

A STUDY OF THE SEISMIC RESPONSE MODIFICATION FACTOR FOR LOG SHEAR  
WALLS

by

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## Abstract

Log construction is becoming increasingly popular throughout the U.S. Currently, seismic coefficients are not provided in model building codes for the design of the log shear walls as a lateral force resisting system for seismic forces. Current design practice is to use a response modification coefficient,  $R$ , of around 4.5. Several tests by other researchers on log shear walls showed strong energy dissipation and good lateral strength with stability after high displacements. This behavior of the log shear wall system is evidence that a higher  $R$  could possibly be used in design. The purpose of this study was to establish a response modification factor for single story log shear walls based on available shear wall tests using the definition of  $R$  provided in ATC-19. This research did not conduct testing according to the protocol and methodology of ATC-63.

This work contains a history of the development of seismic design provisions in the U.S. and the evolution of the response modification coefficient. Common log construction practices are reviewed, with reference to *ICC 400- Standard on Design and Construction of Log Structures*. Using data provided by other researchers from physical testing and computer modeling of various types of log shear walls, an  $R$  of 6.0 is proposed based on the provisions of ATC-19. Finally, recommendations for further research to fully understand the behavior of the log shear wall system, including possible archetypes required by the methodology set forth in ATC-63, are provided.

# Table of Contents

List of Figures .....	vi
List of Tables .....	ix
Acknowledgements .....	x
1 Introduction.....	1
2 The Evolution of Seismic Codes and the Response Modification Factor, $R$ .....	3
2.1 Seismic Design Prior to the Blue Book (1927-1959) .....	4
2.1.1 UBC, International Conference of Building Officials .....	5
2.1.2 The National Building Code of the Building Officials and Code Administrators International .....	10
2.1.3 Standards from ANSI and ASCE.....	11
2.2 The Dynamic Approach (1959 -1972).....	12
2.2.1 Changes to Base Shear.....	12
2.2.2 New Ideas.....	14
2.3 Changes to the Methodology, ATC-3 06 and NEHRP (1973-1986).....	15
2.3.1 ATC-3 06 Methodology.....	16
2.3.1.1 The Equivalent Lateral Force Procedure (ELFP) .....	17
2.3.1.1.1 Design Earthquake Ground Motion .....	19
2.3.1.1.2 Ground Motion Parameters .....	21
2.3.1.1.3 Design Elastic Response Spectra .....	21
2.3.1.1.4 Fundamental Period of Vibration .....	22
2.3.1.2 The Response Modification Factor, $R$ .....	23
2.3.2 National Earthquake Hazards Reduction Program, 1979 .....	25
2.4 Stress Design versus Strength Design (1987-1997).....	26
2.4.1 1988 Blue Book .....	27
2.4.2 Quantifying the Structural Response Modification Factors, ATC-19 .....	28
2.4.2.1 Pushover Analysis.....	29
2.4.2.1.1 Period-dependant Strength Factor, $R_s$ .....	30
2.4.2.1.2 Period-dependant Ductility Factor, $R_u$ .....	31

2.4.2.1.3	Redundancy Factor, $R_R$ .....	32
2.4.2.1.4	Damping Factor, $R_\xi$ .....	32
2.4.3	Adjustments to Seismic Provisions in Model Building Codes .....	33
2.5	Current Provisions, IBC and ASCE 7 (1998-2009) .....	34
2.5.1	2000 IBC .....	34
2.5.2	2003-2009 IBC .....	36
2.6	FEMA 350/ATC-63 (2009- present) .....	36
2.6.1	Adjusted Collapse Margin Ratio, ACMR .....	40
3	Log Shear Wall Construction Practices .....	43
3.1	Structural Log Characteristics .....	44
3.1.1	Log Species and Grade .....	44
3.1.2	Log Profile .....	46
3.1.3	Moisture Content/Settlement .....	47
3.2	Shear Wall Reinforcing .....	48
3.2.1	Log-Log Connectors .....	49
3.2.1.1	Bolt .....	49
3.2.1.2	Lag Screw .....	49
3.2.1.3	Drift Pin .....	50
3.2.2	Hold-down Connectors .....	50
3.3	Common Foundation Anchorage Methods .....	51
4	Behavior of Log Shear Walls Subject to Lateral Seismic Loads .....	53
4.1	Expected Behavior of Log Shear Walls .....	53
4.2	Shear Wall Behavior Demonstrated in Physical Testing .....	54
4.2.1	Gorman and Shrestha, Washington State University, 2002 .....	54
4.2.2	Popovski, Forintek Canada Corporation, 2002 .....	58
4.2.3	Snyder, Montana State University, 2003 .....	63
4.2.4	Scott, Oregon State University, 2004 .....	65
4.2.5	Yeh, 2006 .....	69
4.2.6	Graham, Washington State University, 2007 .....	71
4.2.6.1	Connection Tests .....	72
4.2.6.2	Full-scale Log Shear Wall Tests .....	74

5	Quantifying the R-Factor for Log Systems .....	78
5.1	$R$ Currently Used for Log Shear Walls.....	78
5.2	Recommended $R$ for Log Shear Walls.....	80
5.2.1	Period-Dependant Strength Factor, $R_s$ .....	84
5.2.2	Period-Dependant Ductility Factor, $R_\mu$ .....	85
5.2.3	Redundancy Factor, $R_R$ .....	86
5.2.4	Review of Calculated $R$ .....	86
6	Recommendations for Future Research.....	91
7	Conclusions.....	92
8	Bibliography .....	94
	Appendix A. Development of the $R \sim K$ and $R_w \sim K$ Relationships .....	97
	A.1 $R \sim K$ Relationship (ATC 3-06, 1978).....	97
	A.2 $R_w \sim K$ Relationship (SEAOC Blue Book, 1988).....	98
	Appendix B. Pushover Results and Bi-linear Approximations of the Test Walls .....	99
	Appendix C. Image/Figure Permissions .....	106
	C.1 Permission for republication of figures from UBC.....	106
	C.2 Permission for republication of figures from ATC publications .....	107
	C.3 Shear tests on Log Shear Walls- Shrestha.....	108
	C.4 "Testing of Lateral Resistance of Handcrafted Log Walls. . . " .....	110
	C.5 Log Bearing Walls Research- by Randy Scott.....	111
	C.6 Images permission- Thesis by Drew Graham .....	112

## List of Figures

Figure 2-1 Seismic probability map of the U.S. Republished courtesy of the ICC; originally published in 1949 UBC.....	8
Figure 2-2 Map of the U.S. showing zones of approximately equal seismic probability. Republished courtesy of the ICC; originally published in the 1958 UBC.....	9
Figure 2-3 1976 USGS seismic accelerations map- estimated maximum acceleration contours on rock, developed by Algermissen and Perkins. Republished courtesy of the ATC; originally published in ATC-3 06.....	16
Figure 2-4 Contour map for effective peak acceleration, $A_a$ . Republished courtesy of the ATC; originally published in ATC-3 06. ....	20
Figure 2-5 Contour map for effective peak velocity, $A_v$ . Republished courtesy of the ATC; originally published in ATC-3 06. ....	20
Figure 2-6 Use of $R$ factors to reduce elastic spectral demands to the design force level. Republished courtesy of the ATC; originally published in ATC-19. ....	24
Figure 2-7 Sample pushover analysis results showing bi-linear approximation using equal energy method.....	29
Figure 2-8 Sample pushover analysis results showing bi-linear approximation using equal energy method and consideration of allowable drift .....	30
Figure 2-9 Illustration of seismic performance factors ( $R$ , $C_d$ and $\Omega_o$ ) as defined by the Commentary to the <i>NEHRP Recommended Provisions</i> . Republished courtesy of FEMA; originally published in FEMA P695. ....	38
Figure 2-10 Illustration of seismic performance factors ( $R$ , $C_d$ and $\Omega$ ) as defined by the ATC-63 methodology. Republished courtesy of FEMA; originally published in FEMA P695.....	39
Figure 3-1 Profiles of timber beam, as defined in ICC 400-2007 .....	45
Figure 3-2 Common log profiles.....	47
Figure 3-3 Through-rod detail showing typical placement at ends of wall and around opening	51
Figure 3-4 Common foundation anchorage details.....	52
Figure 4-1 Typical response of wall type 1, two hold-down anchors (Shrestha & Gorman, 2002), courtesy of Washington State University Wood Materials and Engineering Laboratory....	56

Figure 4-2 Typical response of wall type 2, three hold-down anchors (Shrestha & Gorman, 2002), courtesy of Washington State University Wood Materials and Engineering Laboratory .....	57
Figure 4-3 Summary of maximum load at prescribed racking deflections (Shrestha & Gorman, 2002), courtesy of Washington State University Wood Materials and Engineering Laboratory .....	58
Figure 4-4 Load-Displacement relationship at the top level of the wall with hardwood pins (Popovski, 2002), courtesy of Forintek Corporation .....	61
Figure 4-5 Load-Displacement relationship at the top level of the wall with corners (Popovski, 2002), courtesy of Forintek Corporation .....	62
Figure 4-6 Profile of logs used for testing by Snyder, 2003 .....	64
Figure 4-7 Typical force-displacement curve of a log shear wall with through-rods in oversized holes demonstrated by Scott's research in 2004 .....	67
Figure 4-8 Force-Displacement relationship for computer models generated by Scott (Scott et al, 2005), republished courtesy of Oregon State University .....	68
Figure 4-9 Log connection configuration for connection tests completed by Graham in 2007 ..	72
Figure 4-10 Load-displacement relationship with backbone curve from cyclic testing superimposed for wall with 1:1 aspect ratio (Graham, 2007), republished courtesy of Washington State University Civil and Environmental Engineering .....	76
Figure 4-11 Load-displacement relationship with backbone curve from cyclic testing superimposed for wall with aspect ratio 2:1 (Graham, 2007), republished courtesy of Washington State University Civil and Environmental Engineering .....	76
Figure 4-12 Load-displacement relationship with backbone curve from cyclic testing superimposed for wall with aspect ratio 4:1 (Graham, 2007), republished courtesy of Washington State University Civil and Environmental Engineering .....	77
Figure B-1 Wall 1- Pushover results with bi-linear approximation, (Popovski, 2002) .....	99
Figure B-2 Wall 2- Pushover results with bi-linear approximation, (Popovski, 2002) .....	100
Figure B-3 Wall 3- Pushover results with bi-linear approximation, (Graham, 2007) .....	100
Figure B-4 Wall 4- Pushover results with bi-linear approximation, (Graham, 2007) .....	101
Figure B-5 Wall 5- Pushover results with bi-linear approximation, (Graham, 2007) .....	101
Figure B-6 Wall 6- Pushover results with bi-linear approximation, (Yeh et al, 2006).....	102

Figure B-7 Wall 7- Pushover results with bi-linear approximation, (Yeh et al, 2006).....	102
Figure B-8 Wall 8- Pushover results with bi-linear approximation, (Scott et al, 2005).....	103
Figure B-9 Wall 9- Pushover results with bi-linear approximation, (Scott et al, 2005).....	103
Figure B-10 Wall 10- Pushover results with bi-linear approximation, (Scott et al, 2005).....	104
Figure B-11 Wall 11- Pushover results with bi-linear approximation, (Scott et al, 2005).....	104
Figure B-12 Wall 12- Pushover results with bi-linear approximation, (Scott et al, 2005).....	105

## List of Tables

Table 2–1 Horizontal force factors, $C$ , from Table 23-C of 1935 UBC and 1949 UBC .....	7
Table 2–2 Horizontal force factors, $C^a$ , from Table 23-C of 1958 UBC .....	9
Table 2–3 Minimum design lateral seismic force, 1950 BOCA.....	10
Table 2–4 $K$ -values from the 1959 SEAOC Blue Book .....	13
Table 2–5 Draft redundancy factors, ATC-19, 1995 .....	32
Table 2–6 Comparison of $R$ through code updates (1978-2005).....	42
Table 4–1 Pushover tests and results of tests by Popovski in 2002.....	59
Table 4–2 Cyclic tests and results of tests by Popovski in 2002 .....	60
Table 4–3 Computer models tested by Scott in 2004 .....	66
Table 4–4 Tests completed by Yeh, 2006.....	70
Table 4–5 Testing Results from full-scale wall tests by Graham in 2007 .....	75
Table 5–1 Description and parameters of walls included in calculation of recommended $R$ .....	82
Table 5–2 Calculation of $\mu$ , $R_\mu$ and $R_S$ for each wall .....	83
Table 5–3 Calculation of $R$ using wall data from Table 5–2 .....	87
Table 5–4 Calculation of $R$ using adjusted effective seismic weight for Wall 3 through Wall 7 88	

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# 1 Introduction

Log construction, which started as a simple structure, has evolved into an engineered building with many openings, multiple levels and ornate architectural features. From the simple structures with hand-hewn logs and wood pins to modern log mansions, stacked log structures gaining popularity in many parts of the U.S. and spreading throughout the world. A stacked log structure is a structure where the walls are constructed of individual logs laid horizontally in a vertical line. Today, more than 400,000 log homes in the U.S. and Canada exist (Popovski, 2002). As stacked log structures gain popularity around the world, new technologies and more complicated structures bring new design questions. We do not know what log-to-log attachment method is best suited for lateral loading. Additionally, tests of individual log shear walls indicate strong energy dissipation but the ASCE 7 seismic provisions do not provide requirements and seismic factors for log shear walls as a seismic lateral force resisting system (LFRS).

This research has a limited scope with two goals. The main goal of the research conducted for this study is determining a reasonable value for the seismic response modification factor,  $R$ , for a single story log shear wall when designed using the Equivalent Lateral Force Procedure (ELFP) of the ASCE 7. The log shear walls in this research have log-to-log connectors between each course and/or through the whole height of the wall. This research uses the 1995 definition provided by the Applied Technology Council (ATC) in ATC-19 to develop  $R$  for log shear walls. A secondary goal of this research is to recommend additional research and testing to determine an  $R$  factor for log shear walls using the methodology prescribed in “Quantification of Building Seismic Performance Factors”, ATC-63/FEMA P695 (ATC, 2009). The  $R$  recommendation and suggestions for further research will take into account all construction and detailing requirements in the *Standard on the Design and Construction of Log Structures, ICC 400-2007*, from the International Code Council (ICC).

Current log shear wall design is based on success of past designs and designer’s judgment of behavior based on a small amount of physical testing. Much speculation has occurred on the behavior of log shear walls. Several sources indicate a standard value for the  $R$  factor of 4.0, but we do not know if this value accurately represents the behavior of the structural system. To determine  $R$  for a log shear wall first we must understand the history of the seismic codes in the

U.S. and thus the  $R$  factor and desired behavior of a LFRS when subjected to seismic lateral loads. Moreover, understanding current construction for log structures is needed to predict possible behaviors and failures.

No comprehensive sources exist on the design and construction of log shear walls and the model building codes do not have provisions specifically for log shear walls. While this study does not claim to present all of the available data on log shear walls, the information included is a comprehensive compilation of readily available sources. This thesis includes a history of seismic codes and the response modification factor, a brief explanation of the different construction methods of log shear walls, recent physical research on the behavior of log shear walls, and a synopsis of current design practices of log shear walls. It proposes a response modification factor for a log shear wall using a defined construction method and recommends further research on specific construction types.

## 2 The Evolution of Seismic Codes and the Response Modification Factor, $R$

The Great San Francisco Earthquake of 1906 marks the beginning of earthquake engineering in the U.S. (Atkinson & Kiland, 2004). Over the next two decades, several seismic events rocked the state of California, and consequently the Structural Engineers Association of Southern California (SEAOSC) was founded in 1929 (Atkinson & Kiland, 2004). The stated purpose of the organization was:

“To advance the science of structural engineering; to assist the public in obtaining dependable structural engineering services; to encourage engineering education; to maintain the honor and dignity of the profession and to enlighten the public with regard to the province of the structural engineer” (Atkinson & Kiland, 2004).

SEAOSC later merged with a similar group from Northern California and the Structural Engineers Association of California (SEAOC) was formed in late 1931 (Atkinson & Kiland, 2004). Publications from the SEAOC Seismology Committee and the ATC, have been the basis of current earthquake design in the U.S. Seismic design methodology has continually evolved since the first introduction of a seismic lateral force equation in 1927 in the Uniform Building Code (UBC), a regional model building code. After each major earthquake, engineers and researchers studied building behavior and failures and changed the building codes based on their observations. The beginning of modern seismic design methodology is found in the first edition of the SEAOC *Recommended Lateral Force Requirements and Commentary* (Blue Book) in 1959 (SEAOC, 2007) and was significantly changed in 1978 with the publication of *ATC 3-06, Tentative Provisions for the Development of Seismic Regulations for Buildings*. ATC 3-06 recommended seismic provisions use a response modification factor,  $R$ , and change the seismic load from stress level to strength level. Since 1978, seismic design methodology and  $R$  have continued to change.

Until the late twentieth century, seismic design was not well understood or practiced in the U.S. The UBC first introduced seismic design in 1927 because of the frequency of large

earthquakes in southern California in the early 1900s, but not until after the publication of the Blue Book in 1959 was seismic design considered mandatory nationwide. Even after adopting the methods in the Blue Book, several model building codes (both regional and local) allowed exemptions for some geographic areas based on historic seismic activity and damage records. The exceptions allowed most geographic regions of the U.S. to prove it was not necessary to consider seismic forces for buildings in those regions. Many areas of the U.S. had no history of an earthquake resulting in major damage because of low population densities or lack of seismic activity. Though records did not show earthquake damages, seismic risk was should have been a major concern in most of the U.S.

Today, building codes in the U.S. base seismic design on proportioning members of the LFRS for expected actions, using linear structural analysis and prescribed lateral forces determined using the Equivalent Lateral Force Procedure (ATC, 1995). Currently, seismic design forces (seismic base shear) are determined by dividing lateral seismic forces occurring at the base of an elastically responding structure by a response modification factor,  $R$  (ATC, 1995). Seismic design has developed through many changes in the calculation of the prescribed lateral forces, base shear and  $R$  to arrive at the current method. This chapter introduces the major steps in the development of seismic design procedures to reach today's current documents.

## **2.1 Seismic Design Prior to the Blue Book (1927-1959)**

SEAOC published the first Blue Book in 1959. Before the Blue Book was published several code writing bodies were in existence and covered different regions of the U.S., but none considered seismic design until after the 1925 Santa Barbara, California earthquake. These regional building codes had a general geographic area of use and, depending on experience, engineers considered the importance of seismic design very differently even within the same region. The three major model building codes and the relative geographic areas of use were as follows (Beavers, 2002). The UBC and the National Building Code (BOCA) will be discussed in detail in later subsections.

- Uniform Building Code (UBC)      West of the Mississippi River
- National Building Code (BOCA)    Upper Midwest and Northeast
- Standard Building Code (SBC)      Southern U.S.

The UBC was the first to include a method for determining seismic forces in 1927. BOCA followed in 1950. Other authoritative bodies, like the Atomic Energy Commission and the Environmental Protection Agency also developed code documents related to seismic design; however, these documents were written specifically for nuclear facilities and landfills (Beavers, 2002). Several other agencies, like the American National Standards Institute (ANSI), prepared standards for seismic design. Most of the other standards were for specific structure types; however ANSI 58.1, *Minimum Design Loads for Buildings and Other Structures*, the only one referenced for general building design by engineers recognizing the need for seismic design but who were outside the jurisdiction of the UBC (Beavers, 2002). In 1952, the American Society of Civil Engineers (ASCE) published *Lateral Forces of Earthquake and Wind* (now incorporated into the ASCE 7, *Minimum Design Loads for Buildings and Other Structures*) which replaced ANSI 58.1 as the standard of choice. The *Building Construction and Safety Code 2003*, NFP 5000, also developed a methodology for general building seismic design, and California adopted this document as their model building code until 2006.

### **2.1.1 UBC, International Conference of Building Officials**

The International Conference of Building Officials (ICBO) was founded in 1922. The UBC addressed seismic design in “Section 2312- Earthquake Regulations” written into the main code body, beginning in 1927 after the 1925 Santa Barbara earthquake (Beavers, 2002). Section 2312 had no content and referred the designer to an appendix section, thereby making seismic design optional, not mandatory even if jurisdictions adopted the provisions into the local codes. In 1943, the first jurisdiction to adopt UBC provisions was the city of Los Angeles (Atkinson & Kiland, 2004). The UBC regulations calculated lateral force without considering structural system effects (Line, 2006). In the 1927 UBC the lateral force was a constant percentage of building weight, as shown in Equation 2.1 (Beavers, 2002).

$$F = 0.075W$$

**Eqn 2.1**

Where:  $F =$  the force in pounds

$W =$  the total dead load tributary to the point under consideration in pounds

In 1935, the UBC introduced a horizontal force factor in lieu of the constant percentage of building weight. The horizontal force factor depended on the type of structural element, as shown in Table 2-1, and the geographic location. Values were not constant across the entire region but were based on seismic zone map of the eleven western states, developed in 1928 by N.H. Heck (Beavers, 2002). Equation 2.2 gives the 1935 UBC equation for lateral force. This force was the required lateral strength of the structural element being designed.

$$F = CW$$

**Eqn 2.2**

Where:  $C =$  horizontal force factor as shown in Table No. 23-C of the UBC (Table 2-1)

$W =$  the total dead load tributary to the point under consideration in pounds

**Table 2–1 Horizontal force factors, C, from Table 23-C of 1935 UBC and 1949 UBC**

Location of Structural Element	Horizontal Force Factor, $C^a$
Floors, roofs, columns and bracing in any story of a building or the structure as a whole <sup>b</sup>	$\frac{0.15}{(N_t + 4.5)}$
Bearing walls, non-bearing walls, partitions, free-standing masonry walls over 6' in height	0.05, but a minimum force of 5 psf normal to the surface of the wall
Cantilever parapet and other cantilever walls, except retaining walls	0.25, in any horizontal direction
Exterior and interior ornamentation and appendages	0.25, in any horizontal direction
Towers, tanks, towers and tanks plus contents, chimneys, smokestacks and penthouses when connected to or a part of a building	0.05, in any horizontal direction
Elevated water tanks and other tower supported structures not supported by a building	0.03, in any horizontal direction

- a: “C” values given are minimum and should be adopted in locations not subject to frequent seismic disturbances as shown in zone 1. For locations in zone 2 “C” shall be doubled. For locations in zone 3, “C” shall be multiplied by 4.
- b:  $N_t$  is number of stories above the story under considerations, provided that for floors or horizontal bracing,  $N$  shall only be number of stories contributing loads. (UBC, 1949)

The zone map developed by Heck that was included in the 1935 UBC, illustrated that certain geographical areas experienced earthquake disturbances more frequently. On May 18, 1940 the first reliable strong motions seismograph record was made of the Imperial Valley earthquake (Atkinson & Kiland, 2004). This seismograph record provided engineers with a view of real-time accelerations and ground motions of a seismic event. As a result of this new knowledge of earthquake motions and history, a zone map entitled “Seismic Probability of the U.S.” was compiled in 1948 by the U.S. Coast and Geodetic Survey (USCGS) (UBC, 1949) and included in the 1949 UBC. The map (Figure 2-1) contained four zones (0-3) and each zone step

indicated an increased risk of damage due to seismic activity. Zone 0 corresponded to no damage and Zone 3 indicated major damage. This map changed slightly in the 1958 edition of the UBC. Figure 2-2 shows the altered map. Observations of building damage from the 1940 Imperial Valley earthquake lead to adjustments of the values of  $C$  given in the UBC appendix.  $C$  increased from the value given based on the seismic zone of the project site. Although it was not explicitly stated, Zone 0 locations did not require additional considerations due for seismic loads.

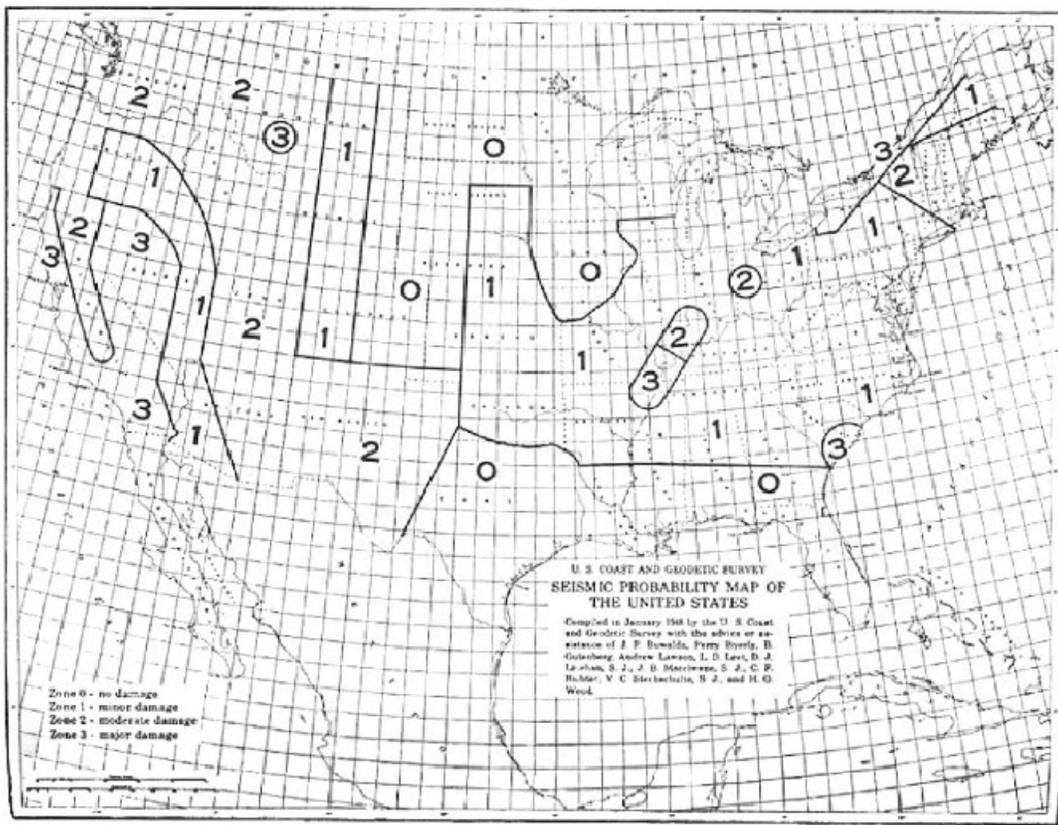


Figure 2-1 Seismic probability map of the U.S. Republished courtesy of the ICC; originally published in 1949 UBC.

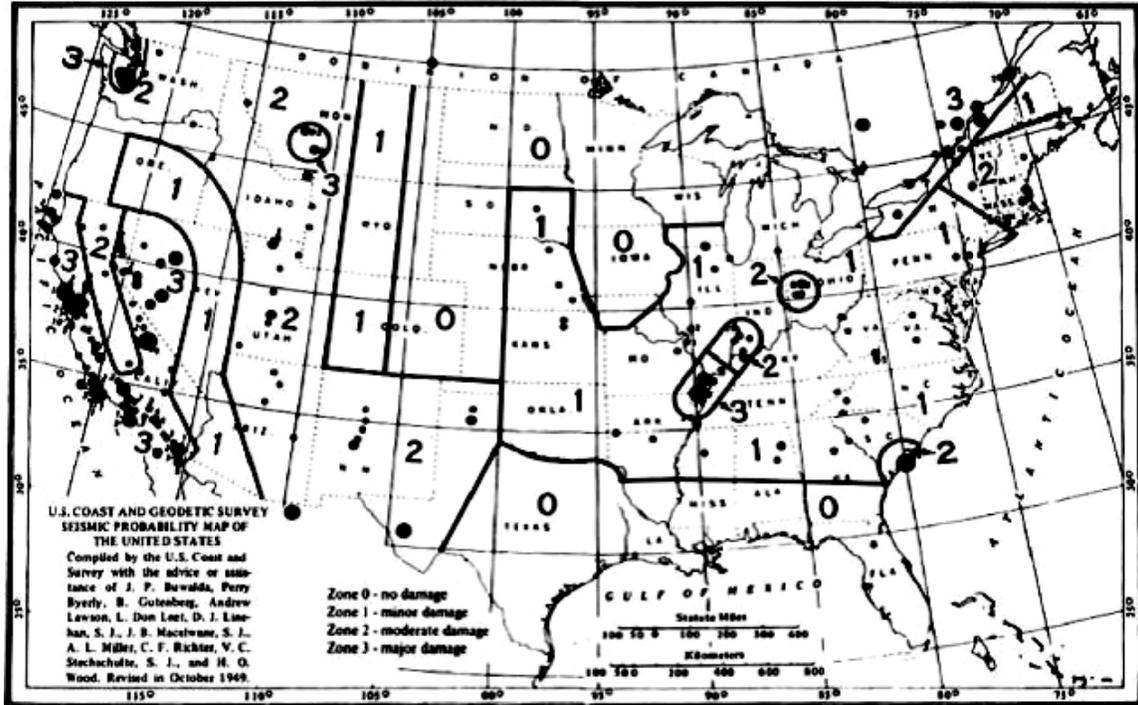


Figure 2-2 Map of the U.S. showing zones of approximately equal seismic probability. Republished courtesy of the ICC; originally published in the 1958 UBC.

Table 2-2 Horizontal force factors,  $C^a$ , from Table 23-C of 1958 UBC

Floors, roofs, columns and bracing in any story of a building or the structure as a whole <sup>b</sup>	_____
Bearing walls, non-bearing walls, partitions, free-standing masonry walls over 6' in height	, but a minimum force of 5 psf normal to the surface of the wall
Cantilever parapet and other cantilever walls, except retaining walls	, in any horizontal direction
Exterior and interior ornamentation and appendages	, in any horizontal direction
Towers, tanks, towers and tanks plus contents, chimneys, smokestacks and penthouses when connected to or a part of a building	, in any horizontal direction
Elevated water tanks and other tower supported structures not supported by a building	, in any horizontal direction

- a: “C” values given are minimum and should be adopted in locations not subject to frequent seismic disturbances as shown in zone 1. For locations in zone 2 “C” shall be doubled. For locations in zone 3, “C” shall be multiplied by 4.
- b:  $N_i$  is number of stories above the story under considerations, provided that for floors or horizontal bracing,  $N$  shall only be number of stories contributing loads. (UBC, 1949)

**2.1.2 The National Building Code of the Building Officials and Code Administrators International**

The Building Officials and Code Administrators International was founded in 1915. The National Building Code published by this organization is known as the BOCA model building code and was predominantly used in the upper midwest and northeastern U.S. In 1950, the BOCA code adopted a seismic design method much like the 1927 UBC method. In this method, the lateral force at the level of interest are calculated as a percentage of building weight, with percentages varying based on building height. The following were the values given as the minimum design lateral seismic force (BOCA, 1950):

**Table 2–3 Minimum design lateral seismic force, 1950 BOCA**

<b>Building Height</b>	<b>Minimum Design Lateral Seismic Force</b>
$h \leq 35$ feet	$\geq 5\%$ of the dead load
$35 \text{ feet} < h \leq 100$ feet	$\geq 10\%$ of the dead load
$100 \text{ feet} < h$	$\geq 20\%$ of the dead load
All parapet walls and exterior ornamentation	= 100% of the dead load of the wall or other projection

In 1955, BOCA adopted the same equation (Eqn 2.2) and horizontal force factor (Table 2-2) as what would later be published in the 1958 UBC method but included the following exceptions (BOCA, 1966):

“In zone “0” of table 14C in appendix K-11 and where local experiences or records do not show loss of life or damage of property, regardless of zone, or when the building complies with any one or more of the following conditions, no earthquake loading shall be required in calculating the structural frame of the building or structure.

- (a) is a 1 or 2 family dwelling
- (b) is a minor accessory building
- (c) is not over 3 stories or 35’ in height
- (d) is the skeleton of frame construction with wind sway bracing as required by the approved engineering practice for the type of frame used and the least dimension of the building is not less than 35% of the height.”

The exceptions eliminated the need for seismic design in most of the jurisdictions where the BOCA code was used for design. Many of the areas governed by BOCA did not have records of strong seismic events since the regions’ settlement. Additionally, those that were recorded (i.e. 1811-1812 New Madrid Fault Zone events in southeast Missouri) did not result in a significant loss of life or damaged property because few people inhabited the area during the time of the severe event (Beavers, 2002). These exemptions, or similar statements, remained even as late as the 1978 edition (Beavers, 2002).

### ***2.1.3 Standards from ANSI and ASCE***

In 1945, ANSI published ANSI A58.1 (Beavers, 2002), the first national standard to consider earthquake loads. The lateral force equation was the same as Equation 2.2, but the  $C$  values differed from those in the 1958 UBC.  $C$  within ANSI A58.1 varied from 0.1 for each story in a building to 1.0 for components of the structure, depending on the member being designed (Beavers, 2002). ANSI A58.1 contained a seismic hazard map displaying the locations of significant seismic events in map form, but the hazard level did not affect the lateral force value. ANSI A58.1 was not widely used after 1952 as a result of the publication of the ASCE

*Lateral Forces of Earthquake and Wind*. Before 1952, engineers who recognized the need for seismic design used ANSI A58.1 as the standard for earthquake loads outside of the jurisdiction that adopted the UBC.

The *Lateral Forces of Earthquake and Wind*, which was eventually incorporated into the *ASCE 7- Minimum Design Loads for Buildings and Other Structures*, replaced the ANSI standard in 1952. This standard is the first to use a dynamic approach to seismic design (Beavers, 2002). ASCE introduced the base shear equation, Equation 2.3, in lieu of the lateral force equations given in ANSI A58.1, the UBC and BOCA (Beavers, 2002). A function relating to the inverse of the structure's natural period defined  $C$  (Beavers, 2002). The model building codes did not adopt this concept until 1961.

$$V = CW \quad \text{Eqn 2.3}$$

Where:  $V$  = lateral seismic base shear in pounds

$$C = \frac{1}{T}$$

$T$  = structure's natural period

$W$  = the total dead load tributary to the point under consideration in pounds

## 2.2 The Dynamic Approach (1959 -1972)

SEAOC published the first Blue Book in 1959 in an attempt to unify the design approach for seismic LFRSs and provide minimum standards to assure public safety in California. During this time, designers observed that structural systems, as well as weight and height, affected the building's response to cyclic lateral forces. This knowledge meant that seismic provisions in the codes of the time needed significant changes.

### 2.2.1 Changes to Base Shear

The 1959 edition of the Blue Book introduced the  $K$  factor to address different building type as well as redefining the  $C$  factor for the base shear equation (Fratessa, 1986). The recommended base shear equation is similar to Equation 2.3, but includes considerations for structural system type (Beavers, 2002):

$$V = KCW$$

Eqn 2.4

Where:  $V$  = lateral seismic base shear in pounds

$$C = \frac{0.05}{\sqrt[3]{T}}$$

$K$  = numerical coefficient based on basic structural system

$T$  = structure's natural period

$W$  = the total dead load tributary to the point under consideration in pounds

Engineers realized the dynamic behavior of the building changed the forces occurring within individual members at any one point in time during a seismic event, especially for flexible structures. Adjusting the base shear by a function of the period accounted for the natural behavior of a building under dynamic loads.

The  $K$  factor was intended to modify the base shear using a number representing bonus characteristics of the structure in question (Fratessa, 1986). The bonus characteristics were intended to account for the unique behavior of the structure type as demonstrated in research and testing as well as damage studies from previous seismic events (Fratessa, 1986). The method of determining the  $K$  factors is not documented; however all subsequent seismic factors have shown similar relationships between bearing wall systems, building frame systems and moment resisting frame systems.

The 1959 Blue Book did not distinguish among materials for the  $K$  values. Structural systems were grouped by system type: bearing wall, building frame, dual systems and moment resisting frames (ATC, 1995). Table 2–3 gives the  $K$  values from the first edition of the Blue Book.

**Table 2–4 K-values from the 1959 SEAOC Blue Book**

<i>Basic Structural System</i>	<i>K</i>
Bearing Wall	1.33
Building Frame	1.00
Dual	0.80
Moment Resisting Frame	0.67

In 1961, the UBC adopted a similar equation into their seismic code requirements, but added consideration of seismic hazard probability, as shown in Equation 2.5. SEAOC considered all of California to have the same high hazard probability but since the UBC covered more than just California and most of its jurisdiction had lower seismic probability, the equation had to address the issue of seismic hazard probability. The consideration of seismic hazard did not change the lateral force result for most of California but it reduced the lateral force for other parts of the country.

$$V = ZKCW$$

**Eqn 2.5**

Where:  $V$  = lateral seismic base shear in pounds

$Z$  = numerical coefficient based on seismic hazard probability from accompanying seismic zone map (Zone 1,  $Z = 0.25$ ; Zone 2,  $Z = 0.50$ ; Zone 3,  $Z = 1.0$ )

$$C = \frac{0.05}{\sqrt[3]{T}}$$

$K$  = numerical coefficient based on basic structural system

$T$  = structure's natural period

$W$  = the total dead load tributary to the point under consideration in pounds

Also in 1961, the UBC moved the seismic provisions from the appendix into the main text making it mandatory to consider seismic forces when designing a structure (Beavers, 2002). Though exceptions still allowed exemptions for some structures, designers had to address the seismic provisions within jurisdictions that adopted the UBC.

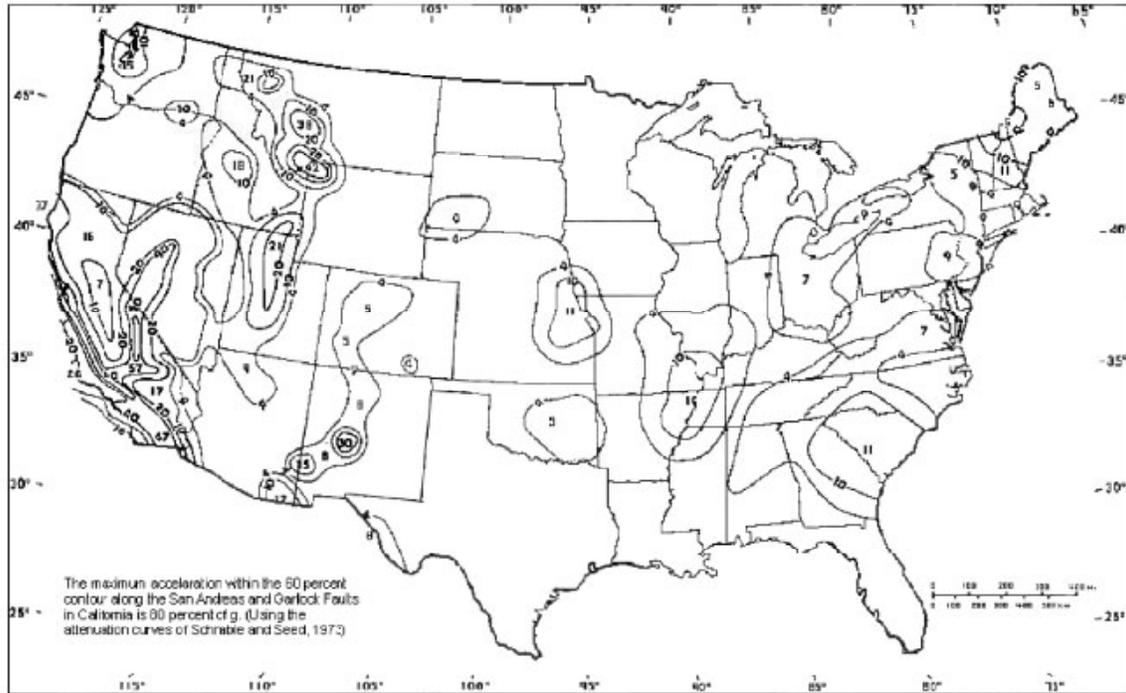
### **2.2.2 New Ideas**

In early February of 1971 the San Fernando earthquake occurred. This massive event, magnitude 6.6 (M6.6) on the Richter scale, allowed engineers to see flaws in the then current seismic provisions. Extensive damage to the Olive View Hospital, designed using the 1970 UBC, led to its demolition after the earthquake. The damage caused by the San Fernando event led to a movement by federal agencies, engineers and scientists to develop better provisions for

designing safer structures. This movement sparked the formation of the ATC in late 1971 (Beavers, 2002). The SEAOC board of directors formed the ATC as a non-profit subsidiary. The Federal Emergency Management Agency (FEMA) and the National Science Foundation (NSF) funded research by the ATC. The first project of this new group was a shake-table test of large-scale prototype structures at the University of California (Atkinson & Kiland, 2004).

### **2.3 Changes to the Methodology, ATC-3 06 and NEHRP (1973-1986)**

In 1973, SEAOC published a new edition of the Blue Book. This edition incorporated into the revisions lessons learned from the San Fernando earthquake and a similar magnitude event in Nicaragua. The phenomenon of soil-structure interaction was a major addition. Also in the early 1970s, ATC quickly received funding for new projects following the presentation of shake table results showing “lamellar tearing” failures in heavy steel structures with welded joints (Atkinson & Kiland, 2004). The results of those tests also caused concern for the robustness of many other types of structural systems. The third ATC project, titled *Tentative Provisions for the Development of Seismic Regulations for Buildings, ATC-3 06*, became the base point for all future seismic codes. At the same time as the ATC research for ATC-3 06, the U.S. Geological Survey (USGS) was updating the seismic hazards map. The new map, published in 1976, was the first probability based map to show estimates of maximum accelerations instead of hazard zones (Figure 2-3).



**Figure 2-3 1976 USGS seismic accelerations map- estimated maximum acceleration contours on rock, developed by Algermissen and Perkins. Republished courtesy of the ATC; originally published in ATC-3 06.**

Finally, the Earthquake Hazards Reduction Act of 1977 called for a government focus group for the development of seismic code provisions. The launch of the FEMA in 1979 brought the National Earthquake Hazards Reduction Program (NEHRP). FEMA collaborated with the NSF, the USGS and the National Bureau of Standards (now the National Institute of Standards and Technology (NIST)) to develop and implement earthquake standards (Hamilton, 2003). The first publication of the NEHRP seismic provisions was an edited version of the ATC-3 06 proposals (Ghosh, 2004).

### ***2.3.1 ATC-3 06 Methodology***

ATC-3 06 embodied many new concepts. The most notable was the introduction of the response modification factor,  $R$ , in lieu of previously used  $K$  factors. ATC-3 06 introduced  $R$ , as part of the shift from allowable stress design to strength level design. Other new concepts included (ATC, 1978):

- building use groups
- seismic performance categories related to the fundamental period of the seismic resisting system of the building
- seismic design requirements for building systems and individual components
- more realistic seismic ground motion maps
- distance earthquake effects on long-period structures
- material design and analysis based on yield stress (strength level design in lieu of allowable stress design)
- guidelines for abatement of seismic hazards in buildings
- guidelines for damage assessment, damage repair/strengthening and potential earthquake hazards within a building

### ***2.3.1.1 The Equivalent Lateral Force Procedure (ELFP)***

The ATC-3 06 methodology determined the required static lateral force capacity of a building based on an assumed behavior of a structure under a dynamic lateral load. The lateral base shear equation included the seismic coefficient,  $C_s$ , and total gravity load of the building (ATC, 1978). The total gravity load of the structure was the total building weight including partitions and permanent equipment, a minimum of twenty-five percent of the live load for storage and warehouse structures and a portion of the snow load (ATC, 1978). Equation 2.6 defines lateral base shear and is the same equation used today in the ASCE 7. The seismic coefficient,  $C_s$ , and the determination of the building gravity load have changed since the ACT-3 06 publication, changes that will be discussed later in this chapter.

$$V = C_s W \qquad \text{Eqn 2.6}$$

Where:  $V$  = lateral seismic base shear in pounds

$C$  = seismic design coefficient (defined in Equations 2.7-2.9)

$W$  = the total dead load tributary to the point under consideration in pounds

This base shear equation did not include the seismic importance factor,  $I$ , as had been included in the base shear equation in the most recent SEAOC Blue Book. The Blue Book introduced  $I$  to represent the importance of the building function.  $I$  increased the base shear of the entire structure for facilities critical to emergency response or life safety. Instead, ATC 3-06 recommended that the designer use different levels of detailing for ductility within a structure to ensure life safety rather than designing the whole building for a higher base shear. The detailing requirements were set forth with Seismic Performance Categories (SPC). Each SPC corresponds to a type of building function. More important functions, such as emergency facilities, require higher levels of detailing. Moreover, the levels of detailing increase as the level of seismic risk increases. Using the SPC, a hospital in an area of low seismic risk would have the same level of detailing as an office building in an area of high seismic risk (Ghosh & Khuntia, 1999).

Equation 2.7 shows the calculation of the seismic coefficient published in ATC-3 06, with limits shown in Equation 2.8 and Equation 2.9. The variables used to determine the coefficient are based on the elastic acceleration response spectra, the elastic design spectra, the LFRS and the structure's fundamental period of vibration. The LFRS determines the response modification factor,  $R$ , which is a force reduction factor. The constants found at the front of the equations are dynamic amplification constants. These constants represent the amplification of accelerations at the higher levels of the structure when a force is applied at the building's base. A brief explanation of the factors contributing to the new ideas in the ATC-3 06 methodology follows the equations. The next subsection discusses the response modification factor in detail.

$$C_s = \frac{1.2A_v S}{RT^{0.67}} \quad \text{Eqn 2.7}$$

$$C_s \leq \frac{2.5A_a S}{R} \quad \text{Eqn 2.8}$$

$$C_s \leq \frac{2A_a S}{R}, \text{ where } S=1.3 \quad \text{Eqn 2.9}$$

Where:  $C_s$  = seismic coefficient

$A_a$  = effective peak acceleration of the design ground motion expressed as a fraction of  $g$

$A_v$  = effective peak velocity related acceleration

$S$  = soil profile coefficient

$R$  = response modification coefficient as given in Table 2-5

$T$  = fundamental period of the building

#### ***2.3.1.1.1 Design Earthquake Ground Motion***

The design earthquake is defined in the commentary of the ATC-3 06 as “the ground motion for which an architect or engineer should have in mind when designing a building which is to provide protection for life safety” (ATC, 1978). At the time of publication, smoothed elastic response spectra were the best tool to describe the design earthquake but were not implemented in seismic provisions. Response spectra are now a central concept in earthquake engineering, with roots in 1932 in the work of M. A. Biot (Chopra, 2007). The current ASCE 7 still uses smoothed elastic response spectra to determine the site coefficients (ASCE, 2005). The smoothed elastic response spectra used for developing the regionalization maps included a five percent natural damping within the structure. The design maps (figures 2-4 and 2-5) intended to show parameters of a design earthquake with an equal probability of exceedance for all areas. Previous codes did not consider probability of exceedance, but looked at historical records to determine the required capacity. One deficiency of smoothed elastic response spectra is the lack of reference to the duration of ground motion (ATC, 1978). The ATC-3 06 committee envisioned that ground motions would last for 20 to 30 seconds but did not explicitly address adjusting base shear to other durations (ATC, 1978).

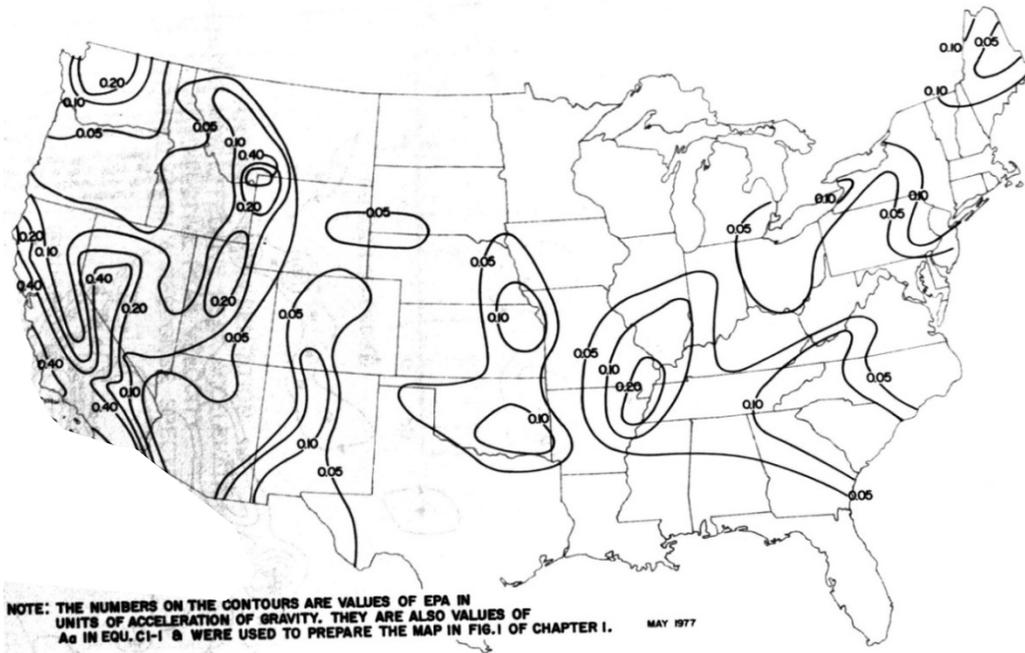


Figure 2-4 Contour map for effective peak acceleration,  $A_a$ . Republished courtesy of the ATC; originally published in ATC-3 06.

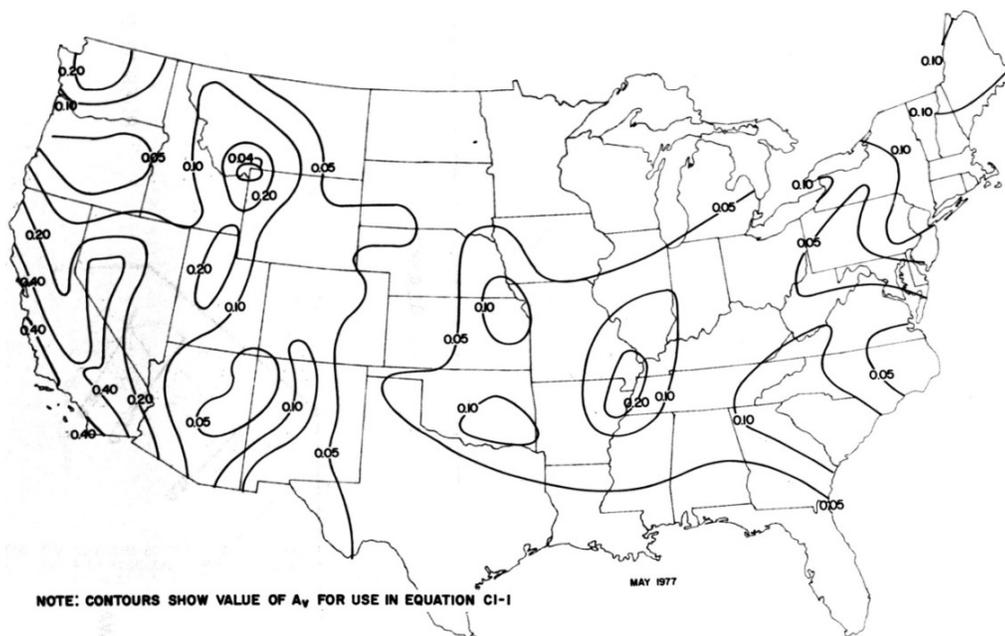


Figure 2-5 Contour map for effective peak velocity,  $A_v$ . Republished courtesy of the ATC; originally published in ATC-3 06.

#### **2.3.1.1.2 Ground Motion Parameters**

ATC-3 06 recognized two parameters to characterize the intensity of the ground motion of the design earthquake: the estimated peak acceleration (EPA) and estimated peak velocity (EPV) (ATC, 1978). The ATC-3 06 Commentary describes these as normalization factors to smooth the elastic response spectra for ground motions of normal duration (ATC, 1978). The EPA is proportional to spectral ordinates for periods ranging from 0.1 to 0.5 seconds and the EPV is proportional to a period of about one second (ATC, 1978). A 2.5 factor normalizes both values to account for the assumed five percent natural damping used to develop the smoothed elastic response spectrum. At any specific location on the map, either the EPA or the EPV will govern the design (ATC, 1978) (see equations 2.7 through 2.9).

The starting point for the development of the EPA and EPV maps for ATC-3 06 was the previously used USGS seismic hazard map, contained in the UBC. County lines defined the boundaries for the different contours of the maps published in ATC-3 06 (ATC, 1978). The map included in ATC-3 06 corresponds to site conditions for firm ground, i.e. shallow deposits of stiff, cohesive soils and dense granular soils, including rock (ATC, 1978). The USGS map stated rock as the control site condition. Adjustments made to this map for ATC-3 06 included relating “firm ground” to “rock” sites, among other small considerations related to time histories and probabilities of reoccurrence (ATC, 1978). The contour maps (figures 2-4 and 2-5) provide the values of  $A_a$  and  $A_v$  used in Equations 2.7, 2.8 and 2.9 (page 18).

#### **2.3.1.1.3 Design Elastic Response Spectra**

The design elastic response spectra presented in ATC-3 06 considered the effects of site conditions and the distance from the seismic source zone (ATC, 1978). The magnitude of the ground motion and the source mechanism were also recognized as contributing factors to the site response, but could not be implemented in the response spectrums. Ground motion magnitude and source mechanism are unique for each seismic event, which would have resulted in several response spectra, complicating the design process.

The provisions define four different site conditions (ATC, 1978):

- Rock
- Stiff soil/ firm ground
- Deep cohesion-less/ stiff clay soil
- Soft-to-medium stiff clay/ sand

These soil types determine the soil profile type and the value of the soil profile coefficient seen in Equation 2.7.

The elastic acceleration response spectra (EARS) provided in ATC-3 06 has a descending branch for long period structural vibrations. The branch in the EARS descends according to the inverse of the structure's natural period,  $T$ . The committee decided that this branch should fall as  $T^{2/3}$  after studying behavior of long period structures (ATC, 1978). As a building's period increases, the committee assumed that the number of stories, and thus the degrees of freedom, also increased. High degrees of freedom enabled designers to concentrate ductility requirements within a few stories, changing the behavior of the structure. Consideration of structural instability becomes more of a concern as the natural period increases. Both of these factors affected the final shape of the elastic design spectra used in ATC-3 06.

#### **2.3.1.1.4 Fundamental Period of Vibration**

Many available methods can determine the fundamental period of a structure. The commentary of ATC-3 06 gives Rayleigh's method as one method to calculate the exact period of vibration of a structure. Rayleigh's method would be time consuming if designers applied the analysis to each project. For this reason, ATC used Rayleigh's method to develop a relationship between structure type and first mode fundamental period (ATC, 1978). ATC introduced the approximate fundamental period of a structure as (ATC, 1978):

$$T_a = C_T h_n^{\frac{3}{4}}, \text{ for moment resisting structures} \quad \text{Eqn 2.10}$$

$$T = \frac{0.05 h_n}{\sqrt{L}}, \text{ for all other structures} \quad \text{Eqn 2.11}$$

where:  $T$  = approximate fundamental period of the structure

$C_T$  = 0.035 for steel frames

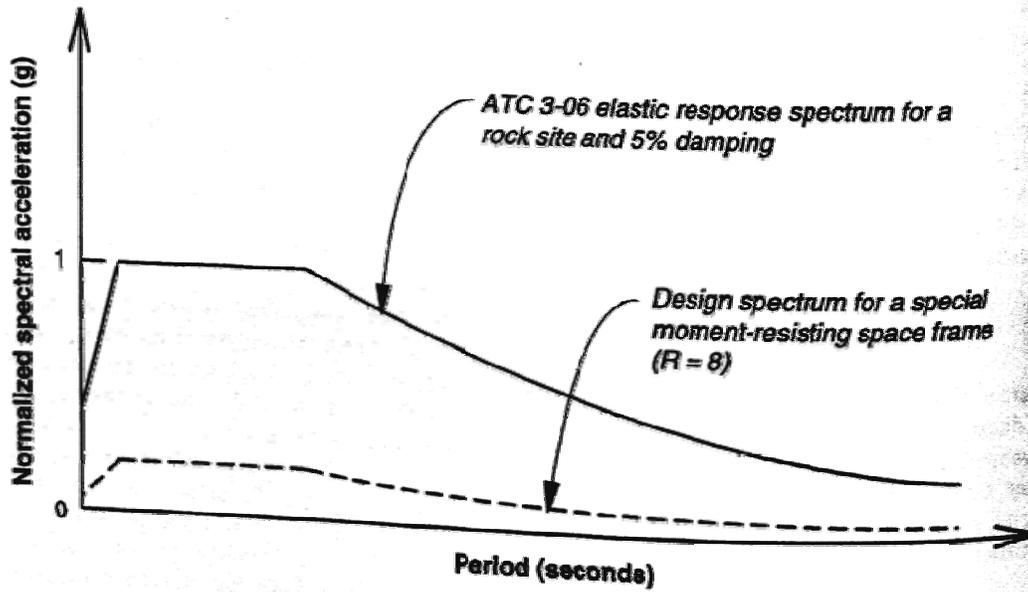
$C_T$  = 0.025 for concrete frames

$h_n$  = the height in feet above the base to the highest level of the building  
 $L$  = the overall length (feet) of the building in the direction of  
consideration

### **2.3.1.2 The Response Modification Factor, $R$**

The basis of the  $R$  factors given in ATC-3 06 considers the structure's inherent toughness, the amount of natural damping, and observed past performance of various types of structural systems. The intent of the factor was to bring structural dynamics into the static design process (ATC, 1995) by reducing the size of the elastic response spectrum.  $R$ , along with  $C_d$  and story drift criteria, was developed because structures have additional strength capacity above the elastic range and yield strength. The approach to seismic engineering presented in ATC 3-06 assumed the structure would move into the inelastic range of certain elements or members to reach a maximum allowable story drift. The maximum allowable story drift is a life safety parameter and ensures the structure remains in a stable position, but not necessarily an undamaged position. Allowing the building to move into the inelastic range makes seismic design more economical while still maintaining the life safety parameters of the code.

In general, the  $R$  factor is a ratio of the forces that would develop in the structure under a specified ground motion if the behavior were entirely elastic compared to the prescribed design forces at the level of significant yield (ATC, 1978).  $R$  reduces the design value of the base shear for the design earthquake, which ensures that the structure could enter the inelastic range if the design earthquake or larger event occurred, see Figure 2-6. Each point on the normalized elastic response spectrum is divided by  $R$  to produce the design spectrum for a given structure type (ATC, 1995).



**Figure 2-6 Use of  $R$  factors to reduce elastic spectral demands to the design force level. Republished courtesy of the ATC; originally published in ATC-19.**

The  $R$  factors given in ATC-3 06 were judgments and did not rely on testing results. Participants in an ATC-3 06 workshop each developed  $R$  factors independently, based on judgment and an  $R$  to  $K$  relationship given by equation 2.12. The  $R$  factors selected varied between participants, depending on the experience with seismic engineering of the participant (ATC, 1995). ATC-3 06 published  $R$  factors that came from the workshop selections. The commentary advises designers to select and apply  $R$  factors carefully, implying that the list is not finite and deviations from the published values are acceptable. One possible reason for deviating from the list is building importance or occupancy category. However, ATC-3 06 recommends that the design account for improved performance for buildings of higher importance with specific detailing set forth in the requirements for each of the SPCs (ATC, 1978).

The maximum  $R$  factor for the structure type considered was the first step in creating the initial selections (ATC, 1995). The definition of the most effective earthquake resisting system was “the system that performed the best in past events” (ATC, 1978). The committee selected special steel moment frames and dual systems of reinforced concrete shear walls and special steel moment frames as the best systems (ATC, 1995). In the dual system, the special steel moment frames in the dual system had to resist at least 25 percent of the required seismic base

shear. The ATC-3 06 committee wanted to develop strength level seismic provisions because the building design industry was trying to move towards strength design and away from allowable stress design. The  $K$  factors of the 1959 Blue Book were the starting point for determining the  $R$  factor. A series of equations relating strength level design base shear (Equation 2.6) to allowable stress design base shear (Equation 2.5) was applied in order to develop the maximum possible  $R \sim K$  relationship (see Appendix A; ATC, 1995). The final  $R \sim K$  relationship was determined to be:

$$R = \frac{5.1}{K} \qquad R \sim K \text{ Relationship, 1978} \qquad \text{Eqn 2.12}$$

The most recently assigned  $K$  value for moment resisting frames, which is assumed to be the most ductile building system, was 0.67, yielding an  $R$  factor of 8.0 for the most efficient systems. The  $R \sim K$  relationship was applied for other structural systems covered in the 1976 UBC and the resulting  $R$  factors were adjusted according to the committee consensus. As demonstrated by this process, the speculation and engineering judgment that played a large role in developing the  $K$  factor was also the basis for the value of  $R$ . The  $R$  factors published in 1978 are in Table 2–3.

### ***2.3.2 National Earthquake Hazards Reduction Program, 1979***

The NEHRP program started in 1979. The Alaska earthquake (M9.0) provided impetus for a government agency focused on reducing earthquake hazards (Beavers, 2002). President Carter issued an executive order in 1979 and merged several disaster response agencies to form FEMA (FEMA, 2009). FEMA collaborated with the NSF, the USGS and the National Bureau of Standards to develop and implement earthquake standards (Hamilton, 2003).

After the publication of the ATC-3 06 document in 1978, the building engineering community examined the provisions. Trial structures, constructed using NEHRP funding, applied the provisions of ATC-3 06. The results of lateral tests on these structures indicated that the original ATC-3 06 needed some changes (Ghosh, 2004). The resulting publication, *NEHRP Recommended Provisions and Commentary for Seismic Regulations for New Buildings and Other Structures 1985 Edition*, was the first edition of what the seismic design community knows, today, as the *NEHRP Provisions*.

The NEHRP committee made very few adjustments to the ATC-3 06 methodology. As shown in Table 2–3, *The Provisions* adjusted a few  $R$  factors. Changes to other parts of the equivalent lateral force procedure are identified in the remainder of this paragraph. NEHRP redefined the snow load contribution to the effective seismic weight to include an exception for locations with less than thirty pounds per square foot snow load. This exception did not delineate whether the snow load used was the ground snow load or flat roof snow load. The maximum calculated period limit changed from a fixed value to a value dependant on site coefficients. (This work does not discuss changes to the modal analysis method and any other analysis method presented in ATC-3 06.)

The design industry recognized that a dynamic approach to earthquake forces was necessary to ensure safe designs. However, the switch to strength methods and the  $R$  factor was slow. In most model building codes the seismic risk factor,  $Z$ , and the USGS Seismic Risk map published in 1976 remained in use until the 2000 edition of the IBC instead of using the  $A_a$  and  $A_v$  factors proposed in ATC 3-06 and NEHRP. The building importance factor also remained in model building codes. Codes ignored the SPCs until the 2000 edition of the IBC, because the seismic zones and the importance factor continued in use.

## **2.4 Stress Design versus Strength Design (1987-1997)**

The ATC-3 06 method determined a strength level earthquake load. At the time, the regional codes determined design loads at the working stress (ASD) level, continuing to rely on circa 1971 codes to determine seismic design requirements. Thus, SEAOC developed a method similar to ATC-3 06 but kept the ASD method for the 1988 Blue Book. The response modification factor for ASD was called  $R_w$ . Regional codes continued to use this method until 1997 when the UBC adopted strength level earthquake forces, incorporating  $R$  into seismic design. In 1995, the ATC published *Structural Response Modification Factors, ATC-19*. ATC-19 provided a review of the  $R$  factors and redefined the term using three characteristics of the structural system, strength, ductility and redundancy. ATC-19 intended to stimulate discussion and further research on the  $R$  factors and how the code should use characteristics of the structural system to determine the coefficient by providing a quantifiable definition. In 1997, ICBO joined BOCA and SBCCI to form the ICC, which uses strength level design.

### 2.4.1 1988 Blue Book

As with the  $R$  factors developed by ATC, the previously defined  $K$  factors determined the maximum value for  $R_w$  (ATC, 1995). The base shear equation changed from Equation 2.13 to Equation 2.14 by adding the  $R_w$  term.

$$V_D = Z I K C S W \quad \text{Eqn 2.13}$$

$$V_D = \frac{Z I C W}{R_w} \quad \text{Eqn 2.14}$$

$$C = \frac{1.25 S}{T^{0.67}} \quad \text{Eqn 2.15}$$

Where:  $V$  = lateral seismic base shear in pounds

$Z$  = numerical coefficient based on seismic hazard probability from accompanying seismic zone map (Zone 1,  $Z = 0.25$ ; Zone 2,  $Z = 0.50$ ; Zone 3,  $Z = 1.0$ )

$I$  = importance factor of the structure, based on building use

$K$  = numerical coefficient based on basic structural system

$C = \frac{0.05}{\sqrt[3]{T}}$  in Equation 2.13 and as defined by Equation 2.15 in Equation 2.14

$T$  = structure's fundamental natural period

$S$  = soil profile coefficient

$W$  = the total dead load tributary to the point under consideration in pounds

$R_w$  = working stress level response modification coefficient

Previously,  $C$  was a function of the fundamental period of vibration of the building and the defined spectral shape. The base shear equation was the same in the 1985 UBC and the ASCE 7-88, but the ASCE 7-88 used a different equation for  $C$  (Equation 2.16).

$$C = \frac{1}{15 T^{1/2}} \quad \text{Eqn 2.16}$$

The Blue Book maintained the same definition but published an equation differing from previous codes (Equation 2.15). Appendix A provides the process of converting the base shear value from equations 2.13 to 2.14. The final  $R_w \sim K$  relationship was determined as:

$$R_w = \frac{5.1}{K} \quad \text{Eqn 2.17}$$

#### ***2.4.2 Quantifying the Structural Response Modification Factors, ATC-19***

ATC-19 began in 1986 with funding from the NSF. Then, in 1991, additional funding from the National Center for Earthquake Engineering Research (NCEER) was used to expand the objectives of the project (ATC, 1995). The full list of project objectives was (ATC, 1995)

1. document the basis for the values assigned to  $R$  factors used in U.S. model building codes
2. review the role of  $R$  factors in seismic design in the U.S.
3. present “state-of-knowledge” on  $R$  factors
4. propose procedures for improving the reliability of  $R$  factors
5. document the use of response modification factors in other countries
6. provide a rational definition of  $R$  using key components defined by state-of-knowledge information
7. provide a framework and methods for defining the key components of  $R$
8. recommend research needed to improve the reliability of construction design using  $R$  factors

The final document was published in 1995 and fulfilled all 8 project objectives. The result of objective 6 was separating the  $R$  factor into three key components related to the structural system (Equation 2.18). The three factors, strength, ductility and redundancy, depended on characteristics of the particular structural system. Pushover analysis results indicated key characteristics of structural system behavior. The key points were the yield displacement, yield strength and strength at the maximum considered displacement. Another consideration of the structural system properties was inherent damping. Though this factor,  $R_\xi$ , was not included in the final calculation of  $R$ , ATC-19 recommended damping as a design consideration.

$$R = R_s R_\mu R_R$$

Eqn 2.18

Where:  $R$  = strength level response modification coefficient

$R_s$  = period dependant strength factor as defined in section 2.4.2.1.1

$R_\mu$  = period dependant ductility factor as defined in section 2.4.2.1.2

$R_R$  = redundancy factor as defined in section 2.4.2.1.3

### 2.4.2.1 Pushover Analysis

Pushover analysis, also called monotonic or quasi-static analysis, tests a structure using prescribed displacement increments until the test reaches failure or the maximum possible displacement. The results of a pushover analysis produce a curve, similar to the solid line seen in Figure 2-7. An approximation of the yield point can be obtained by creating a bi-linear approximation of the pushover results, using the equal energy method. The equal energy method balances areas between the pushover results and the bi-linear approximation. Figure 2-7 shows an example bi-linear approximation equating Areas 1 and 2.

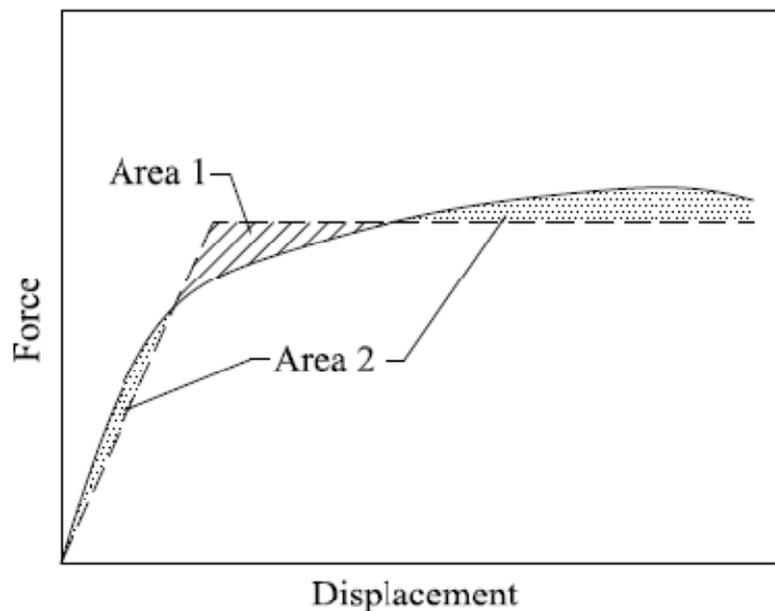
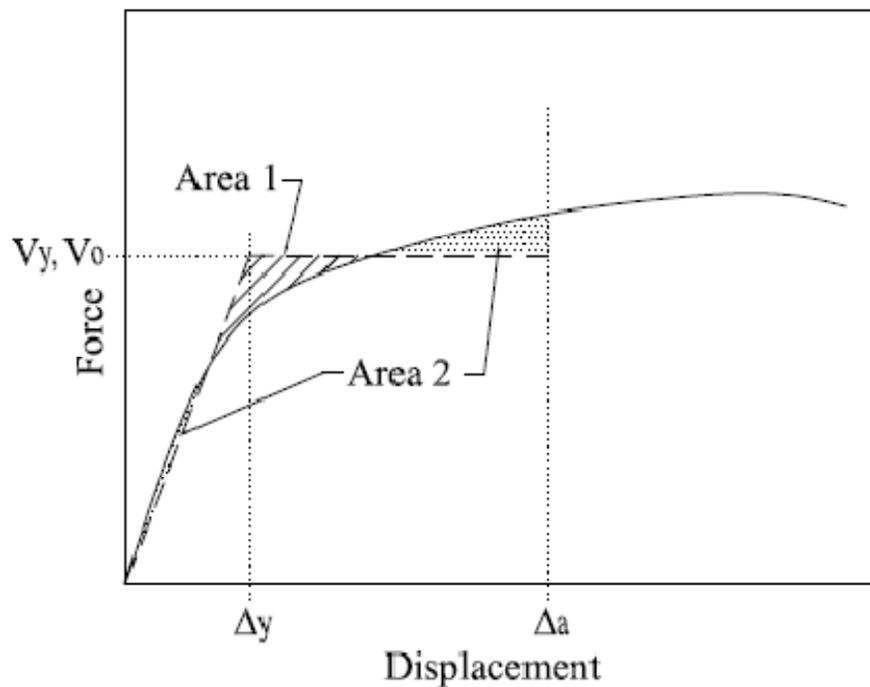


Figure 2-7 Sample pushover analysis results showing bi-linear approximation using equal energy method

To generate a bi-linear approximation of results, a maximum displacement must be considered. In Figure 2-7, the maximum displacement was the end of the test, and was limited by either the testing equipment or the failure displacement of the specimen. ATC-19 recommends setting a maximum displacement equal to the allowable inelastic deflection set forth by model building codes, as shown in Figure 2-8. The bi-linear approximations are adjusted accordingly, still keeping the areas above and below the pushover results line equal. The bi-linear approximation determines the key points needed for the  $R$  calculation: the yield displacement,  $\Delta_y$ ; yield strength,  $V_y$ ; and strength at the maximum considered displacement,  $V_o$ .  $V_y$  and  $V_o$  are considered equal because of the shape of the bi-linear approximation.



**Figure 2-8 Sample pushover analysis results showing bi-linear approximation using equal energy method and consideration of allowable drift**

#### **2.4.2.1.1 Period-dependant Strength Factor, $R_s$**

The strength factor,  $R_s$ , is period dependant and is the ratio of the lateral strength at the maximum considered drift,  $V_o$ , to the required strength,  $V_d$  or design base shear (see Equation 2.19). ATC-19 related the  $R$  factors published in ATC-3 06 to quantifiable structural system

characteristics. When ATC-3 06 was published, the base shear equation was Equation 2.5 (page 14) so it was used to determine  $V_d$ , the design base shear.

$$R_s = \frac{V_o}{V_d} \quad \text{Eqn 2.19}$$

Where:  $R_s$  = period dependant strength factor

$V_o$  = available lateral strength (Figure 2-8)

$V_d$  = required lateral strength determined from 1971 UBC base shear equation (Eqn 2.13)

#### 2.4.2.1.2 Period-dependant Ductility Factor, $R_\mu$

The ductility factor,  $R_\mu$ , is period dependant and based on the ductility ratio. The ductility ratio is the ratio of the yield displacement to the allowable displacement or maximum considered displacement. Several teams of researchers, such as Miranda and Bertero or Nassar and Krawinkler, each developed methods to determine the period dependant ductility factor from the ductility ratio and the fundamental period of the structure. All methods produced similar results, so the method selected for determining  $R_\mu$  has no significant effect on the outcome of  $R$ . Equations 2.20 through 2.23, developed by Miranda and Bertero in 1994, present one method for rock, alluvium or soft soil sites (ATC, 1995).

$$R_\mu = \frac{(\mu-1)}{\phi} + 1 \quad \text{Eqn 2.20}$$

$$\phi = 1 + \frac{1}{10T-\mu T} - \frac{1}{2T} e^{-1.5 (\ln T - 0.6)^2}, \text{ for rock sites} \quad \text{Eqn 2.21}$$

$$\phi = 1 + \frac{1}{12T-\mu T} - \frac{2}{5T} e^{-2 (\ln T - 0.2)^2}, \text{ for alluvium sites} \quad \text{Eqn 2.22}$$

$$\phi = 1 + \frac{T_g}{3T} - \frac{3T_g}{4T} e^{-3 (\ln T/T_g - 0.25)^2}, \text{ for soft soil sites} \quad \text{Eqn 2.23}$$

where:  $\mu$  = ductility ratio,  $\Delta_a/\Delta_y$

$T$  = fundamental natural period of the structure

$T_g$  = predominant period of the ground motion

#### 2.4.2.1.3 Redundancy Factor, $R_R$

The number of moment frames, braced frames or shear walls of equivalent strength and deformation capacity in each orthogonal direction of the LFRS of the structure characterizes the redundancy of a structural system (ATC, 1995). A system with little redundancy in the LFRS would have a lower redundancy factor than that of a system with greater redundancy. The more redundant a structural system, the higher the redundancy factor.  $R_R$  cannot, however, be larger than one. ATC-19 proposed draft values of the redundancy factor. ATC-19 published  $R_R$  values to encourage research and thought on the effects of redundancy on the behavior of a structural system under lateral seismic loads. The committee did not intend the values to be used in design (ATC, 1995) because the effects of redundancy had not been studied in depth.

**Table 2–5 Draft redundancy factors, ATC-19, 1995**

<i>Lines of Vertical Framing</i>	<i>Draft Redundancy Factor</i>
2	0.71
3	0.86
4	1.00

#### 2.4.2.1.4 Damping Factor, $R_\xi$

Damping refers to the amount of energy dissipation in a structure, not considering whether the damping occurs from hysteretic behavior or viscous damping devices installed in the system. Damping occurring from hysteretic behavior, termed natural damping or inherent damping, is typically assumed to be 5% equivalent viscous damping. Viscous damping comes from damping devices within the structural system. Research indicates that most steel frames have natural damping of about 5% and shear walls have natural damping of about 7% to 8%. Type and arrangement of interior and exterior columns not part of the LFRS can affect the natural damping in a structural system. Since elements outside the LFRS can affect the inherent damping in a structure, ATC-19 recommended against altering the  $R$  factor of a structural system due to natural damping. Altering  $R$  for supplemental or viscous damping is acceptable. ATC-19 recommended some values for reducing the design base shear due to viscous damping.

### 2.4.3 Adjustments to Seismic Provisions in Model Building Codes

The UBC adopted the methods of the 1988 Blue Book in its 1990 edition. This method remained in the 1994 UBC. The adoption of the strength level lateral seismic force came with the 1997 UBC. The 1997 UBC was the first code to show earthquake load combinations with both horizontal and vertical earthquake forces. The redundancy factor, recommended by ATC-19, was termed a reliability/redundancy factor,  $\rho$ , for determining the horizontal load effects.  $\rho$  increased the horizontal component of the design earthquake forces for elements in the structure not part of the LFRS. The story shear resisted by any single element in the story was determined  $\rho$ . By placing the redundancy factor outside of the  $R$  factor, the UBC provided constant  $R$  factors for all buildings of a particular structural system and still accounted for the effects of redundancy in determining of the required element strengths. The redundancy factor was not to be less than 1.0 or more than 1.5.

The base shear equation published in the 1997 UBC differed from the equation of ATC-3 06 and the 1985 NEHRP Provisions. It used the soil profile to determine a seismic coefficient, which was based on the seismic zone as in previous editions and not the ground motion parameters of ATC-3 06. The 1997 UBC still considered building importance in the base shear calculation, and did not address seismic performance categories (SPCs), as had been suggested by ATC-3 06. Equations 2.24 through 2.26 show the base shear equation of the 1997 UBC.

$$V = \frac{C_v I}{R T} W \quad \text{Eqn 2.24}$$

$$0.11 C_a I W \leq V \leq \frac{2.5 C_a I}{R} W, \text{ seismic zones 1-3} \quad \text{Eqn 2.25}$$

$$\frac{0.8 Z N_v I}{R} W \leq V \leq \frac{2.5 C_a I}{R} W, \text{ seismic zone 4} \quad \text{Eqn 2.26}$$

Where:  $C_v$  = seismic coefficient, based on seismic zone

$I$  = seismic importance factor

$T$  = elastic fundamental period of vibration, in seconds, of the structure in the direction under consideration

$$T = C_t (h_n)^{3/4}$$

$C_t$  = numerical coefficient, similar to those set forth in ATC 3-06

$h_n$  = height in feet above the base to the highest level of the structure

$C_a$  = seismic coefficient, lower bound for base shear

$N_v$  = near source coefficient for seismic zone 4

## **2.5 Current Provisions, IBC and ASCE 7 (1998-2009)**

The ICC began in 1997 with the merger of the ICBO, BOCA and SBCCI. In 2000, the ICC published the first code resulting from the merger, the 2000 edition of the International Building Code (IBC). The earthquake regulations of the IBC 2000 differed from the 1997 UBC and were based on the 1997 NEHRP Provisions. The ASCE 7-98 had incorporated many of the recommendations of the ATC-3 06 and subsequent NEHRP documents. Some states, such as California, still imposed stricter requirements, or referenced other standards for determining earthquake forces, rather than adopting the IBC at the local level. California adopted the *Building Construction and Safety Code 2003*, NFP 5000-03 as the model building code for seismic design, but reversed the decision in 2006 when the state adopted the IBC as the model building code.

ASCE 7-98 based seismic base shear on the 1997 *NEHRP Provisions*. Beginning with the 2006 IBC, the ASCE 7-05 is adopted by reference for determining earthquake forces, though some information is still contained in the IBC. The IBC 2006 offers an alternative method for determining site class, but permits the user to forgo its requirements if the ASCE 7-05 is used. It is expected that future model building codes will adopt the ASCE 7 by reference without repeating information.

### **2.5.1 2000 IBC**

The 2000 IBC differed considerably from the 1997 UBC. The most significant changes from the 1997 UBC to the 2000 IBC were:

1. adopting seismic design ground motion parameters,  $S_{DS}$  and  $S_{D1}$
2. replacing the seismic zone map with the spectral acceleration maps
3. using the 2500-year return period on the maximum considered earthquake, making the probability that a structure will fail in an earthquake equal for all parts of the country
4. eliminating the near source factor for seismic zone 4
5. changing from Seismic Zones 1-4 to Seismic Design Categories (SDC) A-F, with F being the most severe

6. requiring the level of detailing in a structure be determined as a function of soil characteristics at the site
7. changing the values of the Importance Factor,  $I$
8. adjusting the values of the Response Modification Factor,  $R$

Many of the changes did not affect the outcome of the design base shear. The use of the SDC required higher levels of detailing for some locations and limited the use of some structural systems to areas of lower seismic hazard. The changes to the  $R$  and  $I$  factors yielded higher base shears.

The redundancy/reliability factor introduced in the 1997 UBC was included in the 2000 IBC, again so that  $R$  factors were constant for each building of a particular structural system. The  $R$  factors published in the 2000 IBC were slightly different from those of the ASCE 7-98. Table 2–5 compares the  $R$  factors. Equation 2.27 shows the IBC 2000 seismic base shear equation and is similar to that of the 1997 UBC. The calculation of the seismic coefficient,  $C_s$  (equations 2.28 through 2.30) is the main difference in the base shear calculation.

$$V = C_s W \quad \text{Eqn 2.27}$$

$$C_s = \frac{S_{DS}}{\left[\frac{R}{I_E}\right]} \quad \text{Eqn 2.28}$$

$$0.044 S_{DS} I_E \leq C_s \leq \frac{S_{D1}}{\left[\frac{R}{I_E}\right]T}, \text{ for SDC A-C} \quad \text{Eqn 2.29}$$

$$\frac{0.50 S_1}{\frac{R}{I_E}} \leq C_s \leq \frac{S_{D1}}{\left[\frac{R}{I_E}\right]T}, \text{ for SDC D-F} \quad \text{Eqn 2.30}$$

Where:  $I_E$  = the occupancy factor

$R$  = the response modification coefficient

$S_{DS}$  = design spectral response acceleration at short period

$S_{D1}$  = design spectral response acceleration at 1.0s period

$S_1$  = maximum considered earthquake spectral response acceleration at 1.0s period

$T$  = fundamental period of the building

$T = 0.1 N$ , for moment-frame structures not exceeding

12 stories with a minimum story height of 10

feet

$$T = C_T h_n^{3/4}, \text{ for any structure}$$

$N$  = number of stories

$C_T$  = building period coefficient, equivalent to 1997 UBC values

$h_n$  = the height in feet above the base to the highest level of the structure

### **2.5.2 2003-2009 IBC**

The seismic regulations of the 2003, 2006 and 2009 editions of the IBC did not change significantly from its first edition in 2000. The section for earthquake forces was rearranged, but the only noticeable difference in calculations is the reliability/redundancy factor. Now,  $\rho$  is a constant equal to 1.0 or 1.3 depending on the SDC and the structural element under consideration. Elements receiving amplified loads due to iterative analysis, like P-delta effects, need not amplify the load a second time to account for redundancy. Structures in SDC E, D or F must use  $\rho$  of 1.3 unless the LFRS design meets specific requirements. Secondly, structural systems are now assigned alphanumeric identifiers to make referencing easier, as there are multiple types of light-framed wood shear walls or steel braced frames. Additionally, the 2006 IBC directly references the ASCE 7, which is now accepted as the national standard for minimum design loads for buildings.

### **2.6 FEMA 350/ATC-63 (2009- present)**

In September of 2004, FEMA awarded a contract to ATC to recommend method for quantifying the building system seismic performance factors and response parameters used in seismic design. ATC-63 developed a new methodology based primarily on previous methods for seismic design, prescribed in *Tentative Provisions for the Development of Seismic Regulations for Buildings, ATC-3 06* and the *Recommended Provisions for Seismic Regulations for New Buildings and Other Structures, 2003*. The committee refined previous methods by reviewing relevant research on collapse simulation, nonlinear response of structures, benchmark studies, expert feedback and evaluations of additional structural systems (ATC, 2009).

As stated previously, current values of  $R$  listed in NEHRP and ASCE 7-05 use the judgment of designers and limited qualitative comparisons to other similar systems. The NEHRP Provisions include more than 75 structural systems, each with an assigned  $R$  factor based on

expert judgment, but many have never been tested or evaluated after major seismic events. ATC-63 provides a rationale for establishing global seismic performance factors including the response modification coefficient,  $R$ , the deflection amplification factor,  $C_d$ , and the system over strength factor,  $\Omega_o$ .

The ATC-63 committee intended the seismic coefficients published in the model building codes be determined using the methodology to prevent structural collapse in a seismic event equivalent to the maximum considered earthquake. Life safety performance is the primary concern. Performance based seismic design (PBSD) is not evaluated using the methodology of ATC-63 because PBSD is outside the life safety intention of the building code. The methodology applies directly to new building structures, but conceptually to non-building structures as well.

Engineers must use parameters and equations of the current seismic provisions, given in the ASCE 7-05, to test structures using the ATC-63 methodology. The ASCE 7-05 and the *NEHRP Provisions* defined the global seismic performance factors as follows in equations 2.31 through 2.33. (Figure 2-9 illustrates the definitions.)

$$R = \frac{V_E}{V} \quad \text{Eqn 2.31}$$

$$\Omega_o = \frac{V_{max}}{V} \quad \text{Eqn 2.32}$$

$$C_d = \frac{\delta}{\delta_E} R \quad \text{Eqn 2.33}$$

where:  $V_E$  = base shear which would develop in the seismic force resisting system if the structure remained entirely linearly elastic for design earthquake ground motions

$V$  = design seismic base shear

$V_{max}$  = maximum strength of the fully yielded system

$\delta_E$  = displacement at the elastic base shear

$\delta$  = allowable story drift limit as defined by the ASCE 7-05



amplification not the  $R$  factor. Since damping can affect the displacement of the structure and determining the true damping in the structure is difficult without full-scale testing, the ATC-63 committee recommended using the  $C_d$  values listed in the ASCE 7-05 for design.

$$R = \frac{S_{MT}}{1.5C_s} \quad \text{Eqn 2.34}$$

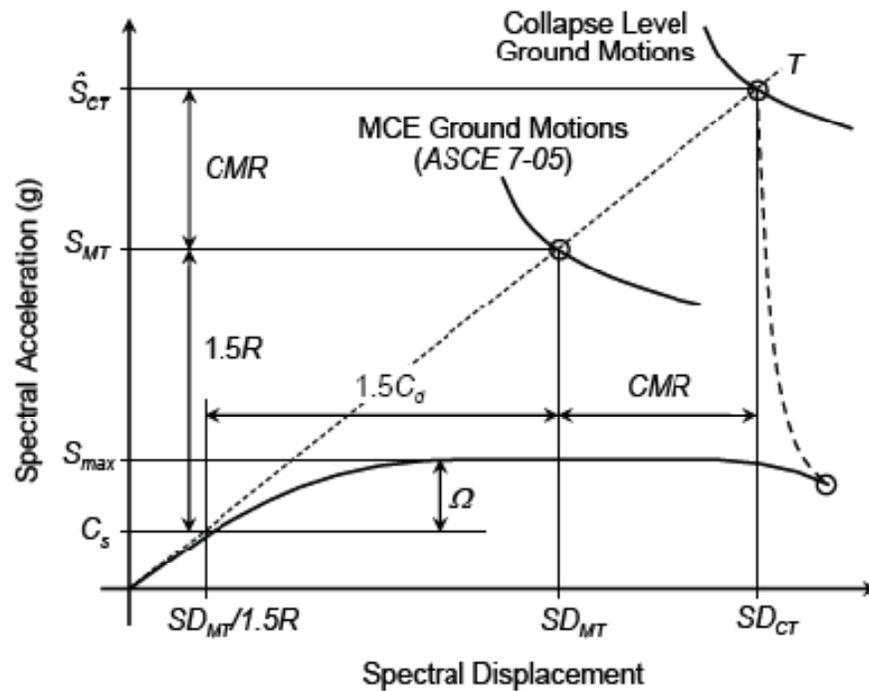
$$\Omega = \frac{S_{max}}{C_s} \quad \text{Eqn 2.35}$$

$$C_d = R \quad \text{Eqn 2.36}$$

Where:  $S_{MT}$  = maximum considered earthquake spectral acceleration at the period of the system

$S_{max}$  = represents the maximum strength of the fully-yielded system (normalized by the effective seismic weight)

$C_s$  = seismic response coefficient



**Figure 2-10 Illustration of seismic performance factors ( $R$ ,  $C_d$  and  $\Omega$ ) as defined by the ATC-63 methodology. Republished courtesy of FEMA; originally published in FEMA P695.**

The methodology proposed in ATC-63 requires testing under pushover and cyclic loading of different archetypes or configurations of a structural system. The selection of archetypes will affect the outcome of the tests. The archetypes should represent all probable configurations of framing, all construction details and all material property variations. Developing several archetypes is critical to developing an  $R$  that represents the most likely behavior of a structural system. Without proper archetypes, the  $R$  factor may not be representative of true system behavior for some cases.

The shape of the backbone curve on the hysteretic diagram and the strength at the maximum considered displacement for each archetype are factors in determining  $R$  for the system in question. The methodology requires calculating the design base shear using the most recent model building code, assuming a trial value of  $R$  for the system. The next step is developing a collapse margin ratio (CMR) and an adjusted collapse margin ratio (ACMR) defined below in section 2.6.1, for the archetype, using the test results and the trial design base shear. If the ACMR is 1.5 or more, the trial value of  $R$  is acceptable and the over strength factor,  $\Omega$ , and displacement amplification factor,  $C_d$ , can then be evaluated. (ATC, 2009)

### **2.6.1 Adjusted Collapse Margin Ratio, ACMR**

ATC-63 requires an ACMR within a required range to establish a new response modification factor for a structural system. The ACMR is an adjusted value of the CMR. The CMR is the ratio of collapse-level ground motions to the maximum considered ground motions. Ground motions rated as “collapse-level” are higher than the maximum considered earthquake ground motions defined in the ASCE 7-05. ATC (2009) defines the CMR as (Equation 2.37):

“Ratio of the median five percent damped spectral acceleration (or displacement) of the collapse level ground motions to the five percent damped spectral acceleration (or displacement) of the MCE ground motions, at the fundamental period of the seismic-force-resisting system”

$$CMR = \frac{\bar{S}_{CT}}{S_{MT}} = \frac{S_{D_{CT}}}{S_{D_{MT}}} \quad \text{Eqn 2.37}$$

Where:  $S D_{CT}$  = spectral displacement at maximum considered earthquake ground motions

$S D_{MT}$  = spectral displacement at collapse level earthquake ground motions

Many factors influence the collapse margin, such as ground motion variability, level of detailing in design and construction variability. To evaluate a system, the methodology requires adjusting the CMR for the influence of these factors. The multiplication of the CMR by the spectral shape factor (SSF) adjusts the CMR to the ACMR. The SSF depends on the archetype's period of vibration or the ductility ratio,  $\mu$ , of the archetype. The definition for both the period of vibration and the ductility ratio in ATC-63 are the same as in ATC-19. The tests must develop an ACMR for each specimen, and proposed an overall ACMR based on the results from each archetype.

Before accepting the overall ACMR for a structural system, a peer review panel must evaluate the results for uncertainty. After the panel accepts the ACMR, if it is within the required range, the panel considers the  $R$  factor used in the trial calculation of base shear acceptable as well.

The methodology in ATC-63 requires extensive physical and computer modeling. ATC-63 requires review by a panel of all steps in developing archetypes, constructing test models, and calculating CMR, ACMR, and design base shear before starting the next step in the process. The amount of work required to generate or validate the  $R$  factors for any structural system is extensive and the generated values may still be conservative, depending on the value of the ACMR and the reliability of the testing results.

The ATC-63 methodology requires more extensive testing to develop  $R$  than the previous definition published in ATC-19. However, the ATC-19 definition is still valid to develop a basis of understanding for system behavior. Until the testing can be completed the ATC-19 definition will remain in use and the  $R$  values published in the ASCE 7 will reflect this definition.

**Table 2-6 Comparison of *R* through code updates (1978-2005)**

Type of Structural System	ATC-3 06 (1978)	NEHRP (1985)	BSSC (1993)	UBC - $R_w$ (1994)	UBC (1997)	IBC (2000) NEHRP (1997)	ASCE 7 (1998) (2000)	IBC (2006) ASCE 7 (2005) *
Bearing Wall Systems								
Light framed walls with shear panels	6.5	6.5	6.5	6	4.5	6.5	6	A.13 6.5
Reinf concrete shear walls	4.5	4.5	4.5	6	4.5	4	4	A.2 4
Reinf. masonry shear walls	3.5	3.5	3.5	6	4.5	2	2	A.9 2
Unreinf/ partially reinf masonry shear walls	1.25	1.25	1.25	-	-	1.5	1.5	A.11 1.5
Building Frame Systems								
Light framed walls with shear panels	7	7	7	7	5.0	7	6.5	B.23 7
Reinforced concrete shear walls	5.5	5.5	5.5	8	5.5	5	5	B.6 5
Reinforced masonry shear walls	4.5	4.5	4.5	8	5.5	2.5	2.5	B.19 2
Braced frames	5	5	5	8	5.6	5	5	B.4 3.25
Unreinf/partially reinf masonry shear walls	1.5	1.5	1.5	-	-	1.5	1.5	B.21 1.5
Moment Resisting Frame System								
Special steel moment frames	8	8	8	12	8.5	8	8	C.1 8
Special reinforced conc. moment frames	7	8	8	12	8.5	8	8	C.5 8
Ordinary steel moment frames	4.5	4.5	4.5	6	4.5	4	4	C.4 3.5
Ordinary reinforced conc. moment frames	2	2	3	5	3.5	3	3	C.7 3
Dual System								
Reinforced concrete shear walls	8	8	8	12	8.5	7	7	D.3 7
Reinforced masonry shear walls	6.5	6.5	6.5	8	5.5	7	7	D.10 5.5
Wood sheathed shear panels	8	8	8	-	-	-	-	-
Braced frames	6	6	6	10	6.5	8	6	D.2 7

\* Equivalent system indicated by associated number from Table 12.2-1 in ASCE 7-05

### 3 Log Shear Wall Construction Practices

Log Builders have used many different methods to construct log structures. A log structure's primary structural elements are formed by a system of structural logs supporting floor and/or roof systems. Log walls, defined as an assembly of individual structural-logs for use as an exterior or interior load bearing wall, shear wall or non-load bearing wall, make up most of the load bearing system in a stacked log structure. Stacked log structures use many different wood species and log types. Log walls are reinforced with a variety of log-log connectors and can be anchored to the foundation in different ways. Many of the current construction methods and practices are assumed adequate and many will be reviewed here. The behavior of a log shear wall under lateral seismic load depends on many construction factors, from type of wood, shape and condition of log, number of courses, type and spacing of log-log connectors, number of openings, reinforcement around openings and method of foundation anchorage. This chapter describes current log construction practices. Chapter 4 will provide an overview of shear wall behavior based on recent physical research.

The ICC's IBC and the American Forest and Paper Association (AFPA) *National Design Specification for Wood Construction* (NDS) contain requirements for conventional wood framing. However, neither the IBC nor the NDS specifically address the design of log shear walls. Thus, the design and construction of log shear walls varies across the U.S. In 2007, ICC published a standard for the design and construction of log buildings, the ICC 400-2007, *Standard on the Design and Construction of Log Structures*, herein referred to as *The Standard*. Prior to *The Standard*, no document written for adoption into local building codes provided parameters for the design or construction of log shear walls. *The Standard* is in a format similar to the AFPA's *Wood Framed Construction Manual for One- and Two- Family Dwellings* and does not limit the designer to one specific construction method; it does cover design requirements for the most common methods. *The Standard* refers the user to the IBC, NDS and other code documents for several design parameters. The following sections discuss the design parameters provided in *The Standard* as they apply to the construction variables affecting the behavior of a log shear wall.

### **3.1 Structural Log Characteristics**

The strength and stiffness of a log shear wall depends not only on construction methods, but also on characteristics of the particular structural logs used. A structural log, used in shear wall construction, is a wood member of any shape and size that have been visually graded and grade marked by an accredited inspection agency and they are stacked horizontally or laid-up vertically to form solid-wood walls (TFBC, n.d.). Glued laminated, edge-glued and/or finger jointed members may be considered structural logs. Given the above definition, many different types of structural logs are available and each have factors that affect building behavior.

The most influential characteristics of structural logs that affect shear wall behavior include log species and grade, moisture content, and log profile. Log species and log profile greatly affect strength and stiffness of the structural log. The moisture content of logs will affect the friction between courses in shear walls (Scott, Leichti, & Miller, 2005), the longevity of the structure and the shrinkage of individual members.

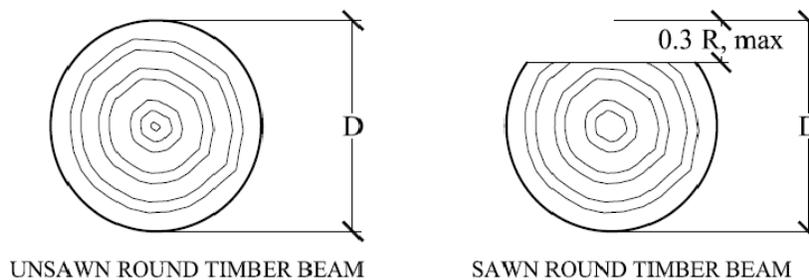
#### ***3.1.1 Log Species and Grade***

In the U.S., log home construction uses more than 70 species of wood (Woodard, 2005). Each geographical area of the country has its own most common species. White Cedar, Lodge Pole Pine, Cypress and Western Red Cedar are most commonly used in construction in the U.S. (Woodard, 2005). Many logs used other parts of the world are imported from the U.S. (Yeh, Chiang, & Lin, 2006). According to a survey completed in Taiwan in 1999, the preferred species is Western Red Cedar because of its durability, machining quality and acquisition cost (Yeh, Chiang, & Lin, 2006). ASTM D 2555 lists the species approved by ICC for log wall construction in *The Standard*, Section 302.2.1.2 (ICC, 2007). Table 302.2(3) of *The Standard* gives design values for various species (ICC, 2007).

Each log species has different strength characteristics and material properties. The modulus of elasticity,  $E$ , varies from a low value of 700,000 psi in grade 3 Aspen, to a high value of 1,700,000 psi in Beam and Header graded Longleaf Pine (ICC, 2007). Allowable stress values vary from species to species depending on grade. Like rectangular sawn lumber in the NDS, allowable stress values for bending, tension parallel to grain, shear parallel to grain, and compression parallel and perpendicular to grain are all tabulated for structural logs. Logs are visually graded according to the requirements of ASTM D 3957 (Green, Gorman, Evans, &

Murphy, 2004). Each grade is delineated by the size and number of growth (strength-reducing) characteristics visible at the time of grading (Breyer, Fridely, Cobeen, & Pollock, 2007).

Structural-log grades fall into two main categories: sawn and unsawn round timber beams (SRTB) and wall logs. The Timber Framing Business Council (TFBC) defines SRTBs as round timbers, shaved or sawn along one side and normally loaded on their flat surface, stressed primarily in bending (TFBC, n.d.). *The Standard* states that the depth of material removed to create the flat surface of the SRTB may not exceed more than three-tenths of the radius of the log at any point (see Figure 3-1; ICC, 2007). SRTBs have greater bending capacity than wall logs and are not often used in a continuous support condition or walls. Some construction conditions may require removing more than three-tenths of the log radius from one side or cutting along multiple sides. These logs are classified and graded as “wall logs” (Pickett & Burke, 2008). For grading, the cross section area of a wall log is the size of the inscribed rectangle (Pickett & Burke, 2008). Wall logs are defined by the TFBC as wood members that are normally stacked horizontally or laid-up vertically to form a load-bearing, solid-wood wall in any building (TFBC, n.d.). *The Standard* permits use of wall logs in locations primarily under bending stress, like SRTBs, but the bending stress design values for wall log grades are lower than SRTBs bending stress values.

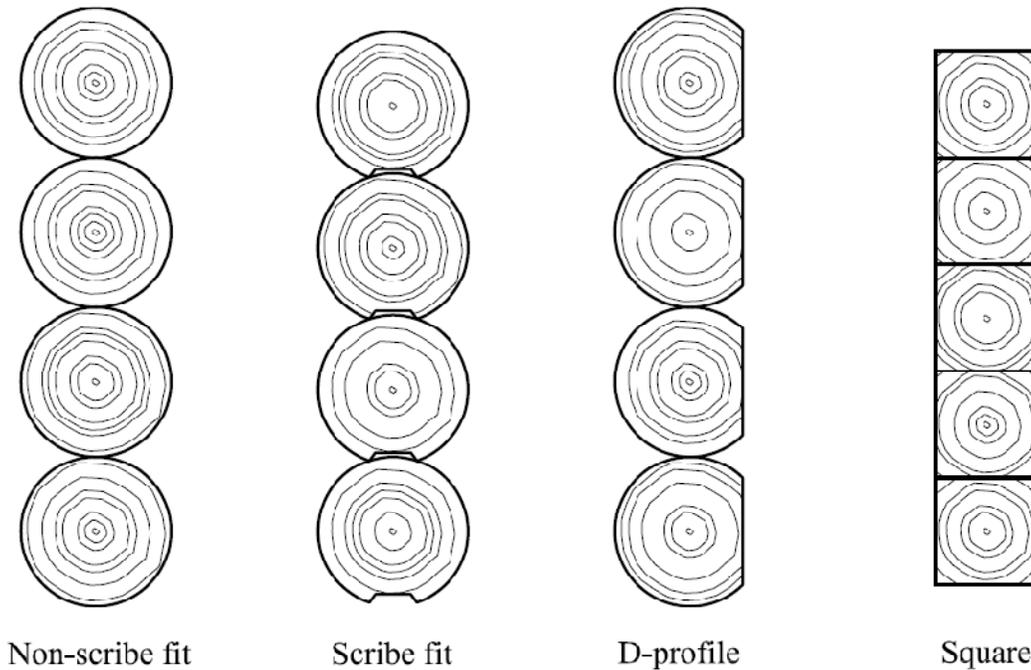


**Figure 3-1 Profiles of timber beam, as defined in ICC 400-2007**

### 3.1.2 Log Profile

Log profiles have many variations. *The Standard* states that any log may be peeled, notched, hewn, sawn, milled or otherwise profiled into a final dimensioned form for installation (ICC, 2007). While log profiles within the same structure need not be equivalent, section 302.2.1.4 of *The Standard* requires that an average profile(s) be established and dimensioned (ICC, 2007).

Log profiles determine how the log shear wall fits together. The amount of log profile that bears on the lower course affects the fit of log shear walls. Significant energy dissipation in shear walls under lateral seismic load occurs through friction between courses. Log shear walls with a tighter fit, typically achieve higher friction resistance and thus have more energy dissipation during initial slip. Today, logs are most commonly either hand hewn or machined for fitting. The traditional log profiles are scribe-fit or non-scribe fit round logs. Square profiles are becoming more common for modern log structures. Notching between courses and at wall intersections is done to ensure tight fit and allow for settlement and shrinkage. Each manufacturer will have standard options for wall joinery and notching within the logs based on their machining capabilities. Figure 3-2 shows basic log profiles but is not an extensive list. Each of the profiles in Figure 3-2 could be achieved with either hand hewing or machining. Hewing or machining of the logs is changing from a field operation to a more controlled shop environment. Many manufacturers will pre-build whole structures, or walls, within the shop for fitting, then disassemble the structure before transporting it to the field. Labels according to exact log location within the building ensure placement of each log in the fitted position to facilitate field erection.



**Figure 3-2 Common log profiles**

### ***3.1.3 Moisture Content/Settlement***

The moisture content of the logs affects the behavior of the structure and the fit of the structure a few seasons after construction. The Timber Frame Business Council (TFBC, n.d.) and *The Standard* define moisture content as “the weight of water in the cell walls and cavities of wood, expressed as a percentage of oven-dry weight.” *The Standard* addresses moisture content in section 302.2.2 (ICC, 2007).

The biggest affect moisture content has in a structure subjected only to gravity loads is shrinkage or settlement. Radial shrinkage is much larger than longitudinal shrinkage. As the structure ages, moisture leaves the logs, causing each course to shrink. Because of the varying moisture content in each log, not all courses will shrink evenly or equally. As the courses shrink, the structure will settle. *The Standard* addresses settlement in section 304 (ICC, 2007). Settlement due to moisture content is included in  $\Delta_{SL}$  or settlement due to slump. The design of connections and openings must account for settlement of the log structure. Without accounting

for settlement of the structure, the fit will change over the structure's lifetime, thus changing shear wall behavior.

Moisture content will affect the coefficient of friction of the log surface. The coefficient of friction value is one of two factors in the friction force, which has been demonstrated to provide a significant amount of energy dissipation in log shear walls.

### **3.2 Shear Wall Reinforcing**

The strength, ductility, spacing and size of the connections used to tie one log course to the log course below it affect the behavior of a log shear wall under lateral seismic loads. Various types of rods, such as lag screws, and drift pins (steel and wood dowels), are very common in connecting courses in modern construction and have similar responses to seismic lateral load. Through-rods and post-tensioned through-rods are used as well, but many designers consider these as hold-down mechanisms to prevent overturning though they still contribute to the response of the shear wall under seismic lateral load. *The Standard* addresses general requirements for mechanical connectors in Section 302.3 and connection design requirements in Section 404 (ICC, 2007).

The strength of an individual mechanical connector or group of connectors connecting two or three members together under static loads can be determined by applying of the Yield-Limit equations and appropriate factors from Chapter 11 of the NDS. The NDS addresses group action factors, connection geometry factors, load duration factors and embedment factors. The group action factor, connection geometry factor and embedment factor are applicable to each individual connector type, but are not the same for each connector type. The type of load determines the load duration factor, so it is constant when considering different types of connectors. Lateral design of a wood structure is to be in accordance with the IBC 2006 using the fastener strengths provided in the NDS. The NDS provides the load duration factors in Section 2.3.2 (AFPA, 2005). As an example, dead loads (i.e. permanent loads) have a lower load duration factor because the load is permanent and the increased likelihood of creep. Seismic loads are assigned a load duration factor of 1.6 found in Table 2.3.2 of the NDS (AFPA, 2005). The NDS allows this increase because the load duration will be short.

It is yet to be determined how a large group of mechanical connectors over several different members will behave and yield under dynamic loads in a log shear wall. The NDS

addresses group action for typical connections, but does not consider friction between the connected members. Chapter 4 presents the behavior of reinforced shear walls and considers the effects of friction.

### ***3.2.1 Log-Log Connectors***

Log shear wall construction uses several types of connectors. Most connectors used today are steel versions of their wood predecessors, but some log home manufacturers still use the more traditional wood pins. The Yield Limit Equations apply to wood or steel dowel-type connectors, but the tabulated reference design values included in the NDS only apply to steel connectors (AFPA, 2005).

The factors applied to the yield limit equations are similar for all dowel-type log-log connectors (bolts, pins, screws). The spacing of the connectors is typically large and edge distance is usually not a concern. Because of the length of the connection, each log-log interface is somewhat similar to a sill plate to foundation connection. The group action factor does not apply to the connection of a sill plate to a foundation or slab because of the long spacing between connector, so it follows that the group action factor would not apply to log-log rod-type mechanical connections (Woodard, 2005). Moreover, detailing of connections enables the designers to meet spacing and end distance requirements without taking the geometry factor into account.

#### ***3.2.1.1 Bolt***

A bolt is a threaded metal rod with nuts used to fasten two pieces of wood together (TFBC, n.d.; ICC, 2007). Bolts must comply with ANSI/ASME B 18.2.1 per section 302.3.1 of *The Standard* (ICC, 2007). Anchorage to the foundation requires bolts to attach sill logs to a sill plate (Scott et al, 2005).

#### ***3.2.1.2 Lag Screw***

A lag screw must comply with the same ANSI/AMSE as a bolt (ICC, 2007). Lag screws are heavy wood screws with square or hexagonal heads threaded into the logs with a wrench. Lag screws are a very common connector in current design practices. The design strength and detailing requirements to ensure the geometry factor is equal to 1.0 are determined in the NDS (AFPA, 2005).

### **3.2.1.3 Drift Pin**

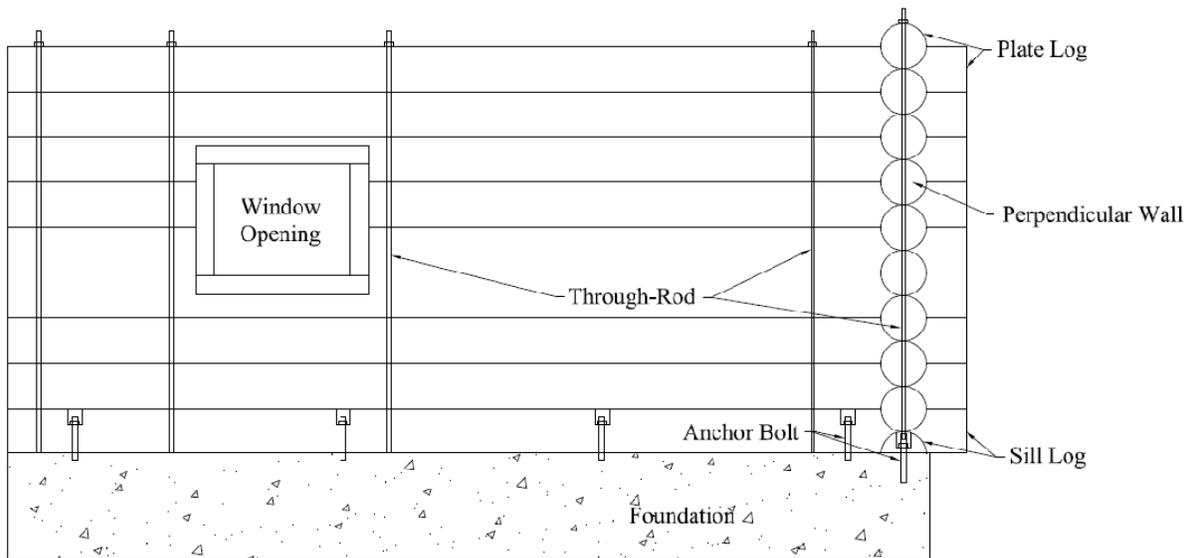
A drift pin or drift bolt is a steel pin of desired length driven through the logs above and into the one below. The holes are pre-dilled at a diameter slightly smaller than the pin's diameter (TFBC, n.d.) to ensure minimal slippage between the log and the connector. Drift connectors span only a single log-log shear plane.

Reductions in yield strength due to the group action factor and geometry factor do not apply to rod type connectors, but drift pins require another consideration. The lack of a bolt head, nut and washer, requires the reference design value be reduced by 25 percent (AFPA, 2005). All other considerations are the same as for bolts.

### **3.2.2 Hold-down Connectors**

Through-rods and post-tensioned through-rods are considered by most engineers as hold-down connectors not log-log connectors. The experimental results summarized in Chapter 4 demonstrate that these methods, if constructed properly, also maintain contact between logs through displacements, so that friction may be used for force resistance strength. Most designers consider this approach unconservative, and ineffective in high seismic areas because of the limited testing of the shear strength of a log shear wall without log-log connectors.

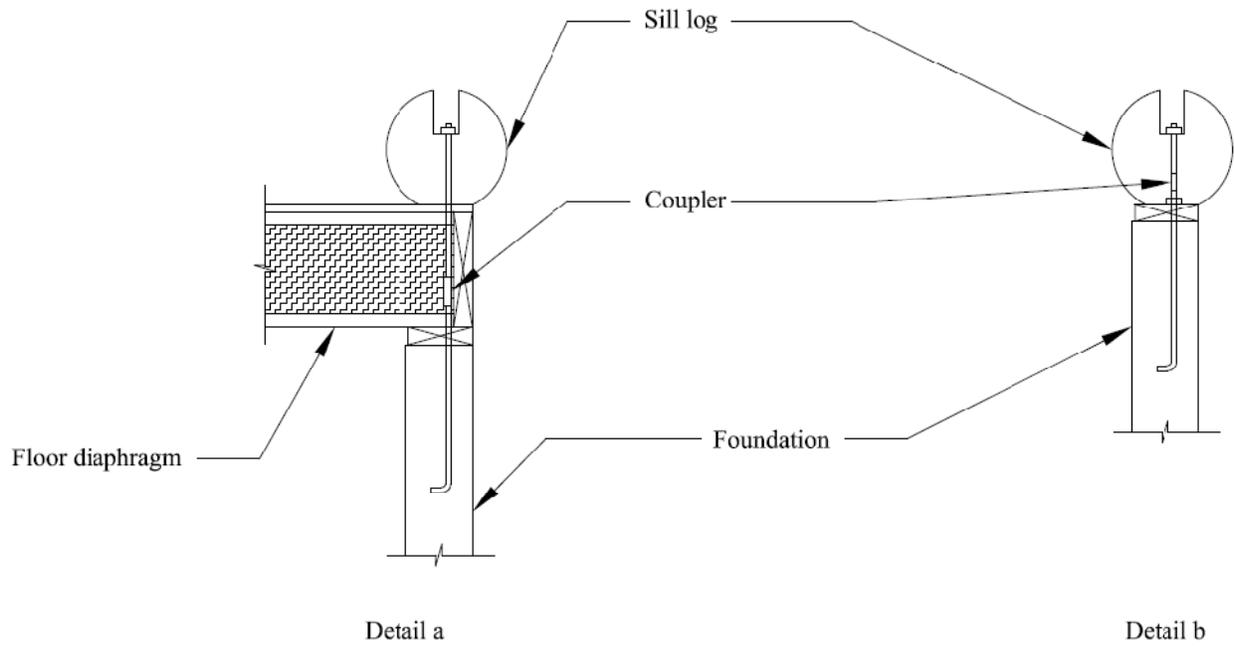
A through-rod is also known as a through-bolt, which the TFBC defines as a threaded metal rod, extending the full height of a wall, fastened at each end with nuts and washers (TFBC, n.d.). These rods provide overall rigidity within the wall but are very difficult to install (Phil Bachofner, personal communication, December 9, 2009). Through-bolts allow the builder to tighten the wall section as/after settlement or shrinkage occurs. Compression or take-up springs adjust for settlement at the top of the through-bolt as it occurs, but require regular maintenance to ensure proper function. The general requirements for log-log bolt connectors discussed in the previous section also apply to through-bolt connectors. Figure 3-3 shows a typical detail of a through-rod. Manufacturers can pre-tension a through-rod for shrinkage or settlement. This method is similar to post-tensioning in concrete slabs. The log shear wall is constructed normally, but before placement of the diaphragm on the top of the wall, the through-rods are tensioned and capped. The cap is recessed into the top log, similar to an anchor bolt head shown in Figure 3-3. As radial shrinkage and settlement occur, the tension in the through-rod maintains contact between log courses without adjustment.



**Figure 3-3 Through-rod detail showing typical placement at ends of wall and around opening**

### **3.3 Common Foundation Anchorage Methods**

Modern log wall construction uses two primary foundation anchorage methods: (1) placing the sill log directly onto the floor diaphragm (“detail a” in Figure 3-4) (Scott et al, 2005) and (2) placing the sill log on the foundation wall (“detail b” in Figure 3-4) (Scott et al, 2005). Foundation anchorage was thought to have a large affect on the behavior of the log wall system during lateral loading until testing completed by Scott in 2004 showed that both anchorage methods behaved similarly (Section 4.2.4). Figure 3-4 shows sketches of the two foundation anchorage methods. In both cases, it is possible to connect the foundation anchor rods to through-bolts in the wall with a coupler.



**Figure 3-4 Common foundation anchorage details**

## **4 Behavior of Log Shear Walls Subject to Lateral Seismic Loads**

A log shear wall typically serves as both the gravity and lateral load bearing system of a log structure. Interior columns and beams typically help support gravity loads for long spans, but are not designed to resist lateral forces. Because log shear walls serve to resist both vertical and horizontal forces, the combined loading will affect the behavior of the system under lateral loads. Log shear walls dissipate energy from lateral forces using a combination of three mechanisms, friction between courses, yielding of log-log connectors and bearing on log-log connectors. The way a log shear wall responds to lateral seismic loads determines the  $R$  factor. Currently, practicing professionals assume a certain behavior and a corresponding  $R$ . The majority of testing on log shear walls has been limited to the effects of a single variable on log shear wall behavior. The test results have not been compiled to form a single resource for the over-all behavior of a log shear wall.

Slip or failure zones for resisting lateral forces in log shear walls are located between courses. Gravity loads can mitigate the effects of shrinkage on friction when using through-rods as shear wall reinforcing. The gravity load maintains friction between the log courses even if the reinforcing no longer holds the courses together tightly. Force-displacement behaviors for lateral resistance depend on the construction details of log construction. Typically, log shear walls with through-rods installed to maintain friction exhibit an ascending load-displacement response before failure, i.e. increase strength because of the effect of wedging caused by log slip between courses. Log shear walls with log-log connectors also exhibit an ascending load-displacement behavior, but this behavior is the result of friction between log courses combined with yield strength of log-log connectors. Detailing the log shear wall to account for shrinkage and settlement of the courses is significant in wall performance because shrinkage and settlement can affect the stiffness of the shear wall. Each of the tests on log shear walls presented in this chapter represent the effect on behavior by changing one or more variables. This chapter presents briefly explains the expected behavior of a log shear wall and presents physical testing results.

### **4.1 Expected Behavior of Log Shear Walls**

Log shear walls are viewed by practicing professionals as a relatively ductile system with good redundancy. The in-plane behavior of a log shear wall LFRS is considered robust and

stable. The out-of-plane stability is the area of largest concern in standard practice, however this stability is maintained with proper construction. Engineers accept that a log shear wall will be capable of large in-plane deflections without sacrificing stability or load bearing capacity.

Engineers understand that friction will provide a large portion of the energy dissipation in a log shear wall, but do not include this in lateral strength calculations. The friction force is considered unreliable and unpredictable. Instead, designs utilize yielding in log-log connectors and a minimal amount of wood crushing for energy dissipation. Engineers expect openings to decrease the stiffness of the shear wall, similar to the behavior of light framed shear walls. Additionally, a graduated displacement from the sill log to the plate log is expected. Finally, log shear walls are not expected to return to their original position after an event; in some cases the residual deformation could be as much as the allowable story drift.

## **4.2 Shear Wall Behavior Demonstrated in Physical Testing**

Most testing took place in the Pacific Northwest, an area where log construction is more common. Taiwan has completed some testing because of the growing popularity of log homes in Asia. The following sections present experimental test methods and results from the available testing on the behavior of log shear walls.

### ***4.2.1 Gorman and Shrestha, Washington State University, 2002***

In 2002, Tom Gorman and Deepak Shrestha, at Washington State University, tested the seismic resistance of log shear walls. The log shear walls were eight feet long full height walls with ten-inch nominal log courses. The log-log surface between each course was tongue and groove. All logs were kiln-dried Ponderosa pine. Five different shear connector configurations were tested and evaluated. Each wall consisted of a half-log first course (anchored to the test frame), ten full log courses and a half-log top course. A 5/8-inch diameter threaded rod attached each sill log (half-log) to the test frame base through predrilled holes in the logs (1 1/4-inch diameter). No further attachment was made (through-rods did not extend into the test-frame base). The test frame anchor termination occurred at the fourth log with a nut and standard washer. Through-rods continued up from the fourth log to the top log. The through-rods started and terminated every fourth log in the same fashion as the anchors. The top half-log was attached

with a nut and take-up spring assembly to maintain rod tension as the logs shrank (Shrestha & Gorman, 2002).

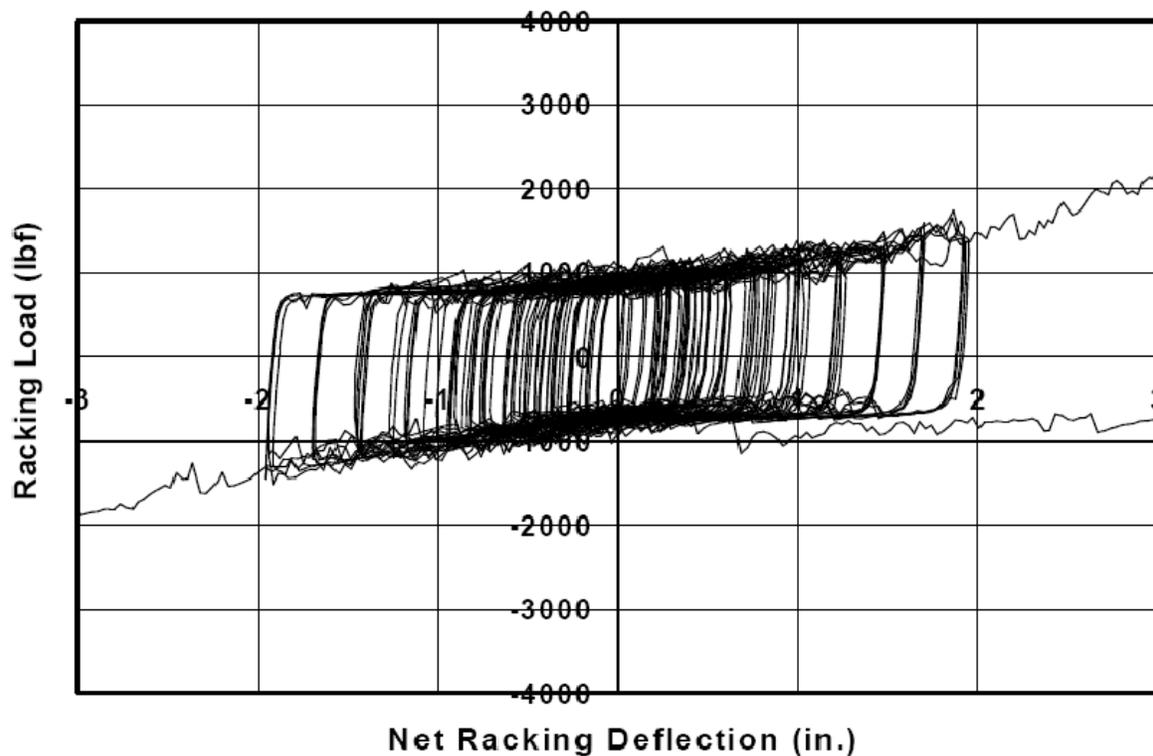
The five different connector configurations tested were

- Wall 1: two 5/8-inch diameter threaded rods secured to the test frame base located 8 inches in from each end of the wall
- Wall 2: three 5/8-inch diameter threaded rods secured to the test frame base with 8 inches end distance and one at the center of wall
- Wall 3: included a corner intersection (28 inches in length) at 16 inches from end of main wall; the main wall was anchored at 8 inches from the free end; at the intersecting end wall courses were fixed together but not anchored to the test frame base; connection between the courses at the intersecting wall were toenails with 2, 16d nails per intersection
- Wall 4: two 5/8-inch diameter threaded rods secured to the test frame base with 8 inches end distance; each course connected to the course below with 12-inch lag screws at 18 inches on center and offset 3 inches from each end of the wall
- Wall 5: two 5/8-inch diameter threaded rods secured to the test frame base with 8 inches end distance and 3/4-inch galvanized pipe inserted as a sleeve at the threaded-rod locations (pipe was interrupted at coupler and nut locations on every fourth log) and outside pipe diameter was 1.1 inches

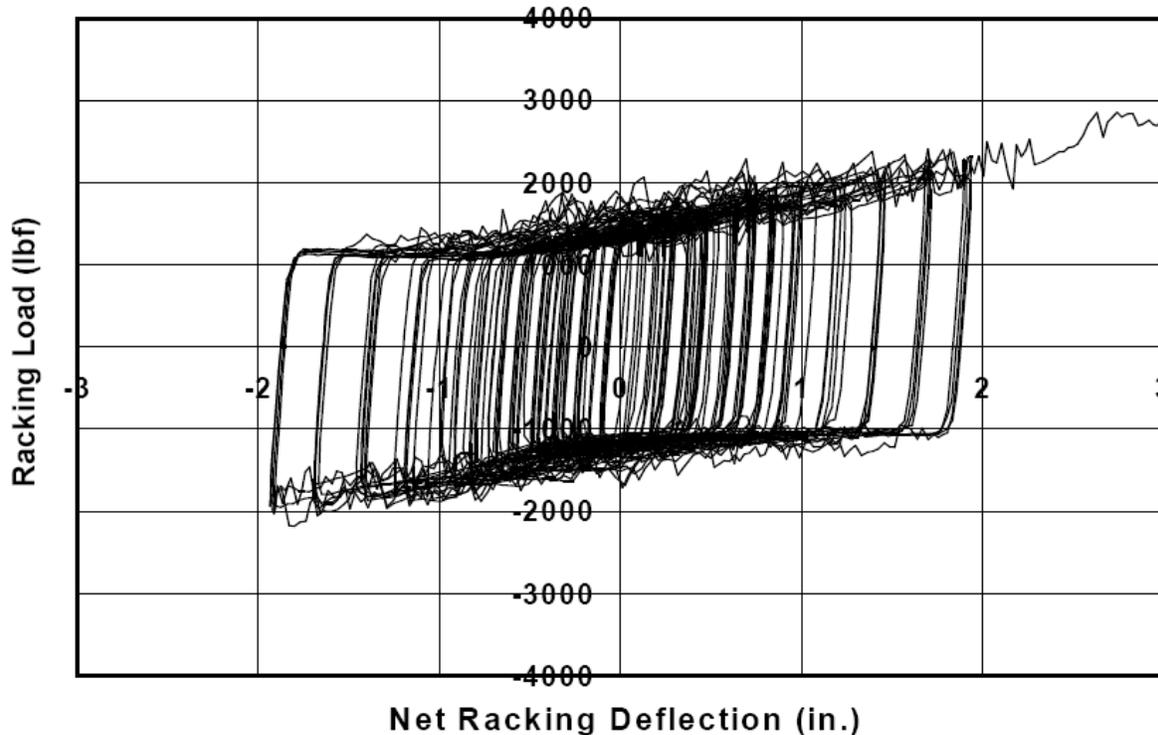
Displacement transducers were mounted on the top and bottom courses to measure both in- and out-of-plane displacement. A double hydraulic ram applied the load into a steel header beam attached to the top half-log. The ram was programmed with SEAOC sequential phased cyclic displacements. Each test was 72 cycles of constant crosshead speed of 1 inch per second. At the end of the 72 cycles, the ram was extended to the maximum stroke possible (about five inches). Each configuration was tested on three identical walls (except wall 5). The monitoring interval for the racking load, horizontal displacements and vertical displacements was 0.1 sec (Shrestha & Gorman, 2002).

Testing results provided hysteresis data, as shown in Figure 4-1 and Figure 4-2. Hysteresis curves showed no significant first major event for the test configurations. The load increased rapidly until static friction was overcome. Friction was the main energy dissipater because very little log damage or connector deformation was visible and the sustained racking

load did not vary based on the wall configurations tested. Each configuration exhibited similar load deflection responses. The intersecting end wall improved the performance. Shrestha recommended ignoring this gain in stiffness since it depended on the intersecting wall and all log shear walls may not have intersecting walls. Shear strength increased with the number of threaded rods until the static friction force of the test condition was reached. Once the applied force overcame the log friction, shear strength was independent of the hold-downs, and depended only on wood bearing strength at the through-rod or log-log connector (Shrestha & Gorman, 2002).



**Figure 4-1 Typical response of wall type 1, two hold-down anchors (Shrestha & Gorman, 2002), courtesy of Washington State University Wood Materials and Engineering Laboratory**



**Figure 4-2 Typical response of wall type 2, three hold-down anchors (Shrestha & Gorman, 2002), courtesy of Washington State University Wood Materials and Engineering Laboratory**

The Wall 4 test with lag screws behaved similarly to the wall with increased number hold-downs (Wall 2), but some tests resulted in shearing of the lag screws. The researchers believed the screws sheared because of the “kerf” sawn into the bottom of the log. The kerf is sawn into the underside of an unseasoned log to prior to drying. As the wood seasons, the kerf opens up, preventing subsidiary stresses due to uneven shrinkage at other points along the log. The opened kerf created a gap in the support along the lag screws; this gap allowed the lag screw to bend and introduced flexibility into the connection. As the slender lag screw deflected, the wall displaced, and the screw was stressed in both bending and shear. When the maximum combined bending and shear stress was reached, the screws failed in shear and the wall strength dropped to that of Wall 1. (Shrestha & Gorman, 2002)

The oversized holes affected the strength added to the log by the through-rods. The oversized hole allowed some bending in the through-rod. The combined loading condition affects the allowable shear strength in the rod. Adding the pipe around the threaded rod did not

increase the shear strength of the wall, but did increase deflection before the connector achieved bearing (Shrestha & Gorman, 2002).

Shrestha created a control chart of the racking load at prescribed deflections for each wall type. As seen in Figure 4-3, Wall 4 performed the best through the cyclic loading if the test condition with the intersecting wall is ignored. Wall 5 demonstrated higher strength provided by the intersecting pilaster, but Shrestha concluded it should not be considered when determining the lateral strength of a log shear wall.

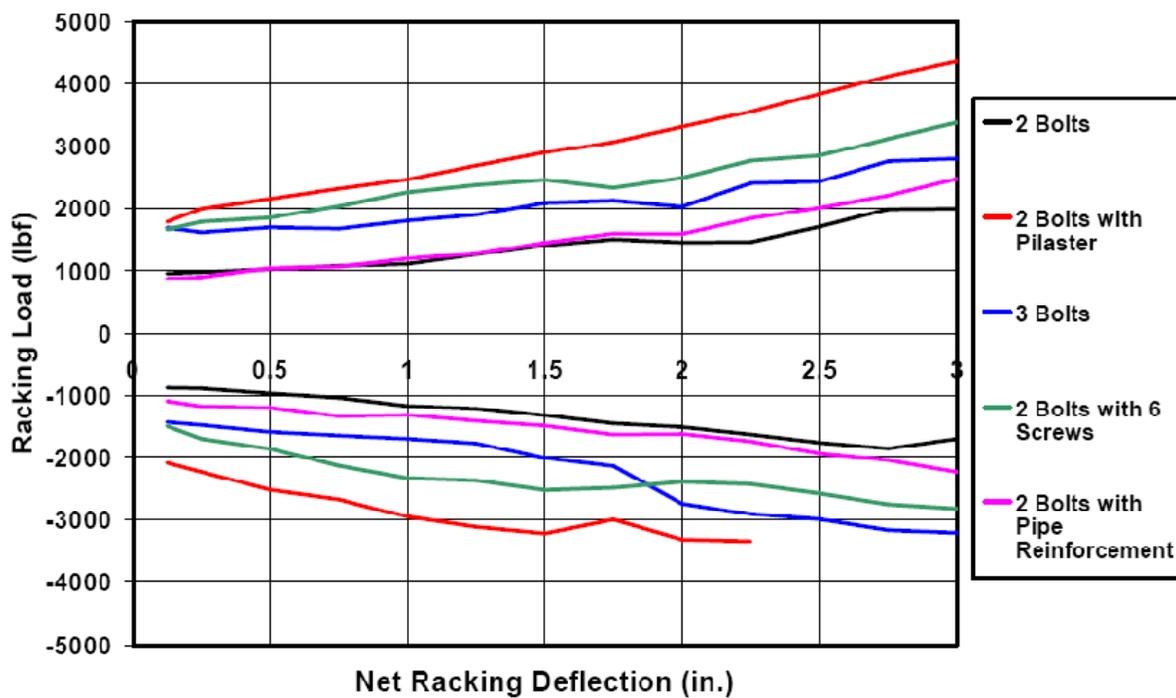


Figure 4-3 Summary of maximum load at prescribed racking deflections (Shrestha & Gorman, 2002), courtesy of Washington State University Wood Materials and Engineering Laboratory

#### 4.2.2 Popovski, Forintek Canada Corporation, 2002

Concurrent to the Washington State research Forintek Canada Corporation performed testing for the International Log Builder’s Association (ILBA) on handcrafted log walls (Popovski, 2002). The ILBA research investigated the effect of wood pins (type and inclusion)

on lateral performance. Popovski varied vertical and horizontal loads, in addition to varying the type of wood pin. The testing continued until it reached the maximum displacement of the testing apparatus (4 1/2 inches for pushover tests and 2 1/4 inches for cyclic tests) (Popovski, 2002).

Two types of quasi-static tests were completed on the test walls, pushover and cyclic. The team used five individual walls to test five configurations and completed ten tests. The walls are described in tables 4-1 and 4-2. Each wall was composed of nine stacked logs, for a height of eight feet and was eight feet long. The log diameters averaged 12 inches, but varied as much as two inches around the average. Inclusion and type of wood pin, vertical load and wall end conditions (intersecting corner) varied for the different wall configurations. The hardwood pins were 1-inch diameter, round pins, placed in a staggered pattern between log courses at the wall ends. The softwood pins were Douglas-Fir square pins of equivalent maximum dimension, placed in the same pattern. The control wall, wall type I with no wood pins or corners, was tested only in pushover. Wall types IV and V were tested under cyclic loads only. Wall types II and III were tested in both pushover and cyclic tests. The pushover test did not load wall types II and III to failure, so these walls were used again for the cyclic testing. The pushover test on these walls affected the initial stiffness in the cyclic test. ILBA provided the five walls. Tables 4–1 and 4–2 show the test results.

**Table 4–1 Pushover tests and results of tests by Popovski in 2002**

Test Number	Wall Type	Test Type	Vertical Load (K)	Wall Configuration	Max Lateral Load (K) <sup>a</sup>	Initial Stiffness (K/in) <sup>a</sup>
1	I	Pushover	1.20	Control	1.379	13.831
2	I	Pushover	2.40	Control	1.965	11.447
3	II	Pushover	1.20	Hardwood pins only	3.271	10.596
4	III	Pushover	1.20	Corners only	2.368	11.675

a: Failure load was not reached during the test

b: Stiffness calculated using  $0.2P_{max}$  and  $0.1P_{max}$

**Table 4–2 Cyclic tests and results of tests by Popovski in 2002**

Test Number	Wall Number	Test Type	Vertical Load (K)	Wall Configuration	Max Lateral Load (K)	Max Uplift (in)
5	III-A	Cyclic	1.20	Corners only	2.247	2.29
6	III-B	Cyclic	10.00	Corners only	6.604	1.03
7	IV	Cyclic	1.20	Softwood pins only	1.709	0.1
8	IV-A	Cyclic	1.20	Hardwood pins only	3.906	2.08
9	V	Cyclic	1.20	Hardwood pins, corners	5.627	2.1
10	V-A	Cyclic	10.00	Hardwood pins, corners	7.935	2.13

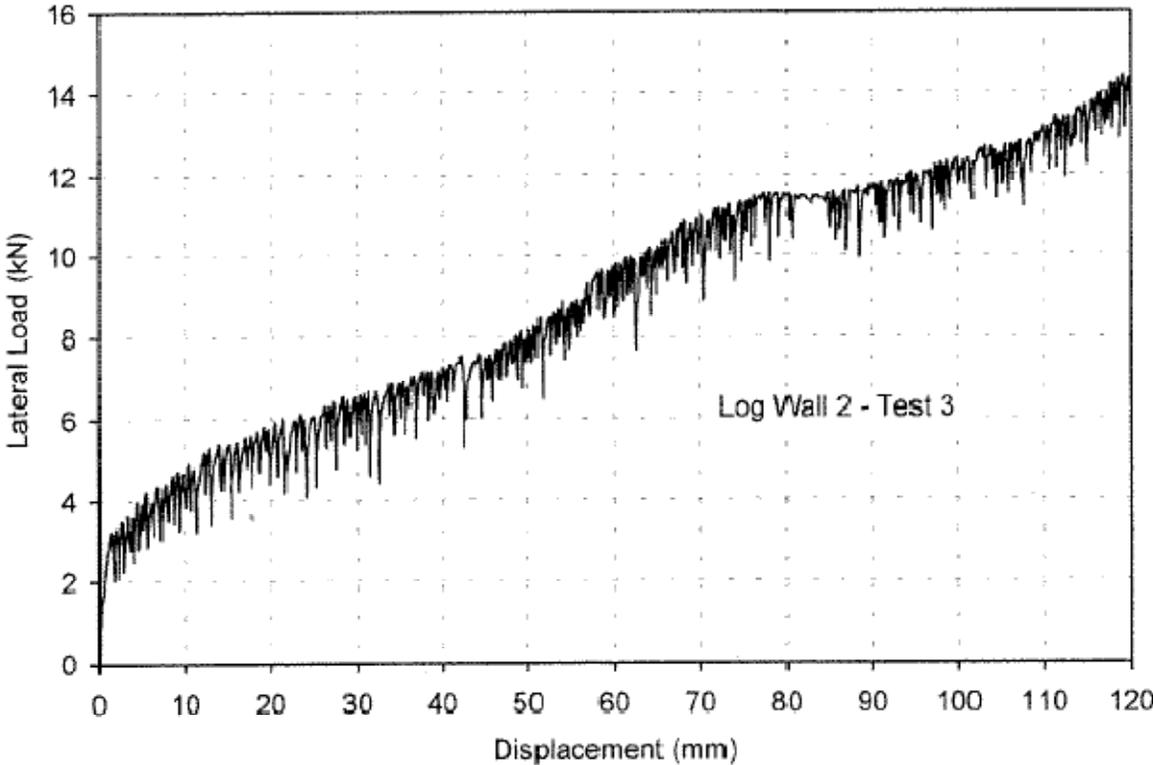
For the pushover test, the constant rate of loading (displacement of the load cell) was 0.3 inches per minute and applied to the top log. Vertical loads remained constant on the walls during the testing. Displacement transducers collected data at logs 4, 6, 7, 8 and 9, with log 1 being the sill log. A fifth transducer measured uplift at the corner with the load cell. Maximum loads recorded resulted from the capacity of the load transducers and not the failure of the wall.

The cyclic tests used the International Organization for Standardization cyclic testing protocol, ISO/DIS 16670 2001. The standard uses a reverse displacement pattern with increasing amplitudes, induced at the top of the wall. The cyclic load tests represented the displacements induced at the top of the wall during an earthquake. Again, vertical loads remained constant on the walls during the testing and a transducer measured uplift of the top log, with respect to the sill log, at the corner with the load cell. Time-histories of the displacement, uplift and applied load were recorded for each test.

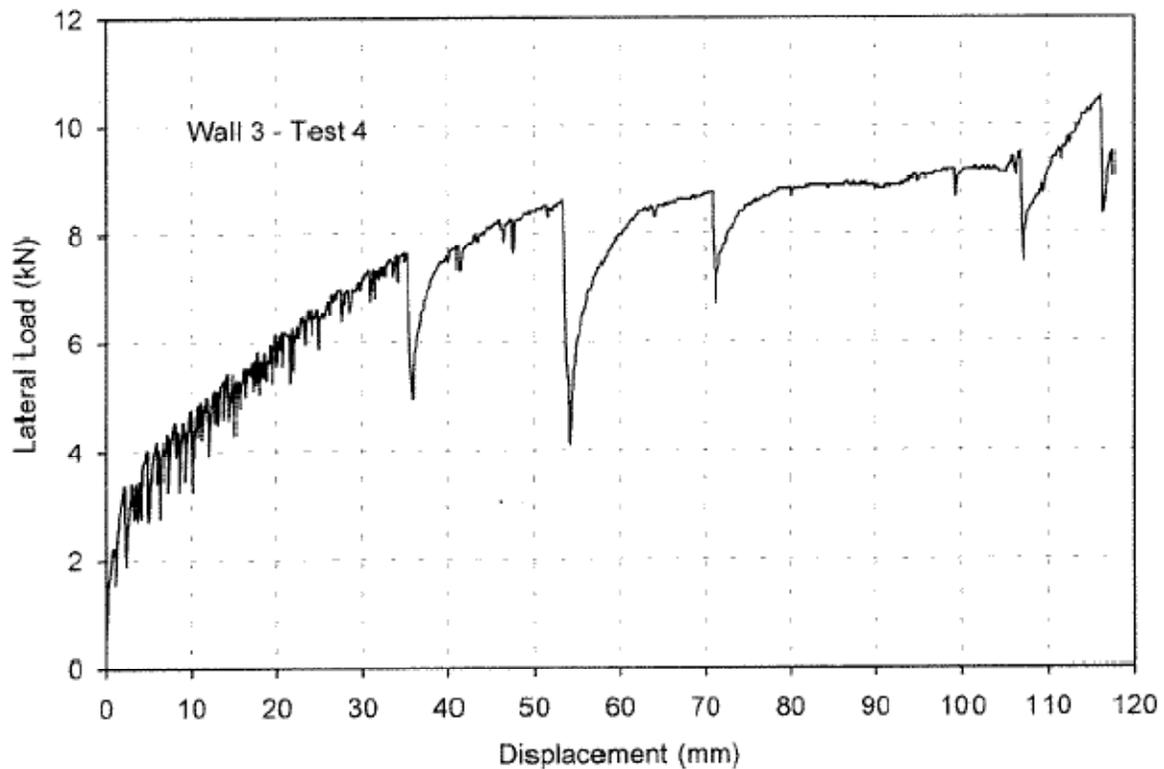
Throughout testing, the stiffness of the wall clearly deteriorated over time in cyclic loading, but wall strength remained almost constant. In some cases, the wall resisted the load, but deformed as much as 5% of the story height. Most building codes limit the allowable story drift to about 2% of story height for shear walls. The ability of a log shear wall to deform without a significant loss of strength is a good attribute for collapse prevention.

Deformation of the walls occurred in pulses or spiked increments. The results of the pushover tests show the pulses clearly as shown in Figure 4-4 and Figure 4-5. The pulses indicated points of overcoming the friction capacity, and the log-log surfaces sliding against each

other. The sliding caused wood crushing and a wedging effect, which built more friction and stopped the slip deformation. Increased gravity loads (test 2) provided more friction, which caused the pulses to be more severe, but occurred at higher loads (Popovski, 2002).



**Figure 4-4 Load-Displacement relationship at the top level of the wall with hardwood pins (Popovski, 2002), courtesy of Forintek Corporatio**



**Figure 4-5 Load-Displacement relationship at the top level of the wall with corners (Popovski, 2002), courtesy of Forintek Corporation**

For the two walls with high vertical loads (test 6 and test 10), the wood pins failed in shear. However, the pin failure was not obvious during the test or in the hysteretic curve generated from the results. The broken pins were between the fifth and sixth courses, roughly in the center of the wall height. This confirmed that log shear walls have a lateral load path that utilizes the shear planes between courses as the transfer mechanism from top to bottom of the wall. This load path is unique to log shear walls.

Walls with hardwood pins and corners demonstrated better lateral energy dissipation than those without pins or intersecting corners. However, uplift was the main source of energy dissipation because lateral deflection was inhibited. The testing indicated strong energy dissipation in the log shear walls with hysteresis curves as wide as or wider at the edges than in the middle. This hysteretic response was better than typical timber connections. Typical timber

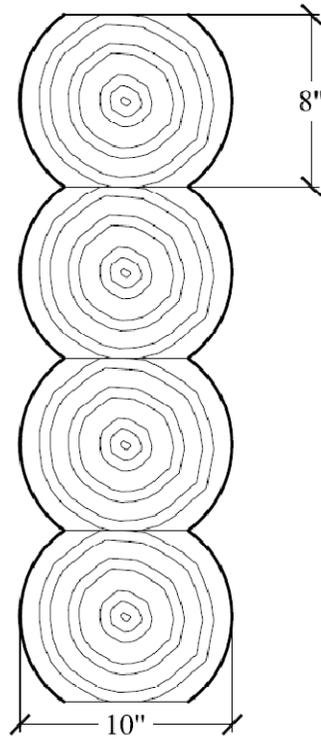
connections yield “pinched” hysteresis curves, or curves that are narrower through the middle than at the ends (Popovski, 2002). The hysteretic and testing response indicated that log walls have desirable behavior in dissipating energy, but when designing for high lateral loads uplift control is necessary.

#### ***4.2.3 Snyder, Montana State University, 2003***

Snyder examined the walls of the Old Faithful Inn in 2003. His study considered the historic construction and recommended reinforcing for the wall as part of the seismic retrofit portion of the Inn’s renovation (Beaudette & Moser, 2009). Beaudette Consulting Engineers used the research to determine the best locations for supplemental pinning. The study at Montana State University tested five wall configurations (Snyder, 2003). The five reinforcing types were:

1. unreinforced
2. pin reinforcing
3. angle bracket reinforcing
4. OlyLog screw reinforcing
5. post-tensioned cable reinforcing

Snyder conducted tests on wall specimens 8-foot tall, 10-foot long shear wall segments, built to match the Old Faithful Inn. Standing, dead Lodge Pole Pine and Engelmann Spruce were professionally cut by a local log home manufacturer. The cross section was eight-inches deep with flat surfaces on the top and bottom and about 10 inches across (Figure 4-6). The results of his tests indicated that OlyLog screws or post-tensioned cables provided the best combination of lateral strength and energy dissipation. Frictional resistance was the primary source of energy dissipation, while lateral strength came from the strength of the wall reinforcing. Testing showed that the logs in testing contained 12% moisture. The moisture content in the logs of the Old Faithful Inn itself were not measurable.



**Figure 4-6 Profile of logs used for testing by Snyder, 2003**

Snyder conducted cyclic displacement tests with varying axial loads. He considered both pure friction behavior and reinforced wall behavior. For pure friction, Snyder considered two cases of overturning by conducting small-scale tests. Results indicated that friction did not increase as the log wall overturns and the corner of rotation digs into the test apparatus (Snyder, 2003).

Hysteresis data for the tests showed very narrow curves, which indicates low energy dissipation, for the angle bracket reinforcing. All other types of reinforcing had wide hysteresis curves with similar areas, indicating strong energy dissipation. The unreinforced log walls showed the highest energy dissipation. However, when given drift limits set by the building code, this level of energy dissipation cannot be fully utilized.

#### ***4.2.4 Scott, Oregon State University, 2004***

In 2004, Randy Scott, at Oregon State University, also studied the lateral performance of log shear walls. Scott conducted both physical research and computer modeling of log shear wall behavior. The physical testing examined the effect of sill log conditions on overall wall performance, which provided parameters for the computer modeling. Scott's computer modeling involved fourteen finite-element models of full walls of equivalent size to the models tested by Gorman and Shrestha. The models varied aspect ratios, locations of through-rods, and locations and sizes of window openings and door openings.

The two different foundation anchorage details tested by Scott can be found in Figure 3-4. Scott used the Consortium of Universities for Research in Earthquake Engineering (CUREE) method for the physical testing (Scott et al, 2005). A series of fully reversed cycles demonstrated the behavior of both foundation anchorage methods (Scott et al, 2005). The test frame used two hydraulic actuators, one to apply vertical loads and the second to apply horizontal loads. Friction, static and quasi-static tests were conducted on specimens of both details.

Four specimens were tested in total:

- Detail a: friction tests
- Detail a1: static test and quasi-static test
- Detail a2: quasi static test
- Detail b: quasi static test

The friction test of "Detail a" determined the coefficient of friction between the sill log and the plywood floor sheathing. The floor joists in "Detail a" did not extend past the thickness of the foundation wall for the "Detail a" test. "Detail a1" and "Detail a2" were similar to "Detail a" but included longer floor joists. Details "a1" and "a2" were used to examine the behavior of the foundation anchorage over the whole structure, so floor joists and sheathing were included to add the effects of diaphragm stiffness. The test of "Detail a2" utilized additional through rods, which ran from the bottom of the floor sheathing to the top of the sill log (Scott et al, 2005). The test of "Detail b" was only quasi-static loading. Friction and static tests were not performed using this detail because floor sheathing properties and diaphragm stiffness would not affect the performance of the connection under lateral loads.

Scott created computer models that closely matched the physical walls built for the research conducted by Gorman and Shrestha of Washington State University in 2002 (see previous section, page 54). The models used eight logs, stacked vertically with through-rods eight inches from the end of the wall, extending the full height. The 14 models each varied roof load, log weight contribution, through-rod tension and coefficient of friction.

Table 4–3 provides a summary of the models generated. Scott used ANSYS for the analysis and used the results from his physical testing to define the friction parameters for the models. The models included log and through-rod properties provided in the ASTM standards governing the materials. Model one was considered the control case. Models eight through 14 are variations of model one, each assessing a wall attribute at the same coefficient of friction.

**Table 4–3 Computer models tested by Scott in 2004**

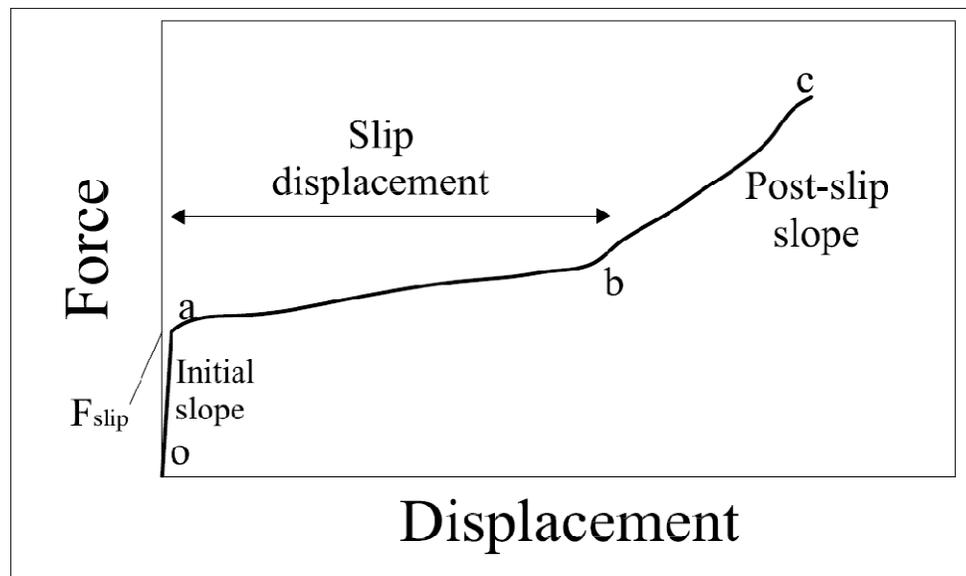
Model #	Aspect Ratio	Roof Load <sup>a</sup> (K)	Log Weight?	Through-Rod Tension <sup>a</sup> (K)	Coefficient of Friction	Opening Size <sup>a</sup> (in.), type	F <sub>slip</sub> <sup>a</sup> (K)
1	1:1	0.00	Yes	1.00	0.40	-	3.86
2	1:1	2.25	Yes	1.50	0.56	-	13.5
3	1:1	0.00	Yes	0.00	0.25	-	0.18
4	1:1	0.00	Yes	0.50	0.40	-	2.08
5	1:1	0.00	Yes	1.50	0.40	-	5.64
6	1:1	0.00	Yes	1.00	0.25	-	2.41
7	1:1	0.00	Yes	1.00	0.56	-	5.40
8	1:1	2.25	Yes	1.00	0.40	-	7.86
9	1:1	2.25	No	1.00	0.40	-	-
10 <sup>b</sup>	1:1	2.25	Yes	1.00	0.40	-	-
11	2:1	2.25	Yes	1.00	0.40	-	-
12	1:1	2.25	Yes	1.00	0.40	24 x 36, window	-
13	1:1	2.25	No	1.00	0.40	40 x 84, door	-
14 <sup>c</sup>	1:1	2.25	Yes	1.00	0.40	-	-

a: Lengths and forces have been converted from S.I. units to English units

b: includes floor diaphragm and anchor bolts

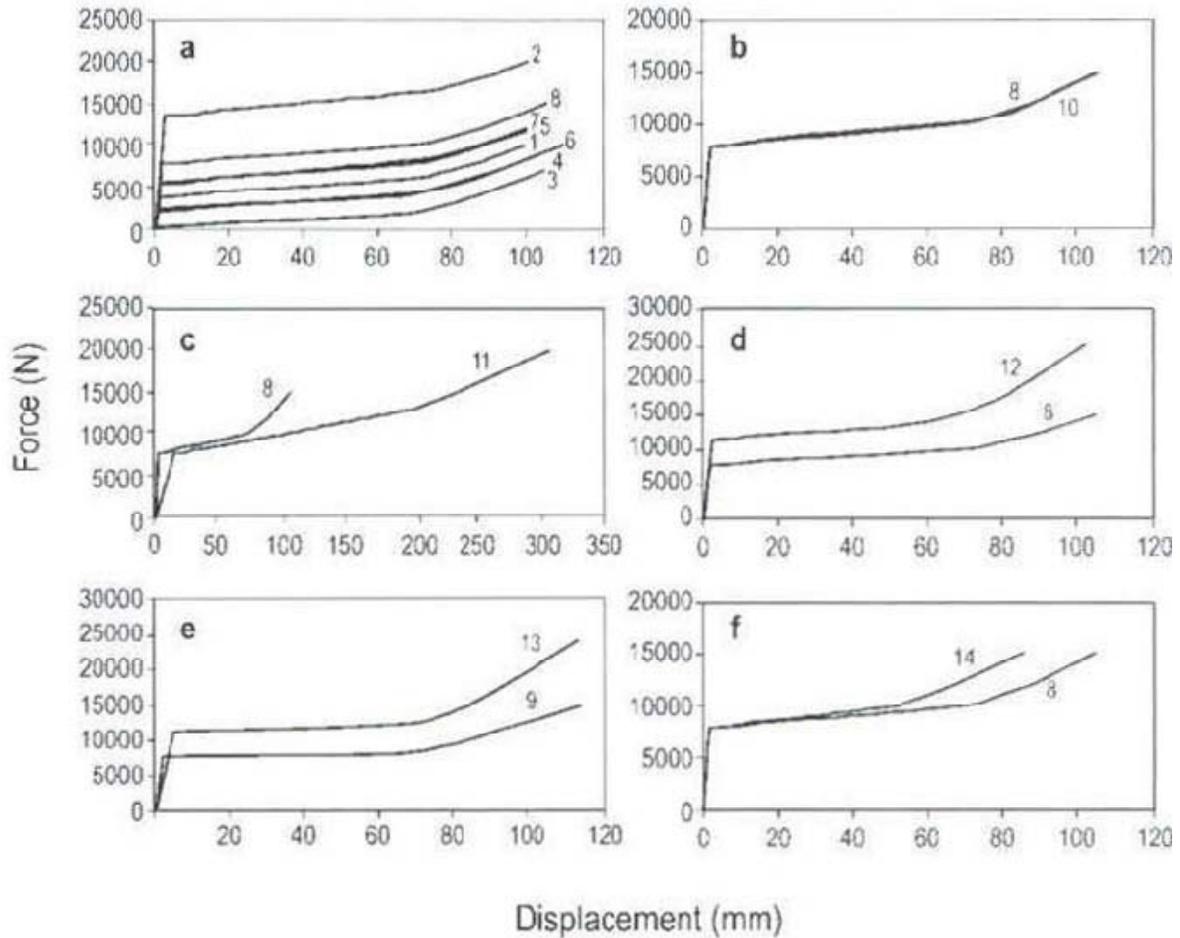
c: through-rod holes oversized by 1/8"

These tests provided results similar to the physical wall details Scott tested previously and the full-scale walls Gorman and Shrestha tested. Four values describe the force-displacement response (initial slope, slip-force ( $F_{slip}$ ), slip displacement, post-slip slope) (Scott et al, 2005) and are illustrated in Figure 4-7.



**Figure 4-7 Typical force-displacement curve of a log shear wall with through-rods in oversized holes demonstrated by Scott’s research in 2004**

The slip force,  $F_{slip}$ , is the initial force required to overcome log-log friction. The force-displacement curve rises rapidly until the load reaches the slip force. Slip-displacement is the displacement that occurs before the log courses achieve bearing on the through-rods. The load increases very little during this period. The force-displacement curve slopes upward again once the log-log connectors achieve bearing at each course. Placement of nuts and couplers on through-rods within the courses can decrease the amount of slip-displacement (Scott et al, 2005) but does not affect overall wall strength. Displacement continues after the connectors achieve bearing on an individual course basis due to wood crushing or yielding of the log-log connectors. Figure 4-8 shows the results of the pushover tests for each of the 14 models tested.



**Figure 4-8 Force-Displacement relationship for computer models generated by Scott (Scott et al, 2005), republished courtesy of Oregon State University**

Friction tests, static tests and quasi-static tests of Scott’s physical sill connection models showed several characteristics of the overall behavior of log shear walls in relation to the foundation connection. First, friction between sill log and the foundation is a large contributor to shear wall energy dissipation, but it decreases throughout cyclic loading. The decrease was about eight percent between the first and fourth cycles of loading. Second, the two connection details tested behaved similarly. Finally, bolt yielding and wood crushing each contribute to the displacement of the shear wall at the sill log location.

In addition, Scott’s finite element models provided good correlation with physical tests on the effects of wall attributes to behavior under lateral loads. The size of the through-rod holes

had no effect on lateral strength as slip-displacement is the only behavior parameter that depends on the hole diameter. The aspect ratio, however, did greatly affect performance. Doubling the aspect ratio decreased the initial stiffness by 88 percent, decreased the post-slip stiffness by 55 percent and increased slip-displacement by 164 percent. Window openings affected on log shear wall behavior positively when through-rods were added on either side of the opening. Placement of the opening did not affect behavior as much as size of the opening. Tall openings, such as doors, decreased the initial stiffness. However, post-slip stiffness and slip force increased when door openings included through-rods in ends of the logs on either side.

#### ***4.2.5 Yeh, 2006***

Yeh completed testing on conventionally framed wood shear walls in 1998. Yeh used those results as a baseline for comparing the results of physical testing of log shear walls, which he completed in 2000. Yeh tested machined logs of D-log profiles. As expected, the conventionally framed shear walls Yeh tested in 1998 showed a decrease in racking strength with added openings. Until Yeh's research, the degree of weakening of a log shear wall due to openings was unknown and considered equivalent to that of a conventionally framed wood shear wall.

Using two sizes of Western Red Cedar machined logs of proportional profiles, Yeh constructed six test walls, two control tests and four with different opening types (Yeh, Chiang, & Lin, 2006). Table 4-4 shows the test wall information. Each wall was 94.5 inches wide and 83.8 inches tall (Yeh et al, 2006). The logs were braced laterally using supporting wood frames to prevent lateral buckling (Yeh et al, 2006). As log connectors, Yeh used lag screws with a diameter of 3/8-inch, 7 7/8- inches long with a 5-inch spiral shank. The penetration depth of the lag screws was 2 3/4-inches. Yeh placed the lag screws at each end. The end distance for the lag screws was 3 1/2- inches, alternating every other course with 4.7 inches. For the control test, Yeh placed an additional row of lag screws in the center of the wall, making the final on center spacing of the lag screws 39.8 inches. For the tests with openings, Yeh installed lag screws at the jamb of each window or door opening, with the same edge distances as the end of the wall. (Yeh et al, 2006)

**Table 4–4 Tests completed by Yeh, 2006**

Wall Name	Log Size (nominal)	Opening Type	Opening Size (in x in)	Opening as a % of Wall Area	Strength as a Ratio to Control
N-4 (control)	4x6	None	-	-	1:1
N-6	6x6	None	-	-	1:2
W-4-35	4x6	Window	35.4 x 31.4	14.0	1:1.69
W-4-70	4x6	Window	70.9 x 31.4	28.1	1:0.72
D-4-35	4x6	Door	35.4 x 68.1	30.5	1:0.64
D-4-70	4x6	Door	70.9 x 68.1	66.9	1:0.28

Yeh tested two specimens of each wall configuration. The racking tests followed the procedures of ASTM E564 and ASTM E72 (Yeh et al, 2006). Loads applied at the top of the wall were increased in four stages. The first 3 stages applied lateral loads of 790, 1,570, 2,360 pounds-forces at a uniform rate of 0.3 inches per minute. After reaching the target load for the first 3 stages, the load was removed and the residual deformation was recorded. The fourth stage load was applied at the same rate as previously until failure. At failure, Yeh recorded maximum load, deformation and failure mode. The total test duration was 30 minutes.

Yeh’s tests showed the load path for an in-plane lateral force applied at one end of the top log travels from the point of applied load through horizontal shear of the lag screws connection between each log course (Yeh et al, 2006). Testing also indicated that the maximum horizontal shear strength of Wall N-6 is 59 percent higher than that of structural light-framing sheathed with 3/8-inch plywood panels (6-inch nail spacing along studs and 2-inch spacing along bottom plate) (Yeh et al, 2006). Additionally, Yeh’s test results indicated that increasing log size increases strength. Increasing the log width from 4 inches to 6 inches doubled the racking strength. Yeh attributed the increase in racking strength to the lower slenderness ratio of the thicker wall. In addition, for small window openings such as those in W-4-35, the lateral strength of the wall increased. Large openings, such as those in W-4-70 and both door opening specimens did decrease strength, but not to the extent expected. Yeh proposed designing the shear wall

using the effective length (overall length minus opening length) as the design length for the lateral strength.

Failure modes of the log shear wall were similar to those of a conventionally framed wood shear wall. The log shear wall behaves and deforms as a cantilevered element. This produces a tensile force on the log-log connectors on the end of the wall near the applied load. This tensile force causes the lag screw to be loaded in withdrawal as well as shear. Failure of the lag screw in withdrawal occurred between the second and third courses. Longer penetration depths would resist higher tensile forces. Deformation of the lag screws at the top courses was larger, causing more displacement between consecutive upper courses than between consecutive courses lower in the wall.

Walls with window openings behaved well, demonstrating higher maximum shear strength than the control specimens. Walls with window openings failed in withdrawal of the lag screws at the corners of the opening. Typically, the lower corner of the opening, closest to the application of load, exhibited withdrawal failure of the lag screws, as did the second and third courses at the loaded end of the wall. Yeh stated that using the effective length of the wall with openings, or the length of uninterrupted shear wall segments, to determine lateral design strength is a possible design approach. The increase in maximum shear strength of a wall with openings is about 1.8 times the maximum shear strength a full wall of equal length, if the shear strength of the wall with openings is computed using the effective length. Wider openings showed decreased lateral shear strength compared to full wall sections, while narrower openings increased in lateral shear strength. Walls with door openings decreased in strength across the board, though not as much as conventionally framed shear walls with door openings.

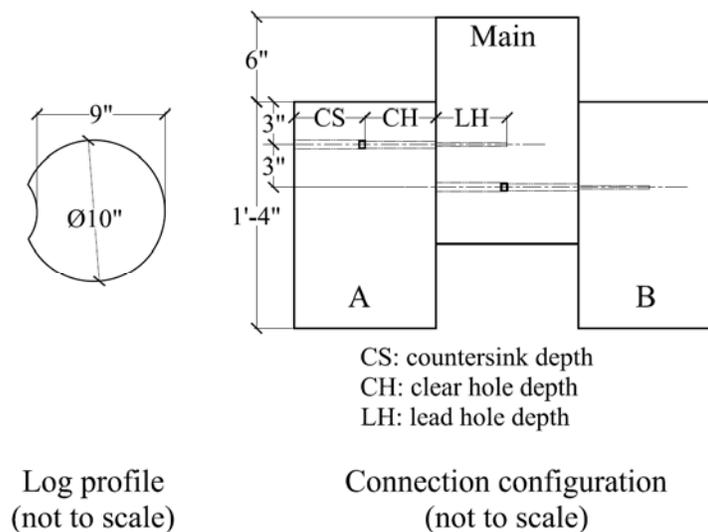
#### ***4.2.6 Graham, Washington State University, 2007***

In 2007, Graham investigated the monotonic and cyclic response of connections using lag screws in log shear walls and the monotonic and cyclic response of log shear walls with varied aspect ratios (Graham, 2007). The connection test used 16-inch long logs in double shear configuration. Graham compared the test results to connection strength values calculated using the NDS. The results of the connection tests served as a baseline for determining the ideal length and diameter of lag screws used to connect courses in full-scale log shear wall models. In

addition, Graham provided experimental data that can be used to develop archetypes and construction types for testing with the methodology of ATC-63/FEMA 350.

#### 4.2.6.1 Connection Tests

The connection tests used lag screws of various lengths and diameters. Logs were cut 16 inches long and were 10 inches in diameter. The profile of the connection logs was a “Swedish Cope” making the stack height of each log 9 inches. Connections were three logs tall (Figure 4-9) and the lag screws were offset at a spacing of 3 inches. Graham drilled lead holes, countersink holes and clear holes according to NDS provisions for lag screw installation. Edgewood Log Homes of Athol, ID provided test logs of Engelmann Spruce, Lodge Pole Pine and Grand Fir for all tests. Preliminary tests were conducted on lag screws of different lengths and diameters to determine the ideal lag screw dimensions. The preliminary tests applied monotonic loads to the connection specimens, at a displacement rate of one-quarter inch per minute. This displacement rate resulted in a time to failure of 20 minutes for each lag screw tested. All lag screws were A307, Grade 2, low carbon steel.



**Figure 4-9 Log connection configuration for connection tests completed by Graham in 2007**

The lag screws used in the monotonic tests were sizes and lengths commonly used in construction in the Pullman, WA area (Graham, 2007):

- 8 inch x 1/2 inch lag screw
- 8 inch x 3/4 inch lag screw
- 12 inch x 1/2 inch lag screw
- 12 inch x 3/4 inch lag screw

Based on the results of the preliminary tests, Graham selected 8-inch long by 1/2-inch diameter lag screw for further research and testing under cyclic loads. This lag length and size produced consistent ductile failures (Graham, 2007) in the preliminary tests. Longer lags (12 inches) of the same diameter experienced brittle failures, while lags of a larger diameter (3/4-inch) caused splitting of the wood members (Graham, 2007). Both brittle failure and member splitting are not desirable behaviors in high seismic regions because they can lead to unpredictable results.

Graham lengthened the 8-inch lag screws to 10 inches for the additional connection tests and full-scale wall tests to account for variance in diameter of logs used in shear wall construction. The additional connection tests consisted of 10 monotonic tests and 15 cyclic tests using 10-inch long by 1/2 -inch diameter lag screws. Graham used the CUREE method of cyclic testing for the cyclic tests and were carried out for 68 cycles, using 18 cycles as primary cycles. A double acting hydraulic actuator of 11,000-pound capacity applied each displacement cycle at 0.5 Hertz. (Graham, 2007)

Connections failed in the following ways:

- pull through of the washers and lag screw heads
- lag screw yielding at each shear plane
- low-cycle fatigue of the lag screws due to repetitive bending

Pullout of the lag screws was minimal and only occurred after large displacements of the logs. The penetration depth of the lag screws of 12 times the minimum diameter of the lag screw was much higher than the NDS minimum requirement of eight times the diameter of the fastener (Graham, 2007). The tests also demonstrated that lag screws with an unthreaded length that extends through the log interface performed better than those that expose the weaker, threaded cross section on the shear plane (Graham, 2007).

#### **4.2.6.2 Full-scale Log Shear Wall Tests**

Graham's full-scale shear wall tests used 3 walls, each with 10-inch long, ½-inch diameter lag screws, with the unthreaded shank portion extending past the shear plane between log courses. The walls were 8 feet high and of varied lengths using 10-inch diameter logs. The aspect ratios (height: length) were 1:1, 2:1 and 4:1. The lag screws were staggered at 6 inches on center at each end of the shear wall for the 1:1 and 2:1 walls and staggered at three inches on center for the 4:1 wall. The minimum end distance for the lag screws was 6 inches, which is longer than required by the NDS. Three anchor bolts (5/8-diameter) were installed in the sill logs, which was a half log, to anchor the wall to the test frame. Ten full courses were stacked on top of the sill log to reach the full height.

Graham carried out both monotonic and cyclic tests on the walls. For the monotonic tests, the load rate was half an inch per minute. The cyclic tests followed the CUREE displacement controlled quasi-static cyclic protocol using 19 primary cycles, with a maximum displacement limit of 4 ½ inches and minimum displacement limit of -4 ½ inches. To represent the behavior conservatively, the walls did not have gravity loads or tension rods between the courses. This decision does not represent normal construction conditions, but does represent the worst case for uplift during a seismic event. The walls did not reach failure during the testing because of the displacement limits of the load apparatus. Table 4–5 shows the results of both testing types. The failure load listed is the load that corresponds to an allowable drift limit of 2.0 inches, as dictated by the ASCE 7 for “All other structures” in Table 12.12-1.

**Table 4–5 Testing Results from full-scale wall tests by Graham in 2007**

Test Type	Aspect Ratio	$V_{\text{peak}}$ (lbs/in)	$V_{\text{failure}}$ (lbs/in) <sup>a</sup>	$V_{\text{peak, design}}$ (lbs/in) <sup>b</sup>	$V_{\text{failure, design}}$ (lbs/in) <sup>c</sup>
Monotonic	1:1	66.7	43.4	24.0	15.4
	2:1	32.5	18.8	11.4	6.9
	4:1	12.6	12.6	9.1	4.6
Cyclic	1:1	69.0	53.7	24.6	18.8
	2:1	30.2	21.1	10.8	7.4
	4:1	22.8	15.4	8.0	5.7

a: Shear strength corresponding to the defined failure deflection of 2.0 inches

b: Peak shear strength divided by a seismic safety factor of 2.8

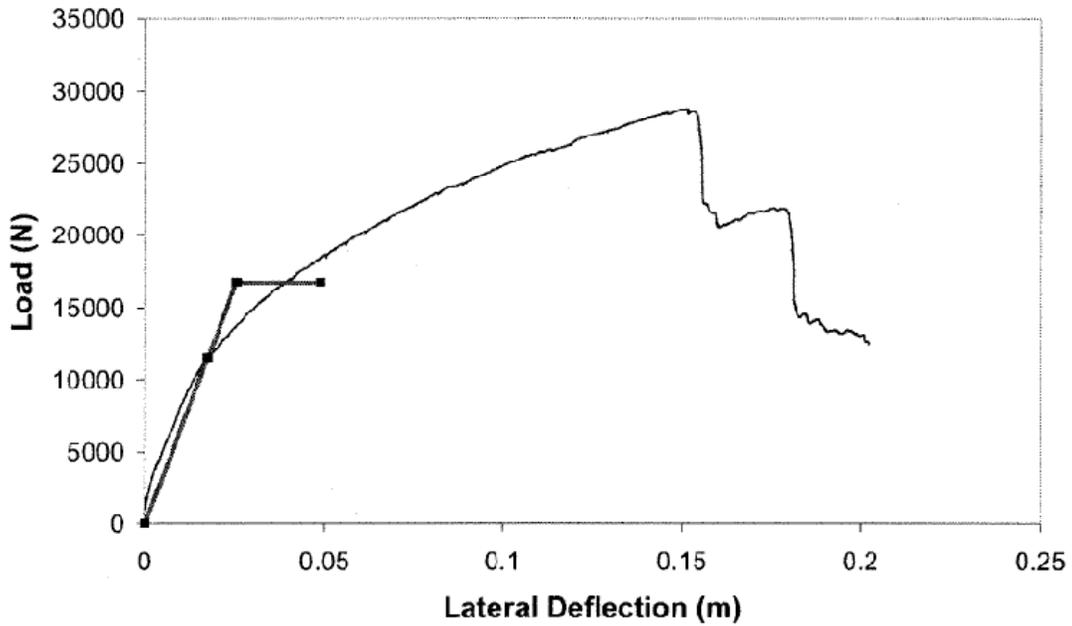
c: Shear strength corresponding to the defined failure deflection divided by a seismic safety factor of 2.8

At the allowable drift limit of 2.0 inches, lag screws showed no evidence of failure. Lower logs exhibited uplift, highest on the end where the load was applied. Slip to achieve bearing on the anchor rods was evident in the sill log, but this slip only accounted for about three percent of the overall wall slip (Graham, 2007).

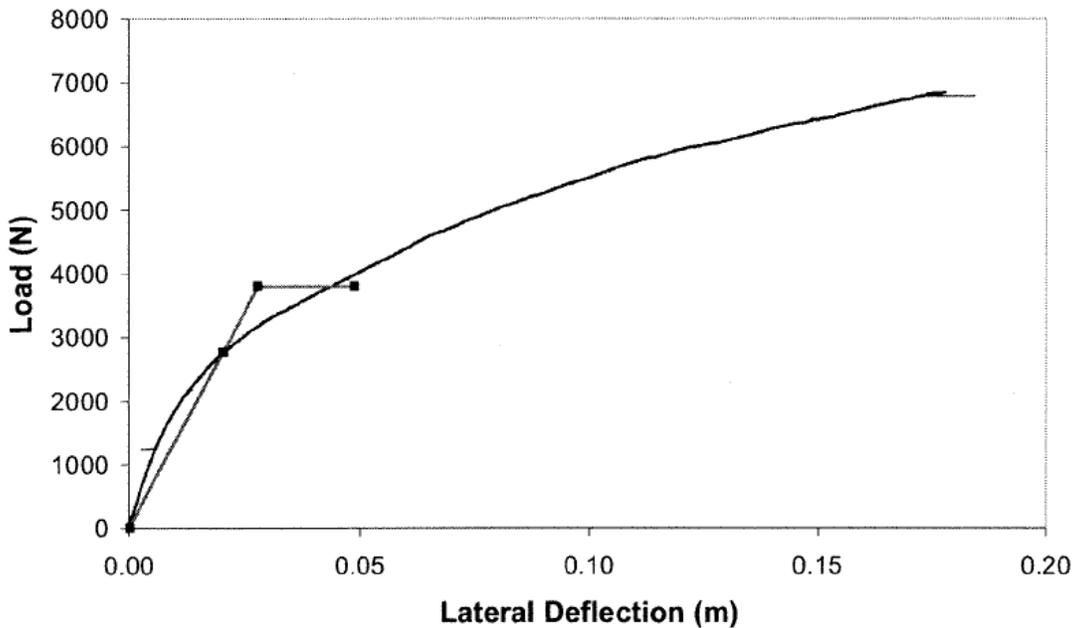
Figure 4-10 through Figure 4-12 show the pushover analysis results from the three full-scale wall tests. Superimposed on the results is the backbone curve developed from the hysteresis graphs. The pushover analysis indicates high ductility and maintaining/increasing strength, even after initial yield.

Graham (2007) concluded after reviewing the testing results:

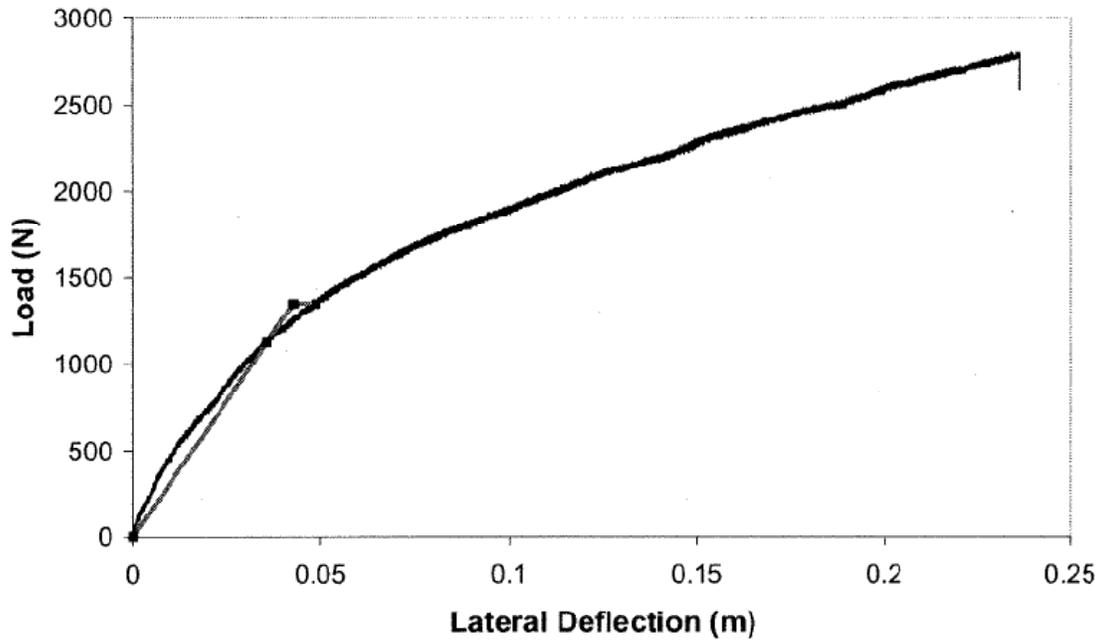
- design anchor bolts to pass up through the sill log and connect in the first full course if half sill logs are used
- restrain uplift at wall ends
- use larger diameter washers to prevent wood crushing at the head of the lag screw and take advantage of the full penetration depth, recommended at 12D



**Figure 4-10 Load-displacement relationship with backbone curve from cyclic testing superimposed for wall with 1:1 aspect ratio (Graham, 2007), republished courtesy of Washington State University Civil and Environmental Engineering**



**Figure 4-11 Load-displacement relationship with backbone curve from cyclic testing superimposed for wall with aspect ratio 2:1 (Graham, 2007), republished courtesy of Washington State University Civil and Environmental Engineering**



**Figure 4-12 Load-displacement relationship with backbone curve from cyclic testing superimposed for wall with aspect ratio 4:1 (Graham, 2007), republished courtesy of Washington State University Civil and Environmental Engineering**

## 5 Quantifying the R-Factor for Log Systems

Log shear walls are constructed all across the U.S. in areas with varying levels of seismic hazard. The lack of model building code guidance has resulted in local plan review boards arriving at different definitions of acceptable designs. As shown in Chapter 4, many different researchers have performed tests on log shear walls, but each test has been too specific to draw general conclusions about shear wall design from the results of the individual study. The availability of test results and engineering judgment both affect the seismic coefficients selected by the engineer during log shear wall design. As a result, seismic coefficients selected for log shear wall design vary based on experience with log shear walls, experience with seismic design and what the local jurisdiction will accept. There are not enough test results to develop code language per the methodology of ATC-63 to incorporate seismic design criteria for log walls into the model building codes or other applicable specifications. However, the definition provided by ATC-19 can be used to approximate an  $R$  for log shear walls. This chapter will present the  $R$  for log shear walls from current design practice and a recommended value developed by evaluating the tests presented in Chapter 4 with the ATC-19 definition of  $R$ .

### 5.1 $R$ Currently Used for Log Shear Walls

Standard practice is to use an  $R$  of 4.0 to 4.5 for log shear walls. However,  $R$  factors for log shear walls used by engineers can vary from 2 to 5.5. Several conditions contribute to the selection of an  $R$  in standard practice:

- type and spacing of connector
- acceptable to plan review committee
- available research/testing results
- familiarity with behavior
- type of corner notching
- log profile

Very little testing is readily available to engineers and most prefer to use more conservative values (lower  $R$ ) until more research has been completed. Designs which utilize higher  $R$  factors use average sized connectors (5/8" to 3/4" diameter) moderately spaced (24" to

36” on center), relying on both shear capacity of connectors for lateral strength and higher ductility from longer spacing in the shear wall for energy dissipation.

Friction between log courses is not considered a viable source of energy dissipation because of the non-constant nature of the friction force. Usually, walls that rely only on through-rods for reinforcing are designed with much lower  $R$  factors than those using dowel-type connectors between each log course. This is because of the variability of the friction force (due to log properties, log joinery, gravity loads), but testing demonstrates friction is a large source of energy dissipation in log shear walls. Friction occurs as the log courses slide across each other. From physics, it is understood that friction force depends on the force normal to the friction plane and the coefficient of friction of the sliding surface. Friction dissipates energy by giving off heat as surfaces slide across each other. Moisture content and log profile both contribute to the coefficient of friction. The force normal to the friction plane for a log shear wall would be dependent on the gravity loads of the structure applied to the wall.

However, earthquake ground motion is unpredictable, with both horizontal and vertical components. A ground motion with a large vertical component could reduce the gravity load acting downward on the friction surface thus reducing friction. Though the reduction in gravity loads would be short because earthquake motion cycles are short, it would reduce the friction force for that point in time. Reducing the friction force causes a reduction in the energy dissipation within the shear wall, changing the behavior of the system under lateral loads. The vertical components of earthquake ground motion are unpredictable, in speed, duration and number of occurrences. This results in the friction force, and energy dissipation due to friction, to be equally unpredictable. Therefore, energy dissipation due to friction forces are not considered a reliable source of energy dissipation by practicing engineers and instead the connector and wood bearing are the only sources of lateral strength used in the design of a log shear wall.

As shown in Chapter 3, many different types of connectors are used in constructing log shear walls. Connector spacing, diameter and material of connectors affect the ductility and lateral behavior of a log shear wall. Out-of-plane stack stability, not lateral strength, typically governs maximum connector spacing (David Roberts, personal communication, October 4, 2009) though smaller spacing of connectors results in a shear wall with higher lateral strength. Typically, engineers view connector type and spacing as the only controllable variable with a

large affect on the ductility of the log wall. Moisture content of the logs and gravity loads are variables outside the engineer's control because material availability controls these variables.

Log manufacturers prefer to use the connector that works best from a construction standpoint for their operation, which can affect the connector selection by the designer. Since most log construction is residential, the log manufacturer is usually involved at the beginning of the design process. Being involved from the beginning, the manufacturer's selection of a log-log connector does not cause design changes. Engineers position connectors around openings, typically six to eight inches from the opening edge, and at the ends of the wall for stability and strength. In some cases, through-rods are placed at the ends of the walls for hold-down, in lieu of log-log connectors. End distance from the end of the wall for log-log connectors or through-rods is often at six to eight inches, alternating at each course if using log-log connectors.

The article, *ICC Standard on Log Construction* by John Showalter in the March 2006, Structure Magazine, provides a good explanation of the reasons for using the 4.0 to 4.5 for  $R$ . In the article, Tom Beaudette, of Beaudette Consulting Engineers (BCE), explains that an  $R$  factor of 4.0 to 4.5 is conservative. Beaudette recommends this value because the  $R$  factor is period dependent (strength and ductility) and redundancy dependent (Showalter & Pickett, 2006). A log shear wall is typically a very flexible element, and has considerable redundancy (if you consider each course as a separate element), similar to a conventionally framed wood shear wall (Showalter & Pickett, 2006). Higher  $R$  factors could be justified if physical testing were available to support the value. BCE designs log shear walls with dowel-type log-log connectors at a maximum spacing of 48 inches on center (Showalter & Pickett, 2006). Log walls that only rely on through-rods at ends and openings should use a lower  $R$  factor of 2.0 to 2.5 (Showalter & Pickett, 2006).

## 5.2 Recommended $R$ for Log Shear Walls

The testing presented in Chapter 4 was evaluated to determine a recommended  $R$  for log shear walls. The ATC-19 definition of  $R$  was used to determine the recommended value. This definition, as previously stated as Equation 2.17, is:

$$R = R_s R_\mu R_R$$

Where:  $R_s$  = period-dependant strength factor

$R_{\mu}$  = period-dependant ductility factor

$R_R$  = redundancy factor

The data used to calculate  $R$ , is from tables 5–1 and 5–2 and the figures located in Appendix B, Pushover Results and Bi-linear Approximations of the Test Walls. The following subsections present the steps used to calculate  $R_S$ ,  $R_{\mu}$  and  $R_R$  for log shear walls. The 12 walls described in Table 5–1 are taken from the testing presented in Chapter 4 and make up the testing sample. Each of these 12 walls represents the effect on lateral behavior of one or more of the following variables:

- Presence of openings
- Size of openings
- Aspect ratio of wall
- Aspect ratio of opening
- Log-log connector or through-rod
- Wood pins or steel connectors

Pushover results of each of the 12 test walls are provided in Appendix B. The bi-linear approximations of these results are shown as dashed lines. Pushover results from the 12 walls were analyzed to produce the ductility ratio, yield displacement and yield strength for each case (Table 5–2). An  $R$  for each of the 12 walls was developed separately to illustrate the behavior of the different construction types (Table 5–3). A second calculation adjusting the effective seismic weight of some test walls provided results that better represent a true constructed condition (Table 5–4). Section 5.2.4 provides a statistical analysis of the 12 separate  $R$  factors generated using the adjusted effective seismic weights. The recommended  $R$  factor for the corresponding construction type follows the statistical analysis.

**Table 5–1 Description and parameters of walls included in calculation of recommended *R***

Wall	Source	Description of openings or connectors	Height (ft)	Profile (ft <sup>2</sup> )	Length (ft)	MC (%)	G	Density (lb/ft <sup>3</sup> )	P (lbs)	W (lbs)
1	Popovski	Hardwood pins at ends	8.000	0.921	8.000	16.000	0.440	26.233	1200.000	1973.144
2	Popovski	Corners, each end, no pins	8.000	0.921	8.000	16.000	0.440	26.233	1200.000	1973.144
3	Graham	1:1 Aspect, 10"x 0.5" Dia lag screws at ends	8.000	0.545	8.000	18.000	0.420	24.980	0.000	522.783
4	Graham	2:1 Aspect, 10" x 0.5" Dia lag screws at ends	8.000	0.545	4.000	16.000	0.420	25.109	0.000	262.738
5	Graham	4:1 Aspect, 10' x 0.5" Dia Lag screws at ends	8.000	0.545	2.000	16.000	0.440	26.233	0.000	137.252
6	Yeh	No openings, 7 7/8" x 0.375" Dia Lag Screws, ends and center	8.000	0.210	8.000	15.200	0.360	21.735	0.000	292.119
7	Yeh	70.9" x 68.1" door opening, 7 7/8" x 0.375" Lag screws, ends and opening	8.000	0.210	8.000	15.200	0.360	21.735	0.000	287.583
8	Scott	10 kN roof load, 6.67 kN thru-rod tension, thru-rods at ends	8.000	1.065	8.000	15.000	0.500	29.666	2248.089	3114.996
9	Scott	10 kN roof load, 4.45 kN thru-rod tension, thru rods at ends	8.000	1.065	8.000	15.000	0.500	29.666	2248.089	3114.996
10	Scott	2:1 Aspect, 10 kN roof load, 4.45 kN thru-rod tension, thru-rods at ends	8.000	1.065	4.000	15.000	0.500	29.666	2248.089	2681.543
11	Scott	10 kN roof load, 4.45 kN thru-rod tension, thru-rods at ends and 2ft x 3ft window	8.000	1.065	8.000	15.000	0.500	29.666	2248.089	2925.360
12	Scott	10 kN roof load, 4.45 kN thru-rod tension, thru-rods at ends, oversized holes	8.000	1.065	8.000	15.000	0.500	29.666	2248.089	3114.996

Table 5–2 Calculation of  $\mu$ ,  $R_\mu$  and  $R_S$  for each wall

Wall	Source	$\Delta_y$ (ft)	$V_y$ (lbs)	$\Delta_m$ (ft)	$V_o$ (lbs)	$(\Delta m - \Delta y)/h$	$\mu = (\Delta m / \Delta y)$	$R_\mu$	$C_{so} = W / V_m$	$C_{sd} = W / V_d$	$R_S$
1	Popovski	0.089	1798.472	0.167	1798.472	0.010	1.881	1.877	0.911	0.146	6.257
2	Popovski	0.089	1854.674	0.167	1854.674	0.010	1.881	1.877	0.940	0.146	6.453
3	Graham	0.148	3821.752	0.167	3821.752	0.002	1.129	1.876	7.310	0.146	50.184
4	Graham	0.082	854.274	0.167	854.274	0.011	2.032	1.877	3.251	0.146	22.320
5	Graham	0.131	292.252	0.167	292.252	0.004	1.270	1.876	2.129	0.146	14.617
6	Yeh	0.108	1500.000	0.167	1500.000	0.007	1.538	1.876	5.135	0.146	35.250
7	Yeh	0.108	575.000	0.167	575.000	0.007	1.538	1.876	1.999	0.146	13.726
8	Scott	0.016	1798.472	0.167	3484.539	0.019	10.160	1.877	1.119	0.146	7.679
9	Scott	0.016	1686.067	0.167	2135.685	0.019	10.160	1.877	0.686	0.146	4.707
10	Scott	0.066	1686.067	0.167	1798.472	0.013	2.540	1.877	0.671	0.146	4.604
11	Scott	0.016	2472.898	0.167	2922.516	0.019	10.160	1.877	0.999	0.146	6.858
12	Scott	0.010	1798.472	0.167	2248.089	0.020	16.933	1.877	0.722	0.146	4.954

### 5.2.1 Period-Dependant Strength Factor, $R_s$

This section covers the calculation of the period-dependant strength factor for the wall in the test sample.  $R_s$  is defined as a period dependant strength factor by ATC-19. This definition is explained in section 2.4.2.1.1 and illustrated by Equation 2.19, shown below for reference:

$$R_s = \frac{V_o}{V_d}$$

Where:  $R_s$  = period dependant strength factor

$V_o$  = available lateral strength (see figures B-1 to B-12 and Table 5-3)

$V_d$  = required lateral strength determined from 1971 UBC base shear equation (see Equation 2.13)

The 1961 UBC Base Shear Equation was used to determine  $V_d$ , as was used in 1978 with the development of  $R$  for other LFRS.

$$ZKCW = C_{sd}W = V_d \quad \text{Eqn 5.1}$$

Substituting the following into Equation 5.1, returns  $C_{sd} = 0.146$  and  $V_d = 0.146 W$ :

$$Z = 1.0, \text{ Seismic Zone 3}$$

$$T = 1.0$$

$$C = \frac{1}{\sqrt{T}} = 1.0$$

$$K = 1.33, \text{ bearing wall system}$$

These values for  $Z$ ,  $T$ ,  $C$ , and  $K$  are selected to correlate with the development of the  $R \sim K$  relationship shown in Appendix A, Development of the  $R \sim K$  and  $R_w \sim K$  Relationships, for other LFRS.

$V_o$  is taken as the strength at the maximum considered drift of 2.0 inches from the bi-linear approximations of each of the 12 test walls. The allowable inelastic drift limit set in the ASCE 7-05 provided the basis for the maximum considered drift. To determine  $V_o$  as a percentage of the effective seismic weight,  $C_{so}$ , the effective seismic weight,  $W$ , was divided by the lateral strength,  $V_o$ , as shown in Equation 5.2.

$$C_{so} = \frac{W}{V_o} \quad \text{Eqn 5.2}$$

Finally, the ratio of  $C_{so}$  to  $C_{sd}$  was calculated and taken as the period dependant strength factor, as shown in Eqn 5.3

$$R_s = \frac{V_o}{V_d} = \frac{C_{so}W}{C_{sd}W} = \frac{C_{so}}{C_{sd}} \quad \text{Eqn 5.3}$$

The period-dependant strength factor for all 12 walls was individually evaluated.

### 5.2.2 Period-Dependant Ductility Factor, $R_\mu$

As stated in Section 2.4.2.1.2, many different researchers have developed a relationship between the ductility of a lateral force resisting system and the period-dependant ductility factor,  $R_\mu$ . All of the relationships return similar results, therefore the selected relationship used in calculating an  $R$  for a LFRS should not affect the results. The definition of  $R_\mu$  used in this study comes from research conducted in 1994 by Miranda and Bertero using data for alluvium sites. Alluvium sites were assumed to return more conservative  $R_\mu$  factor.

$$R_\mu = \frac{(\mu-1)}{\phi} + 1 \quad \text{Eqn 6.6}$$

$$\phi = 1 + \frac{1}{12T-\mu T} - \frac{2}{5T} e^{-2(\ln T-0.2)^2}, \text{ for alluvium sites} \quad \text{Eqn 6.7}$$

Where:  $\mu$  = ductility ratio,  $\Delta_a/\Delta_y$

$T$  = fundamental natural period of the structure

$$T = C_T h_n^{3/4} = 0.0951 \text{ seconds}$$

$$C_T = 0.20$$

$$h_n = 8.0 \text{ feet}$$

To determine the ductility ratio,  $\mu$ , the maximum considered displacement,  $\Delta_a$ , was set at 2.0 inches (50mm), and the yield displacement,  $\Delta_y$ , was taken from the bilinear approximations of the pushover analyses. Appendix B shows the bi-linear approximations for each wall.

Deflection of the out-of-plane walls was considered in selecting the maximum considered drift. The out-of-plane walls are walls that intersect the shear wall at a 90-degree angle, or perpendicular. Out-of-plane walls were considered stable at an out-of-plane story drift less than half the log diameter. At a maximum deflection of two inches, the out-of-plane wall is considered to remain stable. In addition, the ASCE 7-05 sets the maximum inelastic story drift for an eight foot tall shear wall story at approximately two inches in Table 12.12-1 (ASCE, 2005).

The fundamental natural period of the structure shown above was calculated according to the 1994 UBC. The ATC-19 definition used the 1994 UBC equation to approximate the fundamental natural period of buildings when developing the definition of  $R$ . The equation for the approximation of the fundamental natural period has been updated since the 1994 UBC. However, the changes shown in the equation given in the ASCE 7-05 and IBC 2006 do not affect the natural period of shear wall systems. The new equation, given in section 12.8 of the ASCE 7, includes adjusted coefficients to represent the flexibility and natural damping of steel moment frames, concrete moment frames and eccentrically braced steel frames. All other structural systems use coefficients equal to the coefficients set forth in the 1994 UBC.

### **5.2.3 Redundancy Factor, $R_R$**

As stated in Section 2.4.3 of this study, the ASCE 7 adjusted for system redundancy different than what was proposed by ATC-19. The  $R$  factors currently published by ASCE 7-05 do not reflect a redundancy factor. The recommended  $R$  factor is intended for use with the seismic provisions of the ASCE 7, so  $R_R$  is set to 1.0 for this recommendation.

### **5.2.4 Review of Calculated $R$**

$R$  was calculated for each sample wall. Table 5–3 shows the results of the calculations, using the parameters described in the previous subsections. All walls had calculated  $R$  factors higher than the maximum value of eight. In Table 5–3, two columns show  $R$  factors. The first column is the calculated wall  $R$  factor, considering the  $R_\mu$  and  $R_s$  determined based on the testing condition. The second  $R$  column considers the maximum allowable  $R$  factor, per the ATC 3-06 recommendations and current values in the ASCE 7-05. With this maximum imposed, all log shear walls considered have a value of 8.0 for  $R$ .

**Table 5–3 Calculation of  $R$  using wall data from Table 5–2**

Wall	Source	$R_{\mu}$	$R_S$	$R_R$	$R_{actual}$	$R_{code}$
1	Popovski	1.877	6.257	1.000	11.742	8.000
2	Popovski	1.877	6.453	1.000	12.109	8.000
3	Graham	1.876	50.184	1.000	94.151	8.000
4	Graham	1.877	22.320	1.000	41.889	8.000
5	Graham	1.876	14.617	1.000	27.426	8.000
6	Yeh	1.876	35.250	1.000	66.145	8.000
7	Yeh	1.876	13.726	1.000	25.756	8.000
8	Scott	1.877	7.679	1.000	14.416	8.000
9	Scott	1.877	4.707	1.000	8.836	8.000
10	Scott	1.877	4.604	1.000	8.641	8.000
11	Scott	1.877	6.858	1.000	12.875	8.000
12	Scott	1.877	4.954	1.000	9.301	8.000

Walls 3 through 7 returned very high  $R_S$  factors. These test walls do not have additional gravity at the top of the wall. As discussed in Chapter 4, the tests represented settled conditions with no through-rods or other take-up mechanism to maintain friction and no applied vertical loads. This set-up yielded a lower effective seismic weight so the percentage of weight resisted by the wall,  $C_{so}$ , was still higher than the required amount,  $C_{sd}$ , yielding a high strength ratio. The low effective seismic weight skewed the data.

**Table 5–4 Calculation of  $R$  using adjusted effective seismic weight for Wall 3 through Wall 7**

Wall	Source	$R_{\mu}$	$R_{\eta}$	$P_{\text{assumed}}$ (lbs)	$W$ (lbs)	$C_{so}$	$R_S$	$R_{\text{actual}}$	$R_{\text{code}}$
1	Popovski	1.877	1.000	1200.000	1973.144	0.911	6.257	11.742	8.000
2	Popovski	1.877	1.000	1200.000	1973.144	0.940	6.453	12.109	8.000
3	Graham	1.876	1.000	2248.089	2683.742	1.424	9.776	18.340	8.000
4	Graham	1.877	1.000	2248.089	2467.038	0.346	2.377	4.461	4.461
5	Graham	1.876	1.000	2248.089	2362.466	0.124	0.849	1.593	1.593
6	Yeh	1.876	1.000	2248.089	2394.149	0.627	4.301	8.071	8.000
7	Yeh	1.876	1.000	2248.089	2394.149	0.240	1.649	3.094	3.094
8	Scott	1.877	1.000	2248.089	3259.481	1.069	7.339	13.777	8.000
9	Scott	1.877	1.000	2248.089	3259.481	0.655	4.498	8.444	8.000
10	Scott	1.877	1.000	2248.089	2753.785	0.653	4.483	8.415	8.000
11	Scott	1.877	1.000	2248.089	3259.481	0.897	6.155	11.555	8.000
12	Scott	1.877	1.000	2248.089	3259.481	0.690	4.735	8.889	8.000

A second calculation, using higher effective seismic weight, was performed on Wall 3 through Wall 7 (Table 5–4). The results indicate behavior similar to the other walls tested, which included axial loads in the test. Wall 4, Wall 5 and Wall 7 had  $R$  factors less than 8.0. Walls 3 and 4 have higher aspect ratios than the 1:1 aspect ratio limit set forth in ICC 400-2007 and were not considered in determining the recommended  $R$  factor. Walls 3 and 4 were excluded because this recommendation considers all requirements of the ICC 400-2007, and walls with higher aspect ratios will not meet the ICC 400-2007 requirements for log shear walls. Wall 7 has a large door opening, which created slender wall sections on either side. The behavior of this wall indicates that a procedure similar to that used for perforated shear walls in conventionally framed construction, given in Section 2305.3.8.2 of the 2006 IBC, may be useful for the design of log shear walls.

The statistical analysis of the returned  $R$  factors, using the actual calculated values from Table B-4, is as follows:

$$R_{\min,1} = 1.59 \text{ (Wall 5, 4:1 Aspect Ratio)}$$

$$R_{\min,2} = 3.09 \text{ (Wall 7, large door opening)}$$

$$R_{\text{avg}} = 9.21$$

$$\text{Standard Deviation, } \sigma = 4.721$$

$$\text{Coefficient of Variation, } c_v = 0.513$$

$$\text{Variance, } \sigma^2 = 22.285$$

Based on the statistics of the sample results and considering the effects of door openings, the  $R$  value for log shear walls is recommended as 6.0. Energy dissipation due to friction is inherent in this  $R$  recommendation. Though the strength capacity added from the friction force does not need to be considered in the lateral strength capacity of a log shear wall however, energy dissipation due to friction must be considered in the behavior. Ignoring friction in the  $R$  for log shear walls would not represent true system behavior.

This recommendation ( $R = 6.0$ ) assumes that not all of the walls in a log shear wall LFRS will have door openings and the system will comply with the following recommended construction:

- 1:1 aspect ratio shall be used, as required by ICC 400-2007, Section 406.1

- Vertical line of fasteners shall be placed at each end of the shear wall and around openings or at 48" on center if no wall openings
- End distance of fasteners shall be at least twice that required by the NDS
- Either lag screws or through rods shall be used for fasteners
- Holes for through rods shall not be oversized
- Take-up springs or manual tensioners shall be installed on through-rods at plate log to adjust for settlement and shrinkage of courses over-time
- Minimum penetration for lag screws shall be 12 times the diameter of the lag screw
- The diameter of the lag screw used should not exceed half an inch
- Lag screw threads should not be located in the shear plane between log courses
- If partial rounds are used for the sill logs anchor bolts shall extend through the sill log and connect into the first full course

These construction parameters correlate with the test specimens used to determine the *R* recommendation. As additional research on log shear walls of different constructions is completed, these parameters may be adjusted if the new testing returns similar behaviors to the testing used in this recommendation.

Log shear wall construction has good energy dissipation, is capable of large lateral deflections and has high lateral strength when friction is considered. Large openings, such as doors, in log shear walls reduce the ductility and lateral strength. Log shear walls with small openings, such as square windows with an area less than 25% of the wall opening, can have higher lateral strength and stiffness if the above construction is followed. Considering the above factors, the recommended *R* ensures the life safety of occupants while taking advantage of log shear wall characteristics. Taking advantage of the log shear wall characteristics will yield more economical designs, requiring fewer log-log connectors in each shear wall.

## 6 Recommendations for Future Research

Additional research is required to fully explain the behavior of a log shear wall system and develop an  $R$  for various construction methods of log shear walls for adoption into model building codes. The recommendation of an  $R$  of 6.0 should serve as a basis of design and for further research. Any  $R$  used in design should be carefully determined after looking at performance of log shear walls demonstrated in physical testing which represent the constructed condition of the structure being designed.

For further research, archetypes should be developed, looking at the performance of all common connector types used. Both size and placement of openings should be considered in developing the archetypes. The methodology of ATC-63 should be followed to develop an  $R$  factor that could then be considered by model building codes. The following is a list of recommended research:

- The effect of multiple openings of varied sizes within the same wall section
- Determination of the maximum opening percentage that does not result in a loss of lateral strength
- The effect of combined connectors, i.e. lag screws at ends and in the body of the wall with through rods on each side of openings
- Long wall sections, to determine ideal spacing of vertical reinforcing
- Varied wood species of identical test construction
- The effect on stiffness of multiple intersecting walls along one shear wall (at ends and points along length)
- The effect on stiffness by intersecting interior walls of conventional wood framing (both designed to accommodate lateral displacement and not designed to accommodate lateral displacement)

All of the above variables will affect the behavior of a log shear wall. In the author's opinion, the effect of opening size and development of a "perforated shear wall" method are the two topics which should be addressed first. The effects of the other variables can be avoided in design of the shear wall, or the element could be ignored for conservatism. Since the architecture of the structure and the placement of openings are not within the engineer's control, further research should be conducted on this variable first.

## 7 Conclusions

Seismic codes continually change as engineers understand more about earthquake forces, motions and effects on structures. From the earliest code recognizing a need for seismic design in 1927 to the current methods, the life safety of building occupants has been the main concern. ATC-63 methodology will require extensive research to determine the performance of building structures under seismic loads. However, this methodology will result in seismic coefficients that more closely represent the behavior of the most common construction types of each building system not the assumed values currently published in building codes. The majority of the  $R$  factors published in current building codes are based on engineering judgment and expectations.

Current codes do not provide specific guidance on designing log shear walls as a LFRS. With the methodology set forth in ATC-63, it is possible to determine a specific  $R$  for log shear wall systems using various construction methods however, research has not been conducted to the extent required for the ATC-63 methodology. As a result, engineers use assumptions on the behavior of a log shear wall in design usually choosing an  $R$  factor ranging from 2.0 to 5.5. These assumed  $R$  factors vary based on the construction method, type of log-log connector used and judgment of the engineer.

The main goal of this research was to develop a recommended  $R$  factor, representative of log shear wall behavior. The ATC-63 methodology of selecting multiple construction variations and comparing behavior was the basis for the recommendation for  $R$  of 6.0 presented in this work. The data used came from previously published results from either corporate or academic researchers. Through the comparisons of the test results and the calculations shown in Chapter 5 of this study, it was demonstrated that current assumptions on log shear wall behavior might be more conservative in nature than thought by design professionals. Utilizing the higher  $R$  of 6.0 will yield more economical structures by lowering the required capacity of log shear walls and requiring fewer connectors without compromising the life safety of the occupants.

The recommended  $R$  of 6.0 was determined by evaluating six different tests conducted on log shear walls and applying the provisions of ATC-19 to the results. In addition, the ICC 400 was referenced as a construction standard for log structures. The out-of-plane walls were considered to remain stable as long as the maximum considered inelastic drift was less than half

the log diameter. In this study, the maximum considered inelastic drift was set at two inches, keeping the out-of-plane walls within the recommended limit for stability.

This recommended  $R$  of 6.0 is limited to the construction type described in section 5.2.4.

This construction is:

- 1:1 aspect ratio shall be used, as required by ICC 400-2007, Section 406.1
- Vertical line of fasteners shall be placed at each end of the shear wall and around openings or at 48" on center if no wall openings
- End distance of fasteners shall be at least twice that required by the NDS
- Either lag screws or through rods shall be used for fasteners
- Holes for through rods shall not be oversized
- Take-up springs or manual tensioners shall be installed on through-rods at plate log to adjust for settlement and shrinkage of courses over-time
- Minimum penetration for lag screws shall be 12 times the diameter of the lag screw
- The diameter of the lag screw used should not exceed half an inch
- Lag screw threads should not be located in the shear plane between log courses
- If partial rounds are used for the sill logs anchor bolts shall extend through the sill log and connect into the first full course

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## Appendix A. Development of the $R \sim K$ and $R_w \sim K$ Relationships

### A.1 $R \sim K$ Relationship (ATC 3-06, 1978)

The fundamental period of the most efficient LFRS (special steel moment frames) was set as one second. The lateral seismic base shear from the 1976 UBC was multiplied by numerical factors to account for differences between strength and allowable stress methods, then set equal to the base shear proposed by ATC 3-06. The equality was then solved for  $R$  in terms of  $K$ , as follows.

$$V_{1976\text{ UBC}} (1.67/1.33) = V_{\text{ATC 3-06}} / 0.9$$

$$Z I K C S_i W \frac{1.67}{1.33} = \frac{1.2 ASW}{0.9RT^{0.67}} \quad \text{Eqn A.1}$$

Substituting the following into the above equation:

$$Z = 1.0$$

$$I = 1.0$$

$$T = 1.0$$

$$S_i = 1.5$$

$$A_v = 0.4$$

$$S = 1.2$$

$$0.1256 K = \frac{0.67}{R} \quad \text{Eqn A.2}$$

$$R = \frac{5.1}{K} \quad \text{Eqn A.3}$$

Because the 1976 UBC listed the  $K$  factor for moment resisting frame systems as 0.67, the corresponding  $R$  factor within ATC 3-06 using the relationship above was determined as 8.0.

## A.2 $R_w \sim K$ Relationship (SEAOC Blue Book, 1988)

The lateral seismic base shear from the 1985 UBC was set equal to the base shear proposed by SEAOC. The equality was then solved for  $R_w$  in terms of  $K$ , as follows.

$$V_{1985 \text{ UBC}} = V_{\text{SEAOC } 1988}$$

$$\mathbf{ZIKCSW} = \frac{\mathbf{ZI CW}}{\mathbf{R_w}} \quad \mathbf{Eqn A.4}$$

Substituting the following into the above equation:

$$Z = 1.0 \text{ (1985 UBC- left side)}$$

$$Z = 0.4 \text{ (1988 SEAOC- right side)}$$

$$I = 1.0$$

$$CS = 0.14 \text{ (1985 UBC- left side)}$$

$$C = 2.75 \text{ (1988 SEAOC- right side)}$$

$$\mathbf{K (0.14) C} = \frac{\mathbf{2.75*0.4}}{\mathbf{R_w}} \quad \mathbf{Eqn A.5}$$

$$\mathbf{R_w} = \frac{\mathbf{7.86}}{\mathbf{K}} \approx \frac{\mathbf{8}}{\mathbf{K}} \quad \mathbf{Eqn A.6}$$

Because the 1976 UBC listed the  $K$  factor for moment resisting frame systems as 0.67, the corresponding  $R_w$  factor within ATC 3-06 using the relationship above was determined as 12.0.

Relating  $R$  and  $R_w$  one can see that:

$$\mathbf{R_w} = \mathbf{1.54 R} \quad \mathbf{Eqn A.7}$$

## Appendix B. Pushover Results and Bi-linear Approximations of the Test Walls

The factors used to calculate  $R$ , presented in Chapter 6 were taken from the following tables and figures. The twelve walls described in Table B-1 make up the testing sample. Pushover results from the twelve walls were analyzed to produce Table B-2. The pushover results are provided following the tables, with the bi-linear approximations shown as dashed lines, starting with Figure B-1. An  $R$  for each of the twelve walls was developed separately to illustrate the behavior of the different constructions. A statistical analysis of the 12 separate  $R$  factors is provided in section 6.4. All data values taken from the pushover results were converted, using standard conversion factors, from millimeters to feet and Newtons to pounds if needed.

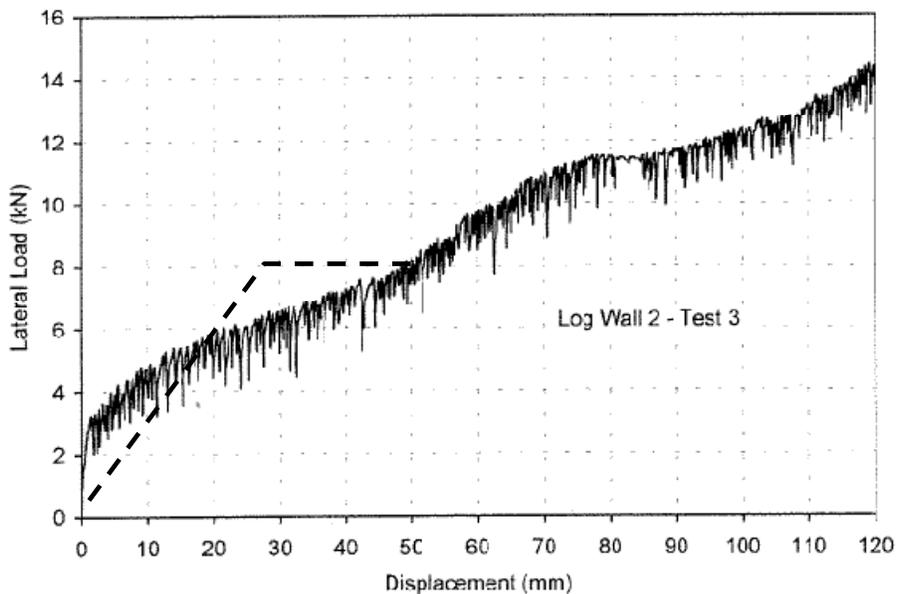


Figure B-1 Wall 1- Pushover results with bi-linear approximation, (Popovski, 2002)

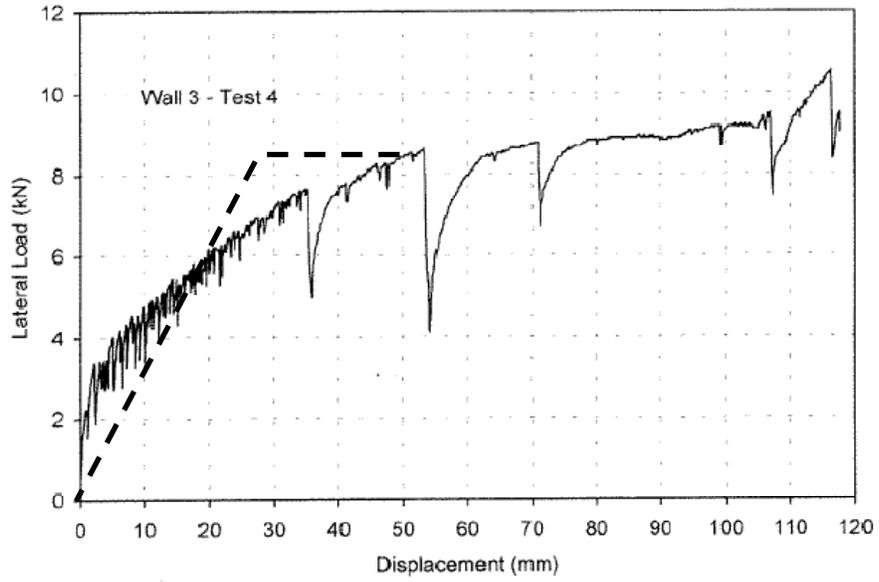


Figure B-2 Wall 2- Pushover results with bi-linear approximation, (Popovski, 2002)

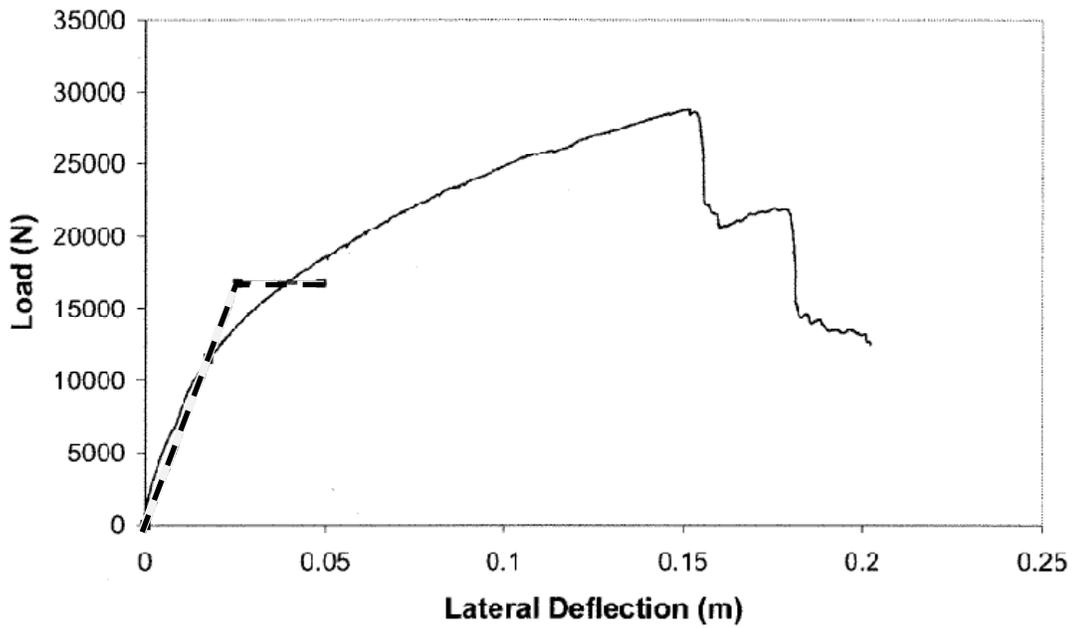


Figure B-3 Wall 3- Pushover results with bi-linear approximation, (Graham, 2007)

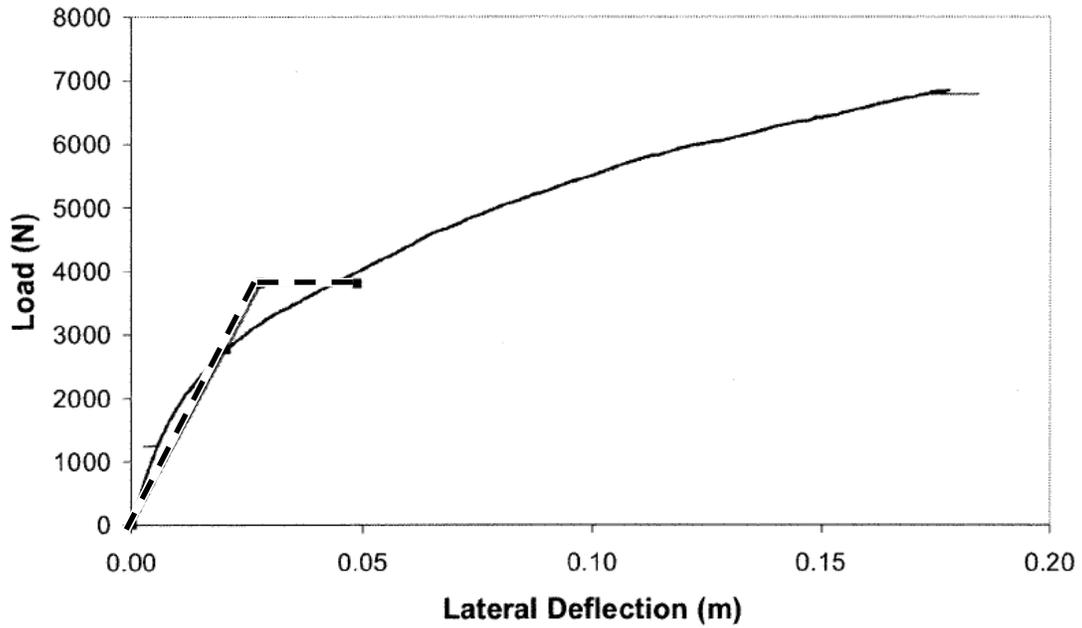


Figure B-4 Wall 4- Pushover results with bi-linear approximation, (Graham, 2007)

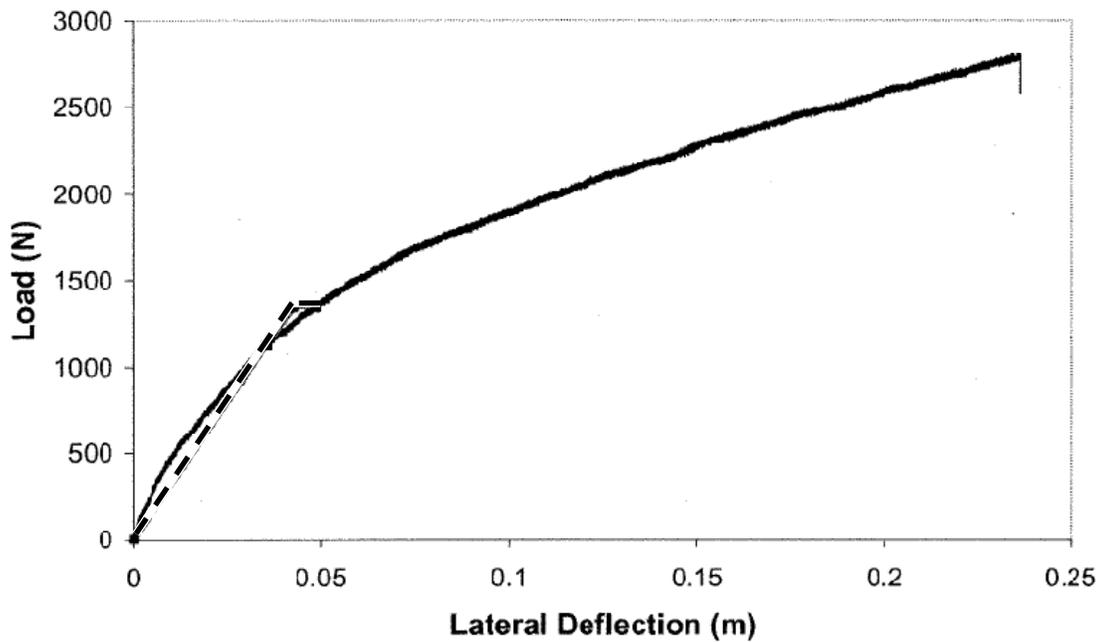
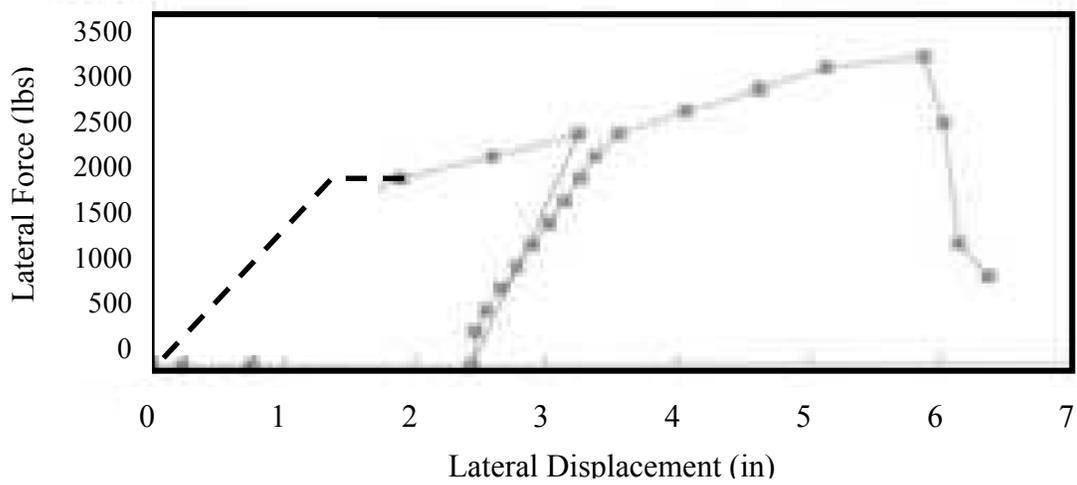
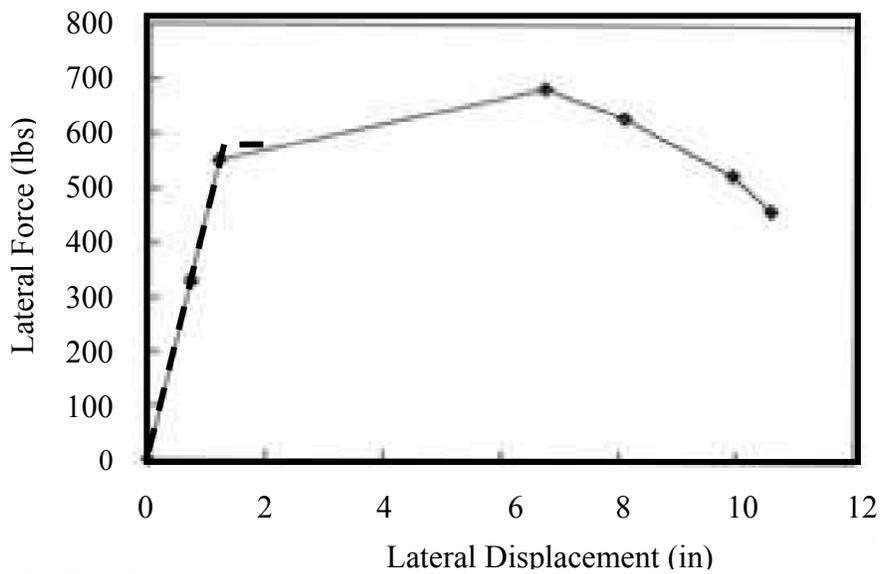


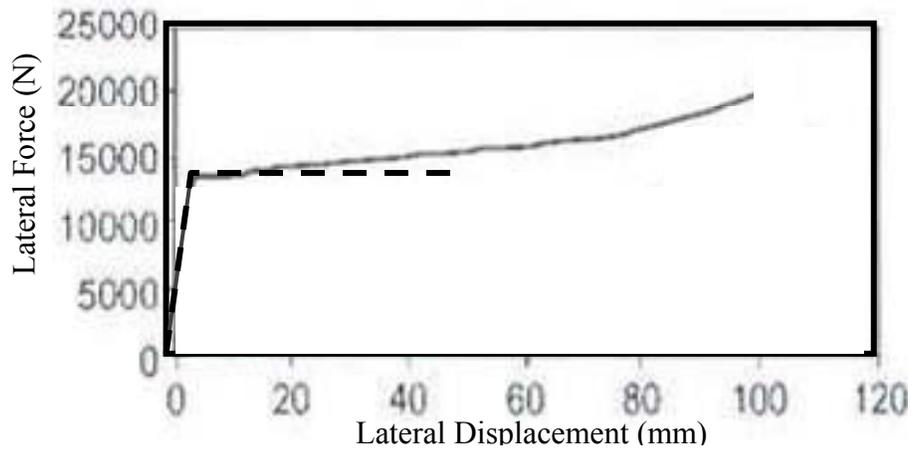
Figure B-5 Wall 5- Pushover results with bi-linear approximation, (Graham, 2007)



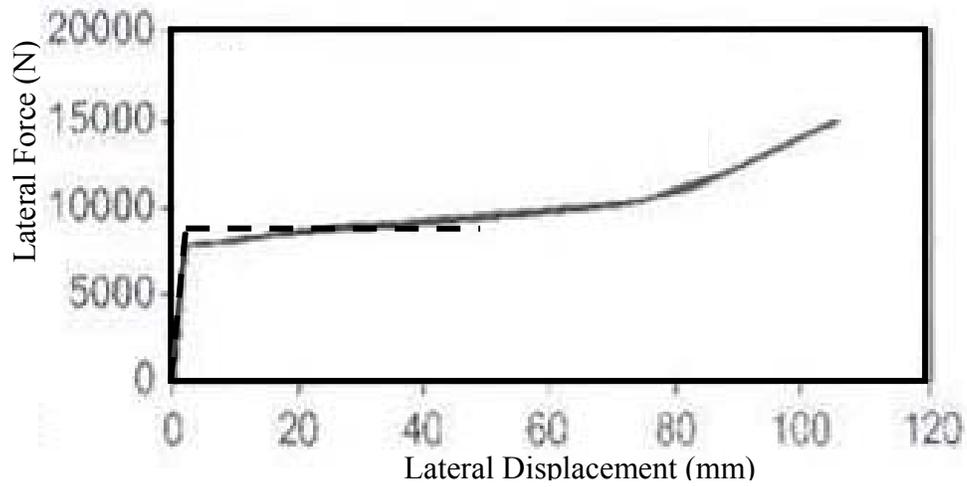
**Figure B-6 Wall 6- Pushover results with bi-linear approximation, (Yeh et al, 2006)**



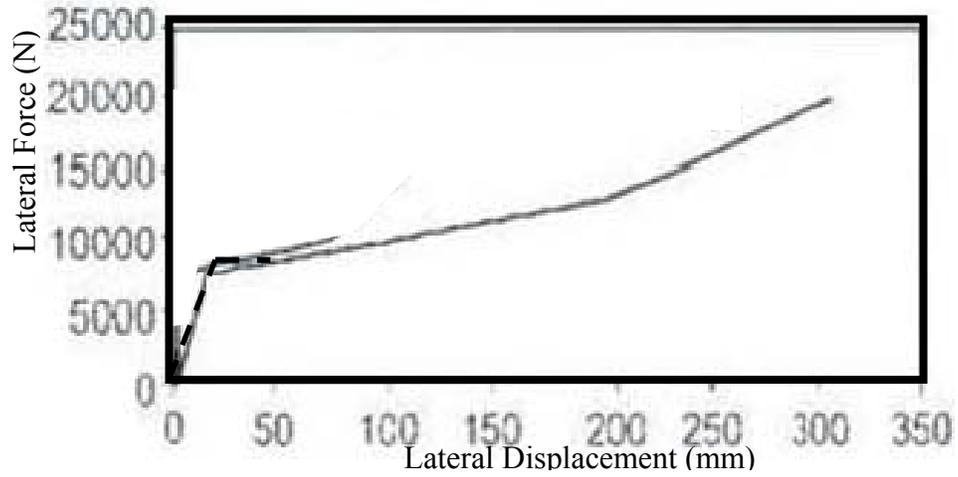
**Figure B-7 Wall 7- Pushover results with bi-linear approximation, (Yeh et al, 2006)**



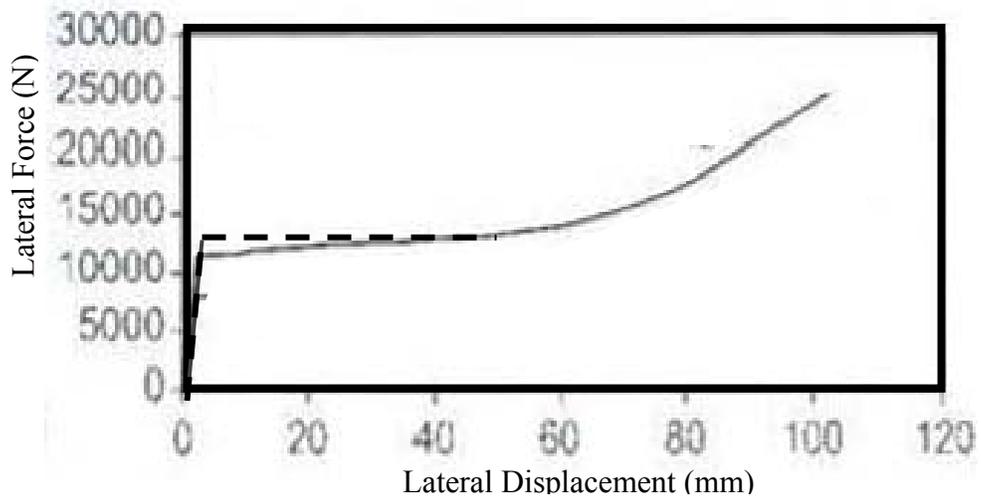
**Figure B-8 Wall 8- Pushover results with bi-linear approximation, (Scott et al, 2005)**



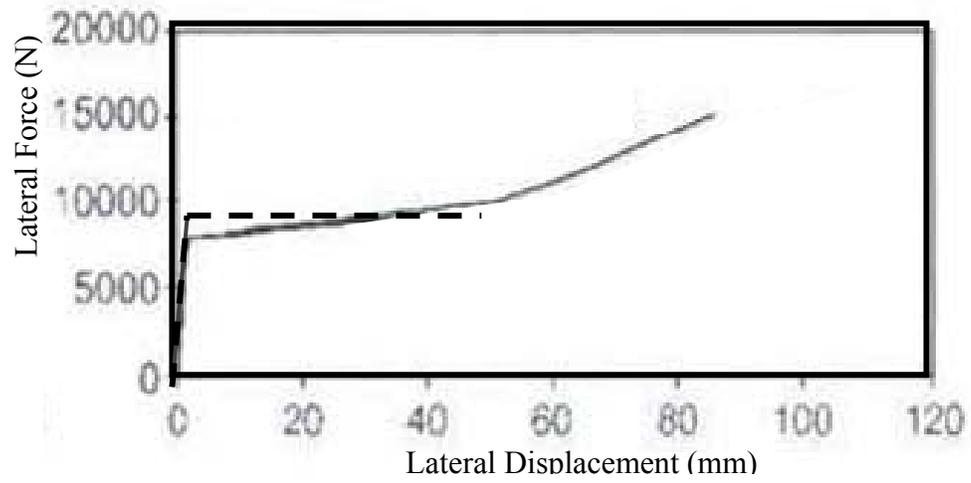
**Figure B-9 Wall 9- Pushover results with bi-linear approximation, (Scott et al, 2005)**



**Figure B-10 Wall 10- Pushover results with bi-linear approximation, (Scott et al, 2005)**



**Figure B-11 Wall 11- Pushover results with bi-linear approximation, (Scott et al, 2005)**



**Figure B-12 Wall 12- Pushover results with bi-linear approximation, (Scott et al, 2005)**

## **Appendix C. Image/Figure Permissions**

The following correspondence was generating in obtaining permission to reprint figures and images from other works. All figures generated by the author unless noted in the caption.

### **C.1 Permission for republication of figures from UBC**

From: Colleen Petry-Johnson <cpjohnson@iccsafe.org>

To: sbutler@ksu.edu

Date: Friday, March 5, 2010; 2:18 PM

Ms. Kessler,

Thank you for contacting the Code Council! Please consider this permission for one-time use of UBC seismic hazard maps for purposes of completion of your thesis. We wish you the best on your thesis and if there is anything else we can do to assist, please feel free to contact me.

Colleen Petry-Johnson  
Production Coordinator

International Code Council, Inc.  
Chicago District Office  
4051 West Flossmoor Road  
Country Club Hills, IL 60478-5795

## C.2 Permission for republication of figures from ATC publications

**Samantha Kessler** <samqbutler@gmail.com>

To: Ayse Hortacsu <ayse@atcouncil.org>

Date: Wednesday, March 31, 2010

Mr. Hortacsu,

I am a graduate student at Kansas State University. I would like to use some figures/images from ATC projects in my thesis. The information is as attached in the word document.

The university does not have a release form. Email permission is accepted and preferred.

Thanks,

Samantha Kessler

[sbutler@ksu.edu](mailto:sbutler@ksu.edu)

[samqbutler@gmail.com](mailto:samqbutler@gmail.com)

316-200-8444

**Ayse Hortacsu** <ayse@atcouncil.org>

To: Samantha Kessler <samqbutler@gmail.com>

Date: Tuesday, April 6, 2010 3:57PM

Dear Samantha,

Thank you for your inquiry.

Please use the following citations in your thesis and you may have ATC's permission to use the figures.

ATC. (1995). *ATC-19 Structural Response Modification Factors*, Redwood City, California.

ATC. (1978). *ATC-3 06 Tentative Provisions for the Development of Seismic Regulations for Buildings*, Redwood City, California.

Good luck with your thesis.

Ayse

### **C.3 Shear tests on Log Shear Walls- Shrestha**

From: Samantha Butler <samqbutler@gmail.com>

To: tgorman@uidaho.edu

Date: Thursday, February 12, 2009; 2:46 PM

Dr. Gorman,

My name is Samantha Butler and I am a graduate student in Architectural Engineering at Kansas State University, Manhattan KS. I have recently started my graduate studies and am working on developing my thesis and required research. Currently, I am interested in quantifying the seismic performance factors (R, Cd, Omega) for log bearing shear walls. One of the articles I read recently used your testing (Shear tests for log home walls, Forest Products Society, Session 5, 2002) as a source. Would you be willing to share you testing parameters and data with me? In addition to using your research as a source, I would like to use the data to calibrate computer models I will use for my research.

At this point, my research will be computer based, using calibrated models (possibly finite element models) to simulate performance of shear wall systems under forces from the "Far-Field" records recommended by FEMA/NEHRP in ATC-63, "Quantification of Building Seismic Performance Factors." Due to time, space and finances, I will be unable to generate my own physical tests in order to calibrate my computer models, so I am collecting data from other research involving log shear walls.

I appreciate your time and assistance in my research. If this is not something that you are willing to share or assist with (by answering questions via email) I understand, but ask that you let me know via email. Thanks for your time.

Sincerely,

Samantha Kessler

From: Gorman, Tom <TGORMAN@uidaho.edu>

To: Samantha Butler <samqbutler@gmail.com>

Date: Thursday, February 19, 2009; 4:07 PM

Dear Samantha,

Sorry to take so long in getting back to you, but the log shear wall testing we did never progressed after Deepak Shrestha left Washington State University. I was able to locate a manuscript that he and I put together, but must admit that the shear analysis portion was not my role, and so I am not really able to provide much more than the attached manuscript. We never did publish the results of the study.

You are certainly welcome to use whatever you wish from the manuscript. If I can find the PowerPoint that we used at the Forest Products Society meeting in 2002 I will forward that on to you, but it was based on the data and materials contained in the manuscript.

I am glad that you are looking at this area; there is still much to be done to provide guidelines for log home manufacturers and builders to meet seismic requirements in an economical and effective manner.

Thomas M. Gorman, Ph.D., P.E.

Professor and Head

Department of Forest Products

University of Idaho

PO Box 441132

Moscow, ID 83844-1132

office: 208-885-7402

## C.4 "Testing of Lateral Resistance of Handcrafted Log Walls. . . "

From: Samantha Kessler <samqbutler@gmail.com>

To: marjan@van.forintek.ca

Date: Wednesday, March 3, 2010, 11:20 AM

Dr. Popovski,

I am a graduate student at KSU working on my master's thesis. I am attempting to compile an extensive literature review of testing completed on log shear walls within my thesis and extrapolate a recommended construction method and seismic coefficient based on the physical research. I have recently read and cited your research for the ILBA titled "Testing of Lateral Resistance of Handcrafted Log Walls Phase I and II" and I am interested in using a few figures to better describe the testing you completed. In order to include figures, I need written permission from the author. As the main author of this report, are you willing to give my permission to use the figures and photographs contained in the document in my master's thesis?

Regards,

Samantha Kessler

Fom: Marjan Popovski <Marjan.Popovski@fpinnovations.ca>

To: Samantha Kessler <samqbutler@gmail.com>

Date: Wednesday, March 3, 2010, 11:30 AM

Dear Samantha,

Thanks for your E-mail. You are welcome to use any photo from the report as needed. Just make the proper reference.

Best regards from the Olympic city,

Marjan Popovski Ph.D., P.Eng.

Senior Scientist & Quality Manager

Building Systems Department

FPInnovations-Forintek

## **C.5 Log Bearing Walls Research- by Randy Scott**

From: Samantha Kessler <samqbutler@gmail.com>

To: "Thomas H. Miller" <millert@enr.orst.edu>

Date: Wednesday, March 3, 2010; 11:26 AM

Professor Miller,

I contacted you a little over a year ago regarding the research completed by Randy Scott in 2004. As I am finishing my thesis, I am finding that including images (photographs of testing apparatus, graphs of results, etc) of the physical testing I am referencing is useful to convey the accuracy of my summary and usefulness of the information. In order to include images from a previously published work, I need written permission from the author or co-author. Are you able/willing to give me permission to use images and figures from Scott's thesis and the associated articles published in the Forest Products Journal?

Regards,

Samantha

From: Tom Miller <thomas.miller@oregonstate.edu>

To: Samantha Kessler <samqbutler@gmail.com>

Date: Wednesday, March 3, 2010; 12:28 PM

Yes, you have my permission.

Thomas H. Miller, PhD, PE

Associate Professor

Assistant Head for Civil Engineering

School of Civil and Construction Engineering

220 Owen Hall

Oregon State University

Corvallis, OR 97331

## C.6 Images permission- Thesis by Drew Graham

From: Samantha Kessler <samqbutler@gmail.com>

To: bender@wsu.edu

Date: Wednesday, March 3, 2010; 11:15 AM

Mr. Bender,

I am a graduate student at KSU working on my master's thesis. I am attempting to compile an extensive literature review of testing completed on log shear walls within my thesis and extrapolate a recommended construction method and seismic coefficient based on the physical research. I have recently read and cited Drew A. Graham's thesis titled "Performance of log shear walls and lag screw connection subjected to monotonic and reverse cyclic loading." Graham generated excellent figures showing his results and I would like to be able to use them in my thesis. As you know, in order to include figures from a previously published work, I need permission from the author. As co-chair of Graham's work, can you provide permission, or does Graham have to provide it? If I need to contact Graham to get permission, do you have any contact information for him?

Thank you for your time.

Samantha Kessler

From: Don Bender <bender@wsu.edu>

To: Samantha Kessler <samqbutler@gmail.com>

Date: Wednesday, March 3, 2010; 11:55 AM

Samantha - feel free to use figures from Drew's thesis and you can cite his MS thesis as the source. We also have two publications out on this work

Graham, D.A., D.M. Carradine, D.A. Bender and J.D. Dolan. 2010. Performance of log shear walls subjected to monotonic and reverse-cyclic loading. ASCE Journal of Structural Engineering 136(1):37-45.

Graham, D.A., D.M. Carradine, D.A. Bender and J.D. Dolan. 2010. Monotonic and reverse-cyclic loading of lag screw connections for log shear wall construction. ASCE Journal of

Materials in Civil Engineering 22(1):88-95.

Donald A. Bender, P.E., Ph.D.

Professor, Civil & Environmental Engineering and  
Director, Composite Materials & Engineering Center  
Washington State University

PO Box 641806

Pullman, WA 99164-1806