Chapter 1

Introduction

1.1 Overview of Shear Walls

Timber is one of the oldest and most frequently used building materials known to man. The advantages of using wood in construction include: it is an extremely versatile raw material, little energy is required to process wood into finished products, it has a less detrimental impact on the environment than many other building materials, it is a renewable resource, and when maintained properly it can be a very durable building material. Early structures were composed of large heavy timber logs and/or beams. Today the emphasis has shifted from heavy timber construction to light-frame building construction. Light-frame building construction is the most frequently used form of residential construction in the United States.

In its infancy, wood-frame construction was a means of forging a primitive shelter, but today, wood-frame construction is an area of major interest to the engineering community. Shear walls, which along with horizontal diaphragms provide resistance to lateral loading such as earthquakes or wind, are one use of timber in low-rise buildings. The combination of shear walls and horizontal diaphragms is often referred to as diaphragm design. Lateral forces are transferred through a structure in the following manner. The wind forces are first transferred from a transverse wall, which is a wall that is perpendicular to the direction of the loading, to a horizontal diaphragm. Earthquake forces amass at the horizontal diaphragm, where the mass of the structure is concentrated, and are a function of inertia. Then the forces are distributed across a horizontal diaphragm and transferred from the horizontal diaphragm to the top of the shear wall. The load is then transferred to the foundation by the shear wall.

A typical shear wall consists of four primary components: framing, sheathing, connectors which attach the framing members and attach the sheathing to the framing, and hold-downs (see Figure 1.1). Framing, which typically consists of dimension lumber such as 38 mm x 89 mm (2 in x 4 in nominal) or 38 mm x 140 mm (2 in x 6 in nominal) members in North America, provides the majority

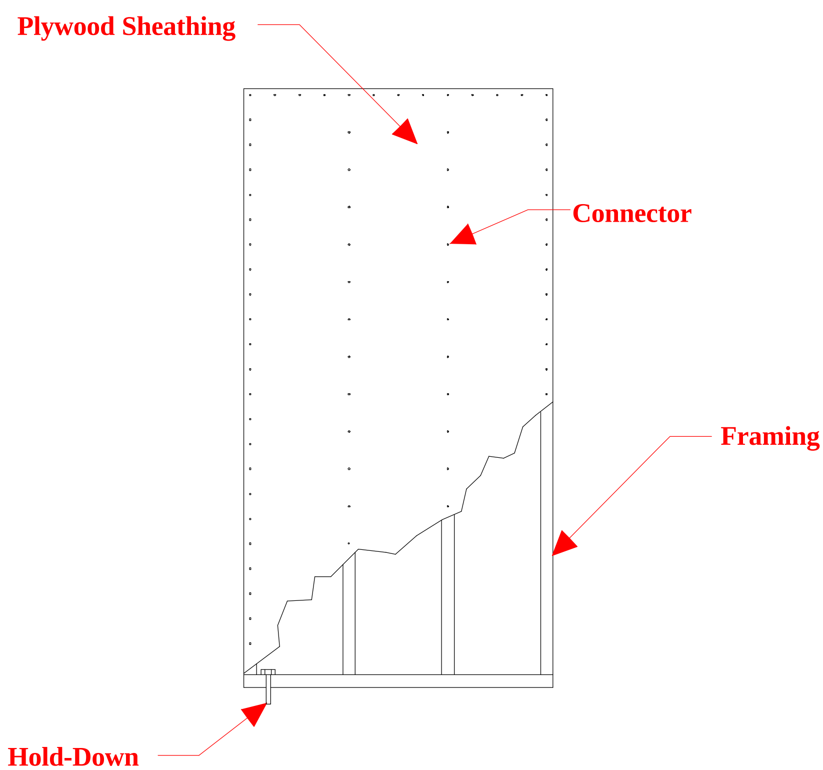

Figure 1.1: Components of a Typical Shear Wall

of the bending resistance of the wall. Sheathing, which typically consists of a structural-use panel such as plywood or OSB, provides the majority of shear resistance in the wall. Originally, horizontal sheathing boards with "let-in corner braces" were used to provide this resistance.

Connectors used to attach sheathing to the framing of a shear wall transfer forces between the sheathing and framing and can also provide significant ductility for the wall. The connectors are typically dowel type fasteners such as nails, screws, or staples. However, elastomeric adhesives are sometimes used along with dowel-type fasteners to attach sheathing to the framing of a shear wall. Adhesive, after it dries, performs as the connector and is responsible for providing resistance to and transferring the shearing forces. Dowel-type fasteners are still required to hold the members in place until the adhesive dries. Framing connectors, which typically consist of nails of various sizes, are responsible for attaching the studs to the top plates, bottom plates, and headers. Hold-downs, which are typically located at the chords, provide resistance to uplift.

Shear walls have been studied for a variety of conditions. However, analysis and experimental testing of shear walls has almost always assumed that a wall is supported by a rigid foundation, such as a block wall. Shear walls are often supported by non-rigid foundations, such as floor joists and stud walls, which would significantly reduce the stiffness at the foundation of the wall. This could have a significant effect on the stiffness of the entire wall system, possibly resulting in increased lateral displacement of the wall and variations in the dynamic base shear.

1.2 Objectives and Scope

The overall objective of the research was to determine the effects of non-rigid foundations on the response of timber shear walls. Ultimately, this information would be used to assist in modifying design procedures for shear walls subjected to monotonic and dynamic loading. Specifically, the following characteristics of timber shear walls were examined:

1.) The effect of joist stiffness on the response of shear walls parallel to and supported by floor joists that are subjected to monotonic and dynamic loading

2.) The effect of hold-downs on the response of shear walls that are subjected to monotonic and dynamic loading

These objectives were satisfied by creating a shear wall numerical model, varying the conditions at the base of the model (specifically joist stiffness and hold-down size), and analyzing the various models using a modified version of the computer program WALSEIZ, titled WALSEIZ1. WALSEIZ1 is a finite element program that uses five elements to model a shear wall. The five elements are: 1.) a common beam element to represent the framing, 2.) a simple four node quadrilateral to represent the sheathing element, and springs to model 3.) bearing between the adjacent sheathing panels, 4.) the connectors between the framing members and between the framing and sheathing members, and 5.) the hold-downs. Each model was subjected to monotonic and dynamic loading and the following information was collected for each run: top-of-the-wall displacement, maximum and minimum element forces, and maximum base shears.

1.3 Significance

As stated previously, although shear walls are sometimes supported by a relatively flexible foundation, there is little or no data on the effects this type of foundation has on the response of a wall. Almost all shear wall research performed to this point has assumed that the wall would rest on a rigid foundation. Researchers agree that the strength of a wall is highly dependent on the stiffness of all of the components that comprise the wall (Schmidt, 1989). Therefore, it is important to know the extent to which the stiffness of the foundation affects the response of shear walls in order to design with adequate confidence.

Chapter 2

Literature Review

2.1 Introduction

Research concerning timber shear walls has continued to expand as their use has increased. An overview of research related to shear walls is presented in this chapter. Research that has been conducted on shear walls will first be covered. Research on the entire structural system will be covered later in the chapter.

2.2 Background

Conventional light-frame housing is the largest form of residential construction in the United States. Timber shear walls are one of the most important components of the light-frame house, and the performance of shear walls needs to be fully understood to make the most efficient use of light-frame construction. Timber shear walls provide resistance to lateral loads, such as wind or seismic loading, in many low-rise buildings. A typical timber shear wall consists of four primary components: the framing, sheathing, connectors joining the sheathing to the framing, and hold-downs. The framing consists of studs, top plates, bottom plates, and headers if openings are present. Studs are oriented vertically and have a typical cross-section measuring 38 mm x 89 mm (2 in x 4 in nominal) or 38 mm x 140 mm (2 in x 6 in nominal). Top plates and bottom plates are oriented horizontally and help to keep the studs in place in the wall. Framing provides the majority of the bending resistance of the wall. Sheathing is attached to the framing of shear walls and helps to provide strength, stiffness, and stability to the walls. The sheathing provides most of the resistance to shearing forces. Originally horizontal sheathing boards with let-in corner braces were used to resist racking forces but the superior performance of panel sheathing has made this type of lateral bracing obsolete. There are three factors that allow sheathing panels to be considered as satisfactory shear panels: 1.) significant force resistance to deformation at the 'design drift', about 3 times the nominal elastic code limit; 2.) significant energy dissipation at deformations well beyond the design drift, but permitting reduced resistance; and 3.) lack of significant damage to the specimen at repeated deformations less than code drift (Zacher & Gray, 1990).

Lateral loads, such as wind, are transferred through a low rise, light-frame structure in the following manner. Wind loading is applied to the transverse wall, which is perpendicular to the shear walls, and then transferred to a horizontal diaphragm, such as a floor or roof system. Seismic loading is typically concentrated at the horizontal diaphragms because of the relatively large mass located there. Load is then transferred from the horizontal diaphragm to the top plate of the shear walls attached to the diaphragm. The shear walls then transfers the load to the foundation. When a load is applied along the top plate of a shear wall, the framing distorts in a manner resembling a parallelogram while the sheathing remains rectangular and rotates slightly, with one side of the panel rising slightly and the opposite side sinking slightly.

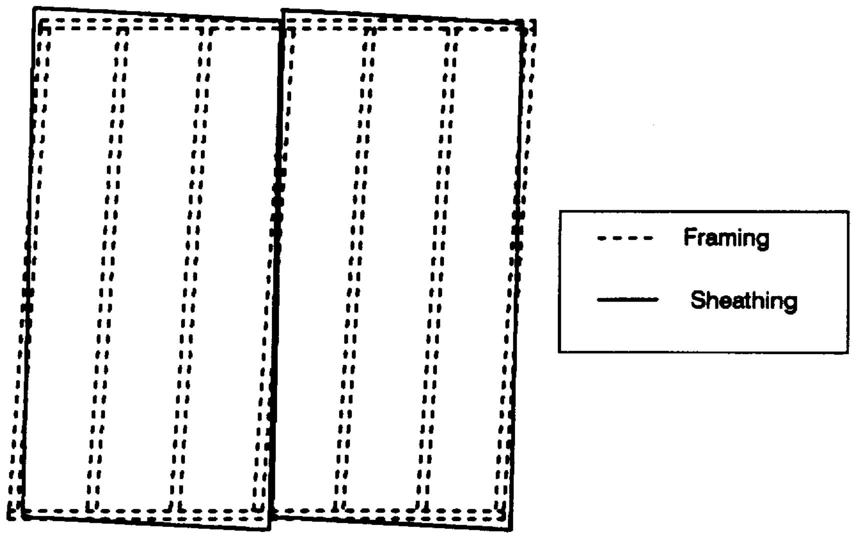

Figure 2.1: Deformation Pattern of a Shear Wall (White, 1995)

2.3 Monotonic Loading

Shear walls are often used to withstand monotonic loading. Monotonic loads are applied slowly in only one direction over an extended period of time, such as a constant pressure in a bulk storage facility. In-plane stiffness and strength are the two most important characteristics of shear walls subjected to monotonic loads. Strength is important for determining the magnitude of load a wall can resist before failure, while stiffness is necessary when determining the amount of deformation, and thereby architectural damage, a structure may experience. Several factors, such as the structural properties of sheathing materials, fastener size and spacing, framing spacing, wall length, and the interaction between the components of a wall, affect the performance of a shear wall

(Polensek and Schimel, 1985; Filatrault, 1990; and White, 1995). As the thickness and stiffness of sheathing panels increase, so does the panels ability to withstand lateral forces, while as the fastener spacing increases the ability of the sheathing and framing to resist force decreases. Framing spacing is vital because as the spacing increases the resistance of the wall decreases. Each connector can only provide a certain amount of resistance to loading, so as the spacing of these connectors increase, the total number of connectors in the wall decreases and thus the resistance decreases. The better the interaction between the components of a wall, the greater force that can be resisted.

Several researchers have made profound conclusions concerning shear walls. Polensek (1976) concluded that the stiffness of wood walls can be significantly increased by adding a few additional nails into the perimeter of the exterior sheathing, sill plate, and header. Additional nails will reduce wall deflections under lateral wind pressures and will also provide better resistance to uplift forces of tornadoes, hurricanes, and seismic excitations. Dolan (1989) found that out-of plane deflections of sheathing are insignificant for 9.5 mm (3/8 in) and thicker panels. Dolan also observed that the connectors that attach sheathing to the framing are very important in the overall performance of the wall. Dolan and White (1992) observed that the use of adhesives to attach the sheathing to the framing of a wall wound increase the stiffness and reduce the ductility of the wall. White (1995) found that walls that are less than 1.2 m (4 ft) in length do not provide sufficient resistance to racking forces and should be avoided in design. Rose and Keith(1995) subjected walls to both monotonic and cyclic loads. The monotonic tests tested racking forces in walls proved that shear strength and stiffness are increased with the addition of gypsum wallboard on the uncovered side of shear walls.

2.3.1 Connections

Connections between the sheathing and the framing members are one of the most important factors influencing the performance of a shear wall. Shear strength and load-deflection characteristics of a connection are of greatest concern when analyzing a shear wall subjected to monotonic loading. McCutcheon (1985) found that the racking strength of a shear wall is dependent on the load-displacement characteristics of the connectors. Shear strength of a connection is vital when testing for the racking strength of a wall. Strength of the connection is a function of load rate, species of wood members, deformation rate and other factors. Girhammar and Anderson (1988) found that the effect of deformation rate is significantly greater for wood than it is for nails.

Research has focused on the relationship between connectors and the response of a shear wall. Effects of nail-slip characteristics on the performance of shear walls has been thoroughly researched. An area that lacks research for shear walls is the effects of loading rate on joint capacity for short duration loads and the use of adhesives (Foschi and Filiatrault, 1990). Zacher and Gray (1990) examined the effects of using nails and staples to connect plywood sheathing in shear walls which were tested both monotonically and dynamically. Properly nailed shear panels were consistent in both monotonic and dynamic loading and showed resistance at drifts well above the necessary design limits. Nails that were over driven did not produce satisfactory resistance levels. Staples that were properly driven did meet resistance criteria but did not achieve as high a level of resistance as properly driven nails. Tests showed that the greatest part of wall distortion is due to nail flexure and movement of the nails in the wood (Zacher and Gray, 1990), demonstrating the importance of the connectors on the response of a structure.

The use of adhesives in shear walls was considered in design by Dolan and White (1992). Foschi and Filiatrault (1990) sought to determine the effects of using nails versus 3M scotch grip wood adhesive 5230 on the performance of shear walls. With use of the adhesive, the walls experienced smaller racking deflections than comparable walls using only nails. The anchorage connection was exposed as the weakest link in walls connected with adhesives through research conducted by Dolan and Madsen (1992) and Foschi and Filiatrault (1990). They also concluded that walls using adhesives were stronger, stiffer, and less ductile than walls using only dowel-type fasteners.

Creep behavior of joints between lumber and plywood is another area that has been researched. Polensek and Jang (1989) collected data on creep behavior with the premise of improving light-frame wood building designs. They developed two models that predicted the creep of joints under variable loads, where the predicted results agreed with data that had been collected from four stepwise load functions.

2.3.2 Shear Wall Models

Researchers have attempted to model the response of shear walls subjected to various conditions. Linear models do not incorporate all of the characteristics of a wall and thus inadequately predict the behavior of a wall so therefore nonlinear models of timber shear walls were created. Itani and Robledo (1984) developed a model with openings simulating doors and windows that was loaded in shear. They used a set of nonlinear springs to model the nonlinearity of nails. Polensek (1986) developed a computer program consisting of the finite-element method, multi-linear material models, and a linear step-by-step procedure that accounts for nonlinear behavior of wood materials, nailed joints, and inter-component gaps that predicted the structural behavior of the connection system between wall, floor, and foundation in wood-frame buildings. The program was called COMPCON and contains an orthotropic rectangular plane strain element, an orthotropic triangular plane strain element, and a gap element. The first two elements were used to represent the behavior of the framing and the sheathing while the third element consists of a set of two perpendicular springs that permit modeling of nail joints and of the contact between adjacent materials. Predicted deflections from the program agreed with experimental results.

Okabe et al. (1985) created a model that was highly influenced by the number of connectors. They analyzed models with varying numbers of connectors in order to analyze the effect of connector number and spacing. Akerlund (1985) developed a model to predict the response of wood shear walls. The model assumed a hinged frame of stiff members (i.e. top plate, bottom plate, sheathing), stiff sheathing, and linear load-slip relations for a single nail. Akerlund assumed a constant stiffness modulus for the nails. The model was in good agreement with results from a finite element model developed by Easley et. al (1982). In 1987, Hata and Sasaki designed a special element to account for the behavior of nails in their model.

Gupta (1987) created a simple nonlinear model in an effort to try and predict the lateral behavior of wood-framed shear walls with stud uplifting. It consisted of a simple finite element model with five degrees-of-freedom for one story walls and seven degrees-of-freedom for two story walls. Additional degrees-of-freedom for the two story model did not significantly change the overall stiffness of the wall. Foschi (1977) developed a general finite element model that modeled nailed connections by fitting a curve to experimental data. Foschi's model is nonlinear and contains a linear and exponential component. His model was the predecessor for several models including Dolan (1989). Dolan (1989) developed a finite element model that accurately predicted the static behavior of shear walls. Dolan's model contained elements for the sheathing, framing, connectors, and bearing between the sheathing panels.

Kasal and Leichti (1992) went about developing a three dimensional detailed model of a wood-frame stud wall utilizing a two dimensional model, by incorporating linear and nonlinear finite elements. The two dimensional models used load deflection characteristics and made the model equivalent to the original substructure by energy concepts. The model could be subjected to shear, torsion, and bending in both the X and Y directions. The approach was developed so as to be capable of analysis of wood-frame stud walls with or without openings. Gap elements were utilized in-between the sheathing elements. Two dimensional shell elements with linear orthotropic properties were used to model the studs and sheathing. Shear stiffness was modeled by a nonlinear and non-conservative diagonal spring and a three dimensional truss created by beam elements pinned at the corners. Results from the analysis of the model agreed with the equivalent model. Chowdhurry (1994) created a simple finite element model in an attempt to predict the global response of wood-frame stud-wall structures. The model included nonlinear diagonal springs to define the shear deformation of the wall. Nonlinear springs on the model boundaries were used to represent the stiffness of connections between the walls and the rest of the structure. White (1995) created a finite-element model to analyze the effects of openings and aspect ratios on the response of a shear wall. The model created by White, which had its basis in the model by Dolan (1989), had elements representing the framing, sheathing, framing to sheathing connections, and bearing between the sheathing panels. Other research has focused on the development of closed-form expressions to characterize the response of timber shear walls. Easley et al. (1982) investigated formulas for wood shear walls. They derived formulas to calculate the sheathing fastener forces, the linear shear stiffness of a wall, and the nonlinear shear load-strain behavior of a wall. The formulas correlated well with results obtained from load tests and of finite element analysis, and can be applied to the design of wood frame shear walls with various sheathing types.

McCutcheon (1985) set out to derive a method for calculating the racking strength of light-frame nailed walls. Results were then compared to actual laboratory tests. There were several assumptions made by McCutcheon including: the sheathing being applied vertically, nails are evenly and symmetrically spaced, deflections are small, and loading is static. There was good agreement between the model and the laboratory tests. The method that McCutheon developed had one short coming in that it was incapable of performing accurate stiffness computations.

McCutcheon (1985) sought to create a new method that would allow more flexibility in the wall construction. Previous models were based on empirical relationships and were only useful for a limited number of wall configurations. The new method is similar to his previous formulation but accounts for the nonlinear behavior of the nails. Simple mathematical power curves are used to define the nonlinear nail behavior and allows the prediction of racking performance of wood shear walls up to moderate levels of deformation (McCutcheon, 1985). Results of the method provided good estimates of displacements due to racking forces.

Analysis of a linear system was used to calculate the distribution of horizontal forces to shear walls. The analysis, performed by Kawai et al. (1990) takes into account the effect of shear rigidity of the floor that the shear wall was resting on. In the process of creating the method of analysis, the vertical frame and the horizontal frame were assumed to only experience shear deformation. The shear wall and floor were also assumed to behave as elastic bodies and the spring constant and shear modulus were assumed to be constant. When compared with exact solutions from experimental tests it was determined that a good approximation could be obtained after several iterations of the calculation method.

2.3.3 Aspect Ratio and Opening Effects

Shear walls often contain openings for windows and doors, and the aspect ratio varies depending on the floor height and length of a structure. These factors have an effect on the response of a wall. Kamiya et al. (1981) examined plywood sheathed walls and the effect wall length had on their racking resistance. Aspect ratios (defined as ratio of the height of a wall to the length of a wall) between one-third and two were used. They found that racking load was proportional to wall length at deformation levels below failure. Researchers such as Gupta (1987) have showed that a longer wall will not have as great a tendency to exhibit uplifting, and shear strength and stiffness will be greater for a longer wall. Griffiths (1976) also examined the effects of aspect ratio on racking resistance of sheathed walls, but the research was extended to include hardboard and insulation board as well as plywood. Griffiths found that racking strength per unit wall length increased with wall length for walls sheathed with plywood, hardboard, and insulation board. Tuomi and McCutcheon

(1975) looked at the effects of additional layers of sheathing and wall length on the response of shear walls and found that additional layers of sheathing increased the stiffness of the wall for all lengths. Wolfe (1983) used the model that Tuomi and McCutcheon (1975) developed and predicted the racking strength of walls with varying lengths, sheathed with continuous gypsum sheathing. The research was not precise enough to accurately extend racking theories to longer walls. Patton-Mallory et al. (1985) sought to examine the affect of shear resistance of small walls with plywood and gypsum sheathing versus full size walls. Wall racking stiffness and strength were compared to wall aspect ratio and effective wall length (Patton-Mallory et al., 1985). Resistance for small walls was found to be proportional to wall length. Meanwhile longer walls did not appear to increase strength proportionally. Longer walls appear to have a higher stiffness per unit length than the shorter walls. Racking strength was found to be proportional to wall length for aspect ratios between 1 and 1/4 but not for walls with aspect ratios greater than 1/4. Research on aspect ratio continued with Gupta (1987) finding that longer walls will show less of a tendency to exhibit uplift than shorter walls. Gupta also found that the longer a wall is the greater the shear stiffness and strength of the wall. The effect of aspect ratios of walls subjected to monotonic loading was studied by White (1995). White found that the strength per unit length of a shear wall less than 1.2 m (4 ft) long was lower than for longer walls. Resistance per unit length was found to decrease as the height of the wall increases. For walls longer than 1.2 m (4 ft), the initial stiffness and maximum base shear was found to increase linearly as the aspect ratio decreases.

Openings have a profound effect on the response of a shear wall. When an opening is present in the wall, it is assumed in design that only the full height portions of the wall provide resistance to the loading (i.e., the portions of the wall above and/or below the wall provide no resistance to loading) (Breyer 1993). Dishough and Fowler (1980) looked at the effects of openings. They compared walls, sheathed on both sides with gypsum, with and without openings in an effort to determine the effects that openings had on the performance of the walls and concluded that a wall with a center opening could be treated as two separate shear walls with the length of the opening considered to encompass the entire height of the wall. Dean et al. (1984) utilized a simple equilibrium analysis and the shear transfer method to examine shear walls with rectangular openings. The material above and below an opening was shown to exhibit structural importance through the research of Ge et al. (1991). The yield strength and lateral stiffness of walls with small openings was studied by Yasumura and Murota (1992). Sugiyama (1993) proposed a simplified calculation method for evaluating the response of a shear wall with openings which has since been adopted for design in high wind regions (AF&PA, 1996). Patton-Mallory et. al. (1985) found that total wall length without openings can be used as a conservative measure of the racking strength of walls with openings, but this procedure tends to overestimate the stiffness of the wall. Rose and Keith (1995) researched wood structural panel shear walls with gypsum wallboard and window/door openings to determine the effect of adding gypsum wallboard to the system. Results showed that for both walls with and without openings the addition of gypsum wallboard added to the stiffness of the wall. White (1995) found that the relative drift of the walls increased as the length of opening increased. Johnson (1996) tested shear walls with openings and found that the ultimate capacity for a shear wall increased as the sheathing area to wall area ratio increased. Heine and Dolan (1997) performed tests on the cyclic response of wood shear walls and found that the capacity and elastic stiffness of a shear wall will decrease as the size of openings increase. Heine and Dolan also found that the inclusion of a hold-down anchor, in the construction of a shear wall, increases the capacity of the wall.

2.4 Dynamic Loading

Design considerations for earthquake forces are of importance, and thus tests involving dynamic loads are necessary in order to accurately set comprehensive design criteria for structures. The inertia effects of a structure cannot be ignored during dynamic loading because of the high frequency of the load. Wood is a good material for use in areas of high seismic potential due to the ability of the connections in wood systems to provide ductility and to dissipate significant amounts of energy through hysteretic damping and friction, its high strength to weight ratio, and the typical redundancy in construction. In dynamic loading ductility along with stiffness becomes an important criteria in the performance of the structure, to prevent excessive displacements and costly architectural damage. A wall needs to have a balance between stiffness and ductility in order to function appropriately.

Shear walls subjected to dynamic loading are similar in some respects to walls subjected to monotonic loading. The framing distorts as a parallelogram while the sheathing retains its rectangular shape and rotates slightly. The resistance of a wall, subjected to dynamic loading, will increase as nail spacing decreases, connector stiffness increases, and sheathing thickness increases. Other factors that affect shear wall performance are load-displacement characteristics of the connectors, species of wood used, framing spacing, length of the wall, and presence of openings in the wall. Some species of wood are capable of resisting greater loads than other species of wood. As framing spacing and opening length increase, the resistance of the wall decreases.

Experimental tests have been performed that sought to define certain response characteristics. Rose and Keith (1995) tested walls subjected to cyclic (dynamic) loads and found that shear strength is governed by the sheathing to framing connectors. Tests also showed that gypsum wallboard attached on the inward facing side of the wall with conventional techniques did not contribute to strength and stiffness of a wall subjected to dynamic loading. For walls with openings there was a reduction of end restraint that was accounted for by multiplying the shear modulus of a fully sheathed wall by a factor of two-thirds. Cyclic load tests on timber shear walls have also been performed by Thurston and Hutchison (1984). The tests were conducted with in-plane slow cyclic load testing carried out on plywood and particle board shear walls and small test specimens. They concluded that wall models sheathed with particle board appear to be stiffer than walls sheathed with plywood. The tests also showed that closer spacing for the framing connectors increased the resistance of the wall. Dolan (1989) performed cyclic tests to investigate the load-deflection characteristics of simple nail connections in shear walls. Dolan developed three numerical models from his research that predict the response of timber shear walls. Dolan (1989) concluded that the fasteners used to attach the sheathing to the framing, governed the load capacity, stiffness, and ductility of the shear walls. Johnson (1996) performed a parametric study to determine the effect of aspect ratio and the presence of openings on the response of timber shear walls. Johnson found that an increase in wall length leads to an increase in base shear while an increase in opening length leads to a decrease in base shear. Johnson also concluded that anchorage connections located at the stud where sheathing panels meet are capable of resisting larger loads than the average force per anchorage. Heine and Dolan (1997) performed tests on the cyclic response of timber shear walls and found that design load values for seismic design should be based on the stabilized response of cyclic tests.

2.4.1 Connections

Traditionally connector design has been based on static loading requirements but designing a wooden structure strictly to static load requirements frequently results in serviceability problems (Kalkert and Dolan, 1992). Research has not been well developed with respect to dynamic or cyclic loading. Connectors loaded dynamically follow hysteretic behavior in deforming. The load-displacement curve is a complex, nonlinear, closed loop. The typical hysteretic curve is pinched on either end of the curve and will have the same load-intercept for all load-displacement relationships. The maximum loading for the loop follows the monotonic load-displacement curve. Kalkert and Dolan (1992) sought to quantify the dynamic behavior of connections by conducting cyclic testing to predict the initial stiffness, stiffness degrade, and damping capacity. They also propose a standard test procedure to follow that tries to ensure that previous as well as future results will be comparable. Dolan (1993) also proposed a new test procedure to evaluate the dynamic properties of connectors manufactured from different materials. The test procedure was developed to allow a direct comparison between different material performance when used as mechanical fasteners as well as different connection configurations. Dolan et al. (1995) performed an experimental study to determine the effect of cyclic loading on the performance and safety of nail and bolt connections, and if the value of 1.6 used as the load duration factor for wind and seismic design in the 1991 NDS is conservative. The study concluded that prior cyclic loading does not have a significant effect on the capacity or ductility of nail and bolt connections when the load levels were as high as 2.0 times the NDS nominal design load. Seismic loading does not seem to have a significant effect on the connections load resisting capacity. Rose and Keith (1995) performed tests on shear walls and found that the shear strength is governed by the sheathing to framing connectors.

Dolan and White (1992) addressed design considerations for the use of adhesives in shear walls. Test results from Foschi and Filiatrault (1990) showed that plywood-sheathed shear walls with nailed connections exhibit a nonlinear force displacement response rather that the idealized elastic-plastic response assumed in the design code (Dolan and White, 1992). Foschi and Filiatrault (1990) analyzed the effects of using nails versus 3M scotch grip wood adhesive 5230 on the performance of shear walls. The adhesive walls attracted larger accelerations and base shears, and had smaller deflections at the top of the wall. Upon inspection of walls that were tested during the initial earthquake simulations, no visual damage was observed for the adhesive walls, while the walls connected with nails showed visual damage. Dolan and Madsen (1992) and Foschi and Filiatrault (1990) concluded that walls using adhesives were stronger, stiffer, and less ductile and they also inferred that the weakest link in adhesive walls would most likely be the anchorage connections. Walls containing adhesives behave nearly linear-elastic with very little plastic deformation thus the structural response factor should be reduced for these walls so as to accurately represent their reduced ductility. Current design procedures also underestimate the shear forces in walls with adhesive connections.

2.4.2 Shear Wall Models

There is a definite need for development of a general structural analysis model for prediction of the lateral stiffness, ultimate lateral load carrying capacity, and complete earthquake response of timber shear walls (Filiatrault, 1990). Early attempts to model dynamic response of shear walls subjected to earthquake forces were made by Medearis (1969), Falk (1986), and Stewart (1987). Medearis (1969) and Falk (1986) based their models on single-degree-of-freedom systems and are not applicable to different wall configurations. These were simple models based on a single degree-of-freedom and could only be applied to specific wall configurations and were capable of analyzing only a few material properties. Stewart (1987) used ideal pinching and stiffness degradation to develop a model consisting of a series of linear components. Dolan (1989) developed two numerical models that sought to predict the behavior of shear walls subjected to dynamic loading. One was a closed-form, mathematical model which could simulate the response of a shear wall subjected to sinusoidal loading, and the other was a finite element model that could simulate the response of a wall subjected to an arbitrary dynamic loading. The finite element program is similar to the model developed for monotonic loading, with a modified load-displacement pattern for the connector element.

Filiatrault (1990) created a single-degree-of-freedom model to perform a nonlinear analysis of shear walls subjected to earthquake excitations and static lateral loads that was based on the connector hysteresis developed by Dolan (1989). The model allows for arbitrary geometry so that walls with or without openings may be analyzed. The model is two dimensional and is incorporated into a shear wall analysis program, SWAP, that was developed for microcomputer usage. The model was validated using data obtained from tests performed by Dolan (1989) on full scale models subjected to racking forces. The model considers the configuration of a wood stud frame with sheathing panels attached with nail connectors. Sheathing is assumed to be thick enough to prevent out of plane deformations, and energy calculations used were based on an equivalent lateral seismic force applied to a rigid based timber shear wall. Results of the tests were relatively good with the ultimate loads predicted by SWAP being within nine percent of the average test peak loads from Dolan's tests. Comparisons with shake table tests show some differences between amplitude and phase of earthquake loads, but the differences were attributed to the use of average nail parameters in the SWAP analysis.

A program was developed by Dolan and White (1995) that was a modified version of Dolan's previous program. The model created by Dolan (1989) was unable to calculate the forces and stresses in the members and also had an extensive analysis time. The new program, named WALSEIZ, had the capability of analyzing shear wall models subjected to static, monotonic, or dynamic loading; calculate displacements at the nodes and forces and stresses in the elements for user specified nodes and elements; and determine the maximum and minimum displacements, forces, and stresses for all of the nodes and elements (White and Dolan, 1995). Finite element analysis is used to develop the model and incorporates a framing element, sheathing element, connector element, and bearing element. The framing element was modeled as a two-dimensional, linear elastic beam element with six degrees-of-freedom. Meanwhile the sheathing element was modeled as a orthotropic plane stress element with eight degrees-of-freedom. The connector elements were modeled as nonlinear-springs. The bearing element was modeled as a bilinear spring. The model was found to correlate to experimental results and was deemed an acceptable way to analyze shear walls subjected to static or dynamic loading.

2.4.3 Aspect Ratio and Opening Effects

Aspect ratio and openings have an effect on the response of shear walls subjected to dynamic loading. The effect of aspect ratios of walls subjected to seismic loading was studied by White (1995). The program (White and Dolan, 1995) described in the previous section was used to perform a parametric study on walls with varying aspect ratio and openings subjected to dynamic loading, and found that the maximum seismic base shear is approximately thirty-five percent greater than the seismic base shear calculated using the criteria in the Uniform Building Code, the maximum seismic base shear for walls with openings is lower than for identical walls without openings, the maximum top-of-the-wall displacements and unit shear increase as the length of an opening increases, and relative drift increases and base shear decreases as the length of the opening in a given wall increases. White also observed that the anchorages located below window openings resists lower shear forces than the average shear force resisted per anchor The results indicated that the structural system quality factor needs to be reduced in the seismic base shear equation to adjust for the lower stiffness of the walls. Some revisions were recommended for the building code. Two of the suggestions made by White was to not use walls less than 1.2 m (4 ft) in length and to reduce the structural system quality factor in the seismic base shear equation.

Other tests involving openings have been performed by Rose and Keith (1995). They conducted cyclic loading tests shear walls sheathed with gypsum wallboard. Results showed that for walls with openings there was a reduction in the end restraint that was accounted for by multiplying the shear modulus of a fully sheathed wall by a factor of two-thirds. They also determined that the shear load value for the composite assembly needed to be modified by a ratio of effective shear stiffness/strength and the resulting factor was consistently between 2.7 and 3.1 for all of the walls tested (Rose and Keith, 1995). Sugiyama (1994) formulated an empirical evaluation on the racking strength of shear walls with openings. He used his work to define a sheathing area ratio that was able to classify walls based on the area of openings present. Johnson (1996) tested shear wall models with various openings subjected to monotonic and cyclic loading and validated Sugiyama's work. He found that the ultimate load increases as the sheathing area to wall area ratio increases. He also attempted to correlate the results from the monotonic and dynamic test results and found that there existed a relatively good correlation between the results. Heine and Dolan (1997) performed tests on the monotonic and cyclic response of timber shear walls. They concluded that the capacity and elastic stiffness of shear walls decrease as the area of openings increases.

2.5 Complete Structural Systems

Shear walls are often isolated when they are tested because it is more economical and convenient. However, the surrounding walls form a portion of a structure, and additional components will affect the response of the wall. A complete structural system with shear walls, floors, foundation, and roof behaves in a different manner than the sum of its components. Each component, if properly designed and constructed, can contribute to the performance of the other components and make them stronger. To fully understand how a structure reacts when exposed to any type of load, a complete system must be analyzed. The system will better predict how a structure will perform than a prediction based on results from tests performed on individual components.

Analytical methods for predicting overall performance of light-frame structures are not well developed (Schmidt and Moody, 1989), but attempts to model light-frame systems date back to Whittemore et al. (1948). Attempts at modeling an entire building have been limited. Only a few tests have been performed in the United States, Canada, Japan, and Australia. A one-story house was subjected to simulated wind and snow loads and analyzed by Dorey and Schriever (1957) under the guidance of the National Research Council of Canada. The test was performed with exterior sheathing purposely omitted from the twenty-four by thirty-six feet house. They found that the windward and leeward walls experienced ballooning deformation while the shear walls exhibited minor racking deformations (from Gupta, 1987). Results indicated that a shear wall is an integral part of the system and can resist certain types of deformations. Hurst (1965) evaluated the performance of a house during various stages of construction. The results confirmed the study performed by Dorey and Schriever (1957) and showed that wall movement under the simulated wind loads was resisted primarily by the plywood exterior wall. Contributions of the interior walls and gypsum wallboard sheathing were not able to be measured (Schmidt and Moody, 1989). Yokel et al. (1973) used simulated wind loads, up to 25 psf, to test a two story house. The walls had gypsum wallboard on both sides and diagonal bracing in the corner. The sheathing and braces added to the loads that the building could withstand and contributed to the overall stiffness of the structure.

Yancey and Somes (1973) evaluated the stiffness and strength of a housing unit typical of a factory-built module (Schmidt and Moody, 1989). They verified drift requirements that had been established for wood-framed buildings under static wind pressures of 20 pounds per square foot. Tuomi and McCutcheon (1974) considered a single story light-frame structure subjected to lateral loading during various stages of construction and sought to determine the effects of window and door openings and interior gypsum board. They determined that nails connecting the sheathing with the framing played a very important role in the overall response of the structure. Properly driven nails created a stronger structure while improperly driven nails reduced the system effect of the structure. Window and door openings were found to reduce the stiffness of the structure and also reduced strength. A simulated wind load and United States design practices were followed in a test of a two story house built by Sugiyama et al. (1976). Stiffness of the structure was found to be influenced not only by the sheathing and the sheathing to framing connectors, but also by several elements such as interior sheathing, which is often neglected in simplistic analysis. A study of a small plywood sheathed house was tested by Hirashima (1981) considered deformation due to loading in two separate directions. The house was able to resist greater loads than a simple shear wall of the same length as the house.

Nelson et al (1985) sought to test the behavior of interior shear walls when the actions of the adjacent systems (i.e., floor, roof, adjacent wall) were included. The tests focused on the length of the wall, its location within the assembly, number of joists beneath, and number of sides onto which paneling was connected. Size and location of the shear wall within the building assembly did affect the ultimate strength while the addition of hardwood paneling on both sides as opposed to one side did not have a significant effect. The effect of the number of floor joists under the wall was inconclusive. Gupta and Kuo (1987) developed a realistic but simple numerical model with an emphasis on perfecting the shear wall behavior of the house. The shear wall was deemed as the most important element for providing stiffness and strength to the house against lateral wind and earthquake loads. The Gupta and Kuo model was in agreement with the Tuomi and McCutcheon (1974) model that was developed eleven years earlier. Bulleit (1987) developed a model based on the Markov model that sought to give the ultimate capacity for a system. The model makes one assumption that cannot be considered as being conservative and must be researched further. The assumption that only one member from the system may fail per any one loading cycle is un-conservative.

Laterally loaded light-frame buildings were the subject of research performed by Schmidt and Moody (1989). They tried to model the composite action of the fasteners, sheathing, and framing but were unable to model the system in three dimensions. They modeled openings as full height, with no partial panels were installed over or under windows and doors. Restraint of panel deformation from frictional and bearing contact with the floor, ceiling, or adjacent panels was ignored. The walls were only subjected to racking loads and were attached to the diaphragm at a single point. The model was named RACK3D and was used to predict the nonlinear response to lateral loading of three-dimensional light-frame structures in which shear walls are the primary load-carrying mechanism. A brittle failure mechanism caused by cyclic loading was discovered by Oliva (1990) that was hidden in static tests. Suzuki (1990) showed that adequately connected cross walls can increase the lateral stiffness of a structural system. McDowall (1991) tested the creep response of an engineered whole house in an effort to predict the effect that it would have on the static and dynamic response of the system and in particular the floor.

A full scale building, the Taisekiji Mutsubo temple, built in Japan utilizing the traditional Japanese woodworking techniques as well as some modern construction techniques, was tested under horizontal load and vibration tests. Kawachi et al. (1990) performed the tests and analyzed the results to determine the effect that shear walls have on an entire structure. The test was unique in that it attempted to predict how an actual building, under construction, would perform. Results showed that the placement of a shear wall at the end of the structure caused a significant increase in the stiffness of the system. Test results also showed the effectiveness of modern construction procedures in resisting vibration.

2.6 Wall Foundation

Often in research it is assumed that a shear wall is supported by a relatively rigid foundation, but situations occur when shear walls are supported by non-rigid foundations such as floor joists or girders, most likely resulting in a reduction in stiffness. The effect of different types of hold-downs was briefly examined by Hutchinson and Thurston (1984). Hold-downs are used to resist the uplift forces induced in walls by lateral forces. The research consisted of shear walls sheathed with particle board and plywood that were connected to the foundation in four different manners. There were two connection configurations for the particle board walls and two for the plywood sheathed walls. The first connection consisted of anchor bolts set in concrete but due to cracking of the concrete foundation the results were disregarded. The second model had the hold down bolts welded to a steel channel that was attached to the wall. The two plywood walls were connected to the foundation via either a steel strap attached to the projecting bearer from the foundation or by welding nails on strips to a steel connector. The first plywood connection (i.e. the steel strap) exhibited shear failure in the bearing plate with nail on strips tying end studs to the bottom plate. The stiffness of foundation connections were observed to have an effect on the deformation of the wall. Stiffer connections exhibited smaller deflections. The effect of foundations was briefly examined by the National Association of Home Builders (1990) when they conducted tests of shear walls in mobile homes. Results from the tests showed that walls resting on flexible foundations could only support seventy percent of the load supported by a wall resting on a rigid foundation. Phillips (1990) conducted tests that showed an anomaly in the stiffness characteristics that could be the result of differing foundation stiffness. Tests were inconclusive and could not accurately show that the foundation stiffness caused the anomaly.

Chords of shear walls are most often attached to the foundation by nailing or bolting through the bottom plate of the wall. These connections are termed hold-downs and serve to transfer forces in the chords of the shear wall into the foundation. The size of the hold-downs may possibly have an effect on the response of the wall but research has not focused on this. Hold-downs are very important in the considerations of the effect of foundation stiffness on the response of the wall in that the hold-downs are responsible for preventing uplift at the chords of the wall. Stewart et al (1982) examined the results of dynamic test that have been performed on hold-down brackets. Results of the study concluded that the addition of stiffness to a foundation increased the seismic resistance for a wall. Conclusions also stated that more research needs to be conducted on the detailing of the hold-down connections that transfer and resist overturning moments and base shears. Zacher and Gray (1990) demonstrated that hold-downs bolted to the end stud attached in single shear are unsatisfactory and hold-downs should be placed in double shear.

Chapter 3

Research Methodology

3.1 Introduction

Research performed on shear walls to the present has always assumed a rigid foundation. However, this is not always the case, such as when a wall is supported by floor joists and/or is not directly above a foundation wall. Therefore, a study was performed to examine the effects that non-rigid foundations have on the static and dynamic response of timber shear walls. This study was performed by creating a numerical wall model, varying the conditions at the foundation of the model, subjecting the model to monotonic and dynamic loading for each foundation condition, and analyzing the various systems using WALSEIZ1, which is a computer program developed specifically for the analysis of timber shear wall models. The effect of foundation stiffness on the following items were examined: maximum horizontal and vertical displacements, maximum seismic base shear, monotonic strength of the wall, and the state of the forces in the wall.

3.2 Analysis Program

WALSEIZ1, which is a modified version of WALSEIZ (White and Dolan, 1995), was the computer program utilized for the study. The program was selected because of its economical benefits when compared to the traditional methods of analysis for timber shear walls, and WALSEIZ1 was chosen because of its ability to provide a detailed force and stress analysis. Analysis of shear walls traditionally has involved constructing and then testing physical walls, often to failure, and then analyzing the results. A significant number of experimental tests would require a substantial financial investment, and to obtain a detailed stress analysis would require a tremendous amount of instrumentation. Computer models allow one person to analyze several walls in a relatively short amount of time and without costly material, labor, and space requirements. With computer models, if new tests with different wall configurations wish to be tested, slight modifications only need to be made to the database and the computer needs to be given time to operate.

WALSEIZ is capable of analyzing shear walls subjected to static, monotonic, or dynamic loading and produces comparable results to those obtained from shear walls tested experimentally (White and Dolan, 1995). It has the capability to calculate nodal displacements, velocities, and accelerations, along with element forces and stresses, as a function of applied load for monotonic loading or as a function of time for dynamic loading. It also has the capability to determine the maximum and minimum nodal displacements, velocity, acceleration, and element forces. WALSEIZ1 has these same capabilities, along with additional logic which allows for the modeling of a non-rigid foundation using linear springs and the modeling of the nails used to attach the framing members using bi-linear springs. The following assumptions were made in the development of the program:

1. The sheathing does not deflect out of plane. This is based on research results concluding out of plane displacements were insignificant in sheathing with dimensions greater than 9.5 mm (3/8 in) and framing spaced less than 610 mm (24 in) on center which are subjected to

lateral loads only (Dolan,1989).

2. Sheathing materials are non-layered orthotropic materials. This assumption simplifies the element, and because the sheathing is loaded in plane, the effect of layering is negligible (White, 1995).

3. Eccentricity in the shear wall due to sheathing being present on only one side of the wall is neglected.

4. Framing members are isotropic, linear-elastic, and homogeneous, while the sheathing is orthotropic, linear-elastic, and homogeneous.

3.2.1 Components

WALSEIZ1 utilizes elements corresponding to framing, sheathing, connectors attaching sheathing to the framing, bearing between adjacent sheathing panels, connectors attaching the framing, and the hold downs/anchor bolts at the foundation of a shear wall. The elements used to model framing and sheathing were formulated by approximating a displacement field, utilizing compatibility and constitutive laws, and imposing equilibrium by means of the principle of virtual work and D'Alembert's principle. The remaining elements were formulated by imposing equilibrium by means of the principle of virtual work.

A common two node, linearly elastic element with three degrees-of-freedom at each node, henceforth referred to as a framing element, is used to model the framing (see Figure 3.1). The

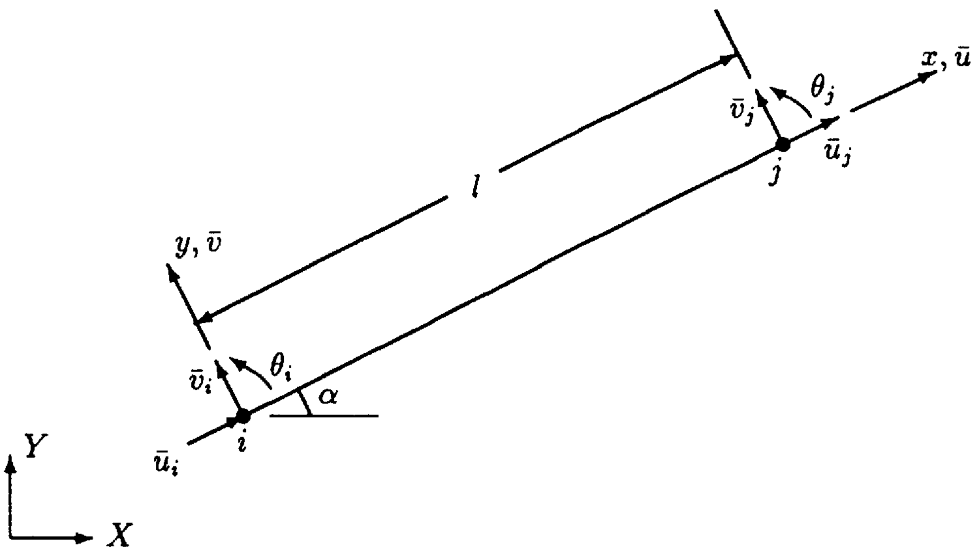

Figure 3.1: Framing Element (White, 1995)

degrees-of-freedom correspond to axial, transverse, and rotational displacements. Shear deformation is neglected in the formulation of the element because the majority of shear force in a shear wall is resisted by the sheathing, and therefore the shear deformation in the framing is minimal. A linear displacement field is utilized for the axial displacements and a cubic displacement field for the transverse displacements. The compatibility or strain displacement relationships are governed by the equations (Jenson, 1983):

$$\epsilon_a = \frac{d\bar{u}}{dx} \qquad (3.1)$$

$$\epsilon_b = -y\frac{d^2\bar{v}}{dx^2}$$

where ϵ_a is the axial strain, ϵ_b is the bending strain, y is the distance from the neutral axis of a cross-section to a point on the cross-section of the beam, $\bar{u}$ is the axial displacement of the element at a point, and $\bar{v}$ is the transverse displacement of the element at a point. The constitutive law is governed by Hooke's law. Equilibrium is then imposed by applying the principle of virtual work along with D'Alembert's principle. This leads to the following relationship (Ugural, 1981):

$$[m]\{\ddot{x}\}+[c]\{\dot{x}\}+[k]\{x\}=\{F\} \tag{3.2}$$

where {F} is the vector containing the element end forces, [m] is the mass matrix, [c] is the viscous damping matrix, $\{\ddot{x}\}$ is the vector containing the nodal acceleration, $\{\dot{x}\}$ is the vector containing nodal velocities, [k] is the stiffness matrix, and $\{x\}$ is the vector containing the nodal displacements. A detailed derivation of the element can be found in Cook et al. (1989).

Sheathing is modeled using a common four node plane rectangular bi-linear element with two degrees-of-freedom at each node (see Figure 3.2). The four node quadrilateral, henceforth referred to as a sheathing element, is being used because the sheathing has a high in-plane stiffness, and typically only minute out-of-plane deflections are expected from the sheathing (Dolan, 1989).

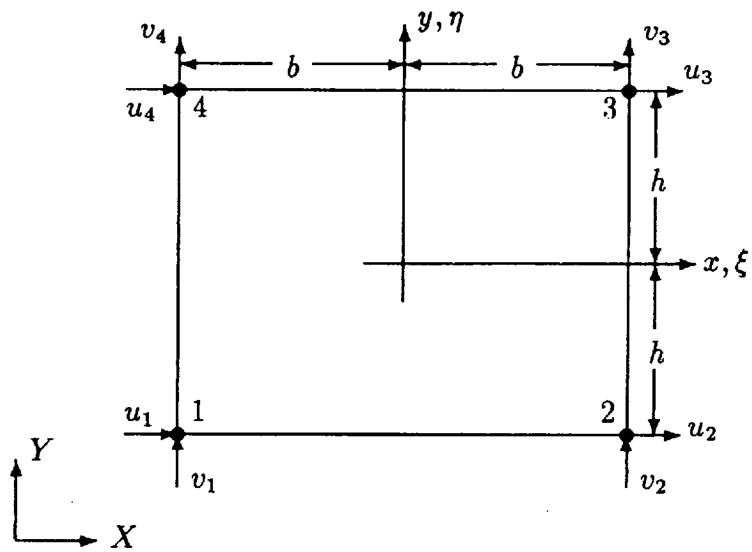

Figure 3.2: Sheathing Element (White, 1995)

A linear displacement field is utilized for displacements along both x- and y-axes. This element was also derived by applying compatibility, constitutive laws, and equilibrium to the approximated displacement fields. Compatibility relationships are given by the following equations (Ugural, 1981):

$$\begin{aligned} \epsilon_x &= \frac{\partial \tilde{u}}{\partial x} \\ \epsilon_y &= \frac{\partial \tilde{v}}{\partial y} \\ \epsilon_{xy} &= \frac{\partial \tilde{u}}{\partial y} + \frac{\partial \tilde{v}}{\partial x} \end{aligned} \tag{3.3}$$

where ϵ_x is the strain in the x-direction, ϵ_y is the strain in the y-direction, ϵ_{xy} is the shear strain, and $\tilde{u}$ and $\tilde{v}$ are displacements along the x and y axes, respectively. Constitutive laws are given by the following expressions:

$$
\begin{aligned}
\sigma_x &= \frac{E_x}{H}\epsilon_x + \frac{E_x \nu_{xy}}{H}\epsilon_y \\
\sigma_y &= \frac{E_y \nu_{yx}}{H}\epsilon_x + \frac{E_y}{H}\epsilon_y \\
\sigma_{xy} &= G_{xy}\epsilon_{xy}
\end{aligned}
\tag{3.4}
$$

where, E_x and E_y are the moduli of elasticity in the x- and y- directions respectively, ν_{xy} and ν_{yx} are poisson's ratios giving a strain in x- or y- direction when there is a stress in the y- or x- direction, G_{xy} is the shear modulus of elasticity, and H is equal to $1-\nu_{xy}\,\nu_{yx}$. Equilibrium is imposed by the use of the principle of virtual work and D'Alembert's principle, resulting in the relationship:

$$[m]\{\ddot{x}\}+[c]\{\dot{x}\}+[k]\{x\}=\{F\} \tag{3.5}$$

The terms from equation 3.5 are the same as were defined in equation 3.2.

A set of two perpendicular non-linear springs, one acting parallel to the X-axis and the second acting parallel to the Y-axis, are used to model the connectors that attach sheathing to the framing. The elements, henceforth referred to as connector elements, utilize the degrees-of-freedom of the sheathing and the framing, and therefore do not contain any independent degrees-of-freedom. The load-displacement relationships for the elements, which are shown in Figures 3.3 and 3.4, were established by curve-fitting experimental data (Dolan, 1989). The curves are governed by the following relationships when subjected to monotonic loading or for deflections larger than previously achieved during a dynamic analysis:

$$|F_{con}| = (P_o + K_1|\Delta|)[1 - exp(\frac{-K_2|\Delta|}{P_o})]$$

$$|F_{con}| = (P_o + K_1|\Delta_{max}|)[1 - exp(\frac{-K_2|\Delta_{max}|}{P_o})] - K_3(|\Delta| - |\Delta_{max}|) \qquad (3.6)$$

and by these relationships when subjected to reversing loading at displacements smaller than the maximum achieved previously during the analysis:

$$\begin{aligned}
F_{con(1)} &= -P_1 + K_4\Delta + [exp(a_1\Delta) - 1] \\
a_1 &= \frac{ln(F_1 + P_1 - K_4u_1 + 1)}{u_1} \\
F_{con(2)} &= -P_1 + K_4\Delta - [exp(a_2|\Delta|) - 1] \\
a_2 &= \frac{ln(-F_2 - P_1 + K_4u_2 + 1)}{u_2} \\
F_{con(3)} &= P_1 + K_4\Delta - [exp(a_3|\Delta|) - 1] \\
a_3 &= \frac{ln(-F_2 + P_1 + K_4u_2 + 1)}{u_2} \\
F_{con(4)} &= P_1 + K_4\Delta + [exp(a_4\Delta) - 1] \\
a_4 &= \frac{ln(F_1 - P_1 - K_4u_1 + 1)}{u_1}
\end{aligned} \qquad (3.7)$$

The load envelope is governed by the same relationship as the connector subjected to monotonic loading. The element is formulated by establishing equilibrium by use of the principle of virtual work, resulting in the relationship:

$$\{r^{int}\}=\{F\} \qquad (3.8)$$

where, $\{r^{int}\}$ is the internal force vector for an element and $\{F\}$ represents the nodal forces on the framing and sheathing elements which are attached.

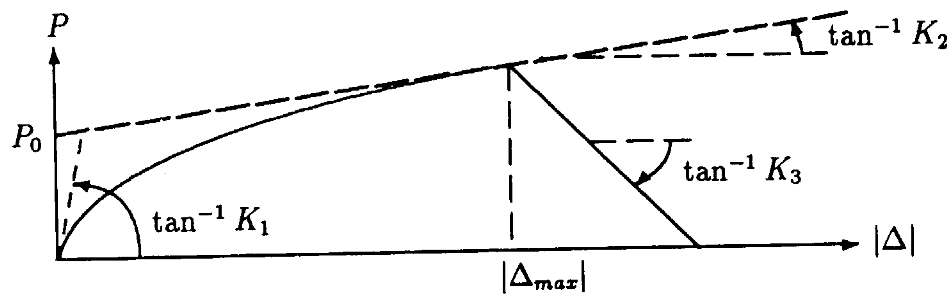

Figure 3.3: Load Displacement Curve for the Connector Elements Subjected to Monotonic Loading (Dolan, 1989)

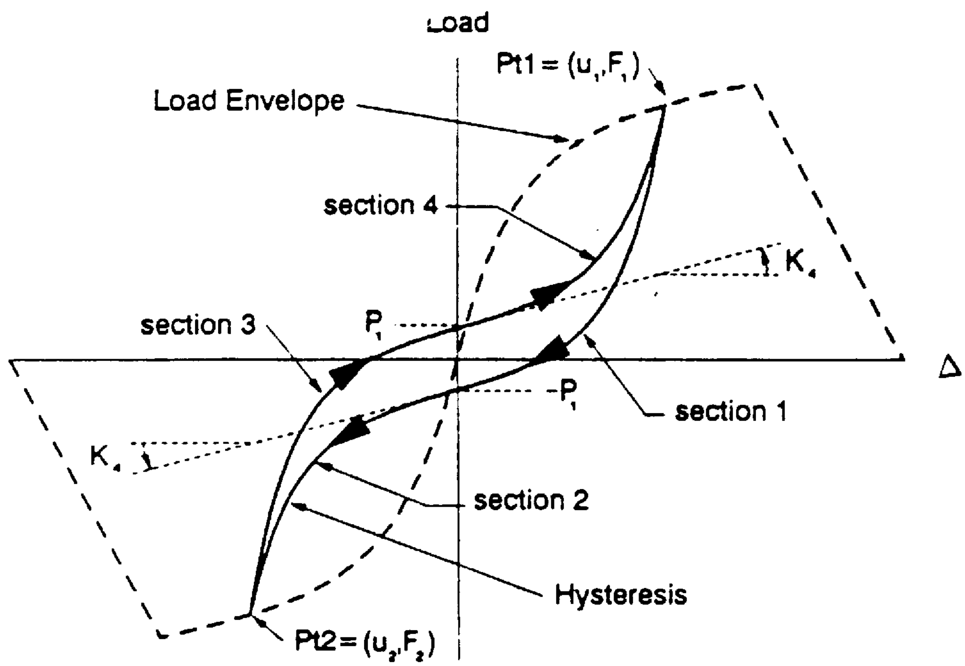

Figure 3.4: Load Displacement Curve for the Connector Elements Subjected to Dynamic Loading (Dolan, 1989)

A bi-linear spring, henceforth referred to as a bearing element, is used to model the effect of one sheathing panel bearing against another sheathing panel. Without the bearing element, the stiffness of the sheathing would be underestimated due to a mathematical overlap of the sheathing created by the lack of resistance in the model (Dolan, 1989). The bi-linear spring has a high stiffness in compression and low stiffness in tension. Therefore, the sheathing panels are allowed to move relatively freely when they do not bear against one another due to the low tensile stiffness of the element, and are not permitted to overlap because of the high compressive stiffness of the element. The element has no independent degrees-of-freedom, but instead utilizes the degrees-of-freedom corresponding to the two adjacent sheathing elements. The element is formulated by establishing equilibrium by use of the principle of virtual work, resulting in the relationship:

$$[k]\{d\}=\{F\} \tag{3.9}$$

where [k] is the stiffness matrix, {d} is the vector containing the displacements of the sheathing elements, and {F} is the vector containing the element end forces for the sheathing elements. Figure 3.5 shows the load versus displacement relationship for the bearing element.

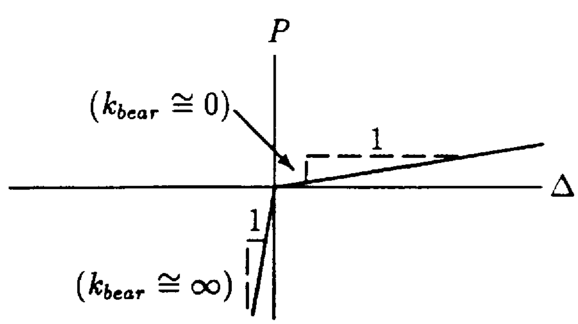

Figure 3.5: Load Displacement Relationship for Bearing Element (White, 1995)

Linear and bi-linear springs were incorporated into the program to model the hold-downs and the connections between framing components. The element was derived by the same principles as the bearing element, with the exceptions that the degrees-of-freedom corresponding to the two adjacent framing elements, not sheathing elements, were used and the spring could be either linear or bi-linear. The bi-linear load-displacement pattern allowed for the low withdrawal resistance for nails inserted into the end grain of a member to be modeled.

Compatibility, constitutive laws, and equilibrium are also used to formulate the equations of motion for the entire shear wall model. Constitutive laws are inherent in the element models and therefore are already imposed for the system model (Holzer, 1990). Compatibility and equilibrium conditions are imposed by the use of the member code derived by Tezcan (1963). Tezcan developed a way to take the individual matrices of each element and combine them to form a system matrix that pertains to the entire model. The final equation of equilibrium is given as:

$$[M]\{\ddot{X}\} + [C]\{\dot{X}\} + \{R^{int}\}=\{F\} \qquad (3.10)$$

where [M] is the system mass matrix, $\{\ddot{X}\}$ is a vector containing the nodal accelerations, [C] is the system viscous damping matrix, $\{\dot{X}\}$ is a vector containing the nodal velocities, $\{R^{int}\}$ is the internal force vector, and {F} is the vector with the applied nodal loads.

3.2.2 Verification and Validation of WALSEIZ1

Verification and validation of WALSEIZ consisted of the following: (1) verification and validation of each element by subjecting an element to a load and comparing the resulting displacements with theoretical results; (2) verification of a wall model by creating identical shear wall models in WALSEIZ and ABAQUS, analyzing the models, and comparing the results; and (3) validation of a wall model by creating and analyzing shear wall models that corresponded to shear walls tested experimentally, and comparing the resulting load-displacement curves and maximum loading for monotonic loading, and time-displacement curves for dynamic loading (White and Dolan, 1995). The results of the verification and validation tests were favorable (White and Dolan, 1995), thereby indicating that the WALSEIZ could adequately model shear walls. Given that the only significant difference between WALSEIZ and WALSEIZ1 is the addition of the frame connector elements, only the following tests were performed to verify and validate WALSEIZ1: (1) verification and validation of the frame connector element, (2) verification of the elements used in both WALSEIZ and WALSEIZ1, and (3) validation of a wall model analyzed using WALSEIZ1.

The first test was performed to ensure that the frame connector element is performing as expected. This was done by constructing a couple of simple framing configurations and subjecting them to a concentrated load. One configuration consisted of two framing elements in a line that were connected by a frame connector element, and the other consisted of a cantilever beam attached to a column with a frame connector element (see Figure 3.6). One end of the frame was fixed, and the other end was subjected to a transverse and axial load of 445 N (100 lb), as is shown in Figure 3.6. The direction of the axial loading was reversed to determine if the bi-linearity of the connector elements held true to form. Displacement of the frame connector element equaled the stiffness of the element divided by the load, thereby indicating that the element was performing as intended.

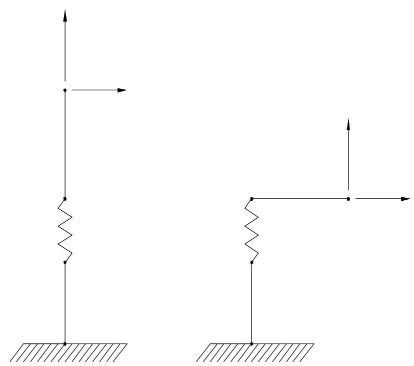

Figure 3.6: Configurations Used to Verify the Frame Connector Element

The second test was performed to ensure that the modification of the program did not adversely affect the performance of elements used in both WALSEIZ and WALSEIZ1. This test was performed by constructing identical shear wall models and analyzing them in WALSEIZ and WALSEIZ1. The models, shown in Figure 3.7, were similar to the models used to validate WALSEIZ (White and Dolan, 1995), with the difference being that hinges are not included in the model because the option for hinges was eliminated from WALSEIZ1. This was done because the frame connector elements were used to model the low torsional resistance of the framing at the connections. The wall model corresponded to 2.4 m by 2.4 m (8 ft by 8 ft) shear walls tested by

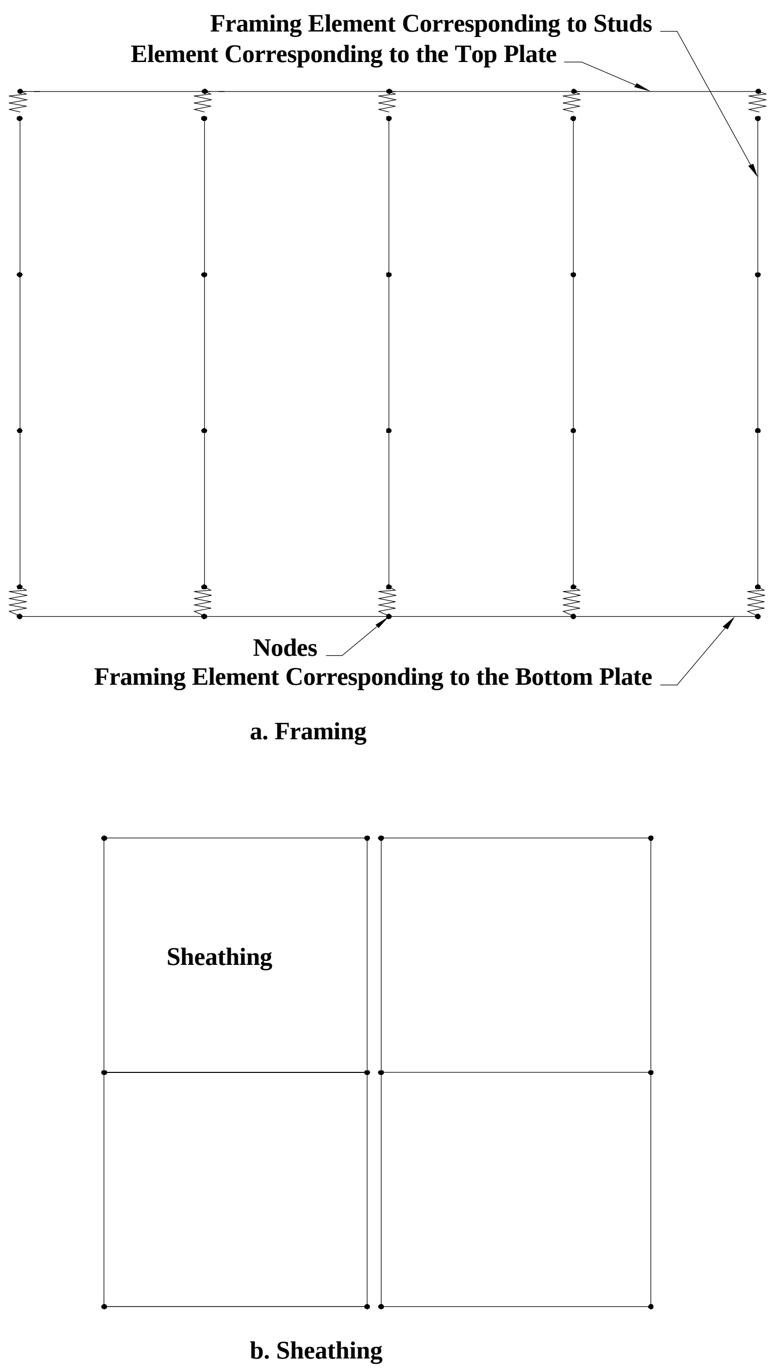

Figure 3.7: Wall Model Used to Verify WALSEIZ1

Dolan (1989). Framing of the walls consisted of studs spaced 61 cm (24 in) apart, which were attached to the header and sill framing members by nailing through the header or sill plate into the end-grain of the stud using two 76 mm (10d or 3 in) common nails. All of the framing was made of 38 x 89 mm (2 x 4 in nominal) Spruce-Pine-Fir ranging in moisture content from 9-14%. The wall was sheathed with 9.5 mm (3/8 in) A-5 Exterior CF BC 142 grade Canadian softwood plywood (CSP) sheathing. The sheathing was attached to the framing with 8d or 63.5 mm (2.5 in) hot dipped galvanized common nails spaced 10 cm (4 in) along the perimeter of a sheathing panel and 15 cm (6 in) along the interior of a sheathing panel. The walls were anchored using steel hold-down connections located at the bottom of the end chords. The asymmetry, due to the sheathing being applied on one side, was not a factor because the wall was restrained by steel tubes at the top and bottom of the wall, which prevented any twisting of the wall.

Framing of the wall models consisted of two series of framing elements each 61 cm (24 in) long which extended the length of the model, one series forming the sill plate and the other the sole plate. Studs and end chords were formed using four framing elements, each 61 cm (24 in) long. Geometric and material properties for the different categories of elements are shown in Table 3.1, where the width of the element is the dimension perpendicular to the face of the sheathing and the depth of the dimension parallel to the face of the sheathing. These dimensions and material properties were chosen to best represent the framing used in the shear wall tests. The sole plate of each model was fixed at the base of each of the studs and the chords. The high density of the top plate modeled the inertial mass associated with the upper stories of a structure. A mass of identical magnitude was used in the experimental tests.

Table 3.1: Geometric and Material Properties for the Framing Elements

Framing Component	Width	Depth	Modulus of Elasticity	Density
	mm (in)	mm (in)	N/cm^2 (lb/in^2)	kg/cm^3 (sl/in^3)
Top plate	89 (3.5)	76 (3.0)	$1.09E^6$ ($1.58E^6$)	$3.29E^{-1}$ ($3.70E^{-1}$)
Bottom plate, Studs	89 (3.5)	38 (1.5)	$1.09E^6$ ($1.58E^6$)	$2.07E^{-4}$ ($2.32E^{-4}$)
End Chords	89 (3.5)	76 (3.0)	$1.09E^6$ ($1.58E^6$)	$2.07E^{-4}$ ($2.32E^{-4}$)

Each sheathing panel consisted of two 1.2 m by 2.4 m (4 ft by 8 ft) sheathing elements that were 9.1 mm (0.361 in) thick. The material properties for the sheathing elements, which were similar for monotonic and seismic loading and based on the design properties for the two materials, were as follows: $E_x t$ = 241306.5 N/cm^2 (349991.0 lb/in^2), $E_y t$ = 171497.5 N/cm^2 (248740.0 lb/in^2), $E_x \nu_{xy} t/H$ = $E_y \nu_{yx} t/H$ = 7862.9 N/cm^2 (11404.4 lb/in^2), $G_{xy} t$ = 24889.7 N/cm^2 (36100.0 lb/in^2), density (ρ) = $6.00E^{-5}$ kg/cm^3 ($6.74E^{-5}$ $slugs/in^3$). The connectors attaching sheathing to the framing, spaced 10 cm (4 in) along the perimeter of the sheathing and 15 cm (6 in) along the interior of the sheathing, were modeled using connector elements governed by the following properties: K_1 = 9559.8 N/cm (5458.8 lb/in), P_o = 1035.2 N (232.7 lb), K_2 = 372.1 N/cm (212.4 lb/in), Δ_{max} = 1.27 cm (0.5 in), K_3 = -875.6 N/cm (-500 lb/in) , P_1 = 333.6 N (75 lb). The properties of the connectors for the walls were based on experimental tests performed by Dolan (1989).

The wall models were subjected to load-controlled monotonic distributed loading applied to the sill plate at increments of 0.88 kN/m (5.0 lb/in), and a dynamic loading of 30 seconds of the acceleration record from the S69E component of the 1954 Kern County, California, earthquake. Loading was in the plane of the walls, as shown in Figure 3.8. Results for the displacements, forces, etc. were identical for both programs, indicating that the addition of the frame connector element did

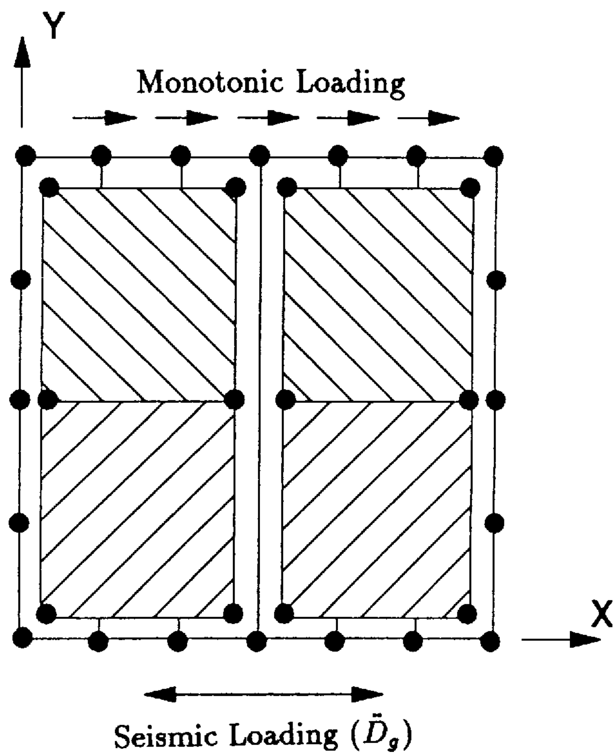

Figure 3.8: Loading Applied to Shear Wall Model (White, 1995)

not adversely affect the performance of the remaining elements.

The third test was performed to ensure that the modified model adequately predicts the response of a timber shear wall subjected to monotonic and dynamic loading. The validation test for

WALSEIZ1 was performed by constructing a model similar to plywood wall model outlined above that was used to verify WALSEIZ, with the difference being that framing connector elements were used in the model. Two sets of framing connector elements were used, one to model the bolts and hold-downs anchoring the wall to the foundation, and the other used to model the nails attaching the top and bottom plates to the chords. Properties of these members, which are listed in Table 3.2, were determined as follows. Lateral stiffness for the anchorages was determined based on the initial stiffness of a 12.7 mm (0.5 inch) diameter bolt attaching a metal plate to a wood side member (Stelmokas, 1995). Vertical stiffness was determined based on the relationship k=EA/L, where E is the modulus of elasticity of the hold-down material, A is the cross-sectional area of the hold-down, and L is the effective length of the hold-down. Rotational stiffness was arbitrarly chosen to reflect the low torsional resistance of the anchorage. Lateral stiffness for the nails was determined based on the initial stiffness of two 76 mm diameter (10d or 3 in diameter) nails attaching two wood members together (Foschi and Bonac, 1977). Vertical stiffness was determined based upon the allowable bearing stress for Spruce-Pine-Fir in compression and was taken as an arbitrarily low value in tension to reflect the low resistance of a nail inserted into the end grain of the main member. Because of the difficulty of the solution to converge when the a bilinear spring was used for dynamic loading, a linear elastic spring was used for the nails for dynamic loading, with the stiffness of the spring being equaling that used in compression. The rotational stiffness was arbitrarly chosen to reflect the low torsional resistance of the nail.

Table 3.2: Properties for the Framing Connector Elements

Component	K_X	K_Y	K_θ
	N/mm (lb/in)	N/mm (lb/in)	N/rad (lb/rad)
Anchorage	$8.8E^3$ ($5.0E^4$)	$3.5E^5$ ($2.0E^6$)	89 (20)
Nail[1]	$8.8E^2$ ($5.0E^3$)	$1.8E^4$ ($1.0E^5$)	44 (10)

[1]For a wall subjected to monotonic loading, K_y = 1.0 in tension

Results of the tests are as follows. A comparison between the experimental monotonic load-displacement curves and curves generated from the program, shown in Figure 3.9, reveals a correlation coefficient of 0.998, which is an excellent comparison. A correlation between the time-displacement curves of the results from WASLEIZ and WALSEIZ1, shown in Figure 3.10, has a correlation coefficient of 0.978, which would indicate that the program is performing as expected. These results would seem to indicate that the program WALSEIZ1 adequately predicts the response of a timber shear wall subjected to monotonic or dynamic loads.

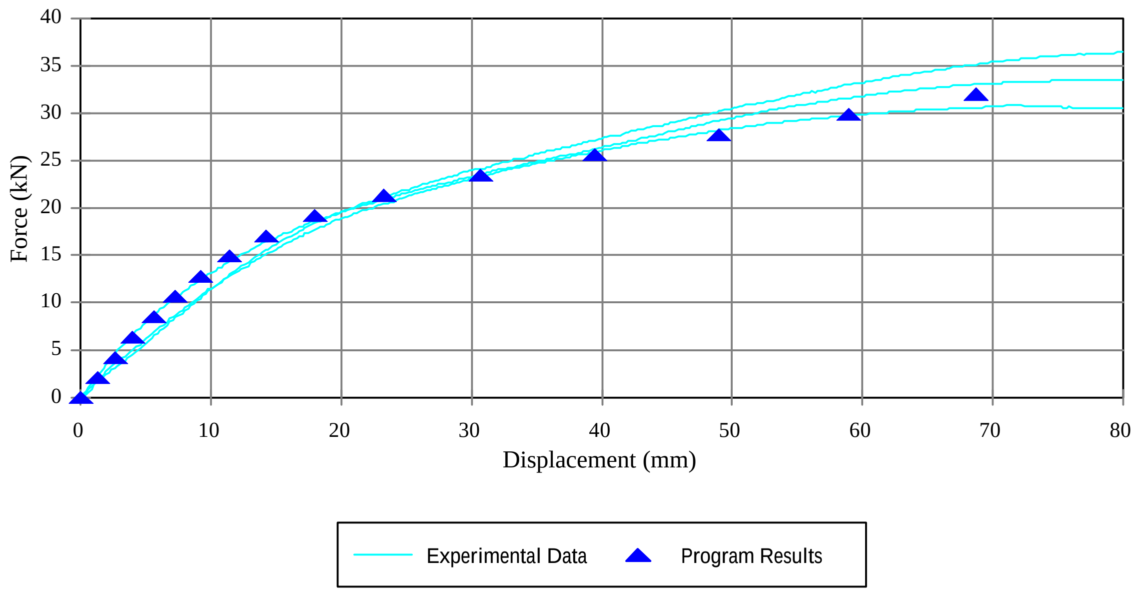

Figure 3.9: Monotonic Load-Displacement Curve from Validation Test for WALSEIZ1

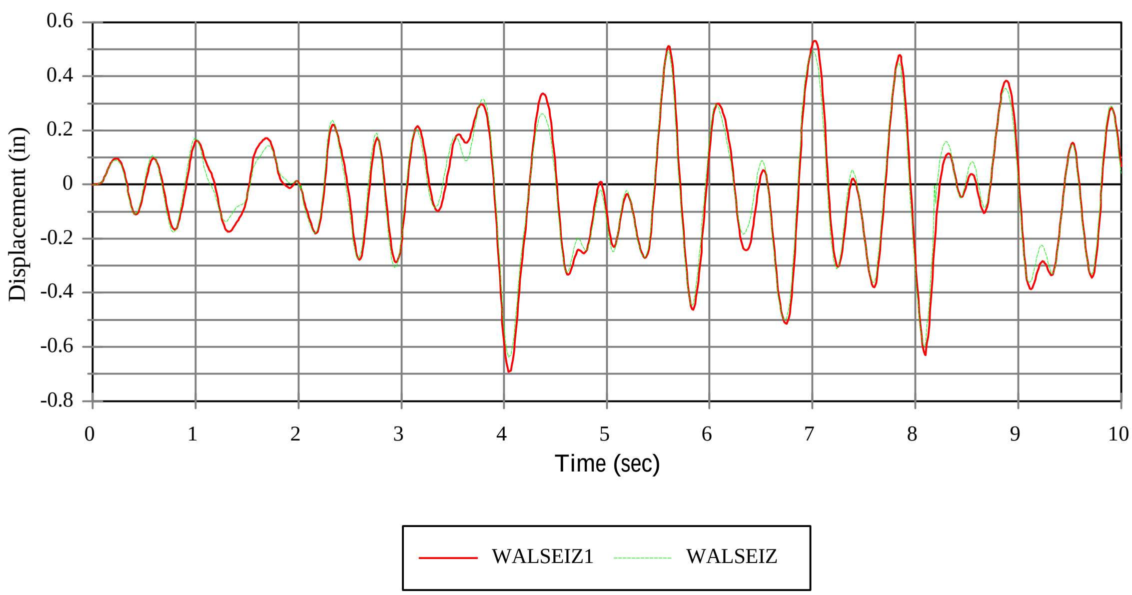

Figure 3.10: Dynamic Time-Displacement Curve from Validation Test for WALSEIZ1

3.3 Shear Wall Models

A 2.4 m by 3.7 m (8 ft x 12 ft) shear wall model was created and analyzed for the study. The wall model corresponded to a common light-frame shear wall with 9.5 mm (3/8 in) thick A-5 Exterior CF BC 142 grade Canadian softwood plywood (CSP) sheathing attached to one side of Spruce-Pine-Fir framing with 8d common nails, which are 63.5 mm (2.5 in) long and 3.33 mm (0.131 in) in diameter. Assumptions in the wall model include:

1. The transverse walls adjacent to the shear wall have no effect on the response of the wall.
2. The load displacement properties of all of the sheathing-to-framing connectors are identical.

The first assumption corresponds to the manner in which most shear wall testing is currently performed and also reflects the lower out-of-plane versus in-plane stiffness for walls. The second assumption was based on the premise that given a large number of connectors, the properties of the individual connectors will "average out" over an entire wall. It also simplifies the analysis.

The wall model (Figure 3.11) is either supported by a 7.3 m (24 ft) long joist that is parallel to the wall, as is shown in Figure 3.12, or a rigid foundation (Figure 3.13), and has hold-down anchors at the chords. One chord of the shear wall is located at a support, the other chord is at the center of the joist. The joist is supported only at its ends. A total of eleven joist sizes and three hold-down sizes were used at the foundation. The following assumptions were made for the foundation:

1. Resistance provided by the floor sheathing is negligible.
2. The wall sheathing does not bear against floor sheathing.
3. The bottom plate of the wall is firmly (rigidly) attached to the joist.

The wall model was subjected to both a distributed monotonic load along the top plate of the wall and seismic loading for each foundation condition.

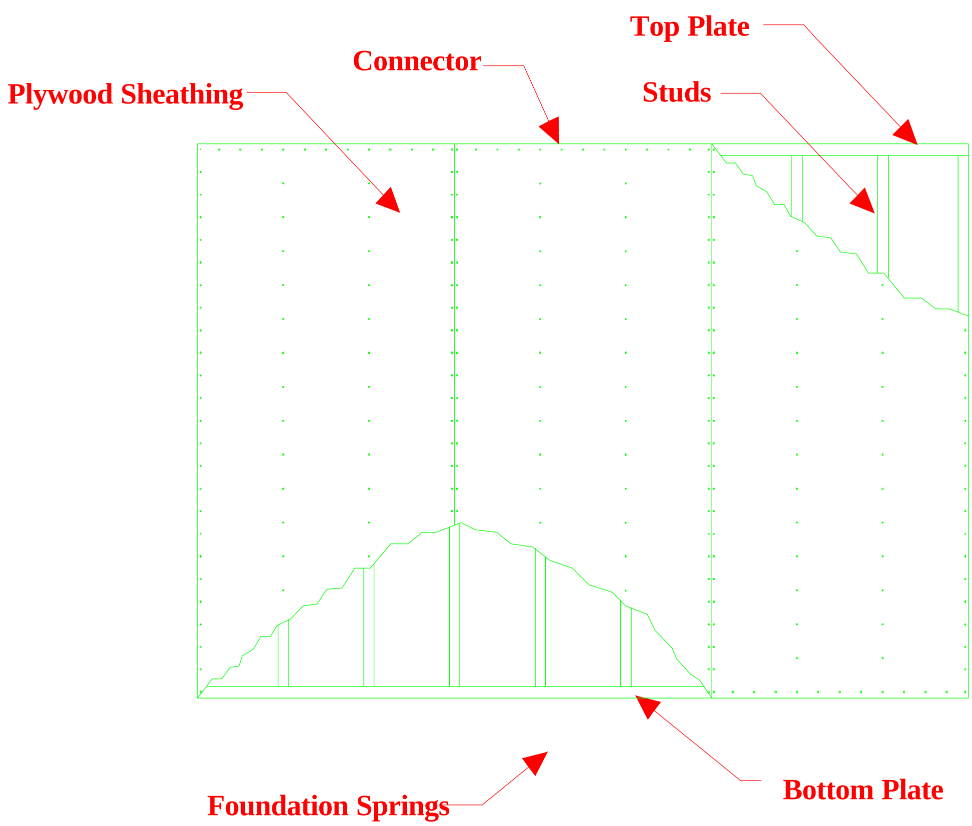

Figure 3.11: 2.4 m x 3.7 m Shear Wall Model

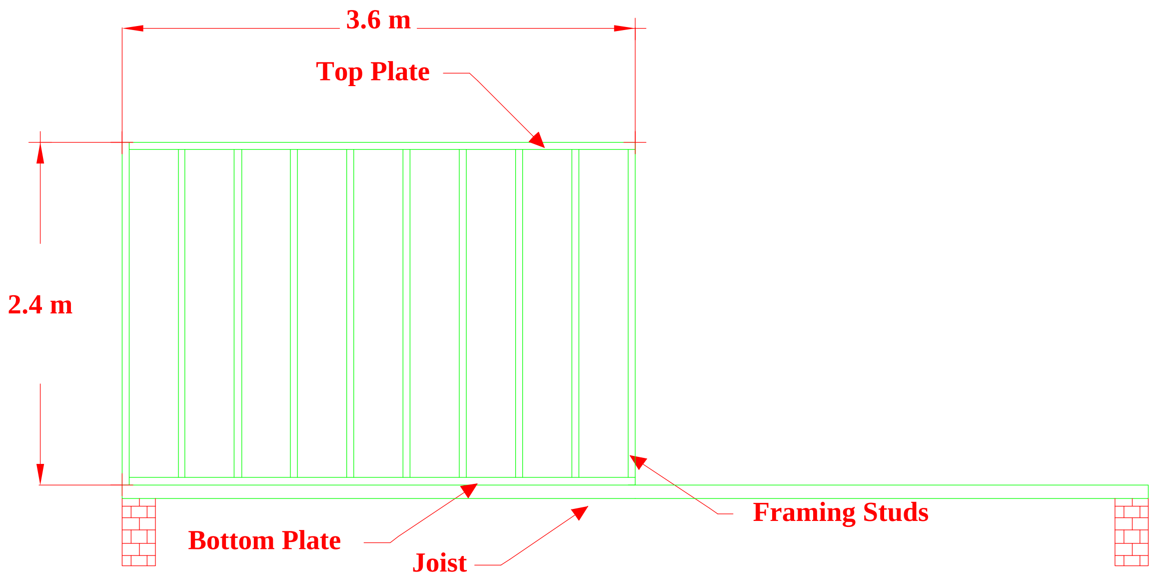

Figure 3.12: Shear Wall Model on a Flexible Foundation

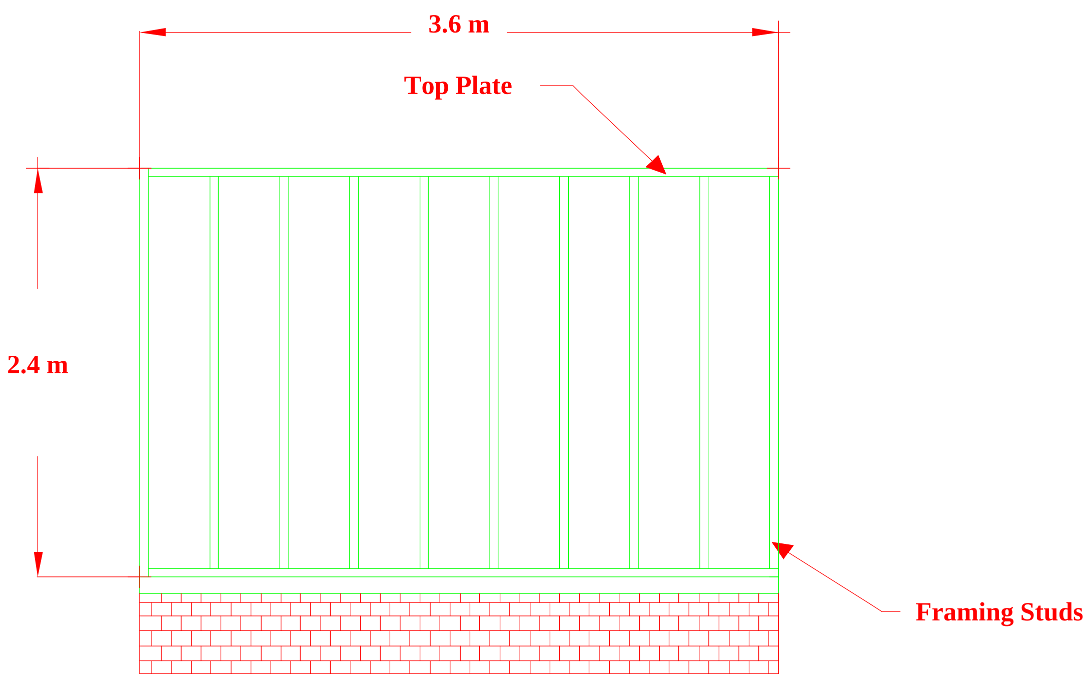

Figure 3.13: Shear Wall Model on a Rigid Foundation

3.3.1 Wall Model Geometry

The wall model utilized in the study was 2.4 m (8 ft) in height and 3.7 m (12 ft) in length. The layout of the framing and sheathing elements for the model is shown in Figure 3.14. The framing consisted of studs spaced 406 mm (16 in) on-center and capped with top and bottom plates. Nails are driven through the top and bottom plates into the end grain of the studs. Framing was modeled by three framing elements of length 813 mm (32 in) each for the studs, framing elements of length 406 mm (16 in) each for the top and bottom plates, and a frame connector element to model the connection between the studs and the top and bottom plates. All of the framing corresponds to 38 mm by 89 mm (2 in by 4 in nominal) Spruce-Pine-Fir, except for the chords and the top plate which corresponds to two 38 mm by 89 mm members nailed together. Properties of the framing elements and frame connector elements are identical to those used to validate the model.

Each sheathing panel is 1.2 m x 2.4 m (4 ft x 8 ft) and oriented such that the long dimension of the panel is vertical, as shown in Figures 3.11. The panels were modeled using two 2.4 m x 2.4 m (4 ft x 4 ft) sheathing elements, as shown in Figure 3.14. Sheathing element material properties that were used correspond to Canadian softwood plywood, and are identical to those used to validate WALSEIZ1.

Shear wall models that were analyzed did not vary in respect to connector configurations. Connectors attaching the sheathing to the framing had identical load-displacement properties and spacing for all models. Connectors modeled correspond to 8d or 63.5 mm (2.5 in) in length by 3.33 mm (.131 in) diameter, hot dipped galvanized common nails. These nails were spaced at 101.6 mm (4 in) intervals along the exterior portion of each sheathing panel and 152.4 mm (6 in) along the interior of each sheathing panel (see Figure 3.15). The load-displacement relationships for the connectors were the same that were used to validate the WALSEIZ and WALSEIZ1 programs. The properties for the connector elements were based on data collected by Dolan (1989).

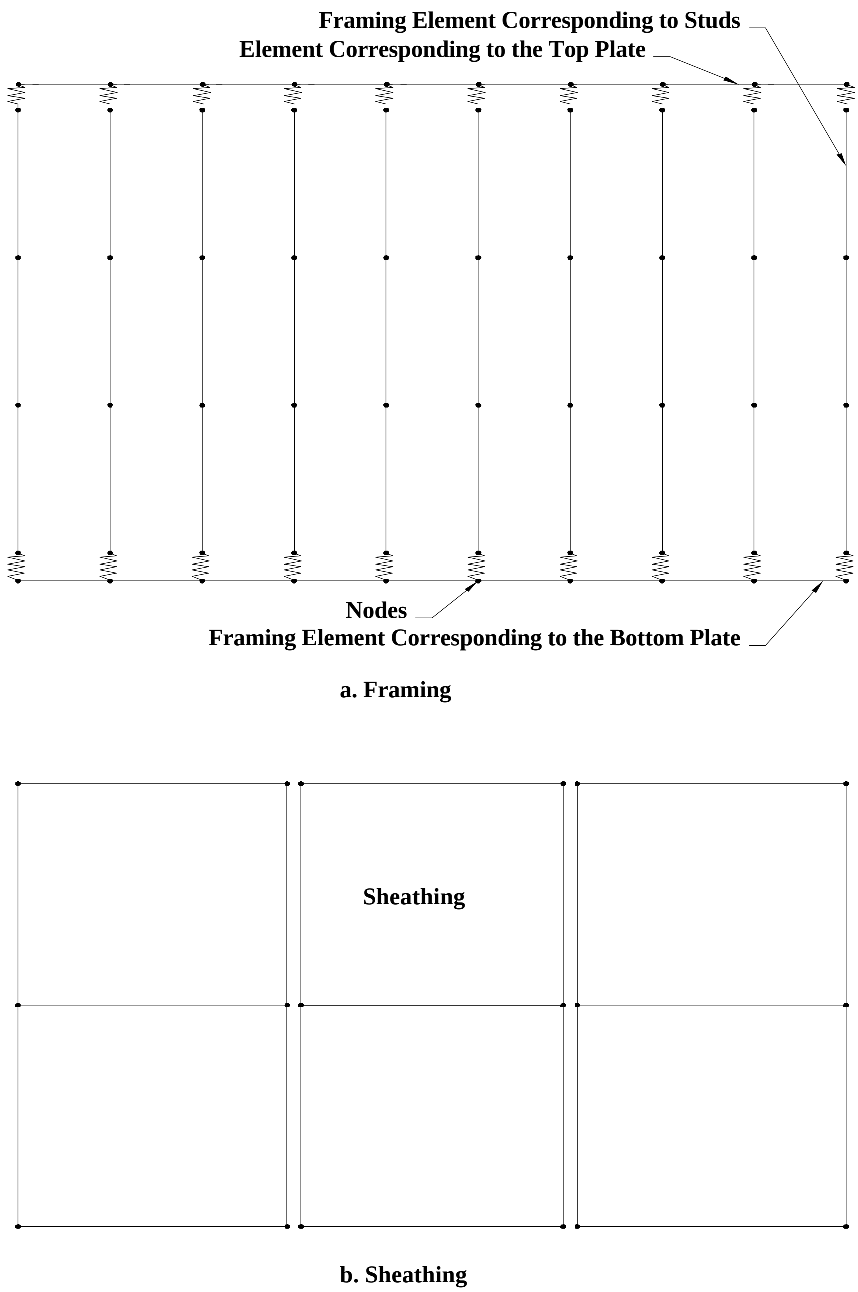

Figure 3.14: Components of Shear Wall Model

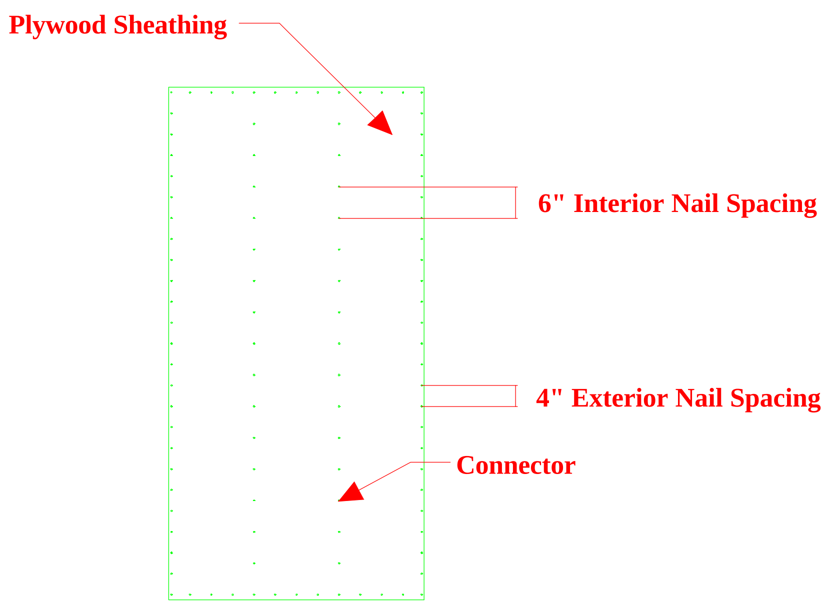

Figure 3.15: Nail Spacing for a Typical Sheathing Panel

3.3.2 Anchorage and Foundation Conditions

The shear wall model is supported by, and parallel to, a floor joist and has bolted hold-down anchors at the chords. Framing elements are used to model the joists and frame connector elements are used to model the hold-down anchors. Two categories of foundations were examined: a flexible foundation, in which a wall is parallel to a floor joist, and a rigid foundation, which corresponds to a wall resting on a concrete block or some other material of high stiffness.

Non-rigid foundation systems are modeled as being 24 feet long with a 12 feet long wall originating at one end and spanning to the center of the foundation (see Figure 3.12). Eleven joist sizes and three hold-down anchor sizes were utilized for a total of thirty-three different foundation conditions for flexible foundations. The sizes, which are listed in Table 3.3, were determined by calculating a moment of inertia for a series of deflection criteria and then using a constant width of

Table 3.3: Dimensions of Beams for Flexible Foundation Walls*

Deflection Criteria	Moment of Inertia		Depth	
	cm^4	in^4	mm	in
L/180	25197	605	325.12	12.8
L/240	33590	807	355.60	14.0
L/300	41994	1009	383.54	15.1
L/330	46193	1110	396.24	15.6
L/360	50393	1211	406.40	16.0
L/420	58792	1412	429.26	16.9
L/480	67191	1614	449.58	17.7
L/510	71389	1715	459.58	18.1
L/540	75589	1816	467.36	18.4
L/600	83988	2017	485.14	19.1
L/720	100786	2421	513.08	20.2

*A width of 98.9 mm (3.5 in) was used for all members.

88.9 mm (3.5 in). The depth was then calculated from the basic equation: $I=bh^3$. The moment of inertia was calculated from the deflection equation with a loading assumed to be equal to 250 plf. The loading includes a 100 psf live load and a 25 psf dead load, these loads are typical design loads for a office building, and a joist spacing of 0.61 m (2 ft). Stiffness of the joists were calculated as a function of the mechanical properties of each member (i.e. size, and cross sectional area). In addition, three rigid foundation models were analyzed, each with a different hold-down anchor size. The rigid foundation, which corresponds to a wall resting on a concrete or masonry foundation, was modeled using linear springs located at the studs.

Hold-down anchors were modeled at the end nodes at the base of the wall. Three different hold-downs were modeled: a 12.7 mm (0.5 in) diameter bolt, a 25.4 mm (1 in) diameter bolt, and a "rigid" hold-down. Vertical stiffness of the hold-down connectors, listed in Table 3.4, varied as a function of the depth of the joist and diameter of the bolt. The vertical stiffness was determined using the relationship K=EA/L. Stiffness for the rigid anchorage conditions remained constant at 3.5×10^7 N/mm (2.0×10^6 lb/in).

Table 3.4: Vertical Stiffness of the Hold-Down Connectors

Deflection Criteria	25.4 mm (1 in) Diameter Anchor	12.7 mm (0.5 in) Diameter Anchor
	kN/mm (kips/in)	kN/mm (kips/in)
L/180	252.2 (1442)	63.1 (360)
L/240	234.4 (1340)	58.6 (335)
L/300	219.0 (1251)	54.7 (313)
L/330	214.3 (1225)	53.6 (306)
L/360	209.8 (1199)	52.4 (300)
L/420	200.3 (1145)	50.1 (286)
L/480	192.5 (1100)	48.1 (275)
L/510	188.9 (1079)	47.2 (270)
L/540	186.2 (1064)	46.6 (266)
L/600	181.2 (1035)	45.3 (259)
L/720	171.8 (982)	42.9 (245)
RIGID	1328.5 (7592)	332.1 (1898)

3.3.3 Loading

The wall model was subjected to both monotonic and dynamic loading for each foundation condition. Monotonic loading consisted of a distributed load applied in the plane of the wall to the elements in the model corresponding to the top plate (see Figure 3.8). This simulates the condition in which the loading is transferred from the horizontal diaphragm to the shear wall. Load was increased at increments of five pounds per linear foot until "failure" (i.e., singular stiffness matrix or strength reduction of connectors) occurs.

Dynamic loading consisted of the S00E component of the 1940 El Centro acceleration record (see Figure 3.16). The record has a maximum velocity of 336 mm/s^2 (13.2 in/s^2) and a maximum acceleration of 3417.1 mm/s^2 (134.5 in/s^2). The acceleration record from this earthquake

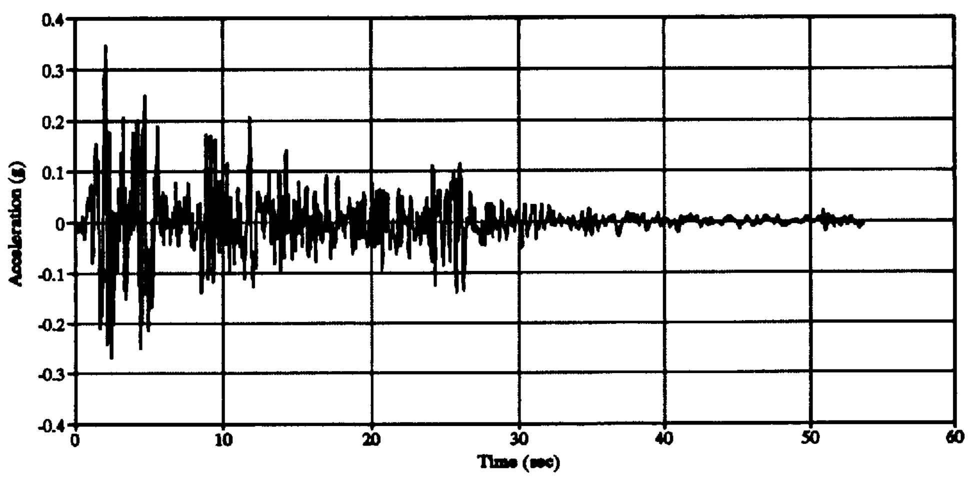

Figure 3.16: Acceleration Record for the S00E Component of the 1940 El Centro Earthquake (White, 1995)

was utilized because it is widely used and accepted today in the world of structural analysis. The ground acceleration acts in the horizontal direction (i.e., parallel to the x-axis of the wall) as shown in Figure 3.17. Only the first ten seconds of analysis will be utilized for this analysis due to the maximum displacements always occurring during the first ten seconds. A model was run for thirty seconds of analysis to verify that the maximum displacement did occur in the first ten seconds (see Figure 3.17).

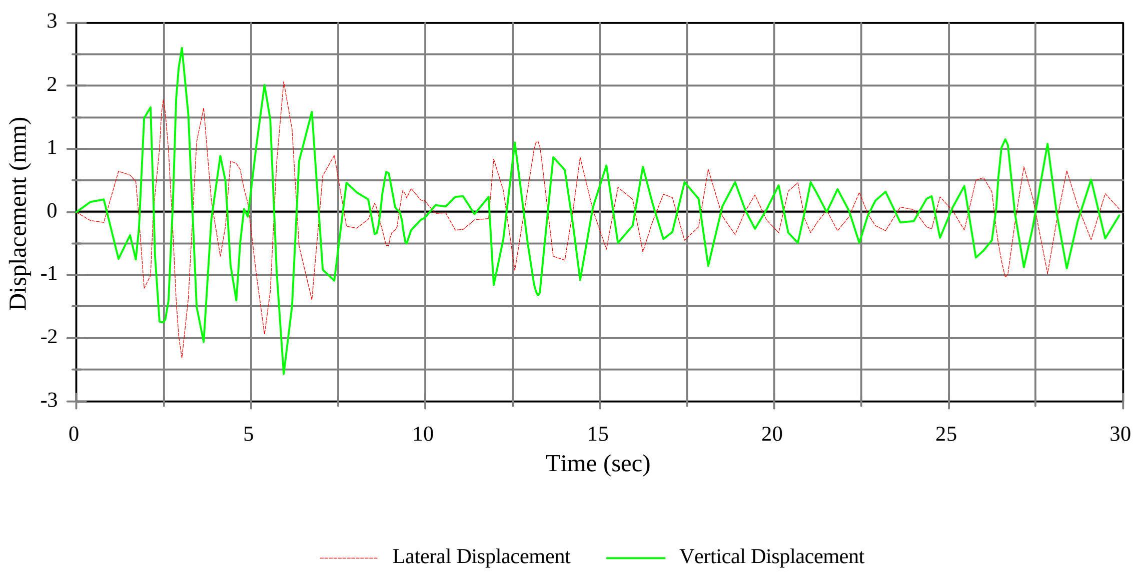

Figure 3.17: Time-Displacement Curve for Thirty Second Analysis

3.4 Data Collected

The following data was collected for each monotonic analysis run: load versus top-of-wall displacement and displacement at a given load (28.8 kN or 6.48 kips), maximum monotonic load, and state of forces throughout the wall. Load verses top-of-wall displacement plot and displacement for a given load were used to determine how overall displacement, wall rotation, and shear displacement vary as a function of the foundation conditions. Maximum load was examined to determine how the maximum resistance in a wall may be affected by the anchorage conditions. State of forces throughout the wall was used to characterize the force distribution in the wall, and to identify points of high stress.

The following data was examined for each dynamic analysis run: maximum top-of-wall displacements and maximum base shear. Maximum top-of-wall displacements is an indicator of the potential magnitude of architectural damage within the wall. The seismic base shear is a very important parameter for this study. The maximum base shear was examined to determine if the stiffness of the foundation needs to be taken into account for design or if the present method of assuming a rigid foundation is conservative. Overall, the results will indicate if the effects of a non-rigid foundation on a timber shear wall are significant enough to warrant their consideration.

3.5 Summary

A study to examine the effect of foundation stiffness on the response of timber shear walls was described. The study involved utilizing the computer program WALSEIZ1 to analyze a shear wall model supported by joists and hold-downs of varying size. The shear wall was modeled using elements corresponding to the framing, sheathing, connectors attaching sheathing to the framing, and connectors attaching the top plate and bottom plate of the wall to the studs. The foundation was modeled using elements corresponding to the joist below the wall and the hold-downs at the chords. The wall model was subjected to monotonic and dynamic loading for each of the foundation conditions, and forces and displacements for each analysis run was recorded.