one in Bhuj, India with a magnitude of 7.9 and killing nearly 100,000 with numbers still rising. The unpredictability of these disasters in conjunction with their speed of action presents a constant danger to those who are unprepared. Probability
formulas exist with a limited degree of accuracy on the occurrence of earthquakes, but the danger is real; they will occur...sometime, somewhere.

Just because inferior and archaic construction in common in third world countries, developed countries must not cease to continue finding ways to improve construction methods and building quality. “In fact, every seismically active country could benefit from large-scale programs of instruction on how to make the home more earthquake resistant.” (Bolt, 1988). To those who are prepared for an earthquake or a hurricane, the danger is imminent, but the resulting destruction is likely to be significantly minimized. Preparedness is an insurance of one's safety – a life insurance, per sè.

Perhaps the most important step in preparing for an earthquake or hurricane is to assure that one's home is structurally sound and will not collapse in the event of strong ground motion or high wind speeds. This can be done in many ways, depending on the type of home one lives in. In North America, the majority of residential construction is light frame timber construction, as are many homes and commercial buildings around the world. Light wood frame construction has a worldwide reputation of performing well during severe earthquakes and hurricanes. “...the single and two story wood frame houses typical of the United States and New Zealand, and the light wooden buildings of Japan are examples of places that are among the safest to be in an earthquake.” (Bolt, 1988). Then the important quote is made that is pertinent to this thesis: “But even in these countries the trend is to experiment with new materials and change the design of ordinary buildings, so that the increase in seismic risk may not be recognized until an
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earthquake occurs.” (Bolt, 1988) This is the fundamental reason that testing of a new structural or architectural concept is so important before it is implemented in the field. Even with the most sophisticated computer models, one cannot know exactly how new structural systems will behave unless they are tested. Even with testing in mind, full-scale testing and the correct simulation of applied forces must be used to simulate actual events, otherwise the testing does not represent an accurate model. This is a real challenge: since earthquake forces vary so much based on historical data, it is impossible to simulate all possible forces the system could experience in its design life. Therefore loading protocols must be devised that encompass all possible governing forces. This is the crux- which are the most governing forces for a particular structure? One can only guess until the structure is actually subjected to real-life forces and the outcome is documented. Until then, simulated loading using “loading protocols” are used for testing. This subject is covered in Chapter 3.

1.2 The Use of Timber

There is much to state about timber being the material of choice for many classes of structures, including small buildings, bridges, and artistic structures. Among the reasons for its popularity are: its great availability, low cost, aesthetic qualities, ease of construction, availability of ready-to-construct items, and the fact that it is a renewable resource. Timber structures are divided into two main categories: heavy timber structures and light frame timber structures. The reasons that light frame timber is so popular in residential and small commercial structures, other than the reasons already mentioned, is
that labour is readily available and cheap for this type of construction, timber is a good insulator and sound dampener, and structural renovations are relatively easy and inexpensive.

More importantly, well-constructed light frame timber structures perform very well in earthquakes because of the excellent ductility though its connections, good energy dissipation characteristics, and lightweight resulting in low dead load inertia forces. An emphasis should be made on "well constructed", because it has been experienced time and time again that walls which have not been nailed correctly and/or detailed correctly have displayed much poorer performance than those which have been constructed well. Examples abound all over the world where light frame timber construction has outperformed concrete, masonry, and steel structures. One striking example is the 1964 Alaska earthquake, where "...under conditions of earth subsidence, most concrete or masonry foundation walls or concrete slabs were destroyed even though the wood frame superstructure was undamaged." (Anderson, 1964).

1.2.1 The Lateral Load Resisting System

The objective of seismic and hurricane design is to ensure that structures will survive minor events without any major damage, and major events without collapse. To meet this objective, all structures must contain a lateral load resisting system, or LLR system, for short. This system restrains the structure against large deflections and collapse from earthquake and wind forces. It is the key system that protects a structure from an
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earthquake. The other essential structural system in every building is the vertical load bearing system, but this is secondary to the LLR system with respect to earthquake and wind resistance. Several types of LLR systems exist such as braced frames, moment frames, and shear walls, shown in Figure 1.1. Each one stiffens structures in the lateral direction, as shown. A structure must have LLR systems that will stiffen it in any direction that the lateral forces are likely to apply. All mass carried by the system creates the inertia force that causes these systems to deflect laterally when ground shaking occurs, as shown below.

![Figure 1.1: Three Types of Lateral-Load Resisting (LLR) Systems](image)

In light frame timber structures, the most common type of LLR system is the shear wall. The conventional shear wall, shown in Figure 1.2, is constructed of 2"x4" vertical members spaced at 16" apart, called studs, and horizontal members on the top and bottom of the studs, called plates. On one side of this grid of studs and plates is nailed a sheathing material such as plywood or OSB (Oriented Strand Board). This sheathing is nailed to both the studs and the plates, and this provides the lateral resistance of the shear
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wall system. The nails are loaded in shear when the wall is loaded laterally as shown in Figure 1.2. Since there are many nailed connections in one wall, and the nails are made of ductile steel, the wall behaves in a relatively ductile manner, meaning that the nails deform permanently thus allowing the wall to deflect a long way before collapsing. A shear wall can be considered a vertical diaphragm, similar to a horizontal light frame timber floor system, which also consists of plywood nailed and glued to members (joists in this case) that act as struts and stiffeners.

Figure 1.2: The exterior forces acting on a Shear Wall are transferred to the nailed connections

In the last few decades, wood frame construction has evolved to include condominiums of 3 or 4 storeys. In commercial construction, 60% of all buildings are three or four stories, usually made of timber (CWC, 1994). In many of these applications of wood construction, large openings in the exterior walls, garages at the first storey, concrete topping on floors, and heavy tiles on roofs are common practice. All these practices create additional demand on the LLR systems of the building due to extra large inertia.
forces produced by the extra large mass, and minimized shear wall lengths – a dangerous combination! Although standard shear walls are commonly increased in number and length to account for these extra forces, designers and architects tend to push the limits by including more openings in their buildings. Consequently, this necessitates innovation in new designs that increase the lateral resistance of wood frame shear walls.

1.2.2 Past Experience with Light Frame Timber Buildings

Examples abound of failure of light frame timber buildings in earthquakes and hurricanes, before and even after full-scale shear wall test results were conducted decades ago. In the Alaska earthquake of 1964, although most light frame timber buildings performed better than the concrete and masonry ones, some buildings with walls that had low racking resistance collapsed. This poor performance was contributed to either poor construction quality, detailing such as openings at corners of shear walls, the absence of sheathing on the walls, or abnormally high displacements applied to the walls due to earth settlements, fissures, and slides (Anderson, 1964) as shown in Figure 1.3.
The San Fernando earthquake of 1971 North of Los Angeles produced one of the highest ground accelerations to date, thus creating a high demand on structures. Although the magnitude of the earthquake was only M6.5 (moment magnitude), compared to the Alaska earthquake of 8.6, the higher ground acceleration was due to soil conditions and the distance from the epicentre to accelerometers. Since the San Fernando epicentre was relatively close to a densely populated area, there was much more structural damage. “Observations at San Fernando provided and ideal supplement to those at Anchorage, Alaska. Anchorage afforded an opportunity to observe behaviour of plywood structures essentially at the design load, while San Fernando showed behaviour at ultimate.” (Tissel, 1973). Nevertheless, light framed wood structures behaved very well and it appeared that design standards were in order, except for tension ties at the ends of shear walls and

Figure 1.3:  Light frame timber building maintains its shape after toppling into an earth crevice while the concrete structure (arrow) in background fails.
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chord continuity. These were the two main things that needed to be worked on, as observed from that earthquake. One storey dwellings performed substantially better than did two storey dwellings, and much better than 1 and 2 storey dwellings. Modern dwellings performed noticeably better than older dwellings, according to the Pacific Fire Rating Bureau.

The Loma Prieta quake of 1989, South of San Francisco, was an M7.1. It caused some four-storey timber buildings to collapse that were built on top of garages - a deplorable design. The top three storeys remained intact while the garages were crushed to pancake thickness. Other than these, most houses were largely undamaged. Noting the superior performance of timber structures over that of the previous earthquakes, it was observed that more was known about light timber construction at this point. (Rainer & Karacabeyli, 1999)

The Northridge earthquake of 1994, North of Los Angeles, was only an M6.7, but produced even greater ground accelerations than the San Fernando quake, with accelerations exceeding that of 1.0g – the acceleration due to gravity. The main structural problem with wood frame buildings was that the first storey of multi-storey buildings collapsed, similar to results of previous quakes. One apartment complex killed 16 people as a result of weak first storeys. Also, irregularities such as large openings, asymmetrical structures, and other architectural features existed in some buildings. One of these types of buildings, called “dingbats”, consisted of “stucco boxes on stilts” and were largely built in California in the 1960’s and 70’s (Figure 1.4). These new and
irregular features led to unexpected behaviour, since some of the features had not been addressed by the design codes of the day (NAHB, 1994).

![Failures of Light Wood Frame Dwellings in Northridge, California caused by weak first stories (car ports). Notice crushed automobiles underneath.](image)

**Figure 1.4:** Failures of Light Wood Frame Dwellings in Northridge, California caused by weak first stories (car ports). Notice crushed automobiles underneath.

The Hyogo-ken Nanbu quake in Japan in 1995, more commonly known here in North America as the "Kobe" quake since it was centred in the large city of Kobe, was the costliest earthquake in modern times with over $100 billion US in damage. Almost all houses that were constructed in the western (North American) style of framing survived the earthquake without much damage at all. The only timber houses that did undergo much damage were the Pre-World War II houses that consisted of post and beam construction with bamboo/clay webbing as infill between the posts (*Figure 1.5*). After this earthquake, western style construction, also known as "2x4 construction", became popular in Japan for residential construction.
As far as hurricanes go, less information on the structural assessment of timber houses is available. One report by the U.S. Dept. of Housing and Urban Development accounts the havoc wreaked by Hurricane Andrew and Hurricane Iniki. In both hurricanes, many light frame timber houses were destroyed. The most common form of structural damage was roofs blown off and windows and doors breaking. Most timber shear walls let houses stay standing unless a blown-off door or window caused the inside pressures to blow the house apart. Thus, timber shear walls do play a role in keeping the house standing through a hurricane, even though some other factors are more important, such as roofs.

1.2.3 History of Timber Shear Wall Testing

What has been summarized is what has been observed in the field. The other important aspect to the history of shear walls is the testing and analysis of shear walls in laboratories. Racking resistance of timber shear walls have been studied utilizing...
empirical tests as far back as 1929. In these early days, George W. Trayer, an engineer with the U.S. Forest Products Laboratory in Madison, Wisconsin, tested and published results on shear walls with various styles of bracing subjected to “end thrust” to simulate racking forces caused by wind. (Trayer, 1929) This essentially pioneered the laboratory testing of shear walls. In those times, walls consisted of 2”x4” studs and wood lath sheathing (2” x ½” x 4’ slats nailed to the studs) combined with wooden bracing between the studs. Trayer compared the results of various sheathing, wood types, and the effects wetting and drying the lumber. Trayer tested walls with openings and even conducted vibration tests using a shaking table! This was definitely a pioneering effort, since vibration tests did not get popular until several decades later. Various other tests were performed on shear walls racking until 1939, when W.E. Wakefield in Ottawa, Canada, compared the rigidity of one-quarter scale walls sheathed with various types of wallboard to those sheathed with shiplap. The conclusion was that wallboards were stiffer than shiplap. The tests included the use of hold down connections and plotted load-displacement curves. (Wakefield, 1939) Similar tests was performed ten years later in Vancouver, B.C. by J.B. Alexander. Beginning the next year, 1950, literature abounds on shear wall tests. The National Building Code began to include standard nail spacing and other parameters for shear walls. J.M. Rudnicki in Ottawa compared special plywood nails to common wire nails in wall racking tests, with the conclusion that wire nails were superior. D.E. Kennedy performed similar tests the same year, also in the Ottawa Forest Products Laboratory, with the conclusions that plywood is just as strong as shiplap, and that sheathing does not contribute to the vertical load strength of the wall.
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1939; comparing shiplap sheathed wall with plywood sheathed wall.

The next year, at the Oregon Forest Products Laboratory, G.H. Atherton and J.W. Johnston performed extensive diagonal sheathed shear wall tests to produce curves charts and tables to aid engineers in the design of wood-framed structures, rather than rule-of-thumb methods as they had been using before. The variation of parameters such as the number of nails, nail spacing, sheathing pattern, and types of end posts were compared with each other with respect to the performance of the walls as a whole. This was a particularly important step in shear wall testing. Throughout the 1950’s, D.E. Kennedy and A.P. Jessome continued to test and compare shear walls with various sheathing. In 1954, J.R. Stillinger and A.P. Johnston examined stud spacing, nails per joint, panel height to length ratios, and nail holding power of one-quarter scale walls at the Oregon Forest Products Laboratory. R.A. Currier at the same lab studied the effects of nail types, wall length vs. strength, and used “stud anchors” to prevent uplift at the end studs during...
racking. He concluded that wire nails were superior to other nails and that wall strength correlated linearly with wall length. In 1958, at the Forest Products Laboratory in Madison, Wisconsin, R.F. Luxford and W.E. Bonser examined the possibility of lessening the vertical load bearing materials in a shear wall, i.e. smaller and less studs. They concluded that stud spacing affected racking strength of walls more than stud size. Also, smaller studs and different stud spacing have a disadvantage because standard mill-sizing of 2"x4" (50mm x 100mm) studs and 16" (406mm) stud spacing was the norm and interior cladding was designed for 16" spacing. In 1963, a comprehensive study was made of the knowledge of shear walls to date in the U.S. and Canada, and suggested new topics of study in this field. This was completed at Pennsylvania State University and the report, prepared for the Federal Housing Administration, touched briefly on many topics, some of which had previously not been covered, including:

- The importance of wall anchorage to foundations as part of the total lateral resisting system, rather than focusing strictly on racking of the panel element.
- Lateral deflection limits and ways to limit plaster and drywall cracking
- Windows and doors jamming due to racking and deflection limits set to avoid this
- Effect of wetting and drying cycles of walls prior to testing
- Failure Modes
- Nail resistance testing
- Importance of finding ways to make walls perform better against racking forces
- Standardized test procedures and measurement methods
By 1965, more was known about the resistance of timber shear walls, but so much more needed to still be known. Standardized testing methods had been introduced and many factors contributing to shear wall resistance were published in a “Guide to Improved Framed Walls for Houses” by the Forest Product Laboratory in Madison Wisconsin. Up to this point in time, the emphasis on timber shear wall testing was on wind force resistance. The 1964 Alaska Earthquake changed this emphasis. In the aforementioned report, it says that design guidelines for vertical loading of timber walls are secondary in importance to the guidelines for racking resistance.

Testing began to really boom after 1964, and to list just the major reports would be unfeasible. Moreover, the highlights of this era of testing has been documented in many theses of the last few years, therefore it is redundant and unnecessary to summarize them here. Only publications that are pertinent to this thesis will be mentioned.

In the last two decades, the scope of shear wall testing (and any structural testing for that matter) has broadened to include analysis and testing techniques that new technology makes possible. One of these new techniques is dynamic testing, which involve reproducing historical earthquake records using shake tables. Although rudimentary shake tables were used as early as 1929, modern technology now makes it possible to quite closely replicate an actual seismic event, unlike the harmonic motion that tables of long ago were limited to. Today, shake tables are used around the world. Another tool that has become possible due to leap and bounds in technology is computer modeling of systems; computer programs are becoming more and more advanced, and it has become
possible to model the behaviour of shear walls responding to a seismic event. Due to the computing power and speed required to perform the analysis, Finite Element Analysis and Time History Analysis have been possible. This advancement in computer analysis is important for many reasons. Computers can be used as analysis tools to model the behaviour of a structure instead of building physical models of the structure and testing them, which is obviously much more expensive and time-consuming, unless one is dealing with a small model and few possible regimes. Dr. R.O. Foschi used computer programs in the 1970's to analyze load-slip characteristics of nails and calculate stress-distributions of diaphragms. Foschi continued to use the computer to calculate Finite Element Models of nail hysterisis behaviour.

In 1989, J.D. Dolan, in his thesis entitled “The Dynamic Response of Timber Shear Walls”, performed tests and developed in-depth computer analysis on the behaviour of timber shear walls under static and dynamic loads. He engineered a device to test timber shear walls on the on the University of British Columbia (UBC) shake table (Figure 1.7). This frame, shown in greater detail in Chapter 8, has been used for all dynamic shear wall tests at UBC, including this thesis. Dolan’s tests and thesis, especially since they were performed and written on the seismically active West Coast, was met with great interest and sparked a wave of dynamic testing of timber structures at UBC that continues to this day.
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Figure 1.7: Shake Table Testing of Timber Shear Walls at UBC using test frame developed by J.D. Dolan: MIDPLY wall test (left) Diagram of testing frame (right).

In 1994, a five-year research program on the lateral load resistance of timber structures was undertaken at Forintek Canada Corp. in Vancouver, B.C. Erol Karacabeyli and Ario Ceccotti were the principle investigators of this study that tested over 60 conventional timber shear walls and other non-conventional shear walls. Walls with openings, gypsum wallboard, various stud, sheathing, and nail types, and various loading protocols were used to determine the effect of each on the performance of the wall, so engineers can benefit from this information when designing timber structures. Karacabeyli and Ceccotti investigated the use of several different cyclic protocols to determine the energy dissipation and resulting wall response to each protocol. One of the discoveries they made was that the peaks of every cycle in the cyclic response of a wall closely followed the first ramp cycle of a wall (monotonic loading). They also tested walls under pseudo-dynamic response, which is testing walls statically (no inertia forces), but using a loading
protocol that is the calculated response of the wall had it been tested dynamically. The input loading is calculated through a time history analysis of the wall, using its stiffness characteristics as measured from cyclic tests.

Others have investigated the use of various loading protocols, energy dissipation, and stiffness degradation, such as Foschi & Filiatrault (1990), Skaggs & Rose (1996), Ming He, Lam, & Prion (1998 - mentioned below), and Dinehart & Shenton (1998).

Many endeavours have been made to modify conventional 2”x4” shear walls in order to increase their lateral strength and ductility without compromising size and cost. As one local example, Ming He, under the direction of Dr. H. Prion, studied the effects of oversized sheathing panels on shear wall behaviour (He, 1999). As part of that ongoing project, J. Durham further investigated shear walls oversized sheathing, conducting static and dynamic tests on them. They concluded that the oversized sheathed walls behaved relatively the same as conventional walls (Prion & Durham, 1998).

Another local example of modifications to conventional shear walls is the “ANTI-RAKKER™” device, also tested at the UBC Earthquake Engineering Laboratory. This device consisted of a steel connection at the corners of the shear wall to dissipate energy and to keep the sheathing connected to the wall under earthquake loading. This device increased the ductility, displacement capacity, and stiffness of the shear walls after the wall had degraded in these properties (Durham, 1996).
Dinehart & Shenton experimented with passive dampers in shear walls. These dampers were of the viscoelastic type and were oriented diagonally across the shear walls. The tests were conducted using a sequential phased displacement cyclic loading protocol, which is explained in Chapter 3. The conclusions were that the dampers increased the maximum load and stiffness by 30% and the energy dissipation by 50%, and that the dampers provide a stable source of energy dissipation at constant amplitude cycling. (Dinehart & Shenton, 1998).

In 1992, J.D. Dolan and M.W. White experimented with gluing of panels to the studs to increase the racking resistance of shear walls. This was previously tried back in 1936 by Luxford et al. The results were that the walls were stiffer than a conventional wall, and would possibly fail at the hold-down connections in an actual earthquake because the wall would experience higher shear forces than a conventional wall. (Dolan & White, 1992).

Other tests have been performed world wide that simulate the actual conditions of a house, such as walls with openings (Dolan & Heine, 1998), multi-storey testing (Ceccotti & Karacabeyli, 1998), and two-storey shake table testing (Yasamura, 1991). In general, tests show that doors and windows significantly decrease the lateral resistance of the walls (20% to 75% decrease, depending on the number of openings) (Dolan & Heine, 1998).
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In Japan, the far-reaching advancements have been made in earthquake testing of timber shear walls. Kamiya et al., Yasamura et al., and several other Japanese engineers and scientists at the Building Research Institute in Japan performed extensive testing using shake table tests since 1981. At the National Research Institute for Earth Science and Disaster Prevention (NIED), run by the Science and Technology Agency of Japan, shake table testing of two storey shear walls have been accomplished, shown in Figure 1.8. Much has been learned from them throughout collaborations and partnerships in research programs. For example, Forintek Canada Corp. has close relations with them – often visiting each other's laboratories to learn new techniques and advancements. Through the combined efforts of these laboratories, more realistic and appropriate mathematical models are developed to predict the behaviour of these structures in an actual event.

Needless to say, such tests are expensive, but short of an actual earthquake, they provide the most realistic answers to the complex problem of seismic behaviour of wood-frame buildings (Rainer & Karacabeyli, 1999).
To summarize the progression of shear wall technology, resistance characteristics of certain walls are tabulated in Table 1.1. This is just to give a rough overview of the strength of shear walls, even though testing methods and otherwise-standard wall properties such as nail properties, plywood quality, stud dimensions, and plate anchoring vary considerably throughout the years. Testing parameters such as the rate of loading affect the test results also. Because the quality and technology of manufactured materials have increased over the years, shear wall performance has increased also.
Table 1.1: Historical Shear Wall Strengths/ Stiffnesses from Static Ramp Tests

<table>
<thead>
<tr>
<th>Name(s) &amp; Year</th>
<th>Description of Wall</th>
<th>Maximum Load (kN/m)</th>
<th>Deflection at Max. Load (mm/m)</th>
</tr>
</thead>
<tbody>
<tr>
<td>A.P. Jessome &amp; D.E. Kennedy 1954</td>
<td>8'x8'ft(2.4mx2.4m), 2&quot; x 4&quot; pine studs-16&quot; o.c., 5/16&quot; D.Fir Plywood, sheathing- horiz, 3.5&quot; common nails - 6&quot; &amp; 12&quot; spacing.</td>
<td>4.1</td>
<td>17.6</td>
</tr>
<tr>
<td>&quot; &quot;</td>
<td>Same as above, but with 7/8&quot; diagonal shiplap sheathing, 5 1/4&quot; dressed, oriented diagonally at 45°, 2-2.5&quot; common nails per board per stud, 4.75&quot; spacing</td>
<td>8.2</td>
<td>21.9</td>
</tr>
<tr>
<td>R.F. Luxford &amp; W.E. Bonser 1958</td>
<td>8'x12', 2x4 studs-16&quot; o.c., ¼&quot; D.Fir Plywood sheathing-vertical, sixpenny nails-5&quot; &amp; 10&quot; spacing,</td>
<td>10.8</td>
<td>n/a</td>
</tr>
<tr>
<td>Isenberg et al. at Penn. State - 1963</td>
<td>8'x8', 2x4 studs-16&quot; o.c., 1&quot;x8' Shiplap sheathing-horiz.w/ diagonal 2x4 let-in brace, 2 nails per board per stud - 6&quot; spacing</td>
<td>9.5</td>
<td>11.5</td>
</tr>
<tr>
<td>&quot; &quot;</td>
<td>Same as above, but with 5/16&quot; D.Fir Plywood sheathing- horiz., 6&quot; &amp; 16&quot; nail spacing</td>
<td>4.8</td>
<td>14.3</td>
</tr>
<tr>
<td>J.D. Dolan at UBC - 1989</td>
<td>8'x8', 2x4 studs- 24&quot; o.c., 3/8&quot; D.Fir Plywood sheathing- vertical., 2x4 blocking, 2.5&quot; galvanized common nails- 4&quot; &amp; 6&quot; spacing, metal corner connectors at top, no dead load</td>
<td>13.7</td>
<td>34.4</td>
</tr>
<tr>
<td>&quot; &quot;</td>
<td>Same as above, but with 3/8&quot; Waferboard sheathing-vertical, and a 45kN dead load applied vertically on wall.</td>
<td>13.9</td>
<td>31.6</td>
</tr>
<tr>
<td>E. Karacabeyli &amp; A. Ceccotti at Forintek, 1996</td>
<td>8'x16', 2x4 SPF studs-16&quot; o.c., 3/8&quot; CSP Plywood sheathing- horiz. 2x4 blocking, 3&quot; common nails-6&quot; &amp; 12&quot; spacing, 2.27 kN/m dead load applied vertically on wall.</td>
<td>8.4</td>
<td>15.6</td>
</tr>
<tr>
<td>J.Durham et al. at UBC - 1998</td>
<td>8'x8', 2x4 studs-16&quot; o.c., 3/8&quot; OSB sheathing-horiz., 2x4 blocking, 2&quot; gun-driven spiral nails-6&quot; &amp; 12&quot; spacing, 9 kN/m dead load applied vertically on wall.</td>
<td>7.1</td>
<td>23.6</td>
</tr>
<tr>
<td>&quot; &quot;</td>
<td>Same as above, but with one oversize 8'x8' sheet of OSB instead of two regular sheets, same nail spacing</td>
<td>9.0</td>
<td>22.4</td>
</tr>
<tr>
<td>&quot; &quot;</td>
<td>Same as above (one oversize OSB sheet), but with a nail spacing of 3&quot; instead of 6&quot; only around perimeter.</td>
<td>14.5</td>
<td>25.0</td>
</tr>
</tbody>
</table>

Notes: 1. Load and Maximum Deflection are listed per metre of wall length for comparison purposes. 2. All dimensions listed in Imperial because of familiarity with terms. 3. Nail spacing is listed in perimeter spacing & interior spacing, respectively. 4. All walls used hold-down connections to prevent uplift leeward end of wall.
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The most common ways that shear walls failed in tests are as follows:

- Nails pulling through the sheathing ("pull-through" failure)
- Nails tearing through the edges of the sheathing ("tear-out" failure")
- Nail fatigue – applies to cyclic loading
- Sheathing Rupture – due to high stresses and strains
- Frame Failure

These ways in which shear walls failed are called failure modes. To this day, these failure modes exist in conventional shear wall testing. No solution to any of these failure modes has yet been used extensively and successfully in the field.

As can be clearly deduced from the destruction by earthquakes and high winds all around the world, the continuing study of shear walls and other lateral load resisting systems is vital to building technology advancements. The next section of this chapter presents the specific innovation that is the subject of this thesis.

1.3 A New Concept

The situations in which conventional shear walls run the risk of providing inadequate lateral resistance are:

- Platform-frame construction with large openings (windows, bays, doors, garages)
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- Manufactured house systems in areas with a high risk of earthquake or high winds
- Multiple-story platform-framed buildings
- Retrofits of older buildings

In these situations, there is a need for increased lateral resistance without encroachment on space, window area, garage area, or wall width. Also, buildings wherein shear walls are not initially used (older Post-and-Beam construction) can benefit from the use of inserted sections of shear walls as a seismic retrofit.

For these applications, a team of scientists and engineers from Forintek Canada Corp. and the University of British Columbia invented the MIDPLY™ wall. The two main inventors were Dr. Erol Varoglu from Forintek Canada Corp. and Prof. S.F. Stiemer of the University of British Columbia. A U.S. Patent was granted in 1998: US5782054: Wood Wall Structure.

The MIDPLY™ system's improved performance against wind and earthquake loads is achieved by rearrangement of wall framing components and sheathing used in standard shear walls, as explained in Chapter 2. The purposes of using conventional building materials are:

- To reduce cost
- To make it easy for contractors to work with the walls
- To keep the same dimensions as conventional shear walls
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The last two purposes are essential in creating a smooth transition from using conventional walls to using the MIDPLY™ wall in the field.

The structural properties of MIDPLY™ wall system were developed through a 3-year testing and analysis project where full-scale test specimens are being subjected to monotonic (ramp), cyclic and dynamic displacement schedules. The objective of this project was to establish the proof-of-concept for the MIDPLY™ wall system design and construction method. The content of this thesis represents the second phase of the MIDPLY™ project. The first phase of the project involved researching various ideas for sheathing-stud-plate connection details, preliminary testing of walls containing these various connections, and coming up with a few standard MIDPLY shear wall concepts to test in the second phase of the project. Chapter 2 will describe the MIDPLY™ wall system in greater detail.
2.1 The MIDPLY™ Wall Configuration

To compare the construction of the MIDPLY™ wall system to conventional shear walls, the characteristics of a typical conventional shear wall must be defined. Several variations of the conventional shear wall are used in the field, with variations in stud size, sheathing type and thickness, nail length, and nail spacing. The MIDPLY™ wall will be compared to the most typical kind of wall. To define what will be hereon referred to as a "conventional shear wall", Table 2.1 lists the components of the typical shear wall presently used in North America in light-frame timber construction.

Table 2.1: Conventional Shear Wall Properties

<table>
<thead>
<tr>
<th>Sheathing:</th>
<th>3/8&quot; to 1/2&quot; S-P-F plywood or OSB</th>
</tr>
</thead>
<tbody>
<tr>
<td>Vertical members (studs)</td>
<td>2&quot;x4&quot; S-P-F studs, stud grade, spaced at 16&quot; on centre</td>
</tr>
<tr>
<td>Nail types and spacing:</td>
<td>2 1/2&quot; nail-gun nails spaced at 6&quot; apart</td>
</tr>
<tr>
<td>Horizontal Plates:</td>
<td>2&quot;x4&quot; S-P-F, grade 2 or better.</td>
</tr>
<tr>
<td>Other Items:</td>
<td>Stud hold-down connections -- optional Blocking -- required for horizontally applied sheathing</td>
</tr>
</tbody>
</table>

The MIDPLY™ wall system differs from the conventional shear wall by using the same materials in a different configuration. The MIDPLY™ wall consists of components
arranged in a way that the lateral load resistance of the system significantly exceeds that in current conventional wall arrangements. Figure 2.1 illustrates the configuration of the MIDPLY™ wall system in comparison to those of a conventional shear wall. The new construction system uses standard building methods as well as standard building materials.

![Diagram of MIDPLY™ wall system compared to conventional shear walls](image)

**Figure 2.1:** MIDPLY™ wall system compared to conventional shear walls

The superior lateral resistance of the MIDPLY™ wall system is attained through the following means:

- A wood-based 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 (*Figure 2.2*), or in triple shear with the addition of exterior sheathing, providing the increased lateral load carrying capacity for the wall.
• Studs in the MIDPLY™ wall system are placed at a $90^\circ$ rotated (about the longitudinal axis) position relative to those 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 increases the lateral load capacity of the MIDPLY™ wall by providing additional edge distance for fasteners on studs, as well as on edges of sheathing panels placed at the mid-plane and also at the exterior face of the wall. This reduces the chip-out failure mode described in Chapter 1.

• The heads of the nails are on the surface of the studs. Therefore they are kept away from the surface of the plywood or OSB and the nail pull-through failure mode is physically prevented.

• Since the nails are nailed into the wide face of the stud, the chances of missing the studs while constructing the walls are greatly reduced. The fewer nails that are missed, the stronger the wall is.

![Figure 2.2: Contrast between Nails in Single Shear and Nails in Double Shear Connections](image-url)
Chapter 2: The MIDPLY™ Shear Wall System

As displayed in Figure 2.2, “single shear” means one shear plane and double shear – two shear planes. The photo on the right side of Figure 2.2 showing the double shear nailed connection after failure was taken from tests performed at Forintek Canada Corp. in the first phase of the MIDPLY™ project. When a shear wall is subjected to loads from earthquakes and high winds, the racking force is taken by the nailed connections, therefore the nailed connections play a major role in the strength of a shear wall. The Canadian Timber Design Code, CSA O89.1 states that the resistance of a nailed connection in double shear can be assumed to be twice that of single shear. Therefore, theoretically speaking, each nail in a MIDPLY™ wall has twice as much strength as a nail in a conventional shear wall, and that is without considering the advantages of pull-through and tear-out failure mode elimination!

The purpose of the MIDPLY™ project was to create a proof-of-concept for the wall so that a final end product could be built and relied upon in to provide superior earthquake and wind-resistant performance. This was done through extensive testing and re-engineering of the system to arrive at the strongest, most ductile, cost-effective, and construction-feasible timber shear wall. Wall elements such as stud size, sheathing size, hold down connections, etc. were continuously re-examined to arrive at the best solution for each element. Certain elements of the MIDPLY™ wall were deemed standard from the start, such as nail size, while others were experimented with and changed throughout the course of the 3-year project. Table 2.2 lists the various elements of the MIDPLY™ wall.
Chapter 2: The MIDPLY™ Shear Wall System

Hold-down connections refer to metal connectors at the end-studs of a shear wall to prevent uplift of the end-studs from racking forces. A shear wall may or may not have hold-downs, depending on its design and length, but the MIDPLY™ always has hold-downs as part of its LLR system. The hold-down is described in Chapter 5, which deals with the design of a better hold-down connection.

Anchor bolts were shown in Figure 1.2 and are their purpose is to keep the shear wall from sliding on its foundation. Anchor bolts are at both the top and bottom of all shear walls to keep the top and bottom plates from sliding under racking forces.

Table 2.2: The Elements of the MIDPLY™ System

<table>
<thead>
<tr>
<th>Element</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>Studs</td>
<td>Size: 39mmx89mm (2&quot;x4&quot; nominal) and 38mmx64mm (2&quot;x3&quot; nominal) Spacing: 16&quot; (406mm) and 24&quot; (610mm) Species: S-P-F (Spruce-Pine-Fir) Grade: Stud Grade, Machine Stress-Rated, and Finger-Joined</td>
</tr>
<tr>
<td>Plates</td>
<td>Size: 2&quot;x4&quot; nominal (39mm x 89mm) and 2&quot;x3&quot; nominal (38mm x 64mm) Species: S-P-F (Spruce-Pine-Fir), and Douglas Fir Grade: Standard &amp; Better, No.2 &amp; Better, Finger Joined</td>
</tr>
<tr>
<td>Sheathing</td>
<td>Size: 4'x 8'(1.2m x 2.4m) sheets - vertically sheathed Thickness: 15/32&quot; (12mm) (1/2&quot; nominal) Exterior Sheathing Thickness: 3/8&quot; nominal (9.5mm) Type: OSB, CSP Plywood, Douglas Fir Plywood</td>
</tr>
<tr>
<td>Nails</td>
<td>Size: 3 1/4&quot; (83mm) long, 1/4&quot; (3mm) diameter Spacing: 3&quot; (75mm) and 4&quot; (100mm) Type: Stanley Stick™ Power Nails (pneumatically driven)</td>
</tr>
<tr>
<td>Hold Downs</td>
<td>Type: 1. Steel triangle hold-down 2. Steel inverted-triangle hold-down Location: bottom of end-studs (4 per wall)</td>
</tr>
<tr>
<td>Anchor Bolts</td>
<td>Type: 3/8&quot; (9.5mm) structural steel bolts through 7/16&quot; vertical holes drilled into top and bottom plates Spacing: Every 2 ft (610mm) on each plate (top and bottom plates)</td>
</tr>
</tbody>
</table>
2.2 MIDPLY™ Project Background - Year 1

This section deals with the progress of the evolution of the MIDPLY™ wall up until the beginning of the second year of the project (1998-1999), which is when the scope of this thesis started. The MIDPLY™ project began in April 1997. To provide valuable advice and to assist in the progress of the project, an industrial advisory group including two prefabricated home manufacturers and a major wood products producer was established. The scope of work completed in the first year of the project was as follows:

- Preliminary analysis of the MIDPLY™ system. A comparative cost analysis was performed with conventional wood-frame shear-wall systems.

- Racking tests of Phase I and Phase II MIDPLY™ wall system. A total of twenty-eight tests on 4.88 m x 2.44 m and twenty-four tests on 2.44 m x 2.44 m MIDPLY™ walls were performed under static ramp (monotonic) and reversed-cyclic loads in Phase I and Phase II. Analysis was conducted to evaluate the performance of various configurations of MIDPLY™ walls in Phase I. The configurations that performed best in Phase I were chosen as standards to test full-scale wall specimens in Phase II.

- Nailed joint tests. Eight nailed joint types were tested and analyzed to find out the best connection details, nailing patterns, and sheathing installation patterns.

- Design and fabrication of hold down for MIDPLY™ wall system.
2.2.1 Cost Comparison between MIDPLY™ and Conventional Shear Walls

The preliminary cost comparison of the MIDPLY™ wall was based on test results of MIDPLY™ walls tested in the first year. It was calculated that the MIDPLY™ walls without exterior sheathing were 20% cheaper than conventional walls of the same type, for equal lateral resistance. MIDPLY™ walls with exterior sheathing were 30% cheaper than conventional walls for equal resistance. These calculations were based on results from conventional walls with 16” (400mm) stud spacing and MIDPLY™ walls with 24” (600mm) stud spacing. The costs taken into account include materials and labour for framing on site. The labour required for a MIDPLY™ wall with or without exterior sheathing was estimated to cost 10% to 50% more than the labour cost of a conventional shear wall.

2.2.2 Testing of Full Scale Walls – Phase I

A total of twenty-eight tests were performed on ten types of 4 ft (2.44m) walls to examine some initial configurations of the MIDPLY™ system. A diagram of the testing apparatus is shown in Figure 2.3. The configurations examined are shown in Figure 2.4. The tests included ramp (monotonic) and reversed-cyclic tests for each configuration. The test methodology (loading protocols and apparatus details) is further described in Chapter 3.
Figure 2.3 shows arrows on both the top and side of the wall specimen. The arrow on the side represents the racking force that is applied to the wall with a hydraulic actuator. This is the main part of all tests – the racking force. This force could be a simple ramp force (monotonic), or it can be programmed to cycle back and forth with increasing amplitude (reversed-cyclic). The other two arrows on the top represent the vertical loads applied in some tests. This vertical load is a constant load of 10,000 lb (44.48 kN) (representing the weight of one storey above the wall).
Chapter 2: The MIDPLY™ Shear Wall System

End stud configurations

<table>
<thead>
<tr>
<th>Type 1</th>
<th>Type 2</th>
</tr>
</thead>
</table>

Stud configurations

<table>
<thead>
<tr>
<th>Type 1</th>
<th>Type 2</th>
</tr>
</thead>
</table>

Top plate configurations

<table>
<thead>
<tr>
<th>Type 1</th>
<th>Type 2</th>
<th>Type 3</th>
<th>Type 4</th>
</tr>
</thead>
</table>

Bottom plate configurations

<table>
<thead>
<tr>
<th>Type 1</th>
<th>Type 2</th>
<th>Type 3</th>
<th>Type 4</th>
</tr>
</thead>
</table>

**Figure 2.4:** Stud and Plate Configurations Tested

The configurations were chosen that had the best combination of test performance and construction feasibility. After the test results were observed, Type 2 was chosen for all four elements as the standard to use for all future tests. Some test results for the ramp tests are shown on *Figure 2.5.* The load-displacement curves are not labelled according
Chapter 2: The MIDPLY™ Shear Wall System

to the different configurations used because there were several other parameters tested in those tests, such as exterior sheathing, sheathing types, and sheathing orientation.

Phase I also involved testing the walls in a reversed-cyclic loading protocol, described in Chapter 3. The reversed cyclic loading subjects the wall to racking forces that slowly oscillate the wall from tension to compression with increasing displacements until the wall fails structurally. Figure 2.6 displays a typical test result of a MIDPLY™ wall under reversed-cyclic loading. The curve shown in Figure 2.6 is known as a hysteresis curve. Hysterisis curves give engineers the parameters (quantitative response characteristics) they need to model and predict the behaviour of the wall in response to earthquake and hurricane-induced racking forces.
2.2.3 Nailed Joint Testing

Nine types of different nailed joints were evaluated under both monotonic and reversed cyclic loads. Six replicates (three monotonic and three cyclic) were used for each joint type. A diagram of the nailed joint test is shown in Figure 2.7. Two nails were used in each specimen. The tests were used to determine the influence of number of shear planes, panel materials, panel thickness, panel orientation, fastener type, and load protocol on the load-slip response, which is directly related to the behaviour of shear walls.
Chapter 2: The MIDPLY™ Shear Wall System

Figure 2.7: Diagram of Nailed Joint Tests

Test results of three of the five parameters studied are tabulated in Table 2.3. Results are listed as the mean values of three replicates. The maximum load $P_{\text{max}}$ and secant stiffness $K$ represent the response of two nails in each joint. It was found that the maximum load ($P_{\text{max}}$) of the joint in double shear is about 80% higher than that tested in single shear. The secant stiffness ($K$) of the joint in double shear is about 3 times that in single shear. The typical test results of joints in double and single shear are shown in Figures 2.8 and 2.9.
### Table 2.3: Nailed Joint Test Results

<table>
<thead>
<tr>
<th>Shear Plane</th>
<th>½&quot; CSP, 3 ¼&quot; Stanley stick nail</th>
<th>¾&quot; CSP, 3 ½&quot; Stanley stick nail</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>( P_{max} ) (kN)</td>
<td>( \delta_u ) (mm)</td>
</tr>
<tr>
<td>DOUBLE</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Monotonic</td>
<td>6.05</td>
<td>27.2</td>
</tr>
<tr>
<td>Cyclic 1&lt;sup&gt;st&lt;/sup&gt;</td>
<td>5.70</td>
<td>13.3</td>
</tr>
<tr>
<td>Cyclic 3&lt;sup&gt;rd&lt;/sup&gt;</td>
<td>4.45</td>
<td>12.4</td>
</tr>
<tr>
<td>SINGLE</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Monotonic</td>
<td>3.68</td>
<td>21.8</td>
</tr>
<tr>
<td>Cyclic 1&lt;sup&gt;st&lt;/sup&gt;</td>
<td>3.03</td>
<td>14.0</td>
</tr>
<tr>
<td>Cyclic 3&lt;sup&gt;rd&lt;/sup&gt;</td>
<td>2.32</td>
<td>7.2</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Fastener</th>
<th>½&quot; CSP</th>
<th>¾&quot; CSP</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>( P_{max} ) (kN)</td>
<td>( \delta_u ) (mm)</td>
</tr>
<tr>
<td>3 ¼&quot; Stanley stick nail</td>
<td>Monotonic</td>
<td>6.05</td>
</tr>
<tr>
<td></td>
<td>Cyclic 1&lt;sup&gt;st&lt;/sup&gt;</td>
<td>5.70</td>
</tr>
<tr>
<td></td>
<td>Cyclic 3&lt;sup&gt;rd&lt;/sup&gt;</td>
<td>4.45</td>
</tr>
<tr>
<td>3 ¼&quot; Common nail</td>
<td>Monotonic</td>
<td>7.30</td>
</tr>
<tr>
<td></td>
<td>Cyclic 1&lt;sup&gt;st&lt;/sup&gt;</td>
<td>6.84</td>
</tr>
<tr>
<td></td>
<td>Cyclic 3&lt;sup&gt;rd&lt;/sup&gt;</td>
<td>5.87</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Panel Type</th>
<th>¾&quot; Panel</th>
<th>⅛&quot; Panel</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>( P_{max} ) (kN)</td>
<td>( \delta_u ) (mm)</td>
</tr>
<tr>
<td>½&quot; CSP</td>
<td>Monotonic</td>
<td>6.05</td>
</tr>
<tr>
<td></td>
<td>Cyclic 1&lt;sup&gt;st&lt;/sup&gt;</td>
<td>5.70</td>
</tr>
<tr>
<td></td>
<td>Cyclic 3&lt;sup&gt;rd&lt;/sup&gt;</td>
<td>4.45</td>
</tr>
<tr>
<td>¼&quot; OSB</td>
<td>Monotonic</td>
<td>5.76</td>
</tr>
<tr>
<td></td>
<td>Cyclic 1&lt;sup&gt;st&lt;/sup&gt;</td>
<td>5.50</td>
</tr>
<tr>
<td></td>
<td>Cyclic 3&lt;sup&gt;rd&lt;/sup&gt;</td>
<td>4.08</td>
</tr>
<tr>
<td>¼&quot; DFP</td>
<td>Monotonic</td>
<td>5.17</td>
</tr>
<tr>
<td></td>
<td>Cyclic 1&lt;sup&gt;st&lt;/sup&gt;</td>
<td>5.06</td>
</tr>
<tr>
<td></td>
<td>Cyclic 3&lt;sup&gt;rd&lt;/sup&gt;</td>
<td>4.15</td>
</tr>
<tr>
<td>¾&quot; CSP</td>
<td>Monotonic</td>
<td>4.77</td>
</tr>
<tr>
<td></td>
<td>Cyclic 1&lt;sup&gt;st&lt;/sup&gt;</td>
<td>5.44</td>
</tr>
<tr>
<td></td>
<td>Cyclic 3&lt;sup&gt;rd&lt;/sup&gt;</td>
<td>4.11</td>
</tr>
</tbody>
</table>

Cyclic 1<sup>st</sup> – load-displacement envelope curve obtained in the first reversed cycle
Cyclic 3<sup>rd</sup> – load-displacement envelope curve obtained in the third reversed cycle
Chapter 2: The MIDPLY™ Shear Wall System

Figure 2.8: Comparison of joint load-slip response under monotonic load

Figure 2.9: Comparison of joint load-slip response under reversed cyclic load
Chapter 2: The MIDPLY™ Shear Wall System

It was found that the load-slip characteristics were similar with ½” Canadian softwood plywood (CSP) and 15/32” Oriented strand board (OSB). The maximum load and displacement of ½” Douglas Fir plywood (DFP) was lower than ½” CSP. Joints with ½” CSP gave a higher maximum load than those with ¾” CSP. Panel orientation had little effect on load-slip response.

The influence of nail type on joint behaviour showed that the properties of the joints made with common nails are higher than those made with Stanley stick power nails. This may be due to the relatively larger diameter of the common nail. Stanley Stick™ power nails were chosen to be used for standard MIDPLY™ walls because of their greater construction feasibility.

The influence of load protocol on joint behaviour showed that joints tested with FCC97 protocol gave higher maximum load and ultimate displacement than those obtained with FCC93 protocol. As shown in Figure 2.10, the FCC93 protocol contains significantly more cycles than the FCC97 protocol before the joint reaches its maximum load, and this leads to more nail fatigue failures under FCC93. This is the primary reason that causes the joint to loose its load-carrying capability under FCC93.
2.2.4 Design and Fabrication of Hold Down for the MIDPLY™ System

A hold-down connection needed to be design that would keep the end-studs from uplifting, as explained on the fifth page of this Chapter. The hold-down that was devised is shown in Figure 2.11. It was designed by Gary Baerg, a UBC intern on the MIDPLY™ project in its first year. The hold-down designed for the MIDPLY™ is not an off-the shelf item because regular shear walls are configured differently, and the typical hold down connections designed for the regular shear walls would not fit on the MIDPLY™ system. The hold-down shown was designed for a maximum uplift load of 10,000 lbs (44kN).
Chapter 2: The MIDPLY™ Shear Wall System

2.2.5 Testing of Full Scale Walls – Phase II

Twenty 8ft x 8ft (2.44m x 2.44m) MIDPLY™ walls were tested in Phase II. The 8ft length wall was chosen as a standard from that point on because the test apparatus could not generate enough load to fail a larger specimen. Secondly, this length would come in handy when comparing the MIDPLY™ to tests performed on regular shear walls in the past because the wall length in most tests was 8ft. Thirdly, the UBC shake-table apparatus can accommodate only an 8ft wall or shorter.
All MIDPLY™ walls in Phase II used the hold-down connection shown in Figure 2.11.

The MIDPLY™ walls in Phase II were tested both monotonically and cyclically to determine the effects of the following parameters on wall performance:

a) Panel Types: ½” OSB vs. ½” CSP
b) Nail Spacing: 6” (150mm) vs. 8” (200mm)
c) Vertical Loads: No vertical load vs. Vertical load of 10,000 lb (44.48kN)
d) Exterior Sheathing: None vs. ⅛” CSP vs. ⅛” Gypsum Wallboard

The resulting load-displacement curves of these tests yielded the following information:

a) The ultimate load of ½” CSP was 10% higher than that of ½” OSB, and the ultimate load was higher also. The stiffness of the CSP wall was about 60% of the OSB wall.

b) The different nail spacing resulted in an insignificant difference in the ultimate load of the walls, therefore the 200mm spacing was made standard.

c) The ultimate loads of the walls with vertical loads were on the average 7% higher than those without vertical loads. This may be attributed to the fact that the vertical loads reduce the uplifting of the end studs.

d) The influence of ½” Gypsum Wallboard (GWB) was insignificant. The walls sheathed with ½” CSP however, were at least 20% stronger than the walls without exterior sheathing. The test apparatus could not fail the walls with CSP exterior sheathing.
A conventional shear wall and a comparable MIDPLY™ wall (same nail spacing, sheathing, stud spacing, etc.) were tested reverse-cyclically to compare their performance. The MIDPLY™ wall used power nails, as opposed to the stronger hand-driven nails used on the conventional wall. The resulting hysterisis curves, shown in Figure 2.12, showed that the MIDPLY™ wall sustained loads over twice that of the conventional wall. Furthermore, the test apparatus could not even fully fail the MIDPLY™ wall because it was so strong, therefore the MIDPLY™ wall could have sustained even higher loads.

Based on these preliminary results, the MIDPLY™ wall also seems to be better against non-structural damage due to drift (total wall deflection). For an equal amount of lateral force applied, the MIDPLY™ deflects less than a conventional shear wall. Therefore, elements in a building that are damage-prone due to large deflections (such as wall plaster, furniture, plumbing, electrical) would be safer in an earthquake with a MIDPLY™ wall as its LLR than a conventional shear wall. However, before such conclusions could be made, more testing needed to be completed.
Figure 2.12: Hysterisis Curves Comparing a Conventional Shear Wall with a MIDPLY™ wall

2.3 Scope of Work and Objectives for Year 2 of MIDPLY™ Project

Year 2 of the MIDPLY™ Project (1998-1999), which constitutes the scope of this thesis, began with the following objectives:

- Design a new hold-down connection to prevent uplift of end studs and perform better than the current hold-down connection.
- Design new anchor bolt connections that will make MIDPLY™ perform better.
Chapter 2: The MIDPLY™ Shear Wall System

- Perform Dynamic Tests of MIDPLY™ walls at the UBC Earthquake Engineering Laboratory and make computer model of MIDPLY™ system’s dynamic performance.
- Examine the performance of MIDPLY™ walls using 2x3 nominal (38mm x 64mm) lumber for studs and plates instead of 2x4 (38mm x 89mm).
- Design “T” and “L” connections to connect two adjacent MIDPLY™ walls and design a floor-to-floor connection.
- Make comparison of MIDPLY™ system to conventional shear walls

The actual completed work exceeded the objectives and is summarized below:

- 40 MIDPLY™ walls tested at Forintek Canada Corp. (Chapter 3)
- New hold down designed and successfully tested (Chapter 4)
- New anchor bolts designed and successfully tested (Chapter 5)
- Examined performance of Finger-Joined Lumber for studs and plates (Chapter 6)
- Examined performance of 2”x3” studs and plates (Chapter 6)
- Performed Dynamic Tests of 6 MIDPLY™ walls at UBC Earthquake Lab. (Chapter 8 & 9)
- Developed a Non-Linear Dynamic Computer Model for the MIDPLY™ system (Chapter 8)
- Developed Commercialization Pamphlet, Video, and Display Model (Chapter 10 & Appendix)
2.4 Plans for Third Year of MIDPLY™ Project

The plans for the next year of the MIDPLY™ project include the following:

- Testing MIDPLY™ walls with openings
- Testing MIDPLY™ walls as inserts into regular shear walls
- Further refining the hold-down connection
- Testing MIDPLY™ walls affected by shrinkage and damaged and repaired walls.
- Developing a full Commercialization Plan
Chapter 3: MIDPLY™ Static Wall Testing

This chapter covers the testing completed at Forintek Canada Corp. in the second year, which includes the 40 monotonic and reversed-cyclic tests. The dynamic testing is covered in Chapter 8.

3.1 Testing Apparatus

The testing apparatus was shown in Chapter 2, Figure 2.3, and will now be described in more detail. The apparatus, shown again in Figure 3.1, indicating its components by number, consists of:

1) A steel base support whereon the shear wall is rigidly mounted (any length up to 16ft (4.88m)). This is fixed support.

2) A wall-mounted guide for the top of the wall and four vertical hydraulic actuators to subject the wall to constant vertical loads of up to 10000 lb (45kN). As can be seen on Figure 3.1, only two of the vertical actuators are used on an 8ft wall. The actuators apply force to steel spreader beam connected rigidly to the top plate of the wall.

3) A Hydraulic actuator capable of 20,000 lb (89kN) situated in line with the top plate of the wall to exert racking forces on the wall through its spreader beam during a test. This actuator is displacement-controlled by an MTS Servo connected to a computer-controlled function generator. The stroke of the actuator is 10” (254mm), which means the walls can be pushed and pulled 5” (125mm) either way (starting from mid-
stroke) in a reverse cyclic test. The actuator can be controlled to a precision of 0.5mm.

4) A Load Cell capable of 20,000 lb (89kN) is situated between the actuator and the spreader beam at the top of the wall to measure the total lateral force exerted on the wall during a test. This information is sent to and recorded by the MTS data acquisition system and plotted in real-time.

5) A Displacement Transducer (LVDT - Linearly Variable Displacement Transducer) located at top left side of the wall to measure the lateral displacement of the wall during a test. This information is sent to the MTS data acquisition system and is plotted real-time along vs. total load. Its maximum stroke is 12" (305mm).

6) An LVDT at the bottom of the end studs of the wall (both ends) to measure the uplift of the end studs during a test.

7) The MTS controller and data acquisition system. Two separate systems: a) A computer-controlled function generator connected to a servo which controls the actuator (displacement controlled). b) The data acquisition system records the data at a sampling rate of 200 samples/sec from the three LVDTs, actuator, and the load cell. (5 channels total). All data is sent to the computer and saved as a separate file for any particular test.

With time as a separate column, there are six columns of data for all MIDPLY™ wall tests. The vertical load is monitored by a separate hydraulic system and pressure gauge but the load is not recorded because it always remains constant throughout shear wall
Chapter 3: MIDPLY™ Static Wall Testing

tests. This test apparatus and data recording system is an approved system according to ASTM Standard E564-95.

Figure 3.1: Testing Apparatus with an 8 ft (2.44m) MIDPLY™ wall

3.2 Loading Protocols

A “loading protocol” refers to the manner in which a force is applied to a test specimen. Other synonyms of “loading protocol” that are often used are: loading schedule, displacement schedule, loading regime, and displacement protocol. The loading protocol is a graph of load or displacement vs. time. All MIDPLY™ test protocols are displacement vs. time, since the actuator is a displacement controlled, rather than load-controlled, as shown in the preceding section of this chapter.
Chapter 3: MIDPLY™ Static Wall Testing

There are two types of loading protocols used in the static tests of the MIDPLY™
system: Monotonic and Reversed Cyclic. The Monotonic protocol is simply a linear
displacement vs. time line graph, shown in Figure 3.2. The Reversed Cyclic protocol is
one that cycles back and forth from positive displacement to negative displacement with
increasing amplitude, as shown on Figure 3.3. The term “reversed-cyclic” is used as
opposed to “cyclic” because of the fact that the displacement goes into the negative range
instead of remaining in the positive range. Therefore, in a reverse-cyclic test, the actuator
is started at a zero point of mid-stroke (5” or 125mm) so it has room to move equidistantly
in either direction.

The loading protocol used in a shear wall test affects the way the wall responds in terms
of stiffness, energy dissipation, and ductility. The characteristics of loading protocols
that affect the shear wall response are: rate of loading, frequency of cycles, number of
cycles, and the rate of increasing and/or decreasing amplitude.

The monotonic protocol pushes the wall at a rate of 5 thousandths of an inch per second
(7.62mm/minute), until the wall fails. This rate is slow enough for the timber strain rate
to “catch up” with the loading. If the wall is loaded too quickly, the result would be a
higher maximum load and a higher initial stiffness. The monotonic protocol is more
representative of a high wind load rather than an earthquake load because of its slow rate
of loading and because it is single-directional.
More than one kind of cyclic loading protocol was used in MIDPLY™ tests. The first two, shown in the preceding chapter in Figure 2.10, were created at Forintek Canada Corp. and therefore carry the "FCC" prefix. These protocols were used in the first phase of the first year of the project and their performance was compared. The reason FCC93 was chosen is because of its great energy demand. FCC97 was chosen because it was similar to other commonly used protocols in terms of cycle frequency and rate of loading and it was used to compare its performance to FCC93.

In the second phase of the first year, the ISO97 protocol, shown on Figure 3.3, was used on all MIDPLY™ tests. This was an internationally accepted protocol and many labs were using it in shear wall tests, therefore the results of MIDPLY™ tests could be compared with many others’ results if the same protocol was used. The protocol
consisted of three cycles per set, followed by a smaller cycle at 25% amplitude of the preceding cycles, as shown in Figure 3.3.

This protocol was used until the end of the third quarter of the second year of the MIDPLY™ project, when the ISO98 protocol was introduced. ISO98, shown on Figure 3.3, differs slightly from ISO97 because the smaller fourth cycle in each series of cycles was eliminated, as shown in Figure 3.3. This smaller cycle is called a "decay cycle" or "degradation cycle", and it was put in the ISO97 protocol to analyze the effect of "slackness" in the wall. This means that, by including smaller cycles after relatively larger cycles, one can determine the amount of slackness developed in the joints and whether the wall has the ability to dissipate energy while the joints are slack.

Figure 3.3: Comparison of the Two Reversed Cyclic Displacement Protocols used in MIDPLY™ 2nd year tests
However, it was discovered that light frame timber shear walls, as opposed to bolted LLR systems, generally do not develop slackness, and therefore the ISO98 protocol eliminated the decay cycle.

Other reversed cyclic load protocols commonly used in shear walls tests around the world include the ASTM and CEN protocols. There are more, such as the SEAOSC (Southern California) and TCCMAR (U.S. & Japan Masonry Committee) test methods, but these won’t be shown due to their limited use. The ASTM protocol uses the “Sequential Phased Displacement” (SPD) method, which is term that refers to the use of sequences of cycles that increase in amplitude by a certain percentage until specimen failure. These sequences include decay/degradation cycles in each sequence. The ASTM protocol was widely used because of its high number of cycles, high energy dissipation, and close resemblance to earthquake time-histories. The ASTM protocol resembles the FCC93 and ISO protocols more than any others. The CEN protocols, developed in Europe, include the CEN-Short and CEN-Long protocols. These protocols have a relatively small number of cycles and are applied at a relatively low velocity, especially the CEN-Long. Because of these characteristics, they do not resemble actual seismic behaviour as closely as the “high cycle frequency” protocols.

Studies have been conducted solely on loading/displacement protocols and how they affect shear wall test results. Among the researchers are Drs. M. He, F.Lam, & H.G.L. Prion (1998); E. Karacabeyli & A. Ceccotti (1998); and T.D. Skaggs & J.D.Rose (1998).
Another displacement protocol used in the 2\textsuperscript{nd} quarter of the second year of the MIDPLY\textsuperscript{TM} project was the “Euro” protocol, shown on Figure 3.4. It consists of two “pre-loading” cycles before a “ramp-to-failure”. The two cycles each hold the wall at a displacement of 9mm for five minutes, which is 40% of the wall capacity, as calculated beforehand from similar walls. After 10 minutes, the ramp load begins and is held until the wall fails.

![Graph showing Euro protocol](image)

\textbf{Figure 3.4:} The Euro protocol used in the 2\textsuperscript{nd} Quarter

Other protocols used in the 2\textsuperscript{nd} year of the MIDPLY\textsuperscript{TM} project are earthquake acceleration time histories, which are \textit{dynamic} test protocols and are explained in Chapter 8.
3.3 Static Test Methodology

3.3.1 The Test Matrix

A test matrix, as the term is used in the MIDPLY™ project, is a chart that describes all parameters of the tests performed; it gives an overview of the tests. The matrix distinguishes one group of tests from the other in terms of which wall parameters are investigated in each group of tests, such as nail spacing, lumber size, type of hold-down connection, etc.

The test matrix for the second year is shown in Table 3.1. This gives an overview of all tests that this thesis is about, except for the dynamic tests, which are shown in Chapter 8. Throughout this thesis, test matrices will appear when test parameters need to be distinguished. Table 3.1 shows that the second year was divided into four quarters and each one was devoted to the investigation of a certain parameter (or more than one parameter in some cases).

The parameters tested in the first quarter were the use of 2”x3” nominal (38mmx64mm) lumber for both studs and plates, and nail spacing. The second quarter parameters tested were the newly designed hold-down connection, and also 2”x3” lumber. The third quarter parameter was the effect of finger-joined lumber for studs and plates. The fourth quarter parameters were the ISO98 protocol, and hysterisis curves to use for modelling of the dynamic test specimens (identical wall specimens tested statically to provide information on how they behave, before the dynamic tests are conducted).
### Table 3.1: MIDPLY™ Test Matrix of Static Tests in the Second Year

<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>Nail spacing</th>
<th>Hold-downs</th>
<th>Variables to be studied</th>
</tr>
</thead>
<tbody>
<tr>
<td>M32-01</td>
<td>Monotonic</td>
<td>44.48</td>
<td>2x4'8&quot; / 2x4'8&quot;</td>
<td>2x4'/2x4'</td>
<td>406</td>
<td>100/200°</td>
<td>Old</td>
<td>Stud spacing</td>
</tr>
<tr>
<td>M32-01a</td>
<td>ISO97</td>
<td>44.48</td>
<td>2x4'/2x4'</td>
<td>2x4'/2x4'</td>
<td>406</td>
<td>100/200°</td>
<td>Old</td>
<td>Stud spacing</td>
</tr>
<tr>
<td>M33-01</td>
<td>Monotonic</td>
<td>44.48</td>
<td>2x3'/2x3'</td>
<td>2x3'/2x3'</td>
<td>406</td>
<td>100/200°</td>
<td>Old</td>
<td>Stud spacing</td>
</tr>
<tr>
<td>M33-02</td>
<td>ISO97</td>
<td>44.48</td>
<td>2x3'/2x3'</td>
<td>2x3'/2x3'</td>
<td>406</td>
<td>100/200°</td>
<td>Old</td>
<td>Stud spacing</td>
</tr>
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<td>2x3'/2x3'</td>
<td>2x3'/2x3'</td>
<td>406</td>
<td>100/200°</td>
<td>Old</td>
<td>Stud spacing</td>
</tr>
<tr>
<td>M34-02</td>
<td>ISO97</td>
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<td>2x3'/2x3'</td>
<td>2x3'/2x3'</td>
<td>406</td>
<td>100/200°</td>
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<td>2x3'/2x3'</td>
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<td>100/200°</td>
<td>Old</td>
<td>Stud spacing</td>
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<td>ISO97</td>
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<td>2x3'/2x3'</td>
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<td>100/200°</td>
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<td>Stud spacing</td>
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<td>100/200°</td>
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<td>Stud spacing</td>
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<td>Cycle 1-Euro</td>
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<td>100/200°</td>
<td>Old</td>
<td>Hold-down</td>
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<td>Cycle 2-Euro</td>
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<td>2x3'/2x4&quot;</td>
<td>610</td>
<td>100/200°</td>
<td>Old</td>
<td>Hold-down</td>
</tr>
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<td>M38-01b</td>
<td>Ramp-Euro</td>
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<td>2x3'/2x4&quot;</td>
<td>2x3'/2x4&quot;</td>
<td>610</td>
<td>100/200°</td>
<td>Old</td>
<td>Hold-down</td>
</tr>
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<td>Cycle 1-Euro</td>
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<td>2x3'/2x4&quot;</td>
<td>610</td>
<td>100/200°</td>
<td>New</td>
<td>Hold-down</td>
</tr>
<tr>
<td>M39-01a</td>
<td>Cycle 2-Euro</td>
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<td>2x3'/2x4&quot;</td>
<td>2x3'/2x4&quot;</td>
<td>610</td>
<td>100/200°</td>
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<td>Hold-down</td>
</tr>
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<td>Ramp-Euro</td>
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<td>2x3'/2x4&quot;</td>
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<td>100/200°</td>
<td>New</td>
<td>Hold-down</td>
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<td>610</td>
<td>100/200°</td>
<td>New</td>
<td>Hold-down</td>
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<td>2x3'/2x3&quot;</td>
<td>610</td>
<td>100/200°</td>
<td>New</td>
<td>Hold-down</td>
</tr>
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<td>2x3'/2x3&quot;</td>
<td>610</td>
<td>100/200°</td>
<td>New</td>
<td>Hold-down</td>
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<td>2x3'/2x3&quot;</td>
<td>610</td>
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<td>New</td>
<td>Stud spacing</td>
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<td>ISO97</td>
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<td>2x3'/2x3&quot;</td>
<td>2x3'/2x3&quot;</td>
<td>406</td>
<td>100/200°</td>
<td>New</td>
<td>FJ lumber</td>
</tr>
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<td>2x3'/2x3&quot;</td>
<td>610</td>
<td>100/200°</td>
<td>New</td>
<td>FJ lumber</td>
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<td>2x3'/2x3&quot;</td>
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<td>2x3'/2x3&quot;</td>
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<td>100/200°</td>
<td>New</td>
<td>FJ lumber</td>
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<td>2x3'/2x3&quot;</td>
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<td>FJ lumber</td>
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<td>2x3'/2x3&quot;</td>
<td>610</td>
<td>100/200°</td>
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<td>2x3'/2x3&quot;</td>
<td>610</td>
<td>100/200°</td>
<td>New</td>
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<td>ISO97</td>
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<td>2x3'/2x3&quot;</td>
<td>610</td>
<td>100/200°</td>
<td>New</td>
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<td>100/200°</td>
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<td>ISO97</td>
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<td>2x3'/2x3&quot;</td>
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<td>100/200°</td>
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<td>610</td>
<td>100/200°</td>
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<td>ISO97</td>
<td>None</td>
<td>2x3'/2x3&quot;</td>
<td>2x3'/2x3&quot;</td>
<td>610</td>
<td>100/200°</td>
<td>New</td>
<td>Load Protocol</td>
</tr>
</tbody>
</table>

1. All tested MIDPLY™ walls used Spruce-Pine-Fir as framing materials, and 12.5 mm OSB as middle sheathing panel connected power nail (diameter = 3.05 mm, length = 82 mm). Studs were spaced at 406mm on center. A vertical load of 44.48 kN was applied on each specimen. Stud grade lumber were used for studs, and standard and better grade lumber were used for top and bottom plates.
2. Two rows of nails at specified nail spacing.
3. Two rows of nails at specified nail spacing. Each row connects different panels.
4. One row of nail was applied at the middle of the stud to prevent panel joint studs from falling apart.
5. Tested after M32-01 reached load capacity of the actuator.
6. Finger joined lumber with SPS 1 and 3 stud grade. 7 FJ lumber with SPS 1 and 3 "standard and better" grade.
8. "New" hold-down refers to inverted-triangle hold-down.
9. 6.4 mm max. displacement, 10 12.7 mm max. displ, 11 25.4 mm max. displ., 12 50.8 mm max. displ., 13 125 mm max. displ.
3.3.2 Static Test Result Parameters

The test matrix lists the wall parameters. After the wall is tested, how does one evaluate how the wall behaved in the test? To evaluate/measure how a wall behaves, we use test result parameters. These parameters are explained in this section.

Failure Mode:

This has already been touched on in Chapters 1 and 2 for conventional shear walls. However, the failure modes for MIDPLY™ are different because certain failure modes are virtually eliminated, as explained in Chapter 2. One new failure mode is transverse buckling due to the vertical loads applied. This is why a third end stud is placed transversely to the two end studs, as shown on Figure 2.4, end stud Type 2. This failure mode is further explained in Chapter 6, Section 6.4. Another failure mode encountered in the MIDPLY™ wall was the disconnection of the studs at the panel joint (the middle stud on an 8ft wall) due to nail fatigue at that joint. This will be further explained in Chapter 6, Section 6.3. A third failure mode of the MIDPLY™ wall is the rupture of the end studs due to uplift forces. This was the reason for the need of a new hold-down connection design, shown in Chapter 4. There exist different kinds of end-stud rupture failure modes, ie. different ways in which the end-stud fails. These failure modes depend on how the wall is loaded – monotonically, cyclically, or dynamically. Chapter 7 (Static Test Results) will delve further into the failure modes and their causes.
Maximum Load ($P_{\text{max}}$)

The maximum load (or ultimate load) of a MIDPLY™ test, denoted $P_{\text{max}}$, is simply defined as the top of the load-displacement curve, shown on Figure 3.5. The maximum load is the most commonly used parameter by which to judge shear wall performance.

Stiffness ($K$)

The stiffness of the MIDPLY™ walls, denoted $K$, is defined as the secant stiffness between 10% and 40% of the maximum load, as shown on Figure 3.5. More specifically, it is the average slope of the load-displacement curve between those points. The secant stiffness is the second most used parameter in comparing shear wall behaviour because it determines how much the wall will displace (deflect) for a certain amount of force, and in a system of other LLR elements, the stiffness will determine how much force the wall will attract. Tangential stiffness is not listed as a parameter in the static tests, but does come into play in the dynamic modelling.

Maximum Displacement ($\delta_u$)

The maximum displacement of a MIDPLY™ wall, denoted $\delta_u$, is defined as the displacement at 80% of the maximum load after the maximum load has been reached, which is on the negative slope of the curve, as shown on Figure 3.5. Due to the strength of the MIDPLY™ walls and the limitations of the static test apparatus, some tests did not reach the point of maximum deflection. For these situations, the maximum displacement
Chapter 3: MIDPLY™ Static Wall Testing

is equal to the highest displacement value on the load-displacement curve, and a note is included where applicable.

**Energy Dissipation (E)**

The energy dissipated in a test is equal to the area enclosed by the hysteresis curves of a cyclic or dynamic test. The total energy dissipated by the system is the work done by hysteretic damping. The method of computing the energy dissipated is shown in Chapter 9, Section 9.1.2.

**Ductility:**

Ductility, when the term refers to a system rather than a material, is qualitatively defined as the ability to deform plastically without a substantial reduction in strength. Nailed joints, like those in shear walls, have a relatively high ductility compared to heavy bolted joints and glued joints. In the MIDPLY™ wall, as in any light frame timber shear wall, the ductility of the system lies almost completely in the nails. Quantitatively, there are several definitions of ductility, depending on the design code that is used and the type of structure that is analyzed (Popovski, 2000). The measurement of ductility is called the ductility ratio, and is usually taken to be the ratio of peak displacement to yield displacement. However, it is unclear on most load-displacement graphs where the point of yield displacement is. On an idealized bi-linear graph, it is concretely defined. On the MIDPLY™ load-displacement graphs, the point of yield displacement will be defined as the displacement corresponding to 40% of maximum load, which is the same point to which the secant stiffness is calculated to. Therefore, the ductility ratio, as
shown on Figure 3.5, is defined as the ratio of maximum displacement ($\delta_u$) to the yield displacement ($\delta$ at 40% of $P_{\text{max}}$). Ductility is not analyzed in all static tests.

![Graph showing important result parameters](image)

**Figure 3.5:** Important Result Parameters Depicted Graphically

### 3.3.3 Testing Summary

To summarize the testing methodology and to add in a few minor points that were not mentioned in this chapter but are part of each MIDPLY™ test, the flowchart in *Figure 3.6* illustrates the steps of a typical MIDPLY™ test.
Chapter 3: MIDPLY™ Static Wall Testing

Create Test Matrix:
- load protocol
- wall parameters
- vertical load (y/n)

Select a Wall from the Matrix and Build Test Specimen

Program the Test Apparatus

Test wall and observe with photos, video record, data recording, & observe failure mode

Take wall apart and make a nail failure diagram.

Download test results from apparatus and put in presentable form.

Analyze results and report result parameters

Use results to create a new Test Matrix and/or determine wall design values

Figure 3.6: The Steps of a MIDPLY™ Wall Test

All MIDPLY™ walls were built on site in the Forintek Canada Corp. Wood Engineering Lab by Mr. Henley Fraser, Mr. Jules Gardy, and myself. Mr. Fraser operated the test apparatus during all tests, including the programming of load protocols. The walls were built using a wall jig, special clamps, and a Bostich™ nail gun. The detailed construction method of a MIDPLY™ wall is listed in Appendix C.

All MIDPLY™ walls were tested within one or two days of construction, except for the dynamic tests, which were conducted approximately a month after construction. The day-to-day temperature and humidity were recorded inside the laboratory in which the dynamic test specimens were kept before testing.
Chapter 4: New Hold Down Connection

4.1 Purpose of Hold Down Connections

As explained in Chapter 2 and shown in Figure 2.11, a special hold down connection was designed for the MIDPLY™ wall in Phase 1 of the first year. The purpose of the hold down is to resist uplift of the end studs during racking, thereby giving the wall greater capacity. The hold down connects the end studs to the foundation below the wall, which could be a concrete foundation if it’s the first storey, or a timber joist floor if it’s on the second storey or higher. In a two-or-more storey scenario, a vertical bolt connects the bottom hold downs on the wall above to the top hold downs on the wall below. The figures in this thesis show the hold downs as being only on the bottom of the wall because only one storey is tested. In a two-or-more storey building, there would be hold downs at the top of the wall as well as the bottom, to provide continuity in the building. Therefore the hold down’s purpose is to resist the overturning moment of the shear wall during racking by providing a load path to the foundation, as shown in Figure 4.1.

In conventional 2x4 shear walls, several different hold down connections can be bought on the market, with Simpson Strong-Tie® being the most popular manufacturer in the US and Canada. These typical hold downs are installed after the wall is built and before the drywall is installed. There is typically one hold down per end stud installed in all shear walls.
4.2 Review of Existing Hold Down Connection

In the MIDPLY™ wall, there are two hold down connection per end stud configuration because there are two end studs at the end of the MIDPLY™ wall. The hold down is positioned as shown on Figure 4.1 - it connects the end studs via five \( \frac{3}{8}'' \) bolts to the foundation below the wall, via one \( 5/8'' \) bolt. It is made of 300W steel and its dimensions were shown in Figure 2.11. This hold down is designed for a higher force than the typical hold down connection in conventional shear walls because of the higher forces encountered in MIDPLY™ walls.

Due to the way shear walls behave in earthquakes and severe winds, the end studs experience both uplift and bending, as illustrated in Figure 4.1.

Figure 4.1: Forces Induced on MIDPLY™ wall due to Existing Hold Down
Due to the fixity these hold downs create at the base, the end studs behave like vertical cantilevers. The triangular piece on the hold down that creates this fixity, as shown on Figure 4.1, because it restrains the end stud against rotation when a lateral force is applied to the top of the wall.

Consequently, it was observed from previous MIDPLY™ tests that the end studs experienced excessive bending in the area of the hold downs and ultimately failed at that location, as shown on Figure 4.2. This type of failure was accelerated when no vertical loads were applied on the wall, since vertical loads counteract uplift forces. The moment is caused by rotation of the end stud due to the lateral deformation of the wall, as shown in Figure 4.1. The moment, combined with the uplift force, causes the end studs to fail.

Since the end studs fail in a brittle manner, the failure mode is undesirable. The reason for this is that a ductile failure is desirable rather than a sudden failure. Brittle failures give little or no warning prior to failure, which can result in collapse. When there is
adequate ductility in the system, the system is less likely to result in total collapse. It was obvious that the existing hold down connection posed a problem that needed to be solved before proceeding with further MIDPLY™ tests.

Furthermore, the existing hold down posed a problem in accessing the 5/8” vertical bolt that connects the hold down to the foundation. The bolt needed to be tightened with a wrench, and there was barley clearance between the triangular gusset plates for the narrowest socket on any available ratchet.

Therefore, the objectives in designing a new hold down connection were as follows:

- To minimize bending of the end-studs during racking
- To avoid causing a brittle failure mode
- To provide greater access to the bolts for tightening the connection

4.3 Analyzing Force on New Hold Down Connection

To design a new hold down connection, the MIDPLY™ wall must be analyzed to determine the force on the hold down. The following simplified equation for shear walls undergoing static loads is used. $T_f$ is the factored load on the hold down:

$$ T_f = V_f \pm \left( \frac{v_f \cdot L_w \cdot H_w}{h} \right) $$

where $V_f$ represents the sum of the vertical loads applied to the wall (if any), $v_f$ is the shear force per unit length due to lateral forces, $L_w$ is the wall length, $H_w$ is the wall
height, and h is the distance between the end studs. As denoted by the subscript f, all loads are factored in this equation. The load factors, as well as any connection design equations, are taken from the Canadian Limit States Timber Design Code, CAN/CSA-O86.1-M89. Since h and Lw are virtually the same in our case, the equation is simplified to:

\[ T_f = V_f \pm (v_f \cdot H_w) \]

The ± term is there because the vertical load is added to one side of the wall (the compression side) and subtracted from the other side of the wall (the tension side). The hold down is designed for uplift i.e. the tension side of the wall, therefore for our purposes the equation is changed to:

\[ T_f = V_f - (v_f \cdot H_w) \]

Therefore we have the equation for a static analysis of the hold down force, but the MIDPLY™ wall needs to be designed to also withstand earthquake and high wind loads, which are not static, but dynamic loads. A dynamic load differs from a static load in that dynamic loads are caused by inertia forces due to mass on the wall that is accelerated by an earthquake or another ground motion or wind motion. Sometimes the dynamic loading tends to govern over the static loading. Therefore, the hold down is designed to cover both loadings. For the dynamic calculation, one term of the equation needs to be changed: the shear force per unit length, v_f. This term can be expressed as an equivalent dynamic force, \( v_{fdyn} \), which is equal to the estimated or calculated relative acceleration at the top of the wall multiplied by the mass, M, at the top of the wall:

\[ v_{fdyn} = (M \cdot a_{top \ of \ wall}) / L_w \]

Where \( L_w \) is the length of the wall (standard 8ft or 2.44m).
Since a shear wall is a multi-degree-of-freedom (MDOF) system, and it has highly non-linear load-displacement relationship, the only way to accurately analyze the true force on the end studs due to an earthquake record is through a non-linear time-stepping dynamic analysis. The program that was chosen is DRAIN-2DX, a versatile structural analysis program written at UC Berkeley which is described in greater detail in Chapter 8. The program converts the MDOF system into a SDOF system by using the “Florence Loop Model”, written by Dr. A. Ceccotti at the University of Florence, to model the hysteresis of the wall. This hysteresis model is explained in Chapter 8, Section 8.1.4. The properties of one of the stiffest MIDPLY™ wall was input into the program and certain earthquake acceleration records were chosen as excitations. The program was run and the peak acceleration output was selected as the value to be used for the hold down design calculation. The acceleration value was: 2.1g (20.6 m/s/s). Therefore the expected force, \( V_{f^{\text{dyn}}} \) is:

\[
V_{f^{\text{dyn}}} = \left( \frac{4500\text{kg} \times 20.6\text{m/s}^2}{2.44\text{m}} \right) = 38.0\text{kN/m}
\]

The mass that was used in the dynamic model is the standard load representing the tributary area of one storey above the wall, which is 4500kg. This is the mass that is used on the UBC shake table test apparatus also.

The worst case scenario for static loading is that there are no vertical loads acting on the system. Therefore \( V \), the vertical load component, is eliminated from the equation for static loading. The term cannot be eliminated from the dynamic load equation because without the mass at the top of the wall, there would be no inertia force acting on the wall, and the gravity acting on the mass creates a vertical load.
Chapter 4: New Hold Down Connection

Therefore, the two equations that the hold down connection must satisfy are:

$$T_f = (v_f \cdot H_w)$$ \hspace{1cm} \text{(Static)}

$$T_f = V_f - (v_{f,\text{dyn}} \cdot H_w)$$ \hspace{1cm} \text{(Dynamic)}

The maximum load that the static test apparatus can handle is 20,000 lb (89kN) therefore the maximum static shear force per unit length is (89kN/2.44m) = 36.5kN/m. The maximum dynamic shear force per metre, \(v_{f,\text{dyn}}\), is 38.0 kN/m, and the vertical load due to the 4500kg mass is \((4500kg*9.81m/s^2) = 44kN\) spread out over the length of the wall.

The resultant vertical load, \(V\), on the hold down is the portion of the load that is distributed linearly along the wall length. Assuming stud spacing at 16\"(406mm), the force on the interior studs is: \((0.406)* (44kN / 2.44m) = 7.32kN\) and the exterior studs receive half that: 3.66kN.

Therefore, with load factors from CAN/CSA-O86.1 being 1.5 for wind and live loads (static), 1.00 for earthquake loads and 0.85 for uplift and stress reversals, these expressions are evaluated to be:

$$T_f = (1.5(36.5kN/m) \cdot 2.44m) = 133.5kN$$ \hspace{1cm} \text{(Static)}

$$T_f = (0.86 \cdot 7.32kN) - ((1.0 \cdot 38.0kN/m) \cdot 2.44m) = 86.4kN$$ \hspace{1cm} \text{(Dynamic)}

Therefore, since there are two hold downs per end stud configuration (two end studs per configuration) one pair of hold downs needed to withstand these loads.

However, actual bearing distribution at the base of a shear wall is different than the simplified analysis above reveals. A more accurate bearing distribution is shown in
Figure 4.3, where the bearing and uplift is shared with the interior studs, not just taken by the end studs acting alone as a couple.

**Figure 4.3:** Diagram of Bearing and Uplift Forces at the base of a Shear Wall without vertical load.

Because of this more accurate representation of the bearing stress at the base of the shear wall, the factored static load to which the pair of hold downs could be subjected is \((1.5 \times 76 \text{ kN}) = 114 \text{ kN}\), instead of 135 kN as computed in the simplified static analysis. The 17,140 lbs (76 kN) is even conservative considering that resultant is spread over a 16” width. Figure 4.3 shows that there are 3 sets of couples that are equal to the 20,000 lb load from the actuator multiplied by the 8ft (2.44m) height.

Therefore, each pair hold downs could theoretically experience a factored tensile load of 114 kN, which is 57 kN per hold down.
Also, the end studs need to withstand this tensile force plus the bending force associated with the eccentricity of the hold down connection. The amount of bending on the stud will depend on the geometry of the hold down connection, therefore it is analyzed after the alternatives have been examined.

4.4 Alternatives For New Hold Downs

After brainstorming and researching existing hold downs on the market, two alternatives were chosen: The Strap-Tie hold down and the Inverted Triangle hold down. These are variations of existing hold downs.

4.4.1 Strap-Tie Hold Down

The Strap-Tie hold down, shown in Figure 4.4, is a variation of a hold down that has been produced by different manufacturers including Simpson Strong-Tie®. The illustration shown in the figure represents a connection that has been modified to withstand 57kN, which is a much higher force than these straps are commonly used for. The strap-tie uses common nails or wood screws and a 7mm thick structural grade steel strap that is attached to the wide face of the stud, as shown. The holes for the nails can be countersunk to provide for lower nail head profiles.
Chapter 4: New Hold Down Connection

Figure 4.4: Strap-Tie Hold Down Connection

The bottom of the strap tie is anchored into the concrete foundation when it is poured. One a wooden foundation, or on a second or higher storey where there is a wood floor, the strap tie can be nailed to the joist below the wall.

The advantages of the strap-tie are its simplicity of installation. It can be put in after the wall is built by using Hilti™ connectors to connect it to the concrete foundation. Moreover, it is inexpensive and can be installed with a hammer and nails. Another advantage is that it can be applied to the side or the end of the wall, and it doesn't apply any eccentricity on the studs if it connected to the side of the studs.

The disadvantages of the strap-tie are the high amount of fasteners and the fact that it makes the application of exterior sheathing difficult. Also, when anchored into concrete
before the wall is built, it has to be positioned very accurately or else it will be misaligned with the wall. This accurate positioning is difficult to achieve in the field.

4.4.2 Inverted Triangle Hold Down

The inverted triangle hold down was conceived from a similar hold down used in Japan, shown in Figure 4.5. The main concept behind this hold down is the elimination of the bending of the end studs. It is the same as the existing hold down but with a modified of triangular gusset, which was creating the bending in the end stud previously.

Figure 4.5: The Japanese Hold Down that the Inverted Triangle Hold Down was modelled after.

The layout and dimensions of the inverted triangle hold down is shown on Figure 4.6. The intent of the inverted triangular gusset is to allow the end studs to rotate and consequently produce primarily a tension force in the end studs.
This hold down was designed according to the ANSI/AF&PA NDS-1997 timber design standard and the CAN/CSA S16.1-M89 steel design code. A spreadsheet, shown in the next section of this chapter, was created to design the hold down so one could determine the results of changing parameters such as number of bolts, steel thickness, grade, etc.

![Diagram of hold down connection]

**Figure 4.6:** The Inverted Triangle Hold Down Connection

The advantages of the inverted hold down are that it does not interfere with sheathing or drywall, it can resist more force than other hold downs, and it is quick and easy to install in a prefabricated home, where the hold downs are attached to the end studs ahead of time. The holes into the concrete foundation can be drilled through the bottom plate before or after the wall is installed, or the threaded rods can be anchored into the concrete while it is poured. However, the disadvantage to this method is that, like the strap-tie, a
high accuracy is needed in positioning the threaded rod. Another disadvantage of this hold down is shrinkage; the threaded rod and the 10 mm bolts would have to be tightened as the wall gets older and the wood shrinks. This tightening sequence for the 16 mm threaded rod would be approximately: 1st tightening in two weeks, the next tightening after a few months, and the final tightening a year after that. The reason for the tightening is to eliminate the slack on the bolt. If there is slack, then during an earthquake, there would be a shock on the end stud during uplift and the end stud would likely fail in a brittle manner.

Out of the two alternatives, the inverted triangle hold down was chosen as the new hold down to try on future tests. From hereon it will be referred to as the “new hold down”.

4.5 Design of the New Hold Down

To arrive at the dimensions shown in Figure 4.6, the flowchart shown in Figure 4.7 was used to design and optimize the hold down for the least material-to-strength ratio. The hold down was designed to affect the end stud at little as possible so the end stud could handle the bending and tension forces imposed by uplift. The end stud, not the hold down, was the governing element in the connection. The reason that the American NDS standard was used instead of the Canadian design code is that the NDS standard takes into account different yield modes which, in this case, led to an increased ultimate design
load for the connection. There is a part of the NDS standard that deals with single shear bolted connections with steel side plates, which was used to design this connection. The ultimate strength of the connection using regular 2x4 SPF (Spruce-Pine-Fir) stud grade studs and 300W structural steel has an ultimate capacity of 46.6 kN. This is under to the calculated requirement of 57kN per hold down. However, this is the maximum that one can attain using these studs, and it can only be increased if a different grade of stud is used, such as Machine Stress Rated (MSR) lumber.

As mentioned before, the stud is the weak link, not the hold down connection. Therefore, MSR studs might be a good idea, but it was decided upon viewing the results of the finite element analysis of the hold down that regular SPF stud grade studs would be used, as usual. If, after testing several walls using the new hold down with stud grade studs, the end studs were seen to be failing, then a higher grade of stud would be considered. An advantage of using regular stud grade studs is simply that the same materials are used throughout the wall. The less special materials that are used, the less the wall is a specialized item, and the cheaper and easier it can be constructed in the field.
Chapter 4: New Hold Down Connection

A structural analysis of the hold down itself was performed to predict its behaviour before using it in full-scale wall testing. This was performed on several models of the hold down before the final dimensions were determined. Through a computer finite element analysis of the hold down connected to the end stud, and using a graphical output of the deflections, it was observed how the hold down would behave in response to a tensile force acting on the end stud. In this way, one could determine the amount of bending on the end stud and in the hold down itself for different designs. Structural analysis software named Visual Analysis™ was used to create a finite element analysis of the hold down (IES, 1999). Figure 4.8 shows the resulting deflection of the new hold
down design (final design) to an applied load of 60kN, which is even higher than the factored load requirement. Sometimes design codes are too restrictive for the needs of engineers, and an acceptable substitution for the code is a structural analysis such as this, which proves that a certain design may be acceptable even though it does not meet the code requirements. The corollary also holds true in some cases, where a structural analysis may conclude that a design meets code standards, but is not adequate for the loads it may encounter. Nevertheless, the structural analysis performed with the computer program named Visual Analysis™ indicates that the hold down connection is adequate for the MIDPLY™ wall.

The following figure shows that the hold down bends near the top of the triangular gusset. This is due to the eccentricity of the vertical 16mm bolt from the load path of the five 10mm bolts. The vertical bolt pulls the triangular gusset away from the vertical strip of the hold down, thus creating a large tension force on the horizontal lip and bending in the vertical strip. Full scale testing will reveal if this is an accurate model, by observation of the maximum deformations in the hold down, and whether they correspond to this model. The maximum deformation and maximum stress location is pointed out in Figure 4.8.
From the results of the structural analysis, the hold down begins to yield at a load of approximately 20kN and remains intact. It does not break until just over 60kN. Therefore the hold down is not expected to withstand the required loads without yielding. This is actually a benefit because the hold down will remain in the linear-elastic range during moderate events and go into the plastic range in large events with greater uplift forces. The yielding of the hold down dissipates energy in the MIDPLY™ system and transfers energy from the end stud to the hold down. The more energy that is transferred...
from the end stud to the hold down, the better. The hold down yields in a ductile manner (thin steel plate) and therefore it is a desirable manner of energy dissipation.

The method of designing certain elements in a structural system to yield and others to remain elastic is called “Capacity Design”. This is a common, modern, and effective way of earthquake-resistant design.

Because wood crushing is not reflected in this structural model, the connection will be able to withstand more load than reflected in the results from the analysis. This is due to the fact that wood crushing will dissipate some of the energy in the connection during uplift.

After the occurrence of large events, the hold down would most likely need to be replaced. This is less time consuming than replacing the end studs, much less the entire wall. The hold down connection is adequate for the MIDPLY™ wall, until full-scale wall testing provides more information.
5.1 Purpose of Anchor Bolt Connections

Anchor bolts connect the bottom plate of a shear wall to the foundation or the floor below. That said, the term “anchor bolt” is a vague one since they vary considerably depending on the type of foundation they are used in. Some are straps or ties rather than bolts. No matter what type, shape, or size they are, anchor bolts exist in some form on all shear walls. Some shear walls not only use anchor bolts on the bottom plate, but the top plate also to connect it to the rest of the structure. Common practice in North America, however, is to nail the top plate of the wall to the floor joists above them, without using anchor bolts. Properly used, anchor bolt connections provide continuity in the structure, much like hold down connections do. Figure 4.1 showed the shear force distributed along the top plate of the wall, and the reaction along the bottom plate. It is this force that the anchor bolts resist – a base sliding force, as it were. Anchor bolts are distributed along the length of the bottom plate.

Commonly used anchor bolts in conventional shear walls are shown in Figure 5.1. The one shown on the left is a metal strap that is placed in the concrete foundation as it is poured. When the shear wall is placed, the strap is bent around the plate and nailed to it. These straps are commonly placed at intervals of 1 to 4 feet (0.3m to 1.3m). The anchor
bolt on the right is exactly that – a bolt with a rebar or Hilti® end that is embedded into the concrete. It can be placed in the concrete as it is poured or else drilled after the wall is put in place. This versatility is its advantage. The bolt protrudes through the bottom plate of the shear wall and is tightened down with a nut and washer. These bolts are usually placed every 2 to 6 feet (0.6m to 2m). Both of these connections prevent base sliding of the wall during lateral loading.

Figure 5.1: Commonly Used Anchor Bolt Connections in Conventional Shear Walls

While these connections work well for conventional shear walls, they do not work for the MIDPLY™ for two reasons: geometry and capacity. First of all, in a MIDPLY™ the sheathing is in the middle of the two plates, therefore the bolt would interfere with the sheathing. The wrap-around strap would not be feasible because the capacity of this connection is adequate for a conventional shear wall, but not for the high forces in a MIDPLY™ wall. Therefore, a custom anchor bolt connection was designed in the first year of the project, as described in the next section.
5.2 Review of Existing Anchor Bolt Connections

The anchor bolt designed for the MIDPLY™ wall is similar to those used in conventional shear walls; they are situated right in the middle of the two plates – in the ½” gap between them. To accommodate anchor bolts in the MIDPLY™ system, the OSB is cut out around the area of the anchor bolt and the bottom and top plates, as shown in Figure 5.2. This cut-out is for wrench access for the anchor bolt, since it needs to be tightened after installation. The cut-out is referred to as a “mouse hole” because of its shape.

The bolts that are used are ½” (12mm) structural grade (A325) bolts with a hardened nut and a thick 2” (50mm) diameter washer that covers both plates. The washer is ridged on the bottom face so it grips the bottom plate and creates more friction resistance against sliding. The spacing is 16” (406mm) centre to centre.

Figure 5.2: Existing Anchor Bolt Connections used in the MIDPLY™ system
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This connection has certain weaknesses. Firstly, the problem with the mouse holes is that they create stress concentrations in the OSB or plywood mid-ply. Several instances of rupture in the mid-ply around the mouse holes have been observed in first year tests, as shown on Figure 5.3. The rupture is caused by increased stress around the holes during racking of the wall.

![Rupture](Figure 5.3 OSB rupture at Mouse Hole locations)

Secondly, cutting out the mouse holes add additional construction cost to the MIDPLY™ system. Thirdly, because the lumber plates may relax during long term load or may change their dimensions due to moisture cycling, the friction may be greatly reduced under real-life conditions, especially in a wet climate. Consequently, this connection may not provide sufficient resistance to the shear force along the MIDPLY™ wall plates. Fourthly, the nailed joint area along the plates is significantly reduced by the mouse holes; if the mouse holes were not there, an additional two nails could be applied to the
perimeter of the OSB or plywood. Therefore, the mouse holes decrease the plate-to-panel connection area, which is a detriment to the system. An alternative method of anchoring the wall needed to be designed.

5.3 Alternatives for New Designs

The objective of designing alternative anchor bolt connections is to transfer the shear force by bearing of fasteners instead of the friction along the MIDPLY™ wall plates, and to eliminate the mouse holes in the mid-ply, thereby avoiding the undesirable panel rupture around these holes. Two different designs of anchor bolts were considered.

5.3.1 Thin Plate Anchor

The first alternative is similar to a conventional anchoring system for wood frame structures; a thin plate anchoring system using nails, screws or small bolts. For the MIDPLY™ system, a special version of this thin plate anchor needed to be designed because of the unique geometry and high forces. The connection that was designed is shown in Figure 5.4. It is meant to be placed on both sides of the wall at a spacing of approximately 24” (610mm) centre to centre. There is flexibility as to where the connections can be placed because it can be nailed to the bottom of the plate anywhere along the plate.

The connection needs to be able to be anchored to both wooden and concrete foundations, depending on the storey they are located. This is why the connection, as shown on Figure 5.4, has bolt holes and nail holes on the bottom of the connection. The
Chapter 5: New Anchor Bolt Connections

Bolt holes can accommodate Hilti® anchoring bolts for concrete, and the nail holes can be used on wooden foundations, with the thin 16 gauge (1.6mm) steel plate bent at 90° so it can be nailed to the floor joists if need be.

Figure 5.4: Thin Plate Anchor Connection Alternative

Common hand nails, 3" long (76mm) are used for the nailed connections and 3/8" (10mm) bolts are used for the bolted connections. The 16 gauge (1.6mm) plate is made of 250MPa steel or higher. The factored resistance of the plate is 7.75kN, therefore the sliding resistance on an eight foot wall is 62kN since 8 plates are used (4 on either side). Calculations show that this resistance is enough for a MIDPLY™ wall, since some of the sliding resistance is taken up by the hold down connections and the friction of the plates on the foundation.
Chapter 5: New Anchor Bolt Connections

The advantages of the thin plate anchor system are that the wall needs no modifications or preparation for the application of these plates (no hole-drilling in plates), it is a ductile connection because of the great number of small fasteners, prevents sliding action despite wood shrinkage or swelling, and it can be installed after the wall is built and placed. The disadvantages of this connection is that it is labour-intensive (great number of fasteners), it gets in the way of the drywall or exterior sheathing, and it is difficult to use second storeys or higher.

5.3.2 Bolt Through Plate

The second alternative to the mouse hole connection is the bolt through plate. As shown in Figure 5.5, this connection simply consists of a bolt through the bottom plate of the wall that is anchored into the foundation, like the anchor bolts in conventional shear walls. The difference between this connection and those used in conventional walls is that this uses 3/8" (10mm) bolts because of the vertically-oriented plates.

![Figure 5.5: Bolt Through Plate Anchor Connection Alternative](image-url)

**Figure 5.5:** Bolt Through Plate Anchor Connection Alternative
As shown, the bolts are placed every 16” on both sides, staggered. That results in a ¾” (10mm) bolt every 8 inches along the entire wall, which yields a factored resistance of 38.6 kN for an 8ft (2.44m) wall. This is adequate, considering that the failure mode is expected to be wood crushing around the bolt hole, and perhaps some bolt bending. These are ductile failure modes, and they dissipate energy. Also, the sliding force is resisted by the friction between the plates on the foundation.

The advantages of the bolt through plate connection are its ease of installation, it prevents base sliding, and it is a ductile connection. Its disadvantages are that the plates need modifications (holes drilled), and if the bolts are placed in the concrete ahead of time, then the bolts must be placed very accurately, or else they will not line up. After considering all factors, this was the connection chosen to be used in future MIDPLY™ wall tests. It will hereon be referred to as the “new anchor bolt” connection.

5.4 Design of the New Anchor Bolt

The new anchor bolt was designed according to the CAN/CSA-O86.1-M89, Section 10.4 for bolts. A regular spreadsheet instead of a decision tree was used to design the connection since the connection was simple. The calculations are shown in Figure 5.6.
### Type: Bolt Through Plate

Section 10.4 in Canadian Timber Design Code & Sec. 7 in CWC Wood Design Manual

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</tbody>
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Parallel to Grain Loading: $P_r = \phi P_u n_s n_F J_F$  
$P_u = p_u (K_D K_{SF} K_T)$

- $\phi = 0.7$
- $p_u = 4.0$ kN for SPF & $?''$ bolt in 64mm thick wood (Table 10.4.4)
- $P_u = 4.6$
- $n_s = 1$
- $n_F = 1$
- $J_F = 1.0$
- $P_r = 3.22$ kN per bolt

8 foot wall has 12 bolts total (6 per side), therefore the wall has a resistance of 38.6 kN

**Figure 5.6:** Design of New Anchor Bolt Connection

In conclusion, this new anchor bolt was tested in all MIDPLY™ walls in the dynamic tests, which is the most demanding on the anchor bolts. The test results in Chapter 9 will prove the validity of this design.
Chapter 6: Testing of 2"x3" and Finger Joined Lumber

In the second year of the MIDPLY™ project, two new wall parameters were tested: 2"x3" and finger joined lumber. 2"x3" lumber was tested throughout the second year, starting in the first quarter. Finger joined lumber was tested in the third quarter. The objectives of testing the effects of these parameters are explained in this chapter.

6.1 Objectives

6.1.1 Testing of 2"x3" Lumber

The main objective of testing MIDPLY™ walls using 2"x3" (38mm x 64mm) lumber was to minimize the materials in a MIDPLY™ wall, while keeping the lateral and vertical load bearing strength at an acceptable level. Reducing the materials result in reducing the manufacturing cost of the walls, hence the incentive to examine the performance of MIDPLY™ walls with 2"x3" studs and plates.

Several lumber mills are able to make 2"x3" lumber, but do not manufacture it in great quantities because of the small demand for this size. The reason for this is that all conventional walls in light timber frame construction are made of either 2"x4" or 2"x6" size lumber. By using 2"x3" lumber instead of 2"x4" lumber, a market for 2"x3" lumber is created, which benefits the lumber manufacturing industry because use is made of
Chapter 6: Testing of 2"x3" and Finger Joined Lumber

lumber that would otherwise be discarded or cut down to smaller sizes. For example, if a 2"x4" piece of lumber does not meet building standards because of knots, wane, or other defects, then it might still be able to be used as a 2"x3" if it is ripped down by one inch. It should be noted that throughout this thesis, either '2"x3"' or '2x3' will be used, as well as '2"x4"' and '2x4', just for simplicity.

The concerns about using 2x3 lumber are as follows:

- Increased potential of lateral buckling of the wall due to vertical loads, because of the reduced cross sectional area of the studs.
- Less nail edge distance and increased potential for missed nails during construction, especially with high-speed nail gun nailing
- Decreased end stud strength due to decreased cross sectional area.

These concerns will be assessed after the walls have undergone lateral loading tests. Since the 2x3 lumber was not available as studs, regular 2x4s were ripped down to 2x3s to build the walls for testing. End studs made of 2x3s were deemed unsatisfactory because of the required strength in this part of the wall. End studs in all MIDPLY™ walls shall be 2x4s.

It was thought prudent to examine the effects of various configurations of MIDPLY™ walls using 2x3s and a combination of 2x3s and 2x4s to arrive at the best wall configuration. Some walls use 2x3s for studs only, while others use them for plates only, and all combinations thereof. The reason for this is the anticipation that small studs
might cause lateral buckling, but therefore 2x3 plates should not be ruled out as an alternative to 2x4 plates. Conversely, if there is a problem with 2x3 plates due to anchor bolt resistance, 2x3 studs should not be ruled out as an option. These various combinations are reflected in the test matrix shown in Table 6.1.

6.1.2 Testing of Finger Joined Lumber

The objective of testing finger joined lumber in the MIDPLY™ wall was similar to that of testing 2x3 lumber – to save material and minimize cost. With finger joined lumber, the cost is the same for a finger joined stud, but the finger joining process utilizes smaller pieces of wood that would otherwise be discarded or made into wood chips. Since finger joined studs are commonly used in conventional construction, it was decided necessary to examine the feasibility of using these same materials in the MIDPLY™. The detail of a finger joined stud is shown in Figure 6.1.

![Finger Joint in Finger Joined Lumber](image)

**Figure 6.1:** Finger Joint in Finger Joined Lumber
Chapter 6: Testing of 2"x3" and Finger Joined Lumber

There are different grades of finger joined lumber. The two categories of finger joined lumber are SPS 1, which has nominal bending moment resistance, and SPS 3, which is specified for vertical use only. These specifications are governed by the NLGA (National Lumber Grades Authority). Various configurations were tested to optimize the use of different grades of finger joined lumber in the wall. The finger joints are designed to not be the weak link in the member, ie. the finger joint is supposed to be stronger than the rest of the member. Using finger joined lumber follows the same philosophy as using 2x3 lumber in that they both make use of lumber that would ordinarily be turned into wood chips or other “lesser value” products.

6.2 Test Matrices

6.2.1 2"x3" Test Matrix

The test matrix for the first quarter of the second year is shown in Table 6.1. As shown, ten tests were performed with various configurations of stud sizes, plate sizes, and nail spacing. In all tests, the old hold down was used, since the new one was being designed during the first quarter. All tests had vertical loads applied, as shown in Note 1 under the matrix.

There are two terms on this matrix that have not yet been addressed in this thesis. First, the wall number represents each new test that is done. The “M” represents “MIDPLY™”. The first number is the numbering system starting at 1 for the first wall
tested in the first year (31 tested in first year). Each wall number has different
parameters, which are shown on the various columns. The extensions "-01" or "-02"
mean it is the same wall, but a different loading protocol is used. If there is an "a" or "b"
behind the extension, it means the same wall has been tested again. The notes on the
bottom of the test matrix explain why it was tested more than once.

Second, the nail spacing column has a subcolumn named "Panel Joint". This refers to the
extra column of nails added in the centre stud, as explained on note 4 under the matrix.
To explain note 4, there was a problem with the panel joint stud, which is the centre stud
where the two panels join. The centre stud sometimes fell apart when the nails failed that
held the studs together. This is further explained in Section 6.3.
### Table 6.1: MIDPLY™ Test Matrix for First Quarter

<table>
<thead>
<tr>
<th>Wall Number</th>
<th>Load Protocol</th>
<th>Top plate</th>
<th>Bottom plate</th>
<th>End stud</th>
<th>Interior stud</th>
<th>Nail Spacing (mm)</th>
<th>Panel Joint</th>
<th>Elsewhere</th>
</tr>
</thead>
<tbody>
<tr>
<td>M32-01</td>
<td>Monotonic</td>
<td>38mm×89mm (2&quot;x4&quot;)</td>
<td>38mm×89mm (2&quot;x4&quot;)</td>
<td>38mm×89mm (2&quot;x4&quot;)</td>
<td>38mm×89mm (2&quot;x4&quot;)</td>
<td>100° 200°</td>
<td></td>
<td></td>
</tr>
<tr>
<td>M32-01a</td>
<td>ISO97</td>
<td>38mm×89mm (2&quot;x4&quot;)</td>
<td>38mm×89mm (2&quot;x4&quot;)</td>
<td>38mm×89mm (2&quot;x4&quot;)</td>
<td>38mm×89mm (2&quot;x4&quot;)</td>
<td>100° 200°</td>
<td></td>
<td></td>
</tr>
<tr>
<td>M33-01</td>
<td>Monotonic</td>
<td>38mm×89mm (2&quot;x4&quot;)</td>
<td>38mm×64mm (2&quot;x3&quot;)</td>
<td>38mm×64mm (2&quot;x3&quot;)</td>
<td>38mm×64mm (2&quot;x3&quot;)</td>
<td>100° 200°</td>
<td></td>
<td></td>
</tr>
<tr>
<td>M33-02</td>
<td>ISO97</td>
<td>38mm×89mm (2&quot;x4&quot;)</td>
<td>38mm×64mm (2&quot;x3&quot;)</td>
<td>38mm×64mm (2&quot;x3&quot;)</td>
<td>38mm×64mm (2&quot;x3&quot;)</td>
<td>100° 200°</td>
<td></td>
<td></td>
</tr>
<tr>
<td>M34-01</td>
<td>Monotonic</td>
<td>38mm×89mm (2&quot;x4&quot;)</td>
<td>38mm×64mm (2&quot;x3&quot;)</td>
<td>38mm×89mm (2&quot;x4&quot;)</td>
<td>38mm×64mm (2&quot;x3&quot;)</td>
<td>100° 200°</td>
<td></td>
<td></td>
</tr>
<tr>
<td>M34-02</td>
<td>ISO97</td>
<td>38mm×89mm (2&quot;x4&quot;)</td>
<td>38mm×64mm (2&quot;x3&quot;)</td>
<td>38mm×89mm (2&quot;x4&quot;)</td>
<td>38mm×64mm (2&quot;x3&quot;)</td>
<td>100° 200°</td>
<td></td>
<td></td>
</tr>
<tr>
<td>M35-01</td>
<td>Monotonic</td>
<td>38mm×64mm (2&quot;x3&quot;)</td>
<td>38mm×64mm (2&quot;x3&quot;)</td>
<td>38mm×89mm (2&quot;x4&quot;)</td>
<td>38mm×64mm (2&quot;x3&quot;)</td>
<td>75°  200°</td>
<td></td>
<td></td>
</tr>
<tr>
<td>M35-02</td>
<td>ISO97</td>
<td>38mm×64mm (2&quot;x3&quot;)</td>
<td>38mm×64mm (2&quot;x3&quot;)</td>
<td>38mm×89mm (2&quot;x4&quot;)</td>
<td>38mm×64mm (2&quot;x3&quot;)</td>
<td>75°  200°</td>
<td></td>
<td></td>
</tr>
<tr>
<td>M36-01</td>
<td>ISO97</td>
<td>38mm×64mm (2&quot;x3&quot;)</td>
<td>38mm×64mm (2&quot;x3&quot;)</td>
<td>38mm×89mm (2&quot;x4&quot;)</td>
<td>38mm×64mm (2&quot;x3&quot;)</td>
<td>64°  200°</td>
<td></td>
<td></td>
</tr>
<tr>
<td>M37-01</td>
<td>ISO97</td>
<td>38mm×64mm (2&quot;x3&quot;)</td>
<td>38mm×64mm (2&quot;x3&quot;)</td>
<td>38mm×89mm (2&quot;x4&quot;)</td>
<td>38mm×64mm (2&quot;x3&quot;)</td>
<td>100°  200°</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

**NOTES:**

1. All tested MIDPLY™ walls used 12.5 mm OSB as middle sheathing panel connected with Stanley stick power nail S12D (diameter = 3.05 mm, length = 82 mm). Studs were spaced at 406mm on centre. A vertical load of 44.5 kN was applied on each specimen.
2. Two rows of nails at specified nail spacing.
3. Two rows of nails at specified nail spacing. Each row connects different panels.
4. One row of nail was applied at the middle of the stud to prevent panel joint studs from falling apart.
5. Tested after M32-01 reached load capacity of the actuator.

The results of this test matrix (first quarter tests) are discussed in Chapter 7, section 7.1.

### 6.2.2 Finger Joined Test Matrix

The test matrix for the third quarter is shown in *Table 6.2*. Various configurations are tried for the studs and plates. SPS 1 (bending resistance) lumber was used exclusively for
the plates, since they are not vertically oriented. SPS 3 (vertical only) lumber was used for the studs, except for the end studs, which varied according to the test matrix.

The following parameters were examined in the third quarter (with corresponding wall numbers):

- Finger joined lumber vs. regular lumber behaviour (M41-01 vs. M40-01b and M45-01 vs. M14-01)

- Capacities of upper and lower bound walls made entirely of finger joined lumber (Upper bound: M41-01, M45-01. Lower bound: M43-01,a,&b)

- The effects of using SPS 1 and SPS 3 grades of finger-joined lumber for the end-studs (M45-01 vs. M41-01a and M44-01 vs. M43-01b)

- Effect of loading protocol on finger-joined MIDPLY™ walls (M42-01 vs. M42-02)

6.2.2.1 Loading Protocols

Four different types of loading protocols were used on the MIDPLY walls in the third quarter: ISO 97 cyclic protocol, Euro protocol, compression ramp protocol, and the tension ramp protocol. The tension ramp test was used in order to determine the complete load-deflection envelope of each wall configuration.
Table 6.2: MIDPLY™ Test Matrix for Third Quarter (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>Comp. Ramp</td>
<td>38mm x 89mm (2” x 4”)</td>
<td>610 mm (24”)</td>
<td>SPS 3</td>
<td>2 x 22.24 kN</td>
</tr>
<tr>
<td>M41-01a</td>
<td>cyclic (ISO 97)</td>
<td>38mm x 89mm (2” x 4”)</td>
<td>610 mm (24”)</td>
<td>SPS 3</td>
<td>2 x 22.24 kN</td>
</tr>
<tr>
<td>M42-01</td>
<td>Comp. Ramp</td>
<td>38mm x 64mm (2” x 3”)</td>
<td>406 mm (16”)</td>
<td>SPS 3</td>
<td>none</td>
</tr>
<tr>
<td>M42-01a</td>
<td>Tension Ramp</td>
<td>38mm x 64mm (2” x 3”)</td>
<td>406 mm (16”)</td>
<td>SPS 3</td>
<td>none</td>
</tr>
<tr>
<td>M42-02</td>
<td>cyclic (ISO 97)</td>
<td>38mm x 64mm (2” x 3”)</td>
<td>406 mm (16”)</td>
<td>SPS 3</td>
<td>none</td>
</tr>
<tr>
<td>M43-01</td>
<td>Cycle 1 (Euro.)</td>
<td>38mm x 64mm (2” x 3”)</td>
<td>610 mm (24”)</td>
<td>SPS 3</td>
<td>none</td>
</tr>
<tr>
<td>M43-01a</td>
<td>Cycle 2 (Euro.)</td>
<td>38mm x 64mm (2” x 3”)</td>
<td>610 mm (24”)</td>
<td>SPS 3</td>
<td>none</td>
</tr>
<tr>
<td>M44-01</td>
<td>Comp. Ramp</td>
<td>38mm x 64mm (2” x 3”)</td>
<td>610 mm (24”)</td>
<td>SPS 1</td>
<td>none</td>
</tr>
<tr>
<td>M45-01</td>
<td>cyclic (ISO 97)</td>
<td>38mm x 89mm (2” x 4”)</td>
<td>610 mm (24”)</td>
<td>SPS 1</td>
<td>2 x 22.24 kN</td>
</tr>
<tr>
<td>M45-01a</td>
<td>Tension Ramp</td>
<td>38mm x 89mm (2” x 4”)</td>
<td>610 mm (24”)</td>
<td>SPS 1</td>
<td>2 x 22.24 kN</td>
</tr>
</tbody>
</table>

NOTES:
1Nail spacing is standard:
Two rows of nails spaced at 100mm on the panel joint stud. Each of these rows connects different panels. One row of nails at 100mm nailed at the middle of the stud between the panel joint to prevent panel joint studs from separating from each other.
Elsewhere: two rows of nails at 200mm.

Until the third quarter, the only ramp test that has been used on the MIDPLY™ walls is the compression ramp test, which means that the wall is subjected to a lateral force in the compression direction, which is left, from the standard viewpoint (recall Figures 3.1 & 3.2). The tension ramp test, which subjects the lateral force in the opposite direction, was used to test the MIDPLY™ wall if the damage on the wall had mainly been on one side.
of the wall. In this way, the other side of the wall could still be tested and the load-displacement graph could be more complete.

Apart from this loading protocol is the Euro Protocol, which was used again (apart from the second quarter) to obtain an accurate wall stiffness value for a typical finger-joined MIDPLY™ wall configuration.

### 6.2.2.2 Wall Configurations

The wall configurations that were tested in the third quarter were based on upper and lower bound wall configurations, loading protocols, and previous MIDPLY™ tests of identically configured walls for comparison of finger-joined lumber with standard lumber.

The upper bound wall is the wall configuration that is expected to provide the highest ultimate load capacity. As shown in Table 6.2, both M45-01 and M41-01 were the upper bound walls tested in this quarter. Walls M45-01 and M41-01 compared the finger-joined grades of the end-studs. M43-01 and M44-01 were constructed of 38mm x 64mm (2in x 3in) studs and plates at a stud spacing of 610mm (24 in) on center. They were the lower bound wall configuration since no vertical loads were applied. Walls M42-01 and M42-02 were tested to compare their behaviour with wall M37-01, a similar wall, except for the finger-joined lumber.
All MIDPLY™ walls, 2.44m in height and length, were constructed with S-P-F FJ SPS 1 and 3 “stud grade” studs and S-P-F FJ SPS 1 and 3 “standard and better” grade top and bottom plates. 12.7mm Oriented Strand Board (OSB) was used for sheathing panels. Stanley stick power nails SI 2D, 3.05 mm in diameter and 82 mm in length, were used to sandwich sheathing panels. All walls from this quarter used the new hold down connection (Chapter 4).

### 6.3 Centre Stud Nailing

In the first year of the MIDPLY™ project, a common problem was the complete failure of the panel joint stud due to all the nails failing which hold the two studs together. What resulted was a separation of the two studs at the panel joint, leaving them both falling to the floor! An example of this problem is shown in Figure 6.2, which shows the panel joint studs completely falling off both walls in two separate wall tests, resulting in severe lateral buckling, and consequently total failure of the wall. The wall shown on the right, M25-02, failed through panel rupture after the panel joint stud fell off.
After having experienced this scenario a few times in first-year reversed cyclic wall tests, the cause of the problem was investigated and a solution tried for the first quarter tests of the second year. It was determined that the cause of the problem was that the nails on the panel joint stud experienced twice the force as those on the other perimeters of the panels since there was only one column of nails per panel at the panel joint stud. Therefore the nails would fail through fatigue sooner than at the other locations. Moreover, the force reversals on the nails, as depicted on Figure 6.3, were especially high on the panel joint due to the relative movement of one panel to the other on reversed-cyclic tests.

**Figure 6.2:** Panel Joint Stud failure through nail fatigue (first year testing)
Different solutions were tried to prevent the separation of the panel joint studs, including bolting the two studs together with 6 - 3/8" bolts. A description of the solution is given in the notes on the two test matrices shown in Tables 6.1 and 6.2. The solution was a separate column of nails nailed in the 1/8" gap between the two panels. This column of nails was there simply to hold the two studs together. Since these nails are not nailed through the panel, they do not experience shear forces, and therefore the studs stay intact. This method was used in the first quarter with excellent results, and therefore it was made a permanent detail in all future MIDPLY™ tests.
6.4 Transverse Buckling of MIDPLY™ walls

Another problem in the first year of the MIDPLY™ project that was solved in the beginning of the second year is that of transverse buckling. Transverse buckling refers to the deflection of the MIDPLY™ wall in the plane transverse to that of the wall. Two examples of this deflection are shown in Figure 6.4. The buckling is caused by the combination of the lateral load and vertical loads applied to the wall.

![Figure 6.4: Examples of extreme transverse buckling (first year testing)](image-url)
To solve the transverse buckling problem, an extra stud was added to the end of the MIDPLY™ wall, which was named the “buckling stud, because its sole purpose was to prevent transverse buckling. The configuration of the buckling stud is shown on Figure 2.4 - Type 2 of the end stud configurations (top right corner). The Type 1 configuration was used in MIDPLY™ tests until this transverse buckling was seen as a problem. The “buckling stud” provides resistance against transverse buckling because its strong axis is oriented in the transverse plane. To accommodate the bolt heads of the hold-down connections, the buckling stud is recessed to fit over the heads of the bolts. This is done by countersinking the stud in the area of the bolt heads. The stud is then nailed onto the end studs with standard 3 ¼” power nails.
Chapter 7: Static Test Results

7.1 First Quarter Results

In the first quarter, the following parameters were tested:

- stud spacing
- 2" x 3" (38mm x 64mm) studs and plates

As a reminder, the test matrix for the first quarter is listed:

<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>Nail spacing (mm)</th>
<th>Hold-downs</th>
<th>Variables Studied</th>
</tr>
</thead>
<tbody>
<tr>
<td>M32-01</td>
<td>Monotonic</td>
<td>44.48</td>
<td>2x4&quot;/2x4&quot;</td>
<td>2x4&quot;/2x4&quot;</td>
<td>406</td>
<td>100 / 200</td>
<td>Old</td>
<td>Stud spacing</td>
</tr>
<tr>
<td>M32-01a</td>
<td>ISO97</td>
<td>44.48</td>
<td>2x4&quot;/2x4&quot;</td>
<td>2x4&quot;/2x4&quot;</td>
<td>406</td>
<td>100 / 200</td>
<td>Old</td>
<td>Stud spacing</td>
</tr>
<tr>
<td>M33-01</td>
<td>Monotonic</td>
<td>44.48</td>
<td>2x4&quot;/2x3&quot;</td>
<td>2x3&quot;/2x3&quot;</td>
<td>406</td>
<td>100 / 200</td>
<td>Old</td>
<td>2x3&quot; lumber</td>
</tr>
<tr>
<td>M33-02</td>
<td>ISO97</td>
<td>44.48</td>
<td>2x4&quot;/2x3&quot;</td>
<td>2x3&quot;/2x3&quot;</td>
<td>406</td>
<td>100 / 200</td>
<td>Old</td>
<td>2x3&quot; lumber</td>
</tr>
<tr>
<td>M34-01</td>
<td>Monotonic</td>
<td>44.48</td>
<td>2x4&quot;/2x3&quot;</td>
<td>2x4&quot;/2x3&quot;</td>
<td>406</td>
<td>100 / 200</td>
<td>Old</td>
<td>2x3&quot; lumber</td>
</tr>
<tr>
<td>M34-02</td>
<td>ISO97</td>
<td>44.48</td>
<td>2x4&quot;/2x3&quot;</td>
<td>2x4&quot;/2x3&quot;</td>
<td>406</td>
<td>100 / 200</td>
<td>Old</td>
<td>2x3&quot; lumber</td>
</tr>
<tr>
<td>M35-01</td>
<td>Monotonic</td>
<td>44.48</td>
<td>2x3&quot;/2x3&quot;</td>
<td>2x4&quot;/2x3&quot;</td>
<td>406</td>
<td>75 / 200</td>
<td>Old</td>
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<td>ISO97</td>
<td>44.48</td>
<td>2x3&quot;/2x3&quot;</td>
<td>2x4&quot;/2x3&quot;</td>
<td>406</td>
<td>75 / 200</td>
<td>Old</td>
<td>2x3&quot; lumber</td>
</tr>
<tr>
<td>M36-01</td>
<td>ISO97</td>
<td>44.48</td>
<td>2x3&quot;/2x3&quot;</td>
<td>2x4&quot;/2x3&quot;</td>
<td>406</td>
<td>64 / 200</td>
<td>Old</td>
<td>2x3&quot; lumber</td>
</tr>
<tr>
<td>M37-01</td>
<td>ISO97</td>
<td>44.48</td>
<td>2x3&quot;/2x3&quot;</td>
<td>2x4&quot;/2x3&quot;</td>
<td>406</td>
<td>100 / 200</td>
<td>Old</td>
<td>2x3&quot; lumber</td>
</tr>
</tbody>
</table>

Notes:
- M32-01a is the same wall as M32-01, and retested, since M32-01 did not fail.
- Nail Spacing split into “Joint” and “Else” categories, meaning panel perimeter nails and interior stud nails respectively.
- Middle row of nails on the panel joint stud was introduced starting with M37-01.
Chapter 7: Static Test Results

7.1.1 Stud Spacing

*Table 7.1* summarizes the test results of MIDPLY™ walls with 610mm and 406mm stud spacing. Walls with 610mm stud spacing (M28-01, M30-01 and M14-01) were tested in the first year of the MIDPLY™ wall project. It was found that walls with 610mm stud spacing buckled at a lateral load of 45kN under reversed cyclic loads. For walls M28-01 and M14-01, the panel joint studs fell apart after nails on these studs broke, then panel sections between the interior stud and the panel joint stud buckled heavily and eventually fractured, as was shown on Figure 6.2.

<table>
<thead>
<tr>
<th>Wall Number</th>
<th>Load Protocol</th>
<th>Cycle</th>
<th>Sheathing Panel Type</th>
<th>Stud Spacing (mm)</th>
<th>$P_{\text{max}}$ (kN)</th>
<th>$\delta_0$ (mm)</th>
<th>K (kN/mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>M28-01</td>
<td>ISO97</td>
<td>1$^{st}$</td>
<td>1/2'' CSP Vertical</td>
<td>610</td>
<td>70.3</td>
<td>79</td>
<td>3.54</td>
</tr>
<tr>
<td></td>
<td></td>
<td>3$^{rd}$</td>
<td></td>
<td></td>
<td>62.2</td>
<td>84</td>
<td>3.77</td>
</tr>
<tr>
<td>M30-01</td>
<td>ISO97</td>
<td>1$^{st}$</td>
<td>1/2'' CSP Vertical</td>
<td>610</td>
<td>75.4</td>
<td>74</td>
<td>3.18</td>
</tr>
<tr>
<td></td>
<td></td>
<td>3$^{rd}$</td>
<td></td>
<td></td>
<td>66.6</td>
<td>75</td>
<td>3.45</td>
</tr>
<tr>
<td>M14-01</td>
<td>ISO97</td>
<td>1$^{st}$</td>
<td>1/2'' OSB Vertical</td>
<td>610</td>
<td>66.6</td>
<td>80</td>
<td>4.70</td>
</tr>
<tr>
<td></td>
<td></td>
<td>3$^{rd}$</td>
<td></td>
<td></td>
<td>56.6</td>
<td>64</td>
<td>5.30</td>
</tr>
<tr>
<td>M32-01</td>
<td>Monotonic</td>
<td></td>
<td>1/2'' OSB Vertical</td>
<td>406</td>
<td>88.6</td>
<td>102</td>
<td>3.87</td>
</tr>
<tr>
<td>M32-01a</td>
<td>ISO97</td>
<td>1$^{st}$</td>
<td>1/2'' OSB Vertical</td>
<td>406</td>
<td>68.8</td>
<td>71</td>
<td>2.60</td>
</tr>
<tr>
<td></td>
<td></td>
<td>3$^{rd}$</td>
<td></td>
<td></td>
<td>54.9</td>
<td>72</td>
<td>2.42</td>
</tr>
</tbody>
</table>

**Notes:**
1. All tested MIDPLY™ walls used stud grade and standard and better grade of Spruce-Pine-Fir as framing members, and Stanley stick power nail S12D (diameter = 3.05 mm, length = 82 mm). A vertical load of 44.48 kN was applied on each specimen. Two rows of nails were spaced at 102 mm (4 inch) on centre at panel joint studs, and 203 mm (8 inch) on centre elsewhere.
2. Load capacity of the actuator was reached.
3. Tested after M32-01 reached load capacity of the actuator.
4. Six bolts were applied at the middle of the stud to prevent panel joint studs from falling apart.
5. Refers to the cycle in each set of cycles in the reversed cyclic loading protocol.
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The load capacity of the actuator was reached under monotonic loading for the wall with 406mm stud spacing (M32-01). No buckling was observed. The panel fractured around the area of the hold-down under compression. This is likely due to the panel stress concentration since the access hole for the installation of the anchor bolt is about 100 mm away from the hold down connection. The wall was then tested under reversed cyclic loads. Although the wall was partially damaged under monotonic load, it still yielded comparable lateral load capacity and displacement at 80% of maximum load to that of walls with 610mm stud spacing. Again, no buckling was observed under reversed cyclic loading.

One of the end studs was broken under reversed cyclic loading. This time, the M32-01a panel fractured near the other hold down (opposite side of the wall as M32-01).

To compare the two different stud spacings, the load-displacement response of each kind is shown in Figure 7.1. These two walls, M14-01 and M32-01a, are identical except for their stud spacing.
Chapter 7: Static Test Results

Figure 7.1: Effect of Stud Spacing on MIDPLY™ walls

It is shown by this graph that the stud spacing does not have much effect on the wall strength. Wall M32-01a was tested monotonically once before the test shown on this graph, and it still came close to reaching the same capacity as M14-01. Therefore, stud spacing is deemed to be a parameter with minor influence on wall capacity/strength.

7.1.2 2" x 3" (38mm x 64mm) Studs and Plates

The test results of MIDPLY™ walls with 38 mm x 64 mm studs or plates are summarized in Table 7.2. In general, walls with 38mm x 64mm studs or plates had comparable lateral load capacity and displacement at 80% of the maximum load to that of walls with 38mm x 89mm studs and plates.
### Table 7.2: Further 1st Quarter Test Results (Effect of 38-mm x 64-mm size lumber)

<table>
<thead>
<tr>
<th>Wall Number</th>
<th>Load Protocol</th>
<th>Cycle</th>
<th>Plate</th>
<th>Stud</th>
<th>Nail Spacing (mm)</th>
<th>P_{max} (kN)</th>
<th>δ_{u} (mm)</th>
<th>K (kN/mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>M32-01</td>
<td>Monotonic</td>
<td></td>
<td>2\times4''/2\times4''</td>
<td>100'/200''</td>
<td>88.6</td>
<td>102</td>
<td>3.87</td>
<td></td>
</tr>
<tr>
<td>M32-01a</td>
<td>ISO97</td>
<td>1st</td>
<td>2\times4''/2\times4''</td>
<td>100'/200''</td>
<td>68.8</td>
<td>71</td>
<td>2.60</td>
<td></td>
</tr>
<tr>
<td>M33-01</td>
<td>Monotonic</td>
<td></td>
<td>2\times3''/2\times3''</td>
<td>100'/200''</td>
<td>80.0</td>
<td>98</td>
<td>4.55</td>
<td></td>
</tr>
<tr>
<td>M33-02</td>
<td>ISO97</td>
<td>1st</td>
<td>2\times3''/2\times3''</td>
<td>100'/200''</td>
<td>70.5</td>
<td>77</td>
<td>5.22</td>
<td></td>
</tr>
<tr>
<td>M34-01</td>
<td>Monotonic</td>
<td></td>
<td>2\times3''/2\times3''</td>
<td>100'/200''</td>
<td>83.9</td>
<td>111</td>
<td>4.42</td>
<td></td>
</tr>
<tr>
<td>M34-02</td>
<td>ISO97</td>
<td>1st</td>
<td>2\times3''/2\times3''</td>
<td>100'/200''</td>
<td>69.1</td>
<td>79</td>
<td>5.43</td>
<td></td>
</tr>
<tr>
<td>M35-01</td>
<td>Monotonic</td>
<td></td>
<td>2\times3''/2\times3''</td>
<td>75'/200''</td>
<td>82.7</td>
<td>92</td>
<td>4.84</td>
<td></td>
</tr>
<tr>
<td>M35-02</td>
<td>ISO97</td>
<td>1st</td>
<td>2\times3''/2\times3''</td>
<td>75'/200''</td>
<td>80.0</td>
<td>83</td>
<td>5.35</td>
<td></td>
</tr>
<tr>
<td>M36-01</td>
<td>ISO97</td>
<td>1st</td>
<td>2\times3''/2\times3''</td>
<td>64'/200''</td>
<td>83.7</td>
<td>94</td>
<td>5.50</td>
<td></td>
</tr>
<tr>
<td>M37-01</td>
<td>ISO97</td>
<td>1st</td>
<td>2\times3''/2\times3''</td>
<td>100'/200''</td>
<td>78.3</td>
<td>98</td>
<td>5.74</td>
<td></td>
</tr>
</tbody>
</table>

**Notes:**

1. All tested MIDPLY™ walls used Spruce-Pine-Fir as framing materials, and 12.5 mm OSB as middle sheathing panel connected with power nail (diameter = 3.05 mm, length = 82 mm). Studs were spaced at 406mm on centre. A vertical load of 44.48 kN was applied on each specimen.
2. Two rows of nails at specified nail spacing.
3. Two rows of nails at specified nail spacing. Each row connects different panels.
4. One row of nails was applied at the middle of the stud to prevent panel joint studs from falling apart.
5. Tested after M32-01 reached load capacity of the actuator.

For walls M33-01 and M33-02, the 38mm x 64mm end studs failed in tension around the hold downs under both monotonic and reversed cyclic loads. The sudden drop in load was observed due to failure in the end studs. This type of failure mode is not desired because studs failing in tension constitute a brittle failure mode. *Figure 7.2* shows the load-displacement response for wall M33-01. Based on the test results of walls M33-01 and M33-02, it was decided that a minimum size of 38mm x 89mm lumber for end studs had to be used. For walls M34-02 and M35-02, no 38mm x 89mm end studs failed under...
reverse cyclic loading. For walls M34-01 and M35-01, however, the 38mm x 89mm end studs failed in tension near the hold down area under monotonic load. This failure was shown in Figure 4.2 (M35-01). This may have been caused by the combined tension stresses and bending moments near the hold downs. As explained in Chapter 4, the bending moment is generated largely by the hold down connection through restraining the rotation of the end stud.

![Figure 7.2: Load-Displacement response of MIDPLY™ wall M33-01](image)

For walls M33-02 and M34-02, the studs at the panel joint separated under reverse cyclic loads. This type of failure mode is also not desired because the studs provide support for the vertical loads due to dead and live loads. Wall M37-01 used the nailing pattern described in Chapter 6 - the row of nails in the gap between the panels (the panel joint).
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The response of wall M37-01 is shown in Figure 7.3. As displayed by the graph, wall M37-01 performed exquisitely, and therefore the nailing pattern used on this wall was used on all subsequent MIDPLY™ tests. The reason that the wall behaved so well is that the nails in the panel joint gap did not experience any shear forces, and therefore they held the panel joint studs together.

![New Nailing Pattern for Panel Joint](image)

**Figure 7.3:** Load-Displacement response of wall M37-01, using 2”x3” studs and plates, and a new nailing pattern for the panel joint stud.

As shown on the graph, wall M37-01 does not have pinched curves, as compared to other walls tested this year. This is due to relatively high residual strength of the wall during a stress reversal. Pinched loops are caused by slackness or loosening of the joints, which in turn are caused by initial or previous cyclic loading. The pinching effect is caused by the
slippage of the joint as it reverses through the void that has been created by the previous cycle(s). Timber joints are known to have pinched curves and the pinching effect is more pronounced with greater fastener sizes. It is not known exactly why wall M37-01 has a relatively "unpinched" curve. Less pinching results in relatively high energy dissipation, which is a desirable quality in a lateral load resisting system. The energy dissipated for wall M37-01 was calculated to be $5.89 \times 10^4$ kN*mm.

The response of wall M37-01, with a maximum load of approximately 80 kN (33 kN/m), indicates that the use of 2"x3" studs and plates in the MIDPLY™ wall is viable.

Meetings were held with industry members of Forintek Canada Corp. and the consensus was to continue testing MIDPLY™ walls with 2"x3" studs and/or plates, especially to include this parameter in the dynamic testing portion of the MIDPLY™ project. The MIDPLY™ wall with 2"x3" members is viewed as the "lower bound" configuration – the least strong out of the MIDPLY™ configurations chosen for final use.

### 7.2 Second Quarter Results

In the second quarter, the main parameter tested was the new hold down connection. In addition to this, the Euro loading protocol was also tested, but this was secondary to testing the hold down. The loading protocol would not actually affect the strength, stiffness or ductility, but would improve ways to measure these performance parameters.
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As shown by the test matrix below, the two other variables in the second quarter were the vertical loads and the member size (2"x3"). The vertical loads were omitted in some tests because that creates more demand on the hold downs. The member sizes varied because of the fact that 2"x3" members should be included, as noted in the previous section of this chapter.

<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>Nail spacing (mm)</th>
<th>Hold-downs</th>
<th>Variables Studied</th>
</tr>
</thead>
<tbody>
<tr>
<td>M38-01</td>
<td>Cycle 1-Euro</td>
<td>None</td>
<td>Top/Bottom</td>
<td>End/Interior</td>
<td>610</td>
<td>100 / 200</td>
<td>Old</td>
<td>Hold-down</td>
</tr>
<tr>
<td>M38-01a</td>
<td>Cycle 2-Euro</td>
<td>None</td>
<td>2x4&quot;/2x4&quot;</td>
<td>2x4&quot;/2x4&quot;</td>
<td>610</td>
<td>100 / 200</td>
<td>Old</td>
<td>Hold-down</td>
</tr>
<tr>
<td>M38-01b</td>
<td>Ramp-Euro</td>
<td>None</td>
<td>2x4&quot;/2x4&quot;</td>
<td>2x4&quot;/2x4&quot;</td>
<td>610</td>
<td>100 / 200</td>
<td>Old</td>
<td>Hold-down</td>
</tr>
<tr>
<td>M39-01</td>
<td>Cycle 1-Euro</td>
<td>None</td>
<td>2x4&quot;/2x4&quot;</td>
<td>2x4&quot;/2x4&quot;</td>
<td>610</td>
<td>100 / 200</td>
<td>New</td>
<td>Hold-down</td>
</tr>
<tr>
<td>M39-01a</td>
<td>Cycle 2-Euro</td>
<td>None</td>
<td>2x4&quot;/2x4&quot;</td>
<td>2x4&quot;/2x4&quot;</td>
<td>610</td>
<td>100 / 200</td>
<td>New</td>
<td>Hold-down</td>
</tr>
<tr>
<td>M39-01b</td>
<td>Ramp-Euro</td>
<td>None</td>
<td>2x4&quot;/2x4&quot;</td>
<td>2x4&quot;/2x4&quot;</td>
<td>610</td>
<td>100 / 200</td>
<td>New</td>
<td>Hold-down</td>
</tr>
<tr>
<td>M40-01</td>
<td>Cycle 1-Euro</td>
<td>44.48</td>
<td>2x3&quot;/2x3&quot;</td>
<td>2x3&quot;/2x3&quot;</td>
<td>610</td>
<td>100 / 200</td>
<td>New</td>
<td>Hold-down</td>
</tr>
<tr>
<td>M40-01a</td>
<td>Cycle 2-Euro</td>
<td>44.48</td>
<td>2x3&quot;/2x3&quot;</td>
<td>2x3&quot;/2x3&quot;</td>
<td>610</td>
<td>100 / 200</td>
<td>New</td>
<td>Hold-down</td>
</tr>
<tr>
<td>M40-01b</td>
<td>Ramp-Euro</td>
<td>44.48</td>
<td>2x3&quot;/2x3&quot;</td>
<td>2x3&quot;/2x3&quot;</td>
<td>610</td>
<td>100 / 200</td>
<td>New</td>
<td>Hold-down</td>
</tr>
</tbody>
</table>

Notes:
- The “100 / 200” nail spacing, with the middle row of nails at the panel joint is standard, since wall M37-01.
- The “New” hold down refers to the one described in Chapter 4.
- The Euro protocol has two cycles, and then a ramp portion.

As shown, three different walls were tested - each three times, under the loading protocol named “Euro”, shown in Chapter 3 (Figure 3.4). The first two walls, M38-01 and M39-01, were identical and they were compared to determine the effect of the new hold down.

Table 7.3 summarizes the test results of the MIDPLY™ walls tested in the second quarter. The two walls with the new hold downs, M39-01b and M40-01b, had a significantly higher ultimate load capacity without an increase in stiffness over the wall with the previous hold down. The average lateral stiffness was 3.41 kN/mm for the wall.
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with the previous hold downs and 3.46 kN/mm for the wall with the new hold downs (Table 7.3). The walls with the new hold downs were also more ductile than the wall with the previous hold down, as shown in Figure 7.4.

### Table 7.3: Test Results of the Second Quarter (Effect of New Hold Downs on MIDPLY™ walls)

<table>
<thead>
<tr>
<th>Wall Number</th>
<th>Load Protocol</th>
<th>Hold-down type</th>
<th>( P_{\text{max}} ) (kN)</th>
<th>( \delta_u ) (mm)</th>
<th>Max. end-stud uplift (mm)</th>
<th>K (kN/mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>M38-01</td>
<td>Cycle 1</td>
<td>Previous</td>
<td>25.9</td>
<td>8.5</td>
<td>1.8</td>
<td>3.46</td>
</tr>
<tr>
<td>M38-01a</td>
<td>Cycle 2</td>
<td>Previous</td>
<td>27.7</td>
<td>8.6</td>
<td>1.4</td>
<td>3.37</td>
</tr>
<tr>
<td>M38-01b</td>
<td>Ramp</td>
<td>Previous</td>
<td>54.8</td>
<td>49</td>
<td>19.1*</td>
<td>3.40</td>
</tr>
<tr>
<td>M39-01</td>
<td>Cycle 1</td>
<td>New</td>
<td>26.2</td>
<td>8.1</td>
<td>1.4</td>
<td>3.56</td>
</tr>
<tr>
<td>M39-01a</td>
<td>Cycle 2</td>
<td>New</td>
<td>27.5</td>
<td>8.3</td>
<td>1.3</td>
<td>3.43</td>
</tr>
<tr>
<td>M39-01b</td>
<td>Ramp</td>
<td>New</td>
<td>73.5</td>
<td>120</td>
<td>16.5*</td>
<td>2.36</td>
</tr>
<tr>
<td>M40-01</td>
<td>Cycle 1</td>
<td>New</td>
<td>28.0</td>
<td>8.0</td>
<td>1.0</td>
<td>3.98</td>
</tr>
<tr>
<td>M40-01a</td>
<td>Cycle 2</td>
<td>New</td>
<td>30.8</td>
<td>8.3</td>
<td>1.2</td>
<td>3.92</td>
</tr>
<tr>
<td>M40-01b</td>
<td>Ramp</td>
<td>New</td>
<td>81.0</td>
<td>111</td>
<td>16.6*</td>
<td>3.98</td>
</tr>
</tbody>
</table>

**Notes:**

* - The LVDT (Linearly Variable Displacement Transistor) is at its maximum reading in these locations. The wall actually uplifted further than the LVDT could read.

Wall M38-01b failed in tension and bending at both the front and the rear end studs at load of 54.8 kN. No transverse buckling was observed on this wall. However, buckling of the wall perpendicular to its plane (transverse buckling) was observed in wall M39-01b and M40-01b and crushing of the panel was observed before the end studs failed. The buckling was greater on wall M40-01b because vertical loads were applied to the wall. The transverse buckling was most noticeable on the studs located in the middle of the panel on the compression side of the wall (left side, on a ramp test). This is due to the
fact that the vertical actuator is located near these studs. The buckling studs prevent the end studs from buckling as much as these studs do.

![Graph showing Load-Displacement response](image)

**Figure 7.4:** Load-Displacement response of MIDPLY™ walls M38-01b, M39-01b, and M40-01b, comparing the performance of the hold downs vs. the previous ones.

As shown in *Figure 7.4*, M40-01b experienced a "stiffening" effect starting at around 57kN. This effect is most likely induced by the plywood panel coming into contact with the lower anchor plate (test apparatus) as the panel rotates. Initially, the plywood panel is kept off the base of the bottom plate by a distance of 10mm. As the wall is laterally loaded, the panels rotate about their centroid. When the edge of the panel makes contact with the base, the panel is restrained and thus can withstand more lateral force. This behaviour is expected to occur in real-life application of the walls.
On M40-01b, the plywood crushed on the compression side of the wall at lateral load of approximately 75kN. This crushing occurred near the top left corner of the left panel and also at the bottom right corner of the left panel (wall loaded from right side).

*Figure 7.5* shows the end stud uplift of all three walls. Wall M40-01b did not experience as much uplift displacement at the end studs as M39-01b did because of the effect of vertical loads. The new hold down connections began to yield at an applied lateral load of approximately 70kN and a horizontal displacement of 70 to 85 mm. This is due to the combination of relatively high rotation of the end studs and restraint of the hold down by its vertical anchoring bolt.

![Graph](image)

*Figure 7.5:* Uplift displacement of MIDPLY™ walls M38-01b, M39-01b, and M40-01b, comparing the performance of the hold downs vs. the previous ones.
Figure 7.6 shows the new hold down under load in test M39-01b. Here we see the deformation of the hold down looks almost identical to the computer-generated structural model of the hold down shown in Chapter 4 (Figure 4.8). The bending is localized near the top of the triangular gusset, and the vertical anchoring bolt is bent at the top, where it connects to the top of the triangular gusset.

Figure 7.6: Uplift M39-01b, showing the deformation of the hold down at the top of the triangular gusset, and the uplift of the end stud.

There was less stress put on the end stud because of this yielding of the hold down. This deformation is desirable since it is a ductile component of the MIDPLY™ system. The hold down was concluded to be greater in strength and ductility, while retaining the same initial stiffness. It was therefore agreed upon by all that this new hold down should be used in future tests as a standard item of the MIDPLY™ system.
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Cyclic testing in future quarters yielded more information on how the hold down deformed.

7.3 Third Quarter Results

In the first quarter, the main parameter were tested was Finger-Joined Lumber. More specifically, the following issues were examined:

- Finger Joined lumber vs. regular lumber behaviour (M41-01 vs. M40-01b and M45-01 vs. M14-01)
- Capacities of upper and lower bound walls made entirely of finger joined lumber (Upper bound: M41-01, M45-01. Lower bound: M43-01, a&b)
- The effects of using SPS 1 and SPS 3 grades of finger joined lumber for the end studs

The test matrix for the third quarter is listed below:

<table>
<thead>
<tr>
<th>Wall Number</th>
<th>Load Protocol</th>
<th>Vertical Load (kN)</th>
<th>Plate ²</th>
<th>Stud ¹</th>
<th>Stud Spacing (mm)</th>
<th>Nail Spacing (mm)</th>
<th>Hold-Downs</th>
<th>Variables Studied</th>
</tr>
</thead>
<tbody>
<tr>
<td>M41-01</td>
<td>Comp. Ramp</td>
<td>44.48</td>
<td>2×4&quot;/2×4&quot;</td>
<td>2×4&quot;/2×4&quot;</td>
<td>610</td>
<td>100/200</td>
<td>New</td>
<td>FJ lumber</td>
</tr>
<tr>
<td>M41-01a</td>
<td>ISO97</td>
<td>44.48</td>
<td>2×4&quot;/2×4&quot;</td>
<td>2×4&quot;/2×4&quot;</td>
<td>610</td>
<td>100/200</td>
<td>New</td>
<td>FJ lumber</td>
</tr>
<tr>
<td>M42-01</td>
<td>Comp. Ramp</td>
<td>None</td>
<td>2×3&quot;/2×3&quot;</td>
<td>2×3&quot;/2×3&quot;</td>
<td>406</td>
<td>100/200</td>
<td>New</td>
<td>FJ lumber</td>
</tr>
<tr>
<td>M42-01a</td>
<td>Tens. Ramp</td>
<td>None</td>
<td>2×3&quot;/2×3&quot;</td>
<td>2×3&quot;/2×3&quot;</td>
<td>406</td>
<td>100/200</td>
<td>New</td>
<td>FJ lumber</td>
</tr>
<tr>
<td>M42-02</td>
<td>ISO97</td>
<td>None</td>
<td>2×3&quot;/2×3&quot;</td>
<td>2×3&quot;/2×3&quot;</td>
<td>406</td>
<td>100/200</td>
<td>New</td>
<td>FJ lumber</td>
</tr>
<tr>
<td>M43-01</td>
<td>Cycle 1-Euro</td>
<td>None</td>
<td>2×3&quot;/2×3&quot;</td>
<td>2×3&quot;/2×3&quot;</td>
<td>610</td>
<td>100/200</td>
<td>New</td>
<td>FJ lumber</td>
</tr>
<tr>
<td>M43-01a</td>
<td>Cycle 2-Euro</td>
<td>None</td>
<td>2×3&quot;/2×3&quot;</td>
<td>2×3&quot;/2×3&quot;</td>
<td>610</td>
<td>100/200</td>
<td>New</td>
<td>FJ lumber</td>
</tr>
<tr>
<td>M43-01b</td>
<td>Comp. Ramp</td>
<td>None</td>
<td>2×3&quot;/2×3&quot;</td>
<td>2×3&quot;/2×3&quot;</td>
<td>610</td>
<td>100/200</td>
<td>New</td>
<td>FJ lumber</td>
</tr>
<tr>
<td>M44-01</td>
<td>ISO97</td>
<td>None</td>
<td>2×3&quot;/2×3&quot;</td>
<td>2×3&quot;/2×3&quot;</td>
<td>610</td>
<td>100/200</td>
<td>New</td>
<td>FJ lumber</td>
</tr>
<tr>
<td>M45-01</td>
<td>ISO97</td>
<td>44.48</td>
<td>2×4&quot;/2×4&quot;</td>
<td>2×4&quot;/2×4&quot;</td>
<td>610</td>
<td>100/200</td>
<td>New</td>
<td>FJ lumber</td>
</tr>
<tr>
<td>M45-01a</td>
<td>Tens. Ramp</td>
<td>44.48</td>
<td>2×4&quot;/2×4&quot;</td>
<td>2×4&quot;/2×4&quot;</td>
<td>610</td>
<td>100/200</td>
<td>New</td>
<td>FJ lumber</td>
</tr>
</tbody>
</table>

Notes:  1 - The studs are all finger joined lumber: SPS 1 and 3 – stud grade
        2 - The plates are all finger joined lumber: SPS 1 and 3 – “standard and better grade.
Chapter 7: Static Test Results

The upper bound wall is the wall configuration that is expected to provide the highest ultimate load capacity. Walls M45-01 and M41-01 were the upper bound walls tested in this quarter. Walls M45-01 and M41-01 compared the finger joint grades of the end studs. The lower bound walls were M43-01 and M44-01, since no vertical loads were applied, stud size was minimum, and stud spacing was maximum. Walls M42-01 and M42-02 were tested to compare their behaviour with wall M37-01, a similar wall except for the finger joined lumber.

Other details of the testing plan was explained in Chapter 6, Section 6.2.2. The results are tabulated in Table 7.4.

<table>
<thead>
<tr>
<th>Wall Number</th>
<th>Load Protocol</th>
<th>End Stud Type</th>
<th>$P_{\text{max}}$ (kN)</th>
<th>$\delta_u$ (mm)</th>
<th>$K$ (kN/mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>M41-01</td>
<td>comp. ramp</td>
<td>SPS 3</td>
<td>71.9</td>
<td>125 $^1$</td>
<td>4.44</td>
</tr>
<tr>
<td>M41-01a</td>
<td>ISO 97</td>
<td>SPS 3</td>
<td>56.4</td>
<td>88.6 $^4$</td>
<td>2.56 $^3$</td>
</tr>
<tr>
<td>M42-01</td>
<td>comp. ramp</td>
<td>SPS 3</td>
<td>60.3</td>
<td>87.2</td>
<td>3.44</td>
</tr>
<tr>
<td>M42-01a</td>
<td>tension ramp</td>
<td>SPS 3</td>
<td>59.2</td>
<td>125 $^1$</td>
<td>1.93 $^3$</td>
</tr>
<tr>
<td>M42-02</td>
<td>ISO 97</td>
<td>SPS 3</td>
<td>62.0</td>
<td>74.4</td>
<td>4.06</td>
</tr>
<tr>
<td>M43-01</td>
<td>Cycle 1 (Euro.)</td>
<td>SPS 3</td>
<td>28.9 $^4$</td>
<td>8.9</td>
<td>3.25</td>
</tr>
<tr>
<td>M43-01a</td>
<td>Cycle 2 (Euro.)</td>
<td>SPS 3</td>
<td>31.1 $^2$</td>
<td>8.9</td>
<td>3.50</td>
</tr>
<tr>
<td>M43-01b</td>
<td>comp. ramp</td>
<td>SPS 3</td>
<td>63.7</td>
<td>108.6</td>
<td>3.94</td>
</tr>
<tr>
<td>M44-01</td>
<td>ISO 97</td>
<td>SPS 1</td>
<td>63.4</td>
<td>61.0</td>
<td>3.97 $^2$</td>
</tr>
<tr>
<td>M45-01</td>
<td>ISO 97</td>
<td>SPS 1</td>
<td>77.3</td>
<td>76.2</td>
<td>5.29</td>
</tr>
<tr>
<td>M45-01a</td>
<td>tension ramp</td>
<td>SPS 1</td>
<td>44.7</td>
<td>81.3 $^2$</td>
<td>0.38 $^3$</td>
</tr>
<tr>
<td>M14-01$^4$</td>
<td>ISO 97</td>
<td>standard</td>
<td>70.6</td>
<td>70.3</td>
<td>4.92</td>
</tr>
<tr>
<td>M40-01b$^5$</td>
<td>comp. ramp</td>
<td>standard</td>
<td>81.0</td>
<td>111</td>
<td>3.98 $^1$</td>
</tr>
</tbody>
</table>

Notes:

1. Maximum actuator displacement is reached (5 in).
2. Load corresponds to 8.9 mm displacement at the top of wall.
3. For tests that are executed on a wall which has already been tested.
4. Included for Comparison Purposes
7.3.1 Finger Joined Lumber vs. Regular Lumber Behaviour

It was found that the secant stiffness values 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 SPS 1 and SPS 3 grade lumber and standard lumber.

*Figure 7.7* displays the response of walls M40-01b and M41-01, which are identical except that M40-01b is built with standard lumber. No visible failure of the end-studs occurred in wall M41-01. Transverse buckling was the only failure mode in this wall. This is due to the presence of vertical loads on the wall, which prevented failure of the end studs. M40-01b and M41-01 behaved similarly until 70kN was reached.
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Figure 7.8 presents the load-displacement responses of walls M45-01 and M14-01. Besides the difference of finger joined vs. standard lumber, new hold downs were used for M45-01, while the old hold downs were used for M14-01. It was noticed that the walls behaved similarly until they approached 76mm lateral displacement. Even though wall M45-01 used the new hold down (subjecting end studs to less bending), the finger joined end studs failed, causing the wall to have virtually no capacity, whereas M14-01 retained about one third of its ultimate capacity and behaved in a ductile manner.

Figure 7.8: Comparison of wall M45-01 and M14-01: finger joined vs. standard lumber in a reversed-cyclic test.
Wall M45-01 failed abruptly at the third compression cycle (76mm); the front right end stud failed in tension at the finger joint. Since the left side of the wall was in good condition, the wall continued to behave well until the test reached the next compression cycle. The right rear end stud then failed and the test was stopped. The brittle failure mode at the end stud was undesirable.

7.3.2 Capacities of Upper and Lower Bound Walls

The upper bound wall, M41-01, achieved an ultimate load of 71.9 kN (16,160 lbs.), which is a relatively high lateral load compared to the average MIDPLY™ wall tested. This is probably due to the applied vertical load, which prevented the end-studs from failing.

The lower bound wall, M43-01b, achieved an ultimate load of 63.7 kN. It was observed that this wall failed at the finger joints of the end-studs. This capacity is similar to that of walls M42-01, M42-02, and M44-01. This is probably because walls M42-01, M42-02, and M44-01 also failed at finger joints of the end studs. The premature failure reduced the capacity of those walls. Figure 7.9 displays the failure of the end-studs. The photo on the left shows the finger joint at the beginning stage of failure.
7.3.3 The Effects of using SPS 1 and SPS 3 for the End Studs

The failure modes of walls M45-01 and M41-01a were different. M41-01a only experienced lateral buckling, whereas M45-01 experienced end stud failure. These two walls behaved differently than expected because the SPS 1 end studs were expected to withstand the uplift forces and buckling forces better than the SPS 3 end studs.

The failure modes of walls M44-01 and M43-01b, which are identical except for the end stud grade and loading protocol, were similar – both tests resulted in end stud failure. A comparison was made between the envelope curve of M44-01 and the load displacement curve of M43-01b, as shown in Figure 7.10. It was found that the two walls attained almost identical ultimate loads of about 63.5 kN and also had nearly identical stiffness values (3.95 kN/mm). Based on these test results, it is indicative that end stud grade does not have a significant effect on the behaviour of the finger joined MIDPLY™ walls.
Figure 7.10: Comparing walls with SPS 1 and SPS 3 end studs

7.3.4 General Conclusions on Tests in the Third Quarter

When finger joined lumber was used for the end studs of MIDPLY™ walls, the lateral load capacity was reduced. This is due to the fact that finger joined lumber is not designed to withstand the magnitude of bending moments and tension forces generated at the end studs of the MIDPLY™ walls. Since the failure mode was predominantly undesirable abrupt failure of the end studs, it was decided to use MSR (machine stress rated) lumber for the end studs in subsequent MIDPLY™ tests. The finger joined lumber behaved well as plates and interior studs. It can be initially concluded from these tests.
Chapter 7: Static Test Results

that finger joined lumber is a viable material to use in the MIDPLY™ walls, except for end studs.

7.4 Fourth Quarter Results

In the fourth quarter, the one parameter that was tested was the loading protocol. The test matrix of the fourth quarter is shown below:

<table>
<thead>
<tr>
<th>Wall Number</th>
<th>Load Protocol</th>
<th>Vertical Load (kN)</th>
<th>Plate Top/Bottom</th>
<th>Stud End/Interior</th>
<th>Stud Spacing (mm)</th>
<th>Nail Spacing Joint/Else</th>
<th>Hold-Downs</th>
<th>Variables Studied</th>
</tr>
</thead>
<tbody>
<tr>
<td>M46-01</td>
<td>ISO98</td>
<td>None</td>
<td>2x4&quot;/ 2x4&quot;</td>
<td>2x4&quot;/ 2x4&quot;</td>
<td>406</td>
<td>100/200</td>
<td>New</td>
<td>Protocol</td>
</tr>
<tr>
<td>M46-01a</td>
<td>ISO98</td>
<td>None</td>
<td>2x4&quot;/ 2x4&quot;</td>
<td>2x4&quot;/ 2x4&quot;</td>
<td>406</td>
<td>100/200</td>
<td>New</td>
<td>Protocol</td>
</tr>
<tr>
<td>M46-01b</td>
<td>ISO98</td>
<td>None</td>
<td>2x4&quot;/ 2x4&quot;</td>
<td>2x4&quot;/ 2x4&quot;</td>
<td>406</td>
<td>100/200</td>
<td>New</td>
<td>Protocol</td>
</tr>
<tr>
<td>M46-01c</td>
<td>ISO98</td>
<td>None</td>
<td>2x4&quot;/ 2x4&quot;</td>
<td>2x4&quot;/ 2x4&quot;</td>
<td>406</td>
<td>100/200</td>
<td>New</td>
<td>Protocol</td>
</tr>
<tr>
<td>M46-01d</td>
<td>ISO98</td>
<td>None</td>
<td>2x4&quot;/ 2x4&quot;</td>
<td>2x4&quot;/ 2x4&quot;</td>
<td>406</td>
<td>100/200</td>
<td>New</td>
<td>Protocol</td>
</tr>
<tr>
<td>M47-02</td>
<td>ISO98</td>
<td>None</td>
<td>2x3&quot;/ 2x3&quot;</td>
<td>2x3&quot;/ 2x3&quot;</td>
<td>610</td>
<td>100/200</td>
<td>New</td>
<td>Protocol</td>
</tr>
<tr>
<td>M47-02a</td>
<td>ISO98</td>
<td>None</td>
<td>2x3&quot;/ 2x3&quot;</td>
<td>2x3&quot;/ 2x3&quot;</td>
<td>610</td>
<td>100/200</td>
<td>New</td>
<td>Protocol</td>
</tr>
<tr>
<td>M47-02b</td>
<td>ISO98</td>
<td>None</td>
<td>2x3&quot;/ 2x3&quot;</td>
<td>2x3&quot;/ 2x3&quot;</td>
<td>610</td>
<td>100/200</td>
<td>New</td>
<td>Protocol</td>
</tr>
<tr>
<td>M47-02c</td>
<td>ISO98</td>
<td>None</td>
<td>2x3&quot;/ 2x3&quot;</td>
<td>2x3&quot;/ 2x3&quot;</td>
<td>610</td>
<td>100/200</td>
<td>New</td>
<td>Protocol</td>
</tr>
<tr>
<td>M47-02d</td>
<td>ISO98</td>
<td>None</td>
<td>2x3&quot;/ 2x3&quot;</td>
<td>2x3&quot;/ 2x3&quot;</td>
<td>610</td>
<td>100/200</td>
<td>New</td>
<td>Protocol</td>
</tr>
</tbody>
</table>

Notes:
1: 6.4 mm maximum displacement. 2: 12.7 mm maximum displacement.
3: 25.4 mm maximum displacement. 4: 50.8 mm maximum displacement.
5: 125 mm maximum displacement.

As shown by the test matrix, there were only two walls tested in the last quarter, each one being tested five times by increasing amplitudes of the ISO 98 load protocol up to the maximum displacement of 125mm. This protocol was the newest internationally accepted reversed-cyclic protocol at the time.

Another purpose of the fourth quarter tests is to acquire the data needed for the dynamic tests, which were carried out the following quarter. The two walls, M46-01 and M47-02,
Chapter 7: Static Test Results

were the same walls that were chosen to be tested in the dynamic tests (shake table tests). The information gathered from the results of these tests was used to plan the shake table tests. More specifically, response parameters such as stiffness, maximum load, and maximum deflection were used in modelling the walls using the computer program DRAIN-2DX, so the dynamic tests could be planned to some degree of precision.

Instead of tabulating the results of the fourth quarter tests, a table comparing similar walls under different loading protocols will be shown. First, the load displacement graphs for walls M46-01 and M47-02 are shown in Figures 7.11 and 7.12 respectively.

![Figure 7.11: Load-displacement graph for the upper bound wall: M46-01a, b, c & d](image)

Figure 7.11: Load-displacement graph for the upper bound wall: M46-01a, b, c & d
Wall M46-01 behaved very well until the second cycle of the last set (M46-01d), at which point the left end stud failed in tension. This is shown on the graph on the top right quadrant, where there is a dip at the top of the curve. The reason the curve came back up after the dip is that the other end stud (backside) took up the load once the front end stud broke. After the end stud broke, the OSB started rupturing in small chunks near the bottom plate. Also, the nails started failing in chip-out failure on the end studs. Therefore the failure mode of M46-01 included three types of failures, and was well distributed on the wall, which is desirable.

![Graph showing load-displacement relationship for M46-01 and other samples.](image)

**Figure 7.12:** Load-displacement graph for the lower bound wall: M47-02, a,b,c & d
Chapter 7: Static Test Results

Wall M47-02 was tested after the M47-01 test failed and was aborted. Wall M47-02 failed after the nailed in the top plate failed. This is more desirable than an end stud failure since it happened more gradually than the typically abrupt end stud failure.

The main purpose of the fourth quarter tests was to test the ISO 98 loading protocol, and therefore Walls M46-01 and M47-02 should be compared to similar, if not identical, walls subjected to other reversed cyclic loading protocols. The parameters by which to measure a protocol were listed in Chapter 3; number of cycles, duration, amplitude, frequency, and pattern. But most important is how the wall behaves in response to the protocol, which is measured by the wall response parameters: maximum load, displacement, energy dissipation, etc. The envelope curve of monotonic tests are also worth comparing to the cyclic ones. Table 7.5 lists the results for similar MIDPLY™ walls under different loading protocols.

Table 7.5: Test Results of some similar MIDPLY™ walls

<table>
<thead>
<tr>
<th>Wall Number</th>
<th>Load Protocol</th>
<th>Vertical Load (kN)</th>
<th>Stud Size / Plate Size</th>
<th>Stud Spacing (mm)</th>
<th>$P_{\text{max}}$ (kN)</th>
<th>$\delta_u$ (mm)</th>
<th>Energy Dissipated (kN*mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>M26-01</td>
<td>Monotonic</td>
<td>None</td>
<td>2 x 4&quot;/ 2 x 4&quot;</td>
<td>406</td>
<td>54.3</td>
<td>108</td>
<td>n/a</td>
</tr>
<tr>
<td>M31-01</td>
<td>ISO97</td>
<td>None</td>
<td>2 x 4&quot;/ 2 x 4&quot;</td>
<td>406</td>
<td>67.9</td>
<td>89</td>
<td>48839</td>
</tr>
<tr>
<td>M46-01d²</td>
<td>ISO98</td>
<td>None</td>
<td>2 x 3&quot;/ 2 x 3&quot;</td>
<td>610</td>
<td>63.7</td>
<td>109</td>
<td>60116</td>
</tr>
<tr>
<td>M43-01b²</td>
<td>Monotonic</td>
<td>None</td>
<td>2 x 3&quot;/ 2 x 3&quot;</td>
<td>610</td>
<td>63.4</td>
<td>61</td>
<td>20962</td>
</tr>
<tr>
<td>M44-01b²</td>
<td>ISO97</td>
<td>None</td>
<td>2 x 3&quot;/ 2 x 3&quot;</td>
<td>610</td>
<td>62.5</td>
<td>80</td>
<td>43012</td>
</tr>
</tbody>
</table>

 Notes:
1. Finger joined lumber
2. New hold down (inverted-triangle hold down)
Chapter 7: Static Test Results

The ultimate loads for walls tested under ISO 97 and ISO98 were similar. The monotonic tests show greater ductility than the cyclic tests. Shortly after they reached their ultimate loads, they exhibited strength degradation. This resulted in a reduced ductility compared to walls tested under monotonic displacement schedule. Table 7.5 shows that the ISO98 walls dissipated significantly more energy than the ISO97 walls. This could be due to the fact that the ISO98 protocol was run five times on different amplitudes, as opposed to the one time the ISO97 was run.

Walls M46-01 and M47-02 were subjected five consecutive ISO98s at different maximum displacement levels. These tests provided important information for the performance of walls at seismically active areas where structures may be subjected to several moderate earthquakes before a major earthquake strike them. From the tests, it was found that walls with previous loading history still provided comparable ductility and ultimate lateral load capacities compared to walls subjected to only one cyclic displacement schedule. The energy dissipation is also clearly superior in the ISO98 protocol.
Figure 7.13: Envelope curves for walls with 2x4" studs/plates and 406mm stud spacing
This concludes the results of the static testing performed at Forintek Canada Corp. in the second year of the MIDPLY™ project. The following two chapters deal with the dynamic testing, and the last chapter deals with the comparison of static and dynamic test results.
Chapter 8: Dynamic Modelling and Testing

Until this chapter of the thesis, the subject has been the static testing and analysis of the MIDPLY™ system. This chapter begins the dynamic testing portion i.e. shake table testing. The chapter is divided into two parts because they are separate projects in themselves.

8.1 Modelling

8.1.1 Objectives

In the context of this thesis, “modelling” constitutes creating a computer model of a MIDPLY™ wall so its response to dynamic excitation can be predicted. The specific objectives of dynamic modelling in the MIDPLY™ project are two-fold:

- To predict the behaviour of the wall in response to the earthquake acceleration time histories, so parameters of the shake table tests can be determined: such as mass, acceleration, displacement, and types of earthquake records needed, and the natural frequency of the system.

- To use this model as a tool for future MIDPLY™ walls so their response can be estimated as elements within a complete structure.
Chapter 8:  Dynamic Modelling and Testing

Modelling reduces the need for further testing; when a wall is accurately modelled, engineers can use the model to more rapidly retrieve information about its behaviour, rather than undertake the laborious task of testing anew.

8.1.2 Structural Dynamic Models

There are many different kinds of structural models for timber structures and connections for dynamic loading. These can be grouped into three categories (Foliente, 1994):

- lumped parameter models
- structural component discrete hinge models
- finite element models

Some models include a combination of two of these types. The DYNWALL program (Dolan, 1989) was a finite element model and SWAP (Filiatrault, 1990) included a structural component for each nailed connection.

All models are either empirical or theoretical, or a combination of both. Empirical models, such as the hysterisis model used on the MIDPLY™ system, rely on previous test results, from which the response characteristics of the structure are taken and input into the program. Theoretical models, such as finite element models, rely on structural theory to analyze the structure; only the material properties and geometry are input into the model.
Every structure is realistically a multi-degree-of-freedom (MDOF) system, often with infinite degrees of freedom. Some structures can be simplified theoretically to a single-degree-of-freedom (SDOF) system for modelling purposes. Single-storey shear walls are one of these systems, although several timber shear wall models exist that are MDOF model, (finite element models). The SDOF model is the approach taken to model the MIDPLY™ shear wall system.

The typical SDOF model has the following three essential components: a mass, spring, and a dashpot. Each represents one component of the classic dynamic equilibrium equation:

\[ ma + cv + kx = 0 \]

The mass, \( m \), is multiplied by the acceleration, \( a \), to produce an inertia force. The dashpot represents the damping in the system, denoted by the damping constant, \( c \), which is multiplied by the velocity, \( v \), to produce a damping force. The spring represents the stiffness of the system and is denoted by the stiffness, \( k \), which is multiplied by the displacement, \( x \), to produce the restoring force.

This is a called a “lumped parameter” model, since all the parameters are lumped into three main components. It could also be described as “discretized”. This is the model used for the MIDPLY™ system, in combination with a hysterisis model, which is explained in detail in section 8.1.4 of this chapter.
Other structural dynamic model groupings are linear & non-linear (material), and 2D & 3D. The model used for the MIDPLY™ system is a non-linear, 2D model. The non-linearity is attributed to the material behaviour. Since it is a shear wall that is being modelled under 2D loading (one-directional earthquake), there is no need for a 3D model other than to model the transverse (out-of-plane) buckling, which would be an overly complex task, given the scope of this project.

8.1.3 Natural Frequency

Apart from the main dynamic model used for the MIDPLY™ wall, a few calculations were made to estimate the natural frequency of the MIDPLY™ system as tested on the shake table. This is an important parameter in the dynamic analysis of structures. A SDOF system will have one natural frequency at which the structure will resonate at a higher amplitude. This is called resonance, and it can be very destructive on a structure. Therefore, the natural frequency, \( f_n \), or the natural period, \( T_n \), which is the reciprocal of the frequency, was calculated for the MIDPLY™ wall. \( f_n \) is related to a structure's mass, material, and geometry. Once it is calculated, earthquake records are selected for the shake table tests that are strong in this frequency, or as close to this frequency as possible, so that the wall may be put through its ultimate test.

To calculate \( f_n \), first the equation of motion for the MIDPLY™ system is formulated. For \( f_n \) calculation purposes, it is assumed that damping is negligible (except for hysteretic damping) and the structural model of the wall is presented in Figure 8.1. The coordinate
system is angular, as opposed to rectangular, to coincide with the main dynamic model of the wall, which also uses that system.

![Dynamic Model of MIDPLY™ wall](image)

**Figure 8.1:** Dynamic Model of MIDPLY™ wall used for $f_0$ calculation purposes

As shown, the mass of the shake table frame and the MIDPLY™ wall itself is lumped as $m_1$, and the main mass is lumped as $m$, which is elevated above $m_1$ by a distance $l_2$. The stiffness of the whole wall is lumped as $k$, which is the effective stiffness of the wall taken from the results of the static wall tests. It is assumed the column is rigid, as can be assumed according to previous tests on the shake table, which makes this a SDOF model.
From this model, the general equation of dynamic equilibrium is taken and tailored to suit the model. Taking moments about the pin base, we get these forces to maintain equilibrium when the model is rotated by an angle of \( \theta \):

The moment due to the angular acceleration of mass = \( J \cdot \ddot{\theta} \)

where \( J \) is the polar moment of inertia and \( \theta \) double dot is the angular acceleration. \( J \) can be written as: \( J = m \cdot k_o^2 \)

where \( k_o \) is the radius of gyration, and it is equal to the length of the column, \( l \).

Therefore, the force due to the acceleration of the main mass, \( m \), is: \( -m \cdot (l_1 + l_2)^2 \cdot \ddot{\theta} \)

the negative sign represents the force acting in the opposite direction of the acceleration.

The force due to the acceleration of the secondary mass, \( m_1 \), is: \( -m_1 \cdot l_1^2 \cdot \ddot{\theta} \)

The force due to the stiffness of the wall (restoring force) is: \( -k \cdot (l_1 \sin \theta) \cdot l_1 \cos \theta \)

the negative sign represents the force acting in the opposite direction of the rotation (displacement).

The force due to the acceleration of gravity acting on the main mass is:

\[
m \cdot g \cdot (l_1 + l_2) \sin \theta
\]

and the same for the secondary mass is: \( m_1 \cdot g \cdot l_1 \cdot \sin \theta \)

Thus we have accounted for all the forces acting on this model, and the resulting equation for dynamic equilibrium is:

\[
[-m \cdot (l_1 + l_2)^2 \cdot \ddot{\theta}] + [-m_1 \cdot l_1^2 \cdot \ddot{\theta}] + [k \cdot (l_1 \sin \theta) \cdot l_1 \cos \theta] + [m \cdot g \cdot (l_1 + l_2) \sin \theta] + [m_1 \cdot g \cdot l_1 \cdot \sin \theta] = 0
\]
and if the angle $\theta$ is relatively small, which it is in this case, then the equation simplifies to:

$$[-m (l_1 + l_2)^2 \ddot{\theta}] + [-m_1 l_1^2 \ddot{\theta}] - (k l_1^2 \theta) + [m g (l_1 + l_2)\theta] + [m_1 g l_1 \theta] = 0$$

Isolating the rotation and acceleration variables, $\theta$ and $\theta \ double \ dot$, we get:

$$\ddot{\theta} [-m (l_1 + l_2)^2 - (m_1 l_1^2)] + \theta [m g (l_1 + l_2) + (m_1 g l_1) - (k l_1^2)] + = 0$$

and rearranged, this becomes:

$$\ddot{\theta} + \left[ \frac{[m g (l_1 + l_2) + (m_1 g l_1) - (k l_1^2)]}{[-m (l_1 + l_2)^2 - (m_1 l_1^2)]} \right] \theta = 0$$

therefore the effective mass, $m_{eff}$, of the system is:

$$[-m (l_1 + l_2)^2 - (m_1 l_1^2)]$$

and the effective stiffness, $k_{eff}$, of the system is:

$$[m g (l_1 + l_2) + (m_1 g l_1) - (k l_1^2)]$$

Since the equation for the natural frequency, $f_n$, is:

$$f_n = \sqrt{\frac{k_{eff}}{m_{eff}}}$$

the natural frequency of our model is:

$$f_n = \sqrt{\frac{[m g (l_1 + l_2) + (m_1 g l_1) - (k l_1^2)]}{[-m (l_1 + l_2)^2 - (m_1 l_1^2)]}}$$

The constants, based on wall M46-01 in the racking frame on the UBC shake table, are as follows:

$$m = 4545 \text{ kg} \quad \text{(main mass on the shake table)}$$

$$m_1 = 500 \text{ kg} \quad \text{(weight of the MIDPLY™ wall, and the rest of the racking frame)}$$

$$\delta_1 = 2.545 \text{ m} \quad \text{(length from hinge at the base of the racking frame to top of wall)}$$

$$\delta_2 = 0.455 \text{ m} \quad \text{(length from top of wall to the centroid of the main mass)}$$
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\[ k = 4.1 \text{kN/mm} = 4.1 \times 10^6 \text{kg/s}^2 \] (secant stiffness of wall M46-01, a, b, & c)

\[ g = 9.807 \text{m/s}^2 \] (acceleration due to gravity)

Inputting the constants into the equation, it yields: \textbf{3.89 Hz}

This is the natural frequency, \( f_n \), for the upper bound wall, which is useful to know so the shake table test parameters can be estimated. The natural frequency for the lower bound wall is not as important, since the maximum displacement, acceleration, and mass is needed to be known for the shake table tests, and those parameters will encompass the lower bound wall requirements also. The natural period, \( T_n \), is the inverse of this, which is \textbf{0.26 seconds}. This constitutes the dynamic modelling by hand. The rest of the modelling is done with a computer, as explained in the following sections of this chapter.

\textbf{8.1.4 The Hysterisis Model}

In Chapter 1, the term “hysterisis” was explained as being a term that describes the load-displacement response curve of a reversed-cyclic or dynamic test. The hysterisis curves (or loops) can be modelled and applied to a structural component. Consequently, the hysterisis model “governs” the way that component behaves throughout the entire dynamic excitation. The hysterisis model used for the MIDPLY™ system is based on reversed-cyclic test data compiled at Forintek Canada Corp.

\textit{Figure 8.2} displays the most common hysterisis models developed for timber structures in the last two decades. Incidentally, hysterisis models began not much more than two decades ago, due to the fact that computers are needed to execute a hysterisis model. A
common element in these models is that they are based partly on the backbone, or “skeleton” curve of the load-deformation relationship determined from cyclic tests. How the curve approaches the backbone curve within a cycle sets these models apart.

Figure 8.2: Different Types of Hysterisis Models for Timber Structures.
Out of the models shown in Figure 8.2, the model by Ceccotti and Vignoli was chosen for modelling the MIDPLY™ system. It is commonly known as the “Florence Loop” or “Florence” model. The Florence model was developed by Drs. A. Ceccotti and A. Vignoli at the University of Florence, Italy, in 1992 (Ceccotti, 1992). It has been used in several timber models worldwide, including bolted connections and conventional shear walls tested at Forintek Canada Corp. Among its preferable qualities is the incorporation of the “pinching effect”, explained in Chapter 7, which is common in timber structures. Figure 8.3 shows the Florence hysterisis model in detail.

Figure 8.3: The Florence Hysterisis Model used in Dynamic Analyses

The Florence model has nine parameters: six stiffness parameters (K1 to K6), two displacement parameters (U1 & U2), and one force parameter (F0). U1 is the displacement at which the first yielding occurs, which is the upper value of the first linear
portion of the load-displacement curve. U2 is the ultimate load \(P_{\text{max}}\). Stiffness value K3 is the degradation stiffness, which is defined by \(P_{\text{max}}\) and the load at \(\delta_u\) (the ultimate displacement at 80% \(P_{\text{max}}\)). The force \(F_0\), controls the degree of pinching by setting the force at which the hysteresis loop crosses the point of zero displacement in a cyclic reversal.

This hysteresis model was input into the structural analysis program DRAIN-2DX (Prakash et.al, 1993), which is capable of time-stepping analysis. Any time-history can be input into the program, and the model will follow the hysteresis behaviour of the Florence Model. The structure to which the hysteresis model is applied is shown in the following section of this chapter.

The Florence Loop model is a Tri-Linear hysteresis model because the envelope curve is composed of three straight (linear) lines, or stiffnesses. There are five types of Florence loops. The one shown in Figure 8.3 is the one that was used for the MIDPLY™ analysis for a “large earthquake”, where the wall goes past the point of maximum load and the load decreases with increasing displacement. Another type of model was used for a “small earthquake”, where the wall remains in the region of the load-displacement curve up to the point of maximum load. The latter model type is shown in Figure 8.4. The reason that two models are used is for modelling accuracy; if the large earthquake model is used to predict the response of a wall subjected to relatively small forces, then the resulting output would not be as accurate as if the small earthquake model was used. An even larger error would result if the small model was used for a large excitation, because
the wall would not fail in this case, resulting is huge forces. Together, these two models cover the full range of forces expected.

Figure 8.4: The Florence Model used for Relatively Small Excitations

The Drain-2DX program takes into account damping aside from the hysteretic damping inherent in the Florence model. This is called “Rayleigh damping” and it represents the damping in the structure in the linear-elastic range of the force-deformation curve. It is mass and stiffness dependent, and it accounts for all damping in the structure other than the hysteretic damping, such as friction of joints, wood crushing, and heat dissipation, if any exists. The damping is input into the program as an “equivalent viscous damping coefficient”. It contains two constants, $\alpha$ & $\beta$, which are the mass-related and stiffness-related damping coefficients, respectively, of the Rayleigh damping. Only the stiffness-related coefficient, $\beta$, is used for the MIDPLY™ model.
The Florence models can be used for single as well as multiple storey buildings or as shear wall inserts in regular buildings. This concludes the explanation of the model itself, and the specific application to the MIDPLY™ wall will be explained in the next section.

### 8.1.5 Methodology

The way the model was implemented into DRAIN-2DX is by the simple model of a shear wall shown in *Figure 8.5*. This model was used previously in dynamic analyses of conventional shear walls, and suited the MIDPLY™ walls also. Only the input parameters were different in the MIDPLY™ wall.

![Figure 8.5: Structural Model used in conjunction with the Florence Model in DRAIN-2DX](image)
The beam and column elements of the wall are stiff enough to resist bending deflections, and the reason they are there is to create the geometry of the model, so deflections can be measured at the top of the wall after the model is run. There is an option in this model to include uplift deflections by adding a vertical spring element at each bottom corner of the wall. This has not been included in the MIDPLY™ model. This model could also be modified to represent a multi-storey building, with uplift springs between each floor, but this is beyond the scope of this thesis. It has been done before for conventional shear walls.

This model uses a rotational spring element in each corner of the wall that follows the force-deformation relationship governed by the Florence Model. Therefore, each stiffness parameter needed to be expressed in N*mm/radian instead of kN/mm as usual. The conversion from the stiffness values taken from the cyclic tests (kN/mm) to the stiffness values on the model (N*mm/radian) is shown on Figure 8.6.
Conversion Problem:

\[
\frac{kN}{x \text{ mm}} \Rightarrow \frac{N \text{ mm}}{\theta \text{ radians}}
\]

\[
\frac{kN \cdot h}{x \text{ mm}} = \frac{N \text{ mm}}{h \theta \text{ radians}}
\]

\[
\left(\frac{kN}{mm}\right) \cdot h^2 \cdot 1000 = \frac{N \text{ mm}}{\text{radian}}
\]

Equally divided by 4 springs:

\[
\left(\frac{kN}{mm}\right) \left(\frac{h^2 \cdot 1000}{4}\right) = \frac{N \text{ mm}}{\text{radian}}
\]

height of wall is 2440 mm:

\[
1.488 \times 10^9 \frac{kN}{mm} = \frac{N \text{ mm}}{\text{radian}}
\]

Figure 8.6:  Stiffness unit conversion from Cyclic Tests to Dynamic Model

The nine model parameters, K1 through K6, U1 & U2, and F0, were taken from the load-deflection diagram as shown on Figure 8.7. Each parameter is the average of the two quadrants (compression and tension). Furthermore, the envelope of the first cycle in each cycle group was used to determine the parameters.
Figure 8.7: Dynamic Model Parameters taken from the Cyclic Test results using a graphical method

Once all nine parameters were determined, there was only one more parameter: damping. As mentioned in the previous section, a Rayleigh damping coefficient, $\beta$, is used to account for the damping in the relatively linear-elastic range of the hysterisis loop. The value for $\beta$ is calculated to be:

$$\beta = \frac{T_n \cdot \zeta}{\pi}$$

where $T_n$ is the fundamental (natural) period of vibration of the structure, and $\zeta$ is the damping ratio (or fraction of...
critical damping). The $T_n$ of the MIDPLY™ wall was calculated earlier to be 0.26 seconds. The $\zeta$ for conventional shear walls will be used for the MIDPLY™ walls also, since they are made of the same materials and fall into the same structure type category (light frame timber shear walls). The value that was used for $\zeta$ was 0.01, or 1%. Any value between 1% and 5% would have been acceptable for this type of structure, but when the ratio was varied between these values, no difference was noticed in the output, so it was kept at 1%. Therefore $\beta = 0.0008$.

This covers all the inputs into the DRAIN-2DX program, except for the earthquake acceleration record that is applied to the base of the model.

Any output can be specified in this program, including deflections, accelerations, forces, or reactions. For the MIDPLY™ model, only the top of wall deflection was specified for the output.

8.1.6. Earthquake Records

The earthquake acceleration records used in the shake table tests were to be selected based on how the computer model performed using the records in a non-linear time history analysis. Therefore, several earthquake records were run on this model and the results were weighed to select which ones were to be used on the shake table.
The criteria used to select an earthquake record to test the MIDPLY™ walls were as follows:

1. **Frequency Content**: Most earthquake records, has peaks at certain dominant frequencies, which is visible when a Fourier analysis is performed on it. If one of these dominant frequencies matches the natural frequency of the wall, then the record is a good selection since it has the potential to be destructive to the wall (resonance). Also, some records have a wide band of frequencies, which is good when the natural frequency of the wall is not exactly known.

2. **Other properties of the acceleration record**: A high number of force reversals, more than one set of strong motion, and a the duration of the shaking can make an earthquake very destructive.

3. **Destructiveness of actual earthquake**: Some earthquake records have been notorious for resulting in structural damage, such as the ones that were mentioned in Chapter 1. Some were even destructive to certain types of buildings, also noted in Chapter 1. These records are beneficial since they have been proven as a threat already, therefore if the wall performs well using these records, then it says a lot for the wall.

4. **Comparison with other shear wall tests**: If the record has been used on other shear wall tests, then the results can be compared.

5. **Feasibility**: Some earthquakes result in displacements and accelerations that the shake table cannot handle, which is the reason the computer model is run ahead of time, determining the resulting displacements.
After several records were run on DRAIN-2DX using the Florence Loop model, the following records were the ones that contained the best combination of the aforementioned criteria:

**The Landers record:** The Landers record was recorded in the Landers earthquake of 1992 in California at the Joshua Tree Firehall, in the East-West direction. The magnitude of the earthquake 7.3 on the Richter scale. The peak ground acceleration was 0.29 g at the Joshua Tree station. The record that was used for the dynamic tests was filtered and scaled up to 0.52g and it is shown on Figure 8.8. The reason it was scaled up was because other tests had been performed on regular shear walls that used the same PGA. The record shown has been filtered so the high frequencies are removed and formatted so the shake table can understand it (UFF format). Before it was used on the shake table, a “post-pulse” was added (one sine wave added at the end of the record to return the shake table displacement to the “zero mark”) using a MathCAD spreadsheet created by Dr. C. Ventura at UBC.

This record was chosen for three reasons. First, its dominant frequency is in the range of 4-5 Hz, which matches the MIDPLY™ wall frequency. Secondly, this is a common record for timber shear wall tests, therefore comparisons can be made with other tests. Thirdly, this earthquake record has two sets of strong motion, which can be very destructive to the structure.
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Figure 8.8: The Landers Earthquake record chosen for the shake table tests.

As shown, the total length of the record is approximately 82 seconds, with about 50 seconds of strong motion. The runs on the shake table were scaled up even more, as determined by the maximum horizontal distance that the shake table can travel, which is explained in the next section: Shake Table Testing. Therefore the maximum acceleration was not 0.35g, but 0.54g. When this record was run on the computer model at a peak acceleration of 0.54g using the M46-01 wall properties on the “large quake model”, the resulting peak displacement at the top of the wall was 16.1 mm.
The Kobe record: The Kobe record is actually named the Hyogo-ken Nanbu earthquake (or the Great Hanshin earthquake), but it is commonly known as the Kobe earthquake since its epicentre was on the island of Awaji near Kobe, Japan on January 17, 1995. It caused much destruction in that city, as explained in Chapter 1. The record that was used for the MIDPLY™ dynamic tests was recorded at the Kobe Observatory of the Japan Meteorological Agency (JMA) in the North-South direction. The earthquake measured 6.8 on the Richter Scale and the ground accelerations were as high as 0.8 g in some places. The original record obtained from the JMA had a peak ground acceleration (PGA) of 0.82 g. After filtering and formatting the signal (UFF format), the PGA was 0.69g, as shown on Figure 8.9. The reduction was due to the displacement limits of the shake table.

The reason this record was chosen was because it was devastating in real-life, it has a few strong peaks, which differs from the Landers record, and other tests in Japan have used this record (comparison purposes).
Figure 8.9: The Kobe earthquake record chosen for the shake table tests

As shown, the total length of the record is approximately 52 seconds, with about 20 seconds of strong motion. When this record was run on the computer model at a peak acceleration of 0.8g and the non-linear analysis was performed, the resulting peak displacement at the top of the wall was 14.5 mm.

The next section of this chapter will explain how these earthquake records were implemented in the shake table testing portion of the MIDPLY™ project.
8.2 Shake Table Testing

The most effective way to test the behaviour of the MIDPLY™ wall in response to earthquake motion is to subject actual recorded earthquake ground motion to the test specimens. Other than building a test specimen and waiting for an earthquake to occur, this can only be done using a shake table, which is a electro-hydraulic controlled seismic simulator onto which a specimen is attached. Shake table tests offer more information on the dynamic behaviour of a structure than simple static tests do. However, shake table testing is expensive and shake tables are “few and far between”. Conveniently, UBC’s Earthquake Engineering Laboratory has a shake table that is capable of handling an 8 foot by 8 foot timber shear wall, and therefore shake table testing was included as part of the MIDPLY™ project.

8.2.1 Objectives

The shake table testing of the MIDPLY™ project was performed to determine the behaviour of the MIDPLY™ walls under dynamic loading ie. earthquake motion. The tests were performed after the fourth quarter of the second year of the MIDPLY™ project. It constitutes the last portion of tests that this thesis covers. The specific objectives of the tests were as follows:

1. To compare the dynamic behaviour of the MIDPLY™ walls to the behaviour under monotonic and cyclic loading.
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2. To produce load-displacement hysterisis graphs, displacement time histories, calculate energy dissipation, and determine the failure modes of the MIDPLY™ wall under moderate and extreme earthquake loading.

3. To test the new connections designed for the MIDPLY™ wall (hold downs and anchor bolts).

8.2.2 Test Apparatus

The UBC Earthquake Engineering Laboratory is located in the Civil Engineering building on campus. The shake table inside consists of a 10' by 10' by 2' thick aluminium table at ground level to which specimens can be bolted. It is shaken by hydraulic actuators that can move the table in two horizontal directions, although only one direction was used for the MIDPLY™ tests. The payload capacity of the table is 156kN (35,000 lbs.), and it can move a horizontal distance of ±3” (76mm), with a maximum acceleration of 2.5g and a maximum velocity of 5 inches/sec (127cm/sec). The table is controlled by a signal processing system that sends signals to the hydraulic actuator to produce a simulated earthquake or any signal that is input. It is a closed loop system, which means that, as the table is shaking, the input motion is adjusted in real-time according to the acceleration readings of the table to create a more accurate output.

A frame, built by J.D. Dolan in 1989 for testing shear walls, was attached to the shake table to test the MIDPLY™ walls, as shown in Figure 8.10. This frame carries a 4550kg mass at the top to produce the inertia force on the wall as the table accelerates. The mass
represents the mass of the top two stories of a typical North American three-storey light timber frame building. The shear wall installed in the frame takes up all the lateral resistance of the frame.

Figure 8.10: Shake Table Test Apparatus with MIDPLY™ specimen

The shear wall is attached to the test frame by anchor bolts at the bottom and by the spreader beam at the top. The spreader beam is connected at each end to small horizontal links attached to the test frame. These links allow a small amount of vertical displacement (uplift) of the wall during tests. The spreader beam and the bottom anchoring beam had to be redesigned for the MIDPLY™ wall since it has a different anchor bolt pattern than a regular shear wall (for which the frame was designed). Also,
the forces on the beams were expected to be much higher than a regular shear wall, so that was another reason they had to be redesigned. These anchor beams were designed by myself according to the Canadian steel design code.

Vertical loads were applied to the wall specimen by means of a pulley system attached to a hydraulic actuator, as shown on Figure 8.11. The actuator was kept at a constant pressure during the tests to apply a constant dead load of 27kN on the wall.

Figure 8.11: The vertical load system on the shake table test frame.
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The walls were fitted with 11 sensors to monitor their behaviour during the tests. The location and types of sensors are shown in Figure 8.12. These were recorded at a sampling rate of 0.005 seconds by a data acquisition equipment and sent to a central computer that converted the data into ASCII format so it could be read by conventional computer programs (MathCAD, spreadsheets). The data from the sensors were used to determine the maximum load, displacement and to produce the necessary graphs listed as Point 2 of the objectives (previous section).

Figure 8.12: Sensor Layout on the Shake Table Apparatus

The accelerometers measured the acceleration of the shake table, the mass, and the top of the wall to determine the force on the wall at any given moment during the tests. The LVDTs (Linearly Variable Displacement Transducers) measured the displacement of various parts of the wall, including uplift, as shown on Figure 8.13.
8.2.3 Test Procedure

8.2.3.1 Test Matrix

Three different types of walls were built (two identical specimens each) and were tested under the Kobe and Landers earthquake records. Each test consisted of up to four repetitions of the earthquake record at increasing intensities until the wall failed. The wall configurations chosen to be tested were the upper and lower bound walls and a wall that had exterior sheathing. The test matrix is shown on Table 8.1. The reason these wall configurations were chosen is as follows: the upper and lower bound walls would give information that ran the entire gamut of MIDPLY™ wall configurations. Knowing the maximum and minimum strength values for the MIDPLY™ wall is beneficial. As for the
wall with exterior sheathing, this was considered to be useful since it would be a common practice to sheath the MIDPLY™ walls when used on the exterior, therefore, accurate strength values needed to be known for this configuration.

### Table 8.1: Test Matrix for the Shake Table Tests

<table>
<thead>
<tr>
<th>Wall No.</th>
<th>Stud Spacing</th>
<th>Stud and Plate Size (inch)</th>
<th>Exterior Sheathing</th>
<th>Vertical Loads</th>
<th>Earthquake Record and PGA scaling levels</th>
</tr>
</thead>
<tbody>
<tr>
<td>M48-01</td>
<td>610 mm</td>
<td>2” x 3”</td>
<td>none</td>
<td>none</td>
<td>Kobe: 0.35g, 0.54g, 0.67g</td>
</tr>
<tr>
<td>M48-02</td>
<td>610 mm</td>
<td>2” x 3”</td>
<td>none</td>
<td>none</td>
<td>Landers: 0.35g, 0.52g, 0.54g</td>
</tr>
<tr>
<td>M49-01</td>
<td>406 mm</td>
<td>2” x 4”</td>
<td>⅞” D.Fir</td>
<td>none</td>
<td>Kobe: 0.35g, 0.54g, 0.67g</td>
</tr>
<tr>
<td>M49-02</td>
<td>406 mm</td>
<td>2” x 4”</td>
<td>⅞” D.Fir</td>
<td>none</td>
<td>Landers: 0.35g, 0.52g, 0.54g</td>
</tr>
<tr>
<td>M50-01</td>
<td>406 mm</td>
<td>2” x 4”</td>
<td>none</td>
<td>27kN</td>
<td>Kobe: 0.35g, 0.54g, 0.67g</td>
</tr>
<tr>
<td>M50-02</td>
<td>406 mm</td>
<td>2” x 4”</td>
<td>none</td>
<td>27kN</td>
<td>Landers: 0.35g, 0.52g, 0.54g</td>
</tr>
</tbody>
</table>

**NOTE:** All walls were 2.44m in length, had new hold down connections on bottom end studs, used OSB 7/16” sheathing for interior panels, used new through plate anchor bolt connections, and had MSR 1600f end studs.

All walls were built at Forintek Canada Corp. two weeks before they were tested, then stored inside where the temperature and humidity were controlled, and then shipped to the UBC shake table the day before they were tested.

The scaling factors for the earthquakes were obtained as follows:

1. **Landers:** The Landers earthquake record was scaled to 0.35g PGA as the lowest scaling because that represents an earthquake of medium intensity (Durham, 1998), and this was repeated for the MIDPLY™ tests for comparison purposes. The next scaling of 0.52g PGA was also used by Durham for the 8’ by 8’ oversized sheathing walls, therefore the same scaling was used for comparison...
purposes. The maximum scaling level of 0.54g PGA was the highest that the UBC shake table could handle based on its displacement limitations.

2. Kobe: The Kobe earthquake record was started at 0.35g in order to compare its behaviour with the Landers earthquake at the same PGA. The same reason applies to using the 0.52g level. The 0.67g PGA was the highest that the shake table could handle.

The plan was to apply these earthquake records at these PGA levels until the wall failed, and if the wall did not fail even after the highest level was applied, there was special wide-band record named “Verteq” that was ready as a “backup record”. The Verteq record did not end up being applied, since the decision was made by project leaders during the testing to rely solely on the Kobe and Landers records, repeated at the maximum intensities until the wall failed.

The anchor bolts used in the shake table tests were the new ones described in Chapter 5 – the through plate anchor bolts. These were chosen because they will be the ones used in the field, and they are also the easiest to attach to the shake table test apparatus. The bolts were ¾” (10mm) diameter bolts attached to the top and the bottom plates.

The end studs used in the shake table tests were MSR (machine stress-rated) studs as opposed to visually graded studs, which were used in all previous MIDPLY™ walls. The reason these were used is that they are stronger. The shake table tests were expected to
apply great impacts to the end studs (more so than cyclic tests) and since this is undesirable type of failure, using stronger end studs was an effort to mitigate this failure. No vertical loads were used in the first two types of walls, since omitting vertical loads challenges the performance of the end studs.

8.2.3.2 Shake Table Test Procedure

The procedure of shake table tests is best listed in chronological point form:

1. The test frame was set up on the shake table by the technician at the UBC Earthquake Engineering Laboratory.
2. All walls are shipped to the laboratory and kept there until each one is tested.
3. Two earthquake records (and backup record) are given to the lab technician in UFF format and entered into the signal processing system.
4. Assembly of wall on the apparatus by myself: first attaching the wall with anchor bolts, then adding the hold down connections, then nailing on the buckling studs.
5. Wall and apparatus instrumented with sensors by the lab technician
6. The impact test (explained below) was performed to determine the natural frequency of the system.
7. The earthquake record was run at 10% PGA (0.035g). (explained below)
8. Three video cameras were set up and the earthquake record was run at the first intensity level, then any damage was recorded on damage diagrams. Then subsequent intensity levels were run until the wall failed, each test being videotaped.
9. After the wall failed, the failure mode was determined, then the impact test was performed to determine the change in the natural frequency of the system.

10. The wall was dismantled and the data of the test was downloaded. Impact tests were performed on the test apparatus to determine the natural frequency of the system. The impact tests (also known as hammer tests) consisted of hitting the top of the apparatus once with a special impact hammer in the direction of shaking. The free vibration was then recorded and analyzed using a Fast Fourier Transform (FFT) to obtain the dominant frequencies of the free vibration. In this way, the natural frequency of the system could be determined accurately before and after the test.

The low-level runs of the earthquake records were run to fine tune the accuracy of the shake table in reproducing the earthquake record. The mass of the apparatus affects the response of the shake table motion, therefore the lab technician needed to make some adjustments by running the records between 10% and 20% of the first maximum PGA. This did not affect the structural properties of the test specimens.

This concludes the chapter on the procedure of shake table testing. The next chapter will cover the results and interpretation thereof.
Chapter 9: Dynamic Test Results

9.1 Data Analysis

9.1.1 Shear Force Calculation

To calculate the effective shear force, $V_{\text{eff}}$, on the wall at all times, both the acceleration at the mass level and the acceleration at the top of the wall was used. The following differential equation for dynamic equilibrium was used to solve for the shear force on the MIDPLY walls:

$$m \ddot{x} + c \dot{x} + kx = 0$$

Breaking up the mass and acceleration term into two terms, one representing the mass and acceleration at the top of the wall, $m_w \ddot{x}_w$, and one representing the mass and acceleration at the inertia mass level, $m_m \ddot{x}_m$, we get:

$$(m_w \ddot{x}_w + m_m \ddot{x}_m) + c \dot{x} + kx = 0$$  \hspace{1cm} \text{Eqn. 9.1}$$

The shear force on the wall, $V_{\text{eff}}$, is calculated to be:

$$V_{\text{eff}} = (m_w \ddot{x}_w + m_m \ddot{x}_m) + c \dot{x}$$
For the damping, the same damping ratio was used as in the DRAIN-2DX analysis: 1%, but this was found to have no significant effect on the hysteresis loops, therefore the damping coefficient was omitted, and the equation became:

\[
V_{ef} = m_w x_w + m_m x_m
\]

Eqn. 9.2

Even when the damping ratio was increased to 5% critical damping, there was still no noticeable difference in the maximum shear force.

To determine the acceleration of the wall and the mass, the data needed to be processed from the accelerometers at the two locations shown in Figure 8.12. The accelerometer at the mass level will be referred to as the "mass accelerometer" and the accelerometer at the top of the wall will be referred to as the "wall accelerometer". In all tests, the output signal from the wall accelerometer contained high quantities of frequencies in the range of 30Hz to 60Hz. This is most likely due to the noise that is recorded when the wall undergoes rupture of its elements. This rupture, in either the sheathing or studs, can result in low amplitude, high-frequency stress waves that are recorded by accelerometers. Since the wall accelerometer is located directly at the linkage connecting the top plate of the wall to the shake-table frame, it records the rupture at a relatively higher intensity than the mass accelerometer.

The noise in the wall accelerometer readings had to be filtered out before it was used for creating force-displacement plots, otherwise inaccurate values for shear would have been interpreted from the plots. Therefore, a MathCad spreadsheet, (Popovski, 1999) was used in connection with signal processing software to filter out certain frequencies of all
signals used for creating all plots from the shake-table tests. The MathCad spreadsheet was tailored to modify the shake table test data from the MIDPLY tests (Appendix D). Both accelerometer and displacement transducer signals were filtered using two different MathCad sheets. The frequencies that it filtered out were below 0.2 Hz and above 18 Hz for the acceleration signals, and below 0.001 Hz and above 15 Hz for the displacement signals. The reason that the filters were set to different levels for displacement and acceleration signals is because the relevant frequency content of each signal. It is inherent of any displacement time history to contain low frequencies as its dominating frequency content, and correspondingly, for acceleration time histories to contain relatively high frequencies. This is explained by the fact that displacement is the result of integrating the acceleration twice. However, when the wall accelerometer readings were filtered, they resulted in amplitude reductions in the order of two, and in some cases, three times. This was because of the large proportions of high frequencies in these readings, which were filtered out completely.

Since the contribution of the wall accelerometer portion of the shear force equation (Eqn. 9.2) is relatively small, it was omitted and the resulting difference in shear force was observed. The resulting difference in shear force was insignificant. Therefore, to avoid dealing with inaccurate values resulting from incorrect filtering of the wall accelerometer, the portion of Eqn. 9.2 containing the wall accelerometer values was omitted in the final shear force equation (Eqn. 9.3). The tributary mass, \( m_w \), which was previously multiplied by wall acceleration in the omitted portion of the shear force equation, was then added to the inertial mass, \( m_m \), to form the final shear force equation, Eqn. 9.3. Because the
tributary mass was then multiplied by a greater acceleration value than it would have
been multiplied by in Eqn. 9.2, the tributary mass, \( m_w \), was reduced to 80% of its value in
the final equation to compensate for the difference in the accelerometer height.

\[
V_{eff} = ((m_m + 0.8m_w) \cdot x_m) \cdot g \left( \frac{h_m}{h_w} \right)
\]

Eqn. 9.3

It was found that by using equation 9.3, the difference between its results and those
obtained from Eqn. 9.2 was less than 1% for all tests. Thus, for the sake of simplicity,
Eqn. 9.3 was used for all load-displacement plots. The tributary mass of the test frame
was calculated to be 200kg and the inertia masses were calculated to be 5261kg, with
cement blocks, steel bracing and brackets included. With the height of the top of the
wall being 2.460m and the height of the hinge where the inertia masses rest on being
2.965m (Figure 8.12), the equation for the shear force becomes:

\[
V_{eff} = ((5260kg + 0.8(200kg)) \cdot x_m) \cdot 9.81 \frac{m}{s^2} \left( \frac{2.965m}{2.460m} \right)
\]

which reduces to:

\[
V_{eff} = 5420 \ kg \cdot x_m \cdot 11.82 \ \frac{m}{s^2}
\]

Final Equation for Shear Force

This final equation, in units of Newtons, was used to create all dynamic load-
displacement plots for the MIDPLY™ wall.

9.1.2 Energy Dissipation Calculation

After the load-displacement graphs were plotted, the calculation of energy dissipation
was just a matter of placing the data of the two columns (load and displacement) into a
Chapter 9: Dynamic Test Results

MathCAD spreadsheet and calculating the area created by the hysteresis loops. This is the same way the program worked for the energy dissipation of the static tests. The same program was not used for the dynamic tests because of the differing data formats. The calculation is:

\[
\text{Energy} := \sum_{i=1}^{n-1} \left[ \frac{1}{2} \left( \Delta_{i+1} - \Delta_i \right) \left( F_{i+1} + F_i \right) \right]
\]

This MathCAD program reads a text file containing two columns of data, the displacement and the load respectively, and computes the energy using the trapezoidal method. Nii Allotey, a friend and fellow UBC student wrote this program and it was not modified in any way. The energy is expressed in kN*mm and the results are tabulated in Section 9.2.

9.1.3 Natural Frequency Analysis

To determine the natural frequency of the MIDPLY™ wall from the impact tests, the data from the impact test recordings were filtered, then input into a MathCAD spreadsheet to determine the Fourier spectra of the free vibration results. Also, a program named “FRF” (Frequency Response Function) (Ventura et al., 1995) was used to double-check the
Chapter 9: Dynamic Test Results

results. The original data from the impact recording is shown on Figure 9.1. The MathCAD sheet performed a fast fourier transform on the data to determine the peak frequencies of the free vibration of the system.

Figure 9.1: Recorded Data from the Impact Test for M48-01 (before shake table test)

A sample output of the MathCAD sheet for wall M48-01 is shown in Figure 9.2. Here we see a very dominant frequency of 4.4 Hz ($T_n = 0.23$ sec).

Figure 9.2: MathCAD Fourier Spectrum of Impact Test for M48-01
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The output of the FRF program is shown in *Figure 9.3*. The resulting dominant frequencies are identical.

Curiously, a peak is seen in both plots around 34 Hz. This is not the second mode of vibration, since that is around 9-10 Hz, and it is not the third either, which is around 12-13 Hz. Therefore, it has been deduced that this frequency is representative of natural frequency of the part of the test frame that the accelerometer is attached to.

![Figure 9.3: FRF Fourier Spectrum of Impact Test for M48-01](image)

Therefore, the natural frequency for the M48-01 test setup is 4.4 Hz. The same procedure was used for all dynamic test specimens, and a table of the results is shown in the next section of this chapter.
9.2 Test Results

A tabulated summary of the maximum values of interest attained in all the dynamic tests is shown in Table 9.1. Positive values occurred in the upper right quadrant of the load-displacement plot, while the negative values occurred in the lower left quadrant.

<table>
<thead>
<tr>
<th>Wall No.</th>
<th>Earthquake Record &amp; PGA</th>
<th>$P_{\text{max}}$ (kN) (+ / -)</th>
<th>$\delta_u$ (mm) (+ / -)</th>
<th>Energy Dissipated (kN*mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>M48-01</td>
<td>Kobe 0.35 g</td>
<td>49.5 / -47.1</td>
<td>23 / -21</td>
<td>2459</td>
</tr>
<tr>
<td>M48-01</td>
<td>Kobe 0.52 g</td>
<td>63.7 / -52.2</td>
<td>61 / -37</td>
<td>2225</td>
</tr>
<tr>
<td>M48-02</td>
<td>Landers 0.35 g</td>
<td>29.5 / -31.5</td>
<td>11 / -12</td>
<td>2488</td>
</tr>
<tr>
<td>M48-02</td>
<td>Landers 0.52 g</td>
<td>52.5 / -52.5</td>
<td>44 / -45</td>
<td>8443</td>
</tr>
<tr>
<td>M48-02</td>
<td>Landers 0.54 g</td>
<td>55.6 / -53.7</td>
<td>99 / -58</td>
<td>6643</td>
</tr>
<tr>
<td>M49-01</td>
<td>Kobe 0.35 g</td>
<td>51.7 / -50.0</td>
<td>14 / -17</td>
<td>2139</td>
</tr>
<tr>
<td>M49-01</td>
<td>Kobe 0.52 g</td>
<td>76.4 / -65.4</td>
<td>29 / -24</td>
<td>5951</td>
</tr>
<tr>
<td>M49-01</td>
<td>Kobe 0.67 g</td>
<td>83.0 / -76.1</td>
<td>97 / -63</td>
<td>7500</td>
</tr>
<tr>
<td>M49-02</td>
<td>Landers 0.35 g</td>
<td>35.1 / -46.1</td>
<td>9 / -11</td>
<td>3598</td>
</tr>
<tr>
<td>M49-02</td>
<td>Landers 0.52 g</td>
<td>46.6 / -50.5</td>
<td>13 / -16</td>
<td>6254</td>
</tr>
<tr>
<td>M49-02</td>
<td>Landers 0.54 g</td>
<td>48.3 / -58.1</td>
<td>15 / -19</td>
<td>6426</td>
</tr>
<tr>
<td>M49-02</td>
<td>Kobe 0.67 g</td>
<td>92.7 / -80.5</td>
<td>53 / -77</td>
<td>9918</td>
</tr>
<tr>
<td>M50-01</td>
<td>Kobe 0.35 g</td>
<td>39.5 / -37.8</td>
<td>21 / -18</td>
<td>2231</td>
</tr>
<tr>
<td>M50-01</td>
<td>Kobe 0.52 g</td>
<td>68.1 / -65.1</td>
<td>54 / -56</td>
<td>8889</td>
</tr>
<tr>
<td>M50-01</td>
<td>Kobe 0.67 g (1\textsuperscript{st})</td>
<td>77.8 / -70.3</td>
<td>105 / -99</td>
<td>11016</td>
</tr>
<tr>
<td>M50-01</td>
<td>Kobe 0.67 g (2\textsuperscript{nd})</td>
<td>55.4 / -54.4</td>
<td>146 / -110</td>
<td>7754</td>
</tr>
<tr>
<td>M50-02</td>
<td>Landers 0.35 g</td>
<td>38.6 / -37.8</td>
<td>9 / -13</td>
<td>3665</td>
</tr>
<tr>
<td>M50-02</td>
<td>Landers 0.52 g</td>
<td>45.9 / -44.2</td>
<td>17 / -18</td>
<td>5783</td>
</tr>
<tr>
<td>M50-02</td>
<td>Landers 0.54 g</td>
<td>53.2 / -42.9</td>
<td>21 / -18</td>
<td>5918</td>
</tr>
<tr>
<td>M50-02</td>
<td>Kobe 0.67 g</td>
<td>75.2 / -56.4</td>
<td>105 / -49</td>
<td>7189</td>
</tr>
</tbody>
</table>

In general, the walls dissipated more energy, reached higher peak loads and displacements, and dissipated more energy than those tested statically. The main failure mode was end-stud failure, although one or two walls failed in other manners. A
complete summary will follow in the next chapter, so this remains the extent of the summary. A load-displacement graph for each tested wall will follow a written description of its behaviour in the tests. The load-displacement graphs contain the significant data from each acceleration amplitude level. Also, a “stitched” graph is shown that contains all the amplitude levels of each wall together on one graph.

9.2.1 Hysterisis Plots & Descriptions of Results

M48-01: Wall M48-01, the weakest wall of the three configurations, showed no signs of failure in the first run - Kobe at 0.35g. Some moderately audible cracks were heard, but no visible failure was noted. Sliding of a magnitude of approximately 10mm occurred at the top plate of the wall, relative to the steel anchor plate it was bolted to. On the second run, Kobe at 0.52g, a few cracks was heard, but no visible failure was noted until the wall failed at the end-stud at 15 seconds into the test (at the largest peak of the record). The wall first failed in tension at the left-front end stud, then on the next acceleration reversal, the left-rear end stud failed, and the test was stopped. The failure occurred at the top bolt of the end-stud, and was labeled as a brash tension failure. The maximum load it achieved was 64 kN, which occurred at a displacement of 47 mm.
After the test was over, the bolts on the bottom plate were loose. This was most likely caused by the MIDPLY™ plate sliding in relation to the steel anchor plate, thereby causing wood crushing on the bottom plate.
M48-01 was taken apart after the test, and it was revealed that not one nail failure occurred in the wall. There were only eight chip-out failures along the bottom plate of the wall. Chip-out failures are failures that occur when the OSB or plywood fails right at the edge (it chips out) in the region of the nail. These failures only occur when the nail is close to the edge of the plywood or OSB (within an inch). Along the studs, there was minimal slotting, therefore the nails did not yield much. Most of the energy was dissipated by the end studs and hold downs.

M48-02: Wall M48-02 performed well in the first two runs - Landers at 0.35g and 0.52g, and failed in the last run - Landers at 0.54g. The failure mode was the same as that of M48-01 - rupture of the front left end stud at approximately 15 seconds into the record, then the rupture of the rear left end stud on the following cycle (17 seconds into the record). Some cracks of the OSB were heard on the 0.52g run, which was the OSB rupturing at the corners. After the test, the studs remained uplifted from the bottom plate by about ¼” to ½” (7 to 9mm). The maximum load of the wall was 56 kN, which occurred at a displacement of 31mm.
After wall M48-02 was taken apart, it was revealed that the whole bottom row of nails on the bottom plate were chipped out (chip-out failure). Also, seven nails had failed in shear on the bottom part of the end studs, both on the left and right sides of the wall. The hold downs remained virtually unbent as were the bolts that connected them to the end studs. The anchor bolts were not as loose as those in wall M48-01 probably because they were tightened more on this wall.

Although the two earthquakes resulted in similar ultimate loads and displacements, the Landers earthquake caused the wall to dissipate much more energy. This was due to the

![Hysterisis Loops of Wall M48-02](image)
fact that the Landers record has a longer time period of strong motion and also that it has more cycles at a moderate amplitude, rather than a few spikes at a high amplitude, like the Kobe record has.

**M49-01:** Wall M49-01, the strongest configuration, lasted through two runs of the Kobe earthquake, 0.35g and 0.52g, before failing at the end stud where, incidentally, a knot was found. The failure occurred about 14 seconds into the record.

The bottom plate cracked longitudinally along the backside (opposite side of the exterior sheathing). Also on the backside bottom plate, there was about a 1” uplift of the studs. This remained after the test was completed. The anchor bolts ripped through the bottom plate in a couple of locations. This destruction on the backside bottom plate was due to the fact that the wall was eccentrically loaded due to the exterior sheathing. It was not symmetric like a standard MIDPLY™ wall. Nevertheless, the wall withstood relatively large loads; the peak load attained was 83 kN, which occurred at a displacement of 29 mm.
Upon inspection after the wall was taken apart, there was only one nail shear failure (on the top row of the bottom plate) and there were six chip out failures. The three end studs that did not fail contained small cracks near the hold down area, which was the beginning stage of a complete failure. There was also some OSB rupture, on the left side of the wall right at the bottom corner of the panel.
Figure 9.7: Hysterisis Loops of Wall M49-01

The hold downs and hold down bolts were noticeably bent – the maximum angle of plastic deformation of the vertical hold down bolt was 16 degrees.

The mid-panel and studs ended up sliding over a ¼” in relation to the bottom plate and staying in that position after the test was completed. Therefore, the nails were cycled to a relatively high displacement. When the wall was taken apart, this was revealed – the nails were bent considerably but not broken, as shown on Figure 9.8.
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Figure 9.8: Nail Yielding on Bottom Plate of Wall M49-01

M49-02: Wall M49-02, the second specimen of the strongest configuration, lasted through all the levels of the Landers earthquake, 0.35g, 0.52g, and 0.54g. Then the Kobe earthquake was applied at 0.68g and it failed at the right rear end stud. The failure was an excellent example of a shear plug failure, as shown in Figure 9.9. A shear plug failure is where the wood ruptures along the outline of a series of bolts, creating a “plug”, as shown in the photo. This is the only test in which the right end stud failed - in all the other end stud failures, the left end stud failed.
Figure 9.9: End stud failure of Wall M49-02 – shear plug failure.

After the wall was taken apart, it was noticed that the hold downs and the hold down bolts were considerably bent, even more so than on wall M49-01. An example is shown on Figure 9.10.

Figure 9.10: Bent Hold Down and bolt on Wall M49-02
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Figure 9.11: Hysteresis Loops of Wall M49-02

M49-02 attained a peak load of 93kN at a displacement of 40 mm. In the last graph in Figure 9.11, the difference between the Landers and the Kobe earthquakes is displayed.
The Kobe record put a lot of demand on the wall in a few cycles, whereas the Landers record had the capacity to put a lot of demand on the wall through resonance, which did not occur in this test. The Kobe record proved to be the most demanding in this test.

![Chip-Out Failures on Wall M49-02](image)

**Figure 9.12:** Chip-Out Failures on Wall M49-02

As shown in *Figure 9.12*, after the wall was taken apart, it was revealed that every nail on the bottom row of nails along the bottom plate failed in chip out failure, just as in wall M48-02. Three of the top row of nails on the bottom plate failed in shear. Also, as shown in the figure above, the OSB ruptured on the corner - about a 4"x 2" piece broke off.

As shown on *Figure 9.13*, the bottom plate failed in tension parallel to grain right at the hold down vertical bolt. This was caused by a force reversal which put a large horizontal force on the vertical hold down bolt, since it was the first bolt along the bottom plate.
This horizontal force pushed against the portion of the bottom plate past the first hold down bolt and caused it to fail in pure tension. The cross sectional area of the bottom plate was reduced to begin with due to the hold for the hold down vertical bolt. Since the hold for the bolt was 11/16" (17mm) in diameter, the cross sectional area was reduced from 5.25 in$^2$ (3387mm$^2$) to 2.8 in$^2$ (1835 mm$^2$). The actual limit for tension on an SPF - standard or better 2x4 is 3.5 MPa as per Table 5.3.1B of the CAN/CSA-O86.1-M89 code. Therefore the force on the bottom chord must have been higher than 6422 N when it broke.

![Figure 9.13: Bottom Plate of Wall M49-02](image)

**Figure 9.13:** Bottom Plate of Wall M49-02

**M50-01:** Wall M50-01 was the first of two walls to be tested with vertical loads. Due to this, it exhibited excellent ductility and well-distributed failure of nails. The behaviour was quite different from that of the walls without vertical loads. It survived all levels of the Kobe record; 0.35g, 0.52g, and 0.67g. Then the Kobe record was reapplied and the vertical load apparatus failed. The cable that applied the force to the wall failed in tension, as shown on **Figure 9.14**. Once the vertical load apparatus failed, the wall
deflected past the limit of the main test frame, and the test was stopped. The fact that the wall failed (in excess deflection) right after the apparatus failed gives a good indication that wall would have survived longer had the apparatus not failed.

![Figure 9.14: Broken Cable on Vertical Load Apparatus](image)

This was the first shake table specimen that did not fail at the end stud. Not one hold down bolt was bent either. At the end of the test, OSB rupture was noticed around the corners of the panel at the panel joint stud. Also, the studs remained uplifted a maximum of one inch (25mm) when the test was completed. The maximum load attained was 78kN, which was achieved at a displacement of 79mm. The load-displacement graphs are shown in Figure 9.15.
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After the wall was taken apart, it was revealed that 28 nails failed in shear, and once again, all nails chipped out on the bottom row of the bottom plate. There was OSB rupture caused by chip-out of nails on all corner of both panels. These ruptures were about 4\" (100mm) by 2\" (50mm).

One striking piece of failure evidence was that the nail holes were slotted up to 1.5\" (38mm) on the end studs and the joint stud. Moreover, these slots were only noticed on the left column of nails on the joint stud. A photo of this is shown in Figure 9.16.
Figure 9.15: Hysteresis Loops of Wall M50-01
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Notice on the load-displacement graphs how the last run of Kobe (on 4th graph) caused the wall to jog over about 90mm and then start cycling back and forth from that point (as if its origin had shifted 90mm). This shift occurred about 14 seconds into the test, and can be better seen in Section 9.2.2, which shows the displacement time histories.

Figure 9.16: Slotted nail holes on left column of nails on M50-01 Panel Joint Stud

M50-02: Wall M50-02 behaved quite differently than its counterpart, M50-01. The difference was in the amount and types of nail failures. This could have been attributed to the type of earthquake record. It survived through all three Landers runs, and by judging the amount of failure in the all after it was taken apart, it most likely would have survived the Kobe run at 0.67g PGA if the vertical load cable had not snapped. The wall behaved excellently before the sudden cable failure. The failure mode after the cable snapped was end stud failure, as shown on Figure 9.17.
Figure 9.17: End Stud failure of Wall M50-02 due to vertical load cable failure.

The maximum load that M50-02 achieved was 75kN, which was attained at a displacement of 54mm.
Figure 9.18: Hysteresis Loops of Wall M50-02
Notice that the same shift in origin occurred in this test - this time to a magnitude of about 40mm. This shift is in the same direction as M50-01, which must mean that strong pulse in the Kobe record occurs in this direction (West). This pulse, or reversal, causes the wall to yield and displace in that direction.

Figure 9.19: Step-by-Step Hysterisis Loop of Kobe 0.67g PGA - wall M50-02
In Figure 9.19 we see that the wall yields in the 2nd graph (14 to 16 seconds), then remains cycling back and forth with an horizontal offset for remainder of the test. This was common in most tests. Moreover, the offset was always to the right (West side) except for wall M49-02, which had an offset to the left (East side). Incidentally, all six tests saw the maximum load occur in the right quadrant of the load-displacement graph, but the greatest displacement was not always in that quadrant. This is most likely due to the fact that the strongest pulse in each record was always in the West direction, but the walls did not always displace in that direction as the pulse occurred. In the case of M49-02, the wall was weakened by that strong Westward pulse, but it held its stiffness right at that point until the next reversal (Eastward) when it displaced to its maximum of 77mm.

After the wall was taken apart, it was revealed that not one nail failed in shear, and seven chip-out failures existed along the bottom plate. The rear end stud failure (the side that failed first) was determined to be a knot-combination tension failure, and the front end stud failure was a brash/splinter combination.

### 9.2.2 Displacement Time Histories

The displacement time histories show how the wall moved in relation to time. They are best used along with load-displacement graphs and ground motion time histories for pinpointing at what time the wall the walls failed, and determining the specific pulses in the ground motions that caused the failures. The displacement is measured as the
difference between the bottom and top of wall and bottom of wall displacement, just as in static tests. Selected displacement-time histories will be shown along with the actual recorded time history of the shake table motion (ground motion) for comparison purposes. All displacement records have been filtered to remove frequencies between lower than 0.001 Hz and higher than 15 Hz. All acceleration records have been filtered to remove frequencies lower than 0.1 Hz and higher than 24 Hz.

M48-01:

![Figure 9.20: Time Histories of Wall M48-01](image)

Figure 9.20: Time Histories of Wall M48-01
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From Figure 9.20, we can observe that the wall yielded significantly due to the pulse at 13.2 seconds in the Kobe record at 0.52g PGA, which was the highest pulse. This incidentally corresponds to a crack heard during the test - perhaps the end stud. Then the wall failed completely (end stud failure) due to the pulse at 18 seconds. The wall ended up displaced at 33mm to the West after the test was over.

M48-02: Only the last two runs of M48-02 are shown since no damage of the wall occurred in the first run (0.35g PGA).

Figure 9.21: Time Histories of Wall M48-02
From Figure 9.21, it is observed that the wall yielded somewhat due to the shaking around between 31 seconds and 35 seconds in the Landers record at 0.52g PGA. The wall then ended up displaced at 5 mm in the Westward direction. When the run at 0.54g PGA was executed, the wall was already "soft" so it deflected a lot in the beginning of the run. Throughout the run, it yielded more and more until the pulse at 13 seconds caused it to fail completely.

**M49-01:** Only the displacement time histories are shown from this point on, since the table acceleration time histories have already been shown for both earthquake records.

![Time Histories of Wall M49-01](image)

**Figure 9.22:** Time Histories of Wall M49-01
On the first run, it is obvious that minimal plastic yielding was caused. The first graph is a good example of the non-destructive response of the wall to a moderate scaling of the Kobe record. The second run shows extensive yielding, which begins at about 15 seconds into the record. The wall softened up due to this run and failed quickly on the last run at 0.67g PGA. However, after the wall displaced to almost 100mm, it still had not completely failed, as shown on the third graph. This displays the superior ductility of the MIDPLY™ wall.

M49-02: This wall displaced little in response to the Landers record, but a lot in the Kobe record.

Figure 9.23: Time Histories of Wall M49-02
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Here we see a good example of non-destructive response to the Landers record in the first three graphs. The last graph shows that the wall failed in the opposite direction than the other walls.

M50-01: After 20 seconds into the first run, this wall already seems to have yielded considerably since the displacement is offset about 2mm after that point, as shown on the first graph. The yielding seems to increase gradually as the tests continue, with a 5mm offset in the third graph (Kobe at 0.67g). Complete failure occurs on the second run of the maximum Kobe acceleration, which happens at 15 seconds into the run.

![Time Histories of Wall M50-01](image)

Figure 9.24: Time Histories of Wall M50-01
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M50-02: Here we see that the wall behaved very well (no offset) until the Kobe run, where it fails at about 15 seconds into the run.

![Figure 9.25: Time Histories of Wall M50-02](image)

9.2.3 Accuracy of Ground Motion Replication

This section shows how well the shake table replicated the input file for the ground motion. Certain table acceleration time histories will be shown along with the input file for comparison purposes. The selected records shown in Figure 9.26 are the ones that deviated most from the input file.
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Input Records

Kobe Earthquake Record, scaled to 0.69 g PGA
Landers Earthquake Record, scaled to 0.54 g PGA

Figure 9.26: Selected shake table acceleration time histories with input records

As shown in Figure 9.26, the shake table output varies considerably from the input files.

The maximum error (deviation from input file) is 39%, which occurred during M49-02,
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Kobe 0.69g PGA run. This error was due to the failure of the wall, which imposed additional inertia forces on the table and affected its accuracy.

It is noticed from the third graph in Figure 9.26 (M49-02 Kobe) that the record was reversed - the negative peaks of the input record are the positive peaks in the output record. This explains why wall M49-02 failed on the opposite end stud than all the other walls.

In conclusion, all the time histories have the same peaks and approximate ratios of amplitude as the input file, which indicates that the shake table replicated the earthquake records quite well.

9.2.4 Impact Test Results

The results of the impact tests are tabulated in Table 9.2. The results show the first three natural frequencies, although only the first natural frequency is of interest. The results are shown for two different accelerometer locations – at the inertia mass level (first row) and at the top of the wall level (second row). The unitless amplitudes are tabulated along with each frequency in order to show how strong each peak was in relation to the other (instead of showing graphs of the spectra).
Table 9.2: Natural Frequencies ($f_n$) of Wall Specimens (Impact Test Results)

<table>
<thead>
<tr>
<th>Wall No.</th>
<th>$f_n^1$ (Hz)</th>
<th>Amplitude (unitless)</th>
<th>$f_n^2$ (Hz)</th>
<th>Amplitude (unitless)</th>
<th>$f_n^3$ (Hz)</th>
<th>Amplitude (unitless)</th>
</tr>
</thead>
<tbody>
<tr>
<td>M49-01 - mass</td>
<td>4.397</td>
<td>0.150</td>
<td>n/a</td>
<td>n/a</td>
<td>12.2</td>
<td>0.030</td>
</tr>
<tr>
<td>&quot;&quot; - top of wall</td>
<td>4.397</td>
<td>0.161</td>
<td>9.25</td>
<td>0.060</td>
<td>12.8</td>
<td>0.035</td>
</tr>
<tr>
<td>M49-02 - mass</td>
<td>4.702</td>
<td>0.126</td>
<td>9.20</td>
<td>0.065</td>
<td>12.9</td>
<td>0.038</td>
</tr>
<tr>
<td>&quot;&quot; - top of wall</td>
<td>4.702</td>
<td>0.067</td>
<td>9.20</td>
<td>0.045</td>
<td>12.9</td>
<td>0.020</td>
</tr>
<tr>
<td>M50-01 - mass</td>
<td>4.336</td>
<td>0.138</td>
<td>9.25</td>
<td>0.060</td>
<td>12.9</td>
<td>0.030</td>
</tr>
<tr>
<td>&quot;&quot; - top of wall</td>
<td>4.336</td>
<td>0.077</td>
<td>9.25</td>
<td>0.040</td>
<td>12.9</td>
<td>0.020</td>
</tr>
<tr>
<td>M50-02 a - mass</td>
<td>4.519</td>
<td>0.059</td>
<td>9.20</td>
<td>0.056</td>
<td>13.0</td>
<td>0.020</td>
</tr>
<tr>
<td>&quot;&quot; - top of wall</td>
<td>4.519</td>
<td>0.034</td>
<td>9.20</td>
<td>0.031</td>
<td>n/a</td>
<td>n/a</td>
</tr>
<tr>
<td>M50-02 b - mass</td>
<td>4.213</td>
<td>0.101</td>
<td>9.20</td>
<td>0.040</td>
<td>12.6</td>
<td>0.042</td>
</tr>
<tr>
<td>&quot;&quot; - top of wall</td>
<td>4.213</td>
<td>0.062</td>
<td>9.20</td>
<td>0.028</td>
<td>12.6</td>
<td>0.020</td>
</tr>
<tr>
<td>M50-02 c - mass</td>
<td>4.152</td>
<td>0.072</td>
<td>9.20</td>
<td>0.045</td>
<td>12.6</td>
<td>0.030</td>
</tr>
<tr>
<td>&quot;&quot; - top of wall</td>
<td>4.152</td>
<td>0.052</td>
<td>9.20</td>
<td>0.025</td>
<td>12.6</td>
<td>0.020</td>
</tr>
</tbody>
</table>

Notes: The data for M48-01 and M48-02 are not available due to computer error. M50-02 was tested before and after the shake table tests – the a refers to before, the b refers to after the 0.35g Landers and c refers to after the 0.54g Landers.

The results of the two locations yielded almost identical results, ranging from 4.2 Hz to 4.7 Hz. The natural frequencies did not seem to depend on the stiffness of the wall very much. The first natural frequency was in good agreement with the structural model shown in Chapter 8, Section 8.1.3. The structural model was calculated to have a $f_n$ of 3.9 Hz, and that was for the relatively stiff wall, therefore the model was considered adequately accurate for natural frequency calculation purposes. Figure 9.27 shows myself performing an impact test.

As shown by Table 9.2, the first fundamental frequency of wall M50-02 changed from 4.52 Hz to 4.15 Hz during the tests from Landers at 0.35g PGA to Landers at 0.54g PGA. This means that the wall "softened" considerably over the course of these two tests. The
second and third fundamental frequency hardly changed at all over the course of these two tests.

Figure 9.27: The Impact test being performed by myself
Chapter 10: Summary and Conclusions

First, the results of each object of study in this thesis will be summarized to give an overview of the behaviour of the MIDPLY™ shear wall system and its specific components. Then conclusions will be drawn in Section 10.4 and recommendations will be made for improvements of the system that should be implemented in the third year of the MIDPLY™ research project.

10.1 Monotonic and Cyclic Tests

The result parameters explained in Chapter 3, along with other performance characteristics such as the behaviour of hold downs, lumber types, and loading protocol will be used to summarize the performance of the MIDPLY™ wall system in the 40 static tests that were completed at Forintek Canada Corp.

Failure Mode: Depending on when the wall was tested in the course of the second year (before or after new hold downs, finger-joined, etc.), the MIDPLY™ walls failed in several different manners: transverse buckling, end-stud failure, plate failure, and panel joint stud rip-off. However, one failure mode was more common than the rest and that is end-stud failure. The end stud usually failed near the top of the hold down connection, especially when no vertical loads were applied. This is one area that needs attention.
Nevertheless, failure modes typical of conventional walls hardly occurred on any MIDPLY™ static tests; pull-through failures were prevented altogether and chip-out failures were greatly reduced compared to conventional shear walls.

**Maximum Load and Displacement:** The maximum load and displacement of selected MIDPLY™ walls are summarized in Table 10.1.

<table>
<thead>
<tr>
<th>Wall Configuration and Number</th>
<th>Vertical Loads</th>
<th>No Vertical Loads</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>$P_{\text{max}}$ (kN)</td>
<td>$\delta_u$ (mm)</td>
</tr>
<tr>
<td>406 mm Stud Spacing: M32-01 &amp; M46-01d</td>
<td>88.6</td>
<td>102</td>
</tr>
<tr>
<td>610 mm Stud Spacing: M40-01b &amp; M39-01b</td>
<td>81.0</td>
<td>111</td>
</tr>
<tr>
<td>Finger Joined Wall: M41-01 &amp; M43-01b</td>
<td>71.9</td>
<td>125</td>
</tr>
<tr>
<td>2&quot;x3&quot; Wall: M36-01 &amp; M42-01</td>
<td>83.6</td>
<td>94</td>
</tr>
</tbody>
</table>

**Notes:** - Unless noted otherwise, all walls listed here have other parameters set to the standard, such as nail spacing, 2"x4" studs and plates, and regular sheathing and nail types. Table 3.1 in Chapter 3 can be used to find out more details of each wall. - The displacement values listed are the displacements attained at 80% $P_{\text{max}}$ on the descending portion of the load-displacement envelopes.

From Table 10.1, it is determined that vertical loads reduce the load capacity of the walls by an average of 18%, while having no effect on peak displacement.

All walls listed have a maximum load of at least 60 kN and a peak displacement of at least 94 mm at 80% $P_{\text{max}}$. Not including the 2"x3" walls, the walls reached a peak
displacement of at least 100 mm. To this date, this level of performance is unheard of for conventional timber shear walls.

**Stiffness:** Figures 10.1 through 10.3 display the envelope curves of similar walls tested statically in the second year of the MIDPLY™ project. Using these graphs, the stiffness values and other parameters can be visualized better than by listing them in tables.

Since vertical load is an important factor affecting the response, the following graphs are split up into "vertical load" and "non-vertical load" categories. Furthermore, the tests with vertical loads numbered much more than the tests without vertical load, therefore the graphs with vertical loads are split into "Reversed-Cyclic" and "Monotonic" tests.

![Figure 10.1: Envelope Curves of Reversed-Cyclic MIDPLY™ tests with Vertical Loads](image)

**Figure 10.1:** Envelope Curves of Reversed-Cyclic MIDPLY™ tests with Vertical Loads
The wall numbers are listed by each curve on the graph so specific details of the walls can be looked up in Table 3.1 in Chapter 3 or throughout Chapter 7.

Figure 10.2: Load-Displacement Graph of Monotonic MIDPLY™ tests with Vertical Loads
As Figures 10.1 through 10.3 display, the initial stiffnesses of the walls are almost identical amongst the cyclic envelopes and very similar amongst the monotonic graphs.

The average stiffness values for the static tests are listed in Table 10.2.

**Table 10.2:** Summary of Average Stiffnesses of Static MIDPLY™ tests

<table>
<thead>
<tr>
<th>Test Type</th>
<th>Average Stiffness, K</th>
</tr>
</thead>
<tbody>
<tr>
<td>Cyclic tests with Vertical Loads</td>
<td>5.94 kN/mm</td>
</tr>
<tr>
<td>Monotonic tests with Vertical Loads</td>
<td>4.35 kN/mm</td>
</tr>
<tr>
<td>Tests without Vertical Loads</td>
<td>3.92 kN/mm</td>
</tr>
</tbody>
</table>

*Note:* All static tests listed in Figures 10.1 through 10.3 are included in these values. Walls M32-01a, M41-01a, M42-01a, and M45-01a are not included because they have been pre-loaded.

**Ductility:** In Figures 10.1 through 10.3, it is shown that for tests with vertical loads, all cyclic envelopes peak around 60mm, whereas the monotonic curves peak past 80mm.
Chapter 10: Summary and Conclusions

The tests without vertical loads vary considerably, however. Vertical loads increase ductility by approximately 20% overall.

As a reminder, the ductility ratio is explained in Chapter 3, Section 3.3.2, and it is depicted graphically in Figure 3.5. Average ductility ratios of the static tests are listed in Table 10.3.

Table 10.3: Summary of Average Ductility Ratios of Static MIDPLY™ tests

<table>
<thead>
<tr>
<th>Test Type</th>
<th>Average Ductility Ratio</th>
</tr>
</thead>
<tbody>
<tr>
<td>Cyclic tests with Vertical Loads</td>
<td>15.4</td>
</tr>
<tr>
<td>Monotonic tests with Vertical Loads</td>
<td>15.2</td>
</tr>
<tr>
<td>Tests without Vertical Loads</td>
<td>12.5</td>
</tr>
</tbody>
</table>

Notes:
1. See note under Table 10.2
2. The ductility ratio for MIDPLY™ walls is defined in Chapt. 3, Sect. 3.3.2

Energy Dissipation: The energy dissipation for each MIDPLY™ wall has been calculated although not listed throughout the thesis. Selected values are listed in Table 10.4.

Table 10.4: Summary of Energy Dissipation of Selected MIDPLY™ walls

<table>
<thead>
<tr>
<th>Wall Configuration &amp; Number &amp; Number</th>
<th>Vertical Loads</th>
<th>Loading Protocol</th>
<th>Energy Dissipated (kN*mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>2&quot;x3&quot; Wall: M36-01</td>
<td>Yes</td>
<td>ISO97</td>
<td>68370</td>
</tr>
<tr>
<td>406 mm Stud Spacing: M46-01d</td>
<td>No</td>
<td>ISO98</td>
<td>60116</td>
</tr>
<tr>
<td>610 mm Stud Spacing: M47-02d</td>
<td>No</td>
<td>ISO98</td>
<td>43012</td>
</tr>
</tbody>
</table>

Notes: Table 3.1 in Chapter 3 can be used to find out more details of each wall.
Chapter 10: Summary and Conclusions

Hold Down Performance: The test results in the second quarter indicate that the new hold downs (inverted triangle) performed better than the old hold downs. Specifically, the resulting increase in $P_{\text{max}}$ was 32% and the increase in ductility was over 50%, without an increase in stiffness. These are remarkable improvements, but they are based on the comparison of two walls: M38-01b and M39-01b.

There is room for improvement in the hold down design, due to continuing end stud failures. It should be noted that the photos of the end stud failures in Figure 7.9 are not relevant to this topic since this was a finger-joint related failure. Recommendations for new hold down solutions are given in Section 10.4.

Use of 2"x3" Lumber: Section 7.1 of Chapter 7 (First Quarter Results) explains the viability of using 2"x3". The average peak load of walls with 2"x3" members was, 80% of that attained by walls with 2"x4" members. The conclusion was that 2"x3" members are viable in MIDPLY™ walls, but should not be used for end studs. MIDPLY™ walls with 2"x3" members are good for lower bound walls, since they efficiently utilize a smaller amount of lumber while achieving 80% $P_{\text{max}}$ of 2"x4" walls, with similar ductility and stiffness.

Use of Finger-Joined Lumber: As explained in Section 7.3 of Chapter 7 (Third Quarter Results), the use of finger joined lumber should be limited to the interior studs and plates. It should not be used for end studs, due to the bending capacity of the finger
joints. The performance of finger joined lumber in MIDPLY™ walls is satisfactory for both SPS1 and SPS 3 grades of finger joining.

**Lumber Grade:** The effect on wall performance due to the use different types of lumber was only recognized in the dynamic tests where MSR (Machine Stress Rated) studs were used for the end studs. This will be summarized in the Section 10.2.

**Loading Protocol:** Static test results show that the use of different loading protocols has a significant effect on the performance of the MIDPLY™ walls.

To summarize, the ISO 98 cyclic protocol resulted in a lower $P_{\text{max}}$ than the ISO 97 protocol. No significant difference in energy dissipation was noted, however.

Monotonic loading protocols, as compared to cyclic protocols, resulted in much better performance in two result parameters: $P_{\text{max}}$ and ductility. Monotonic tests resulted in slightly higher $P_{\text{max}}$ and 50% higher ductility than cyclic protocols. All protocols resulted in similar stiffness values.

The Euro protocol proved to be similar to the monotonic protocol, and will most likely not be used again, since no advantage over the monotonic protocol was apparent.
Chapter 10: Summary and Conclusions

10.2 Dynamic vs. Static Results

To summarize, three MIDPLY™ wall configurations (2.44 m in length and height) were tested on the UBC shake table using a mass of 4550 kg. Wall M48, representing the lower bound configuration, showed no sign of damage at 0.35g PGA under both Kobe and Landers earthquakes, but failed at 0.52g of Kobe and 0.54g of Landers earthquakes, respectively. Walls M49 and M50, representing the medium and the strongest walls within the three wall configurations, showed no visual damage at 0.35g PGA, 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.

To compare and contrast the performance of the MIDPLY™ walls under static and dynamic loads, the same result parameters as in Section 10.1 will be used. This is used in conjunction with Figure 10.5, which can be compared to Figures 10.1 through 10.3.

First, load-displacement graphs of all the dynamic tests are shown on the same scale and the same page so they can be compared with each other. These graphs are shown in Figure 10.4.
There was a significant difference between the performance of lower bound and upper bound walls (M48 vs. M49 & M50), as clearly seen on Figures 10.4 and 10.5. As far as the comparison of the earthquake records, The Landers and Kobe earthquake records were similar in their demands on the MIDPLY™ wall. For the lower bound MIDPLY™ walls, M48, the Landers record resulted in higher ductility, whereas the Kobe record resulted in higher peak load. For walls with exterior sheathing, M49, the reverse was true.

A similar comparison can be made between the application of vertical loads and exterior sheathing. It is shown by comparing M49 and M50 that the application of exterior sheathing (M50) increases $P_{\text{max}}$, whereas the application of vertical loads (M50) increases ductility.
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Figure 10.4: Hysteresis Loops of all MIDPLY™ Dynamic Tests
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The walls mostly failed in tension at the top bolt of the end-stud indicating 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 mid-to-high acceleration tests. Due to the application of vertical loads, the walls experienced excellent ductility and well-distributed failure of nails along the bottom plates and panel joint studs.

A graph of all the dynamic test envelopes is shown in Figure 10.5 to compare and contrast against those of the static envelopes. Using the following figure, the result parameters can be directly compared against those of the static tests - Figures 10.1 through 10.3.

Figure 10.5: Envelope curves of all Dynamic Tests of MIDPLY™ walls
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From this graph, it is concluded that the dynamic tests generally resulted in less ductility than the static tests, while attaining similar peak loads. Because of the large difference amongst the dynamic test results, comparisons with static results should be made independently rather than generalizing.

**Maximum Load and Displacement:** The maximum load and displacement of selected MIDPLY™ walls are summarized in *Table 10.5.*

<table>
<thead>
<tr>
<th>Wall Configuration and Number</th>
<th>Kobe Record</th>
<th>Landers Record</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>P&lt;sub&gt;max&lt;/sub&gt; (kN)</td>
<td>δ&lt;sub&gt;u&lt;/sub&gt; (mm)</td>
</tr>
<tr>
<td>Lower Bound Wall: M48-01 &amp; M48-02</td>
<td>63.7</td>
<td>53</td>
</tr>
<tr>
<td>Exterior Sheathing: M49-01 &amp; M49-02</td>
<td>83.0</td>
<td>51</td>
</tr>
<tr>
<td>Upper Bound Wall: M50-01 &amp; M50-02</td>
<td>77.8</td>
<td>127</td>
</tr>
</tbody>
</table>

*Notes:* - Unless noted otherwise, all walls listed here have standard nail spacing, new (inverted triangle) hold downs, and regular sheathing and nail types.
- *Table 8.1* in Chapter 8 can be used to find out more details of each wall.
- The displacement values listed are the displacements attained at 80% P<sub>max</sub> on the descending portion of the load-displacement envelopes.

Comparing the lower bound walls; wall M47-02d with M48-01, the Kobe test achieved almost identical peak loads than the ISO 98 test, but with much less ductility. In general, the dynamic tests resulted in less displacement and similar peak loads than the static tests. Walls M49 and 50 cannot be directly compared since they do not have identical static counterparts.
Stiffness: As Figure 10.5 displays, the initial stiffnesses of the walls are a little more varied than in the static tests. The average stiffness values for the dynamic tests are listed in Table 10.6.

<table>
<thead>
<tr>
<th>Wall Configuration and Number</th>
<th>Stiffness (kN/mm)</th>
<th>Kobe Record</th>
<th>Landers Record</th>
</tr>
</thead>
<tbody>
<tr>
<td>Lower Bound Wall: M48-01 &amp; M48-02</td>
<td>5.3</td>
<td>5.6</td>
<td></td>
</tr>
<tr>
<td>Exterior Sheathing: M49-01 &amp; M49-02</td>
<td>6.1</td>
<td>5.8</td>
<td></td>
</tr>
<tr>
<td>Upper Bound Wall: M50-01 &amp; M50-02</td>
<td>5.1</td>
<td>5.0</td>
<td></td>
</tr>
</tbody>
</table>

Note: The stiffnesses are based on the envelope curves shown in Figure 10.5.

The stiffnesses were generally higher than those of the static tests. This could be due to rate of loading, which causes higher stiffness in general. Surprisingly, the walls with vertical loads, M50, had lower stiffnesses than those without vertical loads, M48. Wall M49 was stiffer due to the application of exterior sheathing.

Ductility: As Figure 10.5 displays, the ductility ratios of the walls are even more varied than the stiffness values. The ductility ratios of the dynamic tests are listed in Table 10.7.
Table 10.7: Summary of Ductility Ratios of all Dynamic MIDPLY™ tests

<table>
<thead>
<tr>
<th>Wall Configuration and Number</th>
<th>Ductility Ratio</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Kobe Record</td>
</tr>
<tr>
<td>Lower Bound Wall: M48-01 &amp; M48-02</td>
<td>12.3</td>
</tr>
<tr>
<td>Exterior Sheathing: M49-01 &amp; M49-02</td>
<td>13.1</td>
</tr>
<tr>
<td>Upper Bound Wall: M50-01 &amp; M50-02</td>
<td>18.1</td>
</tr>
</tbody>
</table>

Note: The ductility ratio for MIDPLY™ walls is defined in Chapt. 3, Sect. 3.3.2

Comparing this table to Table 10.3, we see that the ductility is a little less than that of the static tests. There is no correlation between the earthquake record or vertical loads and ductility.

Energy Dissipation: The total energy dissipated by each wall in the dynamic tests was much less than those of the static tests. The reason for this is that the dynamic tests inflicted a few large acceleration peaks, whereas the cyclic tests increased the amplitude of cycles more gradually, thereby undergoing more cycles and hence more energy dissipation. The energy dissipation values are listed in Table 10.8.

Table 10.8: Summary of Energy Dissipation of all Dynamic MIDPLY™ tests

<table>
<thead>
<tr>
<th>Wall Configuration and Number</th>
<th>Energy Dissipated (kN/mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Kobe Record</td>
</tr>
<tr>
<td>Lower Bound Wall: M48-01 &amp; M48-02</td>
<td>4684</td>
</tr>
<tr>
<td>Exterior Sheathing: M49-01 &amp; M49-02</td>
<td>15590</td>
</tr>
<tr>
<td>Upper Bound Wall: M50-01 &amp; M50-02</td>
<td>29890</td>
</tr>
</tbody>
</table>

Note: In the first two rows, more runs were completed in the Landers tests than in the Kobe tests. Also, walls M49-02 and M50-02 end in the Kobe run.
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To make a direct comparison, M47-02d under the ISO 97 loading protocol dissipated 245% more energy than M48-02 and 918% more energy than M48-01.

Hold Down Performance: The inverted triangle hold down was used for all dynamic tests and it did not perform as well as expected. In the static tests, it performed very well, resulting in higher loads and ductility, but the dynamic tests proved otherwise, resulting in end stud failure in five of the six tests.

However, in each end stud failure, it was not the hold down that failed, but the end stud. This means that the hold down connection imposed too much force, whether axial or bending, on the end stud during earthquake motion. A recommendation for a new type of hold down is shown in Section 8.4.

The poor performance of the end studs and hold downs suggest that they should be the main focus in improving the performance of the MIDPLY™ wall in dynamic loading. An improved hold down design is expected to significantly improve performance.

Use of 2"x3" Lumber: As shown in Table 10.4 (M47-02d) and all the tables in Section 10.2 listing data for M48, the performance of 2"x3" lumber is viable and an excellent means to save lumber. Using 2"x3" lumber does result in slightly lower ultimate strength (P_{\text{max}}), but this configuration is applicable in situations where less lateral capacity is acceptable.

Loading Protocol: The results of the dynamic tests showed that the Kobe and Landers earthquake records differed only in their energy dissipation (Kobe being higher in
general). Comparing these with the cyclic and monotonic protocols, the earthquake records dissipate less energy. The earthquake records were well chosen, since they were able to fail the MIDPLY™ walls given the limitations of the shake table apparatus. It is suspected that other earthquake records would not have been as destructive.

**Lumber Type:** For the end studs, the MSR lumber stood up well against the high forces induced upon them during the dynamic tests. Although the studs eventually failed, the use of MSR lumber for end studs should be kept as a viable alternative for the MIDPLY™ system. With a new hold down design, the alternative can be reconsidered.

**New Anchor Bolts Performance:** The new “bolt-through-plate” anchor bolt design was used on the dynamic tests and it worked very well. There was minimal slipping of the plates in relation to the base and the top spreader beam (in the order of a few millimetres). Also, the failure mode of OSB rupture at the mouse holes was eliminated. This is a significant breakthrough for anchor bolts in the MIDPLY™ system, and should be kept standard for all MIDPLY™ walls in the future.

One measure to note is that the "bolt-through-plate" anchor bolts need to be torqued tightly in order to achieve adequate performance. Otherwise it slips easily in relation to the base. The limit of tightness should be judged by observing the wood crush a few millimetres.
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It is anticipated that the buildability of MIDPLY™ walls in the field using these kind of anchor bolts will not be high due to the high accuracy needed in placing them, but their performance is high.

10.3 Comparisons with Other Shear Wall Designs

The static tests resulted in superior results over any other light frame timber shear wall system on which information is published. The load-displacement graphs speak for themselves. Figure 10.6 shows the difference between a conventional shear wall, a "strong" shear wall, a MIDPLY™ wall. The “strong” shear wall contains 5/8” thick CSP plywood, 4” nail spacing, and blocking. The standard wall contains ½” thick OSB, 6” nail spacing, and no blocking.

Figure 10.6: Load-Displacement Relationship of Various Shear Walls
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Furthermore, reference is made to Table 1.1: "Historical shear wall strengths and stiffnesses from static ramp tests". Comparing the values listed in this table with MIDPLY™ static test results, the MIDPLY™ wall is much higher is maximum load. For example, the highest value in the table – 14.5 kN/m maximum load using oversize sheathing is less than the lowest MIDPLY™ wall value – M38-01b, which attained 22.5 kN/m. Also, the highest deflection (displacement) at maximum load was 34.4 mm, which is less than the minimum value attained by MIDPLY™ walls in monotonic tests, also by M38-01b, at 49 mm.

With regards to data published about other walls on ductility, energy dissipation, and stiffness, conventional walls tested in the seismic program at Forintek Canada Corp. are inferior in compared to the MIDPLY™ wall.

10.4 Conclusions and Recommendations

In general, the second year of the MIDPLY™ project was a success. Both the static and dynamic tests resulted in favourable results; the advantages of the MIDPLY™ system outweighed the disadvantages. The strengths of the MIDPLY™ system are:

- Unparalleled strength and ductility
- High energy dissipation
- Chip-out, nail tear-through, and nail pull-out failure modes prevented
- Less costly per unit resistance
- Can be utilized in both new and existing wood-frame construction
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and its weaknesses are:

- End stud failure mode common
- Difficult to build the wall on-site

However, with an improved hold down design, the MIDPLY™ wall can resist higher lateral loads, increase ductility, and dissipate more energy.

The working lateral loads of the MIDPLY™ system range between 50kN and 90kN, the stiffnesses range from 3.4 kN/mm to 6.1 kN/mm and the ductility ratios range from 8 to 18. These values are for walls with no pre-loading history.

Several standards have been developed in the past year of the project, including nail spacing, stud spacing, stud size, stud and plate grade, and sheathing thickness and type. These standards should be maintained in the MIDPLY™ so the system does not vary too much. A few “champions” should be selected and implemented in the field - perhaps the configurations tested on the shake table.

To conclude on the experimentation with different materials and connection designs: The use of both 2"x3" and finger-joined lumber is a viable alternative in the MIDPLY™ wall - as long as they are not used for end studs. The use of this material for plates and interior studs does not reduce the capacity of the MIDPLY™ wall in any significant way. Furthermore, the new through plate anchor bolts worked very well and should be made.
standard on all future MIDPLY™ walls. MSR end studs should also be made standard, unless new hold downs are designed that do not result in end stud failure.

One recommendation is to use steel rods as hold down connections. These rods could be 5/8” diameter, just like the vertical hold down bolt on the existing hold down. The rods would run from the top to the bottom of the wall on either side, as shown on Figure 10.7.

![Figure 10.7: Recommendation of using Steel Rods as Hold Downs](image)

There are also quality control recommendations for building the MIDPLY™ wall. These are listed in Appendix E.

Positive technical results from the testing of the MIDPLY™ wall system necessitate a plan for the commercialisation of the system. Among the issues surrounding
commercialisation, of primary importance is the identification of an industry partner (or partners). Once these partners are identified, a target market can be chosen that best fits the strengths of the partners. Further decisions to be made involve the degree of premanufacture of the system, and the resulting production and distribution issues. A more detailed commercialisation plan will follow when the industrial partners and the target markets are determined.

Potential uses of the MIDPLY™ Wall System are:

- Pre-Fabricated House Construction
- Seismic and Wind Structural Upgrading
- Narrow shear walls in platform frame construction (ie. walls next to garage or window openings)
- Shear wall insert in Japanese post-and-beam construction between posts and beams

The MIDPLY™ wall system is currently being considered by a large construction company in the design of several multi-storey wood-frame buildings.

Another Successful Partnership between Forintek Canada Corp. and The University of British Columbia


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Appendix A: Wall Damage / Nail Failure Diagrams

One the following two pages, examples of nail failure diagrams are shown so the reader can view how nail failures were documented throughout the course of the MIDPLY™ project.

These are actual copies of the Failure Diagrams for MIDPLY™ wall M50-01.

The first page shows the diagram that was used at the UBC shake table facility to monitor and record data as the tests were executed.

The second page shows the diagram that was used when the wall was taken apart at Forintek's facility at a later date.

In addition to these diagrams, one still camera and three video cameras were set up in different directions that recorded each test.
Shake Table Test Damage Report

Wall Number: MS0-01
Date: May 12, 1999
Earthquake Record: Kobe
Scaling: (0.35, 0.52, 0.69, 0.89)

Notes: The wall has vertical loads of 6250

No bent anchorbolts (to date) must mean that all tests so far wood cracks around bolt hole (for each high

Vertical load cable snapped on last test
2nd 0.69g test.

Visible no OSB rupture or end-stud failure.

Some of the hold down bolts were bent

All anchorbolts studs were uplifted 1" almost all. Were still tight.

Huge 2" tears noslots observed.
Diagram used at Forintek's facility
Appendix B: MIDPLY™ Full Scale Model

A 0.75m x 0.75m full-scale model of the MIDPLY™ Shearwall system, complete with wiring, outlets, insulation, vapour barrier and drywall finish. This model was built by myself and can be inspected at the Forintek Canada Corp. Wood Engineering laboratory on UBC's campus.
Appendix C: MIDPLY™ Wall Construction Procedure

MIDPLY™ Method of Construction of an 8’ by 8’ wall for testing at the Forintek Laboratory:

Materials needed to build an 8’ by 8’ MIDPLY wall:
- 6 - 2x4 end studs (4 end-studs, and two buckling-studs)
- 6 (24” spacing) or 10 (16” spacing) 2x3 or 2x4 studs.
- 4 2x3 or 2x4 plates (8’3” long)
- 4 hold down connections with 5 3/8” bolts with nut and washers each, and 1 5/8” threaded rod with nut and washers each.
- 3 ¼” nails- approx. 250 (24” spacing) or 300 (16” spacing), and approx. 20 small hand nails.
- Hold down connections: 12- ½” bolts with large washers (16” spacing) or other anchor alternative.
- 2- 4’x8’ sheets of OSB or Plywood.

Tools needed to build an 8’ by 8’ MIDPLY wall:
- 1 Jigsaw
- 1 Chopsaw or Mitre saw, or Rotary Saw
- 2 ratchet or open-ended wrenches for bolts.
- 1 air nail gun.
- 1 9”+ clamp, and 1 12”+ clamp
- 1 tape measure, pencil, hammer, nail puller, and square.

Procedure:

Note: Only OSB is used in this outline, but Plywood can be substituted in for OSB if required.

1. Cut pieces of lumber to size:
   - Studs- 91 ¾” (with 2x3 plates)  
   - 89 ¾” (with 2x4 plates)  
   - Plates- 99” long

2. Drill holes in the lumber:
Appendix C: MIDPLY™ Wall Construction Procedure

End Studs - 5- 3/8” Diam. holes for hold downs at 5 1/2”, 7 1/2”... 13 1/2” from bottom, centered on the width of the stud.

Plates - 2- 5/8” Diam holes 6 1/2” from either end of bottom plate. (centered)

Bucking Studs - Drill countersink holes to match heads of bolts through the end stud. 7/8” Diam holes at 5 1/2”, 7 1/2”... 13 1/2” from the end at 1/2” from both sides of stud (see Figure 1).

3. Mark 16” or 24” centerlines on plates:
   Start with the midpoint of the plate (49 1/2” from the ends) and draw a centerline. Measure out 16” or 24” from there both ways. The last stud will always be 1 1/2” from the end of the plate (so there won’t be a perfect 16” or 24” between the end stud and the next stud).

4. Lay down the plates and studs on the flat. Attach the hold down to the end studs and connect it to the bottom plate with a 5/8” diameter threaded rod at least 9” long. For the test apparatus, screw the threaded rod into the steel base plate. Hammer a 2 1/2” hand nail about a quarter inch to a half-inch into the center stud at the top and bottom of the centerline. This is a spacer and guide for the OSB.

5. Square the wall, do this by measuring diagonally across corner-to-corner of the wall.

6. Cut mouse holes into the top and bottom of the OSB with a jigsaw. These holes are usually 3 1/2” wide by 4 1/2” high. They can be any shape as long as the anchor bolts can be accessed through them. The mouse holes are situated directly over the anchor bolt locations, wherever those are on the wall.

7. Place the OSB on top of the plates and studs. Place the sheets tight to the nails nailed in step#4. Make sure the OSB is 3/8” from the top and bottom of the plate edges. (96” sheet is 3/4” shorter than regular wall height.)

8. Clamp the wall from top to bottom at the center of the wall, so the plate edges are 3/8” away from the edges of the OSB. (easy way to check for 3/8” is to stick the end of a 3/8” bolt in the gap).

   Also, make sure that the end studs are pushed out about 1/16” past the edge of the OSB (The OSB should be recessed). This is so the buckling studs will fit on the end studs.

9. Hand-nail the OSB to the plates and studs with a couple nails at each corner of the OSB sheets as shown in Figure 1. These nails should be short, small nails (short...
Appendix C: MIDPLY™ Wall Construction Procedure

gypsum nails do nicely); they are just there to hold the OSB sheet in the correct position (preventing it from sliding) until the wall is actually nailed. These nails must be driven flush with the sheet or further. Make sure that the end-studs stick out past the OSB by about 1/16”.

10. Lay down the second set of plates and studs, just as the first set was laid down. These plates and studs should fit well on top of the OSB.

11. Put in the anchor bolts and attach hold-down connections. The top anchor bolts must be put in before the plates are nailed (for our test apparatus) The end of the top plate at the actuator position should line up perfectly.

12. After everything in lined up exactly, draw points on the plates and studs where the nails will be nailed.
   2 nails at every 4” at the panel-joint studs, and 2 at every 8” everywhere else. Remember the middle row of nails at panel joint studs. Then clamp the wall, starting with the middle stud, and nail the stud with the nail gun. Repeat the clamp at each stud location, before nailing it. 3 1/4 nails should be used for the wall.

13. Put on the buckling stud with nails into the end-studs. Lift wall up in order to do this. The wall is lifted by putting a lifter bar on the two middle top anchor bolts.

14. Check for missed nails by flipping the wall onto the opposite side. Re-nail any missed nails.

15. Wall is ready for testing or installation.

16. Put wall on the test apparatus by first lining up the holes on bottom steel plate with those on the I-beam.
Appendix D: Shake Table Data Filtering - MathCAD

(Filtering of MIDPLY™ Shake Table Recorded Acceleration Signals)

Time Step (sec):
\[
\Delta := 0.005
\]
Duration of record (sec):
\[
T_d := 49.25
\]
\[
N := \frac{T_d}{\Delta}
\]= 9.8510^3
\]
i := 0, N - 1, j := 0
\[
\text{UNF}_{i,j} := \text{READ}('m4801v005_1.txt')
\]
\[
t := i \cdot \Delta \quad k := 0 \quad d := \text{UNF}_{<0>}
\]

[Graph of acceleration signal]

\[
F_d := \text{mag cfft}(d)
\]
\[
m := 0 \cdot \frac{N}{2} \quad \frac{1}{N \cdot \Delta} \cdot m
\]

[Graph of frequency spectrum]

Number of points used for averages
\[
\text{Navr} := 50
\]
Appendix D: Shake Table Data Filtering - MathCAD

\[
UF_{avr, j} := \frac{\sum_{i=0}^{Navr-1} UNF_{i,j}}{Navr} \quad UF_{avr} = 0.03
\]

\[
UF_{i,j} := UNF_{i,j} - UF_{avr, j}
\]

Windowing the signal:

\[
Win := \text{taprec}(N)
\]

\[
UF_{i,j} := UF_{i,j} \cdot Win_{i,j}
\]

\[e := UF < 0\]

Plot of Windowed vs. unwindowed signal

Filtering the input record to remove any of the high and low frequencies:

1. Setup the filter parameters:
   \[
   \text{Default set for filtering above 24 and below 0.1 Hz}
   \]

\[
\text{Cutoff} := 0.12 \quad \text{Cutfreq} := \frac{1}{\Delta} \quad \text{Cutfreq} = 24
\]

\[
\text{Cutoff1} := 0.0005 \quad \text{Cutfreq1} := \frac{1}{\Delta} \quad \text{Cutfreq1} = 0.1
\]

\[
L := \text{iirlow(butte(4), Cutoff)} \quad xf := 0, 0.001.5 \quad M := \text{iirhigh(butte(2), Cutoff1)}
\]

Static and Dynamic Testing of the MIDPLY™ Shear Wall System
**Appendix D: Shake Table Data Filtering - MathCAD**

\[ \text{FHL (data)} := \begin{align*} 
\text{col} & \rightarrow \text{data}^j \\
\text{highpass response (col, M, N)} \\
A & \leftarrow \text{response (highpass L, N)} \\
F^j & \leftarrow A 
\end{align*} \]

\[ \text{FH (data)} := \begin{align*} 
\text{col} & \rightarrow \text{data}^j \\
A & \leftarrow \text{response (col, M, N)} \\
F^j & \leftarrow A 
\end{align*} \]

UF (unfiltered), FH (Filtered high pass), FHL (Filtered low and high pass)

\[ \text{UFil} := \text{FHL (UF)} \quad \text{v1} := \text{UNF} \quad \text{v2} := \text{UFil} \]

\[ \text{fv1} := \text{mag cfft (v1)} \quad \text{fv2} := \text{mag cfft (v2)} \quad k := 0 \ldots \frac{N - 2}{2} \]

\[ \text{max (UFil)} = \quad \text{mir (UFil)} = \quad i := 0 \ldots N - 2 \quad C := 0 \]

\[ \text{max (UNF)} = 0.19 \quad \text{mir (UNF)} = -0.133 \quad f_k := \frac{k}{(N - 2) \Delta} \]

---

*Static and Dynamic Testing of the MIDPLY™ Shear Wall System*
Appendix D: Shake Table Data Filtering - MathCAD

1. Output the "UFil" file, which is the filtered data of the original file

   WRITEPRN ("filtered.prn") := UFil

2. Output the "ffourier" and "unffourier" files, which are the fourier transforms of the filtered file and of the original (unfiltered) file, respectively.
   
   Note: this has been changed to only the filtered file. The FFT of the unfiltered file was deemed unimportant.

   WRITEPRN ("ffourier.prn") := fv2
Appendix E: Recommendations for Quality Control

In order to assure a proper performance of the MIDPLY™ Wall System in real-life applications, quality control guidelines on the manufacturing of MIDPLY™ wall should be developed.

There are two options for MIDPLY™ wall construction. Prefabrication is one. Regarding quality assurance, it offers all the advantages of mass production and supervised fabrication.

The second option is manufacturing on the site. The MIDPLY™ Wall System uses regular lumber and tools used in the construction of standard shear walls. Quality assurance guidelines for the construction, manufacture and installation of the MIDPLY™ walls are approximately the same as for conventional stud walls. Based on the laboratory experience, we offer the following extra cautions:

- the holes for the inverted-triangle hold-downs should be drilled in the centre of the 38mm face of the end stud to assure a maximum edge distance on both sides of the bolts.
Appendix E: Recommendations for Quality Control

- 2400f MSR lumber must be used for the end studs in the case of inverted-triangle hold-downs due to high tension forces in the studs. Stud-grade lumber can be used when steel rods are used as hold-downs.

- because the holes for the inverted-triangle hold-downs and anchor bolts are drilled through 38mm (1.5 in.) narrow face of the stud, they have to be drilled at the centre of the plates and end studs to assure a maximum edge distance on both sides of the bolts. Also, straight pieces should be used for the studs, and large knots around holes should be avoided to ensure a good drilling.

- the inverted triangle hold-downs should be attached on the end studs before the framing members are nailed to the sheathing in order to increase the assembling speed.

it is advisable to mark the framing members at the specified nail spacing before nailing the wall components to assure the uniform nail spacing.