CYCLICAL TESTING OF ORIENTED STRAND BOARD SHEATHED AND STAPLED WOOD SHEAR WALLS IN ACCORDANCE WITH INTERNATIONAL CONFERENCE OF BUILDING OFFICIALS ACCEPTANCE CRITERIA 130

by

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A thesis submitted to the faculty of
The University of Utah
in partial fulfillment of the requirements for the degree of

Master of Science

in

Civil Engineering

Department of Civil and Environmental Engineering

The University of Utah

August 2003

ABSTRACT

Inelastic response of wood-framed structural walls when subjected to code design level seismic forces necessitated a study of the current design and analysis methods of wood shear walls. Wood shear walls are constructed from an assembly of components such as sheathing, fasteners, studs, and light gauge metal hold-down devices. Each component affects the response of the shear wall element. Most elements constructed in the field have never been tested as a complete assembly, and most components have only been tested with monotonic (static) tests.

The goals of the research were to: (1) test complete assemblies of wood shear walls with fully reversed cyclical test protocol as specified by the acceptance criteria of current building codes (1997 Uniform Building Code and 2000 International Building Code); (2) better understand the relationship of each components to the wall's performance; and (3) examine simple modifications that will help improve the wall's performance.

TABLE OF CONTENTS

Page

ABSTRACT iv

LIST OF TABLES vii

LIST OF FIGURES viii

LIST OF ACCRONYMS.. x

1. INTRODUCTION. 1

1.1 Seismic Design History and Performance of Wood Shear Walls 1
1.2 Performance-Based Design 4
1.3 Project Goals 7

2. LITERATURE REVIEW 9

2.1 Design Criteria for Wood Shear Walls 9
2.1.1 Yield Limit Equations for Nails 10
2.1.2 Staple Capacities 12
2.1.3 Test Protocol for Code Allowable Design Loads 12
2.1.4 New Analysis Trends 15
2.1.4.1 AC130 16
2.1.4.2 PFC-5485 ICBO Evaluation Report 19
2.1.4.3 Cyclical Test Procedures 21
2.2 ATC Report R-1, Cyclic Testing of Narrow Plywood Shear Walls 28
2.3 Comparison of Static and Dynamic Response of Timber Shear Walls 32
2.4 Comparison of Static and Dynamic Response of Timber Shear Walls, Discussion 39
2.5 Preliminary Testing of Wood Structural Panel Shear Walls Under Cyclic Loading 40
2.6 Cyclic Performance of Perforated Wood Shear Walls with Oversize OSB Panels 46
2.7 Findings from Cyclic Testing of Plywood Shear Walls 50

2.8 Hold-Down Connectors and Wood Member End Post Capacity 53

3. TESTING PROGRAM 56
3.1 Introduction 56
3.2 Test Procedures and Goals 58
3.3 Results to Acquire 63

4. SPECIMEN SPECIFICATIONS 66

4.1 Materials 66
4.2 Assemblies 68
4.3 Code Published Wall Capacities 72
4.4 Free-Body Diagram of Hold-Down Configuration 75

5. TEST RESULTS 79

5.1 Introduction 79
5.2 Individual Panel Test Results 81
5.3 AC130 Allowable Design Loads 106

6. CONCLUSION 113

7. RECOMMENDATIONS 116

Appendices

A. STRUCTURAL ENGINEERS ASSOCIATION OF SOUTHERN CALIFORNIA TEST PROTOCOL 119

B. PANEL SPECIFICATION DRAWINGS 125

C. TEST SETUP PHOTOS 131

D. TEST DATA AND HYSTERESIS LOOPS 135

E. CALCULATIONS PER ACCEPTANCE CRITERIA 130 154

F. FEMA 273/356 *m* VALUE CALCULATIONS 157

REFERENCES 163

LIST OF TABLES

Table Page

2.3 Ductility Comparison 36

3.1 Test Specimen Specifications 57

3.2 Structural Engineers Association of Southern California
Test Protocol 59

4.1 Shear Wall Deflection Calculations 76

5.1 Tabulated Test Results 107

5.2 Calculations of Acceptance Criteria 130 Design Loads 109

LIST OF FIGURES

Figure Page

2.1 Comparison of SEAOSC and FCC loading protocol 26

3.1 Test setup 61

3.2 Data acquisition devices 62

3.3 Hysteresis loop with bilinear segments 65

4.1 End-wall strap before installation 69

4.2 End-wall strap installed 69

4.3 Concrete foundation section 70

4.4 Shear wall deflection versus h/w ratio 75

4.5 Free-body diagram of eccentric hold-down 77

5.1 Panel 1 fasteners and sheathing failure at the sill plate 82

5.2 Panel 2 sill plate splitting along centerline 85

5.3 Panel 3 sheathing tearing and end wall stud crack 87

5.4 Hysteresis load-displacement curves panel 3 88

5.5 Hysteresis load displacement curve panel 4 90

5.6 Failure components panel 4 91

5.7 Panels 7, 8, and 9 elevation drawing 93

5.8 Base configuration of panel 7 94

5.9 End-wall stud strap panel 7 96

Figure Page

5.10 End-wall stud, hold-down, and sill failure panel 7. 97

5.11 End-wall strap crushing the sill plates, panel 7 98

5.12 Hysteresis load displacement curve panel 7 99

5.13 DT and actuator arm displacement versus time 103

5.14 Ultimate load and wall uplift per panel 105

5.15 AC130 force deformation bilinear segments 112

LIST OF ACRONYMS

ACRONYM	NAME
AC130	Acceptance criteria for premanufactured wood-shear walls
AITC	American Institute of Timber Construction
APA	American Plywood Association
ASTM	American Society of Testing Materials
ATC	Applied Technology Council
DCR	Demand capacity ratio
DT	displacement transducer
FCC	Fornitek Canada Corporation
FEMA	Federal Emergency Management Association
FME	First major event
G’	Shear modulus
h/w	Height–to-width
IBC	International Building Code
ICBO	International Conference of Building Officials
LRFD	Load Resistance and Factored Design
LVDT	Linear variable displacement
M	Element demand modifier
MCE	Maximum considered earthquake
NDS	National design specification
NEHRP	National Earthquake Hazard Reduction Program
NER	National Evaluation Report
OSB	Oriental Strand Board
Plf	Pounds per linear feet
Q_{CE}	Expected strength
Q_{UD}	Earthquake demand
R	Response modification factor
SEAOSC	Structural Engineers Association of Southern California
SLS	Strength limit state
SPD	Sequential phase displacement
SPF	Spruce pine fir
UBC	Uniform Building Code
V_u	Ultimate shear strength
YLS	Yield limit state
Δ_m	Mean displacement at strength limit state
Δ_S	Strength level design displacements
Δ_{SC}	Strength level design displacements

1. INTRODUCTION

1.1 Seismic Design History and Performance of Wood Shear Walls

Seismic events in the last two decades have allowed engineers and scientists to measure seismic forces on existing structures and to inspect the damage incurred by seismic-resisting elements. Studies of these events have revealed that the code forces for seismic design are drastically underestimated (Chopra 1995). This underestimation is due to the processes (used in current new building-design codes) of calculating seismic forces and distributing those forces to lateral load-resisting elements. Specifically, seismic forces determined in accordance with the design codes are divided by a response modification factor (R) that is representative of the building lateral load system's overstrength factor. These factors have been set based on experience and judgment of those who wrote the building codes (Federal Emergency Management Association (FEMA) 274 1997). One- and two-story, wood-framed, bearing wall structures that have higher natural frequencies than taller buildings will experience some of the highest seismic forces during an event. These buildings are designed using a code-specified R-value of 5.5. The R-value that reduces the force distributed to shear walls during the design process was not verified with testing and has not been equaled during testing of wood shear-wall panels by agencies charged with determining code-allowable load-design values.

Another design procedure that has historically been used in the building codes, is the linear design procedure. Linear procedures are easy to apply but are only applicable when the structure has sufficient strength to remain nearly elastic when subjected to the design earthquake demand and when the building has regular geometries and distributions of mass and stiffness (FEMA 274 1997). When a building's lateral load-resisting elements are stressed past their elastic limit, their stiffness degrades. Inelastic deflections calculated from linear procedures are inaccurate. The stiffness degradation is not accounted for when maximum inelastic response displacements (Δ_m) are calculated in the 1997 Uniform Building Code (UBC). This displacement is determined by multiplying strength-level design displacements (Δ_s) by R and .7. The assumption that $\Delta_m = 0.7 \times R \times \Delta_s$ is based on Newmark's postulations from 30 years ago and research summarized by Miranda and Bertero (Recommended Lateral Force Requirements and Commentary, Structural Engineers Association of Southern California (SEAOSC) (Blue Book 1996).

After the 1994 Northridge earthquake, inspections revealed that certain wood-framed shear walls did not perform as expected by the engineering community. Specifically, tall, narrow shear walls, with height-to-width (h/w) ratios between 2:1 and 3½:1, had higher lateral deflections and uplift deflections than engineers anticipated. This high lateral deflection is due to shear-wall slenderness and hold-down anchor performance. SEAOSC made the following recommendations: (1) limit shear-wall h/w ratios; (2) provide structural lumber members with a minimum thickness of 2 ½″ (3x) boundary member at all boundary conditions of "heavily loaded" walls; (3) reduce the current code published allowable design loads for wood-framed shear walls by 25% (Rose, 1998), until cyclical load testing can be performed to verify these design loads;

and (4) perform testing of hold-downs under high displacement cyclical loading. These recommendations are directed at improving the performance of wood-shear walls by decreasing the effects of an *R*-value that may be nonconservative and help keep seismic response elastic by reducing the h/w ratios. In addition, the 3x-boundary member requirement will help the performance of wood-shear walls in the inelastic range by eliminating some of the failures in boundary members.

In 1997, the UBC required wood-framed shear walls in seismic zone 4 to have a maximum h/w ratio of 2:1 that was reduced from 3½:1 in the previous code editions. This requirement has been maintained in subsequent documents (FEMA 302 and International Building Code (IBC) 2000 new building-design provisions) for site classifications in seismic zone 4 and also in seismic zone 3 locations. In addition, shear walls with loads over 500 pounds per linear foot (plf) were required to have 3x-boundary members at all panel edges. These changes still left other design issues concerning wood-shear walls unresolved, specifically the recommendation for a reduction of current allowable design loads and hold-down anchor performance.

Investigating the allowable design loads published in UBC, IBC, and the current shear-wall design practices revealed the following: (1) capacities for shear walls are based on yield-limit equations and verified with static tests (Tissell 1996); (2) values for stapled sheathing are based on proportional limit equations, with very few having been verified with laboratory testing (National Evaluation Report 272 (NER) (1997); (3) hold-down capacities are published in manufacturers' literature; and (4) no compensation is made to account for the eccentricities typically found in shear walls.

1.2 Performance-Based Design

Performance-based design was developed to offer a methodology of upgrading the seismic performance of existing structures in a transparent manner and to provide building codes where the user has a clear understanding of the level of performance that the building is being designed to meet. In order to meet these criteria FEMA 273/356 were developed. FEMA 273 is the 1997 *Guidelines for Seismic Rehabilitation of Buildings*, and FEMA 356 is the *2000 Prestandard for Seismic Rehabilitation of Buildings.* The *Guidelines and Prestandard* parallel the new building-design model codes FEMA 302 and IBC (2000)) seismic design methodologies for determining seismic design forces. The distribution of these forces to the lateral load-resisting elements differs from the rehabilitation guidelines and the new building-design model codes. New- building codes reduce the design earthquake, which is two-thirds multiplied by the maximum considered earthquake (MCE), by the *R*-value whereas the *Guidelines and Prestandard* require the expected strength (Q_{CE}) of the lateral force system multiplied by the element demand modifier (*m*) to exceed the design earthquake force (also 2/3 times MCE).

There is a correlation between *R* and *m* used in design procedures. Both are used to reduce the required elastic strength of the lateral load-resisting system. However, where *R* is based on experience and judgment, *m* is based on experimental results; its values are 40% to 50% lower than *R*-values for the equivalent performance objective and the same structure type.

Another major difference between the new building-design codes and the performance-based rehabilitation codes is the use of linear and nonlinear design procedures. FEMA 273/356 requires that, when linear procedures are used, the demand capacity ratio (DCR) be checked for each lateral-load resisting element. If any of the ratios are found to be greater than two, then no in-plane or out-of-plane irregularities are allowed. The DCR is calculated by dividing the earthquake demand (Q_{UD}) by Q_{CE}. There is no reduction for *m* in this calculation; thus, this is equivalent to using an *m* value of 2. Keeping the DCR at a maximum of 2 assures two conditions: (1) the building will remain essentially in the elastic range when subjected to design earthquake forces; and (2) the design procedures are accurate. New building-design provisions have no provisions to mathematically verify ductility of the lateral load-resisting system. There is an inelastic deflection limitation, with an amplification of the calculated elastic deformation, but this provides no assurance of ductility.

The design capacities published in FEMA 273/356 differ from the new building-design provisions. Allowable load-design capacities used in current codes are based on published tabulated data. Design capacities published in FEMA 273/356 are based on Q_{CE} and *m*. Determination of Q_{CE} is based on the element's response to cyclical loads. If the element is a deformation-controlled element, Q_{CE} is based on the mean level of resistance at the deformation level anticipated. If the element is a force-controlled element, the Q_{CE} is the mean yield strength minus one standard deviation. Q_{CE} of wood-shear walls in FEMA 273/356 may be derived from one of three methods. For strength-controlled acceptance criteria: (1) use a yield load of 0.8 multiplied by the ultimate shear strength (V_u), determining V_u from Table 8-3 in FEMA 273; or (2) determine Vu from Equation 8-4 in FEMA 273. Use a factor of 0.8 multiplied by the ultimate load of the element's static test results to determine cyclical ultimate load values that are acceptable for walls with h/w ratios of 1 or greater. Neither of the methods is based on direct conversion from the tabulated loads found in the new building-design provisions.

No design values for staples are published in FEMA 273/356. The lack of unpublished design values for staples leaves the default property method Section 8.3.2.5 in FEMA 273/356 as the only alternative to testing in order to determine Q_{CE} values for stapled shear walls. This method differs from the equations described above. The method provides for conversion of allowable strength fastener values (presumably obtained from values in NER 272 or other International Conference of Building Official (ICBO) approved published values) to yield values. In FEMA 273/356, this method is described differently. In FEMA 273, this method is determined by multiplying the allowable stress value by 2.16 times 1.6 and times 0.8 (or as the Code states, multiply the allowable stress value by 2.8) to convert the allowable strength value to a yield-limit value or expected-strength value. The determination of these coefficients is explored later. In FEMA 356, this "format conversion" is described as multiplying the allowable load values and all adjustment values for the fastener except load duration by $2.16/\phi$, where ϕ is the specified load resistance and factored design (LRFD) resistance factor, which is 0.65 for connections. The 356 methodology produces a slightly lower multiplier of 2.65 compared to the multiplier of 2.8 in FEMA 273, that implies that there is still debate over the expected strength values to be used for wood-shear walls. This uncertainty is especially true for shear walls fastened with staples, since there is even less testing and design values for these elements.

1.3 Project Goals

Recognizing the need to verify code capacities of stapled, Oriental Strand Board (OSB) sheathed shear walls and the effects of eccentric hold-downs, the research project was undertaken. The research project was designed to help understand the behavior of specific shear-wall components. This understanding was achieved by testing single shear panel elements with stapled sheathing and eccentric hold-downs and then modifying the components to address highly stressed areas. Nine panels were tested. After the first three tests provided a baseline of a standard panel, modifications were made to the remaining panels to strengthen highly stressed areas and increase performance.

The first goal of the research was to develop force deformation curves (hysteresis loops) for the shear-wall elements. These curves allow for the analysis of allowable design loads, elastic and inelastic behavior, and for the effects of framing modifications made to improve wall performance. The hysteresis loops were developed from data recorded by the testing equipment during the loading of the shear walls. The hysteresis loops show negative and positive deformation and demonstrate the symmetry of the cyclic response. From the hysteresis loops, design loads were also calculated. The calculated loads allow a comparison of current published loads to the experimentally derived loads. Bilinear curves enveloping the hysteresis loops were developed to determine elastic and inelastic force deformation relationships. These curves will also allow for an examination of current modifier factors *R* and *m*. Finally, after an initial baseline group of panels was tested, sequential modifications were made to the test panels. These modifications were made to determine if simple framing modifications could improve shear-wall performance in the full range of the response spectrum.

The test protocol used followed AC130, a 1997 UBC-adopted acceptance criteria for wood-shear panels. AC130 specifies the use of the sequential phase displacement loading protocol developed by SEAOSC, the data acquisition requirements, and provides a method for determining allowable design loads for each specimen group tested.

REFERENCES

American Institute of Timber Construction. (1994). *Timber construction manual*, 4th Ed., Englewood, CO.

American Society for Testing Materials. (1995). "Standard practice for static load test for shear resistance of framed walls for buildings." American Society for Testing Materials E, 564-95.

Applied Technology Council. (1995). "Cyclic testing of narrow plywood shear walls." ATC-R-1. Redwood City, CA

Breyer, D. (1993). *Design of wood structures*, 3rd Ed., New York.

Chopra, A. (1995). *Dynamics of structures*. 1st Ed., Upper Saddle River, NJ.

Dinehart, D. and Shenton, H. (1998, June). "Comparison of static and dynamic response of timber shear walls." *J. of Struct. Engrg.*, ASCE 124(6), 686-.695.

EQE International, Inc. (1999, December). "Results of cyclic (reversed) load testing for shear resistance of wood framed plywood shear walls with USP lumber connector hold-downs." *Structural Engineers Association of Southern California*, 1-22; 30-39.

Federal Emergency Management Agency (1998). NEHRP "Recommended provisions for seismic regulations for new buildings and other structures and commentary." *FEMA 302,303*. Washington D.C.

Federal Emergency Management Agency. (1997). NEHRP "Guidelines for the seismic rehabilitation of buildings, and commentary." *FEMA 273,274*. Washington D.C.

He, M, Magnusson, H, Lam, F, and Prion, H. (1999, January). "Cyclic performance of perforated wood shear walls with oversized OSB panels." *J of Struct. Engrg.*, ASCE 125(1), 10-18.

ICBO Evaluation Service, Inc. (2001, July). "Acceptance criteria for prefabricated wood shear walls, AC130." Whittier, CA.

ICBO Evaluation Service, Inc. (1999, February). "Strong-wall shear panels. evaluation report PFC-5485." Whittier, CA.

International Code Council (ICC). (2000). *International Building Code,* United States of America.

International Conference of Building Officials, (ICBO). (1997). *Uniform Building Code*, Whittier, CA.

Karabeyli, E., Dolan, J., Ceccotti, A., and Ni, C. (1999, July). "Discussion of comparison of static and dynamic response of timber shear walls." *J. of Struct. Engrg*., ASCE 125 (7), 796-797.

National Evaluation Service for the International Staple Nail and Tool Association. (1997). "Power driven staples and nails for use in all types of building construction." *National Evaluation Report 272*. Whittier, CA.

"Commentary on the national design specifications for wood construction." *NDS Commentary* (1993). American Forest and Paper Association, Washington, D.C. 92-93.

Nelson, R. (2001). "Hold-down connectors and wood member end-post capacity." *Structure*, September 2001 36-41.

Rose, J. (1994). "Northridge, California earthquake structural performance of buildings in San Fernando Valley, California (January 17,1994)." *American Plywood Association Report No. T94-5*. Tacoma, WA.

Rose, J. (1998). "Preliminary testing of wood structural panel shear walls under cyclic (reversed) loading." *American Plywood Association Report No. 158*. Tacoma, WA.

Shipp, J., Erickson, T. and Rhodebeck, M. (2001). "Findings from cyclic testing of plywood shear walls." *Structure*, March 2001 27-31.

Structural Engineers Association of California. (1996). "Recommended lateral force requirements and commentary." *Blue Book* 6th Ed. Sacramento, CA

Structural Engineers Association of California. (1999). "Recommended lateral force requirements and commentary." *Blue Book* 7th Ed. Sacramento, CA

Structural Engineers Association of California. (2000). "Seismic design manual." Vol. 1 and 2. Sacramento, CA

Structural Engineers Association of Southern California. (1996). "Standard method of cyclic reversed load test for shear resistance of framed walls for buildings." Structural Engineers Association of Southern California, Whittier, CA.

Tissell, J., and Elliott J. (1997). "Plywood diaphragms." *American Plywood Association Research Report No. 138.* Tacoma, WA

Tissell, J. (1996) "Wood structural panel shear walls." *American Plywood Association Research Report No. 154.* Tacoma, WA