Chapter 1

1. Introduction

1.1 Background

Currently, between one and two million new homes are built in the United States each year, predominantly with light timber framing. At the same time, the cost of lumber has been increasing due to population growth and a larger awareness of the environmental impact of deforestation. For this reason, efficient utilization of our lumber supply is very important. Ideally, the construction industry wants to build stronger, safer buildings that can withstand hurricane and earthquake loading while at the same time use less wood. In order to accomplish this, the actual load capacity, stiffness, and ductility of structures must be known.

Timber structures have a good track record when it comes to withstanding nature's disasters. Shear walls and diaphragms are the main structural elements used to resist the forces of seismic and wind loads. Two main reasons for their effectiveness is that: 1) wood structures are relatively light in mass and 2) the occurence of a large number of redundancies. Currently, shear walls are designed for earthquake loads based on monotonic rather than the cyclic capacity of the shear wall. For this reason, many structures are either being over-designed, wasting material; or worst, some structures may be under-designed, resulting in unsafe buildings.

Typically, structures are designed to remain elastic under static forces. However, it is unrealistic to expect a structure to remain elastic during a major earthquake. It is essential that structures are detailed and designed for sufficient ductility so that the building remains structurally safe when inelastic behavior is exhibited.

Tests conducted as part of this thesis applied monotonic loading and cyclic loading, similar to that experienced during an earthquake, to full scale shear walls to determine their monotonic and cyclic behavior. The information provided in this thesis, along with the data collected from many other researchers, will help future engineers design wood structures more effeciently.

1.2 Objectives

The objective of this thesis investigation was to report and utilize the data gathered concerning long shear walls with openings. This main objective can be broken into four sub-objectives as follows: 1) to use the results of full-scale monotonic and cyclic tests of shear walls with openings to evaluate Sugiyama's and Matsumoto's model, 2) develop a model to predict the strength and stiffness of wood shear walls in this investigation, 3) report on the monotonic and cyclic behavior of shear walls in this investigation, and 4) investigate the relationship between monotonic and cyclic strength and stiffness performance of wood frame shear walls.

1.3 Design Codes

For determining design earthquake loading, the static lateral force procedure (UBC, 1994) is used for most wood framed buildings. This procedure converts the actual dynamic loading a structure experiences during an earthquake into equivalent static forces. With the equivalent static forces, structures can be designed using the National Design Specification (NDS) for wood construction.

When a shear wall contains no openings for doors or windows, the total shear applied to the wall is divided by its total length to determine the unit shear on the wall. Often, shear walls have door and window openings, resulting in less stiffness and capacity than a fully sheathed wall. Currently, the unit shear for a wall with openings is determined by dividing the total shear load by the sum of the length of full height segments of the wall. In other words, the shear load is divided by the full length of wall minus the lengths of the doors and windows.

More experimental data of the monotonic and cyclic strength of shear walls with openings is needed to further verify code values, ensuring conservative design values.

1.4 Sugiyama

Two independent groups have tested one-third scale shear walls under dynamic loading which correlate well with a design methodology derived by Sugiyama. This method is based on actual strength of components of the shear wall, and how they work as a composite system, resulting in a better assessment of the utlimate capacity compared with current design procedures. As Section 1.2 stated, this thesis will further verify that the design values derived from this methodology will correlate well with the actual strength of full-scale wall, under monotonic and cyclic loading.

There are several limitations of Sugiyama's design procedure:

(1) The ratio of the depth to width in the wall space above and/ or below openings must be more than ¼, and
(2) The ratio of sheathed wall area and total wall area shall not be more than 55%.

1.5 Limitations of Study

It is well known that structural and mechanical properties of wood vary. Due to the expense and time associated with testing full-scale walls, only one wall speciman of each configurations was tested for each type of loading (monotonic and cyclic loading). Although there was a grade of standard or better required for all structural members, there are many factors, such as knot location, checks and splits, that effect member strength. It must be realized that data collected in this test falls in a range of accepted values, which depends on the quality of the structural members used and the skill of the carpenter. If one wall configuration were tested ten times, each test would result in slightly different data. A statistical analysis of these ten tests would provide a more accurate prediction of wall performance.

1.6 Document Organization

This thesis is organized such that:

Chapter Two presents results of previous analytical and experimental research on monotonic and cyclic shear wall tests.

Chapter Three describes the wall specimans used in this study, including materials and configuration. The loading applied to the walls and associated data from the eight sensors is also discussed.

Chapter Four presents the monotonic results and analysis.

Chapter Five presents the results and analysis of cyclic shear walls tests.

Chapter Six compares the monotonic and cyclic shear wall tests and shows relationships found between shear wall performance under the two loading patterns.

Chapter Seven summarizes conclusions determined, with special attention given to answering the objectives of this investigation

Chapter 2

2. Literature Survey

2.1 Introduction

This chapter provides an overview of previous research performed on topics relevant to this thesis. Emphasis has been placed on shear wall tests, but work pertaining to modeling and connection tests has also been included. The majority of work has focused on monotonic racking behavior of shear walls, but advances in technology have allowed cyclic performance to be evaluated more in the past decade.

Compilations of work pertinent to timber engineering have been published, several of which are presented here. Dorwick and Smith (1986) discuss the principles of design and analysis of shear walls, and review research under cyclic loading. Soltis and Falk (1992) summarize seismic performance of low-rise wood buildings in the late 1980's and early 1990's.

2.2 Background

Shear resistance in early wooden structures was provided by horizontal board sheathing. In the 1940's, plywood sheathing was introduction and a shift in construction practices occurred. Plywood (and fiberboard) sheathing provided both cost and labor saving advantages over horizontal board practices. In 1949, the Federal Housing Administration (FHA) established minimum performance standards for plywood sheathing. A minimum racking strength of 5200 lbf (23 kN) for an 8 ft. by 8 ft. wall section was set based on previous testing of the performance of horizontal board sheathing. In 1955, the Uniform Building Code (UBC) recognized shear wall design values based on tests performed by the American Plywood Association (APA). For the remaining part of the 20th century, extensive research into the performance of shear walls has been conducted including modeling, racking and cyclic performance, effect of openings, effect of nail size and scheduling, wall length and sheathing type.

Light-frame walls perform three distinct structural functions: a) transfer loads from upper floors or the roof to the foundation, b) resist normal loading (wind) and transfer this load to either the foundation, floor or roof diaphragm, or to a perpendicular wall, and c) act as a shear diaphragm to transmit lateral load to the foundation (Wolfe, 1983). The majority of research conducted has investigated the performance of light-frame walls acting as shear diaphragms. Typically, shear walls are assumed to act alone rather than as part of an integrated structural system.

McCutcheon (1985) reports that racking behavior of a sheathed wall primarily depends upon the lateral load-slip characteristics of the nails that fasten the sheathing to the frame. When a wall panel is under a racking load, the stud frame distorts as a parallelogram, while the sheathing retains its original shape. As a result, corner nailing distorts more than interior nailing. Also, it was found that the direction of distortion of the corner nails was approximately along the diagonals of the sheathing. Although nail slip is the major factor affecting racking stiffness, shear deformation of the sheathing can also play an important role.

Timber structures have performed well in the past under seismic conditions due to the numerous redundant members and the ability to resist large drifts. Relative to other construction materials, low-rise timber buildings generally have higher natural frequencies and similar coefficient of critical damping. The low mass of timber structures reduces inertial forces and their motion is nearly the same as the ground during a seismic event. Soltis (1984) examined typical low-rise timber structure performance during actual earthquakes. His investigation found that inadequate lateral support was a primary cause of actual failures. This was attributed to nonsymmetrical arrangement of racking walls and/or large openings.

2.3 Modeling and Design Approaches

Numerous researchers and designers have tackled different approaches to modeling and design of shear walls under racking and cyclic displacements. This section presents a brief review into modeling and design of shear walls.

Foschi (1977) developed a finite element program to model wood diaphragms. The model considers interaction between: the cover, the frame, the connections between frame members, and the cover-frame connections. Load-deformation characteristics of the nailing were assumed non-linear. A finite element program was developed by Itani and Cheung (1984) that considered nonlinear load-deflection behavior of sheathed wood diaphragms. The model is general and does not impose restrictions regarding sheathing arrangements, load application, and geometry of the diaphragms. This makes the model more feasible for analyzing openings. Dolan (1989) improved upon Foschi's finite element model and developed two finite element models (one for monotonic and one for time-step dynamic loading) to predict the behavior of timber shear walls. The models were verified by forty-two, 8 ft. by 8 ft. shear wall tests and shown to give good predictions of ultimate capacity and load-deformation characteristics. The models accounted for: ultimate capacity of sheathing connector, bearing between adjacent sheathing elements and out-of-plane bending effects of the sheathing. White and Dolan (1995) improved upon the work Dolan and Foschi. This finite element program was able to calculate forces and stresses of the elements and improved upon the analysis time.

Numerous models have been presented based on stress analysis resulting from racking displacements. Tuomi and McCutcheon (1977, 1978) provided a racking model based on a linear nail load/distortion relationship. Easley et al (1982) developed formulas for the analysis of typical wood-frame shear walls which were shown to be in good agreement with actual shear wall tests and a finite element program. The formulas are valid only if no separation occurs between framing members and the sill plates. Gupto and Kuo (1985) developed a model to determine the load capacity of shear walls based on a sinusoidal nail force distribution. Uplifting of the framing members from the bottom plate is not allowed. A variation of the model assumed that the studs were infinitely rigid in bending, reducing the number of degrees-of-freedom from six to three. Both models were comparable with finite element models, but were more suitable for repetitive nonlinear analysis. Gupto and Kuo (1987) refined their model to allow uplifting of the studs. McCutcheon (1985) examined racking deformations in wood shear walls. McCutcheon gives a method for determining ultimate strength of shear walls assuming a linear nail load-slip relationship. However, nail behavior is non-linear and ultimate strength determined from his method must take this assumption into account. Gutkowski and Castillo (1988) developed a mathematical model to analyze shear wall performance. Nonlinear behavior due to sheathing gaps was included in the model. SaRibeiro and SaRibeiro (1991) improved the modeling of the load-slip behavior of nailed joints to include a large range of nailed wood joints under lateral load. Various moisture contents and specific gravity's were examined, as well as the effect of side member thickness and nail diameter. Kamiya (1988) performed a linear analysis of the buckling behavior of plywood sheathed walls.

Static methods are discussed by Diekmann (1989) for design and analysis of shear wall forces. Diekmann assumes that: (1) the wall behaves as a rigid body, (2) bending resistance is provided solely by the boundary elements, (3) shear resistance is provided solely by the sheathing and (4) the shear is uniformly distributed along the length of the sheathing. Kawii et al (1990) proposed a method for calculating the distribution of horizontal forces to shear walls based the shear rigidity of the floor. Potter (1989) developed a computer program to calculate the distribution of seismic shear loads to timber shear panels.

Itani et al (1982) presented a procedure for estimating racking resistance by simulating each panel of sheathing by a pair of diagonal springs. Stiffness of the springs is based on the stiffness of an individual nail used to fasten the sheathing. Similarly, Naik et al (1984) modeled a shear wall panel using a hinged square frame of rigid bars stiffened by two linear diagonal springs of equal stiffness.

Ge et al (1991) developed a model that analyzed the effects of openings on the racking stiffness and resistance of walls. Their model assumes a fully sheathed wall and modifies the stiffness and strength based on openings, accounting for sheathing above and below openings. First, the apparent shear modulus of full panels of sheathing are modified to account for any cutouts, accounting for the size and location of the cutout. Second, the part of the panel that forms the opening is removed analytically by adding an appropriately sized panel of equal but negative stiffness. Sugiyama and Matsumoto (1993a, 1993b, 1994) developed a method to calculate racking strength of shear walls with openings. One-third scale racking tests were performed and shown to be in good agreement with predicted strength values.

Several models consider response of an entire structure when subjected to racking displacements. Ge (1991) provides a model to predict the response of a typical wood-frame house subjected to wind loads. Schmidt and Moody (1988) developed a simple structural analysis model to predict the behavior of light-frame buildings under lateral load.

Energy absorption of shear walls is of critical importance in understanding shear wall performance under cyclic conditions. Tembulkar and Nau (1987) investigated the role of hysteretic modeling in earthquake energy absorption and dissipation. Foliente et al (1993) examined modeling of the dynamic response of wood structures, energy dissipation and hysteresis behavior. Foliente (1995) provides a general hysteresis model for single- and multiple-degree-of-freedom wood joints and structural systems. Hysteresis shapes produced by the model compared favorably with experimental hysteresis loops of wood joints with (1) yielding plates, (2) yielding nails, and (3) yielding bolts. Bulleit (1986) developed a Markovian model for wood systems subjected to cyclic loading. Bulleit assumed the probability of a member failing in the next loading cycle is related only to the present state of the system, not to how the present state is reached. Capacity design in seismic events is discussed by Buchanan et al (1990). Yamanouchi et al (1990) modeled the seismic performance of three-story wooden houses having a steel or reinforced concrete first floor. Results recommended that the first story should consist of a ductile-moment resisting frame to avoid damage into the upper wood floors. Filiatrault (1990) used a model to determine the effect of friction devices in shear walls to dissipate energy. Due to the characteristic pinched hysterisis loops of timber shear walls, large drifts are required to dissipate energy. Friction devices would allow energy to be dissipated at lower drifts with less structural damage.

Knowledge gained from modeling and testing needs to be applied to design of future structures. Stewart and Dean (1989) present a design procedure for shear walls. Due to the relative stiffness of the sheathing and nailing, it is assumed that the sheathing is rigid and fixed to pin-jointed framing members with uniformly spaced flexible nail fasteners. The design procedure utilized the simple diaphragm theory (which forms the basis of NZS 3603:1981). It is assumed that any nail forces acting perpendicular to the framing forces can be ignored. Dean, et al (1984) developed an equilibrium analysis method for design of shear walls with openings. Calculations showed that the effective stiffness of the sheathing is controlled by the nailing stiffness rather than the shear stiffness of the sheathing. The nailing is generally at its allowable design load when the plywood stress is only half or less of its allowable. Sugiyama and Matsumoto (1994) presented an empirical equation for determining racking capacity of shear walls with openings. This empirical equation predicts the shear load ratio (the ratio of the strength of a shear wall with openings to the strength of a fully sheathed shear wall) based on the amount of openings a wall contains. The results of Rose and Keith (1995) were found to be in agreement with Sugiyama's simplified empirical prediction method. The perforated shear wall design approach, based on the work of

Sugiyama and Matsumoto (1994), is discussed by Douglas (1994). The perforated shear wall method allows conservative predictions of capacity for shear walls with openings. Sheathing above and below openings is accounted for in this design approach.

Current design procedures (Uniform Building Code (UBC), 1994) for shear walls in seismic zones 3 and 4 are based on monotonic racking tests rather than cyclic shear wall tests. Only full-height paneling is included in shear resistance. Sheathing above and below openings is not utilized by the UBC. The allowable design capacity is determined using design tables based on sheathing thickness, nail size, and nail spacing. Allowable design capacity is based on a minimum factor of safety of 2.8.

2.4 Cyclic Connection Tests

Connection tests provide valuable information regarding shear wall performance since the behavior of a sheathed wall depends upon the lateral load-slip characteristics of the nails that fasten the sheathing to the frame. Nails attaching the sheathing to the framing are the primary source of energy dissipation under cyclic displacements through inelastic deformation of the nailing or through friction between the nails and the wood (Polensek and Bastendorff, 1987).

Dean (1988) performed a thorough investigation into the cyclic behavior of joints connecting sheathing to timber framing. The dependence of nail bending shape and nail withdrawal on the nail and sheathing dimensions was examined. Dolan (1989) performed cyclic tests on nailed connections. Soltis and Mtenga (1985) examined the strength of nailed wood joints subjected to dynamic loads. At small deformations, the increase in joint capacity due to a higher rate of loading is offset by decreased joint capacity due to load cycling. The effect of load rate of nailed joints was investigated by Girhammar and Anderson (1988). Various thickness of members and angles of load to grain direction were tested. Results showed an increase in dynamic strength relative to static strength, but higher deformations in the wood. Dolan and Gutshall (1994) examined the monotonic and cyclic properties of nailed and bolted wood connections. They found no significant effect due to prior cyclic loading as high as 2.0 times the 1991 NDS nominal design load on capacity or ductility. Foliente (1993) indicated that the response of a joint is dependent on the loading and prior load history. Damping of nailed joints was investigated by Polensek and Bastendorff (1987). Lumber species affect damping and stiffness properties of joints the most, while the angle between shear force and lumber grain, and nail size are also significant.

2.5 Shear Wall Tests

This section presents an overview of full scale shear wall testing and has been broken up into: racking performance, dynamic performance, narrow shear walls, openings, and adhesives.

2.5.1 Racking Performance

The standard procedure for determining racking performance of shear walls is per the American Society of Testing Materials (ASTM) E72. This test procedure uses steel hold-down rods to resist over-turning forces. A 'stop' is placed at the end of the 8 ft. by 8 ft. wall to prevent lateral slipping.

Due to criticism of ASTM E72 because of the use of the rigid hold-down mechanism, ASTM E564 was developed. ASTM E564 uses tie-down anchorage at attach the end studs to the foundation and does not include a 'stop' to prevent lateral slipping.

The is an abundance of racking tests of shear walls without openings. Sugiyama and Suzuki (1975a,b) performed racking tests to examine the effect of nail type and spacing, plywood thickness, single or double sheathing (i.e. plywood and gypsum) and the spacing of nail connecting sheathing to the bottom plate. Tuomi and Gromala (1977) examined rate of loading, sheathing material, and let-in corner bracing. Ten types of structural flakeboard and two types of plywood were examined using ASTM E-72 by Price and Gromala (1980). The effect of moisture content was also examined. Easley et al (1982) examined seventeen shear walls of various size, stud spacing, and nailing schedules. Test results were compared with an analysis method developed by Easley. Griffiths (1984) examined racking loads on 8 ft. x 8 ft. shear wall panels with various sheathing and small variation in nailing schedules. Griffiths found ASTM E-72 unsuitable for racking tests because it over restrains the panel giving unrealistic failures. Seven shear wall assemblies were tested by Nelson, et al (1985) to simulate wind load. They investigated the size of the shear wall and the number of joists beneath the shear wall. They found that the connection of the shear wall to the floor on the windward side of the wall was typically the location of failure. Wolfe (1983) examined the contribution of gypsum board to racking performance. Panel orientation and wall length were also examined. Dolan (1989) performed racking tests of walls sheathed with either plywood or waferboard monitoring out-of-plane bending, sole plate uplift, separation of framing joints, and load-deflection behavior. The American Plywood Association (Tissel, 1990) examined racking performance of shear walls. Tests were performed using ASTM E-72 to examine unblocked shear walls, stapled shear walls, sheathing over metal framing, double-sided walls, panels over gypsum sheathing, and the effect of stud spacing and width.

2.5.2 Dynamic Performance

Cyclic evaluation of shear walls is essential to understand the behavior and better design shear walls to withstand a seismic or hurricane event. Medearis and Young (1964) examined the effect of cyclic loading on plywood shear walls to determine energy absorption properties. These early tests took five to six hours to complete with an average of 500 dial gage readings taken per tests. Sheathing thickness, nail size, nail schedules, and the effect of renailing previously tested shear walls were examined.

Freeman (1977) reports results of shear wall tests performed by URS/ John A. Blume & Associates examining rate of loading and sheathing. Shear walls sheathed with gypsum wallboard, plaster, plywood and gypsum and plywood were tested. Cyclic loading was applied at 0.7 Hz, 1 Hz, 2 Hz, and 10 Hz, with progressively increasing displacements.

An examination of cyclic performance characteristics has provided important insight into shear wall behavior in a seismic event. Dean et al (1986) tested eleven 8 ft. by 8 ft. shear wall tests under monotonic, sinusoidal dynamic and actual earthquake (1940 El Centro) conditions. Falk and Itani (1987) examined natural frequencies, damping ratios, and nonlinear stiffness characteristics of diaphragms. Ten floor, ceiling, and wall diaphragms ranging in size from 8 x 24 ft. to 16 x 28 ft. were tested. Two shear walls with openings were examined. Dolan (1989) performed nineteen free-vibration tests of 8 ft. by 8 ft. shear walls to determined the change in natural frequency after sinewave, frequency sweep and earthquake tests. Both Dolan (1989) and Falk and Itani (1987) found natural frequency and stiffness to reduce after loading. Stewart et al (1984) examined ductility, hold-down performance and stiffness degradation for the seismic performance of plywood sheathed shear walls. Karacabeyli and Ceccotti (1996) are currently involved in a five year research project examining the lateral load resistance of timber structures and have released early test results. Concern is expressed for a universal protocol for cyclic testing. Filiatrault and Foschi (1991) performed three shear wall tests based on actual earthquake displacement patterns. The San Fernando, El Centro, and Romania earthquakes were examined. Ductility and energy absorption characteristics of shear walls were examined by Leiva-Aravena (1996). Equivalent viscous damping ratios were found to range from 0.20 to 0.40, depending on displacement and cycle number.

Yasumura (1992) compared performance of monotonic loading and reversed cyclic loading. Maximum load resistance was 9 to 33% smaller for walls experiencing reversed cyclic lateral loading than monotonic loading. Ductility and equivalent viscous damping ratios data was also determined.

Pseudo dynamic tests were performed by Kamiya et al (1996) to evaluate the relationship between maximum deflection response and the ratio of the mass which the wall supports to the ultimate strength of the wall.

Porter (1987) presented the Sequential Phased Displacement (SPD) loading procedure at the Third Meeting of the Joint Technical Coordinating Committee on Masonry Research (TCCMAR). The purpose of the SPD loading pattern was to establish a uniform testing procedure so data from cyclic tests are comparable. SPD loading ‘entails reversed-cyclic loading of progressively larger magnitudes until a first major event, followed by stabilization and degradation cycles before progressing to the next increment of higher displacement until termination.’ Porter also discusses ductility and cyclic stiffness calculations. The Structural Engineers Association of Southern California (SEAOSC) modified the test protocol developed by Porter and it has been submitted to ASTM for consideration as a standard test method. The SEAOSC loading procedure is conducted at a loading rate between 0.2 and 1.0 Hz.

2.5.3 Narrow Shear Walls

Commonly, shear walls consist of several narrow shear wall panels separated by door and window openings. The Applied Technology Council (1995) conducted a limited study to examine the static and dynamic performance of narrow plywood sheathed shear walls with the maximum allowable 3.5 to 1 height to width ratio. McDowall and Halligan (1989) performed a series of shear wall tests to determine the effect of sliding glass doors in walls resulting in short wall segments which vary in length from less than 300 mm to 600 mm. These small wall segments only contain two studs per panel. This study concluded that allowable design racking loads should be stiffness based. Tissel and Rose (1994) conducted pseudo-static cyclic loading of shear wall panels with height to width ratios of: 2:1, 4:1 and 6:1 that are commonly found adjacent to large openings such as garage doors. They recommend including a 12 inch deep header (extending from the adjacent opening) the full width of the narrow shear wall panel and the use of double studs at each end. They recommend the use of tie-down anchorage on each end of the panel to achieve the maximum strength of the narrow panels.

2.5.4 Openings

Typical shear walls in residential and commercial construction contain door and window openings. Patton-Mallory et al (1985), McDowall and Halligan (1989), Yasumara and Sugiyama (1984), Yasumara (1986), and Rose and Keith (1995) all examined the effect of openings on racking performance. Results of Patton Mallory et al (1985) indicated that wall strength should be obtained only from full-height sheathing. Effective length of a shear wall was defined as the full length of the wall minus the length of openings. Yasumura and Sugiyama (1984) tested one-third scale models to investigate the influence of the shape and area of openings. Rose and Keith (1995) examined the effect of openings under both monotonic and cyclic loading. Shear resistance of gypsum board and plywood sheathing were examined and found to be additive during monotonic loading. Shear resistance of gypsum board was only additive for small displacements during cyclic loading. Sheathing above and below openings was found to add to the shear resistance of the shear wall. Kamiya (1990) examined the effect of openings on floor diaphragms. Effect of the location of an opening of equal area was examined.

2.5.5 Adhesives

Although this investigation is not concerned with glued connections in shear walls, the use of adhesive is common in wood construction. Only a small portion of the work pertaining to adhesives is presented, but typical shear wall performance characteristics obtained when adhesives are used in conjunction with nailing is given here.

The monotonic and dynamic performance of timber shear walls fastened with both nails and wood adhesive and nails alone were examined by Filiatrault and Foschi (1991). Test results showed that adhesive increases stiffness of walls, but results in a more brittle failure. Load-deflection curves of shear walls fastened with both glue and nails were nearly linear until failure. Dolan and White (1992) examine design considerations of wood adhesives in the performance of shear walls. Buildings with glued shear walls are most likely to fail at the anchorage connection during a seismic event due to higher shear forces.

2.6 Wall Assembly and Building Tests

To understand the interaction between structural assemblies, full scale building tests are performed. Boughton and Reardon (1984) built a full-scale house and subjected the structure to uplift and racking forces to simulate cyclone conditions. Stiffness of the diaphragm was found to be greater than the sum of the individual components. For loads up to twice the design load, overall stiffness of the walls were more affected by the connections to the floor than the nailing details. Walls that resisted uplift forces were not subjected to racking loads.

Leichti and Kasal (1992) examined load sharing of shear walls in a full house structural assembly based on relative stiffness.

Suzuki (1990) performed a one-third scale model of a building to examine the effect of cross walls on lateral stiffness. The shear walls did not contain openings but the two cross walls contained two openings equal to 20% of the total wall area each. A floor diaphragm was placed above the four walls. The cross walls did not affect racking strength much, but was effective in preventing rotation of the shear walls. A full-scale building test was performed by Gebremedhin (1992,1994). The investigation examined end wall stiffness, the effect of door openings equivalent to 25% and 50% of the area of the end wall on stiffness, and the effect of steel strapping, plywood sheathing, and stitch screws on end wall stiffness. Hirashima and Suzuki (1996) built and tested a two story post and beam structure. They conducted a force vibration test, eccentric load, and racking load until failure.

A full-scale house was built and tested to verify and analytical method developed by Gupto and Kuo (1987). Two of the four walls contained unsymmetric openings. Nelson et al (1985) examined the affect of the location of shear walls within the assembly of the structure on racking strength. Thurston (1994) examined shear walls with openings in full wall assemblies under cyclic loading. Tie-down anchorage was not used in most tests and nailing of the sheathing to the bottom plate prevented uplift. There was little difference in racking strength between walls where sheathing panels joint at window openings and those where panels are cut for the openings. Sheathing force distribution in walls with openings was not found to be uniform. Sheathing forces directly below a window were found to be particularly high.

A Buddist style temple using traditional Japanese woodworking techniques was built and tested. Results are reported in Kawachi (1990). Horizontal racking tests and vibration tests were performed. Arima et al (1990) performed nondestructive testing of full-scale houses to measure the frequency response curve and to evaluate the relationship between natural frequency and the racking resistance of the house.

Sakamoto et al (1990) tested a base isolated two story house using monotonic horizontal loading and damped free vibration. A comparison was made between the actual response of the base isolated building and the simulated response of a fixed base model. Base isolation was found to be very effective in energy absorption during an earthquake event.

2.7 Summary

An overview of pertinent research on shear walls has been presented including: modeling, cyclic connection performance, monotonic and cyclic full-scale shear walls tests and building tests. Dynamic testing is essential to better understand cyclic behavior of shear walls and will enable future design for shear walls in seismic areas to be based on dynamic performance rather than monotonic racking performance.

Chapter 3

3. Test Specimens and Procedures

3.1 Introduction

This chapter describes the wall configurations investigated, materials used to construct the wall specimens, test equipment, and test procedures used for monotonic and sequential phased displacement loaded shear walls.

3.2 Specimens

A total of ten walls were included in this investigation, all 40 ft. (12.2 m) long and 8 ft. (2.4 m) high. Figure 3.1 shows the five different configurations of walls that were investigated and illustrates opening locations. Opening dimensions and sheathing area ratio for each specimen are listed in Table 3.1. Each wall used the same type of framing, sheathing, nails, and nailing patterns. Wall "A" is fully sheathed and used for comparison purposes.

Displacement of the walls occurred at the top left corner of the wall configurations shown in Figure 3.1. Because the four walls with openings were unsymmetrical, direction of loading can effect shear wall performance. Location of openings is a minor component effecting strength and stiffness properties of shear walls and is not considered in this investigation. It would be expected that the strength and stiffness of Walls B and E would be lower if displaced from the top right corner rather than if displaced from the top left corner because there are less sheathing panels adjacent to the load cell. Conversely, Walls C and D would have higher strength and stiffness when displaced from the opposite side of the wall.

Table 3.1:Opening sizes for the five shear wall configurations examined

Wall	doors	windows[1]	r
A	N/A	N/A	1.0
B	6 ft.-8 in. x 4 ft. (2.0m x 1.2m)	5 ft.-8 in. x 7 ft.-10½ in. (1.7m x 2.4m)	0.76
C	6 ft.-8 in. x 4 ft. (2.0m x 1.2m)	4 ft. x 11 ft.-10½ in. (1.2m x 3.6m) 4 ft. x 7 ft.-10½ in. (1.2m x 2.4m)	0.55
D	6 ft.-8 in. x 4 ft. (2.0m x 1.2m) 6 ft.-8 in. x 12 ft. (2.0m x 3.7m)	4 ft. x 7 ft.-10½ in. (1.2m x 2.4m)	0.48
E[2]	N/A	N/A	0.30

1: The top of each window is located 16 in. (406 mm) from the top of the wall.
2: Wall E has studs 16 in. (406 mm) o.c. for the full length of wall.

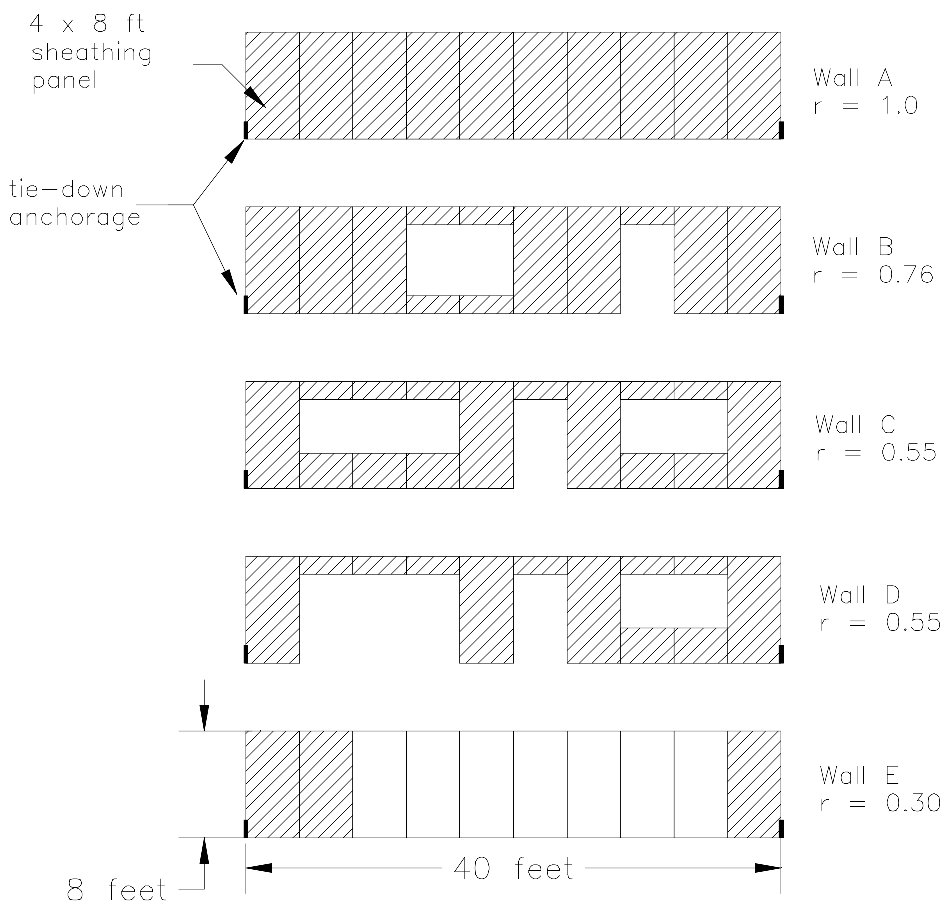

Figure 3.1- Wall configurations examined

The five wall configurations were compared based on sheathing area ratio. Sugiyama (1994) defined sheathing area ratio, ‘r’, based on: a) the ratio of the area of openings to the area of wall and b) the length of wall with full height sheathing to the total

length of the wall. Sugiyama’s classification of shear walls is used in this research, and is defined as:

$$r = \frac{1}{1 + \frac{\sum A_i}{H \cdot \sum L_i}} \tag{3.1}$$

where ΣA_i is the sum of the area of openings, H is the height of the wall, and ΣL_i is the sum of full height sheathed wall segments. These variables are illustrated in Figure 3. 2. By definition, a fully sheathed wall has a sheathing area ratio of 1.0, and sheathing area ratio decreases toward zero as total opening size increases. It is noted that sheathing area ratio does not consider location of the openings.

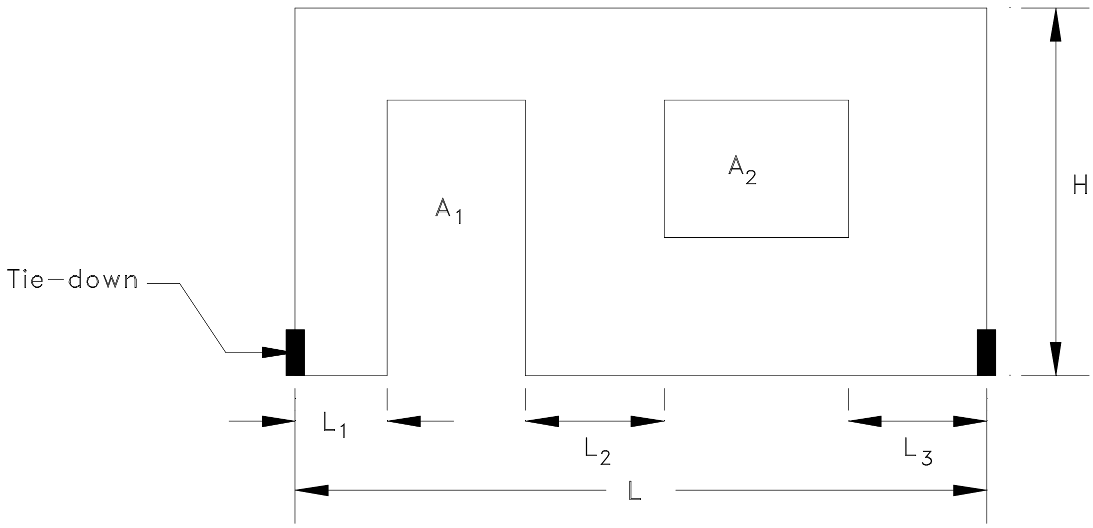

Figure 3.2- Shear wall sheathing area ratio variables

3.3 Materials

Information concerning the materials and construction details used are included in Table 3.2. Listed are the headers and jack stud sizes used around openings and the tie-down anchor type.

The wall framing consisted of double top plates, single bottom plates, double end studs, and double or triple studs around doors and windows. Studs were spaced 16 in. (406 mm) on center. All framing consisted of spruce-pine-fir purchased from a local lumber yard. Members were graded standard or better.

Exterior sheathing used was 15/32 in. (12 mm), 3 ply, structural 1 plywood. All full height panels used were 4 ft. by 8 ft. (1.2 by 2.4 m) and oriented vertically. Plywood was cut to fit above and below the doors and windows.

Interior sheathing was 4 ft. by 8 ft. (1.2 m by 2.4 m) sheets of ½ in. (13 mm) gypsum wallboard, oriented vertically. As with the plywood, the gypsum was cut to fit above and below the doors and windows. All joints in the interior sheathing were taped and covered with drywall compound. The taped joints dried for a minimum of 3 days prior to testing.

Both interior and exterior sheathing were able to rotate past the text fixture at the top and bottom (i.e. the steel test fixture was narrower than the wood framing used for the top and bottom plates.)

Two tie-down anchors were used on each wall, one at each double stud at the ends of the wall specimens (approximately 40 ft. (12 m) apart). Simpson Holdown model HTT22 with 5/8 in. (190 mm) diameter anchor bolts were used. Tie-down anchors were attached to the bottom of the end studs by thirty-two (32) 16d (0.147 in. (4 mm) diameter and 3.25 in. (82 mm) length) sinker nails. A 5/8 in. (190 mm) diameter SEA grade 2 bolt connected the tie-down, via the bottom plate, to the rigid structural steel tube test fixture.

Table 3.2: Wall materials and construction data for shear wall specimens

Component	Material/ Construction Data
Framing Members	Standard and better, Spruce-Pine-Fir
Sheathing:	
Exterior	Plywood, 15/32 in. (12 mm), 3 ply, Structural 1. 4 ft. x 8 ft. sheets installed vertically.
Interior	Gypsum wallboard, ½ in. (13 mm), installed vertically, joints taped
Headers:	
4 ft. (1.2 m) opening	(2) 2 x 4 ’s (38 mm x 89 mm) with intermediate later of 15/32 in. (12 mm) plywood. One jack stud on each end.
7 ft.-10 ½ in. (2.4 m) opening	(2) 2 x 8 ‘s (38 mm x 184 mm) with intermediate layer of 15/32 in. (12 mm) plywood. Two jack studs at each end.
11ft.-10 ½ in. (3.6m) opening	(2) 2 x 12 ‘s (38 mm x 286 mm) with intermediate layer of 15/32 in. (12 mm) plywood. Two jack studs at each end.
Tie-down	Simpson HTT 22, 5/8 in. (19 mm) diameter A307 bolt.
Anchor Bolts	5/8 in. (19 mm) diameter A307 bolt with 3 in. (76 mm) square ¼ in. (6 mm) steel plate washers with (1) anchor bolt within 12 in. (305 mm) from end of each panel.

Table 3.3 shows the fastener schedule used in constructing the wall specimens. Four different types of nails were used in construction of the wall specimens. 16d (0.162 in. (4 mm) diameter and 3.5 in. (89 mm) length) bright common nails were used for the framing, 8d (0.131 in. (3 mm) diameter and 2.5 in. (64 mm) length) bright common nails were used to attach the plywood sheathing to the frame, 16d (0.147 in. (4 mm) diameter and 3.25 in. (83 mm) length) sinker nails were used for attaching tie-down anchors to the end studs, and 13 gage x 1-½ in. (38 mm) drywall nails were used for attaching gypsum wallboard. A nail spacing of 6 in. (152 mm) perimeter and 12 in. (305 mm) field was used for the plywood sheathing and 7 in. (178 mm) perimeter and 10 in. (254 mm) field for the gypsum wallboard. The tie-down anchors were attached to the double end studs using 16d sinker nails, one located in each of the 32 pre-punched holes in the metal anchor. A307 or SEA grade 2 bolts were used to make all attachments to the steel structural tube test fixture. All bolts were 5/8 in. (16 mm) diameter National Coarse thread.

Table 3.3: Fastener schedule used for shear wall specimens

Connection	#/ Type Nails	Nail Spacing
Framing		
Top plate to top plate (face nail)	16d common	per foot (305 mm)
Top or bottom plate to stud (end nail)	2-16d common	per stud
Stud to stud (face nail)	2-16d common	24 in. (610 mm) o.c.
Stud to header (toe nail)	2-16d common	per stud
Stud to sill (end nail)	2-16d common	per stud
Header to header (face nail)	16d common	16 in. (406 mm) o.c. along edges
Tie-down anchor	16d sinker A307 5/8 in. bolt	32 total to end stud per tie-down to foundation
Sheathing:		
Plywood	8d bright common	6 in. (152 mm) edge/ 12 in (305 mm) field. (2 rows 8d common for end stud)
Gypsum wall board	13 ga x 1½in. (3/8 in. head)	7 in. (178 mm) edge/ 10 in. (254 mm) field

3.4 Wall Orientation

Shear wall tests were performed in a horizontal position as shown in Figures 3.3 and 3.4. The wall was raised approximately 16 in. (407 mm) above the ground to allow instruments and the load cell sufficient clearance to be attached to the wall. The bottom plate was secured to a fixed steel structural tube 24 in. (610 mm) on center with 5/8 in. (16 mm) diameter bolts and 3 x 3 inch (76 x 76 mm) square ¼ in. (6 mm) steel plate washers. The oversize of bolt holes was limited to 1/32 in. (1 mm) to minimize slip. A hydraulic actuator, with a range of ±6 in. (152 mm), was attached to the top right corner of each shear wall (for the configurations shown in Figure 3.1) via a steel tube. The steel tube was used to distribute the loading to the double top plate in the wall. The steel tube and the double top plate were connected using 5/8 in. (16 mm) diameter bolts 24 in. (610 mm) on center. The bolts used to attach the specimens to the steel tube were located 12 in. (305 mm) from the end stud of all interior wall segments. Eight casters were attached to the structural tube to allow horizontal motion, as shown in Figure 3.3(b). The casters were fixed parallel to loading, allowing no rotation. The amount of friction created by the wheels was obtained by testing.

The magnitude of the friction was negligible when compared to the capacity of the walls, but all recorded loads were corrected for this bias.

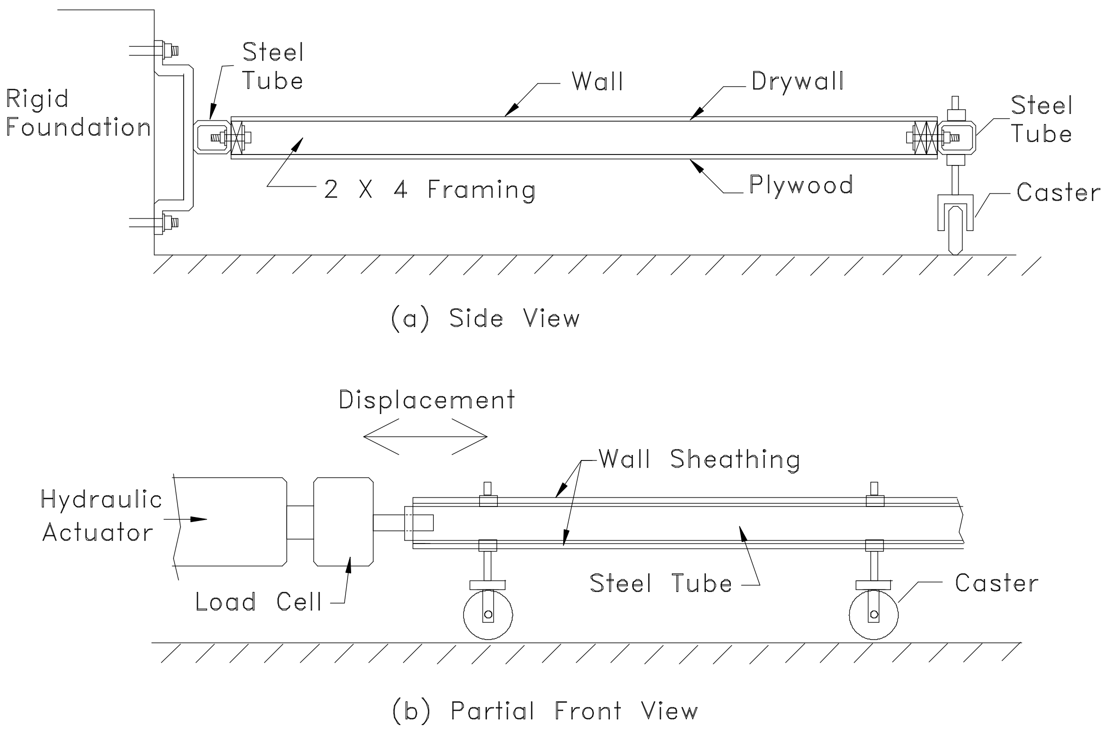

Figure 3.3- Wall attachment to test fixture

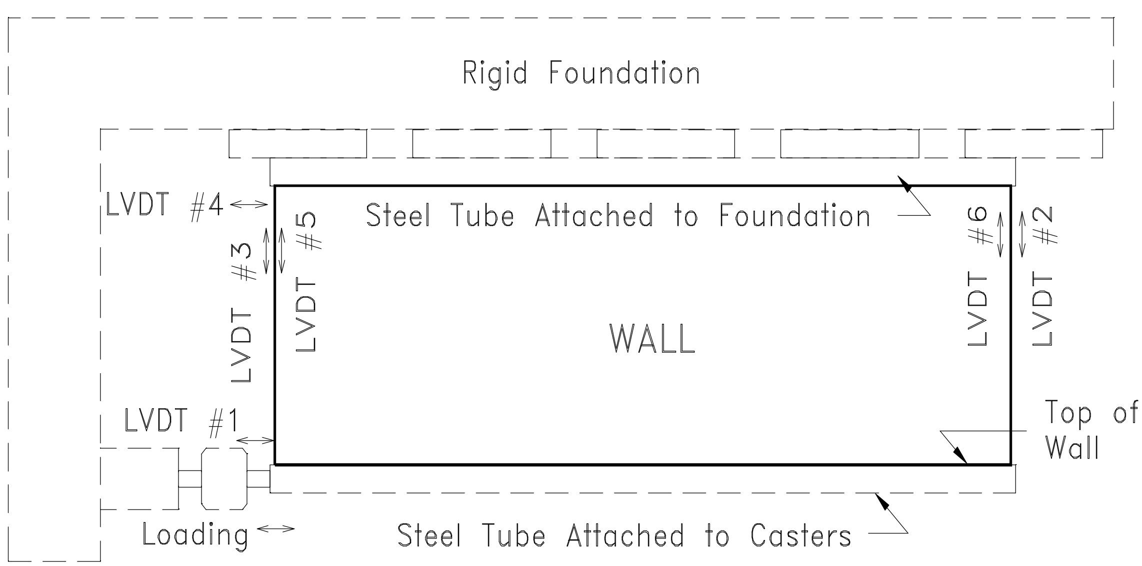

Figure 3.4- Sensor locations on plan view of wall specimen

3.5 Data Acquisition System

Location of the six LVDT's used to measure displacement of the wall during testing are shown in Figure 3.4.

LVDT #1 was located adjacent and parallel to where the load was applied, measuring the displacement of the top of the wall relative to a fixed reference point.

LVDT #2 measured the amount of crushing of the end studs in the bottom sill plate.

LVDT #3 measured the amount of uplift of the end stud relative to the bottom sill plate. All data recorded was corrected to compensate for amplifications caused by the geometry of the LVDT fixtures ensuring the actual uplift / compression displacements of the end studs were reported. The reported movement of the end studs was determined at 1.5 in. (38 mm) from the end of the wall (i.e. between the double end studs) near the bottom plate.

LVDT #4 measured horizontal displacement of the bottom plate relative to a fixed point. This measurement allows rigid body translation of the wall to be subtracted from the global displacement to obtain interstory drift (or racking). Interstory drift was calculated as LVDT #1 - LVDT #4.

LVDT #5 and LVDT #6 were attached to the end studs and tie-down anchors. These sensors measured the slip, if any, of the tie-down anchors relative to the end stud.

In addition to the six external LVDT's, two sensors attached to the hydraulic actuator measured the displacement of the load cell and the load applied to the shear wall, and were used to control the servo-hydraulic system.

3.6 Monotonic Shear Wall Test Procedure

Monotonic tests were one-directional, measuring load resistance as the load cell displaced the top of wall six inches over a ten minute period. The bottom plate was anchored to the test fixture and only lateral movement of the top plate was allowed as discussed previously in this chapter. Data from the 6 external LVDT's and 2 internal hydraulic actuator sensors described above was collected 10 times per second. Each wall configuration was tested once.

3.7 Sequential Phased Displacement (SPD) Shear Wall Test Procedure

Each wall configuration was also built to undergo sequential phased displacement (SPD) loading. Load resisted by the wall was recorded as the load cell was displaced according to the SPD loading pattern. Data concerning the load and drift of each wall was used to compute the cyclic stiffness, hysteritic energy dissipation, potential energy, and the equivalent viscous damping ratio. Data from the 6 external LVDT's and 2 internal load cell sensors was collected fifty times per second.

The SPD loading sequence is shown in Figure 3.5. The loading cycle for cyclic tests consisted of two displacement patterns. The first pattern gradually displaced the wall to its anticipate yield displacement. Elastic behavior of the wall was observed in this section of the test. The second displacement pattern occurred once the wall passed it's anticipated yield displacement (i.e. inelastic behavior). The displacement was a triangular, sinusoidal ramp function with a frequency of 0.5 Hz.

The first displacement pattern consisted of reversed-cyclic displacements for three cycles at each incremental level at low, elastic behavior displacement levels. The first set of three cycles displaced the wall at approximately 25% of first major event (FME). The second set of three cycles displaced the wall 50% of the FME and the final set of three cycles displaced the wall at 75% of the FME. The next cycle displaced the wall to approximately the FME, at which point the second displacement pattern began.

Once yielding occurred, a sequential phased displacement loading procedure was used. In SPD loading, the displacement of each set of cycles was based on the previous set of cycles. Peak displacement of a set of cycles was 100% of the FME higher than the peak displacement of the previous set of cycles. The first peak cycle of a set was followed by three decay cycles, with each magnitude being 25% less than the previous cycle (i.e. the first decay cycle was 75% of the peak displacement, second was 50%, and third was 25%). Following the decay cycles were three cycles at the peak displacement. Three cycles were determined to be sufficient in order to obtain a "stabilized" response. "Stabilized" response is defined as when the load resisted by the wall when displaced the same magnitude in two successive cycles does not decrease more than 5%.

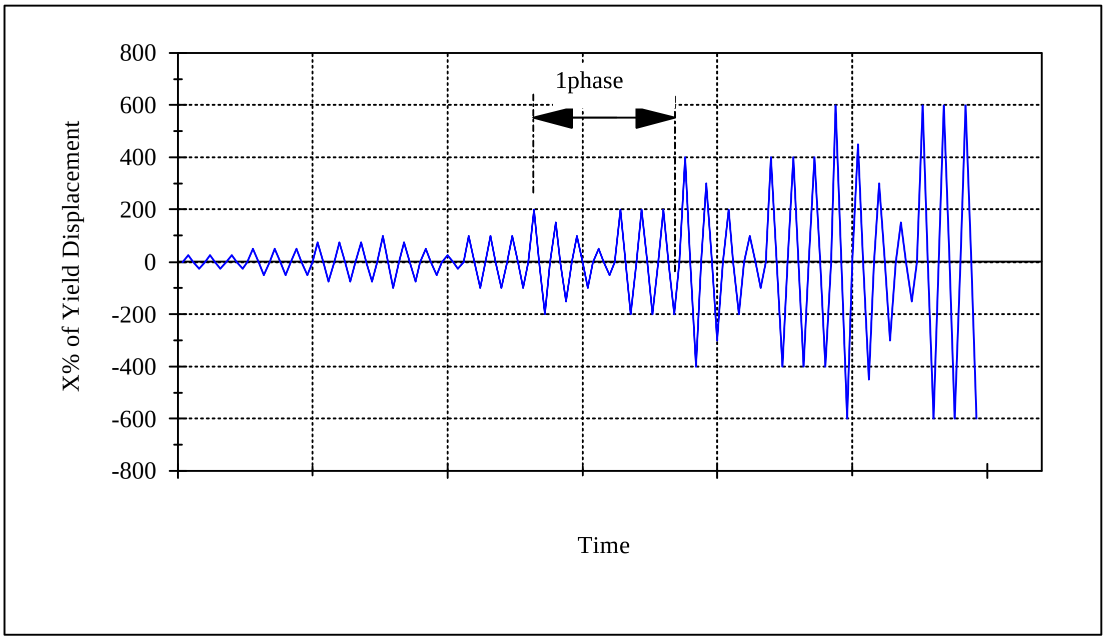

Figure 3.5- Sequential phased displacement (SPD) loading sequence

3.8 Summary

The test equipment, materials used to construct the wall specimens, wall configurations studied, and the displacement patterns used have been described.

Ten 8 ft. by 40 ft. (2.4 m by 12.2 m) walls were constructed with five different opening configurations. Each of the five wall configurations was tested with a monotonic load and a sequential phased displacement (SPD) load. Results and discussion of data collected from the tests are presented in the next two chapters.