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Justin D. Holman for the degree of Master of Science in Civil Engineering and Wood Science and presented on September 20, 2019

Title: Load Path Investigation in a Light-Frame Wood House with Complex Geometry

Abstract approved:

Rakesh Gupta Thomas H. Miller

The objective of this study was to investigate load paths in an existing light-frame wood structure with complex geometry, using previously established computer modeling methods. The structural model is derived from an existing, multi-level residence located in Corvallis, Oregon. For the purpose of this study, the existing structure was simplified in some respects, then analyzed using a commercial finite-element software. Modeling methods developed in previous research were used for both gravity and wind loads in accordance with the 2014 Oregon Structural Specialty Code (OSSC), and ASCE 7-10. The modeling and analysis were done using SAP2000 for behavior within the linear elastic range using frame and shell elements. Results from the study showed gravity load paths and the governing load combinations for different parts of the structure. Wind investigations revealed load concentrations at corners and openings due to uplift pressure on the roof, as well as overturning forces from lateral wind loads on the walls. Additionally, load concentrations are dependent on wind direction and the loading type, as well as the amount of dead load present. Load path investigations show that for this asymmetrical structure, the shear carried in each wall is dependent on the wind direction, and the stiffness of the wall. By adding an additional shear wall to the structure, the base shear redistributes with shear wall loads closest to the added shear wall seeing the greatest reductions. Finally, stress concentrations in the roof sheathing under wind loads revealed that a truss extending up the slope of the roof, perpendicular to the other trusses, can cause high stress concentrations due to the stiffness it adds in comparison to the rafters surrounding it.

Load Path Investigation in a Light-Frame Wood House with Complex Geometry

by
Justin D. Holman

A THESIS

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I understand that my thesis will become part of the permanent collection of Oregon State University libraries. My signature below authorizes release of my thesis to any reader upon request.

Justin D. Holman, Author

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CONTRIBUTIONS OF AUTHORS

Dr. Rakesh Gupta and Dr. Thomas H. Miller provided technical guidance and support during the research stages. They also provided insight and editing for the final manuscript.

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INTRODUCTION

A well-designed structure balances two key aspects 1) adequate strength to resist the loads applied to it, and 2) an efficient design so as to not waste unnecessary resources or time in construction. To meet these conditions, it is imperative to understand the flow of loads through a particular structure so that locations that have the greatest load concentrations can be strengthened. Light-frame wood structures are statically indeterminate, and often contain very complex structural systems, the behavior of which often cannot be captured by simple hand calculations or 2D modeling methods.

Due to their lightness, one particular concern with these structures is their performance under high wind loading. Direct property damage from natural disasters in the U.S. each year exceeds $35 billion, and between 1970 and 2012 approximately 70 percent of insured losses were from wind damage (Holmes 2017). Hurricanes such as Harvey in 2017, and tornadoes such as the one in Joplin, Missouri in 2011, reveal that there are still deficiencies in the design, construction, and inspection of light-frame wood structures, and opportunities for building codes to be updated.

The failure of roof-to-wall connections, particularly on gable roofs, has been the focus of much study, as well as wall-to-foundation connections that have failed under extreme wind loading (van de Lindt et al. 2007). Sheathing loss in hip roofs, while less common than in gable roofs, has been noted to occur not only via connection failures, but also due to structural damage to framing members in the roof (Stevenson 2017). These failures can occur in truss framed roofs, but are much more prevalent in stick-framed roofs with long unsupported members and toe-nail connections at each end of the members.

Understanding the behavior of these complex light-frame wood structures can lead to proper retrofitting of existing structures, as well as in the design of new structures, in areas at risk to severe wind loads. The best method to gain understanding of load flow requires physical testing of either full-scale or scale model structures. However, such research can be both time consuming and expensive. Computer models have been developed to reduce costs and permit study of a wide variety of structures and loadings. Martin et al. (2011) used a SAP 2000 model based on research done at the University of Florida, with physical tests on a 1/3rd scale rectangular structure, as well as wind tunnel testing on a 1:50 scaled model. Pfretzschner et al. (2014) built upon the methods of Martin et al. (2011) and compared results to physical testing conducted by Paevere et al. (2003) on an L-shaped building. Thus, these methods can now be used to investigate more complex existing and new structures with reasonable confidence that the results will accurately reflect the behavior of the actual structure.

OBJECTIVES

The primary objectives of this project were: (1) to apply the computer modeling methods developed by Martin et al. (2011) and Pfretzschner et al. (2014) to analyze an existing residential structure with a complex geometry; and (2) to investigate wind and gravity load paths for that building and determine geometric features that cause load concentrations.

RESEARCH APPROACH

The modeling methods used in this study were developed by Martin et al. (2011), and further refined by Pfretzschner et al. (2014) using SAP2000 finite-element analysis software. As with the previous two projects, an emphasis was put on practical modeling procedures, modeling connections simply as either, pinned, rigid, or with a spring. Further, all modeling and analysis was done within the elastic range, and thus, no non-linear analysis was done. For all materials, industry standard properties were used, with the exception of shear stiffness for the plywood sheathing; for this, results from Pfretzschner et al. (2014) were adopted.

Using these previously developed methods, the model was subjected to:

1. **Gravity Load Investigation:** Dead, live, and snow loads, as well as combinations thereof, were applied as uniform loads to the shell elements representing the sheathing.
2. **Wind uplift investigation:** Wind loads, in accordance with the all-heights method in the 2014 Oregon Structural Specialty Code (OSSC), were applied as uniform loads orthogonal to all of the house’s external surfaces. Due to the building’s geometric complexity, wind coming from all four orthogonal directions was considered. Once modeled, uplifts at anchor and hold-down points were evaluated.

3. **Wind base shear investigation:** Wind loads were applied in the same manner as in the uplift investigation, however, instead of examining the uplift forces, the shear forces in the anchor bolts were evaluated. Additionally, a modified structure was created by sheathing one of the interior walls, so as to investigate the influence of adding an extra shear wall.

Results for these investigations are discussed in the manuscript, and detailed results can be found in Appendixes G-I

Manuscript:

Load Path Investigation in a Light-Frame Wood House with Complex Geometry

Justin Holman, Rakesh Gupta, Thomas H. Miller

To be submitted to the Frontiers in Built Environment journal.

ABSTRACT: The objective of this study was to investigate load paths in an existing light-frame wood structure with complex geometry, using previously established computer modeling methods. The structural model is derived from an existing, multi-level residence located in Corvallis, Oregon. For the purpose of this study, the existing structure was simplified in some respects, then analyzed using a commercial finite-element software. Modeling methods developed in previous research were used for both gravity and wind loads in accordance with the 2014 Oregon Structural Specialty Code (OSSC), and ASCE 7-10. The modeling and analysis were done using SAP 2000 for behavior within the linear elastic range using frame and shell elements. Results from the study showed gravity load paths and the governing load combinations for different parts of the structure. Wind investigations revealed load concentrations at corners and openings due to uplift pressure on the roof, as well as overturning forces from lateral wind loads on the walls. Additionally, load concentrations are dependent on wind direction and the loading type, as well as the amount of dead load present. Load path investigations show that for this asymmetrical structure, the shear carried in each wall is dependent on the wind direction, and the stiffness of the wall. By adding an additional shear wall to the structure, the base shear redistributes with shear wall loads closest to the added shear wall seeing the greatest reductions. Finally, stress concentrations in the roof sheathing under wind loads revealed that a truss extending up the slope of the roof, perpendicular to the other trusses, can cause high stress concentrations due to the stiffness it adds in comparison to the rafters surrounding it.

INTRODUCTION

Every year in the U.S. direct property damage from natural disasters exceeds $35 billion, and between 1970 and 1999 approximately 70 percent of insured losses were from wind damage (Holmes 2017). Wind storms such as hurricanes Katrina in 2005 and Harvey in 2017, as well as tornadoes such as the one seen in 2011 in Joplin Missouri have shown that wind damage is still a major issue, particularly for light-framed residential houses, even ones built under more recent building codes. Due to their lightweight nature, these residential structures are particularly susceptible to uplift forces, so it is important to understand where failures tend to occur, and what can be done to help mitigate the damage.

A well-designed structure is both strong enough to resist the loads applied to it, as well as efficiently designed, to reduce cost, materials, and construction time. To accomplish this, one needs to understand how the loads flow through the structure so that we can strengthen and properly design all of the structural elements and connections along that path. Various aspects of the building's overall geometry, as well as the stiffness and orientations of individual elements, affect the building's response to different types of loadings. For wind loads, one needs to investigate how the house's geometry affects the uplift forces, to know which hold-downs must be strengthened. Similarly, the wind-induced shear force distribution on the foundation is needed to ensure that the anchor bolts are sufficient.

Analyzing the behavior of a structure can be done either using physical tests or computer modeling methods. To develop good computer modeling methods, it is important to calibrate the model with a (hopefully full-scale) physical test. This was done by Martin et al. (2011) modeling a rectangular building that had previously undergone physical wind tunnel testing, and also by Pfretzschner et al. (2014) with an L-shaped house that previously experienced physical testing as well. Both models were linear-elastic in SAP2000 and produced results similar to the physical tests.

A one-third scale physical model tested at the University of Florida served as the basis for Martin et al. (2011). The material properties for the rectangular building were based on industry standards and sheathing was modeled as solid shell elements with its stiffness modified for different nail spacing. The computer analysis was consistent with the physical tests for both lateral and uplift wind load cases.

Pfretzschner et al. (2014) built upon the methods established by Martin et al. (2011) to model an L-shaped house that had undergone full-scale physical model testing by Paevere (2002). The house was one-story but included several doors and windows that added to the geometric complexity. Pfretzschner et al. (2014) investigated the effects of lateral as well as uplift wind loads similar to Martin et al. (2011), and showed that re-entrant corners and wall openings cause load concentrations. Also, particularly due to the asymmetrical nature of the building, load concentrations are affected by wind direction, meaning, wind loads from all four directions must be investigated. The modeling methods established by Pfretzschner et al. (2014) were the primary source for this current investigation with a more complex geometry.

Malone (2013) furthered the research by comparing load paths in light-frame versus timber frame structures. His model was not based upon any physically tests, rather it relied upon the verified modeling methods of Martin et al. (2011) and Pfretzschner et al. (2014). Malone (2013) investigated not only wind, but also gravity loads, something the previous studies had not. Dead loads were incorporated into the member's self-weight, while live and snow loads were applied directly to shell elements, similar to how wind loads had been applied in previous studies.

Aspects of all three studies will be applied in this current investigation. Modeling methods are primarily based upon Martin et al. (2011), wind applications are similar to Pfretzschner et al. (2014), and gravity loadings similar to that of Malone (2013).

OBJECTIVES

The two primary objectives of this project were: (1) to apply the computer modeling methods with SAP2000, developed by Martin et al. (2011) for a rectangular building, and Pfretzschner et al. (2014) for an L-shaped building, to an existing residential structure with a complex geometry; and (2) to investigate the effects of wind and gravity loads on a light-frame wood structure with a complex geometry to determine load paths and geometric features that can cause concentrations of forces.

MODELING METHODS

The Modeled House

The modeled house in this study was based upon one actually located in Corvallis, Oregon (Figure 1). The existing house has three stories, one of which is a daylight basement, and was built using light-framed wood construction.

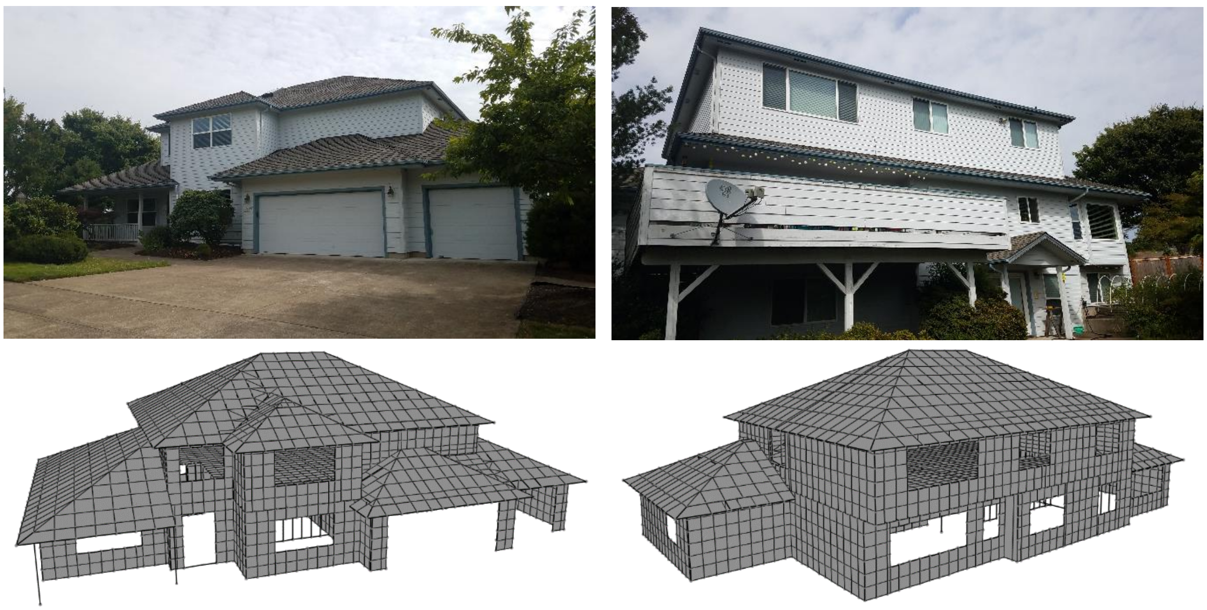


Figure 1: Top: Front (left) and back (right) views of the house.

Bottom: Front (left) and back (right) views of the computer model.

Several modifications were made during the process of converting the existing structure to a computer model. These changes were made either to simplify the modeling, or because the house plans were not entirely clear or complete. First, the original house is three levels with a daylight basement. This daylight basement was removed because it contained a concrete retaining wall. The primary focus of this research is on investigating load paths in light-frame wood structures, so the concrete wall would introduce an unnecessary complicating factor in several respects.

Other changes included reducing the height of the first story from 274 cm (9-feet) to 244 cm (8-feet) so as to have more consistency in wall stiffness, removing the first floor bay windows located on the "north" face of the house to eliminate some complexity, and several other minor dimensional changes to simplify the geometry of the structure and make it more consistent for modeling purposes.

The structure has several interior walls, and with the exception of the garage walls, most are not considered sheathed. The west-most interior wall was originally modeled as not being sheathed. However, to study the influence of sheathing this wall had on load paths, sheathing was added to the wall for a modified version of the structure that is compared to the original structure later in this investigation.

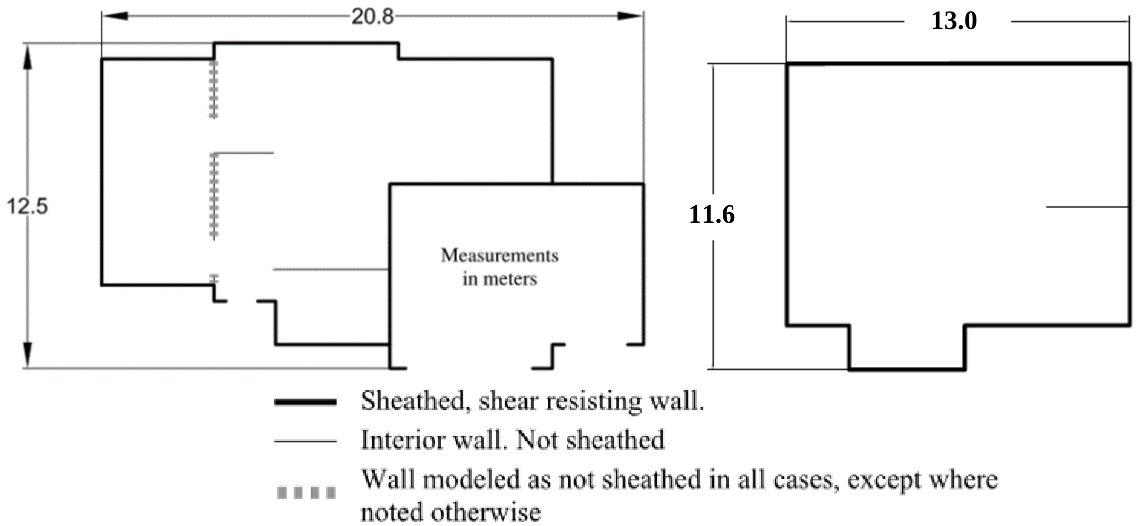


Figure 2: Left: First floor floorplan. Right: Second floor floorplan.

The house's diaphragms are the 2nd floor, which consists of 13 mm (1/2 in) plywood on top of BCI 90 I-joists, as well as the roofing systems. No sheathing was assumed to be in the ceilings to create diaphragms.

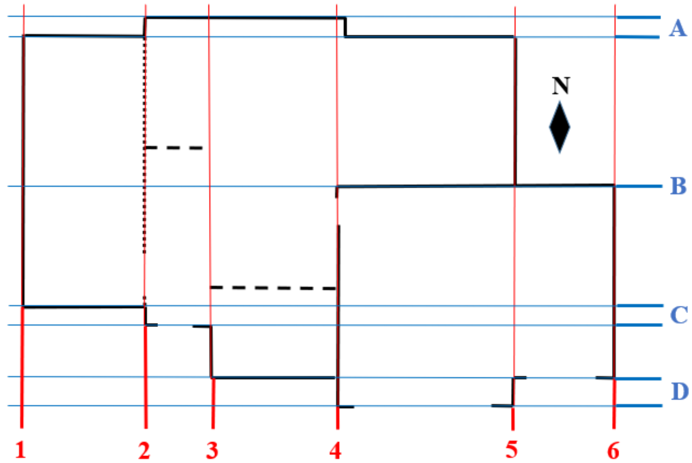


Figure 3: Wall lines.

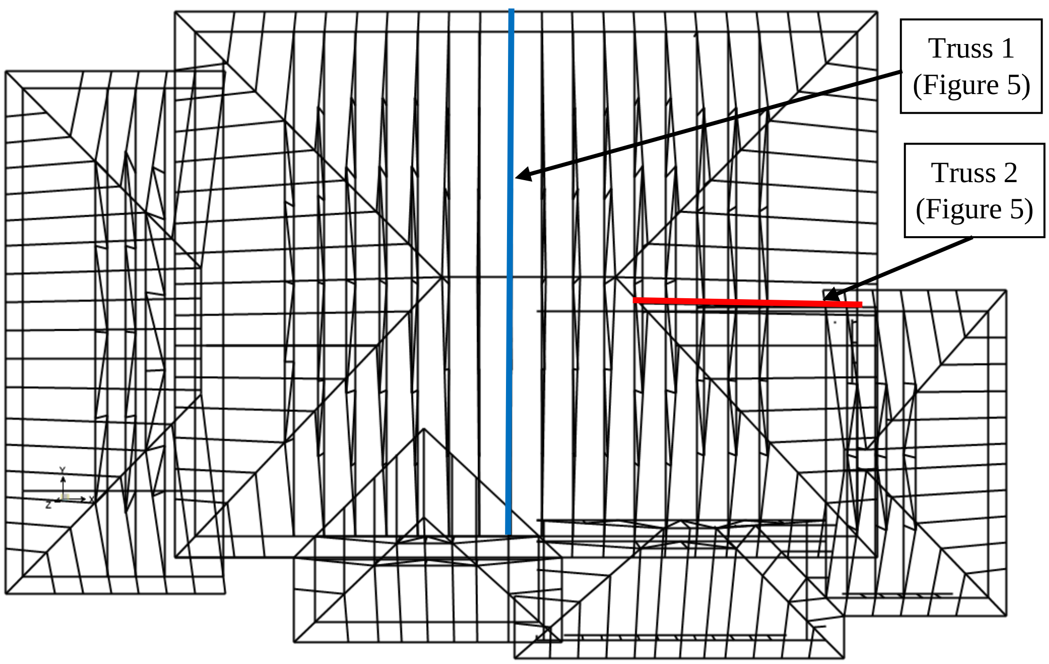


Figure 4: Plan view of roof framing.

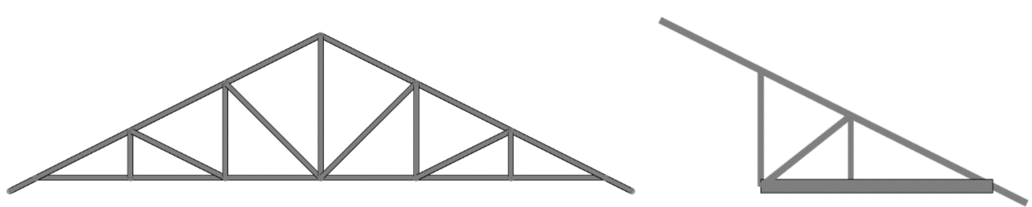


Figure 5: Left: Truss 1 in Figure 4. Right: Truss 2 in Figure 2

The roof system (Figure 4) is made up of various trusses, an example of which can be seen in Figure 5 (for details on all trusses see Appendix B). The majority of the trusses run north-to-south with the exception of: (1) over the protrusion and garage on the south side of the building; and (2) a single truss running east to west up the slope of the eastern second story roof (see Truss 2 in Figures 4 and 5). This truss will be important to the results.

The framing members for the structure, including all studs, top and bottom plates, rafters, and floor I-joists, were modeled using SAP 2000 frame elements, with each member being modeled through its centerline. In cases where multiple members were nailed together (double studs, top plates, and trusses), they were modeled as one solid member. All material properties are based on the National Design Specification (NDS) (AWC 2015), with the exception of the I-joists used in the second floor with material properties from Boise Cascade. All top and bottom plates, as well as the top and bottom chords of all trusses were modeled as continuous members in accordance with both Martin et al. (2011) and Pfretzschner et al. (2014).

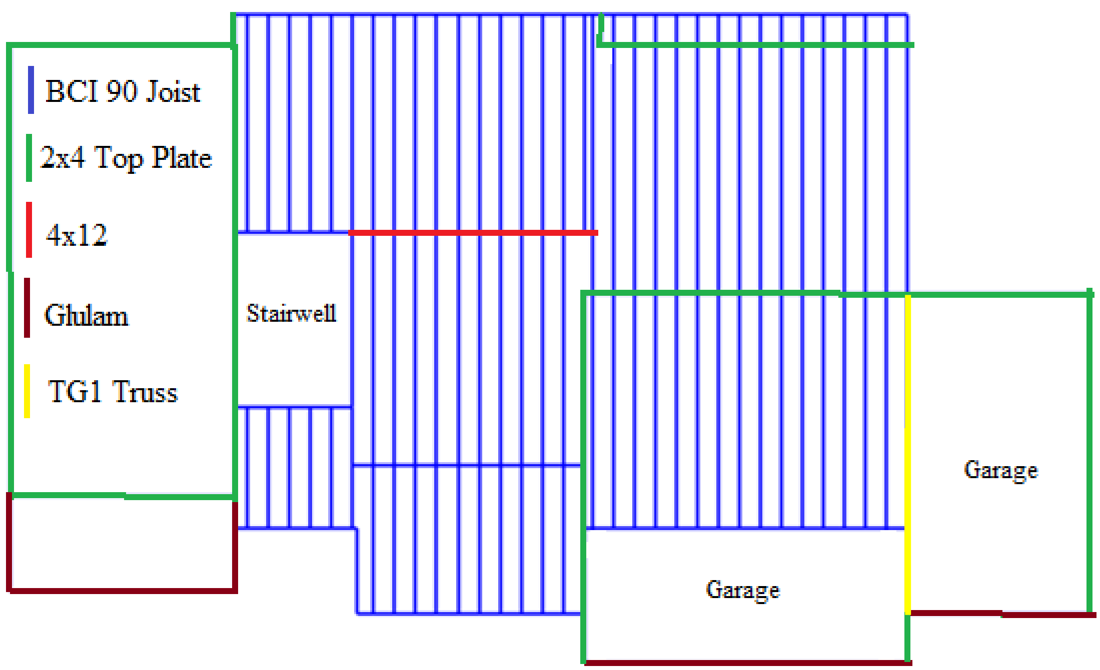


Figure 6: Second floor framing plan showing I joists, as well as top plates and supporting beams.

Sheathing

Since few details were available on the sheathing used for the house that was modeled in this study, the methods and material properties used by Pfretzschner et al. (2014) were adopted. Wall and roof sheathing elements were modeled using the SAP 2000 layered shell feature, which allows for multiple materials and layers within one shell element. All wind-resisting walls were modeled with both 9.7 mm (3/8") plywood as well as GWB (gypsum wall board) layers, while the roof was modeled using only one 12.5 mm (1/2") plywood layer. When modeled, each shell was not broken into 1.22 m x 2.44 m (4 ft x 8 ft) sheets as plywood on the typical house would be, but was modeled as if the plywood were one continuous sheet, as done in both Martin et al. (2011) and Pfretzschner et al. (2014). The shells were then meshed into smaller elements, as seen in Figure 8. It is important when meshing that nodes intersect with frame members, and if there are shell elements connecting to each other, that the nodes align, so that the stresses may be properly transferred. As such, due to the unusual geometry of the house, unlike Pfretzschner et al. (2014), a single shell could not be used to model each side of the house, but rather several shells were assigned to each wall or roof area so that the above conditions could be met.

The material properties used for the sheathing and gypsum wall board (GWB) are based off of properties calculated by Pfretzschner et al. (2014). Plywood properties were calculated using OSULaminates (Nairn 2007) based on the thickness and orientation of each layer and the total number of layers. These properties are used for each of the walls within the house, with the exception of the G12 shear stiffness; this had to be calculated separately. The G12 property varies approximately linearly, based on wall length as discussed by Pfretzschner et al. (2014), and shown in Figure 7. See Appendix B for a detailed discussion on determining G12.

Gypsum wallboard (GWB) properties were assigned based on values from the Gypsum Association (2010), and G12 values for GWB were found in a similar manner as for the plywood.

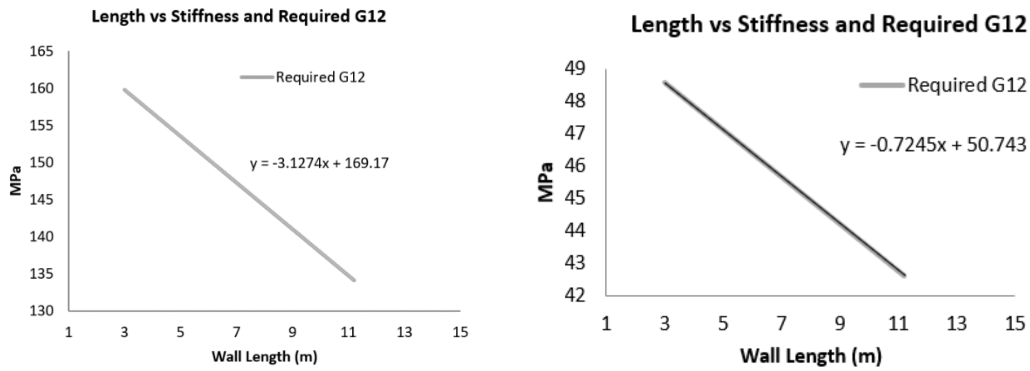


Figure 7: (Left): G12 stiffness for plywood sheathing vs. wall length. (Right): G12 stiffness of GWB vs. wall length. (Pfretzschner et al. (2014))

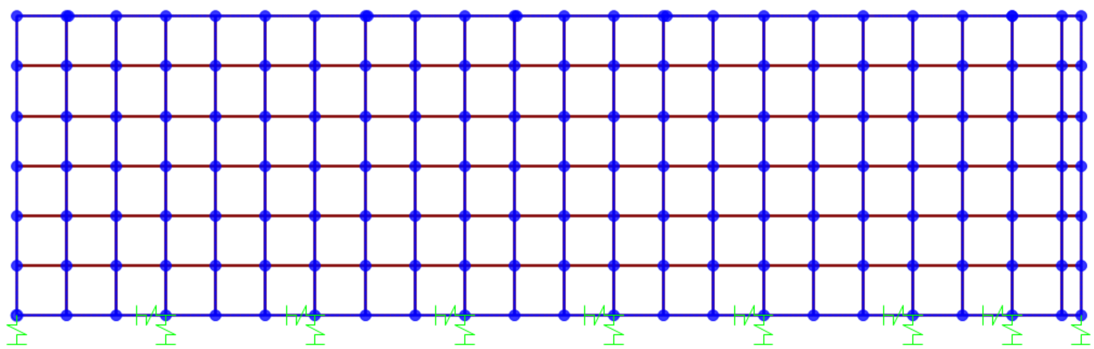


Figure 8: Example of a meshed wall. Vertical lines as well as top and bottom lines are frame elements, while the interior horizontal lines divide the shell into smaller elements. The vertical springs at each end represent hold downs, while the intermediate horizontal and vertical springs represent anchor bolts.

Frame connections

The connections between all framing members are modeled as either pinned or rigid; no partially fixed or spring connections were used. Wall studs are modeled as pinned in the “in-plane’ direction with the wall and rigid in the “out-of-plane” direction. This allows for lateral load to be transmitted properly through the sheathing.

Truss web members have pinned connections, while the top and bottom chords are continuous. The top members are pinned to each other, as well as to the bottom chord. However, the bottom chord has a fixed connection with the wall’s top plate.

Wall Anchorage

Anchor bolts and hold-downs were both modeled using linear springs located at joints where studs meet the bottom plate. In an actual house these hold downs and anchor bolts would be located slightly off the stud; however, for modeling purposes it is easier to locate them beneath the studs, and the stiffness and load flow is not significantly affected. Anchor bolts had three orthogonal springs to resist both base shear and vertical loads, while hold downs only consisted of a single vertical spring. As with Martin et al. (2011), the common Simpson Strong-Tie HUD2 hold down was used; the axial stiffness of which can be found in Simpson Strong-Tie (2012). Also in accordance with Martin et al. (2011), the shear stiffness of the anchor bolts was based upon a procedure recommended by the American Wood Council, and the axial stiffness determined by testing done by Seaders (2004). Hold-downs are located at each corner, while anchor bolts are spaced no more than 1.22 m (4 feet) apart, in accordance with the International Residential Code (IRC) Section R403.1.6.

Table 1: Hold-down and anchor bolt stiffness (Martin et al. 2011)

Item	**X-Direction - Shear (kN/cm)**	**Y-Direction - Shear (kN/cm)**	**Z-Direction - Axial (kN/cm)**	**Source**
Hold-Down	---	---	61.3	*Simpson Strong Tie (2008)*
Anchor Bolts	114	114	61.3	*NDS (AWC 2015 and Seaders 2004)*

Material Properties:

All framing members, except for the I-joists used in the second floor diaphragm, are considered to be either Douglas Fir #1 & Better, or Douglas Fir #2. The material properties were taken from the NDS (AWC 2015) with incision, wet service, and temperature factors applied (all of which were 1.0 in this case).

Table 2: Material properties of framing members

Item	Species – Grade	MOE (GPa)		Poisson's Ratio	Density (kg/m^3)	Poisson's Ratio and Density source
Truss top chords	DF- #1 & Btr	12.4	NDS (AWC 2015)	.374	510	Wood Handbook (USDA 1999)
Studs, top and bottom plates, posts and beams, other truss members	DF- #2	11.0		.374	510	
Glulams	Western Species 24F-V5	11.7		.295	600	

Sheathing values for the plywood and GWB can be found below in Table 2, and in Figure 7 for G12 properties.

Table 3: Material Properties of sheathing elements

Material	Properties	Source
Plywood Sheathing (Roof)	E_1 = 8280 MPa (1201 ksi) E_2 = 2393 MPa (347 ksi) U_{12} = 0.011 G_{12} = 482 MPa (70 ksi)	*Pfretzschner et al. (2014)* *OSULaminates (Nairn 2007)* *(Flexural Properties)*
Plywood Sheathing (Walls)	E_1 = 7017 MPa (1018 ksi) E_2 = 3657 MPa (530 ksi) U_{12} = 0.016	*Pfretzschner et al. (2014)* *OSULaminates (Nairn 2007)* *(In-Plane Properties)*
Gypsum Wallboard (Walls)	E_1 = 1820 MPa (264 ksi) U_{12} = 0.3	*Gypsum Association (2010)*

Flooring

The second floor diaphragm consists of plywood sheathing over I-joists serving as floor beams. Since no details were available, the properties of these I-joists were also used to model the edges of the diaphragm between the first and second floors.

Loadings

The loadings considered in this study include: dead, live, snow, and wind. The ASD load combinations used can be seen in Table 4.

Table 4: Loading combinations used in this study

ASD Gravity Load Combinations	ASD Wind Load Combinations
D $D + L$ $D + S$ $D + 0.75L + 0.75S$	$0.6D + 0.6W$ $1.0D + 0.6W$

Building dead loads were incorporated into the SAP 2000 model of the structure in the self-weight of the materials, as well as with uniform area loads applied to each shell as extra weights for walls, roofs, windows and doors based on building weight estimates from Boise Cascade.

Table 5: Additional uniform dead loads applied to shell elements (Boise Cascade).

Item	**Walls**	**Doors & Windows**	**Floor**	**Roof**
Loading	239 Pa (5 psf)	143 Pa (3 psf)	192 Pa (4 psf)	192 Pa (4 psf)

Live loading was only considered for the 2nd story, and was applied as a uniform 1.92 kPa (40 psf) in accordance with the ASCE 7-10 Chapter 4 (ASCE 2010) . For snow loading, a uniform minimum of 978 Pa (20 psf) was applied to the entire roof. This loading was considered over a projected area. Sliding snow and unbalanced loading scenarios were not considered for this study

Wind loading analysis was done in accordance with the 2014 OSSC, using the "All-Heights Method" that can be found in section 1609.6. This method is similar to the Directional Procedure in the ASCE 7-10 (ASCE 2010). The ASCE 7-10 wind speed for this region is 120 mph, and the house is considered to be exposure category B on flat terrain.

In this procedure, a uniform area wind pressure is applied perpendicularly to each external surface of the house. The pressure on each surface is determined by equation 16-35 in the 2014 OSSC, and using the C_{net} factor found on Table 1609.6.2. Both positive and negative internal pressure cases were investigated, as well as conditions 1 and 2 for both of these. Furthermore, due to the asymmetry of the structure, these wind loads had to be applied from each direction. This results in there being a total of 16 different wind load applications. For the purposes of this study, the wind direction refers to the direction from which the wind is coming and which will be considered the "windward wall".

Table 6: Wind loads applied perpendicular to wall and roof elements. A positive internal pressure is considered to be pushing outward from the interior of the building, while a negative internal pressure is a suction.

		Windward Wall (kPa)	Leeward Wall (kPa)	Side Wall (kPa)	Windward Roof (kPa)	Leeward Roof (kPa)	Parallel to Ridge (kPa)
Positive Internal Pressure	Condition 1	-0.49	0.58	0.75	0.53	0.75	1.23
	Condition 2	-0.49	0.58	0.75	-0.07	0.75	1.23
Negative Internal Pressure	Condition 1	-0.82	0.24	0.40	0.18	0.40	0.89
	Condition 2	-0.82	0.24	0.40	-0.42	0.40	0.89

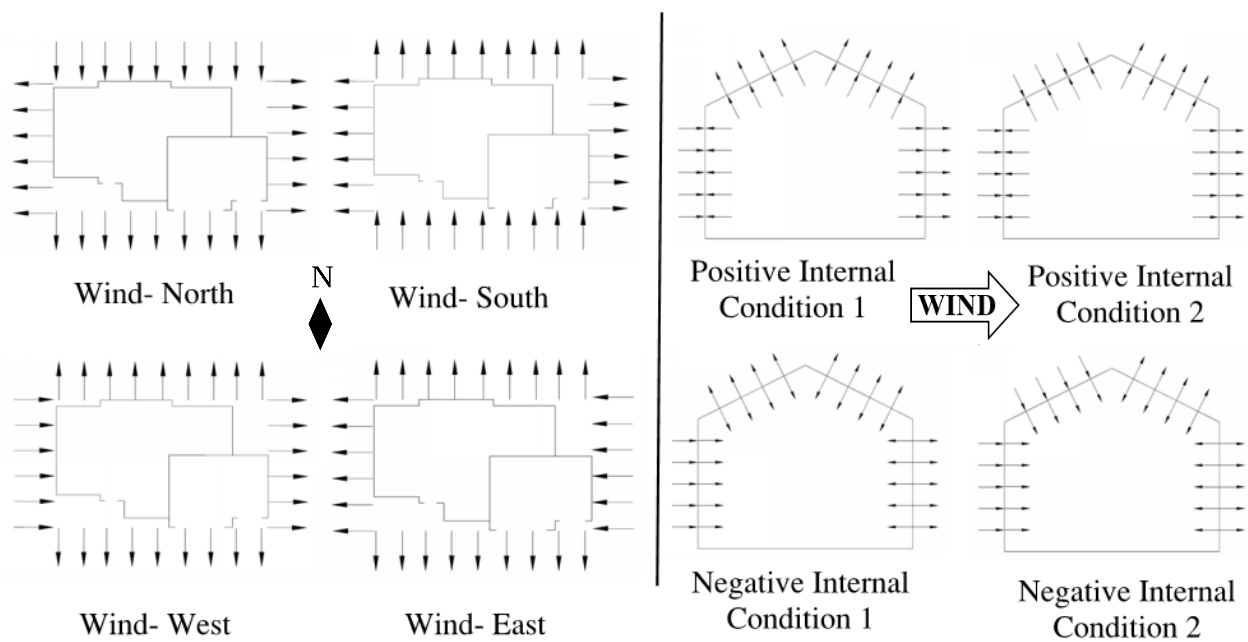


Figure 9: (Left): Wind load directions. (Right: Positive and negative internal load cases, as well as conditions 1 and 2.

Model Verification

Verification of the modeling methods was done in previous studies by Martin et al. (2011) and Pfretzschner et al. (2014). To ensure that the current model exhibited correct behavior, several wall elements were looked at independently to confirm their behavior was consistent with walls in Pfretzschner et al. (2014). Additionally, several lateral and horizontal loads were placed on the structure to confirm equilibrium of reaction forces and applied loads.

RESULTS AND DISCUSSION

Gravity loads

The purpose of the gravity load investigation is to evaluate where gravity loads for this structure concentrate, and which load combinations govern different reactions. Additionally, the stresses on roof and floor shells are examined to evaluate stress concentrations there as well.

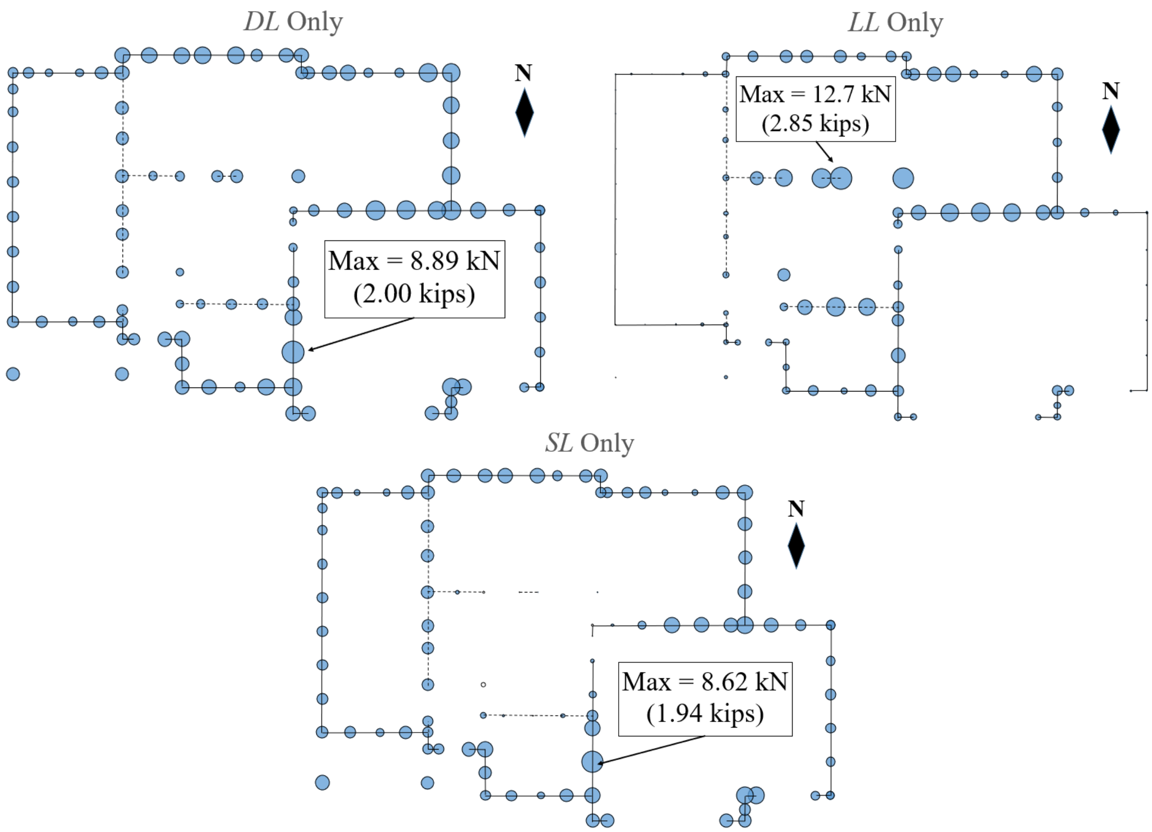


Figure 10: Plots of reactions for each individual gravity load type (dead, live, and snow). Larger bubbles represent larger reactions at that location.

Gravity load reactions are seen in Figure 10. Dead loads are spread proportionally throughout the structure with loads under the second story being greater, and concentrations near corners, with decreases under windows. Live load reactions are most heavily concentrated at the interior walls and columns, while there is little to no loading under the east- and west-most portions of the house that are not under the second story, thus having no live loading on top of them. Snow load reactions are relatively evenly distributed on the exterior walls, but there is little to no loading on the interior walls under the second story, aside from the garage walls

These loadings then can be combined using the load combinations in Table 3 to produce each of the plots in Figure 11. Figure 12 was then generated by comparing these three plots and taking the maximum reaction. From this, the governing load combination at each point can be seen. The east- and west-most portions of the house, which have no second story, and thus no live loads on top of them, are governed by the $D + S$ combination. Interior walls and columns on the first floor are generally governed by the $D + L$ combination, while exterior walls surrounding the second floor are governed by the $D + 0.75L + 0.75S$ combination. There are a few notable exceptions, however: (1) The eastern half of the north garage wall (wall line B in Figure 3) supports a second story interior wall, which in turn supports a truss going up the slope of the east-facing second story roof, and as such carries a lot of snow load as well. (2) The west-most interior wall (wall line 2 in Figure 3) has the two roofs meeting here, supplying additional snow load, and the wall is not supporting much live load, as it runs parallel to the floor joist. (3) Probably the largest oddity at first glance is that the $D + 0.75L + 0.75S$ combination governs the reactions between the two garage doors. This is due to a truss that spans north to south over the garage and carries much of the second floor live loads that overhang the garage (see Figure 5).

To evaluate the stress concentrations in the roof and floor sheathing, Von Mises stress maps were created. The roof stress map in Figure 13 is based upon the D + S loading combination, while the floor stress map is based on the D + L combination. The largest stresses on the roof are concentrated around the truss going up the slope of the eastern upper roof (Truss 2 in Figures 4 and 5). For the second floor sheathing, the stresses tend to concentrate around walls.

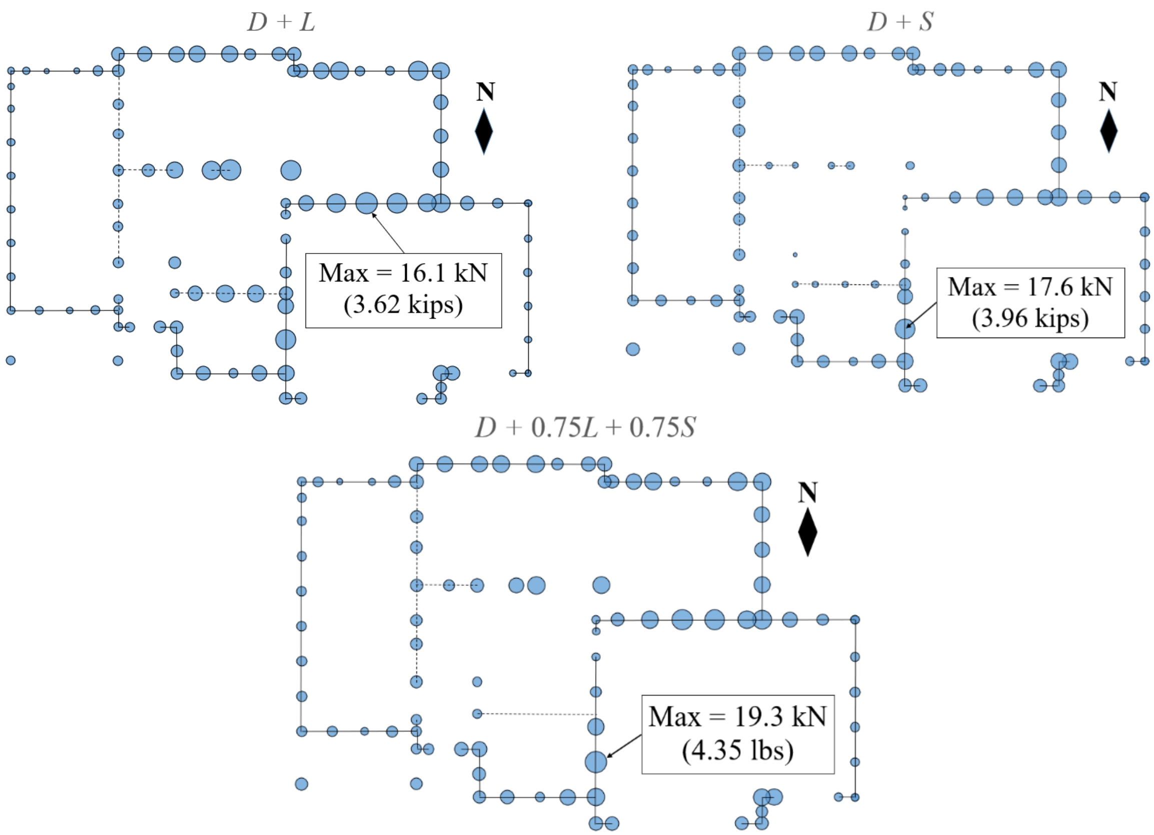


Figure 11: Plots of each gravity load combination. Larger bubbles represent larger reactions at that location.

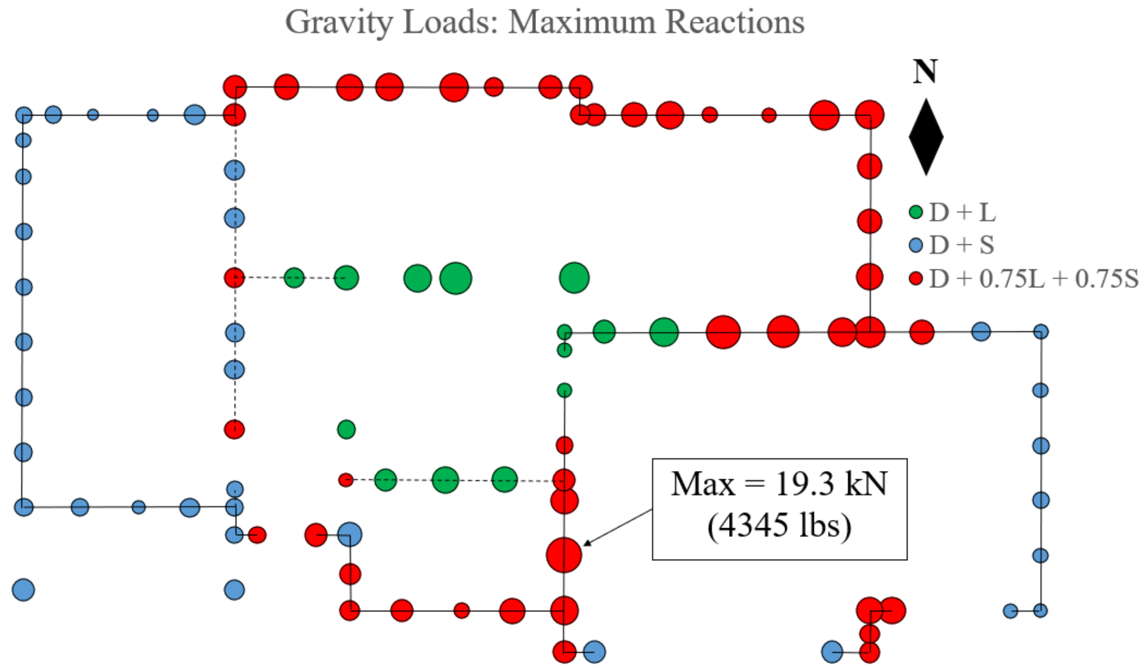


Figure 12: Plot showing the maximum load and governing load combination at each reaction. Larger bubbles indicate larger reactions. Colors indicate the governing load combination.

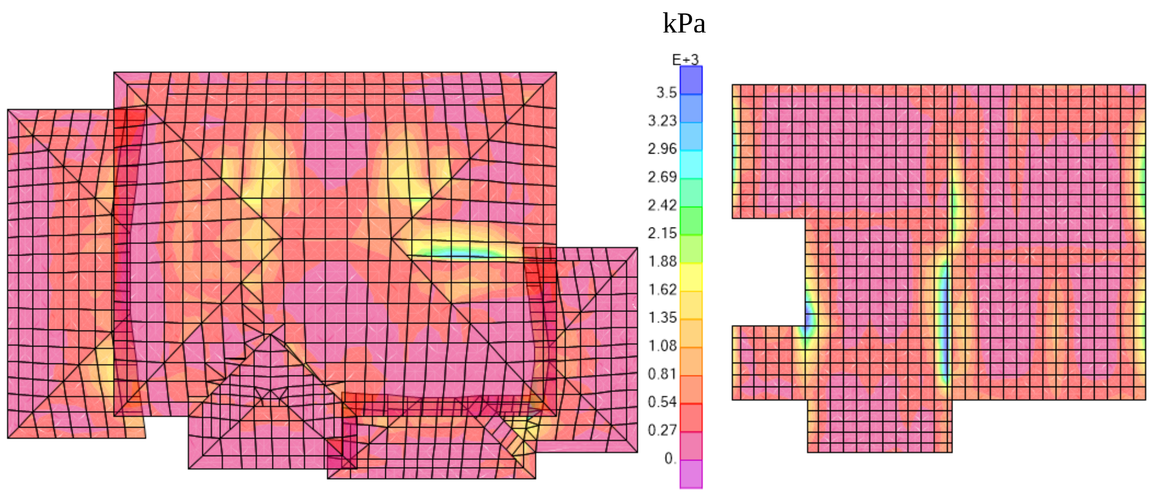


Figure 13: Left: Plot of maximum stresses in the roof shell based on *D* + *S* loading combination. Right: Plot of maximum stresses in the second story floor shell based on *D* + *L* loading combination. Von Mises stresses are in kPa

Wind Loads

Three primary aspects of wind load paths were investigated during this study: 1) vertical uplift reactions; 2) horizontal base shear reactions; and 3) stress concentrations in the sheathing on the roof. For all three, all 16 wind load cases were investigated using both the $0.6D + 06W$ and $1.0D + 06W$ loading combinations.

Uplift

Table 7 displays the total uplift on the structure due to wind for each wind loading case. Positive internal pressure with condition 1 (in bold) provides the largest total net uplift, though this may not be true for each individual base reaction. Wind from the east and west directions also result in greater uplift than wind from the north and south for this particular structure. This is due to the fact that the greatest uplift loads are applied to roofs parallel to the wind direction, and in this structure, which is wider in the E-W direction than in the N-S direction, there is more roof surface area facing north and south than east and west. Further, it can also be seen that wind from the north and the east produces greater uplift than wind from the south and west, respectively. This is due to more roof surface area, and less wall surface area, on the south and west sides of the building.

Table 7: Total uplift on entire structure for each wind loading case and wind direction. A negative load indicates an uplift on the structure while a positive force indicates a downward force.

Direction	Positive Internal – Cond. 1 (kN)	Positive Internal – Cond. 2 (kN)	Negative Internal Cond. 1 (kN)	Negative Internal Cond. 2 (kN)
North	-144	-121	-90	-67
South	-142	-115	-88	-61
West	-148	-123	-95	-69
East	**-151**	-134	-97	-80

To evaluate the uplift reactions on a more localized level, the plots below in Figures 14-16 display the uplift force at each reaction, with empty bubbles representing positive uplift forces and solid bubbles downward forces, for both the north and east wind directions.

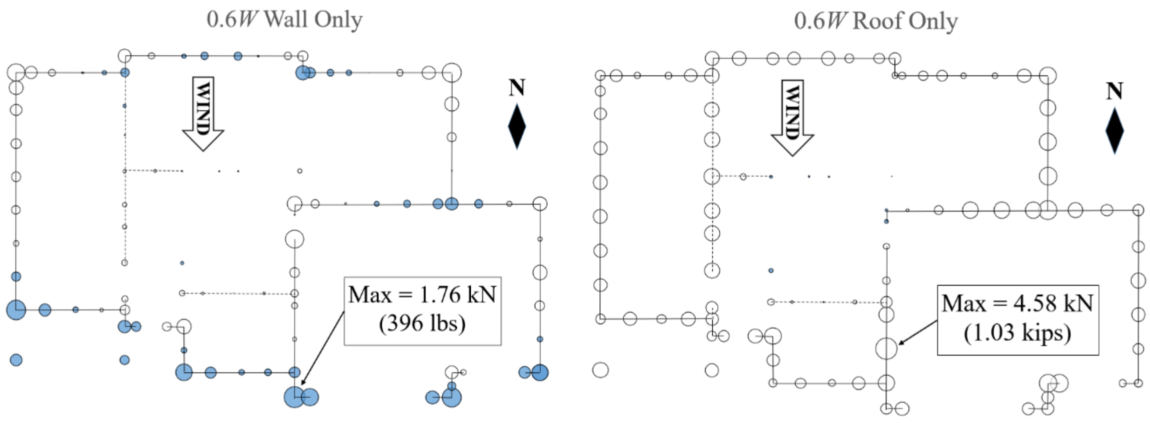


Figure 14: On the left is a plot of reactions from wind loads only applied on the walls, and not on the roof. On the right is a plot of reactions from wind loads only applied on the roof. Wind loading is using positive internal pressure and condition 1 and is from the north.

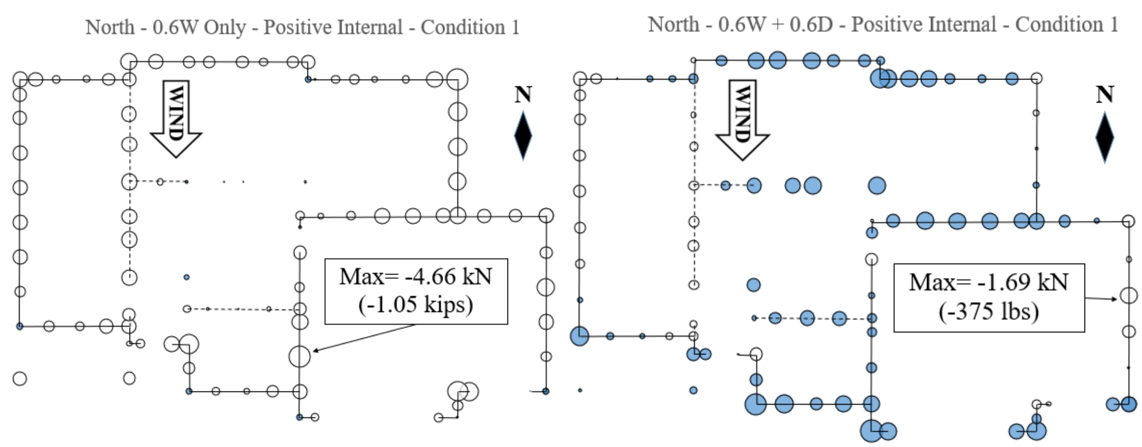


Figure 15: On the left is a plot of the reactions from both wall and roof wind loading using the positive internal pressure, condition 1 loading scenario from the north. On the right is the 0.6*D* + 0.6*W* case.

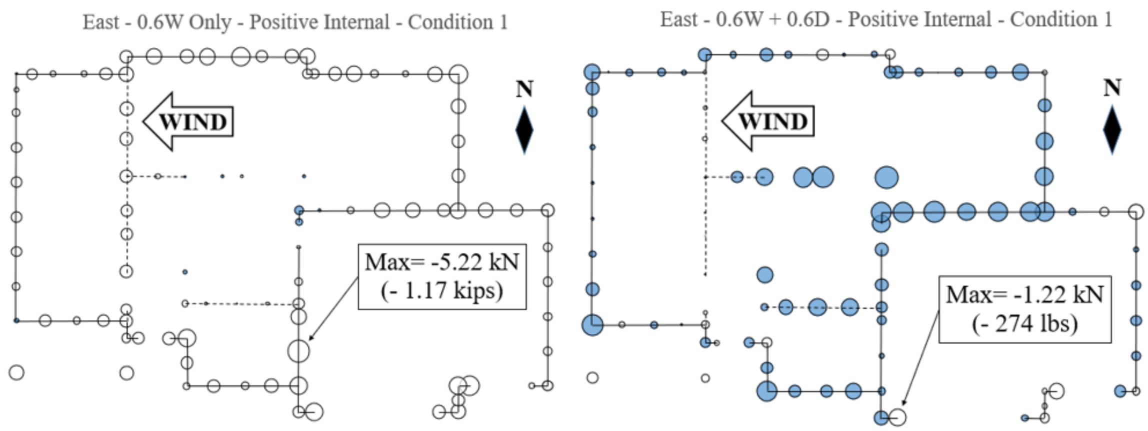


Figure 16: On the left is a plot of the total reactions from both wall and roof wind loading using the positive internal pressure, condition 1 loading scenario from the east. On the right is the 0.6*D* + 0.6*W* case.

From the plots in Figures 14-16, certain general patterns can be seen, as well as load concentrations specific to this structure. Also, when combined with dead loads, the resulting uplifts look significantly different.

Looking at the wind load-only plots, it can be seen that:

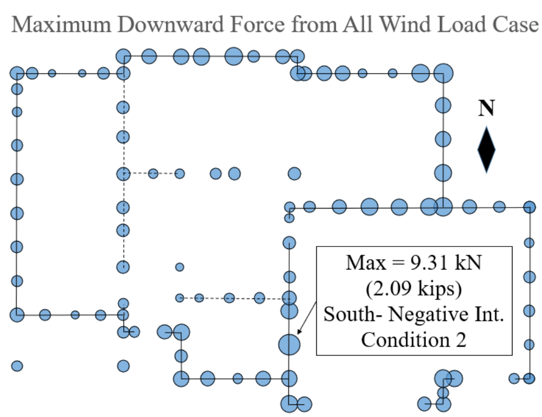
1) For walls that are parallel to the wind direction, uplift tends to be higher on the windward corners. This is due to overturning moments which result in uplift reactions closest to the windward corner, and downward reactions closer to the leeward corner. This is largely the result of overturning moments created by wind pressure on the walls as can be seen in Figure 15. The uplift from roof pressure is generally less at the corners; however to counterbalance the overturning moments from wind pressure on the walls, uplift is produced at the windward corners, and downward forces at the leeward corner, with reaction forces varying between them. Note that the wind pressures on the walls always result in zero net uplift.

2) Uplift is reduced under windows, and increased around doorways. This can be seen most prominently at the corners of the garage (Figure 16).

Due to the floor rafters running north and south, the incorporation of the dead load in the 0.6*W* + 0.6*D* plots has the greatest influence on the north and south exterior walls under the second story, as well as the north garage wall. The east- and west-most exterior walls carry less dead load resulting in higher net uplift. The western interior wall (wall line 2), which is under the second story and thus carries greater dead loads, also experiences net uplift. This is due to this particular wall experiencing uplift both from the upper roof to the east of it, as well as the lower roof to the west of it.

One particular reaction stands out in the wind load-only plots. This reaction located in the southern part of the west garage wall (see the maximum uplift note for the wind load-only plots in Figures 14 and 16), is the result of uplift forces from both the upper roof, as well as the truss spanning east to west over the top of the garage, who's loads predominantly flow down to that reaction as well. This large tributary area results in both high uplift from wind loads, but also greater dead loads, making the net uplift using the $0.6W + 0.6D$ load combination minimal.

The $0.6W + 0.6D$ and $0.6W + 1.0D$ load combinations can be applied using all four wind load cases, with wind in all four directions to result in the maximum and minimum plots shown in Figure 17. These plots show the absolute maximum downward force at each reaction from all 32 possible cases, and the maximum uplift from all 32 cases (Note: not all reactions have an uplift force for their minimum reaction).



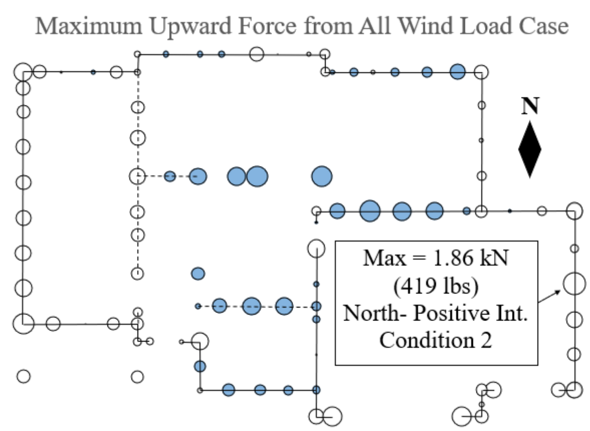


Figure 17: Left: Absolute maximum downward force at each reaction considering all 32 wind and dead loading cases. Right: Absolute maximum upward force at each reaction considering all 32 loading cases and both wind load combinations (note: not all reactions experience uplift). For both plots, solid bubbles indicate a downward force, while empty bubbles indicate an uplift force

As can be seen in Figure 17 above, corners and ends of walls tend to have the highest uplift reactions, as well as around openings such as the garage door. This is consistent with the findings of Pfretzschner et al. (2014). The maximum downward reactions are concentrated around the exterior walls under the second story, and the garage wall, due to the increased dead loads on these walls. Also note that the maximum downward force is significantly less than the maximum downward force seen for the gravity loads only in Figure 12, thus gravity loads control in downward forces.

Lateral Loads

The base shear created by wind loads is shown in Table 8. For lateral loading only, the $0.6W + 0.6D$ load case was investigated. Conversely to uplift loads, the greatest total base shear generally occurs for wind directions perpendicular to the longer side of the building and under condition 2 loading. Additionally, the greatest base shear occurs when the side of the house with the greater wall area compared to roof area is considered to be the windward wall. This means for this model the wind from the north with negative internal pressure and condition 2 loading results in the greatest base shear (see Table 8).

Table 8: Total base shear from 0.6*W* + 0.6*D* load case

Direction	Positive Internal – Cond. 1 (kN)	Positive Internal – Cond. 2 (kN)	Negative Internal – Cond. 1 (kN)	Negative Internal – Cond. 2 (kN)
North	50.4	61.9	51.7	**63.2**
South	46.3	60.0	45.0	58.7
West	32.2	44.4	32.6	44.8
East	39.3	48.0	38.9	47.3

Base shear is divided among up into six wall lines for the north and south wind load cases, and 4 wall lines for east and west wind load cases, as seen in Figures 17 and 18. Base shear for the negative internal pressure and condition 2 loading case for winds from both the north and south can be seen in Figure 18. Shear along walls 1, 2, 4 and 6 remained relatively constant between the north and south loadings, however, for the northern wind wall 5 experiences greater base shear, and for the southern wind wall 3 carries more of the base shear. This is because, for cases where negative internal pressure is used, and for walls that do not go extend through the entire building, such as walls 3 and 5, the shear will be higher when the windward wall is on their side of the building. The effect for walls that go from the windward wall to the leeward wall is significantly less.

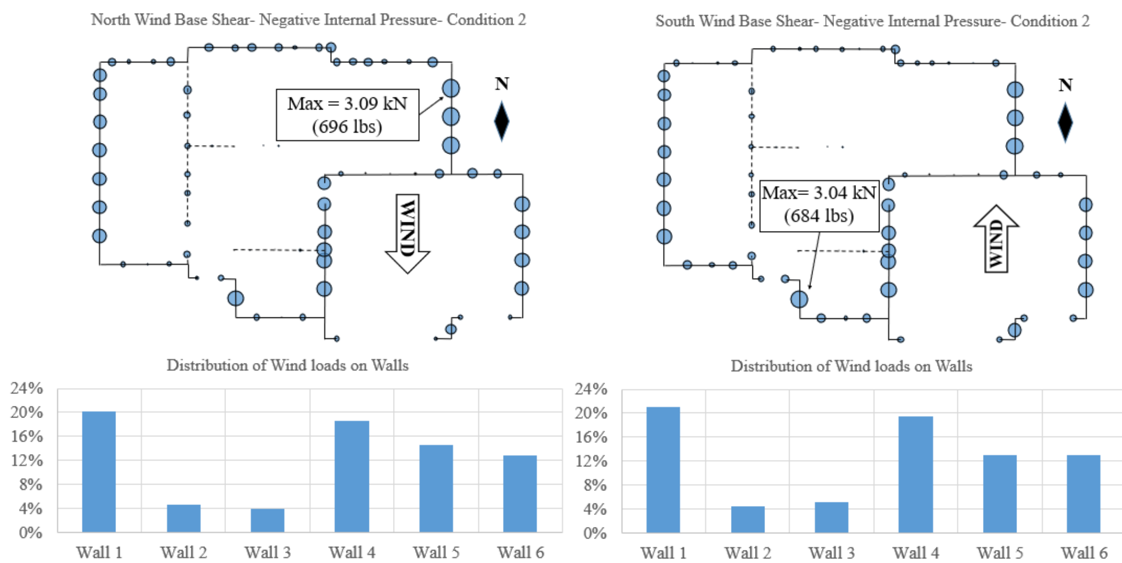


Figure 18: (Left): Base shear from northern wind. (Right): Base shear from southern wind

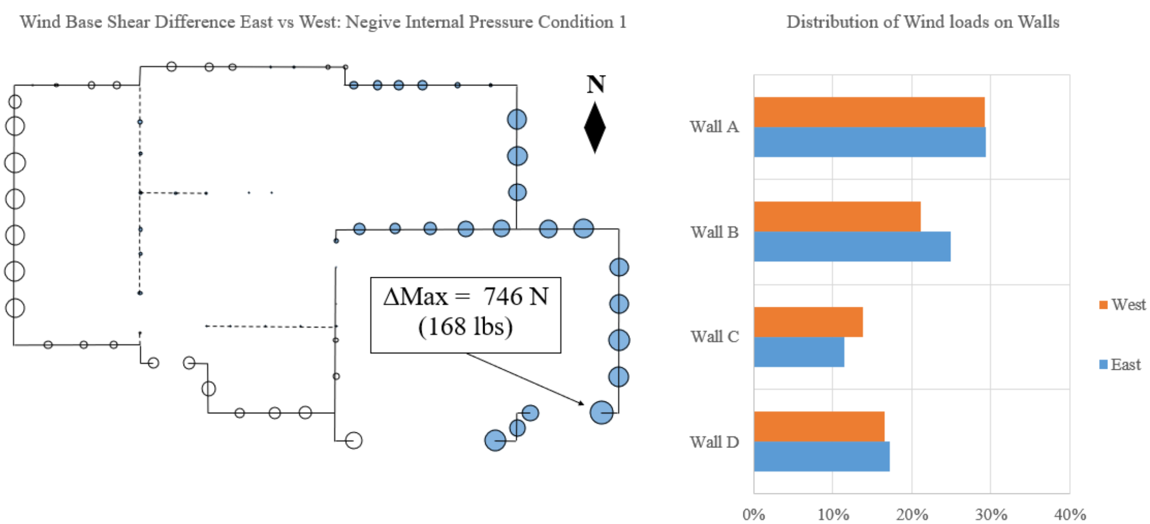


Figure 19: Comparison of base shear for an eastern vs. western wind. Positive loads indicate a higher base shear from an eastern wind, while negative loads indicate a higher base shear from a western wind.

The east and the west wind-induced base shears show little difference Δ along any of the four wall lines. The northern garage wall (wall line B) shows the largest difference since the wall is not continuous through the building as are most of the other wall lines. In addition, since positive internal pressure produced the controlling load, the wind loads are more balanced on the windward and leeward walls, which balances the wind shear distribution.

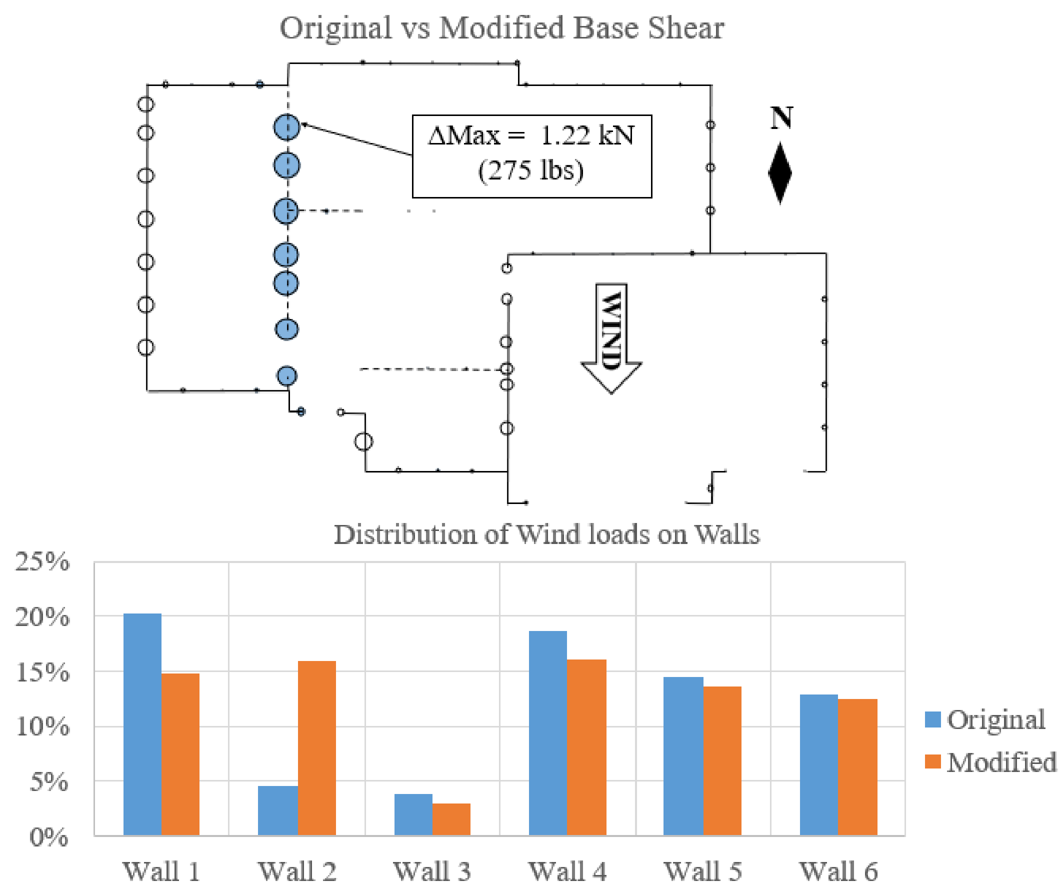


Figure 20: Comparison of the base shear in the original structure vs. the modified structure with an additional shear wall. The positive values indicate an increase in base shear in the modified structure compared to the original structure, while the negative values indicate a decrease in the base shear.

To investigate the changes in base shear flow, an additional shear wall was added at Wall 2 for just this portion of the study. Despite being an interior wall, this wall is now assumed to have the same material properties as the other shear walls, as described in the Materials and Methods section. Figure 20 shows a comparison of the base shear distribution for the original model of the house without sheathing for Wall 2, and the modified model with sheathing for Wall 2. Unsurprisingly, Wall 2 sees the greatest increase in the percentage of the base shear it carries. The walls closest to Wall 2, particularly Walls 1 and 3, experience as much as a 25% reduction in the base shear they are carrying. Walls 4, 5 and 6 experience some reduction in their base shear as well, but the further from Wall 2, the less of a reduction they receive.

Roof Shell Stresses

Von Mises stresses produced in the roof shell are shown in Figure 21. It must be noted that due to the OSSC (2014) codified and simplified method being used to represent wind loads in this study, with wind pressures uniform over each surface, the results will not reflect the some of the roof stress concentrations actually found under severe wind loading conditions, where wind pressure concentrations are seen at the edges and corners of the roof. However, these stress plots do reveal behavior about the roof and the locations where stresses concentrate even under more uniform loading. For both the northern wind and eastern wind plots, positive internal pressure with condition 1 loading was used with the 0.6 D + 0.6 W load combination. The northern wind plot reveals one very bright spot on the eastern slope of the upper roof. This stress concentration is due to the sudden increase in stiffness provided by Truss 2 in Figures 4 and 5 that extends up the slope there. For the eastern wind, there is no single spot of intense stress concentration, however, there are several locations that see large stresses. On the upper roof of the northern side of the building, two areas of increased stress can be seen. These occur in an area where the trusses are getting longer, but do not yet extend to the center ridge. But, why do these concentrations not occur on the south side of the roof also? For the western stress concentration, this is due to the asymmetry of the roof. A portion of the southwest part of structure and roof protrudes, which changes the stress concentrations for the entire side of the roof. For the eastern hotspot, on the east side of the roof, several of the trusses before the center ridge do not span the full length of the roof, but rather are broken in half, with each half resting on the wall that also supports the truss going up the eastern slope. This along with the influence of the protrusion in the roof is what creates the asymmetry in stress concentrations.

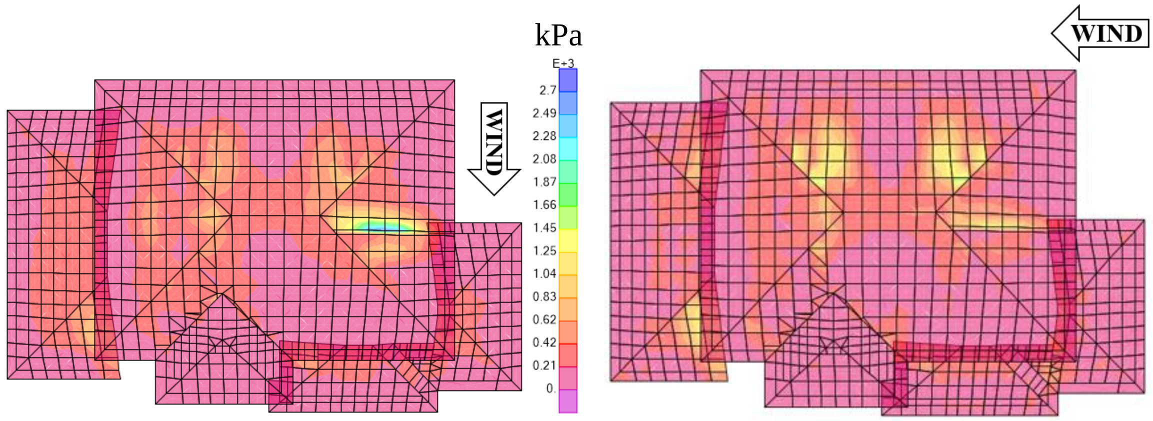


Figure 21: Von Mises Stress contour maps for a northern wind (Left) and Eastern wind (Right). For both, positive internal pressure and condition 1 were used.

Conclusions:

Following the linear-elastic modeling methods developed using SAP 2000, and verified with physical testing by Martin et al. (2011) and Pfretzschner et al. (2014) for light-framed wood structures, an actual light-frame wood residential structure with considerable geometric complexity was analyzed and load paths in the structure examined for both gravity and wind loads.

Three different load path investigations were performed to observe the effects of gravity loads and wind uplift and lateral loads. Conclusions drawn from these three load path investigations are as follows:

1. Gravity load paths in the structure revealed some trends as to which load combinations govern the design of different elements in the structure. The $D + 0.75L + 0.75S$ load combination governs most of the reactions, while interior reactions under the second floor tend to be governed by the $D + L$ combination (since no snow load is supported), and many of the reactions under the exterior walls are governed by the $D + S$ combinations. (See Figure 12)

2. The uplift created by the wind loads varied both locally at each reaction, as well as globally, for the net uplift on the structure, based on wind direction and loading case. The greatest uplift is generally concentrated at the corners of the structure, as well as around doorways and openings. Conversely, the least uplift is seen under windows, and on interior walls that do not have a direct load path from the roof.
3. The base shear created by wind loads is also influenced by wind direction and load case. For walls that do not extend through the entire structure, the greatest loadings occur when they are connected to the windward wall.
4. When an additional shear wall is added as an interior wall, the base shear redistributes with the walls closest to the new shear wall seeing the greatest reduction in load; however, walls further away do still see a reduction in their base shear reactions.
5. Under both snow and wind loadings, the truss extending up the slope of the eastern second floor roof creates the highest stress concentrations in the roof sheathing, due to its stiffness compared to the rafters beside it. From this it can be seen that anomalies in the structure, and particularly the roof structure, can cause stress concentrations.
6. The model generally behaved in a similar manner to those simpler house models created by Martin et al. (2011) and Pfretzschner et al. (2014), and the results validate and further develop several conclusions from their research, particularly in regards to stress concentrations around openings and at corners.

In future studies, more refined and accurate methods of modeling wind forces in ASCE 7 can be used to further investigate load paths and the effects of geometric irregularities and wind pressure concentrations.

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CONCLUSIONS AND RECOMMENDATIONS

Understanding load paths in light-framed wood houses and understanding the impact of geometric features particularly in regards to wind loading is important in helping to update existing building codes to be more effective and to guide the design of specific buildings. In Martin et al. (2011) and Pfretzschner et al. (2014), linear-elastic modeling methods for simple light-framed wood buildings were developed. In this study, those methods were applied to investigate wind and gravity load paths in an existing residential structure of greater geometric complexity.

The gravity load path investigation included comparisons of three main load combinations: $D + L$, $D + S$, and $D + 0.75L + 0.75S$, then determined which controlled at each anchor both in the foundation. The $D + S$ load combination controlled at most anchor bolts not located under the second story, and $D + L$ controlled for several locations not under an exterior wall. The remaining anchors were controlled by the $D + 0.75L + 0.75S$ combination.

Wind loadings were investigated using the all-heights method found in the 2014 Oregon Structural Specialty Code (OSSC). Due to the geometric complexity of the structure, the wind loads were applied from all four orthogonal directions. The first part of the investigation involved uplift load paths. The wind load combinations examined were: $0.6W$ only, to evaluate only the flow of wind loads in a structure, then $0.6W + 0.6D$, and $0.6W + 1.0D$ as realistic load cases. It was seen that the wind direction and wind loading case influenced both uplift at particular anchors and hold-downs, as well as the total global uplift force. However, regardless of direction, some patterns as to where loads concentrated emerged. When looking at the $0.6W$ only load case, higher uplift forces around corners, doorways, and other large openings, as well as less uplift under windows were observed. While many of these patterns remained when dead loads were introduced, anchor bolts under the second story, particularly on the north and south sides of the structure saw less uplift, due to the flow of the dead loads.

The second part of the wind investigation involved examining the base shear created by the wind loadings. In this case as well, both the base shear at individual anchor bolts, as well as the global base shear were dependent on wind direction and load case. It was discovered, however, that the wind load case that created the greatest base shear in each direction was for negative internal pressure and load case 2 on the roof. In this loading case, walls that do not extend fully through the structure in the direction of the wind tend to see higher base shear if they are on the windward side of the house compared to being on the leeward side. Additionally, to examine the effects of adding another shear wall, sheathing was added to an interior wall, and the base shear from the modified structure was compared to the original structure. The addition of a shear wall most impacted the base shear of the wall lines closest to it, however, the addition also slightly decreased the base shear of even wall lines further away.

Finally, an analysis of the roof was conducted to evaluate Von Mises stress concentration points. Under the simulated wind pressure conditions, stresses were largest on the upper portion of the roof, particularly at the ridge. In addition, likely due to the asymmetry of the roof, the northern side of the roof experienced higher stress concentrations from E-W winds than did the south side. The greatest stress concentration however was observed on the eastern slope of the roof near a truss that runs east to west. This is the only truss in this part of the roof structure to run in this direction, and the additional stiffness it provides compared to the surrounding rafters creates the stress concentration.

There are still many aspects of both this structure, as well as with investigating load paths in light-frame residential structures in general, that could be subject to future research. For this model, wind loadings from different angles of attack, as well as using more refined methods from ASCE 7, could be used. The use and development of more comprehensive wind analysis procedures, particularly for buildings with irregular geometry, is essential to more accurately model wind load paths. Additionally, the investigation of seismic loads could be also be investigated with future models. Finally, the development of modeling methods that allow for the investigation of structures using multiple structural materials, particularly concrete in addition to wood, could be useful in evaluating residential structures with such as this one with a daylight basement under wind and seismic loading.

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APPENDIX A:

EXTENDED LITERATURE REVIEW

Full Building Behavior

For evaluating the response of a given structure, the most accurate method of evaluating and understanding its behavior under various loading conditions is to perform physical testing on full-scale models, or to a lesser extent, scaled-down physical models. However, since this can be time consuming and cost prohibitive, computer modeling of these structures can provide an efficient alternative. First, due to the highly indeterminate nature of light-framed wood structures, proper modeling methods must be established. This requires the verification of the models and procedures against physical testing data. For this study, the modeling methods were primarily established by Martin et al. (2011) and Pfretzschner et al. (2014) whose models and loading scenarios were based upon wind tunnel testing and physical testing of a 1/3rd scale model, conducted by the University of Florida, and scale model testing conducted by Paevere et al. (2003). Malone (2013) then applied the methods used in these previous studies to investigate an existing timber frame house located in Vermont, and a theoretical light-frame version of the same structure.

Sub-assemblies

Members

As with all three previous studies noted above, the material properties for all members were determined using industry standards. For places where the exact member size and grade were not known, they had to be assumed. The joint connections, particularly in roof trusses, in reality have some level of partial fixity, and several studies have been conducted considering them as such; however, Limkatanyoo (2003) offered the simplest solution and that is to consider each joint either rigid or pinned, and led to good results. Due to the simplicity and adequate accuracy, this was the method adopted by Martin et al. (2011), Pfretzschner et al. (2014), and Malone (2013).

Shear walls

In light-frame wood structures, shear walls transfer lateral loads between diaphragms and the ground. Typically, shear walls consist of either oriented strand board (OSB) or plywood, and possibly with gypsum wall board as well. However, unlike for frame members with widely available material properties, there is more complexity in determining the stiffness of a shear wall.

Doudak (2005) conducted several physical tests on shear walls to determine the effects of openings. Using SAP 2000, Doudak (2005) then modeled the walls using a complex procedure to model each fastener. Since this is generally an impractical method for modeling all the shear walls in an entire structure, Martin et al. (2011) examined an equivalent energy method developed by Kasal and Leichti (1992), then did a corollary study to determine the effects of nail spacing, comparing results to studies on physical shear walls conducted by Sinha (2007), Langlois (2002), and Lebeda (2002). Finally, since the study conducted by Martin (2010) only included plywood sheathing modeled as a thick shell element, Pfrezschner et al. (2014) modeled shear walls using a layered shell element that allows for both the sheathing and the GWB to be modeled as part of one element. A similar technique was used in this study as well.

Roof

Roof sub-systems have often in the past been modeled as a system rather than having each member modeled. Kasal (1992), Collins (2005 a,b) and Doudak (2005) all modeled their roofing systems as large elements, which neglects the individual stiffness of each truss, and leads to inaccurate load distributions. This method becomes impossible with increasingly complex roof systems, so would not be applicable for this current study. However, Martin et al. (2011) and subsequent OSU studies modeled each individual truss member, and while more time consuming, it has produced results with greater accuracy while modeling complex roofing systems.

Modeling Methods Verification

Accurately modeling wind loads can be difficult, and there is no single method. The three main approaches involve either using results from wind tunnel testing, physical testing, or using codified methods.

Most codified wind load analysis base their wind pressures on three-second gust wind speeds. However, due to their destructive nature, and the difficulty in measuring wind speeds, research continues to refine our understanding of hurricanes and tornadoes, and the design wind speeds and pressure distributions that should be used in design. Lombardo et al. (2015) investigated the Joplin tornado and compared the wind speed estimates based on tree fall patterns to the Enhanced Fujita (EF) scale, which estimates the intensity of the tornado based on the severity of building damage, finding that the tree fall results showed much higher wind speeds than previously predicted. For design purposes, most localities in hurricane regions use hurricane wind speeds produced by a Monte Carlo simulations developed by Vickery et al (2009).

In 1997 the Florida Coastal Monitoring Program was started to gather wind pressure data on full-scale, low-rise buildings mainly located in Florida, as well as a few in the Carolinas. These houses allow pressure transducers to be mounted to the wall and roofs immediately before a hurricane hits, and record real-time pressures on actual houses with complex geometries. Liu et al. (2009) compared results from several of these structures to wind tunnel testing and found that the ASCE 7-05 wind provisions underestimate, sometimes dramatically, peak uplift pressures.

The research conducted by Martin et al. (2011) was based on wind tunnel and physical testing at the University of Florida by Datin (2009). The wind load testing involved simulated hurricane winds on a 1:50 scale model of the building, and from these results Martin et al. (2011) was able to examine three critical loading scenarios: 1) the maximum uplift at a corner, 2) the maximum uplift on the ridge, and 3) the global maximum uplift at all points. The physical testing was done on a one-third scale model that Martin et al. (2011) then modeled as a full-scale building. Datin (2009) used a grid of pneumatic actuators to simulate wind loads on the roof and load cells to measure roof-to-wall and wall-to-foundation loads. Martin et al. (2011) then compared these loads to results from a SAP 2000 model to validate the overall behavior.

Pfretzschner et al. (2014) used similar modeling methods as Martin et al. (2011), but based the model on physical testing done by Paevere et al. (2003). Paevere et al. (2003) conducted static and cyclical physical testing on a relatively simple, full-scale, L-shaped structure which included interior walls, large openings and windows, increasing the geometric complexity from the rectangular one modeled by Martin et al. (2011). Pfretzschner et al. (2014) then took the basic L-shaped design and investigated the effects of changing the length on one end of the wings of the "L". Both physical and model results showed that the effects of the re-entrant corner were dependent on the wind direction as well as the locations of in-plane shear walls.

Wind Damage

In recent years in the U.S., the annual wind and surge damage has been approximated to be $10 billion or more (Li et al. 2012), and as population increases, and more people move towards the coasts, this number will rise. Recent hurricanes such as Andrew in 1992, Katrina in 2005, and Harvey in 2016, and tornados such as the Bridge Creek-Moore Tornado in 1999 and the one in Joplin in 2011, only serve to underscore the need for more robust wind-related code provisions and construction details, and a better understanding of wind load paths in structures.

Keeping roof sheathing from being torn off is vitally important. Loss of roof sheathing can: 1) lead to interior property damage; 2) create greater internal pressures, as the house becomes only a partially enclosed structure; and 3) weaken the roof diaphragm for lateral load resistance. Various roof features can lead to more or less damaged roof sheathing or even structural damage in other locations. According to a report by the U.S. Department of Agriculture, upon inspecting damage caused by hurricane Camille (U.S. Department of Agriculture), there was significantly less damage to hip roofs than gable roofs, even for houses within the same neighborhood. Roof-to-wall connections are particularly a concern in houses with gable roofs, though are not as common of an issue in houses with hip roofs. Stevenson (2012) found from damage in several tornadoes that hip roofs, while seeing some damage from sheathing connections or roof-to-wall connections, also can experience more failures from framing members, leading to sheathing failures. Ahmad et al. (2002) investigated the impact of varying overhang ratios on hip roofs with a 30-degree pitch, finding that larger overhangs lead to higher overall pressure on the roof. Additionally, increasing the elevation of the overhangs leads to increased pressure as well.

Wind Load Investigation

Accurately investigating wind load paths can be very complex, and because of the strong impact geometry has on wind loadings, often requires wind tunnel testing to be truly accurate. Building codes, as noted above, can often under-estimate wind speeds and pressures, and do not offer much help with regards to structures with complex geometries. A possible solution to the geometry problem would be a design-assisted database (DAD). Proposed by Mensah (2010), this database would allow for a realistic and validated method of simulating wind loads for various building components. However, while very likely more accurate and conservative, there is still a significant amount of work to be done before such a database could be used in design for an actual complex roof structure.

For this study, the all-heights method from the 2014 OSSC was used due to its simplicity to apply to a complex structure. Using this method, a uniform wind pressure was applied to each wall or roof without any edge or corner load concentrations. This is in contrast to Pfretzchner et al. (2014) who used the ASCE 7-05 MFWRS method 1, and Malone (2013) who used the ASCE 7-05 MFWRS envelope procedure. However, like

Pfretzschner et al. (2014), to deal with the geometric complexity of the structure, this study also used an example in Mehta and Coulbourn (2010) as a basis for wind load investigations, and considered wind loading for all four orthogonal directions. Pfretzschner et al. (2014) also investigated the effects of corner wind loads, particularly looking at wind blowing into the building’s re-entrant corner. Jameel et al. (2015) concluded that maximum wind pressures typically occur when wind comes from an angle of incidence between 15- and 75-degrees with the corner of the structure, depending on the structure’s geometry. However, while only wind loadings orthogonal to the structure were investigated in this study, wind loadings from other angles could be investigated in future studies.

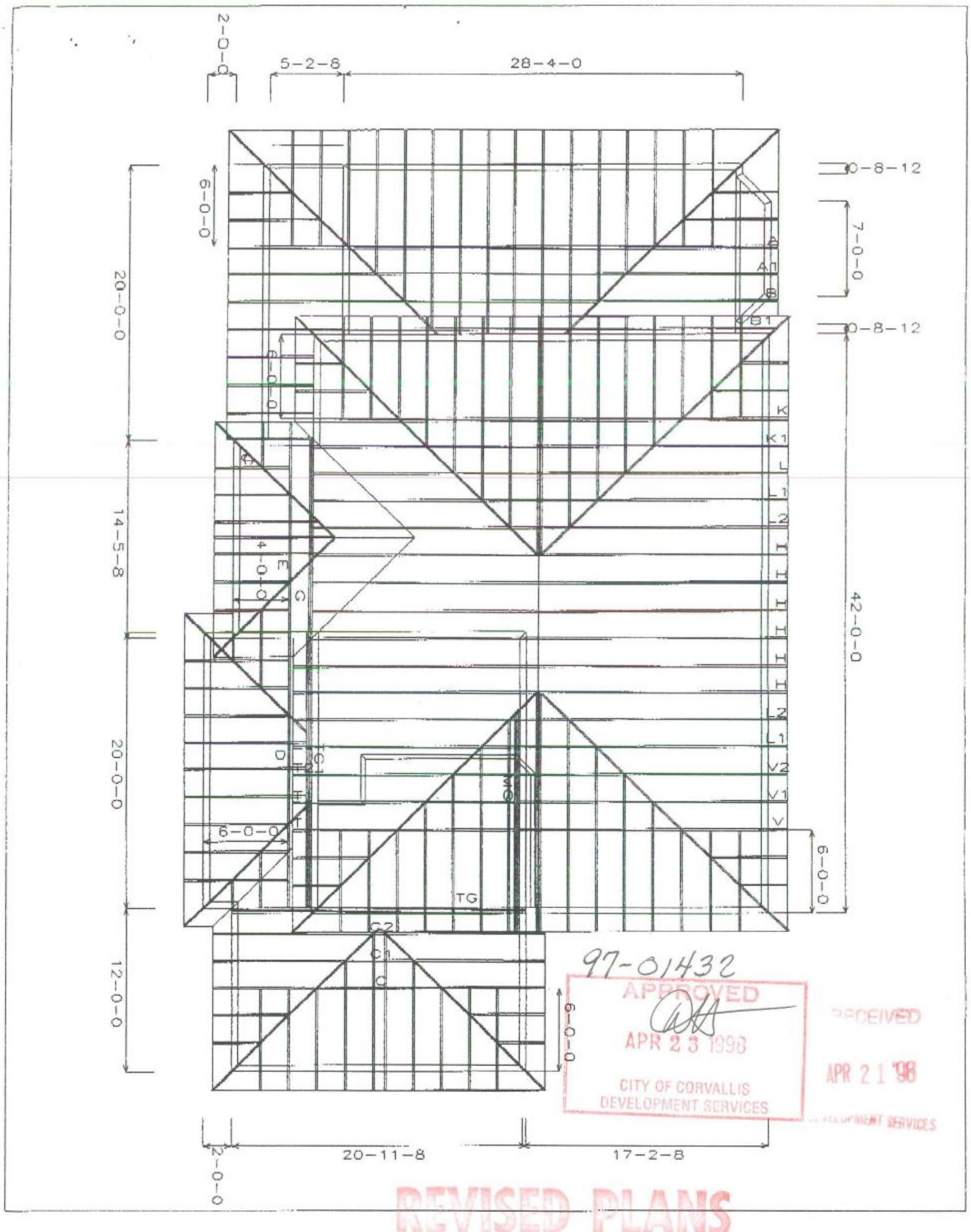
Gravity Loads

In addition to wind loading scenarios, Malone (2013) investigated several gravity load cases. In addition to dead loads, which had been modeled using material weights in each previous study at OSU, Malone (2013) investigated load paths including snow and live loads. Like Malone (2013), in this current study snow and live loads were applied directly to the roof and floor sheathing elements respectively. In addition, in this current study, uniform dead loads were applied to the sheathing elements to account for additional dead load from architectural elements, something that was not done in previous studies.

APPENDIX B:

ORIGINAL BUILDING PLANS

APR-21-'98 07:37 ID:RELCO ROOF/FLOOR INC TEL NO:503 995-6311 #156 P04



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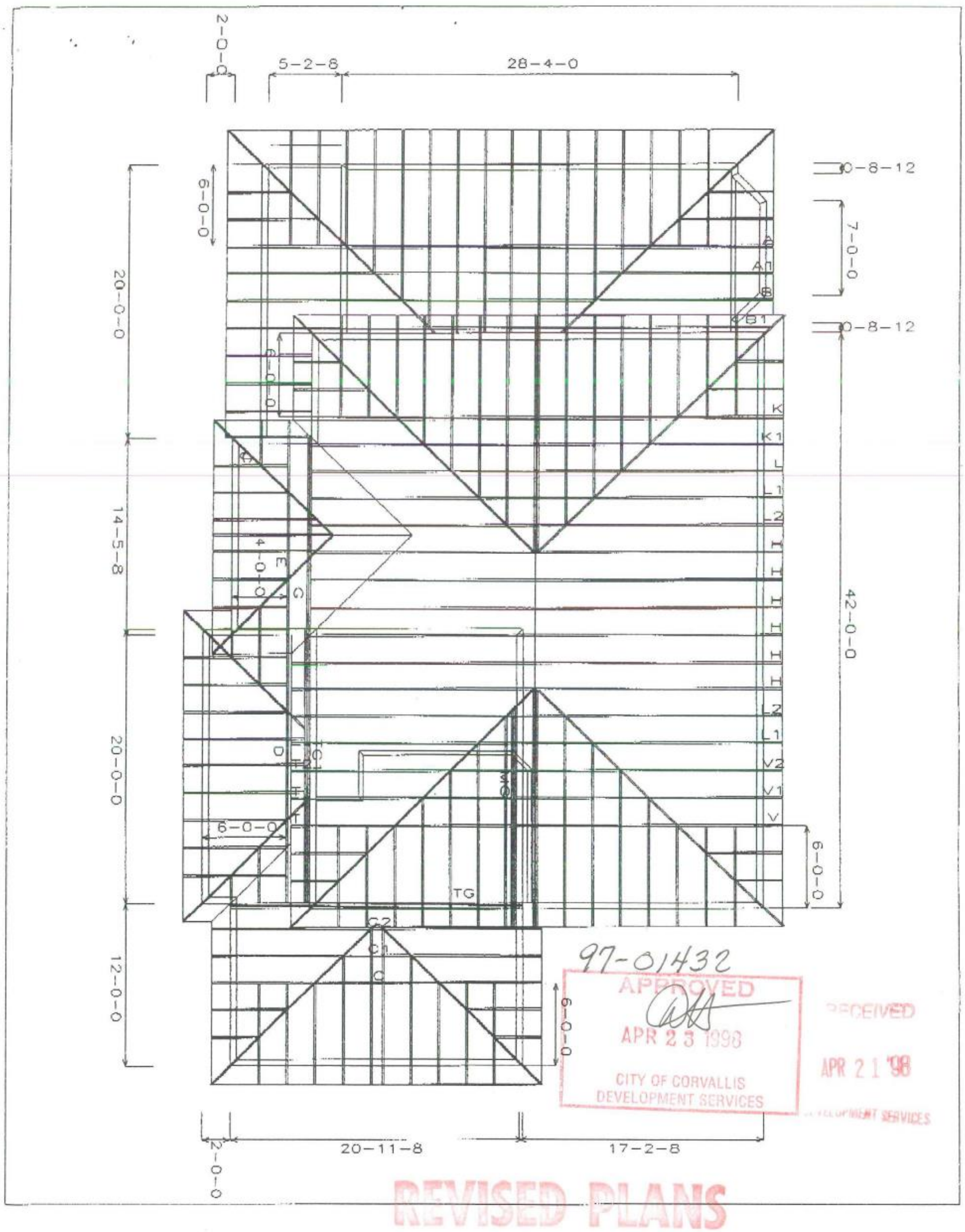


Crescent Valley Builders, Inc. 4685 Arrowood

===<<<<ACES-32 Ver. 1.1>>>>===========[951528]============<<<<RELCO>>>>==========

Customer : C-V-B
Project #: 351
Span : 34-6

Truss ID : A
Quantity : 2

Thu Apr 9 17:03:25 1998
Family # : 324
Top Pitch : 6/12

==Bld(2/12/1998),v1.1===

TOP CHORD	BOTTOM CHORD	WEBS		REACTIONS	SIZE
1-2= 2764	7-8= 3495	2-12=-4635	5-9=-587	12=-4606	5.50
2-3=-1595	8-9= 3540	2-11= 5841	6-9= 1901	7=-2276	5.50
3-4=-4588	9-10= 5290	3-11=-2128	6-8= 161	1= 727	3.50
4-5=-4588	10-11= 1595	3-10= 3255			
5-6=-5290	11-12=-3781	4-10=-817			
6-7=-3930	12-1=-2481	5-10=-763			

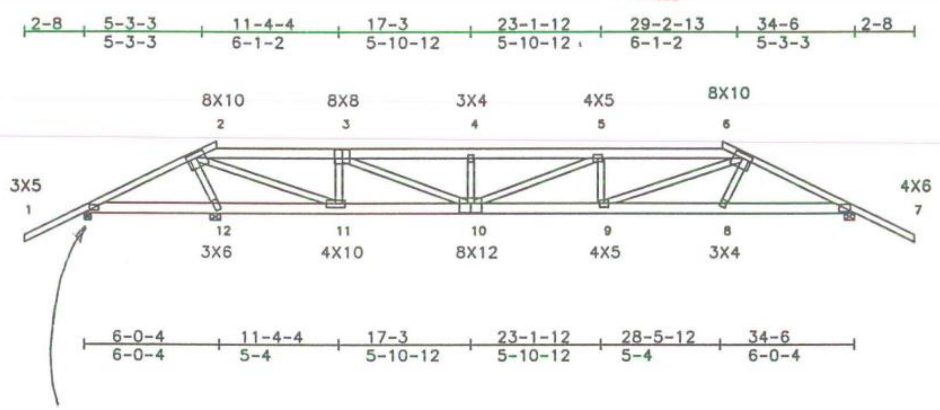
APR 13 1998

PROFESSIONAL ENGINEER
ENGINEER
15,766
EXP 6-30-98
OREGON
JAN. 25, 1992
REDONGYU

PLATE OFFSETS (X=LEFT,Y=TOP):[j2=5,2.5],[j3=4,5],[j6=5,2.5],[j10=6,3],[j12=4,1.5],(SYMMETRIC PLATING TURNED ON)

RECEIVED
APR 20 '98
DEVELOPMENT SERVICES

BRACE FLAT PART OF T/C AT 48 IN O.C. FROM JTS 2 TO 6.



PROVIDE ANCHORAGE AT BRG JT. 1 FOR 800 LBS. UPLIFT.

L. HL TO PK:5-10-10
LEFT HEIGHT:0-4-3

SPAN:34-6

RISE:3-3-11

R. HL TO PK :5-10-10
RIGHT HEIGHT:0-4-3

==

LOADING (PSF)	L	D	MAX STRESSES	MINIMUM GRADE OF LUMBER
TOP	25	10	TOP 1-2=0.363	TOP CHORD:2X4 No.1&Btr GR DF-L
BOTT	0	7	BOTT 9-10=0.446	BOT CHORD:2X6 No.2 GR DF-L
			LL.DEFL.@9=0.15 < L/240	WEBS :2X4 STANDARD 91 HF

==

STR.INC.: LUMB = 1.15 PLATE = 1.15
REPETITIVE STRESSES NOT USED

SPACING : 24.0 in. o. c.
NO. OF MEMBERS = 2

LOADING	STRESS INCREASE LUMBER	STRESS INCREASE PLATE	LOADING TYPE	PANEL(PLF) / JOINTS(LBS)			
1	1.15	1.15	UNIFORM	1- 2= 70	2- 6= 139	6- 7= 70	7- 1= 28
			CONCENTRATED	2= 358	6= 358		

APPROVED
APR 28 1998
CITY OF CORVALLIS
DEVELOPMENT SERVICES

CHORD: 2-3; 3-4; 4-5; 5-6 TO BE 2X6 No.2 GR DF-L
WEB: 2-11 TO BE 2X4 No.1&Btr GR DF-L
2 MEMBERS NAILED TOG. W/1 ROW(S) OF .131x3 in. NAILS 11 in. o.c.(TOP CHS.),AND 1 ROW(S) OF .131x3 in. NAILS 12 in. o.c(BOTT. CHS.)
For Webs use 1 ROW of NAILS 12 in. o.c.
DEFLECTION(IN.) L.L= 0.15,D.L=0.10,T.L=0.24
PLATES ARE MITEK M20-165,142 MANUFACTURED FROM ASTM A 446 GRD A GALVANIZED STEEL(EXCEPT AS SHOWN)
PLATE MUST BE INSTALLED ON EA. FACE OF JOINT,SYMMETRICALLY(EXCEPT AS SHOWN)DESIGN CONFORMS W/NDS DESIGN SPECS, UBC-ICBO,TPI-91
THIS DESIGN IS FOR TRUSS FABRICATION ONLY.FOR PERMANENT AND TEMPORARY BRACING(WHICH IS ALWAYS REQD)CONSULT BLDG ARCHITECT OR ENGINEER

REVISED PLANS 97-01432

===<<<<ACES-32 Ver. 1.1>>>>==========[951531]===========<<<<RELCO>>>>=========

Customer : C-V-B
Project #: 351
Span : 34-6

Truss ID : B
Quantity : 1

Thu Apr 9 17:03:42 1998
Family # : 324
Top Pitch : 6/12

==Bld(2/12/1998),v1.1==

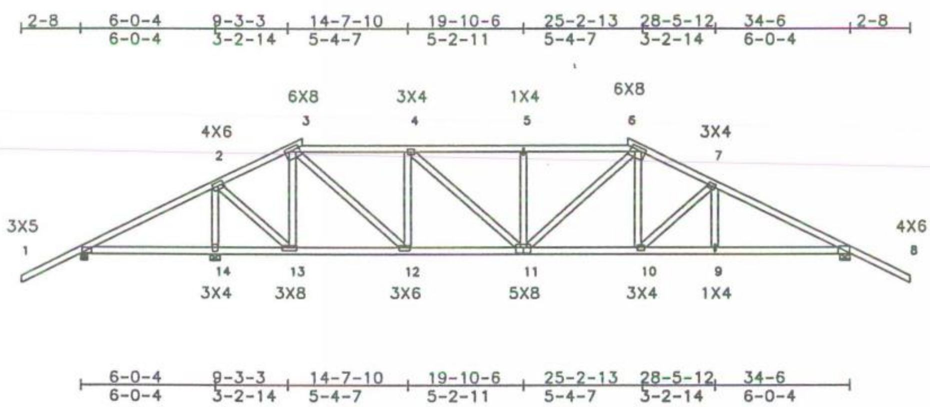
TOP CHORD		BOTTOM CHORD		WEBS		REACTIONS - SIZE	
1-2= 547	7-8=-1912	8-9= 1692	14-1=-488	2-14=-1674	5-11=-371	14=-1739	5.50
2-3=-499		9-10= 1692		2-13= 1282	6-11= 227	8=-1358	5.50
3-4=-1295		10-11= 1454		3-13=-818	6-10= 284	1=-230	3.50
4-5=-1625		11-12= 1295		3-12= 1127	7-10=-327		
5-6=-1625		12-13= 445		4-12=-667	7-9= 63		
6-7=-1629		13-14=-488		4-11= 443			

APR 13 1998

PROFESSIONAL ENGINEER
ENGINEER
5,706
EXP
OREGON
JAN. 25, 1992
REDONGYU

PLATE OFFSETS (X=LEFT,Y=TOP):[j3=4,2.5],[j6=4,2.5],[j11=4,2],(SYMMETRIC PLATING TURNED ON)

BRACE FLAT PART OF T/C AT 24 IN O.C. FROM JTS 3 TO 6.



L. HL TO PK:10-4-5
LEFT HEIGHT:0-4-3
SPAN:34-6
RISE:5-3-11
R. HL TO PK :10-4-5
RIGHT HEIGHT:0-4-3

==

LOADING	(PSF)		MAX STRESSES		MINIMUM GRADE OF LUMBER	
	L	D				
TOP	25	10	TOP	1-2=0.444	TOP CHORD:2X4	No.1&Btr GR DF-L
BOTT	0	7	BOTT	9-10=0.320	BOT CHORD:2X4	No.1&Btr GR DF-L
			LL.DEFL.@11=0.08 < L/240		WEBS :2X4	STANDARD 91 HF

==

STR.INC.: LUMB = 1.15 PLATE = 1.15
REPETITIVE STRESSES USED

SPACING : 24.0 in. o. c.
NO. OF MEMBERS = 1

DEFLECTION(IN.) L.L= 0.08,D.L=0.06,T.L=0.14
PLATES ARE MITEK M20-165,142 MANUFACTURED FROM ASTM A 446 GRD A GALVANIZED STEEL(EXCEPT AS SHOWN)
PLATE MUST BE INSTALLED ON EA. FACE OF JOINT,SYMMETRICALLY(EXCEPT AS SHOWN)DESIGN CONFORMS W/NDS DESIGN SPECS, UBC-ICBO,TPI-91
THIS DESIGN IS FOR TRUSS FABRICATION ONLY.FOR PERMANENT AND TEMPORARY BRACING(WHICH IS ALWAYS REQD)CONSULT BLDG ARCHITECT OR ENGINEER

APPROVED
APR 23 1998
CITY OF CORVALLIS
DEVELOPMENT SERVICES

```
===&lt;&lt;&lt;&lt;ACES-32 Ver. 1.1&gt;&gt;&gt;&gt;==========[ 951529 ]==========&lt;&lt;&lt;&lt;RELCO&gt;&gt;&gt;&gt;=========
  Customer  : C-V-B                                         Thu Apr  9 17:03:32 1998
  Project #: 351                 Truss ID   : A1            Family #   : 324
  Span      : 34-6               Quantity   : 1             Top Pitch  : 6/12
==Bld(2/12/1998),v1.1===================================================================
```

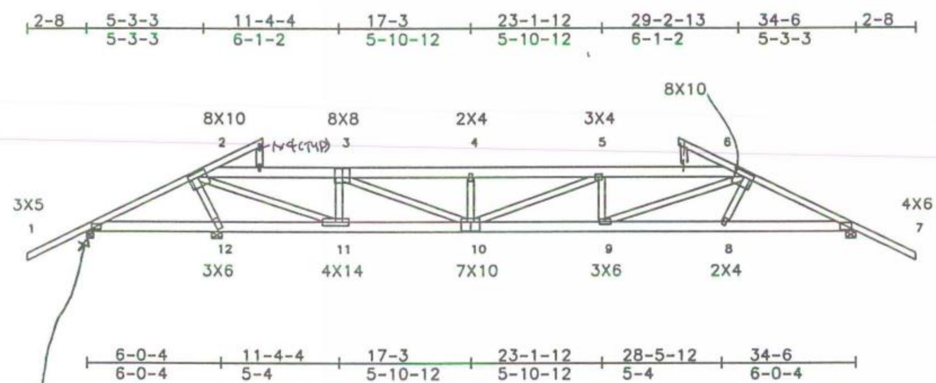
TOP CHORD	BOTTOM CHORD	WEBS		REACTIONS - SIZE	
1-2= 1384	7-8= 1623	2-12=-2221	5-9=-316	12=-2212	5.50
2-3=-804	8-9= 1646	2-11= 2899	6-9= 1010	7=-1275	5.50
3-4=-2268	9-10= 2575	3-11=-1055	6-8= 81	1= 160	3.50
4-5=-2268	10-11= 804	3-10= 1593			
5-6=-2576	11-12=-1865	4-10=-413			
6-7=-1825	12-1=-1242	5-10=-334			

APR 13 1998

PROFESSIONAL ENGINEER
ENGINEER
15,766
EXP 6/30/98
OREGON
JAN. 25, 1992
REDONGYU

PLATE OFFSETS (X=LEFT,Y=TOP):[j2=5,2.5],[j3=4,5],[j6=5,2.5],[j10=5,2],[j12=4,1.5],(SYMMETRIC PLATING TURNED ON)

BRACE FLAT PART OF T/C AT 24 IN O.C. FROM JTS 2 TO 6.



PROVIDE ANCHORAGE AT BRG JT. 1 FOR 200 LBS. UPLIFT.

```
L. HL TO PK:5-10-10                                           R. HL TO PK :5-10-10
LEFT HEIGHT:0-4-3          SPAN:34-6         RISE:4-3-11      RIGHT HEIGHT:0-4-3
===================================================================================
LOADING   (PSF)          MAX STRESSES                MINIMUM GRADE OF LUMBER
          L    D         TOP    1-2=0.487            TOP CHORD:2X4   No.1&Btr GR DF-L
TOP      25   10         BOTT   9-10=0.428           BOT CHORD:2X6   No.2 GR DF-L
BOTT      0    7         LL.DEFL.@10=0.14 < L/240    WEBS     :2X4   STANDARD 91 HF
===================================================================================
STR.INC.: LUMB = 1.15 PLATE = 1.15                   SPACING : 24.0 in. o. c.
REPETITIVE STRESSES USED                             NO. OF MEMBERS = 1
```

CHORD: 2-3; 3-4; 4-5; 5-6 TO BE 2X6 No.2 GR DF-L
WEB: 2-11 TO BE 2X4 No.1&Btr GR DF-L
DEFLECTION(IN.) L.L= 0.14,D.L=0.10,T.L=0.24
PLATES ARE MITEK M20-165,142 MANUFACTURED FROM ASTM A 446 GRD A GALVANIZED STEEL(EXCEPT AS SHOWN)
PLATE MUST BE INSTALLED ON EA. FACE OF JOINT,SYMMETRICALLY(EXCEPT AS SHOWN)DESIGN CONFORMS W/NDS DESIGN SPECS, UBC-ICBO,TPI-91
THIS DESIGN IS FOR TRUSS FABRICATION ONLY.FOR PERMANENT AND TEMPORARY BRACING(WHICH IS ALWAYS REQD)CONSULT BLDG ARCHITECT OR ENGINEER.

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APR 23 1998
CITY OF CORVALLIS
DEVELOPMENT SERVICES

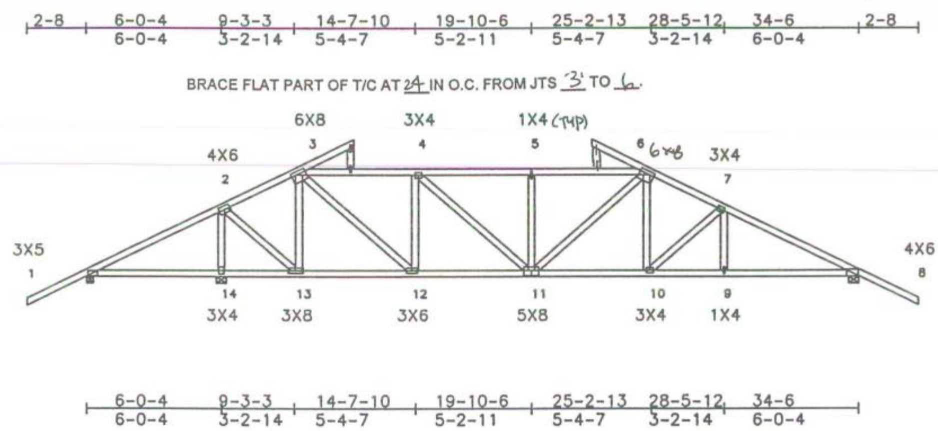
```
===<<<<ACES-32 Ver. 1.1>>>>==========[ 951532 ]===========<<<<RELCO>>>>=========
  Customer : C-V-B                                          Thu Apr  9 17:03:50 1998
  Project #: 351               Truss ID   : B1              Family #   : 324
  Span     : 34-6              Quantity   : 1               Top Pitch : 6/12
==Bld(2/12/1998),v1.1==============================================================
```

TOP CHORD		BOTTOM CHORD		WEBS		REACTIONS	SIZE
1-2= 547	7-8=-1912	8-9= 1692	14-1=-488	2-14=-1674	5-11=-371	14=-1739	5.50
2-3=-499		9-10= 1692		2-13= 1282	6-11= 227	8=-1358	5.50
3-4=-1295		10-11= 1454		3-13=-818	6-10= 284	1=-230	3.50
4-5=-1625		11-12= 1295		3-12= 1127	7-10=-327		
5-6=-1625		12-13= 445		4-12=-667	7-9= 63		
6-7=-1629		13-14=-488		4-11= 443			

APR 13 1998

PROFESSIONAL ENGINEER
ENGINEER
15966
EXP 6/30/98
OREGON
JAN. 25, 1992
REDONG YU

PLATE OFFSETS (X=LEFT,Y=TOP):[j3=4,2.5],[j6=4,2.5],[j11=4,2],(SYMMETRIC PLATING TURNED ON)




```
L. HL TO PK:10-4-5                                              R. HL TO PK :10-4-5
LEFT HEIGHT:0-4-3          SPAN:34-6           RISE:6-3-11      RIGHT HEIGHT:0-4-3
==================================================================================
LOADING    (PSF)          MAX STRESSES                MINIMUM GRADE OF LUMBER
          L     D         TOP    1-2=0.444            TOP CHORD:2X4  No.1&Btr GR DF-L
TOP      25    10         BOTT   9-10=0.320           BOT CHORD:2X4  No.1&Btr GR DF-L
BOTT      0     7         LL.DEFL.@11=0.08 < L/240    WEBS     :2X4  STANDARD 91 HF
==================================================================================
STR.INC.: LUMB = 1.15 PLATE = 1.15                    SPACING : 24.0 in. o. c.
REPETITIVE STRESSES USED                              NO. OF MEMBERS = 1
```

DEFLECTION(IN.) L.L= 0.08,D.L=0.06,T.L=0.14
PLATES ARE MITEK M20-165,142 MANUFACTURED FROM ASTM A 446 GRD A GALVANIZED STEEL(EXCEPT AS SHOWN)
PLATE MUST BE INSTALLED ON EA. FACE OF JOINT,SYMMETRICALLY(EXCEPT AS SHOWN)DESIGN CONFORMS W/NDS DESIGN SPECS, UBC-ICBO,TPI-91
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DEVELOPMENT SERVICES

```
===<<<<ACES-32 Ver. 1.1>>>>==========[ 951540 ]==========<<<<RELCO>>>>==========
  Customer : C-V-B                                          Thu Apr  9 17:04:32 1998
  Project #: 351                Truss ID   : K              Family #   : 324
  Span     : 32-3               Quantity   : 2              Top Pitch  : 6/12
==Bld(2/12/1998),v1.1=================================================================
```

TOP CHORD	BOTTOM CHORD	WEBS		REACTIONS - SIZE	
1-2=-5245	7-8= 4659	2-12= 146	5-9=-1197	1=-2767	5.50
2-3=-7509	8-9= 4659	2-11= 3153	6-9= 3153	7=-2767	5.50
3-4=-8424	9-10= 7509	3-11=-1197	6-8= 146		
4-5=-8424	10-11= 7509	3-10= 1017			
5-6=-7509	11-12= 4659	4-10=-741			
6-7=-5245	12-1= 4659	5-10= 1017			

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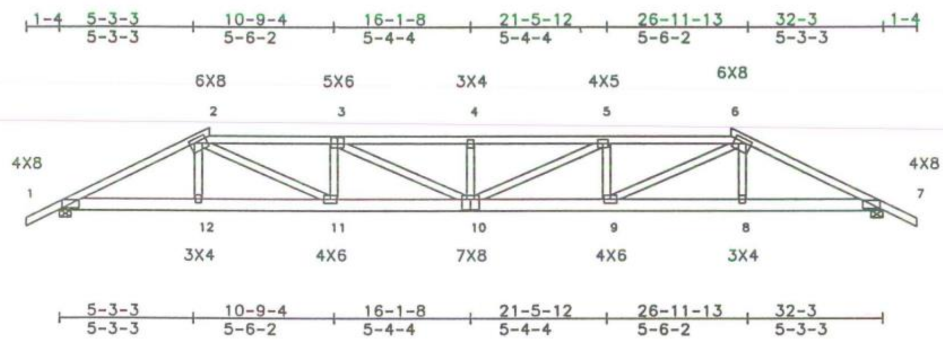
PROFESSIONAL ENGINEER
ENGINEER
EXP. 6-30-98
OREGON
JAN. 25, 1992
REDONG YU

PLATE OFFSETS (X=LEFT,Y=TOP):[j2=4,2.5],[j3=3,3],[j6=4,2.5],[j10=4,2],(SYMMETRIC PLATING TURNED ON)

BRACE FLAT PART OF T/C AT 24 IN O.C. FROM JTS 2 TO 6.




```
L. HL TO PK:5-10-10                                          R. HL TO PK :5-10-10
LEFT HEIGHT:0-4-3         SPAN:32-3         RISE:3-3-11      RIGHT HEIGHT:0-4-3
=====================================================================================
LOADING   (PSF)        MAX STRESSES                MINIMUM GRADE OF LUMBER
          L    D       TOP   3-4=0.833             TOP CHORD:2X4  No.1&Btr GR DF-L
TOP      25   10       BOTT 10-11=0.592            BOT CHORD:2X6  No.2 GR DF-L
BOTT      0    7       LL.DEFL.@10=0.30 < L/240 WEBS     :2X4  STANDARD 91 HF
=====================================================================================
STR.INC.: LUMB = 1.15 PLATE = 1.15                 SPACING : 24.0 in. o. c.
REPETITIVE STRESSES NOT USED                       NO. OF MEMBERS = 2
```

LOADING	STRESS INCREASE LUMBER	PLATE	LOADING TYPE	PANEL(PLF) / JOINTS(LBS)			
1	1.15	1.15	UNIFORM	1- 2= 70	2- 6= 139	6- 7= 70	7- 1= 28
			CONCENTRATED	2= 358	6= 358		

2 MEMBERS NAILED TOG. W/1 ROW(S) OF .131x3 in. NAILS 11 in. o.c.(TOP CHS.),AND 1 ROW(S) OF .131x3 in. NAILS 12 in. o.c(BOTT. CHS.)
For Webs use 1 ROW of NAILS 12 in. o.c.
DEFLECTION(IN.) L.L= 0.30,D.L=0.20,T.L=0.50
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DEVELOPMENT SERVICES

```
===<<<<ACES-32 Ver. 1.1>>>>==========[ 951541 ]===========<<<<RELCO>>>>=========
  Customer : C-V-B                                         Thu Apr  9 17:04:40 1998
  Project #: 351               Truss ID  : K1              Family #  : 324
  Span     : 32-3              Quantity  : 1               Top Pitch : 6/12
==Bld(2/12/1998),v1.1===========================================================
```

TOP CHORD	BOTTOM CHORD	WEBS		REACTIONS - SIZE	
1-2=-2446	7-8= 2173	2-12= 74	5-9=-605	1=-1447	5.50
2-3=-3613	8-9= 2173	2-11= 1593	6-9= 1593	7=-1447	5.50
3-4=-4075	9-10= 3613	3-11=-605	6-8= 74		
4-5=-4075	10-11= 3613	3-10= 514			
5-6=-3613	11-12= 2173	4-10=-375			
6-7=-2446	12-1= 2173	5-10= 514			

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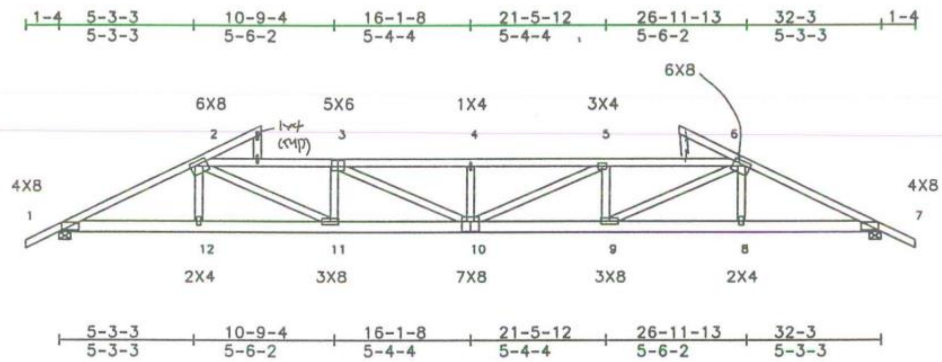
PROFESSIONAL ENGINEER
ENGINEER
15766
EXP. 6-30-98
OREGON
JAN. 25, 1992
REDONGYU

PLATE OFFSETS (X=LEFT,Y=TOP):[j2=4,2.5],[j3=3,3],[j6=4,2.5],[j10=4,2],(SYMMETRIC PLATING TURNED ON)

BRACE FLAT PART OF T/C AT 24 IN O.C. FROM JTS 2 TO 6.




```
L. HL TO PK:5-10-10                                      R. HL TO PK :5-10-10
LEFT HEIGHT:0-4-3         SPAN:32-3       RISE:4-3-11     RIGHT HEIGHT:0-4-3
===============================================================================
LOADING    (PSF)       MAX STRESSES                 MINIMUM GRADE OF LUMBER
           L     D     TOP    3-4=0.741             TOP CHORD:2X4  No.1&Btr GR DF-L
TOP       25    10     BOTT 10-11=0.564             BOT CHORD:2X6  No.2 GR DF-L
BOTT       0     7     LL.DEFL.@10=0.29 < L/240     WEBS     :2X4  STANDARD 91 HF
===============================================================================
STR.INC.: LUMB = 1.15 PLATE = 1.15                  SPACING : 24.0 in. o. c.
REPETITIVE STRESSES USED                            NO. OF MEMBERS = 1
```

DEFLECTION(IN.) L.L= 0.29,D.L=0.19,T.L=0.48
PLATES ARE MITEK M20-165,142 MANUFACTURED FROM ASTM A 446 GRD A GALVANIZED STEEL(EXCEPT AS SHOWN)
PLATE MUST BE INSTALLED ON EA. FACE OF JOINT,SYMMETRICALLY(EXCEPT AS SHOWN)DESIGN CONFORMS W/NDS DESIGN SPECS, UBC-ICBO,TPI-91
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CITY OF CORVALLIS
DEVELOPMENT SERVICES

===<<<<ACES-32 Ver. 1.1>>>>==========[951542]==========<<<<RELCO>>>>=========

Customer : C-V-B
Project #: 351 — Truss ID : L
Span : 32-3 — Quantity : 1

Thu Apr 9 17:04:47 1998
Family # : 324
Top Pitch : 6/12

==Bld(2/12/1998),v1.1==

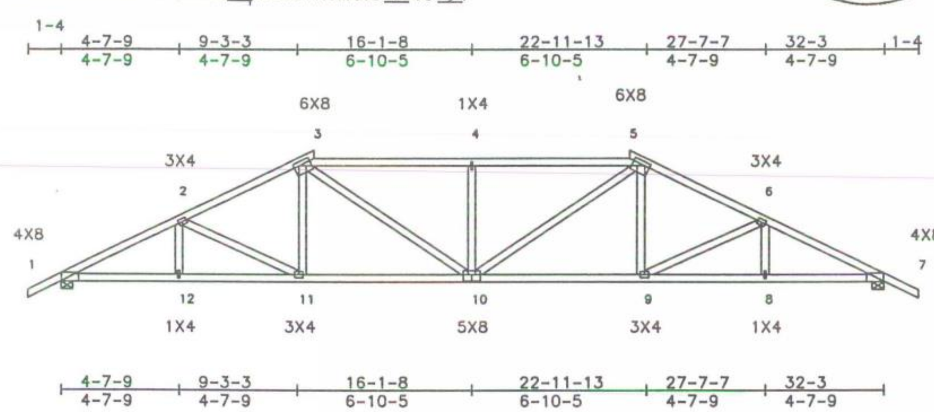
TOP CHORD	BOTTOM CHORD	WEBS		REACTIONS - SIZE	
1-2=-2440	7-8= 2151	2-12= 63	5-9= 238	1=-1447	5.50
2-3=-2063	8-9= 2151	2-11=-347	6-9=-347	7=-1447	5.50
3-4=-2264	9-10= 1842	3-11= 238	6-8= 63		
4-5=-2264	10-11= 1842	3-10= 511			
5-6=-2063	11-12= 2151	4-10=-480			
6-7=-2440	12-1= 2151	5-10= 511			

APR 13 1998

PROFESSIONAL ENGINEER
EXP. 6-30-98
OREGON
JAN. 25, 1992
REDONG YU

PLATE OFFSETS (X=LEFT,Y=TOP):[j3=4,2.5],[j5=4,2.5],[j10=4,2],(SYMMETRIC PLATING TURNED ON)

BRACE FLAT PART OF T/C AT 20 IN O.C. FROM JTS 3 TO 5.



L. HL TO PK:10-4-5 — R. HL TO PK :10-4-5
LEFT HEIGHT:0-4-3 — SPAN:32-3 — RISE:5-3-11 — RIGHT HEIGHT:0-4-3

==

LOADING (PSF)	L	D	MAX STRESSES	MINIMUM GRADE OF LUMBER
TOP	25	10	TOP 3-4=0.978	TOP CHORD:2X4 No.1&Btr GR DF-L
BOTT	0	7	BOTT 8-9=0.421	BOT CHORD:2X4 No.1&Btr GR DF-L
			LL.DEFL.@10=0.13 < L/240	WEBS :2X4 STANDARD 91 HF

==

STR.INC.: LUMB = 1.15 PLATE = 1.15 — SPACING : 24.0 in. o. c.
REPETITIVE STRESSES USED — NO. OF MEMBERS = 1

DEFLECTION(IN.) L.L= 0.13,D.L=0.09,T.L=0.22
PLATES ARE MITEK M20-165,142 MANUFACTURED FROM ASTM A 446 GRD A GALVANIZED STEEL(EXCEPT AS SHOWN)
PLATE MUST BE INSTALLED ON EA. FACE OF JOINT,SYMMETRICALLY(EXCEPT AS SHOWN)DESIGN CONFORMS W/NDS DESIGN SPECS, UBC-ICBO,TPI-91
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CITY OF CORVALLIS
DEVELOPMENT SERVICES

```
===<<<<ACES-32 Ver. 1.1>>>>==========[ 951544 ]==========<<<<RELCO>>>>=========
  Customer : C-V-B                                          Thu Apr  9 17:05:01 1998
  Project #: 351                Truss ID   : L2             Family #   : 324
  Span     : 32-3               Quantity   : 2              Top Pitch  : 6/12
==Bld(2/12/1998),v1.1==========================================================
```

TOP CHORD	BOTTOM CHORD	WEBS		REACTIONS - SIZE
1-2=-2440	7-8= 2151	2-12= 63	5-9= 238	1=-1447 5.50
2-3=-2063	8-9= 2151	2-11=-347	6-9=-347	7=-1447 5.50
3-4=-2264	9-10= 1842	3-11= 238	6-8= 63	
4-5=-2264	10-11= 1842	3-10= 511		
5-6=-2063	11-12= 2151	4-10=-480		
6-7=-2440	12-1= 2151	5-10= 511		

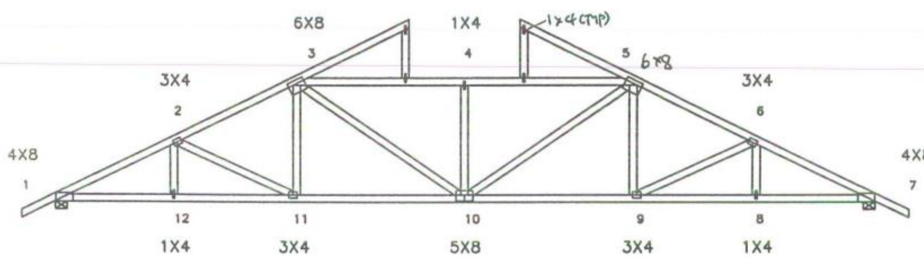
APR 13 1998

PROFESSIONAL ENGINEER
ENGINEER
5,786
EXP 6/30/98
OREGON
JAN. 25, 1992
REDONG YU

PLATE OFFSETS (X=LEFT,Y=TOP):[j3=4,2.5],[j5=4,2.5],[j10=4,2],(SYMMETRIC PLATING TURNED ON)

1-4 | 4-7-9 | 9-3-3 | 16-1-8 | 22-11-13 | 27-7-7 | 32-3 | 1-4
4-7-9 | 4-7-9 | 6-10-5 | 6-10-5 | 4-7-9 | 4-7-9

BRACE FLAT PART OF T/C AT 20 IN O.C. FROM JTS 3 TO 5.



4-7-9 | 9-3-3 | 16-1-8 | 22-11-13 | 27-7-7 | 32-3
4-7-9 | 4-7-9 | 6-10-5 | 6-10-5 | 4-7-9 | 4-7-9

```
L. HL TO PK:10-4-5                                          R. HL TO PK :10-4-5
LEFT HEIGHT:0-4-3        SPAN:32-3        RISE:7-3-11        RIGHT HEIGHT:0-4-3
================================================================================
LOADING  (PSF)       MAX STRESSES                 MINIMUM GRADE OF LUMBER
        L    D       TOP   3-4=0.978              TOP CHORD:2X4  No.1&Btr GR DF-L
TOP    25   10       BOTT  8-9=0.421              BOT CHORD:2X4  No.1&Btr GR DF-L
BOTT    0    7       LL.DEFL.@10=0.13 < L/240     WEBS     :2X4  STANDARD 91 HF
================================================================================
STR.INC.: LUMB = 1.15 PLATE = 1.15                SPACING : 24.0 in. o. c.
REPETITIVE STRESSES USED                          NO. OF MEMBERS = 1
```

DEFLECTION(IN.) L.L= 0.13,D.L=0.09,T.L=0.22
PLATES ARE MITEK M20-165,142 MANUFACTURED FROM ASTM A 446 GRD A GALVANIZED STEEL(EXCEPT AS SHOWN)
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CITY OF CORVALLIS
DEVELOPMENT SERVICES

```
===<<<<ACES-32 Ver. 1.1>>>>==========[ 951543 ]==========<<<<RELCO>>>>=========
 Customer : C-V-B                                       Thu Apr  9 17:04:54 1998
 Project #: 351                  Truss ID   : L1        Family #   : 324
 Span     : 32-3                 Quantity   : 2         Top Pitch  : 6/12
==Bld(2/12/1998),v1.1=============================================================
```

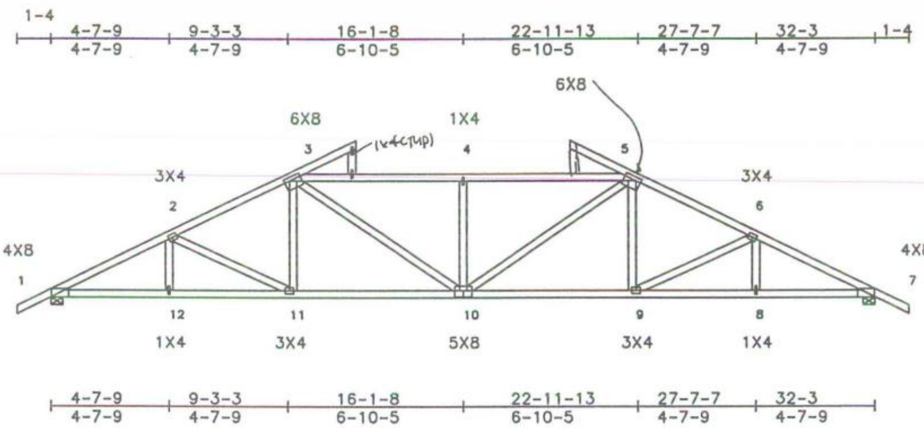
APR 13 1998

TOP CHORD	BOTTOM CHORD	WEBS		REACTIONS -	SIZE
1-2=-2440	7-8= 2151	2-12= 63	5-9= 238	1=-1447	5.50
2-3=-2063	8-9= 2151	2-11=-347	6-9=-347	7=-1447	5.50
3-4=-2264	9-10= 1842	3-11= 238	6-8= 63		
4-5=-2264	10-11= 1842	3-10= 511			
5-6=-2063	11-12= 2151	4-10=-480			
6-7=-2440	12-1= 2151	5-10= 511			

PLATE OFFSETS (X=LEFT,Y=TOP):[j3=4,2.5],[j5=4,2.5],[j10=4,2],(SYMMETRIC PLATING TURNED ON)

PROFESSIONAL ENGINEER
OREGON
JAN. 25, 1992
REDONG YU

BRACE FLAT PART OF T/C AT 20 IN O.C. FROM JTS 3 TO 5.




```
L. HL TO PK:10-4-5                                        R. HL TO PK :10-4-5
LEFT HEIGHT:0-4-3        SPAN:32-3        RISE:6-3-11     RIGHT HEIGHT:0-4-3
===============================================================================
LOADING   (PSF)          MAX STRESSES           MINIMUM GRADE OF LUMBER
          L    D         TOP    3-4=0.978       TOP CHORD:2X4   No.1&Btr GR DF-L
TOP       25   10        BOTT   8-9=0.421       BOT CHORD:2X4   No.1&Btr GR DF-L
BOTT      0    7         LL.DEFL.@10=0.13 < L/240 WEBS  :2X4   STANDARD 91 HF
===============================================================================
STR.INC.: LUMB = 1.15 PLATE = 1.15                SPACING : 24.0 in. o. c.
REPETITIVE STRESSES USED                          NO. OF MEMBERS = 1
```

DEFLECTION(IN.) L.L= 0.13,D.L=0.09,T.L=0.22
PLATES ARE MITEK M20-165,142 MANUFACTURED FROM ASTM A 446 GRD A GALVANIZED STEEL(EXCEPT AS SHOWN)
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CITY OF CORVALLIS
DEVELOPMENT SERVICES

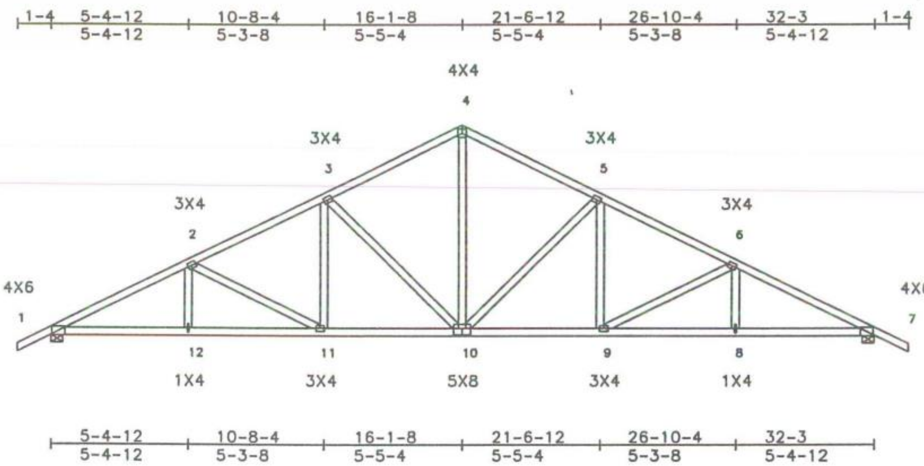
===<<<<ACES-32 Ver. 1.1>>>>>==========[951545]==========<<<<RELCO>>>>>==========
Customer : C-V-B — Thu Apr 9 17:05:07 1998
Project #: 351 — Truss ID : H — Family # : 110
Span : 32-3 — Quantity : 6 — Top Pitch : 6/12
==Bld(2/12/1998),v1.1===

TOP CHORD	BOTTOM CHORD	WEBS		REACTIONS - SIZE	
1-2=-2390	7-8= 2112	2-12= 73	5-9= 267	1=-1447	5.50
2-3=-1947	8-9= 2112	2-11=-417	6-9=-417	7=-1447	5.50
3-4=-1465	9-10= 1741	3-11= 267	6-8= 73		
4-5=-1465	10-11= 1741	3-10=-607			
5-6=-1947	11-12= 2112	4-10= 929			
6-7=-2390	12-1= 2112	5-10=-607			

APR 13 1998

PROFESSIONAL ENGINEER
15766
EXP. 6-30-98
OREGON
JAN. 25, 1992
REDONG YU

PLATE OFFSETS (X=LEFT,Y=TOP):[j10=4,2],(SYMMETRIC PLATING TURNED ON)



L. HL TO PK:18-0-5 — R. HL TO PK :18-0-5
LEFT HEIGHT:0-4-3 — SPAN:32-3 — RISE:8-4-15 — RIGHT HEIGHT:0-4-3

==

LOADING (PSF)	L	D	MAX STRESSES	MINIMUM GRADE OF LUMBER
TOP	25	10	TOP 2-3=0.347	TOP CHORD:2X4 No.1&Btr GR DF-L
BOTT	0	7	BOTT 11-12=0.388	BOT CHORD:2X4 No.1&Btr GR DF-L
			LL.DEFL.@10=0.11 < L/240	WEBS :2X4 STANDARD 91 HF

==

STR.INC.: LUMB = 1.15 PLATE = 1.15 — SPACING : 24.0 in. o. c.
REPETITIVE STRESSES USED — NO. OF MEMBERS = 1

DEFLECTION(IN.) L.L= 0.11,D.L=0.07,T.L=0.18
PLATES ARE MITEK M20-165,142 MANUFACTURED FROM ASTM A 446 GRD A GALVANIZED STEEL(EXCEPT AS SHOWN)
PLATE MUST BE INSTALLED ON EA. FACE OF JOINT,SYMMETRICALLY(EXCEPT AS SHOWN)DESIGN CONFORMS W/NDS DESIGN SPECS, UBC-ICBO,TPI-91
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APPROVED
APR 23 1998
CITY OF CORVALLIS
DEVELOPMENT SERVICES

```
===<<<<ACES-32 Ver. 1.1>>>>==========[ 951551 ]==========<<<<RELCO>>>>=========
  Customer  : C-V-B                                          Thu Apr  9 17:05:33 1998
  Project #: 351               Truss ID   : V2               Family #   : 323
  Span      : 17-6-4           Quantity   : 1                Top Pitch : 6/12
==Bld(2/12/1998),v1.1==========================================================
```

TOP CHORD	BOTTOM CHORD	WEBS	REACTIONS - SIZE	
1-2=-1120	5-6= 0	2-7= 78	1=-835	5.50
2-3=-1128	6-7= 997	2-6= 142	5=-723	1.59
3-4=-1128	7-1= 997	3-6=-439		
4-5=-681		4-6= 1221		

APPROVED FOR MITEK INDUSTRIES INC.

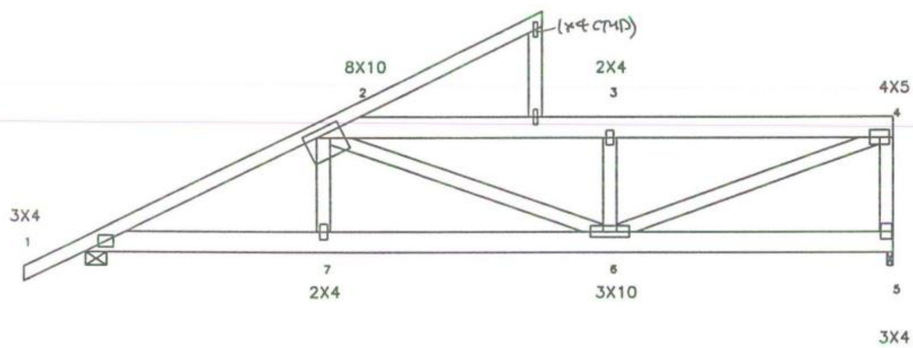
APR 13 1998

PROFESSIONAL ENGINEER 15,766 OREGON JAN. 25, 1992 REDONGYU EXP. 6-30-98

PLATE OFFSETS (X=LEFT,Y=TOP):[j2=5,2.5],(SYMMETRIC PLATING TURNED ON)

1-4   5-3-3   11-4-11   17-6-4
5-3-3   6-1-9   6-1-9

BRACE FLAT PART OF T/C AT 48 IN O.C. FROM JTS 2 TO 4.



5-3-3   11-4-11   17-6-4
5-3-3   6-1-9   6-1-9

```
L. HL TO PK:5-10-10
LEFT HEIGHT:0-4-3        SPAN:17-6-4        RISE:5-3-11        RIGHT HEIGHT:2-11-12
===============================================================================
LOADING   (PSF)          MAX STRESSES               MINIMUM GRADE OF LUMBER
          L    D         TOP    1-2=0.352           TOP CHORD:2X4   No.1&Btr GR DF-L
TOP      25   10         BOTT   6-7=0.206           BOT CHORD:2X6   No.2 GR DF-L
BOTT      0    7         LL.DEFL.@6=0.03 < L/240    WEBS      :2X4  STANDARD 91 HF
===============================================================================
STR.INC.: LUMB = 1.15 PLATE = 1.15                  SPACING : 24.0 in. o. c.
REPETITIVE STRESSES USED                            NO. OF MEMBERS = 1
```

CHORD: 2-3; 3-4 TO BE 2X6 No.2 GR DF-L
WEB: 4-6 TO BE 2X4 No.1&Btr GR DF-L
DEFLECTION(IN.) L.L= 0.03,D.L=0.02,T.L=0.05
PLATES ARE MITEK M20-165,142 MANUFACTURED FROM ASTM A 446 GRD A GALVANIZED STEEL(EXCEPT AS SHOWN)
PLATE MUST BE INSTALLED ON EA. FACE OF JOINT,SYMMETRICALLY(EXCEPT AS SHOWN)DESIGN CONFORMS W/NDS DESIGN SPECS, UBC-ICBO,TPI-91
THIS DESIGN IS FOR TRUSS FABRICATION ONLY.FOR PERMANENT AND TEMPORARY BRACING(WHICH IS ALWAYS REQD)CONSULT BLDG ARCHITECT OR ENGINEER.

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CITY OF CORVALLIS
DEVELOPMENT SERVICES

===<<<<ACES-32 Ver. 1.1>>>>===========[951550]===========<<<<RELCO>>>>==========

Customer : C-V-B   Thu Apr 9 17:05:29 1998
Project #: 351   Truss ID : V1   Family # : 323
Span : 17-6-4   Quantity : 1   Top Pitch : 6/12

==Bld(2/12/1998),v1.1==

TOP CHORD	BOTTOM CHORD	WEBS	REACTIONS - SIZE	
1-2=-1120	5-6= 0	2-7= 78	1=-835	5.50
2-3=-1128	6-7= 997	2-6= 142	5=-723	1.59
3-4=-1128	7-1= 997	3-6=-439		
4-5=-681		4-6= 1221		

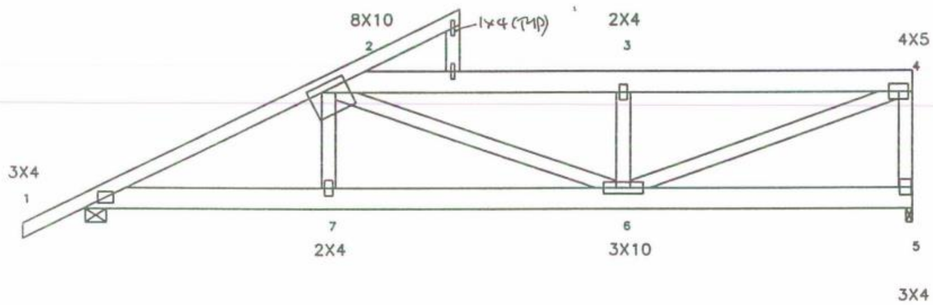
APPROVED FOR MITEK INDUSTRIES APR 13 1998

PROFESSIONAL ENGINEER 15,766 EXP. 6-30-98 OREGON JAN. 25, 1992 REDONGYU

PLATE OFFSETS (X=LEFT,Y=TOP):[j2=5,2.5],(SYMMETRIC PLATING TURNED ON)

1-4 | 5-3-3 | 11-4-11 | 17-6-4
5-3-3 | 6-1-9 | 6-1-9

BRACE FLAT PART OF T/C AT 48 IN O.C. FROM JTS 2 TO 4.



5-3-3 | 11-4-11 | 17-6-4
5-3-3 | 6-1-9 | 6-1-9

L. HL TO PK:5-10-10
LEFT HEIGHT:0-4-3   SPAN:17-6-4   RISE:4-3-11   RIGHT HEIGHT:2-11-12

===

LOADING (PSF)	L	D	MAX STRESSES	MINIMUM GRADE OF LUMBER
TOP	25	10	TOP 1-2=0.352	TOP CHORD:2X4 No.1&Btr GR DF-L
BOTT	0	7	BOTT 6-7=0.206	BOT CHORD:2X6 No.2 GR DF-L
			LL.DEFL.@6=0.03 < L/240	WEBS :2X4 STANDARD 91 HF

===

STR.INC.: LUMB = 1.15 PLATE = 1.15   SPACING : 24.0 in. o. c.
REPETITIVE STRESSES USED   NO. OF MEMBERS = 1

CHORD: 2-3; 3-4 TO BE 2X6 No.2 GR DF-L
WEB: 4-6 TO BE 2X4 No.1&Btr GR DF-L
DEFLECTION(IN.) L.L= 0.03,D.L=0.02,T.L=0.05
PLATES ARE MITEK M20-165,142 MANUFACTURED FROM ASTM A 446 GRD A GALVANIZED STEEL(EXCEPT AS SHOWN)
PLATE MUST BE INSTALLED ON EA. FACE OF JOINT,SYMMETRICALLY(EXCEPT AS SHOWN)DESIGN CONFORMS W/NDS DESIGN SPECS, UBC-ICBO,TPI-91
THIS DESIGN IS FOR TRUSS FABRICATION ONLY.FOR PERMANENT AND TEMPORARY BRACING(WHICH IS ALWAYS REQD)CONSULT BLDG ARCHITECT OR ENGINEER.

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APR 23 1998
CITY OF CORVALLIS
DEVELOPMENT SERVICES

```
===<<<<ACES-32 Ver. 1.1>>>>==========[ 951549 ]==========<<<<RELCO>>>>=========
  Customer : C-V-B                                          Thu Apr  9 17:05:25 1998
  Project #: 351                 Truss ID  : V              Family #  : 323
  Span     : 17-6-4              Quantity  : 1              Top Pitch : 6/12
==Bld(2/12/1998),v1.1=========================================================
```

TOP CHORD	BOTTOM CHORD	WEBS	REACTIONS - SIZE	
1-2=-2504	5-6= 0	2-7= 155	1=-1502	5.50
2-3=-2362	6-7= 2227	2-6= 146	5=-1486	1.59
3-4=-2362	7-1= 2227	3-6=-870		
4-5=-1402		4-6= 2558		

APPROVED FOR MITEK INDUSTRIES INC.

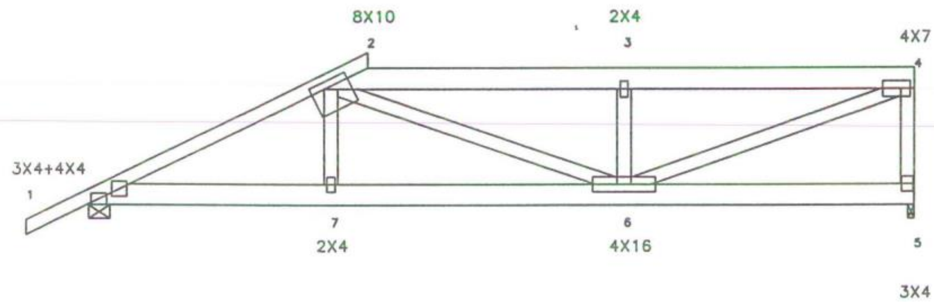
APR 1 0 1998

PROFESSIONAL ENGINEER
ENGINEER
15,766
EXP. 6-30-98
OREGON
JAN. 25, 1992
REDONG YU

PLATE OFFSETS (X=LEFT,Y=TOP):[j2=5,2.5],[j4=4.5,2],(SYMMETRIC PLATING TURNED ON)

BRACE FLAT PART OF T/C AT 24 IN O.C. FROM JTS 2 TO 4.

1-4 | 5-3-3 | 11-4-11 | 17-6-4
5-3-3 | 6-1-9 | 6-1-9



5-3-3 | 11-4-11 | 17-6-4
5-3-3 | 6-1-9 | 6-1-9

```
L. HL TO PK:5-10-10
LEFT HEIGHT:0-4-3        SPAN:17-6-4        RISE:3-3-11        RIGHT HEIGHT:2-11-12
===============================================================================
LOADING  (PSF)           MAX STRESSES                MINIMUM GRADE OF LUMBER
         L    D          TOP   2-3=0.740             TOP CHORD:2X4  No.1&Btr GR DF-L
TOP      25   10         BOTT  6-7=0.464             BOT CHORD:2X6  No.2 GR DF-L
BOTT     0    7          LL.DEFL.@6=0.07 < L/240     WEBS     :2X4  STANDARD 91 HF
===============================================================================
STR.INC.: LUMB = 1.15 PLATE = 1.15                   SPACING : 24.0 in. o. c.
REPETITIVE STRESSES NOT USED                         NO. OF MEMBERS = 1
```

LOADING	STRESS INCREASE LUMBER	PLATE	LOADING TYPE	PANEL(PLF) / JOINTS(LBS)		
1	1.15	1.15	UNIFORM	1- 2= 70	2- 4= 139	5- 1= 28
			CONCENTRATED	2= 358		

CHORD: 2-3; 3-4 TO BE 2X6 No.2 GR DF-L
WEB: 4-6 TO BE 2X4 No.1&Btr GR DF-L
DEFLECTION(IN.) L.L= 0.07,D.L=0.05,T.L=0.11
PLATES ARE MITEK M20-165,142 MANUFACTURED FROM ASTM A 446 GRD A GALVANIZED STEEL(EXCEPT AS SHOWN)
PLATE MUST BE INSTALLED ON EA. FACE OF JOINT,SYMMETRICALLY(EXCEPT AS SHOWN)DESIGN CONFORMS W/NDS DESIGN SPECS, UBC-ICBO,TPI-91
THIS DESIGN IS FOR TRUSS FABRICATION ONLY.FOR PERMANENT AND TEMPORARY BRACING(WHICH IS ALWAYS REQD)CONSULT BLDG ARCHITECT OR ENGINEER.

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DEVELOPMENT SERVICES

===<<<<ACES-32 Ver. 1.1>>>>==========[951552]==========<<<<RELCO>>>>=========

Customer : C-V-B
Project #: 351
Span : 11-6

Truss ID : MG
Quantity : 2

Thu Apr 9 17:05:37 1998
Family # : 201
Top Pitch : 6/12

==Bld(2/12/1998),v1.1==

TOP CHORD	BOTTOM CHORD	WEBS	REACTIONS - SIZE	
1-2=-5506	4-5= 4926	2-5= 4703	1=-2814	5.50
2-3= 0	5-1= 4926	2-4=-5586	4=-4937	2.64
3-4=-441				

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APR 13 1998

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OREGON
JAN. 25, 1992
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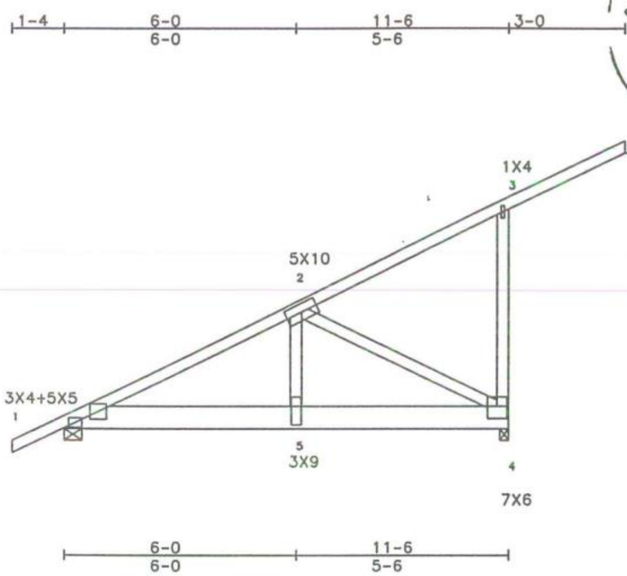
PLATE OFFSETS (X=LEFT,Y=TOP):[j5=6,1.5],(SYMMETRIC PLATING TURNED ON)



L. HL TO PK:12-10-5
LEFT HEIGHT:0-4-3 SPAN:11-6 RISE:7-7-3 RIGHT HEIGHT:6-1-3

==

LOADING	(PSF)		MAX STRESSES	MINIMUM GRADE OF LUMBER
	L	D	TOP 1-2=0.495	TOP CHORD:2X4 No.1&Btr GR DF-L
TOP	25	10	BOTT 4-5=0.941	BOT CHORD:2X8 No.1&Btr GR DF-L
BOTT	0	7	LL.DEFL.@5=0.05 < L/240	WEBS :2X4 STANDARD 91 HF

==

STR.INC.: LUMB = 1.15 PLATE = 1.15 SPACING : 24.0 in. o. c.
REPETITIVE STRESSES NOT USED NO. OF MEMBERS = 2

LOADING	STRESS INCREASE LUMBER	STRESS INCREASE PLATE	LOADING TYPE	PANEL(PLF) / JOINTS(LBS)
1	1.15	1.15	UNIFORM	1- 3= 70 4- 5= 691 5- 1= 14
			CONCENTRATED	5= 2800

WEB: 2-5 TO BE 2X4 No.1&Btr GR DF-L
2 MEMBERS NAILED TOG. W/1 ROW(S) OF .131x3 in. NAILS 12 in. o.c.(TOP CHS.),AND 2 ROW(S) OF .131x3 in. NAILS 4 in. o.c(BOTT. CHS.)
For Webs use 1 ROW of NAILS 12 in. o.c.
DEFLECTION(IN.) L.L= 0.05,D.L=0.04,T.L=0.09
PLATES ARE MITEK M20-165,142 MANUFACTURED FROM ASTM A 446 GRD A GALVANIZED STEEL(EXCEPT AS SHOWN)
PLATE MUST BE INSTALLED ON EA. FACE OF JOINT,SYMMETRICALLY(EXCEPT AS SHOWN)DESIGN CONFORMS W/NDS DESIGN SPECS, UBC-ICBO,TPI-91
THIS DESIGN IS FOR TRUSS FABRICATION ONLY.FOR PERMANENT AND TEMPORARY BRACING(WHICH IS ALWAYS REQD)CONSULT BLDG ARCHITECT OR ENGINEER.

APR-21-'98 07:36 ID:RELCO ROOF/FLOOR INC TEL NO:503 995-6311 #156 P02

==<<<<ACES-32 Ver. 1.1>>>>==========[933391]==========<<<<RELCO>>>>========

Customer : C-V-B
Project #: 351 Truss ID : TG
Span : 20-11-8 Quantity : 3

Sun Apr 5 13:06:17 199
Family # : 313
Top Pitch : 6/12

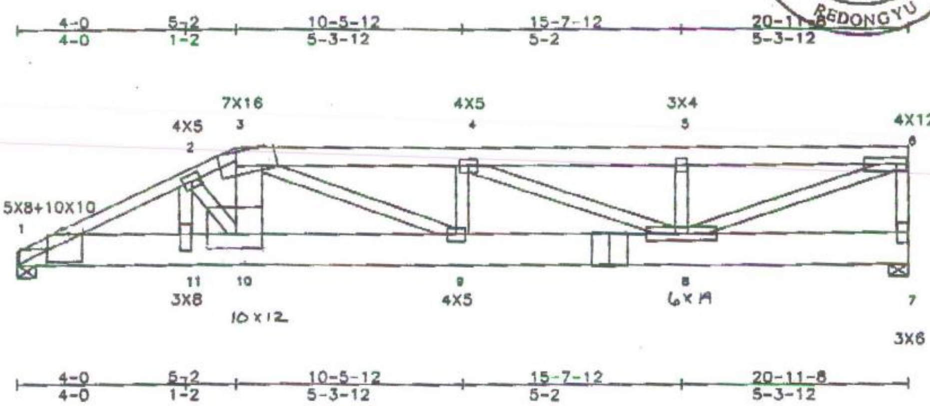
==B(d(2/12/1998),v1.1==

TOP CHORD	BOTTOM CHORD	WEBS		REACTIONS - SIZE	
1-2=-25738	7-8= 0	2-11= 2239	5-8=-1710	1=-11580	5.50
2-3=-24359	8-9= 18890	2-10=-2653	6-8= 12529	7=-6016	5.50..
3-4=-18891	9-10= 21787	3-10= 11508			
4-5=-11433	10-11= 23231	3-9=-3176			
5-6=-11433	11-1= 23231	4-9= 1372			
6-7=-5979		4-8=-8171			

PLATE OFFSETS (X=LEFT,Y=TOP):[j3=8,5],[j6=9,2],[j10=8,8],[j11=5,1.5],(SYMMETRIC PLATING TURNED ON)

BOTT CHORD SPLICES:8-9=10X10;

PROFESSIONAL ENGINEER 13766 EXP 6-30-98 JAN. 25, 1998 EDONGYU



L. HL TO PK:5-9-5
LEFT HEIGHT:0-4-3 SPAN:20-11-8 RISE:2-11-3 RIGHT HEIGHT:2-11-3

==

LOADING (PSF)	L	D	MAX STRESSES	MINIMUM GRADE OF LUMBER
TOP	25	10	TOP 1-2=0.780	TOP CHORD:2X4 No.1&Btr GR DF-L
BOTT	0	7	BOTT 10-11=0.862	BOT CHORD:2X10 No.2 GR DF-L
			LL.DEFL.@9=0.19 < L/240	WEBS :2X4 STANDARD 91 HF

==

STR.INC.: LUMB = 1.15 PLATE = 1.15 SPACING : 24.0 in. o. c.
REPETITIVE STRESSES NOT USED NO. OF MEMBERS = 3

LOADING	STRESS INCREASE LUMBER	PLATE	LOADING TYPE	PANEL(PLF) / JOINTS(LBS)			
1	1.15	1.15	UNIFORM	1- 3= 70	3- 6= 331	7-10= 14	10- 1= 195
			CONCENTRATED	10= 9131	11= 1758		

CHORD: 3-4; 4-5; 5-6 TO BE 2X6 No.2 GR DF-L
WEB: 3-10 TO BE 2X8 No.2 GR DF-L
WEB: 6-8 TO BE 2X4 No.1&Btr GR DF-L
3 MEMBERS NAILED TOG. W/2 ROW(S) OF .131x3 in. NAILS 7 in. o.c.(TOP CHS.),AND 1 ROW(S) OF .131x3 in. NAILS 6 in. o.c(BOTT. CHS.)
For Webs use 1 ROW of NAILS 12 in. o.c.
****NOTE:Special hanger is reqd. to dist. bottom chord concentrated load(s) to all plies.****
DEFLECTION(IN.) L.L= 0.19,D.L=0.13,T.L=0.32
PLATES ARE MITEK M20-165,142 MANUFACTURED FROM ASTM A 446 GRD A GALVANIZED STEEL(EXCEPT AS SHOWN)
PLATE MUST BE INSTALLED ON EA. FACE OF JOINT,SYMMETRICALLY(EXCEPT AS SHOWN)DESIGN CONFORMS W/NDS DESIGN SPECS, UBC-ICBO,TPI-91
THIS DESIGN IS FOR TRUSS FABRICATION ONLY.FOR PERMANENT AND TEMPORARY BRACING(WHICH IS ALWAYS REQD)CONSULT BLDG ARCHITECT OR ENGI

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===<<<<ACES-32 Ver. 1.1>>>>==========[ 951535 ]==========<<<<RELCO>>>>=========
  Customer : C-V-B                                          Thu Apr  9 17:04:09 1998
  Project #: 351                  Truss ID  : C2            Family #  : 324
  Span     : 20-11-8              Quantity  : 1             Top Pitch : 6/12
==Bld(2/12/1998),v1.1=========================================================
 . TOP CHORD     BOTTOM CHORD     WEBS          REACTIONS - SIZE        APPROVED FOR MITEK INDUSTRIES INC.
  1-2=-1421      5-6= 1264       2-8= 72        1=-1085     5.50
  2-3=-1751      6-7= 1264       2-7= 543       5=-1085     5.50
  3-4=-1751      7-8= 1264       3-7=-409
  4-5=-1421      8-1= 1264       4-7= 543
                                 4-6= 72

PLATE OFFSETS (X=LEFT,Y=TOP):[j2=5,2.5],[j4=5,2.5],[j7=4,2],(SYMMETRIC PLATING TURNED ON)
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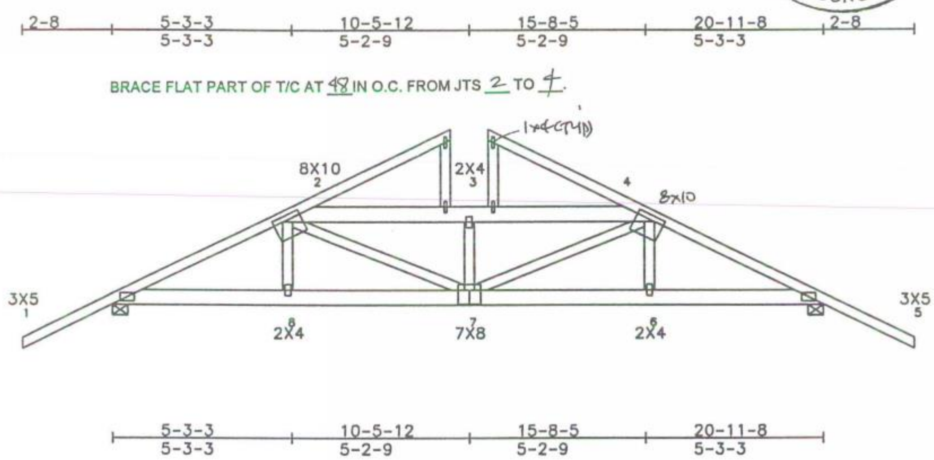
APR 13 1998

PROFESSIONAL ENGINEER
15166
EXP. 6-30-98
OREGON
JAN. 25, 1992
REDONG YU

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L. HL TO PK:5-10-10                                       R. HL TO PK :5-10-10
LEFT HEIGHT:0-4-3          SPAN:20-11-8      RISE:5-3-11      RIGHT HEIGHT:0-4-3
==============================================================================
LOADING   (PSF)         MAX STRESSES               MINIMUM GRADE OF LUMBER
         L     D        TOP    1-2=0.376           TOP CHORD:2X4  No.1&Btr GR DF-L
TOP     25    10        BOTT   7-8=0.229           BOT CHORD:2X6  No.2 GR DF-L
BOTT     0     7        LL.DEFL.@7=0.06 < L/240    WEBS     :2X4  STANDARD 91 HF
==============================================================================
STR.INC.: LUMB = 1.15 PLATE = 1.15                   SPACING : 24.0 in. o. c.
REPETITIVE STRESSES USED                             NO. OF MEMBERS = 1
```

CHORD: 2-3; 3-4 TO BE 2X6 No.2 GR DF-L
DEFLECTION(IN.) L.L= 0.06,D.L=0.04,T.L=0.09
PLATES ARE MITEK M20-165,142 MANUFACTURED FROM ASTM A 446 GRD A GALVANIZED STEEL(EXCEPT AS SHOWN)
PLATE MUST BE INSTALLED ON EA. FACE OF JOINT,SYMMETRICALLY(EXCEPT AS SHOWN)DESIGN CONFORMS W/NDS DESIGN SPECS, UBC-ICBO,TPI-91
THIS DESIGN IS FOR TRUSS FABRICATION ONLY.FOR PERMANENT AND TEMPORARY BRACING(WHICH IS ALWAYS REQD)CONSULT BLDG ARCHITECT OR ENGINEER

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APR 23 1998
CITY OF CORVALLIS
DEVELOPMENT SERVICES

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===<<<<ACES-32 Ver. 1.1>>>>==========[ 951534 ]==========<<<<RELCO>>>>=========
  Customer : C-V-B                                              Thu Apr  9 17:04:04 1998
  Project #: 351                  Truss ID   : C1              Family #   : 324
  Span     : 20-11-8              Quantity   : 1               Top Pitch  : 6/12
==Bld(2/12/1998),v1.1============================================================
  TOP CHORD    BOTTOM CHORD   WEBS          REACTIONS - SIZE      APPROVED FOR MITEK INDUSTRIES INC.
  1-2=-1421    5-6= 1264      2-8= 72       1=-1085   5.50
  2-3=-1751    6-7= 1264      2-7= 543      5=-1085   5.50
  3-4=-1751    7-8= 1264      3-7=-409
  4-5=-1421    8-1= 1264      4-7= 543
                              4-6= 72
```

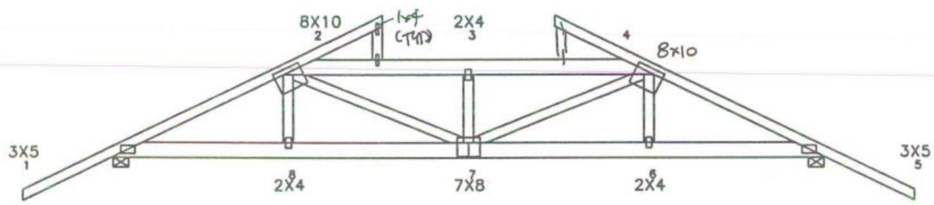
APR 13 1998

PROFESSIONAL ENGINEER
ENGINEER
15,766
EXP. 6-30-98
OREGON
JAN. 25, 1992
REDONGYU

PLATE OFFSETS (X=LEFT,Y=TOP):[j2=5,2.5],[j4=5,2.5],[j7=4,2],(SYMMETRIC PLATING TURNED ON)

2-8 | 5-3-3 / 5-3-3 | 10-5-12 / 5-2-9 | 15-8-5 / 5-2-9 | 20-11-8 / 5-3-3 | 2-8

BRACE FLAT PART OF T/C AT 48 IN O.C. FROM JTS 2 TO 4.



5-3-3 / 5-3-3 | 10-5-12 / 5-2-9 | 15-8-5 / 5-2-9 | 20-11-8 / 5-3-3

```
L. HL TO PK:5-10-10                                             R. HL TO PK :5-10-10
LEFT HEIGHT:0-4-3         SPAN:20-11-8      RISE:4-3-11         RIGHT HEIGHT:0-4-3
=================================================================================
LOADING  (PSF)        MAX STRESSES                MINIMUM GRADE OF LUMBER
        L    D        TOP    1-2=0.376            TOP CHORD:2X4   No.1&Btr GR DF-L
TOP    25   10        BOTT   7-8=0.229            BOT CHORD:2X6   No.2 GR DF-L
BOTT    0    7        LL.DEFL.@7=0.06 < L/240     WEBS     :2X4   STANDARD 91 HF
=================================================================================
STR.INC.: LUMB = 1.15 PLATE = 1.15                  SPACING : 24.0 in. o. c.
REPETITIVE STRESSES USED                            NO. OF MEMBERS = 1
```

CHORD: 2-3; 3-4 TO BE 2X6 No.2 GR DF-L
DEFLECTION(IN.) L.L= 0.06,D.L=0.04,T.L=0.09
PLATES ARE MITEK M20-165,142 MANUFACTURED FROM ASTM A 446 GRD A GALVANIZED STEEL(EXCEPT AS SHOWN)
PLATE MUST BE INSTALLED ON EA. FACE OF JOINT,SYMMETRICALLY(EXCEPT AS SHOWN)DESIGN CONFORMS W/NDS DESIGN SPECS, UBC-ICBO,TPI-91
THIS DESIGN IS FOR TRUSS FABRICATION ONLY.FOR PERMANENT AND TEMPORARY BRACING(WHICH IS ALWAYS REQD)CONSULT BLDG ARCHITECT OR ENGINEER

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APR 2 3 1998
CITY OF CORVALLIS
DEVELOPMENT SERVICES

===<<<<ACES-32 Ver. 1.1>>>>==========[951533]==========<<<<RELCO>>>>=========

Customer : C-V-B		Thu Apr 9 17:03:58 1998
Project #: 351	Truss ID : C	Family # : 324
Span : 20-11-8	Quantity : 1	Top Pitch : 6/12

==Bld(2/12/1998),v1.1==========

TOP CHORD	BOTTOM CHORD	WEBS	REACTIONS - SIZE	
1-2=-3218	5-6= 2863	2-8= 142	1=-1941	5.50
2-3=-3835	6-7= 2863	2-7= 1085	5=-1941	5.50
3-4=-3835	7-8= 2863	3-7=-820		
4-5=-3218	8-1= 2863	4-7= 1085		
		4-6= 142		

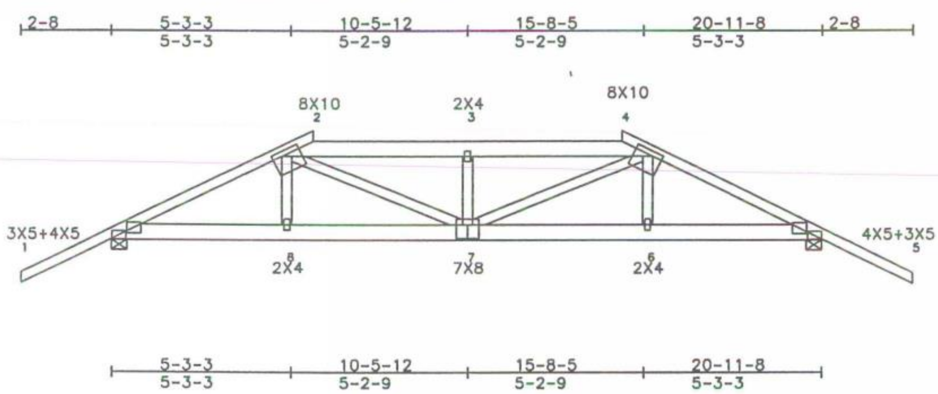
APPROVED FOR MITEK INDUSTRIES INC.

APR 13 1998

PROFESSIONAL ENGINEER
ENGINEER
5,766
EXP. 6-30-98
OREGON
JAN. 25, 1992
REDONGYU

PLATE OFFSETS (X=LEFT,Y=TOP):[j2=5,2.5],[j4=5,2.5],[j7=4,2],(SYMMETRIC PLATING TURNED ON)

BRACE FLAT PART OF T/C AT 24 IN O.C. FROM JTS 2 TO 4.



L. HL TO PK:5-10-10 R. HL TO PK :5-10-10
LEFT HEIGHT:0-4-3 SPAN:20-11-8 RISE:3-3-11 RIGHT HEIGHT:0-4-3

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LOADING (PSF)	L	D	MAX STRESSES	MINIMUM GRADE OF LUMBER
			TOP 1-2=0.669	TOP CHORD:2X4 No.1&Btr GR DF-L
TOP	25	10	BOTT 7-8=0.518	BOT CHORD:2X6 No.2 GR DF-L
BOTT	0	7	LL.DEFL.@7=0.12 < L/240	WEBS :2X4 STANDARD 91 HF

==========

STR.INC.: LUMB = 1.15 PLATE = 1.15 SPACING : 24.0 in. o. c.
REPETITIVE STRESSES NOT USED NO. OF MEMBERS = 1

LOADING	STRESS INCREASE LUMBER	PLATE	LOADING TYPE	PANEL(PLF) / JOINTS(LBS)			
1	1.15	1.15	UNIFORM	1- 2= 70	2- 4= 139	4- 5= 70	5- 1= 28
			CONCENTRATED	2= 358	4= 358		

CHORD: 2-3; 3-4 TO BE 2X6 No.2 GR DF-L
DEFLECTION(IN.) L.L= 0.12,D.L=0.08,T.L=0.21
PLATES ARE MITEK M20-165,142 MANUFACTURED FROM ASTM A 446 GRD A GALVANIZED STEEL(EXCEPT AS SHOWN)
PLATE MUST BE INSTALLED ON EA. FACE OF JOINT,SYMMETRICALLY(EXCEPT AS SHOWN)DESIGN CONFORMS W/NDS DESIGN SPECS, UBC-ICBO,TPI-91
THIS DESIGN IS FOR TRUSS FABRICATION ONLY.FOR PERMANENT AND TEMPORARY BRACING(WHICH IS ALWAYS REQD)CONSULT BLDG ARCHITECT OR ENGINEER

APPROVED
APR 2 [illegible]
CITY OF CORVALLIS
DEVELOPMENT SERVICES

APR-21-'98 07:37 ID:RELCO ROOF/FLOOR INC TEL NO:503 995-6311 #156 P03

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===<<<<ACES-32 Ver. 1.1>>>>===========['933390 ]============<<<<RELCO>>>>=========
  Customer : C-V-B                                         Sun Apr  5 13:06:10 1998
  Project #: 351                  Truss ID  : TG1          Family #   : 324
  Span     : 20-0                 Quantity  : 3            Top Pitch  : 6/12
==Bld(2/12/1998),v1.1=================================================================
```

TOP CHORD	BOTTOM CHORD	WEBS		REACTIONS - SIZE	
1-2=-23234	7-8= 17982	2-12= 3371	5-9= 5152	1=-11905	5.50
2-3=-19191	8-9= 17982	2-11=-4536	6-9=-2622	7=-9131	3.25
3-4=-17387	9-10= 14311	3-11= 6770	6-8= 1402		
4-5=-17387	10-11= 15258	3-10= 2862			
5-6=-17629	11-12= 21015	4-10=-241			
6-7=-19773	12-1= 21015	5-10= 4135			

APR 06 1998

PLATE OFFSETS (X=LEFT,Y=TOP):[j3=7,2.5],[J5=7,2.5],[j12=6,1.5],(SYMMETRIC PLATING TURNED ON)

BOTT CHORD SPLICES:9-10=10X10;

BRACE FLAT PART OF T/C AT 24 IN O.C. FROM JTS 3 TO 5.

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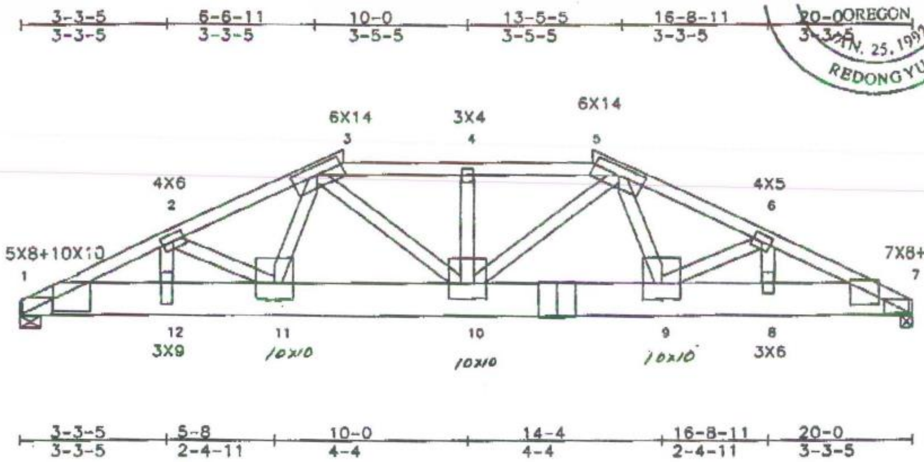


```
L. HL TO PK:7-3-15                                               R. HL TO PK :7-3-15
LEFT HEIGHT:0-4-3          SPAN:20-0          RISE:3-11-7        RIGHT HEIGHT:0-4-3
======================================================================================
LOADING   (PSF)        MAX STRESSES                   MINIMUM GRADE OF LUMBER
          L    D       TOP    1-2=0.669               TOP CHORD:2X4  No.1&Btr GR DF-L
TOP      25   10       BOTT 11-12=0.818               BOT CHORD:2X10 No.2 GR DF-L
BOTT      0    7       LL.DEFL.@10=0.18 < L/240  WEBS     :2X4  STANDARD 91 HF
======================================================================================
STR.INC.: LUMB = 1.15 PLATE = 1.15                    SPACING : 24.0 in. o. c.
REPETITIVE STRESSES NOT USED                          NO. OF MEMBERS = 3
```

LOADING	STRESS INCREASE LUMBER	PLATE	LOADING TYPE	PANEL(PLF) / JOINTS(LBS)
1	1.15	1.15	UNIFORM	1- 7= 70 7- 9= 507 9-10= 807 10- 1= 1240
			CONCENTRATED	9= 1245

CHORD: 10-11; 11-12; 12-1 TO BE 2X10 No.1&Btr GR DF-L
WEB: 3-11 TO BE 2X4 No.1&Btr GR DF-L
3 MEMBERS NAILED TOG. W/1 ROW(S) OF .131x3 in. NAILS 12 in. o.c(TOP CHS.),AND 16d NAILS 12 in. o.c.(BOTT. CHORD)
For Webs use 1 ROW of NAILS 12 in. o.c.
****NOTE:Special hanger/connection is reqd. to dist. bottom chord uniform load(s) to all plies.****
DEFLECTION(IN.) L.L= 0.18,D.L=0.12,T.L=0.30
PLATES ARE MITEK M20-165,142 MANUFACTURED FROM ASTM A 446 GRD A GALVANIZED STEEL(EXCEPT AS SHOWN)
PLATE MUST BE INSTALLED ON EA. FACE OF JOINT,SYMMETRICALLY(EXCEPT AS SHOWN)DESIGN CONFORMS W/NDS DESIGN SPECS, UBC-ICBO,TPI-91
THIS DESIGN IS FOR TRUSS FABRICATION ONLY.FOR PERMANENT AND TEMPORARY BRACING(WHICH IS ALWAYS REQD)CONSULT BLDG ARCHITECT OR ENGINEER

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DEVELOPMENT SERVICES

===<<<<ACES-32 Ver. 1.1>>>>==========[951546]==========<<<<RELCO>>>>=========

Customer : C-V-B		Thu Apr 9 17:05:14 1998
Project #: 351	Truss ID : T	Family # : 323
Span : 14-5-4	Quantity : 1	Top Pitch : 6/12

==Bld(2/12/1998),v1.1===

TOP CHORD	BOTTOM CHORD	WEBS	REACTIONS - SIZE	
1-2=-1908	5-6= 0	2-7= 133	1=-1235	5.50
2-3=-1504	6-7= 1695	2-6=-220	5=-1242	1.50
3-4=-1504	7-1= 1695	3-6=-631		
4-5=-1179		4-6= 1735		

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EXP. 6-30-98
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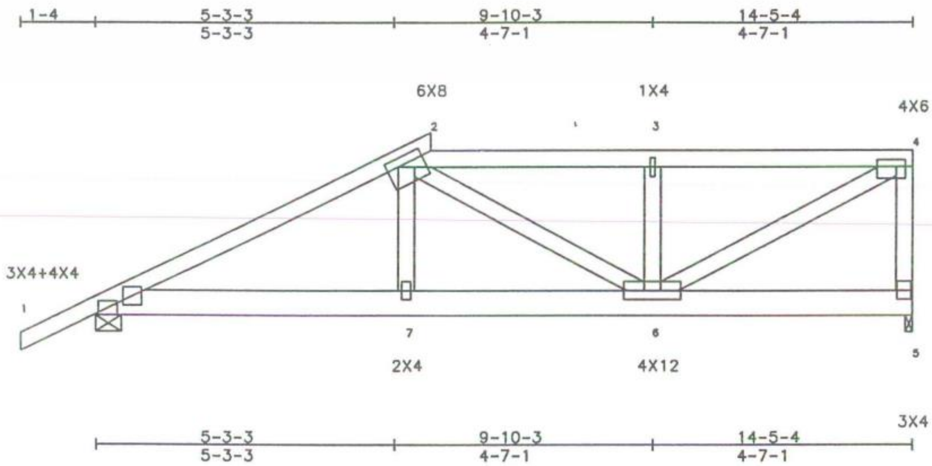
JAN. 25, 1992
REDONG YU

PLATE OFFSETS (X=LEFT,Y=TOP):[j2=4,2.5],[j4=4,1.5],(SYMMETRIC PLATING TURNED ON)

BRACE FLAT PART OF T/C AT 24 IN O.C. FROM JTS 2 TO 4.



L. HL TO PK:5-10-10
LEFT HEIGHT:0-4-3 SPAN:14-5-4 RISE:3-3-11 RIGHT HEIGHT:2-11-12

==

LOADING (PSF)			MAX STRESSES	MINIMUM GRADE OF LUMBER	
	L	D	TOP 2-3=0.676	TOP CHORD:2X4	No.1&Btr GR DF-L
TOP	25	10	BOTT 6-7=0.334	BOT CHORD:2X6	No.2 GR DF-L
BOTT	0	7	LL.DEFL.@6=0.04 < L/240	WEBS :2X4	STANDARD 91 HF

==

STR.INC.: LUMB = 1.15 PLATE = 1.15 SPACING : 24.0 in. o. c.
REPETITIVE STRESSES NOT USED NO. OF MEMBERS = 1

LOADING	STRESS INCREASE LUMBER	PLATE	LOADING TYPE	PANEL(PLF) / JOINTS(LBS)		
1	1.15	1.15	UNIFORM	1- 2= 70	2- 4= 139	5- 1= 28
			CONCENTRATED	2= 358		

DEFLECTION(IN.) L.L= 0.04,D.L=0.03,T.L=0.07
PLATES ARE MITEK M20-165,142 MANUFACTURED FROM ASTM A 446 GRD A GALVANIZED STEEL(EXCEPT AS SHOWN)
PLATE MUST BE INSTALLED ON EA. FACE OF JOINT,SYMMETRICALLY(EXCEPT AS SHOWN)DESIGN CONFORMS W/NDS DESIGN SPECS, UBC-ICBO,TPI-91
THIS DESIGN IS FOR TRUSS FABRICATION ONLY.FOR PERMANENT AND TEMPORARY BRACING(WHICH IS ALWAYS REQD)CONSULT BLDG ARCHITECT OR ENGINEER

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DEVELOPMENT SERVICES

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===&lt;&lt;&lt;&lt;ACES-32 Ver. 1.1&gt;&gt;&gt;&gt;===========[ 951547 ]===========&lt;&lt;&lt;&lt;RELCO&gt;&gt;&gt;&gt;==========
  Customer : C-V-B                                          Thu Apr  9 17:05:18 1998
  Project #: 351                 Truss ID   : T1            Family #    : 323
  Span     : 14-5-4              Quantity   : 1             Top Pitch : 6/12
==Bld(2/12/1998),v1.1=====================================================================
```

TOP CHORD	BOTTOM CHORD	WEBS	REACTIONS - SIZE	
1-2=-833	5-6= 0	2-7= 67	1=-706	5.50
2-3=-702	6-7= 740	2-6=-43	5=-594	1.50
3-4=-702	7-1= 740	3-6=-319		
4-5=-562		4-6= 810		

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APR 13 1998

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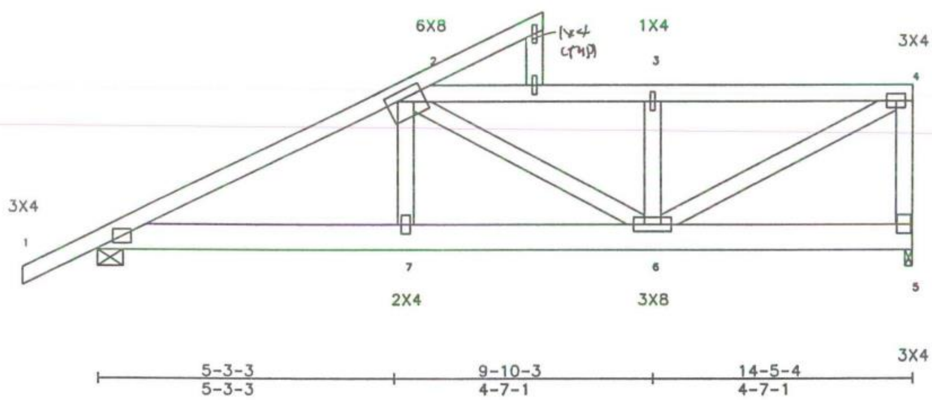
EXP. 6-30-98
OREGON
JAN. 25, 1992
REDONGYU

PLATE OFFSETS (X=LEFT,Y=TOP):[j2=4,2.5],(SYMMETRIC PLATING TURNED ON)

1-4 5-3-3 9-10-3 14-6-4
5-3-3 4-7-1 4-7-1

BRACE FLAT PART OF T/C AT 48 IN O.C. FROM JTS 2 TO 4.




```
L. HL TO PK:5-10-10
LEFT HEIGHT:0-4-3          SPAN:14-5-4       RISE:4-3-11       RIGHT HEIGHT:2-11-12
==========================================================================================
LOADING   (PSF)            MAX STRESSES                 MINIMUM GRADE OF LUMBER
        L     D            TOP    1-2=0.333             TOP CHORD:2X4  No.1&Btr GR DF-L
TOP    25    10            BOTT   6-7=0.146             BOT CHORD:2X6  No.2 GR DF-L
BOTT    0     7            LL.DEFL.@6=0.02 < L/240      WEBS     :2X4  STANDARD 91 HF
==========================================================================================
STR.INC.: LUMB = 1.15 PLATE = 1.15                   SPACING : 24.0 in. o. c.
REPETITIVE STRESSES USED                             NO. OF MEMBERS = 1
```

DEFLECTION(IN.) L.L= 0.02,D.L=0.01,T.L=0.03
PLATES ARE MITEK M20-165,142 MANUFACTURED FROM ASTM A 446 GRD A GALVANIZED STEEL(EXCEPT AS SHOWN)
PLATE MUST BE INSTALLED ON EA. FACE OF JOINT,SYMMETRICALLY(EXCEPT AS SHOWN)DESIGN CONFORMS W/NDS DESIGN SPECS, UBC-ICBO,TPI-91
THIS DESIGN IS FOR TRUSS FABRICATION ONLY.FOR PERMANENT AND TEMPORARY BRACING(WHICH IS ALWAYS REQD)CONSULT BLDG ARCHITECT OR ENGINEER

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DEVELOPMENT SERVICES

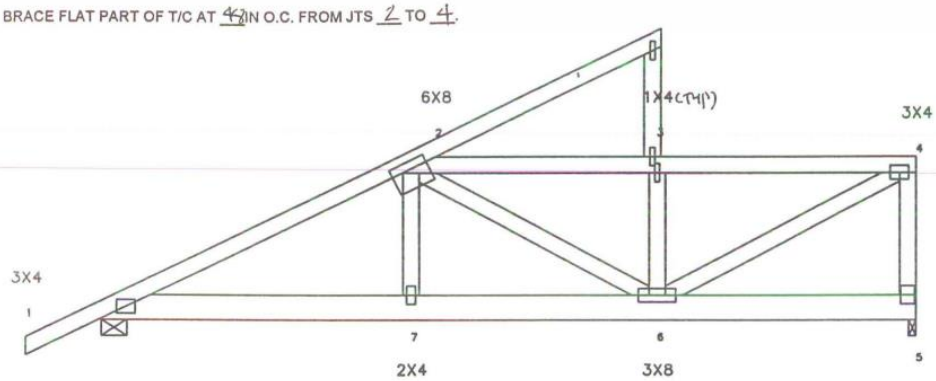
```
===<<<<ACES-32 Ver. 1.1>>>>==========[ 951548 ]==========<<<<RELCO>>>>=========
  Customer  : C-V-B                                      Thu Apr  9 17:05:22 1998
  Project #: 351                Truss ID   : T2          Family #   : 323
  Span      : 14-5-4            Quantity   : 1           Top Pitch  : 6/12
==Bld(2/12/1998),v1.1==========================================================
```

TOP CHORD	BOTTOM CHORD	WEBS	REACTIONS - SIZE		APPROVED FOR MITEK INDUSTRIES INC.
1-2=-833	5-6= 0	2-7= 67	1=-706	5.50	
2-3=-702	6-7= 740	2-6=-43	5=-594	1.50	
3-4=-702	7-1= 740	3-6=-319			
4-5=-562		4-6= 810			

PLATE OFFSETS (X=LEFT,Y=TOP):[j2=4,2.5],(SYMMETRIC PLATING TURNED ON)

1-4 | 5-3-3 / 5-3-3 | 9-10-3 / 4-7-1 | 14-5-4 / 4-7-1

BRACE FLAT PART OF T/C AT 48 IN O.C. FROM JTS 2 TO 4.



5-3-3 / 5-3-3 | 9-10-3 / 4-7-1 | 14-5-4 / 4-7-1

```
L. HL TO PK:5-10-10
LEFT HEIGHT:0-4-3        SPAN:14-5-4        RISE:5-3-11        RIGHT HEIGHT:2-11-12
=================================================================================
LOADING   (PSF)          MAX STRESSES                 MINIMUM GRADE OF LUMBER
         L    D          TOP    1-2=0.333             TOP CHORD:2X4  No.1&Btr GR DF-L
TOP     25   10          BOTT   6-7=0.146             BOT CHORD:2X6  No.2 GR DF-L
BOTT     0    7          LL.DEFL.@6=0.02 < L/240      WEBS     :2X4  STANDARD 91 HF
=================================================================================
STR.INC.: LUMB = 1.15 PLATE = 1.15                    SPACING : 24.0 in. o. c.
REPETITIVE STRESSES USED                              NO. OF MEMBERS = 1
```

DEFLECTION(IN.) L.L= 0.02,D.L=0.01,T.L=0.03
PLATES ARE MITEK M20-165,142 MANUFACTURED FROM ASTM A 446 GRD A GALVANIZED STEEL(EXCEPT AS SHOWN) APR 13 1998
PLATE MUST BE INSTALLED ON EA. FACE OF JOINT,SYMMETRICALLY(EXCEPT AS SHOWN)DESIGN CONFORMS W/NDS DESIGN SPECS, UBC-ICBO,TPI-91
THIS DESIGN IS FOR TRUSS FABRICATION ONLY.FOR PERMANENT AND TEMPORARY BRACING(WHICH IS ALWAYS REQD)CONSULT BLDG ARCHITECT OR ENGINEER.

PROFESSIONAL ENGINEER
EXP. 6-30-98
OREGON
JAN. 25, 1992
REDONGYU

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APR 23 1998
CITY OF CORVALLIS
DEVELOPMENT SERVICES

===<<<<ACES-32 Ver. 1.1>>>>==========[951536]==========<<<<RELCO>>>>=========

Customer : C-V-B
Project #: 351
Span : 20-0

Truss ID : D
Quantity : 1

Thu Apr 9 17:04:15 1998
Family # : 324
Top Pitch : 6/12

==Bld(2/12/1998),v1.1==

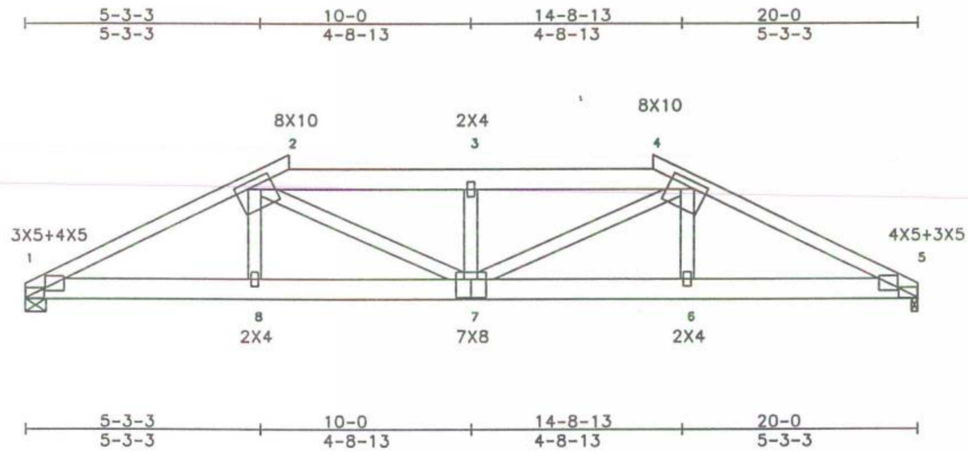
TOP CHORD	BOTTOM CHORD	WEBS	REACTIONS - SIZE	
1-2=-3073	5-6= 2781	2-8= 135	1=-1650	5.50
2-3=-3569	6-7= 2781	2-7= 952	5=-1639	1.75
3-4=-3569	7-8= 2733	3-7=-756		
4-5=-3107	8-1= 2733	4-7= 898		
		4-6= 138		

APPROVED FOR MITEK INDUSTRIES INC.

PROFESSIONAL ENGINEER 15,766 EXP. 6-30-98 OREGON JAN. 25, 1992 REDONGYU

PLATE OFFSETS (X=LEFT,Y=TOP):[j2=5,2.5],[j4=5,2.5],[j7=4,2],(SYMMETRIC PLATING TURNED ON)

BRACE FLAT PART OF T/C AT 24 IN O.C. FROM JTS 2 TO 4.



L. HL TO PK:5-10-10
LEFT HEIGHT:0-4-3

SPAN:20-0

RISE:3-3-11

R. HL TO PK :5-10-10
RIGHT HEIGHT:0-4-3

===

LOADING (PSF)	L	D	MAX STRESSES	MINIMUM GRADE OF LUMBER	
TOP	25	10	TOP 4-5=0.649	TOP CHORD:2X4	No.1&Btr GR DF-L
BOTT	0	7	BOTT 6-7=0.491	BOT CHORD:2X6	No.2 GR DF-L
			LL.DEFL.@7=0.11 < L/240	WEBS :2X4	STANDARD 91 HF

===

STR.INC.: LUMB = 1.15 PLATE = 1.15
REPETITIVE STRESSES NOT USED

SPACING : 24.0 in. o. c.
NO. OF MEMBERS = 1

LOADING	STRESS INCREASE LUMBER	PLATE	LOADING TYPE	PANEL(PLF) / JOINTS(LBS)			
1	1.15	1.15	UNIFORM	1- 2= 70	2- 4= 139	4- 5= 70	5- 1= 28
			CONCENTRATED	2= 358	4= 358		

CHORD: 2-3; 3-4 TO BE 2X6 No.2 GR DF-L
DEFLECTION(IN.) L.L= 0.11,D.L=0.07,T.L=0.18
PLATES ARE MITEK M20-165,142 MANUFACTURED FROM ASTM A 446 GRD A GALVANIZED STEEL(EXCEPT AS SHOWN)
PLATE MUST BE INSTALLED ON EA. FACE OF JOINT,SYMMETRICALLY(EXCEPT AS SHOWN)DESIGN CONFORMS W/NDS DESIGN SPECS, UBC-ICBO,TPI-91
THIS DESIGN IS FOR TRUSS FABRICATION ONLY.FOR PERMANENT AND TEMPORARY BRACING(WHICH IS ALWAYS REQD)CONSULT BLDG ARCHITECT OR ENGINEER

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CITY OF CORVALLIS
DEVELOPMENT SERVICES

```
===<<<<ACES-32 Ver. 1.1>>>>==========[ 951538 ]==========<<<<RELCO>>>>=========
  Customer : C-V-B                                         Thu Apr  9 17:04:22 1998
  Project #: 351                                           Family #  : 324
  Span     : 14-7-8              Truss ID   : E            Top Pitch : 6/12
                                 Quantity   : 1
==Bld(2/12/1998),v1.1==========================================================
 TOP CHORD     BOTTOM CHORD    WEBS          REACTIONS - SIZE      APPROVED FOR MITEK INDUSTRIES INC.
 1-2=-1707     5-6= 1509       2-8= 73       1=-1047     5.50      APR 13 1998
 2-3=-2143     6-7= 1509       2-7= 683      5=-1047     5.50
 3-4=-2143     7-8= 1509       3-7=-419
 4-5=-1707     8-1= 1509       4-7= 683
                               4-6= 73

PLATE OFFSETS (X=LEFT,Y=TOP):[j2=4,2.5],[j4=4,2.5],(SYMMETRIC PLATING TURNED ON)
```

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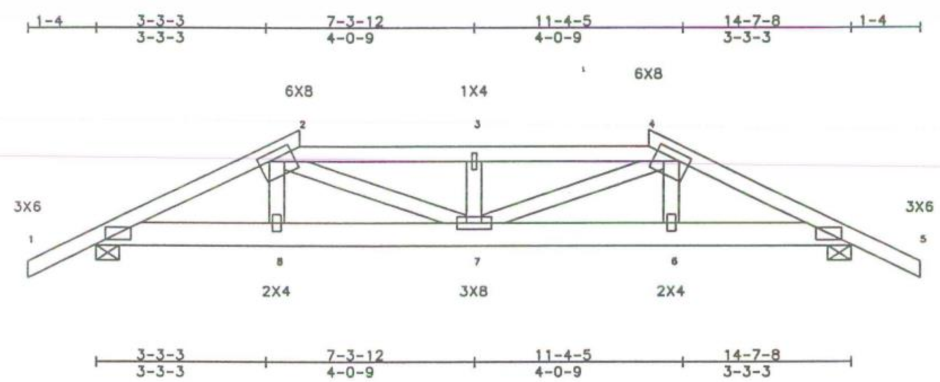
18,766
OREGON
JAN. 25, 1992
REDONG YU

BRACE FLAT PART OF T/C AT 24 IN O.C. FROM JTS 2 TO 4.




```
L. HL TO PK:3-7-13                                          R. HL TO PK :3-7-13
LEFT HEIGHT:0-4-3         SPAN:14-7-8        RISE:2-3-11    RIGHT HEIGHT:0-4-3
================================================================================
LOADING   (PSF)         MAX STRESSES              MINIMUM GRADE OF LUMBER
          L    D        TOP   2-3=0.438           TOP CHORD:2X4  No.1&Btr GR DF-L
TOP       25   10       BOTT  6-7=0.264           BOT CHORD:2X6  No.2 GR DF-L
BOTT      0    7        LL.DEFL.@7=0.06 < L/240   WEBS     :2X4  STANDARD 91 HF
================================================================================
STR.INC.: LUMB = 1.15 PLATE = 1.15                   SPACING : 24.0 in. o. c.
REPETITIVE STRESSES NOT USED                         NO. OF MEMBERS = 1

LOADING STRESS INCREASE  LOADING        PANEL(PLF) / JOINTS(LBS)
        LUMBER  PLATE    TYPE
  1     1.15    1.15     UNIFORM        1- 2= 70   2- 4= 104   4- 5= 70   5- 1= 21
                         CONCENTRATED      2= 157     4= 157

DEFLECTION(IN.) L.L= 0.06,D.L=0.04,T.L=0.10
PLATES ARE MITEK M20-165,142 MANUFACTURED FROM ASTM A 446 GRD A GALVANIZED STEEL(EXCEPT AS SHOWN)
PLATE MUST BE INSTALLED ON EA. FACE OF JOINT,SYMMETRICALLY(EXCEPT AS SHOWN)DESIGN CONFORMS W/NDS DESIGN SPECS, UBC-ICBO,TPI-91
THIS DESIGN IS FOR TRUSS FABRICATION ONLY.FOR PERMANENT AND TEMPORARY BRACING(WHICH IS ALWAYS REQD)CONSULT BLDG ARCHITECT OR ENGINEER
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APR 2 0 1998
CITY OF CORVALLIS
DEVELOPMENT SERVICES

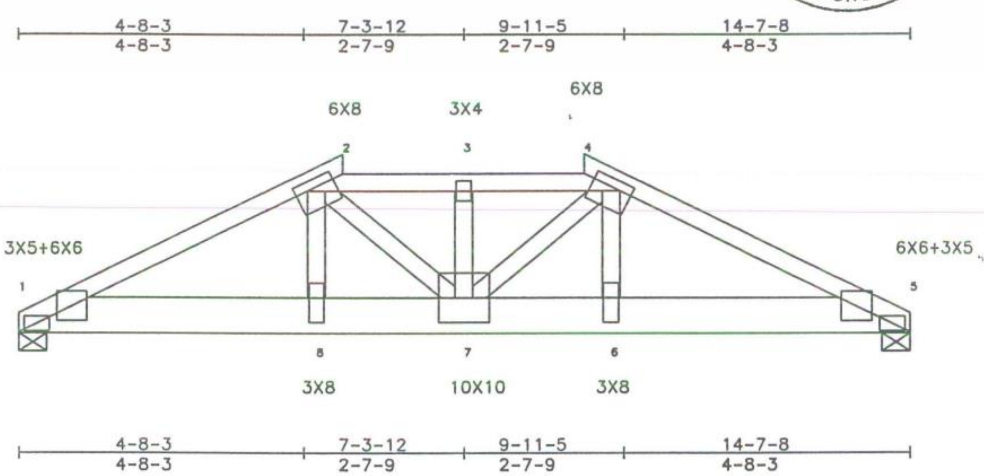
```
===<<<<ACES-32 Ver. 1.1>>>>==========[ 951539 ]==========<<<<RELCO>>>>=========
  Customer : C-V-B                                            Thu Apr  9 17:04:27 1998
  Project #: 351                 Truss ID   : G               Family #    : 324
  Span     : 14-7-8              Quantity   : 2               Top Pitch : 6/12
==Bld(2/12/1998),v1.1==========================================================
 TOP CHORD     BOTTOM CHORD    WEBS          REACTIONS - SIZE        APPROVED FOR MITEK INDUSTRIES INC.
 1-2=-7774     5-6= 6945       2-8= 2299     1=-5095     5.50
 2-3=-8057     6-7= 6945       2-7= 1460     5=-5095     5.50
 3-4=-8057     7-8= 6945       3-7=-184
 4-5=-7774     8-1= 6945       4-7= 1460
                               4-6= 2299

PLATE OFFSETS (X=LEFT,Y=TOP):[j2=4,2.5],[j4=4,2.5],[j6=5,1.5],[j8=5,1.5],(SYMMETRIC PLATING TURNED ON)
```

APR 15 1998

PROFESSIONAL ENGINEER
ENGINEER
15766
EXP 6-30-98
OREGON
JAN. 25, 1992
REDONG YU

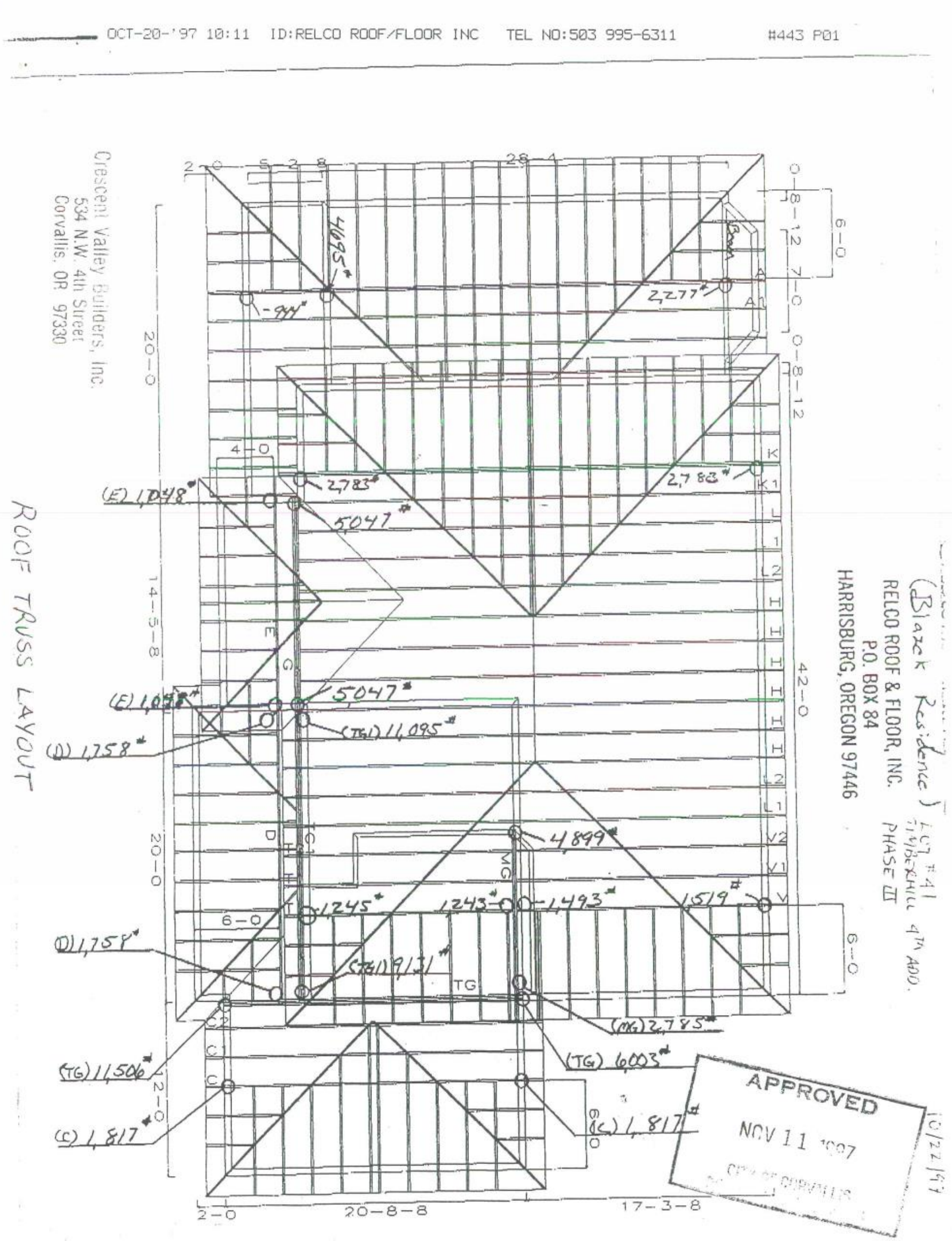
BRACE FLAT PART OF T/C AT 24 IN O.C. FROM JTS 2 TO 4.




```
L. HL TO PK:5-2-13                                            R. HL TO PK :5-2-13
LEFT HEIGHT:0-4-3          SPAN:14-7-8        RISE:3-0-3      RIGHT HEIGHT:0-4-3
================================================================================
LOADING   (PSF)          MAX STRESSES               MINIMUM GRADE OF LUMBER
          L     D        TOP    1-2=0.410            TOP CHORD:2X4  No.1&Btr GR DF-L
TOP      25    10        BOTT   7-8=0.872            BOT CHORD:2X8  No.2 GR DF-L
BOTT      0     7        LL.DEFL.@7=0.08 < L/240     WEBS     :2X4  STANDARD 91 HF
================================================================================
STR.INC.: LUMB = 1.15 PLATE = 1.15                   SPACING : 24.0 in. o. c.
REPETITIVE STRESSES NOT USED                         NO. OF MEMBERS = 2

LOADING STRESS INCREASE  LOADING       PANEL(PLF) / JOINTS(LBS)
        LUMBER  PLATE    TYPE
   1     1.15   1.15     UNIFORM        1- 5= 70     5- 1= 649
```

2 MEMBERS NAILED TOG. W/1 ROW(S) OF .131x3 in. NAILS 12 in. o.c.(TOP CHS.),AND 2 ROW(S) OF .131x3 in. NAILS 5 in. o.c(BOTT. CHS.)
For Webs use 1 ROW of NAILS 12 in. o.c.
DEFLECTION(IN.) L.L= 0.08,D.L=0.06,T.L=0.14
PLATES ARE MITEK M20-165,142 MANUFACTURED FROM ASTM A 446 GRD A GALVANIZED STEEL(EXCEPT AS SHOWN)
PLATE MUST BE INSTALLED ON EA. FACE OF JOINT,SYMMETRICALLY(EXCEPT AS SHOWN)DESIGN CONFORMS W/NDS DESIGN SPECS, UBC-ICBO,TPI-91
THIS DESIGN IS FOR TRUSS FABRICATION ONLY.FOR PERMANENT AND TEMPORARY BRACING(WHICH IS ALWAYS REQD)CONSULT BLDG ARCHITECT OR ENGINEER



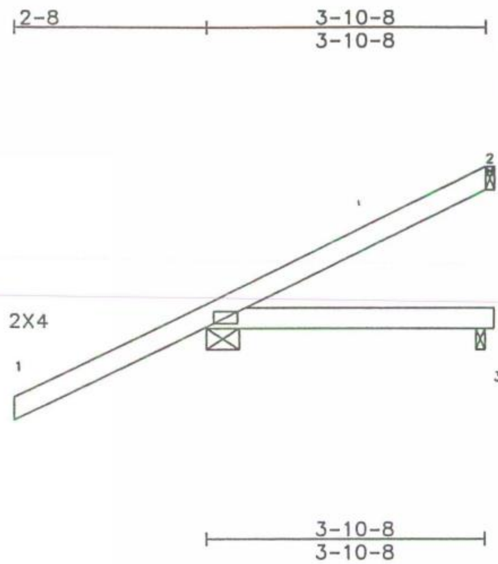
OCT-20-'97 10:11 ID:RELCO ROOF/FLOOR INC TEL NO:503 995-6311 #443 P01
ROOF TRUSS LAYOUT
Crescent Valley Builders, Inc.
534 N.W. 4th Street
Corvallis, OR 97330
(Blazek Residence)
RELCO ROOF & FLOOR, INC.
P.O. BOX 84
HARRISBURG, OREGON 97446
PHASE III
APPROVED

```
===<<<<ACES-32 Ver. 1.1>>>>==========[ 951537 ]==========<<<<RELCO>>>>=========
  Customer : C-V-B                                      Thu Apr  9 17:04:20 1998
  Project #: 351                 Truss ID  : Q          Family #  : 205
  Span     : 3-10-8              Quantity  : 4          Top Pitch : 6/12
==Bld(2/12/1998),v1.1=========================================================
 TOP CHORD    BOTTOM CHORD   WEBS         REACTIONS - SIZE     APPROVED FOR MITEK INDUSTRIES INC.
 1-2= 0       3-1= 0                      1=-375    5.50
 2-3= 0                                   3=-25     1.50
                                          2=-125    1.50

(SYMMETRIC PLATING TURNED ON)
```

APR 13 1998

PROFESSIONAL ENGINEER
ENGINEER
15766
EXP. 6-30-98
OREGON
JAN. 25, 1992
REDONG YU




```
L. HL TO PK:4-4
LEFT HEIGHT:0-4-3        SPAN:3-10-8        RISE:2-3-7        RIGHT HEIGHT:2-3-7
==============================================================================
LOADING   (PSF)          MAX STRESSES              MINIMUM GRADE OF LUMBER
          L    D         TOP   1-2=0.189           TOP CHORD:2X4  No.1&Btr GR DF-L
TOP      25   10         BOTT  3-1=0.038           BOT CHORD:2X4  No.1&Btr GR DF-L
BOTT      0    7         LL.DEFL. < L/240          WEBS     :2X4  STANDARD 91 HF
==============================================================================
STR.INC.: LUMB = 1.15 PLATE = 1.15                 SPACING : 24.0 in. o. c.
REPETITIVE STRESSES USED                           NO. OF MEMBERS = 1

DEFLECTION(IN.) L.L= 0.00,D.L=0.00,T.L=0.00
PLATES ARE MITEK M20-165,142 MANUFACTURED FROM ASTM A 446 GRD A GALVANIZED STEEL(EXCEPT AS SHOWN)
PLATE MUST BE INSTALLED ON EA. FACE OF JOINT,SYMMETRICALLY(EXCEPT AS SHOWN)DESIGN CONFORMS W/NDS DESIGN SPECS, UBC-ICBO,TPI-91
THIS DESIGN IS FOR TRUSS FABRICATION ONLY.FOR PERMANENT AND TEMPORARY BRACING(WHICH IS ALWAYS REQD)CONSULT BLDG ARCHITECT OR ENGINEER.
```

APPROVED
APR 2 3 1998
CITY OF CORVALLIS
DEVELOPMENT SERVICES

1
97-95

Crescent Valley Builders, Inc.
534 N.W. 4th Street
Corvallis, OR 97330

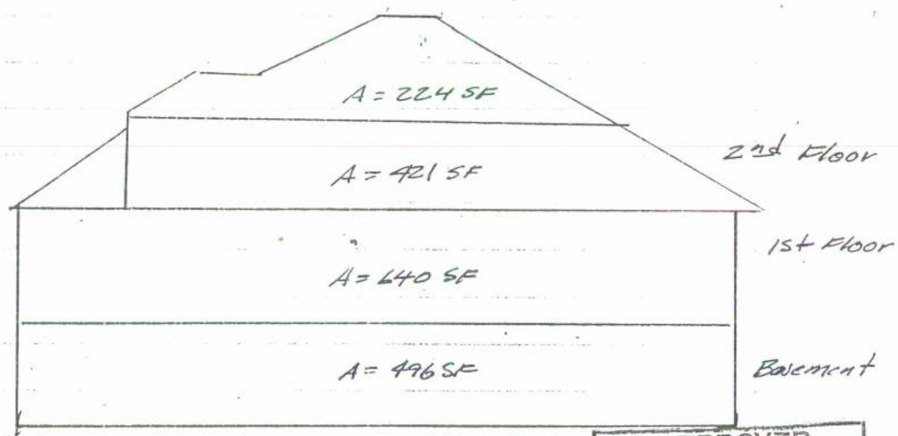
Oct 29, 1997

Crescent Valley Builders
534 NW 4th St
Corvallis Oregon
97330

757-0076
753-4940 FAX

REGISTERED PROFESSIONAL ENGINEER
4265
OREGON
MAY 2, 1958
Gordon Bakewell Wallace
Renew 6-30-98

Scott / Blazek Residence
Timberhill 4th Addition Phase III
Lot #41



APPROVED
NOV 11 1997
CITY OF CORVALLIS
DEVELOPMENT SERVICES

Front to Rear

Wind 80 MPH

Roof LL+DL = 25+15 = 40 psf

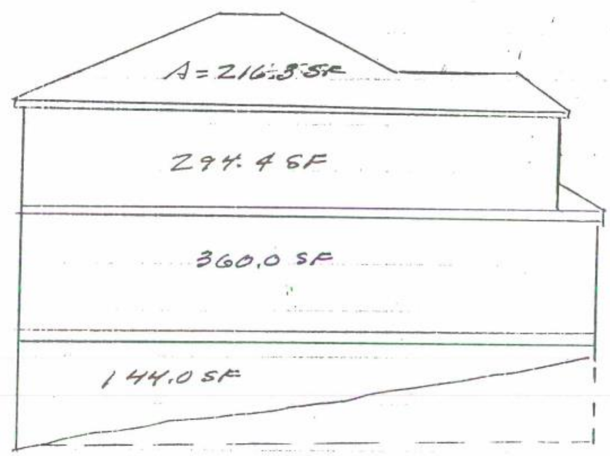
Floor LL+DL = 40+15 = 55 psf.

RECEIVED
NOV 07 '97
DEVELOPMENT SERVICES

ADDENDUM

Scott

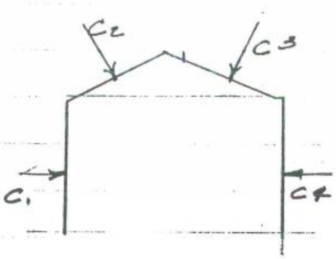
2
97-95



Left to Right

80 MPH

$q = 12.0$ psf



$C_1 = 0.70$ $p_1 = 8.4$ psf

$C_2 = -1.40$ $p_2 = -16.8$ psf

$C_3 = -1.00$ $p_3 = -12.0$ psf

$C_4 = -0.95$ $p_4 = -11.4$ psf

$C_1 + C_4 = (0.70) + (0.95) = 1.65$

$P_{1+4} = (1.65)(12) = 19.8$ PSF.

Scott

$\frac{3}{97\text{-}95}$

Wind Force

Front to Rear

Force on Roof = (224)(19.8) = 4435.2 #

Force on 2nd Floor = (421)(19.8) = 8335.8 #

Force on 1st Floor = (640)(19.8) = 12672 #

Force on Daylite Basement =

= (496)(19.8) = 5654.4# Suction

Force on 2nd Floor Ceiling

$= 4435 + \frac{8336}{2} = 8603\#$

Force on 1st Floor Ceiling

$= 4435 + 8336 + \frac{12672}{2} = 19107\#$

Force on First Floor

$= 4435 + 8336 + 12672 + \frac{5654}{2} = 28270\#$

Force on Top of Foundation

= 4435 + 8336 + 12672 + 5654 = 31097 #

62 - ½" Anchor Bolts

Scott

4
92-95

Wind Force

Left to Right

Force on Roof = (216)(19.8) = 4276.8 #

Force on 2nd Floor = (294)(19.8) = 5821.2 #

Force on 1st Floor = (360) (19.8) = 7128.0 #

Force on Basement = (144) (19.8) = 2851.2 #

Force on 2nd Floor Ceiling

$= 4277 + \frac{5821}{2} = 7187.5$ #

Force on 1st Floor Ceiling

$= 4277 + 5821 + \frac{7128}{2} = 13662$ #

Force on 1st Floor

$= 4277 + 5821 + 7128 + \frac{2851.2}{2} = 18651.6$ #

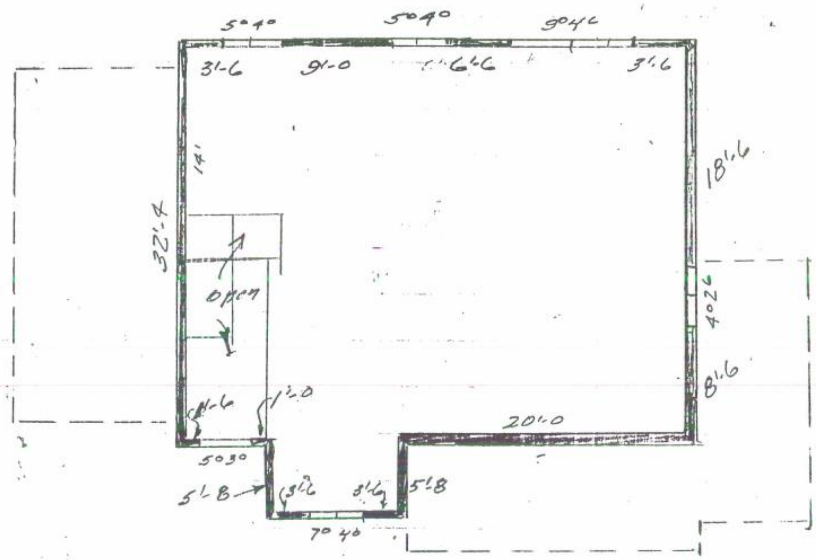
Force on top of Foundation

$= 4277 + 5821 + 7128 + 2851 = 20077$ #

Scott

5
97-95

2nd Floor



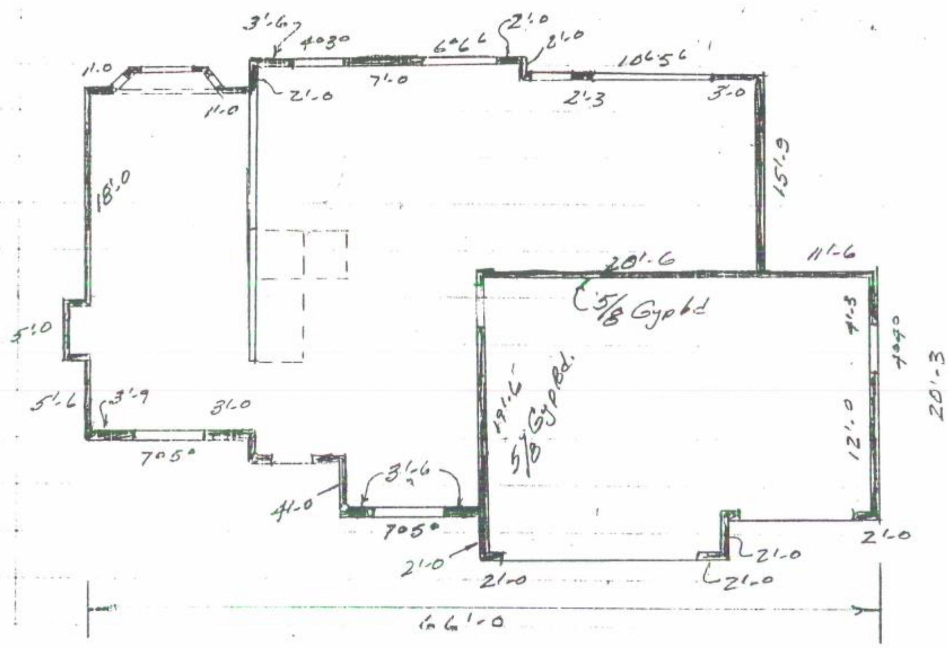
Front

Rear Wall	3.5+9+6.5+3.5	=	22.5LF
Front Wall	1.5+1.0+3.5+3.5+20	=	29.5LF
			52.0LF
Left Wall	14.0+5.67+5.67	=	25.34LF
Right Wall	18.5+8.5	=	27.00LF
			52.34LF

Scott

6
97-95

1st Floor



Front to Rear Walls.

$5.5 + 18 + 2 + 2 + 4 = 31.5$ LF

$2 + 2 + 12 + 4.25 + 15.75 = 36.0$ LF

67.5 LF

5/8 Gypbd $\frac{19.5}{6}$ = 3.25 Equivalent to ½" C.D.X

70.75 LF

Left to Right Walls

$3.5 + 7.0 + 2 + 2.25 + 3 = 17.75$ LF

$3.75 + 3.0 + 3.5 + 3.5 + 11.5 = 25.25$

43.0 LF

$\left(\frac{20.5}{6}\right)$ Gypbd = 3.42 Equivalent

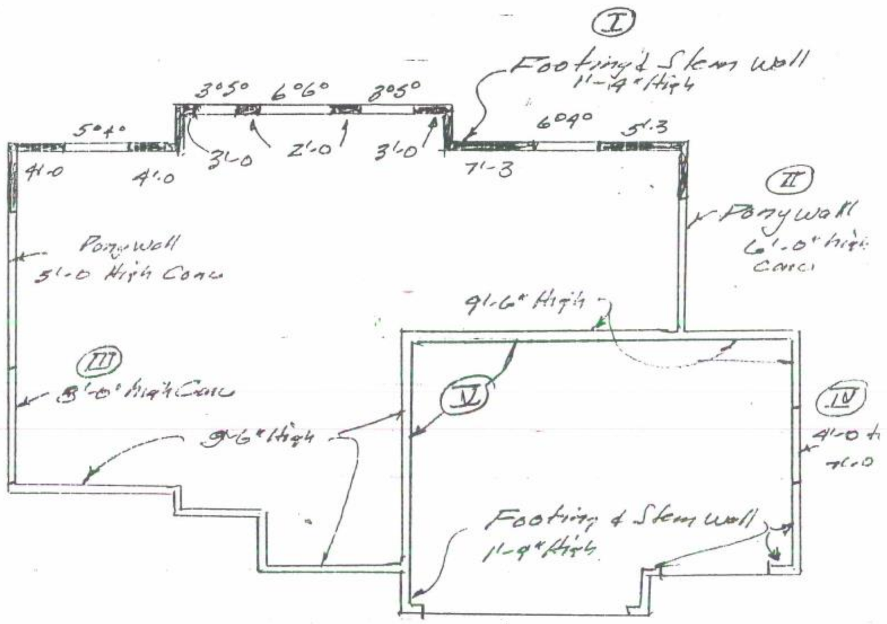
46.48 LF

Scott

7

97-95

Basement



Front

I Daylite Basement 6" Thick

II Pony 2'-0-6'-0 High 8" Thick

III Basement Wall 8'-0" High 8" Thick

IV Wall 4'-0" to 7'-0 High 8" Thick

V Basement Wall to Garage 8" Thick

Note: Horizontal #4 Rebar to be continuous around Corners.

Front to Rear.

L. wall 28'-4

Center Wall 22'-0

Right Wall 15'-9 + 20'-3

= 36'-0

E = 86'-4

Left to Right

Front Fdn 66'-6 LF

Rear Wall 4+4+3+2+2

+3+5.5+7 = 30.5

97.0 LF

Scott

8
97-95

2nd Floor Ext. Wall Sheathing

Front to Rear

Shear in Sheathing = $\frac{8603}{52.34}$ = 164.37 #/LF

Left to Right

Shear in Sheathing = $\frac{7188}{52}$ = 158.23 #/LF

Second Floor Sheathing

1/2 Plywood or equal

Nails: 6" OC Perimeter

16" OC Field

7d Common.

1st Floor Exterior Wall Sheathing

Front to Rear

Shear in Sheathing = $\frac{19107}{70.75}$ = 270.06 #/LF

Left To Right

Shear in Sheathing = $\frac{13662}{46.48}$ = 293.93 #/LF

Use 1/2" Plywood Sheathing or equal

Nail: 6" OC Perimeter

16" OC Field

7d Common

Capacity 300 #/LF

Block all Edges

Scott

9
97-95

Basement Walls

Front to Rear. Force = 28270 #

Shear Force = $\frac{28270}{86}$ = 328.7 #/LF

Left to Right Force = 18652 #

Shear Force = $\frac{18652}{93.5}$ = 199.5 #/LF

Assume: Average Weight on Foundation = 400 #/LF

Friction of Wood on Conc.

C_f = 0.35

F_f = (0.35)(400) = 140 #/LF

Net loading on Anchor Bolts = 329 − 140 = 189 #/LF max

Anchor Bolts: Parallel Grain 960 #

1½" Plates Normal to Grain 480 #

½" Bolts

Note that the first floor system will transfer Wind loading to all full height basement Walls.

Wind loading on Foundation

Front to Rear 31097 #

Left to Right 20077 #

Scott

10
97-95

Use 1/2 x10 Anchor Bolts ←

Full Height Basement Walls III, IV, V 8"
1/2" x10" Bolts @ 3'-0" oc & 1'-0 from corners

Pony Walls II 8"
1/2" x10" Bolts @ 6'-0" oc and ends of plates

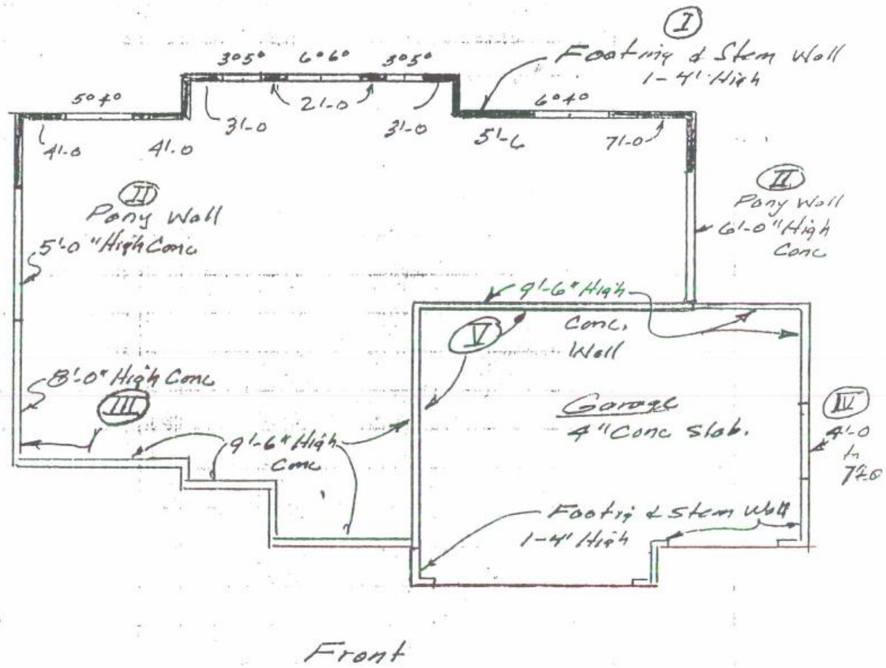
Daylite Basement Wall I 6"
1/2" x10" Bolts @ 6'-0" oc and 1'-0" from Corners

Crescent Valley Builders

7
97-20

BLAZEK / SCOTT

Basement



I Daylite Basement 6" Thick

II Pony Wall 2'-0 – 6'-0" High 8" Thick

III Basement Wall 8'-0" High 8" Thick

IV Wall 4'-0 – 7'-0" High 8" Thick

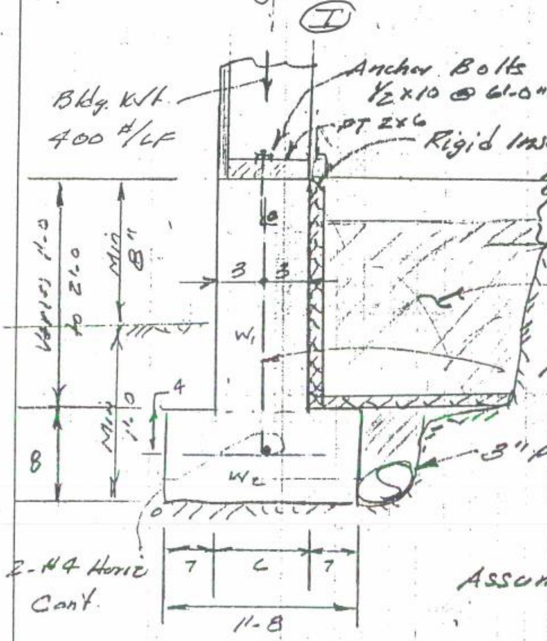
V Basement Wall to Garage 8" Thick

Note: Horiz #4 Rebar to be Continuous around Corners.

Front to Rear
L Wall 28'-4"
Center Wall 22'-0
R. Wall 15'-9 + 20'-3 = 36'-0
86'-4"

Left to Right
Front Fdn. Wall 68'-0
Rear wall 4+4+3 +2+2 +3+5.5+7 30.5
93.5'

Crescent Valley Bldrs 3-18-97 97.20



REGISTERED PROFESSIONAL ENGINEER 4265 OREGON MAY 2, 1958 Gordon Bakewell Wallace

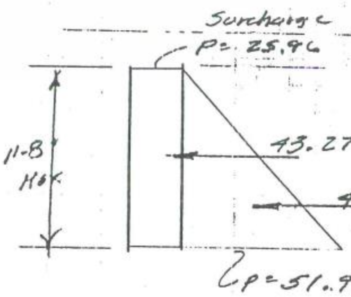
Renew 6-30-98

Assume: Floor LL 50 psf
Floor DL 50 psf

Backfill wt 120 pcf
$\phi = 36°$

$W_1 = 150$ #/LF
$W_2 = 167$ #/LF
317 #/LF

$P = wh \tan^2(45° \pm \frac{\phi}{2})$
$= (120)h \tan^2(45° - \frac{36}{2})$
$= (120h)(0.2596) = 31.154\,h$



Moment at Top of Footing

$M_1 = (43.27)\left(\frac{1.667}{2}\right) = 36.06$ ft#/LF
$M_2 = (43.27)\left(\frac{1.667}{3}\right) = 24.04$ ft#/LF
60.10 ft#/LF

0.0601 ftK/LF
$= 0.7212$ inK/LF

$A_s = \frac{M}{f_s j d} = \frac{0.7212}{(20)(0.87)(3)} = 0.014$ in²/LF

#4 Verticals @ 2'-0" o.c.

97-20

Check Over turning.

Overturning:

$(43.27)\left(\frac{1.667}{2}+0.667\right) = 64.92$ ft#/ft

$(43.27)\left(\frac{1.667}{3}+0.667\right) = 52.89$

117.81 ft#/ft

Resisting: Ftg. $(317)\left(\frac{1.667}{2}\right) = 264.17$ ft#

backfill $(0.58)(2.0)(120)\left(\frac{16\frac{1}{2}}{12}\right) = 191.4$ ft#

455.57 ft#/ft

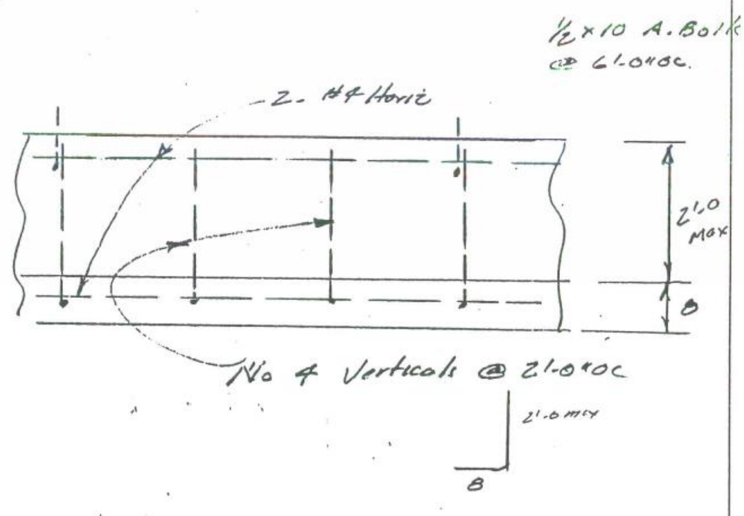
Bld. $(400)\left(\frac{1.667}{2}\right) = 333.33$ ft#/ft

788.90 ft#/ft

$\Sigma Wt = 317 + 95.7 + 400$

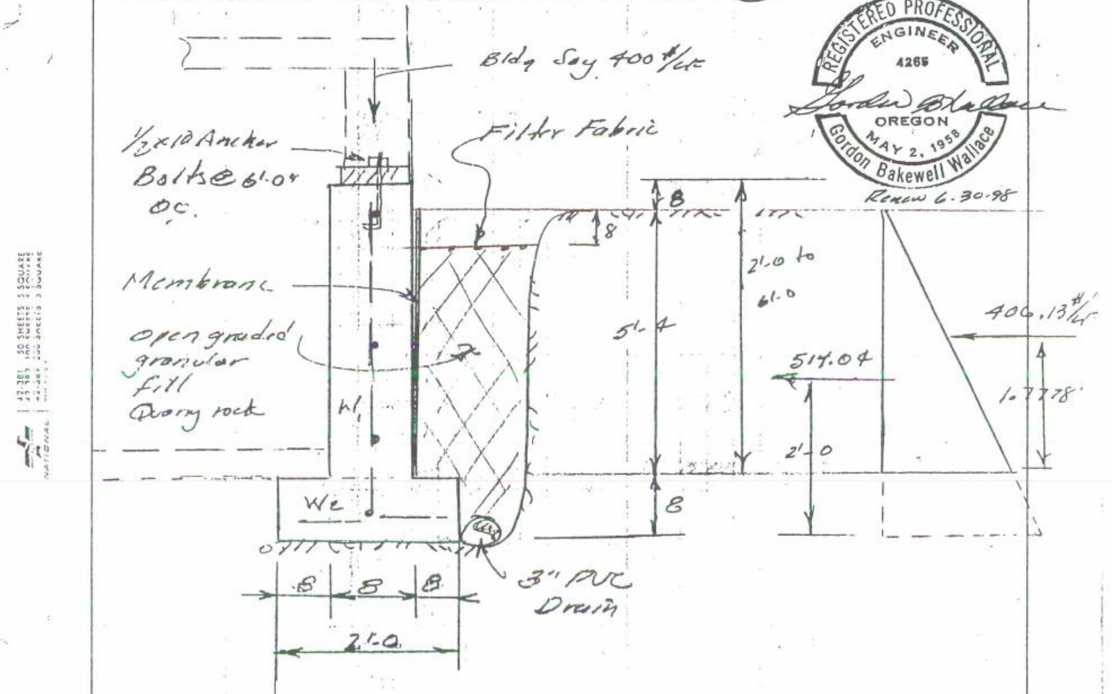
$= 812.7$ #/ft

$Av. p = \left(\frac{812.7}{1.667}\right) = 487.62$ psf.



Crescent Valley Bldg — 3-18-97 — 97.20

Foundation Wall 2'-0 to 6'-0 High (II)



Assume: Soil wt 110 #/ft³
Backfill ϕ 30°

$P = wh \tan^2 (45° \pm \frac{\phi}{2})$

$= (110h) \tan^2 (45° \pm \frac{30}{2}) = (0.2596)(110)h = 28.558\,h$

$W_1 = 600$ #/LF

$W_2 = 200$ #/LF

Overturning Moment about "O" = (2)(514) = 1028 '#

Resisting Moment

Bldg	(400)(1.0)	= 480.0 #/LF
W_1	(600)(1.0)	= 720.0
W_2	(200)(1.0)	= 240.0
Backfill	(0.667)(110)(4)	= 293.3
		1733.3 ft #/LF

6'-0" High Fdn

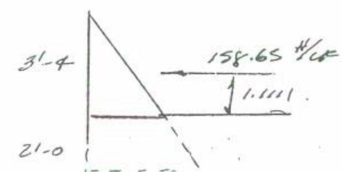
97-26

Moment at Top of Footing

$$M = (406.1)(1.78) = 722.91\ '^{\#}/_{LF} = 0.72\ '^{k}/_{LF} = 8.67\ ^{in\,k}/_{LF}$$

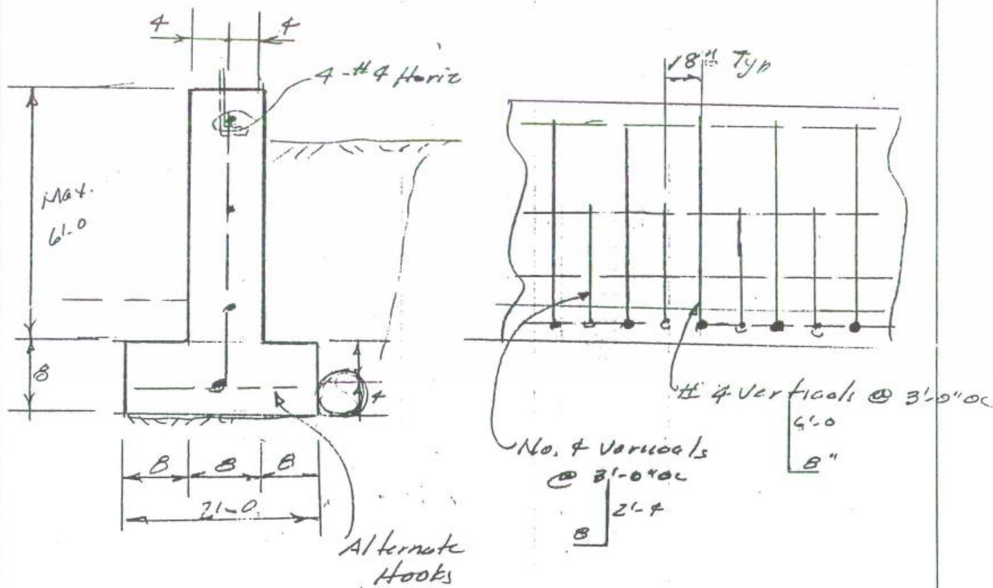
$$A_s = \frac{M}{f_s j d} = \frac{8.67}{(20)(0.87)(4)} = 0.125\ ^{in^2}/_{LF}$$

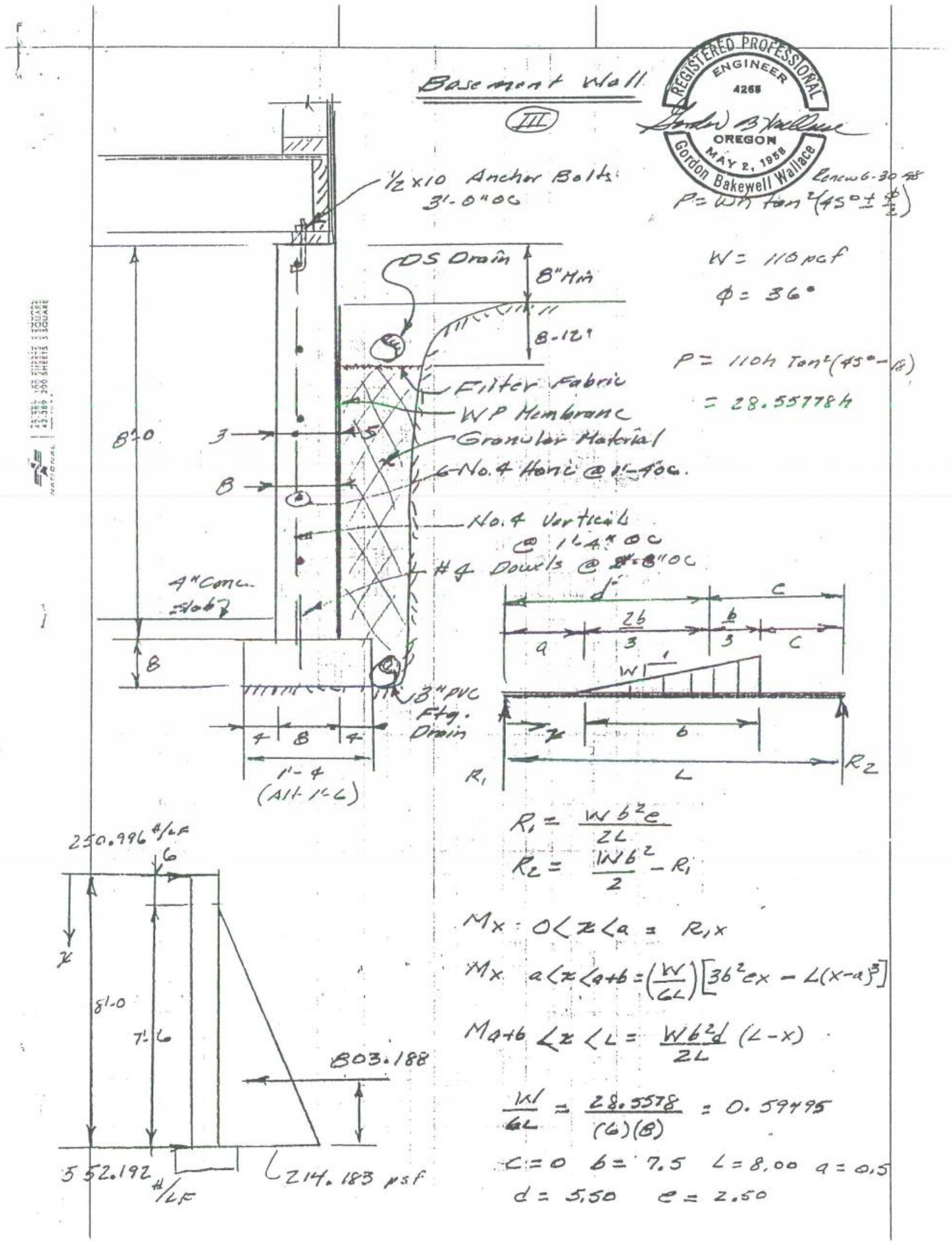
Momoment 21-0" above Top of Footing



$$M = (158.65)(1.1111) = 176.28\ ft^{\#}/_{LF} = 0.18\ '^{k}/_{LF} = 2.115\ ^{in\,k}/_{LF}$$

$$A_s = \frac{M}{f_s j d} = \frac{2.115}{(20)(0.87)(4)} = 0.0304\ ^{in^2}/_{LF}$$





Basement Wall
III
REGISTERED PROFESSIONAL ENGINEER 4265 OREGON MAY 2, 1958 Gordon Bakewell Wallace
Renew 6-30-98
1/2 x10 Anchor Bolts 3'-0" OC
P = wh tan²(45° ± φ/2)
W = 110 pcf
φ = 36°
P = 110h Tan²(45° − 18)
= 28.557784
DS Drain
8" Min
8-12"
Filter Fabric
WP Membrane
Granular Material
6-No.4 Horiz @ 1'-4" oc.
No.4 Vertical @ 1'-4" OC
#4 Dowels @ 8"OC
4" Conc. Slab
8'-0
8
3" PVC Ftg. Drain
4 8 4
1'-4 (All 1"-L)
R1 = Wb²e / 2L
R2 = Wb²/2 − R1
Mx : 0 < x < a = R1x
Mx a < x < a+b = (W/6L)[3b²ex − L(x−a)³]
Ma+b < x < L = Wb²d/2L (L − x)
W/6L = 28.5578/(6)(8) = 0.59495
c = 0 b = 7.5 L = 8.00 a = 0.5
d = 5.50 e = 2.50
250.996 #/LF
803.188
214.183 psf
552.192 #/LF
8'-0
7'-6

$$A_s = \frac{M}{f_s j d} = \frac{M}{(22)(0.87)(d)} = 0.0522466 \frac{M}{d}$$

When $d = 4"$ $A_s = 0.0130616\ M$

$d = 5$ $A_s = 0.0104493\ M$

$$3 b^2 e = (3)(7.5)^2(2.50) = 421.875$$

x	$(3b^2e)x$	$(x-a)$	$L(x-a)^3$	$[3b^2ex - L(x-a)^3]$	M ft#	M in k	A_s in² d = 4"	A_s in² d = 5"
0	0	-0.5	-1.00	-1.00	0		0	
0.5	210.94	0	0	210.94	125.499	1.506	0.020	0.0157
1	421.88	0.5	1.00	420.88	250.403	3.005	0.039	0.031
2	843.75	1.5	27.00	816.75	485.925	5.831	0.076	0.061
3	1265.63	2.5	125.00	1140.63	678.618	8.143	0.106	0.085
4	1687.50	3.5	343.00	1344.50	799.910	9.599	0.125	0.100
5	2109.38	4.5	729.00	1380.38	821.257	9.855	0.129	0.103
6	2531.25	5.5	1331.00	1200.25	714.089	8.569	0.112	0.0895
7	2953.13	6.5	2197.00	756.13	449.860	5.398	0.0705	0.056
8	3375.00	7.5	3375.00	0	0	0	0	0

6" Conc. Wall
d = 4"

x	No. 4 Verticals
0	∞
0.5	120"
1	61.5
2	31.5
3	22.6
4	19.2
5	18.6
6	21.4
7	34.04
8	∞

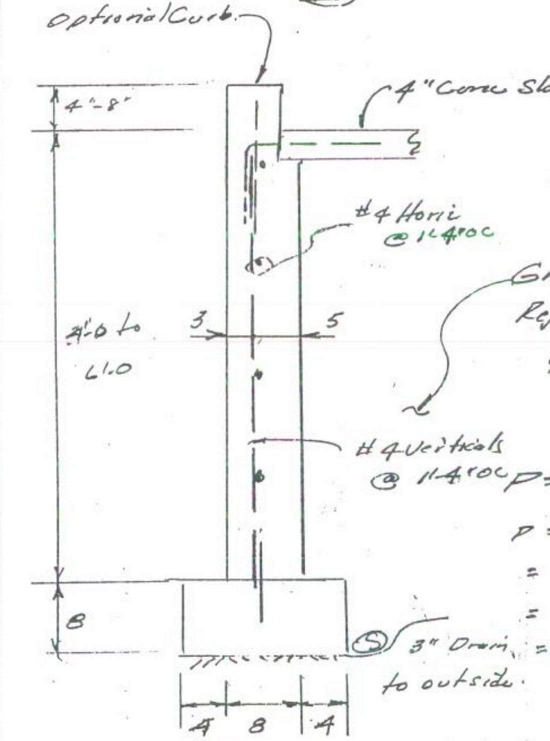
8" Conc. Wall
d = 5"

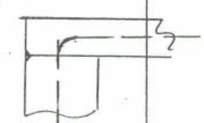
x	No. 4 Verticals
0	∞
0.5	153"
1	77
2	39.3
3	28.2
4	24.0
5	23.3
6	26.8
7	42.9
8	∞

Crescent Valley Bldg. | 3-18-97 | 97-29

Abutment Wall 4'-0 to 6'-0 W/o Curb.

(IV)





Surcharge 150 psf

Gravel Backfill
Reject Material
$\phi = 26°$
$W = 120$ pcf.

REGISTERED PROFESSIONAL ENGINEER 4265 OREGON MAY 2, 1958 Gordon Bakewell Wallace

Renew 6-30-98

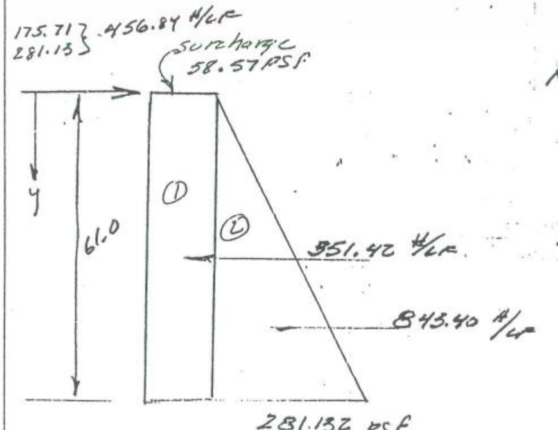
$p = wh\tan^2\left(45° \pm \frac{\phi}{2}\right)$

$p = wh\tan^2\left(45° - \frac{26}{2}\right)$

$= 120h\tan^2 32°$

$= (120h)(0.39046)$

$= 46.8554h.$



$M_1y = \frac{w_1 y}{2}(L-y) = \left(\frac{58.57}{2}\right)y(L-y)$

$= 29.285\,y(L-y)$

$M_2y = \frac{W_2 y}{3L^2}(L^2-y^2)$

$= \frac{(843.40)(y)}{(3)(6)^2}\,L^2-y^2$

$= 7.809\,y(L^2-y^2)$

97-20

2

4'-0 to 6'-0

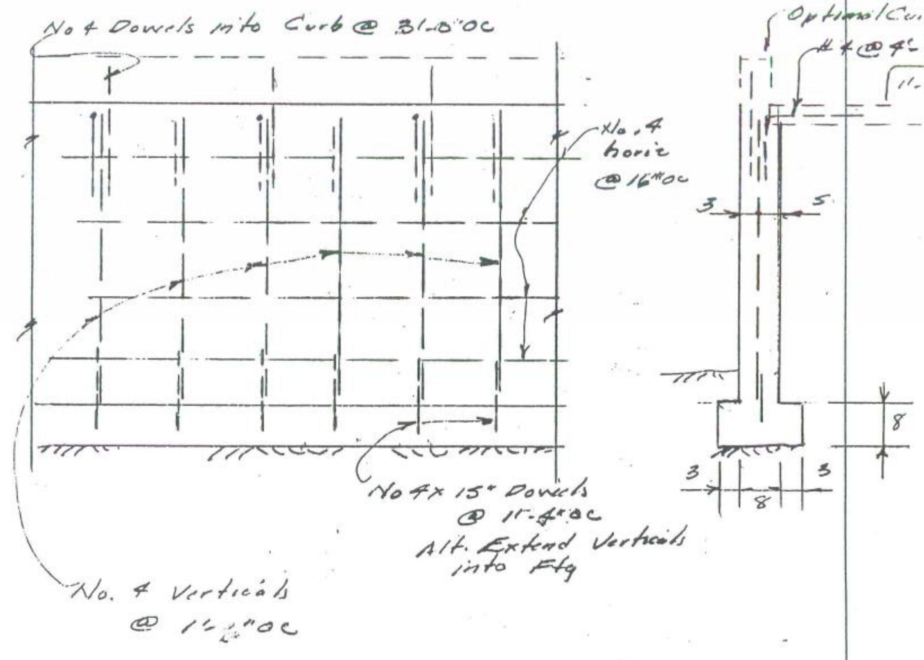
$$M_1 = \left(\frac{W_1 y}{2}\right)(L-y) \qquad M_2 = \left[\frac{W_2 y}{3L^2}\right](L^2-y^2)$$

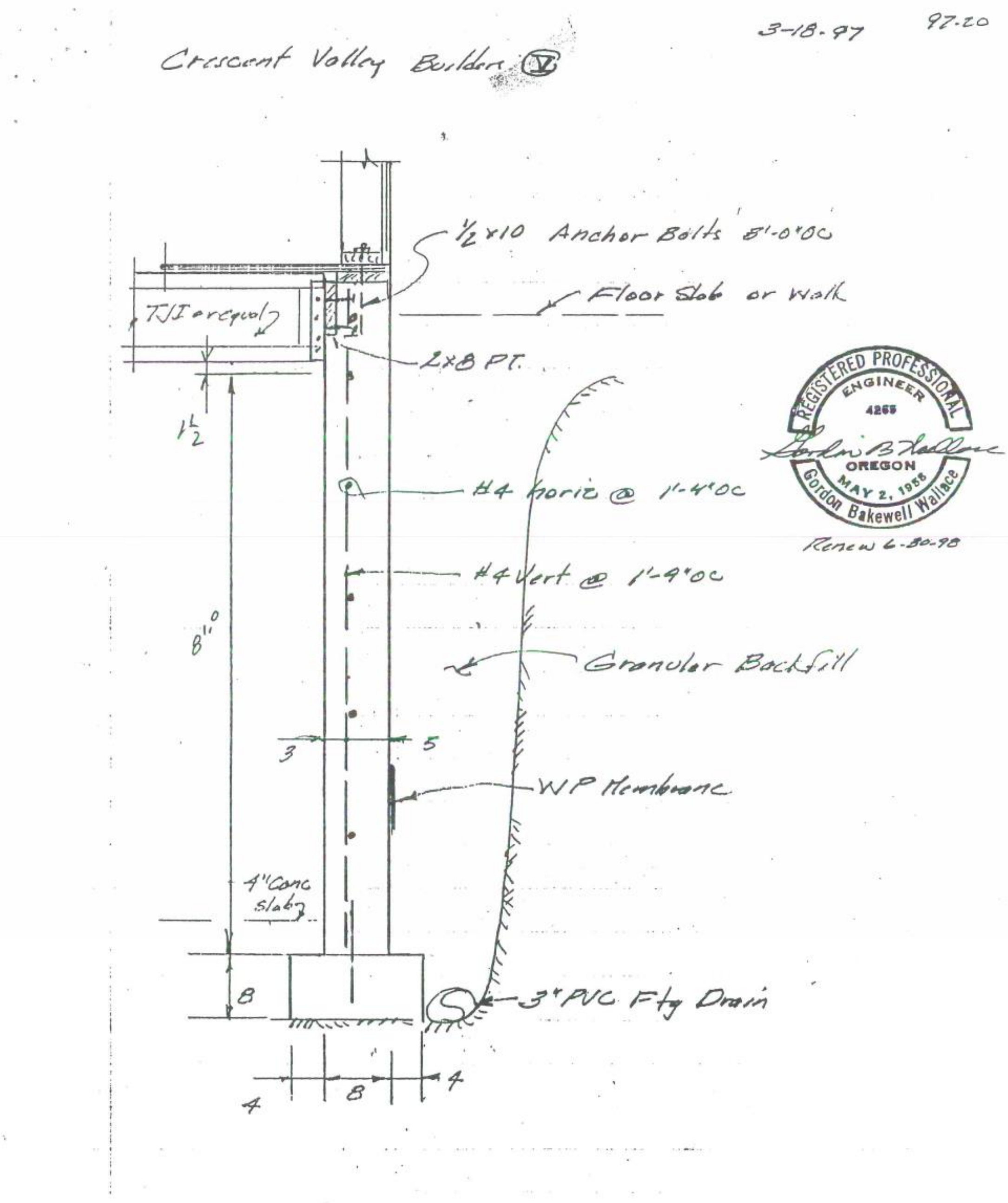
y	L-y	L^2-y^2	M_1 29.285(y)(L-y)	M_2 (y) 7.080922)(L^2-y^2)	ΣM #'/LF	ΣM in-k/LF	d=3' As in²/LF	d = 5'
0	6	36.00	0	0	0	0	0	0
1	5	35.00	146.425	277.832	394.257	4.731	0.0906	0.0544
2	4	32.00	234.280	453.179	687.459	8.250	0.158	0.0948
3	3	27.00	263.565	573.555	837.120	10.045	0.192	0.1155
4	2	20.00	234.280	566.474	800.754	9.609	0.184	0.1104
5	1	11.00	146.425	389.451	535.876	6.431	0.123	0.0738
6	0	0	0	0	0	0	0	0

$$A_s = \frac{M}{f_s j d} = \frac{M}{(20)(0.87)d} = \frac{M}{17.4d} = 0.05747\frac{M}{d}$$

when d = 3" $\quad A_s = 0.019157M$

d = 5" $\quad A_s = 0.011494\ M$





3-18-97
97-20
Crescent Valley Builders
½ x10 Anchor Bolts 5'-0"OC
Floor Slab or Walk
TJI or equal
2x8 PT.
1½
8'-0
#4 horiz @ 1'-4"OC
#4 Vert @ 1'-4"OC
Granular Backfill
3
5
WP Membrane
4" Conc slab
8
3" PVC Ftg Drain
4
8
4
REGISTERED PROFESSIONAL ENGINEER
4265
OREGON
MAY 2, 1958
Gordon Bakewell Wallace
Renew 6-30-98

Crescent Valley Builders

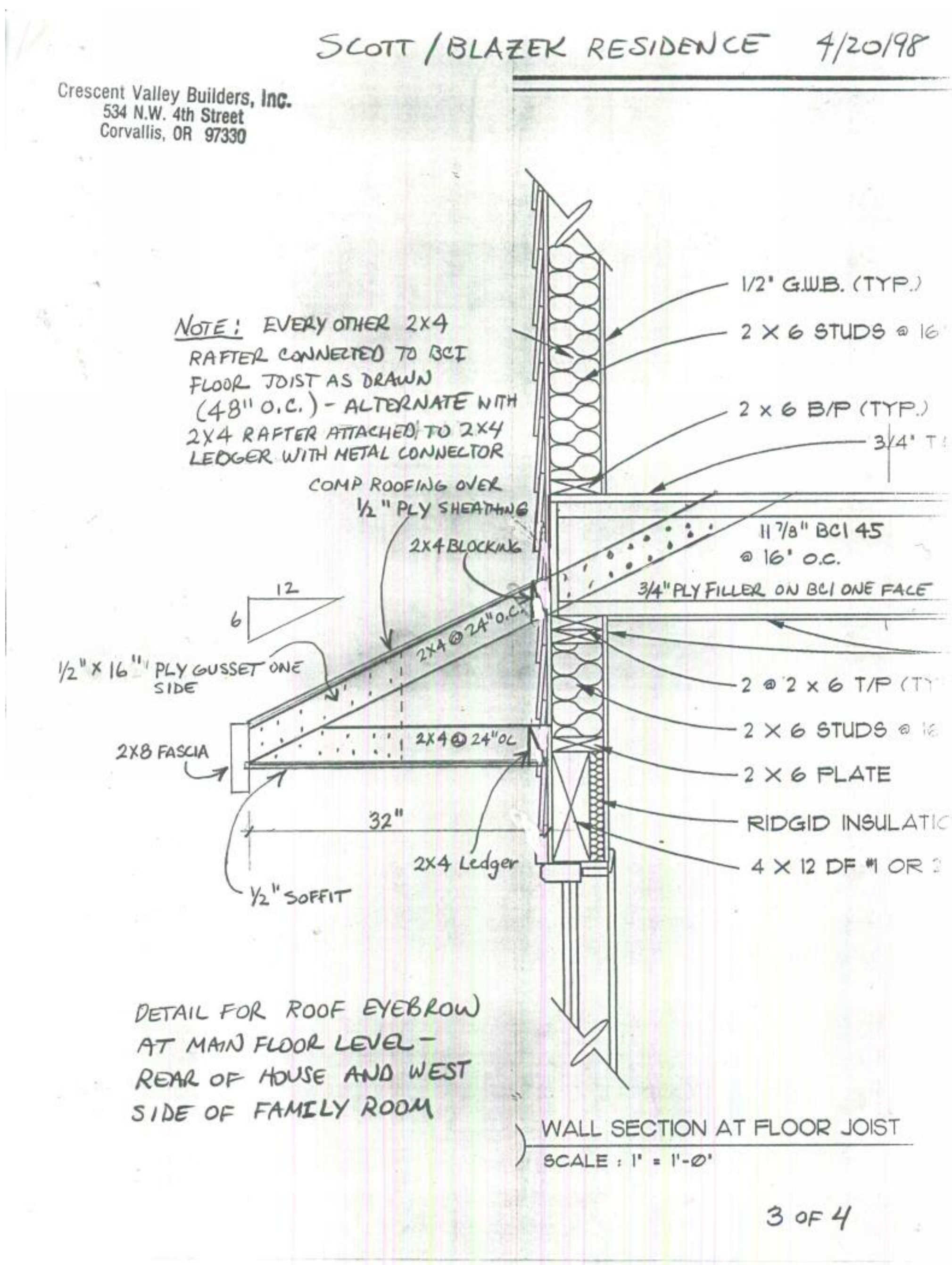
10/97-20

Use 1/2 x 10 Anchor Bolts

Full Height Basement Walls. III, IV & V
Bolts @ 3'-0" OC & 1'-0" from Corners.

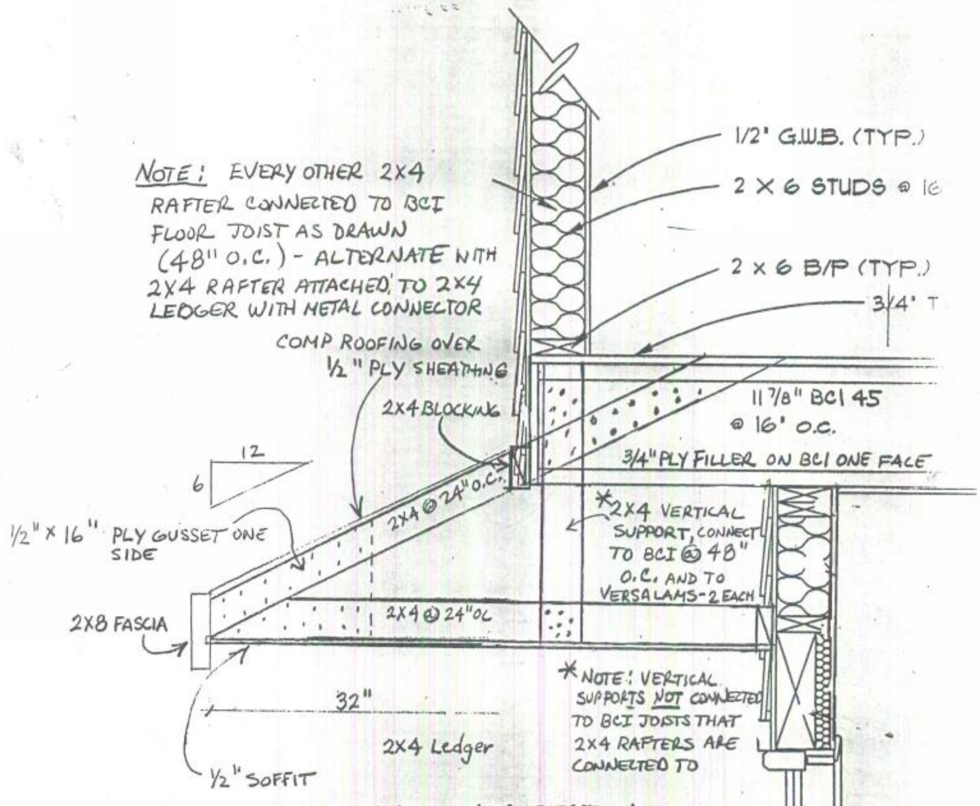
Pony Walls II
Bolts @ 6'-0" OC and ends of Plates.

Daylite Basement Wall I
Bolts @ 6'-0" OC. and 1'-0" from Corners.


SCOTT / BLAZEK RESIDENCE 4/20/98
Crescent Valley Builders, Inc.
534 N.W. 4th Street
Corvallis, OR 97330
NOTE: EVERY OTHER 2X4 RAFTER CONNECTED TO BCI FLOOR JOIST AS DRAWN (48" O.C.) - ALTERNATE WITH 2X4 RAFTER ATTACHED TO 2X4 LEDGER WITH METAL CONNECTOR
COMP ROOFING OVER ½" PLY SHEATHING
2X4 BLOCKING
1/2" G.W.B. (TYP.)
2 X 6 STUDS @ 16
2 x 6 B/P (TYP.)
3/4"
11 7/8" BCI 45 @ 16" O.C.
3/4" PLY FILLER ON BCI ONE FACE
12
6
2X4 @ 24" O.C.
1/2" X 16" PLY GUSSET ONE SIDE
2X4 @ 24" OC
2X8 FASCIA
32"
2X4 Ledger
½" SOFFIT
2 @ 2 x 6 T/P
2 X 6 STUDS @
2 X 6 PLATE
RIDGID INSULATI
4 X 12 DF #1 OR
DETAIL FOR ROOF EYEBROW AT MAIN FLOOR LEVEL - REAR OF HOUSE AND WEST SIDE OF FAMILY ROOM
WALL SECTION AT FLOOR JOIST
SCALE : 1" = 1'-0"
3 OF 4

SCOTT / BLAZEK RESIDENCE 4/20/98

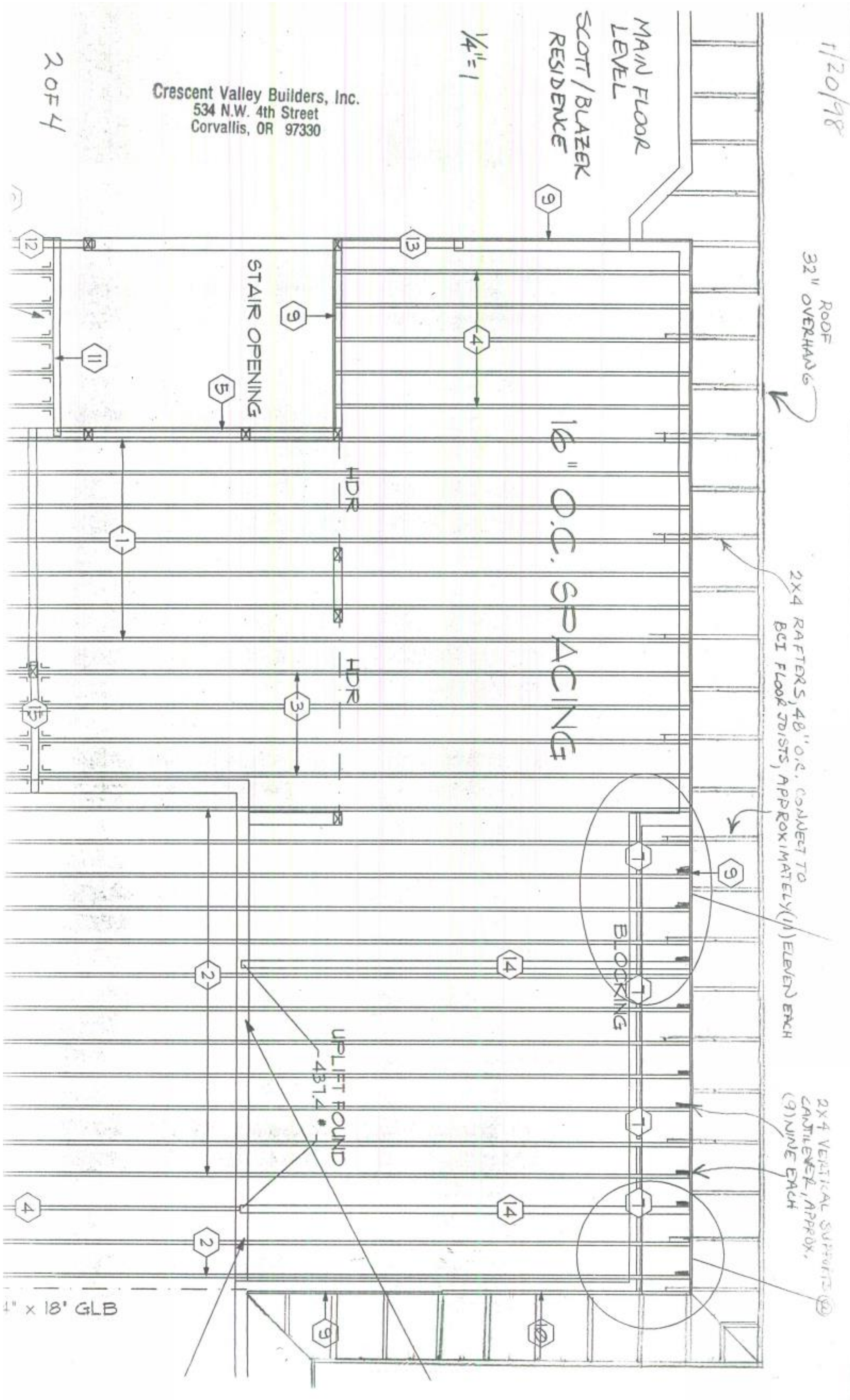
Crescent Valley Builders, Inc.
534 N.W. 4th Street
Corvallis, OR 97330



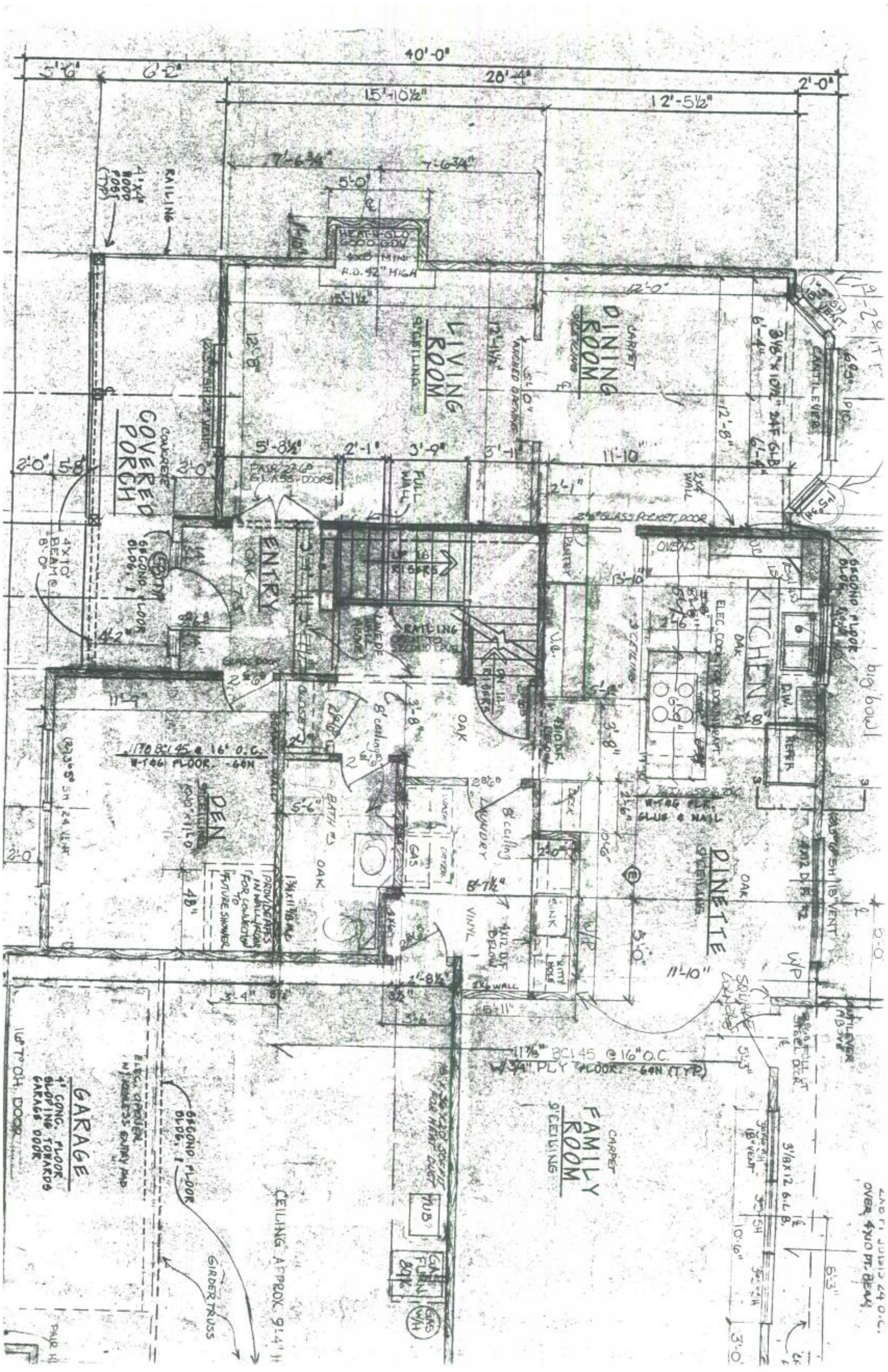
CANTILEVER SECTION
AT FAMILY ROOM - REAR

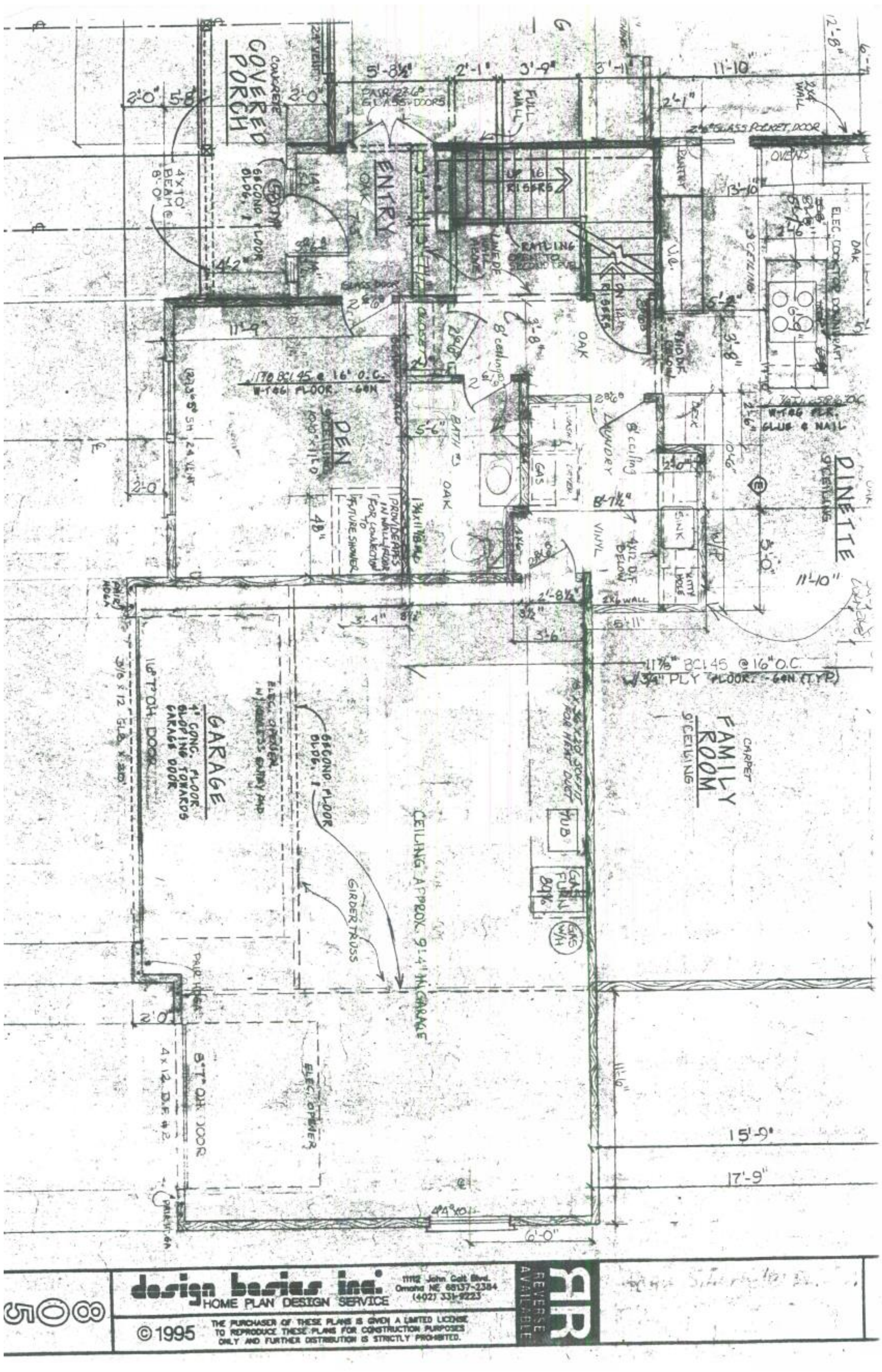
DETAIL FOR ROOF EYEBROW
AT MAIN FLOOR LEVEL -
REAR OF HOUSE AND WEST
SIDE OF FAMILY ROOM

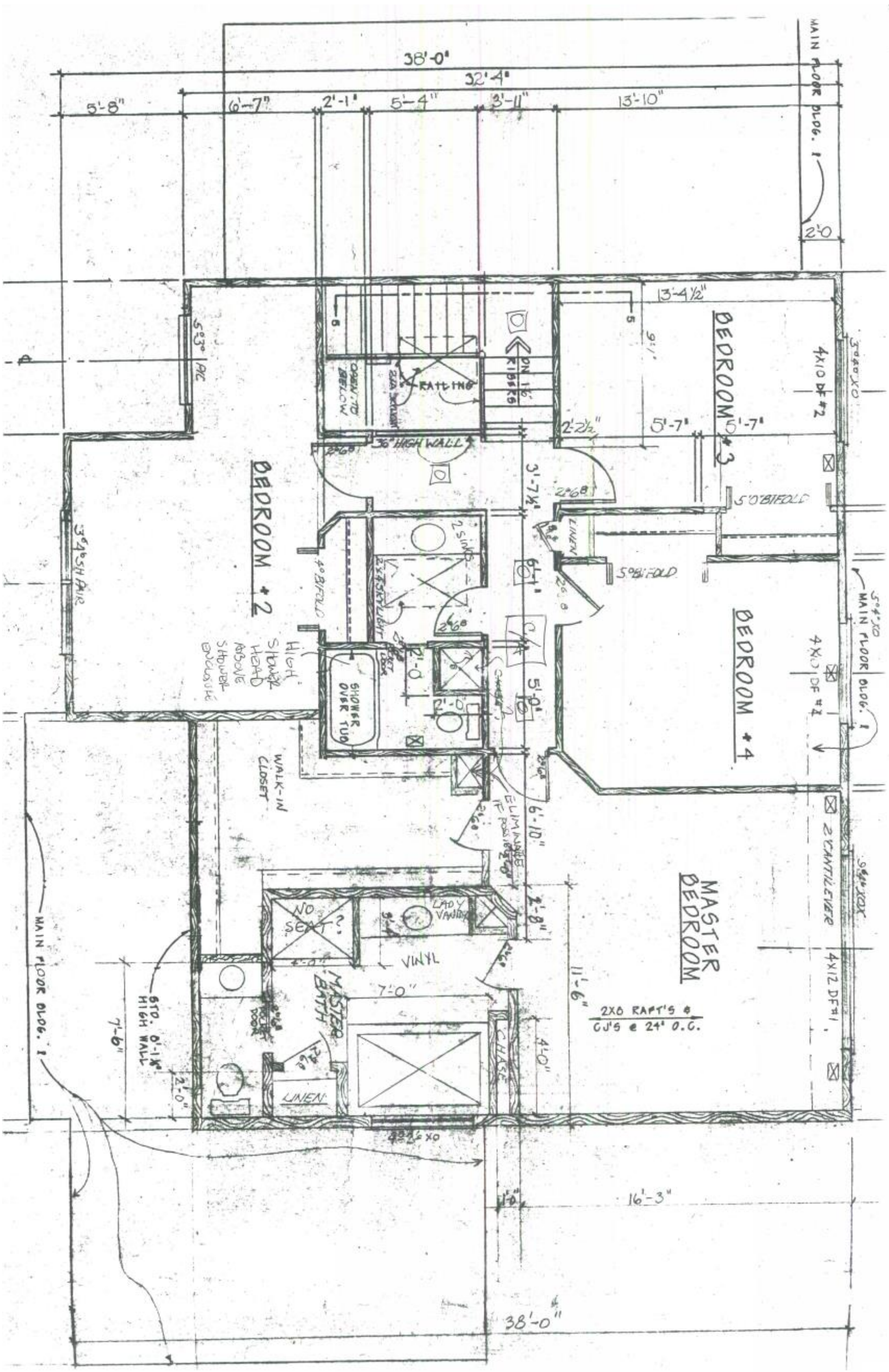
4 OF 4

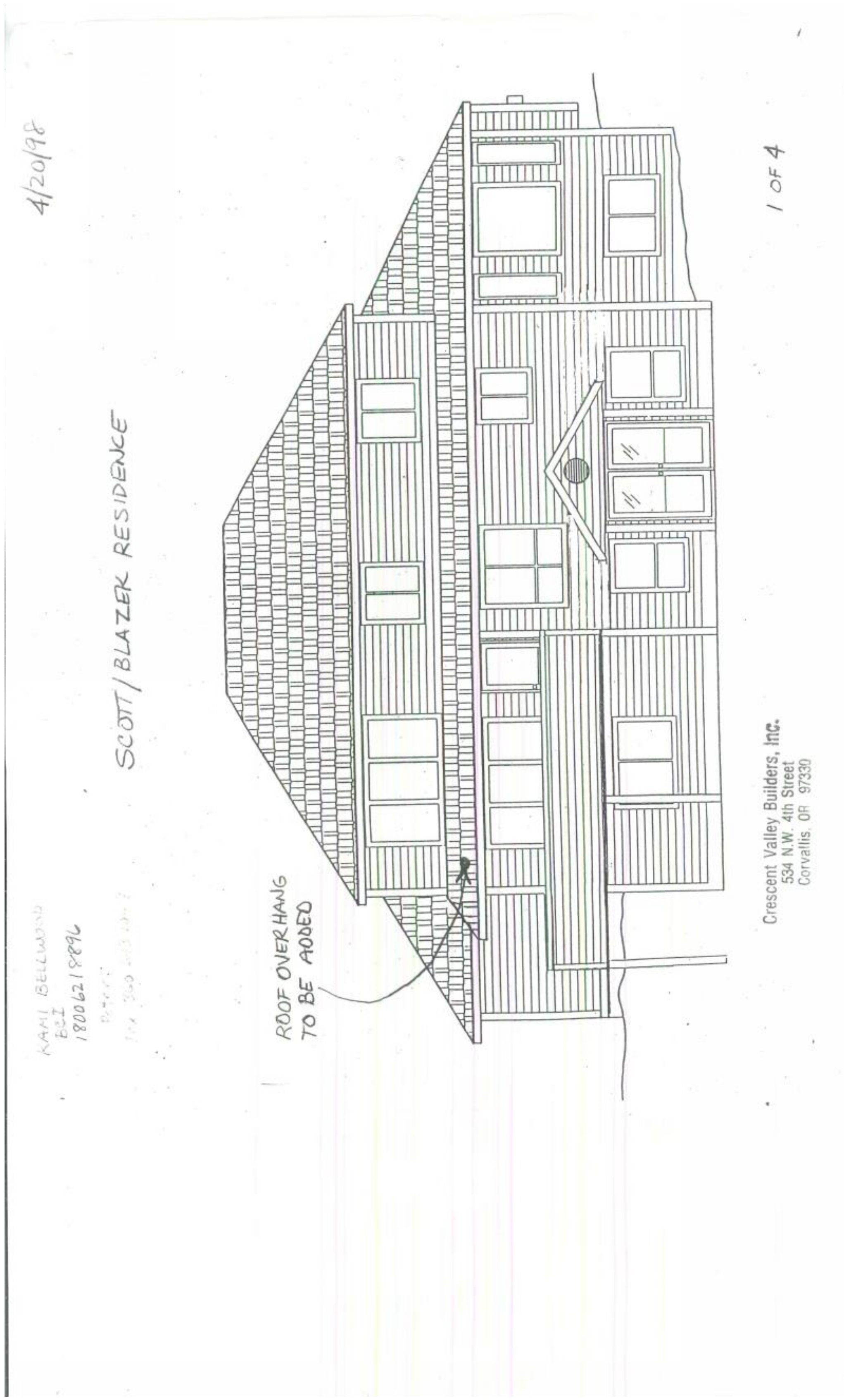


Crescent Valley Builders, Inc.
534 N.W. 4th Street
Corvallis, OR 97330
2 OF 4
MAIN FLOOR LEVEL
SCOTT / BLAZER RESIDENCE
1/4"=1'
1/20/98
ROOF 32" OVERHANG
STAIR OPENING
16" O.C. SPACING
HDR
HDR
2X4 RAFTERS, 48" O.C. CONNECT TO BCI FLOOR JOISTS, APPROXIMATELY (11) ELEVEN EACH
BLOCKING
UPLIFT FOUND 431.4 #
2X4 VERTICAL SUPPORTS CANTILEVER, APPROX. (9) NINE EACH
x 18' GLB


40'-0"
LIVING ROOM
DINING ROOM
KITCHEN
DINETTE
FAMILY ROOM
GARAGE
DEN
ENTRY
COVERED PORCH


COVERED PORCH
ENTRY
DEN
GARAGE
FAMILY ROOM
DINETTE
design basics inc.
HOME PLAN DESIGN SERVICE
©1995
THE PURCHASER OF THESE PLANS IS GIVEN A LIMITED LICENSE TO REPRODUCE THESE PLANS FOR CONSTRUCTION PURPOSES ONLY AND FURTHER DISTRIBUTION IS STRICTLY PROHIBITED.
REVERSE AVAILABLE


38'-0"
32'-4"
BEDROOM #2
BEDROOM #3
BEDROOM #4
MASTER BEDROOM
MASTER BATH
WALK-IN CLOSET
LINEN
VINYL
SHOWER OVER TUB
MAIN FLOOR BLDG. 1
16'-3"
38'-0"


4/20/98
SCOTT / BLAZEK RESIDENCE
ROOF OVERHANG TO BE ADDED
1 OF 4
Crescent Valley Builders, Inc.
534 N.W. 4th Street
Corvallis, OR 97330

Appendix C: Building Model Details

Building Layout:

The house that was modeled for this study was modified from the original plans found in Appendix B. The modified building dimensions can be found below in Figures C-1 and C-2. The biggest change is the removal of the daylight basement.

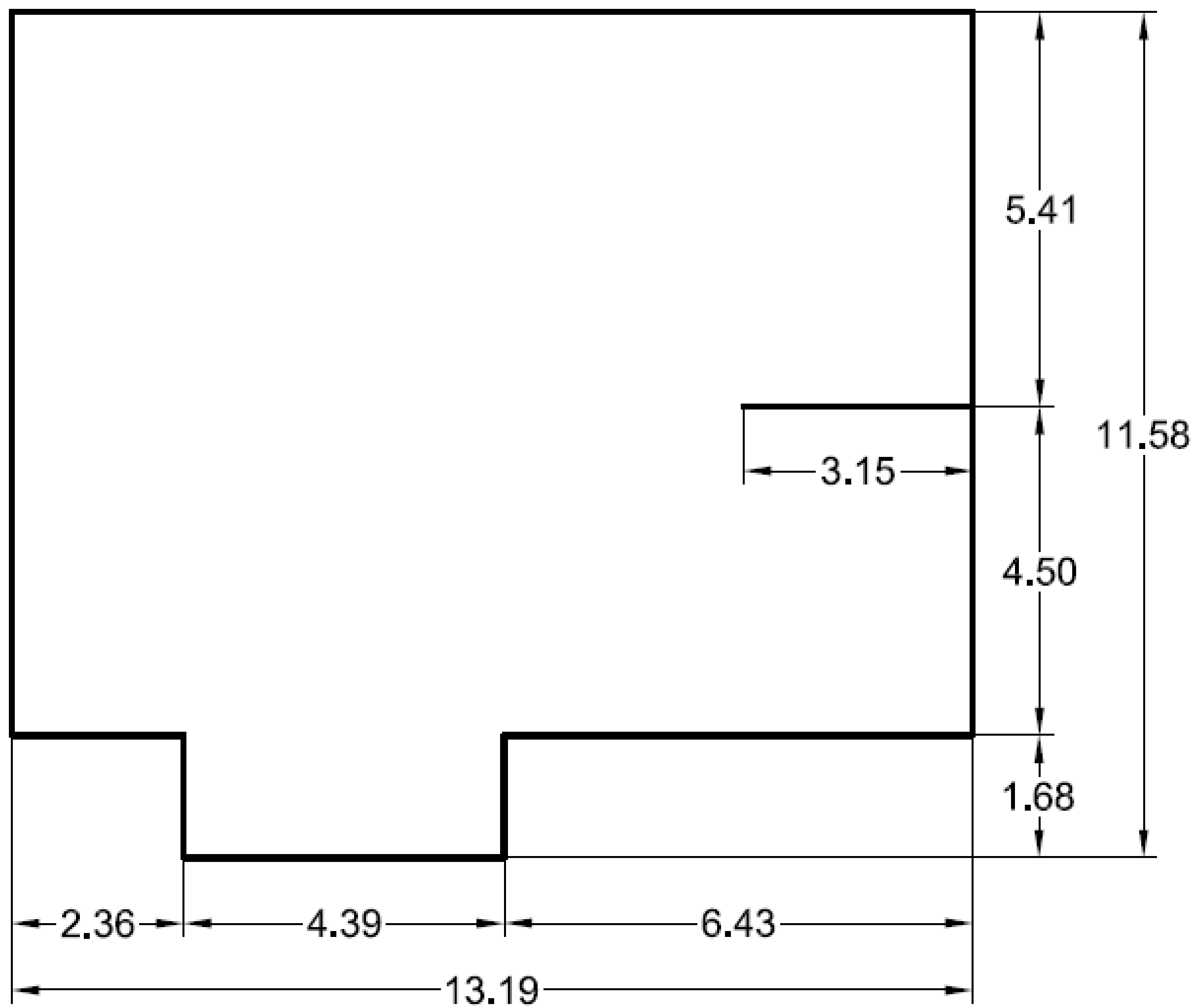


Figure C 1. Second floor layout. (dimensions in meters)

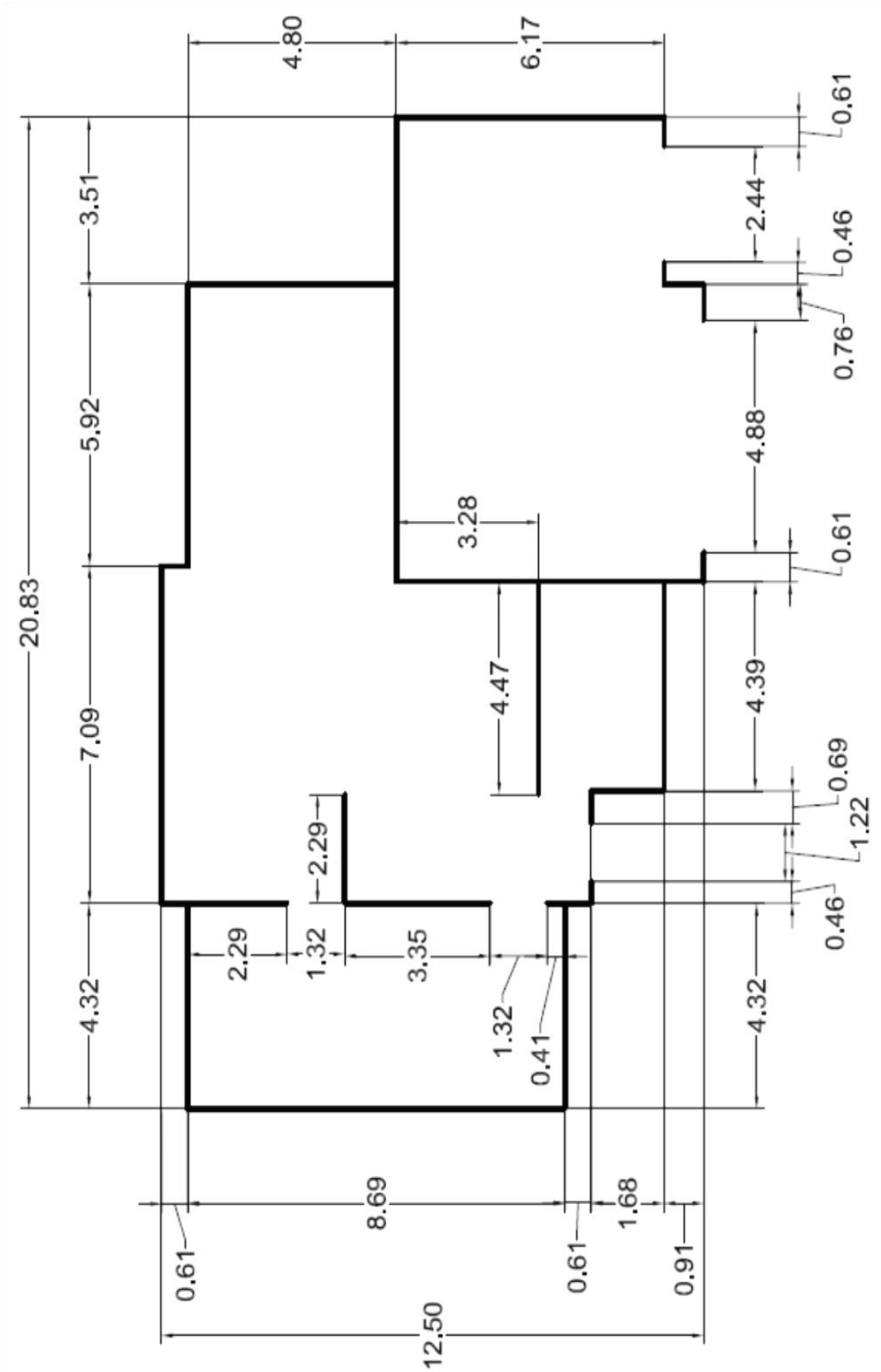


Figure C 2. First floor layout (dimensions in meters)

Framing Members

Members used in this study include those found in Table C-1, as well as all member sizes and grades found in the truss details Appendix B. The BCI joists used for the second floor are assumed to be 11-7/8" BCI 90, the property values of which can be found in BCI's *Western Species Guide*.

Table C 1. Member usage and sizes. Does not include roof truss members, or BCI joists.

Member Usage	Member Size W x H (mm)	Material
Wall Stud	38 x 140	DF #2
Double Wall Stud (used at corners and next to windows	76 x 140	DF #2
Roof Rafters	38 x 184	DF #2
Angle Rafter on Roof	38 x 235	DF #2
Window and door headers + Window sills	76 x 184	DF #2
First floor Bottom plate	140 x 38	DF #2
Top plate (not used between 1st and 2nd floors)	140 x 76	DF #2
Glulams spanning over garage doors	171 x 254	24F-V5

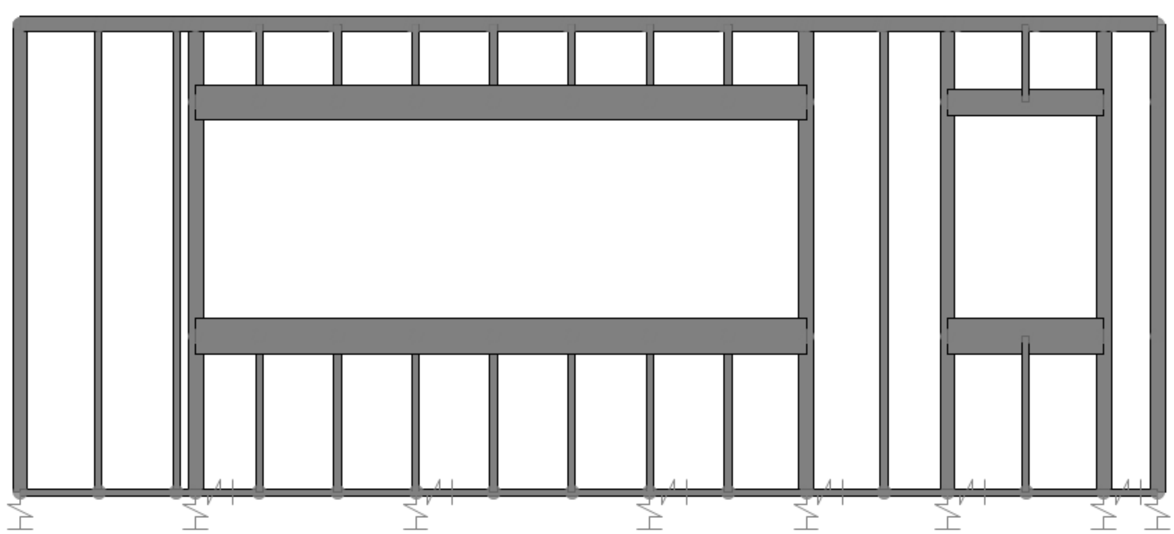


Figure C 3. Example of wall. (First floor, eastern side of north wall. View from exterior.).

APPENDIX D

SHELL MODELING

Modeling shells

The shell modeling procedure used for this study is based upon the methods used by and the results of Pfretzchner et al. (2014). Pfretzchner et al. (2014) used a similar process to Martin et al. (2011), while incorporating a layered shell element that allows for both the plywood and gypsum wall board stiffnesses to be taken into account.

Both Martin et al. (2011) and Pfetzchner et al. (2014) used a single shell element for each wall and roof section, but due to geometric complexity, in this study walls and roofs are modeled using multiple shell elements.

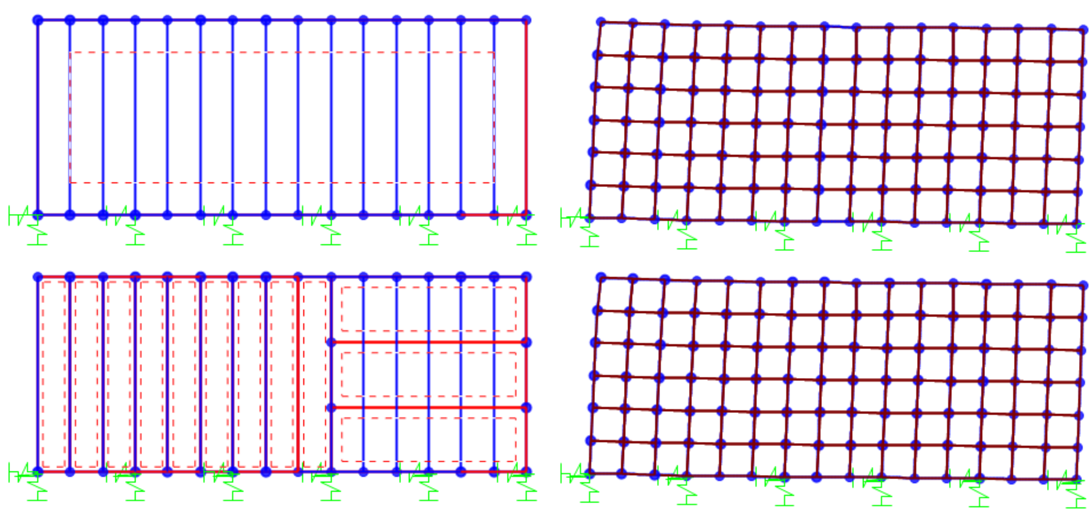


Figure D 1. Top: wall with single shell element. Bottom: Wall with several shell elements. Both walls have the same deflection when subjected to similar loading.

Wall Stiffness

Due to limited information for plywood sheathing, for sake of consistency and ease, the material properties for shell elements used in this study were taken from Pfretzschner et al. (2014). All properties aside from the G_{12} stiffness in the walls are consistent throughout.

Table D 1. Material properties of plywood and gypsum wall board.

Material	Properties	Source
Plywood Sheathing (Roof)	E_1 = 8280 MPa (1201 ksi) E_2 = 2393 MPa (347 ksi) U_{12} = 0.011 G_{12} = 482 MPa (70 ksi)	*Pfretzschner et al. (2014) OSULaminates (Nairn 2007) (Flexural Properties*
Plywood Sheathing (Walls)	E_1 = 7017 MPa (1018 ksi) E_2 = 3657 MPa (530 ksi) U_{12} = 0.016	*Pfretzschner et al. (2014) OSULaminates (Nairn 2007) (In-Plane Properties)*
Gypsum Wallboard (Walls)	E_1 = 1820 MPa (264 ksi) U_{12} = 0.3	*Gypsum Association (2010)*

The G_{12} stiffness for walls varies relatively linearly with the wall's length and is modeled based on a calibration procedure outlined in Pfretzchner et al. (2014). Figure D1 shows the results of this procedure for 9.5 mm plywood with 152 mm nail spacing from Pfretzschner et al. (2014), which is used for all walls in this study. Figure D2 simillarly gives the G_{12} stiffness for the wallboard relative to the wall length, again from Pfretzschner et al. (2014).

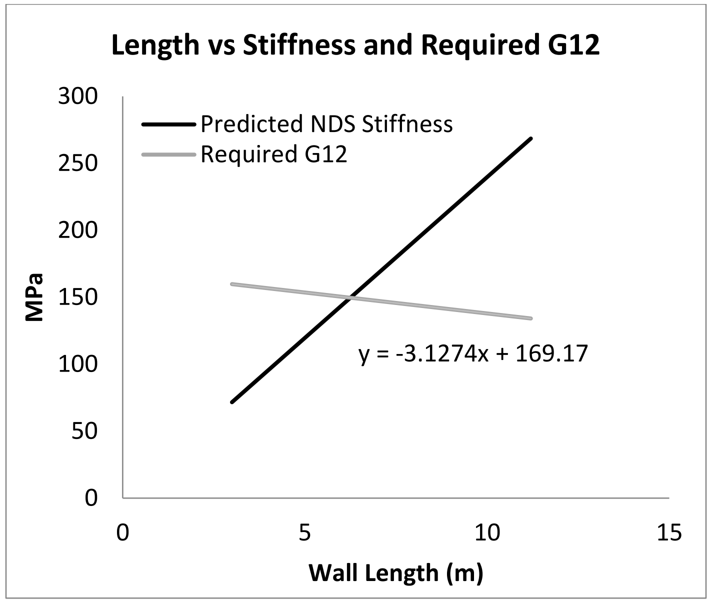


Figure D 2. Wall stiffness and required G_{12} for 9.5mm plywood with 15.2 cm nail spacing (Source: Pfretzchner et al. 2014)

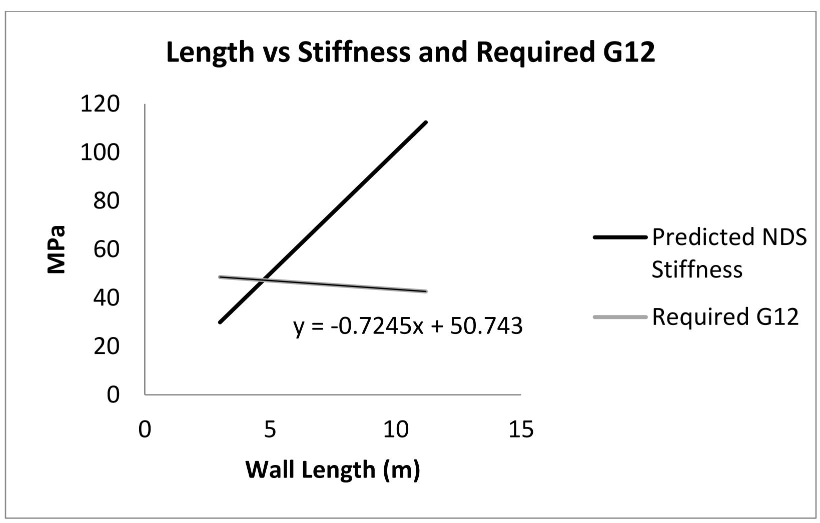


Figure D 3. Wall stiffness and required G_{12} for 13mm gypsum wall board (Source: Pfretzchner et al. 2014)

APPENDIX E

SNOW LOAD INVESTIGATION

Snow loading for this study was based on Section 1608 of the 2014 OSSC, in accordance with Chapter 7 of ASCE 7-10 and the *Snow Load Analysis for Oregon* (2011). The ground snow load based on the location of the house being modeled is 19.0 psf. The "Use of Map" Part I lays out adjustments that may be used for sites in steep terrain, as well as for site-specific case studies, neither of which apply here. Part II defines the minimum roof snow load as 20 psf, which will govern (see notes on ASCE 7-10 section 7.3 below). The criteria for applying a 5 psf rain-on-snow surcharge are defined in Part III:

1) All roofs with a slope less than 1:12.
2) Roofs of any slope that constrain runoff.

Since the all roof slopes are greater than 1:12, and the house has a continuous gutter system, which is not considered to constrain runoff, the 5 psf rain-on-snow surcharge does not apply.

Section 7.3 of ASCE 7-10 defines the flat roof snow load for a building, and Section 7.4 defines the Sloped Roof Snow Load. Since both are less than the minimum 20 psf , the 20 psf minimum will govern. Section 7.5 is not applicable as the roof of this structure does not contain continuous beam systems. Sections 7.6-7.9 contain several loading cases that may be applicable to this structure, but will not be investigated in this study. Section 7.6 provides provisions for unbalanced snow loadings. These provisions apply for hip roofs with a slope of less than 7:12, so they would be applicable in a complete design. Since there are lower roofs on this structure, Drifts onto Lower Roofs as detailed in Section 7.7

and Sliding Snow Loads could also be investigated, but will not be for this study. The rain-on-snow surcharge load in Section 7.10 is addressed above in the 2014 OSSC, and Sections 7.11 and 7.12 are not applicable. Thus, 20 psf was used as the roof snow load.

APPENDIX F

WIND LOAD INVESTIGATION

The wind loading procedure used in this study is the all-heights method found in section 1609.6 of the 2014 OSSC. For this method, a uniform pressure is applied over each surface based on Equation F-1 below. While wind tunnel testing shows that wind loads concentrate towards the corners, this method was selected because it is both simple and should produce reasonable results for the overall load flow through the structure.

The structure must meet the following conditions specified in section 1609.1 to use this method:

1) The structure is
    a. Less than 22.9 m in height. (This structure is approximately 7.5 m tall, so this condition is met. OK)
    b. Has a height-to-least-width ratio of 4 or less. (No height-to-width ratio exceeds 1 for this structure. OK)
2) The structure is not sensitive to dynamic effects. (OK)
3) The structure is not located on a site with channeling effect. (No channeling effects at the site of this structure. OK)
4) The building must be a simple diaphragm building. (The code definition of a simple diaphragm building is somewhat unclear, but assumed to be true. OK)
5) Only applies for roofs with a slope less than 45 degrees. (OK)

Loadings

Similar to Pfretzschner et al. (2014), due to the asymmetry of the house, wind loading must be considered from all four directions. Only orthogonal loadings perpendicular to the major axes of the building are investigated in this study with corner wind loads are not considered. The loading on each surface is defined by Equation F-1 below (Equation 16-35 in the 2014 OSSC).

$$P_{net} = 0.00256V^2K_zC_{net}K_{zt} \qquad \text{(Equation F-1)}$$

Table F 1. Coefficient definitions.

Coefficients	Value	Source
Risk Category	II	ASCE 7-10
Site	B	ASCE 7-10
V (wind speed)	52.8 m/s (120 mph)	Figure 1609A in the 2014 OSSC
K_z	0.64	ASCE 7.10 Table 27.3-1
K_{zt}	1.0	ASCE 7.10 Section 26.8

Table F 2. C_{net} coefficients and resultant pressures for each wind loading case. C_{net} coefficients from Table 1609.6.2 in the 2014 OSSC.

			Windward Wall	**Leeward Wall**	**Side Wall**	**Windward Roof**	**Leeward Roof**	**Parallel to Ridge**
Positive Internal Pressure	**Condition 1**	Cnet	0.43	-0.51	-0.66	-0.47	-0.66	-1.09
		P (kPa)	**0.49**	**-0.58**	**-0.74**	**-0.53**	**-0.74**	**-1.23**
	Condition 2	Cnet	0.43	-0.51	-0.66	0.06	-0.66	-1.09
		P (kPa)	**0.49**	**-0.58**	**-0.74**	**0.07**	**-0.74**	**-1.23**
Negative Internal Pressure	**Condition 1**	Cnet	0.73	-0.21	-0.35	-0.16	-0.35	-0.79
		P (kPa)	**0.82**	**-0.24**	**-0.39**	**-0.18**	**-0.39**	**-0.89**
	Condition 2	Cnet	0.73	-0.21	-0.35	0.37	-0.35	-0.79
		P (kPa)	**0.82**	**-0.24**	**-0.39**	**0.42**	**-0.39**	**-0.89**

APPENDIX G:

WIND UPLIFT

Maximum UPLIFT force displayed on each chart.

North:

Wind Only

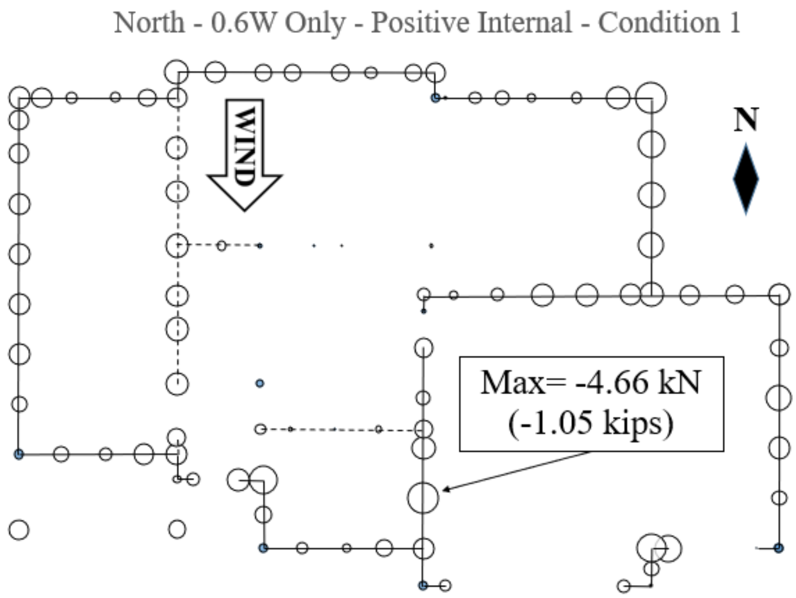


Figure G 1: North Wind Only. Positive internal pressure, Condition 1

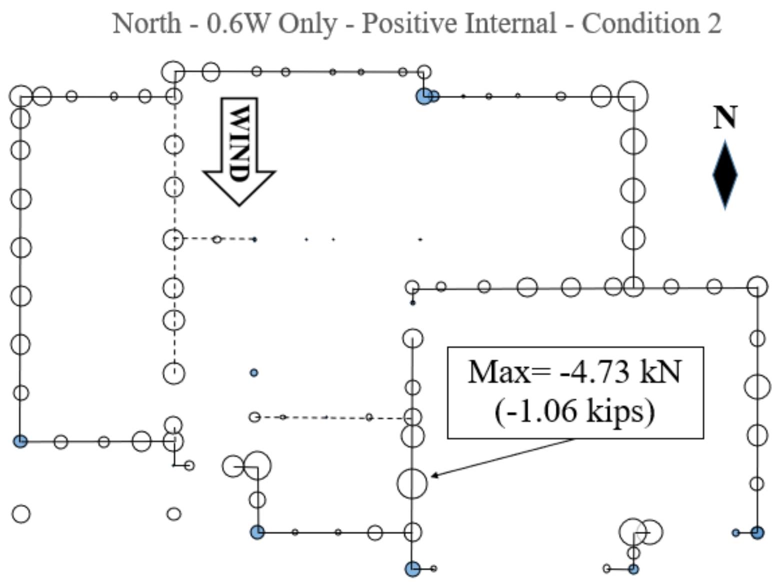


Figure G 2: North Wind Only. Positive Internal Pressure, Condition 2

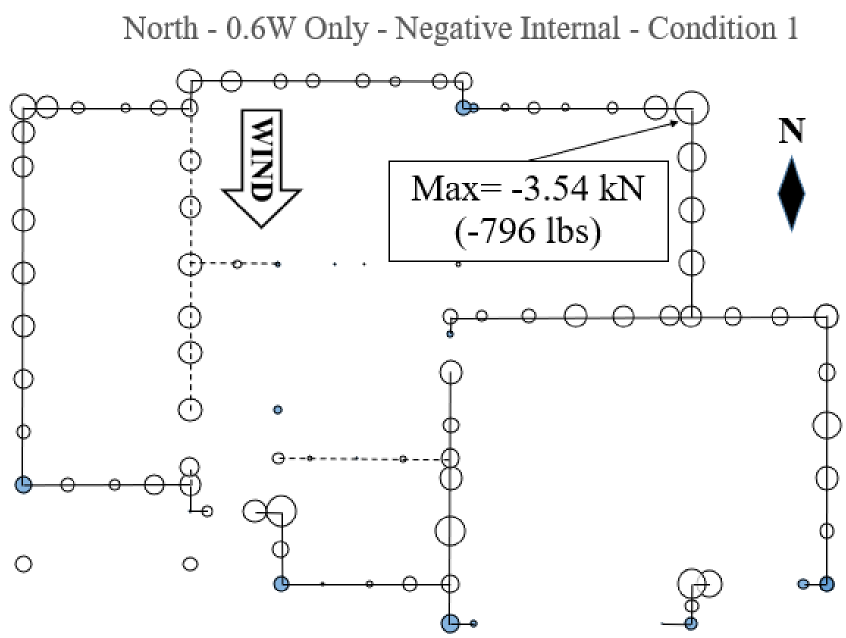


Figure G 3: North Wind Only. Negative internal pressure, Condition 1

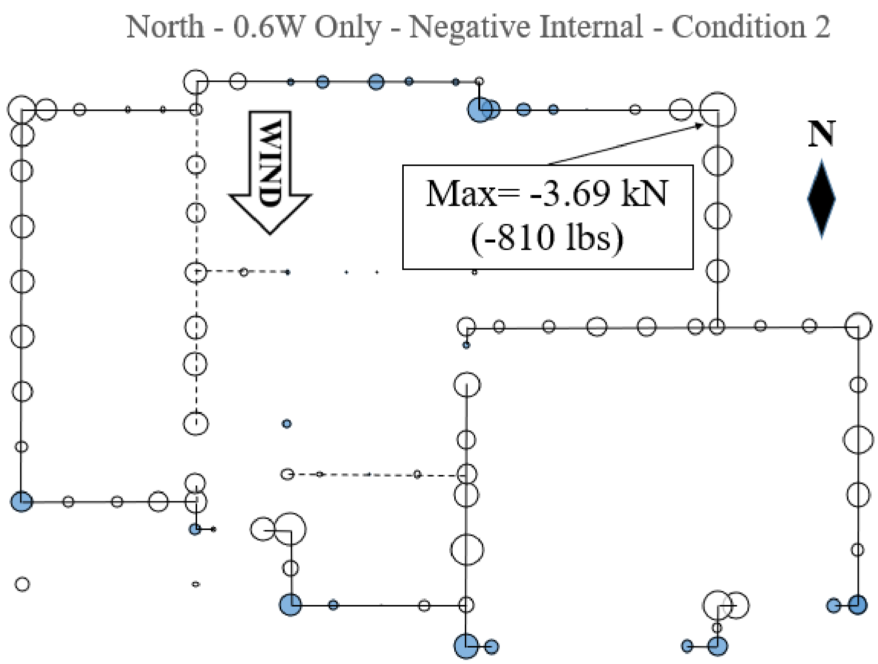


Figure G 4: North Wind Only. Negative internal pressure, Condition 2

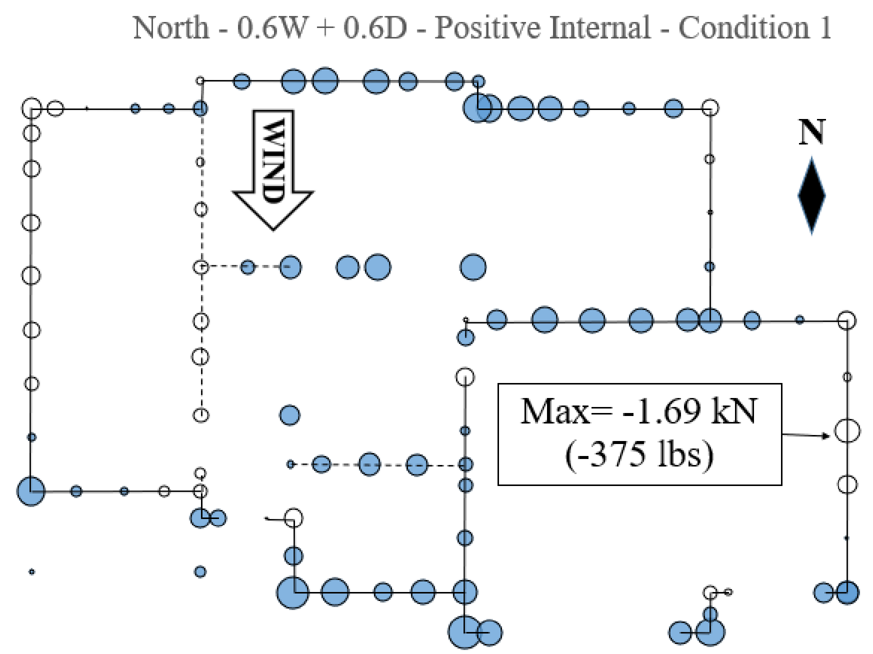


Figure G 5: North 0.6W + 0.6D. Positive internal pressure, Condition 1

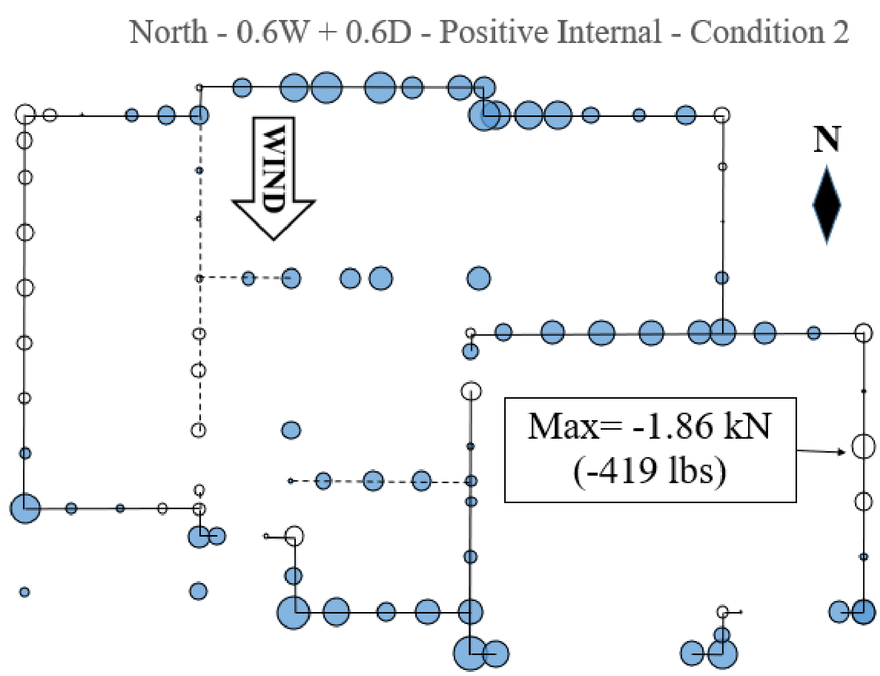


Figure G 6: North 0.6W + 0.6D. Positive internal pressure, Condition 2

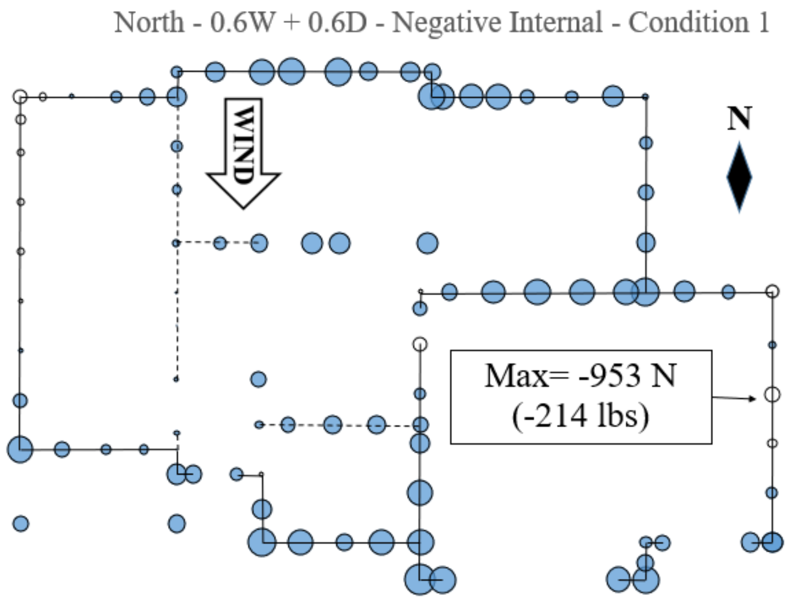


Figure G 7: North 0.6W + 0.6D. Negative internal pressure, Condition 1

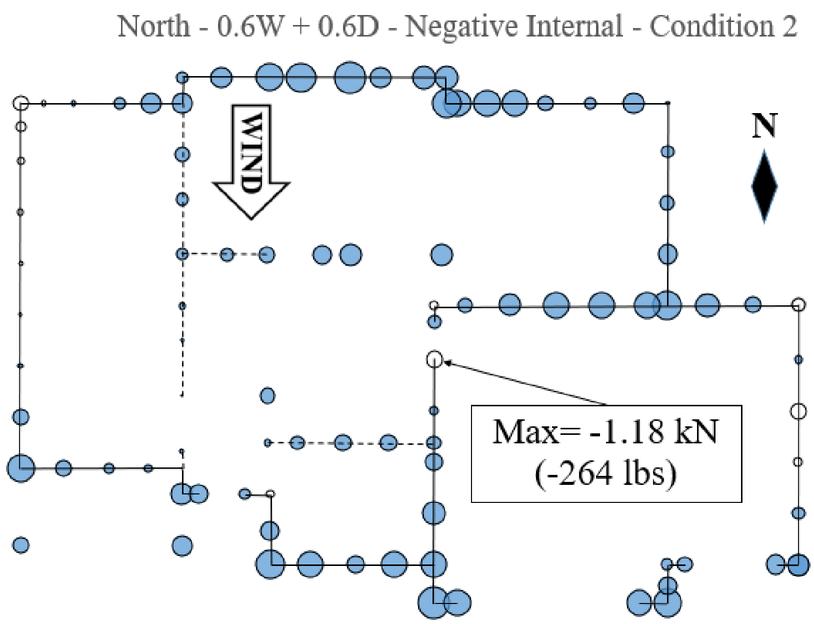


Figure G 8: North 0.6W + 0.6D. Negative internal pressure, Condition 2

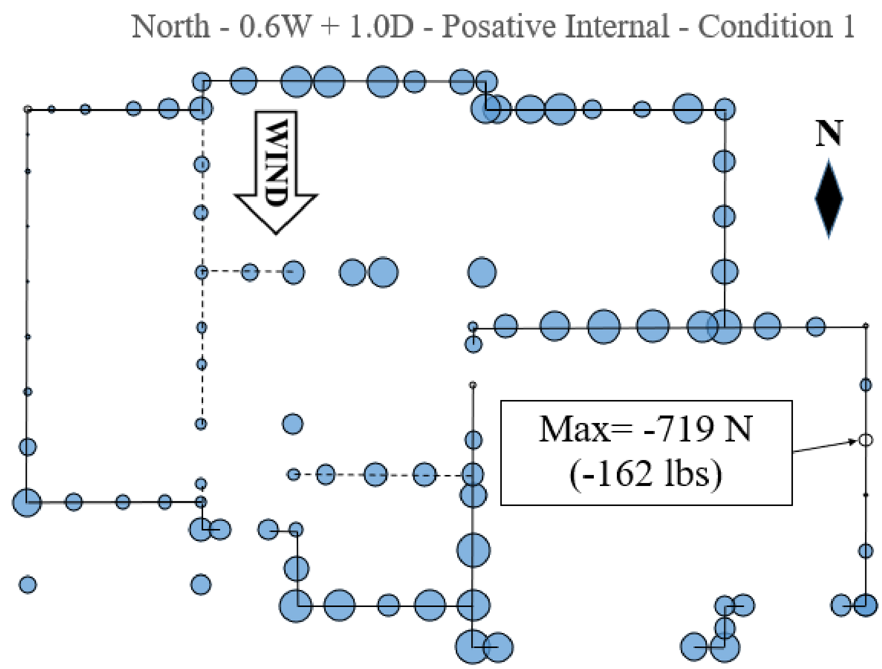


Figure G 9: North 0.6W + 1.0D. Positive internal pressure, Condition 1

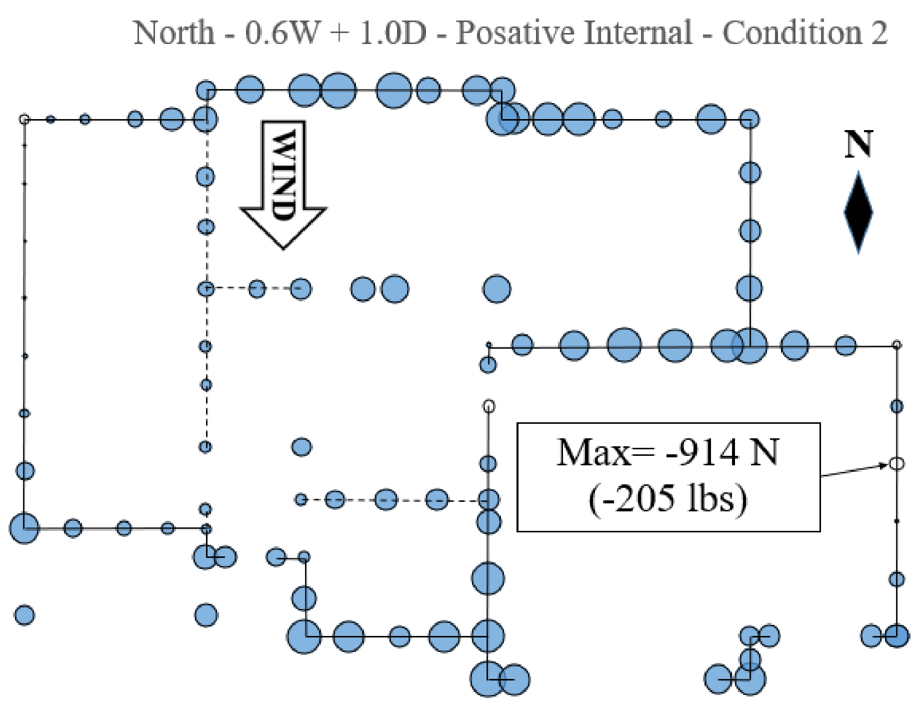


Figure G 10: North 0.6W + 1.0D. Positive internal pressure, Condition 2

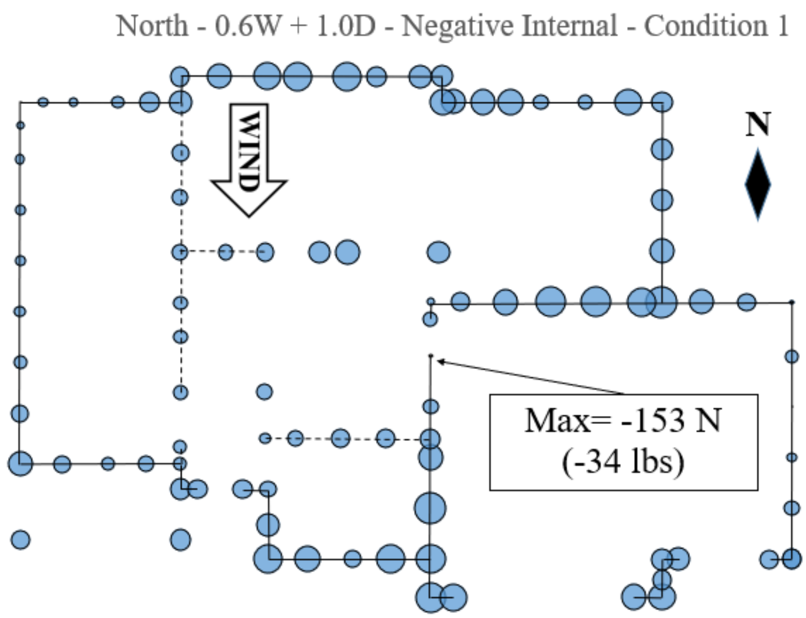


Figure G 11: North 0.6W + 1.0D. Negative internal pressure, Condition 1

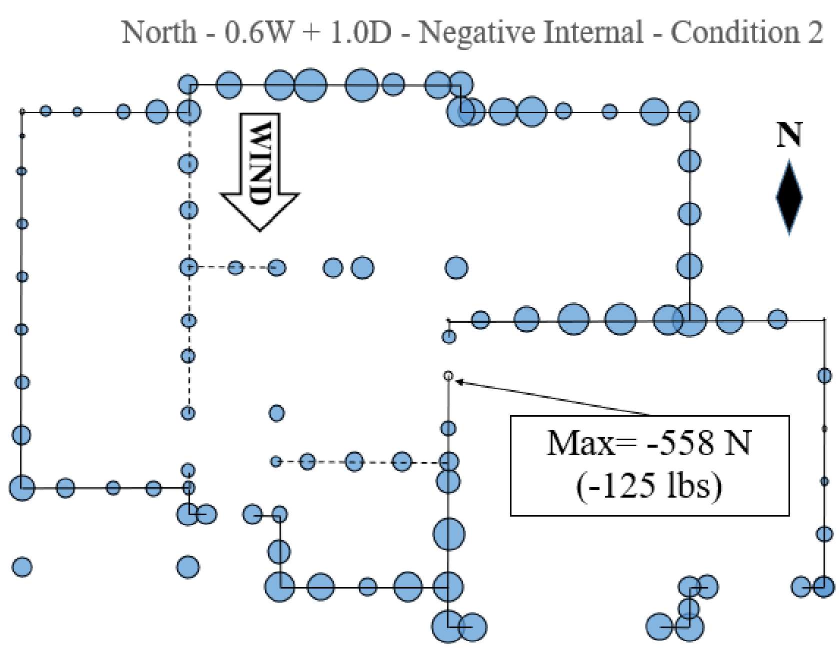


Figure G 12: North 0.6W + 1.0D. Negative internal pressure, Condition 2

South:

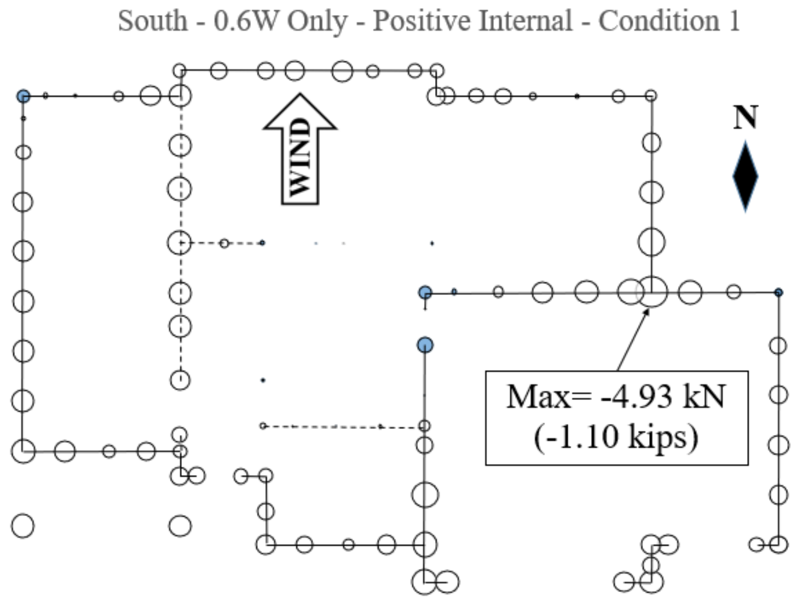


Figure G 13: South Wind Only. Positive internal pressure, Condition 1

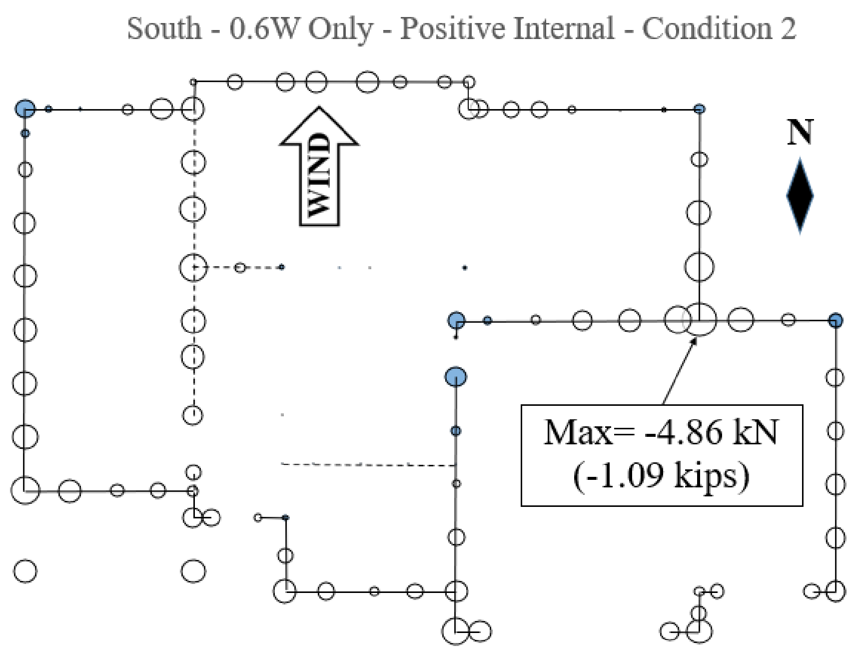


Figure G 14: South Wind Only. Positive internal pressure, Condition 2

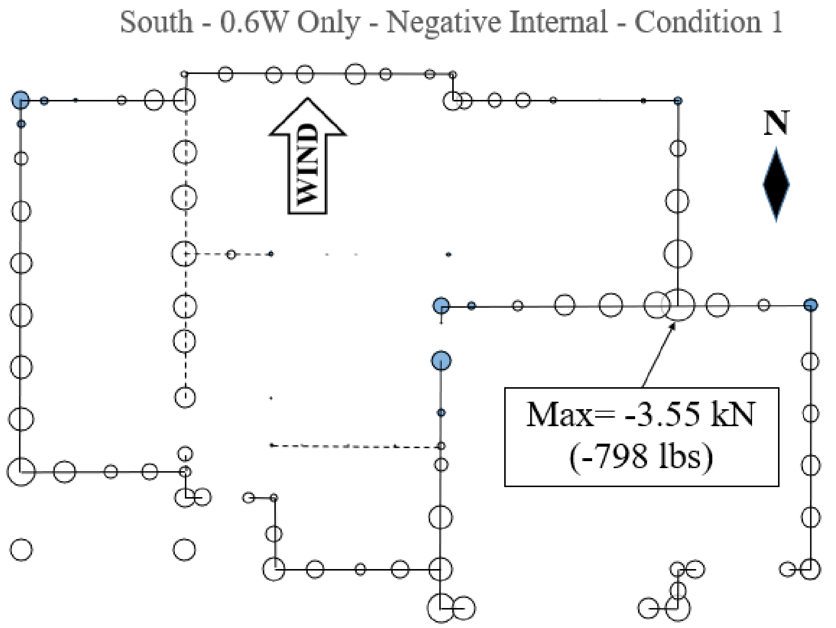


Figure G 15: South Wind Only. Negative internal pressure, Condition 1

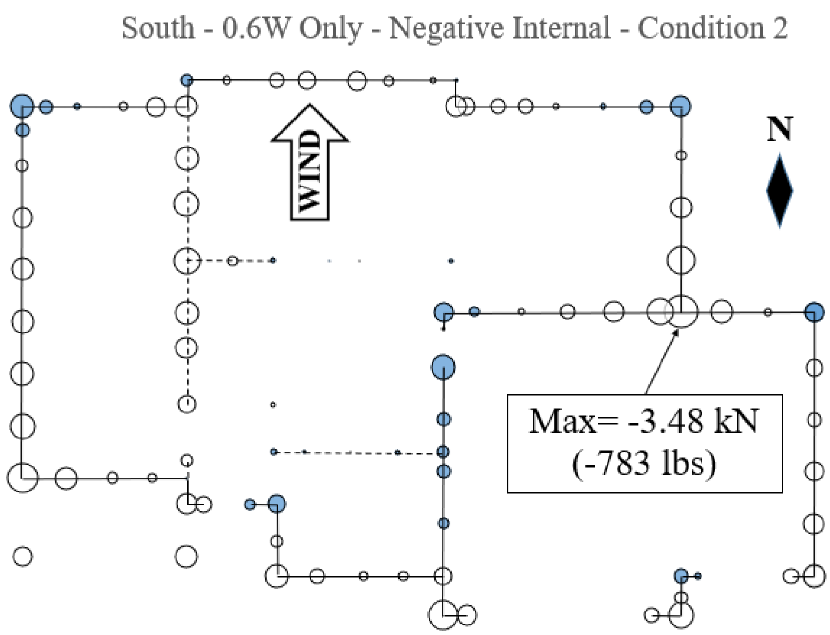


Figure G 16: South Wind Only. Negative internal pressure, Condition 2

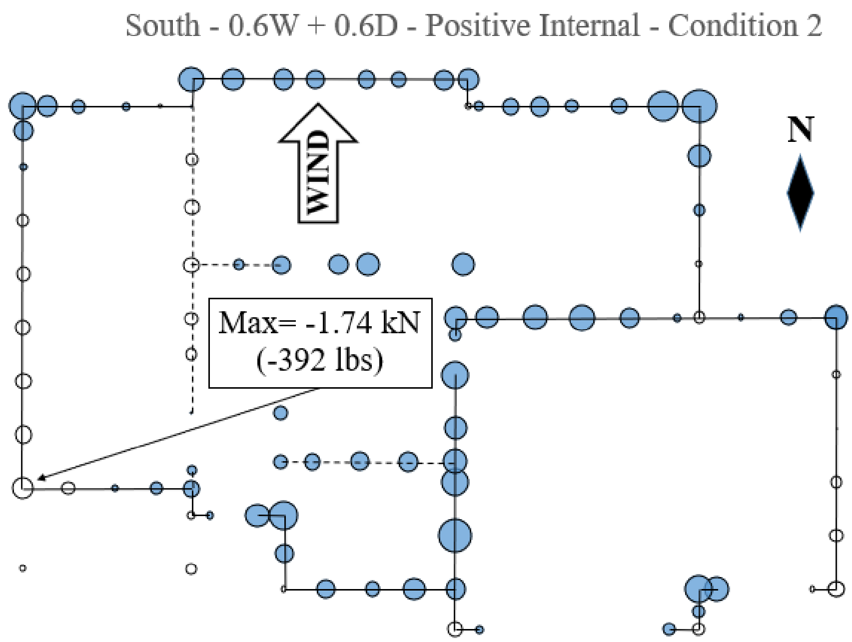


Figure G 17 : South 0.6W + 0.6D. Positive internal pressure, Condition 1

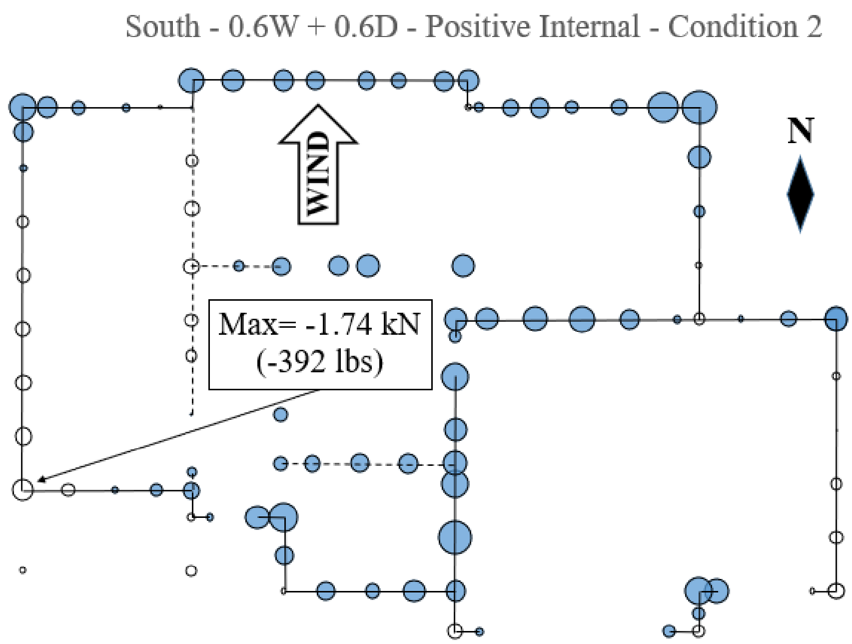


Figure G 18: South 0.6W + 0.6D. Positive internal pressure, Condition 2

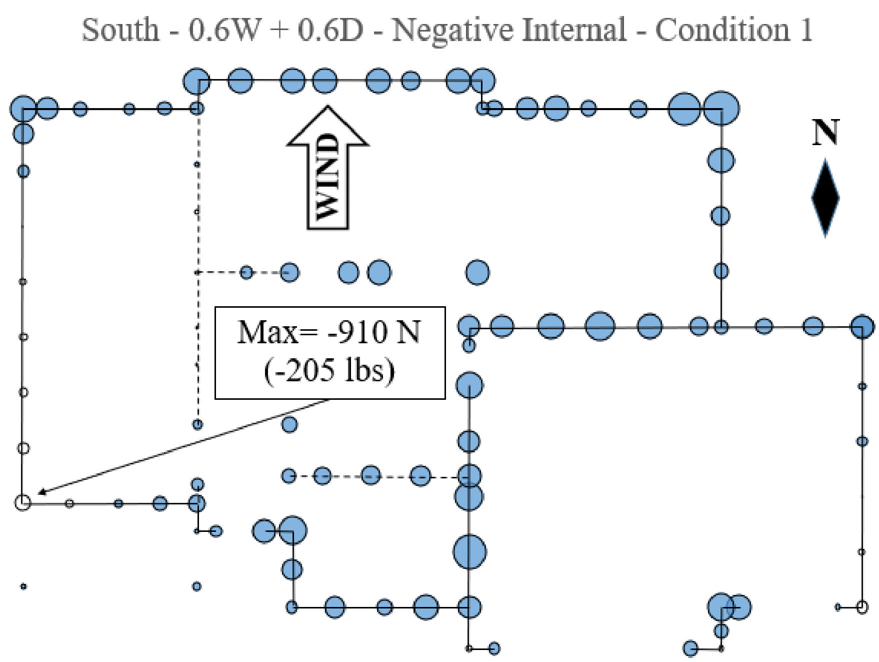


Figure G 19: South 0.6W + 0.6D. Negative internal pressure, Condition 1

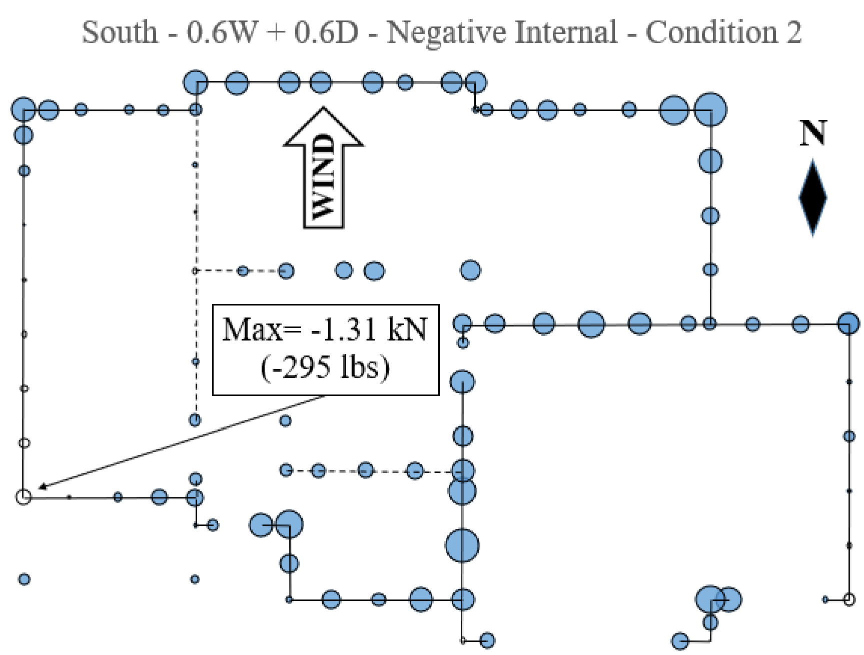


Figure G 20: South 0.6W + 0.6D. Negative internal pressure, Condition 2

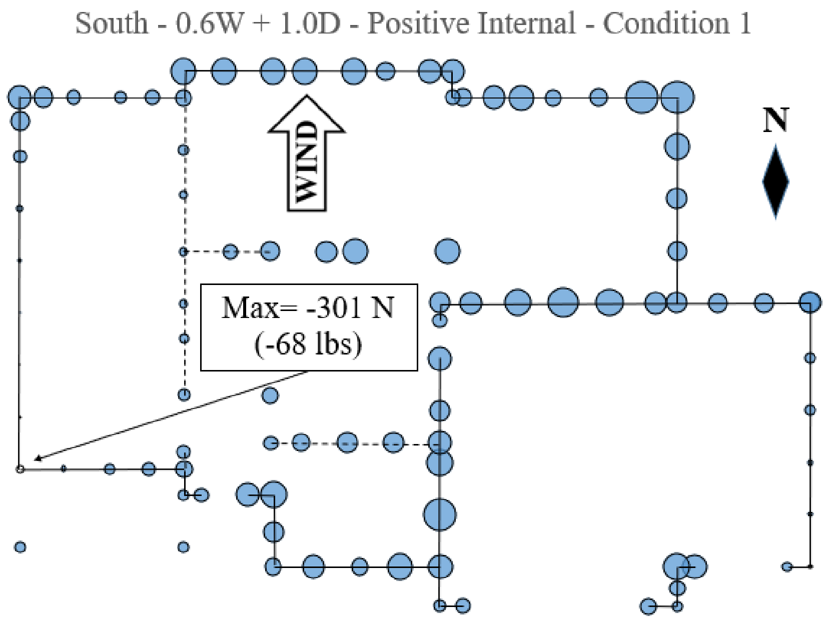


Figure G 21: South 0.6W + 1.0D. Positive internal pressure, Condition 1

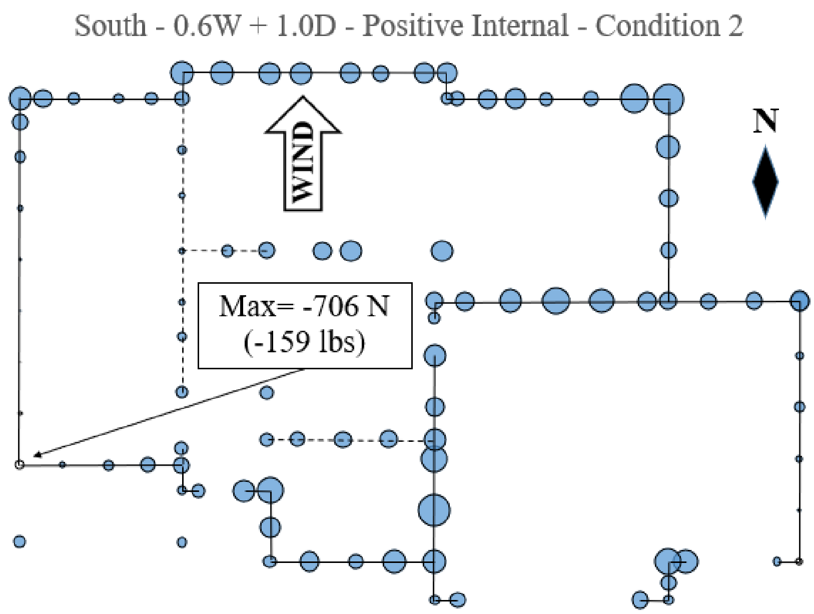


Figure G 22: South 0.6W + 1.0D. Positive internal pressure, Condition 2

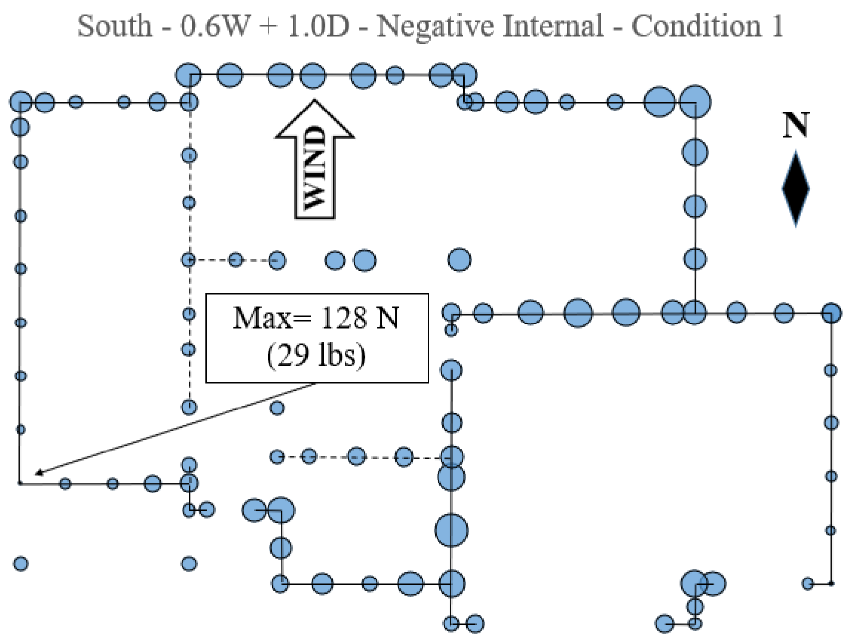


Figure G 23: South 0.6W + 1.0D. Negative internal pressure, Condition 1

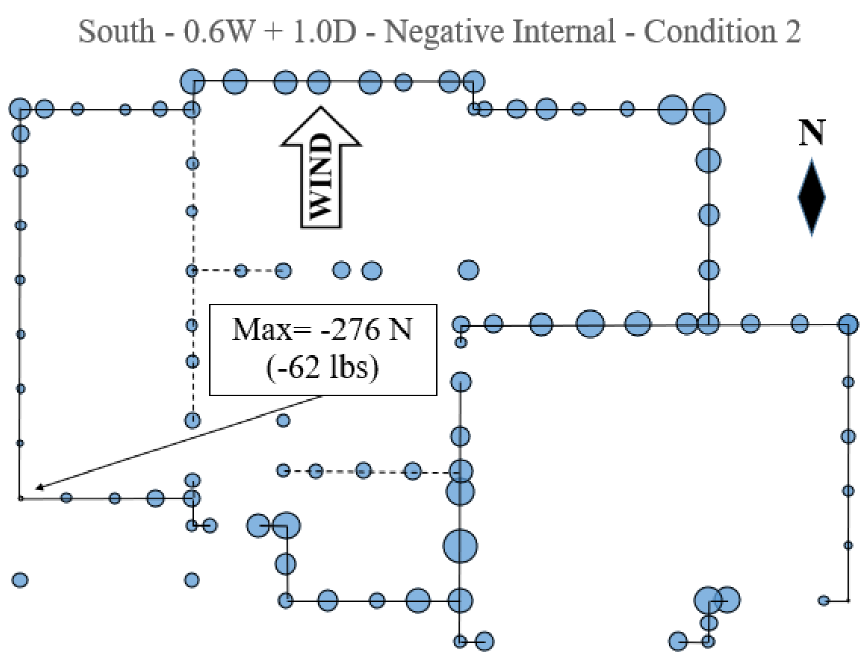


Figure G 24: South 0.6W + 1.0D. Negative internal pressure, Condition 1

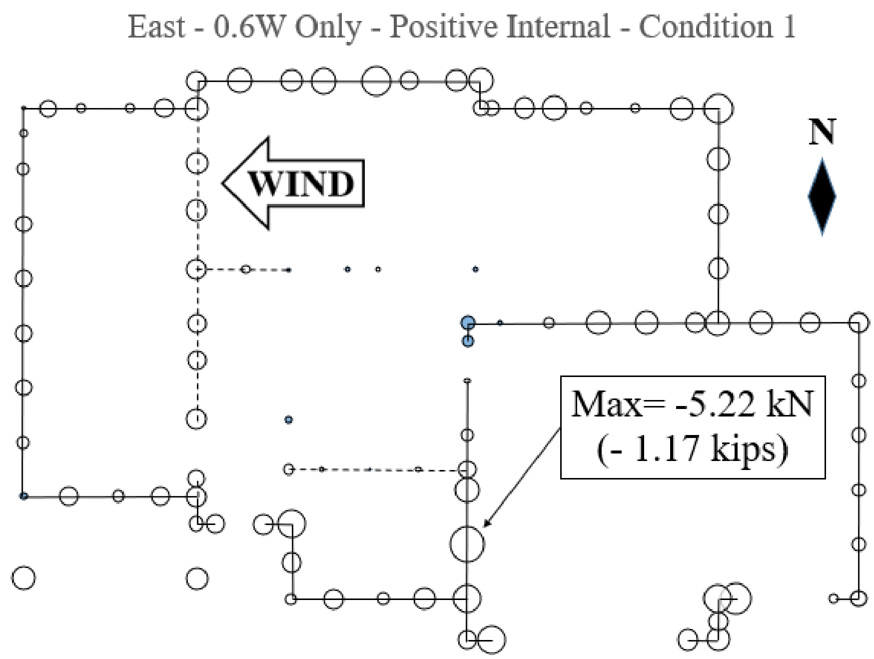


Figure G 25 : East Wind Only. Positive internal pressure, Condition 1

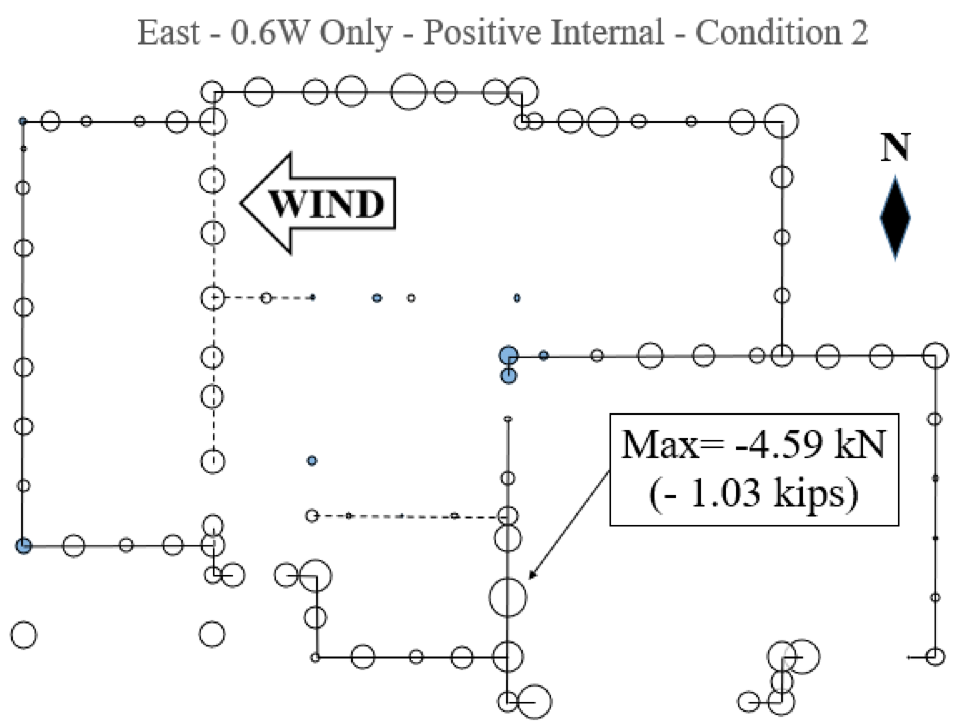


Figure G 26: East Wind Only. Positive internal pressure, Condition 2

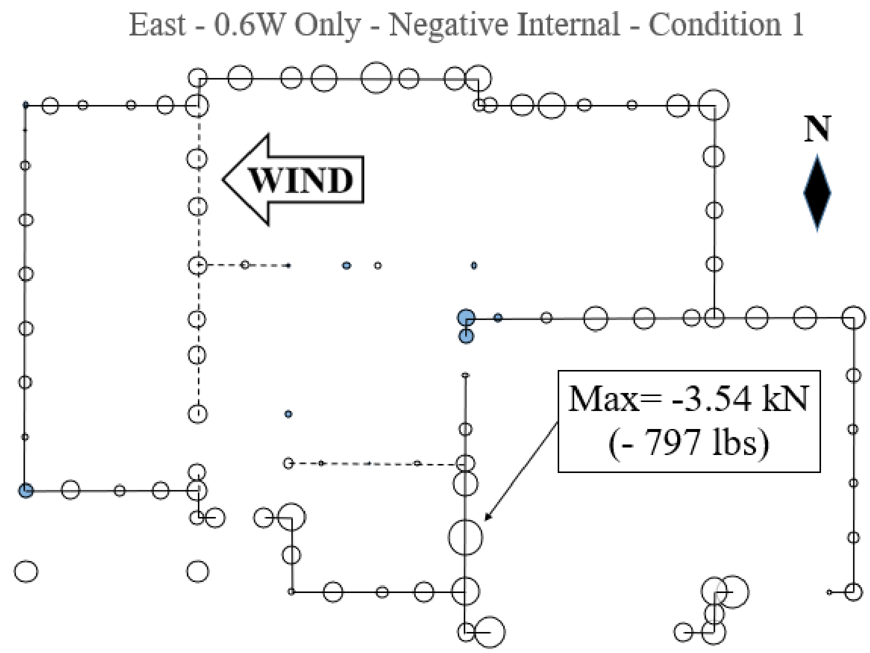


Figure G 27: East Wind Only. Negative internal pressure, Condition 1

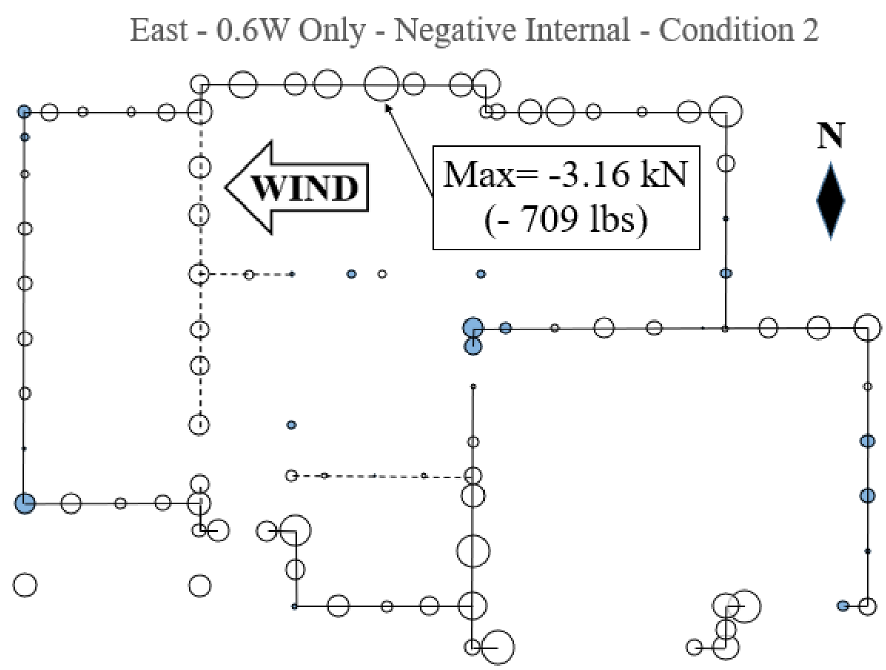


Figure G 28: East Wind Only. Negative internal pressure, Condition 2

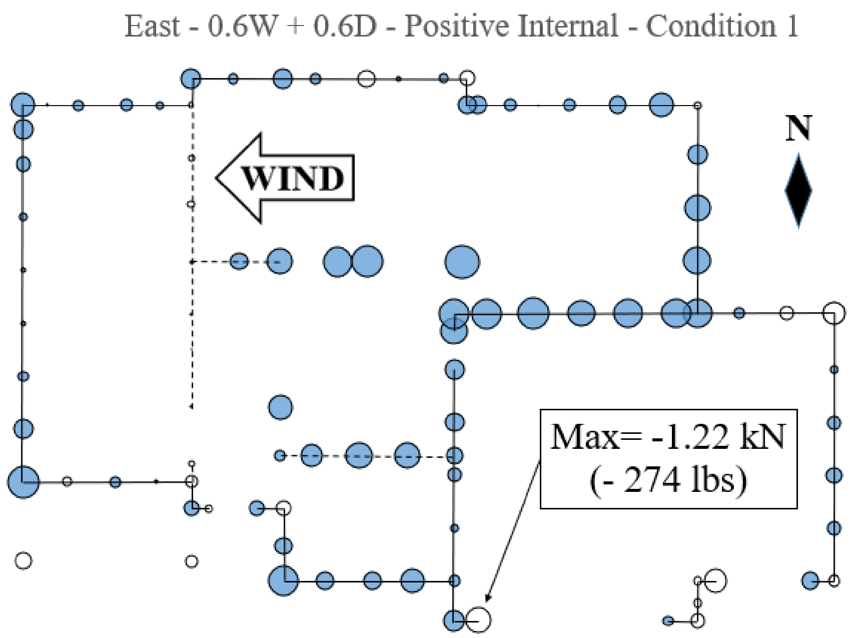


Figure G 29: East 0.6W + 0.6D. Positive internal pressure, Condition 1

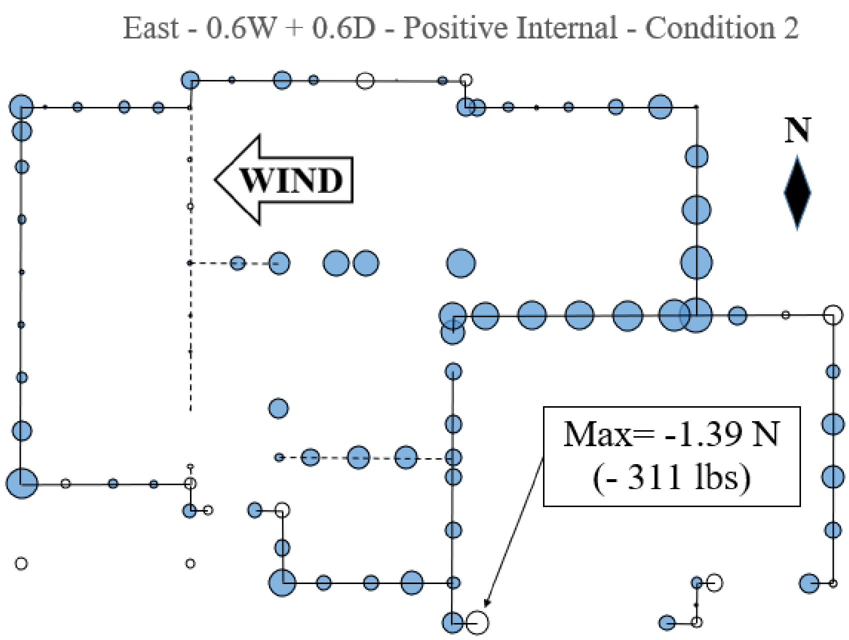


Figure G 30: East 0.6W + 0.6D. Positive internal pressure, Condition 2

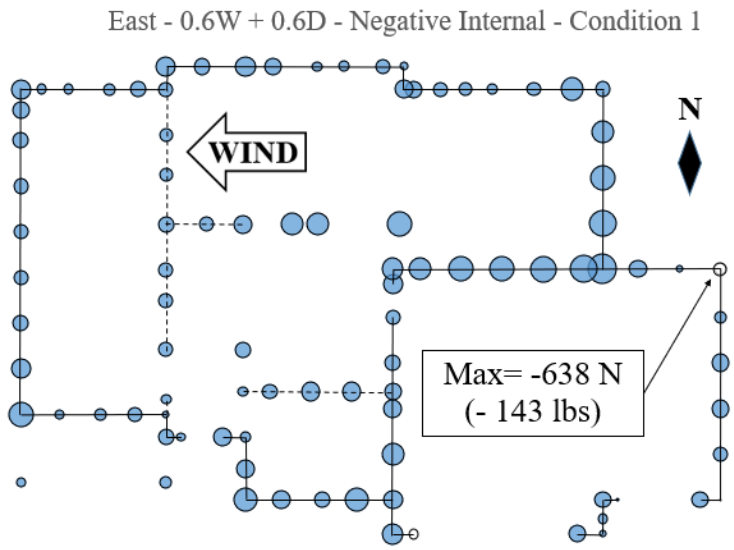


Figure G 31: East 0.6W + 0.6D. Negative internal pressure, Condition 1

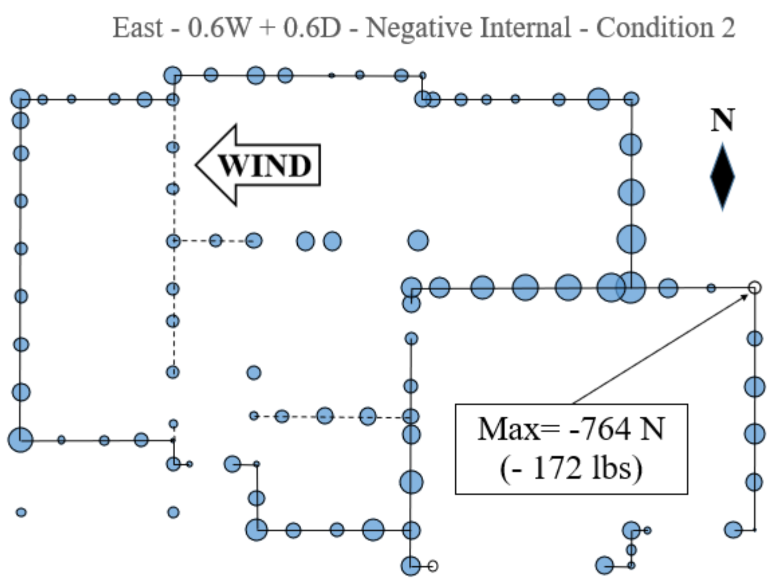


Figure G 32: East 0.6W + 0.6D. Negative internal pressure, Condition 2

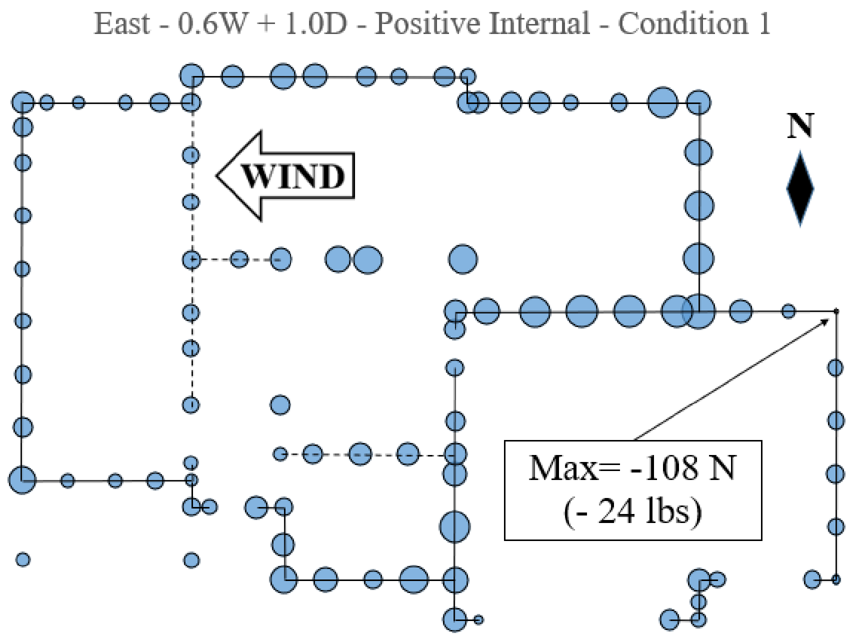


Figure G 33: East 0.6W + 1.0D. Positive internal pressure, Condition 1

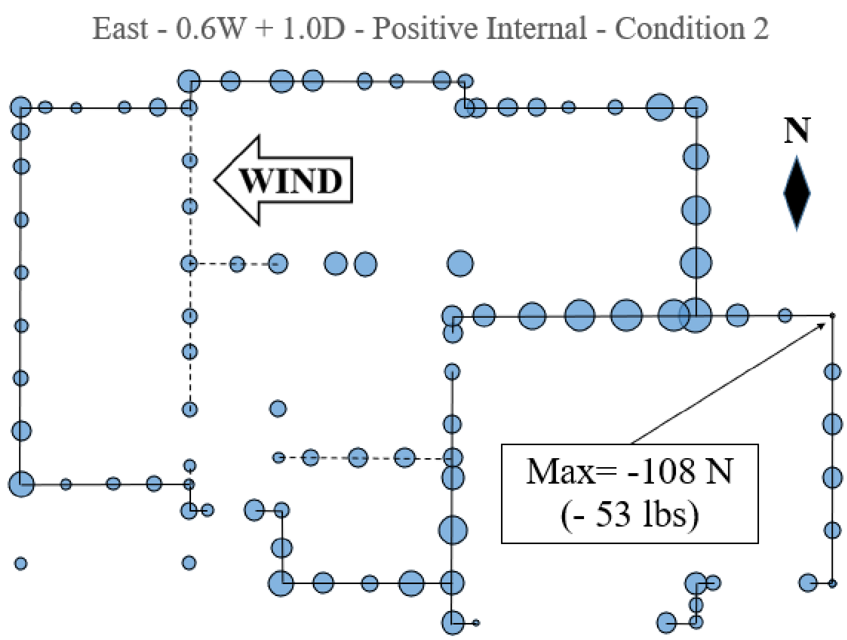


Figure G 34: East 0.6W + 1.0D. Positive internal pressure, Condition 2

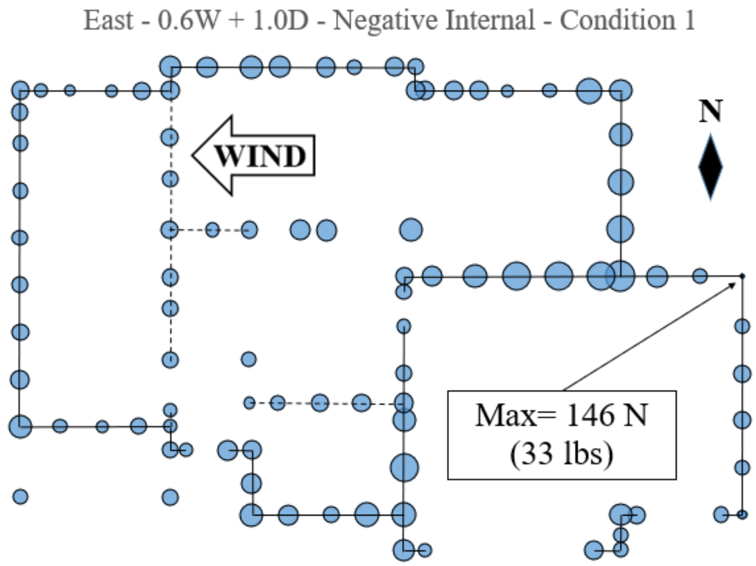


Figure G 35: East 0.6W + 1.0D. Negative internal pressure, Condition 1

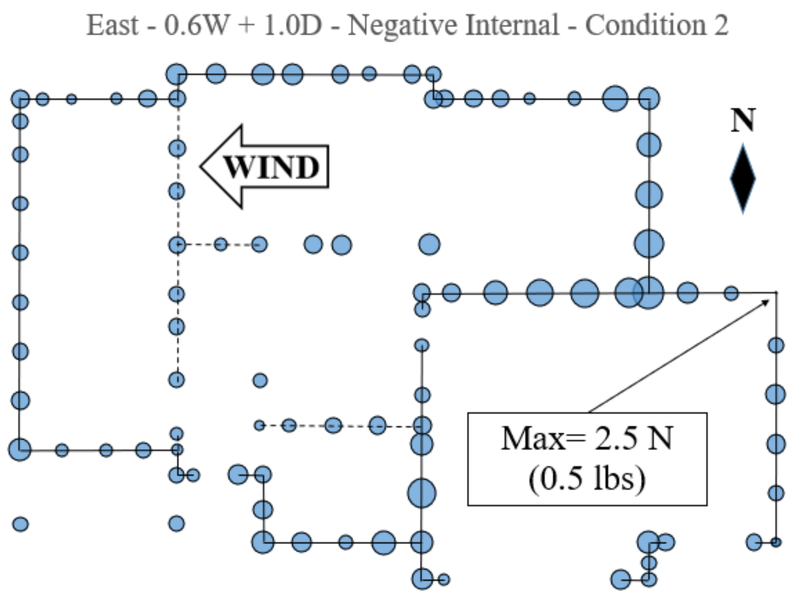


Figure G 36: East 0.6W + 1.0D. Negative internal pressure, Condition 2

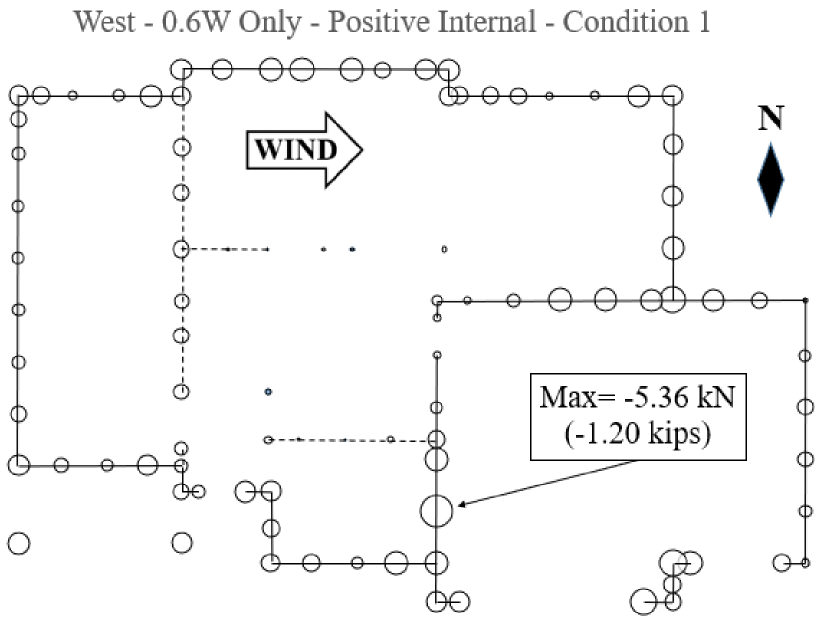


Figure G 37: West Wind Only. Positive internal pressure, Condition 1

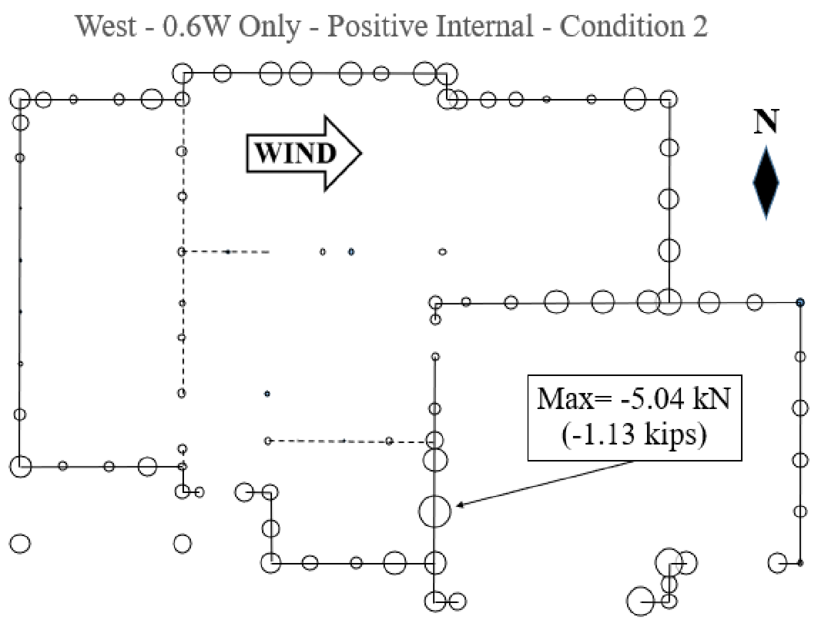


Figure G 38: West Wind Only. Positive internal pressure, Condition 2

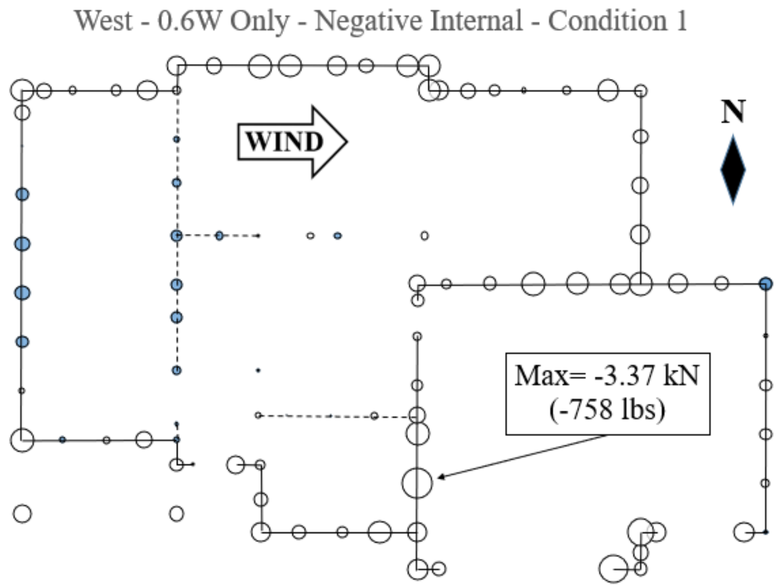


Figure G 39: West Wind Only. Negative internal pressure, Condition 1

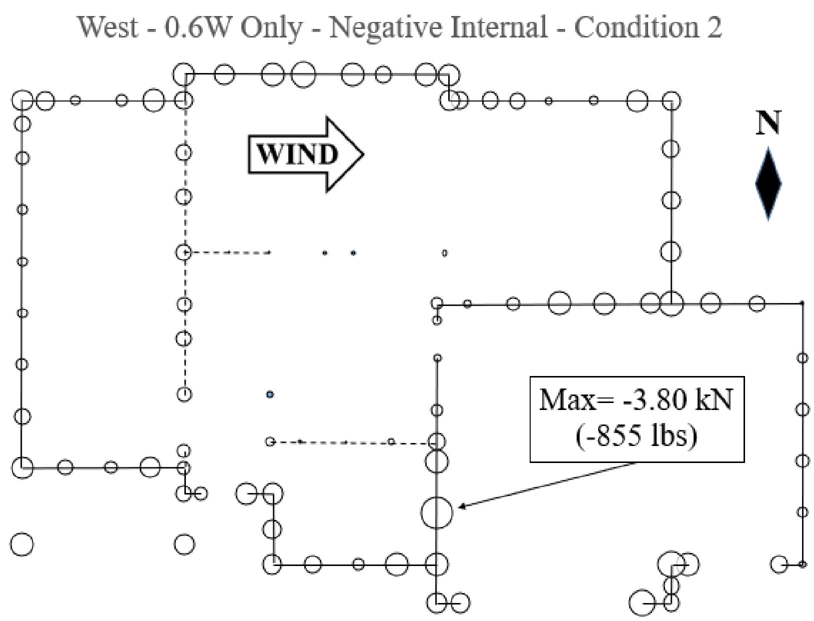


Figure G 40: West Wind Only. Negative internal pressure, Condition 2

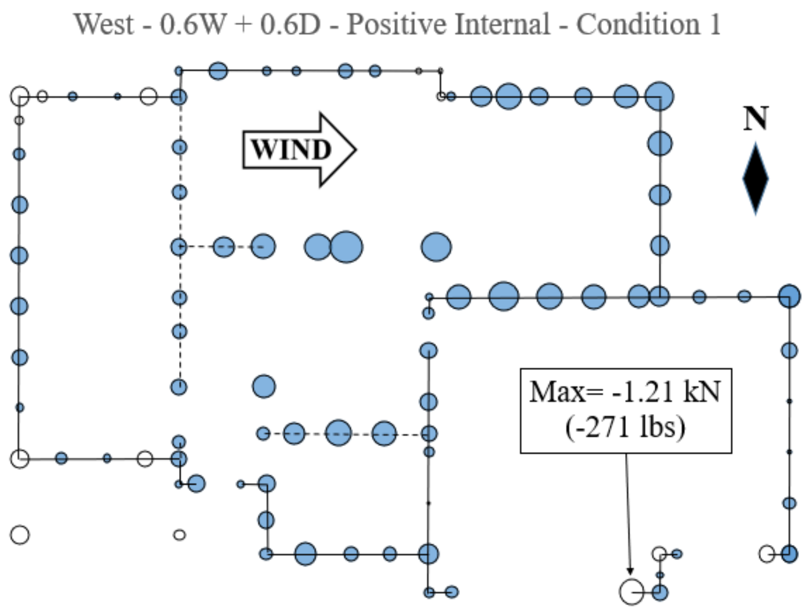


Figure G 41: West 0.6W + 0.6D. Positive internal pressure, Condition 1

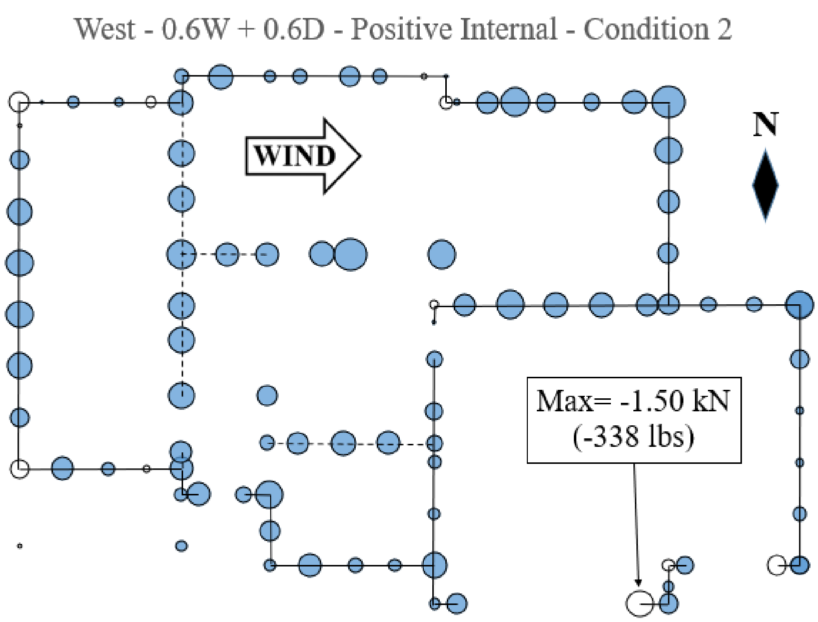


Figure G 42: West 0.6W + 0.6D. Positive internal pressure, Condition 2

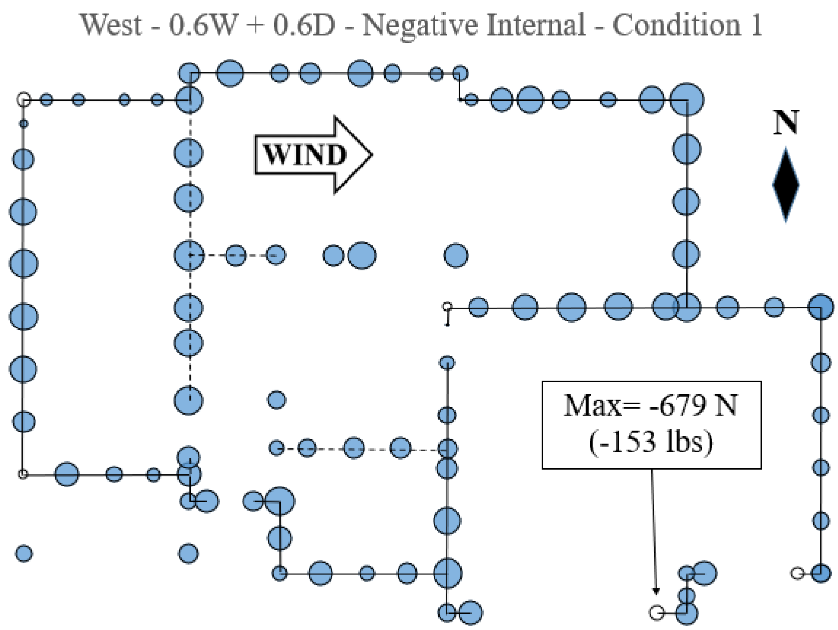


Figure G 43; West 0.6W + 0.6D. Negative internal pressure, Condition 1

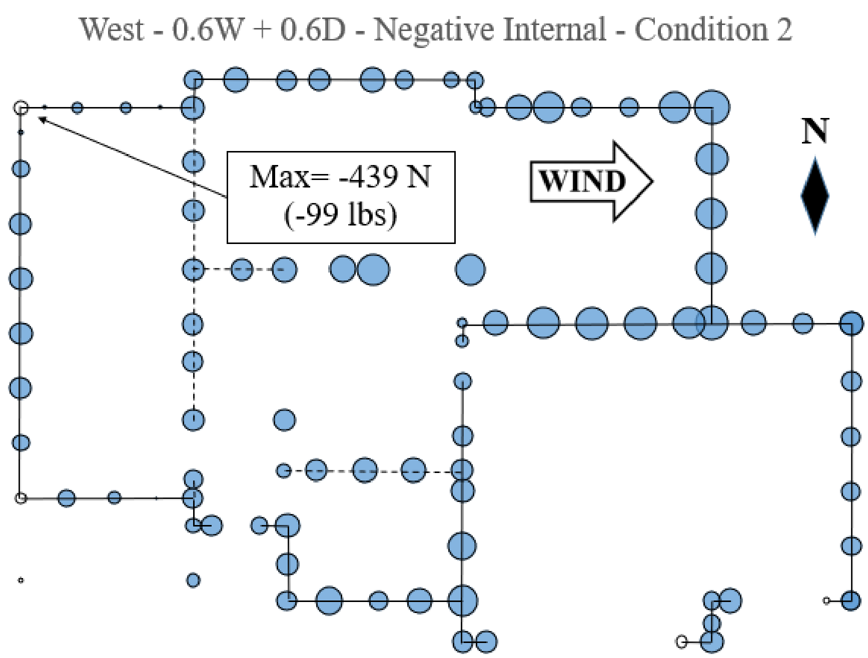


Figure G 44: West 0.6W + 0.6D. Negative internal pressure, Condition 2

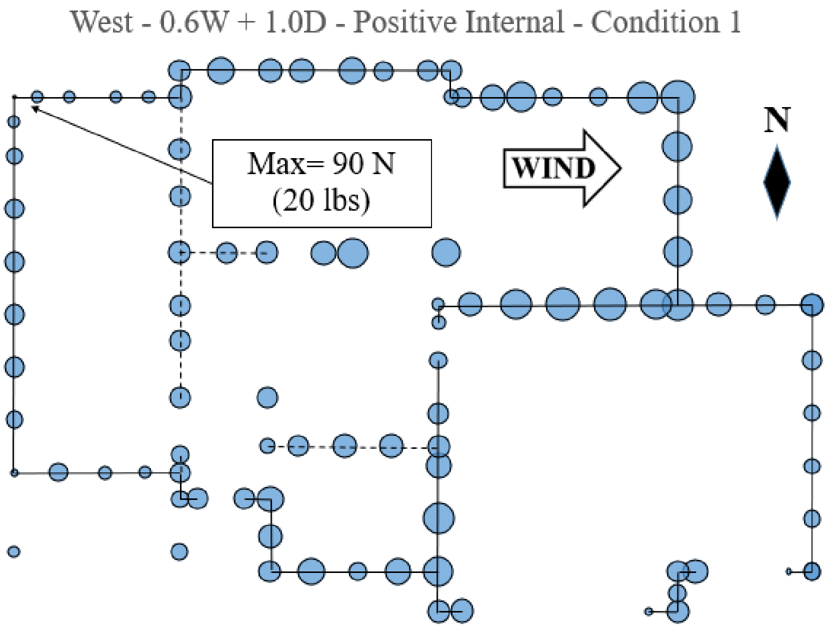


Figure G 45: West 0.6W + 1.0D. Positive internal pressure, Condition 1

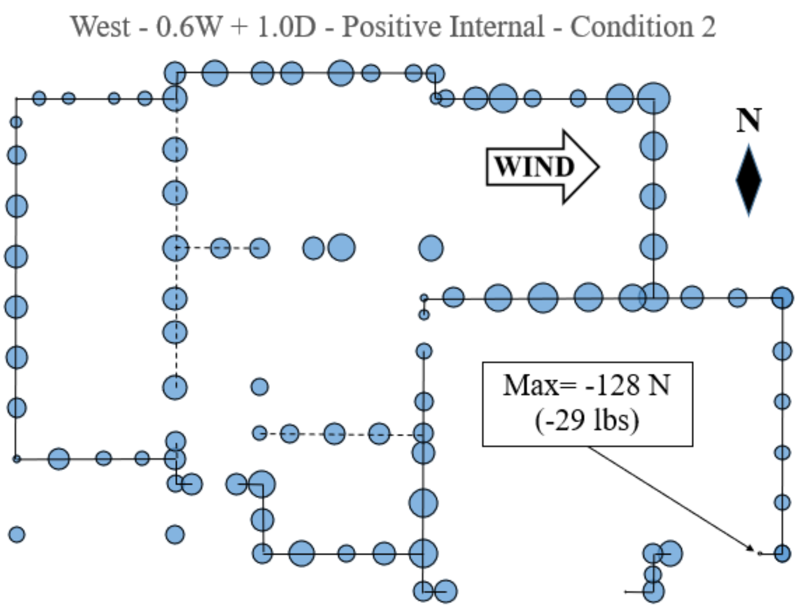


Figure G 46: West 0.6W + 1.0D. Positive internal pressure, Condition 1

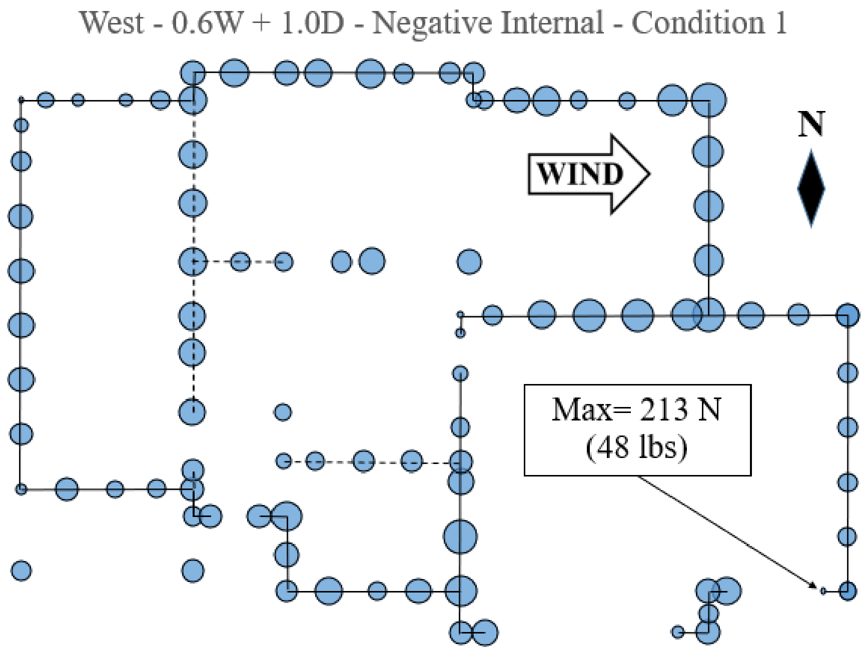


Figure G 47: West 0.6W + 1.0D. Negative internal pressure, Condition 1

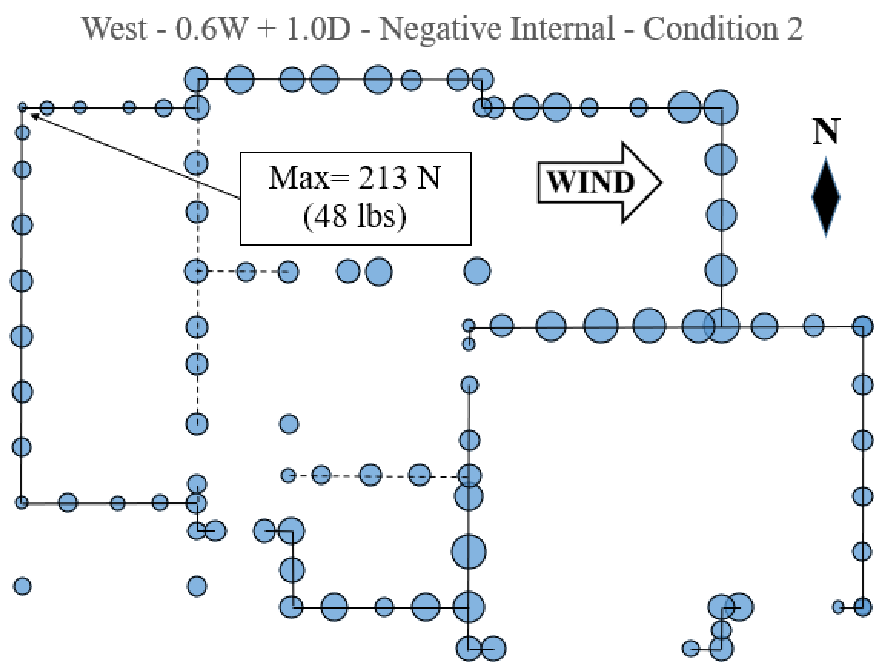


Figure G 48: West 0.6W + 1.0D. Negative internal pressure, Condition 1

APPENDIX H

WIND BASE SHEAR

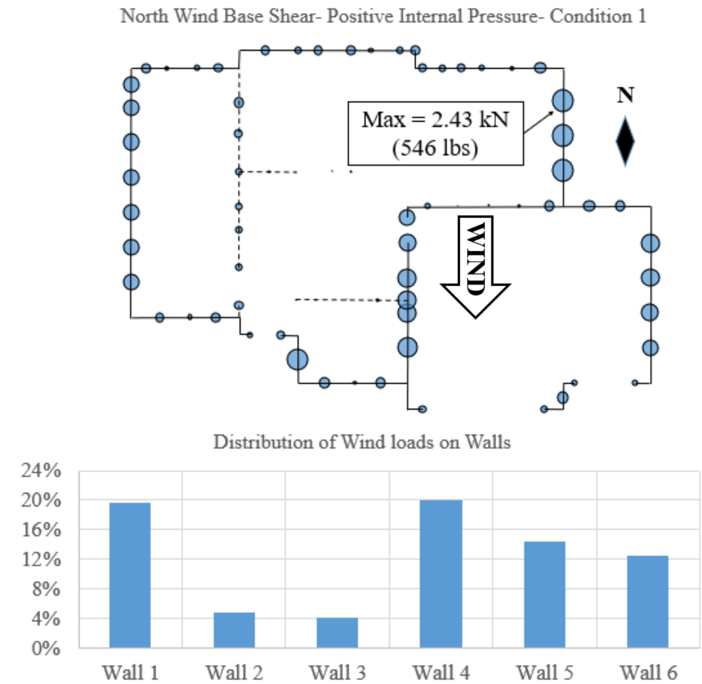


Figure H 1: North, 0.6W Base shear. Positive internal pressure, condition 1

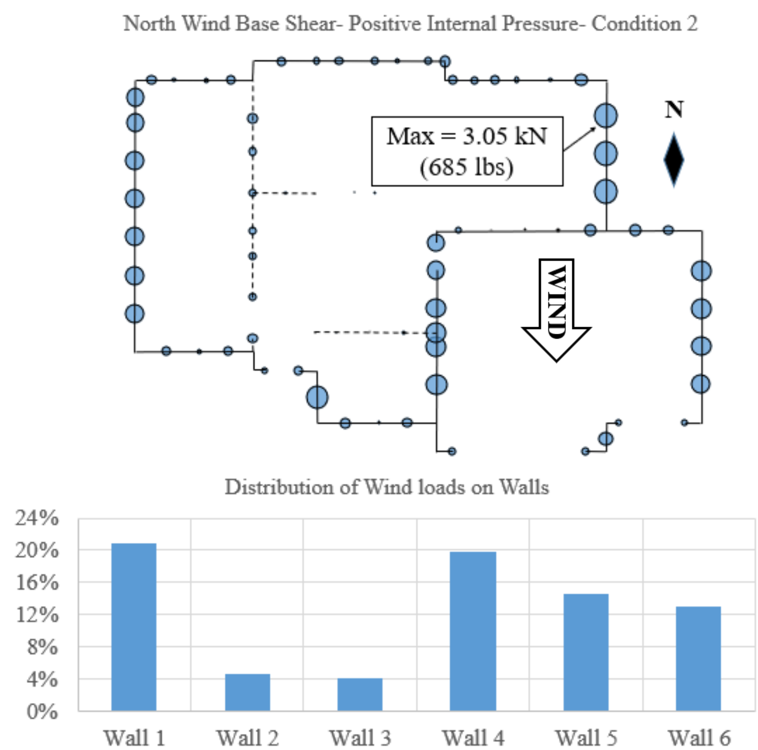


Figure H 2: North, 0.6W Base shear. Positive internal pressure, condition 2

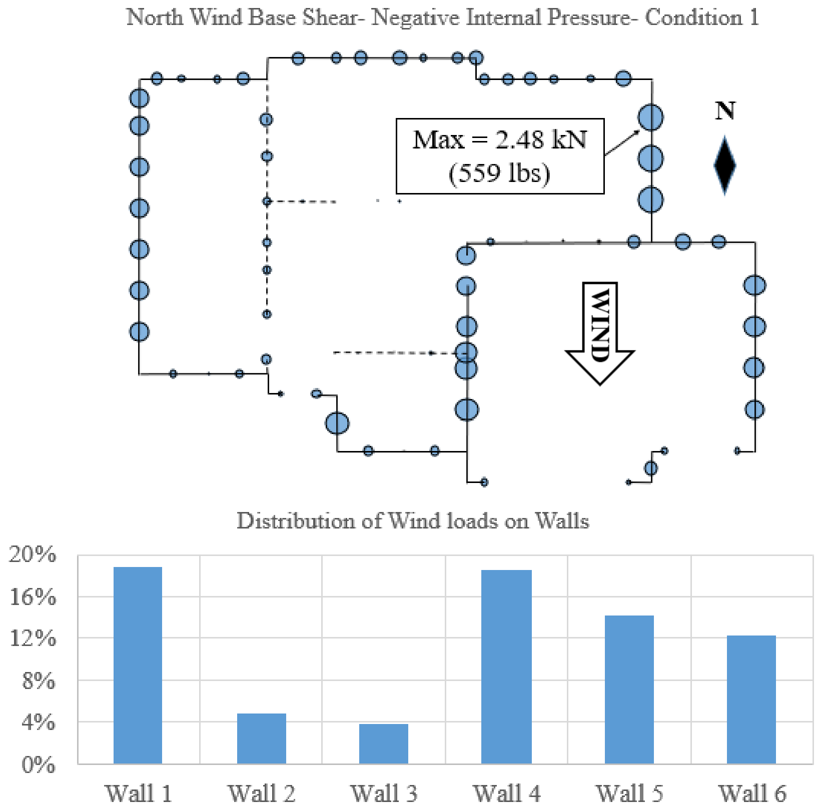


Figure H 3: North, 0.6W Base shear. Negative internal pressure, condition 1

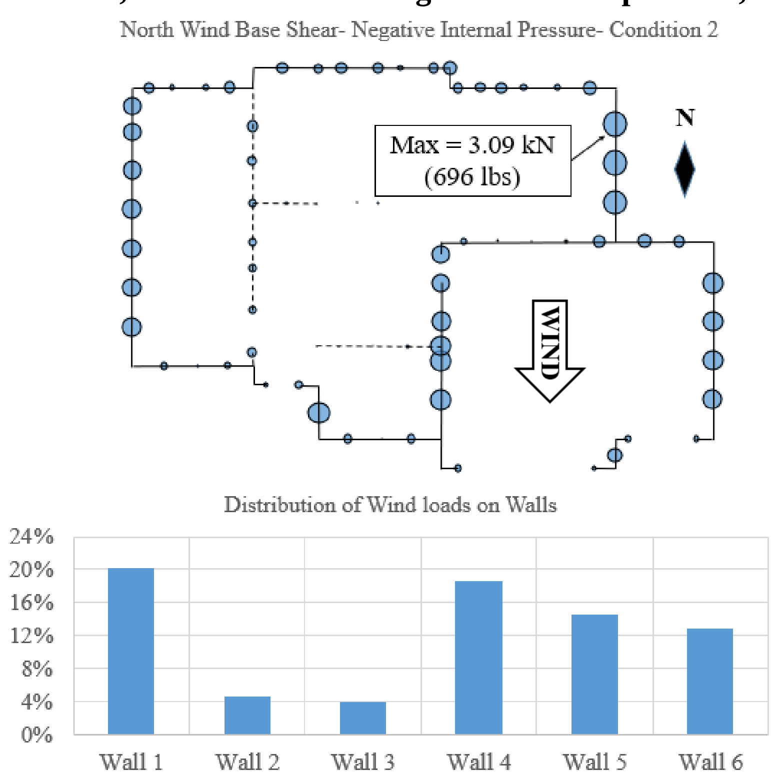


Figure H 4: North, 0.6W Base shear. Negative internal pressure, condition 2

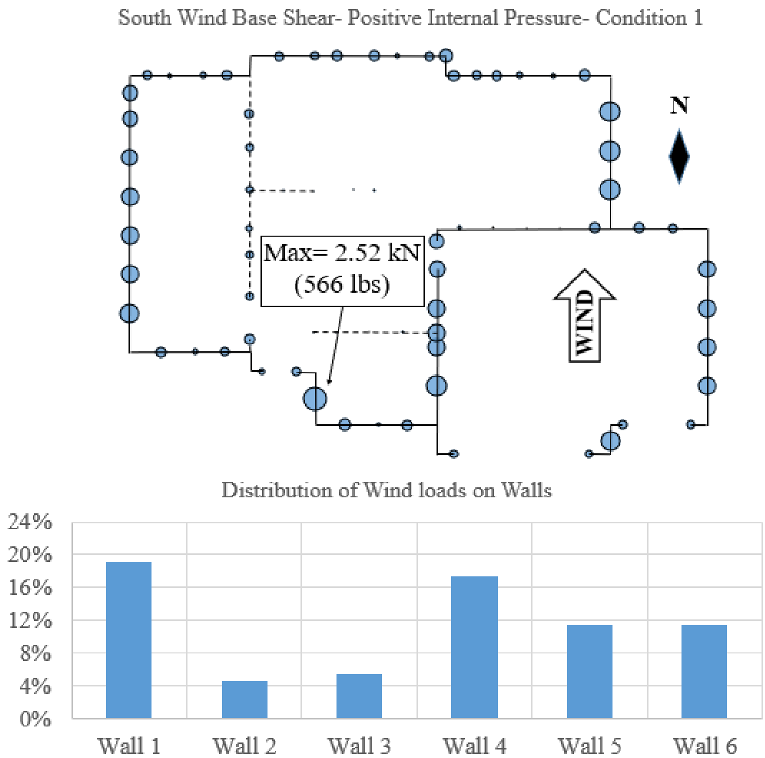


Figure H 5: South, 0.6W Base shear. Positive internal pressure, condition 1

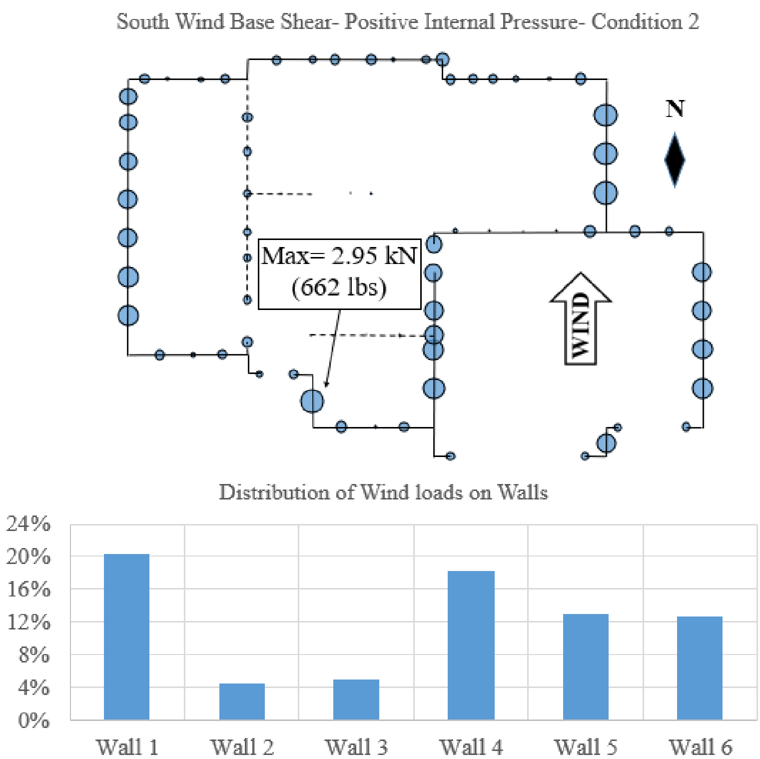
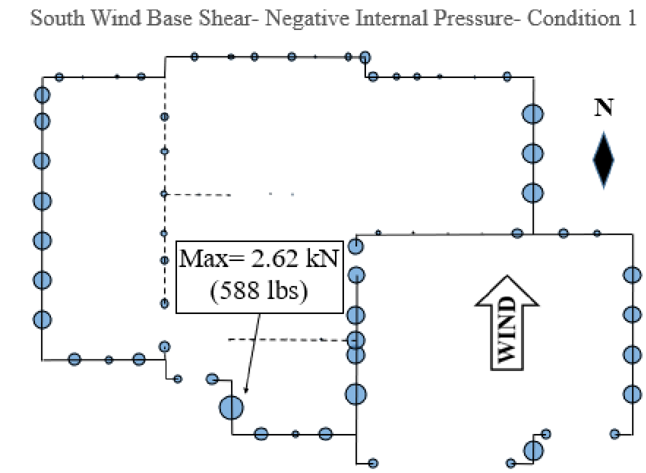


Figure H 6: South, 0.6W Base shear. Positive internal pressure, condition 2



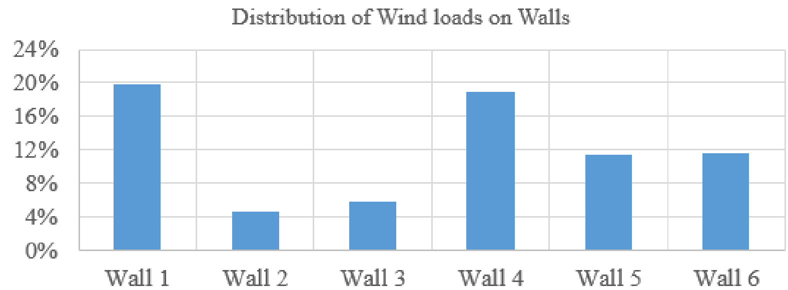
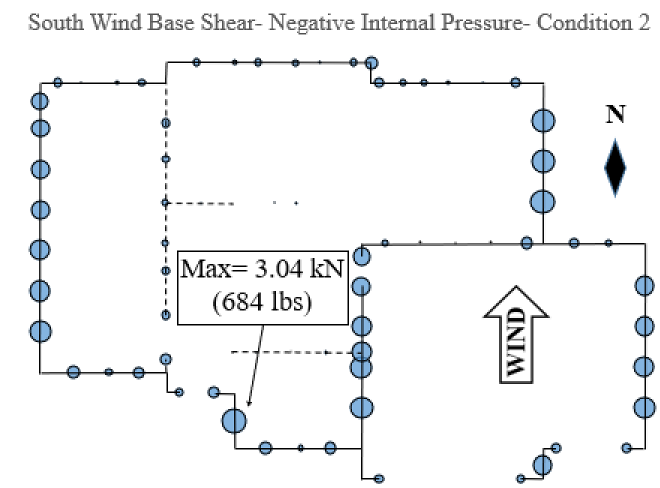


Figure H 7: South, 0.6W Base shear. Negative internal pressure, condition 1



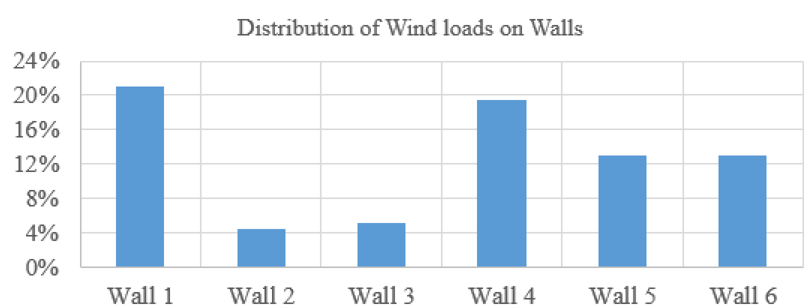


Figure H 8: South, 0.6W Base shear. Negative internal pressure, condition 2

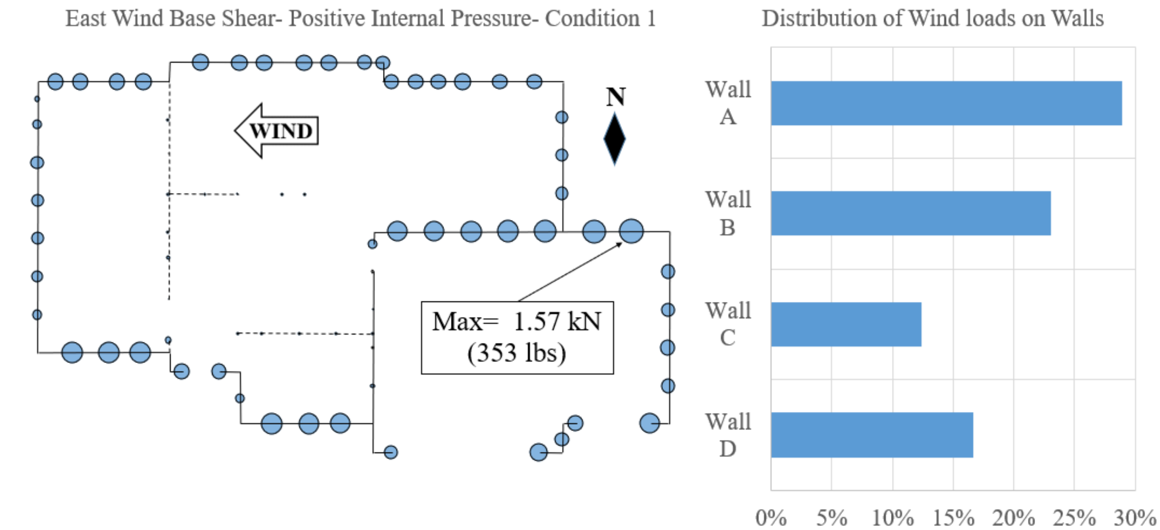


Figure H 9: East, 0.6W Base shear. Positive internal pressure, condition 1

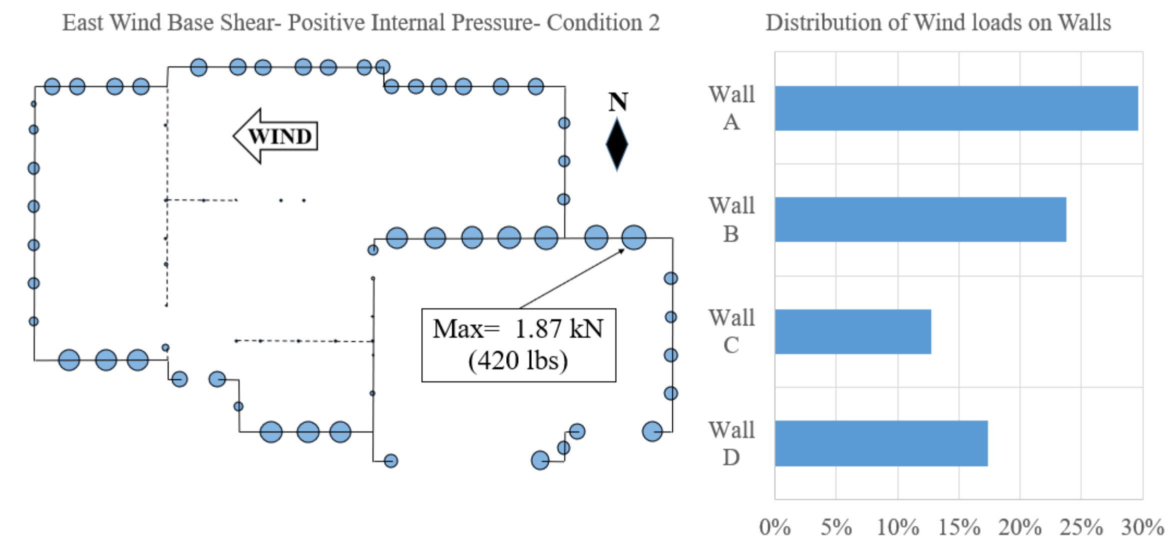


Figure H 10: East, 0.6W Base shear. Positive internal pressure, condition 2

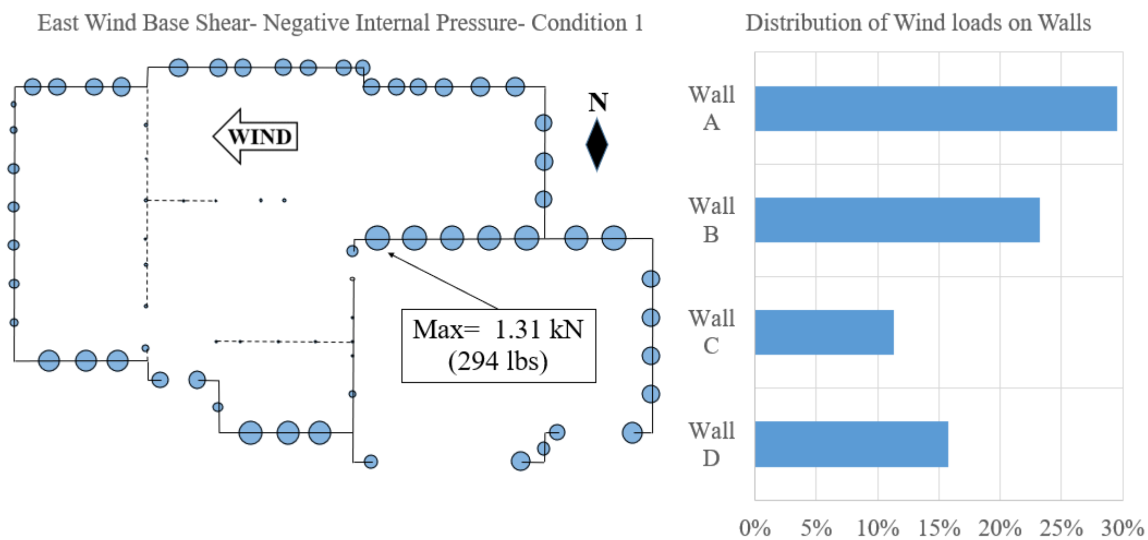


Figure H 11: East, 0.6W Base shear, Negative internal pressure, condition 1

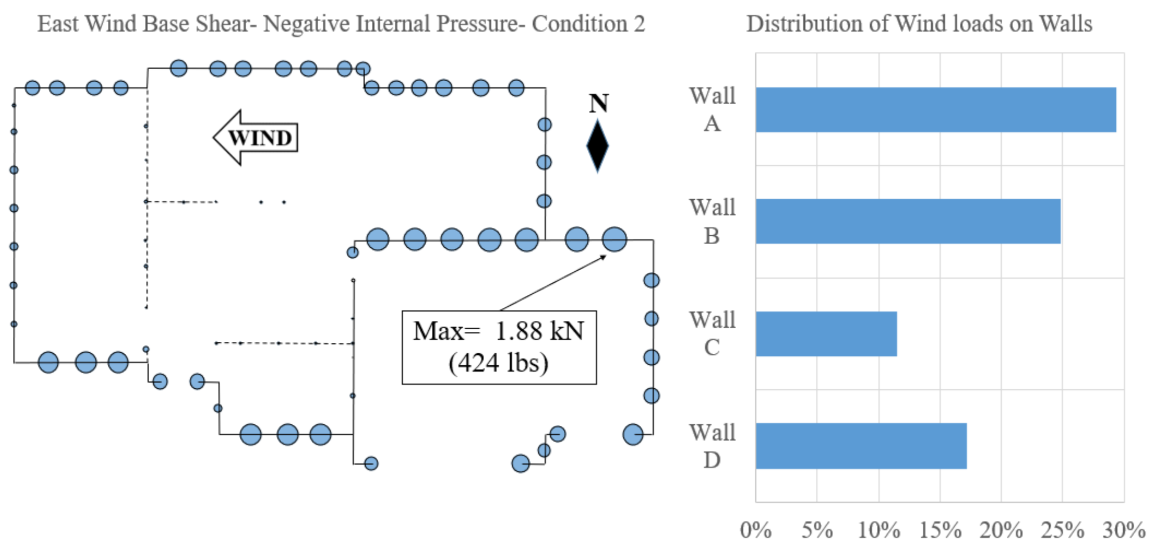


Figure H 12: East, 0.6W Base shear. Negative internal pressure, condition 2

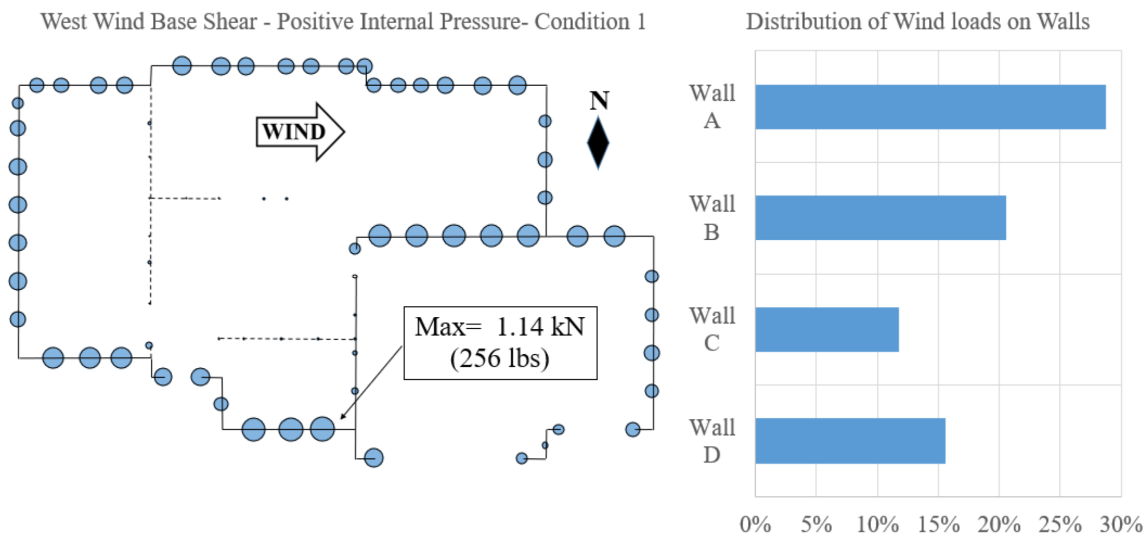


Figure H 13: West, 0.6W Base shear. Positive internal pressure, condition 1

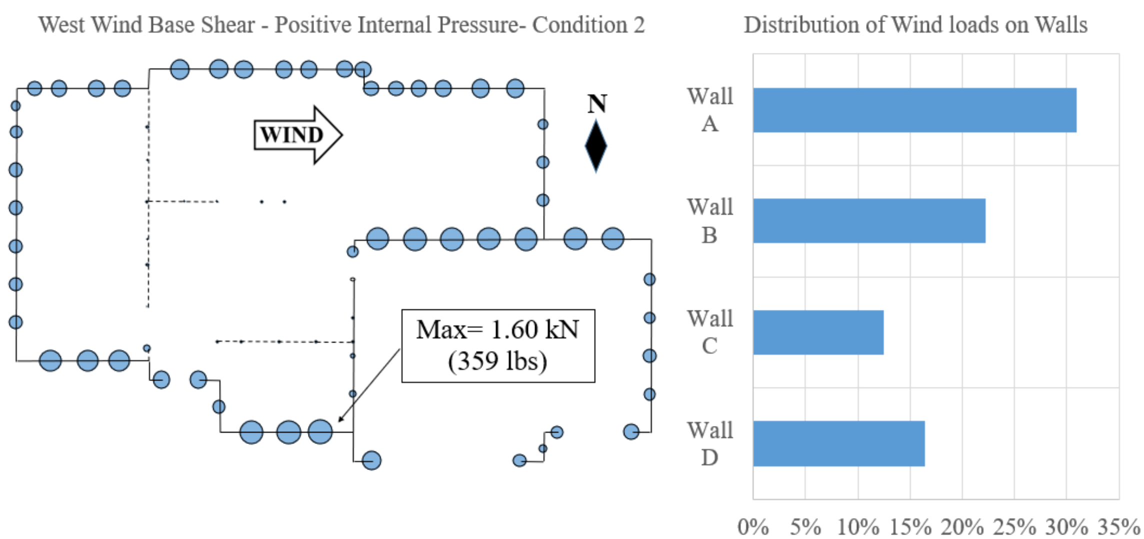


Figure H 14: West, 0.6W Base shear. Positive internal pressure, condition 2

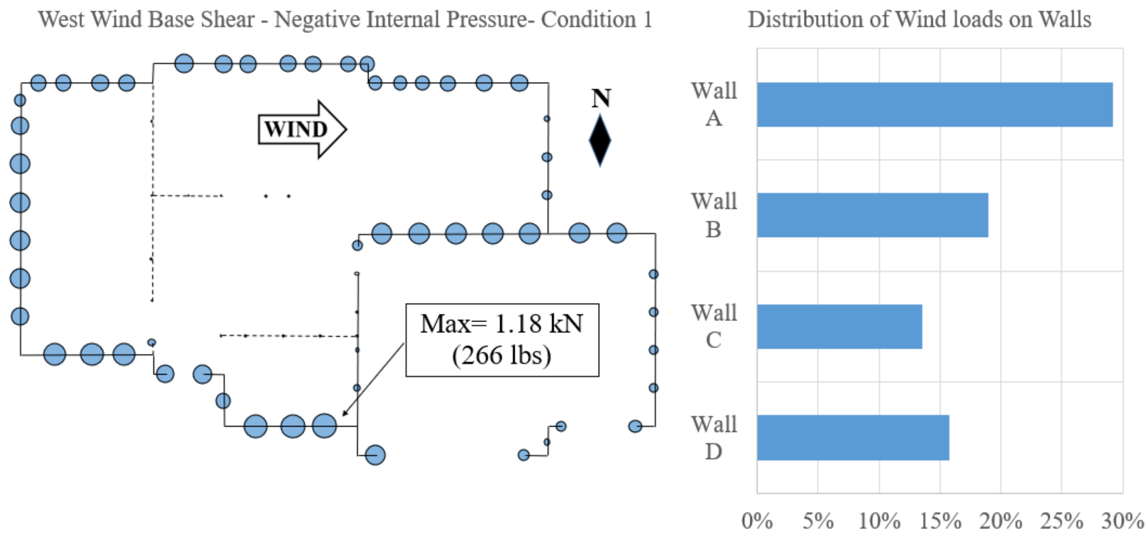


Figure H 15: West, 0.6W Base shear. Negative internal pressure, condition

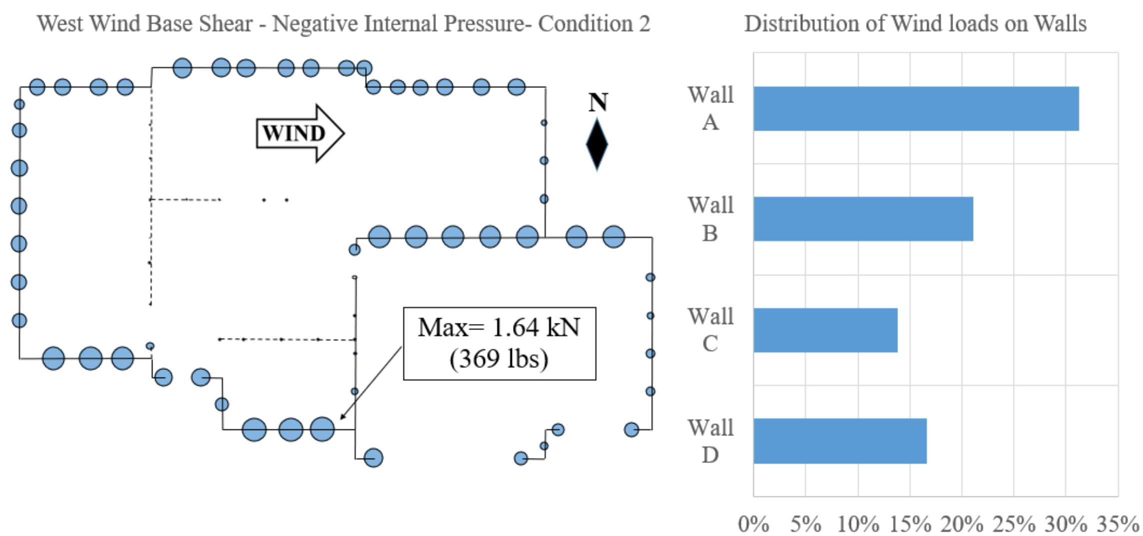
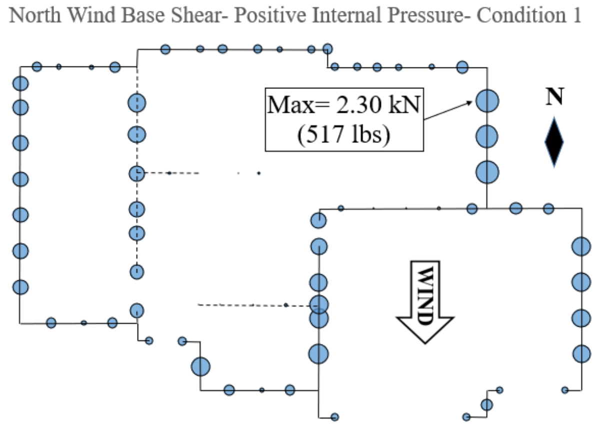


Figure H 16: West, 0.6W Base shear. Negative internal pressure, condition 2

Modified Structure with a shear wall added to "Wall 2"



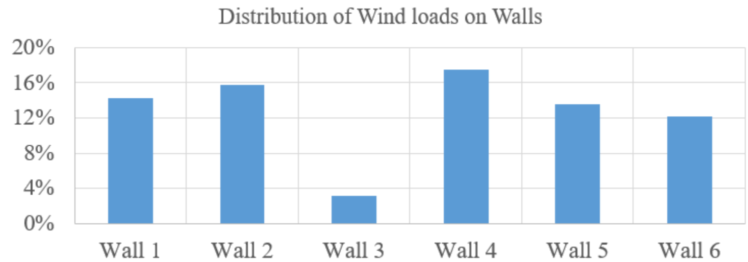
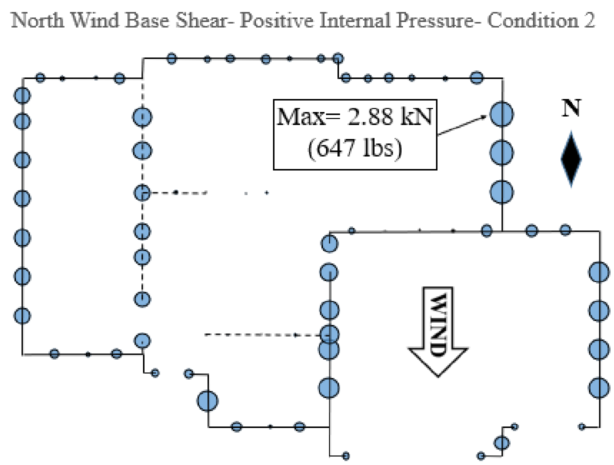


Figure H 17: Modified structure base shear. North 0.6W, positive internal pressure, condition 1.



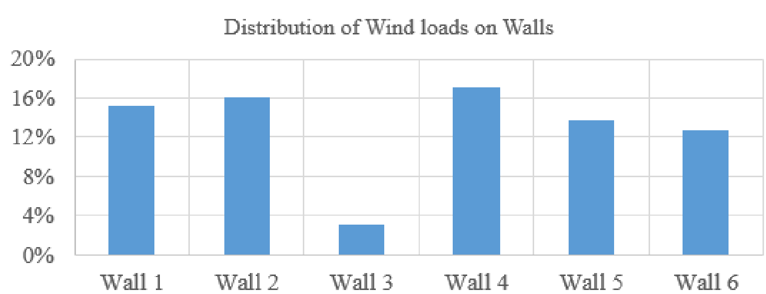
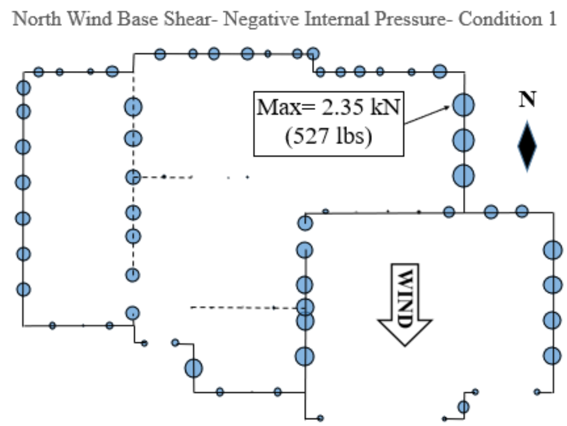


Figure H 18: Modified structure base shear. North 0.6W, positive internal pressure, condition 2.



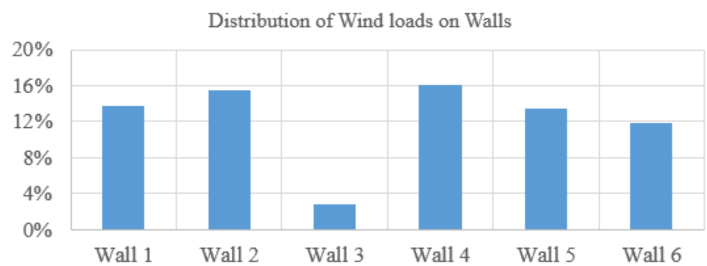
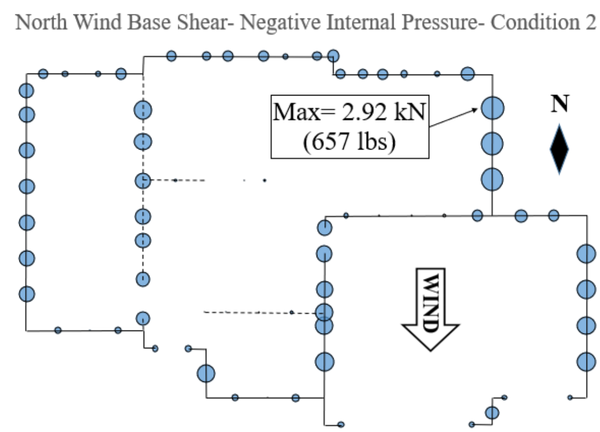


Figure H 19: Modified structure base shear. North 0.6W, negative internal pressure, condition 1.



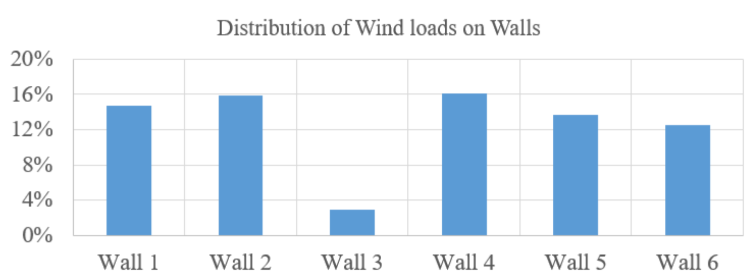


Figure H 20: Modified structure base shear. North 0.6W, negative internal pressure, condition 2.

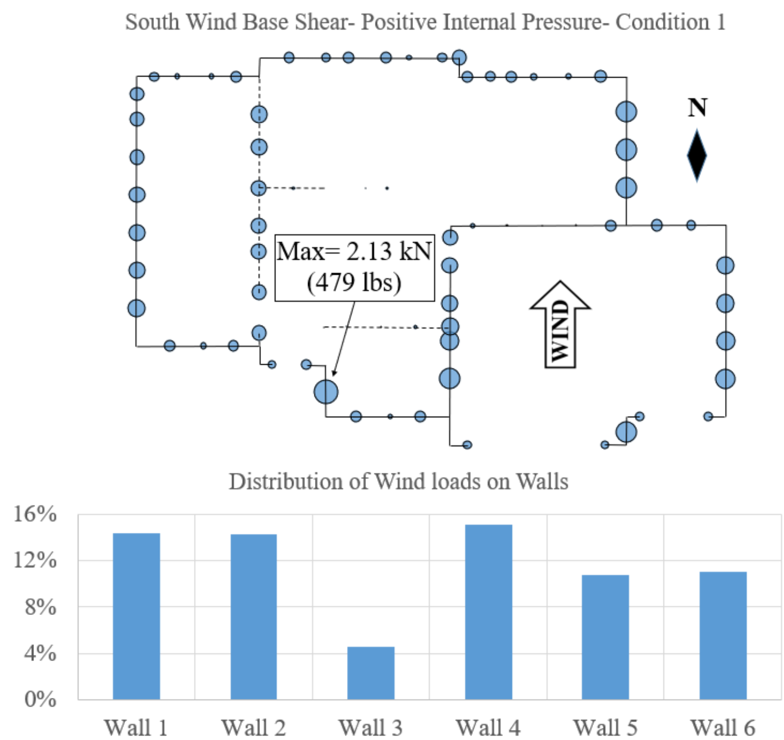


Figure H 21: Modified structure base shear. South 0.6W, positive internal pressure, condition 1.

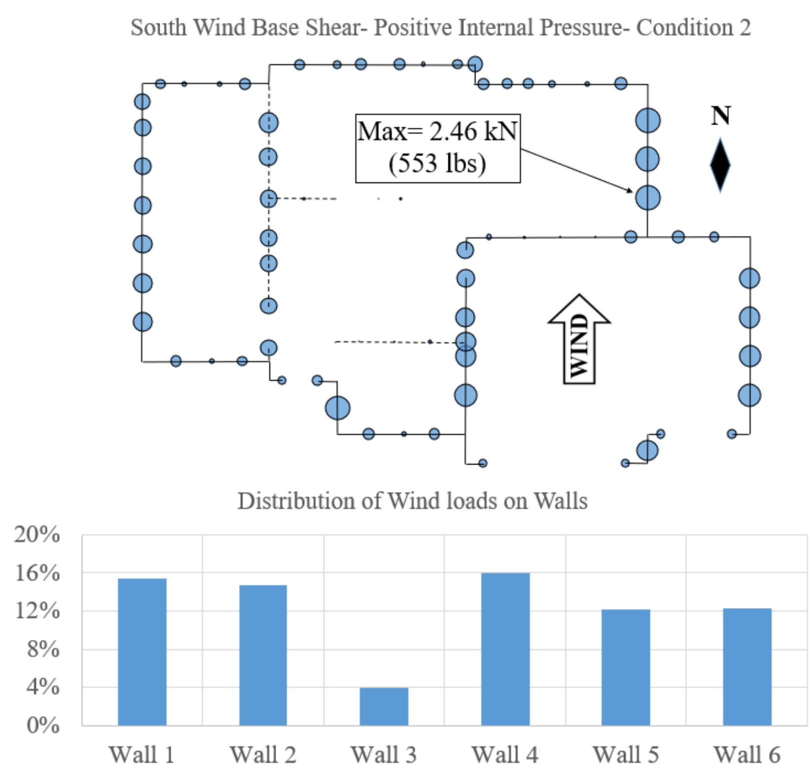


Figure H 22: Modified structure base shear. South 0.6W, positive internal pressure, condition 2.

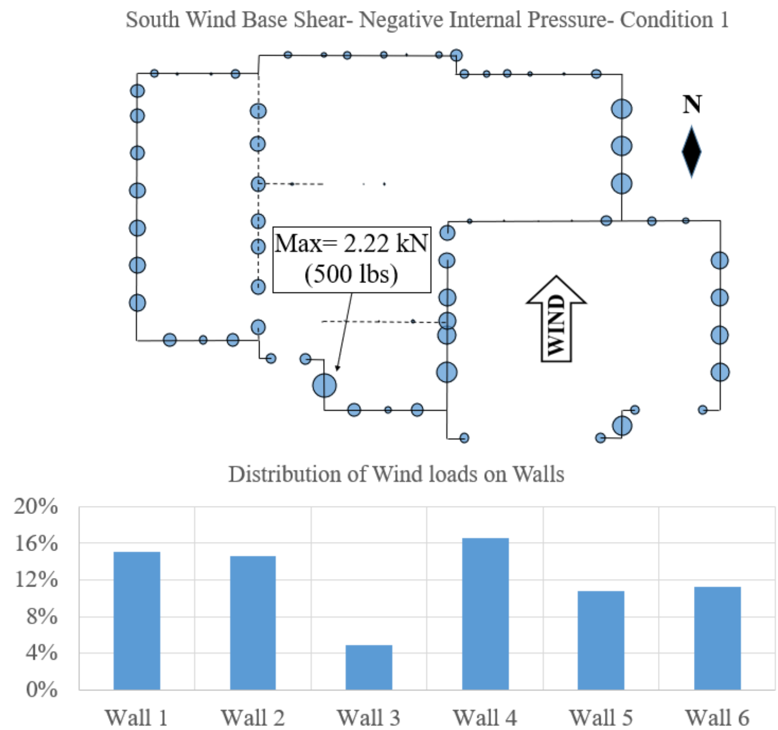


Figure H 23: Modified structure base shear. South 0.6W, negative internal pressure, condition 1.

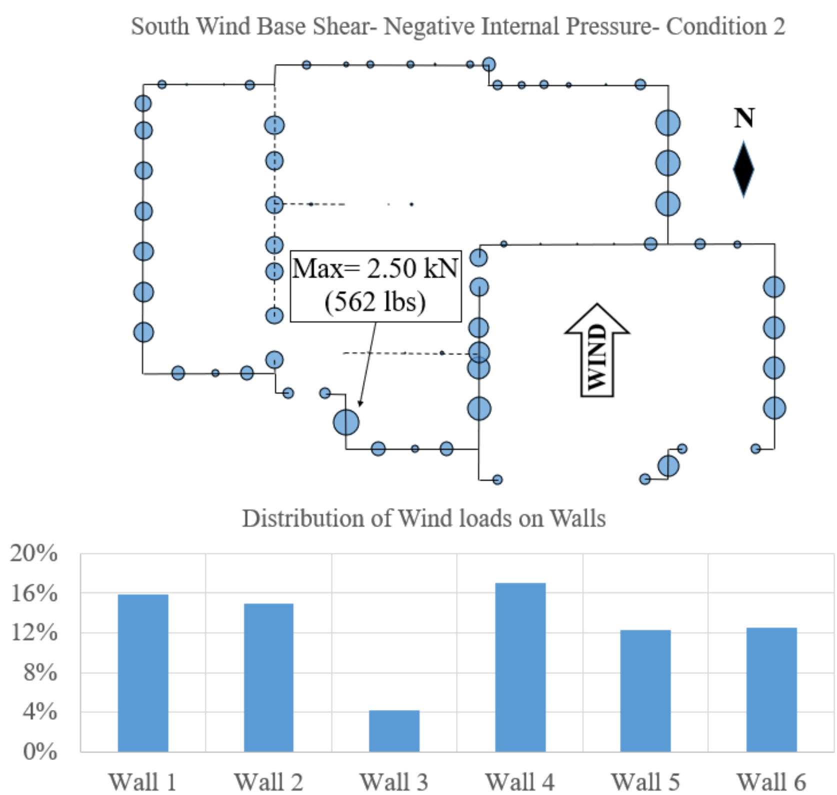


Figure H 24: Modified structure base shear. South 0.6W, negative internal pressure, condition 2.

APPENDIX I

ROOF SHELL STRESSES

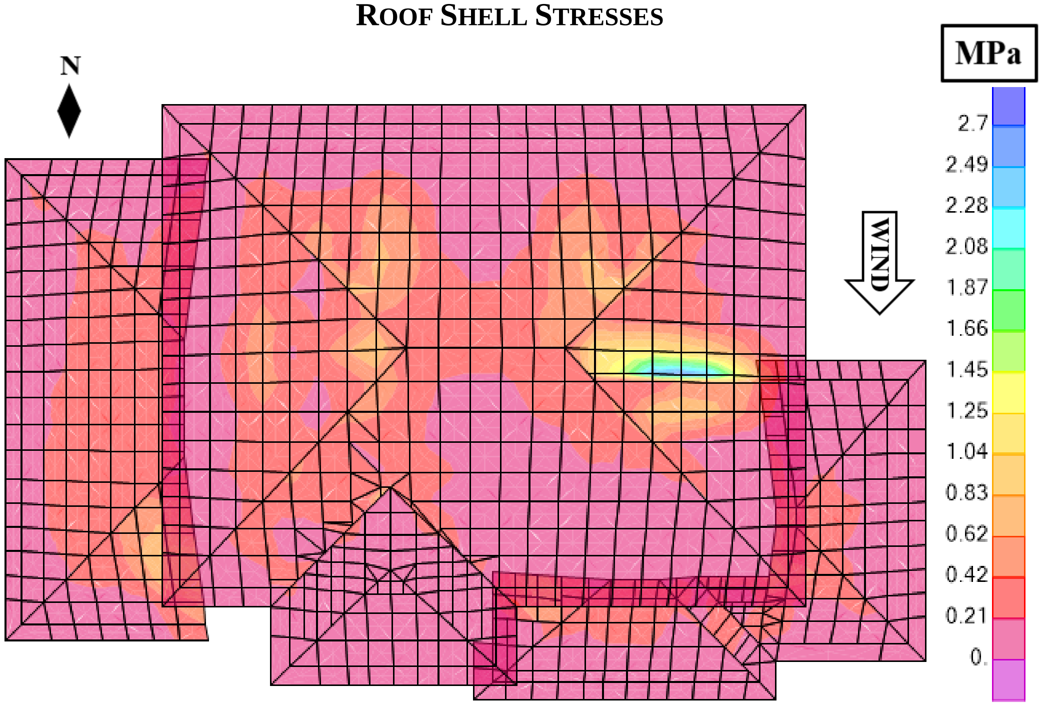


Figure I 1: 0.6W from North. Positive internal pressure, condition 1. Max Von Mises shell Stress.

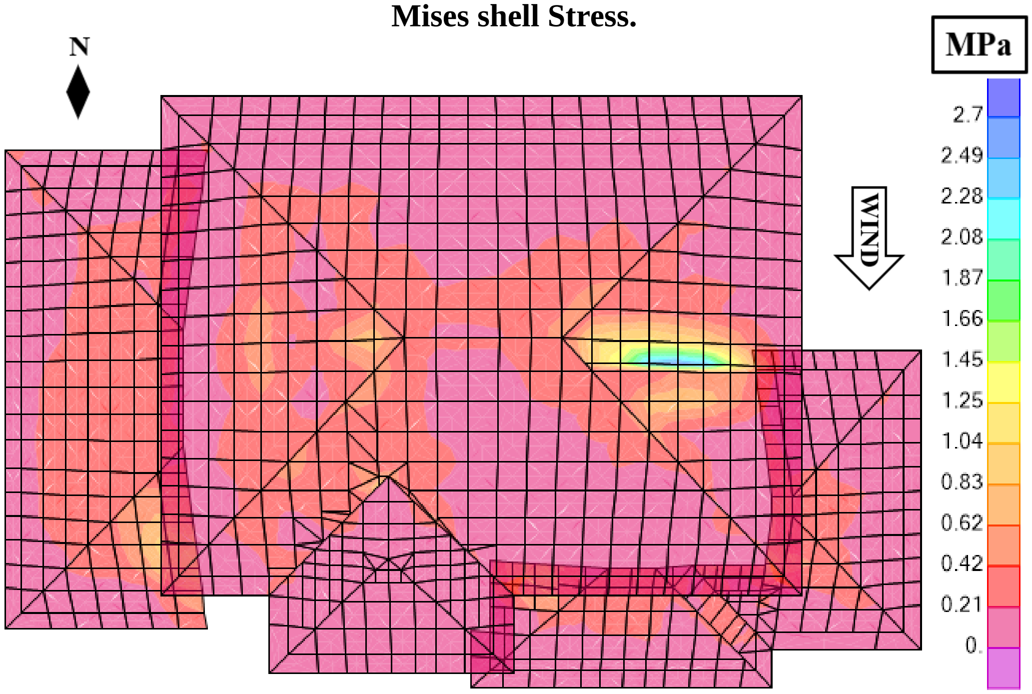


Figure I 2: 0.6W from North. Positive internal pressure, condition 2. Max Von Mises shell Stress.

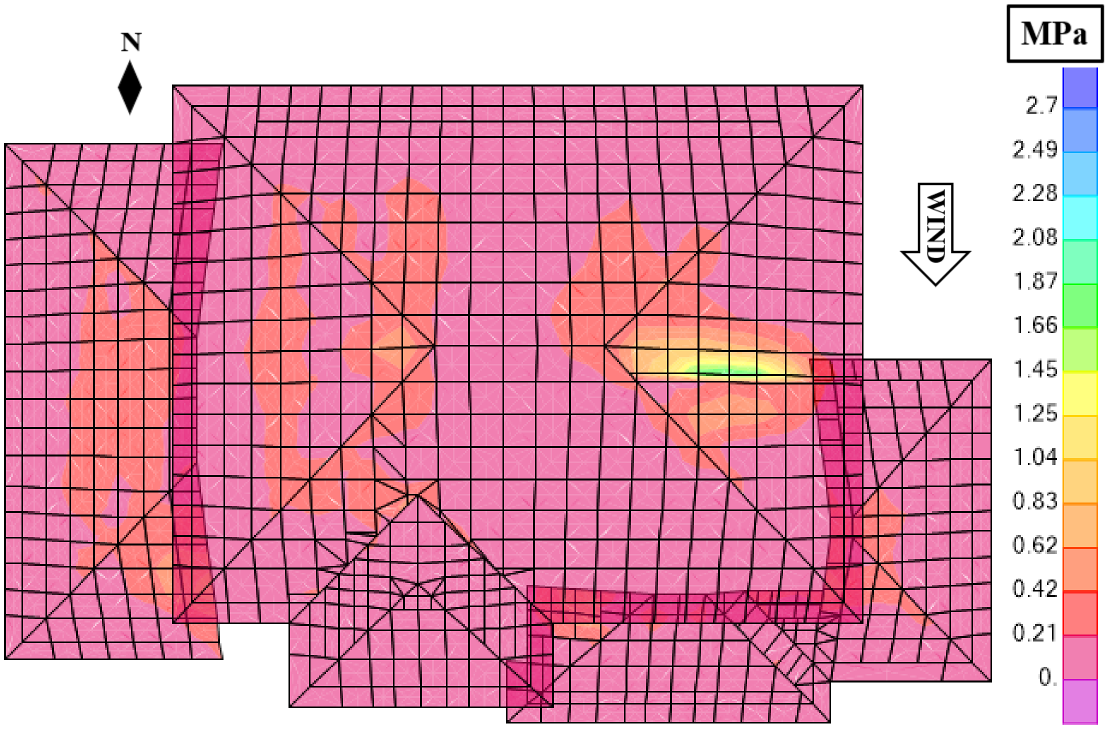


Figure I 3: 0.6W from North. Negative internal pressure, condition 1. Max Von Mises shell Stress.

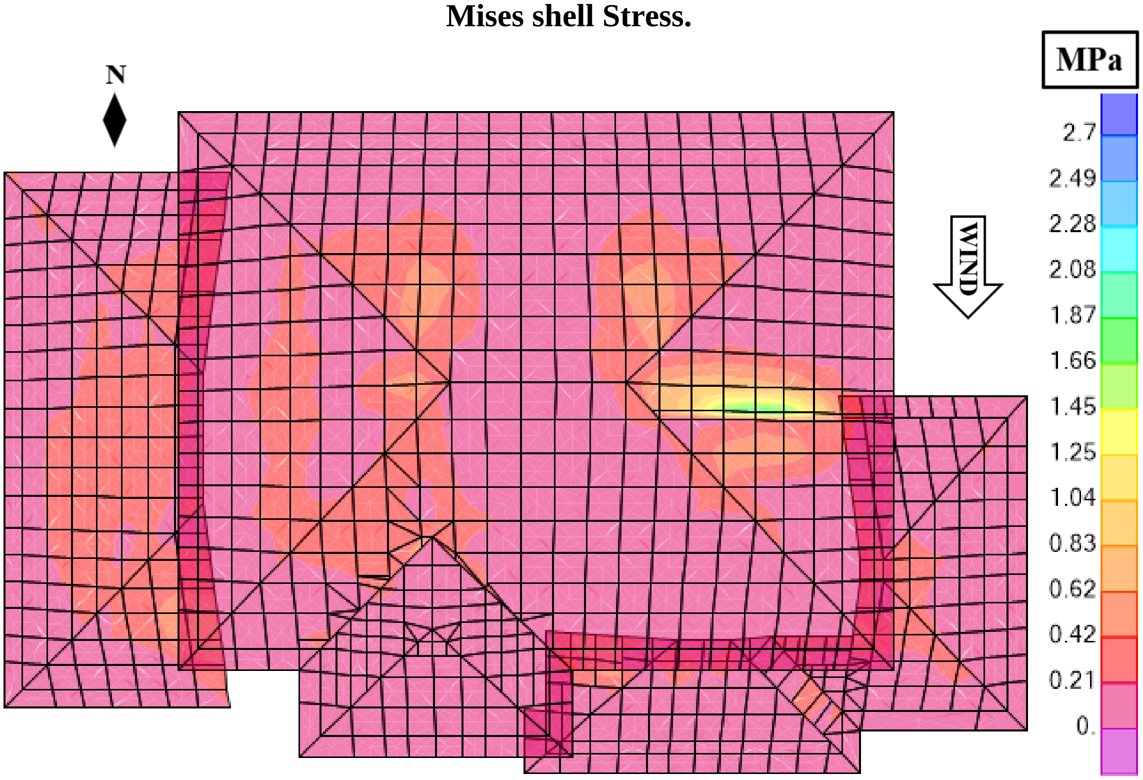


Figure I 4: 0.6W from North. Negative internal pressure, condition 2. Max Von Mises shell Stress.

South:

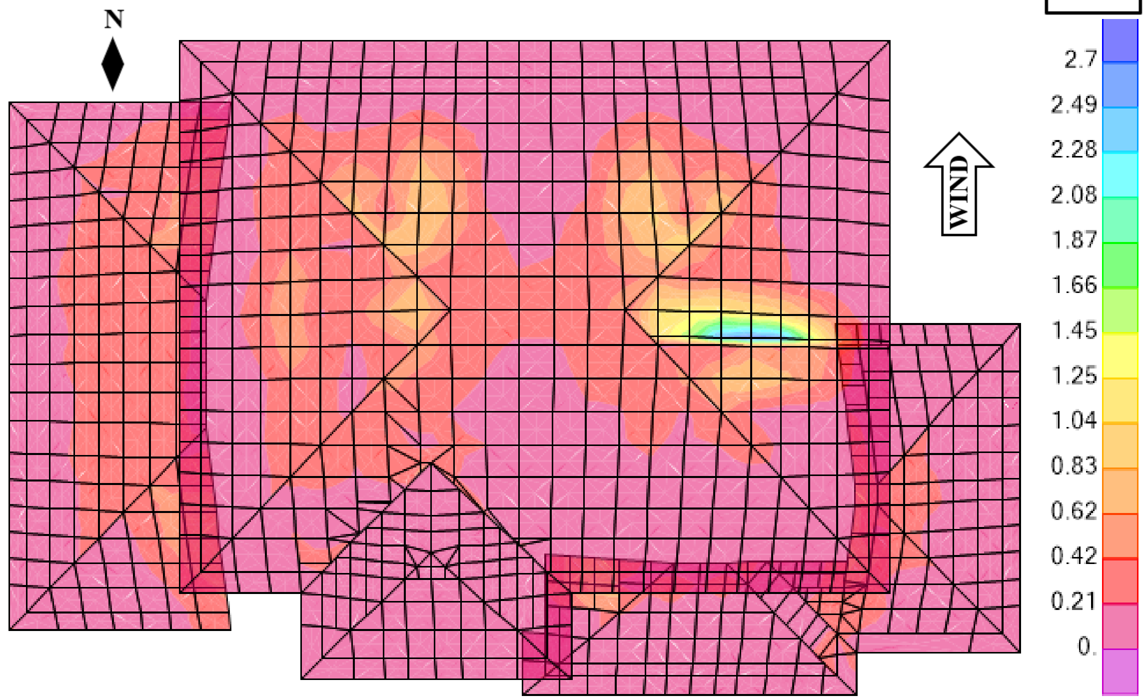


Figure I 5: 0.6W from South. Positive internal pressure, condition 1. Max Von Mises shell Stress.

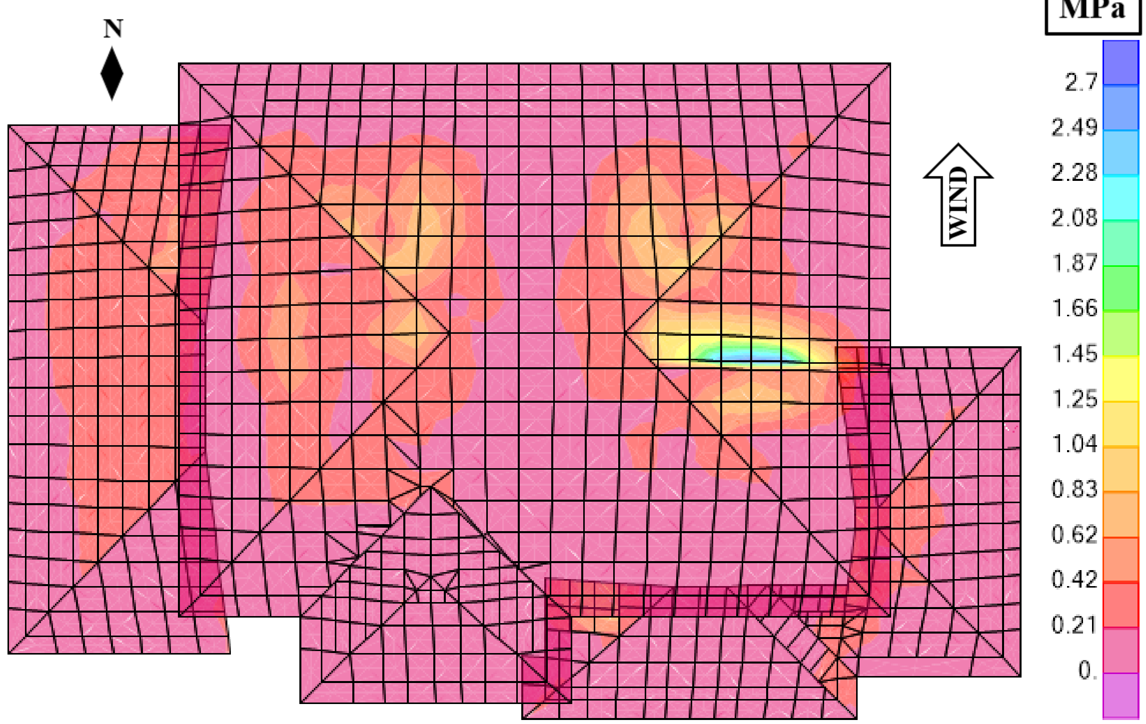


Figure I 6: 0.6W from South. Positive internal pressure, condition 2. Max Von Mises shell Stress.

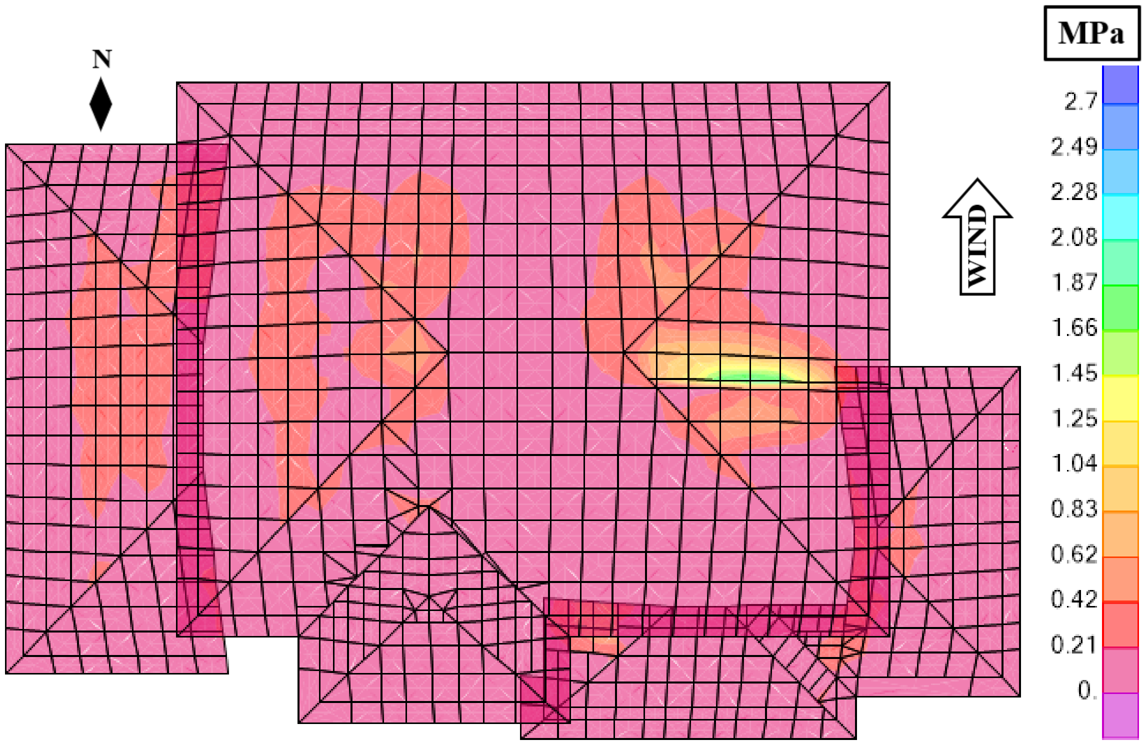


Figure I 7: 0. 6W from South. Negative internal pressure, condition 1. Max Von Mises shell Stress.

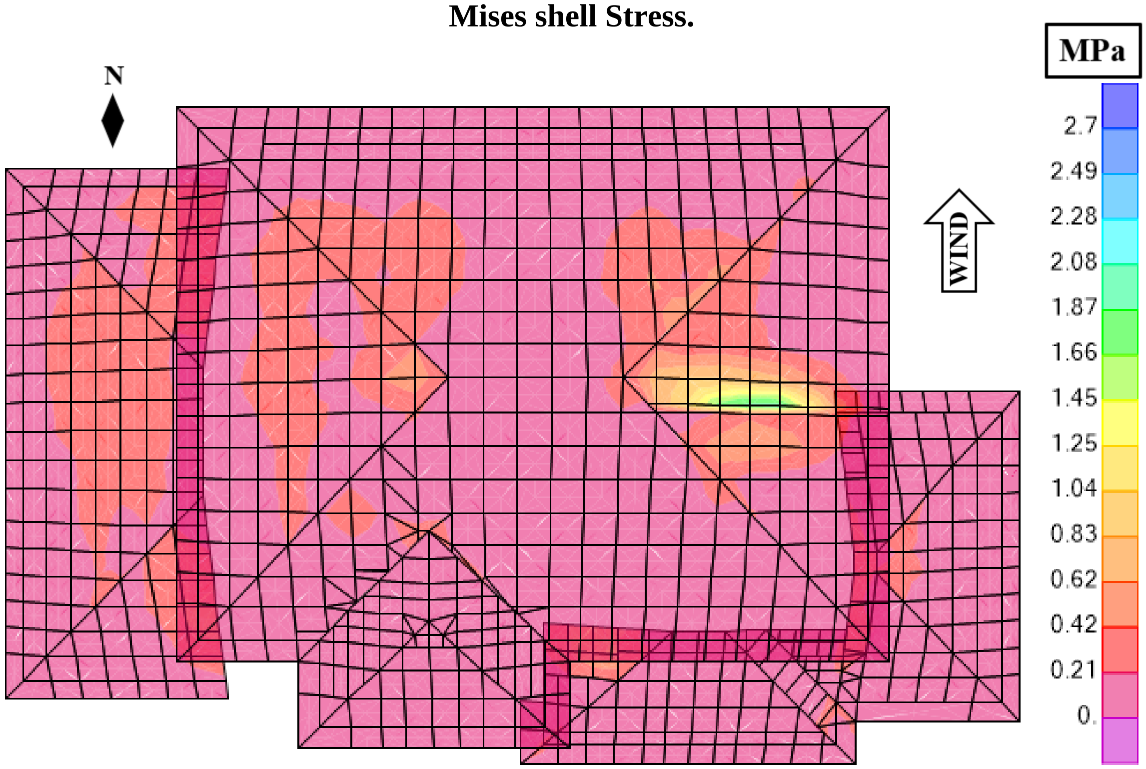


Figure I 8: 0.6W from South. Negative internal pressure, condition 2. Max Von Mises shell Stress.

East:

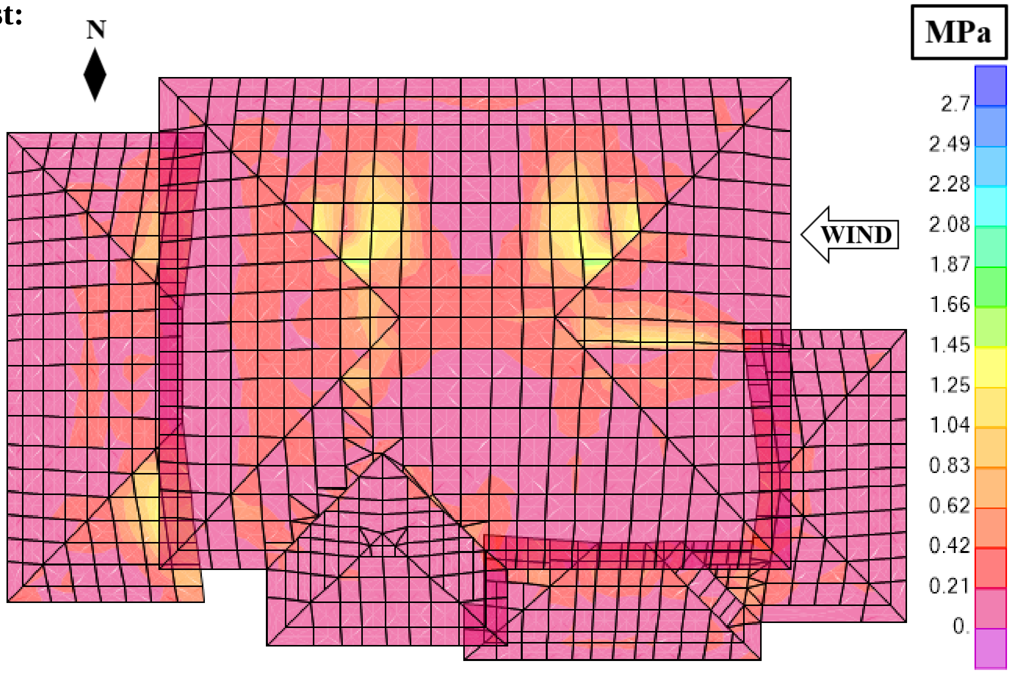


Figure I 9: 0.6W from East. Positive internal pressure, condition 1. Max Von Mises shell Stress.

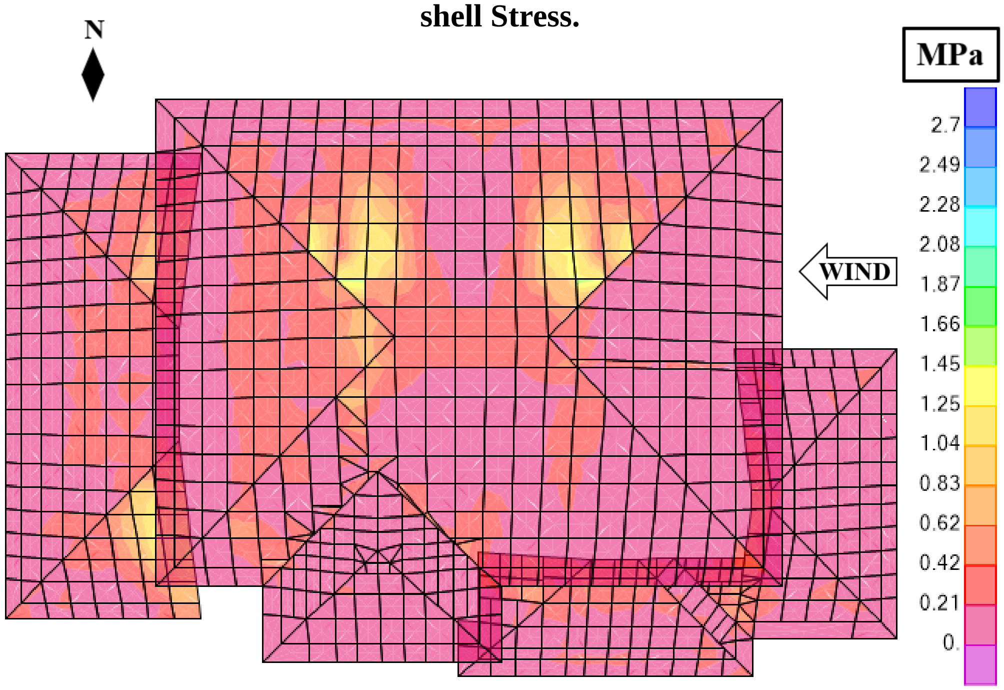


Figure I 10: 0.6W from East. Positive internal pressure, condition 2. Max Von Mises shell Stress.

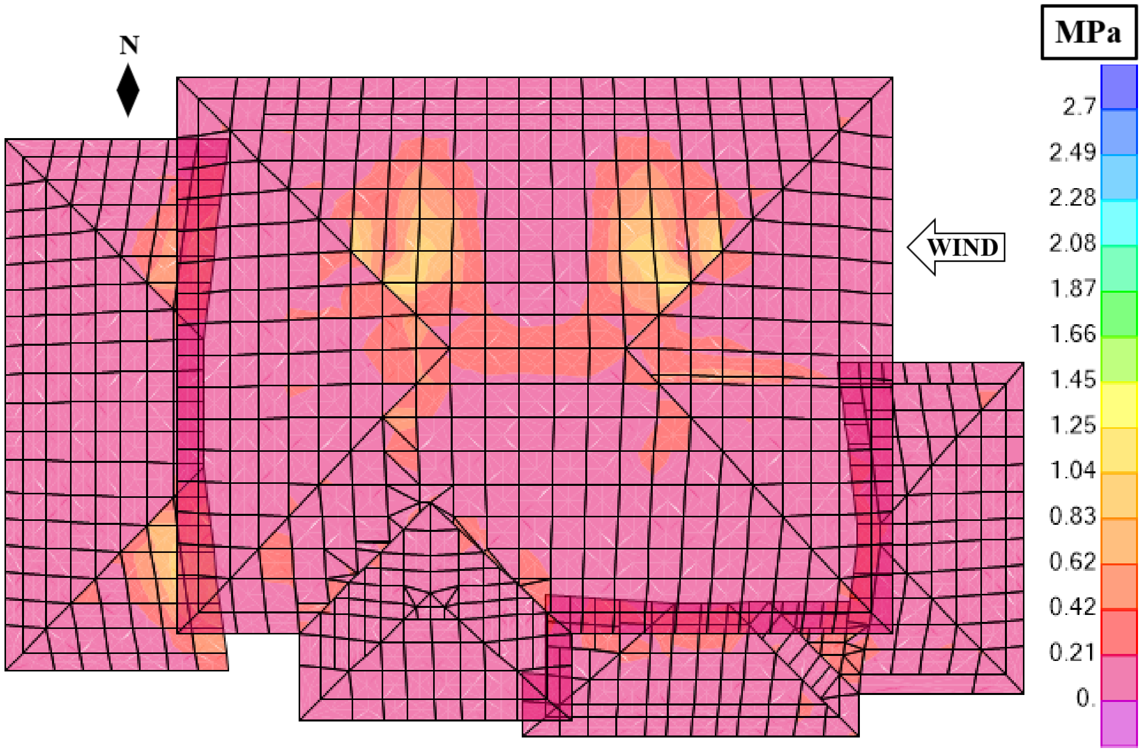


Figure I 11: 0.6W from East. Negative internal pressure, condition 1. Max Von Mises shell Stress.

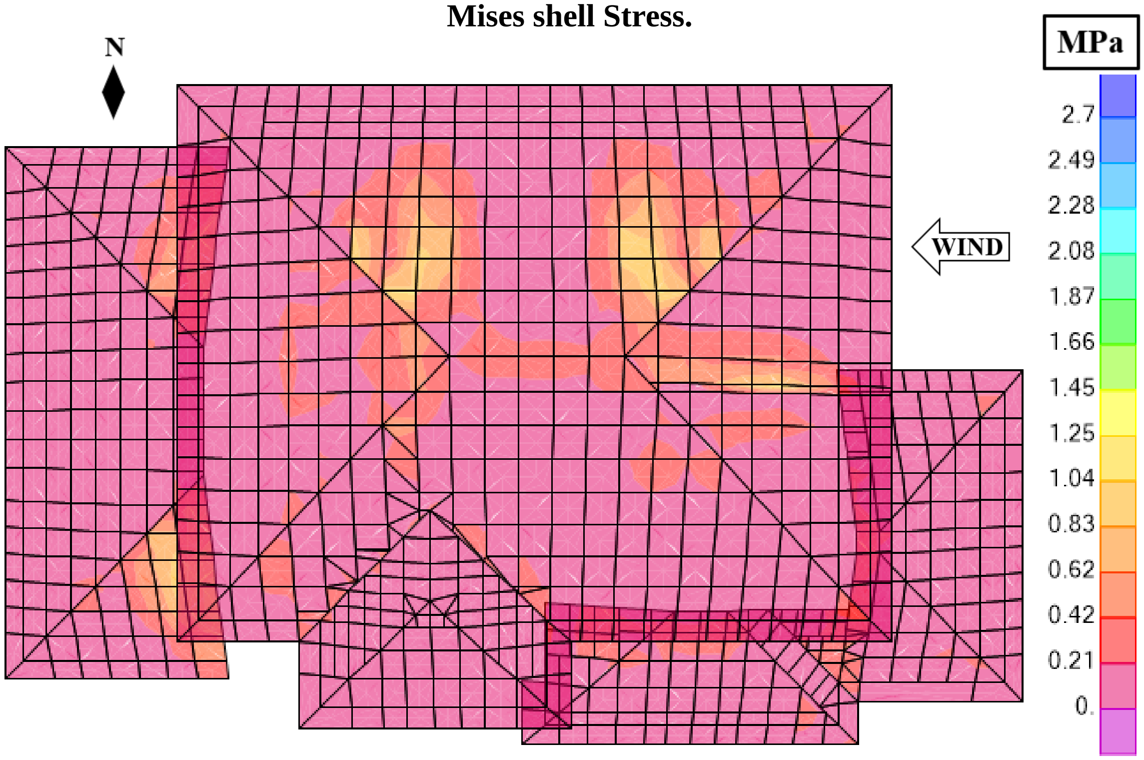


Figure I 12: 0.6W from East. Negative internal pressure, condition 2. Max Von Mises shell Stress.

West:

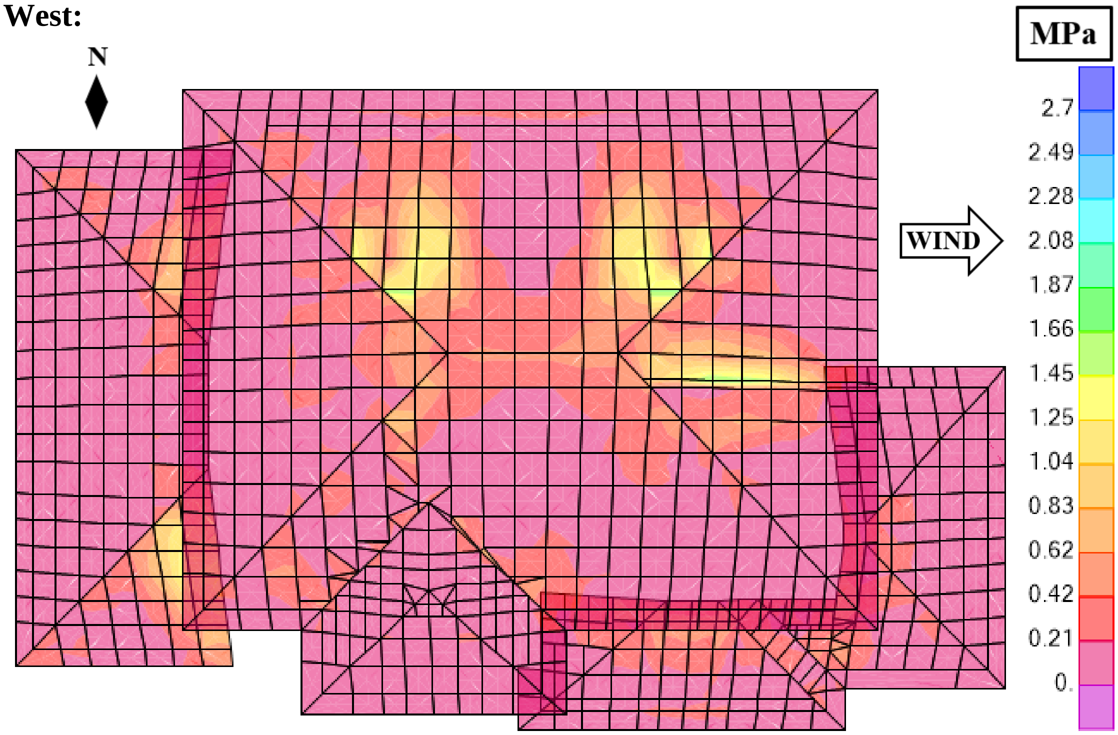


Figure I 13: 0.6W from West. Positive internal pressure, condition 1. Max Von Mises shell Stress.

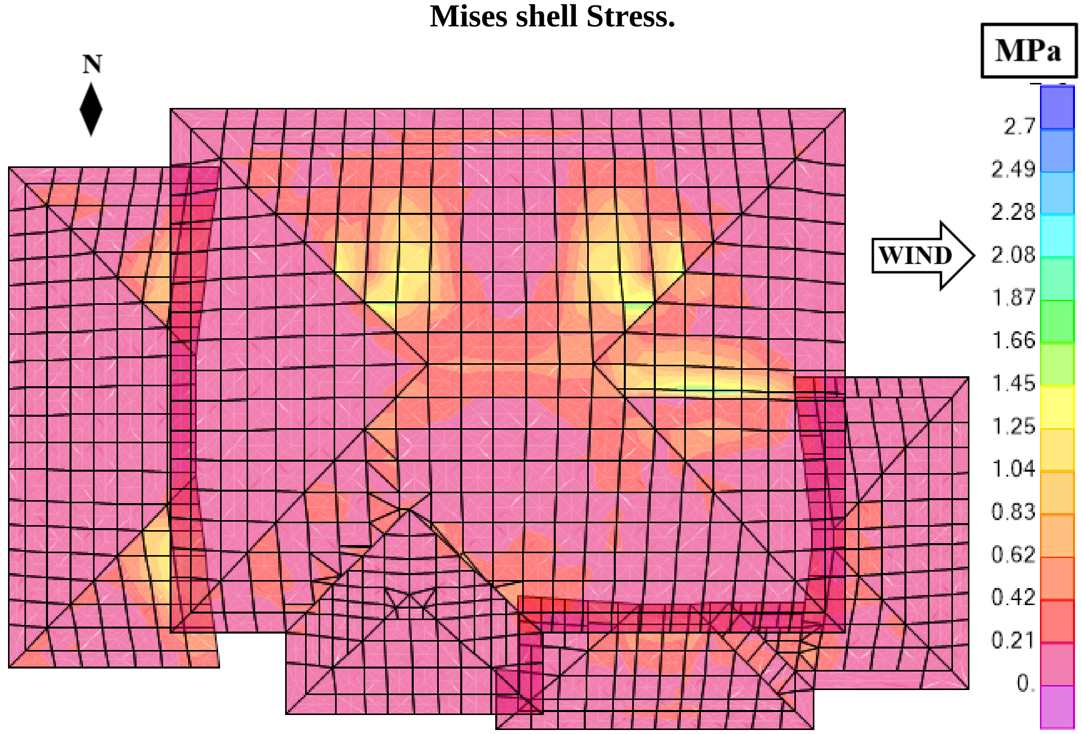


Figure I 14: 0.6W from West. Positive internal pressure, condition 2. Max Von Mises shell Stress.

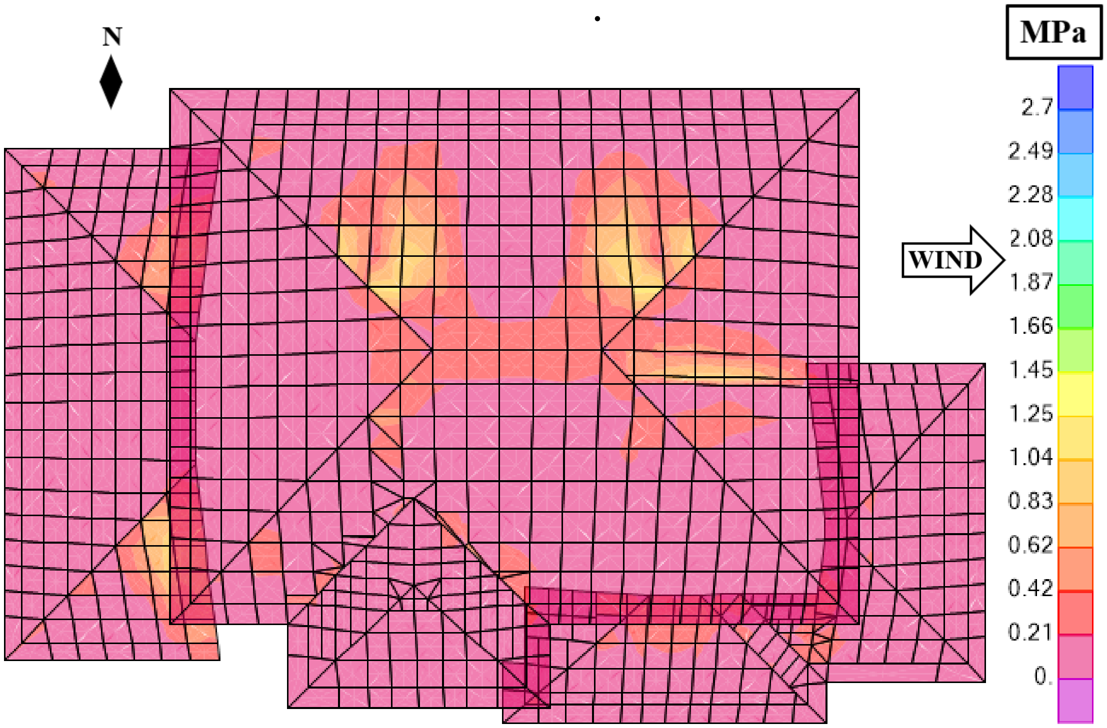


Figure I 15: 0.6W from West. Negative internal pressure, condition 1. Max Von Mises shell Stress.

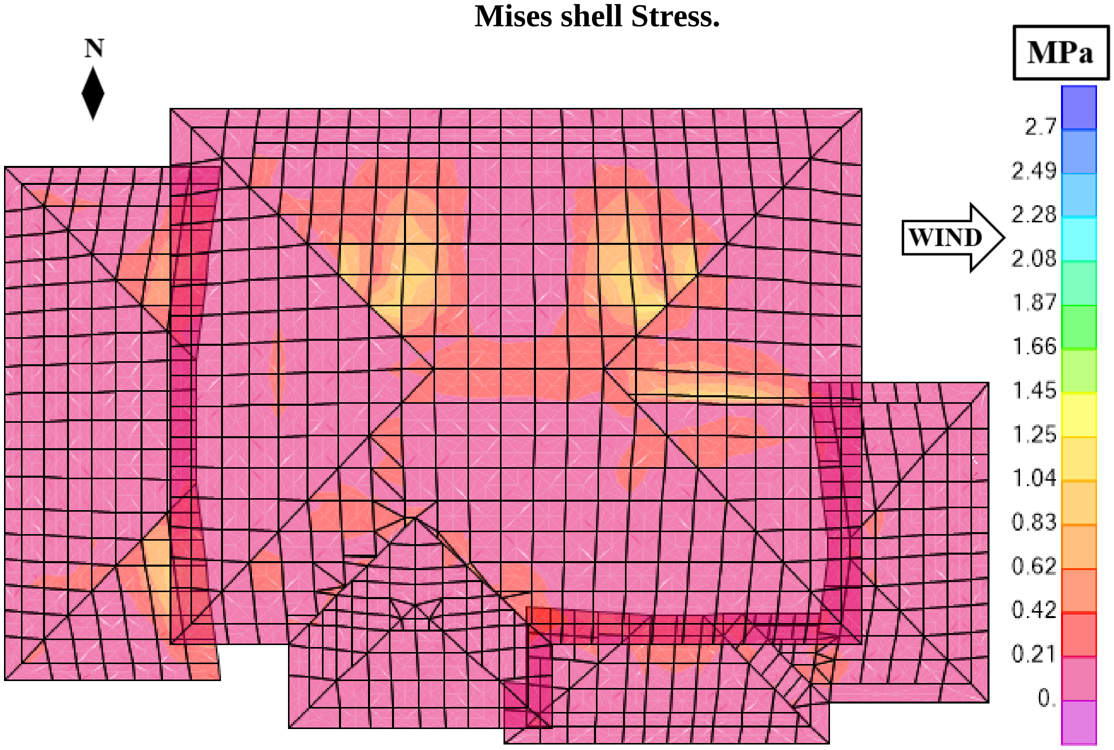


Figure I 16: 0.6W from West. Negative internal pressure, condition 2. Max Von Mises shell Stress.