Seismic Design of a Current Woodframe Structure and Study of Innovative Products and Damping Systems in Wood Construction

by

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Submitted to the Department of Civil and Environmental Engineering on May 14, 2007 in partial fulfillment of the requirements for the Degree of Masters of Engineering in Civil & Environmental Engineering

Abstract

Wood structures have seen resurgence in popularity over the past several decades, especially in Western States of America, such as California. The industry keeps creating new structural wood products of exceptional strength, versatility, and reliability. Wood-frame structures offer a more sustainable answer, but need to be carefully detailed in high seismic zone.

The objective of this work is to describe the seismic design of a current woodframe structure. Moreover, this thesis aims to present the innovation occurring in the market of wood construction. New engineered wood products are introduced as well as a review of the new developments and researches that are being made to incorporate damping systems such as viscoelastic and hysteretic dampers, in the ultimate goal of obtaining an optimum earthquake-resistant wood structure.

Thesis Supervisor: Jerome J. Connor
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My parents have always told me that knowledge will benefit your family, your society, and most of all, yourself. I truly believe that my studies have proved to be the most valuable tools in facing challenges that have come my way and are sure to greet me in the coming years.

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Introduction

Woodframe construction is the predominant method for building homes and multi-family structures in the United States; in California, about ninety percent of residential construction consists of wood structures. For centuries, wood has been favored as a building material because of its strength, economy, workability, and is also environmentally friendly. Finally, wooden buildings have a good reputation when subjected to seismic events. They can resist catastrophic earthquakes while sustaining only minimal damage.

Woodframe construction is being used, more widely now, in commercial and industrial buildings. This market growth causes wood to be put off-limits to harvesting. Higher quality trees are being used, ultimately restricting the availability of high-quality lumber. Furthermore, sawn lumber limits the size and grade that can be used in construction. Thus, when loads become large or the span becomes longer, the use of sawn lumber becomes unfeasible. This is where engineered wood products become of critical and practical use in the construction market. Through technology, smaller, faster growing, lower quality trees are engineered to become excellent wood products. These products have greatly expanded building options and methods in all forms of residential and commercial construction.

Woodframe structures seem to be safer to live in, in seismic areas, compared to traditional heavier buildings. However, while building codes and standards emphasize life safety issues, structural and non structural damage can cause economical problems. Furthermore, the height of woodframe construction is currently limited to approximately four stories. This restriction is mainly due to uncertainties in understanding the dynamic response of taller woodframe construction and the non-structural limitations. New challenges are being faced in developing a new seismic design philosophy based on performance-based design. In addition to this philosophy, supplemental innovative damping systems are being studied to obtain optimum earthquake-resistant wood structures.

The objective of this work is to provide an overview of a current woodframe construction, presenting the seismic design requirements, detailing the different structural components of the lateral force resisting system, and designing the lateral framing of a typical four-story apartment located in a high seismic zone. Moreover, the thesis provides information on the recent engineered wood products. It also gives an overview of the different techniques and researches that have been started in the area of providing innovative damping systems to obtain an optimum earthquake-resistant wood structure.

Scope of Chapter I

Chapter 1 provides an overview of a current woodframe construction. The chapter provides an introductory design process to the estimation of lateral seismic loads and the associated structural behavior of low-rise wood buildings. These seismic design requirements are based on the provisions of the 1997 Uniform Building Code (as well as the 2001 California Building Code). The chapter ends with the seismic design of a woodframe four-story apartment located in Los Angeles, California, region of high seismic area.

Scope of Chapter II

Chapter 2 provides a detailed description of the new engineered wood products available in the market. These products are able to enhance the structural performance of the building, creating a greater market growth in the residential and commercial construction. New technologies are discussed utilizing traditionally less desirable species, smaller trees, and lower quality trees, but resulting in the production of excellent wood products. This chapter also raises the issue of sustainability. Indeed, engineered wood products (EWP) offer higher yields from a given log. This would permit the reach of a more sustainable environment in a much polluted industry.

Scope of Chapter III

Chapter 3 provides a literature review of the different techniques and researches that have been started in order to obtain an optimum earthquake resistant structure. The chapter describes innovative damping systems that are being studied to understand the improvement on a woodframe construction. Moreover, this part introduces the new philosophy that engineers should start to learn when designing wood structures.

I. Overview of Current Wood-frame Construction

A. Seismic Design Requirements

Earthquake activities result in various types of ground motion as seismic waves. When passing through a structure, those waves subject the structure primarily to lateral forces and to a lesser degree to vertical forces. The structure should be able to withstand vertical and lateral movements without losing strength; it needs to resist deformations without developing high stress concentrations.

The objective of this section is to give an introductory design process to the estimation of lateral seismic loads and the associated structural behavior of low-rise wood buildings. These seismic design requirements are based on the provisions of the 1997 Uniform Building Code (similar to 2001 California Building Code).

This motion occurs at the base of the structure resulting in dynamic loads. Those loads are then distributed throughout the structure based on the stiffness of each structural elements and mass distribution (stiffness representing restoring forces and distribution of mass being the inertial forces). In order to account for those seismic loading, the most accurate way would be to run some dynamic analysis. However, for the design of low-rise wood building, dynamic analysis can be replaced with simplified analytical techniques, provided in the building codes such as equivalent static force or equivalent lateral force procedures.

1. Equivalent Static Lateral Procedure

This procedure entails applying static loads on a structure with magnitudes and direction approximating the effects of dynamic loading caused by earthquakes. Those forces are concentrated lateral forces occurring at each floor and roof levels, where the mass concentration is at its highest. Additionally, the higher the elevation, the larger the forces are.

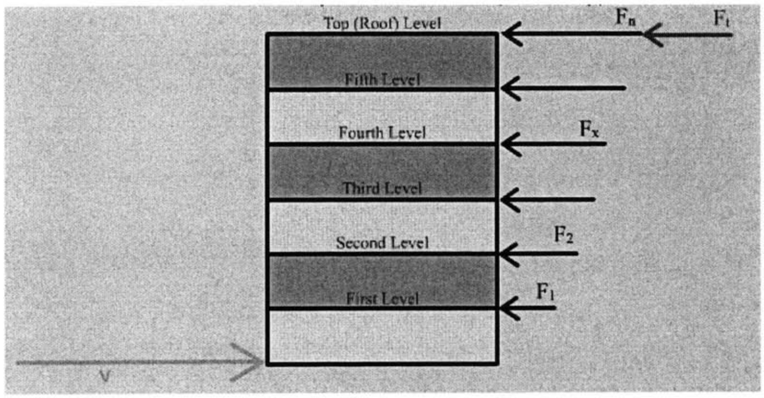


Figure 1: Equivalent Static Lateral Force Schematic
(CUREE Caltech Project, 2000)

Equivalent Static Lateral Forces	Force Description
V	Base Shear Force (associated with ground motion at base of structure)
Fx	Lateral story force applied at each story level
Ft	Additional lateral force applied at the top level of structure (UBC)

Table 1: Equivalent Static Lateral Forces Description

The distribution of the lateral story forces Fx corresponds to the fundamental mode of vibration of a cantilevered structure. Ft, the additional lateral force at the top level, is here to represent the collection of the higher modes of vibration. It can also be noted that the summation of Fx and Ft should be equivalent to the base shear force, V, applied to the structure due to seismic ground motion.

UBC provisions (and CBC provisions) are developed on the concept of the base shear. This force represents the horizontal reaction at the base of the building required to balance the inertia force. This force is developed over the height of the building due to the earthquake. It is the result of the maximum lateral force expected from a seismic ground motion at the base of the structure. This force is calculated based on five criteria: soil conditions at the site, proximity to geological faults, the level of ductility and overstrength depending on the total weight of structure, the fundamental period of vibration of the structure under dynamic loading, and the probability of major seismic ground motion.

a) Probability of major seismic ground motion

This criterion can be assessed by the graph found below (Figure 2). The map is divided into seismic zone ranging from Zone 0 (region with no seismic activity) to Zone 4 (region with high seismic activity).

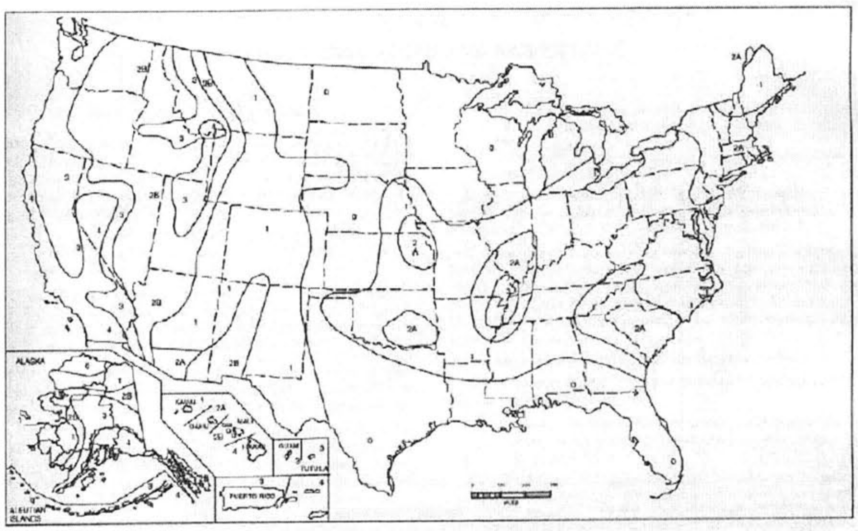


FIGURE 16-2—SEISMIC ZONE MAP OF THE UNITED STATES
For areas outside of the United States, see Appendix Chapter 16.

Figure 2: Seismic Zone Map of the United States

(*UBC 1997, Vol. 2, CHAPTER 16, DIV. III, SESMIC DESIGN.FIGURE 16-2)

It is clear here that California is situated in a Zone 4, increasing the probability of suffering from seismic ground motion.

A structure, designed in a Zone 4, will therefore need to follow certain formulas in calculating the base shear:

$$\mathbf{V = Cv\ I\ W / R\ T} \qquad \text{(UBC Equation 30-4)}$$

In addition to this, lower and upper bound values are calculated as follow. Lower bounds tend to represent structures with relatively large fundamental periods, while the upper bound tends to govern for structures with low fundamental periods.

V < 2.5Ca I W / R	Upper Bound - (UBC Equation 30-5)
V > 0.11Ca I W / R	Lower Bound - (UBC Equation 30-6)
V > 0.8ZNv I W / R	Lower Bound for Zone 4 -(UBC Equation 30-7)

Terms	Description	Criteria Correspondence
Cv	Seismic Coefficient (for velocity controlled region)	(1): soil conditions at the site (2): proximity to geological faults
I	Importance Factor	-
W	Total Seismic Dead Load	-
R	Ductility & Over strength Factor	(3): the level of ductility and over strength depending on the total weight of structure
T	Fundamental Period of Structure	(4): the fundamental period of vibration of the structure under dynamic loading
Ca	Seismic Coefficient (for acceleration controlled region)	(1): soil conditions at the site (2): proximity to geological faults
Z	Zone Factor – Magnitude of Peak Acceleration	(2): proximity to geological faults
Nv	Near-Source Factors (for Zone 4)	(2): proximity to geological faults

Table 2: Description of Terms found in Base Shear Calculations

I: Importance Factor

This factor is an additional safety factor used to increase the load based on the occupancy of the structure. For example, hospitals, emergency buildings, hazardous facilities have an importance factor of 1.25. This is a precaution to make sure those buildings will remain operational during earthquake activities.

However, a residential or office wood structure usually corresponds to a standard building and its resultant importance factor is 1.00. UBC Table 16-K (Appendix p.79) summarizes the different importance factor depending on the occupancy of the structure to be designed.

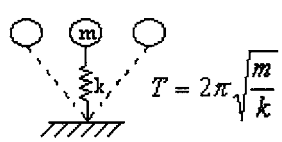
b) Fundamental period of vibration of the structure under dynamic loading

The fundamental period of the building can be estimated using the information given in UBC Section 1630.2.2.

Indeed, UBC provides a simplified method for calculating T, which is based on the height of the building, h_n (in feet):

$T = Ct (h_n)3/4$ (UBC Equation 30-8)

($Ct = 0.02$ for wood structures)



In general: Low rise - short T
High rise - longer T

c) Level of ductility and overstrength depending on the total weight of structure

In a general sense, R is the measure of the ability of the building to deform and dissipate energy without collapsing. This factor also accounts for the inelastic structural behavior of the structure. UBC Table 16-N (Appendix p.80) specifies the values of R for different framing schemes. Those factors have mainly been derived from observed building performance under earthquakes as well as from analytical and experimental research. All R values are greater than unity and thus will reduce the base shear V. The more ductile the structural system, the higher R it is.

Some typical values of R are presented below:

8.5 Steel Eccentrically Braced Frame

5.5 Concrete Shear Walls

For low-rise wood buildings, the typical values of R range from 2.8 (for heavy timber braced frames) to 6.5 (for light frame wood buildings). A value of 5.5 is usually taken for light woodframe of structure, with less than four stories and that have shear walls supporting gravity and lateral loads.

d) Proximity to geological faults

Few factors are used in the estimation of the base shear such as Cv, Ca, Z, and Nv. These factors take into account the proximity of the structure to geological faults.

Table 16-R and 16-Q (Appendix p.81) can be used to obtain the values of Cv and Ca, seismic dynamic response spectrum values. Cv and Ca account for how the building and soil can amplify the basic ground acceleration or velocity. It should be noted that in the highest seismic regions (Zone 4), Cv and Ca depend on the seismic source type (Table 16-U, Appendix p.81). This seismic source type is a function of the earthquake magnitude expected for a given fault and the slip rate of that fault.

Additionally, in Zone 4 region, the additional lower bound calculation for shear requires two more factors: Z and Nv. Z, Zone Factor, is associated with the magnitude of peak ground acceleration. It is 0.40 for a Zone 4 (San Francisco /Los Angeles for example).
Nv, referring to "Near-Source factor", accounts for the higher ground accelerations expected in regions close to fault rupture zone. Values of Z and Nv can be found in UBC Table 16-I and 16-T (Appendix p.79, p.81).

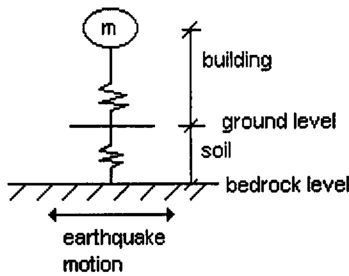
e) Soil conditions at the site

The soil conditions of the site are also considered by the factors Cv and Ca, seismic coefficient for velocity and acceleration controlled region. These values depend on the soil profile type as defined in Table 16-J (Appendix p.79). Six different soil profiles are defined in this table as well as in Table 3, from Sa to Sf:

Description	Type
Hard Rock	S_A
Rock	S_B
Very Dense Soil & Soft Rock	S_C
Stiff Soil	S_D
Soft Soil	S_E
UBC 1629.3.1	S_F

Table 3: Soil Profiles

The soil layers beneath a structure can affect the way the structure responds to a seismic ground motion.



If the period of vibration of the structure is close to that of the underlying soil, the bedrock motion will be amplified and the building will experience larger motions than predicted without Cv and Ca. If no geotechnical investigation has been done on the site, a soil profile of $\mathbf{S_D}$ is used.

Determination of Earthquake Forces

- First compute the seismic dead weight w_i, for each floor and the roof. This weight typically includes only the unfactored dead load. The story values can be added to obtain the total seismic dead load of the building.
- Then, compute the base shear V as thoroughly described in sections above.
- Compute the additional lateral force Ft, acting at the top of the structure:

$F_t = 0$	for $\mathbf{T} < 0.7s$	
$F_t = 0.07\,\mathbf{T}\,\mathbf{V}$	for $0.7s < \mathbf{T} < 3.57s$	(UBC Equation 30-14)
$F_t = 0.25\,\mathbf{V}$	for $\mathbf{T} > 3.57s$	

- Compute $\Sigma w_i h_i$ where i goes from 1 to the number of stories. This value will be constant for all Fx. hi corresponds to the height from the base of the building to story i.
- Compute Fx, the story forces at story x, as shown below

$$F_x = \frac{(V - F_t) w_x h_x}{\sum_{i=1}^{n} w_i h_i}$$

2. Simplified Lateral Procedure

For low-rise standard occupancy, an alternate procedure is offered to calculate the base shear V and story forces Fx. This method can be used for light frame wood structure of no more than three-story high. This can be found in UBC Section 1629.8.2, and Section 1630.2.3.

In this simplified procedure, the fundamental period of vibration of the structure and the height of each floor level are not considered anymore, as can be seen in the formulas below:

$\mathbf{V = 3\ C_a W / R}$ (UBC Equation 30-11)

$\mathbf{F_x = 3\ C_a w_x / R}$ (UBC Equation 30-12)

It can also be noted that in this method, the additional force at the top of the structure, Ft, has been omitted. The effects of other vibration modes are not taken into account.

3. Diaphragm Forces

Diaphragm forces correspond to the seismic lateral force applied to the perimeter of each floor and roof diaphragm. In typical wood structures, the floors and roof systems are designed to act as horizontal diaphragms. These will help transfer the applied lateral forces into the shear walls (described in the next section) supporting the diaphragms on each side. The figure below shows a wood diaphragm carrying a uniformly distributed load (applied lateral loads). The shear forces on each side represent the unit shear load transferred to the shear walls, with

v = (wL) / (2b)

w= uniformly distributed lateral load

L= Diaphragm length perpendicular to lateral load

b = Diaphragm length parallel to lateral load

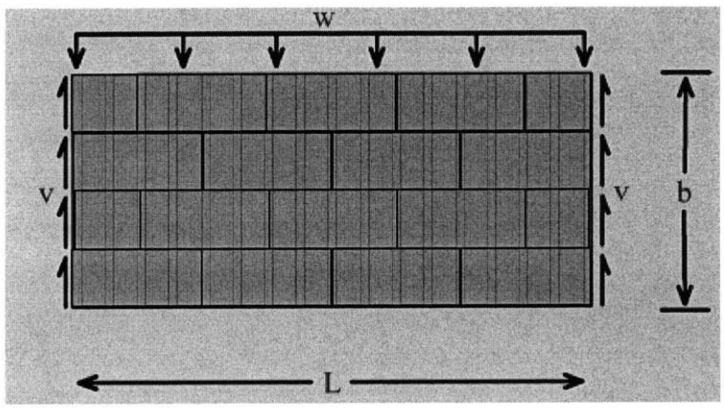


Figure 3: Wood Diaphragm Carrying Uniformly Distributed Load

(CUREE Caltech Project, 2000)

UBC Section 1633.2.9 proposes the following equation to obtain an approximation of the diaphragm forces:

$$F_{px} = \frac{F_t + \sum_{i=x}^{n} F_i}{\sum_{i=x}^{n} w_i} (w_{px})$$

(UBC Equation 33-1)

Lower and upper bounds are also specified in the Uniform Building Code as followed:

$F_{px} > 0.5\ C_a\ I\ w_{px}c$ (Upper Bound)

$F_{px} < C_a\ I\ w_{px}c$ (Lower Bound)

w_{px}= fraction of building weight lumped with diaphragm at level x

B. Lateral Force Resisting System (LRFS)

1. Introduction to Shear Wall

As discussed above in section A.3, diaphragms are the horizontal elements of the building, namely the roof and floors. The forces generated from seismic or wind activities will be transmitted through the diaphragm to shear walls or frames acting as the vertical elements of the lateral-force-resisting system of the structure. Shear walls can be designed as vertical deep cantilever beams supported by the foundation. In the same manner, diaphragms can be designed as horizontal beams transferring lateral loads to the shear walls.

In wood construction, along with the diaphragms, frames, and foundation, shear walls belong to the load path. Those elements must be adequately interconnected in order to provide a continuous load path. Indeed, one main concern in seismic design is to ensure this continuous path to foundation. Figure 4, Figure 5, and Figure 6 represent the different phases of load transfer.

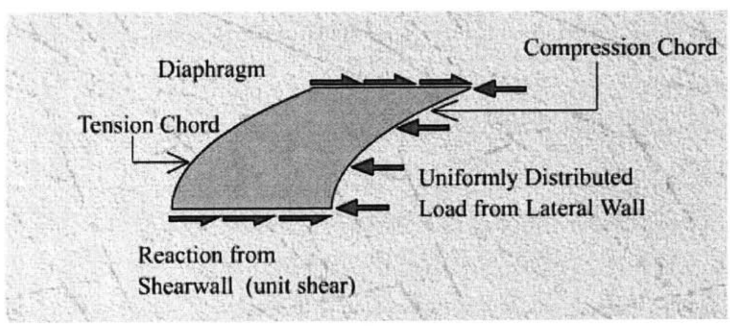


Figure 4: Load Transfer from Lateral Wall to Horizontal Diaphragm
(CUREE Caltech Project, 2000)

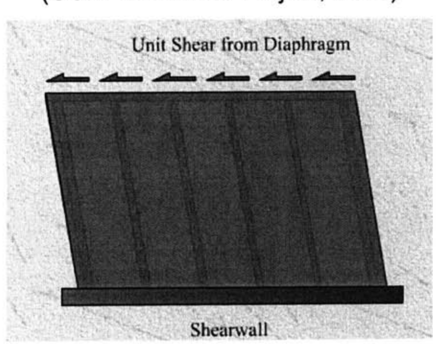


Figure 5: Load Transfer from Diaphragm to Shear Wall
(CUREE Caltech Project, 2000)

Shear walls serve two main functions: strength and stiffness. In terms of strength, shear walls must provide necessary lateral strength to resist the horizontal diaphragm forces resulting from seismic activities. Their strength also ensure the transfer of those horizontal forces to the next element in the load path (other shear walls, foundation ...) In terms of stiffness, shear walls should provide enough lateral stiffness to prevent the roof or floor above from excessive side-sway. Stiff enough, the shear walls should prevent the framing members from racking off their respective supports.

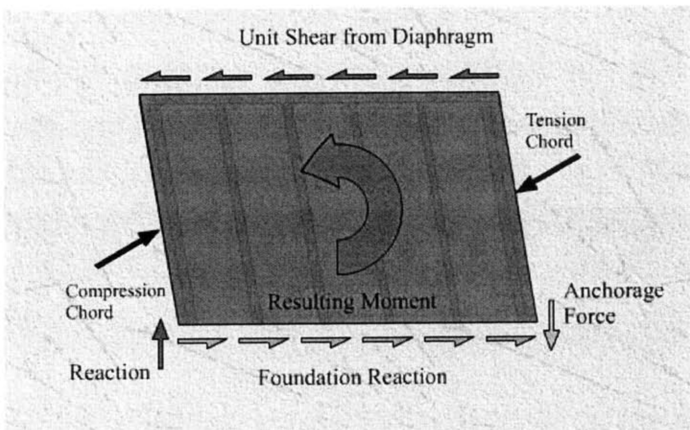


Figure 6: Load Transfer from Shear Wall to Foundation
(CUREE Caltech Project, 2000)

Typical shear walls consist of woodframe stud walls, dimension lumber framework, connected together with nails, and covered with a structural sheathing material like plywood (see section II.B.5 for material details), insulations panels or finishing panels such as drywall. The figure below (Figure 1) shows a typical woodframe shear wall construction, presenting the four main part of such system: framing members, sheathing, nails, and hold-downs. The latter provide the connection to the foundation to resist uplift forces resulting from applied moments. Hold-downs connectors are required at the corners of each shear wall to prevent the walls from overturning. Additionally, the length of the shear wall is determined by the location of those hold-downs. The top plate is used to connect the studs by end nails. Nailing plays an important role in shear wall construction. The performance of the plywood shear walls is highly based on the ductility and energy dissipative properties of nailed joints between the sheathing and framework.

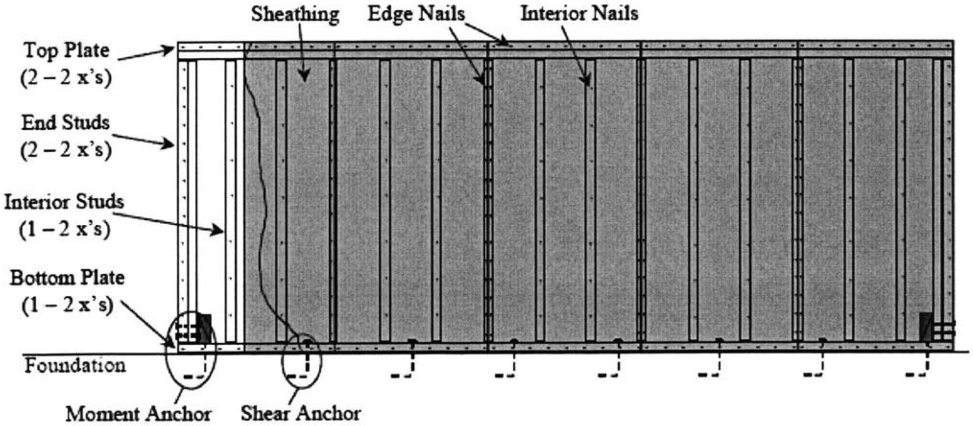


Figure 7: Typical Woodframe Shear Wall Construction
(Robert N. Emerson)

2. Shear Wall Design

In wood construction, there exists two ways of designing shear walls, both following very straight forward procedures: Segmented design and Perforated design.

a) Segmented Shear Wall Design (SSW)

This traditional method starts by dividing the walls into segments of full-height sheathing. That is, it does not take into account segments above or below openings in walls (such as windows or doors). The lengths of all the full-height segments are added and used to resist shear forces. This design provides a conservative estimate of the total length of wall resisting the applied forces since it does not take into account sections of walls that can provide lateral resistance (i.e. yellow walls on Figure 8)

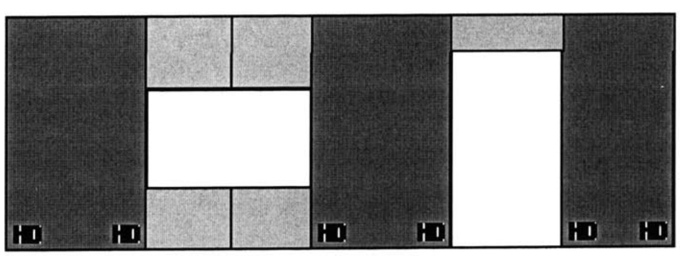


Figure 8: Segmented Shear Walls
(CUREE Caltech Project, 2000)

The design shear capacity, V, is calculated by the equation below:

$$V = v\ \Sigma bi$$

where V represents the total allowable shear capacity of wall (lb), v is the allowable shear capacity per unit length (lb/ft), and Σbi is the sum of the total length of full-height sheathing segments.

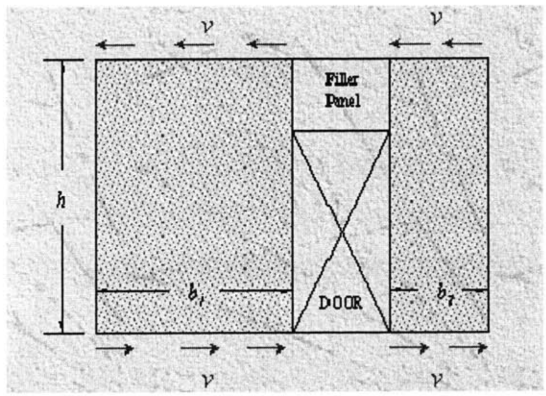


Figure 9: SSW Determination of Shear Capacity Schematic
(CUREE Caltech Project, 2000)

The shear capacity per unit length is obtained depending on the sheathing grade and thickness as well as the nail size and spacing. Such relation can be found in UBC Table 23-II-I-1, entitled "Allowable Shear for Wood Structural Panel Shear Walls" (Appendix p.83).Table 4 represents a shear wall schedule used by designers at a local structural company (Design Plus Inc.) as well as by contractors during the construction process of a structure. This schedule determines the shear capacity of unit length for different configurations proposed by the company.

MARK	SHEATHING MATERIALS	PANEL NAILING		BLK'G TO SILL DBL PL CONN	ANCHOR BOLT OPT.	EMB. DEPTH	SHEAR plf
		PERIMETER	FIELD				
1	1/2" CDX PLYWOOD STR. I	10 d @ 6" o.c.	10 d @ 12" o.c.	A35 @ 24" o.c. or 16 d @ 6" o.c.	5/8" Ø @ 48" o.c. 3/4" Ø @ 6'-0" o.c.	9"	255
2	1/2" CDX PLYWOOD STR. I	10 d @ 4" o.c.	10 d @ 12" o.c.	A35 @ 16" o.c. or 16 d @ 4" o.c.	5/8" Ø @ 32" o.c. 3/4" Ø @ 48" o.c.	9"	382
3	1/2" CDX PLYWOOD STR. I	10 d @ 3" o.c.	10 d @ 12" o.c.	A35 @ 8" o.c.	5/8" Ø @ 32" o.c. 3/4" Ø @ 32" o.c.	9"	498
4	1/2" CDX PLYWOOD STR. I	10 d @ 2" o.c.	10 d @ 12" o.c.	A35 @ 8" o.c.	5/8" Ø @ 24" o.c. 3/4" Ø @ 32" o.c.	9"	652
5	3/8" CDX PLYWOOD	8 d @ 6" o.c.	8 d @ 12" o.c.	A35 @ 24" o.c. or 16 d @ 6" o.c.	5/8" Ø @ 48" o.c. 3/4" Ø @ 6'-0" o.c.	9"	198
6	5/8" GYPBD. BOTH SIDES	6 d cooler @ 7" o.c.	6 d cooler @ 7" o.c.	16 d @ 8" o.c.	1/2" Ø @ 48" o.c. 5/8" Ø @ 6'-0" o.c.	9"	30
	PORTLAND CEMENT PLAST. BOTH SIDE	1 1/2 - #11 nails or #1t go. staples		16 d @ 6" o.c.	5/8" Ø @ 32" o.c. 3/4" Ø @ 48" o.c.	9"	180
7	7/8" PORTLAND CEMENT PLAST.	1 1/2 - #11 nails @ 6" o.c.		A35 @ 32" o.c. or 16 d @ 8" o.c.	1/2" Ø @ 48" o.c. 5/8" Ø @ 6'-0" o.c.	9"	90

Table 4: Shear Wall Schedule (Typical Zone 4 Construction)

(Courtesy of Design Plus Inc., 2006)

b) Perforated Shear Wall Design (PSW)

In this procedure, all sheathed portions of the shear wall are used to resist overturning and lateral loads (green areas shown in Figure 10). The entire wall section acts as a brace which will take into account the weakening caused by openings in the wall. Moreover, in this method, only two hold-downs are required for each wall, one at each end.

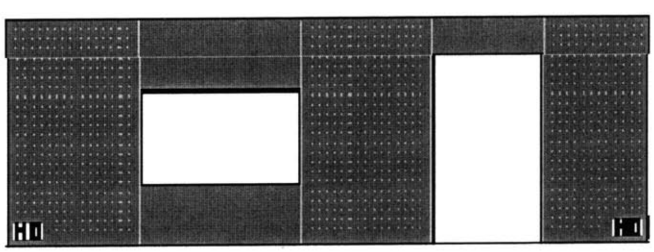


Figure 10: Perforated Shear Walls

(CUREE Caltech Project, 2000)

The design procedure is very similar to the segmented shear wall design. Indeed, the same table (UBC Table 23-II-I-1, Appendix p.83), to obtain the unit shear capacity, v, of a given wall. However, a shear capacity adjustment C_o must be tabulated to account for the openings in walls; this adjustment factor relates to the percentage of full-height sheathing in the wall and is always less than unity. This percent of full-height sheathing is calculated by the equation below:

$$\% = \Sigma bi / L$$

where L is the total length of the wall, bi is the length of the full-height sheathing segment.

A table in appendix p.83 presents the complete tabulated factors.

Finally, the total shear force is calculated in a similar manner to SSW design with:

$$V = C_o v \Sigma bi$$

Comparing both methods, it can be noted that the SSW yields a higher design shear capacity than the PSW method, sometimes being too conservative. Moreover, the SSW method requires hold-downs at the bottom corners of each full-height shear wall segment to resist overturning. More hold-downs mean more labor needed to install them causing the project to cost more.

It should also be noted that building codes (International Building Code and Uniform Building Code) have imposed limits on the dimensions of wood-frame shear walls, requiring a minimum wall length for any given wall height. This restriction rises from the poor performance of tall and narrow shear walls during previous earthquakes. For a wall of constant height, it has been showed that the stiffness grows exponentially as the wall length increases. UBC Table 23-II-G (Appendix p.82) provides the requirements depending on the location of the structure and the type of shear wall construction used.

3. Shear Wall Connectors

Designing shear wall does not permit many mistakes to occur for the engineer. In fact, if carefully followed, the design can be smoothly and accurately made. However, during a seismic activity, the behavior of timber structures is fully dependent on the behavior of its joints. Wood usually performs linearly and elastically, where failure is brittle. Wood has a low capability of dissipating energy, except if in compression with loads perpendicular to its grain. The joints should then be more ductile than the timber parts themselves. The detailing of the joints is therefore very important in seismic design and additionally, in the construction phase. The quality and workmanship of those connections are crucial in the success of shear wall behavior during seismic activities. The following section describes different connectors and also presents some problems occurring on the job site.

a) Foundation Connectors

Hold-Downs

As previously discussed, hold-downs are the connectors used at each end of the shear wall to prevent the wall from overturning. They are connected to the end stud or post of the shear wall. Indeed, seismic activities shake the shear wall back and forth and engender uplift forces on both ends of the shear wall. Hold-downs should transmit the tensile force from the chord (Figure 6) to the foundation of the structure.

The grade and size of the lumber help determine how much uplift the framing member can take and help design the connection of a hold-down device to the framing member. Table 5 reflects on this property. Many companies selling those products provide tables with allowable tension loads (Table 6).

Holdown Product	Stud Size	Douglas Fir-Larch Grade	Catalog Value	Tension, lbs	Compression, lbs		
				Net Section	8 Ft. Stud	Sill or Sole Plate	
						Hem fir	DF-L
		No. 1		12,078	7,695		
		No. 2		10,288	7,209		
HD 8A	4 x 4	Construction	7,460	7,753	6,840	4,961	7,656
		Standard		4,473	6,327		
		Stud		5,905	5,965		

Table 5: Effect of Lumber Type on a Given Hold-down Product

(Association of Bay Area Governments Technical Manual)

Model No.	Allowable Tension Loads DF/SP (133/160)							Allowable Tension Loads SPF/HF (133/160)						
	Wood Member Thickness							Wood Member Thickness						
	1½	2	2½	3	3½	4½	5½	1½	2	2½	3	3½	4½	5½
HD2A	1555	2055	2565	2775	2775	2775	2760	1320	1740	2165	2570	2565	2565	2550
HD5A	1870	2485	3095	3705	4010	4010	3980	1585	2110	2625	3130	3645	3700	3680
HD6A	2275	2980	3685	4405	5105	5460	5510	1870	2470	3065	3680	4280	5055	5020
HD8A	3220	4350	5415	6465	7460	8065	7910	2710	3655	4530	5480	6350	7470	7330
HD10A	3945	5540	6935	8310	9540	10235	9900	3275	4600	5745	7045	8160	9500	9195
HD14A	—	—	—	—	11080	13645	13380	—	—	—	—	9495	11950	12485
HD15	—	—	—	—	—	16345	15305	—	—	—	—	—	14355	13810

Table 6: Allowable Tension Loads for Different Hold-downs Models

(Simpson Strong-Tie Company Inc., 2007)

Figure 11 shows a structural detail of a typical hold-down used in residential building with flat foundation.

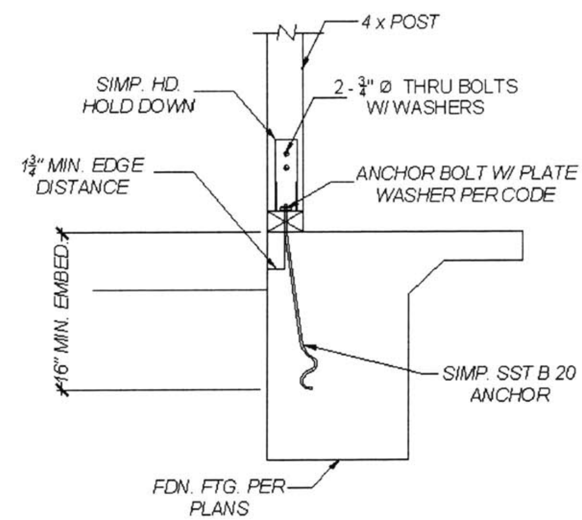


Figure 11: Typical Hold-Down Detail used in Residential Structure (Zone 4)

(Courtesy of Design Plus Inc., 2006)

The correct placement of hold-downs is also very important on the job site. In fact, during the Northridge 1994 Earthquake, many wood-frame buildings suffered a great deal of structural damage. Many of these damages were partly due to quality control deficiencies. A study showed that misplaced hold-downs caused reductions in strength and absorbed energy of wood shear walls when undergoing monotonic and cyclic loadings: about 42% of loss (Lebeda, Gupta, Rosowsky, Dolan, 2004).

Anchor Bolts

Anchor Bolts (sill plate bolts) are the second type of foundation connectors. These bolts are evenly spaced along the bottom length of the shear wall and primarily resist sliding action from lateral loads. They are embedded at a calculated depth in the foundation concrete slab as shown in Figure 11.

b) Blockings

For shear walls in seismic zones, it is important to keep all wood panels fastened to framing members. This is why blockings must be provided when two panels are not supported between framing members, i.e. wall heights exceed available panel lengths. It is important to keep all sheathing panel edges correctly fastened because if not, the shear wall can lose up to two third of the strength when all edges are fastened.

Moreover, blockings are also installed when shear walls are designed with openings. Blocks are installed between the studs on each side of the opening. Metal straps, described in the next section, are nailed to the blocks to reinforce the openings. The picture below was taken on a residential job site located in Los Angeles.

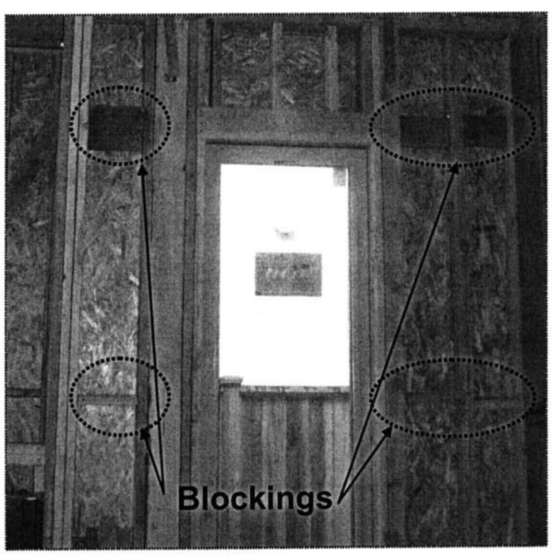


Figure 12: Blockings Located on Each Side of Door Openings

(Courtesy of Nina Mahjoub, 2007)

c) Metal Strap

As explained above, one use of metal straps are to help reinforce the openings in a shear wall. They can also be used as hold-downs to connect the end studs or posts below a floor. Figure 13 is a picture of metal straps used in a residential project, where they are used to connect the studs from the second floor to the first floor. There must be long enough to pass through the floor framing all the way to the end studs. A required number of nails (given by the manufacturer) must be provided between the strap and the stud to ensure the strong connection.

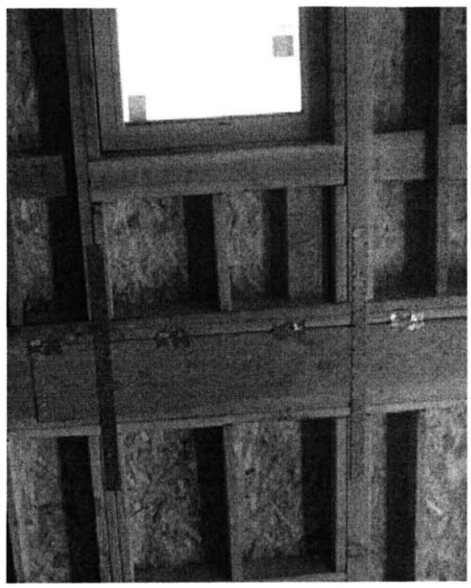


Figure 13: Metal Straps used as Hold-downs from Floor to Floor

(Courtesy of Nina Mahjoub, 2007)

d) Fasteners

The strength of those wood sheathed shear walls mostly comes from the strength of the fasteners. Here, nails are the preferred fasteners. In fact, compared to bolts or screws, they cost less to install and are easier to install thanks to nail guns.

Nails are preferred because they are more ductile, which result in a better absorption of seismic energy. In fact, screws might offer a better holding power in tension, but they are less ductile; this property is necessary to prevent brittle fracture to occur during cyclic loading.

When seismic activity strikes, nails tend to want to pull through the structural panel sheathing. Therefore, many requirements need to be followed during the construction process. In fact, nails should be driven flush with the surface of the sheathing, avoiding any overdriven nails. The overdriven nails reduce the shear wall strength by reducing the thickness of the sheathing. Moreover, nails should not be installed too close to the edge of sheathing. This should prevent prematurely failure due to earthquake motions. Nails that are improperly installed have no value to the good performance of the sheathing connection.

Common nails are favored to fasten sheathing because they have higher strength and stiffness compared to box, cooler, or sinker nails; they have larger nail shank diameters decreasing splitting of wood.

C. Lateral Analysis of a typical residential wood construction

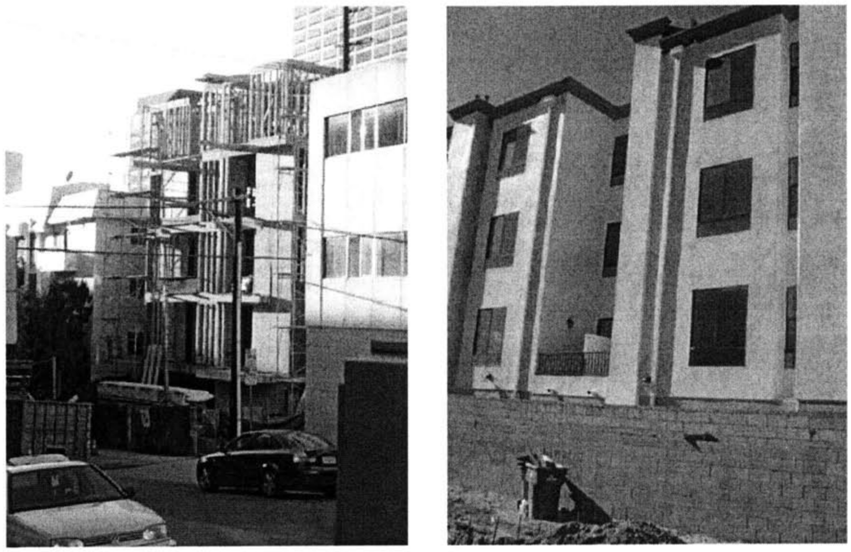


Figure 14: Two Timber Apartments in High Seismic Region (Left- under construction, Right- ready for use)

(Courtesy of Nina Mahjoub, 2007)

In order to demonstrate some design methodologies in practice, a virtual four-story apartment has been taken in Los Angeles, California, region of high seismic area.
The objective of this section is to describe the seismic design of this structure. The main structural material used in this design is wood (lumber and engineered wood).

The structural design comprises the calculation of the following:

- Design loads
- Wind loads and factors
- Seismic loads and factors
- North-South and East-West shear walls
- Posts, Hold-down and Strap Capacities
- Overturning Moments for N-S & E-W Walls
- Horizontal Diaphragms
- Anchorage to Concrete
- Shear Wall Deflection

1. Loads and Factors

a) Design Loads

Those design loads were taken from the design of a regular residential construction. The dead loads are approximate and can vary depending on the material used. However, they remain quite precise in the domain of wood design.

A.1 provides reference to the UBC and CBC Chapter 16, where different formulas and graphs help define the wind and seismic factors.

1) STRUCTURAL DESIGN DATA:

A.) ROOF LOADS:

Roofing: Allow		=	2.2	psf
5/8" Plywood Sheathing		=	1.8	
11-7/8" TJS Joists @ 16"		=	3.3	
5/8" Gyp. Board		=	2.8	
Insulation: 8"		=	2.4	
Roof Slope: Rip Framing		=	2.5	
Sprinklers Allow		=	1.5	
Miscellaneous Allow		=	0.5	
	SUM OF D.L.	=	17.0	psf

B.) TYPICAL FLOOR LOADS:

Floor covering: allow		=	1.5	psf
1-1/2" Elastizell		=	13.0	
3/4" Plywood Sheathing		=	2.3	
14" TJS Joists @ 16"		=	3.9	
5/8" Gyp. Board		=	2.8	
Sprinklers Allow		=	1.5	
Miscellaneous Allow		=	1.0	
	SUM OF D.L.	=	26.0	psf

C.) 3-1/2" NW CONC. TOPG.:	=	42.0	psf
D.) 12" NW CONC. FLOOR:	=	145.0	psf
E.) EXT. WALL DEAD LOAD:	=	13.00	psf
F.) INT. WALL DEAD LOAD:	=	10.00	psf
D.) STAIR/EXIT LOADS:	=	25.0	psf

b) Wind Loads and Factors

P = Ce * Cq * qs * Iw

a.) WIND LOAD FACTORS:

EXPOSURE	=	B	
IMPORTANCE FACTOR, Iw	=	1.0	
BASIC WIND LOAD	=	70	mph

qs: STAGNATION PRESSURE = 12.6 PSF

Cq: PRESSURE COEFFICIENT

1. Primary frame system (method 1):		
Roof (Flat)	=	0.7
wall (windward)	=	0.8

2. Elements & Components:		
Parapets	=	1.3

3. Elements & Components:		
Wall Corners	=	1.5
Roof Eaves (Slope < 2:12)	=	2.3

Ce: PRESSURE COEFFICIENT

HEIGHT, h ft.	
0-15 =	0.62
20 =	0.67
25 =	0.72
30 =	0.76
40 =	0.84

b.) VERTICAL DISTRIBUTION OF WIND PRESSURE

WHERE: P = Ce * Cq * qs * Iw

	PRESSURE COEFFICIENT, Cq				
	0.7	0.8	1.3	1.5	2.3
ELEV., h ft.	**WIND PRESURE, P psf**				
0-15	5.47	6.25	10.16	11.72	17.97
20	5.91	6.75	10.97	12.66	19.42
25	6.35	7.26	11.79	13.61	20.87
30	6.70	7.66	12.45	14.36	22.02
40	7.41	8.47	13.76	15.88	24.34

c) Seismic Loads and Factors

SEISMIC FACTORS:

IMPORTANCE FACTOR I			=	1.00
REDUCTION, R (T. 16-N)			=	4.5
ZONE, Z			=	0.4
SEISMIC SOURCE TYPE			=	B
SOIL TYPE			=	S_D
Na			=	1.00
Nv			=	1.11
Ca =	0.44	X Na	=	0.440
Cv =	0.64	X Na	=	0.710

STRUCTURE PERIOD:

Ct	=	0.020	
h_n	=	45	ft
$T = Ct * (h_n)^{3/4}$	=	0.347	sec.

BASE SHEAR:

	V =	W (Cv I) / (RT)		=	0.454	* W
0.285	Vmax =	W (2.5 Ca I) / R		=	0.244	* W
	Vmin =	W (0.11 Ca I)		=	0.048	* W
	$Vmin_{(Z4)}$ =	W (0.8ZNvI) / R		=	0.079	* W
	GOVERNING BASE SHEAR			=	0.285	* W

CALCULATE BUILDING WEIGHT, W:

Disc.	Length (ft)	Width (ft)	DL			
RF	80	80	0.027	=	172.8	k
4TH	80	80	0.041	=	262.4	k
3RD	80	80	0.041	=	262.4	k
2ND	80	80	0.041	=	262.4	k
			SW	=	960	k

EARTHQUAKE LOADS:

RELIABILITY/REDUNDANCY FACTOR:			
$\rho = 2 - 20/[r_{max}(A_B^2)]$	=	1.0	
Ω_0	=	2.8	

Eh = BASE SHEAR, V						
Eh = V =	0.285	X	960.0	=	273.6	k
Ev = VERTICAL COMPONENT				=	0.00	k
E =	ρ Eh	+	Ev			
E =	274	+	0	=	273.6	k
Em = Ω_0 Eh				=	766.1	k

VERTICAL DISTRIBUTION OF FORCES

BUILDING PERIOD:

T (s) = 0.347 < 0.7

USE: Ft = 0

V = 273.6 k

Ft = 0 k

LATERAL SHEAR FORCES:

Fx = (V - Ft) Wx hx / {SUM OF (Wi hi)}

LEVEL	Wx	hx	Wx hx	Fx
RF	172.8	41	7084.8	**82.08**
4TH	262.4	31	8134.4	**94.24**
3RD	262.4	21	5510.4	**63.84**
2ND	262.4	11	2886.4	**33.44**
TTL:	**960**		**23616**	**273.6**

LATERAL DIAPHRAGM FORCES:

Fpx = Wpx (Ft + {SUM OF Fi}) / {SUM OF Wi}

Fpx (min) = 0.5 Ca I Wpx

Fpx (max) = 1.0 Ca I Wpx

LEVEL	Wpx	Fpx	ΣFpx	ΣWpx
RF	172.8	82.1	82	172.8
4TH	262.4	94.2	176	435.2
3RD	262.4	63.8	240	697.6
2ND	262.4	33.4	274	960
TTL:	**960**			

LEVEL	Fpx (min)	Fpx	Fpx (max)	REQ'D Fpx
RF	38.016	82.1	76.032	**76.03**
4TH	57.728	106	115.456	**106.3**
3RD	57.728	90.3	115.456	**90.34**
2ND	57.728	74.8	115.456	**74.78**

2. North-South and East-West Shear Walls

SEISMIC LOAD:

H = (TA) X (SEISMIC LOAD PER S.F.[1,2,3,4,5]) + H $_{\text{FROM LEVEL ABOVE}}$
v = H / L

Here, only the north-south shear walls calculations will be shown. For all detailed calculations, please see appendix from p.87.

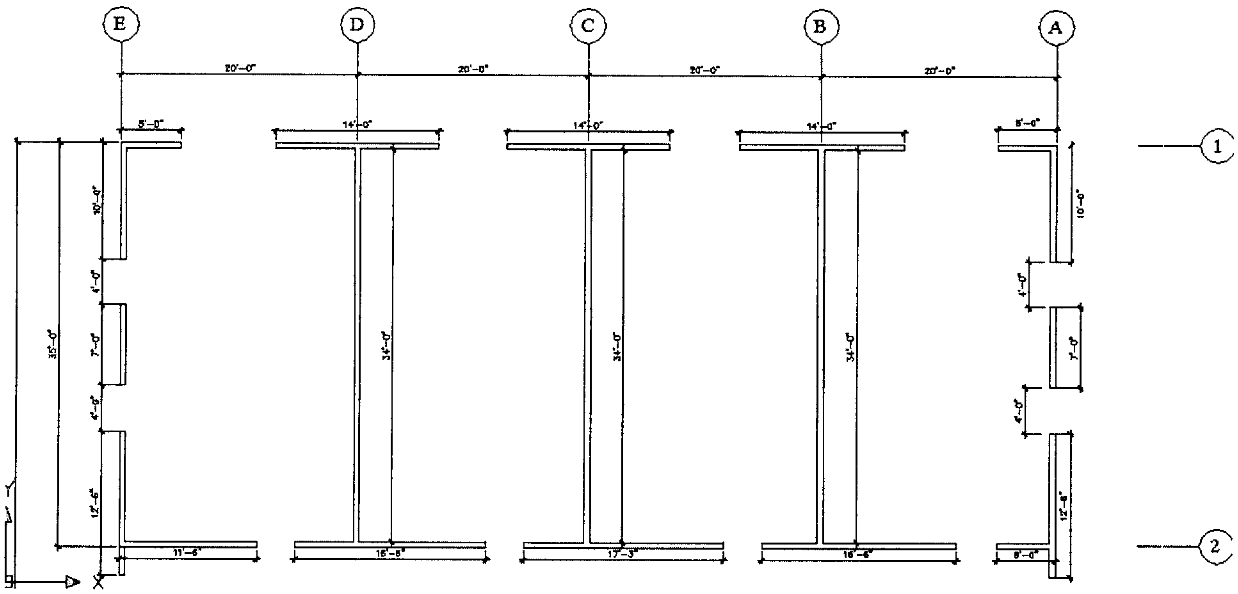


Figure 15: Portion of Typical Floor Plan of Design Structure

(See Appendix p.84 for detailed and entire floor plan)

Using the seismic loads and factors found above, we can obtain the type of shear wall needed to sustain seismic ground activity. Table 4 presents the different types of shear wall available in this seismic region and will be used to define which shear wall to use. For example, line 1 needs shear wall of type 1. This means that a sheathing material of ½" CDX Plywood Str. 1 is needed, with a panel nailing of 10d @ 6" on center in the perimeter and 10d @ 12" on center in the field. Blockings (A35) to sill double plate connections are required at 24" on center. Anchor bolts options are 5/8" diameter bolts at 48" on center or ¾" diameter bolts at 6" on center, with an embedment depth of 9". This type of shear wall can take up to 255 PLF of shear.

SHEAR WALLS SUPPORTING THE ROOF LEVEL:

Wall	Net Wall Length L (ft)	Tributary Area TA (SF)				Seismic Load Per SF[1]	Trib. Seis. Load, H_{TA} lbs.	SHEAR, v plf	Shear Panel Type per Table 4
SW1	52	ROOF:	80*17.5	=	1400	9.161	12825	247	**1 - 255 plf**
SW2	66.75	ROOF:	80*22.5	=	1800	9.161	16489	247	**1 - 255 plf**
SW3	66.75	ROOF:	80*22.5	=	1800	9.161	16489	247	**1 - 255 plf**
SW4	52	ROOF:	80*17.5	=	1400	9.161	12825	247	**1 - 255 plf**

1 - SEISMIC LOAD PER S.F.= 82,080 lb. / (6,400 ft2 * 1.4)

SHEAR WALLS SUPPORTING THE 4TH LEVEL:

Wall	Net Wall Length L (ft)	Tributary Area TA (SF)				Seismic Load Per SF[1]	Trib. Seis. Load, H_{TA} lbs.	SHEAR, v plf	Shear Panel Type per Table 4
SW1	52	FLOOR	80*17.5	=	1400	10.518	27550	530	**4 - 652 plf**
SW2	66.75	FLOOR	80*22.5	=	1800	10.518	35421	531	**4 - 652 plf**
SW3	66.75	FLOOR	80*22.5	=	1800	10.518	35421	531	**4 - 652 plf**
SW4	52	FLOOR	80*17.5	=	1400	10.518	27550	530	**4 - 652 plf**

2 - SEISMIC LOAD PER S.F.= 94,240 lb. / (6,400 ft2 * 1.4)

SHEAR WALLS SUPPORTING THE 3RD LEVEL:

Wall	Net Wall Length L (ft)	Tributary Area TA (SF)				Seismic Load Per SF[1]	Trib. Seis. Load, H_{TA} lbs.	SHEAR, v plf	Shear Panel Type per Table 4
SW1	56	FLOOR	80*17.5	=	1400	7.125	37525	670	**2#2 - 764 plf**
SW2	66.75	FLOOR	80*22.5	=	1800	7.125	48246	723	**2#2 - 764 plf**
SW3	66.75	FLOOR	80*22.5	=	1800	7.125	48246	723	**2#2 - 764 plf**
SW4	56	FLOOR	80*17.5	=	1400	7.125	37525	670	**2#2 - 764 plf**

3 - SEISMIC LOAD PER S.F.= 63,840 lb. / (6,400 ft2 * 1.4)

SHEAR WALLS SUPPORTING THE 2ND LEVEL:

Wall	Net Wall Length L (ft)	Tributary Area TA (SF)				Seismic Load Per SF[1]	Trib. Seis. Load, H_{TA} lbs.[7]	SHEAR, v plf	Shear Panel Type per Table 4
SW1	62	FLOOR:	80*17.5	=	1400	3.732	42750	690	**2#2 - 764 plf**
SW2	68.75	FLOOR:	80*22.5	=	1800	3.732	54964	799	**2#3 - 996 plf**
SW3	68.75	FLOOR:	80*22.5	=	1800	3.732	54964	799	**2#3 - 996 plf**
SW4	62	FLOOR:	80*17.5	=	1400	3.732	42750	690	**2#2 - 764 plf**

4 - SEISMIC LOAD PER S.F.= 33,440 lb. / (6,400 ft2* 1.4)

3. Posts, Hold-down, and Strap Capacities

The tables below represent different allowable strap and hold-down tension loads. Those tables will be used when calculating the necessary anchorage of the structure to the foundation and to connect floor to floor shear walls.

Strap or Hold-Down	LARR Capacity	Studs & Posts	0.75 LARR
MSTI36	1270	2 - 2X	
MSTI48	2355	2 - 2X	
MSTI60	3445	2 - 2X	
MST60	4830	2 - 2X	
MST72	6420	2 - 2X	
HD2A	2775	2 - 2X	2081.25
HD5A	3705	2 - 2X	2778.75
HD6A	4405	2 - 2X	3303.75
HD8A	6465	2 - 2X	4848.75
HD10A	8310	2 - 2X	6232.5
HD14A	11080	1 - 4X	8310
Z4-T2 (28-8)	13162	2 - 4X	
Z4-T2 (46-8)	17535	2 - 4X	
Z4-T2 (85-8)	24355	2 - 4X	
Z4-T2 (48-9x)	31174	2 - 6X	
Z4-T2 (68-10x)	46761	2 - 6X	

Table 7: Design Hold-Down Capacities for Overturning Moment

Strap	GAGUE	END NAILING	Studs & Posts	NAILS UNIT CAPACITY lb.	REDUCED NAIL CAPACITY lb.	LARR Capacity
MSTI36	12	7	2 - 2X	118	88.5	619.5
MSTI48	12	13	2 - 2X	118	88.5	1150.5
MSTI60	12	19	2 - 2X	118	88.5	1681.5
MSTI72	12	25	2 - 2X	118	88.5	2212.5
MST60	10	23	2 - 2X	141	105.75	2432.25

Table 8: Design Allowable Strap and Hold-down Seismic Tension Loads for Floor to Floor

Strap	GAGUE	END NAILING	Studs & Posts	NAILS UNIT CAPACITY lb.	REDUCED NAIL CAPACITY lb.	LARR Capacity
ST6224	16	14-16d	2 - 2X	135	101.25	1417.5
ST6236	14	20-16d	2 - 2X	136	102	2040
MSTI 36	12	18-10d	2 - 2X	120	90	1620
MSTI 60	12	30-10d	2 - 2X	120	90	2700
MSTI 72	12	32-10d	2 - 2X	120	90	2880
MST 37	12	21-16d	2 - 2X	141	105.75	2220.75
MST 48	12	23-16d	2 - 2X	141	105.75	2432.25
MST 60	10	28-16d	2 - 2X	149	111.75	3129

Table 9: Design Allowable Strap and Hold-down Seismic Tension Loads for Drag Strut

4. Overturning Moments for N-S and E-W walls

In this section as well, only the case of the North-South shear walls between roof and fourth level as well as the walls between fourth floor and third floor level will be presented. For entire calculation information, please see Appendix from p.88.

CHECK OVERTURNNG MOMENT IN THE N-S DIRECTION:
WITH UNIFORM RESISTIVE LOADS)

(1)	(2)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)	(13)	(14)	(15)
Wall Name	**Wall Length Li (ft)**	**Force, H (lb.)**	**Total Wall Length L (ft)**	**OTM from above (ft.-lb)**	**Total OTM (ft.-lb.)**	**Resistive Wall Length, Li (ft)**	**Resistive Load Description**	**Resistive Load, w (lb)**	**RM (ft.-lb.)**	**Tension/ Comp. T/C (lb.)**	**REMARKS**	**Conn**	**FACTORED SEIS. COMP. LOAD, P (lb.)**
	FROM ELEVS.	FROM SHEAR WALLS	FROM SHEAR WALLS	(7)ABOVE =	(2)x(3) x (4)/(5) + (6) =	FROM PLAN		(9) =	(10)x (8)/2 =	{(7) - [(11) x 0.9]} / (8) =			{[(7) x 3.92] + [(9) x 1.2]} / (2) =

WALLS BETWEEN ROOF AND 4TH LEVEL: (3): **Height:(ft)** **8.36**

SW1	5	12825	52	0	10,309	5	WALL DL: Li*h*13 ROOF DL: Li*13 *17	1648	4121	1,320	UPLIFT	HD2A, 2081 lbs.	9,072
SW2	5	16489	66.75	0	10,326	5	WALL DL: Li*h*13 ROOF DL: Li*4*18	883	2209	1,668	UPLIFT	HD2A, 2081 lbs.	8,626
SW3	5	16489	66.75	0	10,326	5	WALL DL: Li*h*10 ROOF DL: Li*13*17	1147	2867	1,549	UPLIFT	HD2A, 2081 lbs.	8,784
SW4	5	12825	52	0	10,309	5	WALL DL: Li*h*10 ROOF DL: Li*16*17	1778	4445	1,262	UPLIFT	HD2A, 2081 lbs.	9,149

WALLS BETWEEN 4TH AND 3RD LEVELS:

SW1	5	27550	52	10,309	32,455	5	WALL DL: Li*(9.5+h)*13 ROOF DL: Li*13*17 FLOOR DL:Li*13*26	3956	9890	4,711	UPLIFT	HD8A, 4848.75 lbs.	27,818
SW2	5	35421	66.75	10,326	32,507	5	WALL DL: Li*(9.5+h)*13 ROOF DL: Li*4*17 FLOOR DL:Li*4*26	2021	5052	5,592	UPLIFT	HD10A, 6232.5 lbs.	26,698
SW3	5	35421	66.75	10,326	32,507	5	WALL DL: Li*(9.5+h)*10 ROOF DL: Li*13*17 FLOOR DL:Li*13*26	3688	9220	4,842	UPLIFT	HD8A, 4848.75 lbs.	27,699
SW4	5	27550	52	10,309	32,455	5	WALL DL: Li*(9.5+h)*10 ROOF DL: Li*16*17 FLOOR DL:Li*16*26	4333	10833	4,541	UPLIFT	HD8A, 4848.75 lbs.	28,045

5. Horizontal Diaphragms

SEISMIC UNIFORM LOAD, w plf	**w = W x Fpx**
SEISMIC LOAD, H lbs.	**H = w x L**
TOTAL DIAPH. SHEAR, V lbs.	**V = 0.5H (IF CANTILEVERED, V = H)**
SHEAR, v plf	**v = V / W**
TRANSVERSE MOMENT, M ft.-lbs.	**$M = wL^2 / 8$ (IF CANTILEVERED, $M = wL^2 / 2$)**
CHORD T, C lbs.	**T = C = M / W**
CHORD STRESS, f_t psi	**$f_t = T / A_{2\text{-}2x}$**

NOTES FOR ALL TABLES:

1 - ROOF UNIT SEISMIC LOAD, Fpx = 76000/(6400*1.4)=8.48 psf

2 - 4TH FLOOR UNIT SEISMIC LOAD, Fpx =106300/(6400*1.4)=11.50 psf

3 - 3RD FLOOR UNIT SEISMIC LOAD, Fpx = 90340/(6400*1.4)=10.1 psf

3 - 2ND FLOOR UNIT SEISMIC LOAD, Fpx = 74780/(6400*1.4)=8.34 psf

4 - CANTILEVERED DIAPHRAGM: V = H, $M = w L^2 / 2$

HORIZONTAL DIAPHRAGM AT THE ROOF:

As $_{2\text{-}2X4}$ = 10.5 in^2

Load Dir.	Net Diaph. Length, L ft.	Net Diaph. Width, W ft.	Unit Seismic Load[1], Fpx psf	Seismic Uniform Load, w plf	Seismic Load, H lbs.	Total Diaph. Shear, V lbs.	SHEAR, v plf	Sheathing Remarks	Transv. Moment, M ft.-lbs.	Chord, T, C lbs.	Chord Stress, f_t psi	Chord Remarks
E-W	35	52	8.480	440.9	15434	7717	148	NOTE 7	67522	1299	123.67	NOTE 5
N-S	20	68	8.480	576.64	11533	5766	85	NOTE 7	28832	424	40.38	NOTE 6

HORIZONTAL DIAPHRAGM AT THE 4TH FLOOR:

As $_{2\text{-}2X6}$ = 16.5 in^2

Load Dir.	Net Diaph. Length, L ft.	Net Diaph. Width, W ft.	Unit Seismic Load[1], Fpx psf	Seismic Uniform Load, w plf	Seismic Load, H lbs.	Total Diaph. Shear, V lbs.	SHEAR, v plf	Sheathing Remarks	Transv. Moment, M ft.-lbs.	Chord, T, C lbs.	Chord Stress, f_t psi	Chord Remarks
E-W	35	52	11.500	598.00	20930	10465	201	NOTE 8	91569	1761	106.72	NOTE 5
N-S	20	68	11.500	782.00	15640	7820	115	NOTE 8	39100	575	34.85	NOTE 6

DIAPHRAGM HORIZONTAL AT THE 3RD FLOOR:

As $_{2\text{-}2X6}$ = 16.5 in^2

Load Dir.	Net Diaph. Length, L ft.	Net Diaph. Width, W ft.	Unit Seismic Load[1], Fpx psf	Seismic Uniform Load, w plf	Seismic Load, H lbs.	Total Diaph. Shear, V lbs.	SHEAR, v plf	Sheathing Remarks	Transv. Moment, M ft.-lbs.	Chord, T, C lbs.	Chord Stress, f_t psi	Chord Remarks
E-W	35	52	10.080	524.16	18346	9173	176	NOTE 8	80262	1544	93.55	NOTE 5
N-S	20	68	10.080	685.44	13709	6854	101	NOTE 8	34272	504	30.55	NOTE 6

HORIZONTAL DIAPHRAGM AT THE 2ND FLOOR:

As $_{2\text{-}2X6}$ = 16.5 in^2

Load Dir.	Net Diaph. Length, L ft.	Net Diaph. Width, W ft.	Unit Seismic Load[1], Fpx psf	Seismic Uniform Load, w plf	Seismic Load, H lbs.	Total Diaph. Shear, V lbs.	SHEAR, v plf	Sheathing Remarks	Transv. Moment, M ft.-lbs.	Chord, T, C lbs.	Chord Stress, f_t psi	Chord Remarks
E-W	35	52	8.340	433.68	15179	7589	146	NOTE 8	66407	1277	77.40	NOTE 5
N-S	20	68	8.340	567.12	11342	5671	83	NOTE 8	28356	417	25.27	NOTE 6

6. Anchorage to Concrete

ANCHORAGE TO CONCRETE AT HOLDOWNS[1]:

Fut (A449) =	120	ksi (1/4 to 1")
	105	ksi (1-1/8 to 1-1/2")
Fut(A307) =	60	ksi
F'c =	4,000	ksi
ϕ =	0.65	
λ =	1	

NOTE: DEFAULT BOLT TYPE IS A449

Anch. Bolt Diam.	Plate Side Length (in)
0.500	2
0.625	2.5
0.750	2.75
0.875	3
1.000	3.5
1.125	4
1.250	4.5

Table 10: Design Anchor Bolt Diameter with Corresponding Plate Side Length Used

HD	BOLT DIAM. (in)
HD2A	0.625
HD5A	0.75
HD6A	0.875
HD8A	0.875
HD10A	0.875
HD14A	1

Z4-T2	BOLT DIAM. (in)
28-8	1
46-8	1
85-8	1
48-9x	1.125
68-10x	1.25

Table 11: Design Hold-Down HD & Z4-T2 Type with Corresponding Bolt Diameter

Notes:

1- BASED ON LABC '02: DIV. II, SEC. 1923

2- THE VALUES ARE FROM OVERTURNING CALCULATIONS FACTORED PER 1923.2.

3- WHERE: 1" < BOLT DIA. < 1-1/2"

4- Pu NEED NOT EXCEED ULTIMATE STRANGTH OF THE ROD PER LABC '02: 1633.2.12.

5-AT EDGE CONDITIONS, ONLY HALF OR A QUARTER OF CONCRETE FAILURE PLANE AREA IS USED, ACCORDINGLY.

LOCATION		Plate Size	Bolt Information			Anchor Area for each side, Ab (in^2)	Concrete Failure Plane Area, Ap (in^2)	Pss [3] (k)	ϕPc (k)	Ultimate Normal Tension from OTM., Pu_{-OTM} (k)	Ultimate Normal Tension from Bolt Capacity[4], Pu-ss (k)	Design Ultimate Normal Tension[4], Pu (k)	Anchorage Concrete
WALL	**Hold-down Type**	**L. & W. (in)**	**DIAM, D (in)**	**EMBED LENTH, le (in)**	**No. EA. SIDE**	**Ab =No.X $\pi D^2/4$ (in2)**	**Ap = $(2\ le + L)^2$**	**Pss = 0.9 Ab f_{ut}**	**$\phi Pc = \phi \lambda 4$ Ap $(f'c)^{0.5}$**	**REACTIONS Pu_{-OTM} = (E1.4 - 0.9DL) x 1.3/l**	**Pu-ss = Pss x 1.3**	**Pu = MIN(Pss, ϕPc)**	
SW1	Z4-T2 (85-8), 24355 lbs.	3.5	1.000	9	1	0.785	462	85	76	34.03	110.27	76.01	>Pu, OK!
SW2	Z4-T2 (46-8), 17535 lbs.	3.5	1.000	9	1	0.785	462	85	76	32.91	110.27	76.01	>Pu, OK!
SW3	Z4-T2 (46-8), 17535 lbs.	3.5	1.000	9	1	0.785	462	85	76	32.91	110.27	76.01	>Pu, OK!
SW4	Z4-T2 (85-8), 24355 lbs.	3.5	1.000	9	1	0.785	462	85	76	32.99	110.27	76.01	>Pu, OK!
SWA	HD8A, 4849 lbs.	3.0	0.875	12	1	0.601	729	65	120	11.28	84.42	84.42	>Pu, OK!
SWB	Z4-T2 (28-8), 13162 lbs.	3.5	1.000	9	1	0.785	462	85	76	26.19	110.27	76.01	>Pu, OK!
SWC	Z4-T2 (28-8), 13162 lbs.	3.5	1.000	9	1	0.785	462	85	76	26.19	110.27	76.01	>Pu, OK!
SWD	Z4-T2 (28-8), 13162 lbs.	3.5	1.000	9	1	0.785	462	85	76	26.19	110.27	76.01	>Pu, OK!
SWE	HD8A, 4849 lbs.	3.0	0.875	9	1	0.601	441	65	73	11.28	84.42	72.52	>Pu, OK!

Table 12: Design Anchorage Concrete Check Calculation for N-S & E-W Shear Walls

7. Shear Wall Deflection

The calculations below represent the check for deflection of the north-south shear walls between the roof and fourth floor level. The entire calculation can be found upon request at ninazadeh@yahoo.com. This check is essential to control the story drift and relies on two main reasons: serviceability and limitation on maximum inelastic response of the wall. The first reason controls the cracking in wall coverings and the second reason is important in seismic design of wood buildings.

UBC Standard 23-2 is used to obtain the following deflections. It accounts for bending, shear, nail deformation, and anchorage slip.

Total shear wall deflection, $\Delta_s = \Delta_b + \Delta_v + \Delta_n + \Delta_a$

E_{wood}	1.7.E+06	PSI
G	9.0.E+04	PSI
F'c	625	PSI
Effective Thickness, t =	0.535	in.
E_{steel}	2.9E+07	PSI

F'c =	625	PSI
$\gamma_{(5/8")}$ = 270,000 $(5/8)^{1.5}$ =	175370	lb./in.
$\gamma_{(3/4")}$ = 270,000 $(3/4)^{1.5}$ =	175370	lb./in.
$\gamma_{(7/8")}$ = 270,000 $(7/8)^{1.5}$ =	220992	lb./in.
$\gamma_{(1")}$ = 270,000 $(1)^{1.5}$ =	220992	lb./in.
Maximum Allowable Drift:		
Δ_M = 0.025 hs =	2.85	in.

Table 13: Various Proprieties for Deflection Calculations

DEVICE	MAX ALLOW CAP., (lb.)	MAX DEFL. @ CAP., (in.)	NO. OF BOLTS	DIA. OF BOLTS, (in.)
HD2A	2775	0.058	2	0.625
HD5A	3705	0.067	2	0.750
HD6A	4405	0.041	2	0.875
HD8A	6465	0.111	3	0.875
HD10A	8310	0.269	4	0.875
HD14A	11080	0.282	4	1.000

DEVICE	MAX ALLOW CAP., (lb.)	MAX DEFL. @ CAP., (in.)	NO. OF BOLTS	DIA. OF BOLTS, (in.)
Z4-T2 (28-8)	13162	0.025	2	1.000
Z4-T2 (46-8)	17535	0.027	4	0.750
Z4-T2 (85-8)	24355	0.027	8	0.625
Z4-T2 (48-9X)	31174	0.032	4	1.000
Z4-T2 (68-10x)	46761	0.036	6	1.000

Table 14: Hold-Downs Allowable Force & Deflection Capacities used for Design

SHEAR WALL INFORMATION										
Wall Name	**Wall Length L = b (ft.)**	**Wall Height, h (ft.)**	**Boundary Member Area, A ($in.^2$)**	**# of Nails per ft**	**ASD Shear Load, v/1.4 (lb./ft)**	**Stg. Shear Load, v (lb./ft)**	**Shear /Nail, Vn (lb.)**	**Nail Deform., en (in.)**	**ASD Uplift T/1.4 (lb.)**	**Stg. Uplift, T (lb.)**
From OTM N-S walls	From OTM N-S walls	From OTM N-S walls	2-2X4	From Plans	N-S SHEAR WALLS		v / (NO. OF NAILS)	en = $(Vn/769)^{3.276}$	From OTM N-S walls	
WALLS BETWEEN ROOF AND 4TH FLOOR:										
SW1	5	9.50	10.5	2	247	345	173	0.0075	1,320	1,848
SW2	5	9.50	10.5	2	247	346	173	0.0075	1,668	2,335
SW3	5	9.50	10.5	2	247	346	173	0.0075	1,549	2,169
SW4	5	9.50	10.5	2	247	345	173	0.0075	1,262	1,766

Table 15: Shear Walls Information used for Design

Wall Name	Device NO.	Device Type	Device Max. Allow. Cap., (lb.)	Max Defl. @ Cap. (in.)	# of Bolts	Dia. Of bolts, (in.)
From OTM N-S walls					ONE SIDE ONLY	
SW1	2	HD2A, 2081 lbs.	2775	0.058	2	0.625
SW2	2	HD2A, 2081 lbs.	2775	0.058	2	0.625
SW3	2	HD2A, 2081 lbs.	2775	0.058	2	0.625
SW4	2	HD2A, 2081 lbs.	2775	0.058	2	0.625

Table 16: Design Tie-Down Device Properties

Wall Name	Device Elong. (in)	Shrink	Crush	Slip	da, (in.)
From OTM N-S walls	Δstraps = TL_0/A_0E OR Δhd = device*T* (Δmax) /MAX LOAD	TJI => No Shrinkage	0.02 IF fc<.73F'c, 0.04 IF fc=F'c WHERE fc = T/A	Δstraps = e_n OR Δhd=No. of device*T/γ+1/32	Σ(DISPL.)
SW1	0.0386	0.000	0.020	0.0451	0.104
SW2	0.0488	0.000	0.020	0.0488	0.118
SW3	0.0453	0.000	0.020	0.0475	0.113
SW4	0.0369	0.000	0.020	0.0445	0.101

Table 17: Design Device Elongation & Assembly Displacement

Wall Name	Cantilevered Action	Sheathing Shear Deformation	Nail Splitting or Bending	Tiedown Assembly	Sum of Deflection. (INCL. 25% INCR.)	Max. Inter-Story Drift
From OTM N-S walls	$8vh^3/EAb$	vh/Gt	$0.75he_n$	hda/b	$1.25\Delta s$	$\Delta_M = 0.7R\Delta s$
SW1	0.0265	0.0681	0.0534	0.1971	0.4314	1.3590
SW2	0.0266	0.0682	0.0537	0.2233	0.4648	1.4640
SW3	0.0266	0.0682	0.0537	0.2144	0.4536	1.4288
SW4	0.0265	0.0681	0.0534	0.1927	0.4259	1.3416

Table 18: Design Shear Wall Deflections

This section resumes the sample of calculations needed to design for lateral loads on a wooden four-story residential apartment located in a high seismic area.

Further calculations can be performed to design for gravity loads. For this phase of design, new products have entered the market, enabling engineers and architects to have more freedom and use stronger wood materials. Chapter II provides a description of the new engineered wood products available.

II. New Technology of Wood Products

A. Introduction to Wood

Woodframe construction is the predominant method of building homes and apartments in the United States. It is also being used, more and more often, in commercial and industrial buildings. Indeed, woodframe buildings are economical and offer design flexibility as well as strength. Pound for pound, wood is stronger than steel because it has a more favorable strength to weight ratio. Choosing wood as a construction material can also be recognized for its environmental attributes. Wood is more energy efficient building product with an R-rating about four hundred times greater than steel and about eight times greater than concrete. It is recyclable, biodegradable, and sustainable over the long term. According to a 1987 study, wood products make up about forty-seven of all industrial raw materials manufactured in the Unites States. Yet, it only consumes four percent of the energy needed to manufacture the total industrial raw materials.

Douglas Fir Larch wood products are commonly used in residential and commercial structures. These structural lumbers are not engineered, but are graded for their performance in load bearing or load-carrying applications.

Figure 16: WWPA "Western Lumber Grading Rules" Grade Stamp

(Accredited Lumber Rules-writing & Grading Agency of the American Lumber Standard Committee, Inc.)

Douglas Fir is dimensionally stable and recognized for its superior strength-to-ratio weight ratio. Its high specific gravity provides excellent nail and plate holding ability. The figure below (Figure 17) shows a typical shear wall using Douglas-Fir Larch wood. These wood products are commonly found in home retail stores. A table can be found in

Appendix p.94, summarizing the different spans for floors and ceiling joist that can be provided with this type of wood.

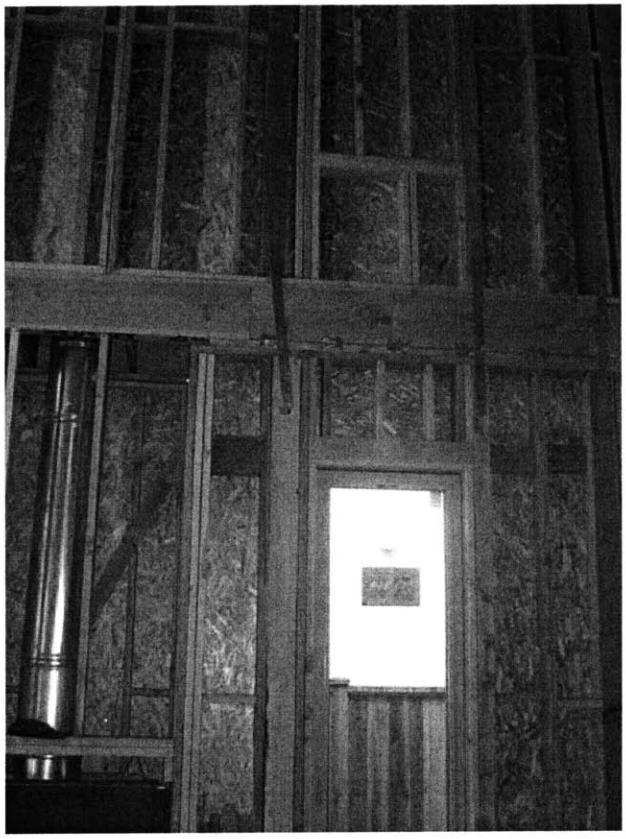


Figure 17: Typical Wood Douglas-Fir Larch Type
(Courtesy of Nina Mahjoub, 2007)

Wood is increasingly being put off-limits to harvesting. Higher quality trees are being used, which ultimately restricts the availability of high-quality lumber. It can also be noted that, even though sawn lumber is manufactured in a large number of sizes and grades, the sectional dimensions and lengths of these members are limited by the size of the trees available. Thus, when the loads become large or the span becomes longer, the use of sawn lumber becomes unfeasible. This is where engineered wood products become of critical and practical use in the construction market. Through technology, smaller, faster growing, lower quality trees are engineered to become excellent wood products. These products have greatly expanded building options and methods in all forms of residential and commercial construction.

B. New Engineered Wood Products

Structural engineered wood products are manufactured by bonding together wood fibers, such as wood strands or veneers, to produce larger composite materials. Through this manufacturing process, the wood product ends up being much more consistently reliable than lumber and can also be identified as stiffer and stronger. During the process of making engineered wood, the product is homogenized, eliminating weak points. This process also utilizes what would have been wood waste otherwise. In other words, those products become more environmentally friendly, stronger, cost-effective and easy to use. Thomas Williamson, executive vice president of Engineered Wood Systems, APA's nonprofit corporation explained that these "engineered wood products have set new performance standards by minimizing both resource and manufacturing defects while enhancing structural integrity."

The bonding process is mainly done through the use of adhesives. Those resins are used under heat and pressure to bind the wood materials (veneer, strands, and boards) and form the final engineered product. The most common binder resin system contains phenol-formaldehyde, urea-formaldehyde, melamine-formaldehyde, and isocyanate. The different types of resins used depend on their suitability in binding their respective products. For example, if cost is taken into account, urea-formaldehyde (UF) is used for particleboard (mostly utilized for the manufacture of furniture or cabinets). If durability is of importance, melamine-formaldehyde resins can be implemented, since they are known for the excellent durability, but are quite expensive. Isocyanate is usually the resin employed in the manufacture of OSB, Oriented Strand Boards (which will be discussed later on in this chapter).

Research is being done to exploit other types of adhesives that could deliver better products: lower costs, more stable, and reduction in formaldehyde's emissions. Those emissions can become a problem, causing bad health effects. Difficulty in breathing can happen if exposed to elevated levels (above 0.1 parts per million). In buildings with significant amounts of new pressed wood products, levels can be greater than 0.3 parts per million. These researches have been able to reveal that for example, soybean-based adhesive could be an option.

1. Glued Laminated Timber (Glulam)

Glulam production in North America reached in 2000 more than 350 million board feet (board feet being the basic unit of lumber measurement equaling 12 x 12 x 1 inches).

Glulam members are stress-rated engineered wood products fabricated from relatively thin laminations (a nominal of one and two inches) of wood. Those laminations are bonded together with strong, waterproof adhesives (described in the previous paragraphs). These "lams" can be end-jointed and glued together to produce any size and length members.

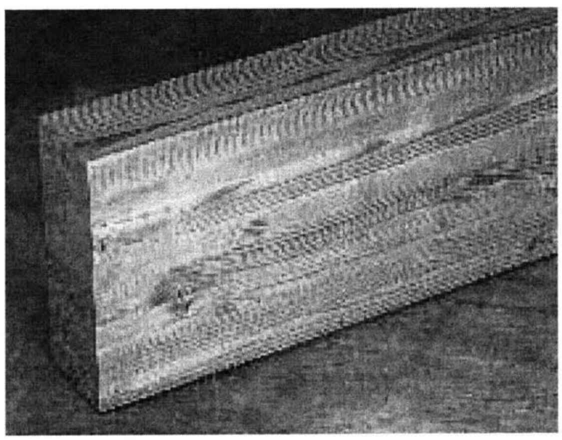


Figure 18: Glulam Beam

Glued Laminated Lumber offers architects and designers a very flexible wood product. Indeed, it can be shaped into many different forms from straight beams to complex curved members. Glulam products have increased design capabilities improving product performance while maintaining a competitive cost.

The higher strength of Glulam also allows for longer clear spans than sawn lumber. They also demonstrate minimal shrinkage and warping since they are fabricated from kiln-dried lumber. Therefore, if we use Glulam beams for our floor system, we would end up with minimal nail popping and a more leveled floor surface.

Figure 19: Floor Glulam Beams

Glulam offers many advantages in the construction phase of a project. Indeed, wood-to-wood connections can be made with typical on-site construction equipment. Other wood members can also be easily attached to the Glulam beams without nailing necessary. Additionally, intermediate supports occur less in this system because of the higher strength and stiffness of those beams.

Another beneficial aspect of Glulam wood products is the smart repartition of laminations. Indeed, high quality laminations are located in parts of the cross section that suffers the highest stresses. If we take the example of a typical Glulam, the location of maximum bending stresses under classic loading is on the outer faces of the beam, near the top and bottom of the beam (see Figure 20). Thus, wood of superior quality is placed in those outer tension and compression zones while lower quality wood is placed near the neutral axis where stresses are lower. Moreover, research has shown that even though the maximum bending compressive and tensile stresses are equal, the tension zone is more critical and thus additional strength requirements are used for those outer laminations.

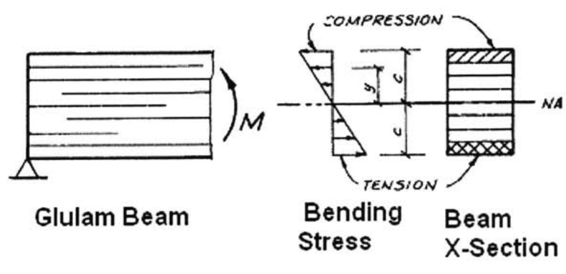


Figure 20: Distribution of Different Laminations in Glulam beams

Despite being considered a composite member (the Glulam comprises different modulus of elasticity throughout its section), a designer can treat the member as a homogeneous material with a rectangular cross section. Transformed sections have been determined and design values have been established accordingly. Therefore, a Glulam design is being carried out the same way as the design of a regular sawn lumber. Table 19 (Reference #) shows a conversion between typical sawn lumber members to their appropriate Glulam members. The complete table with detailed specifications can be found in Appendix p.95)

DOUGLAS FIR - LARCH
Dry Service Conditions
Simple Span, Uniformly Loaded

GLUED LAMINATED TIMBER CONVERSION TABLES

Glulam Design Values:	F_{bx}, psi	E_x, psi
	2,400	1,800,000

DOUGLAS FIR - LARCH LUMBER & TIMBER CONVERSIONS

1997 NDS Lumber & Timber Design Values:		F_b, psi	E, psi
Dimension Lumber, 2 to 4 inches thick and 5 inches and wider:	Select Structural:	1,500	1,900,000
	No. 1:	1,000	1,700,000
Timbers – Beams & Stringers, having a least dimension of 5 inches or greater:	Select Structural:	1,600	1,600,000
	No. 1:	1,350	1,600,000

DOUGLAS FIR - LARCH LUMBER & TIMBER SECTIONS NOMINAL SIZE thickness x depth	GLULAM SECTIONS, width (in.) x depth (in.)			
	ROOF BEAMS SNOW LOAD Load Duration Factor = 1.15		FLOOR BEAMS Load Duration Factor = 1.00	
	SELECT STRUCTURAL	No. 1	SELECT STRUCTURAL	No. 1
DIMENSION LUMBER				
3 x 8	3 1/8 x 6	3 1/8 x 6	3 1/8 x 7 1/2	3 1/8 x 7 1/2
3 x 10	3 1/8 x 7 1/2	3 1/8 x 6	3 1/8 x 9	3 1/8 x 9
3 x 12	3 1/8 x 9	3 1/8 x 7 1/2	3 1/8 x 12	3 1/8 x 10 1/2
3 x 14	3 1/8 x 9	3 1/8 x 7 1/2	3 1/8 x 13 1/2	3 1/8 x 13 1/2
4 x 6	3 1/8 x 6	3 1/8 x 6	3 1/8 x 6	3 1/8 x 6
4 x 8	3 1/8 x 7 1/2	3 1/8 x 6	3 1/8 x 9	3 1/8 x 7 1/2
4 x 10	3 1/8 x 9	3 1/8 x 7 1/2	3 1/8 x 10 1/2	3 1/8 x 10 1/2
4 x 12	3 1/8 x 10 1/2	3 1/8 x 9	3 1/8 x 12	3 1/8 x 12
4 x 14	3 1/8 x 12	3 1/8 x 10 1/2	3 1/8 x 15	3 1/8 x 15
4 x 16	3 1/8 x 13 1/2	3 1/8 x 10 1/2	3 1/8 x 16 1/2	3 1/8 x 16 1/2
MULTIPLE PIECE LUMBER				
[2] 2 x 6	3 1/8 x 6	3 1/8 x 6	3 1/8 x 6	3 1/8 x 6
[2] 2 x 8	3 1/8 x 7 1/2	3 1/8 x 6	3 1/8 x 7 1/2	3 1/8 x 7 1/2
[2] 2 x 10	3 1/8 x 9	3 1/8 x 7 1/2	3 1/8 x 10 1/2	3 1/8 x 9
[2] 2 x 12	3 1/8 x 9	3 1/8 x 7 1/2	3 1/8 x 12	3 1/8 x 12
[3] 2 x 8	5 1/8 x 7 1/2	5 1/8 x 7 1/2	5 1/8 x 7 1/2	5 1/8 x 7 1/2
[3] 2 x 10	5 1/8 x 7 1/2	5 1/8 x 7 1/2	5 1/8 x 10 1/2	5 1/8 x 9
[3] 2 x 12	5 1/8 x 9	5 1/8 x 7 1/2	5 1/8 x 12	5 1/8 x 12
[4] 2 x 8	5 1/8 x 7 1/2	5 1/8 x 7 1/2	5 1/8 x 9	5 1/8 x 7 1/2
[4] 2 x 10	5 1/8 x 9	5 1/8 x 7 1/2	5 1/8 x 10 1/2	5 1/8 x 10 1/2
[4] 2 x 12	5 1/8 x 10 1/2	5 1/8 x 9	5 1/8 x 13 1/2	5 1/8 x 12
TIMBERS				
6 x 8	5 1/8 x 7 1/2	5 1/8 x 7 1/2	5 1/8 x 7 1/2	5 1/8 x 7 1/2
6 x 10	5 1/8 x 9	5 1/8 x 7 1/2	5 1/8 x 10 1/2	5 1/8 x 10 1/2
6 x 12	5 1/8 x 10 1/2	5 1/8 x 9	5 1/8 x 12	5 1/8 x 12
6 x 14	5 1/8 x 12	5 1/8 x 10 1/2	5 1/8 x 13 1/2	5 1/8 x 13 1/2
6 x 16	5 1/8 x 13 1/2	5 1/8 x 12	5 1/8 x 16 1/2	5 1/8 x 16 1/2
6 x 18	5 1/8 x 15	5 1/8 x 13 1/2	5 1/8 x 18	5 1/8 x 18
6 x 20	5 1/8 x 18	5 1/8 x 16 1/2	5 1/8 x 19 1/2	5 1/8 x 19 1/2
8 x 10	6 3/4 x 9	6 3/4 x 9	6 3/4 x 10 1/2	6 3/4 x 10 1/2
8 x 12	6 3/4 x 10 1/2	6 3/4 x 10 1/2	6 3/4 x 12	6 3/4 x 12
8 x 14	6 3/4 x 12	6 3/4 x 12	6 3/4 x 13 1/2	6 3/4 x 13 1/2
8 x 16	6 3/4 x 13 1/2	6 3/4 x 13 1/2	6 3/4 x 16 1/2	6 3/4 x 16 1/2
8 x 18	6 3/4 x 16 1/2	6 3/4 x 15	6 3/4 x 18	6 3/4 x 18
8 x 20	6 3/4 x 18	6 3/4 x 16 1/2	6 3/4 x 19 1/2	6 3/4 x 19 1/2
8 x 22	6 3/4 x 19 1/2	6 3/4 x 18	6 3/4 x 22 1/2	6 3/4 x 22 1/2

Table 19: Glued Laminated Timber Conversion Table

(American Institute of Timber Construction)

2. Fiber Reinforced Glued Laminated

Fiber Reinforced Polymers (FRP's) are integrated into conventional Glulam beams to enhance the structural performance of those products to ultimately create greater market growth. High-strength fiber reinforced polymers are adhesively bonded to Glulam beams increasing the stiffness and bending strength of the final product (see Table 20). Those panels or layers of FRP's are positioned in the zone where tensile stresses occur (see Figure 21). Indeed, those layers have high tensile strength and stiffness compared to the regular wood in the member. Therefore, higher stresses can develop in the tension of the beam before failure occurs. The bending strength is increased because the FRP panels do not contain strength-reducing characteristics, such as knots and slope of grains along with end joints. A small percentage of FRP (about one percent) added to a Glulam beam is only needed to obtain stronger member.

	Typical FRP Glulam Beam	**Typical Douglas Fir Beam**
Tensile Strength	143,000 psi	22,400 psi
Modulus of Elasticity	10,500,000 psi	1,950,000 psi

Table 20: Comparison of Typical Mechanical Properties

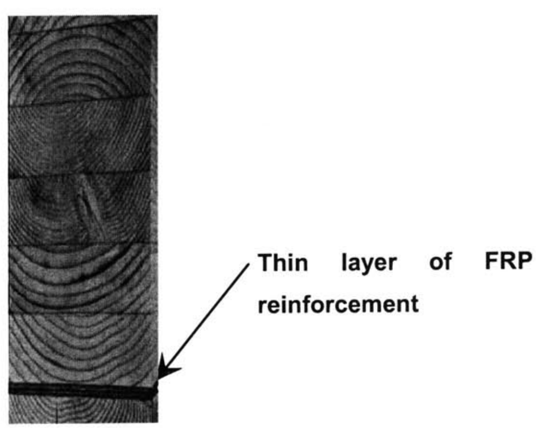


Figure 21: Reinforced Glulam Cross Section

Several advantages make the usage of this product reliable. For example, the FRP Glulam beam is smaller than an equivalent conventional member, with about one width narrower and several laminations shallower than the conventional beam carrying the same load. This detail introduces two advantages: lower cost and sustainability. Focusing on the latter, FRP Glulam can be considered a "green" material even though they have not yet being recognized by sustainable organization such as LEED (Leadership in Energy and Environmental Design). The amount of wood resource needed for a given project is significantly reduced when FRP beams are used. Table 21 shows a comparison of a FRP beam and equivalent conventional beam based on their size, weight, and cost (Gilham, Williamson, 2007).

	Beam Size	Weight	Cost
Conventional Beam #1	$14^{1/4}$ x 90	33,040 lb	$15,430
FRP Beam #1	$10^{3/4}$ x 75	20,770 lb	$12,665
Conventional Beam #2	$12^{1/4}$ x $70^{1/2}$	16,475 lb	$7,835
FRP Beam #2	$10^{3/4}$ x 57	11,690 lb	$7,130

Table 21: Size, Weight, & Cost Comparison of FRP Beams with Equivalent Wood Beam

Finally, it can be noted that the design of such beam relates to the design of a reinforced concrete beam. Indeed, the amount of FRP reinforcement in a Glulam beam can be increased or decreased depending on the strength and stiffness requirements for the beam. This is analogue to the design of a reinforced concrete beam where we use steel rebars to reinforce the capacity of a concrete beam.

3. Structural Composite Lumber

Structural Composite Lumber (SCL) is a family of reconstituted lumber products, offering particularly uniform strength and stiffness properties as well as being almost warp and split free. SCL is fabricated by layering dried wood veneers or strands with adhesives into blocks of material, each layer oriented in the same direction. Because different species can be used interchangeably, the veneering and gluing process of large timbers can therefore be made from a combination of fast-growing species and from relatively small trees. The three types of commercially available structural composite lumber are laminated veneer lumber (LVL), parallel-strand lumber (PSL), and oriented-strand lumber (OSL).

a) Laminated Veneer Lumber (LVL)

LVL is fabricated from layers of veneers with their grains all parallel to the long axis of the stock for maximum strength. LVL is commonly used for header, beam, hip, and valley rafter elements. The figure below (Figure 22) shows a sample of Laminated Veneer Lumber.

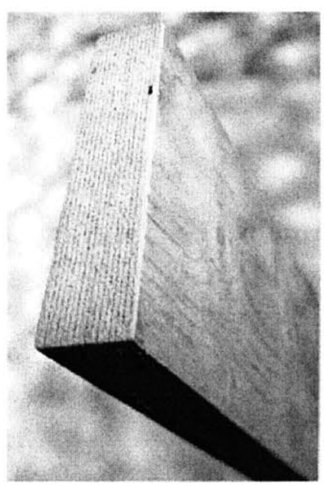


Figure 22: Sample of Laminated Veneer Lumbers (LVL)
(Selkirk Truss Limited, 2001)

One advantage of this product is its higher strength compared to lumber. Indeed, LVL has about twice the bending strength of an equivalent lumber beam. It can also be noted that the strength of this wood product is very predictable. LVL is also used to make I-joist flanges as will be described in the next section.

b) Parallel Strand Lumber (PSL)

Similar to LVL, Parallel Strand Lumber starts as a pile of veneers. One difference is that PSL uses lower grade trees infused with defects. PSL has the same usage as LVL, such as beam or header, but is also utilized as load bearing columns.

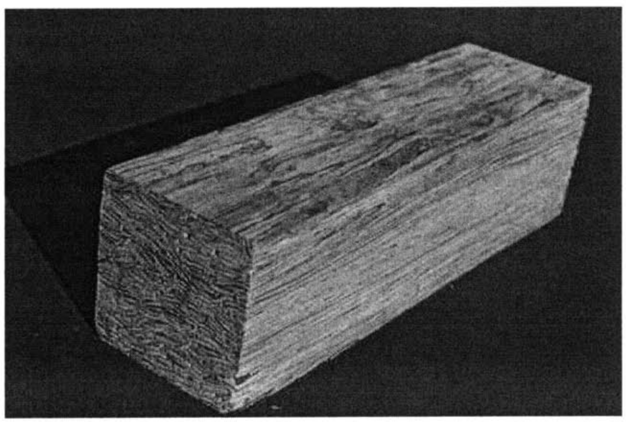


Figure 23: Parallel Strand Lumber Sample
(TRADA, 2006)

Another factor favoring the use of this product is its resistance to moisture-induced warpage, much better than with LVL. If the structural elements will be exposed to elevated moisture conditions during construction, PSL can be safely used. In fact, its composition allows a preservative treatment to penetrate the core of the product to provide protection from termites and other wet weather defects.

c) Oriented Strand Lumber (OSL)

In the case of Oriented Strand Lumber, the strands used in its fabrication are oriented, formed into large mat, and finally pressed. Their usage is primarily oriented towards studs' components.

Many companies offer their own OSL products. For example, a Canadian company, Ainsworth introduces a new application for its 0.8E Durastrand OSL Rimboard (Figure 24), which can sustain more flexural loads than conventional lumber products of the same size. They advertise their product as a good structural decision for spanning openings, eliminating the need to install a separate structural component. They believe that their product makes a viable and cost-effective alternative for short-span beams and headers. This information can be verified on their website: http://www.ainsworth.ca/

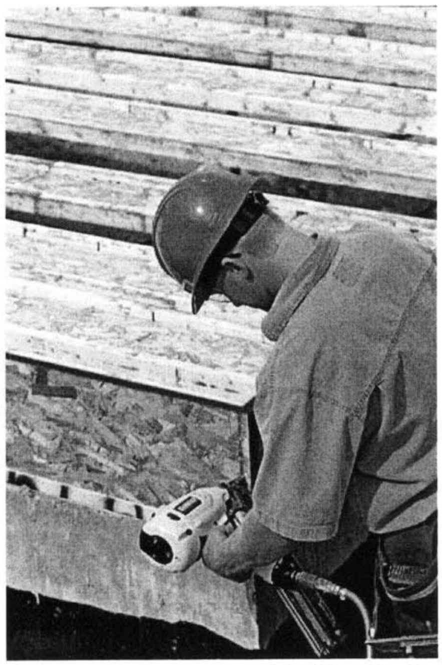


Figure 24: OSL Rimboard
(Ainsworth Company, 2007)

4. Wood I-joists

I-Joists are engineered wood products principally designed for long span applications in floor systems as well as for long roof rafters. They are composed of two horizontal components called flanges and vertical components called a web. Figure 25 provides a figurative description of the different components of a typical I-joist as well as some sample products of different sizes.

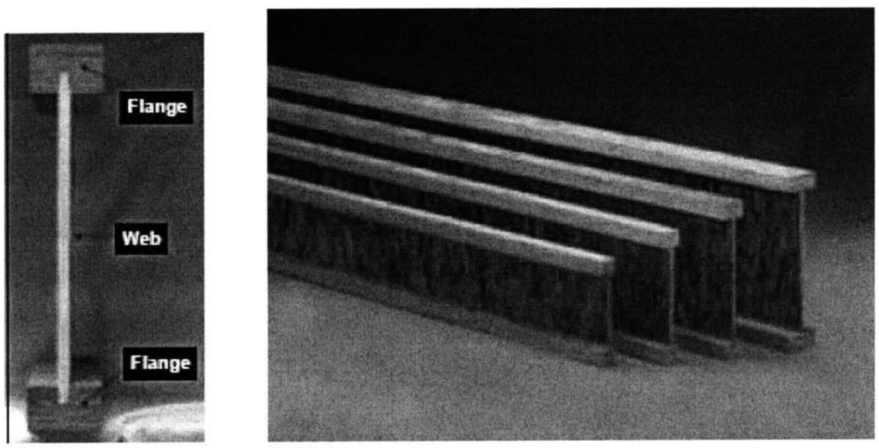


Figure 25: I-Joist Configuration & Sample Products
(American Forest & Paper Association, 2006)

The I-shape offers advantages such as a better engineering configuration. In fact, this shape allows the most efficient usage of wood necessary to carry design loads. Most of a beam's stress is along the top and bottom edges. Therefore, the center of the beam can be removed since it is redundant. This produces large weight and material savings without reducing the overall strength of the beam. It is said that I-joists require up to fifty percent less wood material to make than a conventional timber beam of same strength.

Flanges are made from end-joined, solid sawn lumber or structural composite lumber. Strong fiber are concentrated in those flanges where the stress in maximum. Webs typically are made of Oriented Strand Board or Plywood. This section is considered strong and thin, but enough to be able to transfer loads to the flanges.

As previously mentioned, I-joists allows long span to be served. Indeed, these products can extent up to sixty feet, distances that regular sawn lumber cannot span. Because of this characteristic, a single continuous joist can be used to span the entire width of a house, which is very efficient during construction. Figure 14 presents a basement floor assembly using I-joists of long spans.

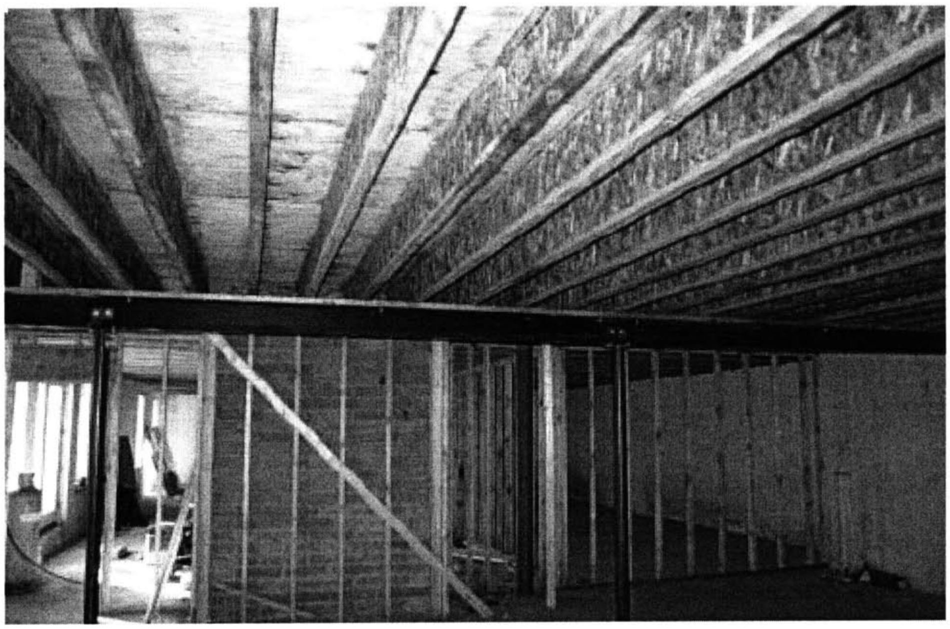


Figure 26: I-Joists in Basement Floor Assembly
(American Forest & Paper Association, 2006)

The manufacture of those products goes through many quality control procedures, making sure that the web-to-flange joint is properly shaped and fixed. I-Joists endure many physical and mechanical property tests to ensure that the products remain within specifications. Examples of such tests are shear and tensile strength tests. Other tests are made to ensure serviceability. Performance requirements are thus carried out for code acceptance.

5. Structural Wood Panels

Structural wood panels are among the engineered wood products mostly used in today's construction market. Two main types of panels are plywood and Oriented Strand Board (OSB).

a) Structural Plywood

Plywood consists of thin layers of veneer, with the grain of adjacent layers at right angles to maximize strength and stability. Indeed, considerable dimensional stability across the width of the plywood is generated from the alternation of the grain direction in adjacent plies. Figure 27 presents a schematic cross section of structural plywood, with the veneer plies.

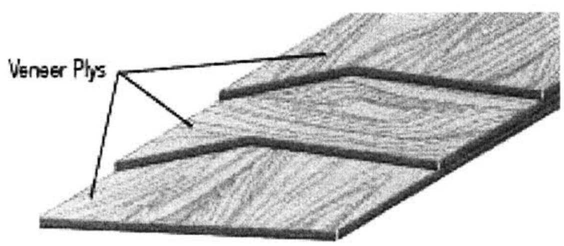


Figure 27: Schematic Structural Plywood
(Eco-Link, 2001)

Plywood must have a minimum number of plies and layers for a specific thickness range. For example, a 15/32 inch Structural 1 Plywood must have at least four plies and three layers.

The laminated construction provides the almost uniform distribution of defects ultimately reducing splitting, especially when compared to regular solid wood. However, plywood is produced from high quality veneer and could be expensive compared to the Oriented Strand Board, briefly described below. Structural plywood is mainly used in siding and sheathing for shear wall construction.

b) Oriented Strand Board (OSB)

Oriented Strand Board is believed to become the most common structural sheathing in North America. The key difference with the structural plywood is the composition of the layers. In fact, OSB is manufactured from waterproof heat-cured resins and with layers of thin, rectangular strands arranged in cross-oriented layers. It is produced in huge, continuous mats, providing a solid panel product with consistent quality with no laps. Additionally, each layer of strands is alternately placed perpendicular to the prior layer providing bending supports in two directions.

OSB can use lower quality fiber than structural plywood and can therefore become much cheaper and is winning over the market of plywood. However, it should be noted that OSB expands more than plywood when it is exposed to moisture. Fasteners can start fracturing the surface of the sheathing because of wetting and expansion. Figure 28 shows typical OSB samples while Figure 29 presents the sheathing of a residential building with OSB.

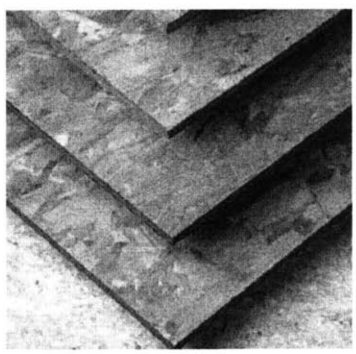


Figure 28: Oriented Strand Board Samples

(Holz Bongartz)

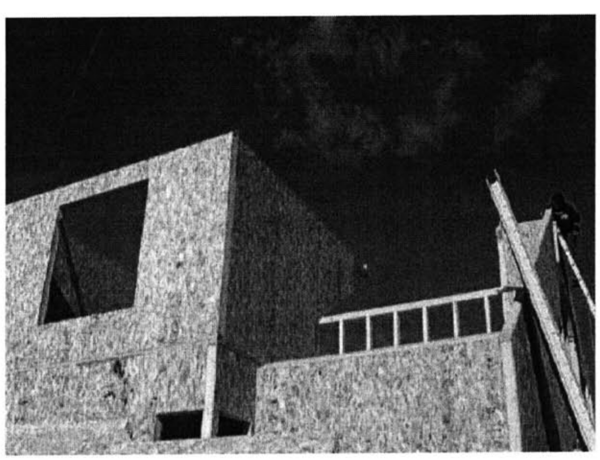


Figure 29: OSB Sheathing of Residential Construction

(APA, 2001)

6. Summary

The graph and figure in this section present information reinforcing the growth of engineered wood products in the residential and commercial construction. New technologies have emerged utilizing traditionally less desirable species, smaller trees, and lower quality trees. However, they have been able to produce excellent wood products. Engineered wood products (EWP) offer higher yields from the log. A more sustainable environment can be reached in this much polluted industry. In fact, with EWP, less waste of material is achieved and lower manufacturing cost is obtained.

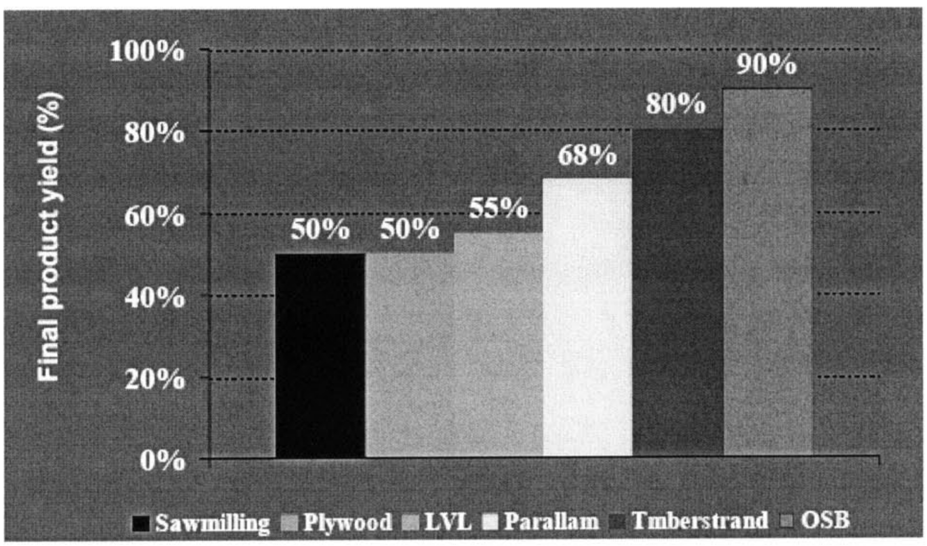


Figure 30: Final Product Yield from Log for Different EWP's

(TJ Weyco, 2002)

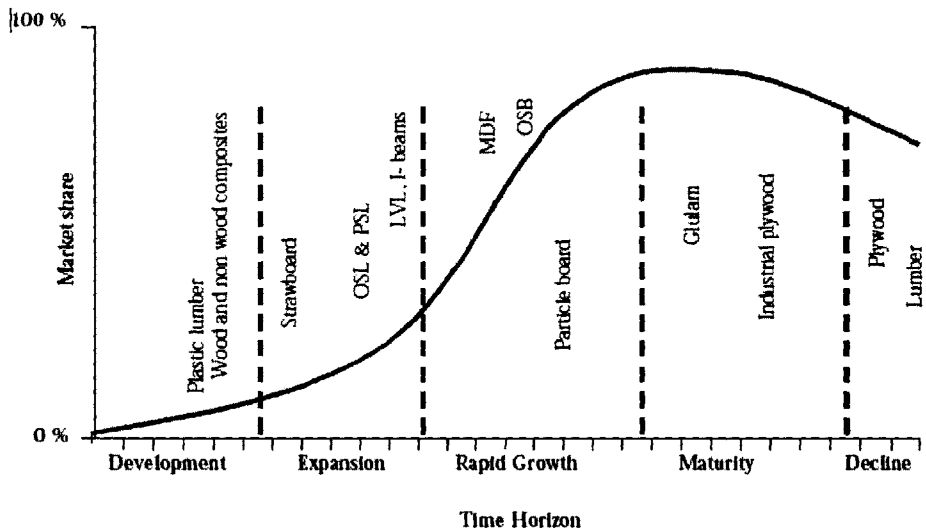


Figure 31: Engineered Wood Products Life Cycle
(Schuler ,2000)

As shown in Figure 31, EWPs continue to evolve and capture market share from conventional wood materials. Those EWPs are also being developed more rapidly in response to changing needs in the market. For example, lumber is losing appeal because its quality and performance decreases as younger and smaller trees are utilized. The costs are increasing and the consumers are becoming more demanding.

Comparing these products to steel and concrete, it is evident that engineered wood products help reduce the energy consumption of the structure. Indeed, wood is known to be the best insulator of all structural building materials; millions of small air cells are trapped within its cellular structure. Taking the example of steel, the material provides about ten times less thermal conductivity than timber, often requiring additional insulation to compensate.

Engineered wood products enhance nature's product, by building on the inherent cellular structure and engineering out natural flaws and weaknesses from the raw material.

III.Literature Review of Innovative Damping Systems

This chapter summarizes the different techniques and researches that have been started in the area of providing supplemental damping in wood structures. It also suggests different topics for future research.

In fact, low-rise woodframe structures experience many structural and non-structural damages during an earthquake. For example, in Los Angeles County, about 60,000 woodframe residences were significantly damaged by the 1994 Northridge earthquake (Holmes and Somers, 1995). The different building codes available for wood structure design carefully address life safety issues. However, new design technologies must be adopted to account for these structural damages (the cost of the damage to woodframe structures was estimated at over twenty billion dollars after the Northridge earthquake; this amount corresponds to about half the total estimated loss from the earthquake (CUREE, 1999)).

The major trend of all those papers is the true need for additional and more precise research on innovative systems and materials for earthquake-resistant wood structures. Many researches and development have been made in improving mainly the damping systems of steel, concrete, and masonry structures. Those innovative applications should now be applied to the wood framework.

During the past few years, analytical investigations have been made on the effect of applying new sorts of damping in wood structures. Those experiments have proven that these new damping systems absorbed an important quantity of the seismic input energy. Additionally, there is an ongoing project where a full-scale townhouse, filled with visco-elastic and hysteretic dampers in walls, has just been tested a few months ago. The results of this experiment are still being analyzed.

A. *Passive energy dissipation system*

Supplemental Damping in Wood-frame Structures (Dinehart, David)

Several researches have concluded that the stiffness of a shear wall decreases linearly with continuous cycling of same amplitude. This stiffness is not stabilized entailing that the durability of the wall continues to decrease. Moreover, it was found that the energy dissipation capacity of the shear wall decreases by approximately twenty percent between the first and second cyclic loading.

Thus, the paper aims toward the urgent need for new and emerging technologies focusing on passive energy dissipation devices in addition to the usage of new materials to obtain an optimum earthquake-resistant wood structure. According to the author, those systems will provide a constant source of energy dissipation that will remain steady during the different cyclic loadings.

There has been mostly analytical research on the application of passive energy dissipation devices in wood-frame walls: slotted friction devices in the corners of panels and fluid damper on one diagonal brace. Nevertheless, those investigations have only been analytical; and although they show an effective increase in dissipation of a large seismic input energy, the result should be confirmed with some experimental research to demonstrate the effects of construction tolerances, wall materials, and other technicalities.

Additionally, the author describes some experimental analysis, such as the testing of a hysteretic damper and viscoelastic dampers installed in walls. These experiments have shown that these dampers provide a constant source of energy dissipation, without impacting the design construction or dimensions of a conventional wall. Finally, the paper presents alternatives applications of viscoelastic material, where viscoelastic polymers could be directly applied to wood, or with VE material introduced between the sheathing and the stud wall. The results show that like similar previous damped wall tests, these materials provide a constant source of energy dissipation. The figure below (Figure 32) presents the comparison between a conventional shear wall and two shear walls with viscoelastic dampers installed via a diagonal bracing and on sheathing-to-stud connections. It is clear that those dampers allow the shear wall less displacement after seismic activities, dissipating more energy than a conventional shear wall.

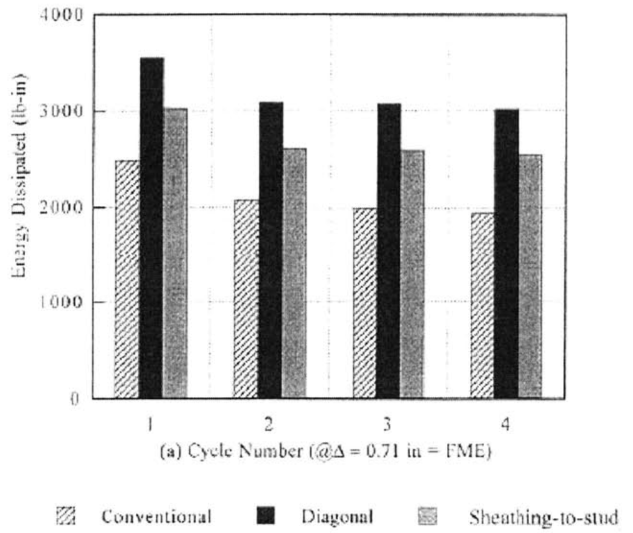


Figure 32: Energy dissipation at constant amplitude cycling amplitude

(Dinehart and Shenton, 1998)

The author also describes the implementation of viscoelasticity polymers directly to wood (Figure 33). Again, results show that this layer of VE polymers improves the energy dissipation capacity of conventional connection by more than thirty percent (Figure 34).

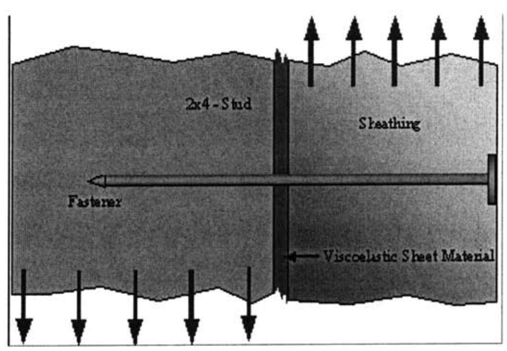


Figure 33: Schematic of VE Material Connection Test Specimen

(David W. Dinehart)

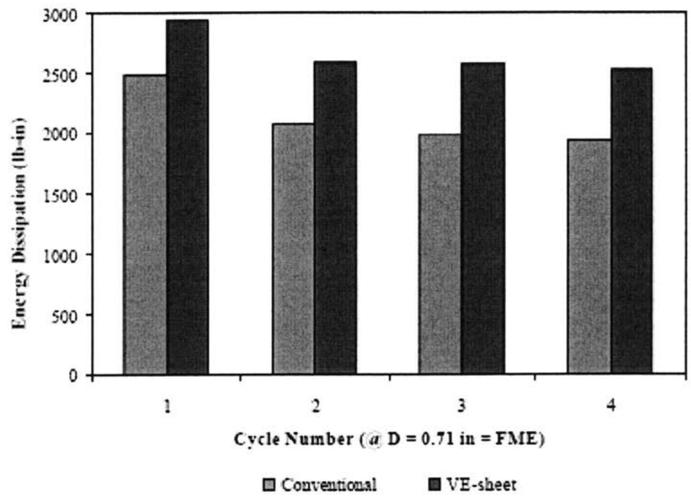


Figure 34: Comparison of Energy Dissipation of Conventional and VE-sheet Shear Walls
(David W. Dinehart)

It is true that those innovative systems improve the seismic performance of low-rise wood buildings. Nevertheless, those supplemental damping seems to be costly, especially if active systems are examined. Passive dampers remain more economical, but still need to provide a system that can be implemented by low level labor and does not require intensive operation. Therefore, it is recommended that future researches also provide a life-cycle cost analysis of those supplemental damping system.

B. From Research to Practice

1. NEESWood Project

There exists an international project intended to design a better earthquake-resistant woodframe building by installing seismic shock absorbers inside walls, NEESWood project (Network Earthquake Engineering Simulation). The objective of this project is to develop a performance based seismic design for mid-rise construction, offering an economic and sustainable option to seismic region developments.

In fact, the height of woodframe construction is currently limited to approximately four stories. This is due to many uncertainties in understanding the dynamic response of taller woodframe construction, non-structural limitations, and potential damage considerations for non-structural finishes. Another area of weakness is encountered when designing wood structure: the elements are analyzed independently without considering the influence of their stiffness and strength on other structural components.

The NEESWood project presented the test of a full-scale, 1800-square-foot townhouse while undergoing seismic testing on a shake table in November 2006. The townhouse was mounted with fluid-filled shock absorbers installed throughout certain walls of the house. Figure 35 is a picture of Professor Michael Symans of Rensselaer University (left) and Andre Filiatrault of the University of Buffalo next to one of the dampers installed in the walls of the NEESWood townhouse. Those professors, along with other universities affiliated professors, supervised the damping tests at the University of Buffalo's Structural Engineering and Earthquake Simulation Laboratory. This project has been funded by the National Science Foundation.

Figure 35: Seismic Damper Installed inside NEESWood Bedroom Wall
(University at Buffalo/Parisi, 2006)

The damper configuration is very similar to the one presented in Figure 36. This configuration provides tremendous advantage on the overall performance of a woodframe construction during seismic activity. Indeed, tests have proven that about 67% of the peak drift was reduced, 45% reduction of the peak base shear, comparing to the behavior of a conventional shear wall (Symans, Fridley, Cofer, and Du, 2001).

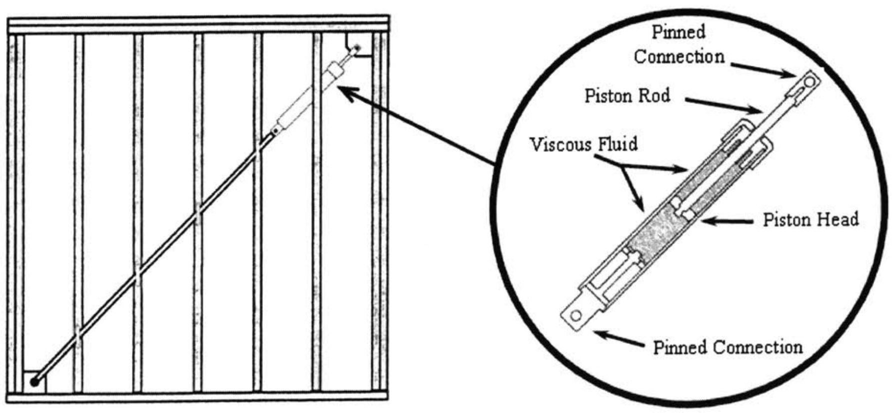


Figure 36: Fluid-filled Viscous Damper Configuration
(Symans, Fridley, Cofer, and Du, 2001)

The dampers used in the experiment have been provided by Taylor Devices (Figure 37). Those dampers have been primarily used in commercial buildings and bridges worldwide, but if the testing ends up successful, Taylor Devices will be able to acquire a brand new market (i.e. residential market).

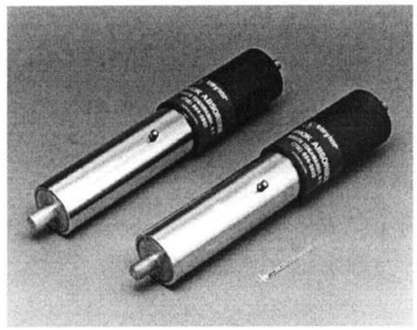


Figure 37: Taylor Seismic Fluid Viscous Damper

(Taylor Devices, Inc., 2007)

The dampers will take the energy of the seismic loading and convert it into heat. This heat will then dissipate into the atmosphere. Even though the temperature of the dampers can rise up to 200° Fahrenheit (93° Celsius), it will only take about fifteen minutes for the temperature to go back to normal.

While these dampers guarantee a better performance of woodframe structure during an earthquake, the cost remains an important obstacle. Taylor Devices Inc. affirms that it is too early to predict the cost to purchase dampers for a home. One estimate of the coast for this kind of damper is about \$300 per damper. However, this does not entail the price of installation. It could cost about \$15,000 (quite a nominal approximation) to install those dampers in an average house.

The NEESWood project has still many experiments to undergo before real changes can take place in the world of wood construction. However, it seems that this project represents the first step in moving toward performance-based design for woodframe structures. In the near future (2009), a six story NEESWood type woodframe structure will be tested on the world's largest shake table in Miki City, Japan. This experiment will permit additional validation of those new design technologies.

2. SAPWood Software

In an effort to promote performance based wood design, NEESWood developed a new analysis tool, SAPWood. This software can be downloaded, along with its user's manual, at http://www.engr.colostate.edu/NEESWood/SAPWood.htm.

SAPWood stands for Seismic Analysis Package for Woodframe. It is a user friendly software providing researchers and engineers an analysis tool that can perform nonlinear seismic analysis of woodframe structures. Thus, this software allows the user to get a better understanding of the structure behavior, moving significantly beyond the current simplified analysis. Many variables can be taken into consideration. Examples are earthquake ground motion, properties of structure, properties of finish materials, and many more. Designers are also allowed to build and analyze woodframe structures beginning at the fastener level, using nonlinear nail elements. Moreover, the designer can perform a time domain analysis (Figure 38) and/or an incremental dynamic analysis of a wood structure model with an earthquake acceleration time series record and be able to view the results of the analysis.

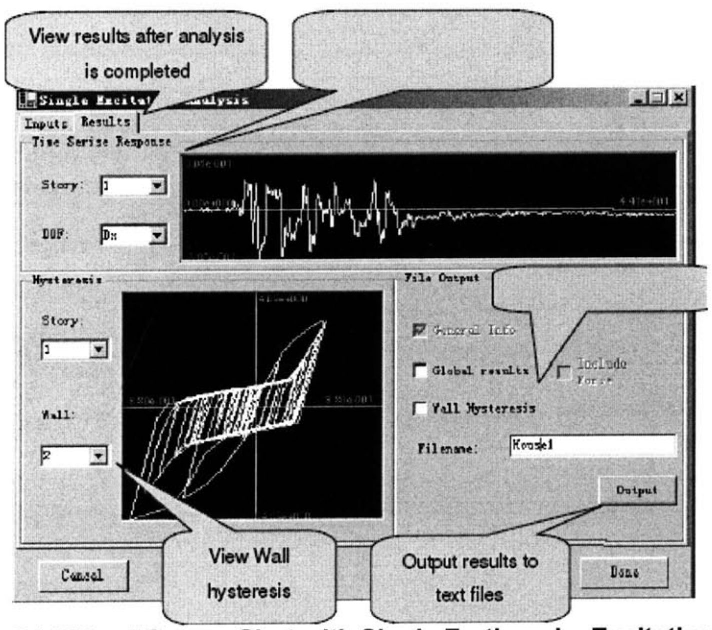


Figure 38: SAPWood Screen Shot with Single Earthquake Excitation Results
(SAPWood User's Manual)

C. *Additional Readings & Idea on Supplemental Damping Systems*

This section provides reference to additional readings on the implementation of supplemental damping systems.

Improved viscoelastic damping for earthquake-resistant wood structures (Joye and Dinehart, 2007)

This paper studies the use of viscoelastic polymeric damping material placed between the wood stud and the sheathing material. Testing has been done and the paper describes the technical aspect of the dampers performance, such as their position in the structure. The implementation of those new dampers have proven to damp out vibrations in wood structures and could eventually be used in earthquake-resistant wood structures.

Seismic Behavior of Wood-framed Structures with Viscous Fluid Dampers (Symans, 2004)

This paper introduces the use of viscous fluid dampers within the wall cavities of wood structures for their seismic protection. Extensive numerical analyses, such as nonlinear finite element models, have been able to demonstrate that those dampers dissipate a significant portion of seismic input energy.

Base Isolation & Supplemental Damping Systems for Seismic Protection of Wood Structures (Symans, 2002)

This paper provides a literature review of the implementation of different types of dampers in woodframe structures. The damping systems explained in this paper are elastomeric and sliding bearings, friction, viscoelastic, hysteric, and fluid viscous dampers. This review demonstrates the advanced seismic-resistant systems available and the need for further investigation to ultimately being able to incorporate those systems in the real construction of woodframe structures.

Disposable Damping System

Many researches seem to be devoted to the implementation of dampers inside the walls. However, the cost remains an important aspect. A new possible technique could be the implementation of a renewable, "sacrificial" damping device. This could possibly save this dilemma if one can find a way to design low-priced dampers. Those dampers could be described as being sacrificial damping device, in the sense that they can be used only for one earthquake; that could explain their low cost. They could also be fairly accessible in the house, much like a fuse box. There should also be located in clever parts of the structural system so that they could be removed after an earthquake for replacement without disturbing the original structural configurations. Japan seems to have introduced a similar system: implementing steel hysteretic dampers – "unbounded braces" in the walls (Samo L. Di and Elnashai A. S., 2005). Those dampers can be replaced after an earthquake. However, additional research and experimental tests should be developed in applying those types of dampers in woodframe structure.

Conclusion

Wood structures have seen resurgence in popularity over the past several decades, especially in Western States of America. In California, about ninety percent of residential construction consists of wood structures. For centuries, wood has been favored as a building material because it can provide strength, economy, and design flexibility. Choosing wood can also be recognized for its environmental attributes. It is recyclable, biodegradable, and sustainable over the long term, consuming only four percent of the energy needed to manufacture the total industrial raw materials while accounting for about half produced in the United States.

Woodframe construction has seen great expansion in the market of commercial and industrial construction. This means that stronger and more flexible wood products are necessary. However, the sectional dimensions and lengths of timber members are limited by the size of the trees available. Moreover, wood is increasingly being put off-limits to harvesting; higher quality trees are being used, ultimately restricting the availability of high-quality lumber. In an effort to solve this problem, the industry keeps creating new structural products, attaining a strong hand on the construction market. Engineered wood products are superior in strength, stability, and uniformity to standard lumber species. In fact, those products, manufactured by bonding together wood fibers, become larger composite materials; the manufacture process permits the achievement of homogenized products, with a decrease in defects and weak points. Those products also help in the development of a more sustainable environment. In fact, they utilize what would have been wood waste otherwise. These stronger and stiffer materials ultimately allow for the design of taller walls resisting greater environmental conditions (like high wind speed or seismic activity).

Nevertheless, restrictions still remain on woodframe construction, especially in region of high seismic zone. Indeed, the height of wooden buildings is currently limited to approximately four stories. This constraint is mainly due to uncertainties in understanding the dynamic response of taller woodframe construction. Along with this restriction rises the issue of the seismic performance of low-rise buildings. In fact, while building codes and standards emphasize life safety issues, wooden structure can experience great structural and nonstructural damage. Thus, research and new techniques aim at developing supplemental damping systems for woodframe structure. Those developments will benefit the society in a greater sense, by reducing damages, human injury, and economic loss.

Several researches have concluded that the stiffness of a conventional shear wall decreases linearly with continuous cyclic loading of same amplitude. Analytical investigations have been made on the effect of incorporating viscoelastic and hysteretic dampers in wood structures. Those dampers have been proven to absorb an important quantity of the seismic input energy. They are able to provide a constant source of energy dissipation that will remain steady during the different cyclic loadings. Performance of such woodframe structures can see a reduction of about fourty percent in peak base shear (compared to conventional shear wall). Overall, those innovative technologies have the potential to deeply influence the design and construction of woodframe structures. The potential improvements could result in a decrease of structural and nonstructural damages. However, full-scale experiments should be more abundant in order to achieve concrete and faster solutions. Finally, new techniques could also be researched, such as renewable, "sacrificial" dampers that would permit the development of lower cost systems, making them accessible to a greater market.

Appendices

1. Appendix Chapter I

a. Uniform Building Code 1997

i. Table 16-I

TABLE 16-I—SEISMIC ZONE FACTOR Z

ZONE	1	2A	2B	3	4
Z	0.075	0.15	0.20	0.30	0.40

NOTE: The zone shall be determined from the seismic zone map in Figure 16-2

ii. Table 16-J

TABLE 16-J—SOIL PROFILE TYPES

SOIL PROFILE TYPE	SOIL PROFILE NAME/GENERIC DESCRIPTION	AVERAGE SOIL PROPERTIES FOR TOP 100 FEET (30 480 mm) OF SOIL PROFILE		
		Shear Wave Velocity, $\bar{v}_s$ feet/second (m/s)	Standard Penetration Test, $\bar{N}$ [or $\bar{N}_{CH}$ for cohesionless soil layers] (blows/foot)	Undrained Shear Strength, $\bar{s}_u$ psf (kPa)
S_A	Hard Rock	> 5,000 (1,500)	—	—
S_B	Rock	2,500 to 5,000 (760 to 1,500)		
S_C	Very Dense Soil and Soft Rock	1,200 to 2,500 (360 to 760)	> 50	> 2,000 (100)
S_D	Stiff Soil Profile	600 to 1,200 (180 to 360)	15 to 50	1,000 to 2,000 (50 to 100)
S_E[1]	Soft Soil Profile	< 600 (180)	< 15	< 1,000 (50)
S_F	Soil Requiring Site-specific Evaluation. See Section 1629.3.1.			

[1]Soil Profile Type S_E also includes any soil profile with more than 10 feet (3048 mm) of soft clay defined as a soil with a plasticity index, $PI > 20$, $w_{mc} \geq 40$ percent and $s_u < 500$ psf (24 kPa). The Plasticity Index, PI, and the moisture content, w_{mc}, shall be determined in accordance with approved national standards.

iii. Table 16-K

TABLE 16-K—OCCUPANCY CATEGORY

OCCUPANCY CATEGORY	OCCUPANCY OR FUNCTIONS OF STRUCTURE	SEISMIC IMPORTANCE FACTOR, I	SEISMIC IMPORTANCE[1] FACTOR, I_p	WIND IMPORTANCE FACTOR, I_w
1. Essential facilities[2]	Group I, Division 1 Occupancies having surgery and emergency treatment areas Fire and police stations Garages and shelters for emergency vehicles and emergency aircraft Structures and shelters in emergency-preparedness centers Aviation control towers Structures and equipment in government communication centers and other facilities required for emergency response Standby power-generating equipment for Category 1 facilities Tanks or other structures containing housing or supporting water or other fire-suppression material or equipment required for the protection of Category 1, 2 or 3 structures	1.25	1.50	1.15
2. Hazardous facilities	Group H, Divisions 1, 2, 6 and 7 Occupancies and structures therein housing or supporting toxic or explosive chemicals or substances Nonbuilding structures housing, supporting or containing quantities of toxic or explosive substances that, if contained within a building, would cause that building to be classified as a Group H, Division 1, 2 or 7 Occupancy	1.25	1.50	1.15
3. Special occupancy structures[3]	Group A, Divisions 1, 2 and 2.1 Occupancies Buildings housing Group E, Divisions 1 and 3 Occupancies with a capacity greater than 300 students Buildings housing Group B Occupancies used for college or adult education with a capacity greater than 500 students Group I, Divisions 1 and 2 Occupancies with 50 or more resident incapacitated patients, but not included in Category 1 Group I, Division 3 Occupancies All structures with an occupancy greater than 5,000 persons Structures and equipment in power-generating stations, and other public utility facilities not included in Category 1 or Category 2 above, and required for continued operation	1.00	1.00	1.00
4. Standard occupancy structures[3]	All structures housing occupancies or having functions not listed in Category 1, 2 or 3 and Group U Occupancy towers	1.00	1.00	1.00
5. Miscellaneous structures	Group U Occupancies except for towers	1.00	1.00	1.00

[1]The limitation of I_p for panel connections in Section 1633.2.4 shall be 1.0 for the entire connector.
[2]Structural observation requirements are given in Section 1702.
[3]For anchorage of machinery and equipment required for life-safety systems, the value of I_p shall be taken as 1.5.

iv. Table 16-N

TABLE 16-N—STRUCTURAL SYSTEMS[1]

BASIC STRUCTURAL SYSTEM[2]	LATERAL-FORCE-RESISTING SYSTEM DESCRIPTION	R	Ω_o	HEIGHT LIMIT FOR SEISMIC ZONES 3 AND 4 (feet) × 304.8 for mm
1. Bearing wall system	1. Light-framed walls with shear panels			
	a. Wood structural panel walls for structures three stories or less	5.5	2.8	65
	b. All other light-framed walls	4.5	2.8	65
	2. Shear walls			
	a. Concrete	4.5	2.8	160
	b. Masonry	4.5	2.8	160
	3. Light steel-framed bearing walls with tension-only bracing	2.8	2.2	65
	4. Braced frames where bracing carries gravity load			
	a. Steel	4.4	2.2	160
	b. Concrete[3]	2.8	2.2	—
	c. Heavy timber	2.8	2.2	65
2. Building frame system	1. Steel eccentrically braced frame (EBF)	7.0	2.8	240
	2. Light-framed walls with shear panels			
	a. Wood structural panel walls for structures three stories or less	6.5	2.8	65
	b. All other light-framed walls	5.0	2.8	65
	3. Shear walls			
	a. Concrete	5.5	2.8	240
	b. Masonry	5.5	2.8	160
	4. Ordinary braced frames			
	a. Steel	5.6	2.2	160
	b. Concrete[3]	5.6	2.2	—
	c. Heavy timber	5.6	2.2	65
	5. Special concentrically braced frames			
	a. Steel	6.4	2.2	240
3. Moment-resisting frame system	1. Special moment-resisting frame (SMRF)			
	a. Steel	8.5	2.8	N.L.
	b. Concrete[4]	8.5	2.8	N.L.
	2. Masonry moment-resisting wall frame (MMRWF)	6.5	2.8	160
	3. Concrete intermediate moment-resisting frame (IMRF)[5]	5.5	2.8	—
	4. Ordinary moment-resisting frame (OMRF)			
	a. Steel[6]	4.5	2.8	160
	b. Concrete[7]	3.5	2.8	—
	5. Special truss moment frames of steel (STMF)	6.5	2.8	240
4. Dual systems	1. Shear walls			
	a. Concrete with SMRF	8.5	2.8	N.L.
	b. Concrete with steel OMRF	4.2	2.8	160
	c. Concrete with concrete IMRF[5]	6.5	2.8	160
	d. Masonry with SMRF	5.5	2.8	160
	e. Masonry with steel OMRF	4.2	2.8	160
	f. Masonry with concrete IMRF[3]	4.2	2.8	—
	g. Masonry with masonry MMRWF	6.0	2.8	160
	2. Steel EBF			
	a. With steel SMRF	8.5	2.8	N.L.
	b. With steel OMRF	4.2	2.8	160
	3. Ordinary braced frames			
	a. Steel with steel SMRF	6.5	2.8	N.L.
	b. Steel with steel OMRF	4.2	2.8	160
	c. Concrete with concrete SMRF[3]	6.5	2.8	—
	d. Concrete with concrete IMRF[3]	4.2	2.8	—
	4. Special concentrically braced frames			
	a. Steel with steel SMRF	7.5	2.8	N.L.
	b. Steel with steel OMRF	4.2	2.8	160
5. Cantilevered column building systems	1. Cantilevered column elements	2.2	2.0	35[7]
6. Shear wall-frame interaction systems	1. Concrete[8]	5.5	2.8	160
7. Undefined systems	See Sections 1629.6.7 and 1629.9.2	—	—	—

N.L.—no limit

[1]See Section 1630.4 for combination of structural systems.

[2]Basic structural systems are defined in Section 1629.6.

[3]Prohibited in Seismic Zones 3 and 4.

[4]Includes precast concrete conforming to Section 1921.2.7.

[5]Prohibited in Seismic Zones 3 and 4, except as permitted in Section 1634.2.

[6]Ordinary moment-resisting frames in Seismic Zone 1 meeting the requirements of Section 2214.6 may use a R value of 8.

[7]Total height of the building including cantilevered columns.

[8]Prohibited in Seismic Zones 2A, 2B, 3 and 4. See Section 1633.2.7.

v. Table 16-Q

TABLE 16-Q—SEISMIC COEFFICIENT C_a

SOIL PROFILE TYPE	SEISMIC ZONE FACTOR, Z				
	Z = 0.075	Z = 0.15	Z = 0.2	Z = 0.3	Z = 0.4
S_A	0.06	0.12	0.16	0.24	$0.32N_a$
S_B	0.08	0.15	0.20	0.30	$0.40N_a$
S_C	0.09	0.18	0.24	0.33	$0.40N_a$
S_D	0.12	0.22	0.28	0.36	$0.44N_a$
S_E	0.19	0.30	0.34	0.36	$0.36N_a$
S_F	See Footnote 1				

[1]Site-specific geotechnical investigation and dynamic site response analysis shall be performed to determine seismic coefficients for Soil Profile Type S_F.

vi. Table 16-R

TABLE 16-R—SEISMIC COEFFICIENT C_v

SOIL PROFILE TYPE	SEISMIC ZONE FACTOR, Z				
	Z = 0.075	Z = 0.15	Z = 0.2	Z = 0.3	Z = 0.4
S_A	0.06	0.12	0.16	0.24	$0.32N_v$
S_B	0.08	0.15	0.20	0.30	$0.40N_v$
S_C	0.13	0.25	0.32	0.45	$0.56N_v$
S_D	0.18	0.32	0.40	0.54	$0.64N_v$
S_E	0.26	0.50	0.64	0.84	$0.96N_v$
S_F	See Footnote 1				

[1]Site-specific geotechnical investigation and dynamic site response analysis shall be performed to determine seismic coefficients for Soil Profile Type S_F.

vii. Table 16-T

TABLE 16-T—NEAR-SOURCE FACTOR N_v[1]

SEISMIC SOURCE TYPE	CLOSEST DISTANCE TO KNOWN SEISMIC SOURCE[2,3]			
	≤ 2 km	5 km	10 km	≥ 15 km
A	2.0	1.6	1.2	1.0
B	1.6	1.2	1.0	1.0
C	1.0	1.0	1.0	1.0

[1]The Near-Source Factor may be based on the linear interpolation of values for distances other than those shown in the table.

[2]The location and type of seismic sources to be used for design shall be established based on approved geotechnical data (e.g., most recent mapping of active faults by the United States Geological Survey or the California Division of Mines and Geology).

[3]The closest distance to seismic source shall be taken as the minimum distance between the site and the area described by the vertical projection of the source on the surface (i.e., surface projection of fault plane). The surface projection need not include portions of the source at depths of 10 km or greater. The largest value of the Near-Source Factor considering all sources shall be used for design.

viii. Table 16-U

TABLE 16-U—SEISMIC SOURCE TYPE[1]

SEISMIC SOURCE TYPE	SEISMIC SOURCE DESCRIPTION	SEISMIC SOURCE DEFINITION[2]	
		Maximum Moment Magnitude, M	Slip Rate, SR (mm/year)
A	Faults that are capable of producing large magnitude events and that have a high rate of seismic activity	$M \geq 7.0$	$SR \geq 5$
B	All faults other than Types A and C	$M \geq 7.0$ $M < 7.0$ $M \geq 6.5$	$SR < 5$ $SR > 2$ $SR < 2$
C	Faults that are not capable of producing large magnitude earthquakes and that have a relatively low rate of seismic activity	$M < 6.5$	$SR \leq 2$

[1]Subduction sources shall be evaluated on a site-specific basis.

[2]Both maximum moment magnitude and slip rate conditions must be satisfied concurrently when determining the seismic source type.

ix. Figure 16-2

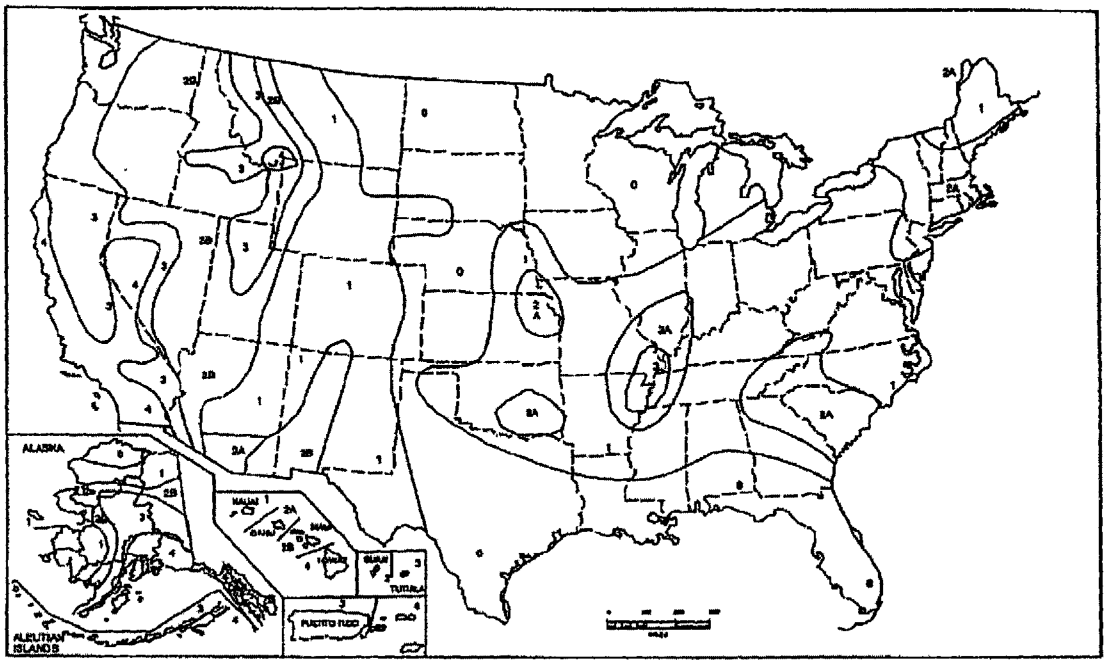


FIGURE 16-2—SEISMIC ZONE MAP OF THE UNITED STATES
For areas outside of the United States, see Appendix Chapter 16.

x. UBC Table 23-II-G

TABLE 23-II-G—MAXIMUM DIAPHRAGM DIMENSION RATIOS

MATERIAL	HORIZONTAL DIAPHRAGMS	SHEAR WALLS
	Maximum Span-Width Ratios	Maximum Height-Width Ratios
1. Diagonal sheathing, conventional	3:1	1:1[1]
2. Diagonal sheathing, special	4:1	2:1[2]
3. Wood structural panels and particleboard, nailed all edges	4:1	2:1[2, 3]
4. Wood structural panels and particleboard, blocking omitted at intermediate joints.	4:1	[4]

[1]In Seismic Zones 0, 1, 2 and 3, the maximum ratio may be 2:1.
[2]In Seismic Zones 0, 1, 2 and 3, the maximum ratio may be $3^1/_2$:1.
[3]In Seismic Zone 4, the maximum ratio may be $3^1/_2$:1 for walls not exceeding 10 feet (3048 mm) in height on one side of the door to a one-story Group U Occupancy.
[4]Not permitted.

xi. UBC Table 23-II-I-1

TABLE 23-II-I-1—ALLOWABLE SHEAR FOR WIND OR SEISMIC FORCES IN POUNDS PER FOOT FOR WOOD STRUCTURAL PANEL SHEAR WALLS WITH FRAMING OF DOUGLAS FIR-LARCH OR SOUTHERN PINE[1,2,3]

PANEL GRADE	MINIMUM NOMINAL PANEL THICKNESS (inches)	MINIMUM NAIL PENETRATION IN FRAMING (inches)	PANELS APPLIED DIRECTLY TO FRAMING: Nail Size (Common or Galvanized Box)[5]	Nail Spacing at Panel Edges (in.) 6	4	3	2	PANELS APPLIED OVER 1/2-INCH (13 mm) OR 5/8-INCH (16 mm) GYPSUM SHEATHING: Nail Size (Common or Galvanized Box)[5]	Nail Spacing at Panel Edges (in.) 6	4	3	2
	× 25.4 for mm			× 25.4 for mm; × 0.0146 for N/mm					× 25.4 for mm; × 0.0146 for N/mm			
Structural I	5/16	1 1/4	6d	200	300	390	510	8d	200	300	390	510
	3/8	1 1/2	8d	230[4]	360[4]	460[4]	610[4]	10d	280	430	550	730
	7/16			255[4]	395[4]	505[4]	670[4]					
	15/32			280	430	550	730					
	15/32	1 5/8	10d	340	510	665	870	—	—	—	—	—
C-D, C-C Sheathing, plywood panel siding and other grades covered in UBC Standard 23-2 or 23-3	5/16	1 1/4	6d	180	270	350	450	8d	180	270	350	450
	3/8			200	300	390	510		200	300	390	510
	3/8	1 1/2	8d	220[4]	320[4]	410[4]	530[4]	10d	260	380	490	640
	7/16			240[4]	350[4]	450[4]	585[4]					
	15/32			260	380	490	640					
	15/32	1 5/8	10d	310	460	600	770	—	—	—	—	—
	19/32			340	510	665	870					
			Nail Size (Galvanized Casing)					Nail Size (Galvanized Casing)				
Plywood panel siding in grades covered in UBC Standard 23-2	5/16	1 1/4	6d	140	210	275	360	8d	140	210	275	360
	3/8	1 1/2	8d	160	240	310	410	10d	160	240	310	410

[1]All panel edges backed with 2-inch (51 mm) nominal or wider framing. Panels installed either horizontally or vertically. Space nails at 6 inches (152 mm) on center along intermediate framing members for 3/8-inch (9.5 mm) and 7/16-inch (11 mm) panels installed on studs spaced 24 inches (610 mm) on center and 12 inches (305 mm) on center for other conditions and panel thicknesses. These values are for short-time loads due to wind or earthquake and must be reduced 25 percent for normal loading.

Allowable shear values for nails in framing members of other species set forth in Division III, Part III, shall be calculated for all other grades by multiplying the shear capacities for nails in Structural I by the following factors: 0.82 for species with specific gravity greater than or equal to 0.42 but less than 0.49, and 0.65 for species with a specific gravity less than 0.42.

[2]Where panels are applied on both faces of a wall and nail spacing is less than 6 inches (152 mm) on center on either side, panel joints shall be offset to fall on different framing members or framing shall be 3-inch (76 mm) nominal or thicker and nails on each side shall be staggered.

[3]In Seismic Zones 3 and 4, where allowable shear values exceed 350 pounds per foot (5.11 N/mm), foundation sill plates and all framing members receiving edge nailing from abutting panels shall not be less than a single 3-inch (76 mm) nominal member and foundation sill plates shall not be less than a single 3-inch (76 mm) nominal member. In shear walls where total wall design shear does not exceed 600 pounds per foot (8.76 N/mm), a single 2-inch (51 mm) nominal sill plate may be used, provided anchor bolts are designed for a load capacity of 50 percent or less of the allowable capacity and bolts have a minimum of 2-inch-by-2-inch-by-3/16-inch (51 mm by 51 mm by 5 mm) thick plate washers. Plywood joint and sill plate nailing shall be staggered in all cases.

[4]The values for 3/8-inch (9.5 mm) and 7/16-inch (11 mm) panels applied direct to framing may be increased to values shown for 15/32-inch (12 mm) panels, provided studs are spaced a maximum of 16 inches (406 mm) on center or panels are applied with long dimension across studs.

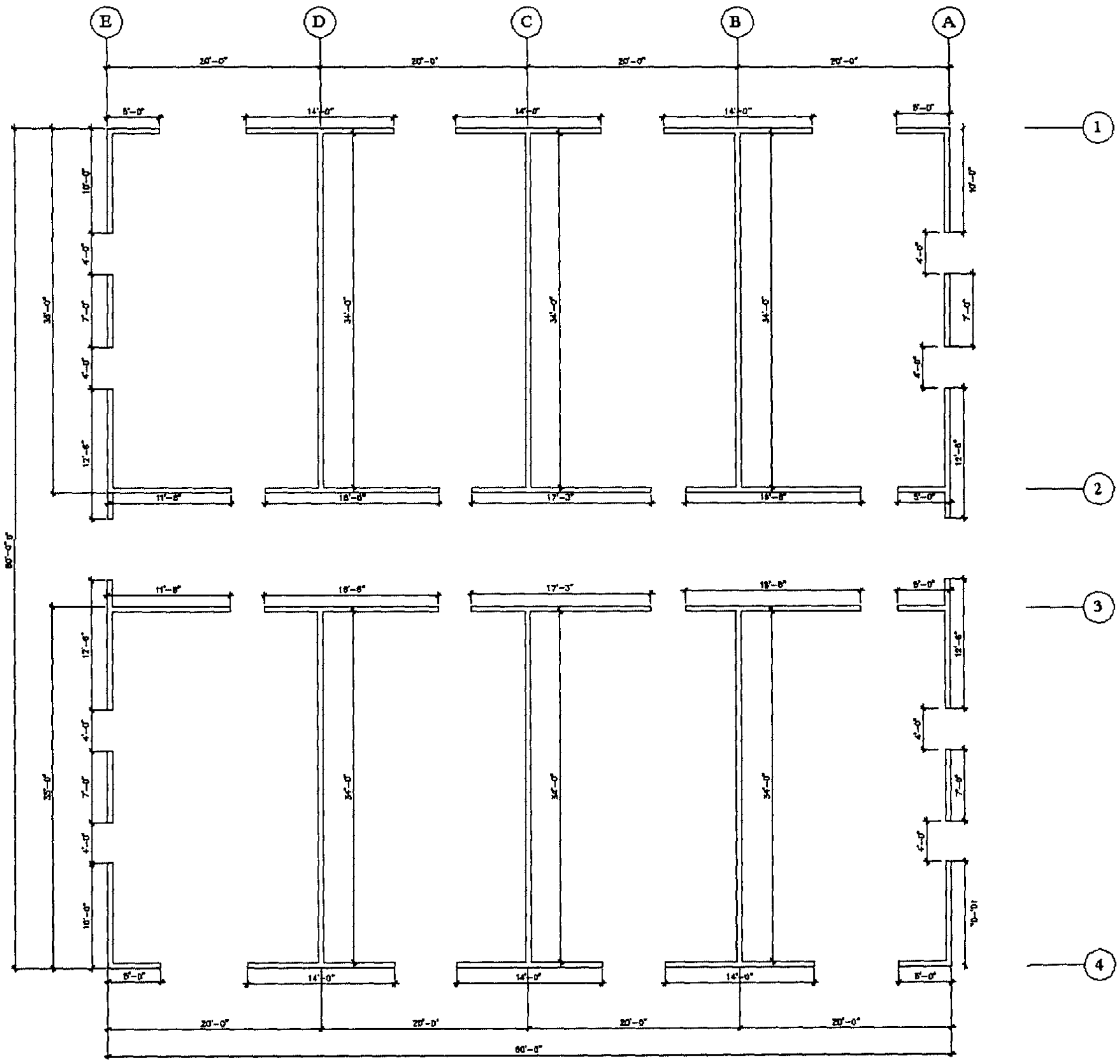
[5]Galvanized nails shall be hot-dipped or tumbled.

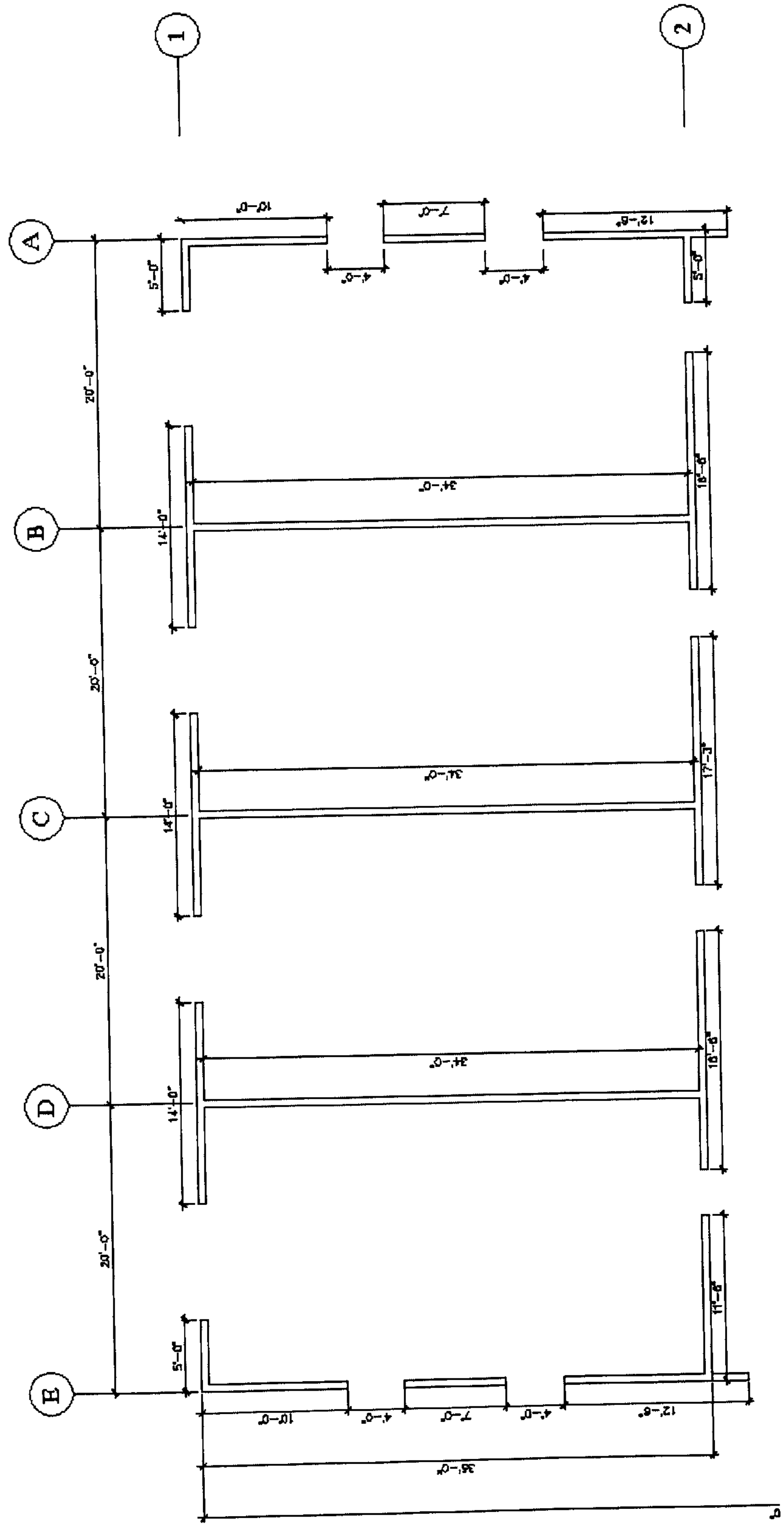
xii. Shear Capacity Adjustment Factor

Maximum Unrestrained Opening Height (Door or Window)					
h/3	h/2	2h/3	5h/6	h	
8 ft wall	2'-8"	4'-0"	5'-4"	6'-8"	8'-0"
10 ft wall	3'-4"	5'-0"	6'-8"	8'-4"	10'-0"
Percent full-height sheathing	Shear Capacity Adjustment Factor (C_o)				
0%	1	0.67	0.5	0.4	0.33
10%	1	0.69	0.53	0.43	0.36
20%	1	0.71	0.56	0.45	0.38
30%	1	0.74	0.59	0.49	0.42
40%	1	0.77	0.63	0.53	0.45
50%	1	0.8	0.67	0.57	0.5
60%	1	0.83	0.71	0.63	0.56
70%	1	0.87	0.77	0.69	0.63
80%	1	0.91	0.83	0.77	0.71
90%	1	0.95	0.91	0.87	0.83
100%	1	1	1	1	1

b. Seismic Design of Four-story Apartment – Calculation Output

i. Typical Floor Plan

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2
A
B
C
D
E
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20'-0"
20'-0"
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5'-0"
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14'-0"
34'-0"
14'-0"
34'-0"
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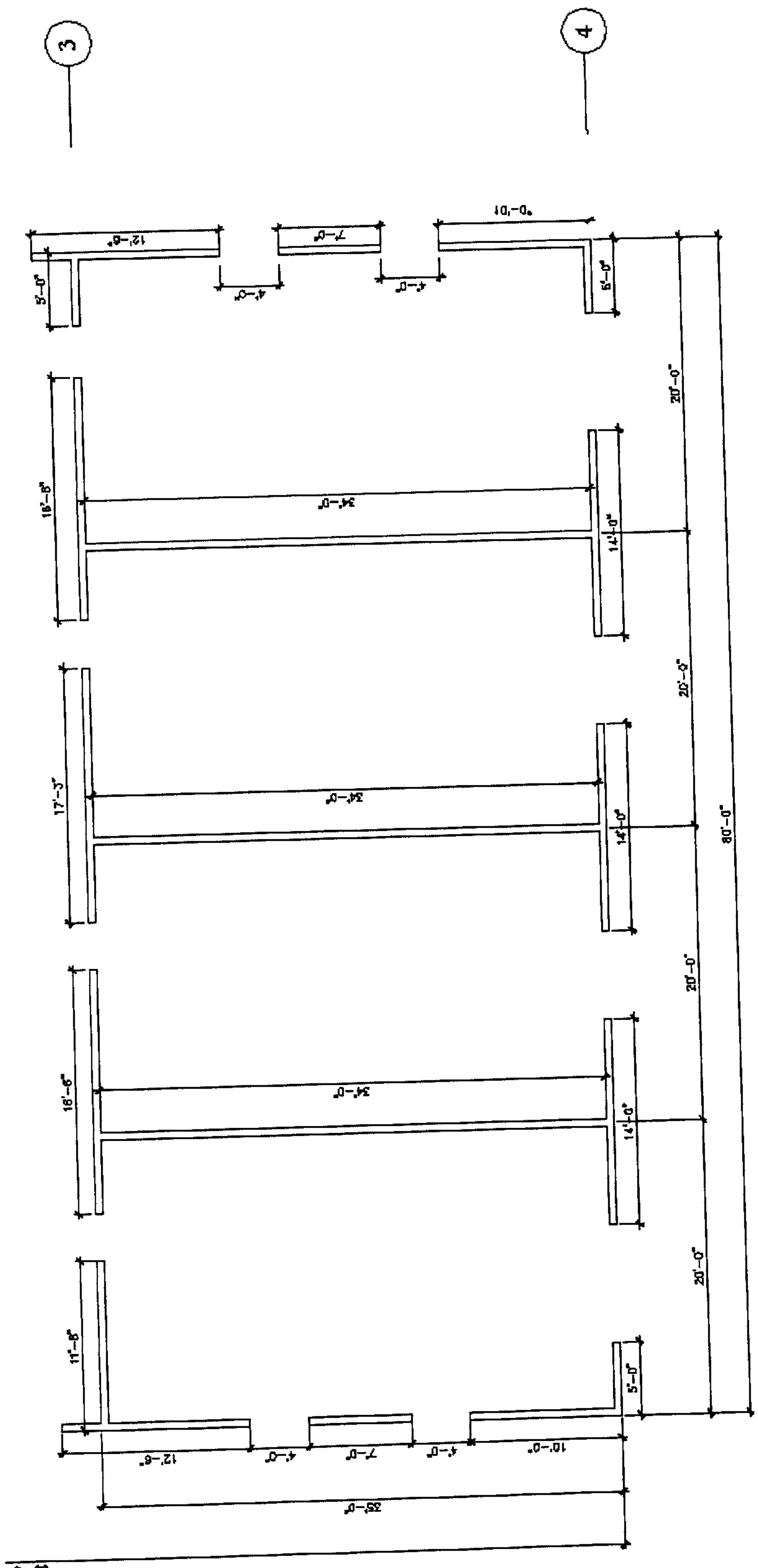
34'-0"
5'-0"
10'-0"
4'-0"
7'-0"
4'-0"
12'-6"
36'-0"


3
4
12'-6"
7'-0"
10'-0"
4'-0"
5'-0"
20'-0"
80'-0"
34'-0"
14'-0"
18'-6"
17'-3"
11'-6"
25'-0"
60'-0"

ii. East-West Shear Walls

SHEAR WALL DESIGN IN SEISMIC E-W DIRECTION:

SEISMIC LOAD:

H = (TA) X (SEISMIC LOAD PER S.F.[1,2,3,4,5]) + $H_{\text{FROM LEVEL ABOVE}}$

v = H / L

SHEAR WALLS SUPPORTING THE ROOF LEVEL:

Wall	Net Wall Length L (ft)	Tributary Area TA (SF)				Seismic Load Per SF[1]	Trib. Seis. Load, H_{TA}, lbs.[7]	SHEAR, v plf	Shear Panel Type per Table 4
SWA	59	ROOF:	80*10	=	800	9.161	7329	124	**1 - 255 plf**
SWB	68	ROOF:	80*20	=	1600	9.161	14657	216	**1 - 255 plf**
SWC	68	ROOF:	80*20	=	1600	9.161	14657	216	**1 - 255 plf**
SWD	68	ROOF:	80*20	=	1600	9.161	14657	216	**1 - 255 plf**
SWE	59	ROOF:	80*10	=	800	9.161	7329	124	**1 - 255 plf**

1 - SEISMIC LOAD PER S.F.= 82,080 lb. / (6,400 ft2 * 1.4)

SHEAR WALLS SUPPORTING THE 4TH LEVEL:

Wall	Net Wall Length L (ft)	Tributary Area TA (SF)				Seismic Load Per SF[1]	Trib. Seis. Load, H_{TA}, lbs.[7]	SHEAR, v plf	Shear Panel Type per Table 4
SWA	59	FLOOR	80*10	=	800	10.518	15743	267	**2 - 382 plf**
SWB	68	FLOOR	80*20	=	1600	10.518	31486	463	**3 - 498 plf**
SWC	68	FLOOR	80*20	=	1600	10.518	31486	463	**3 - 498 plf**
SWD	68	FLOOR	80*20	=	1600	10.518	31486	463	**3 - 498 plf**
SWE	59	FLOOR	80*10	=	800	10.518	15743	267	**2 - 382 plf**

2 - SEISMIC LOAD PER S.F.= 94,240 lb. / (6,400 ft2 * 1.4)

SHEAR WALLS SUPPORTING THE 3RD LEVEL:

Wall	Net Wall Length L (ft)	Tributary Area TA (SF)				Seismic Load Per SF[1]	Trib. Seis. Load, H_{TA}, lbs.[7]	SHEAR, v plf	Shear Panel Type per Table 4
SWA	59	FLOOR	80*10	=	800	7.125	21443	363	**2 - 382 plf**
SWB	68	FLOOR	80*20	=	1600	7.125	42886	631	**4 - 652 plf**
SWC	68	FLOOR	80*20	=	1600	7.125	42886	631	**4 - 652 plf**
SWD	68	FLOOR	80*20	=	1600	7.125	42886	631	**4 - 652 plf**
SWE	59	FLOOR	80*10	=	800	7.125	21443	363	**2 - 382 plf**

3 - SEISMIC LOAD PER S.F.= 63,840 lb. / (6,400 ft2*1.4)

Wall	Net Wall Length L (ft)	Tributary Area TA (SF)				Seismic Load Per SF[1]	Trib. Seis. Load, H_{TA} lbs.[7]	SHEAR, v plf	Shear Panel Type per Table 4
SWA	59	FLOOR:	80*10	=	800	3.732	24429	414	**3 - 498 plf**
SWB	68	FLOOR:	80*20	=	1600	3.732	48857	718	**2#2 - 764 plf**
SWC	68	FLOOR:	80*20	=	1600	3.732	48857	718	**2#2 - 764 plf**
SWD	68	FLOOR:	80*20	=	1600	3.732	48857	718	**2#2 - 764 plf**
SWE	59	FLOOR:	80*10	=	800	3.732	24429	414	**3 - 498 plf**

4 - SEISMIC LOAD PER S.F.= 33,440 lb. / (6,400 ft2 * 1.4)

xiii. Overturning Moments for N-S & E-W Walls

Overturning Moments for North-South Walls Continued

(1)	(2)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)	(13)	(14)	(15)
Wall Name	**Wall Length Li (ft)**	**Force, H (lb.)**	**Total Wall Length L (ft)**	**OTM from above (ft.-lb)**	**Total OTM (ft.-lb.)**	**Resistive Wall Length, Li (ft)**	**Resistive Load Description**	**Resistive Load, w (lb)**	**RM (ft.-lb.)**	**Tension/ Comp. T/C (lb.)**	**REMARKS**	**Conn**	**SEIS. COMP. LOAD, P**
	FROM ELEVS.	FROM SHEAR WALLS	FROM SHEAR WALLS	(7)ABOVE =	(2)x(3) x (4)/(5) + (6) =	FROM PLAN		(9) =	(10)x (8)/2 =	{(7) - [(11) x 0.9]} / (8) =			{[(7) x 3.92] + [(9) x 1.2]} / (2)

WALLS BETWEEN 3RD AND 2ND LEVELS: (3): Height:(ft) 8.36

SW1	5	37525	56	32,455	60,465	5	WALL DL: Li*(19+h)*13 ROOF DL: Li*13*17 FLOOR DL:Li*13*26*2	6263	15659	9,274	UPLIFT	Z4-T2 (28-8), 13162 lbs.	51,163
SW2	5	48246	66.75	32,507	62,720	5	WALL DL: Li*(10+h)*13 ROOF DL: Li*4*17 FLOOR DL:Li*4*26*2	3158	7896	11,123	UPLIFT	Z4-T2 (28-8), 13162 lbs.	51,068
SW3	5	48246	66.75	32,507	62,720	5	WALL DL: Li*(19+h)*10 ROOF DL: Li*13*17 FLOOR DL:Li*13*26*2	5853	14633	9,910	UPLIFT	Z4-T2 (28-8), 13162 lbs.	52,684
SW4	5	37525	56	32,455	60,465	5	WALL DL: Li*(19+h)*10 ROOF DL: Li*16*17 FLOOR DL:Li*16*26*2	6888	17220	8,993	UPLIFT	Z4-T2 (28-8), 13162 lbs.	51,537

WALLS BETWEEN 2ND AND PODIUM LEVELS: **Height : (ft)** **10.77**

SW1	5	42750	62	60,465	97,595	5.00	WALL DL: Li*(28.5+h)*13	2553	6381	18,370	UPLIFT	Z4-T2 (85-8), 24355 lbs	78,046
SW2	5	54964	68.75	62,720	105,772	5.00	WALL DL: Li *(28.5+h)* 10 ROOF DL: Li *16 * 17 FLOOR DL: Li *16 * 26*3	9564	23909	16,851	UPLIFT	Z4-T2 (46-8), 17535 lbs	88,663
SW3	5	54964	68.75	62,720	105,772	5.00	WALL DL: Li *(28.5+h)* 10 ROOF DL: Li *4.5 * 16 FLOOR DL: Li *16*26*3	9564	23909	16,851	UPLIFT	Z4-T2 (46-8), 17535 lbs	88,663
SW4	5	42750	62	60,465	97,595	5.00	WALL DL: Li *(28.5+h)* 10 ROOF DL: Li *5 * 17 FLOOR DL: Li * 5 * 26*3	4339	10846	17,567	UPLIFT	Z4-T2 (85-8), 24355 lbs	79,118

Overturning Moments for East - West Walls

(1) Wall Name	(2) Wall Length Li (ft)	(4) Force, H (lb.)	(5) Total Wall Length L (ft)	(6) OTM from above (ft.-lb)	(7) Total OTM (ft.-lb.)	(8) Resistive Wall Length, Li (ft)	(9) Resistive Load Description	(10) Resistive Load, w (lb)	(11) RM (ft.-lb.)	(12) Tension/ Comp. T/C (lb.)	(13) REMARKS	(14) Conn	(15) SEIS. COMP. LOAD, P
	FROM ELEVS.	FROM SHEAR WALLS	FROM SHEAR WALLS	(7)ABOVE =	(2)x(3) x (4)/(5) + (6) =	FROM PLAN		(9) =	(10)x (8)/2 =	{(7) - [(11) x 0.9]} / (8) =			{[(7) x 3.92] + [(9) x 1.2]} / (2) =

WALLS BETWEEN ROOF AND 4TH LEVELS:

SWA	7	8.36	7329	59	0	7,269	WALL DL:Li*h*13	761	2663	696	UPLIFT	HD2A, 2081 lbs	4,527
SWB	34	8.36	14657	68	0	61,267	WALL DL:Li*h*13	3695	62817	139	UPLIFT	HD2A, 2081 lbs	9,281
SWC	34	8.36	14657	68	0	61,267	WALL DL:Li*h*13	3695	62817	139	UPLIFT	HD2A, 2081 lbs	9,281
SWD	34	8.36	14657	68	0	61,267	WALL DL:Li*h*13	3695	62817	139	UPLIFT	HD2A, 2081 lbs	9,281

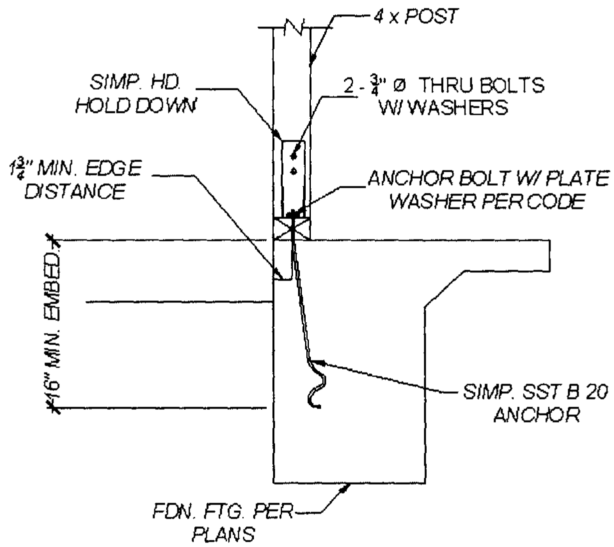
WALLS BETWEEN 4TH AND 3RD LEVELS:

SWA	7	8.36	15743	59	7,269	22,884	WALL DL: Li *(9.5+h)* 10	1250	4376	2,707	UPLIFT	HD5A, 2778 lbs	13,565
SWB	34	8.36	31486	68	61,267	192,877	WALL DL: Li *(9.5+h)* 10	6072	1032 31	2,940	UPLIFT	HD6A, 3304 lbs	25,881
SWC	34	8.36	31486	68	61,267	192,877	WALL DL: Li *(9.5+h)* 10	6072	1032 31	2,940	UPLIFT	HD6A, 3304 lbs	25,881
SWD	34	8.36	31486	68	61,267	192,877	WALL DL: Li *(9.5+h)* 13	6072	1032 31	2,940	UPLIFT	HD6A, 3304 lbs	25,881

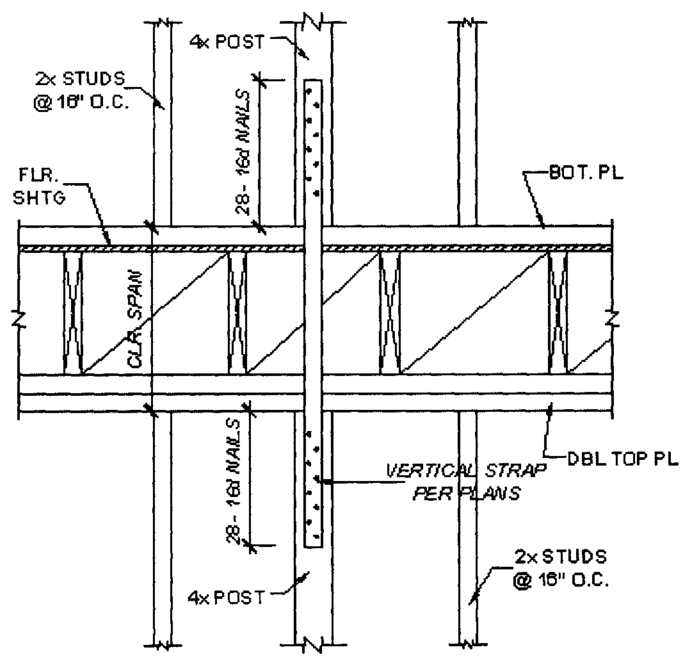
WALLS BETWEEN 2ND AND PODIUM LEVELS:

SWA	7	10.77	24429	59	44,152	44,152	WALL DL: Li *(28.5+h)* 13 ROOF DL: Li * 16 * 17 FLOOR DL: Li * 16 * 26*3	1421 4	4974 7	4,371	UPLIFT	HD8A, 4849 lbs.	50,734
SWB	34	10.77	48857	68	372,13 9	372,139	WALL DL: Li *(28.5+h)* 10	1335 2	2269 81	12,67 5	UPLIFT	Z4-T2 (28-8), 13162 lbs.	81,250
SWC	34	10.77	48857	68	372,13 9	372,139	WALL DL: Li *(28.5+h)* 10	1335 2	2269 81	12,67 5	UPLIFT	Z4-T2 (28-8), 13162 lbs.	81,250
SWD	34	10.77	48857	68	372,13 9	372,139	WALL DL: Li *(28.5+h)* 10	1335 2	2269 81	12,67 5	UPLIFT	Z4-T2 (28-8), 13162 lbs.	81,250

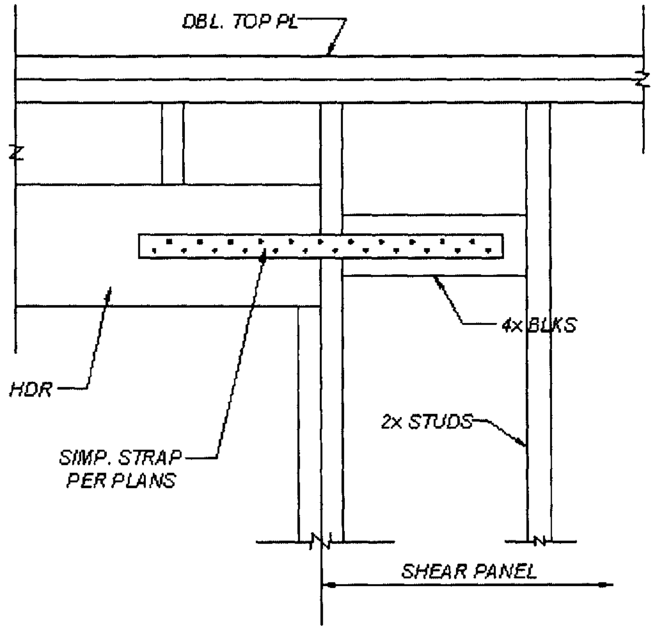
iii. Structural Details of Typical Residential in High Seismic Area



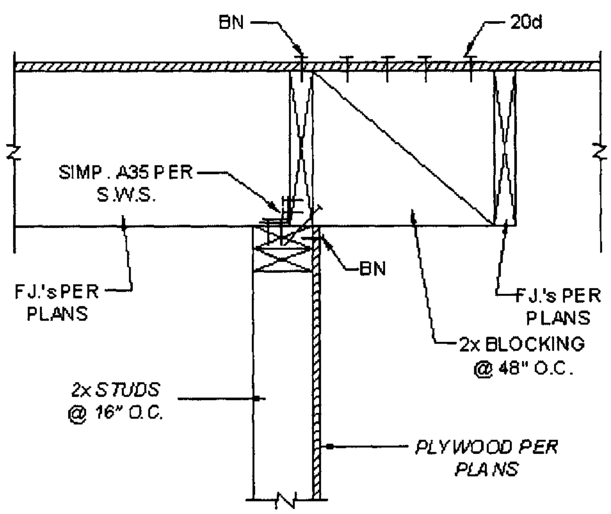
HOLD DOWN DETAIL



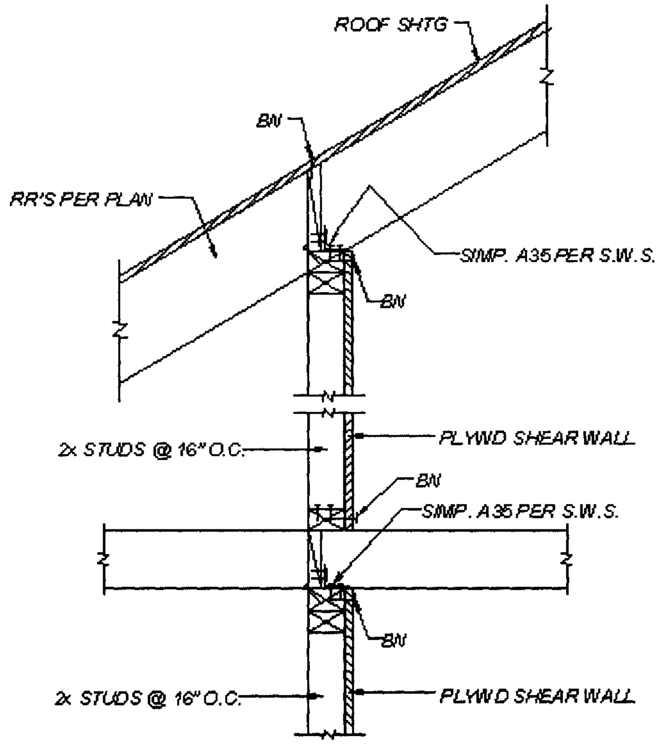
HOLD DOWN STRAP



DRAG STRAP

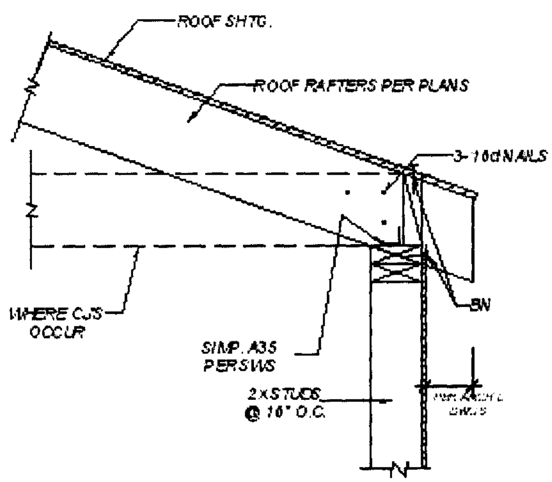


SHEAR TRANSFER @ FLR.



RAFTERS PERP. TO SHEAR WALL

SHEAR TRANSFER @ ROOF



SHEAR TRANSFER @ ROOF

2. *Appendix Chapter II*

a. Span Table for Douglas Fir Larch Lumber

SPAN TABLES BASED ON <u>DOUGLAS FIR-LARCH LUMBER</u> AS GRADED BY UBC SECTION 2303. OTHER SPECIES MAY CALCULATE DIFFERENTLY.
UNIFORM BUILDING CODE-1997

TABLE 23-IV-J-1 - ALLOWABLE SPANS FOR <u>FLOOR JOISTS</u>, 40# PER SQ. FT. LIVE LOAD, NOT TO EXCEED A DEFLECTION OF L/360.
STRENGTH: 40 PSF L.L. + 10 PSF D.L.

SIZE	SPACING	GR. NO. 1 $E=1.7 \times 10^6$	GR. NO. 2 $E=1.6 \times 10^6$	DESIGN VALUE-BENDING F_b GRADE NO.-1 NO.-2
2 x 6	12"	10' - 11"	10' - 9"	2x6 1170 1120
	16"	9' - 11"	9' - 9"	
	19.2"	9' - 4"	9' - 2"	2x8 1080 1035
	24"	8' - 8"	8' - 6"	
2 x 8	12"	14' - 5"	14' - 2"	2x10 990 950
	16"	13' - 1"	12' - 10"	2x12 900 865
	19.2"	12' - 4"	12' - 1"	
	24"	11' - 5"	11' - 3"	
2 x 10	12"	18' - 5"	18' - 0"	
	16"	16' - 9"	16' - 5"	
	19.2"	15' - 9"	15' - 5"	
	24"	14' - 7"	14' - 4"	
2 x 12	12"	22' - 5"	21' - 11"	
	16"	20' - 4"	19' - 11"	
	19.2"	19' - 2"	18' - 9"	
	24"	17' - 9"	17' - 5"	

TABLE 23-IV-J-3 - ALLOWABLE SPANS FOR <u>CEILING JOISTS</u> USING DOUGLAS FIR-LUMBER USING SHEETROCK FINISH, NOT TO EXCEED A DEFLECTION OF L/240, 10 PSF L.L + 5 PSF D.L. <u>ALSO</u> USE FOR ACCESSORY AND AG. BLDGS. WITH METAL ROOFING.

SIZE	SPACING	GR. NO. 1 $E=1.7 \times 10^6$	GR. NO. 2 $E=1.6 \times 10^6$	DESIGN VALUE-BENDING F_b GRADE NO.-1 NO.-2
2 x 4	12"	12' - 8"	12' - 5"	2X4 1350 1295
	16"	11' - 6"	11' - 3"	
	24"	10' - 0"	9' - 10"	2x6 1170 1120
2 x 6	12"	19' - 11"	19' - 6"	2x8 1080 1035
	16"	18' - 1"	17' - 8"	
	24"	15' - 9"	15' - 6"	2x10 990 950
2 x 8	12"	- -	25' - 8"	
	16"	23' - 10"	23' - 4"	2x12 900 865
	24"	20' - 10"	20' - 5"	
[1]2 x 10	12"	- -	- -	
	16"	- -	- -	
	24"	26' - 0"	26' - 0"	

b. Glued Laminated Timber Conversion Table

DOUGLAS FIR - LARCH
Dry Service Conditions
Simple Span, Uniformly Loaded

GLUED LAMINATED TIMBER CONVERSION TABLES

Glulam Design Values:	F_{bx}, psi	E_x, psi
	2,400	1,800,000

DOUGLAS FIR - LARCH LUMBER & TIMBER CONVERSIONS

1997 NDS Lumber & Timber Design Values:		F_b, psi	E, psi
Dimension Lumber, 2 to 4 inches thick and 5 inches and wider:	Select Structural:	1,500	1,900,000
	No. 1:	1,000	1,700,000
Timbers – Beams & Stringers, having a least dimension of 5 inches or greater:	Select Structural:	1,600	1,600,000
	No. 1:	1,350	1,600,000

DOUGLAS FIR - LARCH LUMBER & TIMBER SECTIONS NOMINAL SIZE thickness x depth	GLULAM SECTIONS, width (in.) x depth (in.)			
	ROOF BEAMS SNOW LOAD Load Duration Factor = 1.15		FLOOR BEAMS Load Duration Factor = 1.00	
	SELECT STRUCTURAL	No. 1	SELECT STRUCTURAL	No. 1
DIMENSION LUMBER				
3 x 8	3 1/8 x 6	3 1/8 x 6	3 1/8 x 7 1/2	3 1/8 x 7 1/2
3 x 10	3 1/8 x 7 1/2	3 1/8 x 6	3 1/8 x 9	3 1/8 x 9
3 x 12	3 1/8 x 9	3 1/8 x 7 1/2	3 1/8 x 12	3 1/8 x 10 1/2
3 x 14	3 1/8 x 9	3 1/8 x 7 1/2	3 1/8 x 13 1/2	3 1/8 x 13 1/2
4 x 6	3 1/8 x 6	3 1/8 x 6	3 1/8 x 6	3 1/8 x 6
4 x 8	3 1/8 x 7 1/2	3 1/8 x 6	3 1/8 x 9	3 1/8 x 7 1/2
4 x 10	3 1/8 x 9	3 1/8 x 7 1/2	3 1/8 x 10 1/2	3 1/8 x 10 1/2
4 x 12	3 1/8 x 10 1/2	3 1/8 x 9	3 1/8 x 12	3 1/8 x 12
4 x 14	3 1/8 x 12	3 1/8 x 10 1/2	3 1/8 x 15	3 1/8 x 15
4 x 16	3 1/8 x 13 1/2	3 1/8 x 10 1/2	3 1/8 x 16 1/2	3 1/8 x 16 1/2
MULTIPLE PIECE LUMBER				
[2] 2 x 6	3 1/8 x 6	3 1/8 x 6	3 1/8 x 6	3 1/8 x 6
[2] 2 x 8	3 1/8 x 7 1/2	3 1/8 x 6	3 1/8 x 7 1/2	3 1/8 x 7 1/2
[2] 2 x 10	3 1/8 x 9	3 1/8 x 7 1/2	3 1/8 x 10 1/2	3 1/8 x 9
[2] 2 x 12	3 1/8 x 9	3 1/8 x 7 1/2	3 1/8 x 12	3 1/8 x 12
[3] 2 x 8	5 1/8 x 7 1/2	5 1/8 x 7 1/2	5 1/8 x 7 1/2	5 1/8 x 7 1/2
[3] 2 x 10	5 1/8 x 7 1/2	5 1/8 x 7 1/2	5 1/8 x 10 1/2	5 1/8 x 9
[3] 2 x 12	5 1/8 x 9	5 1/8 x 7 1/2	5 1/8 x 12	5 1/8 x 12
[4] 2 x 8	5 1/8 x 7 1/2	5 1/8 x 7 1/2	5 1/8 x 9	5 1/8 x 7 1/2
[4] 2 x 10	5 1/8 x 9	5 1/8 x 7 1/2	5 1/8 x 10 1/2	5 1/8 x 10 1/2
[4] 2 x 12	5 1/8 x 10 1/2	5 1/8 x 9	5 1/8 x 13 1/2	5 1/8 x 12
TIMBERS				
6 x 8	5 1/8 x 7 1/2	5 1/8 x 7 1/2	5 1/8 x 7 1/2	5 1/8 x 7 1/2
6 x 10	5 1/8 x 9	5 1/8 x 7 1/2	5 1/8 x 10 1/2	5 1/8 x 10 1/2
6 x 12	5 1/8 x 10 1/2	5 1/8 x 9	5 1/8 x 12	5 1/8 x 12
6 x 14	5 1/8 x 12	5 1/8 x 10 1/2	5 1/8 x 13 1/2	5 1/8 x 13 1/2
6 x 16	5 1/8 x 13 1/2	5 1/8 x 12	5 1/8 x 16 1/2	5 1/8 x 16 1/2
6 x 18	5 1/8 x 15	5 1/8 x 13 1/2	5 1/8 x 18	5 1/8 x 18
6 x 20	5 1/8 x 18	5 1/8 x 16 1/2	5 1/8 x 19 1/2	5 1/8 x 19 1/2
8 x 10	6 3/4 x 9	6 3/4 x 9	6 3/4 x 10 1/2	6 3/4 x 10 1/2
8 x 12	6 3/4 x 10 1/2	6 3/4 x 10 1/2	6 3/4 x 12	6 3/4 x 12
8 x 14	6 3/4 x 12	6 3/4 x 12	6 3/4 x 13 1/2	6 3/4 x 13 1/2
8 x 16	6 3/4 x 13 1/2	6 3/4 x 13 1/2	6 3/4 x 16 1/2	6 3/4 x 16 1/2
8 x 18	6 3/4 x 16 1/2	6 3/4 x 15	6 3/4 x 18	6 3/4 x 18
8 x 20	6 3/4 x 18	6 3/4 x 16 1/2	6 3/4 x 19 1/2	6 3/4 x 19 1/2
8 x 22	6 3/4 x 19 1/2	6 3/4 x 18	6 3/4 x 22 1/2	6 3/4 x 22 1/2

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