A STUDY OF WOOD BASED SHEAR WALLS SHEATHED WITH
OVERSIZE ORIENTED STRAND BOARD PANELS

by

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ABSTRACT

A new wood based shear wall system built with nonstandard large dimension oriented strand board panels has been developed. The project described in this thesis consists of studies to 1) experimentally investigate and quantify the structural performance of the new shear wall system under monotonic and cyclic lateral loading conditions, considering different types and spacings of panel-frame nail connectors, and 2) analytically predict and model the performance of the new shear wall system.

A test facility has been built for full scale shear walls of up to 2.4 x 7.3 m in dimension where both lateral and vertical loads can be applied simultaneously onto the wall assembly. The experimental program has been divided into a static and a cyclic test phase. Thirteen shear wall systems with standard 1.2 x 2.4 m and with oversize panels have been tested to investigate the influence of 1) panel size, 2) panel-frame nail connector type, 3) panel-frame nail connector spacing, and 4) cyclic loading protocol on the overall shear wall behaviour.

A database on the structural performance of wood based shear walls sheathed with oversize oriented strand board panels has been generated. The database includes information on strength, stiffness and ductility properties, energy dissipation and failure modes of shear walls under different testing conditions. A substantial increase in both stiffness and lateral load carrying capacity was achieved by shear walls built with oversize panels as compared to standard panels. A further reduction in nail spacing around the perimeter of the full size panel increased the lateral load resistance to more than double that of regular walls. The failure modes in the shear walls were shown to be substantially different under monotonic and currently used cyclic test conditions. The former was mainly nail withdrawal while the latter was dominated by low cycle nail fatigue failures. A new cyclic loading protocol was proposed and tested, which resulted in failure modes similar to those observed in dynamic earthquake tests. In examining the dissipated energy as the area under the
load deformation curve, it seems that the walls built with the standard panels can dissipate more energy under cyclic loading, compared to the walls built with oversize panels. Furthermore, a group of tests has been conducted to investigate the load-slip characteristics of panel-frame connectors.

A nonlinear finite element analysis program has been adopted to predict and model the performance of shear walls built with oversize panels. The program results had good agreement with the initial part of the nonlinear behaviour of the shear walls in the tests, which is before the panel-frame connectors failed. It could not, however, follow the test results when the connectors failed and further refinements to the program have been suggested to incorporate nail failure occurrence.
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1. INTRODUCTION

Wood based shear wall systems are an efficient means of enclosing residential and commercial space. They provide the building with enclosure and with sufficient strength and stiffness to resist vertical loads, transverse wind loads and in-plane lateral forces imposed by wind and seismic loadings. These systems typically consist of three major components: lumber frame members, sheathing panels and nail connectors. Wood based shear wall systems have an excellent history of performance against lateral loading especially in residential applications. A good example is the recent earthquake in Kobe, Japan in January 1995; wooden buildings constructed using the North American Frame Construction Method equipped with shear walls resisted the strong earthquake accelerations even on sites of serious liquefaction (Yasumura 1996). Even though they have many advantages, the study and development of wood based shear walls have never stopped. With the increasing demand for high level construction standards, more and more new materials and configurations have been applied in shear wall systems. Furthermore, changes in North American demographics and industry restructuring towards value-added products have caused the Canadian wood product industries’ expansion from traditional residential construction markets into the commercial construction market while trying to maintain and protect the current competitive advantage in residential construction.

In the past, much experimental work was done on the structural behaviour of wood based shear wall systems. Full size shear walls were tested to quantify the ultimate load carrying capacity of the system under static loads (Tissel and Elliott 1977; COFI 1979; TECO 1980, 1981; Atherton 1982, 1983; Adams 1987). Full scale studies of buildings under lateral loads were also conducted (Gupta and Kuo 1987; Sugiyama et al. 1988; Boughton 1988; Stewart et al. 1988; Kasal and Leichti 1992). To understand the performance of shear walls when subjected to earthquakes, studies were conducted to evaluate the performance of full size shear walls under reversed cyclic (Foliente and
Zacher 1994; Rose 1995; Karacabeyli 1995), pseudo dynamic (Kamiya 1988), and dynamic (Stewart 1987; Dolan 1989) load conditions. It is commonly accepted that the load-deflection behaviour and energy absorbing characteristics of shear walls are decided mainly by the joint characteristics. Therefore, studies of connector behaviour under both static and cyclic loading conditions related to shear walls have also been investigated extensively (Wilkinson 1972; Foschi 1974; Hunt and Bryant 1990; Karacabeyli 1996).

Many models have been proposed to analyze and predict the performance of wood based shear walls subjected to lateral loads (Polensek 1976; Foschi 1977; Tuomi and McCutcheon 1978; Easley et al. 1982; Itani and Cheung 1984; Gupta and Kuo 1985; Dolan and Foschi 1991; Foliente 1995). Various approaches were adopted in these models, ranging from linear to nonlinear analyses and from variational principles, differential equations to finite element method. A three-dimensional finite element model was developed recently to investigate the response of complete light-frame wood structures (Kasal et al. 1994). These studies formed a solid foundation of knowledge contributing to the understanding of the structural performance of wood based shear wall systems and their components. From these studies, it is clear that the critical characteristics in the shear wall performance evaluated are strength, stiffness, ductility and energy dissipation capabilities.

For many years, plywood has been a predominant sheathing material in shear wall construction. Since the 1970’s, oriented strand board has been introduced as a new sheathing material. Many scientists studied the performance of oriented strand board in shear wall systems and made a comparison to other sheathing materials (Rose 1995; Karacabeyli 1995; White and Dolan 1995). For the first time in Canada, the overall mechanical properties of oriented strand board were studied (Lau and Lam 1989).

Up till now, previous research efforts have focused on walls built with either plywood or OSB panels of standard 1.2 x 2.4 m dimensions as sheathing materials. The discontinuity between panels, as dictated by standard panel dimensions, has a significant impact on the performance and
load carrying capacity of the shear walls, even if some construction compensations have been adopted. The compensations include the use of blocking elements to provide more continuity between panels in cases where the sheathing panels are oriented horizontally and the reduction of nail spacing along the seams to provide higher shear wall stiffness.

In Canada some oriented strand boards mills have the capability to produce panels of up to $3.3 \times 7.3$ m in size. Currently these large panels are cut into standard sizes for conventional building practice.

Preliminary finite element analyses have shown that a significant increase in the load carrying capacity of $2.4 \times 7.3$ m shear walls modelled with oversize panels compared to standard panels can be achieved. Experimental work was needed, however, to quantify potential structural advantages of using larger dimension panels especially for commercial construction where building geometry such as height, length, and width typically exceed those in residential construction. Here, larger lateral forces will be introduced into the building systems and the use of prefabricated shear wall systems using the large nonstandard panels would be especially well suited. Furthermore, with a comprehensive database and verified model, optimization of the wall assembly would be possible through reduction of panel thickness, redistribution of the nailing pattern, and shortening of the required length of wall, while achieving the same lateral capacity as walls built with regular size panels.

The overall objective of the study was to investigate the structural performance of wood based shear wall systems built with nonstandard large dimension oriented strand board panels. Sub-objectives of the study were:

1) to build a test facility for full scale shear walls of up to $2.4 \times 7.3$ m in dimension where both lateral and vertical loads can be applied simultaneously onto the wall assembly;
2) to experimentally investigate the structural performance of shear wall systems built with oversize panel under monotonic and cyclic lateral loading conditions, compared to those built with regular panels;

3) to experimentally investigate the influence of the types and spacings of panel-frame connectors on the shear wall behavior;

4) to experimentally investigate the shear wall performance under various cyclic loading protocols;

5) to develop a database on the load-slip characteristics of panel-frame connectors, as required by analytical programs; and

6) to apply the nonlinear finite element analysis program to predict and model the behavior of shear walls built with oversize panels.
2. BACKGROUND

2.1. Structures and roles of wood based shear walls

Modern shear walls are typically constructed from three basic components: frame members (plates and studs), sheathing (plywood, oriented strand board, particleboard, or gypsum board), and fasteners (nails or staples). Conventional walls have frame members sized 38 mm × 89 mm or 38 mm × 140 mm and studs spaced 410 mm or 610 mm apart. Stud lengths generally provide a 2.4 m finished ceiling height. Usually the perimeter of a frame is of the same size as the studs, with the same or higher grade and sometimes doubled or tripled up to carry concentrated loads from upper storeys. In some situations, diagonal braces are added to the frame to increase the in-plane stiffness of the wall, although the stiffness of the frame itself, compared to the sheathing, plays only a secondary role. Exterior sheathing usually consists of nailed 1.2 × 2.4 m panel material such as plywood or oriented strand board, while the main interior sheathing material is gypsum board, or drywall.

The main role of shear wall in a building is to resist lateral loads caused by wind and ground movement during an earthquake. These lateral loads can be either the forces transferred from horizontal roof and floor diaphragms to the shear wall elements, or forces directly applied to shear walls themselves. In addition, shear walls are required to support vertical loads from the weight of the structure (dead load), snow and rain weight (live load) and vertical earthquake movement. Shear walls must also be anchored down to resist uplift forces caused by wind and earthquakes. Therefore, shear walls must have sufficient strength and rigidity in their plane to carry these loads.

From a structural point of view, a wood frame wall is a very efficient and forgiving system. Because of the large number of closely spaced members and fasteners, a highly redundant system is created to carry transverse, vertical and in-plane lateral loads. For in-plane loading the shear stiffness and strength of the sheathing elements play a key role in resisting lateral loads. Without the
sheathing, the frame members have very low capacity to resist lateral loads. The structural sheathing also enhances load sharing among adjacent elements. The most important component to affect shear wall performance, however, in terms of both stiffness and strength, is the performance of the fasteners connecting the sheathing to the frame (frame-panel connection) and, to a lesser extent, the connection between framing members. For all the connections, nails are the most common fasteners. As a direct result of their importance regarding shear wall behaviour, changes in fastening and sheathing are the major structural concerns in the study of wood shear walls. An important issue is the fact that the load-slip characteristics of the fasteners are highly nonlinear. Consequently, shear deformations of shear walls do not vary linearly with loads.

Shear walls also need to withstand loads, such as wind pressure, in the direction perpendicular to the plane of shear walls (out-of-plane loading). Shear walls subjected to this kind of load are usually designed as beam-columns. Although this mode of loading is beyond the scope of this thesis, these loads can decrease critical load carrying capacity significantly and cause wall buckling.

2.2. Historical review

Research on wood based shear walls dates back to 1927 (Peterson 1983). Most of the research before the end of 1930 (Bolin 1937) was related to earthquake damage analysis of wood construction, which can be considered to be the first period that basically dealt with urgent problems. In the next decade, people paid more attention to the investigation of the structural properties of wood walls, floors and roofs. In this period, researchers were not satisfied with solving scattered problems but were interested in developing a basic understanding of the mechanics of wood products and to provide a standard for design and manufacture. Until the 1940’s conventional light-frame structures used diagonal bracing for shear resistance. Even after panel-type sheathing became popular, diagonal bracing remained the standard. In 1949, a guideline was issued for the acceptance
of panel sheathing in lieu of diagonal bracing, which formed the basis of subsequent standards for construction.

During the 1950's and 1960's, investigators took advantage of previous information and developed it to enrich the knowledge in this field. The majority of this work was experimental in nature which focused on the relative influence of parameters such as sheathing type and orientation, fastener type and spacing, and diaphragm geometry. The analyses were simplified because of a lack of understanding of the static and dynamic behaviour of the walls. Many full-size shear walls or even full scale houses were tested. Up to that time, the research work and design guidelines for wood shear walls were mostly based on experience and testing results.

The rapid development of computers since their inception in the early 1950's provided the possibility to use advanced analysis methods to study wood shear walls. Since the mid-1960's, the finite element method had been introduced in this area. In 1967, Amana and Booth published the results of theoretical studies on nailed and glued plywood stressed skin components. The concept of nail modulus, used to account for fastener stiffness, was first presented in Amana and Booths' paper (Falk and Itani 1989). In 1972, a group of researchers at Oregon State University, in cooperation with the lumber industry, used the finite element method to predict deflections and stresses as well as the ultimate load capacity of walls (Polensek 1976). During this period, many formulae and models for wood shear walls were developed. Some researchers began to study the nonlinear behaviour of shear wall connections and to consider the wood shear wall members as orthotropic materials (Foschi 1977). The results from these theoretical analyses were experimentally verified.

In the last twenty years, significant progress was made in the study of shear wall behaviour. Researchers made great efforts to develop new analytical techniques to predict shear wall performance. Advanced principles of mechanics were applied to overcome the drawbacks in earlier studies which used simple beam theory. Nowadays, numerical analysis methods such as the finite element method combined with a variational approach are employed extensively. More recent
studies tend to gradually shift from linear to nonlinear modelling; from static one-directional loading to dynamic loading; from plane deformation to 3-D deformation and from structures without openings to those with openings.

2.3. Structural analysis models for walls under monotonic loading conditions

2.3.1. Tuomi and McCutcheon's model

In 1978, Tuomi and McCutcheon modelled a single panel wall with an assumed parallelogram distortion shape, using the principle of energy conservation. They proposed that when a wall is subjected to a lateral load, the nail connectors are deformed and the stud frame distorts as a parallelogram while the sheathing remains rectangular (Figure 2.1). The external load is resisted by the nails which absorb all the energy. The load-deflection relationship for a single nail was assumed to be linear. A single sheet of sheathing was fastened around the perimeter of the sheet to the frame. Two equal horizontal forces act on the top and bottom of two opposite corners which form a moment, while its center of rotation is located at the center of panel. The authors believed that the lateral strength of fasteners was the most important parameter needed for predicting racking strength; therefore, they focused on the behaviour of the nails in this study. With materials such as plywood having a high lateral nail resistance, they found that joint failures may occur in the lumber rather than in the sheathing. This model assumed that the individual nail distorts linearly following a certain load-distortion ratio, so the total energy absorbed by the nails can be calculated by summing up the energy absorbed by each individual nail. They believed that the interior nails followed the same distortion pattern of the perimeter nails; therefore, the geometric distribution rather than mechanical performance of interior nails would affect the racking strength of the wall. Several assumptions in this model were contradicted by experimental results, for example, the assumption that the racking strength of a panel...
per unit length is almost constant, and the assumption that the four corner nails distort along the lines of the sheathing’s diagonals.

2.3.2. Easley et al.’s model

Another shear wall model was presented by Easley et al. in 1982. It was assumed that not only the frame but also the sheathing deform as parallelograms. Both vertical edges of the frame rotate equally and the horizontal edge remains horizontal during the deformation (Figure 2.2). The wall is simply supported and the fastener forces in the panel consist of both horizontal and vertical components. Having defined the relative displacement, the nail forces can be calculated by means of simple force or moment equilibrium. It was also assumed that the vertical nail force component is proportional to the distance from the vertical center line of the panel. The proportionality would be true for the vertical slip. It would also be true for the vertical nail force component when the force-slip relationship is linear. But the proportionality of both the vertical force and slip is inconsistent when the force-slip relationship is nonlinear.

Figure 2.2 Easley et al.’s model and assumed pattern of sheathing fastener forces
2.3.3. Gupta and Kuo's model

Gupta and Kuo (1985) used a strain energy method to model a shear wall specimen similar to that analyzed by Easley et al. In this model, for a wall under a horizontal force, the frame would deform into a parallelogram, with a shear angle $\gamma$ relative to its former vertical position (Figure 2.3).

The sheathing resists this deformation and as a result the horizontal and vertical edges have angles $\alpha$ and $\beta$, respectively, relative to the relevant frame members. Compared with this model, Easley’s model assumes $\beta=0$ and Tuomi-McCutcheon’s model constrains $\alpha$ and $\beta$ such that the relative displacement of corners is along the diagonal. Here, both $\alpha$ and $\beta$ are non-zero and independent.

The nail forces are generated due to relative displacements between the frame and the sheathing. It was assumed that both side studs will be deformed sinusoidally with an amplitude $w$, while the top plate was assumed to remain straight. Considering shear strain energy in the sheathing, bending strain energy for both studs, and the nail strain energy, the general force-deflection relationships could be derived. Finally, this study concluded that the behaviour of wood based shear walls is primarily governed by the nail force-slip characteristics. The bending stiffness of studs and the shear stiffness of the sheathing play a secondary role in defining the load-deformation properties.
2.3.4. Foschi’s model

Foschi was one of the pioneer researchers who used the finite element method in his shear wall study. In 1977, he presented a nonlinear finite element model for shear walls. A shear wall assembly was divided into four basic structural elements: panel, frame, frame-frame connection and panel-frame connection. The sheathing material was considered as linear elastic orthotropic plane stress elements and frame members as linear elastic beam elements. The frame-frame connectors and panel-frame connectors were idealized as 3-degree-of-freedom and 2-degree-of-freedom spring elements with nonlinear load-slip relationship (Figure 2.4):

\[ P = \left( m_0 + m_1 \Delta \right) \left( 1 - \exp\left( -k \Delta / m_0 \right) \right) \]  

(2.1)

This model reproduced the nonlinear behaviour observed in the tests. It was found that almost all of the observed non-linearity is due to the nonlinear behaviour of the nailing between the panel and the framing. The effect of variability in the connectors’ properties on the overall diaphragm deflections was relatively minor when compared to the effect of variability on the stiffness properties of the sheathing. In this model, the load-slip relationship of connectors was usually considered as ideal elasto-plastic (\( \tan^{-1} m_1 = 0 \)), while the failure part of connectors (softening) was not taken into account at that time.

2.4. The cyclic testing protocols used in shear wall studies

To understand and improve the performance of timber structures under earthquake loading and to validate and improve seismic design methods, many investigations were conducted using a number of different standards developed for quasi-static cyclic testing of timber joints with
mechanical fasteners and structures. Several variations of these test procedures have been developed by some researchers. The term "quasi-static" testing, usually associated with cyclic tests, indicates that loads are applied at rates slow enough so that material strain rate effects do not influence the results (Leon and Deierlein 1996). Compared to shake table or dynamic test methods, a "quasi-static" or cyclic test method is relatively easier to conduct on a full-scale wall assembly and to explain its results. When based on the same test protocol, results from different tests can be compared to provide a consistent basis in developing the database for building codes. Critical to these tests is the choice of an appropriate loading history in which several important parameters must be considered. These parameters include the maximum load, yield point, ductility and stiffness, all of which are obtained from a monotonic push-over test. Furthermore, the amplitude, frequency and number of cycles, and the loading rate in the cyclic test have to be decided upon. Currently, several different test protocols, such as ASTM-SPD (1993), CEN (1995), FCC-Forintek (1994) and others used in Australia and New Zealand, are available. A recognized international standard for conducting cyclic load testing of shear walls does not exist, however. In these test procedures, different definitions of parameters apply, causing difficulties when comparisons of test results from various procedures are made.

The yield point is a key indicator in defining the cyclic test schedule or loading history. Experimental results show that the selection of the yield point could dramatically affect the shear wall performance such as strength, ductility and energy dissipation. Since wood material does not show a typical yield behaviour like steel, it is difficult to obtain a common definition of yield point without resorting to subjective judgment. Another related factor is ductility, which is a measure of deformation capacity of a system; i.e., the ability for a system to undergo deflection in the inelastic range without substantial reduction in strength. Although the definitions of ductility in most test procedures, including the three above-mentioned protocols, are the same (the ratio between the maximum slip and the yield slip, $D = \Delta_u / \Delta_{yield}$) differences in the definition of yield slip and
maximum slip can cause the calculated ductility to vary substantially. For some typical tests, for example, these values could vary from 5 to 11, where the FCC protocol gives the smallest value and the CEN protocol the largest. Experimental results from the current study and other studies indicated that the ductility values from tests following a single protocol displayed little variation, which shows that such variations do not seem realistic and that a standardized definition is needed.

The energy dissipation of a shear wall system, obtained from the area enclosed by the hysteresis loops, is an important index to evaluate its performance under earthquake loading. The most extensively used method to calculate the energy dissipation in a system is the so called "equivalent viscous damping ratio", which assumes that the dissipated energy of the actual damping force is equivalent to that of a viscous damper and is defined as

$$\hbox{eq} = \left(\frac{1}{2\pi}\right)\left(\frac{\Delta W}{W}\right)$$ (2.2)

where, $\Delta W$ is dissipated energy in one circle, and $W$ is the potential energy.

In a single-degree-of-freedom system, for which the equation of motion is

$$M\ddot{x} + C\dot{x} + Kx = F$$ (2.3)

the damping coefficient $C$ is determined by

$$C = 2\hbox{eq} \sqrt{MK}$$ (2.4)

As a general rule, a higher damping capacity, affected by the equivalent viscous damping ratio, mass and initial stiffness, is the single-most factor to control seismic response. Although this method was originally developed for the linear elastic systems, many researchers still use it for comparing the damping characteristics and for numerical modelling of the dynamic response of structures, even though the specimens were loaded beyond the elastic limit.
2.4.1. ASTM protocol

The ASTM protocol (ASTM 1993 and 1996) is also referred to as the sequential phased displacement (SPD) procedure, which was developed cooperatively in 1987 by U.S. and Japanese engineers and scientists participating on the Joint Technical Coordinating Committee on Masonry Research (TCCMAR). This procedure consists of reversed-cyclic displacements of progressively increasing magnitude until a first major event (FME) occurs (FME is defined as an event at which the first significant limit state occurs; here it implies the yield limit state). The loading is then followed by a stabilization and degradation cycle before progressing to the next higher increment of displacements. This process is repeated until failure (Figure 2.5).

Figure 2.5 ASTM loading history
To obtain the yield slip and maximum slip, the load-slip curve from monotonic test is converted to an elastic-perfectly plastic system with equivalent energy, which is defined by two lines (Figure 2.6). The first line represents the elastic part of the elastoplastic curve, obtained by connecting the origin and the point on the curve at the load 0.4$P_{\text{max}}$. The second line is a horizontal line at a load level equal to or greater than 0.8$P_{\text{max}}$. The yield slip ($A_{\text{yield}}$) then is defined at the intersect of these two lines and the maximum slip ($A_{\text{u}}$) is determined by equating areas $A_1$ and $A_2$ as shown in Figure 2.6. The cyclic frequency is a maximum of 1.0 Hz.

2.4.2. CEN protocol

The CEN protocol is one of a series of European standards for testing of joints made with mechanical fasteners. It includes two procedures: a general one (CEN Long), when the determination of

Figure 2.6 Definition of yield and ultimate displacement in different protocols

Figure 2.7 CEN protocol: (a) CEN Long; (b) CEN Short
the complete cyclic load-slip performance is required, and a particular one (CEN Short), when only
the determination of main performance at a pre-determined ductility level is required. The CEN
Long procedure (Figure 2.7 (a)) consists of number of cycle groups of three cycles each (except for
the first and the second cycle groups which consist of a single cycle) to reach 25%, 50%, 75%,
100%, 200%, 400%, ... of the yield slip ($\Delta_{\text{yield}}$) until failure or a slip of 30 mm is reached. The CEN
Short procedure (Figure 2.7 (b)) consists only of three identical cycles and then applies a
unidirectional load to the joint until failure. The amplitude is defined as the product of ductility and
yield slip ($D \times \Delta_{\text{yield}}$).

The yield slip ($\Delta_{\text{yield}}$) is defined by the intersection of two lines. The first line is determined
as that drawn through the points on the load-slip curve between $0.1P_{\text{max}}$ and $0.4P_{\text{max}}$. The second line
is the tangent to the curve having an inclination of $1/6$ of the first line (Figure 2.6). The maximum
slip ($\Delta_s$) is taken as either the displacement when the load has dropped to 80% of the maximum load,
or a slip of 30 mm, whichever occurs first in the test. A constant rate of slip, between 0.02 and 0.2
mm/sec, is prescribed for the cyclic test.

2.4.3. FCC - Forintek protocol

The FCC protocol consists of a
sequence of sinusoidal cycle groups,
with each cycle group containing three
identical cycles. The amplitude of each
cycle group is taken as a percentage of
the nominal yield slip ($\Delta_{\text{yield}}$) with
alternating increase and decrease of the
amplitude until specimen failure (Figure 2.8). The nominal yield slip is defined as the displacement
at a load level equal to half of the maximum load obtained during a monotonic test. Here the

![Figure 2.8 FCC-Forintek protocol](image-url)
maximum slip ($\Delta_w$) is defined as the displacement at the maximum load (Figure 2.6). A constant frequency of 0.5 Hz (2 seconds per cycle) is used.

### 2.5. Effect of shear wall geometric configurations

Several researchers (Patton-Mallory et al. 1985; Sugiyama 1994; Griffiths and Enjily 1996; Dolan 1996) studied how shear wall performance varies with wall geometric configurations, nail spacing and vertical load. The results indicated that the racking strength and stiffness of shear walls increase proportionally with the increase of wall length. Plywood sheathing provided higher racking strength compared to OSB panels. Double-sheathed walls had almost twice the racking strength as single-sheathed walls. Vertical loads provide shear walls with higher lateral load resistance and stiffness, but the influence of vertical loads decreases with wall length. The nail spacing around the panel perimeter has a significant influence on shear wall performance. Smaller nail spacing results in a higher racking resistance, although it is not directly proportional to the number of fasteners. The nail spacing on internal studs is not critical to wall performance and often is twice the perimeter spacing. In investigating the effect of openings, they found that under monotonic loading condition the decrease in the shear wall strength was related to the area of the openings. The empirical equations by Patton-Mallory (the ratio of effective wall length) and by Sugiyama (the sheathing area ratio) were tried to calculate the strength loss due to openings. The experimental results show that the former tends to overestimate the initial stiffness and ultimate load capacity of shear walls with openings (Patton-Mallory et al. 1985), while the latter provides a conservative estimation of the shear load ratio at ultimate capacity (Johnson and Dolan 1996). The predominant failure mode for shear walls with openings is nail failure, especially near the corners of openings. The nail failure mode is the same as that in shear walls without openings.

All of the above effects are considered as factors in shear wall design in addition to the typical specifications based on materials properties to calculate allowable shear resistance.
2.6. Oriented strand board (OSB)

Oriented strand board panelling was introduced in the housing construction industry of North America in the early 1970’s as an engineered sheathing material. OSB is comprised of thin wooden strands (averaging 102 mm long and 25 mm wide) dried, and coated with resin and wax. The strands are oriented in cross directional layers and then bonded under high temperature and pressure with a water resistance adhesive resulting in a structural panel equivalent to plywood in its major mechanical properties. One of the advantages of this panel is that it makes efficient use of small diameter logs and residual timber which is not suitable for plywood and other lumber products, thus providing highly economical and environmental benefits. In Canada, OSB panels are “performance rated”, meeting CSA-O325 requirements. Performance standards ensure that the panels satisfy the requirements of the intended end use, such as strength and stiffness properties, while providing flexibility in manufacturing. OSB is typically manufactured in either standard dimensions of 1.2 × 2.4 m or, more recently, in non-standard dimensions ranging from 0.9 × 1.8 m to 3.3 × 7.3 m, with its thickness from 6 mm to 38 mm. In the last decades OSB has taken a large market share from plywood with a current output of 9.6 million m$^3$ (1994) which equalled 50% of plywood output in North America. It is estimated that by the year 2000, OSB will account for 82% of Canada’s structural panel production and 44% of US output.
3. THE SHEAR WALL MODEL AND ANALYTICAL PROGRAM

Wood based shear wall systems are highly redundant and may display complicated stress and deformation fields when subjected to external forces. To better predict and simulate the performance of shear walls, finite element analysis programs have been developed. When the different structural components, such as panel, frame and fasteners, are correctly modelled and assembled, finite element procedures can provide satisfactory solutions. Due to the high nonlinearity of shear wall systems, however, nonlinear analysis procedures are necessary, which have only become available relatively recently.

The finite element program - Shear Wall/Diaphragm Analysis Program (DAP) was developed by Foschi (1993) to analyze wood based shear wall systems. The program was used extensively in this study to predict the performance of the different shear wall configurations with regular and oversize panels. In this chapter, a brief summary of the program is given to show the basic concepts and assumptions.

3.1. The shear wall model

A typical shear wall system consists of four basic structural components: sheathing panel, frame, connections between frame members, and connections between panel and frame. Based on the model described in DAP, an illustration of the model is shown in Figure 3.1. The sheathing panel is divided into a number of two-dimensional twelve-node cubic isoparametric elements with two displacement degrees of freedom at each node, namely in the x and y directions. It
is assumed that the panel is subjected to a plane stress field and has elastic orthotropic material properties. The frame members consist of a number of one-dimensional beam elements with three degrees of freedom at each node: displacements in $x$ and $y$ directions and rotation $\theta$. The deformations of the members include lateral deflection $w$ and axial displacement $u$. Their mechanical properties are assumed to be linear elastic. Nonlinear spring elements are used to model two kinds of nail connectors, either between frame members or between panel and frame. The nonlinearity of the shear wall assembly is therefore solely determined by the behaviour of the nail connectors.

3.2. Panel members

In this study, no body forces and initial stresses are considered.

3.2.1. Formulation of the panel finite elements based on the principle of potential energy

In the panel, the total potential energy $V$ is the summation of energy contributions from each element, $V^e$,

$$V = \sum e V^e$$  \hspace{1cm} (3.1)$$

For one element, the potential energy in matrix form is

$$V^e = \Omega^e + U^e = -\int_s [F^T u ds + \frac{1}{2} \int_\Omega e^T e dV]$$  \hspace{1cm} (3.2)$$

where:

$\Omega^e$: potential energy of external loads in one element,

$U^e$: strain energy in one element,

$F$: surface loads (tractors) acting on the element,

$u$: the element displacement vector.

The element strains are
\[ \varepsilon = [\varepsilon_x \quad \varepsilon_y \quad \gamma_{xy}] = \begin{bmatrix} \frac{\partial u}{\partial x} & \frac{\partial v}{\partial y} & \frac{\partial u}{\partial x} + \frac{\partial v}{\partial y} \end{bmatrix} \] (3.3)

Since

\[ \varepsilon = Bs \] (3.4)

and

\[ \sigma = D\varepsilon = DBs \] (3.5)

where

\( s \): element nodal displacement vector, \( s = [u_1 \quad u_2 \ldots u_{12} \quad v_1 \quad v_2 \ldots v_{12}] \),

\( B \): strain-displacement transformation matrix (see Section 3.2.2),

\[ \sigma = \begin{bmatrix} \sigma_x & \sigma_y & \tau_{xy} \end{bmatrix} \] (3.6)

\[ D = \begin{bmatrix} E_x & \frac{\nu_{xy} E_x}{1 - \nu_{xy}^2} & 0 \\ \frac{1 - \nu_{xy}^2}{E_x} & \frac{\nu_{xy} E_x}{1 - \nu_{xy}^2} & 0 \\ 0 & 0 & G_{xy} \end{bmatrix} \] (3.7)

where

\( E_x, E_y \): Young’s moduli,

\( G_{xy} \): shear modulus,

\( \nu_{xy}, \nu_{yx} \): Poisson’s ratios.

Rewriting Equation (3.2), yields

\[ \Omega^e = -\int_{\Omega} [F] \cdot N \cdot ds = -S^T s \]

\[ U^e = \frac{1}{2} \int_{\Omega} [s]^T B^T DB s \, dV = \frac{1}{2} [s]^T \cdot k \cdot s \] (3.8)
where the load vector $S$ and stiffness matrix of an element $k$ are given by

$$S^T = \int_\gamma F^T N ds$$

$$k = \int \int B^T DB dV$$

From the requirement for equilibrium, the potential energy has to be stationary, i.e.,

$$\delta V = \delta \Omega + \delta U = 0$$

This statement is known as the Principle of Stationary Potential Energy. Apply this equation to each individual element,

$$\delta V^e = \delta \Omega^e + \delta U^e$$

$$\frac{\partial V^e}{\partial s} = \frac{\partial \Omega^e}{\partial s} + \frac{\partial U^e}{\partial s} = \frac{\partial (-S^T s)}{\partial s} + \frac{\partial (s^T ks)}{2\partial s} = -S^T + s^T k = 0$$

$$\therefore \ S = ks$$

The global stiffness matrix $K_P$ is then assembled from the element stiffness matrices $k_i$.

3.2.2. Shape functions

Consider a natural coordinate system $(\xi, \eta)$ in the panel element (Figure 3.2). The shape functions $N$ in terms of this natural system are
\[ N_1 = \frac{9}{32} (1 - \xi)(1 - \eta) \left[ -10 + 9(\xi^2 + \eta^2) \right] \]
\[ N_2 = \frac{9}{32} (1 - \xi^2)(1 - \eta)(1 - 3\xi) \]
\[ N_3 = \frac{9}{32} (1 - \xi^2)(1 - \eta)(1 + 3\xi) \]
\[ N_4 = \frac{9}{32} (1 + \xi)(1 - \eta)(1 - 3\xi) \]
\[ N_5 = \frac{9}{32} (1 + \xi)(1 - \eta^2)(1 - 3\eta) \]
\[ N_6 = \frac{9}{32} (1 + \xi)(1 - \eta^2)(1 + 3\eta) \]
\[ N_7 = \frac{9}{32} (1 + \xi)(1 + \eta)(1 - 3\xi) \]
\[ N_8 = \frac{9}{32} (1 - \xi^2)(1 + \eta)(1 + 3\xi) \]
\[ N_9 = \frac{9}{32} (1 - \xi^2)(1 + \eta)(1 - 3\xi) \]
\[ N_{10} = \frac{9}{32} (1 - \xi)(1 + \eta)(1 - 3\xi) \]
\[ N_{11} = \frac{9}{32} (1 - \xi)(1 - \eta^2)(1 + 3\eta) \]
\[ N_{12} = \frac{9}{32} (1 - \xi)(1 - \eta^2)(1 - 3\eta) \]

(3.12)

Figure 3.2 Panel finite element

In the isoparametric formulation, the element displacements are interpolated by

\[ u = \sum_{i=1}^{12} N_i u_i \]
\[ v = \sum_{i=1}^{12} N_i v_i \]

(3.13)
or in matrix form

$$u = Ns$$

(3.14)

Since

$$\frac{\partial N_i}{\partial \xi} = \frac{\partial N_i}{\partial x} \frac{\partial x}{\partial \xi} + \frac{\partial N_i}{\partial y} \frac{\partial y}{\partial \xi}$$

$$\frac{\partial N_i}{\partial \eta} = \frac{\partial N_i}{\partial x} \frac{\partial x}{\partial \eta} + \frac{\partial N_i}{\partial y} \frac{\partial y}{\partial \eta}$$

(3.15)

in matrix form

$$\begin{bmatrix} \frac{\partial N_i}{\partial \xi} \\ \frac{\partial N_i}{\partial \eta} \end{bmatrix} = \begin{bmatrix} \frac{\partial x}{\partial \xi} & \frac{\partial y}{\partial \xi} \\ \frac{\partial x}{\partial \eta} & \frac{\partial y}{\partial \eta} \end{bmatrix} \begin{bmatrix} \frac{\partial N_i}{\partial x} \\ \frac{\partial N_i}{\partial y} \end{bmatrix}$$

(3.16)

where

$$J = \begin{bmatrix} \frac{\partial x}{\partial \xi} & \frac{\partial y}{\partial \xi} \\ \frac{\partial x}{\partial \eta} & \frac{\partial y}{\partial \eta} \end{bmatrix}$$

(3.17)

is called the Jacobian matrix. Inverse Equation (3.16) to obtain the relations between the element coordinates and the natural coordinates

$$\begin{bmatrix} \frac{\partial N_i}{\partial x} \\ \frac{\partial N_i}{\partial y} \end{bmatrix} = J^{-1} \begin{bmatrix} \frac{\partial N_i}{\partial \xi} \\ \frac{\partial N_i}{\partial \eta} \end{bmatrix}$$

where

$$J^{-1} = \frac{1}{|J|} \begin{bmatrix} \frac{\partial y}{\partial \eta} & -\frac{\partial y}{\partial \xi} \\ -\frac{\partial y}{\partial x} & \frac{\partial y}{\partial \xi} \end{bmatrix}$$

(3.18)

then the element coordinates and the strain-displacement transformation matrix $B$ can be obtained as follows
\[ x = \sum_{i=1}^{12} N_i x_i \quad y = \sum_{i=1}^{12} N_i y_i \]  

(3.19)

\[
B = \begin{bmatrix}
\frac{\partial N_1}{\partial x} & 0 & \frac{\partial N_2}{\partial x} & 0 & \ldots & \frac{\partial N_{12}}{\partial x} & 0 \\
0 & \frac{\partial N_1}{\partial y} & 0 & \frac{\partial N_2}{\partial y} & \ldots & 0 & \frac{\partial N_{12}}{\partial y} \\
\frac{\partial N_1}{\partial y} & \frac{\partial N_1}{\partial x} & \frac{\partial N_2}{\partial y} & \frac{\partial N_2}{\partial x} & \ldots & \frac{\partial N_{12}}{\partial y} & \frac{\partial N_{12}}{\partial x}
\end{bmatrix}
\]  

(3.20)

So that the stains, \( \varepsilon \), can be calculated by \( B \) and displacements \( s \) as given in Equation (3.4).

### 3.2.3. Numerical integration

A 4 \times 4 Gaussian quadrature scheme is employed to solve the components in Equation (3.4):

\[
\int_{-1}^{1} \int_{-1}^{1} F(\xi, \eta) d\xi d\eta = \sum_{i=1}^{4} \sum_{j=1}^{4} W_i W_j F(\xi_i, \eta_j)
\]  

(3.21)

where

\( W_i, W_j \): Gaussian weights,

\( \xi_i, \eta_i \): local coordinates of Gaussian points,

shown in Figure 3.3.

The calculated stresses within the panel element are located at these Gaussian points.

### 3.3. Frame members

A frame element of length \( L \) between two nodes \( i \) and \( j \) is shown in Figure 3.4. Cubic shape functions are used to calculate the lateral deflection, while linear shape functions are used to represent the axial displacement.

The strain energy in a frame element is
\[
U^e = \frac{1}{2} \left[ L \int_0^L E I \left( \frac{d^2 w}{dx^2} \right)^2 \, dx + L \int_0^L E A \left( \frac{du}{dx} \right)^2 \, dx \right]
\]

(3.22)

The lateral deflection \( w \) and axial displacement \( u \) of an element can be represented by the shape functions \( N_i \) and \( M_i \) and the nodal displacements, \( w, \theta, \) and \( u \):

\[
w = N_1^F(x)w_i + N_2^F(x)\theta_i + N_3^F(x)w_j + N_4^F(x)\theta_j
\]
\[
u = M_1^F(x)u_i + M_2^F(x)u_j
\]

(3.23)

where the shape functions are as follows:

\[
N_1^F(x) = 1 - 3 \left( \frac{x}{L} \right)^2 + 2 \left( \frac{x}{L} \right)^3
\]
\[
N_2^F(x) = L \left( \frac{x}{L} \right)^2 - 2 \left( \frac{x}{L} \right)^3 + \left( \frac{x}{L} \right)^3
\]
\[
N_3^F(x) = 3 \left( \frac{x}{L} \right)^2 - 2 \left( \frac{x}{L} \right)^3
\]
\[
N_4^F(x) = L \left[ - \left( \frac{x}{L} \right)^2 + \left( \frac{x}{L} \right)^3 \right]
\]
\[
M_1^F(x) = 1 - \left( \frac{x}{L} \right)
\]
\[
M_2^F(x) = \left( \frac{x}{L} \right)
\]

(3.24)

From the principle of potential energy (Equations (3.2) to (3.9)), the stiffness matrix of the frame element is

\[
k_F = \begin{bmatrix}
  k_1 & 0 & 0 & -k_1 & 0 & 0 \\
  0 & k_{11} & k_{12} & 0 & k_{13} & k_{14} \\
  0 & k_{21} & k_{22} & 0 & k_{23} & k_{24} \\
 -k_1 & 0 & 0 & k_1 & 0 & 0 \\
  0 & k_{31} & k_{32} & 0 & k_{33} & k_{34} \\
  0 & k_{41} & k_{42} & 0 & k_{43} & k_{44}
\end{bmatrix} = \begin{bmatrix}
  \frac{EA}{L} & 0 & 0 & 0 & 0 & -\frac{EA}{L} & 0 & 0 \\
  0 & \frac{12EI}{L^3} & \frac{6EI}{L^2} & 0 & 0 & -\frac{12EI}{L^2} & \frac{6EI}{L} & 0 \\
  0 & \frac{6EI}{L^2} & \frac{4EI}{L} & 0 & 0 & -\frac{6EI}{L} & \frac{2EI}{L} & 0 \\
 -\frac{EA}{L} & 0 & 0 & \frac{EA}{L} & 0 & 0 & 0 & 0 \\
  0 & -\frac{12EI}{L^3} & -\frac{6EI}{L^2} & 0 & 0 & 0 & -\frac{6EI}{L} & \frac{2EI}{L} \\
  0 & \frac{6EI}{L^2} & \frac{2EI}{L} & 0 & 0 & 0 & -\frac{6EI}{L} & \frac{4EI}{L}
\end{bmatrix}
\]

(3.25)
where

$$k_{ij} = \int_0^L EIN_i^F N_j^F \, dx$$

(3.26)

The global stiffness matrix $K_F$ is assembled from the stiffness matrices of all beam elements of the frame members $k_{Fi}$.

3.4. Panel-frame connections

The panel elements are attached to frame elements by nail connectors. A nail is assumed to be located at a pair of points, $C$ and $C'$, which belong to the panel element and frame element, respectively, and are overlapped before loading. Therefore, the relative displacement, $|\Delta|$, between points $C$ and $C'$ is defined as the deformation of the nail at the point $C$ (Figure 3.5). $|\Delta|$ is composed of two components, i.e., $\Delta w$ and $\Delta u$, which are normal to the frame member and along the member, respectively. By using Equations (3.14) and (3.24), and considering the angle $\psi$ (transforming the natural coordinates to $x'$-$y'$ coordinates), the relative displacements are

$$\Delta w = w_C - w_{C'} = \sum_{k=1}^{12} (-N_k u_k \sin \psi + N_k v_k \cos \psi) - \left(N_1^F w_j + N_2^F \theta_i + N_3^F w_j + N_4^F \theta_j \right)$$

$$\Delta u = u_C - u_{C'} = \sum_{k=1}^{12} (N_k u_k \cos \psi + N_k v_k \sin \psi) - \left(M_1^F u_j + M_2^F u_j \right)$$

(3.27)
or in matrix form

\[
\Delta w = N_w^T s_c \\
\Delta u = N_u^T s_c
\]  

(3.28)

The strain energy of the frame element with the force \( P \) and a nail density \( \lambda \) is

\[
\Omega_c^e = \int_0^L \lambda (P dz \cos(\phi - \alpha)) dx = \int_0^L \lambda P \left( \frac{\Delta u}{|A|} d(\Delta u) + \frac{\Delta w}{|A|} d(\Delta w) \right) dx
\]  

(3.29)

where \( dz \) is a virtual displacement.

Its variational equation can be expressed as the sum of linear part \( k_c^e \) and a nonlinear part \( W_c^e \)

\[
\delta \Omega_c^e = k_c^e s_c - W_c^e
\]  

(3.30)

and \( k_c^e \) and \( W_c^e \) can be expressed as follows

\[
k_c^e = \int_0^L \lambda \left( k_C^w N_u N_u^T + k_C^w N_u N_u^T \right) dx
\]

\[
W_c^e = \int_0^L \lambda \left[ k_C^u \Delta u - P \frac{\Delta u}{|A|} N_u + \left( k_C^w \Delta w - P \frac{\Delta w}{|A|} \right) N_w \right] dx
\]  

(3.31)

Here a four-point Gaussian quadrature scheme is used again. Then the global matrices for the panel-frame connection, \( K_c \) and \( W_c \), are assembled by the element matrices \( k_c^e \) and \( W_c^e \), respectively.

The nonlinear load-slip relationship of a nail connection shown in Figure 3.6 can be described by the following formula:

\[
P = \begin{cases} 
(P_0 + P_1 \Delta) \left( 1 - e^{-\frac{\Delta u}{f}} \right) & \Delta \leq E_0 \\
(P_0 + P_1 E_0) \left( 1 - e^{-\frac{\Delta u}{f}} \right) - \left( \Delta - E_0 \right) m_g & \Delta > E_0
\end{cases}
\]  

(3.32)

where
\( P \): applied load,
\( \Delta \): slip,
\( P_0 \): the intercept of the asymptote,
\( P_1 \): the asymptotic stiffness,
\( K \): the initial stiffness,
\( E_0 \): the slip at peak load,
\( m_E \): the slope of the failure branch

Above parameters are obtained from the nail tests.

3.5. The frame-frame connections

By the same procedure used in Section(3.4), the relative displacements between two points \( i \) and \( j \) along the member axes \( x' \) and \( y' \), and the relative rotation are (referring to Figure 3.7)

\[
\Delta u = \left[ (u_i - u_j) \cos \psi + (v_i - v_j) \sin \psi \right]
\]
\[
\Delta v = \left[ -(u_i - u_j) \sin \psi + (v_i - v_j) \cos \psi \right]
\]
\[
\Delta \theta = (\theta_i - \theta_j)
\]

(3.33)

or in matrix form

\[
\Delta u = T_u^T s_s \quad \Delta v = T_v^T s_s \quad \Delta \theta = T_\theta^T s_s
\]

(3.34)

Employing the energy method, the variational equation can again be obtained as

\[
\delta \Omega_S^\varepsilon = k_S^\varepsilon s_s - W_S^\varepsilon
\]

(3.35)

where the linear part \( k_S^\varepsilon \) and nonlinear part \( W_S^\varepsilon \) are
\( k_S = k_u T_u T_u^T + k_v T_v T_v^T + k_\theta T_\theta T_\theta^T \)

\[ W_S^e = \left( k_u \Delta u - P_x \frac{\Delta u}{\Delta u} \right) T_u + \left( k_v \Delta v - P_y \frac{\Delta v}{\Delta v} \right) T_v + \left( k_\theta \Delta \theta - P_\theta \frac{\Delta \theta}{\Delta \theta} \right) T_\theta \]

(3.36)

where the load-slip relationship of the nail in all three directions is

\[
P_{(x, y, \phi)} = \begin{cases} 
    (P_0 + P_\Delta) \left(1 - e^{-\left(K_{\Delta}/P_\Delta\right)}\right) & \Delta \leq E_0 \\
    (P_0 + P_\Delta E_0) \left(1 - e^{-\left(K_{E_0}/P_\Delta\right)}\right) - (\Delta - E_0)m_E & \Delta > E_0
\end{cases}
\]

(3.37)

The global matrices for the frame-frame connection \( K_S \) and \( W_S \) are assembled from the element matrices \( K_S^e \) and \( W_S^e \), respectively.

Now, the stiffness matrices for panel elements \( K_P \), frame elements \( K_F \) and the linear stiffness matrices of two groups of connection elements \( K_C \) and \( K_S \) can be assembled into \( K \), the linear part of the global stiffness matrix of the shear wall system. The matrices for two groups of connection elements \( W_C \) and \( W_S \) can be assembled into \( W \), the nonlinear part of the global stiffness matrix of the system.

\[
K = K_P + K_F + K_C + K_S
\]

\[
W = W_C + W_S
\]

(3.38)

Applying the principle of potential energy and letting \( R \) be the load vector and \( r \) be the displacement vector, the governing equation of the shear wall system is

\[
R = Kr - W(r)
\]

(3.39)

By iteration, the displacements of the whole system can be obtained

\[
r_0 = K^{-1} R
\]

\[
r_{i+1} = K^{-1} R + K^{-1} W(r_i)
\]

(3.40)
3.6. The flow chart of Diaphragm Analysis Program

The Diaphragm Analysis Program, when used in this study, has been expanded to 250 frame elements, 100 cover elements and 1000 nodes with 400 nodes for frame elements (originally the program capacity was 60 frame elements, 10 cover elements and 180 nodes). Another modification to the program was carried out to calculate cases subjected to not only live loads but constant dead loads in both concentrated and distributed forms simultaneously. The main flow chart of this program is shown in Figure 3.8 and the detailed flow chart for the key part, SADT, is shown in Figure 3.9. The program can output the nodal displacements for panel and frame elements. It can also provide, if required, force and stress data in frame elements and stress data in panel elements.

![Flow Chart](image)

Figure 3.8 DAP main flow chart
Figure 3.9 The flow chart of SADT
4. EXPERIMENTAL PROCEDURES

4.1. Experimental facilities

A test setup was designed and built to conduct monotonic and cyclic shear tests on a 2.4 × 7.3 m shear wall with regular or oversize OSB panels. The test setup included a steel framework, lateral and vertical load actuators, and test control and data acquisition systems.

Figure 4.1 and Photo 4.1 show the shear wall test setup assembly, which had a total length of 12.5 m and height of 3.7 m. The test specimen was mounted vertically between the base beam and the load distribution beam using thirty six 12.7 mm Ø class 5 steel bolts on its top and bottom sides. The base beam was in turn bolted to the ground by five 38 mm Ø steel bolts to provide a rigid foundation restricting the bottom of the shear wall from any possible movement. The load distribution beam was attached to the top of the shear wall to transfer the in-plane lateral live load and vertical dead load to the wall. Two 7.6 m long W150 × 22 steel I beams were used as the load distribution beam and base beam. Two sets of lateral support frames at the two ends of the load distribution beam prevented the out-of-plane displacement of shear wall. Two roller guides were installed vertically on each support frame to allow the load distribution beam to move freely in the longitudinal direction of shear wall during the test. A reaction frame made from three bolt-connected members was set up along the long axis of the wall specimen to support the lateral load actuator. One W310 × 143 steel I beam, 3.7 m in height, was used as the reaction column, and two pipes (HSS 101.6 O.D. × 6.35) were used as braces to stiffen the reaction frame. The frame was fixed to the ground by 50.8 mm Ø bolts and welded to the base beam at one end.
Figure 4.1 The scheme of the shear wall test setup assembly
A programmable double-acting hydraulic actuator with an inline load cell was mounted between the wall specimen and the reaction frame at a distance of 2.67 m (Figure 4.1 (b)) from the ground. The capacity of the hydraulic actuator was 222 kN, from which either monotonic or cyclic lateral load could be applied to the shear wall specimen. The loading rates used in the tests were 0.13 mm/sec for monotonic tests and 0.25 Hz to 0.083 Hz in cycle frequencies for cyclic tests. Six hydraulic jacks, each with a working pressure capacity of 17.24 MPa, were mounted to the test floor at the base and attached through 2.1 m long 12.7 mm Φ steel rods to the top of load distribution beam. Three jacks were mounted on each side of the wall and placed symmetrically about the center of the wall with a spacing of 2.44 m. These jacks provided a static distributed vertical load to the wall through the top of the load distribution beam. In all of the tests a vertical load of 9.12 kN/m was used which represented a gravity load of 5.0 kPa over an area of 7.3 x 7.3 m, equivalent to the weight of the second floor of a two-storey building, supported by four walls.

A MTS Micro-controller (458.10) and Material Testing Function Generator (Exact-340) were used to drive the actuator in a displacement control mode to develop the required lateral loading
pattern. A 386/25 personal computer data acquisition system with 16 A/D channels and LabTech Notebook data acquisition software were used to collect the experimental data which included the lateral load, vertical load, actuator displacements, lateral displacements at the top of the shear wall, uplift at the base of the shear wall, and relative movements between panel and frame at various locations (Figure 4.2). The sampling frequencies were 1 Hz for the monotonic load condition and 10 Hz for cyclic load conditions.

where the measurements of transducers are as follows:
A: lateral load and displacement of actuator,
B: vertical load of jacks,
C: lateral displacement of shear wall,
D: absolute movement of wall member at different locations,
E: relative movement between wall member at different locations,
F: diagonal deformation of panel,
G: relative movement between panels for shear wall with regular panels only.
4.2. Test shear wall configurations

In all the shear wall specimens, 2.4 × 7.3 m in dimension, No. 2 and better Spruce Pine Fir 38 × 89 mm dimensional lumber members were used as framing members. This choice was based on the consideration of the influence of lumber quality on experimental output and the compatibility of the test results with those of other researchers. It was felt that lumber with less defects and more uniform mechanical properties would ensure that the experiments would generate more consistent output, especially with the small number of replications. The wall frame included stud members and plate members. The stud members were spaced 400 mm apart. The top plate and the end studs consisted of double members while the bottom plate and the interior studs consisted of single members. At mid-height of the walls built with regular panels, either continuous blocking components (Photo 4.1) or staggered blocking components (Photo 4.2) were installed. The installation of the continuous blocking components was done by end-nailing one end of the blocking and toe-nailing the other end to the framing member. The installation of staggered blocking involved
only end-nailing. The moisture content of the lumber ranged from 9% to 12% during the tests and its average specific gravity was 0.42.

Performance Rated W24 Oriented Strand Board (CSA-0325.0-M88), 9.5 mm thick, was used as sheathing panels. The single panel dimensions used in the study were 1.2 × 2.4 m (regular) and 2.4 × 7.3 m (oversize), respectively. All of the sheathing panels were oriented with the long axis of the panels parallel to the length of the wall. In the shear walls built with the regular size panels, the panels were staggered. The panel layouts for shear walls built with regular panels (a) and oversize panels (b) are shown in Figure 4.3. The elastic constants which were used in the finite element analysis were obtained from a study of OSB at Forintek Canada Corp. (Lau and Lam 1989). The reasons were that (1) the elastic constants for both OSB panel types may not differ much because of
similar layout configuration; (2) the testing facility for all elastic constants of OSB panels was not available during the study; and (3) minor variations of these constants were found from a parameter study to have a negligible effect on the output from the finite element analysis.

The frame members were connected by 76 mm coil-fed common nails (in the lower center of Photo 4.3) driven by a Bostitch-N80CB air nail gun. The sheathing panels were connected to the framing members with 50 mm spiral nails (nail gun driven, shown in the upper left of Photo 4.3) and 50 mm common nails (hand driven, shown in the upper right of Photo 4.3). For the interior attachment of the sheathing panel to the frame members, the nail spacing was 300 mm throughout, while for the perimeter attachment, nail spacing ranged from 76 mm to 152 mm.

Photo 4.3 The nails used in shear wall tests
4.3. Test program

Thirteen shear walls were tested with one replicate per test type (Table 4.1). The following four test conditions were investigated:

(1) **Panel dimensions**: Two panel dimensions, regular $1.2 \times 2.4$ m for tests 1, 4 and 7 and oversize $2.4 \times 7.3$ m for the other ten tests, were used. In the walls sheathed with regular panels, continuous blocking was installed in walls 1 and 4 and staggered blocking in wall 7.

(2) **Nail types for the panel-frame connections**: 50 mm common nails in tests 5 and 6 and spiral nails in all other tests were used.

(3) **Nail spacing around the perimeter of panels**: Three types of nail spacing were investigated. They were:

- **Type one**: 152 mm around all edges, in tests 1 to 7,
- **Type two**: 76 mm around all edges, in tests 8, 9, 12 and 13, and
- **Type three**: 76 mm at four $0.813 \times 0.813$ m corners and 152 mm along remaining edges, in tests 10 and 11.

The total number of nail connections between panel and framing were 424 for the walls with regular panels (375 for staggered blocking) and 281 to 409, depending on nail spacing, for the walls with oversize panels.

(4) **Loading schedules**: Walls 1, 2, 5, 7, 8 and 10 were tested under a monotonic loading scheme, and walls 3, 4, 6, 9, 11, 12 and 13 were tested under a cyclic loading scheme. For the cyclic tests, the Forintek protocol was adopted in walls 3 to 11, the CEN Short protocol in wall 12 and a newly proposed protocol in wall 13.
### Table 4.1 Shear wall test program

<table>
<thead>
<tr>
<th>Wall No.</th>
<th>Panel size (m)</th>
<th>Blocking types</th>
<th>Lateral load types</th>
<th>Nails for sheathing</th>
<th>Nail spacing (mm)</th>
<th>No. of nails</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>1.2 x 2.4</td>
<td>Continuous</td>
<td>Monotonic</td>
<td>Spiral (Gun)</td>
<td>152</td>
<td>424</td>
</tr>
<tr>
<td>2</td>
<td>2.4 x 7.3</td>
<td>N/A</td>
<td>Monotonic</td>
<td>Spiral (Gun)</td>
<td>152</td>
<td>281</td>
</tr>
<tr>
<td>3</td>
<td>2.4 x 7.3</td>
<td>N/A</td>
<td>Cyclic (F)</td>
<td>Spiral (Gun)</td>
<td>152</td>
<td>281</td>
</tr>
<tr>
<td>4</td>
<td>1.2 x 2.4</td>
<td>Continuous</td>
<td>Cyclic (F)</td>
<td>Spiral (Gun)</td>
<td>152</td>
<td>424</td>
</tr>
<tr>
<td>5</td>
<td>2.4 x 7.3</td>
<td>N/A</td>
<td>Monotonic</td>
<td>Common (Hand)</td>
<td>152</td>
<td>281</td>
</tr>
<tr>
<td>6</td>
<td>2.4 x 7.3</td>
<td>N/A</td>
<td>Cyclic (F)</td>
<td>Common (Hand)</td>
<td>152</td>
<td>281</td>
</tr>
<tr>
<td>7</td>
<td>1.2 x 2.4</td>
<td>Staggered</td>
<td>Monotonic</td>
<td>Spiral (Gun)</td>
<td>152</td>
<td>375</td>
</tr>
<tr>
<td>8</td>
<td>2.4 x 7.3</td>
<td>N/A</td>
<td>Monotonic</td>
<td>Spiral (Gun)</td>
<td>76</td>
<td>409 (128)</td>
</tr>
<tr>
<td>9</td>
<td>2.4 x 7.3</td>
<td>N/A</td>
<td>Cyclic (F)</td>
<td>Spiral (Gun)</td>
<td>76</td>
<td>409 (128)</td>
</tr>
<tr>
<td>10</td>
<td>2.4 x 7.3</td>
<td>N/A</td>
<td>Monotonic</td>
<td>Spiral (Gun)</td>
<td>76 and 152</td>
<td>325 (44)</td>
</tr>
<tr>
<td>11</td>
<td>2.4 x 7.3</td>
<td>N/A</td>
<td>Cyclic (F)</td>
<td>Spiral (Gun)</td>
<td>76 and 152</td>
<td>325 (44)</td>
</tr>
<tr>
<td>12</td>
<td>2.4 x 7.3</td>
<td>N/A</td>
<td>Cyclic (C)</td>
<td>Spiral (Gun)</td>
<td>76</td>
<td>409 (128)</td>
</tr>
<tr>
<td>13</td>
<td>2.4 x 7.3</td>
<td>N/A</td>
<td>Cyclic (N)</td>
<td>Spiral (Gun)</td>
<td>76</td>
<td>409 (128)</td>
</tr>
</tbody>
</table>

 suppression symbol. Forintek cyclic test protocol, C - CEN Short cyclic test protocol, N - the newly proposed cyclic test protocol.

The number in parentheses shows the additional number of nails by using a reduced nail spacing, compared to the wall with a normal 152 mm nail spacing.

4.4. Loading Scheme

4.4.1. The monotonic loading scheme

The loading rate for the monotonic tests was approximately 0.13 mm/sec based on recommendations in the ASTM Standard E564-73 and in compliance with tests done previously by Forintek Canada Corp.

4.4.2. The Forintek cyclic loading scheme

According to the definition in Section 2.5.3, the maximum load ($P_{max}$), yield point ($\Delta_{yield}$), from the corresponding monotonic tests, and cycle frequencies for shear walls 3, 4, 6, 9 and 11 are listed in Table 4.2. To obtain consistent results for both monotonic and cyclic tests and for the ease of actuator control, the values for the yield points were taken from the displacement of the hydraulic cylinder, instead of that of the shear wall.
Table 4.2: The parameters in cyclic tests using Forintek cyclic test protocol

<table>
<thead>
<tr>
<th>Wall</th>
<th>$P_{\text{max}}$ (kN)</th>
<th>$\Delta_{\text{yield}}$ (mm)</th>
<th>Loading rate</th>
<th>Cycle group</th>
<th>Cycle frequency (Hz)</th>
<th>Cycle group</th>
<th>Cycle frequency (Hz)</th>
</tr>
</thead>
<tbody>
<tr>
<td>3</td>
<td>82.21</td>
<td>6.0</td>
<td></td>
<td>1 to 23</td>
<td>0.25</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>4</td>
<td>62.77</td>
<td>10.0</td>
<td></td>
<td>1 to 19</td>
<td>0.25</td>
<td>20 to 22</td>
<td>0.083</td>
</tr>
<tr>
<td>6</td>
<td>71.13</td>
<td>6.0</td>
<td></td>
<td>1 to 13</td>
<td>0.25</td>
<td>14 to 23</td>
<td>0.125</td>
</tr>
<tr>
<td>9</td>
<td>125.21</td>
<td>10.0</td>
<td></td>
<td>1 to 18</td>
<td>0.25</td>
<td>19 to 22</td>
<td>0.125</td>
</tr>
<tr>
<td>11</td>
<td>84.27</td>
<td>8.0</td>
<td></td>
<td>1 to 17</td>
<td>0.25</td>
<td>18 to 22</td>
<td>0.125</td>
</tr>
</tbody>
</table>

The amplitudes of cycle groups as a percentage of the yield point for these walls are listed in Table 4.3. The displacement amplitudes of these cycle groups are graphically illustrated in Figure 2.8. Please note that the amplitude schedule is identical for these walls up to cycle group 18. Typically at this cycle group, the maximum load had already been well exceeded. Both the amplitude schedules and the cycle frequencies were adjusted beyond cycle group 18 so that the hardware data storage capability was not exceeded.

Table 4.3: Displacement amplitude schedule of various cycle groups as a percentage of $\Delta_{\text{yield}}$.

<table>
<thead>
<tr>
<th>Cycle Group</th>
<th>Wall No. 3</th>
<th>Wall No. 4</th>
<th>Wall No. 6</th>
<th>Wall No. 9</th>
<th>Wall No. 11</th>
<th>Wall No. 3</th>
<th>Wall No. 4</th>
<th>Wall No. 6</th>
<th>Wall No. 9</th>
<th>Wall No. 11</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>25%</td>
<td>25%</td>
<td>25%</td>
<td>25%</td>
<td>25%</td>
<td>13</td>
<td>250%</td>
<td>250%</td>
<td>250%</td>
<td>250%</td>
</tr>
<tr>
<td>2</td>
<td>50%</td>
<td>50%</td>
<td>50%</td>
<td>50%</td>
<td>50%</td>
<td>14</td>
<td>350%</td>
<td>350%</td>
<td>350%</td>
<td>350%</td>
</tr>
<tr>
<td>3</td>
<td>25%</td>
<td>25%</td>
<td>25%</td>
<td>25%</td>
<td>25%</td>
<td>15</td>
<td>300%</td>
<td>300%</td>
<td>300%</td>
<td>300%</td>
</tr>
<tr>
<td>4</td>
<td>100%</td>
<td>100%</td>
<td>100%</td>
<td>100%</td>
<td>100%</td>
<td>16</td>
<td>400%</td>
<td>400%</td>
<td>400%</td>
<td>400%</td>
</tr>
<tr>
<td>5</td>
<td>50%</td>
<td>50%</td>
<td>50%</td>
<td>50%</td>
<td>50%</td>
<td>17</td>
<td>350%</td>
<td>350%</td>
<td>350%</td>
<td>350%</td>
</tr>
<tr>
<td>6</td>
<td>150%</td>
<td>150%</td>
<td>150%</td>
<td>150%</td>
<td>150%</td>
<td>18</td>
<td>450%</td>
<td>450%</td>
<td>450%</td>
<td>450%</td>
</tr>
<tr>
<td>7</td>
<td>100%</td>
<td>100%</td>
<td>100%</td>
<td>100%</td>
<td>100%</td>
<td>19</td>
<td>400%</td>
<td>500%</td>
<td>400%</td>
<td>400%</td>
</tr>
<tr>
<td>8</td>
<td>200%</td>
<td>200%</td>
<td>200%</td>
<td>200%</td>
<td>200%</td>
<td>20</td>
<td>500%</td>
<td>600%</td>
<td>500%</td>
<td>500%</td>
</tr>
<tr>
<td>9</td>
<td>150%</td>
<td>150%</td>
<td>150%</td>
<td>150%</td>
<td>150%</td>
<td>21</td>
<td>450%</td>
<td>700%</td>
<td>450%</td>
<td>450%</td>
</tr>
<tr>
<td>10</td>
<td>250%</td>
<td>250%</td>
<td>250%</td>
<td>250%</td>
<td>250%</td>
<td>22</td>
<td>600%</td>
<td>800%</td>
<td>600%</td>
<td>600%</td>
</tr>
<tr>
<td>11</td>
<td>200%</td>
<td>200%</td>
<td>200%</td>
<td>200%</td>
<td>200%</td>
<td>23</td>
<td>750%</td>
<td>700%</td>
<td>700%</td>
<td>700%</td>
</tr>
<tr>
<td>12</td>
<td>300%</td>
<td>300%</td>
<td>300%</td>
<td>300%</td>
<td>300%</td>
<td></td>
<td></td>
<td></td>
<td>300%</td>
<td>300%</td>
</tr>
</tbody>
</table>
4.4.3. The CEN Short cyclic loading scheme

The cyclic testing schedule for wall 12 was based on the CEN Short cyclic loading scheme (Section 2.5.2). Here, the ductility factor $D = 3$, used for deciding the cyclic amplitude, was cited from the National Building Code of Canada (NRCC 1990). The amplitude, yield slip, loading rate and other relevant parameters for shear wall 12 are listed in Table 4.4. The graph of the amplitude schedules is shown in Figure 2.7 (b).

Table 4.4 The parameters in the cyclic test using CEN Short cyclic test protocol

<table>
<thead>
<tr>
<th>Wall</th>
<th>$P_{\text{max}}$ (kN)</th>
<th>$\Delta_{\text{yield}}$ (mm)</th>
<th>$D$</th>
<th>$v$ (mm)</th>
<th>No. of cycles</th>
<th>Loading rate (mm/sec)</th>
</tr>
</thead>
<tbody>
<tr>
<td>12</td>
<td>125.2</td>
<td>9.5</td>
<td>3</td>
<td>28.5</td>
<td>3</td>
<td>0.4</td>
</tr>
</tbody>
</table>

where

$P_{\text{max}}$: the maximum load carrying capacity of shear wall in monotonic test (wall 8),

$\Delta_{\text{yield}}$: the yield slip, defined as in Figure 2.6,

$v$: the amplitude of cycles, $= D \times \Delta_{\text{yield}}$.

Compared with the amplitudes in the early stage of the tests using the Forintek cyclic protocol, the amplitudes of the cycles in the CEN Short protocol were substantially higher. The basic consideration for this high amplitude was to observe if the wall could recover from heavily stressed cyclic loading to support even a higher load.

4.4.4. A newly proposed cyclic loading scheme

A new cyclic loading scheme was proposed to take into consideration more realistic failure modes, and the high yield slip and avoid the arbitrariness in defining test parameters in the CEN Short cyclic test procedure. This proposed loading scheme was used in shear wall test No. 13. The cyclic loading scheme consisted of two groups of cycles, three identical cycles in each group, and one unidirectional loading (push-over) until the wall failed. In the first cycle group, the amplitude, $v_1$, equaled the displacement at 50% of the maximum load obtained during a monotonic test of wall 8,
and in the second cycle group, the amplitude, \( v_2 \), equaled the displacement at 80% of the maximum load. All the parameters and cyclic amplitude schedule are illustrated in Table 4.5 and Figure 4.4, respectively.

![Figure 4.4 The cyclic schedule for wall 13](image)

Table 4.5 The parameters used in cyclic test of shear wall 13

<table>
<thead>
<tr>
<th>Wall</th>
<th>( P_{\text{max}} ) (kN)</th>
<th>( v_1 = \Delta_0.5P_{\text{max}} ) (mm)</th>
<th>( v_2 = \Delta_0.8P_{\text{max}} ) (mm)</th>
<th>Loading rate (mm/sec)</th>
</tr>
</thead>
<tbody>
<tr>
<td>13</td>
<td>125.2</td>
<td>7.47</td>
<td>18.24</td>
<td>0.4</td>
</tr>
</tbody>
</table>

(Note: One of the bracing pipes of the reaction frame was removed without notice which caused a significant reduction of the frame stiffness. This resulted in a lower amplitude in the first cycle group, which was equal to a displacement at 35% of peak load. The scheduled cycle groups were applied after an adjustment was made. Therefore, the actual number of cycle groups were three, instead of two.)
5. EXPERIMENTAL RESULTS

5.1. Experimental results on wall performance

In compliance with ASTM Standard E 564-76 and the calculation methods used by Forintek Canada Corp. (Karacabeyli 1995), the following data, shown in Table 5.1, were obtained from shear wall tests:

- $P_{\text{max}}$: the maximum load carrying capacity of shear wall, kN.
- $\Delta_{\text{max}}$: total displacement of shear wall at failure when the load dropped to 40 kN in monotonic tests or the maximum displacement among all cycle groups in cyclic tests, measured at the top plate of shear wall, which includes both the shear deflection of the wall and deflections caused by the anchorage system, mm.
- $\Delta_u$: the displacement of shear wall at the maximum load, mm.
- $\Delta_{\text{yield}}$: the yield slip which is the displacement of shear wall at half of the maximum load, mm.
- $S_u$: ultimate shear strength of shear wall, kN/m.

$$S_u = \frac{P_{\text{max}}}{L} \quad (5.1)$$

where: $L = 7.315\text{m}$, length of shear wall measured parallel to the loading direction.

- $G'$: shear stiffness of shear wall, MN/m.

$$G' = \frac{P_{\text{max}}H}{2\Delta_{\text{yield}}L} \quad (5.2)$$

where: $H = 2.438\text{m}$, height of shear wall.

- $D$: ductility factor.

$$D = \frac{\Delta_u}{\Delta_{\text{yield}}} \quad (5.3)$$
Table 5.1 Summary of shear wall test results (refer to Table 4.1 for wall configurations)

<table>
<thead>
<tr>
<th>Wall</th>
<th>$P_{\text{max}}$</th>
<th>$\Delta_{\text{max}}$</th>
<th>$\Delta_u$</th>
<th>$\Delta_{\text{yield}}$</th>
<th>$S_u$</th>
<th>$G'$</th>
<th>$D$</th>
<th>$N$</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>62.77</td>
<td>82</td>
<td>54.04</td>
<td>9.82</td>
<td>8.581</td>
<td>1.07</td>
<td>5.5</td>
<td>424</td>
</tr>
<tr>
<td>4</td>
<td>+59.29, -51.69</td>
<td>79</td>
<td>31.93</td>
<td>12.62</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>424</td>
</tr>
<tr>
<td>2</td>
<td>82.21</td>
<td>46</td>
<td>33.27</td>
<td>5.88</td>
<td>11.239</td>
<td>2.33</td>
<td>5.7</td>
<td>281</td>
</tr>
<tr>
<td>3</td>
<td>+64.07, -59.40</td>
<td>43</td>
<td>15.60</td>
<td>4.51</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>281</td>
</tr>
<tr>
<td>5</td>
<td>71.13</td>
<td>41</td>
<td>21.92</td>
<td>3.54</td>
<td>9.724</td>
<td>3.35</td>
<td>6.2</td>
<td>281</td>
</tr>
<tr>
<td>6</td>
<td>+61.25, -56.69</td>
<td>36</td>
<td>15.78</td>
<td>4.56</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>281</td>
</tr>
<tr>
<td>7</td>
<td>54.73</td>
<td>84</td>
<td>50.97</td>
<td>10.32</td>
<td>7.482</td>
<td>0.88</td>
<td>4.9</td>
<td>375</td>
</tr>
<tr>
<td>8</td>
<td>125.21</td>
<td>51</td>
<td>38.07</td>
<td>7.40</td>
<td>17.117</td>
<td>2.81</td>
<td>5.1</td>
<td>409</td>
</tr>
<tr>
<td>9</td>
<td>+101.54, -99.80</td>
<td>60</td>
<td>21.46</td>
<td>5.45</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>409</td>
</tr>
<tr>
<td>12</td>
<td>+112.94, -109.25</td>
<td>52</td>
<td>23.38</td>
<td>5.00</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>409</td>
</tr>
<tr>
<td>13</td>
<td>+115.33, -110.01</td>
<td>52</td>
<td>31.95</td>
<td>6.16</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>409</td>
</tr>
<tr>
<td>10</td>
<td>84.27</td>
<td>48</td>
<td>33.75</td>
<td>6.50</td>
<td>11.520</td>
<td>2.16</td>
<td>5.2</td>
<td>325</td>
</tr>
<tr>
<td>11</td>
<td>+71.35, -66.24</td>
<td>46</td>
<td>19.83</td>
<td>3.77</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>325</td>
</tr>
</tbody>
</table>

Note: + : extension of the hydraulic cylinder, - : contraction of the hydraulic cylinder.

5.2. Discussion of shear wall tests under monotonic loading conditions

The major parameters for the six shear walls (1, 2, 5, 7, 8 and 10), which were tested under monotonic loading conditions, are listed in Table 5.2.
Table 5.2 Major test results for shear walls under monotonic loading conditions

<table>
<thead>
<tr>
<th>Wall</th>
<th>Major changes from Wall 1</th>
<th>$P_{\text{max}}$</th>
<th>$\Delta_v$</th>
<th>$G'$</th>
<th>$D$</th>
<th>$N$</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>-</td>
<td>62.77</td>
<td>54.04</td>
<td>1.07</td>
<td>5.5</td>
<td>424</td>
</tr>
<tr>
<td>2</td>
<td>Oversize panel</td>
<td>82.21</td>
<td>33.27</td>
<td>2.33</td>
<td>5.7</td>
<td>281</td>
</tr>
<tr>
<td>5</td>
<td>Oversize panel, common nails</td>
<td>71.13</td>
<td>21.92</td>
<td>3.35</td>
<td>6.2</td>
<td>281</td>
</tr>
<tr>
<td>7</td>
<td>Staggered blocking</td>
<td>54.73</td>
<td>50.97</td>
<td>0.88</td>
<td>4.9</td>
<td>375</td>
</tr>
<tr>
<td>8</td>
<td>Oversize panel, perimeter nail reinforced</td>
<td>125.21</td>
<td>38.07</td>
<td>2.81</td>
<td>5.1</td>
<td>409</td>
</tr>
<tr>
<td>10</td>
<td>Oversize panel, corner nail reinforced</td>
<td>84.27</td>
<td>33.75</td>
<td>2.16</td>
<td>5.2</td>
<td>325</td>
</tr>
</tbody>
</table>

Figure 5.1 provides the load-displacement curves for shear walls 1, 2 and 7. The spiral nails in a conventional 152 mm spacing were used in these three walls as the connectors between panel and frame. The differences were that wall 2 was sheathed with oversize panel and walls 1 and 7 were sheathed with regular panels. Continuous blocking was installed in wall 1, while staggered blocking was installed in wall 7.
The test results show that wall 2 (oversize panel) had higher static load capacity (31% increase compared to wall 1 and 50% for wall 7), higher stiffness (118% increase from wall 1 and 165% from wall 7), higher ductility (4% increase from wall 1 and 16% from wall 7), and smaller deflection (45% decrease in both walls) compared to walls 1 and 7 (regular panels). All these improvements were achieved even though the number of nail connectors between panel and frame was reduced by 34% from 424 nails to 281 nails when an oversize panel was used. This is an indication that the panel shear rigidity significantly contributes to the overall wall stiffness. Furthermore, it would be interesting to observe the performance of the shear wall with oversize panel if the same number of nails as in regular shear walls were used. In the tested walls, the edges of sheathing panels were damaged severely by nail crushing, embedding or pulling through (Photo 5.1). This was the critical failure mode of the shear walls which was to be expected since the shear force was distributed throughout the panel, while discontinuities at panel edges formed the weak link. A logical improvement was thus to strengthen the nail connections around the perimeter of the walls.

Photo 5.1 The torn edge of regular panel at the mid height of shear wall
Figure 5.2 shows the test results of the nail reinforcement study. Wall 8 with a reduced nail spacing of 76 mm around the perimeter of the wall had a significantly higher strength (52%) and stiffness (21%) compared to wall 2 which had regular 152 mm nail spacing. Its ductility is also comparable to the other walls. An important fact is that the total of number of nails used in wall 8 was still less than that used in the conventional wall assembly such as wall 1 (409 nails vs. 424 nails), with its load capacity and stiffness doubled (Table 5.2). Wall 10, reinforced only at four 0.8 × 0.8 m corners by using 76 mm nail spacing (adding 11 more nails), did not present any significant variation in both load capacity and stiffness, compared to wall 2.

Figure 5.2 Load-displacement curves for shear walls 2, 8 and 10
In the monotonic tests of walls 1 and 7 (regular panels), connection failures were first observed at the edges of panels along the blocking. Initially the failures were observed in the form of nail withdrawal from blocking and framing members. As the test progressed, failures in the form of nail heads embedding into and pulling through the panels were observed. In the pull-through failure mode, nails were bent in one direction, like a "J" (Photo 5.2 (b)), and in withdrawal failure mode, nails were still connected to both panel and frame and deformed into an "S" shape (Photo 5.2 (a)). The dominant failure mode was nail pull-through (up to 75%). Approximately 3/4 of these nail pull-through failures occurred at the lower edge of upper panels along blocking in the loaded end, and the other 1/4 occurred at the upper edge of the lower panels along the blocking in the other end. This phenomenon can be explained by the fact that two small panels were attached on the upper row of panels which resulted in high discontinuity and reduced stiffness as compared to the lower row of panels. This unsymmetrical arrangement may further cause a higher bending moment in the stud at the top ends of the loading side leading to a larger deflection. It was observed that under lateral loading, the end studs tended to deform into a shape like a loaded cantilever beam (Photo 5.3) and the interior studs into a "S" shape (Photo 5.4 (a)). The stud deformation was more severe in the wall with regular panels than in the wall with oversize panel (Photo 5.4 (b)), due to the panel discontinuity. In the monotonic tests of walls sheathed with over-sized panels, connection failures always initiated along the bottom edge of the
wall in the form of nail withdrawal from the framing members and progressed upward towards the mid-height of the wall. Among failed nails, the dominant failure mode was nail pull-through as before (Photo 5.2 (b)) while a small proportion of connector failures was nail withdrawal (Photo 5.2 (a)). Clearly, it was the pull-through mode that dominated and dictated the shear wall strength. More appropriate methods should be investigated to prevent this nail pull-through from occurring. An increase of the panel thickness would be one option, although it might not be the most efficient, because the currently used panel still has a big margin in its shear resistance and a thicker panel might not necessarily have a much higher pull-through resistance. In all tests, for both regular and oversize panels, no buckling was detected.

Photo 5.3 The deformation of end studs
The shear resistance of walls built with regular panels was affected by the type of blocking: wall 7 showed a 13% decrease in its load capacity and a 18% decrease in its stiffness when compared to wall 1 (Figure 5.1 and Table 5.2). This was to be expected since the discontinuity along the panel edges at the mid height of the wall was more severe in the staggered blocking case compared to the continuous blocking case. As most failures initiated at mid height of walls 1 and 7, it is clear that the discontinuities were critical to the wall performance. As an extreme case where the discontinuities were completely removed (wall 2), the improvement in wall performance was even more significant. One can further assume that the lateral load capacity and stiffness of conventional shear walls would be further reduced if blocking was not installed.
In the test to determine the effect of nail types, common nails were used as the connectors between panel and frame in wall 5, with the other parameters identical to shear wall 2. As seen in Figure 5.3, wall 5 had a slightly higher stiffness and a lower strength than wall 2 which implied that the common nail connections were slightly stiffer but weaker in holding wood members than the spiral nail connections. An interesting phenomenon in wall 5 was the occurrence of a saw-tooth shaped pattern in the descending branch of the load displacement curve. This is a complicated load-slip mechanism involving frictional forces between the nails and the frame members, large deformation kinematics, embedment of the nail shank into the frame and embedment of nail head into the panel.

![Figure 5.3 Load-displacement curves for shear walls 2 and 5](image)

5.3. Discussions of the shear wall tests under cyclic loading conditions

Seven tests were carried out to study the influence of panel dimension (wall 3-oversize panel vs. wall 4-regular panels), nail reinforcement (wall 9-perimeter nail reinforced and wall 11-corner nail reinforced vs. wall 3-regular nail spacing), nail type (wall 6-common nails vs. wall 3-spiral nails)
and cyclic test protocols (wall 12-CEN Short protocol and wall 13-new protocol vs. wall 9-FCC protocol) on the shear wall performance under cyclic loading conditions. The major parameters for these shear walls are listed in Table 5.3. Their hysteresis loops in cyclic tests with corresponding monotonic envelope curves are also shown in Figure 5.4 to Figure 5.7, Figure 5.9, Figure 5.12 and Figure 5.13.

Table 5.3 Major test results for shear walls under cyclic loading conditions

<table>
<thead>
<tr>
<th>Wall</th>
<th>Major changes from wall 3</th>
<th>$P_{\text{max}}$</th>
<th>$\Delta_{\text{max}}$</th>
<th>$\Delta_\text{u}$</th>
<th>$N$</th>
</tr>
</thead>
<tbody>
<tr>
<td>3</td>
<td></td>
<td>+64.07, -59.40</td>
<td>43</td>
<td>15.60</td>
<td>281</td>
</tr>
<tr>
<td>4</td>
<td>Regular panel</td>
<td>+59.29, -51.69</td>
<td>79</td>
<td>31.93</td>
<td>424</td>
</tr>
<tr>
<td>6</td>
<td>Common nails</td>
<td>+61.25, -56.69</td>
<td>36</td>
<td>15.78</td>
<td>281</td>
</tr>
<tr>
<td>9</td>
<td>Perimeter nail reinforced</td>
<td>+101.54, -99.80</td>
<td>60</td>
<td>21.46</td>
<td>409</td>
</tr>
<tr>
<td>11</td>
<td>Corner nail reinforced</td>
<td>+71.35, -66.24</td>
<td>46</td>
<td>19.83</td>
<td>325</td>
</tr>
<tr>
<td>12</td>
<td>CEN Short protocol</td>
<td>+112.94, 109.25</td>
<td>52</td>
<td>23.38</td>
<td>409</td>
</tr>
<tr>
<td>13</td>
<td>New protocol</td>
<td>+115.33, -110.01</td>
<td>52</td>
<td>31.95</td>
<td>409</td>
</tr>
</tbody>
</table>

5.3.1. Performance of shear walls with different configurations

The results illustrate that wall 3 (oversize panel, shown in Figure 5.4) had an 11% increase in load carrying capacity and the same decrease in deflection (45%) as in the monotonic tests compared to wall 4 (regular panels, shown in Figure 5.5) with 143 or 34% less nails. After nail reinforcement, wall 9 (perimeter reinforced, Figure 5.6) and wall 11 (corner reinforced, Figure 5.7) provided approximate by a 60% and 10% increase in the maximum load capacity compared to wall 3 (regular nail spacing), respectively. As in the monotonic tests, meaningful comparisons between wall 9 (409 nails) and wall 4 (424 nails) can be made as the number of nails in both cases was similar. Wall 9 had a 82% increase in its load carrying capacity compared to wall 4. The test results also show that wall 9 had the smallest strength degradation (<2%) at both compression and tensile sides within a cycle (Figure 5.6), while in all other walls this degradation could be as much as 8%.
Figure 5.4 Cyclic load-displacement curve and static envelope for wall 3

Figure 5.5 Cyclic load-displacement curve and static envelope for wall 4
Figure 5.6 Cyclic load-displacement curve and static envelope for wall 9

Figure 5.7 Cyclic load-displacement curve and static envelope for wall 11
A further investigation is necessary to find the optimal nail reinforcement pattern. From previous tests, it was observed that nails had the maximum deformation along the edges of the panels, while in the central area of the panel, nails were essentially intact (Figure 5.8). It seems that in future studies the nail spacing for interior connection between panel and studs could be larger and unevenly arranged, and higher nail density around the perimeter of panels, or at wall corners may be considered.

![Figure 5.8 The distributions of failed nails along the height of wall](image)

(a) regular panels (wall 4)  (b) oversize panel (wall 6)

Note that an error in the actuator control system caused an instant pull-back of the hydraulic cylinder causing a peak load of up to 70kN, applied to wall 11 prior to the planned cyclic test. A degradation of wall stiffness can be observed in the cyclic test when the wall was subjected to a loading in the pull-back direction (lower left part in Figure 5.7). It seemed that this impact load did not reduce the load capacity and stiffness of the wall in the push-forward loading condition (upper right part in Figure 5.7).

Common nails were used as the connectors between panel and frame in wall 6, shown in Figure 5.9, under cyclic loading conditions. When compared to wall 3, wall 6 had a slightly lower
peak load. The test result also showed that the peak load in wall 6 was reached two cycle groups earlier than that in wall 3, indicating a weaker reversed bending resistance of common nails.

Figure 5.9 Cyclic load-displacement curve and static envelope for wall 6

To determine the reason why the failure occurred mostly along the bottom edge of the wall, the horizontal joint deformations along the top and bottom edges were measured. The measurements were taken from all the cycles in walls 12 and 13. The difference of joint deformations between the top and bottom edges in the first cycle for two walls are illustrated in Figure 5.10 and Figure 5.11, respectively. The figures and Photo 5.5 showed clearly that the joints located on the bottom edge deformed more than that on the top edge and this difference occurred at the very early stage of the tests, which implies that the top joints had higher stiffness than the bottom joints. The difference in stiffness in joints may be caused by the structural unsymmetry (double top plates and single bottom plate) so that the forces were unevenly distributed along the height of the studs.
Figure 5.10 The comparison of joints deflection in wall 12

Figure 5.11 The comparison of joints deflection in wall 13
Photo 5.5 Larger movement in bottom joints
Under cyclic load, the predominant failure mode was different from the monotonic load cases, although failures occurred at the same locations. Instead of nail withdrawals, most connectors failed in low cycle fatigue fractures (Photo 5.6 (a) for spiral nails and (b) for common nails, and Photo 5.7) due to reversed bending. For example, along the bottom edge of walls 4 (regular panel) and 6 (over-sized panel), 100% and 84% of the nails failed in fatigue fracture, respectively. The fatigued nails, however, were observed as early as the eleventh cycle group or the 33rd cycle. This failure mode was not observed in buildings subjected to earthquakes or during previous dynamic tests on shear walls. It was felt that the failure mode was influenced by the loading history, more appropriate cyclic test protocols were adopted (Section 5.3.2) for subsequent tests.
5.3.2. Comparison among cyclic loading schedules

Walls 12 and 13 were tested under CEN Short protocol and a newly proposed protocol, respectively. The cyclic test results of shear walls 12 and 13 as well as wall 9 (Forintek protocol) are shown in Figure 5.6, Figure 5.12 and Figure 5.13.
Figure 5.12 Cyclic load-displacement curve and static envelope for wall 12

Figure 5.13 Cyclic load-displacement curve and static envelope for wall 13
There was a noticeable sequential increase in load carrying capacity from walls 9 to 12 to 13. Wall 9 reached to its maximum load at 102 kN until nails started to fail in low cycle fatigue fracture and then followed a rapid decrease in its load capacity from the peak load to a very low level. In the CEN Short test of wall 12, due to a high initial cycle amplitude, which equaled the displacement at 95% of peak load in wall 8, the wall reached its maximum load capacity (113 kN) in the first cycle and could never regain this level during the subsequent push-over stage. This indicated that unrecoverable damage occurred in nail joints at the very beginning of the test so that the effect of the whole schedule cannot be quantified. It was observed, however, that the nails did not fail in fatigue (Photo 5.8). In the new cyclic test schedule for wall 13, three cycle groups (see Section 4.4.4) with lowered amplitudes at $\Delta_{0.35P_{\text{max}}}$, $\Delta_{0.5P_{\text{max}}}$ and $\Delta_{0.8P_{\text{max}}}$ levels were applied first (the maximum load was 107 kN). Wall 13 was then pushed over, reaching a maximum lateral resistance level of 115 kN. These results show that the wall load carrying capacity was significantly dependent on the cyclic loading protocol. Furthermore, nail fatigue failures were not observed using the new proposal cyclic test sequence (Photo 5.8). Therefore it can be concluded that this new protocol provided a better loading history to overcome the weakness in the other two protocols. In both tests of walls 12 and 13, the nail failure mode and location were similar to that in walls under monotonic loading as shown in Photo 5.2.
There is a necessity to evaluate the degradation in ultimate lateral resistance of shear wall under cyclic loading. The degradation is measured by a percentage drop in load from the first cycle to subsequent cycles. Table 5.4 lists the degradation values in walls 12 and 13.

Table 5.4 Degradation of load capacity occurring in cycles

<table>
<thead>
<tr>
<th>Wall</th>
<th>Cycle group</th>
<th>Loading</th>
<th>Degradation (%)</th>
<th>Degradation (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td>from 1st to 2nd cycle</td>
<td>from 2nd to 3rd cycle</td>
</tr>
<tr>
<td>12</td>
<td>1</td>
<td>+</td>
<td>12.9</td>
<td>6.9</td>
</tr>
<tr>
<td></td>
<td></td>
<td>-</td>
<td>11.2</td>
<td>6.9</td>
</tr>
<tr>
<td>13</td>
<td>1</td>
<td>+</td>
<td>4.5</td>
<td>1.0</td>
</tr>
<tr>
<td></td>
<td></td>
<td>-</td>
<td>6.5</td>
<td>0.0</td>
</tr>
<tr>
<td>2</td>
<td></td>
<td>+</td>
<td>8.2</td>
<td>4.9</td>
</tr>
<tr>
<td></td>
<td></td>
<td>-</td>
<td>3.3</td>
<td>3.4</td>
</tr>
<tr>
<td>3</td>
<td></td>
<td>+</td>
<td>13.1</td>
<td>6.0</td>
</tr>
<tr>
<td></td>
<td></td>
<td>-</td>
<td>10.0</td>
<td>8.1</td>
</tr>
</tbody>
</table>
The maximum strength impairment occurred between the first and second cycles. A reduced strength impairment occurred between the second and third cycles. This trend agrees with results reported by Daneff et al. (1996) that continuous degradation after the third cycle was not observed. This implies that the degradation in shear wall strength is mainly concentrated in the first three cycles and three identical cycles within a cycle group were sufficient. The maximum strength deterioration observed in two walls was less than 14%. The stiffness degraded in the same way as the strength.

In wall 13, the degradation of strength and stiffness, in the first cycle group with small displacements, was not significant due to the elastic behaviour of the shear wall, but it progressively increased with the increase of shear wall displacement. The strength drop on the positive loading side (+) was slightly larger than on the negative loading side (-) in the second cycle, since the degradation occurred within the first cycle. The same trends were also observed in other cyclic tests.

5.3.3. Energy Dissipation

Under cyclic loading, energy dissipation occurred through internal friction, yielding of the nails and non-recoverable deformation (damage) in the wall assembly. A measure of this energy dissipation was obtained by evaluating the area of the hysteresis loops from the load-displacement curves for each load cycle. For walls 3, 4, 6, 9 and 11, the dissipated energy within each cycle group ($U_i$) and the total dissipated energy at the end of a cycle group ($U_T$) are shown in Table 5.5. Figure 5.14 to Figure 5.15 also graphically illustrate the energy dissipation characteristics of the walls. Since slight differences exist in loading schedule, the results are only comparable up to cycle 18. It is clear that more energy dissipation occurred in wall 4 (regular panel) compared to walls 3, 6, 9 and 11 (over-sized panel) due to its larger displacement. The maximum amount of energy dissipated within a cycle group occurred in three high amplitude cycle groups 10, 12 or 14 in all five cases, after which nail fatigue developed progressively. The total dissipated energy by wall 11 was fairly close to that by wall 3, although its peak energy dissipation occurred earlier. Wall 9 had the highest energy
dissipation among all walls sheathed with over-sized panels. The difference of this high energy
dissipation in wall 9 from wall 4 is that wall 9 can support much heavier lateral load at a relative
small deformation while wall 4 went to a larger deflection under a lighter lateral load. Figure 5.16
shows the normalized dissipated energy within a cycle group as a percentage of the peak dissipated
energy in cycle group 14 for walls 3, 4 and 6, in cycle group 10 for wall 9 and in cycle group 12 for
wall 11. It can be observed that the normalized dissipated energy of all five walls was similar until
cycle group 12 after which the normalized dissipated energy significantly reduced in wall 9.

Table 5.5 Dissipated energy in shear wall during cyclic loading.

| Cycle Group No. | Wall 3 | | | Wall 4 | | | Wall 6 | | | Wall 9 | | | Wall 11 |
|-----------------|-------|---|---|-------|---|---|-------|---|---|-------|---|---|-------|---|
| No. i           | U_i  | U_Ti | N m | U_i  | U_Ti | N m | U_i  | U_Ti | N m | U_i  | U_Ti | N m | U_i  | U_Ti | N m |
| 1               | 19    | 19   | N m | 70   | 70   | N m | 17   | 17   | N m | 66   | 66   | N m | 27   | 27   | N m |
| 2               | 99    | 118  | N m | 248  | 318  | N m | 103  | 120  | N m | 308  | 375  | N m | 149  | 175  | N m |
| 3               | 19    | 137  | N m | 60   | 378  | N m | 20   | 140  | N m | 60   | 435  | N m | 32   | 207  | N m |
| 4               | 394   | 531  | N m | 755  | 1133 | N m | 430  | 570  | N m | 1134 | 1568 | N m | 468  | 676  | N m |
| 5               | 96    | 627  | N m | 181  | 1314 | N m | 95   | 665  | N m | 272  | 1840 | N m | 130  | 805  | N m |
| 6               | 741   | 1368 | N m | 1267 | 2581 | N m | 816  | 1481 | N m | 2108 | 3948 | N m | 954  | 1759 | N m |
| 7               | 291   | 1659 | N m | 451  | 3032 | N m | 375  | 1856 | N m | 849  | 4797 | N m | 374  | 2134 | N m |
| 8               | 1138  | 2797 | N m | 1845 | 4877 | N m | 1262 | 3118 | N m | 3078 | 7875 | N m | 1493 | 3627 | N m |
| 9               | 551   | 3348 | N m | 824  | 5701 | N m | 666  | 3784 | N m | 1545 | 9420 | N m | 765  | 4392 | N m |
| 10              | 1516  | 4864 | N m | 2475 | 8176 | N m | 1713 | 5497 | N m | 4611 | 13580| N m | 2243 | 6635 | N m |
| 11              | 837   | 5701 | N m | 1227 | 9403 | N m | 984  | 6481 | N m | 2275 | 15855| N m | 1215 | 7850 | N m |
| 12              | 1881  | 7582 | N m | 2985 | 12388| N m | 1923 | 8404 | N m | 4029 | 19884| N m | 2345 | 10195| N m |
| 13              | 1103  | 8685 | N m | 1662 | 14050| N m | 1073 | 9477 | N m | 1004 | 20889| N m | 556  | 10751| N m |
| 14              | 1919  | 10604| N m | 3339 | 17389| N m | 1920 | 11397| N m | 1140 | 22028| N m | 955  | 11706| N m |
| 15              | 778   | 11382| N m | 1977 | 19366| N m | 1100 | 12497| N m | 694  | 22722| N m | 490  | 12196| N m |
| 16              | 1033  | 12415| N m | 2900 | 22266| N m | 1817 | 14314| N m | 700  | 23423| N m | 751  | 12946| N m |
| 17              | 531   | 12946| N m | 1950 | 24216| N m | 893  | 15207| N m | 555  | 23978 |N m | 447  | 13394| N m |
| 18              | 765   | 13711| N m | 1837 | 26053| N m | 1260 | 16467| N m | 168  | 24146 |N m | 728  | 14122| N m |
| 19              | 429   | 14140| N m | 2415 | 28468| N m | 765  | 17232| N m | 833  | 25029 |N m | 472  | 14593| N m |
| 20              | 618   | 14758| N m | 2459 | 30927| N m | 1034 | 18266| N m | 1054 | 26083 |N m | 693  | 15287| N m |
| 21              | 392   | 15150| N m | 2247 | 33174| N m | 688  | 18954| N m | 592  | 26675 |N m | 466  | 15753| N m |
| 22              | 572   | 15722| N m | 2898 | 36072| N m | 1075 | 20029| N m | 873  | 27548 |N m | 752  | 16505| N m |
| 23              | 536   | 16258| N m | 834  | 20863| N m | | | | | | | | | |
Figure 5.14 Energy dissipation characteristics ($U_j$) within each cycle group

(a) $U_j$ in walls 3, 4 and 6

(b) $U_j$ in walls 9 and 11
(a) $U_{TI}$ in walls 3, 4 and 6

(b) $U_{TI}$ in walls 9 and 11

Figure 5.15 Cumulative energy dissipation ($U_{TI}$)
The energy dissipation in each cycle or cycle group and the cumulative energy dissipation in walls 12 and 13 are shown in Table 5.6, Figure 5.17 and Figure 5.18, respectively. Within a cycle group, the maximum energy absorption always happened during the first cycle.

Figure 5.16 Normalized dissipated energy within a cycle group
Table 5.6 Energy dissipation in shear walls 12 and 13 under cyclic loading

<table>
<thead>
<tr>
<th>Cycle</th>
<th>Wall 12</th>
<th></th>
<th>Wall 13</th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>$U_i$ (Nm)</td>
<td>$U_{Ti}$ (Nm)</td>
<td>$U_i$ (Nm)</td>
<td>$U_{Ti}$ (Nm)</td>
</tr>
<tr>
<td>1</td>
<td>3219</td>
<td>3219</td>
<td>160</td>
<td>160</td>
</tr>
<tr>
<td>2</td>
<td>2043</td>
<td>5262</td>
<td>128</td>
<td>288</td>
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<tr>
<td>3</td>
<td>1711</td>
<td>6973</td>
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<td>412</td>
</tr>
<tr>
<td>4</td>
<td></td>
<td>560</td>
<td></td>
<td>972</td>
</tr>
<tr>
<td>5</td>
<td></td>
<td>453</td>
<td></td>
<td>1424</td>
</tr>
<tr>
<td>6</td>
<td></td>
<td>398</td>
<td></td>
<td>1823</td>
</tr>
<tr>
<td>7</td>
<td></td>
<td>2131</td>
<td></td>
<td>3954</td>
</tr>
<tr>
<td>8</td>
<td></td>
<td>1423</td>
<td></td>
<td>5377</td>
</tr>
<tr>
<td>9</td>
<td></td>
<td>1203</td>
<td></td>
<td>6581</td>
</tr>
</tbody>
</table>

Figure 5.17 Energy dissipation characteristics ($U_i$) within each cycle group
In general, by examining all the experimental results, it can be concluded that (1) the load-displacement curves were highly nonlinear and did not show a distinct elastic part and yield point, (2) the ductility ratios obtained from all monotonic tests did not depend on panel dimension, (3) in cyclic tests, the envelope curves and the first cycle loops in each cycle group were very close to each other until the ultimate lateral resistance of a shear wall was reached, and (4) the hysteretic loops showed a good symmetric shapes on both sides.

Figure 5.18 Cumulative energy dissipation ($U_T$)
6. THE STATIC NAIL TESTS

6.1. Description of static nail tests

Thirteen groups of nail tests under static loading conditions were conducted by Sieber as part of this project to investigate the performance of nails as the connectors in lumber-panel construction and to provide valid nail load-slip relationships required by the model in the finite element program DAP. A detailed description of the tests and data are contained in a report by Sieber et al (1997). The effect of several variables such as nail type, number of nails, nail spacing, lumber orientation and seasoning were studied. Each specimen was constructed with a panel member and a lumber member, using the same materials as in the shear wall experiments. The grain direction of the lumber member was oriented parallel or perpendicular to the applied load. The two members of a specimen were connected by 50 mm spiral nails (Gun driven) or common nails (Hand driven). All specimens were conditioned at 20°C and 65% relative humidity for over 14 days, except for groups 7, 8 and 13 which were freshly nailed, as contrasting groups. The target moisture content in specimens was 12% and the average specific gravity of the lumber used in specimens was approximately 0.42. Groups 1 to 8 consisted of specimens built with one or three (spaced at 50 or 150 mm) spiral nails. Groups 9 to 13 consisted of specimens built with one or three common nails. The tests were conducted on a MTS 800-20 universal testing machine. The lumber located at the lower end of the specimen was fixed to the MTS table, while the panel located at the upper end of the specimen was connected to the MTS loading unit. A unidirectional tensile load was applied to the specimen to cause a shear deformation similar to the situation in a shear wall. The loading rate was 2.54 mm/min for all of the tests. The displacements between panel and lumber were measured by a linear voltage displacement transducer (LVDT) mounted on the specimen.

Table 6.1 lists the configurations of testing specimens. Figure 6.1 shows the test setup.
Table 6.1 The configurations of specimens

<table>
<thead>
<tr>
<th>Group No.</th>
<th>Replication</th>
<th>Orientation of lumber to load</th>
<th>Nail type (50mm)</th>
<th>Number of nails</th>
<th>Nail spacing (mm)</th>
<th>Conditioning time (day)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>16</td>
<td>parallel</td>
<td>spiral</td>
<td>1</td>
<td>-</td>
<td>14</td>
</tr>
<tr>
<td>2</td>
<td>16</td>
<td>perpendicular</td>
<td>spiral</td>
<td>1</td>
<td>-</td>
<td>14</td>
</tr>
<tr>
<td>3</td>
<td>16</td>
<td>parallel</td>
<td>spiral</td>
<td>3</td>
<td>50</td>
<td>14</td>
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<tr>
<td>4</td>
<td>16</td>
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<td>parallel</td>
<td>spiral</td>
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<td>150</td>
<td>14</td>
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<td>spiral</td>
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<td>150</td>
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<tr>
<td>7</td>
<td>16</td>
<td>parallel</td>
<td>spiral</td>
<td>1</td>
<td>-</td>
<td>0 - 1</td>
</tr>
<tr>
<td>8</td>
<td>16</td>
<td>perpendicular</td>
<td>spiral</td>
<td>1</td>
<td>-</td>
<td>0 - 1</td>
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<tr>
<td>9</td>
<td>16</td>
<td>parallel</td>
<td>common</td>
<td>1</td>
<td>-</td>
<td>14</td>
</tr>
<tr>
<td>10</td>
<td>16</td>
<td>perpendicular</td>
<td>common</td>
<td>1</td>
<td>-</td>
<td>14</td>
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<tr>
<td>11</td>
<td>5</td>
<td>parallel</td>
<td>common</td>
<td>3</td>
<td>150</td>
<td>14</td>
</tr>
<tr>
<td>12</td>
<td>5</td>
<td>perpendicular</td>
<td>common</td>
<td>3</td>
<td>150</td>
<td>14</td>
</tr>
<tr>
<td>13</td>
<td>3</td>
<td>parallel</td>
<td>common</td>
<td>1</td>
<td>-</td>
<td>0 - 1</td>
</tr>
</tbody>
</table>

MTS loading unit

9.5mm OSB Panel

50mm spiral or common nails

38x89 Lumber

LVDT

MTS Table

Board for one-nail specimen

Board for three-nail specimen

(a) lumber oriented parallel to loading direction
6.2. Test results

The static nail test results are summarized in Table 6.2. The definitions used in the table are as follows:

- $P_{\text{max}}$: maximum load per nail within a group,
- $\Delta_{\text{max}}$: averaged relative displacement between panel and plate at $P_{\text{max}}$ within a group,
- $K$: averaged initial stiffness within a group, obtained from regression analysis by fitting load-slip curve shown in Equation (3.32) to test data,
- $D$: averaged ductility factor within a group, defined as $\Delta_{\text{max}} / \Delta_{\text{yield}}$,
- $\Delta_{\text{yield}}$: the displacement at $0.5P_{\text{max}}$.

Figure 6.2 and Figure 6.3 list the combined and averaged load-slip curves from four major specimen groups to show the effects of nail type and orientation on the peak load and stiffness.
(i) Load parallel to lumber grain (Group 1)  (ii) Load perpendicular to lumber grain (Group 2)

(a) Individual load-displacement curves

(i) Load parallel to lumber grain (Group 1)  (ii) Load perpendicular to lumber grain (Group 2)

(b) Averaged load-displacement curves

Figure 6.2 Spiral nail joints, one nail

(i) Load parallel to lumber grain (Group 11)  (ii) Load perpendicular to lumber grain (Group 12)

(a) Individual load-displacement curves
(i) Load parallel to lumber grain (Group 11)  
(ii) Load perpendicular to lumber grain (Group 12)

(b) Averaged load-displacement curves

Figure 6.3 Common nail joints, one nail

Table 6.2 Summary of results in static nail tests

<table>
<thead>
<tr>
<th>Group No.</th>
<th>$P_{\text{max}}$ (N)</th>
<th>$\Delta_{\text{max}}$ (mm)</th>
<th>$K$ (kN/m)</th>
<th>$D$</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>ave.</td>
<td>cov (%)</td>
<td>ave.</td>
<td>K</td>
</tr>
<tr>
<td>1</td>
<td>1118</td>
<td>15.16</td>
<td>13.3</td>
<td>518.5</td>
</tr>
<tr>
<td>2</td>
<td>1162</td>
<td>13.38</td>
<td>11.8</td>
<td>536.1</td>
</tr>
<tr>
<td>3</td>
<td>1128</td>
<td>3.10</td>
<td>16.1</td>
<td>567.2</td>
</tr>
<tr>
<td>4</td>
<td>1144</td>
<td>3.39</td>
<td>12.6</td>
<td>864.4</td>
</tr>
<tr>
<td>5</td>
<td>1188</td>
<td>3.71</td>
<td>14.9</td>
<td>610.6</td>
</tr>
<tr>
<td>6</td>
<td>1049</td>
<td>2.99</td>
<td>12.1</td>
<td>774.3</td>
</tr>
<tr>
<td>7</td>
<td>1118</td>
<td>15.58</td>
<td>14.7</td>
<td>667.3</td>
</tr>
<tr>
<td>8</td>
<td>1159</td>
<td>10.26</td>
<td>11.8</td>
<td>544.9</td>
</tr>
<tr>
<td>9</td>
<td>1166</td>
<td>13.33</td>
<td>12.9</td>
<td>1206.1</td>
</tr>
<tr>
<td>10</td>
<td>1105</td>
<td>11.42</td>
<td>8.8</td>
<td>1559.0</td>
</tr>
<tr>
<td>11</td>
<td>1150</td>
<td>1.98</td>
<td>11.6</td>
<td>1686.9</td>
</tr>
<tr>
<td>12</td>
<td>1308</td>
<td>4.23</td>
<td>10.1</td>
<td>2198.7</td>
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<tr>
<td>13</td>
<td>1222</td>
<td>7.58</td>
<td>6.3</td>
<td>1610.0</td>
</tr>
</tbody>
</table>

6.2.1. The types of nails

Test results show that the specimens built with spiral nails and common nails do not have a significant difference in maximum load capacity in both loading directions. For example, the average maximum loads in the specimens built with spiral nails in both parallel and perpendicular loading directions were 1118 N and 1162 N, respectively, while for specimens built with common nails, these values were 1166 N and 1104 N, respectively. The common nails, however, provided the
specimens with much higher initial stiffness than spiral nails in both loading directions. For example, the stiffness values of common nail joints was 1206 kN/m loaded parallel to lumber grain and 1559 kN/m loaded perpendicular to lumber grain, comparing to 518 kN/m and 536 kN/m, respectively, in spiral nail joints. Furthermore, common nail joints showed a higher ductility factor than spiral nail joints.

6.2.2. The lumber orientation

In comparing the peak loads in joints with the lumber oriented perpendicular to applied load to the joints with the lumber oriented parallel to applied load, spiral nail joints built with one nail and three nails (50mm spacing) showed a somewhat higher peak load (about 4%), while those built with three nails (150 mm spacing) showed a lower peak load (-13%). In common nail joints this trend was reversed. In general, both spiral and common nail joints with the lumber oriented perpendicular to applied load have an initial stiffness about 3% to 30% higher when compared to parallel to grain loading. Therefore, it may concluded that the lumber orientation with respect to the applied load does not affect the load carrying capacity of specimens significantly, but affects their stiffness.

6.2.3. Nail spacing and number of nails

Two nail spacings were used in spiral nail tests: 50 and 150 mm. The nail spacing was not studied in common nail joints. The variation in nail spacing when the specimens were loaded in both directions did not cause any significant difference in load capacity and stiffness per nail of specimens (the difference were all less than 10%). Both spiral nail joints and common nail joints built with 1 or 3 nails showed very little difference in peak load and stiffness per nail. This implies that for the tested specimens, the group effect was negligible.

6.2.4. Conditioning duration

In contrast with the specimens which were kept in a conditioning chamber for over 14 days, some groups of freshly nailed specimens (group 7, 8 and 13) were tested. It can be noted that the specimens freshly built with spiral nails showed a slightly higher stiffness than the aged specimens,
but this difference was not significant from a statistical point of view and did not show any difference in the peak load. The specimens freshly built with common nails, when loaded parallel to lumber grain, provided higher initial stiffness and peak load than the aged specimens. No specimens loaded perpendicular to lumber grain were tested. This result indicates that the relaxation of wood fibers may not affect spiral nails as much as common nails which rely more on the fiber damping force for their withdrawal resistance.

6.2.5. Failure modes

Generally the nail joints failed in a mixed failure mode, either by nail withdrawal from the lumber or nail pulling through the panel. The wood density and local mechanical properties of panel affect the failure modes. Specimens with lower wood density were more likely to fail in withdrawal. With increasing wood density, the occurrence of pull through failure increased. This tendency was stronger for common nails and freshly nailed spiral nails than for cured spiral nails.

6.3. The parameters for the panel-frame nail connection and numerical examples

By fitting load-slip curves from the nail tests using regression analysis, two groups of parameters for either spiral nail or common nail connections between panel and frame were obtained, as shown in Table 6.3. The definitions of the parameters refer to Equation (3.32). The curves drawn from these parameters combined with averaged nail test data are shown in Figure 6.4.

<table>
<thead>
<tr>
<th>Nail</th>
<th>Loading direction to lumber grain</th>
<th>$P_0$ (N)</th>
<th>$P_1$ (kN/m)</th>
<th>$K$ (kN/m)</th>
<th>$E_0$ (m)</th>
<th>$m_E$ (kN/m)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Spiral</td>
<td>parallel</td>
<td>926</td>
<td>13</td>
<td>579</td>
<td>0.0130</td>
<td>18</td>
</tr>
<tr>
<td></td>
<td>perpendicular</td>
<td>751</td>
<td>34</td>
<td>561</td>
<td>0.0125</td>
<td>44</td>
</tr>
<tr>
<td>Common</td>
<td>parallel</td>
<td>918</td>
<td>20</td>
<td>1206</td>
<td>0.0120</td>
<td>20</td>
</tr>
<tr>
<td></td>
<td>perpendicular</td>
<td>853</td>
<td>27</td>
<td>1559</td>
<td>0.0080</td>
<td>17</td>
</tr>
</tbody>
</table>

Table 6.3 The parameters for both spiral nail and common nail
(i) Load parallel to lumber grain  
(ii) Load perpendicular to lumber grain  

(a) Spiral nail joints, one nail

(i) Load parallel to lumber grain  
(ii) Load perpendicular to lumber grain  

(b) Common nail joints, one nail

Figure 6.4 Predicted nail load-slip curves and averaged nail test data

The numerical examples from DAP for walls 1, 2, 5, 8 and 10 with the experimental curves are shown in Figure 6.5. The comparison in the peak load for those walls is listed in Table 6.4.
(a) Shear wall 1 - regular panel, spiral nails with 152 mm perimeter spacing

(b) Shear wall 2 - oversize panel, spiral nails with 152 mm perimeter spacing

Figure 6.5 Comparison between test results and numerical results
(c) Shear wall 5 - oversize panel, common nails with 152 mm perimeter spacing

(d) Shear wall 8 - oversize panel, spiral nails with 76 mm perimeter spacing

Figure 6.5 Comparison between test results and numerical results (Cont’d)
(e) Shear wall 10 - oversize panel, spiral nails with 76 mm spacing at four corners

Figure 6.5  Comparison between test results and numerical results (Cont’d)

Table 6.4  Comparison of the peak loads from the test and numerical analysis

<table>
<thead>
<tr>
<th></th>
<th>Wall 1</th>
<th>Wall 2</th>
<th>Wall 5</th>
<th>Wall 8</th>
<th>Wall 10</th>
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<tr>
<td>$P_{\text{max}}$ (kN)</td>
<td>Test</td>
<td>62.8</td>
<td>82.2</td>
<td>71.1</td>
<td>125.2</td>
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<tr>
<td></td>
<td>DAP</td>
<td>52.3</td>
<td>57.0</td>
<td>57.0</td>
<td>95.0</td>
</tr>
<tr>
<td>error (%)</td>
<td>-16.7</td>
<td>-30.7</td>
<td>-19.8</td>
<td>-24.1</td>
<td>-21.1</td>
</tr>
</tbody>
</table>

In general, curves obtained from the finite element analysis (DAP) followed the experimental curves quite well until the load level reached approximately 60% of the maximum load. The predicted maximum lateral load values were 16% to 30% lower than the experimentally obtained values. It can be noticed that the separation of two curves occurs when the nail displacements pass the point corresponding to the maximum load in the nail load-slip curves used in the program and in most cases the program stopped without convergence. This implied that the program has a difficulty to deal with the post-peak-load (softening) part of the nail load-slip curve.
A study comparing the structural performance of shear wall systems built with regular and nonstandard large dimension oriented strand board panels under monotonic and cyclic lateral loading conditions was described in detail.

Under monotonic loading, a substantial increase in both stiffness and lateral load carrying capacity in shear walls built with oversize panels (with 281 nails) was observed when compared with those built with regular panels (with 424 nails). A further increase in both stiffness and strength was achieved in shear walls (with 409 nails) with closer nail spacing around the wall perimeter. The shear resistance of shear walls built with regular panels was affected by the presence and arrangement of blocking. The ductility ratios obtained from all tests did not vary significantly for both panel dimensions. The test results showed that common nails were stiffer but weaker than spiral nails in holding wood framing members together.

Under cyclic loading, the shear wall tested with a newly proposed cyclic test schedule provided the highest lateral resistance compared to the walls with the same configuration but tested with Forintek or CEN Short protocols. The difference in lateral resistance may be attributed to more realistic failure modes obtained in the new cyclic test protocol. The maximum degradation in strength and stiffness occurred in the second cycle within a cycle group in all cases.

When examining the dissipated energy as the total area under the load deformation curve, walls built with the regular panel layout dissipated more energy under cyclic loading as compared to walls built with oversize panels.

The experimental results indicated that not only the sheathing panel but also the connection between panel and frame played key roles in shear wall performance. In monotonic tests the dominant failure mode in the nails connecting the sheathing to the frame was nail pull-through. The dominant nail failure mode in cyclic tests by using the Forintek protocol was low cycle fatigue
fracture, which was substantially different from those observed in monotonic tests. When the new
protocol was used, no nail fatigue was observed, which implied that the loading history is important
to control the nail failure mode and the final result. In shear walls sheathed with oversize panels the
failures all occurred along the bottom edge and in those sheathed with regular panels the failure
occurred along the panel edge at the mid height and bottom of the wall.

A nonlinear finite element analysis program has been adopted to predict and model the
performance of shear walls built with oversize panels. The program presented a good agreement
with the early part of the nonlinear behaviour of shear walls obtained in the experiments; it was,
however, not able to follow the testing results when the connectors tend to fail. A group of panel-
frame joints connected by either spiral nails or common nails have been tested to investigate their
nonlinear load-slip characteristics, from which the parameters required for the finite element analysis
were derived.

Although the shear wall systems with described configurations were extensively studied and
a corresponding database has been generated at this stage, further studies on the following topics are
needed:

1. the overall behaviour of shear wall built with oversize panel with openings;

2. the optimized distribution and innovation of the nail connections between panel and
frame to produce the maximum lateral resistance;

3. an appropriate cyclic load schedule;

4. a series of dynamic experiments with realistic earthquake input;

5. improvement of the finite element program to better reflect realistic failure modes.

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8. BIBLIOGRAPHY


TECO (1980). “Summary of racking tests conducted on waferboard sheathing.” Report to the Canadian Waferboard Association, TECO, Oregon. USA.


9. APPENDIX 1. SUPPLEMENTARY INFORMATION OF THE STATIC NAIL TESTS

9.1. Nail test specimens

Two types of specimens were designed and tested in the static nail tests. Type I, as shown in Figure 9.1, was tested first. It consisted of a piece of 38 x 89 mm lumber sandwiched between two pieces of panelling with two nails on each side. The materials and nails were the same as those used in the shear wall experiments. Due to the difficulty in building such a specimen with the lumber grain oriented perpendicular to the loading direction and the tendency of the panel to separate from the lumber under load, a second type of specimens (Type II) was also configured, as described in Chapter 6. This type of specimen was designed in such a way that the potential overturning moment from eccentric loading was minimized to obtain “pure” nail shear effect.

![Figure 9.1 Type I specimen used in nail tests](image)

9.2. Parameters for nail load-slip models in shear wall studies

The parameters used in the current study and from previous research are listed in Table 9.1. The materials used for the nail specimens for these studies are listed in Table 9.2.
Table 9.1 The parameters required by nail load-slip model in DAP

<table>
<thead>
<tr>
<th>Resource</th>
<th>Nail</th>
<th>Loading direction to lumber grain</th>
<th>$P_0$ (N)</th>
<th>$P_1$ (kN/m)</th>
<th>$K$ (kN/m)</th>
<th>$E_0$ (m)</th>
<th>$m_e$ (kN/m)</th>
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<tr>
<td>Current shear wall study</td>
<td>Spiral (Type I)</td>
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<tr>
<td></td>
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<td>578</td>
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<tr>
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<td>579</td>
<td>0.0130</td>
<td>18</td>
</tr>
<tr>
<td></td>
<td></td>
<td>perpendicular</td>
<td>751</td>
<td>34</td>
<td>561</td>
<td>0.0125</td>
<td>44</td>
</tr>
<tr>
<td></td>
<td>Common (Type II)</td>
<td>parallel</td>
<td>918</td>
<td>20</td>
<td>1206</td>
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<td>20</td>
</tr>
<tr>
<td></td>
<td></td>
<td>perpendicular</td>
<td>853</td>
<td>27</td>
<td>1559</td>
<td>0.0080</td>
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<tr>
<td>Foschi &amp; Bonac¹</td>
<td>Common</td>
<td>parallel</td>
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<td>657</td>
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<td>-</td>
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<td></td>
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<td>-</td>
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<tr>
<td>VPIST²</td>
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<td>(Waferboard)</td>
<td>800</td>
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<td>831</td>
<td>0.0127</td>
<td>500</td>
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<tr>
<td></td>
<td></td>
<td>(Plywood)</td>
<td>972</td>
<td>42.4</td>
<td>972</td>
<td>0.0127</td>
<td>500</td>
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</table>

Table 9.2 The materials of nail test specimens

<table>
<thead>
<tr>
<th>Resource</th>
<th>Nails and Diameter (mm)</th>
<th>Lumber</th>
<th>Panel</th>
</tr>
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<tbody>
<tr>
<td>Current study</td>
<td>Spiral, 2.67 mm &amp;</td>
<td>38 x 89 SPF, No.2 &amp; btr.</td>
<td>9.5 mm OSB</td>
</tr>
<tr>
<td></td>
<td>Common, 2.82 mm</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Foschi &amp; Bonac</td>
<td>Common, 2.54 mm</td>
<td>Douglas-fir</td>
<td>12.5 mm Douglas-fir plywood</td>
</tr>
<tr>
<td>VPIST</td>
<td>Common, 3.00 mm</td>
<td>38 x 89 SPF</td>
<td>9.5 mm Waferboard and Plywood</td>
</tr>
</tbody>
</table>

9.3. Influence of nail load-slip parameters variation on DAP

In this part, shear wall 8 is used as an example to show the influence of nail load-slip parameters variation on numerical results. This wall was built with oversize panel attached to the frame with spiral nails in a 76 mm perimeter spacing.

The parameters obtained from nail test data by using both Type I and Type II specimens were used as model inputs for DAP program to produce the load-displacement relationship of shear walls. They provided similar numerical results, as shown in Figure 9.2.

![Figure 9.2 Comparison of DAP results by using parameters from Type I and Type II specimens](image)

DAP output with the variations in $P_0$ and $K$ or in $P_1$ and $E_0$ are shown in Figure 9.3 and Figure 9.4, respectively. In Figure 9.3, the parameters were from (1) averaged nail test data, (2) a nail load-slip curve with the maximum $P_0$ value, and (3) a nail load-slip curve with the maximum $K$ value. The varied $P_1$ and $E_0$ values for Figure 9.4 are listed in Table 9.3.
Figure 9.3 Influence of variations in $P_0$ and $K$ on DAP outputs
(b)

Figure 9.4 Influence of variations in $P_1$ and $E_0$ on DAP output

Table 9.3 Variations in $E_0$ and $P_1$

<table>
<thead>
<tr>
<th>Curves</th>
<th>Loading direction to lumber grain</th>
<th>$P_1$</th>
<th>$E_0$</th>
</tr>
</thead>
<tbody>
<tr>
<td>DAP result 1 (Averaged)</td>
<td>Parallel</td>
<td>13</td>
<td>0.0130</td>
</tr>
<tr>
<td></td>
<td>Perpendicular</td>
<td>34</td>
<td>0.0125</td>
</tr>
<tr>
<td>DAP result 2 (Doubled $E_0$ and $P_1$)</td>
<td>Parallel</td>
<td>26</td>
<td>0.0250</td>
</tr>
<tr>
<td></td>
<td>Perpendicular</td>
<td>68</td>
<td>0.0250</td>
</tr>
<tr>
<td>DAP result 3 (Adjusted $E_0$ and $P_1$)</td>
<td>Parallel</td>
<td>40</td>
<td>0.0180</td>
</tr>
<tr>
<td></td>
<td>Perpendicular</td>
<td>68</td>
<td>0.0180</td>
</tr>
<tr>
<td>DAP result 4 (Doubled $P_1$)</td>
<td>Parallel</td>
<td>26</td>
<td>0.0130</td>
</tr>
<tr>
<td></td>
<td>Perpendicular</td>
<td>68</td>
<td>0.0125</td>
</tr>
<tr>
<td>DAP result 5 (Doubled $E_0$)</td>
<td>Parallel</td>
<td>13</td>
<td>0.0250</td>
</tr>
<tr>
<td></td>
<td>Perpendicular</td>
<td>34</td>
<td>0.0250</td>
</tr>
</tbody>
</table>

The results indicate that the fluctuations in $P_0$ and $K$ in a group of nail tests may not significantly affect the DAP outputs; however, the changes in $P_1$ and $E_0$ influenced the DAP outputs. Even though result 3 in Figure 9.4 provides the best fit to the experimental curve, the parameters have gone out of the ranges of nail test data, as shown in Figure 9.5. One reason for the differences
between numerical curves and the experimental curves at the high load level may be that too few Gaussian points (four Gaussian points in the current version) were used along the element length in DAP (Foschi\(^3\) 1997). Work is underway to modify the DAP program to verify this reasoning.

\[ \text{(a) Lumber grain parallel to loading direction} \]

\[ \text{(b) Lumber grain perpendicular to loading direction} \]

Figure 9.5 The parameters in result 3 were out of the range of the test data