2. LITERATURE REVIEW

2.1 Design Criteria for Wood Shear Walls

One of the goals of this research was to investigate the published design capacities for wood-shear walls. A shear wall's element capacity is a function of several component strengths and their integration. Current design methods to determine shear-wall capacities use the following method: (1) determine the required fastener size and spacing, from the tributary lateral loads, in plf; and (2) analyze the design seismic loads on the element for uplift effects to specify the appropriate manufactured hold-down. Integration of the hold-down component with other components within the shear-wall element is rarely considered.

Published design loads for the fastener spacing are based on specimens of the uniform size 8′ x 8′ and subjected to static loads. Under these conditions, the effects of slenderness and uplift are minimal, and load factors or over-strengths measured may be greater than the factors when measured under cyclical loading. A discussion of the sheathing and fastener component design values demonstrates the derivation of published code and approved design values. The effects of aspect ratios and hold-down type on element performance are also examined.

Allowable load design values for shear walls, published in new building-design provisions, are based on the National Design Specification (NDS) equations for fasteners. The allowable load tables are forced based; in other words, the capacities are based on allowable fastener strengths, without consideration to element deflection. The nail loads are determined by yield-limit equations that were adopted by the NDS in 1991. Before 1991, proportional limit equations were used to determine these values. The yield-limit equations were derived from different failure modes of the individual fasteners and are applicable to a variety of fasteners, side members, and materials. Then, the yield-limit equations were "calibrated" to match the yield point of proportional limit equations based on a yield displacement of 5% of the nail diameter (NDS Commentary 1993). For diaphragms, the yield-limit equations are used to determine the strength of each nail based on nail diameter, edge distance, spacing, sheathing thickness, and framing-member thickness. Adjustment factors are applied for penetration, diaphragm construction, and load duration; then, the total load in plf is computed. Deflection is not accounted for from these equations. Rather, deflection is calculated using the equations discussed in Section 3.3 of this thesis. The limitations for deflection in the 1997 UBC are determined by taking the calculated elastic deflection (Δ_s) and multiplying by 0.7 times R to determine if it is within limits of the building code criteria. This limitation is 2.5% of the story height. Since the yield-limit equations represent different failure modes, examination of the equations below discusses the failure modes they represent.

2.1.1 Yield Limit Equations for Nails

There are four yield modes represented by four equations that model the failure of nails in single shear between sheathing and wood-framing members. The equations represent the following modes:

1. Crushing in the side member, $Z = D \times t_s \times F_{es} / K_D$

2. Plastic hinge and crushing in the main member, $Z = k_1 \; x \; D_p \; x \; F_{em} \,/\, (\, K_D \; x \; (1{+}2R_e))$

3. Plastic hinge and crushing in the side member, $Z = k_2 \; x \; D \; x \; ts \; x \; Fem \,/\, (K_D \, x \; (2{+}R_e))$

4. Two plastic hinges, one on each side of the shear plane, $Z = D^2/K_D \;\; (2 \; x \; F_{em} \, x \; F_{yb}/(3 \; x \; (1{+}R_e)))^{\wedge 1/2}$

The lowest value of the above equations controls the design value. *Z* is the nominal dowel-bearing strength; *D* is the nail diameter; ts is the sheathing thickness; F_{es} is the dowel-bearing strength of the sheathing; F_{em} is the dowel-bearing strength of the framing members; F_{yb} is the bending yield strength of the nails; k_1 and k_2 are factors that consider relative member strengths, sheathing thickness, and nail penetration; and K_D is a constant-based nail diameter American Institute of Timber Construction (AITC) (1994). It is clear that the equations consider multiple failure mechanisms. When these equations are used for diaphragm connections, the result is multiplied by a strength factor to account for additional strength in nailed diaphragms (C_{di}). The value of C_{di} is 1.1. This increase accounts for test values that have consistently measured higher than the calculated values (Bryer 1993). Currently, staple capacities are determined using the older proportional limit equations, since the yield-limit equation values have not been test verified for staples. Tissel and Elliot (1997) in American Plywood Association (APA) Report 138, stated that C_{di} should not be used when calculating stapled diaphragm allowable loads. This statement demonstrate that the equations are not considered applicable for all types of diaphragms and is another justification for cyclical load testing to verify stapled shear-wall load capacities.

2.1.2 Staple Capacities

Staple capacities are published in NER 272 by the International Staple, Nail, and Tool Association (ISNTA) and not in the UBC. Capacities of stapled connections were based on the proportional limit model equations of NER 272 (1997). These equations have been in use since the 1950s. The use of these equations will eventually be changed, but only when research to correlate the yield-limit equations of staples is completed. The proportional limit model is simply $p = K \times D^{\wedge 3/2}$. Few static tests have been performed to verify the staple design values, and very few, if any, dynamic tests have been conducted. None of the most recent dynamic tests performed on wood-framed shear walls have utilized staples.

2.1.3 Test Protocol for Code-Allowable Design Loads

The testing criterion used to verify the tabulated nail and staple shear-wall capacities in new building-design codes for the design of wood-shear walls is a static loading procedure based on American Society of Testing Materials (ASTM) E-72. This criterion is described in *APA Research Report 154*. This report is a summary of shear-wall tests on various framed and fastened specimens, all having the same outside dimensions. The testing procedure specifies the required loading, racking, assembly, and panel specifications. An 8′ x 8′ specimen (note the h/w ratio is 1) framed with 2″ x 4″ or 2″ x 6″ studs at either 16″ or 24″ on-center is sheathed, fastened, and placed in the test assembly. This assembly is designed to allow the top of the shear wall to deflect laterally but not out of plane of the load. The test specimen sits on a rigid 4″ x 6″ piece of timber bolted into the concrete, and the specimen is bolted to the timber. A stop on the end of the wall opposite the load prevents the wall from sliding. Two steel rods embedded in the concrete and secured to the top plate provide resistance to the uplift force. Since the loading is static, the hold-down is only provided on the loaded end. This loading allows the specimen that is 8′ wide and 8′ high, to be loaded in a manner that will cause the sheathing and fasteners to fail in shear prior to any other component of the shear wall failing. However, this type of loading also limits the applicability of the tests to shear walls with h/w ratios of 1. These square walls do not have the h/w aspect ratios of the walls that have raised concerns in post-earthquake inspections.

The sheathing used on the test specimens is configured in two different orientations. One is oriented with the 8′ side in the vertical direction to model blocked shear walls, and in the other orientation the 8′ side is in the horizontal direction to model unblocked shear walls. At the center of the walls that have the sheathing spanning vertically, and when the panel edges have a fastener spacing of 3″ or less, a 3″ x 4″ stud is used to adjoin the two 8′ sides of the sheathing. The ticker member is used to prevent splitting due to the proximity of the fasteners.

The loading sequence of ASTM E72 is applied in four cycles. The first cycle is a load to 790 pounds (99 plf), the second is a load to 1,570 pounds (196 plf), the third is a load to 2,360 pounds (295 plf), and the fourth is a load that will continue increasing until the wall fails. Since most walls are designed for loading in excess of 350 plf, ASTM E72 does not stress the walls to their design values until the fourth cycle. Some researchers have seen this as applying too many cycles prior to stressing the wall to failure and, therefore, have modified their loading procedures (Tissell 1996).

Tissell (1996) modified the ASTM E72 criteria when he conducted tests for APA 154. Tissell increased the loading sequence of the first cycle equal to the target design shear based on code-allowable design loads. The second cycle was loaded to twice the target design shear, and then a five-minute, no-load condition was observed to allow the wall to recover. The third cycle was the final cycle; at this time, the wall was pushed until it failed. Deflection of the shear walls was measured at the opposite end of the applied load. A dial gauge was used between the end of the wall and a steel frame that was designed to support a deflection gauge. The gauge measured total deflection of the wall, including the slip between fasteners, separation of wall studs from the sill plate, and any internal movement between framing members and the wall. Because the hold-down device extends from the concrete to the top plate, there is no separation between the end stud where the load is applied and the sill plate. However, crushing of the sill at the opposite end of the wall from where the load is applied can occur and is, in fact, measured by the deflection gauge at that end of the panel.

APA Research Reports 154 and 138 test a variety of sheathing thicknesses, fastener configurations, and materials for shear wall and horizontal diaphragm construction. Overall, the reports show that the values for shear walls developed from the yield-limit equations and the proportional-limit models are acceptable and provide an adequate factor of safety for use in design based on a static test and a h/w of 1. The safety factors ranged from 2 to 4. The code tabular values for shear walls with common nails are based on yield-limit equations verified through testing. Values for other fasteners are similarly developed. For example, NER 272 (The Power Driven Fastener Capacity Report) is developed strictly from the use of equations with some values verified from testing of 8′ x 8′ panels, primarily by APA. The test specimens all have h/w of one. As the h/w is increased, the load on the individual fasteners in the corners of the shear wall panels is increased due to eccentricities from the hold-downs and deformation incompatibility between the sheathing and the framing. Nelson (2001) discussed these effects in detail.

2.1.4 New Analysis Trends

Current code values for allowable loads on wood-shear walls are consistently being analyzed and critiqued as new codes are published that aim to improve the seismic performance of structures. The Structural Engineers Association of California (SEAOC) (1999) Blue Book is the seventh edition of the *Recommended Lateral Force Requirements and Commentary*, that summarizes the recommendations of the Structural Engineers Association of California for Earthquake Resistance Design of Structures. The Building Seismic Safety Council developed the 1997 National Earthquake Hazard Reduction Program (NEHRP) FEMA 273 and 274 provisions in a parallel effort with SEAOSC. The 1997 NEHRP serves as the source document for the 2000 IBC that will replace the 1997 UBC as the standard code in the United States.

The Blue Book presents current sections of the 1997 UBC and notes the recommended modifications to each section. Chapter 8 covers the wood construction section. The most noted modification to the wood-design section of the 1997 UBC was the maximum allowable h/w ratios allowed for shear walls. The Blue Book limits these ratios to 2:1 in seismic zones 3 and 4. The Blue Book recommends this h/w restriction because of deflection concerns and damage to slender shear walls observed after the 1994 Northridge earthquake. The book also acknowledges ongoing methods to improve performance of slender shear walls and states; as performance, predictability, and quality control are improved, the maximum allowable h/w ratios will increase.

2.1.4.1 AC130.

The effective inelastic response of wood-shear walls is critical to the building's seismic performance. As architectural designs require the use of higher aspect ratio shear walls, construction methods to develop higher load capacities and aspect ratios are being developed. The Simpson Strong-Tie Company has tested and sells premanufactured shear walls that have ICBO-approved capacities that are from 25% to 50% higher than UBC maximum-tabulated loads and have h/w ratios up to 5 to 1. These approved capacities were accomplished according to the ICBO Acceptance Criteria for Premanufactured Wood Shear Walls, AC130.

The 1997 UBC allowed for alternate design and construction methods by stating:

> Horizontal and vertical diaphragms sheathed with wood structural panels may be used to resist horizontal forces not exceeding those set forth in Table 23-II-H for horizontal diaphragms and Table 23-II-I-1 for vertical diaphragms, or may be calculated by principles of mechanics without limitation by using values of nail strength and wood structural panel shear values as specified elsewhere in this code (UBC 1997, Vol. 2 pp 279, 280).

This statement, coupled with Section 104.2.8 of the 1997 UBC states:

> The provisions of this code are not intended to prevent the use of any material, alternate design or method of construction not specifically prescribed by this code, provided any alternate has been approved and its use authorized by the building official. The building official may approve any such alternate, provided the building official finds that the proposed design is satisfactory and complies with the provisions of this code and that the material, method or work offered is, for the purpose intended, at least the equivalent of that prescribed in this code in suitability, strength, effectiveness, fire resistance, durability, safety, and sanitation. The building official shall require that sufficient evidence or proof be submitted to substantiate any claims that may be made regarding its use. The details of any action granting approval of an alternate shall be recorded and entered in the files of the code enforcement agency allowing for the development of alternative construction methods (UBC 1997, Vol. 1 1 p.2).

The purpose of AC130 is to provide procedures for recognizing lateral racking loads on prefabricated wood-shear panels in ICBO evaluation reports. This acceptance criterion has allowed Simpson Strong-Tie Company to develop prefabricated shear walls that exceed allowable loads, and aspect ratios prescribed in the code for field-constructed shear walls. Based on this criterion, only configurations tested are approved for building design, with design loads determined from test data. In addition, all materials used in construction of the panels must be code approved (i.e., framing members, sheathing, fasteners, and connectors).

The design loads for allowable stress design are determined as described below. The allowable stress design load is based on the lesser of the following criteria:

1. Drift limit:
    a. The mean displacement at the strength limit state, Δ_m.
    b. Using Δ_m, calculate Δ_s from UBC Equation 30-17,
       ($\Delta_s = \Delta_m / 0.7 / R$).
    c. From the first cycle backbone curve, determine the force corresponding to Δ_s. This force corresponds to the strength-level factored resistance.
    d. In accordance with Section 1612.3 of the UBC, this strength-level factored resistance shall then be divided by a factor of 1.4 to determine the appropriate allowable stress design level resistance.
2. The average ultimate capacity of the panel with a safety factor of 2.0.

These criteria use a maximum drift limit to ascertain design values computed, and correspond to strength values at deflection of less than 2.5% of story height. By maintaining this drift limit, AC130 assures a ductility of 0.7 x *R*. This drift limit does not measure energy dissipation that could vary significantly from specimen to specimen, depending upon critical component construction details.

Performance-based codes (FEMA 273) use *m* values to account for an element's ductility. *m* values are based on the element's expected strength, elastic stiffness, and inelastic stiffness values, and they are inversely proportional to the desired performance level. The greater the expected performance, the lower the *m* value used in design. The *m* values are based on expected strength elastic stiffness and inelastic stiffness values that are analogous to the reduction of ultimate loads by 1.4 times 0.7 times *R* in AC 130. The *m* values are multiplied by expected strengths Q_{CE} to determine if the resultant is greater than the calculated seismic loads. Since the *m* values are correlated to specific performance criteria, the formulations ascertain the expected performance based on design objectives. Unlike performance-based codes, current codes through the adoption of AC130 limit the ductility to 0.7 multiplied by *R*. *R* is a subjective value that may or may not maintain a certain level of inelastic capacity. FEMA 273/356 uses a more reliable factor, *m*, and provides different *m* values for different levels of desired performance.

AC130 designates a standard cyclical test procedure for code-design variations and determines the results by considering strength and deformation. The current code-accepted loads for shear walls (stapled or nailed) are based on static loading. The AC130 requirements signal a change in the methodology used for determination of lateral force-resisting systems from modeling with static tests to dynamic load testing in wood-framed construction design. Eventually, all wood shear wall fasteners will be dynamically verified. The first test performed for this thesis addressed that issue by dynamically testing 4′ wide x 8′ tall shear-wall components sheathed with 7/16 sheathing and fastened with 16GA staples.

The testing protocol specified that AC130 is the *Standard Method of Cyclic/(Reversed) Test for Shear Resistance of Framed Walls for Buildings* SEAOSC testing procedures. A summary of the testing protocol and required results is in Appendix A.

In order to be submitted for acceptance, AC130 requires a complete report on the tests. The requirements include: (1) details; (2) descriptions of the wall assembly consisting of dimensions, details of attachments, location of load cell, strain gauges, deflection gauges, and other test items; and (3) description of construction material (grade of material, thickness of material, yield point, tensile strength, compressive strength, density, moisture content, manufacturer of components, and source of supply). Drawings should also be supplied showing plan elevation and cross-sections as well as any other section required for assembly. The results should include hysteresis loops, a complete record of displacements, and maximum displacement. Photographs of the failure locations are also helpful, as are the signatures of all of those present to witness the tests.

2.1.4.2 PFC-5485 ICBO Evaluation Report.

The ICBO Evaluation Report (PFC-5485) for the Simpson Strong-Tie Company strong wall shear panel demonstrates the necessary requirements to meet AC130. The report demonstrates the UBC allowance to exceed the tabulated design criteria if proper testing and development are performed for the structural system submitted. The report clearly states that the vertical dimensions in Table 23-II-G of the UBC are exceeded. It also identifies that the allowable shear loads in the report are based on cyclical testing in accordance with AC130, thus allowing the design values determined from testing to be approved. In accordance with AC130, the drift at the allowable shear load is also provided so that the engineer can verify that the UBC story-drift provisions are met.

In order to achieve the performance stated from the shear panels, Simpson Strong-Tie Company made some changes to typical shear wall construction. The strong walls are modified at the end studs where glue-laminated timber is used in lieu of dimensional lumber, the sheathing is 15/32″ thick structural 1 OSB or plywood, and nailed with .148″diameter by 2 ¼″ long (.148″Φ x 2 ¼″) nails. The nail size and sheathing thickness used maximize the values from the limit equations and provide the high-shear values under the cyclical load testing. The design of these walls also includes metal straps to reinforce the shear wall along the perimeter. Hold-downs are mounted 24″ above the sill plate to reduce the load on the shear wall corners. Hold-down anchor bolts used are Simpson SSTB that are significantly longer than standard anchor bolts and are deformed in a manner that will increase the pull-out cone area of the concrete. When used on a garage portal, the walls are designed to be installed under the exterior wall's double top plate or under the garage door beam.

Simpson Strong-Tie Company's development and successful completion of the PFC-5485 demonstrate the extraordinary potential that can be achieved by customized traditional wood-framed construction. The areas of concern that were found to be critical in post-earthquake and hurricane inspections, namely, proper construction techniques and stress concentrations, are addressed with controlled prefabricated construction of the panels, special inspection requirements for anchor-bolt installation, and light gauge metal strap reinforcement. Perhaps, with greater special inspection requirements, a shear-wall element could be designed for field construction that can also achieve greater than code-specified shear capacities and h/w ratios.

2.1.4.3 Cyclical Test Procedures.

Prior to examining research involving shear panels and shear panels with openings, the method of loading the shear panels should be further examined. Current requirements for cyclical testing of existing code-allowable design strengths stemmed from the poor performance of narrow-braced wall panels and shear walls that carried high loads during the Northridge earthquake. Any test methods that are not rigorous should not be considered. SEAOSC testing procedures were specifically developed to evaluate the shear stiffness and shear strength of a typical section of a framed wall (SEAOSC) (1996). Future research will likely use this testing criterion. Several of the research papers reviewed while writing this thesis have reverted back to the test methods used in ASTM E72 or developed a performance friendly cyclic test. Performance friendly test criteria could camouflage areas of poor performance that may be exposed in real seismic events.

Researchers should adopt a uniform test procedure so that data can be assembled on different fasteners and sheathing thicknesses and compared uniformly. Existing code capacities for nails are based on equations that were developed for static loads and verified with static-load tests. In addition, design values for staples that are more commonly used in light frame wood construction than are nails, have not been verified through extensive testing. The model equations used to determine capacities are more than 50 years old. Testing is needed where the sheathing is fastened with staples and the loading is cyclical to better model seismic forces. The needs are twofold: (1) more test data are needed from cyclically loaded shear panels as well as tests that explore various h/w ratios, and (2) simple detailing methods must be developed to improve performance. These are the main issues that the current research investigated. Prior to describing the performed testing, a review of current cyclical test criteria and various research papers that test wood-shear panels is appropriate.

Recent research on shear panels investigated the effects of the loading criteria (static versus cyclical), the sheathing type (OSB versus plywood), and the effects of openings in shear walls. Traditional loading for testing wood-shear panels has been in accordance with ASTM E-564/E-72 test procedures. ASTM procedures consist of a static loading. These ASTM tests have produced ultimate loads having a load factor from 2.8 to 3.5 over the design loads recognized by the building codes (Tissell 1993). Much of the latest research focuses on comparing cyclic test loading to static loading. The movement to cyclic loading was accelerated by the 1994 Northridge earthquake event and post-earthquake investigations. These investigations revealed flaws in the current construction and building code practices, poor performance of tall narrow shear walls, and poor performance of eccentric hold-downs.

> After the Northridge, California, earthquake in January 1994, evaluation of shear wall performance by cyclic load tests was encouraged by a local building code change adopted by the City of Los Angles, Department of Building and Safety in response to recommendations from the Structural Engineers Association of Southern California (SEAOSC). (APA research report: *Preliminary Testing of Wood Structural Panel Shear Walls Under Cyclic (Reversed) Loading*, p. 5).

The recommendation reduced the allowable design capacity of shear panels to 25% of the values published in the building code until cyclic load tests could be conducted to confirm previously recognized values. This recommendation has caused the most recent research to focus on the change to cyclic testing. (Applied Technology Counsel (ATC R-1, 1995); Dinehart & Shenton (1998); Rose (1998); Ming, Magnusson, Lam, & Prion (1999)). All investigations performed cyclical tests on wood-shear walls.

The aforementioned articles focused on the differing results between static loading and cyclic loading of shear wall test specimens. Dinehart and Shenton (1998) found that load factors of shear walls (ultimate loads divided by design allowable loads) decayed with each load cycle beyond the strength limit state (SLS), whereas the static test having only one cycle could not show this decay. Dinehart and Shenton also found that failure modes between the two testing methods differed dramatically. The failure mode of the static test was the sheathing pulling away from the framing, with the bottom plate and the end-wall studs splitting. In the dynamic case, the failure mode was confined to the sheathing nails that fatigued and broke off. As shown in my prior review of the loading procedure for the cyclic load tests, the majority of the cycles exceeded the yield point of the walls. Dinehart and Shenton stated that fastener fatigue failure is not consistent with post-earthquake inspections, and they also questioned the testing procedures. However there are many reasons why the failure mechanisms witnessed in post-earthquake inspections differ from what is witnessed from testing. Explanations could be that during earthquakes, shear walls could not be developed to their full capacity due to poor construction and design practices (e.g., incorrect hold-down installation, improper fastener edge distances, eccentric hold-downs, and h/w ratios that create greater tension and compression forces at the wall's base). These poor construction practices would cause failures in other modes (e.g., in hold-down devices, sill plates, end-wall studs, and panel corners).

Dinehart and Shenton (1998) examined the yield-mode equations for a test specimen and used 15/32″ sheathing with 8d common nails at 4″ on-center to 2″ x 4″ studs at 16″ on-center. Yield limit equations predict a Mode IIIs type failure, which is a plastic hinge forming in the nail and crushing the side member. This type of initial failure would lead to nails fatiguing and breaking off in the sheathing. Therefore, their test results show that the cyclical testing stresses the shear wall nailing and sheathing, whereas the static testing stresses other components of the wall that fail prior to the sheathing and nails failing. If shear-wall elements were detailed and constructed according to more stringent standards, individual components would not fail, shear walls would carry higher shear loads, and nails would fatigue during an earthquake. Recent code changes stress component detailing to prevent failures observed in earthquakes and static testing. In addition, earthquake duration cannot be predicted, including aftershocks. If a new test method, reveals a new observation of a failure mode, the new test should not be dismissed or modified to prevent this type of failure during testing.

The tests performed for APA Research Report 158 were all cyclic. All of the failures for the walls using .148″ϕ nails failed due to fastener fatigue, starting in the lower corners of the panel, and around the hold-downs. This finding is consistent with the findings of Dinehart and Shenton (1998). The static test performed in *APA Research Reports 138 and 154* did not indicate failure by fastener fatigue but by failure of other components of the shear-wall system. Testing trends showed that the cyclic loading caused a reduction in the fastener strength when compared to the static test results. These trends were due to the fatigue failures of the fasteners that govern maximum loads achieved during testing.

Recently, cyclical or dynamic testing for research was performed using one of two testing protocols. The two testing protocols were Fornitek Canada Corporation (FCC) and SEAOSC. Figure 2.1 shows that the two tests are similar in the number of cycles in each test; however, the tests differ by the percentage the load is increased past the yield point, FME, and in frequency specified. The SEAOSC protocol specifies a higher frequency of 1 Hz at earlier cycles and .2 Hz for large displacement cycles. The FCC protocol specifies .25 Hz at lower displacements for cycles 1 to 57 and .083 Hz for cycles 58 to 66.

He, Magnusson, and others (1999) discussed the trend that extensive yielding occurs in the FCC protocol testing that is similar to the SEAOSC testing protocol but shorter in duration and higher in displacement. They believe that this type of dynamic load causes nail fatigue due to the high frequency of the cycles and the prolonged loading of the fasteners. They recommended a third dynamic loading that cycles smaller loads initially slower and then goes into a static mode of pushover until failure. Their new test cycles two or three times for each loading, starting with a load of .5 P_{max} (static) to .8 P_{max} and finally to a pushover until failure statically. These tests are similar to that of ASTM E-564's optional cyclic test. The low cyclic frequency of this test and the finish in a pushover mode seem to revert the test to a more static testing than a dynamic testing.

The ATC's cyclic testing of narrow plywood shear walls (ATC R-1 1995) used a dynamic loading similar to the FCC protocol loading discussed above. The failure modes in both the static and dynamic tests were similar to failures found in other tests. The

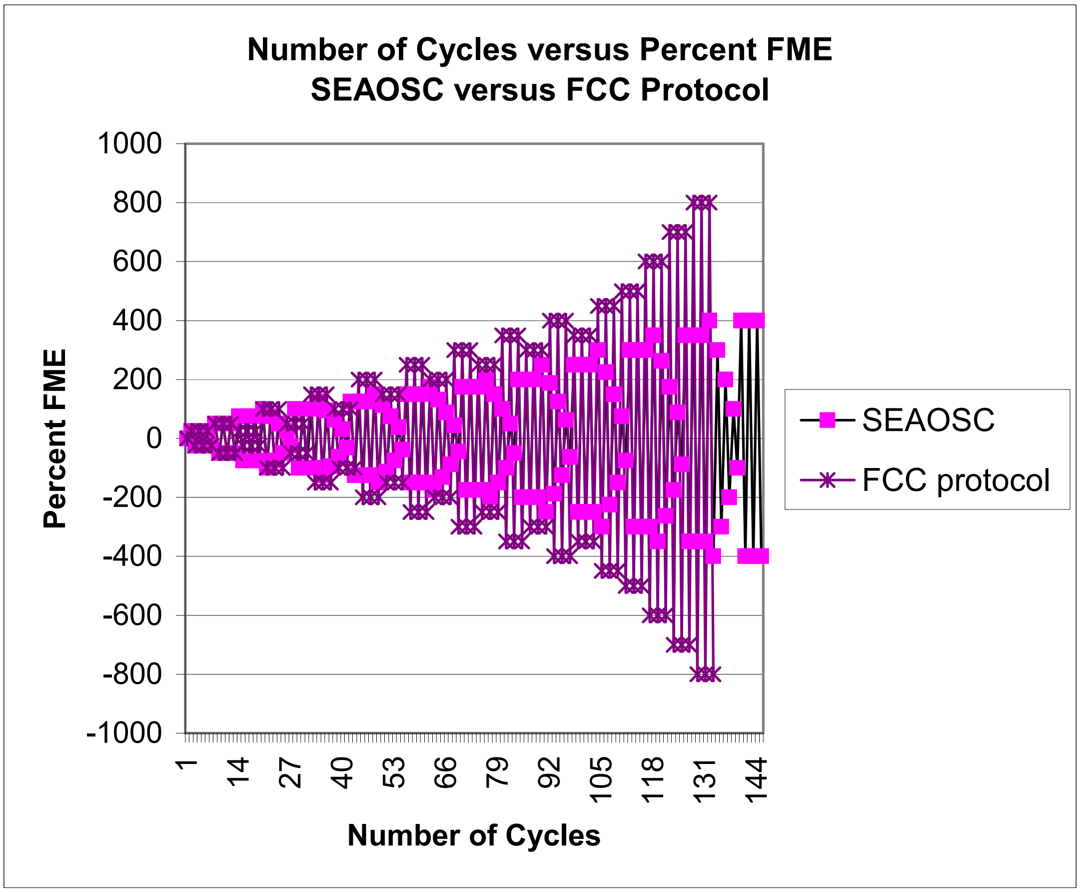

Figure 2.1. Comparison of SEAOSC and FCC loading protocol.

static failures occurred from initial movement in the hold-down device that gave way to stiffening and then vertical cracking in the sill plate. The sill plate began to twist due to the uplift from the sheathing on only one side of the shear wall. Final failure occurred when the sill plate failed. The dynamically loaded test began to show failure when the bottom sill plate suffered inelastic compression at the ends of the wall, demonstrating higher vertical loads due to the narrow shear panel. Looseness in the hold-down device brought on a vertical crack in the sill plate and, finally a pulling out of plywood nails from the framing as the panel neared its maximum load.

Clearly, cyclical load testing causes different failure mechanisms from statically loaded tests. It is still uncertain that testing type represents failure modes that occur during an earthquake more accurately. However, some form of cyclic loading must more accurately mimic earthquake loads.

Regardless of the testing, wall designs must account for the high compression and tension forces on the ends of these panels. Moreover, testing should be done in a manner that accurately stresses the component being tested. If the goal of the test is to determine shear panel load tables, then the walls should be restrained so that the shear panel and its connections are the limiting factors. Not understanding how the components will act when constructed together is why a complete system approach is desired. The specimen tested should be identical to the specimen specified in the field.

Both Dinehart and Shenton (1998) and Rose (1998), investigated tests that were conducted to determine the load factors of walls when loaded dynamically. They performed tests that verified that the dynamic test produces load factors around 2.5 when compared to the tabulated code-design values for shear walls. In most tests, the load factor decreased with additional cycles beyond the wall's ultimate strength (Dinehart & Shenton 1998). Based on the research mentioned above and the recent ICBO movement to verify load factors with cyclical tests, the testing procedure of choice is dynamic.

2.2 ATC Report R-1, Cyclic Testing of Narrow Plywood Shear Walls

ATC Report R-1 (1995) was one of the first research efforts to look at the performance of narrow shear panels under reverse cycle dynamic loading. The ATC was motivated to conduct this research due to the lack of prior cyclical testing of narrow shear walls and the associated hold-down devices. Concerns by some members of the SEAOCS about the seismic-resisting characteristics of narrow plywood-sheathed wall panels and the code allowance for shear-wall ratios to reach 3½ to 1 h/w ratio also prompted the need for the research. Previous testing of shear panels was performed using static tests of walls with h/w ratios of 1 or greater (*APA Research Report 154*). These tests do not provide an accurate load-deflection curve for narrow shear walls incorporating hold-down devices. For this report, the ATC performed static and dynamic testing of the shear panels. This testing report would allow a comparison of the traditional testing results versus the cyclical testing results as well as the development of hysteresis loops from the cyclical tests.

The test specimens were 27 ½″ x 8′-0″; 2″ x 4″ wall panels sheathed with 3/8″ structural 1 plywood with 8d common nails at 4″ on-center at edges and 12″ on-center field nailing. End-wall studs and top plates were doubled and a single stud in the center of the wall and a single sill plate were used. All studs were Douglas Fir construction grade. The test specimens were mounted on a concrete base that was bolted to the concrete floor. The actuator was applied to the top of the wall where lateral bracing was provided. The hold-downs used at the ends of the shear wall were 9″, 12 gauge 5,000 pounds capacity eccentric hold-downs, bolted to the end-wall framing with two ¾″ diameter through bolts in ¾″ tight-fitting holes. The vertical hold-down capacity was developed with a single ¾″ rod anchored to the foundation. An interesting note is that the hold-down anchors passed through oversized holes in the sill plate so that all of the lateral resistance was transferred to the two 5/8″ diameter Hilti expansion bolts drilled into the concrete and attaching to the sill plate. The Hilti bolts were installed in line and were symmetric to the hold-down bolts.

The testing procedure included static and dynamic tests. The static tests were performed by gradually applying a horizontal displacement at the rate of 2″ per minute until the ability to resist the maximum horizontal applied load was significantly diminished. The dynamic tests were a series of applied displacement excitations. Loads were applied as a series of three-period sine waves within a 1.5-second time period (2Hz). The first series was equal in amplitude to 50% of the code-rated capacities, with increments increasing by 50% of the code-rated capacities. When the resistance of the wall began to level off, the incremental increases were reduced to 25% of the code-rated capacity. The deformation amplitude was increased by this amount in each subsequent loading until failure occurred. The amplitude of failure was 6″ positive and 3″ negative. Part of the limitations in this portion of the testing was in the actuator operating at a high stroke and still maintaining the frequency of 2 Hz that is still one of the higher frequency tests performed.

The static test results demonstrated an ultimate panel load of 2.2 kips, with approximately 8.3″ of deflection. The failure mechanisms were a combination of hold-down deformation, sill bending and twisting (from cross-grain tension), and hold-down bolts splitting the end-wall studs. The hold-down deformation initiated when the applied force exceeded 750 pounds. The hold-down stiffened after this initial deflection, but the initial deformation caused bending in the sill plate back to the anchor bolt and cracking the sill plate vertically along a plane that corresponded to the tip of the nails. The sill plate started to undergo twisting as the plywood pulled upward at the edge of the plate. (This sill plate cracking was the same type of failure that occurred in Panel 4 of the thesis tests). At 2,200 pounds, the cracking became splitting, and the wall failed. The sill plate broke up, splitting at the corner and at the position of the Hilti expansion bolts. The double 2″ x 4″ studs also fractured vertically along the plane of the hold-down bolts.

Dynamic test results demonstrated failure mechanisms in the hold-down device. Deformation in the hold-down device and bolt slippage occurred in the first two to three cycles. Vertical cracks appeared in the sill plates that were most likely from cross-grain tension on the sill plate attributed to the eccentricity between the anchor bolts and the plywood. Deterioration of the sill plate was not as great as in the static case. When the deteriorated panels were subject to repeated shear deformation of a maximum of 10″, nails suffered fatigue and typically broke off ¾″ below the head. At this point, the wall no longer continued to act as a shear wall. Recorded load and deflection data indicated the horizontal slip at the base. Diagonal movement of the plywood was insignificant, .08″ at the ultimate load.

The conclusions and recommendations of the report were that static and dynamic loads demonstrated initial deformation of the hold-down device, causing the walls to have substantial inherent flexibility. While the ultimate magnitudes of the allowable loads were of the same order of magnitude, the failure mechanisms were dissimilar. The dynamic tests that are probably more indicative of earthquake loads, demonstrated several mechanisms of failure. The common initiation of failure, however, is the hold-down device. The initial deformation of this device led to excessive deflection of the shear panel, far more than the allowable code, and ultimately too great for shear panels supporting second stories. Two suggestions to improve the design of these panels are: (1) the development of strap-type hold-downs that are common today; and (2) the use of L-shaped metal ties on the inside corners, opposite the sheathing, between the studs and sill plate. The authors also stated that designers and engineers specifying these narrow shear walls need to be aware of their inherent flexibility and to distribute the loads to wall panels accordingly. This inherent flexibility exceeded the code allowable for inelastic drift limit. This point is commonly overlooked in the design of single-family residences.

In order to improve the engineering of narrow shear panels, a standardized test procedure is required so that tests can be run to develop better hold-downs and fastening requirements. Presently, this work has progressed, and Simpson Strong-Tie Company has developed strap-type hold-downs and “predeflected” hold-downs to eliminate the initial deformation that was responsible for so much of the test wall’s initial deflection. This paper, ATC R-1 used the test procedures from ATC-24. In July 1997, ICBO, through AC130, adapted as the accepted loading criteria the SEAOSC’s “Standard Method of Cyclic Load Test for Shear Resistance of Framed Walls for Buildings.”

The appendix of ATC R-1 explored the use of two types of hold-downs: (1) the manufactured strap-type hold-downs; and (2) a custom-made hold-down. The strap-type hold-down failed at the reduced section of the hold-down yet provided approximately

40% less horizontal deflection than the previously tested panels with the bolted hold-downs. The custom hold-down that was designed with the goal of eliminating any slack in the connection, performed only slightly better than the hold-downs used in the original test. (This poor performance is due to the hold-down being the eccentric type.) During one dynamic test with the custom hold-down, the lag bolts failed prematurely. The bolts were retightened, and the test was performed again on the same panel. The hold-down performed better in controlling the initial deflection probably because of the previous loading predeflecting the hold-down.

ATC R-1 focused on the cyclical testing of narrow shear walls. This research demonstrated how cyclical testing causes different failure mechanisms than static tests. ATC R-1 also demonstrates how narrow shear walls have deflections that exceed the code allowable for inelastic deformation. Typically, these deflections are not realized when designing from load tables and are another example of an erroneous design process. The deflection limitation is a result of the high-aspect ratio and the eccentric hold-down.

2.3 Comparison of Static and Dynamic Response of Timber Shear Walls

Dinehart and Shenton (1998) examined performance differences between dynamic and static testing of plywood and OSB-sheathed, wood-framed shear walls. The authors acknowledged that the vast majority of construction in the United States consists of wood-framed buildings and that the lateral load resistance of these structures relies on the wood-shear walls. Current published building code-design capacities are based on static testing. The authors, noting that wind and seismic loads are dynamic in nature, wished to examine the differences the static loading and cyclical loading would have on the plywood and OSB sheathed shear walls. In 1996 the SEAOSC also developed a draft standard for testing wood-framed shear walls. Dinehart and Shenton choose this standard for the cyclical loading in their tests. This procedure was conducted using full reverse cycles of increasing amplitude and at frequencies between .2 Hz and 1 Hz. The test sequence is referred to as a sequential phase displacement (SPD) test. This procedure is fully examined in Appendix A; therefore the details will not be repeated. The test specimen used was an 8′x8′ shear wall with double end-wall studs, a double top plate, and a single sill plate. The plywood shear walls and the OSB walls were nailed with 8d nails on 4″ on-center. The shear panels were anchored to the steel frame with 12, ¾″ diameter anchor bolts, and two hold-down anchors, one at each end. The authors chose the high number of anchor bolts to ensure that the base would not slide along the rigid base. Since these walls have a h/w ratio of 1, the uplift forces experienced in ATCR-1 shear panels would be greater.

The tests used static loading procedures of ASTM E-64 and the dynamic SPD procedure of SEAOSC. Four test specimens (two sheathed with plywood and two sheathed with OSB) were tested according to ASTM E-64. Eight test specimens (four sheathed with plywood and four sheathed with OSB) were tested in accordance with the SEAOSC testing procedure.

The static test results demonstrated the yield limit, the ultimate load capacity of the panels, and stiffness. The results were as expected. The ultimate loads were reached at 2.5 and 2.4 times the design allowable load. The stiffness recorded at the end of each load cycle was slightly higher for the OSB walls than for the plywood walls, yet the plywood walls had a slightly higher ultimate load. The static test indicated that the OSB is stiffer but less ductile. Failure mechanisms for the static test were similar to the failure mechanisms in the static ATC tests. Along the sheathing edges, the plywood tended to lift away from the wood frame, pulling the nails with it. Another component failure was the sill plate split parallel to the grain at the uplift corner. The splitting occurred along the ends of the panel edge nailing in the bottom plate. (This failure is similar to the ATC R-1 failure mechanism for the static test they performed). No mention of hold-down failure was recorded which was probably due to the low h/w ratio of the walls.

The dynamic SPD test results revealed ratios of ultimate loads to allowable design loads for the dynamic tests that were slightly lower than those determined by the static tests. The plywood ultimate load ratio was 2.4, and the OSB ultimate load was 2.1. Both of these loads were lower than the 2.5 and 2.4 respective load factors for the static tests. In comparing the two different materials' performances during the SPD test, it is interesting to note that the stiffness of the plywood after several cycles was greater than the OSB. This resulting stiffness is opposite of the results of the static test.

The damage observed during the dynamic testing was confined to sheathing fasteners. Fastener failure occurred when the nails sheared off at the face of the studs or pulled out of the studs. Little or no damage to the interior field nailing was noted. Another difference in failure mechanisms between the two tests was that the interior studs did not twist, and the sill plate did not crack and split; both of that were failure mechanisms in the static tests. Damage started in the corners of the sheathing and then spread along the panel edge as the fasteners picked up additional load. The plywood and OSB panels behaved slightly differently from one another during the dynamic testing.

The main difference between the plywood and the OSB was that the OSB fractured at the corners, demonstrating some brittle behavior.

Comparing the static and dynamic test results revealed three major differences in the static and dynamic responses of the shear walls. First, at the same ultimate loads, the corresponding displacements measured during the dynamic tests were significantly lower (by approximately 33%); second, the decline in wall stiffness after ultimate load was reached was greater in the dynamic test; and third, the mechanism of failure was different between the two tests.

Dinehart and Shenton (1998) calculated a notable difference between the ductility of the static tests and the dynamic tests. The dynamic tests indicated a lower ductility that was defined as the displacement at ultimate load divided by the displacement at yield load. When calculating the ductility for the static and dynamic tests, the authors assumed the same yield displacement of 6 mm (0.236″). From this assumption, they demonstrated the decrease in wall ductility when comparing static and dynamic tests. Ductility is defined as the property of a material to be drawn out to a considerable extent before rupture; therefore to state that the dynamic ductility is lower than the static ductility is erroneous. This is erroneous due to the number of cycles the element experiences in the dynamic test. The SPD test has 61 cycles past the static yield point, with approximately 49 of these at a force equal to or greater than the static yield point. With this in mind, why do Dinehart and Shenton claim “static ductility is greater than the dynamic ductility” (p. 691)? What Dinehart and Shenton should state is that in cyclical applications the ductility of a wood-framed shear wall may be insufficient to provide the required inelastic response to seismic design loads.

An important assumption made by Dinehart and Shenton (1998) with the ductility calculations was the value of the assumed yield displacement (Δ_y). The importance of Δ_y is demonstrated in Table 2.1. Table 2.1 shows that if the assumed yield displacement occurred 2 mm (0.0787″) earlier in the dynamic tests than in the static tests, it would lessen the difference in ductility between the two tests. In fact, the calculated plywood ductility showed no difference between the two tests. Reducing the assumed yield displacement by 2 mm or 33% matches the difference between the ultimate load displacements of the two tests, since the authors reported that the ultimate loads found in the dynamic tests were comparable to the static ultimate load; however, it occurred at a 33% smaller displacement.

Table 2.1 Ductility Comparison

Sample	Average dynamic Δ_{ULT}	Average static Δ_{ULT}	Assumed dynamic Δ_{yield}	Assumed static Δ_{yield}	Dynamic Ductility	Static Ductility
a) Plywood	54.6	83.1	4	6	13.65	13.85
OSB	46.6	79.2	4	6	11.65	13.2

Note: All dimensions are in mm.

Dinehart and Shenton (1998) justified their support of a reduction in tabulated code-allowed shear-wall values based on the reduced ductility demonstrated in the dynamic tests. The differences between measured dynamic Δ_y and static Δ_y ductility should form the bases of a ductility comparison. The fact that a reversed cycle dynamic load test would show less calculated ductility than a static test is intuitive by the nature of the two different tests. What should be studied is energy dissipation. The amount of energy absorbed by a specimen in the SEAOSC SPD test is much higher than the static test. The wall, of course, will lose ductility after excursions into the higher cycles of the test. In addition, this loss of ductility in the dynamic tests is probably why the 1997 UBC through AC130 accepts a load factor of 2 for wood-shear walls tested dynamically rather than the traditional load factor of 3 that was determined after static tests of various shear-wall configurations (Tissell 1993).

Dinehart and Shenton (1998) next discussed the impact on shear-wall design. Their findings suggested supporting a reduction in allowable shear-wall loads. They began their justification by noting that the seismic design of buildings, typically framed with sheathed shear walls as the lateral-load resisting elements, is performed by using a simplified equivalent static load, rather than a dynamic load. This analogy is an interesting approach to the subject matter being researched. As a result of their test comparisons, Dinehart and Shenton stated, "An equivalent static procedure is necessary for design of timber structures until a design friendly dynamic analytical mechanism is developed" (p. 693). Any concerns with the code model determining lateral load forces should focus on linear analysis procedures as well as overstrength factors assumed in design building codes and available in the lateral-load resisting system.

More research is needed on the current design values for shear-wall capacities that are based on model equations and verified with static tests on shear walls with h/w ratios of 1 or less. This thesis confirmed that this research is needed. The question that surfaces is, "While an 8′x8′ shear wall may demonstrate ductility in the latter cycles of an extensive dynamic load test, will narrower shear walls that depend more on the boundary elements for stiffness perform as well?" Certainly, the authors were alarmed by the reduced ductility demonstrated by the dynamic tests even though the dynamic tests still produced load factors of 1.7 and greater.

What engineers should understand from Dinehart and Shenton's (1998) research is how allowable shear wall load tables are constructed, how the values are used in design, and what load deformation relationships are developed from dynamic loading. The allowable design loads should be based on a dynamic load test rather than on a static load test. Just as significant is jumping from verifying loads based on a panel with a h/w ratio of 1 and then applying those values to a panel with a h/w ratio as high as 3½ to 1.

The dynamic tests have provided a look into inelastic behavior of shear walls and demonstrated the decay into the latter cycles. The 8′x8′ shear walls still performed well, maintaining load factors above 1.7 after the ultimate load was reached and serious permanent deformation had occurred. This testing justified only the current load tables for the h/w ratio of the wall and fasteners tested. However Dinehart and Shenton failed to address the more critical design and analysis issues of h/w ratios, inelastic behavior, over-strength factors, and demand-capacity ratios of the lateral-load resisting system.

2.4 Comparison of Static and Dynamic Response of Timber Shear Walls, Discussion

Karacabeyli et al. (1999) responded to the Dinehart and Shenton (1998) research. The main point of the response was to state that they disagreed with a reduction in published allowable strength shear-wall design loads that Dinehart and Shenton recommended. They referenced their own research that states that the ultimate load experienced by a shear wall during a seismic event is no greater than the load experienced in the first cycle of the SEAOSC test protocol. Karacabeyli et al. believed that the test protocol used by Dinehart and Shenton contained many more displacement cycles than an earthquake would impose on a structure, and therefore, the results did not justify a reduction in the allowable design loads. With the SEAOSC procedure lasting more than 2 ½ minutes, their point may be well taken. The Elcentro ground motion data are only 31.2 seconds in length (Chopra 1995). However, to suggest a design earthquake is of certain duration is ignorant. In addition, the performance of a structure in lesser earthquakes is also a consideration. To rebuff a testing standard due to duration would jeopardize understanding the complete range of the shear wall's performance.

One area that previous researchers do agree on is that the common cause for fastener failure in post-earthquake inspection is nail pullout, not nails breaking within the lumber (ATC Report R-1, 1995), Dinehart and Shenton (1998), Karacabeyli et al. (1999). This failure mechanisms may be a difference in performance between pneumatically driven nails that are much more common in building construction, and hand-driven nails that are commonly used in laboratory experiments. Pneumatically driven nails are typically shorter. Rose (1998) found that a greater portion of the shorter pneumatic nails pulled out and then fractured in the lumber when compared to longer hand-driven nails.

Some researchers have tried to point to the length of the SEAOSC testing procedure (also known as the TCCMAR testing procedure, (Rose 1998)) as the reason for the nail's fatigue rather than pull out, but the real cause may be fastener length (Rose). Therefore, it is inconclusive whether or not the findings of Dinehart and Shenton (1998) support the SEAOSC recommendation to reduce the shear load values in the UBC tables by 25%. In 1997, the UBC did not lower the allowable load design values as recommended by SEAOSC and Dinehart and Shenton.

2.5 Preliminary Testing of Wood Structural Panel Shear Walls Under Cyclic Loading

Rose (1999) investigated the performance of 8′x8′ shear walls subjected to dynamic, fully reversed, cyclical loads, as specified by SEAOSC. The purpose was to provide preliminary data on load displacement characteristics and load capacity of shear walls when subjected to dynamic loads, to study two different cyclic load-displacement test protocols, and to evaluate whether or not gypsum wall board installed on one side of the wall contributes to overall strength and stiffness. The tests were constructed with 8′x8′ shear wall and sheathed with three different thicknesses of plywood and OSB. The sheathing thicknesses (15/32″, 7/16″, and 3/8″) were structural one grade. Fasteners were 10 common nails (0.148″ diameter x 3″ in length) and 8d common nails (0.131″ diameter x 2 ½″ in length) hand-driven nails, but with one exception, a test with 10d common short nails (0.148″ diameter x 2 1/8″ in length). Fastener spacing varied from 3″ on-center for the 8d common nails and 4″ on-center for the 10d common nails.

The hold-downs used for this test were specially constructed from welded structural steel components that are significantly stronger than the cold-rolled 14 GA material typically used in wood-framed residential and commercial construction. The purpose of using these hold-downs was to minimize deflection from the hold-downs. Since the rated capacity of the walls, according to the UBC design table, is in excess of 350 plf; 3″ x 4″ sill plates and center studs were used. The end-wall elements used were 4″ x 4″s assuring that the failure mechanism would involve the sheathing and fasteners. Both the h/w ratio of the walls and the beefy construction of boundary components that have been the cause of failure in other tests (ATC R-1) predicted the failure mechanism.

The tests proved this prediction. The dynamic loading of the test specimens was performed using a 55 kip, double-action actuator connected to a hinged column. This linkage amplified the displacement and reduced the applied force of the actuator. The force displacement relationship was established for each cycle. The test procedure was based on the FME of the element. Rose (1998) found that this to occur at 0.8″ that compares favorably with Dinehart and Shenton's (1998) findings of 0.75″ for 8′x8′ walls. The load-displacement relationship curve (hysteresis loop) developed from the tests showed significant decay in the structure's rigidity after 10,000 pounds that is approximately 2.5 times the design load (510 plf). These loads and displacements were comparable to the results published by Dinehart and Shenton, with comparable load factors for plywood. Both tests calculated a load factor of 2.5 times the design load.

The shear panels' load displacement relationships were linear and repeatable up to a displacement of 0.48″. The maximum shear load at this displacement was on average 1.4 times the allowable design load. Rose (1998) continued to describe the hysteresis loops developed. At the next significant displacement, 0.96″ (1% of the shear-wall height), the hysteresis loops were still repeatable but nonlinear. As the test panels were subjected to repeated cycles, the displacement increased up to 1.6″ without any increase in load. This “expected maximum shear strength” provides the basis of performance for shear walls subjected to cyclical loading. Beyond 1.4″ to 1.6″ of the strength limit state, yielding and resulting fatigue failures of fasteners resulted in decreasing shear load capacity at each repeated cycle. The hysteresis loops were also not repeatable and the shear load was not stabilized.

Comparing the test results to previous static test results indicates that the shear strength for repeated cyclic tests averaged 18% lower than for static tests. Rose’s (1998) test also produced strengths that were less than the loads determined from other tests (Schmid 1995); however, there were load application differences. In Schmid’s test the load was transmitted through a guide that lay on top of the sheathing, thus preventing rotation of the sheathing component and increasing the load capacity. Rose believed this was analogous to having a bearing load on the panel such as a floor joist or roof trusses. In summary, considering the results from Schmid’s test and Rose’s results, I believe that the 25% reduction in the shear wall allowable loads implemented by the city of Los Angeles is conservative for walls with h/w ratios of 1 or greater.

Comparing the test results between the plywood and the OSB, the wall displacements were consistent between the two; however, the strength limit state for OSB was approximately 10% to 20% lower than with plywood. Rose stated that this is due to the higher density of the OSB that actually changes the fastener failure mechanism from failure between the sheathing and framing to failure in the framing. The higher density OSB caused the shear walls to be less ductile than the softer plywood. This relationship is something that additional testing would have to verify.

Rose (1998) tested one panel sheathed with gypsum wallboard on one side and plywood on the other. Initially, the gypsum provides additional stiffness, but this additional stiffness deteriorated rapidly when the yield strength of the wall was reached. No additional overstrength was provided.

Rose (1998) also tested a panel with pneumatically driven nails. These nails were the same diameter as the10d common nails but were shorter in length. This panel was also sheathed with OSB. The wall demonstrated lower shear strength but a greater "energy dissipation" than the shear walls with the longer nails. The failure mechanism revealed that a greater proportion of nails withdrew from the framing but fewer fractured. Withdrawal failure is consistent with an increased displacement without a significant reduction in cycled shear strength. The shear walls with the longer hand-driven fasteners showed a decrease in wall strength at a displacement of 1.6″; this panel with shorter pneumatic nails did not show a decrease in shear strength until 2″ of displacement. Then the reduction in shear strength of the pneumatic panel was gradual. This contradicts one of the concerns of Dinehart and Shenton (1998), which was that the wall with the longer hand-driven nails had a sharp decline in shear strength. Rose recommended more studies relating to the length of fasteners tested with cyclical loading to wall performance. Rose also reported that the greater energy dissipation may be offset by the fact that excessive deflection may not be tolerated in a structure.

Rose (1998) tested one panel with a different SPD cyclical test. The main difference between this test and the SEAOSC test is that it is shorter and contains no decay cycles; it is only a ramped test. The results matched the identical panel tested to the SEAOSC. Rose concluded that this new test could be an alternative test to the

SEASOC test, but saw no reason to deviate from the standardized procedure offered by SEAOSC.

APA used specially designed hold-down anchors for these tests. The slip was limited to .039″ at the expected maximum shear strength of the wall. This limited slip indicated that the forces in the hold-downs were high, although not recorded, and eliminated the uplift forces from being transferred into the panel sheathing and anchor bolts. Rose stressed the importance of limiting uplift in any shear-wall test. The uplift failure mechanism (the tearing of the sill plate and end-wall stud failures) controlled the results of ATCR-1.

The design and construction considerations that Rose's (1998) test revealed were the importance of the end-wall studs and hold-down devices. If the wall is to achieve its strength limit state, then every component of the element must be able to achieve this state. The end-wall studs and hold-downs must not fail prematurely if the strength limit state of the sheathing and fasteners is to be reached. This failure sequence may seem obvious, but the allowable shear-wall design loads are currently based on 8′x8′ panels. The h/w ratio of this element is such that the end-wall forces can be met with standard design practices. During the design process, the allowable loads, based on the 8′x8′ panels, are used for the design of narrower walls; yet, the design of the hold-down and end-wall studs are based on the allowable design load calculated from simple overturning expressions for a rigid element. The load factors and stiffness of these elements are not considered; therefore, the reaction of the wall element after the yield-limit state is reached is uncertain. With narrower shear walls, as demonstrated in ATCR-1, slippage in the hold-downs caused the sheathing to tear at the bottom plate causing premature panel failure.

In the design analysis section, Rose (1998) demonstrates how to use the information in the hysteresis loops to determine the length of shear walls required for a building. Rose used both the yield-limit state (YLS) and the strength-limit state (SLS) test values. The required lengths determined from both methods are then compared and the more conservative value is used. The calculations using the YLS values are analogous to using the ASD calculations in the UBC. The design loads used are based on the static force procedure of Section 1630.2. The calculations using the SLS values are used with an ultimate strength design method. The required seismic capacity was developed from the design response spectra in Figure 16-3 of the UBC. A capacity reduction factor of ϕ is applied to determine the required length of shear wall. This section gives the designer an idea of how to take results from tests and to determine design capacities to be used according to the UBC.

In Rose's (1998) report the Appendix contains plots of the hysteresis loops. The first three tests that are plywood panels have a FME that occurs at 0.8″, and the load is slightly less than 10,000 pounds. The fourth test, OSB sheathing, has a similarly shaped backbone curve, but it peaks at a much lower level than the first three tests (at almost 8 kips compared with 10 kips for the plywood). The unrepeatability of the curves after the ultimate load is reached indicates that under dynamic loading the decay of the structure is rapid and load-bearing capacity is quickly lost.

Initially, after the Northridge earthquake, SEAOSC recommended reducing the allowable design loads by 25% for shear walls specified in the 1994 UBC. Rose (1998) did not support these recommendations based on the preliminary test results. A review of the SEAOC (1999) indicates that this recommendation has been eliminated. However, the boundary conditions used in these tests are not commonly found in standard construction practices. The 4″ x 4″ end-wall studs are not required by code; however, the intermediate 3″ x 4″ studs at the internal panel edges and at the 3″ x 4″ sill plate are required by the UBC. These components were critical in allowing the sheathing fasteners to reach failure mechanism and should be recognized for their importance by designers.

2.6 Cyclic Performance of Perforated Wood Shear Walls with Oversize OSB Panels

He et al. (1999) reviewed previous studies of shear walls with openings under static and dynamic load conditions. Several studies extrapolated equations to model the experimental results. The empirical formulas derived resulted in additional formulas that indicated that the reduction in shear strength of a wall was directly proportional to the area of the opening. The equations developed by Patton-Mallory and Sugiyama (1985) were examined and found to be inaccurate as shear panels became tall and slender and deflection became a more crucial design parameter. One goal of He et al. was to determine design criteria that would accurately model wood shear walls with openings under seismic conditions. Another purpose was to investigate the structural performance of perforated wood-shear walls constructed with standard-sized and oversized sheathing, and to analyze the failure mechanism.

He et al. (1999) compared an 8′ tall x 24′ wide wall sheathed with standard 4′ wide x 8′ tall OSB sheets with a wall of the same size, sheathed with one oversized sheet of OSB. The other comparison was between an 8′ tall x 24′ wide wall with openings sheathed with standard-sized 4′ wide x 8′ tall OSB sheets against a wall sheathed with an oversized sheet with the openings cut out. The authors constructed eight test elements two of each configuration: (1) two elements without openings and with oversized panels, (2) two elements without openings and with standard-sized panels, (3) two elements with openings and with oversized panels, and (4) two elements with openings and with standard-sized panels.

He et al. (1999) installed the test elements beneath a steel WF beam and bolted the top plate to the beam to conduct the tests. The end of the beam was then connected to an actuator that was attached to a braced steel WF column. The braced column supporting the actuator was stiff enough to ignore any lateral deflection. A uniform vertical load was applied along the length of the wall to simulate a two-story dead load and to eliminate the need for any hold-downs.

The framed wall configuration was standard 2″ x 4″ framing at 16″ on-center and fastened together with 0.105″ diameter x 3″ long nails; these were not 10d common nails as stated by the authors, but were pneumatic 3″ nails. A 10d common nail has a diameter of 0.148″ that is significantly thicker. The sheathing used was all 3/8″ thick performance-rated OSB. The sheathing was fastened to the wall studs with 0.105″ diameter x 2″ long pneumatic spiral nails. The spacing of the fasteners varied. The walls were framed with standard framing practices with two studs around all openings and at wall ends and top plate.

Both the nail types and the spacing used was peculiar. Field nailing was consistent at 12″ on-center for all panels. Walls sheathed with standard size panels were nailed at 6″ on-center to all panel edges, and oversized sheathed panels were nailed along the panel edges at 3″ and 4″ on-center. The authors justified this change in the panel edge nail spacing by stating, “These nail spacing patterns were chosen to keep the total number of nails in the various walls similar” (p. 12). This nail spacing is curious because they were comparing standard panels and oversized panels without the same panel edge spacing, especially in areas critical to the wall element. Because they chose to use the same number of nails in each wall, the panel edge nail spacing on the wall ends and around openings was closer with the oversized panels (which had fewer edges) than the standard-sized panels. This nail spacing will give the oversized panels a decisive advantage in load-bearing capacity. From the previous tests it was shown that the critical nailing was to the sill plates and the end-wall studs. The nailing pattern selected by He et al. (1999) allows only for a comparison of the types of failure between the two wall types, not a load capacity comparison.

The loading schemes used by He et al. (1999) were static and cyclic types. In all, eight panels were tested; four under static loading and four under cyclic loading. Each loading type was applied to one panel from each configuration. However, they proposed a new cyclic loading protocol after only one test to the FCC test protocol. The remaining three walls were tested to the new protocol. The new protocol was developed to eliminate nail fatigue and yielding induced during the preultimate portion of the test. A significant difference was found between the FCC cyclic protocol and He et al. proposed protocol. First the frequency of the proposed protocol is .01 to .002 Hz, and the FCC protocol is .25 Hz. Second the FCC protocol’s time duration is 280 seconds, with displacements exceeding the yield displacement for 80% of the test.

He et al. (1999) proposed that test protocol last five times as long as the FCC protocol and have only 7 cycles. The displacements for 95% of the test were only 80% of the ultimate load. They proposed a cyclic test that was more of a static test than a cyclic test. Their reasoning for changing the cyclical test protocol was that the FCC tests led to nail fatigue that they did not believe is a common failure mode found in post-earthquake inspections. I believe that He et al. (1999) should reconsider their position on this test protocol.

The test results and discussions presented by He et al. (1999) compared all of the test elements by graphing the load versus displacement for each element and showing the static and cyclic test on the same graph. The static loads provided a great envelope for the hysteresis curve developed from the cyclic tests. The different panel edge nail spacing hindered comparing the load-resisting capacities of the different elements. The oversized panel elements fared better when compared with the standard-sized panel elements of the same configuration. Another result of not providing the same panel edge nailing for all specimens was that the large panel element with openings was stiffer than the standard-size panel element without openings. I believe that this result would have been different if the same panel edge nail spacing was provided. The failure mechanisms did provide adequate information on how to reinforce the test elements so the components would act as one. As suspected, the failure of the elements with openings and sheathed with the over-sized panels occurred at the corners of the openings on a 45° angle into the sheathing. This failure is the same failure mechanism that occurred on the element with openings and sheathed with standard-sized panels. I speculate that with proper nailing, the element without openings and sheathed with standard-size panels could have achieved loads equal to the element with openings and sheathed with an oversized panel. The importance of reinforcing around openings is illustrated by the failure mechanism. Straps across headers and vertically up the trimmers would provide a ductile reinforcement for this failure area.

He et al. (1999) concluded that the openings in wood-based shear walls caused a significant decrease in shear strength and stiffness in the walls because of the reduced effective sheathing area. They also compared the oversized panel's performance to the standard-sizes panels fastened with the tighter panel edge nail spacing. Since the panel edge nail spacing was not the same, this comparison should not be made. The greatest error of He et al. was their conclusion regarding the selection of a suitable cyclic loading protocol. They state their new protocol better represents the amount of energy dissipation expected in an earthquake while avoiding unrealistic failure mechanisms. Their new protocol has 7 cycles and is closer to the ASTM E72 static test than a true cyclical loading test. This area of their research should be revisited and restated.

2.7 Findings from Cyclic Testing of Plywood Shear Walls

Shipp et al. (2001) discussed the findings of recently conducted full-scale cyclic tests of one-story, wood-framed plywood shear walls. They investigated the performance of eccentric hold-down devices in wood-framed shear walls. Wood-framed shear-wall design values in the UBC and IBC are based on static tests. They investigated the effects of cyclical testing and hold-down eccentricity to determine if neglecting this eccentricity during design is nonconservative. Shipp et al. believe the overturning forces for short walls (h/w ratio of greater than two) are underestimated when this hold-down eccentricity is ignored.

The walls tested were designed to accentuate any bending in the end posts (Shipp et al. 2001). To allow this to happen, the end posts were sized using the IBC and UBC allowable stress in tension, including the load duration (C_D) factor of 1.33. The hold-downs were also overstressed in the test, some by as much as 50%. Testing with undersized hold-downs would allow for the observation of how the mechanism would perform under extreme loading conditions. The test panels were either 4′ x 8′ or 8′x8′ panels, sheathed with 3/8″ or ½″ structural 1 plywood and with various nailing sizes and spacing. The tests were run in accordance with the testing protocol specified in SEAOSC.

The test setup involved a hydraulic actuator connected to a vertically hinged lever arm that reduce the forces of the actuator; this is a common setup with actuators that are too powerful for wood-framed wall capacities. The lever arm was then attached to a horizontal load beam that was connected to the top plate of the shear walls. Four 5/8″ ϕ anchor bolts were spaced along the base of the wall and hold-downs were installed on the inside of the end-wall posts.

Shipp et al. (2001) stated that the SLS of all but one of the walls was controlled by nail fatigue, sheathing edge tearing, and nail withdrawal at the lower corners of the wall, with nail fatigue being the most common. They also compared the force deformation curves of the 2:1 h/w ratio wall to the 1:1 h/w ratio wall. They noted that different eccentric hold-down devices had only a small affect on the SLS capacity of the wall.

The recommendations and conclusions developed by Shipp et al. (2001) were that the overall wall performance would be improved by using stiffer hold-downs, and the

SLS capacity of 2:1 ratio walls is 75% of the 1:1 ratio walls, not one-half as one might expect. However, since the 2:1 ratio walls are 25% more flexible during a seismic event, more load will be carried by the 1:1 ratio walls if both walls are in alignment with one another. Shipp et al. believes that force distribution during a seismic event should be modeled during the design process that is similar to the methods used in concrete and masonry shear-wall design.

Shipp et al. (2001) observed the performance of the end posts to be acceptable and design of the end posts should be based on the net area of the posts. Ignoring the eccentricity of the hold-downs as a design practice was also acceptable. They stated that ignoring the eccentricity was deemed acceptable, even though the limitation on the capacity of the wall was the nail failure in the lower corners of the wall. Shipp et al. also believed this fastener failure was partially due to the eccentricity of the hold-downs. To address this eccentricity, the authors proposed that a new hold-down method be developed that would eliminate this eccentricity, or that stiffer hold-downs be used. The new design requires stopping the sill plate short of the end-wall post and extending the post down into a u-shaped stirrup. The stirrup has a raised base with an anchor bolt directly beneath the post. This would eliminate any eccentricity in the shear wall.

Shipp et al. were remiss in stressing that by using their design methods and testing the 8′x8′ wall panels have a YLS load factor of only 1.5 to design loads and for 4′ x 8′ wall panels a load factor of 1.25 to design loads. The SLS load factors are 2 and 1.5 for 8′ wide and 4′ wide walls, respectively. The load factors for the 4′ x 8′ walls are below the acceptable limits of AC130 strength criteria. It would appear that for walls with 2:1

h/w ratios and narrower and using eccentric hold-downs, the published code allowable strength design capacities should be reduced.

Shipp et al. missed the opportunity to emphasize how their tests demonstrated the affects of the eccentric hold-down, and how this eccentricity caused stress concentrations in the lower corners of the walls. This stress concentration leads to premature wall failure at levels that do not provide significant overstrength factors to published code design capacities.

2.8 Hold-Down Connectors and Wood Member End Post Capacity

Nelson (2001) discussed the effects of eccentric hold-downs on the stresses in end-wall posts in wood-framed shear walls. According to Nelson, this type of hold-down has been under scrutiny since the Northridge earthquake. Nelson was concerned with the apparent lack of attention to the design of the end-post and the added stress to the end-posts from the eccentric hold-downs. A September 1999 hearing by the ICBO issued a committee action on the issues of eccentricity in hold-downs. The committee action would require the manufacturers to state that the design of the wood end-wall members must be checked. The hold-down capacity listed in the design catalogues covers only the metal device. Several hold-down manufacturers were present and agreed to this action.

Nelson (2001) also discussed the previously reviewed article by Shipp, et al., and referenced an EQE report prepared for USP and available on the Internet. Nelson stated concerns about the article and the report's acceptance of the wall failures found in the testing. The testing clearly demonstrates the failures caused by the eccentric hold-downs, but accepts the final test values since they are higher than the code-design numbers.

Nelson (2001) used static analysis of the shear walls, investigations of field failures, and designs where the engineers ignored the wood capacity in designing the hold-down to demonstrate why the hold-down eccentricities cannot be ignored. From Nelson's static analysis of shear walls and from the results of previous tests, Nelson explained the mechanics of a shear wall resisting lateral loads with eccentric hold-downs. Due to the shear deformation of the sheathing, the sheathing fasteners attached to the end-wall studs put the stud in double curvature. Tension occurs at the base and inside of the end-wall studs, as well as at the top and outside of the end-wall studs on the side of the wall that the load is applied to. This load from the sheathing fasteners comes from the sheathing deforming differently than the wood frame. The sheathing is in pure shear and the wood frame is in tension and compression. The difference in deflection within the element causes additional tension in the end-wall studs that is not accounted for in the uplift calculations. The additional tension could overload the end-wall studs at their reduced section through the hold-down fasteners, causing failure in this component. Nelson's analysis demonstrates the complexity of the wood-shear wall element. This complexity is overlooked in the design practice because design calculations analyze the shear-wall element as one component and ignore the interaction of the many components within the element.

Nelson's (2001) conclusion that the end-wall posts or multistuds are not being analyzed thoroughly during the design process is applauded. With the new code requirements for 3x-boundary members at sills and panel edges of high-load shear walls, the weak link in a shear-wall element can now move to the eccentric type hold-down and its connection to the end-wall studs and sill plates. Nelson's research should be continued with testing that demonstrates how different h/w ratios affect the additional fastener load to the end-wall components.