A REPORT ON THE EFFECTS OF WIND SPEED ON TIMBER CONSTRUCTION

by

JOSHUA HUENEFELD

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Major Professor
Kimberly Waggle Kramer, PE, SE

Abstract

Main lateral force resisting systems (MLFRS) in timber buildings consist of two components: diaphragms and shear walls. Diaphragms are used to collect the shear induced by the lateral force at each of the levels. The shear is transferred from the diaphragms to the shear walls via plywood sheathing and connections. The shear walls transfer shear to the sill plate via plywood sheathing and then into the foundation via anchors.

Two approaches for designing shear wall are: the segmented shear wall approach and the perforated shear wall approach. The segmented shear wall approach uses only full height segments to resist shear; each individual segment must be designed to resist the shear and overturning force induced by the lateral load. The perforated shear wall approach uses both full height segments and segments around openings to resist shear; the wall as a whole is used to resist shear and overturning forces induced by the lateral load.

This report examines one-, two-, and three-story timber buildings located in three different wind regions: a) 115 mph, b) 140 mph, and c) 160 mph. This report presents the design process for the MLFRS components and a comparison of the designs for each of the buildings. The purpose of this report is to determine how the design changes depending on the magnitude of the lateral load, the height of the building, and the approach used to design the shear walls.

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Acknowledgements

Thank you to everyone who made this paper possible, especially my past and present teachers who gave me the knowledge needed to write this report.

Dedication

This report is dedicated to my parents, Randy and Kathy, who have always encouraged me to be the best that I can be.

Chapter 1 - Introduction

In timber construction, the main lateral force resisting system (MLFRS) consists of two main components: (1) wood diaphragms and (2) shear walls. These components act together to resist vertical gravity loads in addition to resisting horizontal shear induced by lateral forces. The diaphragms, consisting of wood sheathing connected to the supporting structure by nails or screws, transfer both lateral and gravity loads into the shear walls.

Shear walls carry the load from the diaphragms to the foundation through shear. These shear walls consist of wood sheathing connected, using either nails or screws, to 2 x or 3 x stud framing. The sheathing may be on one side or both sides of the studs, depending on the required capacity of the shear wall; single-sided sheathing is the most common. Shear wall capacity can be determined by any of the following methods: (1) segmented shear wall design, (2) perforated shear wall design or (3) force-transfer shear wall design. The force-transfer method is allowed by the 2012 International Building Code (IBC), however this method requires more detailing than the other two methods and is beyond the scope of this report.

For both components of the MLFRS, the intensity of the lateral loads is used to determine the type of sheathing, the type of connectors, and the spacing of connectors. For diaphragms, gravity loads are used, except in areas of high wind, to determine sheathing thickness. Lateral loads are used to determine the orientation of the wood sheathing and the blocking requirements for the diaphragms. For shear walls, the intensity of the lateral load is used to determine the thickness of the sheathing. Lateral load intensity also determines the required length and location of the shear walls, as well as the anchoring to prevent both sliding and uplift. With all of the different components of both diaphragms and shear walls that must be considered when designing a MLFRS, it is very important to understand how these components fit together, how loads are transferred between elements, and how the intensity of the lateral force affects the overall design. The focus of this report is to examine an example building under varying conditions. The building is placed in three different locations within the United States with varying wind strength level speeds, a) 115 mph, b) 140 mph, and c) 160 mph, in order to ascertain the impact that lateral forces due to wind have on the design of the MLFRS for timber structures. The building is varied in height in order to show the impact on the design of one-, two- and three-story buildings, for a total of nine structures for this parametric study. The exterior dimensions for each of the buildings are 60 ft. by 100 ft. The floor-to-floor height for each of the buildings is 12 ft., giving mean roof heights of 12ft., 24 ft. and 36 ft. respectively. Each of the buildings has a flat roof with a 2 ft. parapet above the mean roof height. The building floor plan is shown in **Figure 1-1**. The basics of MLFRS component design, including equations and the governing building codes, are presented and then each of the nine designs are presented

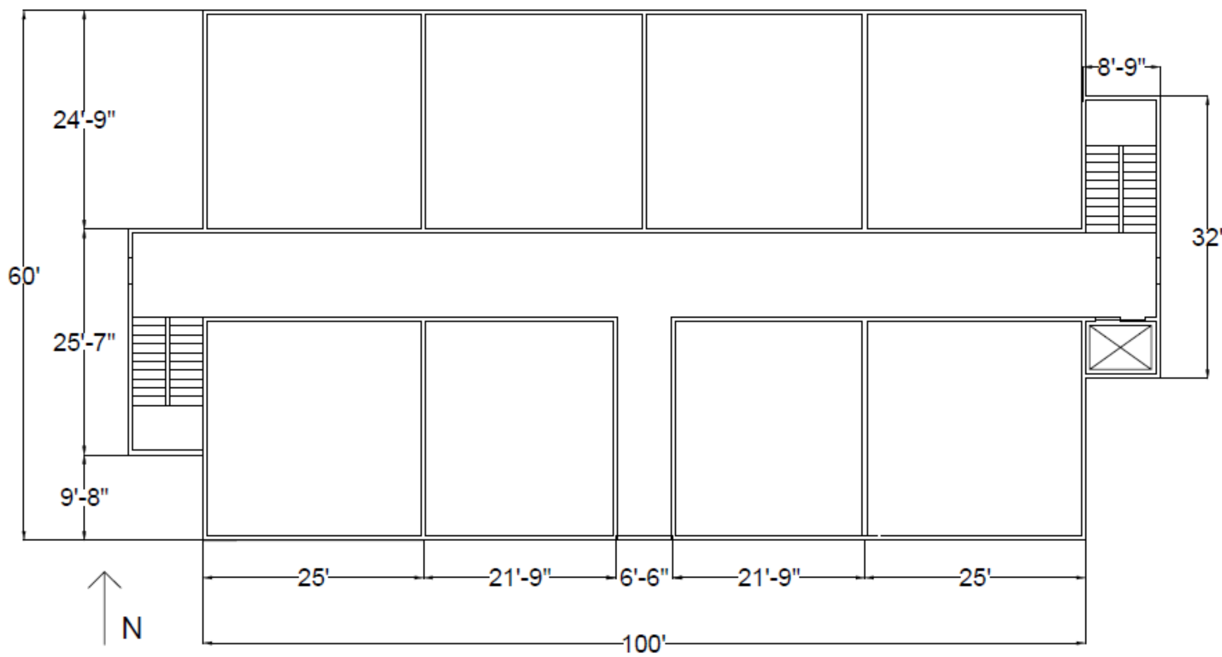

Figure 1-1: Typical Floor Plan

Chapter 2 - MLFRS Components

Diaphragms

Diaphragms are one of the main components that must be considered when designing any building's main lateral force resisting system. The purpose of diaphragms is to transfer the lateral load into the shear walls, which carry the load to the foundation. This lateral load is due to either seismic or wind loading; the focus of this report is strictly lateral loads due to wind loads. The total wind load applied to each diaphragm can be determined by taking the wind load, in pounds per square foot, multiplied by the tributary width. This width for floor diaphragms is half the story height below plus half the story height above, assuming simply supported wall studs. This width for roof diaphragms is half the story height below plus the full height of the parapet, assuming simply supported wall studs and a cantilevered parapet. This gives a distributed wind load into the diaphragm, *w*, in pounds per linear foot (plf).

Since the diaphragm is plywood, it can be designed as a "flexible" diaphragm per ASCE 7-05 Section 12.3.1.1 rather than a "rigid" diaphragm. In a flexible diaphragm, the forces are calculated using statics; the diaphragm can be idealized into a simply supported member spanning from shear wall to shear wall. This is because the diaphragm lacks the stiffness to make the building act as a unit, therefore there are no additional forces induced into the shear walls due to torsion. The total shear in a flexible diaphragm can be determined by multiplying the distributed wind load, *w*, by half the length of the wall perpendicular to the wind, *L*. Typically, a distributed unit shear, *v*, is used when designing a diaphragm, therefore the total shear must be divided by the length of wall parallel to the wind, *b*. This can be expressed by Equation 2-1 and is depicted in **Figure 2-1**:

$$v = \frac{wL}{2b} \qquad \text{(EQ 2-1)}$$

Where: v = distributed unit shear, plf.

w = distributed wind load into diaphragm, plf.

L = length of wall perpendicular to wind, ft.

b = length of wall parallel to wind, ft.

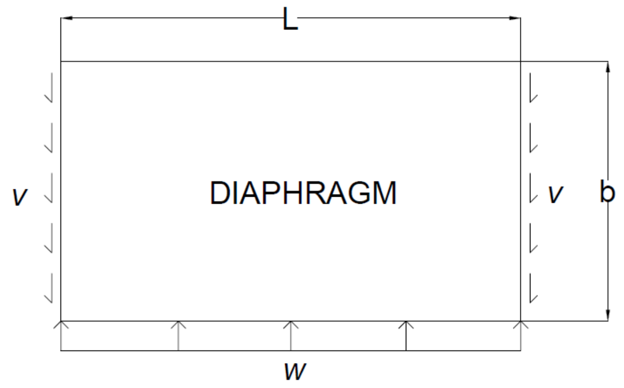

Figure 2-1: Diaphragm Unit Shear

Once the distributed unit shear is determined, the sheathing and nailing pattern can be selected. For this, Tables 4.2A and 4.2B provided by the American Forest and Paper Association and the American Wood Council within the 2008 Special Design Provisions for Wind and Seismic (SPDWS) can be used. These values are the nominal shear capacities of the diaphragms and are based on the type and thickness of sheathing, the size and spacing of nailing, the thickness of diaphragm framing members, and the layout of the sheathing. Since the listed capacities are nominal shear capacities, they must be adjusted for Allowable Stress Design (ASD) and Load and Resistance Factor Design (LRFD). For this report, ASD is used because it is the primary design method used for timber construction; for ASD, all values from the SDPWS design table must be divided by 2.0 (SPDWS 2008 Sec. 4.2.3).

When designing the diaphragm, the layout of the panels can greatly affect the capacity of the diaphragm. This is because sheathing typically comes in 4ft. x 8 ft. sections, therefore each sheet has a strong and weak direction due to the orientation of the plies of the sheathing; the sheathing is strongest for in-plane shear when the short, 4 ft., dimension is parallel to the diaphragm framing, with the lateral load also applied parallel to the framing. Six different layouts for sheathing must be considered (APA 2007):

- Case #1: Lateral loading is parallel to the framing, with the short dimension of the sheathing parallel to the framing. The sheathing is staggered perpendicular to the load with the continuous panel edge perpendicular to the framing.
- Case #2: Lateral loading is perpendicular to the framing, with the short dimension of the sheathing parallel to the framing. The sheathing is staggered perpendicular to the load with the continuous panel edge parallel to the framing.

- Case #3: Lateral loading is perpendicular to the framing, with the short dimension of the sheathing parallel to the framing and the continuous panel edge perpendicular to the framing.
- Case #4: Lateral loading is parallel to the framing, with the short dimension of the sheathing parallel to the framing and the continuous panel edge parallel to the framing.
- Case #5: Lateral loading is parallel to the framing, with the short dimension of the sheathing parallel to the framing and continuous panel edges both perpendicular and parallel to the framing.
- Case #6: Lateral loading is perpendicular to the framing, with the short dimension of the sheathing parallel to the framing and continuous panel edges both perpendicular and parallel to the framing.

For any diaphragm, two cases will apply; one for when wind is applied in the transverse direction and another when wind is applied in the longitudinal direction. **Figure 2-2** depicts each of the diaphragm sheathing cases.

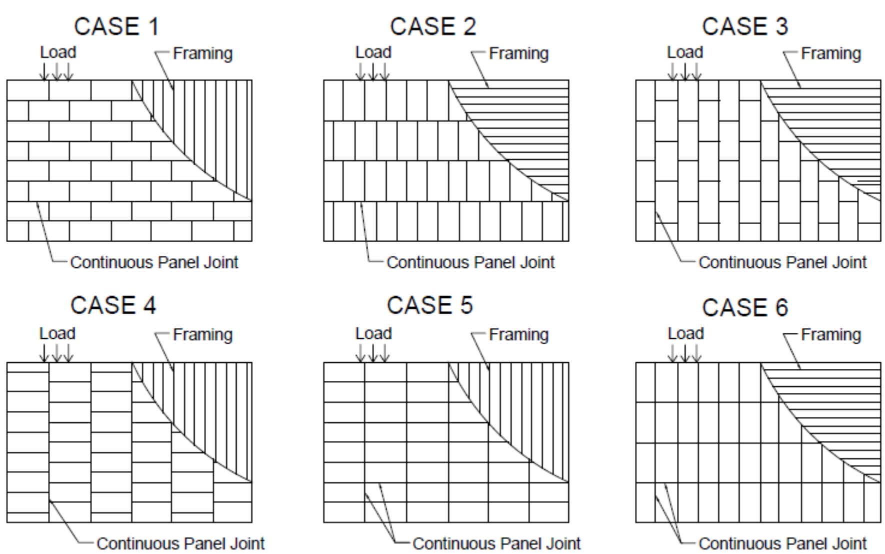

Figure 2-2: Diaphragm Sheathing Cases

When choosing the sheathing cases, it is important to look at the blocking options that are available for each of the cases. Diaphragm blocking, typically in the form of 2 x members, is provided along the unsupported edges of each of the sheathing panels. This increases the overall in-plane shear capacity of the diaphragm. In the case of an unblocked diaphragm, the governing failure mode is buckling of the unsupported panel edge; this means that there is a point at which increasing the number of nails per panel will not increase the shear capacity of the diaphragm. Providing blocking along these edges changes the governing failure mode to nail shear, therefore increasing the number of nails per panel will increase the panel's capacity, up to a certain point. When comparing an unblocked and a blocked diaphragm with the same nailing pattern, the blocked diaphragm will have a capacity 1.5 to 2 times greater than that of the unblocked diaphragm (APA 2007). Due to their greater capacities, and for ease of comparison, only blocked diaphragms are considered for this report. These diaphragms must meet the aspect ratio of 4:1 listed in **Table 2-1,** where L is the length of the long side of the diaphragm and W is the length of the short side (SDPWS 2008 Sec. 4.2.4). The purpose of these ratios is to prevent excessive deflection of the diaphragm and to allow for the use of accepted diaphragm deflection equations provided by the 2008 SDPWS Section 4.2.2.

Table 2-1: Maximum Diaphragm Aspect Ratios (L/W)

Wood Structural Panels, Unblocked	3 : 1
Wood Structural Panels, Blocked	4 : 1
Single-Layer Straight Lumber Sheathing	2 : 1
Single-Layer Diagonal Lumber Sheathing	3: 1
Double-Layer Diagonal Lumber Sheathing	4 : 1

In addition to designing the sheathing to resist the lateral loads, the diaphragm chords – the members on either side of the diaphragm perpendicular to the applied lateral force – must be designed to resist the axial load induced by the lateral load. This axial load can be calculated by determining the moment, *M*, caused by the distributed wind load, *w*, and then dividing by the width of the diaphragm parallel to the wind, *b*. This can be expressed by Equation 2-2 and is depicted in **Figure 2-3**:

$$P = \frac{M}{b} = \frac{wL^2}{8b} \qquad \text{(EQ 2-2)}$$

Where: P = axial load (tension or compression), lbs.

w = distributed wind load into diaphragm, plf.

L = length of wall perpendicular to wind, ft.

b = width of the diaphragm parallel to wind, ft.

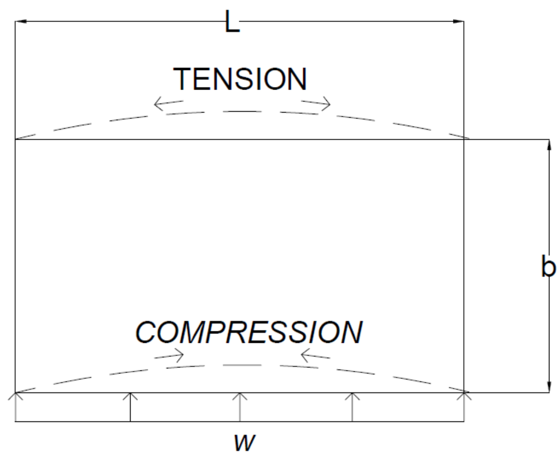

Figure 2-3: Diaphragm Chord Forces

This axial load can be either tension or compression, depending on the chord member location and the direction of the lateral loading.

Once the axial load is known, the chords are designed to resist these loads. The chords are designed as tension and compression members using the equations provided by the 2012 NDS for tension and compression capacities. These chords consist of multiple members stacked and spliced together by nails. Multiple wood members must be used because building widths are greater than the standard lengths of wood members. When determining the stress due to the axial force in pounds per square inch, the units typically used to compare loading to capacity, one of the members in the chord must be neglected when determining the tensile force; this results in a higher tensile stress than compressive stress. The reason one of the members is neglected for tension is because at the chord splices one non-continuous member occurs; the tensile force normally carried by the non-continuous member must be carried by the continuous members at splices. When designing for compression, all of the members in the chords can be considered continuous along the length of the diaphragm, therefore the full area of each chord can be used to resist the compressive force. **Figure 2-4** shows a splice detail for a chord with two members.

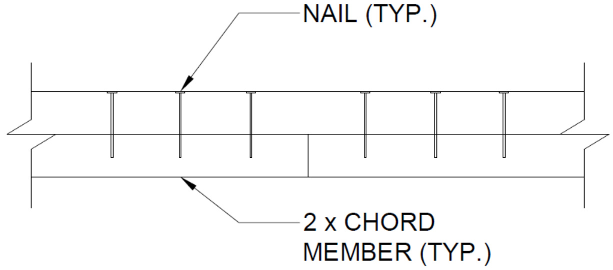

Figure 2-4: Chord Splice Detail

In addition to designing the sheathing and chords to resist lateral loads, the diaphragm must also be designed to resist gravity loads. The joists themselves are designed to resist the gravity loads determined using accepted codes and standards; the spacing, size, and span of these joists are dependent on these loads. Bending and shear capacities of these joists can be determined using capacity equations within in the 2012 NDS. Alternatively, TJI joists can be specified based on span tables provided by the manufacturer. The sheathing must also be designed to resist the gravity loads; sheathing capacity is based on the spacing of joists, type of sheathing, and the thickness of sheathing and can be determined using span tables within the 2012 IBC. In areas with lower wind speeds, gravity loads will often govern over lateral loads when sheathing is being selected for diaphragms.

Shear Walls: Segmented vs. Perforated

Lateral loads are transferred from the diaphragm sheathing into the sill joist and then into the top plate of the shear wall. The load is then transferred to the sill plate, via the sheathing and nailing, and into the foundation through the shear wall anchors. When designing the shear walls for a timber structure, two main design methodologies are used. 1) the segmented shear wall approach, uses only full height panels (sections of shear walls with no openings) to resist the applied lateral load, therefore each segment acts as an individual shear wall. 2) the perforated shear wall approach, uses the sheathing above and below the openings along with the full height sections of panels to resist the applied lateral load, therefore all of the segments act as a single shear wall. Both methods are shown in **Figure 2-5**. Both of these methods are allowed by the 2012 IBC and are widely used in structural consulting. Both methods are compared within this report to determine which is more effective for design purposes.

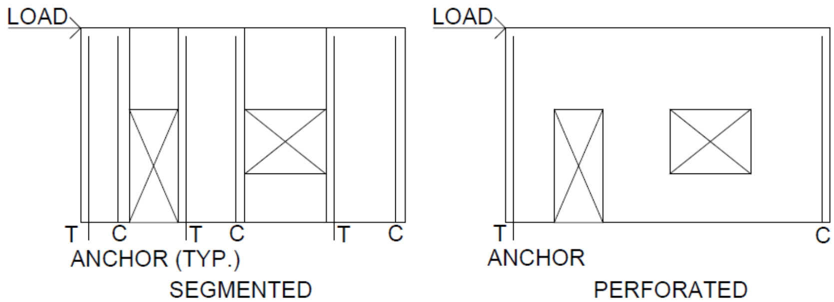

Figure 2-5: Shear Wall Types

Segmented Shear Wall Approach

In the segmented shear wall approach, the section of shear wall must not contain any openings and must also meet the aspect ratios listed in **Table 2-2** (SDPWS 2008 Sec. 4.3.4). The purpose of these ratios is to ensure that the shear wall deflection can be accurately predicted using accepted shear wall deflection equations provided by the 2008 SDPWS Section 4.3.2. The height of a shear wall panel for a multi- story building is the floor-to-floor height (the section of shear wall spanning between diaphragms) rather than the entire height of the wall. For this report, the distance from the top of diaphragm sheathing to the top of diaphragm sheathing is used for the floor-to-floor height. If openings occur, the section of shear wall above and below the opening is not taken into account. If the section exceeds the listed aspect ratio, a ratio of the panel height, H, to the panel width, W, it cannot be used. The segmented shear wall method is based on the idea that each of the shear wall sections act as individual shear walls; each section is designed for the moment and the resulting couple in the shear wall chords in each segment.

Table 2-2: Maximum Shear Wall Aspect Ratios (H/W)

Wood Structural Panels, Unblocked	2: 1
Wood Structural Panels, Blocked	3 ½ : 1
Diagonal Sheathing, Conventional	2 : 1
Particleboard, Blocked	2 : 1

To determine the shear capacity of a wall, an engineer must first determine which wall segments may be considered as the effective shear wall panels. This is done by determining the aspect ratios of each of the individual full height panels and comparing these values to the appropriate maximum allowable aspect ratio. Once this is determined, the widths of the individual effective shear panels are added together to determine the shear wall's effective length

(length of the shear wall that can be used to transfer lateral forces). The distributed unit shear, *v*, in pounds per linear foot, acting on the sections can now be determined. This is done by summing the shear forces that are transferred into the wall from the diaphragms above and dividing by the effective length of the shear wall. This can be expressed by Equation 2-3:

$$v = \frac{V}{\sum L} \qquad \text{(EQ 2-3)}$$

Where: v = distributed unit shear, plf

V = total shear in shear wall, lbs.

$\sum L$ = sum of shear wall lengths, ft.

Once the loading is known, the type of sheathing, the type and spacing of nails, and whether sheathing one or both sides of the shear wall will be used to carry the load is determined. These items are typically designed using Tables 4.3A and 4.3B in the 2008 SDPWS. These tables give the nominal shear capacity of the shear walls based on the type and thickness of sheathing and the size and spacing of nails. Since the listed capacities are nominal shear capacities, they must be adjusted. For this report, ASD is used therefore all values from the SDPWS design table must be divided by 2.0 (SPDWS 2008 Sec. 4.3.3).

Along with the sheathing, both the chords– the exterior members on either end of a shear wall segment- and the anchors for each of the shear wall segments must also be designed. The force, *P*, in each of the chords is for a single story building is determined by multiplying the distributed unit shear by the overall height of the segment, *h*. This is the same as dividing the overturning moment for each segment by the distance between chords to get the axial load. This can be expressed by Equation 2-4 and is depicted in **Figure 2-6**:

$$P = vh \qquad \text{(EQ 2-4)}$$

Where: P = axial load (tension or compression), lbs.

v = distributed unit shear, plf

h = height of the segment, ft.

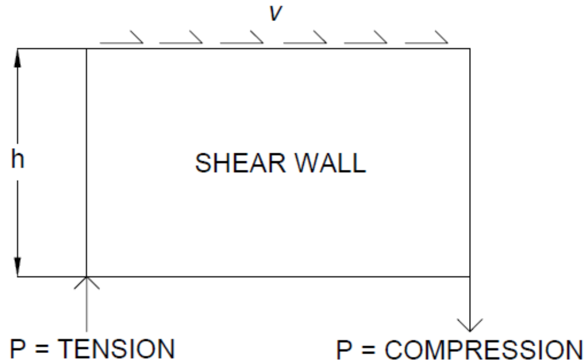

Figure 2-6: Shear Wall Chord Forces

For multi-story buildings, the total chord force is the sum of the distributed shear multiplied by the distance to the shear wall base for each level. For example, consider a two-story building. The total chord force is the distributed unit shear induced at the roof multiplied by the mean roof height plus the unit shear induced at the second floor multiplied by the height from the ground to second floor. This can be written as:

$$P = v_{roof}h_{roof} + v_{floor}h_{floor} \qquad \text{(EQ 2-5)}$$

Note that the unit shear for the second floor is not the total collected shear, but rather the load induced by the wind on that level; the roof shear is not included.

Once the chord force is known, the capacities of the chords for both compression and tension must be determined using equations provided within the 2012 NDS. These capacities are compared to the calculated axial load to determine if the chords are adequate. Anchors for each individual segment must also be designed to resist both the overturning moment and the distributed unit shear; an anchoring device is required at end of each shear wall segment, per SDPWS 2008 Section 4.3.6.4.2.

Perforated Shear Wall Approach

The perforated shear wall method uses the sheathing above and below openings as well as the full height panels to resist lateral forces; it is based on the assumption that the entire wall acts as a single unit (Breyer 2007). The framing above and below the openings is not specifically designed to transfer forces. Instead, the inherent continuity of the wall provided by typical construction methods is relied upon to transfer forces around openings. Typical construction methods include double studs on either side of the opening, a 2 x10 header with cripple studs above (if required), and, for window openings, a subsill composed of a double plate supported by cripple studs beneath; sheathing is nailed to the opening’s framing members. **Figure 2-7** gives an

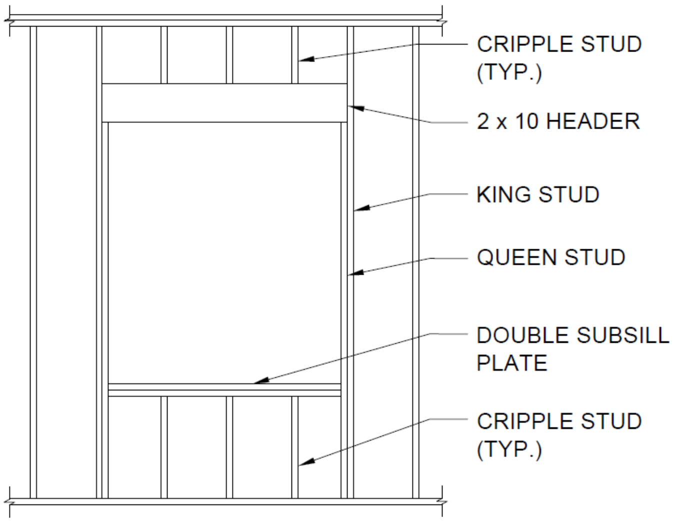

Figure 2-7: Typical Window Opening

example of typical framing for a window opening. Because the framing is not specifically designed to transfer forces, the shear wall capacity and stiffness are lower than what it could be with proper framing detailing (Breyer 2007). The design process accounts for these two facts within the equations. The main advantages of this method over the segmented shear wall method are that the number of tie-down anchors required and the number of chord members are reduced. The wall is assumed to act as a single unit, therefore the overturning moment acts on the entire wall rather than individual segments.

The perforated shear wall method was first introduced by Professor Hideo Sugiyama at the University of Tokyo in 1981. He created an empirical equation that took into account the shear capacity and stiffness of the wall as a whole. This equation was later tested extensively in the United States by several groups, including the American Plywood Association (APA), and was found to be correct. The original equation was adjusted to make it similar to the segmented shear wall approach in the hopes of it being accepted more easily by engineers. The perforated shear wall approach appeared for the first time in the 1995 edition of the American Forest and Paper Association's *Wood-Frame Construction Manual for One- and Two-Family Dwellings* (Douglas 1994). Since then, this method has been accepted by multiple codes and standards.

Before the perforated shear wall method may be used, certain requirements laid out by the IBC must be met as well as the requirements by 2008 SDPWS Section 4.3.5.3. These requirements are based on the testing discussed above and are provided to ensure that the shear wall acts as a single unit. Since these requirements are based on testing, they are subject to change in the future as further testing is completed. These requirements are the following (SDPWS 2008 Sec. 4.3.5.3):

1. A full height shear wall segment shall be located at each end of a perforated shear wall.

2. The maximum unit shear capacity for wind loading is 2,435 plf for both single-sided and double-sided sheathing.
3. Where out-of-plane offsets occur, the portions of wall on either side of the offset must be designed separately.
4. Collector elements for the shear must be provided along the entire length of the perforated shear wall.
5. The elevations at both the top and bottom of the shear wall must be constant throughout the entire length of the shear wall.
6. The maximum shear wall height allowed is 20 ft.

For perforated shear wall design, the distributed unit shear and the chord forces require more calculations than segmented shear walls. The wall is now acting as a single unit rather than as individual segments; therefore, the interaction between the full height segments and the segments around openings must be considered. Each of the full height segments within the perforated shear wall has a different degree of restraint for overturning forces and thus different capacities. This means that some of the full height segments, those that are fully restrained, are taking a larger portion of the load than those that are partially restrained. Because of these differences in capacities, a shear capacity adjustment factor, C_o, must be included to adjust for the combination of fully and partially restrained segments (Breyer 2007). This factor is used in the equations used to determine the distributed unit shear and the axial chord forces. This factor can be determined using Table 4.3.3.4 in the 2008 SDPWS, and is based on the height of the openings in the shear wall and the percentage of the shear wall that consists of full height segments. Values for C_o vary from 0.36 to 1.0, with 1.0 being the maximum value. For perforated shear walls, the equation for distributed unit shear, *v*, per SDPWS 2008 Sec. 4.3.6.4.1.1 is:

$$v = \frac{V}{C_o \sum L} \qquad \text{(EQ 2-6)}$$

Where: v = distributed unit shear, plf.

V = total shear in shear wall, lbs.

C_o = shear capacity adjustment factor

$\sum L$ = sum of lengths of full height segments, ft.

A smaller C_o value yields a larger unit shear and a larger C_o value yields a smaller unit shear. Ideally, perforated shear walls will have only a few small openings, giving a large C_o value, thereby giving the lowest possible unit shear. The maximum is set at 1.0 because greater values would result in a reduction in the distributed shear.

The forces in the chords also account for the shear capacity adjustment factor. For perforated shear walls, the equation for chord forces, *P*, per SDPWS 2008 Sec. 4.3.6.1.2 is:

$$T = C = \frac{Vh}{C_o \sum L} \qquad \text{(EQ 2-7)}$$

Where: *T (or C)* = axial load (tension or compression), lbs.

h = height of the segment, ft.

V = total shear in shear wall, lbs.

C_o = shear capacity adjustment factor

$\sum L$ = sum of lengths of full height segments, ft.

As with the unit shear, the greater C_o is, the lower the axial force is and the lower C_o is, the greater the axial force.

Once the forces are determined, the design is similar to that of the segmented shear wall method, in that Tables 4.3A and 4.3B in the 2008 SPDWS are used to determine the type of sheathing and the type and spacing of all nailing. The chords must also be designed, only instead of looking at the chords in each of the individual full height segments, only the chords at the ends of the perforated shear wall are considered. The chord capacities are determined using equations provided within the 2012 NDS. Shear and overturning anchors must also be designed, however instead of designing and laying out overturning anchors for each of the full height segments, overturning anchors for the shear wall as a whole are designed and laid out. The shear anchors are placed evenly along the entire length of the shear wall. Anchors must also be provided to resist a continuous distributed uplift force equal to the distributed unit shear along the entire length. The shear anchors that have already been provided along the shear wall are typically adequate enough to resist this uplift force without requiring additional anchors.

For both methods, several other components must be considered when designing a shear wall. For example, shear wall framing, the size and spacing of the studs, must be designed to resist both axial (gravity) and lateral (components and cladding) loads. The drag struts, members used to carry the lateral loads into the effective shear wall panels, must also be designed. Stud framing will be discussed later; however drag struts are beyond the scope of this report.

Chapter 3 - Anchorage

Anchor Design

Two types of anchors must be considered in shear wall design: shear anchors and overturning anchors. Shear anchors must be able to resist in-plane loading, due to MLFRS wind loads, and out-of-plane loading, due to components and cladding wind loads. The purpose of these anchors is to connect the shear walls to the foundation to allow for the transfer of shear; without these anchors, the building would slide off of its foundation. For segmented walls, shear anchors are calculated for the individual segments; for perforated shear walls, shear anchors are calculated for the entire length of the shear wall. Shear anchors in perforated shear walls must also be able to resist an upward distributed force equal to the horizontal distributed shear (SDPWS 2008).

Overturning anchors are designed to resist the moment that is created during wind loading. Without these anchors, the wall would overturn. Segmented shear walls require overturning anchors at each end of every segment while perforated shear walls require overturning anchors only at the ends of the overall shear wall.

To determine the number of shear anchors required, divide the shear in the panel under consideration by the shear capacity of the chosen anchor. This can be expressed by Equation 3-1:

$$N = \frac{vb}{Z'} \qquad \text{(EQ 3-1)}$$

Where: N = number of anchors

v = distributed shear, plf.

b = width of panel, ft.

Z' = capacity of anchor, lbs., NDS 2012 Chp. 11

For in-plane loading, v is the total distributed unit shear; for out-of-plane loading, v is the wind pressure times half of the first story height. For segmented shear walls, b is the width of the segment; for perforated shear walls, b is the total length of full height segments in the shear wall. Designers must determine the number of shear anchors required for both in-plane and out-of-plane loading and then select the greater of the two. These anchors are distributed evenly along the width of the wall segment under consideration, *b* (Breyer 2007).

Overturning anchors are designed to resist tensile forces, therefore to determine the area of the overturning anchors required, multiply the chord force (See **Chapter 2**) by an ASD adjustment factor, *Ω,* and then divide by the tensile yield strength of the anchor. Set this equal to the area of a circle and rearrange to determine the minimum diameter of the anchor. This equation is derived by rearranging AISC 360 Equation D2-1 for tensile yielding in members and can be expressed by Equations 3-2 and 3-3:

$$A = \frac{\pi d^2}{4} = \frac{P\Omega}{F_y} \qquad \text{(EQ 3-2)}$$

$$d = \sqrt{\frac{4P\Omega}{\pi F_y}} \qquad \text{(EQ 3-3)}$$

Where: A = minimum area of the anchor, in^2

d = minimum diameter of the anchor, in.

P = chord force (ASD), lbs.

Ω = ASD adjustment factor, 1.67

F_y = tensile strength of anchor, psi.

Typically overturning anchors are only at the ends of the shear walls, however they can be distributed if the overturning force is large; this allows for the use of smaller anchors. Distributing the overturning anchors does require additional calculations when determining the force that each anchor must resist, therefore it is not recommended in timber design.

Types of Anchors

Several anchoring options are available when determining the type of overturning anchoring system to use. These anchoring systems can be placed in three categories: (1) hold-downs with threaded anchors, (2) embedded hold-downs, and (3) threaded rods with bearing plates. Hold-downs with threaded anchors provide brackets nailed, screwed or bolted to all shear wall chord members. These brackets are also attached to concrete anchors that are embedded within the foundation. Embedded hold-downs require embedding one end of the anchor into the concrete while attaching the other end to the chord member via bolts or nails. Threaded rods with bearing plates are typically used for multiple story buildings. For this system, steel rods are run from the foundation to the top of the shear wall's top plate, where the rod is secured; a bearing plate is provided to ensure that the rod does not rupture through the top plate of the wall (Vilasineekul 2011). **Figure 3-1** shows examples of each of these anchoring systems.

Each of these types of anchors is available from manufacturers such as Simpson Strongtie or Hilti and can be chosen using tables provided within product catalogs. These tables are based on the type of overturning anchor, type of wood, size of the chord member, size of the concrete anchor and, for hold-downs and embedded anchors, the number of fasteners (Simpson 2011). As the design portion of this report shows, sometimes two anchors are required per chord to meet the required strength. In some cases, the chord size must be increased to accommodate larger anchors. For all of the designs discussed within this report, hold-down overturning anchors are used.

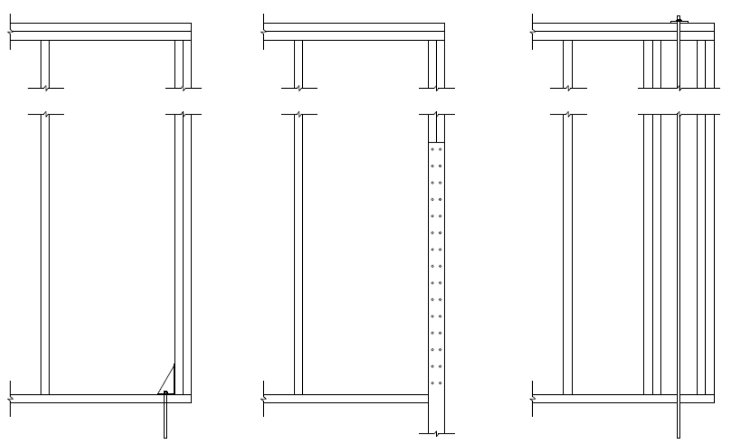

Figure 3-1: Anchoring System Assemblies

Shear anchoring is typically achieved by using anchor rods with plate washers attached to the sill plate of the shear wall. The 2008 SDPWS requires the use of plate washers for all anchors that are used to resist in-plane shear; this reduces the likelihood of failure of the sill plate due to cross-grain bending, thereby encouraging a ductile limit state for the shear wall. This is especially important when high strength sheathing is being used because the sheathing panels have a high enough capacity that the sill plate will fail before the sheathing if plate washers are not provided. These washers are to have minimum dimensions of 0.229” x 3” x 3” and must be within ½” of the sheathed edge of the sill plate (SDPWS 2008). **Figure 3-2** shows a typical plate washer assembly detail. Instances when plate washers may be ignored can be found in the 2008 SDPWS Section 4.3.6.4.3. The key requirement for each of these instances is that the anchors must be loaded in shear only. It is recommended that plate washers be used for all instances because the likelihood of anchors being loaded in shear only over the life of the anchor is small (Vilasineekul 2011).

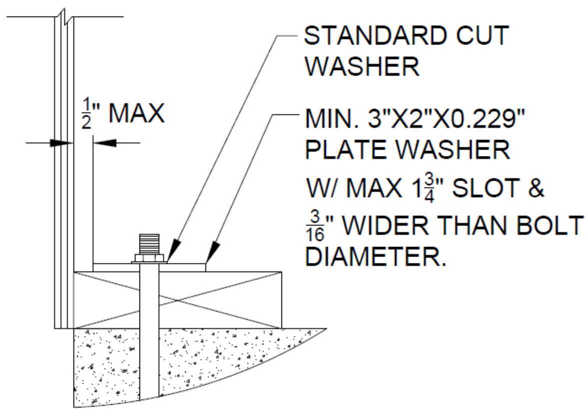

Figure 3-2: Typical Plate Washer Assembly

In addition to the different anchoring systems available, several types of anchor bolts/threaded rods are available, including anchor bolts with plate washers, deformed anchor bolts, and concrete screw anchors. The types of anchors being used depends on the required capacity per anchor, the amount of space available, and the budget of the project. The most common of these anchors is the anchor bolts with plate washers. Each of these types of anchor bolts is shown in **Figure 3-3**. These anchors should be designed using Appendix D of the latest ACI code; the design of these anchors is beyond the scope of this report.

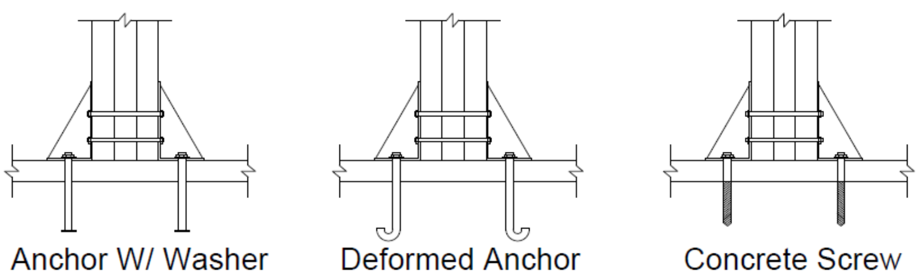

Figure 3-3: Types of Anchor Bolts

Chapter 4 - Codes and Standards

The governing building code for this report will be the 2012 IBC. The wind loads will be determined using the design procedures given in the American Society of Civil Engineers (ASCE) Standard 7-10 for MLFRS and C&C, which calculates wind at strength levels. Dead loads will also be calculated using ASCE 7-10 minimum dead load tables. The capacities for all wood members will be calculated using the 2012 NDS and the diaphragm and shear wall capacities will be calculated using the 2008 SDPWS because the 2012 edition has yet to be published.

2012 IBC

Chapter 23,*Wood*, of the 2012 IBC deals with the design and construction of timber structures. Within the chapter, multiple tables dealing with the span ratings of sheathing for gravity loading are given. Live loads and partitions loads can be found in Chapter 16, Table 1607.1 and Section 1607.5 respectively. These are utilized when analyzing the sheathing capacity when subjected to gravity loads.

ASCE 7-10

ASCE 7-10, Chapter 27, *Wind Loads on Buildings –MWFRS*, provides the procedure for calculating the MLFRS wind pressures for building of all heights; this procedure is used when determining the wind loads applied to the buildings examined within this report.

Several factors affect the MLFRS wind pressures on a building, including the height, usage, and location of the building. When calculating wind pressures, the velocity pressure must first be determined. The velocity pressure equation is given by ASCE 7-10 Equation 27.3-1:

$$q_z = 0.00256 K_z K_{zt} K_d V^2 \qquad \text{(EQ 4-1)}$$

Where: q_z = velocity pressure at height z

K_d = wind directionality factor, ASCE 7-10 Sec. 26.6

K_z = velocity pressure exposure coefficient, ASCE 7-10 Sec. 27.3.1

K_{zt} = topographic factor, ASCE 7-10 Sec. 26.8.2

V = basic wind speed, ASCE 7-10 Sec. 26.5

In the above equation, K_z accounts for both the height and the location of the building with regards to surrounding buildings. K_{zt} accounts for the building's locations with regard to topography (hills, escarpments, etc.). K_d accounts for the wind direction and is only used in conjunction with load combinations. V accounts for both the building's location and the building's usage (importance). This has changed from previous ASCE 7 Standards where the importance factor was separate from the wind speed. It is left to the reader to investigate each of these terms further.

To calculate the wind pressures on a building, both the internal and external pressure must be taken into account. This is done by using ASCE 7-10 Equation 27.4-1:

$$p = qGC_p - q_i(GC_{pi}) \qquad \text{(EQ 4-2)}$$

Where: p = wind pressure, psf.

$q = q_z$ velocity pressure at height z for windward walls, ASCE 7-10 Equation 27.3-1

$q = q_h$ velocity pressure at mean roof height, h, for leeward walls, ASCE 7-10 Equation 27.3-1

$q_i = q_h$ velocity pressure at mean roof height, h, (conservatively) ASCE 7-10 Equation 27.3-1

G = gust factor, ASCE 7-10 Sec. 26.9

C_p = external pressure coefficient, ASCE 7-10 Figures 27.4-1, 27.4-2, 27.4-3

(GC_{pi}) = internal pressure coefficient, ASCE 7-10 Table 26.11-1

For this equation, the internal pressure coefficient, (GC_{pi}), can be either a positive or negative value, therefore the wind pressures must be calculated twice. When determining which condition governs, the sign convention must be considered. The sign convention for the current equations is based on positive pressures acting towards the surface in question and negative pressures acting away from the surface. When calculating the overall wind loads for a building, the loads that will be transferred into the diaphragms and then into the shear walls, both the windward and leeward wind pressures must be added together; since the governing wind load is acting towards the building and the leeward is acting away, they add together to give a greater force.

Other components that must be considered for wind loading include parapet loading and components and cladding loads. Information on MLFRS parapet loading can be found in ASCE 7-10 Section 27.4.5. Note that the equation accounts for both sides of each parapet, therefore it is not necessary to calculate individual windward and leeward pressures for each parapet.

Components and cladding loads are loads that are applied to individual elements. These wind loads are greater than the loads calculated by the MLFRS procedure because certain regions of a building will experience greater wind loads than others. For this report the individual elements include all framing members. The procedure for calculating components and cladding loads is similar to the procedure used in calculating the MLFRS loads and can be found in ASCE 7-10 Section 30.4.

Tables C3-1 and C3-2 from ASCE 7-10 Commentary are used to calculate the dead loads for the examples within this report. ASCE 7-10 can also be used to calculate snow loads, however for the locations that were chosen snow is not a governing factor therefore it is not investigated.

2012 NDS & 2008 SDPWS

The 2012 NDS provides equations for calculating the capacities in wood members. For this report, members in shear, bending, tension, and compression will be investigated. Each of these equations involves multiplying base wood properties, based on the species and grade of timber being used, by correction factors. These correction factors are based on how and where the member is being used. The capacity equations for ASD, which can be found in the 2012 NDS, Table 4.3.1 are:

$$\text{Shear:} \quad F_v' = F_v C_D C_m C_t C_i \qquad \text{(EQ 4-3)}$$

$$\text{Bending:} \quad F_b' = F_b C_D C_m C_t C_L C_F C_{fu} C_i C_r \qquad \text{(EQ 4-4)}$$

$$\text{Tension:} \quad F_t' = F_t C_D C_m C_t C_F C_i \qquad \text{(EQ 4-5)}$$

$$\text{Compression:} \quad F_c' = F_c C_D C_m C_t C_F C_i C_p \qquad \text{(EQ 4-6)}$$

All base wood properties can be found in the 2012 NDS Supplement Tables 4A/B/C. For this report, Douglas Fir Larch will be used for all members. The correction factors can be found in their respective sections per the 2012 NDS Chapter 4. Equations are also provided within SDPWS Section C4 to calculate diaphragm and shear wall deflections. It is left to the reader to further investigate each of these equations and factors.

When looking at the capacities, each of these is given in terms of stress with units of pounds per square inch, therefore all loads must be converted into units of stress. For tension and compression members, this is simply a matter of dividing the force within the member by the area of the member. For bending, the stress is calculated by using principles of mechanics of materials. The equation for stress due to bending, f_b, can be expressed by Equation 4-7:

$$f_b = \frac{Mc}{I} = \frac{M}{S} \qquad \text{(EQ 4-7)}$$

Where: M = applied bending moment, lb-in.

c = distance to extreme fiber in tension, in.

I = moment of inertia of the section, in^4

S = section modulus of the section, in^3

The stress due to shear is calculated using the principle of shear flow from mechanics of materials. Stress due to shear, f_v, for rectangular sections only can be expressed by Equation 4-8:

$$f_v = \frac{3V}{2bd} \qquad \text{(EQ 4-8)}$$

Where: V = applied shear, lbs.

b = width of member, in.

d = depth of member, in.

For both diaphragms and shear walls, sheathing capacities are calculated using tables within the 2008 SDPWS, as has been previously discussed. These tables are based on the type, thickness, and layout of sheathing in addition to the size and spacing of nails. These tables, in conjunction with the 2012 NDS equations, will be used to design the MLFRS for several different cases in the following sections.

Chapter 5 - Building Design Comparisons

Report Parameters

The parameters of this parametric study are the following: (1) lateral loads are caused by wind, (2) height of the buildings vary from one-story (12 ft mean roof height) to three-story (36 ft mean roof height). Buildings of various heights are investigated in order to gain a better understanding of how the design of diaphragms and shear walls change with both increased lateral forces and increased height. The reason that this report is limited to a three-story building is because most timber buildings fall within the one- to three-story range in the United States. (3) Wind loads vary: a) 115 mph, b) 140 mph, and c) 160 mph. The exact locations are discussed in more detail under the appropriate section. The building is an office building with a normal occupancy in a commercial development on flat terrain.

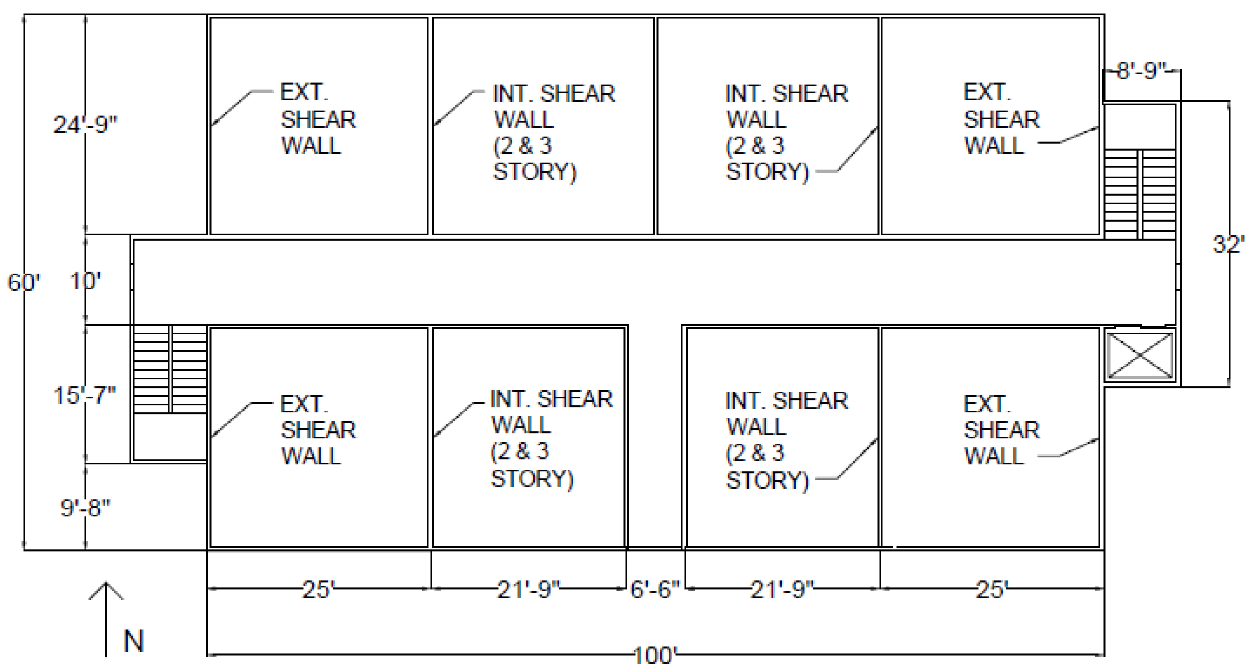

Figure 5-1: Typical Building Floor Plan

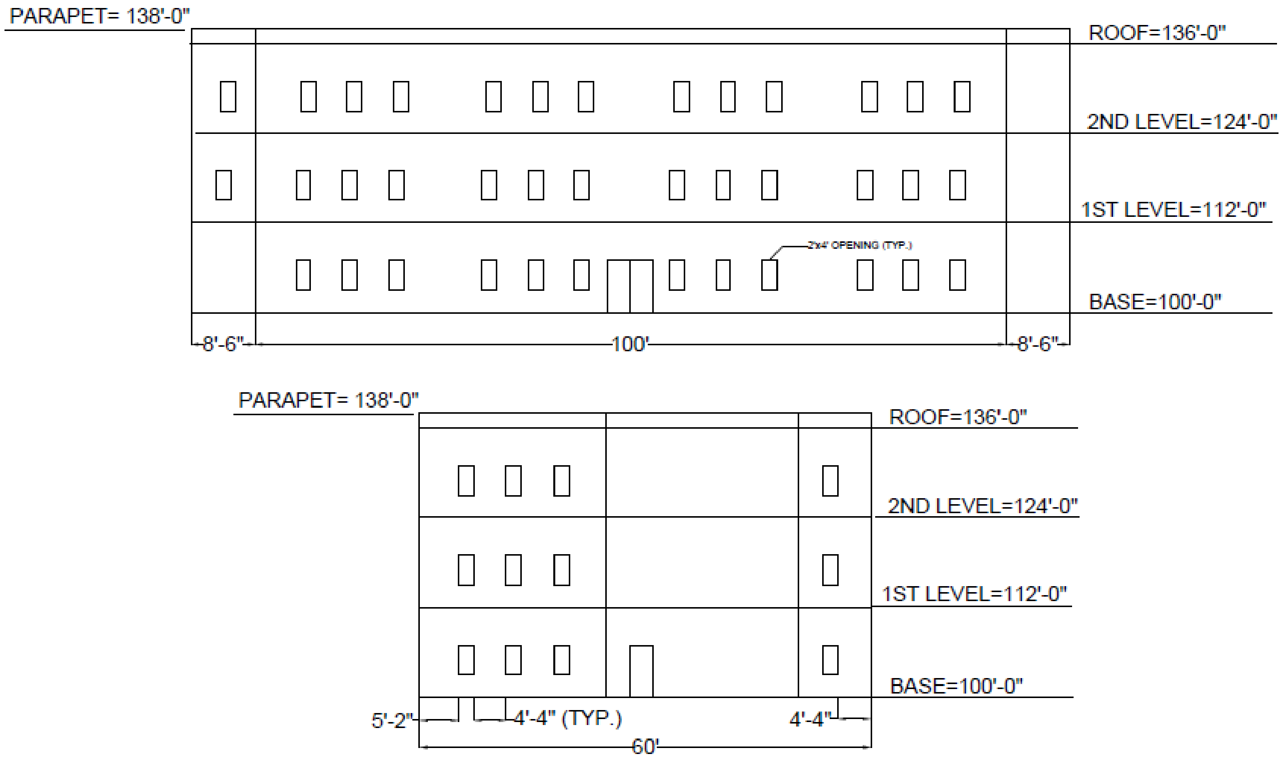

Figure 5-2: Building Elevations

The floor plan and elevations for the building are shown in **Figure 5-1** and **Figure 5-2**. This floor plan is the same for each floor, except the main entrance hallway is eliminated on upper floors and the single story buildings do not have stairways. The elevation shown is for the three-story configuration. For the other configurations, the floor-to-floor height (12 ft.) is the same and a flat roof, sloped at 0.25 inches per 1 ft., with a 2 ft. parapet is used for all building heights. Windows are all 2 ft. x 4 ft. The overall dimensions for the building are 60 ft. x 100 ft., with a stairway on either end for the multi-story configurations. The building is designed so that both exterior and interior walls can be used as blocked shear walls and all diaphragms are blocked for ease of comparison. The structural systems for the stair towers and elevator shaft are assumed to resist the lateral forces applied to them; therefore, they do not affect the rectangular structure's shear walls. For this floor plan, the north/south direction is considered the transverse direction and the east/west direction is considered the longitudinal direction. Dead loads for the roof and all floors are 20 psf. The roof live loads are 20 psf. Live loads are 50 psf. with an additional 15 psf. partition load; this live load has been reduced for this report to increase the overturning force that must be resisted by the overturning anchors. Ground snow loads are location dependent (ranges from 0 to 25 psf) as are the wind loads.

Locations

Manhattan, Kansas is the first location to be considered. The design wind speed according to ASCE 7-10 is 115 mph. Manhattan was chosen because its design wind speed is the most common wind speed for the 48 contiguous states. Houston, Texas is the second location, for which the design wind speed according to ASCE 7-10 is 140 mph. Houston was chosen because its design wind speed is the most common wind speed among coastal cities. Miami, Florida is the third location, for which the design wind speed according to ASCE 7-10 is 160 mph. Miami was chosen because it has higher wind speeds than any other major city in the contiguous 48 states. Three cases shall be considered: one-story, two-story and three-story. The diaphragms and shear walls shall be designed for each of the cases and then a comparison is made between the three cases for the three locations.

One-Story Comparison

The first case is a one-story building. This building has a mean roof height of 12 ft. with a 2 ft. parapet. The building is designed to resist gravity and lateral wind loading. The design includes sheathing, framing, nailing, and anchoring. Calculations include checks for both capacity and serviceability. An example of the calculations is presented in **Appendix A**.

Diaphragms

The roof diaphragm is the only diaphragm for this building. The transverse direction governs for this building, therefore it is the direction that is examined. A blocked diaphragm with a Case #1 sheathing layout is used to determine the required sheathing and nailing for each of the buildings.

The diaphragm for the building in the 115 mph zone consists of 3/8” plywood sheathing with 6d nails at 6” O.C. along the boundary of the diaphragm, 6d nails at 6” O.C. at the other panel edges, and 6d nails at 12” O.C. in the field. The diaphragm framing is 14” TJI joists spaced at 16” O.C. and the diaphragm chords are 2-2” x 6” DF-L continuous members.

The diaphragm for the building in the 140 mph zone consists of 3/8” plywood sheathing with 6d nails at 6” O.C. along the boundary of the diaphragm, 6d nails at 6” O.C. at the other panel edges, and 6d nails at 12” O.C. in the field. The diaphragm framing is 14” TJI joists spaced at 16” O.C. and the diaphragm chords are 2-2” x 6” DF-L continuous members.

The diaphragm for the building in the 160 mph zone consists of 3/8” plywood sheathing with 6d nails at 4” O.C. along the boundary of the diaphragm, 6d nails at 6” O.C. at the other panel edges, and 6d nails at 12” O.C. in the field. The diaphragm framing is 14” TJI joists spaced at 16” O.C. and the diaphragm chords are 2-2” x 6” DF-L continuous members. **Table 5-1** summarizes the three diaphragm designs.

Table 5-1: Single-Story Diaphragm Summary

Roof Diaphragm			
Wind Speed	**115 mph**	**140 mph**	**160 mph**
Sheathing Thickness	3/8"	3/8"	3/8"
Boundary Nailing	6d at 6" O.C.	6d at 6" O.C.	6d at 4" O.C.
Other Panel Edge Nailing	6d at 6" O.C.	6d at 6" O.C.	6d at 6" O.C.
Field Nailing	6d at 12" O.C.	6d at 12" O.C.	6d at 12" O.C.
Framing	14" TJI at 16" O.C.	14" TJI at 16" O.C.	14" TJI at 16" O.C.
Chords	2-2" x 6" DF-L	2-2" x 6" DF-L	2-2" x 6" DF-L

The diaphragms for the three buildings have very similar designs; the only difference is the smaller nail spacing at the diaphragm boundary for the building in the 160 mph zone. The designs are similar because the magnitude of the lateral load that is transferred into the roof diaphragm is relatively low. The diaphragm for the first building has twice the required capacity (<50% stressed); this is because gravity loads determined the sheathing thickness. The diaphragm for the second building was based on the lateral loading and had adequate capacity (80% stressed). The diaphragm for the third building was based on lateral loading and required smaller nail spacing than the other two, however the required capacity was only slightly above the capacity of the other two diaphragms (106% stressed if same design is used). A complete summary for each of the diaphragm designs can be found in **Appendix B**.

Shear Walls

The shear walls examined for this building are the east and west shear walls. These shear walls are in the transverse direction and are shorter in length than the longitudinal shear walls, therefore they have a larger distributed shear than the longitudinal shear walls. For the segmented shear wall approach, the thinnest full height segment is 4’-4” wide; this is the segment that will be examined in this report because the chord and anchor forces are the greatest. The total length of full height segments is 42’-6” and the total length of wall is 60’-0”.

The sheathing and nailing for shear walls for the building in the 115 mph zone did not differ when comparing the segmented and perforated approaches; both used 3/8” structural panel sheathing nailed at 6” O.C. with 6d nails.

The sheathing and nailing for shear walls for the building in the 140 mph zone did not differ when comparing the segmented and perforated approaches; both used 3/8” structural panel sheathing nailed at 4” O.C. with 6d nails. An alternative to this design is to use 3/8” structural panel sheathing nailed at 6” O.C. with 8d nails for both shear wall design approaches.

The sheathing and nailing for shear walls for the building in the 160 mph zone differed when comparing the segmented and perforated approaches; both used 3/8” structural panel, however the segmented shear wall requires 6d nails at 4” O.C. while the perforated shear wall requires 6d nails at 3” O.C. An alternative to this design is to use 3/8” structural panel sheathing nailed at 4” O.C. with 8d nails for both shear wall design approaches. This is allowed by footnote 2 of SDPWS Table 4.3A because the shear wall framing is 16” O.C. **Table 5-2** summarizes the shear wall designs for all cases.

Table 5-2: Single-Story Shear Wall Summary

Exterior Segmented Shear Walls			
Wind Speed	**115 mph**	**140 mph**	**160 mph**
Sheathing Thickness	3/8"	3/8"	3/8"
Boundary Nailing	6d at 6" O.C.	6d at 4" O.C.	6d at 4" O.C.
Exterior Perforated Shear Wall			
Wind Speed	**115 mph**	**140 mph**	**160 mph**
Sheathing Thickness	3/8"	3/8"	3/8"
Boundary Nailing	6d at 6" O.C.	6d at 4" O.C.	6d at 3" O.C.

For any shear wall design multiple acceptable sheathing/nailing options are available; the options given within this section are similar so that a comparison can be made. Comparing the three different buildings shows that the type and spacing of nailing is more important than the sheathing thickness. For each of the buildings, the sheathing was 3/8” thick, however as the wind speed was increased the nail size was increased or the nail spacing was decreased.

The shear wall framing, 6” Douglas Fir Larch (DF-L) studs at 16” O.C., did not change between the three buildings, because the component and cladding wind loads were similar in magnitude for each of the buildings. The end chords for the segmented shear walls were 2” x 6” studs for each building; the end chords for the perforated shear walls were 2-2” x 6”. The overturning anchors for the segmented shear wall approach were 1/2” diameter for each of the buildings; the overturning anchors for the perforated were 5/8” diameter. For both approaches the diameter of anchor required for the selected hold-downs was greater than the calculated required diameter of the anchor rod alone. While the type of hold-down remained the same for the segmented shear wall approach, the size of the hold-downs and the number of fasteners required per hold-down increased as the wind speed increased for the perforated shear walls. The shear anchors are ½” diameter for all of the buildings. The number of anchors per segmented section was 2 for all cases. For the perforated shear walls, the number of anchors increased from 10 to 19 anchors. A complete summary for each of the shear wall designs can be found in **Appendix B**.

Two-Story Comparison

The second case considered is a two-story building. This building has a mean roof height of 24 ft. with a 2 ft. parapet. The building is designed to resist lateral wind loading. The design includes sheathing, framing, nailing, and anchoring. Calculations include checks for both capacity and serviceability. An example of the calculations are presented in **Appendix A**.

Diaphragms

The first level and roof diaphragms are examined for this building. The transverse direction governs for this building, therefore it is the direction examined. Blocked diaphragms with Case #1 sheathing layouts are used to determine the required sheathing and nailing for each of the buildings.

The floor and roof diaphragms for the building in the 115 mph zone both consist of 3/8” plywood sheathing with 6d nails at 6” O.C. along the boundary of the diaphragm, 6d nails at 6” O.C. at the other panel edges, and 6d nails at 12” O.C. in the field. The floor diaphragm framing is 16” TJI joists spaced at 16” O.C. and the diaphragm chords are 2-2” x 6” DF-L continuous members. The roof diaphragm framing is 14” TJI joists spaced at 16” O.C. and the diaphragm chords are 2-2” x 6” DF-L continuous members.

The floor and roof diaphragms for the building in the 140 mph zone both consist of 3/8” plywood sheathing with 6d nails at 6” O.C. along the boundary of the diaphragm, 6d nails at 6” O.C. at the other panel edges, and 6d nails at 12” O.C. in the field. The floor diaphragm framing is 16” TJI joists spaced at 16” O.C. and the diaphragm chords are 2-2” x 6” DF-L continuous members. The roof diaphragm framing is 14” TJI joists spaced at 16” O.C. and the diaphragm chords are 2-2” x 6” DF-L continuous members.

The floor and roof diaphragms for the building in the 160 mph zone both consist of 3/8” plywood sheathing with 6d nails at 6” O.C. along the boundary of the diaphragm, 6d nails at 6” O.C. at the other panel edges, and 6d nails at 12” O.C. in the field. The floor diaphragm framing is 16” TJI joists spaced at 16” O.C. and the diaphragm chords are 2-2” x 6” DF-L continuous members. The roof diaphragm framing is 14” TJI joists spaced at 16” O.C. and the diaphragm chords are 2-2” x 6” DF-L continuous members. **Table 5-2** summarizes the three diaphragm designs.

Table 5-3: Two-Story Diaphragm Summary

Floor Diaphragm			
Wind Speed	**115 mph**	**140 mph**	**160 mph**
Sheathing Thickness	3/8"	3/8"	3/8"
Boundary Nailing	6d at 6" O.C.	6d at 6" O.C.	6d at 6" O.C.
Other Panel Edge Nailing	6d at 6" O.C.	6d at 6" O.C.	6d at 6" O.C.
Field Nailing	6d at 12" O.C.	6d at 12" O.C.	6d at 12" O.C.
Framing	16" TJI at 16" O.C.	16" TJI at 16" O.C.	16" TJI at 16" O.C.
Chords	2-2" x 6" DF-L	2-2" x 6" DF-L	2-2" x 6" DF-L
Roof Diaphragm			
Wind Speed	**115 mph**	**140 mph**	**160 mph**
Sheathing Thickness	3/8"	3/8"	3/8"
Boundary Nailing	6d at 6" O.C.	6d at 6" O.C.	6d at 6" O.C.
Other Panel Edge Nailing	6d at 6" O.C.	6d at 6" O.C.	6d at 6" O.C.
Field Nailing	6d at 12" O.C.	6d at 12" O.C.	6d at 12" O.C.
Framing	14" TJI at 16" O.C.	14" TJI at 16" O.C.	14" TJI at 16" O.C.
Chords	2-2" x 6" DF-L	2-2" x 6" DF-L	2-2" x 6" DF-L

The diaphragms for the three buildings have the same design. This is because the loads are low enough, due to the low wind speed for the first building and the use of interior shear walls in the other two, that minimum nailing can be used. As will be shown in the following section, interior shear walls are required for the two buildings in the higher wind regions in order to resist the wind loading. Similar loading for the 1st elevated level and roof diaphragm for each of the buildings results in the diaphragms having the same design. The floor diaphragm has a larger tributary width than the roof diaphragm, however the parapet has a greater design wind pressure than the wall; this results in similar loading for the two diaphragms. Gravity loading governed the thickness of the sheathing for all cases. A complete summary for each of the diaphragm designs can be found in **Appendix B**.

Shear Walls

The shear walls examined for this building are the east and west shear walls. These shear walls are in the transverse direction and are shorter in length than the longitudinal shear walls, therefore they have a larger distributed shear than the longitudinal shear walls. For the segmented shear wall approach, the thinnest full height segment is 4’-4” wide; this is the segment examined in this report because the chord and anchor forces are the greatest. The total length of full height segments is 42’-6” and the total length of wall is 60’-0”. Two of the designs require interior shear walls. The eastern and western most interior walls are used as shear walls because they are continuous for the height of the building. These walls have four 3’-0” doors each, evenly distributed. The thinnest full height segment for the interior shear walls is 6’-4” wide. The total length of full height segments is 38’-0” and the total length of wall is 60’-0”.

The sheathing and nailing for shear walls for the building in the 115 mph zone differed when comparing the segmented and perforated approaches. The segmented approach used 3/8” sheathing with edge nailing of 8d nails at 4” O.C. from the base to the 1st level and then 8d nails at 6” O.C. from the 1st level to the roof. The perforated approach used 3/8” sheathing with edge nailing of 8d nails at 3” O.C. from the base to the 1st level and then 8d nails at 6” O.C. from the 1st level to the roof. The end chords for the segmented and perforated shear wall approaches were 2-2”x6”.

For the 140 mph zone, interior shear walls were required to resist the loading. For the exterior shear walls, 3/8” sheathing with 6d nails at 6” O.C. for the full height of the shear wall was used for both approaches. For the interior shear walls, the segmented approach used 3/8” sheathing with edge nailing of 8d nails at 3” O.C. from the base to the 1st level and then 8d nails at 6” O.C. from the 1st level to the roof. The perforated approach used 15/32” sheathing with edge nailing of 10d nails at 3” O.C. from the base to the 1st level and then 10d nails at 6” O.C.

from the 1st level to the roof. The end chords for the interior segmented shear walls were 2-2” x 6” and 3-2”x6” for the perforated approach.

For the 160 mph zone, interior shear walls were required to resist the loading. For the exterior shear walls, the segmented approach used 3/8” sheathing with 6d nails at 6” O.C. for the full height of the shear wall was used for both approaches. The perforated approach used 3/8” sheathing with edge nailing of 6d nails at 4” O.C. from the base to the 1st level and then 6d nails at 6” O.C. from the 1st level to the roof. For the interior shear walls, the segmented approach used 15/32” sheathing with edge nailing of 10d nails at 3” O.C. from the base to the 1st level and then 10d nails at 6” O.C. from the 1st level to the roof. The perforated approach used 15/32” sheathing with edge nailing of 10d nails at 2” O.C. from the base to the 1st level and then 10d nails at 4” O.C. from the 1st level to the roof. The end chords for the interior segmented and perforated shear wall approaches were 3-2”x6”. **Table 5-4** summarizes the shear wall designs for all cases.

Table 5-4: Two-Story Shear Wall Summary

Exterior Segmented Shear Walls			
Wind Speed	**115 mph**	**140 mph**	**160 mph**
Sheathing Thickness	3/8"	3/8"	3/8"
Boundary Nailing (B to 1)	8d at 4" O.C.	6d at 6" O.C.	6d at 6" O.C.
Boundary Nailing (1 to R)	8d at 6" O.C.	6d at 6" O.C.	6d at 6" O.C.
Interior Segmented Shear Wal			
Wind Speed	**115 mph**	**140 mph**	**160 mph**
Sheathing Thickness	--	3/8"	15/32"
Boundary Nailing (B to 1)	--	8d at 3" O.C.	10d at 3" O.C.
Boundary Nailing (1 to R)	--	8d at 6" O.C.	10d at 6" O.C.
Exterior Perforated Shear Wall			
Wind Speed	**115 mph**	**140 mph**	**160 mph**
Sheathing Thickness	3/8"	3/8"	3/8"
Boundary Nailing (B to 1)	8d at 3" O.C.	6d at 6" O.C.	6d at 4" O.C.
Boundary Nailing (1 to R)	8d at 6" O.C.	6d at 6" O.C.	6d at 6" O.C.
Interior Perforated Shear Wall			
Wind Speed	**115 mph**	**140 mph**	**160 mph**
Sheathing Thickness	--	15/32"	15/32"
Boundary Nailing (B to 1)	--	10d at 3" O.C.	10d at 2" O.C.
Boundary Nailing (1 to R)	--	10d at 6" O.C.	10d at 4" O.C.

The first notable difference in designs is the requirement for interior shear walls for the two buildings in the higher wind regions. As a result, the exterior shear walls were lightly loaded and required minimum nailing (6d at 6" O.C.) for both buildings, with the exception of the perforated approach in the 160 mph building. The next notable difference is that the size of nail required increased and/or the spacing of nails decreased from the segmented to the perforated approach. This is more apparent with the two-story building than it was with the single story building.

The shear wall framing, 6" Douglas Fir Larch (DF-L) studs at 16" O.C., did not change between the three buildings, because the component and cladding wind loads were similar in magnitude for each of the buildings. The overturning anchors for the segmented and perforated shear wall approaches were 7/8" diameter for the main shear walls (exterior for the 115 mph and interior for the 140 mph and 160 mph). For both approaches the diameter of anchor required for the selected hold-downs was greater than the calculated required diameter of the anchor rod alone. The size of the hold-downs, number of hold-downs, and the number of fasteners required per hold-down increased as the wind speed increased for both approaches. The shear anchors are ½" diameter for all of the buildings. The number of anchors per segmented section for the main shear walls increased from 2 to 5. For perforated shear walls, the number of anchors for the main shear walls increased from 22 to 33 anchors. A complete summary for each of the shear wall designs can be found in **Appendix B**.

Three-Story Comparison

The third case considered is a three-story building. This building has a mean roof height of 36 ft. with a 2ft. parapet. The building is designed to resist lateral wind loading. The design includes sheathing, framing, nailing, and anchoring. Calculations include checks for both capacity and serviceability. An example of the calculations is presented in **Appendix A**.

Diaphragms

The first level, second level, and roof diaphragms are examined for this building. The transverse direction governs for this building, therefore it is the direction examined. Blocked diaphragms with Case #1 sheathing layouts are used to determine the required sheathing and nailing for each of the buildings.

The floor and roof diaphragms for the building in the 115 mph zone both consist of 3/8" plywood sheathing with 6d nails at 6" O.C. along the boundary of the diaphragm, 6d nails at 6" O.C. at the other panel edges, and 6d nails at 12" O.C. in the field. The floor diaphragm framing is 16" TJI joists spaced at 16" O.C. and the diaphragm chords are 2-2" x 6" DF-L continuous members. The roof diaphragm framing is 14" TJI joists spaced at 16" O.C. and the diaphragm chords are 2-2" x 6" DF-L continuous members.

The floor and roof diaphragms for the building in the 140 mph zone both consist of 3/8" plywood sheathing with 6d nails at 6" O.C. along the boundary of the diaphragm, 6d nails at 6" O.C. at the other panel edges, and 6d nails at 12" O.C. in the field. The floor diaphragm framing is 16" TJI joists spaced at 16" O.C. and the diaphragm chords are 2-2" x 6" DF-L continuous members. The roof diaphragm framing is 14" TJI joists spaced at 16" O.C. and the diaphragm chords are 2-2" x 6" DF-L continuous members.

The 1st level and roof diaphragms for the building in the 160 mph zone both consist of 3/8" plywood sheathing with 6d nails at 6" O.C. along the boundary of the diaphragm, 6d nails at 6" O.C. at the other panel edges, and 6d nails at 12" O.C. in the field. The 2nd level diaphragm consists of 3/8" plywood sheathing with 6d nails at 4" O.C. along the boundary of the diaphragm, 6d nails at 6" O.C. at the other panel edges, and 6d nails at 12" O.C. in the field. The floor diaphragm framing is 16" TJI joists spaced at 16" O.C. and the diaphragm chords are 2-2" x 6" DF-L continuous members. The roof diaphragm framing is 14" TJI joists spaced at 16" O.C. and the diaphragm chords are 2-2" x 6" DF-L continuous members. **Table 5-3** summarizes the three diaphragm designs.

The diaphragms for the three buildings have similar designs. This is because the loads are low and the tributary widths for each diaphragm are similar, resulting in similar loads. The only difference was that the 2nd level diaphragm boundary nailing in the 160 mph building required a closer spacing. Gravity loading governed the thickness of the sheathing for all cases. A complete summary for each of the diaphragm designs can be found in **Appendix B**.

Table 5-5: Three-Story Diaphragm Summary

First Floor Diaphragm			
Wind Speed	**115 mph**	**140 mph**	**160 mph**
Sheathing Thickness	3/8"	3/8"	3/8"
Boundary Nailing	6d at 6" O.C.	6d at 6" O.C.	6d at 6" O.C.
Other Panel Edge Nailing	6d at 6" O.C.	6d at 6" O.C.	6d at 6" O.C.
Field Nailing	6d at 12" O.C.	6d at 12" O.C.	6d at 12" O.C.
Framing	16" TJI at 16" O.C.	16" TJI at 16" O.C.	16" TJI at 16" O.C.
Chords	2-2" x 6" DF-L	2-2" x 6" DF-L	2-2" x 6" DF-L
Second Floor Diaphragm			
Wind Speed	**115 mph**	**140 mph**	**160 mph**
Sheathing Thickness	3/8"	3/8"	3/8"
Boundary Nailing	6d at 6" O.C.	6d at 6" O.C.	6d at 4" O.C.
Other Panel Edge Nailing	6d at 6" O.C.	6d at 6" O.C.	6d at 6" O.C.
Field Nailing	6d at 12" O.C.	6d at 12" O.C.	6d at 12" O.C.
Framing	16" TJI at 16" O.C.	16" TJI at 16" O.C.	16" TJI at 16" O.C.
Chords	2-2" x 6" DF-L	2-2" x 6" DF-L	2-2" x 6" DF-L
Roof Diaphragm			
Wind Speed	**115 mph**	**140 mph**	**160 mph**
Sheathing Thickness	3/8"	3/8"	3/8"
Boundary Nailing	6d at 6" O.C.	6d at 6" O.C.	6d at 6" O.C.
Other Panel Edge Nailing	6d at 6" O.C.	6d at 6" O.C.	6d at 6" O.C.
Field Nailing	6d at 12" O.C.	6d at 12" O.C.	6d at 12" O.C.
Framing	14" TJI at 16" O.C.	14" TJI at 16" O.C.	14" TJI at 16" O.C.
Chords	2-2" x 6" DF-L	2-2" x 6" DF-L	2-2" x 6" DF-L

Shear Walls

The shear walls examined for this building are the east and west shear walls. These shear walls are in the transverse direction and are shorter in length than the longitudinal shear walls, therefore they have a larger distributed shear than the longitudinal shear walls. For the segmented shear wall approach, the thinnest full height segment is 4’-4” wide; this is the segment examined in this report because the chord and anchor forces are the greatest. The total length of full height segments is 42’-6” and the total length of wall is 60’-0”. All of the designs require interior shear walls. The eastern and western interior walls are used because they are continuous for the height of the building. These walls have four 3’-0” doors each, evenly distributed. The thinnest full height segment for the interior shear walls is 6’-4” wide. The total length of full height segments is 38’-0” and the total length of wall is 60’-0”.

For the 115 mph zone, the exterior shear walls used 3/8” sheathing with 6d nails at 6” O.C. for the full height of the shear wall was used for both approaches. For the interior shear walls, the segmented approach used 3/8” sheathing with edge nailing of 8d nails at 3” O.C. from the base to the 1st level and 8d nails at 3” O.C. from the 1st level to the 2nd level, and 8d nails at 6” O.C. from the 2nd level to the roof. The perforated approach used 19/32” sheathing with edge nailing of 10d nails at 3” O.C. from the base to the 1st level and 10d nails at 4” O.C. from the 1st level to the 2nd level, and 10d nails at 6” O.C. from the 2nd level to the roof. The end chords for the interior segmented shear walls were 3-2” x 6” and 3-2”x6” for the perforated approach.

For the 140 mph zone, the exterior shear walls used 3/8” sheathing with 6d nails at 6” O.C. for the full height of the shear wall was used for both approaches, with the exception of 6d nails at 4” O.C. for the perforated approach from the base to the 1st level. For the interior shear walls, the segmented approach used 3/8” sheathing on both sides with edge nailing of 8d nails at 4” O.C. from the base to the 1st level, 3/8” sheathing on both sides with edge nailing of 8d nails at 6” O.C. from the 1st level to the 2nd level, and 3/8” sheathing on one side with edge nailing of 8d nails at 6” O.C. from the 2nd level to the roof level. The perforated shear wall approach is not allowed to be used in this case because the required nominal shear capacity is greater than 2435 plf, per SDPWS 2008, Section 4.3.5.3. The end chords for the interior segmented shear walls were 4-2” x 6”.

For the 160 mph zone, the exterior shear walls use 3/8” sheathing with 6d nails at 4” O.C. from the base level to the 1st level and 6d nails at 6” O.C. for the remainder of the shear wall for both approaches. For the interior shear walls, the segmented approach used 3/8” sheathing on both sides was used for the entire height with edge nailing of 8d nails at 3” O.C. from the base to the 1st level, 8d nails at 6” O.C. from the 1st level to the 2nd level, and 8d nails at 6” O.C. from the 2nd level to the roof level. The perforated shear wall approach is not allowed to be used in this case because the required nominal shear capacity is greater than 2435 plf, per SDPWS 2008, Section 4.3.5.3. The end chords for the interior segmented shear walls were 5-2” x 6”. **Table 5-6** summarizes the shear wall designs for all cases.

Table 5-6: Three-Story Shear Wall Summary

Exterior Segmented Shear Walls			
Wind Speed	**115 mph**	**140 mph**	**160 mph**
Sheathing Thickness	3/8"	3/8"	3/8"
Boundary Nailing (B to 1)	6d at 6" O.C.	6d at 6" O.C.	6d at 4" O.C.
Boundary Nailing (1 to 2)	6d at 6" O.C.	6d at 6" O.C.	6d at 6" O.C.
Boundary Nailing (2 to R)	6d at 6" O.C.	6d at 6" O.C.	6d at 6" O.C.
Interior Segmented Shear Wal			
Wind Speed	**115 mph**	**140 mph**	**160 mph**
Sheathing Thickness	3/8"	3/8" Both Sides (B to 2)	3/8" Both Sides (B to R)
Boundary Nailing (B to 1)	8d at 3" O.C.	8d at 4" O.C.	8d at 3" O.C.
Boundary Nailing (1 to 2)	8d at 3" O.C.	8d at 6" O.C.	8d at 6" O.C.
Boundary Nailing (2 to R)	8d at 6" O.C.	8d at 6" O.C.	8d at 6" O.C.
Exterior Perforated Shear Wall			
Wind Speed	**115 mph**	**140 mph**	**160 mph**
Sheathing Thickness	3/8"	3/8"	3/8"
Boundary Nailing (B to 1)	6d at 6" O.C.	6d at 4" O.C.	6d at 4" O.C.
Boundary Nailing (1 to 2)	6d at 6" O.C.	6d at 6" O.C.	6d at 6" O.C.
Boundary Nailing (2 to R)	6d at 6" O.C.	6d at 6" O.C.	6d at 6" O.C.
Interior Perforated Shear Wall			
Wind Speed	**115 mph**	**140 mph**	**160 mph**
Sheathing Thickness	19/32"	--	--
Boundary Nailing (B to 1)	10d at 3" O.C.	--	--
Boundary Nailing (1 to 2)	10d at 4" O.C.	--	--
Boundary Nailing (2 to R)	10d at 6" O.C.	--	--

The main difference in the designs is the requirement for sheathing on both sides of the interior shear walls for the two buildings in the higher wind regions. As a result, the nailing for all three buildings is similar. Sheathing both sides, however, requires more materials and results in higher construction costs. The perforated shear wall approach is not allowed to be used for the interior shear walls for the two buildings in the higher wind regions because the required shear capacity of the shear walls exceeds that allowed by the SDPWS.

The shear wall framing, 6” Douglas Fir Larch (DF-L) studs at 16” O.C. for the exterior walls and 12” O.C. for the interior walls, did not change between the three buildings. The difference in spacing is due to the interior walls having a larger tributary width. The overturning anchors for the segmented shear wall approach increased from 7/8” to 1 ¼” diameter for the interior shear walls. The diameter of anchor required for the selected hold-downs was greater than the calculated required diameter of the anchor rod alone. The type of the hold-downs and the number/type of fasteners required per hold-down increased as the wind speed increased. The shear anchors are ½" diameter for all of the buildings. The number of anchors per segmented section for the interior shear walls increased from 4 to 8. A complete summary for each of the shear wall designs can be found in **Appendix B**.

Chapter 6 - Conclusions

This report has examined how the design of both timber shear walls and diaphragms changes depending on the height of the building, location of the building, and the design approach used. This report has shown that as the height of the building increases in the same location, the magnitude of the force in the shear walls increases while the magnitude of the force in each diaphragm remains relatively constant. The increase in force in the shear walls increases the required sheathing thickness/nail size and/or decreases the required nail spacing.

This report has shown that one story buildings in high wind regions require only exterior shear walls with minimum nail size, 6d, at reasonable spacing. Diaphragm designs remain the same for all of the regions, with 6d nails at closer spacing for the building in the 160 mph region. This report has also shown that buildings in high wind regions of more than one story typically require both exterior and interior shear walls to resist the induced load. The addition of interior shear walls results in reduced spans for the diaphragms. Reduced diaphragm spans result in minimum nailing, 6d nails at 6" O.C., for most of the diaphragms. The only exception to this is the 2nd level for the building in high wind regions because the pressures and the tributary area are the greatest.

This report has shown how the design of shear walls changes depending on whether the segmented shear wall approach or the perforated shear wall approach is used. The segmented shear wall approach requires each panel to be designed and anchored individually. The perforated shear wall approach requires that the wall be designed and anchored as a whole. For single story buildings, the sheathing and nailing are nearly identical for both approaches. For two- story buildings using the perforated shear wall approach, the size of nail is larger and/or the spacing of the nails is smaller when compared to the segmented shear wall approach for all regions. For three- story buildings using the perforated shear wall approach, the size of nail is larger and/or the spacing of the nails is smaller when compared to the segmented shear wall approach for the 115 mph region. For the higher wind regions, perforated shear walls are not allowed to be used because the required nominal capacity is greater than the maximum permitted nominal load of 2435 plf. Segmented shear walls in the higher wind regions require sheathing on both sides to resist the induced lateral load.

The conclusions that can be drawn from these results are that the perforated shear wall approach requires the following adjustments when compared to the segmented shear wall approach:

- Closer nail spacing, therefore more nails overall.
- Larger nails in some cases.
- Thicker plywood sheathing in some cases.
- Larger, but fewer, overturning anchors.
- Calculations are more tedious and may require more time to complete.

For timber buildings in single story buildings, the method that is used to design the shear walls does not truly matter because the end results are nearly identical. However, the greater the wind pressure and/or the taller the building, the greater the differences in the final design when comparing the two approaches. Ultimately it is up to the engineer to use either of the two approaches, however this report has shown that the segmented shear wall approach is the more economical approach for multi-story buildings and buildings in high wind regions.

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Appendix A - Example Calculations

This section contains a single set of calculations used to create this report. The calculations are for the two-story building in the 115 mph zone. These calculations cover the calculation of the wind loads, shear wall design, floor diaphragm design, and roof diaphragm design. Similar calculations were completed for the other eight setups and are available upon request from the author.

Table A- 1: Wind Loads

[redacted]

[redacted]

[redacted]

[redacted]

[redacted]

[redacted]

Table A- 2: Segmented Shear Wall Approach

[redacted]

[redacted]

[redacted]

[redacted]

[redacted]

Table A- 3: Perforated Shear Wall Approach

[redacted]

[redacted]

[redacted]

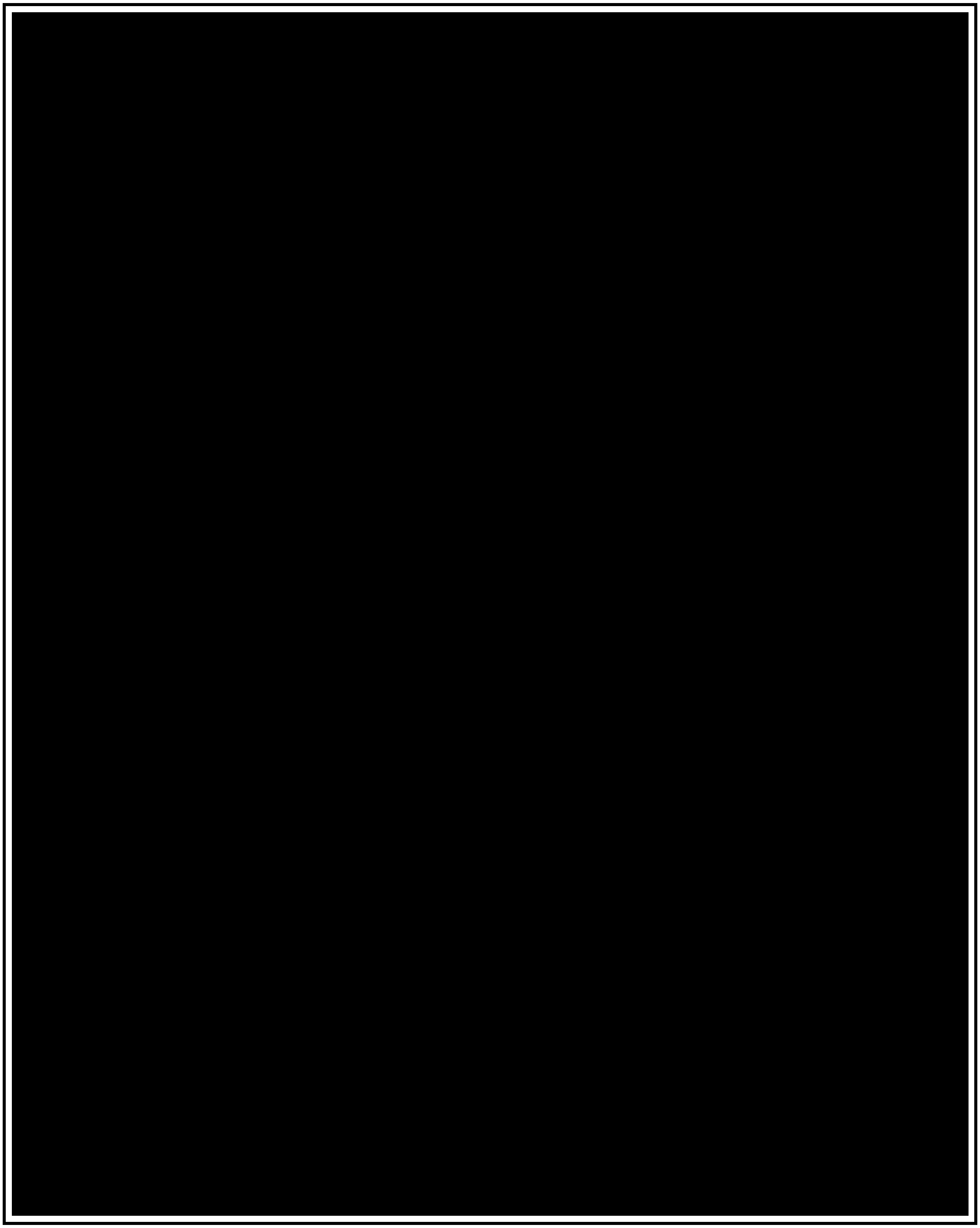
Combined Bending and Compression				NDS 2012 Section 3.9.2
fc/F'c =	0.284324			
Is fc <FcE1?	YES			
l_e/d2	50			
RB	5.18545			
FbE	27669.42			
fc/F'cE1 =	0.263166			
fb/FbE	0.022916			
Fce2	203.856			
Fb'1	1840			
Check #1 =	0.548526	<1		
Check #2 =	0.960303	<1		
Studs adequate in shear?	YES			
Studs adequate in bending and compression?	YES			
Are the studs adequate?	YES			
Shearwall Anchorage				
Shear Anchors				
Total Factored Shear	22585.26	lb		
Dia. of Anchor	0.5			
$Z_{Parallel}$	650	lb/anchor		NDS 2012 Table 11E
C_d	1.6			NDS 2012 Table 2.3.2
C_m	1			NDS 2012 Table 10.3.3
C_t	1			NDS 2012 Table 10.3.4
C_g	1			NDS 2012 Section 10.3.6
C_Δ	1			NDS 2012 Section 11.5.1
Z'	1040	lb/anchor		NDS 2012 Table 10.3.1
Required Number of Anchors	21.71659	anchors		
Number of Anchors to Use	22	anchors		
Max Spacing	2.857143	ft		
	34.28571	in		
Overturning Anchors				
Fy	45000	psi		*Note: Anchor capacities based on
Overall Panel Anchor Force	9438.847	lb		P=FyAg/1.67 Per AISC
Required Area Per Anchor	0.350286	in^2		
Required Diameter Per Anchor	0.667831	in		
Tension Ties/Holddowns				
Type	Simpson Strongtie HDC10/22-S			Product Catalog
Nailing	24-SDS 1⁄4"x2 1⁄2"			Product Catalog
Capacity	9665	lb		Product Catalog
Is the holddown adequate?	YES			
Tension Anchors				
t=vmax	531.4178	plf		
F_y	45000	psi		
Diameter of Anchor	1	in		
Area of Anchor	0.785398	in^2		
Anchor Capacity	21163.42	lb		*Note: Anchor capacities based on
Max Spacing	39.82445	ft		P=FyAg/1.67 Per AISC
	477.8935	in		

[redacted]

[redacted]

Table A- 4: First Level Diaphragm

[redacted]



[redacted]

Table A- 5: Roof Diaphragm

[redacted]

[redacted]

[redacted]

Appendix B - Calculation Summaries

Table B- 1: Single Story, 115 MPH Design Summary

Shear Walls		
	Segmented	**Perforated**
	Sheathing	
Sheathing Type	Wood Structural Panels - Sheathing	Wood Structural Panels - Sheathing
Sheathing Thickness	3/8" or greater	3/8" or greater
Minimum Nail Penetration	1 1/4"	1 1/4"
Nail Size	6d	6d
Edge Nailing	6d at 6" O.C.	6d at 6" O.C.
Field Nailing	6d at 12" O.C.	6d at 12" O.C.
	Studs	
Type of Wood	Douglas Fir Larch No. 1	Southern Pine No. 1
Depth of Stud (in)	5.5	5.5
Width of Stud (in)	1.5	1.5
Spacing (in O.C.)	16	16
	Chords[1]	
Type of Wood	Douglas Fir Larch No. 1	Douglas Fir Larch No. 1
Depth of Chord (in)	5.5	5.5
Width of Chord (in)	1.5	3
	In-Plane Anchorage[2]	
Min. Dia. of Overturning Anchors (in.)	0.163	0.362
Dia, of Shear Anchor (in.)	0.5	0.5
Number of Shear Anchors	1	10
Max Spacing of Shear Anchors (in.)	#DIV/0!	80.00
Type of Holddown	Simpson Strongtie DTT2Z w/ 1/2" Bolt	Simpson Strongtie HDU2-SDS2.5 w/ 5/8" Bolt
Holddown Screws	8-SDS 1⁄4"x1 1⁄2"	6-SDS 1⁄4"x2 1⁄2"
	Out-of--Plane Anchorage[3]	
Dia, of Shear Anchor (in.)	0.5	0.5
Number of Shear Anchors	12	12
Max Spacing of Shear Anchors (in.)	60.00	60.00

1: For segmented shear walls, chords are for each segment and for perforated chords are for the whole shearwall.

2: For segmented shear walls, in-plane anchorage is for each individual segment and for perforated chords are for the whole shearwall.

3: For segmented and perforated shear walls, out-of-plane anchorage is for the entire wall.

Diaphragms	
	Roof
Sheathing	
Sheathing Type	Plywood Sheathing
Sheathing Thickness	3/8"
Span Rating	24/0
Sheathing Case	1
Minimum Nail Penetration	1 1/4"
Nail Size	6d
Edge Nailing	6d at 6"
Boundary Nailing	6d at 6"
Field Nailing	6d at 12"
Joists	
Type of TJI Joist	TrusJoist 14" TJI 360
Spacing (in.)	16
Blocking[1]	
To be determined	
Chords	
Type of Wood	Douglas Fir Larch No. 1
Number of Chord Members	2
Depth Per Member (in.)	5.5
Width Per Member (in.)	1.5

1: It is left to the engineer of record to determine the blocking requirements.

Table B- 2: Single Story, 140 MPH Design Summary

Shear Walls		
	Segmented	**Perforated**
Sheathing		
Sheathing Type	Wood Structural Panels - Sheathing	Wood Structural Panels - Sheathing
Sheathing Thickness	3/8" or greater	3/8" or greater
Minimum Nail Penetration	1 1/4"	1 1/4"
Nail Size	6d	6d
Edge Nailing	6d at 4" O.C.	6d at 4" O.C.
Field Nailing	6d at 12" O.C.	6d at 12" O.C.
Studs		
Type of Wood	Douglas Fir Larch No. 1	Southern Pine No. 1
Depth of Stud (in)	5.5	5.5
Width of Stud (in)	1.5	1.5
Spacing (in O.C.)	16	16
Chords[1]		
Type of Wood	Douglas Fir Larch No. 1	Douglas Fir Larch No. 1
Depth of Chord (in)	5.5	5.5
Width of Chord (in)	1.5	3
In-Plane Anchorage[2]		
Min. Dia. of Overturning Anchors (in.)	0.198	0.440
Dia, of Shear Anchor (in.)	0.5	0.5
Number of Shear Anchors	2	14
Max Spacing of Shear Anchors (in.)	51.6	55.38
Type of Holddown	Simpson Strongtie DTT2Z w/ 1/2" Bolt	Simpson Strongtie HDU4-SDS2.5 w/ 5/8" Bolt
Holddown Screws	8-SDS 1⁄4"x1 1⁄2"	10-SDS 1⁄4"x2 1⁄2"
Out-of--Plane Anchorage[3]		
Dia, of Shear Anchor (in.)	0.5	0.5
Number of Shear Anchors	18	18
Max Spacing of Shear Anchors (in.)	40.00	40.00

1: For segmented shear walls, chords are for each segment and for perforated chords are for the whole shearwall.

2: For segmented shear walls, in-plane anchorage is for each individual segment and for perforated chords are for the whole shearwall.

3: For segmented and perforated shear walls, out-of-plane anchorage is for the entire wall.

Diaphragms	
	Roof
Sheathing	
Sheathing Type	Plywood Sheathing
Sheathing Thickness	3/8"
Span Rating	24/0
Sheathing Case	1
Minimum Nail Penetration	1 1/4"
Nail Size	6d
Edge Nailing	6d at 6"
Boundary Nailing	6d at 6"
Field Nailing	6d at 12"
Joists	
Type of TJI Joist	TrusJoist 14" TJI 360
Spacing (in.)	16
Blocking[1]	
To be determined	
Chords	
Type of Wood	Douglas Fir Larch No. 1
Number of Chord Members	2
Depth Per Member (in.)	5.5
Width Per Member (in.)	1.5

1: It is left to the engineer of record to determine the blocking requirements.

Table B- 3: Single Story, 160 MPH Design Summary

Shear Walls		
	Segmented	Perforated
Sheathing		
Sheathing Type	Wood Structural Panels - Sheathing	Wood Structural Panels - Sheathing
Sheathing Thickness	3/8" or greater	3/8" or greater
Minimum Nail Penetration	1 1/4"	1 1/4"
Nail Size	6d	6d
Edge Nailing	6d at 4" O.C.	6d at 3" O.C.
Field Nailing	6d at 12" O.C.	6d at 12" O.C.
Studs		
Type of Wood	Douglas Fir Larch No. 1	Southern Pine No. 1
Depth of Stud (in)	5.5	5.5
Width of Stud (in)	1.5	1.5
Spacing (in O.C.)	16	16
Chords[1]		
Type of Wood	Douglas Fir Larch No. 1	Douglas Fir Larch No. 1
Depth of Chord (in)	5.5	5.5
Width of Chord (in)	1.5	3
In-Plane Anchorage[2]		
Min. Dia. of Overturning Anchors (in.)	0.226	0.503
Dia, of Shear Anchor (in.)	0.5	0.5
Number of Shear Anchors	2	19
Max Spacing of Shear Anchors (in.)	51.6	40.00
Type of Holddown	Simpson Strongtie DTT2Z w/ 1/2" Bolt	Simpson Strongtie HDU5-SDS2.5 w/ 5/8" Bolt
Holddown Screws	8-SDS 1⁄4"x1 1⁄2"	14-SDS 1⁄4"x2 1⁄2"
Out-of--Plane Anchorage[3]		
Dia, of Shear Anchor (in.)	0.5	0.5
Number of Shear Anchors	23	23
Max Spacing of Shear Anchors (in.)	31.30	31.30

1: For segmented shear walls, chords are for each segment and for perforated chords are for the whole shearwall.

2: For segmented shear walls, in-plane anchorage is for each individual segment and for perforated chords are for the whole shearwall.

3: For segmented and perforated shear walls, out-of-plane anchorage is for the entire wall.

Diaphragms	
	Roof
Sheathing	
Sheathing Type	Plywood Sheathing
Sheathing Thickness	3/8"
Span Rating	24/0
Sheathing Case	1
Minimum Nail Penetration	1 1/4"
Nail Size	6d
Edge Nailing	6d at 6"
Boundary Nailing	6d at 4"
Field Nailing	6d at 12"
Joists	
Type of TJI Joist	TrusJoist 14" TJI 360
Spacing (in.)	16
Blocking[1]	
To be determined	
Chords	
Type of Wood	Douglas Fir Larch No. 1
Number of Chord Members	2
Depth Per Member (in.)	5.5
Width Per Member (in.)	1.5

1: It is left to the engineer of record to determine the blocking requirements.

Table B- 4: Two-Story, 115 MPH Design Summary

Shear Walls		
	Segmented	Perforated
Base to 1st Level Sheathing		
Sheathing Type	Wood Structural Panels	Wood Structural Panels
Sheathing Thickness	3/8" or greater	3/8" or greater
Minimum Nail Penetration	1 3/8"	1 3/8"
Nail Size	8d	8d
Edge Nailing	8d at 4" O.C.	8d at 3" O.C.
Field Nailing	8d at 12" O.C.	8d at 12" O.C.
1st Level to Roof Sheathing		
Sheathing Type	Wood Structural Panels	Wood Structural Panels
Sheathing Thickness	3/8" or greater	3/8" or greater
Minimum Nail Penetration	1 3/8"	1 3/8"
Nail Size	8d	8d
Edge Nailing	8d at 6" O.C.	8d at 6" O.C.
Field Nailing	8d at 12" O.C.	8d at 12" O.C.
Studs		
Type of Wood	Douglas Fir Larch No. 1	Douglas Fir Larch No. 1
Depth of Stud (in)	5.5	5.5
Width of Stud (in)	1.5	1.5
Spacing (in O.C.)	16	16
Chords[1]		
Type of Wood	Douglas Fir Larch No. 1	Douglas Fir Larch No. 1
Depth of Chord (in)	5.5	5.5
Width of Chord (in)	3	3
In-Plane Anchorage[2]		
Min. Dia. of Overturning Anchors (in.)	0.623	0.668
Dia, of Shear Anchor (in.)	0.5	0.5
Number of Shear Anchors	2	22
Max Spacing of Shear Anchors (in.)	51.6	34.29
Type of Holddown	Simpson Strongtie HDC10/22-SDS2.5 w/ 7/8" Bolt	Simpson Strongtie HDC10/22-SDS2.5 w/ 7/8" Bolt
Holddown Screws	24-SDS 1⁄4"x2 1⁄2"	24-SDS 1⁄4"x2 1⁄2"
Out-of--Plane Anchorage[3]		
Dia, of Shear Anchor (in.)	0.5	0.5
Number of Shear Anchors	10	10
Max Spacing of Shear Anchors (in.)	72.00	72.00

1: For segmented shear walls, chords are for each segment and for perforated chords are for the whole shearwall.

2: For segmented shear walls, in-plane anchorage is for each individual segment and for perforated chords are for the whole shearwall.

3: For segmented and perforated shear walls, out-of-plane anchorage is for the entire wall.

Diaphragms		
	1st Level	Roof
Sheathing		
Sheathing Type	Plywood - 3 Ply	Plywood - 3 Ply
Sheathing Thickness	3/8"	3/8"
Span Rating	24/0	24/0
Sheathing Case	1	1
Minimum Nail Penetration	1 1/4"	1 1/4"
Nail Size	6d	6d
Edge Nailing	6d at 6"	6d at 6"
Boundary Nailing	6d at 6"	6d at 6"
Field Nailing	6d at 12"	6d at 12"
Joists		
Type of TJI Joist	TrusJoist 16" TJI 560	TrusJoist 14" TJI 360
Spacing (in.)	16	16
Blocking[1]		
To be determined		
Chords		
Type of Wood	Douglas Fir Larch No. 1	Douglas Fir Larch No. 1
Number of Chord Members	2	2
Depth Per Member (in.)	5.5	5.5
Width Per Member (in.)	1.5	1.5

1: It is left to the engineer of record to determine the blocking requirements.

Table B- 5: Two-Story, 140 MPH Design Summary

Exterior Shear Walls		
	Segmented	**Perforated**
Base to 1st Level Sheathing		
Sheathing Type	Wood Structural Panels	Wood Structural Panels
Sheathing Thickness	3/8" or greater	3/8" or greater
Minimum Nail Penetration	1 1/4"	1 1/4"
Nail Size	6d	6d
Edge Nailing	6d at 6" O.C.	6d at 6" O.C.
Field Nailing	6d at 12" O.C.	6d at 12" O.C.
1st Level to Roof Sheathing		
Sheathing Type	Wood Structural Panels	Wood Structural Panels
Sheathing Thickness	3/8" or greater	3/8" or greater
Minimum Nail Penetration	1 1/4"	1 1/4"
Nail Size	6d	6d
Edge Nailing	6d at 6" O.C.	6d at 6" O.C.
Field Nailing	6d at 12" O.C.	6d at 12" O.C.
Studs		
Type of Wood	Douglas Fir Larch No. 1	Douglas Fir Larch No. 1
Depth of Stud (in)	5.5	5.5
Width of Stud (in)	1.5	1.5
Spacing (in O.C.)	16	16
Chords[1]		
Type of Wood	Douglas Fir Larch No. 1	Douglas Fir Larch No. 1
Depth of Chord (in)	5.5	5.5
Width of Chord (in)	1.5	1.5
In-Plane Anchorage[2]		
Min. Dia. of Overturning Anchors (in.)	0.379	0.407
Dia, of Shear Anchor (in.)	0.5	0.5
Number of Shear Anchors	1	9
Max Spacing of Shear Anchors (in.)	#DIV/0!	90.00
Type of Holddown	2 Simpson Strongtie DTT2Z w/ 1/2" Bolt	2 Simpson Strongtie DTT2Z w/ 1/2" Bolt
Holddown Screws	8-SDS 1⁄4"x1 1⁄2"	8-SDS 1⁄4"x1 1⁄2"
Out-of--Plane Anchorage[3]		
Dia, of Shear Anchor (in.)	0.5	0.5
Number of Shear Anchors	15	15
Max Spacing of Shear Anchors (in.)	48.00	48.00

1: For segmented shear walls, chords are for each segment and for perforated chords are for the whole shearwall.

2: For segmented shear walls, in-plane anchorage is for each individual segment and for perforated chords are for the whole shearwall.

3: For segmented and perforated shear walls, out-of-plane anchorage is for the entire wall.

Interior Shear Walls		
	Segmented	**Perforated**
Base to 1st Level Sheathing		
Sheathing Type	Wood Structural Panels	Wood Structural Panels
Sheathing Thickness	3/8" or greater	15/32" or greater
Minimum Nail Penetration	1 3/8"	1 1/2"
Nail Size	8d	10d
Edge Nailing	8d at 3" O.C.	10d at 3" O.C.
Field Nailing	8d at 12" O.C.	10d at 12" O.C.
1st Level to Roof Sheathing		
Sheathing Type	Wood Structural Panels	Wood Structural Panels
Sheathing Thickness	3/8" or greater	15/32" or greater
Minimum Nail Penetration	1 3/8"	1 1/2"
Nail Size	8d	10d
Edge Nailing	8d at 6" O.C.	10d at 6" O.C.
Field Nailing	8d at 12" O.C.	10d at 12" O.C.
Studs		
Type of Wood	Douglas Fir Larch No. 1	Douglas Fir Larch No. 1
Depth of Stud (in)	5.5	5.5
Width of Stud (in)	1.5	1.5
Spacing (in O.C.)	16	16
Chords[1]		
Type of Wood	Douglas Fir Larch No. 1	Douglas Fir Larch No. 1
Depth of Chord (in)	5.5	5.5
Width of Chord (in)	3	4.5
In-Plane Anchorage[2]		
Min. Dia. of Overturning Anchors (in.)	0.695	0.758
Dia, of Shear Anchor (in.)	0.5	0.5
Number of Shear Anchors	4	26
Max Spacing of Shear Anchors (in.)	25.32	28.80
Type of Holddown	2 Simpson Strongtie HDU8-SDS2.5 w/ 7/8" Bolt	2 Simpson Strongtie HDU8-SDS2.5 w/ 7/8" Bolt
Holddown Screws	20-SDS 1⁄4"x2 1⁄2"	20-SDS 1⁄4"x2 1⁄2"

1: For segmented shear walls, chords are for each segment and for perforated chords are for the whole shearwall.

2: For segmented shear walls, in-plane anchorage is for each individual segment and for perforated chords are for the whole shearwall.

3: For segmented and perforated shear walls, out-of-plane anchorage is for the entire wall.

Diaphragms		
	1st Level	**Roof**
Sheathing		
Sheathing Type	Plywood - 3 Ply	Plywood - 3 Ply
Sheathing Thickness	3/8"	3/8"
Span Rating	24/0	24/0
Sheathing Case	1	1
Minimum Nail Penetration	1 1/4"	1 1/4"
Nail Size	6d	6d
Edge Nailing	6d at 6"	6d at 6"
Boundary Nailing	6d at 6"	6d at 6"
Field Nailing	6d at 12"	6d at 12"
Joists		
Type of TJI Joist	TrusJoist 16" TJI 560	TrusJoist 14" TJI 360
Spacing (in.)	16	16
Blocking[1]		
To be determined		
Chords		
Type of Wood	Douglas Fir Larch No. 1	Douglas Fir Larch No. 1
Number of Chord Members	2	2
Depth Per Member (in.)	5.5	5.5
Width Per Member (in.)	1.5	1.5

1: It is left to the engineer of record to determine the blocking requirements.

Table B- 6: Two-Story, 160 MPH Design Summary

Exterior Shear Walls		
	Segmented	Perforated
Base to 1st Level Sheathing		
Sheathing Type	Wood Structural Panels	Wood Structural Panels
Sheathing Thickness	3/8" or greater	3/8" or greater
Minimum Nail Penetration	1 1/4"	1 1/4"
Nail Size	6d	6d
Edge Nailing	6d at 6" O.C.	6d at 4" O.C.
Field Nailing	6d at 12" O.C.	6d at 12" O.C.
1st Level to Roof Sheathing		
Sheathing Type	Wood Structural Panels	Wood Structural Panels
Sheathing Thickness	3/8" or greater	3/8" or greater
Minimum Nail Penetration	1 1/4"	1 1/4"
Nail Size	6d	6d
Edge Nailing	6d at 6" O.C.	6d at 6" O.C.
Field Nailing	6d at 12" O.C.	6d at 12" O.C.
Studs		
Type of Wood	Douglas Fir Larch No. 1	Douglas Fir Larch No. 1
Depth of Stud (in)	5.5	5.5
Width of Stud (in)	1.5	1.5
Spacing (in O.C.)	16	16
Chords[1]		
Type of Wood	Douglas Fir Larch No. 1	Douglas Fir Larch No. 1
Depth of Chord (in)	5.5	5.5
Width of Chord (in)	3	3
In-Plane Anchorage[2]		
Min. Dia. of Overturning Anchors (in.)	0.433	0.465
Dia, of Shear Anchor (in.)	0.5	0.5
Number of Shear Anchors	1	11
Max Spacing of Shear Anchors (in.)	#DIV/0!	72.00
Type of Holddown	Simpson Strongtie HDU4-SDS2.5 w/ 5/8" Bolt	Simpson Strongtie HDU5-SDS2.5 w/ 5/8" Bolt
Holddown Screws	10-SDS 1⁄4"x2 1⁄2"	14-SDS 1⁄4"x2 1⁄2"
Out-of--Plane Anchorage[3]		
Dia, of Shear Anchor (in.)	0.5	0.5
Number of Shear Anchors	20	20
Max Spacing of Shear Anchors (in.)	36.00	36.00

1: For segmented shear walls, chords are for each segment and for perforated chords are for the whole shearwall.

2: For segmented shear walls, in-plane anchorage is for each individual segment and for perforated chords are for the whole shearwall.

3: For segmented and perforated shear walls, out-of-plane anchorage is for the entire wall.

Interior Shear Walls		
	Segmented	**Perforated**
Base to 1st Level Sheathing		
Sheathing Type	Wood Structural Panels	Wood Structural Panels
Sheathing Thickness	15/32" or greater	15/32" or greater
Minimum Nail Penetration	1 1/2"	1 1/2"
Nail Size	10d	10d
Edge Nailing	10d at 3" O.C.	10d at 2" O.C.
Field Nailing	10d at 12" O.C.	10d at 12" O.C.
1st Level to Roof Sheathing		
Sheathing Type	Wood Structural Panels	Wood Structural Panels
Sheathing Thickness	15/32" or greater	15/32" or greater
Minimum Nail Penetration	1 1/2"	1 1/2"
Nail Size	10d	10d
Edge Nailing	10d at 6" O.C.	10d at 4" O.C.
Field Nailing	10d at 12" O.C.	10d at 12" O.C.
Studs		
Type of Wood	Douglas Fir Larch No. 1	Douglas Fir Larch No. 1
Depth of Stud (in)	5.5	5.5
Width of Stud (in)	1.5	1.5
Spacing (in O.C.)	16	16
Chords[1]		
Type of Wood	Douglas Fir Larch No. 1	Douglas Fir Larch No. 1
Depth of Chord (in)	5.5	5.5
Width of Chord (in)	4.5	4.5
In-Plane Anchorage[2]		
Min. Dia. of Overturning Anchors (in.)	0.794	0.866
Dia, of Shear Anchor (in.)	0.5	0.5
Number of Shear Anchors	5	33
Max Spacing of Shear Anchors (in.)	18.99	22.50
Type of Holddown	2 Simpson Strongtie HDQ8-SDS3 w/ 7/8" Bolt	2 Simpson Strongtie HDQ8-SDS3 w/ 7/8" Bolt
Holddown Screws	20-SDS 1⁄4"x3"	20-SDS 1⁄4"x3"

1: For segmented shear walls, chords are for each segment and for perforated chords are for the whole shearwall.

2: For segmented shear walls, in-plane anchorage is for each individual segment and for perforated chords are for the whole shearwall.

3: For segmented and perforated shear walls, out-of-plane anchorage is for the entire wall.

Diaphragms		
	1st Level	**Roof**
Sheathing		
Sheathing Type	Plywood - 3 Ply	Plywood - 3 Ply
Sheathing Thickness	3/8"	3/8"
Span Rating	24/0	24/0
Sheathing Case	1	1
Minimum Nail Penetration	1 1/4"	1 1/4"
Nail Size	6d	6d
Edge Nailing	6d at 6"	6d at 6"
Boundary Nailing	6d at 6"	6d at 6"
Field Nailing	6d at 12"	6d at 12"
Joists		
Type of TJI Joist	TrusJoist 16" TJI 560	TrusJoist 14" TJI 360
Spacing (in.)	16	16
Blocking[1]		
To be determined		
Chords		
Type of Wood	Douglas Fir Larch No. 1	Douglas Fir Larch No. 1
Number of Chord Members	2	2
Depth Per Member (in.)	5.5	5.5
Width Per Member (in.)	1.5	1.5

1: It is left to the engineer of record to determine the blocking requirements.

Table B- 7: Three-Story, 115 MPH Design Summary

Exterior Shear Walls		
	Segmented	Perforated
Base to 1st Level Sheathing		
Sheathing Type	Wood Structural Panels	Wood Structural Panels
Sheathing Thickness	3/8" or greater	3/8" or greater
Minimum Nail Penetration	1 1/4"	1 1/4"
Nail Size	6d	6d
Edge Nailing	6d at 6" O.C.	6d at 6" O.C.
Field Nailing	6d at 12" O.C.	6d at 12" O.C.
1st to 2nd Level Sheathing		
Sheathing Type	Wood Structural Panels	Wood Structural Panels
Sheathing Thickness	3/8" or greater	3/8" or greater
Minimum Nail Penetration	1 1/4"	1 1/4"
Nail Size	6d	6d
Edge Nailing	6d at 6" O.C.	6d at 6" O.C.
Field Nailing	6d at 12" O.C.	6d at 12" O.C.
2nd Level to Roof Sheathing		
Sheathing Type	Wood Structural Panels	Wood Structural Panels
Sheathing Thickness	3/8" or greater	3/8" or greater
Minimum Nail Penetration	1 1/4"	1 1/4"
Nail Size	6d	6d
Edge Nailing	6d at 6" O.C.	6d at 6" O.C.
Field Nailing	6d at 12" O.C.	6d at 12" O.C.
Studs		
Type of Wood	Douglas Fir Larch No. 1	Douglas Fir Larch No. 1
Depth of Stud (in)	5.5	5.5
Width of Stud (in)	1.5	1.5
Spacing (in O.C.)	16	16
Chords[1]		
Type of Wood	Douglas Fir Larch No. 1	Douglas Fir Larch No. 1
Depth of Chord (in)	5.5	5.5
Width of Chord (in)	3	4.5
In-Plane Anchorage[2]		
Min. Dia. of Overturning Anchors (in.)	0.461	0.403
Dia, of Shear Anchor (in.)	0.5	0.5
Number of Shear Anchors	1	9
Max Spacing of Shear Anchors (in.)	#DIV/0!	90.00
Type of Holddown	Simpson Strongtie HDU4-SDS2.5 w/ 5/8" Bolt	Simpson Strongtie HDU5-SDS2.5 w/ 5/8" Bolt
Holddown Screws	10-SDS 1⁄4"x2 1⁄2"	14-SDS 1⁄4"x2 1⁄2"
Out-of--Plane Anchorage[3]		
Dia, of Shear Anchor (in.)	0.5	0.5
Number of Shear Anchors	11	11
Max Spacing of Shear Anchors (in.)	65.45	65.45

1: For segmented shear walls, chords are for each segment and for perforated chords are for the whole shearwall.

2: For segmented shear walls, in-plane anchorage is for each individual segment and for perforated chords are for the whole shearwall.

3: For segmented and perforated shear walls, out-of-plane anchorage is for the entire wall.

Interior Shear Walls		
	Segmented	**Perforated**
Base to 1st Level Sheathing		
Sheathing Type	Wood Structural Panels	Wood Structural Panels
Sheathing Thickness	3/8" or greater	19/32" or greater
Minimum Nail Penetration	1 3/8"	1 1/2"
Nail Size	8d	10d
Edge Nailing	8d at 3" O.C.	10d at 3" O.C.
Field Nailing	8d at 12" O.C.	10d at 12" O.C.
1st to 2nd Level Sheathing		
Sheathing Type	Wood Structural Panels	Wood Structural Panels
Sheathing Thickness	3/8" or greater	19/32" or greater
Minimum Nail Penetration	1 3/8"	1 1/2"
Nail Size	8d	10d
Edge Nailing	8d at 4" O.C.	10d at 4" O.C.
Field Nailing	8d at 12" O.C.	10d at 12" O.C.
2nd Level to Roof Sheathing		
Sheathing Type	Wood Structural Panels	Wood Structural Panels
Sheathing Thickness	3/8" or greater	19/32" or greater
Minimum Nail Penetration	1 3/8"	1 1/2"
Nail Size	8d	10d
Edge Nailing	8d at 6" O.C.	10d at 6" O.C.
Field Nailing	8d at 12" O.C.	10d at 12" O.C.
Studs		
Type of Wood	Douglas Fir Larch No. 1	Douglas Fir Larch No. 1
Depth of Stud (in)	5.5	5.5
Width of Stud (in)	1.5	1.5
Spacing (in O.C.)	12	12
Chords[1]		
Type of Wood	Douglas Fir Larch No. 1	Douglas Fir Larch No. 1
Depth of Chord (in)	5.5	5.5
Width of Chord (in)	4.5	4.5
In-Plane Anchorage[2]		
Min. Dia. of Overturning Anchors (in.)	0.845	0.752
Dia, of Shear Anchor (in.)	0.5	0.5
Number of Shear Anchors	4	28
Max Spacing of Shear Anchors (in.)	25.32	26.67
Type of Holddown	2 Simpson Strongtie HDU8-SDS2.5 w/ 7/8" Bolt	2 Simpson Strongtie HDQ8-SDS3 w/ 7/8" Bolt
Holddown Screws	20-SDS 1⁄4"x2 1⁄2"	20-SDS 1⁄4"x3"

1: For segmented shear walls, chords are for each segment and for perforated chords are for the whole shearwall.

2: For segmented shear walls, in-plane anchorage is for each individual segment and for perforated chords are for the whole shearwall.

3: For segmented and perforated shear walls, out-of-plane anchorage is for the entire wall.

Diaphragms			
	1st Level	2nd Level	Roof
Sheathing			
Sheathing Type	Plywood - 3 Ply	Plywood - 3 Ply	Plywood - 3 Ply
Sheathing Thickness	3/8"	3/8"	3/8"
Span Rating	24/0	24/0	24/0
Sheathing Case	1	1	1
Minimum Nail Penetration	1 1/4"	1 1/4"	1 1/4"
Nail Size	6d	6d	6d
Edge Nailing	6d at 6"	6d at 6"	6d at 6"
Boundary Nailing	6d at 6"	6d at 6"	6d at 6"
Field Nailing	6d at 12"	6d at 12"	6d at 12"
Joists			
Type of TJI Joist	TrusJoist 16" TJI 560	TrusJoist 16" TJI 560	TrusJoist 14" TJI 360
Spacing (in.)	16	16	16
Blocking[1]			
To be determined			
Chords			
Type of Wood	Douglas Fir Larch No. 1	Douglas Fir Larch No. 1	Douglas Fir Larch No. 1
Number of Chord Members	2	2	2
Depth Per Member (in.)	5.5	5.5	5.5
Width Per Member (in.)	1.5	1.5	1.5

1: It is left to the engineer of record to determine the blocking requirements.

Table B- 8: Three-Story, 140 MPH Design Summary

Exterior Shear Walls		
	Segmented	**Perforated**
Base to 1st Level Sheathing		
Sheathing Type	Wood Structural Panels - One Side	Wood Structural Panels - One Side
Sheathing Thickness	3/8" or greater	3/8" or greater
Minimum Nail Penetration	1 1/4"	1 1/4"
Nail Size	6d	6d
Edge Nailing	6d at 6" O.C.	6d at 4" O.C.
Field Nailing	6d at 12" O.C.	6d at 12" O.C.
1st to 2nd Level Sheathing		
Sheathing Type	Wood Structural Panels - One Side	Wood Structural Panels - One Side
Sheathing Thickness	3/8" or greater	3/8" or greater
Minimum Nail Penetration	1 1/4"	1 1/4"
Nail Size	6d	6d
Edge Nailing	6d at 6" O.C.	6d at 6" O.C.
Field Nailing	6d at 12" O.C.	6d at 12" O.C.
2nd Level to Roof Sheathing		
Sheathing Type	Wood Structural Panels - One Side	Wood Structural Panels - One Side
Sheathing Thickness	3/8" or greater	3/8" or greater
Minimum Nail Penetration	1 1/4"	1 1/4"
Nail Size	6d	6d
Edge Nailing	6d at 6" O.C.	6d at 6" O.C.
Field Nailing	6d at 12" O.C.	6d at 12" O.C.
Studs		
Type of Wood	Douglas Fir Larch No. 1	Douglas Fir Larch No. 1
Depth of Stud (in)	5.5	5.5
Width of Stud (in)	1.5	1.5
Spacing (in O.C.)	16	16
Chords[1]		
Type of Wood	Douglas Fir Larch No. 1	Douglas Fir Larch No. 1
Depth of Chord (in)	5.5	5.5
Width of Chord (in)	3	4.5
In-Plane Anchorage[2]		
Min. Dia. of Overturning Anchors (in.)	0.562	0.491
Dia, of Shear Anchor (in.)	0.5	0.5
Number of Shear Anchors	2	14
Max Spacing of Shear Anchors (in.)	51.6	55.38
Type of Holddown	Simpson Strongtie HDC10/22-SDS2.5 w/ 7/8" Bolt	Simpson Strongtie HDU8-SDS2.5 w/ 7/8" Bolt
Holddown Screws	24-SDS 1⁄4"x2 1⁄2"	20-SDS 1⁄4"x2 1⁄2"
Out-of--Plane Anchorage[3]		
Dia, of Shear Anchor (in.)	0.5	0.5
Number of Shear Anchors	20	20
Max Spacing of Shear Anchors (in.)	36.00	36.00

1: For segmented shear walls, chords are for each segment and for perforated chords are for the whole shearwall.

2: For segmented shear walls, in-plane anchorage is for each individual segment and for perforated chords are for the whole shearwall.

3: For segmented and perforated shear walls, out-of-plane anchorage is for the entire wall.

Interior Shear Walls		
	Segmented	Perforated
Base to 1st Level Sheathing		
Sheathing Type	Wood Structural Panels - Both Sides	N/A: Nominal Capacity > 2435 plf
Sheathing Thickness	3/8" or greater	N/A
Minimum Nail Penetration	1 3/8"	N/A
Nail Size	8 d	N/A
Edge Nailing	8d at 4" O.C.	N/A
Field Nailing	8d at 12" O.C.	N/A
1st to 2nd Level Sheathing		
Sheathing Type	Wood Structural Panels - Both Sides	N/A: Nominal Capacity > 2435 plf
Sheathing Thickness	3/8" or greater	N/A
Minimum Nail Penetration	1 3/8"	N/A
Nail Size	8d	N/A
Edge Nailing	8d at 6" O.C.	N/A
Field Nailing	8d at 12" O.C.	N/A
2nd Level to Roof Sheathing		
Sheathing Type	Wood Structural Panels - One Side	N/A: Nominal Capacity > 2435 plf
Sheathing Thickness	3/8" or greater	N/A
Minimum Nail Penetration	1 3/8"	N/A
Nail Size	8d	N/A
Edge Nailing	8d at 6" O.C.	N/A
Field Nailing	8d at 12" O.C.	N/A
Studs		
Type of Wood	Douglas Fir Larch No. 1	N/A
Depth of Stud (in)	5.5	N/A
Width of Stud (in)	1.5	N/A
Spacing (in O.C.)	12	N/A
Chords[1]		
Type of Wood	Douglas Fir Larch No. 1	N/A
Depth of Chord (in)	5.5	N/A
Width of Chord (in)	6	N/A
In-Plane Anchorage[2]		
Min. Dia. of Overturning Anchors (in.)	1.029	N/A
Dia, of Shear Anchor (in.)	0.5	N/A
Number of Shear Anchors	6	N/A
Max Spacing of Shear Anchors (in.)	15.192	N/A
Type of Holddown	2 Simpson Strongtie HD12 w/ 1" Bolt	N/A
Holddown Screws	4 - 1" Stud Bolts	N/A

1: For segmented shear walls, chords are for each segment and for perforated chords are for the whole shearwall.

2: For segmented shear walls, in-plane anchorage is for each individual segment and for perforated chords are for the whole shearwall.

3: For segmented and perforated shear walls, out-of-plane anchorage is for the entire wall.

Diaphragms			
	1st Level	2nd Level	Roof
Sheathing			
Sheathing Type	Plywood - 3 Ply	Plywood - 3 Ply	Plywood - 3 Ply
Sheathing Thickness	3/8"	3/8"	3/8"
Span Rating	24/0	24/0	24/0
Sheathing Case	1	1	1
Minimum Nail Penetration	1 1/4"	1 1/4"	1 1/4"
Nail Size	6d	6d	6d
Edge Nailing	6d at 6"	6d at 6"	6d at 6"
Boundary Nailing	6d at 6"	6d at 6"	6d at 6"
Field Nailing	6d at 12"	6d at 12"	6d at 12"
Joists			
Type of TJI Joist	TrusJoist 16" TJI 560	TrusJoist 16" TJI 560	TrusJoist 14" TJI 360
Spacing (in.)	16	16	16
Blocking[1]			
To be determined			
Chords			
Type of Wood	Douglas Fir Larch No. 1	Douglas Fir Larch No. 1	Douglas Fir Larch No. 1
Number of Chord Members	2	2	2
Depth Per Member (in.)	5.5	5.5	5.5
Width Per Member (in.)	1.5	1.5	1.5

1: It is left to the engineer of record to determine the blocking requirements.

Table B- 9: Three-Story, 160 MPH Design Summary

Exterior Shear Walls		
	Segmented	**Perforated**
Base to 1st Level Sheathing		
Sheathing Type	Wood Structural Panels - One Side	Wood Structural Panels - One Side
Sheathing Thickness	3/8" or greater	3/8" or greater
Minimum Nail Penetration	1 3/8"	1 3/8"
Nail Size	8 d	8d
Edge Nailing	8d at 4" O.C.	8d at 4" O.C.
Field Nailing	8d at 12" O.C.	8d at 12" O.C.
1st to 2nd Level Sheathing		
Sheathing Type	Wood Structural Panels - One Side	Wood Structural Panels - One Side
Sheathing Thickness	3/8" or greater	3/8" or greater
Minimum Nail Penetration	1 3/8"	1 3/8"
Nail Size	8d	8d
Edge Nailing	8d at 6" O.C.	8d at 6" O.C.
Field Nailing	8d at 12" O.C.	8d at 12" O.C.
2nd Level to Roof Sheathing		
Sheathing Type	Wood Structural Panels - One Side	Wood Structural Panels - One Side
Sheathing Thickness	3/8" or greater	3/8" or greater
Minimum Nail Penetration	1 3/8"	1 3/8"
Nail Size	8d	8d
Edge Nailing	8d at 6" O.C.	8d at 4" O.C.
Field Nailing	8d at 12" O.C.	8d at 12" O.C.
Studs		
Type of Wood	Douglas Fir Larch No. 1	Douglas Fir Larch No. 1
Depth of Stud (in)	5.5	5.5
Width of Stud (in)	1.5	1.5
Spacing (in O.C.)	16	16
Chords[1]		
Type of Wood	Douglas Fir Larch No. 1	Douglas Fir Larch No. 1
Depth of Chord (in)	5.5	5.5
Width of Chord (in)	3	4.5
In-Plane Anchorage[2]		
Min. Dia. of Overturning Anchors (in.)	0.642	0.561
Dia, of Shear Anchor (in.)	0.5	0.5
Number of Shear Anchors	2	18
Max Spacing of Shear Anchors (in.)	51.6	42.35
Type of Holddown	Simpson Strongtie HDC10/22-SDS2.5 w/ 7/8" Bolt	2 Simpson Strongtie HDU5-SDS2.5 w/ 5/8" Bolt
Holddown Screws	24-SDS 1⁄4"x2 1⁄2"	14-SDS 1⁄4"x2 1⁄2"
Out-of--Plane Anchorage[3]		
Dia, of Shear Anchor (in.)	0.5	0.5
Number of Shear Anchors	21	21
Max Spacing of Shear Anchors (in.)	34.29	34.29

1: For segmented shear walls, chords are for each segment and for perforated chords are for the whole shearwall.

2: For segmented shear walls, in-plane anchorage is for each individual segment and for perforated chords are for the whole shearwall.

3: For segmented and perforated shear walls, out-of-plane anchorage is for the entire wall.

Interior Shear Walls		
	Segmented	**Perforated**
Base to 1st Level Sheathing		
Sheathing Type	Wood Structural Panels - Both Sides	N/A: Nominal Capacity > 2435 plf
Sheathing Thickness	3/8" or greater	N/A
Minimum Nail Penetration	1 3/8"	N/A
Nail Size	8 d	N/A
Edge Nailing	8d at 3" O.C.	N/A
Field Nailing	8d at 12" O.C.	N/A
1st to 2nd Level Sheathing		
Sheathing Type	Wood Structural Panels - Both Sides	N/A: Nominal Capacity > 2435 plf
Sheathing Thickness	3/8" or greater	N/A
Minimum Nail Penetration	1 3/8"	N/A
Nail Size	8d	N/A
Edge Nailing	8d at 4" O.C.	N/A
Field Nailing	8d at 12" O.C.	N/A
2nd Level to Roof Sheathing		
Sheathing Type	Wood Structural Panels - Both Sides	N/A: Nominal Capacity > 2435 plf
Sheathing Thickness	3/8" or greater	N/A
Minimum Nail Penetration	1 3/8"	N/A
Nail Size	8d	N/A
Edge Nailing	8d at 6" O.C.	N/A
Field Nailing	8d at 12" O.C.	N/A
Studs		
Type of Wood	Douglas Fir Larch No. 1	N/A
Depth of Stud (in)	5.5	N/A
Width of Stud (in)	1.5	N/A
Spacing (in O.C.)	12	N/A
Chords[1]		
Type of Wood	Douglas Fir Larch No. 1	N/A
Depth of Chord (in)	5.5	N/A
Width of Chord (in)	7.5	N/A
In-Plane Anchorage[2]		
Min. Dia. of Overturning Anchors (in.)	1.176	N/A
Dia, of Shear Anchor (in.)	0.5	N/A
Number of Shear Anchors	8	N/A
Max Spacing of Shear Anchors (in.)	10.85142857	N/A
Type of Holddown	2 Simpson Strongtie HD19 w/ 1 1/4" Bolt	N/A
Holddown Screws	5 - 1" Stud Bolts	N/A

1: For segmented shear walls, chords are for each segment and for perforated chords are for the whole shearwall.

2: For segmented shear walls, in-plane anchorage is for each individual segment and for perforated chords are for the whole shearwall.

3: For segmented and perforated shear walls, out-of-plane anchorage is for the entire wall.

Diaphragms			
	1st Level	2nd Level	Roof
Sheathing			
Sheathing Type	Plywood - 3 Ply	Plywood - 3 Ply	Plywood - 3 Ply
Sheathing Thickness	3/8"	3/8"	3/8"
Span Rating	24/0	24/0	24/0
Sheathing Case	1	1	1
Minimum Nail Penetration	1 1/4"	1 1/4"	1 1/4"
Nail Size	6d	6d	6d
Edge Nailing	6d at 6"	6d at 6"	6d at 6"
Boundary Nailing	6d at 6"	6d at 4"	6d at 6"
Field Nailing	6d at 12"	6d at 12"	6d at 12"
Joists			
Type of TJI Joist	TrusJoist 16" TJI 560	TrusJoist 16" TJI 560	TrusJoist 14" TJI 360
Spacing (in.)	16	16	16
Blocking[1]			
To be determined			
Chords			
Type of Wood	Douglas Fir Larch No. 1	Douglas Fir Larch No. 1	Douglas Fir Larch No. 1
Number of Chord Members	2	2	2
Depth Per Member (in.)	5.5	5.5	5.5
Width Per Member (in.)	1.5	1.5	1.5

1: It is left to the engineer of record to determine the blocking requirements.