VU VILLA STRUCTURAL DESIGN AND REHABILITATION

by

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ABSTRACT

The Vu Villa is a bar and pizzeria which has operated out of the first floor of the Campana building in Butte, Montana since 1967. Construction of the Campana building started in 1894 and was completed before the end of the 19th century. The Campana building is a two-story building which primarily consists of exterior brick masonry walls and wood framing on the interior. Within the last several decades significant sag has been identified in the second-floor of the building as well as an adjacent staircase. In addition, renovation activities have been discussed for the upstairs which would involve replacing the existing upstairs apartments with an open-space floor plan which could be utilized as a conference area.

The Vu Villa Structural Design and Rehabilitation project presents the proposed preliminary design to address the existing sag, and accommodate renovations associated with a change-in-use of the second-floor of the Vu Villa. A new structural system was designed as part of the project which would remedy the existing structural problems while accommodating the renovation that has been proposed. Due to significant expenses associated with renovation work, the design attempts to minimize the number of structural members requiring replacement by evaluating existing members for structural adequacy. Members specifically identified for replacement included the roof beams, second-floor columns, second-floor beams, and first-floor columns. Total costs anticipated to complete the work were approximately $620,000.

The Vu Villa Structural Design and Rehabilitation report discusses the design objectives and assumptions, and anticipated loading conditions, member design calculations, and structural software modeling results that demonstrate the functionality of the new structural system. In addition, it discusses several additional safety and design considerations warranting evaluation, if the project were to be undertaken. The design report appendices present the architectural and engineering drawings, technical specifications, calculations, estimated costs, and background information related to the project.
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1.0 INTRODUCTION/BACKGROUND

1.1. History of the Vu Villa and Campana Building

The Vu Villa, located at 521 West Park Street in Butte, Montana, is known to locals as one of the iconic bars and pizzerias in the area. The Vu Villa operates out of the Campana Building, which dates back more than a century. The Campana Building like many of the buildings in the historical district of uptown Butte has a long and storied history. Based on literature from the time, Rocco Campana constructed the building that bears his namesake in 1894. Upon the building’s completion, its first business, the Campana Grocery Store, opened around the same time. After Rocco’s passing in 1898, the ownership of the store was unclear, though it appears to have stayed in his family (Ogrin, 2000).

In 1939, records show that Rocco’s son, Rocco Campana Jr. opened the Parkway bar which operated out of the same building. The Parkway, which was described as a ‘wonderful neighborhood bar’, continued operation until a man by the name of Pete Vucurovich purchased it in 1967. At that point, the bar’s name changed to the Vu Villa (the House of Vucurovich). Pete maintained ownership of the Vu Villa until it was purchased by one of his bartenders, Charlie Delaney. Delaney continued the tradition of operating the Vu Villa Bar out of the Campana Building and proceeded to expand the enterprise to include a pizza parlor (Ogrin, 2000). Since then, the bar has changed owners’ multiple times, but has maintained its namesake and local popularity.

1.2. Existing Building

The Vu Villa currently functions as both a pizza parlor which operates out of the east side of the first floor, and a bar on the west side. The second story is setup as adjacent apartments but have not been inhabited for some time. The Vu Villa is separated into four ‘bays’; an east and west bay on the first floor, and replica bays on the second floor. The west bays’ are approximately 29-feet (ft) by 61-ft, 7-inches (in). The east bays’ are 25-ft by the same, 61-ft 7-in (Beaudette Consulting Engineers, 2013).

The existing structural system for the Vu Villa consists of two-by-six wood roof joists that are oriented east-west and appear to run the length of the roof. These joists provide the support for the sheathing and roofing materials. The existing wood-framed walls which connect the roof to the second-floor act as the main support of the joists and roof system.

At the second-floor level the structural system consists primarily of the sheathing and two-by-twelve wood joists. The second-floor joists, similar to the roof joists, are oriented east-west and are supported by several long span girders, as well as an interior brick masonry wall. Two columns and a wood-framed load-bearing wall which runs along the staircase provide the primary interior support of the west bay second-floor. Th east bay second-floor is mostly supported by the interior load-bearing walls that separate the dining area from the kitchen.
The first-floor connects the rest of the structural system to the Vu Villa’s foundations. The foundations for the structure are a slab-on-grade for the west bay and a rubble-stone foundation with an undeveloped basement for the east (Beaudette Consulting Engineers, 2013). The foundations support five original masonry brick walls, commonly referred to in this report as unreinforced masonry (URM) walls. Four of the URM walls are located along the north, south, east, and west boundaries of the building. The fifth URM wall runs through the approximate center of the building, separating the two bays, and is adjoined to the URM walls on the north and south side of the building. Four of the five walls consist entirely of URM brick while the southern-most wall (the store-front side) is a hybrid of both URM and wood components. With the exception of the two columns and two walls mentioned above, these five walls concentrate and channel all structural forces to the foundations.

Both first-floor bays are primarily an open-space plan with limited framing. In the east bay is an open area with extensive seating for people dining on the pizza side. Approximately 27-ft from the east bay entrance is a wood-framed load-bearing wall that separates the existing dining area from the kitchen. No other apparent load-bearing components exist within the east bay. The west bay, similar to the east, has an open-floor plan that extends to what appears to be a ‘floating’ wall or a wall not carrying any of the structures load. This apparent floating-wall lies approximately 41-ft from the southern wall. The two columns support the primary girder which runs north-south in the west bay.

The second-floor consists entirely of wood-framed walls for both the east and west bays. It is assumed that a variety of the walls within the second-floor framing system, currently transfer the forces from the loading of the joists and roof system.

For the existing layout of the building, see the Existing Site Drawings in Appendix A.

### 1.2.1. Structural Deficiencies

As one would expect with a 125-year old building, the Vu Villa has started to demonstrate minor structural defects. According to Montana Cadastral (a Montana State Library (MSL) provided, property recordkeeping service) the Vu Villa’s first-floor remains in ‘normal’ physical condition. The second floor of the Vu Villa, however, is noted as being in ‘poor’ physical condition (Montana State Library, 2019).

A structural investigation and follow up inspection of the building was conducted in 2013, by the structural engineering company Beaudette Consulting Engineers, Inc. (BCE) (Beaudette Consulting Engineers, 2013). The investigation and subsequent inspection concluded that there was significant sag of the second-story floor-system. The deflection appears primarily in the west bay of the building, which is home to the bar side of the structure. BCE identified the likely cause of the deflection (another term for sag) as the removal of a load-bearing wall that was previously located north of the stairs (Beaudette Consulting Engineers, 2013).
To-date, no further action has been taken to address the structural defects of the building. In addition to the noted sag, it appears as though the existing beams and framing system are the same ones originally erected when the Vu Villa was first constructed in 1893 and 1894.

1.2.2. Existing Occupancy and Usage

The Campana building currently functions as two occupancy types. On the first-floor, both the bar and pizza sides of the Vu Villa can be characterized as assembly areas which according to the American Society of Civil Engineers 7-02: Minimum Design Loads for Buildings and Other Structures (ASCE 7-02) “Table 1-1” means it is considered a class III building (American Society of Civil Engineers, 2003). A class III building according to ASCE 7-02 is “a building that represents a substantial hazard to human life in the event of failure” (American Society of Civil Engineers, 2003).

The second-floor of the Vu Villa, in the past has functioned as two adjacent apartments. As was mentioned above the structural system still reflects the second-floor’s past usage. Unlike assembly areas, apartments represent a lesser hazard to human life and therefore fall into ASCE 7-02’s building class II.

1.3. Proposed Building

The goal of the Vu Villa Design and Rehabilitation project is to render a final building product that provides structural stability and accommodates a change in the usage of the second-floor. The proposed changes to the Vu Villa consist of both addressing the existing structural deficiencies of the second-floor and renovating the upstairs into an open-space floor plan that could be used as a conference or assembly area. Both desired outcomes from the project are discussed in detail below.

1.3.1. Updating Components to Address Structural Issues

The first objective of the Vu Villa Design and Rehabilitation project is to remedy the existing sag previously identified for the second-floor. Based on previous information and several site evaluations, it is believed that the existing second-floor beams are responsible for the noted deflection. Therefore, the proposed solution is to replace both the second-floor beams and corresponding first-floor columns with replacement members.

While replacement of the beams and columns is required, it is an additional goal of the design to minimize the impact to other structural members, such as joists and sheathing. The purpose for minimizing additional removal and impacts is primarily based on the owner’s desire to reduce the cost of such a project and allow for continued operation of the bar and restaurant.
1.3.2. Occupancy & Usage Change

The second goal of the Vu Villa Design and Rehabilitation project is to alter the usage and layout of the second-floor from apartments to an assembly area which can be used for conferences, events, and a secondary location for patrons of the Vu Villa. In order to implement such a renovation, a new structural support system will be required and demolition of the existing apartment framing will need to be completed. To additionally address safety and functionality concerns, the change in occupancy will require ensuring that the updates comply with both the International Building Code (IBC) and International Existing Building Code (IEBC). This will require verifying that existing members are capable of handling the increased building capacity and that such members meet the permitted loading requirements of a class III building.

2.0 DESIGN CRITERIA

The following section sets forth the design objectives, assumptions, and constraints of Vu Villa Structural Design and Rehabilitation project and describes the proposed design.

2.1. Design Objectives

Three primary design objectives are addressed as part of the Vu Villa Structural Design and Rehabilitation project, as discussed in Section 1.0 above. These criteria establish the basis for the design and denote the specific end goals. The design objectives are:

- Design a system which remedies the existing sag of the second-floor;
- Design a system that replaces the existing roof- and second-floor structural members and accommodates the change in usage of the second-floor from apartments to an assembly area; and
- Preserve all existing structural members that do not require replacement.

The structural system, specified in Section 2.4 of this report, achieves all objectives.

2.2. Design Assumptions

The following assumptions were made related to the Vu Villa Structural Design and Rehabilitation project:

- All initial surveys and investigations were accurate and correct;
- The existing foundations are adequate to support the new design and assumed loading conditions; and
- The existing first-floor members (i.e. joists, sheathing, and girders) are capable of transferring the new design and assumed loading conditions to the foundation.
2.3. Design Constraints

The following constraints were identified for the Vu Villa Structural Design and Rehabilitation project. The design shall:

- Minimize impact to existing structural members;
- Optimize design to reduce costs; and
- Provide a design which can be implemented while keeping the Vu Villa operational.

2.4. Proposed Design

The proposed design to achieve the objectives discussed above involves replacing the first-floor columns and second-floor beams of the existing system with new members sized based on new loading conditions. The design changed the location and span of beams and columns slightly in order to optimize the members size and ensure a more natural load path for the second-floor.

Four 1st Floor Columns will be installed, two in each bay with the exterior and interior URM walls providing the other vertical supports for the first-floor. The columns will be centered on the midpoint of each bay and will be equally spaced along the depth of the bays. Similarly, the 2nd Floor Beams will be centered in the middle of each bay with three beams collectively spanning the depth of both bays; for a total of six 2nd Floor Beams. The splice condition for the beams will occur directly centered over the 1st Floor Columns.

The support system for the roof and second-floor will mirror that of the second- and first-floor. The 2nd Floor Columns will be placed centered on the splice condition of the 2nd Floor Beams. However, unlike the first-floor support system, no full length URM wall exists in the center of the second story. Therefore, columns of similar orientation and spacing to the other columns on the second-floor will be placed directly over the interior URM wall on the first-floor. Three Roof Beams shall span north-south, similar to the 2nd Floor Beams, with each terminating either centered on the 2nd Floor Columns or at the exterior URM walls.

Bolted, Nailed, and Simpson Strong-Tie® hardware connections will be utilized to transfer vertical and lateral loads from the Roof Beams, through the primarily load path, down to the 1st Floor Columns and finally to the foundations. In addition, the existing roof- and second-floor joists will be fastened to the replacement beams by the method of toe-nailing each joist with a specified number and size of nails. For further information on the layout, details, and specifications of the proposed structural system refer to the Design Drawings in Appendix A and Technical Specifications in Appendix B. For the methods and logic used in determining the loading conditions and design of the Vu Villa, refer to Section 3.0.

3.0 METHODS OF DESIGN

In structural design, there are two common philosophies for comparing structural member’s strength to applied stresses, Allowable Stress Design (ASD), and Load and Resistance Factor Design (LRFD) (Quimby, 2008). In general, they both accomplish similar goals of ensuring that
a designed structural member can support its applied load. However, each approach takes
dissimilar steps to reaching the solution. ASD consists of comparing actual stresses to an
allowable stress which is determined from a member’s section properties, strength, and a safety
factor (Quimby, 2008). Meanwhile, LRFD compares the member’s actual strength to a factored-
stress based on loading conditions (Quimby, 2008). The Vu Villa Structural Design and
Rehabilitation Project utilized LRFD as the primary design methodology.
Critical steps in completing an LRFD-based design for the Vu Villa Structural Design and
Rehabilitation project included identifying the anticipated loading conditions, optimizing
structural member’s section properties and material, specifying the connections to effectively
transfer loads from the roof members to the foundation, and compiling an engineer’s estimate of
the cost of the project. The manner in which each step was performed is discussed in greater
detail in the remainder of Section 3.0.

3.1. Loading Conditions

The primary resources used in evaluating the loading conditions for the Vu Villa Structural
Design and Rehabilitation project included both provisions from the 2012 IBC and from ASCE 7-
02. In addition, loads for specific members were collected from the Design of Wood Structures
ASD/LRFD, Rosboro X-Beam Technical Guide, and the website Engineering Toolbox and their
webpage “Density of Various Wood Species”. Specific references and detailed calculations for
determining the loading on the Vu Villa are noted in the Loading Calculations in Appendix C.

3.1.1. Dead Loads

Dead loads are defined by ASCE 7-02 as “the weight of all materials of construction incorporated
into the building” (American Society of Civil Engineers, 2003). For estimating the anticipated
dead loads, a conservative assumption was made for all likely components of the structure that
may bear on each replacement member. For example, it was assumed that components the Roof
Beam would need to support, included:

- Built-Up-Roofing (5-Ply with Gravel);
- Roof Sheathing (3/4-inch Plywood);
- Loose Insulation;
- 2x6 Roof Joists at 16-inch on-center (o.c.) spacing;
- 1” Gypsum Drywall Ceiling; and
- Self-Weight of the Roof Beam.

Each of the components were compiled in a spreadsheet and approximate weights of each were
determined from several of the sources listed above (International Code Council, Inc., 2011;
Rosboro, 2019; The Engineering Toolbox, 2004). The summation of each of the component’s
weights corresponded with the design dead load for that member. This process was completed
individually for the Roof Beams, 2nd Floor Columns, 2nd Floor Beams, and 1st Floor Columns. It
was also assumed that the weight each previous member had to support, would also bear on the
following member in the load path.
It is also important to note, in instances where a component was known (joists for example) the dead load was assumed to be that material. In situations where a component was unknown, the worst-case was assumed for that component (i.e. roofing material). The component may not always agree with what is actually presently in place at the Vu Villa but makes conservative assumptions as to the type of material.

3.1.2. Live Loads

Live loads according to *ASCE 7-02* are loads produced by using or occupying a building or structure (American Society of Civil Engineers, 2003). Determination of live loads for each member was done with a similar approach to the dead loads. Each member was specifically evaluated based on equations and tables provided in both *ASCE 7-02* and the *2012 IBC* for roof and floor live loads. Logically, the roof live load applied to the Roof Beams and 2nd Floor Columns while the floor live loads applied to the 1st Floor Columns and 2nd Floor Beams. “Equation 4-1” of *ASCE 7-02* provides the standard live load reduction calculation for determining the floor live loads. Similarly, *ASCE 7-02* provides “Equation 4-2” for determining roof live loads (American Society of Civil Engineers, 2003).

3.1.3. Snow Loads

For determining the snow load applicable to the Vu Villa Structural Design and Rehabilitation project, the calculations follow the progression of *ASCE 7-02* “Section 7.3” (American Society of Civil Engineers, 2003). An online interactive map resource from Medeek Design was used for determining the ground snow load for Silver Bow County (Medeek Design, 2017). The returned value was compared with the *2012 IBC* “Figure 1608.2” (International Code Council, Inc., 2011). As the Medeek Design snow load was a more conservative value, it was the ground snow load assumed for the project. In addition, the factors specified in *ASCE 7-02* “Equation 7-1” were determined from *ASCE 7-02* “Table 7-2”, “Table 7-3”, and “Section 4.9.1” based on the building’s roof pitch, exposure, terrain roughness, insulation level, and risk category.

3.1.4. Wind Loads

Wind loads were evaluated using the simplified procedure described in *ASCE 7-02* “Section 6.4”. The simplified procedure was used as the Vu Villa met all the building and site requirements described in *ASCE 7-02* “Section 6.4.1.1” (American Society of Civil Engineers, 2003). The basic wind speed was first determined by using *ASCE 7-02* “Figure 6-1”, however, the wind speed from the figure was significantly lower than one would expect. Due to this, the *2012 IBC*’s “Figure 1609B” was consulted and inversely it seemed to provide an extremely conservative value. Due to the discrepancy between codes, local maximum wind data between 1980 and 2019 was acquired and evaluated from Weather Underground and a more representative wind speed was selected (Weather Underground, 2019).
For the parameters specified in ASCE 7-02 “Equation 6-1”, the net design wind pressures and adjustment factor for building height and exposure were determined from ASCE 7-02 “Figure 6-2”; while the importance factor was selected from ASCE 7-02 “Table 6-1” (American Society of Civil Engineers, 2003).

With all known parameters, the design wind pressure was then calculated. The maximum design wind pressure for each story was then translated into a uniform load acting at the roof- and second-floor diaphragms by multiplying by the tributary height of that diaphragm.

### 3.1.5. Seismic Loads

Seismic loads, which are those loads on a building induced by shaking from an earthquake, were determined for the Vu Villa Structural Design and Rehabilitation project following the design progression identified in ASCE 7-02 “Section 9.0”. The first step in developing the loads was determining the maximum ground motion accelerations and response spectra. This was done with the use of a United States Geological Survey (USGS) interactive map located on the USGS website (United States Geological Survey, 2018). Once the maximum spectral response parameters were established, the adjusted maximum and design spectral response accelerations were tabulated using ASCE 7-02 “Equations 9.4.1.2.4” and “Equation 9.4.1.2.5”, respectively. From there, the design response spectrum curve was developed in accordance with ASCE 7-02 “Section 9.4.1.2.6” (American Society of Civil Engineers, 2003). This provided the needed design spectral response to complete the rest of the calculations required for determining the seismic loads.

The vertical seismic force was then determined using the equation denoted in ASCE 7-02 “Section 9.5.2.7”. The lateral design forces for the Vu Villa were tabulated using ASCE 7-02 “Section 9.5.2.6.4.4” for the roof and second-floor diaphragms, and ASCE 7-02 “Section 9.5.5.4” for the vertical elements (i.e. shear walls) (American Society of Civil Engineers, 2003). The resulting forces were concentrated loads in pounds or kips and then had to be divided by a total length of the respective diaphragms to determine the uniform load acting across each. This simplified the comparison of wind and seismic loads in the LRFD load combinations discussed in the Section 3.1.6.

### 3.1.6. LRFD Load Combination Equations

With all applicable dead, live, snow, wind, and seismic loads known, the next step was to factor the loads, or in other words, apply safety factors to them. This is accomplished through the use of the LRFD load combination equations. Each equation considers varying permutations of the loads specified above and each individual load is multiplied by different factors. Once each of the loads and the associated factor have been multiplied, the summation is taken of all loads associated with that equation. The greatest summed load from each of the seven equation is selected as the ‘ultimate load’ and denotes the value to be used in the remainder of the design. Figure 1 below shows the seven specified equations related to LRFD.
Figure 1: LRFD Load Combination Equations (Gupta, 2014)

The equations were applied to both vertical and lateral loads separately but simultaneously to determine the ultimate loads in each plane. An additional step was required due to the fact the design being implemented consisted primarily of wood members. For wood design, the ultimate loads determined from the LRFD load combination equations must also be divided by the time effect factor ($\lambda$) as it is specified in “Appendix N” of the National Design Specification (NDS) for Wood Construction (American Wood Council, 2016). The time effect factor varies for each of the seven load combinations from 0.6 to 1.0 depending on the equation and onsite conditions (American Wood Council, 2016). Dividing the equation by the time effect factor is strictly for selection of the ultimate load and does not provide the actual ultimate load. After the loads have been divided by the time effect factors, the largest remaining value indicates the equation from which the ultimate load results, however, the ultimate load that is used in further calculations is that value prior to dividing it by the time effect factor.

Ultimate loads determined from these equations were then applied in the design and verification of existing and proposed members through principles of engineering statics and mechanics. It was with these factored loads that the remainder of the design was completed. For all load combination calculations and spreadsheets refer to Appendix C of this report.

3.1.7. Chord and Drag Analysis

In order for the lateral loads determined in Section 3.1.6 to be further applied to members such as the existing diaphragms and shear walls, a chord and drag analysis was required to determine the shear and bending forces that the diaphragm is subject to. Writing for Structure Magazine, Kurt
Voigt analogized diaphragms to a ‘short, deep beam’ (Voigt, 2013). In essence, lateral loads acting at each diaphragm level behave as a uniform load and the diaphragm as a beam subject to shear and bending. In this case, the internal forces resisting shear within the diaphragm are referred to as ‘drag’ forces while bending is synonymous with ‘chord’ forces. Much like you would expect, in order to determine these forces, all one has to do is solve for the internal forces of the diaphragm ‘beam’. See Figure 2 below for demonstration on the behavior as diaphragms as a beam.

![Diagram of Diaphragm Beam Behavior](image)

**Figure 2:** Diaphragm Beam Behavior (Breyer, Cobeen, Fridley, & Pollock, 2015)

As described, the chord and drag analysis in Appendix C followed the same progression. Using engineering statics, the reactions were determined for both the roof and second-floor diaphragms. Once the reactions were known, material mechanics were used to determine the internal shear and maximum bending moment based on the applied lateral load. As one would expect, the maximum shear occurs at the boundaries of the diaphragm and shear walls, as the shear walls provide the primary resistance against the lateral loads. For the chord forces, the maximum moment occurs at the center of the diaphragm, the greatest distance from the shear wall where no resistance is provided.

With known internal forces, the unit shear value was determined by dividing the internal shear by the depth of the diaphragm it is acting over. This provided a force in pounds per lineal feet (plf) which, as discussed later, was used in determining the minimum requirements for the diaphragms. Similarly, the maximum moment can be divided by the depth of the diaphragm as well. When this is done, the resultant force is both the compression in the top of the diaphragm, as well as the tension in the bottom member of the diaphragm. The chord forces were later used to determine nailing requirements for diaphragm which is discussed in further detail in Section 3.3.1, below.
The same methodology used for determining the diaphragm unit shear was also used for assessing the existing capacity of the URM walls. By using the lateral load for shear walls that was based on the seismic load on vertical elements, the drag forces along the URM walls were also determined.

### 3.2. Replacement Member Design

Following the development of the Vu Villa’s loading conditions, the next step was designing the replacement members and their associated connections. This was done primarily from the guidelines of the 2015 NDS and the use of the *Simpson Strong-Tie*® website and product catalog. The design of members followed the basic load path which started at the Roof Beams and ended at the 1st Floor Columns. Structural members were designed first and the connections were specified once each member’s size and dimensions were known. Design methods for replacement members consisted of first using design spreadsheets, and hand-calculations supporting those spreadsheets to determine the proposed members size. Once sizes were known, a model of the proposed structural system was developed in AutoDesk’s Robot Structural Analysis (RSA), a structural analysis software. The loading conditions determined above were loaded into RSA and applied to the constructed model. Reports were run based on LRFD load combination equations and the report verified that the proposed members were adequate for specified loads. Once the replacement members capacities had been verified in RSA, the connections were designed for each using similar spreadsheets and hand-calculations. The following section explains the basic design procedure in sizing the members and specifying their connections. The Design Calculations in Appendix D provide all spreadsheets, hand-calculations, and RSA reports used in the Vu Villa Structural Design and Rehabilitation project.

#### 3.2.1. Roof Beam Design

As mentioned above, the first member designed was the Roof Beams. The proposed layout for the nine beams is as shown in Figure 3 below.
Conceptually, three different loading conditions were considered when designing the beams. The first was the loading on the member vertically from the dead, live, and snow loads on the top of beam. This will be the most common loading condition for the beams. In this plane, the beam was evaluated for its bending capacity, strength capacity, and deflection (a serviceability consideration). Figure 4 below demonstrates the gravity loading condition that was considered.
The second condition considered was loading along the beam perpendicular to its length. This was due to the loads acting in the transverse or east-west direction. This loading will most often occur during large wind or seismic events. For this condition, only bending and shear capacities were considered. Deflection was not of major concern as the member would not consistently be subject to the load and therefore serviceability was not a consideration. Figure 5 provides a representation of the beams loading in this case.

![Figure 5: Roof Beam Transverse Loading Condition](image)

The third and final loading condition was for loads acting in the longitudinal direction or north-south. Unlike the other conditions, loading in this direction sees the beam acting as what is referred to as a collector. The *Seismic Design Provisions for Wind and Seismic* (SDPWS) document produced by the American Wood Council (AWC) defines a collector as a diaphragm boundary member which is oriented parallel to the applied load and transfers the load to the vertical resisting elements (American Wood Council, 2014b). In a more basic sense, the beams, when loaded in the longitudinal direction, will act as a column resting on its side. Similar to the transverse loading condition, these forces will be likely due to wind and seismic events as well. For this loading condition, the primary capacities of the member which require consideration are the members tension and compression capacities. See Figure 6 below for a representation of the longitudinal loading condition for the Roof Beams.
For each of the load conditions, the associated reactions and ultimate internal stresses acting on the beam were determined using principles of engineer statics and mechanics. For the first two loading conditions, this consisted of calculating the reactions, internal shear, and internal moment that would be applied to a beam based on those conditions. For the third loading condition, the reactions and internal tensions and compression forces were determined. Since the applied loads had already been factored, the internal stresses that were calculated were the ultimate loads that would occur on the beam. Based on engineering judgement and the unlikely event of ultimate loads from each condition occurring on the member at the same time, each loading condition was considered individually when designing the beam.

Once the loading conditions were understood and established for the Roof Beams, the next step in the design process was to run the calculations for various combinations of wood beam types, species, and sectional properties (i.e., breadth and depth). The basic design methodology for flexure (bending), shear, and deflection are outlined in “Section 3.3”, “Section 3.4”, and “Section 3.5” of the 2015 NDS. Similarly, design considerations and equations for determining the compressive and tensile capacities are defined in “Section 3.6” and “Section 3.8” of the 2015 NDS (American Wood Council, 2016).

In order to use the equations referenced above, the reference design values needed to first be determined. The reference design values are denoted in the design as $F_b$, $F_v$, $F_c$, $F_t$, $E$, and $E_{\text{min}}$ for bending strength, shear strength, compressive strength, tensile strength, the modulus of elasticity, and minimum modulus of elasticity, respectively. These values can be found in the NDS Supplement - National Design Specification Design Values for Wood Construction (2015 NDS Supplement) or provided by a product supplier. In the case of the Vu Villa Design and Rehabilitation project, the values were pulled from “Table 5A” of the 2015 NDS Supplement reference design value tables (American Wood Council, 2014a).

Once the reference design values were established, they were corrected by use of the adjustment factors specified in “Table 5.3.1” for each individual design value (American Wood Council, 2016). The various subsections of “Section 5.3” detail the specific adjustment factors and logic.
(American Wood Council, 2016). For complete logic and justification of factors used, refer to the Design Calculations in Appendix D of this report.

When all applicable factors had been identified, the adjusted reference design value was then calculated. This was done by taking the original reference design value from the table and multiplying by the associated factors as they are identified in “Table 5.3.1” of the 2015 NDS (American Wood Council, 2016). The adjusted reference design values are identified in the design calculations with an apostrophe following the letter. In example, for the flexure or bending strength, the initial reference design value is \( F_b \), after it has been adjusted with the appropriate factors, it is denoted as \( F'_b \). It should be noted that the adjusted referenced design values are actually the design strengths used to determine the allowable capacity of the members.

With the known flexure, shear, tensile, and compressive strengths known, the allowable bending moment, shear, compression, and tension were determined for the beams. This was done using the rearranged version of the equations provided in the NDS (American Wood Council, 2016) and by Gupta in Principles of Structural Design (Gupta, 2014):

\[
\begin{align*}
\Phi M_n &= f'_b S \quad (3.2.1-1) \\
\Phi P_n &= f'_c A \quad (3.2.1-2) \\
\Delta &= \frac{5wl^4}{384E'I} \quad (3.2.1-3) \\
\Phi V_n &= \frac{2}{3} f'_v A \quad (3.2.1-4) \\
\Phi T_n &= f'_t A \quad (3.2.1-5)
\end{align*}
\]

where,
A – Cross-Sectional Area of the Beam (inches squared, in\(^2\));
\( E' \) – Adjusted Modulus of Elasticity of Beam (pounds per square inch, psi);
\( F'_b \) – Adjusted Bending/Flexural Strength of Beam (psi);
\( F'_c \) – Adjusted Compressive Strength of Beam (Parallel to Load) (psi);
\( F'_t \) – Adjusted Tensile Strength of Beam (psi);
\( F'_v \) – Adjusted Shear Strength of Beam (psi);
l – Span of the Beam (feet, ft);
I – Moment of Inertia of Beam (inches to the fourth, in\(^4\));
W – Uniform Load acting on the Beam (pounds per foot, lb/ft);
\( \Phi M_n \) – Allowable Bending Moment of Beam (kip-feet, k-ft);
\( \Phi P_n \) – Allowable Force on Beam (kips, k);
\( \Phi T_n \) – Allowable Tensile Force on Beam (k);
\( \Phi V_n \) – Allowable Shear Force on Beam (k).

After each of the member strengths had been tabulated, they were then compared to the ultimate loads from the three conditions that were previously determined. The comparison confirmed the
beams ability to support the specified loading conditions. If the tabulated strengths were all greater than the ultimate stresses then it was determined that the specified beam was an adequate design. Though not shown in the calculations, this was an iterative process that evaluated multiple types, species, and dimensions of beams. The proposed beam was selected over others, based on comparisons of availability and cost.

3.2.2. 2nd Floor Column Design

The next replacement member designed, following the primary load path from the roof level down, was the 2nd Floor Columns. Six 2nd Floor Columns are proposed, with each located at the splice conditions between Roof Beams. Similar to the beams, the columns are equally spaced in both the north-south and east-west directions. Figure 7 shows the proposed locations for the each of the 2nd Floor Columns.

![Figure 7: 2nd Floor Columns Proposed Layout - Plan View](image)

Loading conditions remained generally the same for the design of the 2nd Floor Columns, as they were for the Roof Beams. The only significant difference that occurred was a slight increase in the dead load due to the need for the column to support its own self-weight.

Per the design, the columns only primary contributing forces are from the roof system and designed Roof Beams gravity load. The entire lateral load was able to be resisted by the URM
walls acting as shear walls (see sections on existing member verification for further information). However, as a matter of conservancy, calculations were also performed demonstrating the 2nd Floor Columns, assuming a moment permitting connection was installed, could support twenty-five percent (25%) of the lateral loads in both directions, as well. With both of these conditions in mind, the only strength parameters that required evaluation for the columns was compression due to the gravity loads, and shear at the base of the column and bending of the column due to an assumed 25% of the applied lateral load.

The design followed a similar logic to that used for the Roof Beams. Using engineering statics and mechanics the ultimate internal forces acting on the columns were evaluated. Several major ‘conservative’ assumptions were required in order to evaluate the columns behavior when subject to a lateral load. The columns were assumed to behave as a moment-frame when subject to the load. For assumptions and the lateral design approach for the columns, see the Design Calculations in Appendix D.

Reference design values were acquired from “Table 4D” of the 2015 NDS Supplement (American Wood Council, 2014a). The applicable adjustment factors were determined from “Table 4.3.1” and tabulated based on “Section 4.3” of the 2015 NDS (American Wood Council, 2016). Once the aforementioned were known, the adjusted reference design values were calculated. The strength capacities were determined for compression, flexure, and shear of several species of available columns. The ultimate loads were then compared to each to verify that the member’s species and section properties were adequate. Again, the most available and cost-effective option was selected.

3.2.3. 2nd Floor Beam Design

The 2nd Floor Beams were the next replacement member designed in the primary load path. The 2nd Floor Beams mirror the Roof Beams layout and their proposed location is centered on the midline of both the east and west bays. The one variation from the Roof Beams is that only six beams are required. This is due to the presence of the interior URM wall which replaces the middle set of beams and separates the east and west bays from one another. The junction of the 2nd Floor Beams with the 1st and 2nd Floor Columns is one of the most critical locations for the design. The beam splice of the 2nd Floor Beams designates the centerline for both the placement of both floor’s columns. Figure 8 below shows the proposed layout for the 2nd Floor Beams.
The design of the 2nd Floor Beams used the same equations and methods discussed for the Roof Beams. The loading conditions are scaled up significantly, however, due to the location of the beams and the increase of the dead and live loads being contributed from the second-floor.

For the vertical and transverse, lateral loading on the beams, the ultimate internal loads were once again flexure, shear, and deflection. Similarly, the longitudinal lateral forces on the second-floor beam were compressive and tensile in nature. In the vertical plane, as is often the case with long beam spans, deflection was the controlling factor. This was the case for both the proposed Roof and 2nd Floor Beams for the Vu Villa Structural Design and Rehabilitation project.

Determination of the beam’s strength followed the common theme of, first determining the reference design values from “Table 5A” of the 2015 NDS Supplement (American Wood Council, 2014a). Then “Section 5.3” of the 2015 NDS was consulted, once again, for specifying the factors, and the adjusted design values were calculated (American Wood Council, 2016). Then using the equation described in Section 3.2.1 above, the strength of the member for each property was determined. These strengths had to be greater than the ultimate internal forces from the factored loads, in order to select the members that were adequate for structural integrity and serviceability. The proposed beam was then determined by reaching out to local suppliers to determine the most readily available and cheapest option.

3.2.4. 1st Floor Column Design

The final member in the primary load path is the 1st Floor Columns. Much like the two sets of beams, the logic and methodology for design and placement of the 1st Floor Columns reflect
those of the 2nd Floor Columns. The 1st Floor Columns were designed to be directly below and centered-on the beam splice condition of the 2nd Floor Beams. In the other direction (east-west), the beams will also be centered on the columns, with an equal distance of clearance from the edge of the columns to the face of the 2nd Floor Beams. For further information related to the installation of the beams and columns refer to the Design Drawings provided in Appendix A.

For design of the 1st Floor Columns, the loads applied to the columns are similar, once again, to those of the 2nd Floor Beams. Likewise, the methods and equations, reflect those of the 2nd Floor Columns. The only load that the 1st Floor Columns will be required to support is the gravity load provided by those members upstream in the primary load path. Like the 2nd Floor Columns, calculations were still performed demonstrating that the columns were capable of resisting 25% of the lateral loads. However, as was mentioned in Section 3.2.2 of this report, the URM walls are capable of resisting the entire lateral load.

Again, the contributing internal forces impacting the columns consisted of the compression from gravity loads, the shear at the base of the column from lateral loads, and the bending of the column due to its cantilevered condition, assuming a moment resisting connection were used.

As discussed in Section 3.2.2, above, the columns strengths were determined from the reference design values from “Table 4D” of the 2015 NDS Supplement (American Wood Council, 2014a), while adjustment factors and the calculated reference design value were established by “Section 4.3” and “Table 4.3.1” of the 2015 NDS (American Wood Council, 2016). The strength of the proposed members was based on its properties and determined by Equations 3.2.1-1, 3.2.1-2, and 3.2.1-4 specified above. The optimized column species and dimensions were determined by a cost comparison and feedback of available products from local providers.

### 3.2.5. Connection Design

Once all replacement members had been designed, the next step was to specify connections by which the loads would be transferred along the primary load path. For the Vu Villa Structural Design and Rehabilitation project, connections were specified for all new joints between adjacent replacement members, as well as for joints where existing members would need to be secured to newly installed members. In general, three connection types were identified which required specification:

- Beam-Column Connections;
- Beam-Joist Connections; and
- Beam-Wall Connections.

Loading for each connection was determined based on the forces that acted on each member and required transferring to the next member in the primary load path. In the interest of this report’s brevity, please refer to the Design Calculations in Appendix D for the determined loads on each connection.
All connections specified for installation were either Simpson Strong-Tie® products, nails, or machine bolts. For detailed connection information refer to the Design Drawings provided in Appendix A or the Technical Specifications provide in Appendix B. Determination of the capacity of connections was from the *Simpson Strong-Tie® Wood Construction Connectors Catalog* load tables (Simpson Strong-Tie, 2019), or the design provisions of “Chapters 11 – Mechanical Connections” and “Chapter 12 – Dowel-Type Fasteners” of the 2015 *NDS* (American Wood Council, 2016).

Several specific considerations that warrant further discussion include the connection of Roof Beams to 2nd Floor Columns, and both Roof Beam and 2nd Floor Beam connections to the exterior URM walls. In the first scenario, apart from the anticipated gravity and lateral loads, there was the additional consideration of uplift from wind which significantly impacted the behavior of the Roof Beams. Due to the uplift forces applied to the unsupported lengths of the Roof Beams, the beam splices required an additional connection which could transfer the compressive forces generated from the uplift of the beams. A unique Simpson strap was required that was capable of transferring the compression forces across the beam splice. The compressive forces and strap are further discussed in the Design Calculations in Appendix D.

Similarly, for the connection of both Roof and 2nd Floor Beams, anchoring was required for both to the existing URM walls. The proposed anchoring method was by the use of a Simpson connection which was coupled by use of an epoxy to the URM walls for both levels and at locations where the 2nd Floor Beams were connected to existing wood beam headers.

### 3.3. Existing Member Verification

Existing member verification was performed in conjunction with the design of the replacement members. Similar to the method of replacement member design, the existing members were verified following the primary load path starting at the roof level. Existing member verification consisted of evaluating the current members in place for their ability to carry the ultimate loads discussed in Section 3.1. An additional goal of the existing member verification was to provide criteria for which an engineer or competent professional could evaluate the existing members during the demolition process and identify those members that required replacement. The three existing members in which verifications were performed included:

- Diaphragms (Roof and 2nd Floor);
- Joists (Roof and 2nd Floor); and
- Load-Bearing Walls (4-Exterior and 1-Interior).

The methods used for verifying the members capacity are discussed in the remainder of Section 3.3.
3.3.1. Diaphragm Verification

Diaphragms are defined by the 2015 SDPWS as “a roof, floor, or other membrane bracing system acting to transmit lateral forces to the vertical resisting elements (American Wood Council, 2014b). In the case of the Vu Villa, the term diaphragm is used to refer to the combination of joists, sheathing, and nails at both the roof and second-floor levels. Diaphragms were one of the members in which identifying the existing components was not possible. Due to this, a set of minimum criteria was developed that could guide decision-making of whether aspects of the diaphragm required replacement or alteration in the field.

Both the roof- and second-floor diaphragms were assessed using the methods and values provided in the 2015 SDPWS (American Wood Council, 2014b) as well as with values from the American Plywood Association’s (APA) Load-Span Tables for APA Structural-Use Panels (American Plywood Association, 2006). The Load-Span Tables for APA Structural-Use Panels provided strength values for a variation of sheathing grades and thicknesses (American Plywood Association, 2006). These values were compared to the factored load that was anticipated on the existing sheathing. From this, a minimum thickness and type of panel was determined. This was done in respect to the gravity loads that would be acting on the panels.

Once minimum criteria for the panel type and thickness were established, the lateral loads were addressed. In order for the diaphragm to resist the lateral loads, a combination of the sheathing type and size, as well as the nailing patterns, needed to be specified. This was done in accordance with the provisions from “Section 4” of the SDPWS in concurrence with the unit shear capacities from “Table 4.2A”, “Table 4.2C”, and “Table 4.2D” (American Wood Council, 2014b).

Based on the performance of the system versus the ultimate lateral loads, minimum criteria were specified for sheathing thickness and orientation, boundary nail patterns and size, and panel edge/interior nail pattern and size. For further information on the verification of the roof and second-floor diaphragms refer to the Design Calculations in Appendix D.

3.3.2. Joist Verification

Unlike the diaphragms, both joists were accessible and the general dimensions and material were able to be determined. In a process reflective of the beam design discussed in Section 3.2, the joists were evaluated for their strength in resisting the calculated ultimate vertical loads, as well as the lateral loads in the longitudinal (north-south) and transverse (east-west) directions. Much like the beams, the internal forces considered for the joist’s strength were bending/flexure and shear for the gravity loads and lateral loads in the longitudinal direction, and compression/tension in the transverse direction.

Due to the inaccessibility of the lengths the joists, the property that was unable to be determined was the grade of the material. Because of this, a minimum grade was established based on the loading condition that could also be used as an evaluation tool during actual construction activities. As is mentioned in the Design Drawings and Technical Specifications denoted in Appendix A and Appendix B, respectively, any existing joist that does not meet the minimum
specified criteria is to be removed and replaced with a substitute meeting the specified structural requirements.

3.3.3. Bearing/Shear Wall Verification

As the last verification step, the existing bearing and shear walls were assessed for their capability of transferring the vertical and lateral loads to the foundations. This was done with both the methods discussed previously for wood portions of the walls and with provisions outlined in the *Building Code Requirements and Specification for Masonry Structures* (The Masonry Society, American Concrete Institute, and Structural Engineering Institute, 2011) for the URM walls.

Five walls were verified, four exterior walls and one interior wall. The southern exterior wall, and store-front side, was a hybrid of both wood components and walls as well as unreinforced masonry brick. The other four walls consisted entirely of masonry brick. All exterior walls stood the full two stories of the building (approximately 30-feet, with the parapet), and the interior wall was the height of the first story, 16-feet.

Each was assessed for their compressive strength and ability to convey the gravity loads from roof level down to the foundation. In addition, they were evaluated for their ability to resist flexure, shear, and overturning generated from lateral loads in both transverse and longitudinal directions. For detailed assumptions and calculations refer to Appendix D of this report.

3.4. Engineer’s Estimate

As stated in the design constraints, the Vu Villa Structural Design and Rehabilitation project was to optimize the design and reduce costs where possible. As is the case in most projects, keeping the expenses to a minimum was a major goal. With costs a major consideration, an engineer’s estimate was put together which projected the expenses associated with the design, demolition, and construction of the project. This was done by estimating the costs of materials and labor, and projecting the duration of the project. Quotes by local distributors were used for approximate material costs, while labor and equipment costs were determined based on online resources and engineering judgement.

Though the removal and replacement of the beams and columns had a clear scope, the other existing members quality and need to be replaced was unknown. Therefore, three costs were developed as part of the estimate. The Lowest-Case (LC), Most-Likely Case (MLC), and Highest-Case (HC) costs for the project were determined based on the number of existing joists and sheathing that require replacement. See Appendix E for all calculations, assumptions, and estimated costs associated with the Vu Villa Structural Design and Rehabilitation project.
4.0 RESULTS

Section 4.0 discusses the ultimate loads, specified members and connections, and final costs of completing the Vu Villa Structural Design and Rehabilitation project. For the Design Drawings and Technical Specifications detailing the final results refer to Appendix A and Appendix B, respectively.

4.1. Loading Conditions

The following section details the results of the loading calculations discussed in Section 3.1. Table 1 below shows the ultimate loads that act on each replacement member or, in the case of lateral loads, act at each relative story. For all calculations and assumptions related to determining the loading conditions for the Vu Villa Structural Design and Rehabilitation project, refer to Appendix C.

Table 1: Ultimate Loading Conditions for the Vu Villa

<table>
<thead>
<tr>
<th>Load Type/Location</th>
<th>Load</th>
<th>Units</th>
</tr>
</thead>
<tbody>
<tr>
<td>Gravity Loads</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Roof Beam</td>
<td>65.9</td>
<td>psf</td>
</tr>
<tr>
<td>2nd Floor Column</td>
<td>66.5</td>
<td>psf</td>
</tr>
<tr>
<td>2nd Floor Beam</td>
<td>194.4</td>
<td>psf</td>
</tr>
<tr>
<td>1st Floor Column</td>
<td>167.0</td>
<td>psf</td>
</tr>
<tr>
<td>Lateral Loads</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Roof Diaphragm</td>
<td>566.8</td>
<td>plf</td>
</tr>
<tr>
<td>2nd Floor Diaphragm</td>
<td>1268.3</td>
<td>plf</td>
</tr>
<tr>
<td>Roof Shear Wall</td>
<td>1342.3</td>
<td>plf</td>
</tr>
<tr>
<td>2nd Floor Shear Wall</td>
<td>1716.2</td>
<td>plf</td>
</tr>
</tbody>
</table>

4.1.1. Dead Loads

The dead loads determined for each replacement member followed the methodology laid out in Section 3.1.1. For the Roof Beams the total dead load was 17.5 pounds per square foot (psf). Dead loads determined for the 2nd Floor Columns, 2nd Floor Beams, and 1st Floor Columns were 18 psf, 34.8 psf, and 36.0 psf, respectively. Table 2 below presents the estimated Roof Beam Dead Loads.
Table 2: Roof Beam Dead Load Calculations

<table>
<thead>
<tr>
<th><strong>Roof Beams:</strong></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>Built-Up-Roofing (5-Ply with Gravel)</td>
<td>6.5 (psf)</td>
</tr>
<tr>
<td>Roof Sheathing (3/4&quot; Plywood)</td>
<td>2.3 (psf)</td>
</tr>
<tr>
<td>Insulation (Loose)</td>
<td>0.5 (psf)</td>
</tr>
<tr>
<td>Roof Joists (2&quot;x6&quot; @ 16&quot; O.C.)</td>
<td>1.4 (psf)</td>
</tr>
<tr>
<td>Ceiling (1&quot; Gypsum Drywall)</td>
<td>5.0 (psf)</td>
</tr>
<tr>
<td>Member Self-Weight</td>
<td>1.8 (psf)</td>
</tr>
<tr>
<td><strong>Total:</strong></td>
<td><strong>17.5 (psf)</strong></td>
</tr>
</tbody>
</table>

4.1.2. Live Loads

Live loads were tabulated as discussed in Section 3.1.2. The applicable live loads for the roof and second-floor was as follows:

- Roof Beams = 18.1 psf;
- 2nd Floor Columns = 18.1 psf;
- 2nd Floor Beams = 86.7 psf; and
- 1st Floor Columns = 68.6 psf.

As you can see, the Roof Beams and 2nd Floor Columns were equivalent for the roof live load. This was strictly because the equation does not change for either. In contrast, the 2nd Floor Beams and 1st Floor Columns values varied due to the live load element factor ($K_{LL}$) in the floor live load equation which varies based on the type of member (i.e. beam, column, etc.) the load is applied to. Table 3 demonstrates the calculations for the roof live load which acts on the proposed Roof Beam.

Table 3: Roof Live Load Calculations

<table>
<thead>
<tr>
<th><strong>Roof Beams:</strong></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>TA, Tributary Area</td>
<td>295.6 (ft$^2$)</td>
</tr>
<tr>
<td>θ, Roof Slope</td>
<td>0.0 (º)</td>
</tr>
<tr>
<td>$L_0$, Initial Live Load</td>
<td>20.0 (psf)</td>
</tr>
<tr>
<td>$R_1$, Tributary Area Reduction Factor</td>
<td>0.9 (psf)</td>
</tr>
<tr>
<td>$R_2$, Slope Reduction Factor</td>
<td>1.0 (psf)</td>
</tr>
<tr>
<td>$L_r$, Roof Live Load</td>
<td>18.1 (psf)</td>
</tr>
</tbody>
</table>

4.1.3. Snow Loads

Snow loads were figured as described in Section 3.1.3. The Vu Villa has a flat roof, with less than a five-degree pitch, therefore the approximation of the snow load used the flat roof snow load equation. The ground snow load was determined based on an interactive map as was
discussed above and was taken as 36.8 psf. After tabulating the design snow load using “Equation 7-1” of *ASCE 7-02* (American Society of Civil Engineers, 2003) the value was determined to be 28.1 psf.

### 4.1.4. Wind Loads

Wind load calculations were performed for both the vertical and lateral directions. Due to the flat roof, the only contributing load acting in the vertical direction was uplift. For conservancy, the uplift was neglected when considering the loads acting on the replacement members. The uplift was taken into consideration only for the design of connections. Uplift on each specific connection was determined based on the tributary area of the joint for the proposed connection multiplied by the design wind pressure applicable to that tributary area. If the tributary area had multiple design wind pressures associated with it, the larger value was used.

As was discussed previously, net design wind pressures were acquired from “Figure 6-2” of *ASCE 7-02* (American Society of Civil Engineers, 2003) for both the vertical and lateral directions. These values were then multiplied by the importance factor and adjustment factor for building height and exposure. The design wind pressures are listed in Table 4.

*Table 4: Design Wind Pressures*

<table>
<thead>
<tr>
<th>Zone: Design Pressure (psf)</th>
<th>Lateral</th>
<th>Vertical</th>
</tr>
</thead>
<tbody>
<tr>
<td>A</td>
<td>22.1</td>
<td>-26.6</td>
</tr>
<tr>
<td>B</td>
<td>16.0</td>
<td>-16.0</td>
</tr>
<tr>
<td>C</td>
<td>16.0</td>
<td>-16.0</td>
</tr>
<tr>
<td>D</td>
<td>16.0</td>
<td>-18.4</td>
</tr>
<tr>
<td>E</td>
<td>-26.6</td>
<td>-16.0</td>
</tr>
<tr>
<td>F</td>
<td>-16.0</td>
<td>-18.4</td>
</tr>
<tr>
<td>G</td>
<td>-16.0</td>
<td>-16.0</td>
</tr>
<tr>
<td>H</td>
<td>-18.4</td>
<td>-16.0</td>
</tr>
</tbody>
</table>

For the lateral forces, the largest design pressure was then used with the associated tributary height to determine the maximum uniform load along both diaphragms. For the roof diaphragm, the lateral load acting in both the longitudinal and transverse directions were 132.5 plf. For the second-floor diaphragm, the maximum uniform load was 309.1 plf.

### 4.1.5. Seismic Loads

As was alluded to in Section 3.1.5 of this report, three primary load types were established for seismic and earthquake related forces. The vertical forces, lateral forces acting on horizontal elements (i.e. diaphragms), and lateral forces acting on vertical elements (i.e. shear walls). For each of these, the load values were determined as was discussed in Section 3.1.5.

The vertical forces were relatively insignificant, with the total force for the roof being 1.0 psf and the second-floor being 2.1 psf. However, as is discussed in Section 4.1.6 below, the lateral forces for both horizontal and vertical elements were the controlling loads. As discussed above, the total lateral force was determined for each level of the building. A uniform load was then formulated by dividing the total force by the length of the diaphragm. These uniform loads were used in the LRFD Load Combination Equations and compared to the design wind loads from Section 4.1.4.
Table 5 defines each lateral load induced by seismic forces on the horizontal and vertical elements for both the roof and second-floor.

*Table 5: Seismic Lateral Loads*

<table>
<thead>
<tr>
<th>Seismic Lateral Loads (plf)</th>
<th>Roof Level</th>
<th>2nd Floor</th>
</tr>
</thead>
<tbody>
<tr>
<td>Loads on Horizontal Elements</td>
<td>566.8</td>
<td>1,268.3</td>
</tr>
<tr>
<td>Loads on Vertical Elements</td>
<td>1,342.3</td>
<td>1,716.2</td>
</tr>
</tbody>
</table>

**4.1.6. LRFD Load Combination Equations**

With the loads tabulated above, the LRFD load combination equations were then used to determine the ultimate vertical and lateral loads acting on each member. For the gravity loads, equation 3 governed for both the Roof Beams and 2nd Floor Columns, this was primarily due to the effect of the roof live load being scaled by a factor of 1.6. Similarly, the 2nd Floor Beams and 1st Floor Columns were governed by equation 2 due to a 1.6 factor applied to the floor live load. In both instances the live loads were the greatest contributor of weight to the structure due to gravity.

For the lateral loads, at both diaphragm levels, the seismic loads controlled. Butte lies within a moderate area for seismic activity and due to the Vu Villa’s protection by surrounding buildings, the wind loads are only approximately a fourth of the seismic loads. As you can see from the load combinations in Figure 1 above, almost all equation’s factors for wind and seismic loads are one and therefore the seismic governed in all instances.

Table 1 at the beginning of Section 4.1 provides the ultimate loads used in the design of replacement members for the Vu Villa Structural Design and Rehabilitation project. Table 6 below shows the calculations and use of the LRFD equations for determining gravity loads on the Roof Beams. For complete load combinations and calculations associated with each, refer to Appendix C.

*Table 6: Roof Beam Gravity Load Combination Equations*

<table>
<thead>
<tr>
<th>EQUATION NO.</th>
<th>LOAD COMBINATION EQUATIONS</th>
<th>ULTIMATE LOAD (psf)</th>
<th>TIME EFFECT FACTOR (λ)</th>
<th>ULTIMATE LOAD/λ (psf)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>1.4D</td>
<td>24.5</td>
<td>0.6</td>
<td>40.8</td>
</tr>
<tr>
<td>2</td>
<td>1.2D + 1.6L + 0.5(Lr or S)</td>
<td>35.0</td>
<td>0.8</td>
<td>43.8</td>
</tr>
<tr>
<td>3</td>
<td>1.2D + 1.6(Lr or S) + L + 0.5W</td>
<td>65.9</td>
<td>0.8</td>
<td>82.3</td>
</tr>
<tr>
<td>4</td>
<td>1.2D + W + L + 0.5(Lr or S)</td>
<td>35.0</td>
<td>1.0</td>
<td>35.0</td>
</tr>
<tr>
<td>5</td>
<td>1.2D + E + L + 0.2S</td>
<td>26.6</td>
<td>1.0</td>
<td>26.6</td>
</tr>
<tr>
<td>6</td>
<td>0.9D + W</td>
<td>15.7</td>
<td>1.0</td>
<td>15.7</td>
</tr>
<tr>
<td>7</td>
<td>0.9D + E</td>
<td>15.7</td>
<td>1.0</td>
<td>15.7</td>
</tr>
</tbody>
</table>

\[ P_u, \text{ ULTIMATE ROOF BEAM LOAD} = 65.9 \text{ (psf)} \]

\[ \lambda, \text{ APPLICABLE TIME EFFECT FACTOR} = 0.8 \]
### 4.1.7. Chord and Drag Analysis

Chord and drag forces were determined by the methods from Section 3.1.7 and were used in the evaluation of diaphragm and shear walls discussed in Section 4.3. Table 7 details the unit drag shear and chord tension/compression for each level.

**Table 7: Roof Level and Second-Floor Chord and Drag Forces**

<table>
<thead>
<tr>
<th>CHORD &amp; DRAG FORCES</th>
<th>Longitudinal</th>
<th>Transverse</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Roof Level</td>
<td>2nd Floor</td>
</tr>
<tr>
<td>Chord Tension/Compression (k)</td>
<td>4.3</td>
<td>2.4</td>
</tr>
<tr>
<td>Diaphragm Unit Drag (lb/ft)</td>
<td>283.4</td>
<td>211.4</td>
</tr>
<tr>
<td>Shear Wall Unit Drag (lb/ft)</td>
<td>671.2</td>
<td>572.1</td>
</tr>
</tbody>
</table>

### 4.2. Replacement Member Design

The designed replacement members consisted of Glued-Laminated (Glulam) timber for both the Roof and 2\textsuperscript{nd} Floor Beams and Douglas Fir-Larch (North) (DFL(N)) species timbers for the 2\textsuperscript{nd} Floor and 1\textsuperscript{st} Floor Columns. Section 4.2 discusses the design members, construction requirements, and specifications related to both the replacement beams and columns. Section 3.2 discusses the design methodology for each member and for further information related to the design refer to the Design Calculations in Appendix D.

#### 4.2.1. Roof Beams

The proposed member is a 5 ½-inch by 21-inch 24F-V4 Douglas Fir/Douglas Fir (DF/DF) Glulam beam. The beams were designed to be of equal span with three consecutive beams spanning the length of the roof in the north-south direction. They were also designed with equal spacing between each set of beams, three sets in total, in the east-west direction. The Roof Beams will support the joists and roof system and transfer the loads from the roof level to the 2\textsuperscript{nd} Floor Columns.

At locations where the beams meet exterior URM walls, they shall rest on a bench cutout within the wall. They shall be connected to the wall with a Simpson HGLBA Beam Seat and two Simpson HTSM20 Straps on either side of the beam to brace against uplift. A minimum air gap of ½-inch shall be provided on all sides of the beam between the existing URM wall and the beam in accordance with “Section 2304.11.2.5” in the 2012 IBC (International Code Council, Inc., 2011).

Where the beams butt-together, they shall rest on the 2\textsuperscript{nd} Floor Columns. The joint (or beam splice) between beams shall be centered on the profile of the column. The beams shall connect to the columns with a Simpson CC66 column cap and four Simpson CTS218 compression/tension straps, two installed on either side of the beams across the splice. Once the beams have been installed, the existing joists shall be toe-nailed to the newly installed beams with 8d common nails.
For complete information on the installation and connections of the Roof Beams refer to the Design Drawings in Appendix A and the Technical Specifications in Appendix B.

4.2.2. 2nd Floor Columns

The 2nd Floor Columns shall be 6-inch by 6-inch DFL(N) Select Structural grade timbers. They shall be installed equally spaced in both the longitudinal and transverse directions with columns located at each Roof Beam splice.

Connection to the Roof Beam shall be as discussed in the previous section. For assembly of the columns to the 2nd Floor Beams, two Simpson HL53 heavy angles and two Simpson HST5PC straps shall be used to transfer the loads across the member’s interface.

4.2.3. 2nd Floor Beams

The 2nd Floor Beams will be 6 ¾-inch by 24-inch 24F-V4 DF/DF Glulams. They will be installed much in the same regards as the Roof Beams, with equal spans of three beams running north-south. In total, there will be six beams, three along each alignment and each will be centered on the east and west bays with equal distance from exterior and interior URM walls.

Beams adjoining the URM walls on the north and south ends of the building will be connected in a similar fashion to the Roof Beams. Simpson HGLBD beam seats will be used to adhere the beams to URM walls. By the time uplift forces reach the 2nd Floor Beams, the forces are negligible. Therefore, no strapping was required to resist uplift as was required for the Roof Beams.

The most active location for connection hardware will be where the 2nd Floor Beams join both the 1st and 2nd Floor Columns. For connections to both columns, Simpson HL53 angles shall be installed in conjunction with Simpson HST5PC straps. Four straps shall be centered on the beam splice and placed along the east and west faces of the 2nd Floor Beams.

Once the 2nd Floor Beams are installed, the second-floor joists shall be adjoined to the beams by toe-nailing the base of the joist to the beam with 10d common nails.

For further information on each connection, see Section 4.2.5 below. For further details and specifications related to the 2nd Floor Beams refer to Appendix A (Design Drawings) and Appendix B (Technical Specifications).

4.2.4. 1st Floor Columns

10-inch by 10-inch Select Structural DFL(N) timbers will be used for the 1st Floor Columns. As with the second-floor, the 1st Floor Columns will be installed at each beam splice. Two columns
shall be equally spaced in the north-south directions and will be placed centered on the midline of the east or west bay.

Connection of the 1st Floor Columns to the 2nd Floor Beams shall be by the materials and methods discussed in Section 4.2.3 and 4.2.5. Connection of the 1st Floor Columns to the slab-on-grade foundation in the west bay or the structural framing in the east bay shall require evaluation of conditions during construction. Due to the unknown state of the subfloor for both bays and the difficulty approximating where in the basement the columns will tie-in, it was decided that until further information could be collected, the connection devices for the 1st Floor Columns to the foundation or first-floor members would be treated as a deferred submittal. Provided the design were to be finalized at some point, the fastening methods for the 1st Floor Columns would be designed and detailed prior to the submission of the final design.

For further details and specifications related to the 1st Floor Columns refer to Appendix A (Design Drawings) and Appendix B (Technical Specifications).

### 4.2.5. Connections

Connections shall follow the schedule listed in Table 7. All connection hardware shall be of the named manufacturer unless the Contractor provides an acceptable alternative. If an alternative hardware manufacturer’s product is desired, Contractor must provide all technical and product information for the Engineer to review and approve prior to implementation.

Connections for both Roof and 2nd Floor Beams will consist of beam seats installed prior to placement of the beams. Beam seats to act as connections at the roof level shall be installed with a 1 \(\frac{7}{16}\)-inch clearance from the interior face of the URM wall while the beam seats for the second-floor shall be installed flush with the associated wood header or URM wall. 1-inch holes will be drilled to accommodate the installation of the threaded rebar of the beam seat into the existing wood header or URM walls. The holes will be filled with MiTek Incredi-Bond Epoxy prior to placing the threaded rebar in the holes to provide a connection between the beam seat and the URM walls. The threaded rebar should be centered in the holes so that the epoxy binds on all sides of the rebar. The Roof or 2nd Floor Beams shall then be properly placed and the bolt holes drilled in the beams to allow for installation of the \(\frac{3}{4}\)-inch bolts. For the Roof Beams, prior to bolt installation, Simpson HTSM20 straps shall be installed and connected to the beam and URM wall with the specified fasteners in Table 7. HTSM20 straps should be placed inside of the beam seat brackets and centered between the bolts of the beam seat.
Table 7: Replacement Connection Schedule

<table>
<thead>
<tr>
<th>Connection Location</th>
<th>Hardware</th>
<th>Fasteners</th>
</tr>
</thead>
<tbody>
<tr>
<td>Roof Beam - 2nd Floor Column</td>
<td>Simpson CC66 Column Cap;</td>
<td>2 - 5/8&quot; Grade 2 Machine Bolts;</td>
</tr>
<tr>
<td></td>
<td>4 - Simpson CTS218 Straps</td>
<td>96 - SD #9 x 1 1/2&quot;</td>
</tr>
<tr>
<td>Roof Beam - Roof Joist</td>
<td>N/A</td>
<td>4 - 8d Common Nails</td>
</tr>
<tr>
<td>Roof Beam - URM Wall</td>
<td>Simpson HGLBA Beam Seat;</td>
<td>2 - 3/4&quot; Grade 2 Machine Bolts;</td>
</tr>
<tr>
<td></td>
<td>2 - Simpson HTSM 20 Straps</td>
<td>20 - 0.148 x 1 1/2&quot; Sinkers;</td>
</tr>
<tr>
<td></td>
<td></td>
<td>8 - 1 1/4&quot; Titen® 2 Screws</td>
</tr>
<tr>
<td>2nd Floor Beam - 1st Floor Column</td>
<td>2 - Simpson HL53 Angles;</td>
<td>2 - 1/2&quot; Grade 2 Machine Bolts;</td>
</tr>
<tr>
<td></td>
<td>2 - Simpson HST5PC Straps</td>
<td>12 - 5/8&quot; Grade 2 Machine Bolts</td>
</tr>
<tr>
<td>2nd Floor Beam - 2nd Floor Column</td>
<td>2 - Simpson HL53 Angles;</td>
<td>2 - 1/2&quot; Grade 2 Machine Bolts;</td>
</tr>
<tr>
<td></td>
<td>2 - Simpson HST5PC Straps</td>
<td>12 - 5/8&quot; Grade 2 Machine Bolts</td>
</tr>
<tr>
<td>2nd Floor Beam - 2nd Floor Joist</td>
<td>N/A</td>
<td>3 - 10d Common Nails</td>
</tr>
<tr>
<td>2nd Floor Beam - URM Wall</td>
<td>Simpson HGLBD Beam Seat</td>
<td>2 - 3/4&quot; Grade 2 Machine Bolts</td>
</tr>
</tbody>
</table>

Roof Beam connection to the 2nd Floor Columns shall consist of Simpson CC66 column caps and four – Simpson CTS218 straps per beam to column connection. The CTS218 straps shall be installed with two on either side of the beam, centered horizontally on the beam splice, and vertically placed on the second and third laminated two-by-sixes from the top. Similarly, the column caps shall be centered on the beam splice and center of the column as shown in the Design Drawings.

For 2nd Floor Beam to column connections (both first- and second-floor), the connection hardware shall be two - Simpson HL53 heavy angles and two – Simpson HST5PC straps. Angles shall be centered on both the column and beam forming right angles as shown on the Design Drawings. Straps shall be centered, horizontally, on the beam splice and shall be installed with 10-inches of strap in contact with the columns surface. Fasteners shall be as specified in Table 7 above.
Joists shall be toe-nailed with the designated fasteners at each location where a joist crosses a beam. For the Roof Beams, four nails shall be installed at each location. For the 2nd Floor Beams, three nails are required for each joist to beam connection and the nails shall alternate sides of the joist so as to ensure a similar number of fasteners adhere each side along the entire length of the joist.

In locations where a gap exists between hardware and replacement members, or between various structural members, a wood spacer shall be installed according to the Technical Specifications and as shown on the Design Drawings. The spacer shall be of equivalent or greater strength than the adjacent members and the proposed size and source of the material must be reviewed and approved by the Engineer on the project.

For further details and specifications, refer to the Design Drawings in Appendix A and the Technical Specifications in Appendix B. For further information on the methods and assumptions for the connection design, refer to the Design Calculations in Appendix D.

### 4.3. Existing Member Verification

As was denoted in the design objectives in Section 2.1, one of the goals of the Vu Villa Structural Design and Rehabilitation project was to preserve existing structural members which did not require replacement. Among these ‘existing members’ were the Roof and 2nd Floor Diaphragms, Joists, and URM walls. Similar methods to the design of the replacement members were used for assessing the structural capacity of each existing member. Based on these calculations, minimum criteria were developed which are intended to be used in the field to identify existing members that are unsatisfactory, structurally speaking, and require replacement. The remainder of Section 4.3 will discuss acceptable criteria for each existing member evaluated. For further information on the specific design methods used to develop the minimum criteria, refer to Section 3.3 in this report and Appendix D (Design Calculations). For minimum provisions, specifications, and details related to the existing members refer to the Design Drawings in Appendix A and to the Technical Specifications in Appendix B.

#### 4.3.1. Diaphragm Verification

Prior to the development of minimum criteria, both the Roof and 2nd Floor Diaphragms were assessed for their existing components. Due to the variety in potential materials that could be currently functioning as diaphragm components in the Vu Villa, a better approximation of the type of materials was desired to limit the minimum criteria scope. Based on visual observation at locations where diaphragm components were exposed, the roof sheathing appeared to have been replaced in the recent past and from a distance seemed to be oriented-strand board (OSB). Because of this, the criteria established for the roof sheathing assumed the material was either OSB or plywood. For the 2nd Floor Diaphragm, no available access existed to visually observe the components. However, the flooring appeared to be aged and was believed to be original. Based on that observation, it was assumed that the sheathing material was not OSB, as OSB did
not originate until around 1980 (American Plywood Association, 2019). Instead, it was assumed that the 2nd Floor Diaphragm consisted of either plywood or wood-plank sheathing. Using the anticipated dead, live, and snow loads acting vertically, it was determined that the minimum thickness for the Roof Diaphragm sheathing was $15/32$-inch. This was the case for both OSB and plywood. For the 2nd Floor Diaphragm, the minimum permissible thickness of plywood sheathing was determined to be $19/32$-inch. In the case that the diaphragm consisted of wood-planks, it was assumed the material would likely be 1-inch by 6-inch DFL(N) planks. In this case the sheathing would need to be placed in a double-diagonal pattern with two layers of planks in order to resist the anticipated lateral loads.

The next step in defining the minimum criteria for the Roof and 2nd Floor Diaphragms was determining the nail sizes and patterns required. This was done by use of the tables previous noted in Section 3.3. The Roof Diaphragm has the following minimum requirements related to the nailing sizes and patterns:

- Boundary nails shall be 8d (penny) common nails or the equivalent and shall be spaced at 6-inches o.c.;
- Boundary nailing shall continue to a minimum of 9-feet, 4-inches from each diaphragm – URM wall boundary;
- Nails along panel edges shall be 6d common nails at 6-inches o.c.; and
- Nails located along the interior sheathing panels shall be 6d common nails at 12-inches o.c.

Similarly, the 2nd Floor Diaphragm criteria shall be, at a minimum for plywood sheathing:

- 10d common nails at 4-inches o.c. for boundaries and along panel edges in the east-west (transverse) direction;
- 10d common nails at 6-inches o.c. for panel edges running north-south (longitudinal); and
- 10d common nails at 12-inches o.c. for interior nailing;

And minimum criteria for double-diagonal plank sheathing shall be:

- 20d common nails with 3 nails per plank - boundary connection; and
- 20d common nails with 2 nails per plank to joist connection.

In addition, for all 2nd Floor Diaphragm scenarios, blocking shall be required. Blocking is the method of fastening common framing members, such as small kickers to adjacent joists along panel edges to provide additional support. Blocking is not required for the Roof Diaphragm.

### 4.3.2. Joist Verification

Minimum criteria for the existing Roof and 2nd Floor Joists were similarly developed based on the ultimate loads discussed within this report. In the same way as the diaphragms, the joists were visually inspected to determine the estimated size of the members. The locations and
accessibility of both Roof and 2nd Floor Joists impacted the ability to verify the dimensions, however, relatively accurate values are believed to have been determined.

Determining the grade and quality of the existing joists was not feasible due to the limited access to the members, therefore, a minimum acceptable grade was determined based on the loading conditions. The Engineer or a Competent Professional shall independently verify, based on engineering judgement, the joists meet the required minimum grade designated in the Technical Specifications. If an existing member does not meet the minimum criteria, it is to be removed and replaced with a similar member that achieves both the specified grade requirements, as well as any applicable constructability requirements.

The Roof Joists shall be 2-inch by 6-inch Select Structural grade DFL(N) lumber or the equivalent and shall be oriented in the east-west direction. The joists shall be spaced at no more than 16-inches o.c. The joists must also span, at a minimum, the distance from URM walls to the replacement Roof Beams, or from beam to beam.

The 2nd Floor Joists shall be 2-inch by 12-inch Number (No.) 1 or Better (BTR) DFL(N) lumber or a structural equivalent. Similarly, they shall be oriented east-west, shall have a maximum spacing of 16-inches o.c., and shall span, at a minimum, from URM walls to 2nd Floor Beams.

It should be noted that the design calculations assumed that lumber dimensions were actual dimensions, not the reduced values based on the standards of lumber production today. This decision was made based on measurements taken at the site which demonstrated that each member was close to the actual (nominal) width and depth.

### 4.3.3. Bearing/Shear Wall Verification

The final existing members which were assessed for their strength capabilities were the existing bearing and shear walls. All five noted bearing/shear walls had components of URM. The north, east, and west exterior walls consisted entirely of URM, as did the interior wall. The southern exterior bearing/shear wall was a hybrid system consisting completely of URM for the second-floor and a combination of wood columns, beams, walls and URM columns on the first-floor.

Methods for evaluating the ability of the bearing/shear walls are discussed in Section 3.3.3 above, and assumptions and further calculation details are provided in the Design Calculations in Appendix D.

Based on the assumptions made, the URM walls were capable of conveying all of the vertical loads of the replacement members and alternative use of the second-floor. However, as Daniel Lazzarini notes in his thesis, URM structures are historically unstable when subject to earthquakes and seismic activity (Lazzarini, 2009). While not in an extremely active seismic area, it is clear from the Design Calculations in Appendix D, that if the Vu Villa were subject to significant lateral forces, URM walls would be incapable of supporting the loads caused by an earthquake. While outside the scope of this project, it is the recommendation of this report, that if renovation and rehabilitation work is performed on the Vu Villa, seismic retrofitting should be
implemented as part of such work. For further information related to URM wall instability and seismic retrofitting refer to Section 6.2 below.

4.4. **Engineer’s Estimate**

Cost was a major consideration in the design methods discussed within this report. The costs developed as part of the engineer’s estimate were strictly limited to the direct costs associated with the design, preparation, and implementation of the Vu Villa Structural Design and Rehabilitation project. No architectural, renovation, electrical, plumbing, or aesthetic costs were considered as part of the estimate. Table 8 below shows the estimated Lowest-Case, Most-Likely Case, and Highest-Case costs.

**Table 8: Lowest-Case, Most-Likely Case, and Highest-Case Costs**

<table>
<thead>
<tr>
<th>TASK</th>
<th>DESCRIPTION</th>
<th>LC</th>
<th>MLC</th>
<th>HC</th>
</tr>
</thead>
<tbody>
<tr>
<td>1.0</td>
<td>ADMINISTRATION, PLANNING, &amp; ENGINEERING</td>
<td>$46,080.00</td>
<td>$46,080.00</td>
<td>$46,080.00</td>
</tr>
<tr>
<td>2.0</td>
<td>MOBILIZATION &amp; PROJECT PREPARATION</td>
<td>$119,580.00</td>
<td>$119,580.00</td>
<td>$119,580.00</td>
</tr>
<tr>
<td>3.0</td>
<td>DEMOLITION/REMOVAL OF REPLACED MEMBERS</td>
<td>$57,656.25</td>
<td>$136,333.01</td>
<td>$372,363.28</td>
</tr>
<tr>
<td>4.0</td>
<td>REPLACEMENT MEMBER INSTALLATION</td>
<td>$50,052.58</td>
<td>$50,052.58</td>
<td>$50,052.58</td>
</tr>
<tr>
<td>5.0</td>
<td>EXISTING MEMBER REPAIR/REPLACEMENT</td>
<td>$7,874.09</td>
<td>$31,232.72</td>
<td>$372,363.28</td>
</tr>
<tr>
<td></td>
<td>TOTAL</td>
<td>$273,368.83</td>
<td>$359,919.68</td>
<td>$619,308.58</td>
</tr>
</tbody>
</table>

The estimated costs of completing the project range from an approximate total of $273,400 to $619,300. The most-likely cost of the project is estimated to be around $360,000.

5.0 **SAFETY CONSIDERATIONS**

The Vu Villa Structural Design and Rehabilitation presents a variety of challenges and one must consider the likely safety concerns that such a project may encounter. Section 5.0 discusses several broad safety topics which should be taken into consideration if such a project was to proceed to an execution phase. The remainder of Section 5.0 will discuss safety concerns and considerations that potentially could be encountered in the event that the Vu Villa Structural Design and Rehabilitation project were to be carried out.

5.1. **Code and Regulatory Safety**

The first opportunity to address safety, happens during the design phase of the project. The 2012 *IBC*, which was adhered to for the purposes of the design discussed in this report, states that its intent is to provide minimum requirements to ensure public health, safety, and general welfare (International Code Council, Inc., 2011). The 2012 *IBC*, acts as the foundation for each code book it references and therefore, ensuring a design is compliant with the proper applicable codes, is the first step to ensuring the safety of those impacted by a design (International Code Council, 2019).
For the structural design components of the Vu Villa Structural Design and Rehabilitation, the 2012 IBC guidelines were followed and the design is believed to be code compliant. If errors or omissions occurred to the design that were identified at a later date, proper action would be taken to correct them.

While the design did consider structural analysis and detailing methodologies permitted by the 2012 IBC, there exist several components to the proposed project that require further consideration. The 2012 IBC, in general, would need to address the all of the following International Code Council (ICC) requirements, including but not limited to:

- Use and Occupancy Requirements;
- All Fire Protection Related Requirements;
- Means of Egress;
- Accessibility;
- Energy Efficiency;
- Wood Construction Requirements;
- Electrical, Mechanical, Plumbing Requirements; and
- Existing Structure Requirements.

Many of the additional considerations discussed above would require code compliance specifically relating to the National Fire Protection Association 101 Life Safety Code (NFPA 101) (National Fire Protection Association, 2018). NFPA 101 states in Chapter 1.2 that its purpose is to provide minimum requirements to a buildings design, operation, and maintenance for protecting lives from fire (National Fire Protection Association, 2018). It is recommended that both the owner, and any professional working on a project related to this report give careful consideration to all applicable code requirements of the NFPA 101.

In addition, to the 2012 IBC and NFPA 101, as well as prior to execution of the project, it should be verified that all proposed work-related activities comply with the 2012 IEBC, as well.

### 5.2. Construction Safety

Once a design is code compliant and protective of public health, safety, and general welfare, the next consideration should be safety during the physical work to be performed. This is perhaps the greatest risk to people’s health and safety.

Several aspects that are likely to be encountered during construction activities include exposure to environmental contaminants both from Butte’s storied mining history as well as those present due to outdated construction practices. The potential for exposure to dust and particles which contain asbestos, heavy metals, and silica dust during both demolition and construction activities are high risks. Mitigation systems which can be used to limit the effects of these threats include:

- Isolation of potential contaminants from work areas;
- High-Efficiency Particulate Arrestance (HEPA) vacuums;
As it relates to fugitive dusts, the concerns do not strictly apply to the construction workers themselves. One must also consider the impact that construction related activities could have on the customers of the Vu Villa if the work was to be done in conjunction with typical bar and restaurant operations. Therefore, isolation and wetting practices should be considered to prevent dust and airborne particulates from making their way to, or accumulating in areas where customers may be exposed. Examples of methods which could be implemented to achieve these results include the use of plastic sheeting at entrance locations to a work area and consistent mopping and cleaning practices in the bar and restaurant while construction activities are taking place.

Another likely threat to the health and safety of construction workers and others that may be onsite is the potential for gravity related injuries. There exists a significant potential that during both demolition and construction activities, for materials and debris to fall and strike workers. Additionally, there is the potential for workers to be exposed to falls, this can occur when working at heights (workers on ladders or working close to edges) and due to failures of the existing structural components (workers falling through the flooring, sheathing, ceilings). Several remedial practices that can guard workers safety from gravity related threats are:

- Use of proper personal protective equipment (PPE), such as hard hats;
- Use of fall protection and restraint systems; and
- Worker site evaluations prior to accessing areas which seem structurally unsafe.

It is the recommendation of this report that only contractors who are qualified and experienced in demolition and rehabilitation work be used for implementing the Vu Villa Structural Design and Rehabilitation project, and that the highest priority be given to safety, and compliance with OSHA regulations.

5.3. Building Maintenance Safety

The final safety topic which warrants consideration related to the Vu Villa Structural Design and Rehabilitation project is building maintenance. In this report, building maintenance refers to the use, functionality, and general upkeep of the building, following the completion of construction activities. Among those concerns related to building maintenance are:

- Functionality of light bulbs and adequate lighting;
- Functionality and maintenance of fire alarms and carbon monoxide detectors;
- Meeting accessibility and escape requirements;
- Upkeep on safety devices; and
- Eliminating threats posed by slips, trips, and falls.
It is worth noting that many of the items discussed in this section are code-based requirements which original implementation and installation would be covered under Section 5.1. The maintenance tasks associated with any newly installed systems are crucial to maintaining the safety of occupants of the building. In order to address the concerns listed above, the owner or maintenance personnel should consider:

- Developing change-out schedules for light bulbs;
- Developing replacement schedules for fire alarms and carbon monoxide detectors;
- Inspecting exit signs and pathways periodically for reasonable passage along access and escape routes;
- Inspect and maintain safety devices, hand rails, and other support tools; and
- Practice good house-keeping techniques to maintain safe and open walking surfaces.

### 6.0 ADDITIONAL DESIGN CONSIDERATION

Due to time and resource constraints, the Vu Villa Structural Design and Rehabilitation project considered only a refined scope of the design that would be required to execute the objectives discussed within this report. In addition, there are many other aspects of the design which warrant further consideration and detailing. Section 6.0 discusses several of those topics that could and are being pursued.

#### 6.1. Structural Staircase Repair

The Vu Villa Sag Report conducted by BCE, Inc. and provided in Appendix F, originated in response to the owner’s concerns over the staircase located along the west exterior URM wall which was experiencing significant deflection. The report identified the removal of a load-bearing wall along the north side of the stairs as the cause for the sag (Beaudette Consulting Engineers, 2013). As an addendum to the Vu Villa Structural Design and Rehabilitation project, a group of Montana Tech undergraduate senior design students are completing an additional analysis and will propose two alternative designs of structural systems aimed at remedying the faltering staircase.

#### 6.2. URM Seismic Retrofit

As was briefly discussed in Section 4.3.3 above, based on the Design Calculations related to the project and the amount of information regarding URM structures vulnerability, there is reason to believe that, should a significant seismic event occur in the vicinity of Butte, there could be a potential threat to both the structural integrity of the Vu Villa as well the health and safety of its inhabitants. To better demonstrate the threat that exists to URM structures, one Federal Emergency Management Agency (FEMA) publication explains that based on statistics from several United States earthquakes with 4,457 URM buildings, five out of six were damaged enough for bricks to fall while one-fifth were damaged to the point of partial or complete collapse (Federal Emergency Management Agency, 2009).
It is the recommendation of this report, that as part of any design and rehabilitation work for the Vu Villa, seismic retrofitting be included within the scope of the project. Seismic retrofitting is defined as “the modification of an existing structure to improve resistance to seismic activity” (Jamal, 2017). There exist many useful resources that provide further information on seismic retrofitting for URM structures. A good reference for typical techniques related to seismic retrofitting is an article on Simpson Strong-Tie’s Structural Engineering Blog titled “Unreinforced Masonry (URM) Buildings: Seismic Retrofit” (Hensen, 2013). An additional recommendation of this report is that any retrofit systems be designed and work completed by experts with previous experience in seismic retrofitting.

6.3. Expert Evaluation of Existing Structural Components

Though outside of the scope of the Vu Villa Structural Design and Rehabilitation project, it is also a recommendation of this report that all existing components, including first-floor components and foundations, be inspected and evaluated for damage, flaws, or general wear from the aging of the building. This is suggested prior to the implementation of any new structural system.

7.0 CONCLUSION/DISCUSSION OF DESIGN

The Vu Villa Structural Design and Rehabilitation project specifically addressed three primary objectives:

- Design a system which remedies the existing sag of the second-floor;
- Design a system that replaces the existing roof- and second-floor structural members and accommodates the change in usage of the second-floor from apartments to an assembly area; and
- Preserve all existing structural members that do not require replacement.

The proposed design specified the locations, dimensions, and materials of nine Roof Beams and six 2nd Floor Columns which will adequately replace the existing structural framing system of the second-floor and support a change in usage to an assembly area. Similarly, the proposed design detailed new locations, dimensions, and materials of six 2nd Floor Beams and four 1st Floor Columns which will replace the existing beam and column system that is believed responsible for the sag conditions.

The design proposes limiting additional costs to the project by analysis and evaluation of other existing members such as joists, nails, sheathing, and the bearing/shear walls to ensure that based on the minimum criteria and analysis performed will adequately support a new system. In order to complete this proposed scope of work, it is estimated that the most-likely case cost of the project will be $360,000.

The intention is to submit the Vu Villa Structural Design and Rehabilitation project and report to the Vu Villa’s owner as supplemental information from which future analysis, design, and
construction can be based on. Please note, this project is strictly informative and is not an approved work product. Any work actually implemented for rehabilitation of the Vu Villa MUST be reviewed and stamped by a licensed engineering professional. For complete information of the design, drawings, and specifications of the Vu Villa Structural Design and Rehabilitation project refer to the appendices of this report.
References


https://www.wunderground.com/dashboard/pws/KMTBUTTE25?cm_ven=localwx_pwsdash
APPENDICES
VU VILLA
STRUCTURAL DESIGN & REHABILITATION PROJECT

PREPARED FOR:
LATIS CORPORATION
521 W. PARK STREET

PREPARED BY:
MONTANA TECHNOLOGICAL UNIVERSITY
1300 W. PARK STREET

GENERAL DRAWING LAYOUT:
1. GENERAL SHEETS
2. EXISTING SITE PLAN SHEETS
3. ARCHITECTURAL SHEETS
4. STRUCTURAL SHEETS
5. CROSS SECTION SHEETS
6. DETAIL SHEETS
REPLACEMENT MEMBER NOTES:

1. REPLACEMENT BEAMS, COLUMNS, AND CONNECTIONS SHALL CONFORM TO ALL PROVISIONS OUTLINED IN THE TECHNICAL SPECIFICATIONS.
2. CONTRACTOR IS SOLELY RESPONSIBLE FOR PROCUREMENT AND DELIVERY OF ALL SPECIFIED REPLACEMENT MATERIALS.
3. ALL Replacement BEAMS, COLUMNS, AND CONNECTIONS SHALL OF THE MATERIALS SPECFIED IN PART 2 OF THE TECHNICAL SPECIFICATIONS.
4. IF DESIGN A CONSTRUCTIBILITY REQUIREMENTS ARE NOT MET: REPLACEMENT BEAMS AND COLUMNS MAY BE 'SISTERED' WITH THEIR ASSOCIATED EXISTING MEMBERS.
5. BEAM SPLICE SHALL BE CENTERED ON SUPPORT COLUMNS AS SHOWN IN THE DRAWINGS.
6. COLUMNS SHALL BE CENTERED OVER THE SPICE CONDITION OF SUPPORT BEAMS WITH EQUAL WIDTH EXISTING ON EXISTING MEMBERS.
7. WHERE APPLICABLE, A MINIMUM OF CLEARANCE SHALL BE PROVIDED ON TOP, SIDS, AND ENDS OF WOOD MEMBERS WHICH HAVE BEEN PREVIOUSLY TREATED WHEN PLACED ON NEW CONCRETE OR WOODEN MEMBERS.
8. WHERE SPECIFIED, CONNECTIONS OF GLULAMS OR EXISTING WOOD BEAMS HEADERS SHALL BE INSTALLED SIMILAR TO GLULAM TO URM WALLS.
9. REPLACE ALL BEAMS AND COLUMNS SHALL BE PLACED, SQUARE, AND LEVEL TO WITHIN STANDARD CONSTRUCTION TOLERANCES AND TO THE PROVISIONS OF APPLICABLE CONSTRUCTION MANUALS.
10. CONTRACTOR SHALL PLACE BEAMS AND COLUMNS IN SUCH A WAY SO AS TO PREVENT MOVING OR PRESTRESSING OF THE MEMBERS PRIOR TO INSTALLATION.
11. NO REGIONS OF INTERSECTIONS SHALL BE MADE IN PLACE REPLACEMENT OF BEAMS OR COLUMNS AS SHOWN ON THE DRAWINGS OR AS INCLUDED IN THE TECHNICAL SPECIFICATIONS.
12. CONTRACTOR SHALL MAINTAIN EXISTING MEMBERS CONDITION IN SUCH A WAY SO AS TO ENSURE THAT REPLACEMENT BEAMS CAN BE INSTALLED AND CONNECTED WITHOUT CHANGING THE LOADS, SHAPES, OR STRENGTH OF THE EXISTING MEMBERS.
13. UNLESS OTHERWISE SPECIFIED, ALL BOLT HOLES SHALL BE DRILLED A MINIMUM OF 1/2" TO A MAXIMUM Z-LESS THAN THE THICKNESS OF THE REPLACEMENT BOLT.
14. ALL MACHINE BOLTS SPECIFIED IN THE DRAWINGS AND TECHNICAL SPECIFICATIONS SHAL BE MINIMUM GRADE 2 BOLTS.
15. ALL CONNECTION HARDWARE SPECIFIED SHALL BE FROM THE DESIGNATED MANUFACTURER UNLESS ENGINEER APPROVES AN EQUIVALENT ALTERNATIVE.

LOADING CONDITIONS:
1. CONTRACTOR SHALL STAGE A PLACE MATERIALS & EQUIPMENT IN SUCH A WAY SO AS TO ENSURE THAT ALOCATED LOGO SHALL CROSS THE EXISTING MEMBERS SPECIFIED BELOW MAY BE EXCEEDED, THE DESIGNER SHALL CONSIDER NULL AND VOID, AND THE ENGINEER SHALL BE NOTIFIED IMMEDIATELY.

SEA LOADS:
- ROOF (BEAMS): 17.5 PSF
- END FLOOR (BEAMS): 16.0 PSF
- END FLOOR (COLUMNS): 14.2 PSF
- END FLOOR: 14.2 PSF
- DIEGo: 68.6 PSF
- END FLOOR LIVE LOAD (BEAMS): 96.7 PSF
- END FLOOR LIVE LOAD (COLUMNS): 68.8 PSF
- SEISMIC LOADS:
- ROOF - GRAVITY: 1.0 PSF
- END FLOOR - GRAVITY: 2.1 PSF
- ROOF DIAPHRAGM - LATERAL: 566.0 PSF
- END FLOOR DIAPHRAGM - LATERAL: 1,598 PSF
- END FLOOR SHEAR WALLS - LATERAL: 1,716 PSF
- SNOW LOADS:
- ROOF: 26.8 PSF
- WIND LOADS:
- UPLIFT - VERTICAL: 20.6 PSF
- STORM (LATERAL): 22.3 PSF
- WIND - INTERIOR: 16.5 PSF

LEGEND:
- PROPOSED FEATURE
- ACH. - PROPOSED CONFERENCE AREA FLOORING
- ARCH. - SIDEWALK/CONCRETE COLUMN
- CONCRETE
- EXISTING NATIVE SUBGRADE
- GLULAM BEAM (CROSS-SECTION/PROFILE VIEW)
- GLULAM BEAM (PLAN VIEW)
- JOIST (PLAN VIEW)
- SIMPSON CONNECTIONS
- URMI WALL (PROFILE VIEW)
- URMI WALL (SECTION VIEW)
- WOOD-FRAMED WALL

AABBREVIATIONS:
- AEC: AMERICAN INSTITUTE OF ARCHITECTS
- AGS: AMERICAN GEOTECHNICAL SOCIETY
- AI: ASHRAE INTERNATIONAL
- AIAG: AMERICAN INSTITUTE OF ARCHITECTS
- ASCE: AMERICAN SOCIETY OF CIVIL ENGINEERS
- AWS: AMERICAN WELDING SOCIETY
- BSI: BRITISH STANDARD INSTITUE
- CSA: CANADIAN STANDARD ASSOCIATION
- DEW: DOUGLAS EVEN GRADE
- DOUG: DOUGLAS FIR/DOUGLAS FIR
- DOUGL: DOUGLAS FIR-LARCH (NORTH)
- EN: EUROPEAN STANDAR
- FFL: FIBER FACED LUMBER
- IFI: INSTITUTE FOR FOREST INDUSTRY
- IEBC: ILLINOIS ELECTRIC BOARD COMMUNITY
- IECC: ILLINOIS ELECTRIC CODE CONVENTION
- IFN: IRELAND FERTILITY NETWORK
- IUB: international building code
- IIBC: INTERNATIONAL BUILDING CODE
- ICD: INTERNATIONAL CODE COMMISSION
- IPI: INTERNATIONAL PROFESSIONAL INSTITUTIONS
- IOM: INTERNATIONAL ORGANIZATION FOR METROLOGY
- ISO: INTERNATIONAL ORGANIZATION FOR STANDARDIZATION
- JAS: JAPAN AGRICULTURAL STANDARDS
- KBC: KANSAS BOARD OF COMMISSIONERS
- KBE: KANSAS BOARD OF ENGINEERS
- KBEA: KANSAS BOARD OF ENGINEERS ASSOCIATION
- KSO: KANSAS STATE BOARD OF OBRADERS
- LBC: LUXEMBURG BUILDING CODE
- MBC: MONTANA BUILDING CODE
- MB: MONTANA BOARD
- MPCA: MONTANA PUBLIC SERVICE COMMISSION
- MRR: MONTANA RESEARCH RESOURCES
- NFC: NORTHERN CONCRETE WERBCOMMISSION
- NFPA: NATIONAL FIRE PROTECTION ASSOCIATION
- NIST: NATIONAL INSTITUTE OF STANDARDATION
- NSC: NATIONAL STANDARDS COUNCIL
- NSF: NATIONAL SCIENCE FOUNDATION
- NYS: NEW YORK STATE BOARD
- PMAC: PORTLAND MATERIALS ADVISORY COMMITTEE
- PMI: PROJECT MANAGEMENT INSTITUTE
- PNSC: PROJECT MANAGEMENT NETWORK
- QAI: QUALITY ASSURANCE INFORMATION
- RAB: RICHMOND AREA BOARD
- SAE: SOCIETY OF AUTOMOBILE ENGINEERS
- SDPWS: SOUTHERN DEVELOPMENT PLANNING WORKERS GROUP
- TMS: THE MINERAL SOCIETY
- USACE: U.S. ARMY CORPS OF ENGINEERS
- VU: VILLA DRAFT PRELIMINARY DESIGN

DEFERRED SUBMITTALS:
1. DESIGN OF CONNECTION DEVICES FOR FASTENING 1ST FLOOR COLUMNS TO FOUNDATION; 1ST FLOOR DIAPHRAGM SHALL BE SUBMITTED PRIOR TO FINAL DESIGN SUBMISSION.
2. DESIGN OF CONNECTION DEVICES FOR FASTENING 2ND FLOOR COLUMNS TO INTERIOR URMI WALLS SHALL BE SUBMITTED PRIOR TO FINAL DESIGN SUBMISSION.
3. DESIGN OF CONNECTIONS FOR EXISTING MEMBERS WHICH REQUIRE CHANGING THE LOADING, SHAPE, OR STRENGTH OF THE EXISTING MEMBERS SHALL BE COMPLETED ON AN AS-NEEDED BASIS.
4. DESIGN OF ReconCILING SYSTEMS FOR URMI WALLS SUSCEPTIBILITY TO SEISMIC LOADS SHALL BE COMPLETED UPON FURTHER DISCUSSION WITH OWNER.

SHEET INDEX:

1. PROPOSED SHEET
2. GENERAL NOTES, LEGEND, AND SHEET INDEX
3. 1ST FLOOR EXISTING PLAN VIEW
4. 2ND FLOOR EXISTING PLAN VIEW
5. ARCHITECTURAL ISOMETRIC VIEW
6. 1ST FLOOR CONSTRUCTION ISOMETRIC VIEW
7. 2ND FLOOR CONSTRUCTION ISOMETRIC VIEW
8. 1ST FLOOR ARCHITECTURAL PLAN
9. 2ND FLOOR ARCHITECTURAL PLAN
10. 1ST FLOOR STRUCTURAL PLAN
11. 2ND FLOOR STRUCTURAL PLAN
12. STRUCTURAL CROSS SECTION A
13. STRUCTURAL CROSS SECTION B
14. STRUCTURAL CROSS SECTION C
15. STRUCTURAL CROSS SECTION D
16. STRUCTURAL CROSS SECTION 1
17. STRUCTURAL CROSS SECTION 2
18. STRUCTURAL CROSS SECTION 3
19. STRUCTURAL CROSS SECTION 4
20. ROOF LEVEL STRUCTURAL PLAN
21. ROOF LEVEL CONNECTION DETAILS
22. ROOF LEVEL COLUMN DETAILS
23. 1ST FLOOR COLUMN DETAILS
24. 2ND FLOOR COLUMN DETAILS
25. 2ND FLOOR DIAPHRAGM MINIMUM CRITERIA DETAILS
26. 1ST FLOOR DIAPHRAGM MINIMUM CRITERIA DETAILS
27. 2ND FLOOR DIAPHRAGM MINIMUM CRITERIA DETAILS

SCALE:
- 1" = 20 FT

DATE: 10/26/19
NOTES:
1. IF REPLACEMET BEAMS AND COLUMNS CAN NOT BE SISTERED TO EXISTING MEMBERS, EXISTING BEAMS AND COLUMNS SHALL BE REMOVED.
2. CONTRACTOR SHALL MINIMIZE IMPACT TO THOSE EXISTING MEMBERS NOT DESIGNATED FOR DEMO ACCORDING TO THE TECHNICAL SPECIFICATIONS.
3. EXISTING JOISTS AND SHEATHING SHALL BE ASSESSED BY THE ENGINEER OR A COMPETENT PROFESSIONAL. IF EXISTING MEMBERS DO NOT MEET SPECIFIED REQUIREMENTS THE MEMBER SHALL BE REPLACED WITH A SIMILAR MEMBER MEETING THE STRUCTURAL REQUIREMENTS.
4. DESIGN BASED ON SURVEY BY OTHERS. ENGINEER ASSUMES NO RESPONSIBILITY FOR VALIDITY OF SURVEY.

EXISTING 1ST FLOOR PLAN

ALL EXISTING 1ST FLOOR MEMBERS WITH THE EXCEPTION OF COLUMNS SHALL BE PRESERVED.

PROPOSED 2ND FLOOR BEAM LOCATIONS (TYP.)

PROPOSED 1ST FLOOR COLUMN LOCATIONS (TYP.)

DESIGN TO ADDRESS BEAM SAG IN THESE APPROX. AREAS

STAIR DEFLECTION TO BE ADDRESSED BY OTHERS

APPROX. LOCATION OF EXISTING BEAM

APPROX. LOCATION OF EXISTING COLUMN (TYP.)

DEMO./REMOVE EXISTING BEAMS & COLUMNS THAT CAN NOT BE SISTERED TO REPLACEMENT MEMBERS

EXISTING -OISTS AND SHEATHING SHALL BE ASSESSED BY THE ENGINEER OR A COMPETENT PROFESSIONAL. IF EXISTING MEMBERS DO NOT MEET SPECIFIED REQUIREMENTS THE MEMBER SHALL BE REPLACED WITH A SIMILAR MEMBER MEETING THE STRUCTURAL REQUIREMENTS.

DESIGN BASED ON SURVEY BY OTHERS. ENGINEER ASSUMES NO RESPONSIBILITY FOR VALIDITY OF SURVEY.

ALL EXISTING 1ST FLOOR MEMBERS WITH THE EXCEPTION OF COLUMNS SHALL BE PRESERVED.

DRAFT
PRELIMINARY DESIGN
NOTES:
1. IF REPLACEMENT BEAMS AND COLUMNS CAN NOT BE SISTERSED TO EXISTING MEMBERS, EXISTING BEAMS AND COLUMNS SHALL BE REMOVED.
2. CONTRACTOR SHALL MINIMIZE IMPACT TO THOSE EXISTING MEMBERS NOT DESIGNATED FOR DEMO ACCORDING TO THE TECHNICAL SPECIFICATIONS.
3. EXISTING JOISTS AND SHEATHING SHALL BE ASSESSED BY THE ENGINEER OR A COMPETENT PROFESSIONAL. IF EXISTING MEMBERS DO NOT MEET SPECIFIED REQUIREMENTS THE MEMBER SHALL BE REPLACED WITH A SIMILAR MEMBER MEETING THE STRUCTURAL REQUIREMENTS.
4. DESIGN BASED ON SURVEY BY OTHERS. ENGINEER ASSUMES NO RESPONSIBILITY FOR VALIDITY OF SURVEY.
NOTE: ARCHITECTURAL DRAWINGS ARE PROVIDED STRICTLY FOR AESTHETIC PURPOSES. FOR ACTUAL ARCHITECTURAL DESIGN, IT IS STRONGLY RECOMMENDED THAT OWNER CONSULT A LICENSED ARCHITECT.

PROJECT SITE LOCATION:
VU VILLA (521 W. PARK ST.)

VU VILLA ISOMETRIC VIEW
NOTE: ARCHITECTURAL DRAWINGS ARE PROVIDED STRICTLY FOR AESTHETIC PURPOSES. FOR ACTUAL ARCHITECTURAL DESIGN, IT IS STRONGLY RECOMMENDED THAT OWNER CONSULT A LICENSED ARCHITECT.

1ST FLOOR ISOMETRIC VIEW

WEST BAY - BAR SIDE

PROPOSED 1ST FLOOR COLUMNS

EAST BAY - PIZZA SIDE

SLAB-ON-GRADE FOUNDATION

RUBBLE-STONE FOUNDATION

1ST FLOOR ISOMETRIC VIEW
NOTE: ARCHITECTURAL DRAWINGS ARE PROVIDED STRICTLY FOR AESTHETIC PURPOSES. FOR ACTUAL ARCHITECTURAL DESIGN, IT IS STRONGLY RECOMMENDED THAT OWNER CONSULT A LICENSED ARCHITECT.
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CONSIDER PROVIDING SECOND EXIT ROUTE TO ADDRESS POTENTIAL FIRE CODE DEFICIENCIES

PROPOSED CONFERENCE AREA

PROPOSED MEN'S RESTROOM

PROPOSED WOMEN'S RESTROOM

PROPOSED WET-BAR

DEMO UPSTAIRS APARTMENTS AND CONSTRUCT OPEN-FLOOR PLAN CONCEPT

2ND FLOOR ARCHITECTURAL PLAN

MONTANA TECHNOLOGICAL UNIVERSITY

VU VILLA

STRUCTURAL DESIGN & REHABILITATION

3/32" = 1'-0"

NOTE: ARCHITECTURAL DRAWINGS ARE PROVIDED STRICTLY FOR AESTHETIC PURPOSES. FOR ACTUAL ARCHITECTURAL DESIGN, IT IS STRONGLY RECOMMENDED THAT OWNER CONSULT A LICENSED ARCHITECT.

CONSIDER PROVIDING SECOND EXIT ROUTE TO ADDRESS POTENTIAL FIRE CODE DEFICIENCIES

PROPOSED CONFERENCE AREA

PROPOSED MEN'S RESTROOM

PROPOSED WOMEN'S RESTROOM

PROPOSED WET-BAR

DEMO UPSTAIRS APARTMENTS AND CONSTRUCT OPEN-FLOOR PLAN CONCEPT

2ND FLOOR ARCHITECTURAL PLAN

MONTANA TECHNOLOGICAL UNIVERSITY

VU VILLA

STRUCTURAL DESIGN & REHABILITATION

3/32" = 1'-0"

NOTE: ARCHITECTURAL DRAWINGS ARE PROVIDED STRICTLY FOR AESTHETIC PURPOSES. FOR ACTUAL ARCHITECTURAL DESIGN, IT IS STRONGLY RECOMMENDED THAT OWNER CONSULT A LICENSED ARCHITECT.

CONSIDER PROVIDING SECOND EXIT ROUTE TO ADDRESS POTENTIAL FIRE CODE DEFICIENCIES

PROPOSED CONFERENCE AREA

PROPOSED MEN'S RESTROOM

PROPOSED WOMEN'S RESTROOM

PROPOSED WET-BAR

DEMO UPSTAIRS APARTMENTS AND CONSTRUCT OPEN-FLOOR PLAN CONCEPT

2ND FLOOR ARCHITECTURAL PLAN

MONTANA TECHNOLOGICAL UNIVERSITY

VU VILLA

STRUCTURAL DESIGN & REHABILITATION

3/32" = 1'-0"

NOTE: ARCHITECTURAL DRAWINGS ARE PROVIDED STRICTLY FOR AESTHETIC PURPOSES. FOR ACTUAL ARCHITECTURAL DESIGN, IT IS STRONGLY RECOMMENDED THAT OWNER CONSULT A LICENSED ARCHITECT.

CONSIDER PROVIDING SECOND EXIT ROUTE TO ADDRESS POTENTIAL FIRE CODE DEFICIENCIES

PROPOSED CONFERENCE AREA

PROPOSED MEN'S RESTROOM

PROPOSED WOMEN'S RESTROOM

PROPOSED WET-BAR

DEMO UPSTAIRS APARTMENTS AND CONSTRUCT OPEN-FLOOR PLAN CONCEPT

2ND FLOOR ARCHITECTURAL PLAN

MONTANA TECHNOLOGICAL UNIVERSITY

VU VILLA

STRUCTURAL DESIGN & REHABILITATION

3/32" = 1'-0"
NOTES:
1. ALL EXISTING 1ST FLOOR MEMBERS, WITH THE EXCEPTION OF COLUMNS, SHALL BE PRESERVED.
2. CONTRACTOR SHALL MAKE EVERY EFFORT TO LIMIT IMPACT TO ALL EXISTING MEMBERS UNLESS OTHERWISE NOTED IN THE DRAWINGS OR TECHNICAL SPECIFICATIONS.
3. REPLACEMENT COLUMNS SHALL BE PLUMB, SQUARE, AND LEVEL TO WITHIN STANDARD ACCEPTABLE TOLERANCES AND TO THE PROVISIONS OF APPLICABLE CONSTRUCTION MANUALS.

1. INSTALL 10" X 10" SELECT STRUCTURAL DFL(N) COLUMNS (TYP.) SEE CS SHEETS

2. DO NOT CONNECT BEAM TO NON-LOAD BEARING WALLS

3. EXISTING RUBBLE STONE FOUNDATION

4. EXISTING URM WALL

5. EXISTING SLAB-ON-GRADE FOUNDATION

6. EXISTING WOOD-FRAMED WALLS (TYP.)

7. EXISTING WOOD-FRAMED BEARING/WALL SUPPORTING 2ND FLOOR JOISTS

8. EXISTING 10" X 10" SELECT STRUCTURAL DFL(N) COLUMNS (TYP.) SEE CS SHEETS

9. INSTALL 10" SELECT STRUCTURAL DFL(N) COLUMNS (TYP.) SEE CS SHEETS

10. EXISTING WOOD-FRAMED BEARING/WALL SUPPORTING 2ND FLOOR JOISTS

11. EXISTING SLAB-ON-GRADE FOUNDATION

12. EXISTING URM WALL
NOTES:
1. DEMOLITION OF EXISTING 2ND FLOOR FRAMING SHALL BE CONDUCTED PREVIOUS-TO OR SIMULTANEOUSLY WITH INSTALLATION OF REPLACEMENT MEMBERS.
2. CONTRACTOR SHALL MAKE EVERY EFFORT TO LIMIT IMPACT TO ALL EXISTING MEMBERS UNLESS OTHERWISE NOTED IN THE DRAWINGS OR TECHNICAL SPECIFICATIONS.
3. REPLACEMENT BEAMS & COLUMNS SHALL BE PLUMB, SQUARE, AND LEVEL TO WITHIN STANDARD ACCEPTABLE TOLERANCES AND TO THE PROVISIONS OF APPLICABLE CONSTRUCTION MANUALS.

EXISTING EXTERIOR URM WALLS (TYP.)
EXISTING 2ND FLOOR J O I N T S @ 1 6 " O . C . ( T Y P . )
INSTALL 6" X 6" SELECT STRUCTURAL DFL(N) COLUMNS (TYP.)
SEE CS SHEETS

EXISTING INTERIOR URM WALL

EXISTING BALCONY

INSTALL 6 3/4" X 24" 24F-V4 DF/DF GLULAMS (TYP.)
SEE CS SHEETS

PROPOSED NON-LOAD BEARING FRAMING FOR 2ND FLOOR BATHROOMS (TYP.)

DO NOT CONNECT BEAM TO NON-LOADING BEARING WALLS

STAIR DEFLECTION SUPPORT DESIGN BY OTHERS
NOTES:
1. DEMOLITION OF EXISTING 2ND FLOOR FRAMING SHALL BE CONDUCTED PREVIOUS-TO OR SIMULTANEOUSLY WITH INSTALLATION OF REPLACEMENT MEMBERS.
2. CONTRACTOR SHALL MAKE EVERY EFFORT TO LIMIT IMPACT TO ALL EXISTING MEMBERS UNLESS OTHERWISE NOTED IN THE DRAWINGS OR TECHNICAL SPECIFICATIONS.
3. REPLACEMENT BEAMS SHALL BE SQUARE AND LEVEL TO WITHIN STANDARD ACCEPTABLE TOLERANCES AND TO THE PROVISIONS OF APPLICABLE CONSTRUCTION MANUALS.

EXISTING ROOF JOISTS @ 16" O.C. (TYP.)
INSTALL 5 1/2" X 21" 24F-V4 DF/DF GLULAMS (TYP.)
SEE CS SHEETS

EXISTING URM WALL (TYP.)

2ND FLOOR STRUCTURAL PLAN
NOTES:

1. ALL EXISTING EXTERIOR MEMBERS (I.E. WALLS, COLUMNS, ETC.) SHALL BE PRESERVED BY THE CONTRACTOR.
   CONTRACTOR SHALL MAKE EVERY EFFORT TO LIMIT IMPACT TO ALL EXISTING MEMBERS UNLESS OTHERWISE
   NOTED IN THE DRAWINGS OR TECHNICAL SPECIFICATIONS.

Scale

Date

Drawn by

Designed by

Checked by

Montana Technological University

VU VILLA

STRUCTURAL DESIGN & REHABILITATION

3/16" = 1'-0"

CS1
NOTES:
1. CONTRACTOR SHALL MAKE EVERY EFFORT TO LIMIT IMPACT TO ALL EXISTING MEMBERS UNLESS OTHERWISE NOTED IN THE DRAWINGS OR TECHNICAL SPECIFICATIONS.
2. REPLACEMENT BEAMS & COLUMNS SHALL BE PLUMB, SQUARE, AND LEVEL TO WITHIN STANDARD ACCEPTABLE TOLERANCES AND TO THE PROVISIONS OF APPLICABLE CONSTRUCTION MANUALS.

1st Floor
0' - 0"  

2nd Floor
16' - 0"

Roof
28' - 0"

EXISTING URM WALLS (TYP.)
RUBBLE - STONE FOUNDATION (TYP.)

1st Offset
3' - 0"

1st Floor
0' - 0"

2nd Floor Beam-Column Connection (TYP.)
See Detail

SECTION B

INSTALL 6 3/4" X 24" 24F-V4 DF/DF GLULAMS
SEE DETAIL

INSTALL 5 1/2" X 21" 24F-V4 DF/DF GLULAMS
SEE DETAIL

INSTALL 6" X 6" SELECT STRUCTURAL DFL(N) COLUMNS
SEE DETAIL

INSTALL 10" X 10" SELECT STRUCTURAL DFL(N) COLUMNS
SEE DETAIL

CS2

Montana Technological University

STRUCTURAL DESIGN & REHABILITATION

VU VILLA

3/16" = 1'-0"

Date
10/25/19

Drawn by
KBS

Designed by
KBS

Checked by
BMK
NOTES:
1. CONTRACTOR SHALL MAKE EVERY EFFORT TO LIMIT IMPACT TO ALL EXISTING MEMBERS UNLESS OTHERWISE NOTED IN THE DRAWINGS OR TECHNICAL SPECIFICATIONS.
2. REPLACEMENT BEAMS & COLUMNS SHALL BE PLUMB, SQUARE, AND LEVEL TO WITHIN STANDARD ACCEPTABLE TOLERANCES AND TO THE PROVISIONS OF APPLICABLE CONSTRUCTION MANUALS.

SECTION C

1. INSTALL 6 3/4" X 24" 24F-V4 DF/DF GLULAMS SEE DETAIL
2. INSTALL 5 1/3" X 21" 24F-V4 DF/DF GLULAMS SEE DETAIL
3. INSTALL 6 3/4" X 24" 24F-V4 DF/DF GLULAMS SEE DETAIL

NOTES:
1. CONTRACTOR SHALL MAKE EVERY EFFORT TO LIMIT IMPACT TO ALL EXISTING MEMBERS UNLESS OTHERWISE NOTED IN THE DRAWINGS OR TECHNICAL SPECIFICATIONS.
2. REPLACEMENT BEAMS & COLUMNS SHALL BE PLUMB, SQUARE, AND LEVEL TO WITHIN STANDARD ACCEPTABLE TOLERANCES AND TO THE PROVISIONS OF APPLICABLE CONSTRUCTION MANUALS.

SCALE 3/16" = 1'-0"
NOTES:
1. ALL EXISTING EXTERIOR MEMBERS (I.E. WALLS, COLUMNS, ETC.) SHALL BE PRESERVED BY THE CONTRACTOR. CONTRACTOR SHALL MAKE EVERY EFFORT TO LIMIT IMPACT TO ALL EXISTING MEMBERS UNLESS OTHERWISE NOTED IN THE DRAWINGS OR TECHNICAL SPECIFICATIONS.
2. MINIMUM 1/2" OF CLEARANCE SHALL BE PROVIDED ON TOP, SIDES, AND ENDS OF WOOD MEMBERS WHICH HAVE NOT BEEN PRESERVATIVELY TREATED WHEN PLACED ON OR NEAR CONCRETE OR MASONRY MEMBERS.

CONSIDER FURTHER EVALUATION & DESIGN TO ADDRESS PARAPET & URM WALLS SUSCEPTIBILITY TO EARTHQUAKE LOADS

SECTION 1
NOTES:
1. CONTRACTOR SHALL MAKE EVERY EFFORT TO LIMIT IMPACT TO ALL EXISTING MEMBERS UNLESS OTHERWISE NOTED IN THE DRAWINGS OR TECHNICAL SPECIFICATIONS.
2. REPLACEMENT BEAMS & COLUMNS SHALL BE PLUMB, SQUARE, AND LEVEL TO WITHIN STANDARD ACCEPTABLE TOLERANCES AND TO THE PROVISIONS OF APPLICABLE CONSTRUCTION MANUALS.
3. WHERE SPECIFIED, CONNECTIONS OF GLULAMS TO EXISTING WOOD BEAM HEADERS SHALL BE INSTALLED SIMILAR TO GLULAMS TO URM WALLS, SEE SHEET D9 FOR INSTALLATION DETAILS.
4. MINIMUM 1/2" OF CLEARANCE SHALL BE PROVIDED ON TOP, SIDES, AND ENDS OF WOOD MEMBERS WHICH HAVE NOT BEEN PRESERVATIVELY TREATED WHEN PLACED ON OR NEAR CONCRETE OR MASONRY MEMBERS.
NOTES:
1. CONTRACTOR SHALL MAKE EVERY EFFORT TO LIMIT IMPACT TO ALL EXISTING MEMBERS UNLESS OTHERWISE NOTED IN THE DRAWINGS OR TECHNICAL SPECIFICATIONS.
2. REPLACEMENT BEAMS & COLUMNS SHALL BE PLUMB, SQUARE, AND LEVEL TO WITHIN STANDARD ACCEPTABLE TOLERANCES AND TO THE PROVISIONS OF APPLICABLE CONSTRUCTION MANUALS.
3. WHERE SPECIFIED, CONNECTIONS OF GLULAMS TO EXISTING WOOD BEAM HEADERS SHALL BE INSTALLED SIMILAR TO GLULAMS TO URM WALLS, SEE SHEET D9 FOR INSTALLATION DETAILS.
4. MINIMUM 1/2" OF CLEARANCE SHALL BE PROVIDED ON TOP, SIDES, AND ENDS OF WOOD MEMBERS WHICH HAVE NOT BEEN PRESERVATIVELY TREATED WHEN PLACED ON OR NEAR CONCRETE OR MASONRY MEMBERS.

INSTALL 5 1/2" X 21" 24F-V4 DF/DF GLULAMS (TYP.) SEE DETAIL

ROOF LEVEL BEAM-WALL CONNECTIONS (TYP.) SEE DETAIL

EXISTING 2ND FLOOR JOISTS @ 16" O.C.

EXISTING URM WALL (TYP.)

ENGINEER/COMPETENT PROFESSIONAL TO ASSESS ROOF & 2ND FLOOR DIAPHRAGMS FOR STRUCTURAL ADEQUACY (SEE TECHNICAL SPECIFICATIONS)

EXISTING NON-LOAD BEARING WALL

PLACE 5 1/2" X 21" 24F-V4 DF/DF GLULAMS (TYP.) SEE DETAIL

EXISTING 2ND FLOOR JOISTS @ 16" O.C.

TOE-NAIL BEAM-JOIST CONNECTIONS WITH 8d COMMON NAILS (TYP.) SEE DETAIL

INSTALL 6" X 6" SELECT STRUCTURAL DFL(N) COLUMNS (TYP.) SEE DETAIL

SCALE 3/16" = 1'-0"

DRAFT PRELIMINARY DESIGN

Montana Technological University

DATE: 10/25/19
DRAWN BY: KBS
DESIGNED BY: KBS
CHECKED BY: BMK

VU VILLA STRUCTURAL DESIGN & REHABILITATION
NOTES:

1. CONTRACTOR SHALL MAKE EVERY EFFORT TO LIMIT IMPACT TO ALL EXISTING MEMBERS UNLESS OTHERWISE
   NOTED IN THE DRAWINGS OR TECHNICAL SPECIFICATIONS.
2. REPLACEMENT BEAMS & COLUMNS SHALL BE PLUMB, SQUARE, AND LEVEL TO WITHIN STANDARD
   ACCEPTABLE TOLERANCES AND TO THE PROVISIONS OF APPLICABLE CONSTRUCTION MANUALS.
3. WHERE SPECIFIED, CONNECTIONS OF GLULAM TO EXISTING WOOD BEAM HEADERS SHALL BE INSTALLED
   SIMILAR TO GLULAM TO URM WALLS, SEE SHEET D9 FOR INSTALLATION DETAILS.
4. MINIMUM 1/2" OF CLEARANCE SHALL BE PROVIDED ON TOP, SIDES, AND ENDS OF WOOD MEMBERS WHICH
   HAVE NOT BEEN PRESERVATIVELY TREATED WHEN PLACED ON OR NEAR CONCRETE OR MASONRY MEMBERS.

DRAWING SHEET 4

SECTION 4

ROOF LEVEL BEAM-WALL
CONNECTIONS (TYP.)
SEE DETAIL

EXISTING URM WALL (TYP.)

EXISTING NON-LOAD
BEARING INTERIOR WALL

INSTALL 6" X 6" SELECT
STRUCTURAL DFL(N)
COLUMNS (TYP.)
SEE DETAIL

TOE-NAIL BEAM-JOIST
CONNECTIONS WITH
8d COMMON NAILS (TYP.)
SEE DETAIL

ROOF LEVEL BEAM-COLUMN
CONNECTIONS (TYP.)
SEE DETAIL

EXISTING 1ST FLOOR
JOIST SYSTEM

EXISTING NATIVE
SUBGRADE MATERIAL (TYP.)

INSTALL 6 3/4" X 24" 24F-V4
DF/DF GLULAMS (TYP.)
SEE DETAIL

TOE-NAIL BEAM-JOIST
CONNECTIONS WITH
10d COMMON NAILS (TYP.)
SEE DETAIL

INSTALL 10" X 10" SELECT
STRUCTURAL DFL(N) COLUMNS
SEE DETAIL

INSTALL 10" X 21" 24F-V4
DF/DF GLULAMS (TYP.)
SEE DETAIL

EXISTING 1ST FLOOR
JOIST SYSTEM

EXISTING NATIVE
SUBGRADE MATERIAL (TYP.)

INSTALL 5 1/2" X 21" 24F-V4
DF/DF GLULAMS (TYP.)
SEE DETAIL

INSTALL CONNECTION TO
WOOD BEAM HEADER
CENTERED OVER
DOORFRAME AS SHOWN
(SEE NOTE 3)
NOTE:
1. IF DESIGN AND CONTRACTIBILTY REQUIREMENTS PERMIT, REPLACEMENT BEAMS MAY BE SISTED TO EXISTING BEAMS BY CONNECTIONS LATER SPECIFIED BY THE ENGINEER.
2. BEAMS SHALL BE SQUARE AND LEVEL WITHIN STANDARD CONSTRUCTION PRACTICES AND APPLICABLE CONSTRUCTION MANUALS.
3. CONTRACTOR SHALL NOT ALTER, INCISE, OR INSTALL BEAMS IN ANY WAY OTHER THAN AS STIPULATED IN THE DRAWINGS AND TECHNICAL SPECIFICATIONS.

PROPOSED 2ND FLOOR COLUMN SEE SHEET D3
PLACE & ORIENT ROOF BEAMS AS SHOWN ON SHEET S3
INSTALL 2ND FLOOR COLUMN PRIOR TO BEAM INSTALLATION (SEE SHEET D2)
EXISTING ROOF SHEATHING
EXISTING ROOF JOISTS (TYP.)

PROPOSED 5 1/2" X 21" 24F-V4 DF/DF GLULAM
PLACE BEAM SPLICE CENTERED ON 2ND FLOOR COLUMN

NOTES:

1. IF DESIGN AND CONTRACTIBILITY REQUIREMENTS PERMIT, REPLACEMENT BEAMS MAY BE SISTED TO EXISTING BEAMS BY CONNECTIONS LATER SPECIFIED BY THE ENGINEER.
2. BEAMS SHALL BE SQUARE AND LEVEL WITHIN STANDARD CONSTRUCTION PRACTICES AND APPLICABLE CONSTRUCTION MANUALS.
3. CONTRACTOR SHALL NOT ALTER, INCISE, OR INSTALL BEAMS IN ANY WAY OTHER THAN AS STIPULATED IN THE DRAWINGS AND TECHNICAL SPECIFICATIONS.

PROPOSED 5 1/2" X 21" 24F-V4 DF/DF GLULAM
PLACE BEAM SPLICE CENTERED ON 2ND FLOOR COLUMN

NOTES:

1. IF DESIGN AND CONTRACTIBILITY REQUIREMENTS PERMIT, REPLACEMENT BEAMS MAY BE SISTED TO EXISTING BEAMS BY CONNECTIONS LATER SPECIFIED BY THE ENGINEER.
2. BEAMS SHALL BE SQUARE AND LEVEL WITHIN STANDARD CONSTRUCTION PRACTICES AND APPLICABLE CONSTRUCTION MANUALS.
3. CONTRACTOR SHALL NOT ALTER, INCISE, OR INSTALL BEAMS IN ANY WAY OTHER THAN AS STIPULATED IN THE DRAWINGS AND TECHNICAL SPECIFICATIONS.

PROPOSED 5 1/2" X 21" 24F-V4 DF/DF GLULAM
PLACE BEAM SPLICE CENTERED ON 2ND FLOOR COLUMN

NOTES:

1. IF DESIGN AND CONTRACTIBILITY REQUIREMENTS PERMIT, REPLACEMENT BEAMS MAY BE SISTED TO EXISTING BEAMS BY CONNECTIONS LATER SPECIFIED BY THE ENGINEER.
2. BEAMS SHALL BE SQUARE AND LEVEL WITHIN STANDARD CONSTRUCTION PRACTICES AND APPLICABLE CONSTRUCTION MANUALS.
3. CONTRACTOR SHALL NOT ALTER, INCISE, OR INSTALL BEAMS IN ANY WAY OTHER THAN AS STIPULATED IN THE DRAWINGS AND TECHNICAL SPECIFICATIONS.

PROPOSED 5 1/2" X 21" 24F-V4 DF/DF GLULAM
PLACE BEAM SPLICE CENTERED ON 2ND FLOOR COLUMN

NOTES:

1. IF DESIGN AND CONTRACTIBILITY REQUIREMENTS PERMIT, REPLACEMENT BEAMS MAY BE SISTED TO EXISTING BEAMS BY CONNECTIONS LATER SPECIFIED BY THE ENGINEER.
2. BEAMS SHALL BE SQUARE AND LEVEL WITHIN STANDARD CONSTRUCTION PRACTICES AND APPLICABLE CONSTRUCTION MANUALS.
3. CONTRACTOR SHALL NOT ALTER, INCISE, OR INSTALL BEAMS IN ANY WAY OTHER THAN AS STIPULATED IN THE DRAWINGS AND TECHNICAL SPECIFICATIONS.

PROPOSED 5 1/2" X 21" 24F-V4 DF/DF GLULAM
PLACE BEAM SPLICE CENTERED ON 2ND FLOOR COLUMN
NOTES:
1. IF DESIGN AND CONTRACTIBILITY REQUIREMENTS PERMIT, REPLACEMENT BEAMS MAY BE SISTERED TO EXISTING BEAMS BY CONNECTIONS LATER SPECIFIED BY THE ENGINEER.
2. BEAMS SHALL BE SQUARE AND LEVEL WITHIN STANDARD CONSTRUCTION PRACTICES AND APPROPRIATE CONSTRUCTION MANUALS.
3. CONTRACTOR SHALL NOT ALTER, MOVE, OR INSTALL BEAMS IN ANY WAY OTHER THAN AS STIPULATED IN THE DRAWINGS AND TECHNICAL SPECIFICATIONS.

EXISTING 2ND FLOOR JOISTS & SHEATHING

PROPOSED 6 2/3" X 24" 24F-V4 DF/DF GLULAM

PLACE 2ND FLOOR BEAM SPLICE CENTERED ON PROPOSED 1ST FLOOR COLUMN

PROPOSED 2ND FLOOR COLUMN SEE SHEET D3

PROPOSED 1ST FLOOR COLUMN SEE SHEET D4

PLACE 2ND FLOOR SHEATHING CENTERED ON PROPOSED 1ST FLOOR COLUMN

EXISTING 2ND FLOOR SHEATHING

PLACE & ORIENT 2ND FLOOR BEAMS AS SHOWN ON SHEET S2

INSTALL 1ST FLOOR COLUMN PRIOR TO 2ND FLOOR BEAM INSTALLATION

NOTES:
1. IF DESIGN AND CONTRACTIBILITY REQUIREMENTS PERMIT, REPLACEMENT BEAMS MAY BE SISTERED TO EXISTING BEAMS BY CONNECTIONS LATER SPECIFIED BY THE ENGINEER.
2. BEAMS SHALL BE SQUARE AND LEVEL WITHIN STANDARD CONSTRUCTION PRACTICES AND APPROPRIATE CONSTRUCTION MANUALS.
3. CONTRACTOR SHALL NOT ALTER, MOVE, OR INSTALL BEAMS IN ANY WAY OTHER THAN AS STIPULATED IN THE DRAWINGS AND TECHNICAL SPECIFICATIONS.

PLACE 2ND FLOOR BEAM SPLICE CENTERED ON PROPOSED 1ST FLOOR COLUMN

PROPOSED 2ND FLOOR COLUMN SEE SHEET D3

PROPOSED 1ST FLOOR COLUMN SEE SHEET D4

PLACE & ORIENT 2ND FLOOR BEAMS AS SHOWN ON SHEET S2

INSTALL 1ST FLOOR COLUMN PRIOR TO 2ND FLOOR BEAM INSTALLATION

NOTES:
1. IF DESIGN AND CONTRACTIBILITY REQUIREMENTS PERMIT, REPLACEMENT BEAMS MAY BE SISTERED TO EXISTING BEAMS BY CONNECTIONS LATER SPECIFIED BY THE ENGINEER.
2. BEAMS SHALL BE SQUARE AND LEVEL WITHIN STANDARD CONSTRUCTION PRACTICES AND APPROPRIATE CONSTRUCTION MANUALS.
3. CONTRACTOR SHALL NOT ALTER, MOVE, OR INSTALL BEAMS IN ANY WAY OTHER THAN AS STIPULATED IN THE DRAWINGS AND TECHNICAL SPECIFICATIONS.

PLACE 2ND FLOOR BEAM SPLICE CENTERED ON PROPOSED 1ST FLOOR COLUMN

PROPOSED 2ND FLOOR COLUMN SEE SHEET D3

PROPOSED 1ST FLOOR COLUMN SEE SHEET D4

PLACE & ORIENT 2ND FLOOR BEAMS AS SHOWN ON SHEET S2

INSTALL 1ST FLOOR COLUMN PRIOR TO 2ND FLOOR BEAM INSTALLATION

NOTES:
1. IF DESIGN AND CONTRACTIBILITY REQUIREMENTS PERMIT, REPLACEMENT BEAMS MAY BE SISTERED TO EXISTING BEAMS BY CONNECTIONS LATER SPECIFIED BY THE ENGINEER.
2. BEAMS SHALL BE SQUARE AND LEVEL WITHIN STANDARD CONSTRUCTION PRACTICES AND APPROPRIATE CONSTRUCTION MANUALS.
3. CONTRACTOR SHALL NOT ALTER, MOVE, OR INSTALL BEAMS IN ANY WAY OTHER THAN AS STIPULATED IN THE DRAWINGS AND TECHNICAL SPECIFICATIONS.

PLACE 2ND FLOOR BEAM SPLICE CENTERED ON PROPOSED 1ST FLOOR COLUMN

PROPOSED 2ND FLOOR COLUMN SEE SHEET D3

PROPOSED 1ST FLOOR COLUMN SEE SHEET D4

PLACE & ORIENT 2ND FLOOR BEAMS AS SHOWN ON SHEET S2

INSTALL 1ST FLOOR COLUMN PRIOR TO 2ND FLOOR BEAM INSTALLATION

NOTES:
1. IF DESIGN AND CONTRACTIBILITY REQUIREMENTS PERMIT, REPLACEMENT BEAMS MAY BE SISTERED TO EXISTING BEAMS BY CONNECTIONS LATER SPECIFIED BY THE ENGINEER.
2. BEAMS SHALL BE SQUARE AND LEVEL WITHIN STANDARD CONSTRUCTION PRACTICES AND APPROPRIATE CONSTRUCTION MANUALS.
3. CONTRACTOR SHALL NOT ALTER, MOVE, OR INSTALL BEAMS IN ANY WAY OTHER THAN AS STIPULATED IN THE DRAWINGS AND TECHNICAL SPECIFICATIONS.

PLACE 2ND FLOOR BEAM SPLICE CENTERED ON PROPOSED 1ST FLOOR COLUMN

PROPOSED 2ND FLOOR COLUMN SEE SHEET D3

PROPOSED 1ST FLOOR COLUMN SEE SHEET D4

PLACE & ORIENT 2ND FLOOR BEAMS AS SHOWN ON SHEET S2

INSTALL 1ST FLOOR COLUMN PRIOR TO 2ND FLOOR BEAM INSTALLATION

NOTES:
1. IF DESIGN AND CONTRACTIBILITY REQUIREMENTS PERMIT, REPLACEMENT BEAMS MAY BE SISTERED TO EXISTING BEAMS BY CONNECTIONS LATER SPECIFIED BY THE ENGINEER.
2. BEAMS SHALL BE SQUARE AND LEVEL WITHIN STANDARD CONSTRUCTION PRACTICES AND APPROPRIATE CONSTRUCTION MANUALS.
3. CONTRACTOR SHALL NOT ALTER, MOVE, OR INSTALL BEAMS IN ANY WAY OTHER THAN AS STIPULATED IN THE DRAWINGS AND TECHNICAL SPECIFICATIONS.

PLACE 2ND FLOOR BEAM SPLICE CENTERED ON PROPOSED 1ST FLOOR COLUMN

PROPOSED 2ND FLOOR COLUMN SEE SHEET D3

PROPOSED 1ST FLOOR COLUMN SEE SHEET D4

PLACE & ORIENT 2ND FLOOR BEAMS AS SHOWN ON SHEET S2

INSTALL 1ST FLOOR COLUMN PRIOR TO 2ND FLOOR BEAM INSTALLATION

NOTES:
1. IF DESIGN AND CONTRACTIBILITY REQUIREMENTS PERMIT, REPLACEMENT BEAMS MAY BE SISTERED TO EXISTING BEAMS BY CONNECTIONS LATER SPECIFIED BY THE ENGINEER.
2. BEAMS SHALL BE SQUARE AND LEVEL WITHIN STANDARD CONSTRUCTION PRACTICES AND APPROPRIATE CONSTRUCTION MANUALS.
3. CONTRACTOR SHALL NOT ALTER, MOVE, OR INSTALL BEAMS IN ANY WAY OTHER THAN AS STIPULATED IN THE DRAWINGS AND TECHNICAL SPECIFICATIONS.

PLACE 2ND FLOOR BEAM SPLICE CENTERED ON PROPOSED 1ST FLOOR COLUMN

PROPOSED 2ND FLOOR COLUMN SEE SHEET D3

PROPOSED 1ST FLOOR COLUMN SEE SHEET D4

PLACE & ORIENT 2ND FLOOR BEAMS AS SHOWN ON SHEET S2

INSTALL 1ST FLOOR COLUMN PRIOR TO 2ND FLOOR BEAM INSTALLATION

NOTES:
1. IF DESIGN AND CONTRACTIBILITY REQUIREMENTS PERMIT, REPLACEMENT BEAMS MAY BE SISTERED TO EXISTING BEAMS BY CONNECTIONS LATER SPECIFIED BY THE ENGINEER.
2. BEAMS SHALL BE SQUARE AND LEVEL WITHIN STANDARD CONSTRUCTION PRACTICES AND APPROPRIATE CONSTRUCTION MANUALS.
3. CONTRACTOR SHALL NOT ALTER, MOVE, OR INSTALL BEAMS IN ANY WAY OTHER THAN AS STIPULATED IN THE DRAWINGS AND TECHNICAL SPECIFICATIONS.
NOTES:
1. IF DESIGN AND CONTRACTABILITY REQUIREMENTS PERMIT, REPLACEMENT COLUMNS MAY BE SISTERED TO EXISTING COLUMNS BY CONNECTIONS LATER SPECIFIED BY THE ENGINEER.
2. COLUMNS SHALL BE FLUSH, SQUARE, AND LEVEL WITHIN STANDARD CONSTRUCTION PRACTICES AND APPLICABLE CONSTRUCTION MANUALS.
3. CONTRACTOR SHALL NOT ALTER, MODIFY, OR INSTALL COLUMNS IN ANY WAY OTHER THAN AS STIPULATED IN THE DRAWINGS AND TECHNICAL SPECIFICATIONS.

INSTALL COLUMN TO MATCH EXISTING ROOF ELEVATION & DIAPHRAGM ELEMENTS

PLACE 2ND FLOOR COLUMNS AS SHOWN ON SHEET S2

CENTER PROPOSED 2ND FLOOR COLUMN ON 2ND FLOOR BEAM SPLICE

INSTALL PROPOSED 2ND FLOOR BEAM PRIOR TO INSTALLATION OF COLUMN

PROPOSED ROOF BEAM SEE SHEET D1

6" X 6" SELECT STRUCTURAL DFL(N) 2ND FLOOR COLUMN

PROPOSED 2ND FLOOR BEAM SEE SHEET D2

ISO VIEW

TOP VIEW

2ND FLOOR COLUMN DETAIL
NOTES:
1. IF DESIGN AND CONTRACTIBILITY REQUIREMENTS PERMIT, REPLACEMENT COLUMNS MAY BE SISTERED TO EXISTING COLUMNS BY CONNECTIONS LATER SPECIFIED BY THE ENGINEER.
2. COLUMNS SHALL BE PLUMB, SQUARE, AND LEVEL WITHIN STANDARD CONSTRUCTION PRACTICES AND APPLICABLE CONSTRUCTION MANUALS.
3. CONTRACTOR SHALL NOT ALTER, INCREASE, OR INSTALL COLUMNS IN ANY WAY OTHER THAN AS STIPULATED IN THE DRAWINGS AND TECHNICAL SPECIFICATIONS.

PROPOSED 10" X 10" SELECT STRUCTURAL DFL(N)
1ST FLOOR COLUMN

PLACE 1ST FLOOR COLUMNS AS SHOWN ON SHEET S1

INSTALL COLUMN TO MATCH EXISTING 2ND FLOOR ELEVATION & DIAPHRAGM ELEMENTS

CONNECTION TO BE SPECIFIED UPON ONSITE EVALUATION OF 1ST FLOOR CONDITIONS

EXISTING 1ST FLOOR SHEATHING/FLOORING

EXISTING 1ST FLOOR JOISTS/SLAB-ON-GRADE FOUNDATION

PROPOSED 2ND FLOOR BEAM SEE SHEET D2

ISO VIEW

TOP VIEW

MAX. 16'
INSTALL 2 - SIMPSON CTS218 STRAPS AS SHOWN

INSTALL SIMPSON CC66 COLUMN CAP AS SHOWN

INSTALL SIMPSON CC66 COLUMN CAP

MIN. 1 1/2"

1/4" GRADE 2 MACHINE BOLTS WITH NUT/WASHER (2 EA.)

PREVIOUSLY INSTALLED 2ND FLOOR COLUMN

CENTER SIMPSON CC66 COLUMN CAP ON BEAM SPLICE AS SHOWN

SD #9 X 1 1/2" SCREWS (24 EA.)

1 1/2" GRADE 2 MACHINE BOLTS WITH NUT/WASHER (4 EA.)

NOTES:
1. SEQUENCING OF ALL CONNECTION INSTALLATION TO ACHIEVE SPECIFIED RESULT IS SOLELY THE RESPONSIBILITY OF THE CONTRACTOR.

DRAFT
PRELIMINARY DESIGN

BEAM-COLUMN CONNECTION DETAIL

PROPOSED 2ND FLOOR COLUMN SEE SHEET D3

PROPOSED ROOF BEAM SEE SHEET D1

RIGHT

FRONT

ISO VIEW

SCALE
1" = 12'

VU VILLA
STRUCTURAL DESIGN & REHABILITATION

ROOF LEVEL
CONNECTION DETAILS

D5

DRAFT
PRELIMINARY DESIGN

Scale 1" = 12'

Date 10/18/2019
Drawn by KBS
Designed by KBS
Checked by BMK
**BEAM-URM WALL CONNECTION DETAIL**

**NOTES:**
1. MINIMUM 2" OF CLEARANCE SHALL BE PROVIDED BETWEEN ALL NON-PRESERVATIVE TREATED WOOD GIRDERS AND MASONRY/CONCRETE ON TOP, SIDES, AND END OF GIRDER.
2. SEQUENCING OF ALL CONNECTION INSTALLATION TO ACHIEVE SPECIFIED RESULT IS SOLELY THE RESPONSIBILITY OF THE CONTRACTOR.

**EXISTING URM WALL (TYP.)**

**PROPOSED ROOF BEAM INSTALLED AFTER SIMPSON HGLBA BEAM SEAT**

**INSTALL 2 - SIMPSON HTSM20 STRAPS ON EITHER SIDE OF BEAM PRIOR TO PLACEMENT OF BEAM**

**INSTALL SIMPSON HGLBA BEAM SEAT AS SHOWN PRIOR TO ROOF BEAM INSTALLATION**

**DRILL 1" HOLE AND FILL VOID WITH MITEK INCREDI-BOND EPOXY. IMMEDIATELY AFTER EPOXY IS INSTALLED PLACE SIMPSON HGLBA REBAR CENTERED IN HOLES**

**INSTALL SIMPSON TITEN 2 SCREWS AS SHOWN FOR HTSM20 STRAP CONNECTION TO FACE OF URM WALL**

**EXISTING URM WALL BENCH**

**PROPOSED ROOF BEAM**

**SIMPSON HGLBA BEAM SEAT PLACED ON URM WALL BENCH**

**0.148 X 1 1/2" RING SHANK NAILS (14 EA.)**

**3/8" GRADE 2 MACHINE BOLTS WITH NUT/WASHER (2 EA.)**

**INSTALL SIMPSON HTSM20 STRAPS ON BOTH SIDES OF ROOF BEAM AS SHOWN**

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**RIGHT**

**FRONT**

---

**SCALE**

1" = 10'

**DATE**

10/19/2019

**DRAWN BY**

KBS

**DESIGNED BY**

KBS

**CHECKED BY**

BMK

**MONTANA TECHNICAL UNIVERSITY**

**DEPARTMENT OF STRUCTURAL DESIGN & REHABILITATION**

**ROOF LEVEL CONNECTION DETAILS**

**DRAFT PRELIMINARY DESIGN**
NOTES:
1. ALL EXISTING MEMBERS SHALL BE MAINTAINED IN THE BEST CONDITION POSSIBLE. IF A JOIST IS INSPECTED BY THE ENGINEER OR COMPETENT PROFESSIONAL AND DETERMINED TO BE INADEQUATE BASED ON THE DESIGN, THE MEMBER SHALL BE REMOVED AND REPLACED IN ACCORDANCE WITH THE TECHNICAL SPECIFICATIONS.
2. SEQUENCING OF ALL CONNECTION INSTALLATION TO ACHIEVE SPECIFIED RESULT IS SOLELY THE RESPONSIBILITY OF THE CONTRACTOR.

EXISTING ROOF JOIST & SHEATHING (TYP.)

8d COMMON NAILS TOE-NAILED IN ACCORDANCE WITH TECHNICAL SPECIFICATIONS

MIN. 1 3/4

START NAIL APPROXIMATELY THE LENGTH OF THE NAIL ABOVE JOIST-BEAM INTERFACE

EXISTING ROOF JOIST

INSTALL 8d COMMON NAILS - 4 EA. PER INTERSECTION BETWEEN JOIST & ROOF BEAM

MIN. 1" (TYP.)

PROPOSED ROOF BEAM SEE SHEET D1

PROPOSED ROOF BEAM

8d COMMON NAILS TOE-NAILED IN ACCORDANCE WITH TECHNICAL SPECIFICATIONS

MIN. 1 1/2

EAST-WEST

NORTH-SOUTH

BEAM-JOIST CONNECTION DETAIL

Scale 1" = 5'

Date 10/19/2019
Drawn by KBS
Designed by KBS
Checked by BMK

D7

NOT TO SCALE
NOTES:
1. WOOD SPACERS SHALL BE SAME OR SIMILAR GRADE AND SPECIES OF WOOD BEAMS AND COLUMNS AND SHALL BE APPROVED BY THE ENGINEER PRIOR TO INSTALLATION.
2. SEQUENCING OF ALL CONNECTION INSTALLATION TO ACHIEVE SPECIFIED RESULT IS SOLELY THE RESPONSIBILITY OF THE CONTRACTOR.

INSTALL 4 EA. - SIMPSON HL53 ANGLES AS SHOWN
PLACE WOOD SPACERS TO PROVIDE ADEQUATE SPACING FOR SIMPSON HST5PC STRAP INSTALLATION (SEE NOTE 1)

INSTALL 2 EA. SIDE - SIMPSON HST5PC STRAPS AS SHOWN
CENTER SIMPSON HST5PC STRAPS ON BEAM SPLICE AS SHOWN

PLACE WOOD SPACERS TO ALIGN BOLT HOLES FOR TOP & BOTTOM SIMPSON HL53 ANGLES (SEE NOTE 1)

CENTER SIMPSON HL53 ANGLE ON BEAM/COLUMNS AS SHOWN

PLACE WOOD SPACERS TO PROVIDE ADEQUATE SPACING FOR SIMPSON HST5PC STRAP INSTALLATION (SEE NOTE 1)

1 2 " GRADE 2 MACHINE BOLTS WITH NUT/WASHER (24 EA.)

CENTER SIMPSON HST5PC STRAPS ON BEAM SPLICE AS SHOWN

INSTALL 2 EA. SIDE - SIMPSON HST5PC STRAPS AS SHOWN

PLACE WOOD SPACERS TO PROVIDE ADEQUATE SPACING FOR SIMPSON HST5PC STRAP INSTALLATION (SEE NOTE 1)

1 2 " GRADE 2 MACHINE BOLTS WITH NUT/WASHER (24 EA.)

RIGHT

BEAM-COLUMN CONNECTION DETAIL

FRONT

PROPOSED 2ND FLOOR BEAM COLUMN SEE SHEET D3

PROPOSED 1ST FLOOR COLUMN SEE SHEET D4

PROPOSED 2ND FLOOR COLUMN SEE SHEET D3

PROPOSED 2ND FLOOR BEAM COLUMN SEE SHEET D2

ISO VIEW
NOTES:
1. MINIMUM 2" OF CLEARANCE SHALL BE PROVIDED BETWEEN ALL NON-PRESERVATIVE TREATED WOOD GIRDERS AND
MASONRY/CONCRETE ON TOP, SIDES, AND END OF GIRDERS.
2. WOOD SPACERS SHALL BE PRESSURE TREATED AND HAVE THE SAME GRADE AND RATED STRENGTH OF 2ND FLOOR
BEAM. SPACER GRADE AND SOURCE MUST BE APPROVED BY THE ENGINEER PRIOR TO INSTALLATION.
3. SEQUENCING OF ALL CONNECTION INSTALLATION TO ACHIEVE SPECIFIED RESULT IS SOLELY THE RESPONSIBILITY OF
THE CONTRACTOR.
4. SOUTHERN 2ND FLOOR BEAMS SHALL REST ON EXISTING WOOD HEADERS AT SOUTH ENTRANCES TO VU VILLA.
BEAM-WALL CONNECTION SHALL BE SAME AS SPECIFIED ON SHEET D9. FOR BEAM CONNECTION TO HEADER AS
FOR URM WALL ON NORTH SIDE OF BUILDING.

RIGHT

EXISTING URM WALL (TYP.)

PROPOSED 2ND FLOOR BEAM
INSTALLED AFTER SIMPSON
HGLBD BEAM SEAT

PLACE WOOD SPACER BETWEEN
EXISTING URM WALL AND 2ND FLOOR
BEAM AS SHOWN (SEE NOTE 2)

2ND FLOOR BEAM MUST BEAR COMPLETELY
ON SIMPSON HGLBD BEAM SEAT

EXISTING WOOD HEADER
(SEE NOTE 4)

INSTALL SIMPSON HGLBD
BEAM SEAT FLUSH WITH URM WALL/
WOOD HEADER FACE AS SHOWN

MIN. 1/4"

MIN. 1/2"

WOOD SPACER (TYP.)

3/4" GRADE 2 MACHINE BOLTS
WITH NUT/WASHER (2 EA.)

DRILL 1/2" HOLES AND FILL VOID
WITH MITEK INCREDI-BOND
EPOXY. IMMEDIATELY AFTER
EPOXY IS INSTALLED PLACE
SIMPSON HGLBD REBAR
CENTERED IN HOLES

EXISTING WOOD HEADER
(SEE NOTE 4)

FRONT

WOOD SPACER (TYP.)

SIMPSON HGLBD BEAM SEAT
PLACED ON URM/WALL BENCH
WOOD HEADER

VU VILLA
STRUCTURAL DESIGN
& REHABILITATION

BEAM-URM WALL CONNECTION DETAIL

Montana Technological University

KBS

10/16/2019

Scale 1" = 12"
BEAM-JOIST CONNECTION DETAIL

EXISTING ROOF JOIST & SHEATHING (TYP.)

10d COMMON NAILS TOE-NAILED IN ACCORDANCE WITH TECHNICAL SPECIFICATIONS

INSTALL 8d COMMON NAILS - 3 EA. PER INTERSECTION BETWEEN JOIST & 2ND FLOOR BEAM

START NAIL APPROXIMATELY 1/2 THE LENGTH OF THE NAIL ABOVE JOIST-BEAM INTERFACE

PROPOSED 2ND FLOOR BEAM SEE SHEET D1

NOTES:
1. ALL EXISTING MEMBERS SHALL BE MAINTAINED IN THE BEST CONDITION POSSIBLE. IF A JOIST IS INSPECTED BY THE ENGINEER OR COMPETENT PROFESSIONAL AND DETERMINED TO BE INADEQUATE BASED ON THE DESIGN, THE MEMBER SHALL BE REMOVED AND REPLACED IN ACCORDANCE WITH THE TECHNICAL SPECIFICATIONS.
2. SEQUENCING OF ALL CONNECTION INSTALLATION TO ACHIEVE SPECIFIED RESULT IS SOLELY THE RESPONSIBILITY OF THE CONTRACTOR.

EXISTING ROOF JOIST

2ND FLOOR CONNECTION DETAILS

EAST-WEST

NORTH-SOUTH

Scale 1" = 5'

DATE 10/19/2019

Drawn by KBS

Designed by KBS

Checked by BMK
NOTES:
1. NAILS SHALL BE 8D COMMON OR EQUIVALENT FOR BOUNDARIES AND 6D COMMON OR EQUIVALENT FOR PANEL EDGES AND INTERIORS.
2. NAIL SPECIFICATIONS ARE MINIMUM REQUIREMENTS. IF NAILS ARE OF GREATER SIZE OR A REDUCED SPACING, NO FURTHER ACTION IS REQUIRED.
3. IF EITHER NAILS OR SHEATHING ARE DEEMED INADEQUATE BY THE ENGINEER OR COMPETENT PROFESSIONAL, THE INADEQUATE MATERIAL SHALL BE REMOVED AND REPLACED IN ACCORDANCE WITH THESE SPECIFICATIONS OR AS APPROVED BY THE ENGINEER.

MIN. 4" X 8" SHEATHING PANELS ORIENTED NORTH-SOUTH AS SHOWN

6D COMMON NAILS @ 12" O.C. OR EQUIVALENT

6D COMMON NAILS @ 6" O.C. OR EQUIVALENT

EDGE/INTERIOR NAILING PLAN

MIN. 8" FROM EDGE OF PANEL TO EDGE/BOUNDARY NAILS (TYP.)

BOUNDARY NAILING PATTERN SHALL CONTINUE TO A MIN. OF 9 - 4" FROM URN WALL INWARD IN BOTH DIRECTIONS.

DIAPHRAGM BOUNDARY

8D COMMON NAILS @ 6" O.C. OR EQUIVALENT

EXISTING ROOF JOISTS (TYP.)

MIN. NAILING AS SPECIFIED (SEE NOTE 1)

1/2" STANDARD OSB OR Sanded Plywood

MINIMUM CRITERIA DETAILS

15" STANDARD OSB OR SANDED PLYWOOD

EXISTING ROOF JOISTS (TYP.)

6D COMMON NAILS @ 12" O.C. OR EQUIVALENT

6D COMMON NAILS @ 6" O.C. OR EQUIVALENT

BOUNDARY NAILING PLAN

10/22/2019

KBS

BMK

VU VILLA
STRUCTURAL DESIGN & REHABILITATION

ROOF DIAPHRAGM MINIMUM CRITERIA DETAILS

Scale 1" = 30'

KBS

Drawn by

Designed by

Checked by

16/22/2019
NOTES:
1. DETAIL 1 & 2 PRESENT MULTIPLE ACCEPTABLE CASES FOR 2ND FLOOR DIAPHRAGM. IF EITHER CASE IS SATISFIED, NO FURTHER ACTION IS REQUIRED.
2. Nail specifications are minimum requirements. If nails are of greater size or a reduced spacing, no further action is required.
3. If either nails or sheathing are deemed inadequate by the engineer or competent professional, the inadequate material shall be removed and replaced in accordance with these specifications or as approved by the engineer.
4. In all cases specified, blocking is required for 2nd floor diaphragm.

MIN. 4" X 8" SHEATHING PANELS ORIENTED NORTH-SOUTH AS SHOWN

FOR BOUNDARY & EAST-WEST PANEL EDGE NAILING: 10d COMMON NAILS @ 4" O.C. OR EQUIVALENT (SEE NOTE 2)

PLAN VIEW

FOR PANEL INTERIOR NAILING: 10d COMMON NAILS @ 12" O.C. OR EQUIVALENT

PLAN VIEW

MIN. 3" FROM EDGE OF PANEL TO ENDBOUNDARY NAILS (TYP.)

FOR NORTH-SOUTH PANEL EDGE NAILING: 10d COMMON NAILS @ 6" O.C. OR EQUIVALENT

PLAN VIEW

MIN. 3" SANDED PLYWOOD

MIN. 1" X 6" DFL(N) DOUBLE-DIAGONAL PLANKS

DIAPHRAGM BLOCKING REQUIRED (SEE NOTE 4)
APPENDIX B – TECHNICAL SPECIFICATIONS
VU VILLA STRUCTURAL DESIGN/REHABILITATION

TECHNICAL SPECIFICATIONS

Kolby Sukut
General Engineering – Civil Option

Montana Tech
2019
SECTION 01 – GENERAL PROVISIONS

PURPOSE

The following section provides general notes, specifications, and provisions related to the Vu Villa Structural Design and Rehabilitation Project.

PART 1 - GENERAL

1.1 PROJECT SCOPE

A. The intent of the Vu Villa Structural Design and Rehabilitation Project is to provide initial structural design information on which the Owner can make further decision of work to be performed on the building.

B. Any Work completed related to the Design shall be in accordance with the guidelines provided in the 2012 International Building Code (IBC).

C. In the event the Engineer or Competent Professional determines an existing member is not of suitable condition or strength, the member will be removed and replaced with a new member of adequate condition and capacity to satisfy the Design.

D. Any design information provided within these technical specifications is considered a draft product and must be reviewed and stamped by a licensed professional engineer prior to Work being completed.

E. The following notes, specifications, and provisions are NOT to be used for construction or renovation activities.

1.2 DEFINITIONS

A. Engineer – Person or entity who is professionally licensed to work as an engineer in the State of Montana and who is responsible for the stamped design and quality assurance on the project.

B. Competent Professional – Person or entity who is specifically responsible for providing quality assurance support on the project but may not be a licensed, practicing engineer.

C. Contractor – Person or entity performing the Work as detailed in the Drawings and Technical Specifications.

1.3 ACRONYMS

A. ACI – American Concrete Institute
B. ANSI – American Nation Standards Institute
C. APA – American Plywood Association
D. ASCE – American Society of Civil Engineers
E. DFL (N) – Douglas Fir-Larch (North)
F. IBC – International Building Code
G. IEBC – International Existing Building Code
H. NDS – National Design Specification
I. O.C. – On-Center (for spacing measurements)
J. PSF – Pounds per Square Foot
K. SDPWS – Special Design Provisions for Wind and Seismic
L. TMS – The Masonry Society
M. URM – Unreinforced Masonry

1.4 CODES & STANDARDS
A. The following codes were used for calculations and design of the proposed structural system for the Vu Villa Structural Design/Rehabilitation project:

   I. ASCE 7-02: Minimum Design Loads for Buildings and Other Structures
   II. Load-Span Tables for APA Structural-Use Panels
   III. TMS 402-11/ACI 530-11/ASCE5-11: Building Code Requirements for Masonry Structures
   IV. 2012 IBC
   V. 2012 IEBC
   VI. 2015 NDS
   VII. 2015 SDPWS

B. Codes do NOT necessarily meet the requirements for locally adopted codes. In the event that any information related to the Design is implemented, the Design is to be reviewed and verified that it complies with the current local code requirements.

C. If codes are not current, design is to be revised to ensure all aspects of Design comply with locally adopted codes.

1.5 DESIGN ASSUMPTIONS
A. The following assumptions were used to define the scope of the project:

   I. The first-floor members and foundation, excluding columns, of the Vu Villa were assumed to be of a sufficient condition and strength to support the newly designed structural system. This was based on the current state of each and a lack of evidence suggesting otherwise.

   II. Existing members that will remain a part of the structural system were assessed and verified based on the estimated quality of material as noted in the design calculations.

   III. The two primary goals of the design are to address:
      (i) The existing sag within the 2nd Floor Beams; and
      (ii) Provide a structural system that can support an assembly area, open space floor plan on the 2nd floor of the building.
1.6  LOADING CONDITIONS

A. The loading conditions used in the design and verification of each structural member provided in the design calculations were derived from the ASCE 7-02 Minimum Design Loads for Buildings and Other Structures. See below for the specific values used in the design.

B.  Dead Load
   I. Roof (Beams) = 17.5 psf
   II. 2nd Floor (Columns) = 18.0 psf
   III. 2nd Floor (Beams) = 34.8 psf
   IV. 1st Floor (Columns) = 36.0 psf

C.  Live Load
   I. Roof Live Load = 18.1 psf
   II. 2nd Floor Live Load (Beams) = 86.7 psf
   III. 2nd Floor Live Load (Columns) = 68.6 psf

D.  Seismic Load
   I. Gravity Loads
      (i) Roof (Gravity) = 1.0 psf
      (ii) 2nd Floor (Gravity) = 2.1 psf
   II. Lateral Loads
      (i) Roof (Vertical Elements) = 1,342.3 plf
      (ii) 2nd Floor (Vertical Elements) = 1,716.2 plf
      (iii) Roof (Horizontal Elements) = 566.8 plf
      (iv) 2nd Floor (Horizontal Elements) = 1,268.3 plf

E.  Snow Load
   I. Snow = 28.1 psf

F.  Wind Load
   I. Gravity Loads
      (i) Uplift on Roof = 26.6 psf
   II. Lateral Loads
      (i) Exterior = 22.1 psf
      (ii) Interior = 16.0 psf

G. Contractor shall stage and place materials and equipment in such a way so as to ensure that no concentrated load will exceed the live loads identified in Section 1.4 (C).

H. Design does not apply to any case in which the specified loads from Section 1.5 are exceeded. If Contractor or Owner believes that there is potential for the specified
loads to be exceeded, the Design is to be considered null and void, and the Engineer is to be notified immediately.

1.7 DISCREPANCIES & OMISSIONS
A. All Design dimensions and conditions shall be validated by the Contractor onsite. Any discrepancies, errors, or omissions identified by the Contractor shall be immediately reported to the Engineer and no Work shall be completed related to the discrepancy without written approval, clarification, or interpretation by the Engineer.

B. Design is based on a survey by others, Engineer takes no responsibility for errors or mistakes in the original survey or any design discrepancies due to errors in the original survey.

END OF SECTION
SECTION 02 – REPLACEMENT BEAMS

PURPOSE

The following section details the specifications and provisions related to replacing the Roof and 2nd Floor Beams associated with the Vu Villa Structural Design and Rehabilitation Project.

PART 1 - GENERAL

1.1 DEFINITIONS

A. Roof Beams – Girders to be installed as part of the proposed primary load path at the roof level and replace the existing partitioned load bearing walls.

B. 2nd Floor Beams – Girders to be installed as part of the primary load path at the second-floor level and replace or reinforce the existing beams supporting the second-floor system.

1.2 CODES & STANDARDS

A. Roof and 2nd Floor Beams shall comply with the following provisions, except where otherwise noted:

I. ANSI A190.1-2012
II. ASTM D 2559
III. ASTM D 3737
IV. ASTM D 7199
V. ASTM D 7247-07a
VI. ASTM D 7341-09

PART 2 - PRODUCTS

2.1 MATERIALS

A. Both Roof Beams and 2nd Floor Beams shall be 24F–V4 DF/DF Glulam.

B. All beams shall have been manufactured in conditions with less than 19% moisture.

C. All beams shall NOT have been exposed to sustained temperatures greater than 150°F during production.

D. All Roof Beams shall have a breadth of 5 ½”, depth of 21”, and a maximum length of 19’-8 ½”.

E. All 2nd Floor Beams shall have a breadth of 6 ¾”, depth of 24”, and a maximum length of 19’-8 ½”.
F. Materials for replacement columns shall be as specified in Section 03 – 2.1 Materials.

G. Materials for connections shall be as specified in Section 04 – 2.1 Materials.

2.2 QUALITY ASSURANCE/QUALITY CONTROL (QA/QC)
   A. Submittals shall be provided to the Engineer by the Contractor certifying that all materials and products meet the requirements and provisions specified herein. Contractor shall not purchase or acquire any material or product without written approval of the associated submittal from the Engineer.

   B. It shall be the sole responsibility of the Contractor to perform all quality control requirements in acquisition, transport, and handling of specified materials and verifying that the materials meet the specifications provided herein.

   C. Engineer has the right to access/perform any necessary quality assurance documentation, inspections, and/or testing to ensure the materials meet the specifications provided herein.

   D. Contractor shall coordinate all necessary inspections by County and State Inspectors as required by local, state, and federal laws and regulations.

PART 3 - EXECUTION

3.1 DEMOLITION/REMOVAL OF EXISTING MEMBERS
   A. Removal of beams, columns, and load bearing walls shall be by the means and methods of the Contractor.

   B. Contractor shall minimize impact to existing ceilings, flooring, joists, sheathing, and other structural or architectural components to the roof or second-floor, not otherwise noted as requiring removal within these specifications or as shown on the Drawings.

   C. If existing beams can be sistered with replacement beams as determined by the Engineer, the existing beams need not be removed and shall be connected to the replacement beams as specified by the Engineer or Competent Professional.

3.2 INSTALLATION
   A. Contractor shall be solely responsible for procurement and delivery of all materials specified in Part 2.

   B. Beams shall be installed as shown on the Drawings and shall be square and level to within the tolerances of standard construction practices and to the provisions of applicable construction manuals.
C. Beams splices shall be centered with the same length of each beam resting on support columns.

D. Where applicable, a minimum clearance of \( \frac{1}{2} \)" air gap shall be provided between beams and all concrete, masonry, and/or mortar.

E. Beams shall be installed without the Contractor bending or prestressing prior to installation.

F. No incisions or alterations shall be made to Beams except those specified in these specifications or as shown on the Drawings.

G. Contractor shall maintain existing members condition in such a way so as to ensure replacement members can be installed and connected to existing members without changing the loading, shape, or strength of the existing members.

END OF SECTION
SECTION 03 – REPLACEMENT COLUMNS

PURPOSE

The following section details the specifications and provisions related to replacing the Roof and 2nd Floor Columns associated with the Vu Villa Structural Design and Rehabilitation Project.

PART 1 - GENERAL

1.1 DEFINITIONS
A. 2nd Floor Columns – Columns to be installed as part of the proposed primary load path at the second-floor level and replace the existing partitioned load bearing walls.

B. 1st Floor Columns – Columns to be installed as part of the primary load path at the first-floor level and replace or reinforce the existing columns supporting the second-floor system.

1.2 CODES & STANDARDS
A. Roof and 2nd Floor Columns shall comply with the following provisions, except where otherwise noted:

I. ASTM D 245-06
II. ASTM D 2555-06

PART 2 - PRODUCTS

2.1 MATERIALS
A. Both 2nd Floor Columns and 1st Floor Columns shall be either rough sawn or S4S Select Structural Douglas-Fir Larch (North).

B. All columns shall have been manufactured in conditions with less the 19% moisture.

C. All columns shall NOT have been exposed to sustained temperatures greater than 150°F during production.

D. All 2nd Floor Columns shall be nominal 6” by 6” columns (i.e. minimum 5 ½” actual dimensions) with a maximum installed length of 12-feet.

E. All 1st Floor Columns shall be nominal 10” by 10” columns (i.e. minimum 9 ½” actual dimensions) with a maximum installed length of 16-feet.

F. Materials for replacement beams shall be as specified in Section 02 – 2.1 Materials.

G. Materials for connections shall be as specified in Section 04 – 2.1 Materials.
2.2 QUALITY ASSURANCE/QUALITY CONTROL (QA/QC)

A. Submittals shall be provided to the Engineer by the Contractor certifying that all materials and products meet the requirements and provisions specified herein. Contractor shall not purchase or acquire any material or product without written approval of the associated submittal from the Engineer.

B. It shall be the sole responsibility of the Contractor to perform all quality control requirements in acquisition, transport, and handling of specified materials and verifying that the materials meet the specifications provided herein.

C. Engineer has the right to access/perform any necessary quality assurance documentation, inspections, and/or testing to ensure the materials meet the specifications provided herein.

D. Contractor shall coordinate all necessary inspections by County and State Inspectors as required by local, state, and federal laws and regulations.

PART 3 - EXECUTION

3.1 DEMOLITION/REMOVAL OF EXISTING MEMBERS

A. Removal of beams, columns, and load bearing walls shall be by the means and methods of the Contractor.

B. Contractor shall minimize impact to existing ceilings, flooring, joists, sheathing, and other structural or architectural components to the roof or second-floor, not otherwise noted as requiring removal within these specifications or as shown on the Drawings.

C. If existing columns can be sistered with replacement columns as determined by the Engineer, the existing columns need not be removed and shall be connected to the replacement columns as specified by the Engineer.

3.2 INSTALLATION

A. Contractor shall be solely responsible for procurement and delivery of all materials specified in Part 2.

B. Columns shall be installed as shown on the Drawings and shall be plumb, square, and level to within the tolerances of standard construction practices and to the provisions of applicable construction manuals.

C. All columns shall be centered over the splice condition of support beams with equal lengths of column width resting on each beam.

D. Where applicable, a minimum clearance of ½” air gap shall be provided between columns and all concrete, masonry, and/or mortar.
E. Columns shall be installed without the Contractor bending or prestressing prior to installation.

F. No incisions or alterations shall be made to Columns except those specified in these specifications or as shown on the Drawings.

G. Contractor shall maintain existing members condition in such a way so as to ensure replacement members can be installed and connected to existing members without changing the loading, shape, or strength of the existing members.

END OF SECTION
SECTION 04 – REPLACEMENT CONNECTIONS

PURPOSE

The following section details the specifications and provisions related to connecting replacement beams and columns to one another, as well as to existing joists and bearing walls associated with the Vu Villa Structural Design and Rehabilitation Project.

PART 1 - GENERAL

1.1 DEFINITIONS

A. Roof Level Connections – Various connection hardware to be installed for the purpose of transferring loads from existing and replacement members at the roof level.

B. 2nd Floor Connections – Various connection hardware to be installed for the purpose of transferring loads from existing and replacement members at the second-floor level.

C. Beam-Column Connections – Connection hardware specifically purposed for connecting replacement beams to replacement columns.

D. Beam-Joist Connections – Connection hardware specifically purposed for connecting existing roof or 2nd floor joists to replacement beams.

E. Beam-Wall Connections – Connection hardware specifically purposed for connecting replacement beams to existing exterior load-bearing unreinforced masonry walls.

F. Simpson – Connection hardware supply company Simpson Strong-Tie.

1.2 CODES & STANDARDS

A. All connections to be installed shall comply with the following provisions, except where otherwise noted:

   I. ANSI/ASME B18.2.1
   II. ASTM A36
   III. ASTM A123
   IV. ASTM A653
   V. ASTM A706
   VI. ASTM A1011
   VII. ASTM F1554-07a
PART 2 - PRODUCTS

2.1 MATERIALS

A. Roof Connections
   I. Beam-Column:
      (i) Simpson CC66 Column Caps
      (ii) Simpson CTS218 Straps
      (iii) Simpson Strong Drive #9 – 1 ½” Screw
      (iv) 5/8” Grade 2 Steel Machine Bolt/Nut/Washer (2 ¾”)

   II. Beam-Joist:
      (i) 8d Common Nails

   III. Beam-Wall:
      (i) Simpson HGLBA Beam Seat
      (ii) Simpson HSTM20 Straps
      (iii) Simpson Titen 2® Screws
      (iv) 0.148” x 3 ¼” Sinkers
      (v) ¼” Grade 2 Steel Machine Bolt/Nut/Washer (6 3/8”)
      (vi) MiTek Incredi-Bond epoxy or equivalent

B. 2nd Floor Connections:
   I. Beam-Column:
      (i) Simpson HL53 Angles
      (ii) Simpson HST5PC Straps
      (iii) 1/2” Grade 2 Steel Machine Bolt/Nut/Washer (6”, 7 5/8”, 10”, and 21 1/2”)

   II. Beam-Joist:
      (i) 10d Common Nails

   III. Beam-Wall:
      (i) Simpson HGLBD Beam Seat
      (ii) 3/4” Grade 2 Steel Machine Bolt/Nut/Washer (7 5/8”)
      (iii) MiTek Incredi-Bond epoxy or equivalent

2.2 QUALITY ASSURANCE/QUALITY CONTROL (QA/QC)

A. Submittals shall be provided to the Engineer by the Contractor certifying that all
   materials and products meet the requirements and provisions specified herein.
   Contractor shall not purchase or acquire any material or product without written
   approval of the associated submittal from the Engineer.

B. It shall be the sole responsibility of the Contractor to perform all quality control
   requirements in acquisition, transport, and handling of specified materials and
   verifying that the materials meet the specifications provided herein.
C. Engineer has the right to access/perform any necessary quality assurance documentation, inspections, and/or testing to ensure the materials meet the specifications provided herein.

PART 3 - EXECUTION

3.1 DEMOLITION/REMOVAL OF EXISTING MEMBERS

A. Removal of beams, columns, and load bearing walls shall be by the means and methods of the Contractor.

B. Contractor shall minimize impact to existing ceilings, flooring, joists, sheathing, and other structural or architectural components to the roof or second-floor, not otherwise noted requiring removal within these specifications or as shown on the Drawings.

C. For Beam-Wall Connections, Contractor shall remove all necessary brick to perform installation of both Roof and 2nd Floor Beams and corresponding connection hardware at the proper height as specified on the Drawings. Contractor shall make every effort to limit the removal of additional bricks from bearing wall. If brick must be taken from areas outside of what is required for the beams, Contractor shall repair or replace masonry units to acceptable working condition as determined by the Engineer.

3.2 INSTALLATION

A. Contractor shall be solely responsible for procurement and delivery of all hardware & dowels specified in Part 2.

B. All connection hardware specified in Part 2 shall be from the designated manufacturer unless Engineer approves an equivalent alternative.

C. Connection hardware shall be installed as shown on the Drawings and to within the recommended provisions, specifications, and tolerances of the manufacturer.

D. Unless otherwise specified, all bolt holes shall be drilled a minimum $\frac{1}{32}$” to a maximum of $\frac{1}{16}$” larger than the diameter of the associated bolt.

E. Contractor shall maintain existing and replacement member’s condition in such a way so as to ensure installation of connection hardware and nails can be achieved without changing the loading, shape, or strength of the existing or replacement members.

F. No incisions, alterations, or drilling shall be performed on replacement or existing members except those required for the implementation of hardware and nails specified herein or as shown on the Drawings.
G. **Roof Connections**

I. **Beam-Column:**

   (i) All connections shall be centered on beam splice with equal spacing on each side of splice location.

   (ii) Simpson CC66 column caps shall be placed in such a way so as to ensure that bottom bracket of cap and associated bolts run directly along the centerline of the column.

   (iii) Top Simpson CTS218 compression/tension strap shall be installed with a clearance of 1 ½” from the top of the beams and centered on the second laminated 2x6 from the top of the beams. Bottom strap shall be placed immediately below the first, centered directly on the third laminated 2x6 from the top.

II. **Beam-Joist:**

   (i) Four nails – two nails on each side of joists shall be installed at locations where a joist rests on a beam.

   (ii) Minimum of 1” of clearance shall be provided from edge of beam to the first nail.

   (iii) Contractor shall make every effort to ‘toe-nail’ nails at an angle of 30 degrees from vertical axis through the bottom of the joist into the beam.

   (iv) Minimum of 1 ½” of nail shall penetrate into the beam.

III. **Beam-Wall:**

   (i) Simpson HTSM20 straps shall be installed between Simpson HGLBA beam seat and Roof Beams. Straps shall be installed so that they are spaced between bolt holes of the HGLBA beam seat as shown on the Drawings.

   (ii) Simpson HTSM20 straps shall be connected to face of unreinforced masonry (URM) wall with 4 - Simpson ¼” x 2 ¼” Titen® 2 screws as shown on the Drawings.

   (iii) Holes for ¼” x 2 ¼” Titen® 2 screws shall be drilled with a 3/16” bit to a minimum hole depth of 2 ¼”. Screw shall then be inserted and embedded to a depth of 1 ¼”.

   (iv) Simpson HTSM20 straps shall be connected to the side face of the Roof Beams with 10 – 0.148” x 1 ½” Ring Shank nails as shown on the Drawings.

   (v) Simpson HGLBA Beam Seats shall be installed between bench of URM Wall and replacement beam with the beam resting directly the beam seat. Simpson HGLBA beam seat shall be installed with a clearance of 1 7/16” from interior face of URM Wall.

   (vi) Simpson HGLBA Beam Seat shall be connected to the Roof Beam with 2 – 3/4” Grade 2 Steel Machine Bolt/Nut/Washer as shown on the Drawings.

   (vii) Simpson HGLBA Beam Seat shall be connected to the URM Wall by drilling holes for rebar dowels of Simpson HGLBA beam seat. MiTek Incredi-Bond epoxy shall then be prepared and used to seal the remaining voids surrounding the rebar dowels. Rebar dowels shall not be installed until holes are filled with specified epoxy.
(viii) Holes for rebar dowels shall be drilled to with a 1” bit to a minimum depth of 12”.
(ix) Holes shall be drilled vertically down, directly through brick unit and shall NOT contact any surrounding mortar.
(x) Contractor shall drill holes with adequate spacing to allow for installation of all rebar dowels for each Simpson HGLBA beam seat.
(xi) Prior to installation of Simpson HGLBA beam seat, Contractor shall prepare MiTek Incredi-Bond epoxy and fill the designated rebar dowel holes.
(xii) Epoxy shall only be installed if ambient temperatures are between 40°F and 100°F.
(xiii) Contractor shall install Simpson HGLBA beam seat immediately following placement of epoxy.
(xiv) Simpson HGLBA beam seat’ rebar dowels shall be centered in installation holes.
(xv) Contractor shall provide a minimum cure time of 3-hours from installation of epoxy/beam seat to installation of Roof Beams.

H. 2nd Floor Connections
I. Beam-Column:
  (i) All beams and columns shall be properly installed prior to the installation of Beam-Column connections.
  (ii) 4 - Simpson HST2PC straps shall be installed on both east and west faces of associated beams and columns as shown on the Drawings.
  (iii) Simpson HST2PC straps shall be centered on beam splice location and will be anchored with 12 – 5/8” machine bolts to the associated beams and columns.
  (iv) Contractor shall install Simpson HST2PC straps in such a way so that the straps are centered on each associated column, with equal spacing on each side of the strap.
  (v) 4 – Simpson HL53 angles shall be installed on the top and bottom (2-each) faces of the 2nd Floor Beams, and along the north/south faces of the 1st Floor and 2nd Floor Columns.
  (vi) Simpson HL53 angles shall be anchored with 8 – ½” machine bolts to the associated beams and columns.
  (vii) Contractor shall install Simpson HL53 angles in such a way so that the angles are centered on each associated column, with equal spacing on each side of the angle.
  (viii) Contractor shall install wood spacers to simplify connections as shown on the Drawings. Spacers shall be of similar grade and species as replacement beams and columns and shall be approved by the Engineer prior to installation.

II. Beam-Joist:
  (i) Three nails shall be installed at locations where a joist rests on a beam with an alternating pattern along the length of the joist of 2 nails on one side and 1 nail on the other.
  (ii) Minimum of 1” of clearance shall be provided from edge of beam to the first nail.
(iii) Contractor shall make every effort to ‘toe-nail’ nails at an angle of 30 degrees from vertical axis through the bottom of the joist into the beam.

(iv) Minimum of 1 7/8” of nail shall penetrate into the beam.

III. Beam-Wall:

(i) Simpson HGLBD Beam Seats shall be installed between bench of URM Wall and Roof Beam with the beam resting directly on the beam seat.

(ii) Simpson HGLBD Beam Seat shall be connected to the Roof Beam with 2 – 3/4” Grade 2 Steel Machine Bolt/Nut/Washer as shown on the Drawings.

(iii) Simpson HGLBD Beam Seat shall be connected to the URM Wall by drilling holes for rebar dowels of Simpson HGLBD beam seat. MiTek Incredi-Bond epoxy shall then be prepared and used to seal the remaining voids surrounding the rebar dowels. Rebar dowels shall not be installed until holes are filled with specified epoxy.

(iv) Holes for rebar dowels shall be drilled to with a 1” bit to a minimum depth of 12”.

(v) Holes shall be drilled vertically down, directly through brick unit and shall NOT contact any surrounding mortar.

(vi) Contractor shall drill holes with adequate spacing to allow for installation of all rebar dowels for each Simpson HGLBD beam seat.

(vii) Prior to installation of Simpson HGLBD beam seat, Contractor shall prepare MiTek Incredi-Bond epoxy and fill the designated rebar dowel holes.

(viii) Epoxy shall only be installed if ambient temperatures are between 40°F and 100°F.

(ix) Contractor shall install Simpson HGLBD beam seat immediately following placement of epoxy and shall position beam seat in such a way that the front of the plate has 1” of clearance to the face of the URM Wall.

(x) Simpson HGLBD beam seat’ rebar dowels shall be centered in installation holes.

(xi) Contractor shall provide a minimum cure time of 3-hours from installation of epoxy/beam seat to installation of Roof Beams.

(xii) Contractor shall install wood spacers to simplify connections as shown on the Drawings. Wood spacers shall be pressure treated and have the same or similar grade and rated strength of 2nd Floor Beam. Spacer grade and source must be approved by the Engineer prior to installation.

END OF SECTION
SECTION 05 – EXISTING MEMBERS

PURPOSE

The following section is intended to provide the minimum criteria for evaluation of existing members by the Contractor, Competent Professional, and/or Engineer for the Vu Villa Structural Design and Rehabilitation Project.

PART 1 - GENERAL

1.1 DEFINITIONS

A. Roof Diaphragm – The horizontal system of members and connections which act to transfer lateral forces at the roof level to the vertical-resisting elements; this includes joists, roof sheathing, and the nails adjoining them.

B. 2nd Floor Diaphragm – The horizontal system of members and connections which act to pass lateral forces at the second-floor level to the vertical-resisting elements; this includes joists, floor sheathing, and the nails adjoining them.

C. Roof Joists – The horizontal members running east and west that support the ceiling and roof systems, and transfer vertical and horizontal loads to the primary beams/girders.

D. 2nd Floor Joists – The horizontal members running east and west that support the ceiling and floor systems, and transfer vertical and horizontal loads to the primary beams/girders.

E. Shear/Bearing Walls – The wood members/walls and URM Walls that act to transfer lateral and vertical forces to the foundation.

1.2 CODES & STANDARDS

A. No codes or standards apply to the existing members as specified herein; it shall be the sole responsibility of the Engineer or Competent Professional to evaluate existing members and determine their structural adequacy.

B. In the event that existing members require replacement, those replacement members shall comply with the provisions, materials, and specifications as defined in Part 2 and Part 3, below.

1.3 EVALUATION OF EXISTING MEMBERS

A. All members associated with the primary vertical and lateral load paths which require direct fastening to replacement members shall be evaluated by an Engineer
or Competent Professional to determine the adequacy of the member’s condition and quality.

B. If it is determined by the Engineer or Competent Professional that an existing member requires replacement, it shall be replaced with a member that at a minimum meets the criteria below.

C. *Part 2 – Products* below defines minimum acceptable materials that existing members must be to achieve adequate structural needs for the project. *Part 3 – Execution* below defines, at a minimum, the existing installed requirements that existing members must meet. Replacement members are permitted to be of a greater strength and/or size than outlined below as needed. Both the Contractor, and Engineer or Competent Professional shall collaborate and select the best material that meets both strength and serviceability needs.

D. If a replacement member is selected other than as specified below, the Engineer must review the selected member to ensure it meets the necessary strength requirements.

**PART 2 - PRODUCTS**

2.1 MATERIALS

A. Roof Diaphragm:
   I. Sheathing shall be 4’x8’ sheets, at a minimum:
      (i) \(\frac{15}{32}\)” standard Oriented Strand Board (OSB); or
      (ii) \(\frac{15}{32}\)” Sanded Plywood.

   II. Nailing shall be, at a minimum:
      (i) 8d Common Nails (diaphragm-shear wall boundaries); and
      (ii) 6d Common Nails (sheathing edge/interior)

B. 2nd Floor Diaphragm:
   I. Sheathing shall be, at a minimum:
      (i) 1”x6” No. 3 DFL(N) Lumber Planks; or
      (ii) \(\frac{19}{32}\)” Sanded Plywood – 4’x8’ sheets.

   II. Nailing shall be, at a minimum:
      (i) 20d Common Nails for Lumber Planks; or
      (ii) 10d Common Nails for Sanded Plywood.

C. Roof Joists:
   I. 2”x6” Select Structural DFL(N)

D. 2nd Floor Joists:
   I. 2”x12” No. 1 & BTR DFL(N)
   II.
E. Shear/Bearing Walls:
   I. URM:
      (i) 3-Wythe Solid Brick with a Running Bond
      (ii) Type N Mortar or mortar of equivalent strength
   II. Wood Members/Walls:
      (i) 2”x6” Dense No. 1 DFL(N) Wall at 16” O.C.
      (ii) 2 – 6”x6” Dense No. 1 DFL(N) Column
      (iii) 8”x8” Dense No. 1 DFL(N) Column
      (iv) 12”x12” No. 1 DFL(N) Beam Header

2.2 QUALITY ASSURANCE/QUALITY CONTROL (QA/QC)
   A. For any existing members requiring replacement, submittals shall be provided to the Engineer by the Contractor certifying that all materials and products meet the requirements and provisions specified herein. Contractor shall not purchase or acquire any material or product without written approval of the associated submittal from the Engineer.
   B. It shall be the sole responsibility of the Contractor to perform all quality control requirements in acquisition, transport, and handling of specified replacement materials and verifying that the materials meet the specifications provided herein.
   C. Engineer or Competent Professional is responsible for quality assurance of all existing members and Contractor shall provide Engineer/Competent Professional adequate time to evaluate and perform documentation, inspections, and/or testing to ensure the materials meet the minimum criteria provided herein.
   D. All replacement members required must be reviewed and approved by the Engineer. Contractor shall provide all necessary technical datasheets, manufacturer’s information, and grade marks necessary for the Engineer to complete review and approval of materials.

PART 3 - EXECUTION

3.1 DEMOLITION/REMOVAL OF EXISTING MEMBERS
   A. Removal of existing members shall be by the means and methods of the Contractor.
   B. Contractor shall minimize impact to ceilings, flooring, joists, sheathing, and other structural or architectural components during removal of other existing members and shall ensure that remaining members are preserved in a functional state as determined by the Engineer.
   C. If any existing members are damaged during demolition or removal activities not specified to be removed, Contractor shall replace members to the same condition or better than it was originally and shall do so at the Contractors expense.
3.2 EVALUATION/DETERMINATION OF MEMBER CONDITION

A. Existing state of members defined in Section 3.1 shall meet the minimum present installed criteria outlined in this section. For further details and specifications refer to the ‘D’ Sheets of the Drawings.

B. Roof Diaphragm:
   I. Minimum of 4’x8’ sheets shall be required and shall be oriented with the longer length of sheets placed in the north-south direction.

   II. 8d Common Nails shall be installed at 6” O.C. spacing along east and west oriented boundaries with URM Walls.

   III. 8d Common Nails shall be installed at 16” O.C. spacing along north and south oriented boundaries with URM Walls.

   IV. Diaphragm boundary nailing shall be continued along all panel locations (i.e. panel edges and interiors) to a distance of 9’-4” directed inward from the URM Walls.

   V. 6d Common Nails shall be installed at 6” O.C. spacing along all panel edges not considered part of the diaphragm boundary.

   VI. 6d Common Nails shall be installed at 12” O.C. spacing along all panel interiors not considered part of the diaphragm boundary.

   VII. Joists are permitted to be unblocked for entire Roof Diaphragm.

C. 2nd Floor Diaphragm:
   I. For 1”x6” No. 3 DFL(N) Lumber Plank sheathing:
      (i) Planks shall be placed in a double diagonal alignment, as shown on the ‘D’ Sheets of the Drawings.
      (ii) 3 each – 20d Common Nails shall be placed at locations of diaphragm boundaries where plank ties into URM Walls.
      (iii) 2 each – 20d Common Nails shall be placed at locations where planks intersect 2nd Floor Joists.

   II. For 19/32” Sanded Plywood sheathing:
      (i) Minimum of 4’x8’ sheets shall be required and shall be oriented with the longer length of sheets placed in the north-south direction.
      (ii) 10d Common Nails shall be installed at 4” O.C. spacing for boundaries and panel edges oriented in the north-south direction.
      (iii) 10d Common Nails shall be installed at 6” O.C. spacing for boundaries and panel edges oriented in the east-west direction.
      (iv) 10d Common Nails shall be installed at 12” O.C. spacing for panel interiors for all 2nd Floor Diaphragm sheathing.
III. Blocking is required for entirety of joists within the 2nd Floor Diaphragm.

D. Roof Joists:
   I. Joists shall be 2”x6” (actual size) Select Structural DFL(N) spaced at a maximum of 16” O.C. oriented along the east-west direction as shown on the Drawings.
   II. Joists shall span at a minimum, the distance from the URM Wall to the replacement beam, or between replacement beams.

E. 2nd Floor Joists:
   I. Joists shall be 2”x12” (actual size) No. 1 & BTR DFL(N) spaced at a maximum of 16” O.C. oriented along the east-west direction as shown on the Drawings.
   II. Joists shall span at a minimum, the distance from the URM Wall to the replacement beam.

F. Shear/Bearing Walls:
   I. All members associated with the shear/bearing walls shall be at a minimum what is specified on the Drawings.
   II. Contractor shall remedy any degraded masonry at locations where installation of replacement members is to occur (i.e. pointing, replacement of brick, etc.).

G. If any variation exists from specifications in this section, Engineer shall be notified immediately and provided time to evaluate the actual conditions of existing members for structural adequacy.

3.3 INSTALLATION OF EXISTING MEMBER REPLACEMENTS
A. Contractor shall be solely responsible for procurement and delivery of all replacement materials for existing members.

B. All replacement members shall at a minimum meet the requirements specified in this section. If lesser materials are required, they shall be reviewed and approved by the Engineer prior to implementation.

END OF SECTION
APPENDIX C – LOADING CALCULATIONS
VU VILLA STRUCTURAL DESIGN/REHABILITATION

LOADING CALCULATIONS

Kolby Sukut
General Engineering – Civil Option

Montana Tech
2019
VU VILLA STRUCTURAL DESIGN/REHABILITATION
MONTANA TECH

GRAVITY LOADS
### ROOF LEVEL BEAMS:

<table>
<thead>
<tr>
<th>EQUATION NO.</th>
<th>LOAD COMBINATION EQUATIONS</th>
<th>ULTIMATE LOAD (psf)</th>
<th>TIME EFFECT FACTOR (λ)</th>
<th>ULTIMATE LOAD/λ (psf)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>1.4D</td>
<td>24.5</td>
<td>0.6</td>
<td>40.8</td>
</tr>
<tr>
<td>2</td>
<td>1.2D + 1.6L + 0.5(Lr or S)</td>
<td>35.0</td>
<td>0.8</td>
<td>43.8</td>
</tr>
<tr>
<td>3</td>
<td>1.2D + 1.6(Lr or S) + L + 0.5W</td>
<td>65.9</td>
<td>0.8</td>
<td>82.3</td>
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<tr>
<td>4</td>
<td>1.2D + W + L + 0.5(Lr or S)</td>
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<td>1.0</td>
<td>35.0</td>
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<tr>
<td>5</td>
<td>1.2D + E + L + 0.2S</td>
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<td>1.0</td>
<td>26.6</td>
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<tr>
<td>6</td>
<td>0.9D + W</td>
<td>15.7</td>
<td>1.0</td>
<td>15.7</td>
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<tr>
<td>7</td>
<td>0.9D + E</td>
<td>15.7</td>
<td>1.0</td>
<td>15.7</td>
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P<sub>u</sub>, ULTIMATE ROOF BEAM LOAD = 65.9 (psf)

λ, APPLICABLE TIME EFFECT FACTOR = 0.8

### 2ND FLOOR COLUMNS:

<table>
<thead>
<tr>
<th>EQUATION NO.</th>
<th>LOAD COMBINATION EQUATIONS</th>
<th>ULTIMATE LOAD (psf)</th>
<th>TIME EFFECT FACTOR (λ)</th>
<th>ULTIMATE LOAD/λ (psf)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
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<td>25.2</td>
<td>0.6</td>
<td>42.1</td>
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<tr>
<td>2</td>
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<td>0.8</td>
<td>44.6</td>
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<tr>
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<td>0.8</td>
<td>83.1</td>
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<tr>
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<td>1.2D + W + L + 0.5(Lr or S)</td>
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<td>1.2D + E + L + 0.2S</td>
<td>27.2</td>
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<td>27.2</td>
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<td>0.9D + W</td>
<td>16.2</td>
<td>1.0</td>
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<tr>
<td>7</td>
<td>0.9D + E</td>
<td>16.2</td>
<td>1.0</td>
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P<sub>u</sub>, ULTIMATE 2ND FLOOR COLUMN LOAD = 66.5 (psf)

λ, APPLICABLE TIME EFFECT FACTOR = 0.8

### 2ND FLOOR BEAMS:

<table>
<thead>
<tr>
<th>EQUATION NO.</th>
<th>LOAD COMBINATION EQUATIONS</th>
<th>ULTIMATE LOAD (psf)</th>
<th>TIME EFFECT FACTOR (λ)</th>
<th>ULTIMATE LOAD/λ (psf)</th>
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</thead>
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<tr>
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<td>86.6</td>
<td>0.8</td>
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<td>1.2D + W + L + 0.5(Lr or S)</td>
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<tr>
<td>5</td>
<td>1.2D + E + L + 0.2S</td>
<td>134.0</td>
<td>1.0</td>
<td>134.0</td>
</tr>
<tr>
<td>6</td>
<td>0.9D + W</td>
<td>31.3</td>
<td>1.0</td>
<td>31.3</td>
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<tr>
<td>7</td>
<td>0.9D + E</td>
<td>32.4</td>
<td>1.0</td>
<td>32.4</td>
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P<sub>u</sub>, ULTIMATE 2ND FLOOR BEAM LOAD = 194.4 (psf)

λ, APPLICABLE TIME EFFECT FACTOR = 0.8

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<th>EQUATION NO.</th>
<th>LOAD COMBINATION EQUATIONS</th>
<th>ULTIMATE LOAD (psf)</th>
<th>TIME EFFECT FACTOR (λ)</th>
<th>ULTIMATE LOAD/λ (psf)</th>
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<tr>
<td>1</td>
<td>1.4D</td>
<td>50.4</td>
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<td>2</td>
<td>1.2D + 1.6L + 0.5(Lr or S)</td>
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<td>208.8</td>
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<tr>
<td>3</td>
<td>1.2D + 1.6(Lr or S) + L + 0.5W</td>
<td>156.7</td>
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<td>195.9</td>
</tr>
<tr>
<td>4</td>
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</tr>
<tr>
<td>5</td>
<td>1.2D + E + L + 0.2S</td>
<td>117.4</td>
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<td>117.4</td>
</tr>
<tr>
<td>6</td>
<td>0.9D + W</td>
<td>32.4</td>
<td>1.0</td>
<td>32.4</td>
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<td>7</td>
<td>0.9D + E</td>
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<td>1.0</td>
<td>32.4</td>
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</tbody>
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P<sub>u</sub>, ULTIMATE 1ST FLOOR COLUMN LOAD = 167.0 (psf)

λ, APPLICABLE TIME EFFECT FACTOR = 0.8
VU VILLA STRUCTURAL DESIGN/REHABILITATION
MONTANA TECH

DEAD LOAD
## VU VILLA STRUCTURAL DESIGN & REHABILITATION

### LOADING CONDITIONS

#### GRAVITY ANALYSIS

### DEAD LOADS

<table>
<thead>
<tr>
<th>Member Type</th>
<th>Quantity</th>
<th>Unit Load (psf)</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Roof Beams:</strong></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Built-Up-Roofing (5-Ply with Gravel)(^1)</td>
<td>6.5</td>
<td></td>
</tr>
<tr>
<td>Roof Sheathing (3/4&quot; Plywood)(^1)</td>
<td>2.3</td>
<td></td>
</tr>
<tr>
<td>Insulation (Loose)(^1)</td>
<td>0.5</td>
<td></td>
</tr>
<tr>
<td>Roof Joists (2&quot;x6&quot; @ 16&quot; O.C.)(^1)</td>
<td>1.4</td>
<td></td>
</tr>
<tr>
<td>Ceiling (1&quot; Gypsum Drywall)(^1)</td>
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<td></td>
</tr>
<tr>
<td>Member Self-Weight(^2)</td>
<td>1.8</td>
<td></td>
</tr>
<tr>
<td><strong>Total:</strong></td>
<td>17.5</td>
<td></td>
</tr>
<tr>
<td><strong>2nd Floor Beams:</strong></td>
<td></td>
<td></td>
</tr>
<tr>
<td>2nd Floor Columns(^3)</td>
<td>17.5</td>
<td></td>
</tr>
<tr>
<td>Roof Sheathing (3/4&quot; Plywood)(^1)</td>
<td>2.3</td>
<td></td>
</tr>
<tr>
<td>Flooring (1&quot; Hardwood)(^1)</td>
<td>4.0</td>
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</tr>
<tr>
<td>2nd Floor Joists (2&quot;x12&quot; @ 16&quot; O.C.)(^1)</td>
<td>2.9</td>
<td></td>
</tr>
<tr>
<td>Ceiling (1&quot; Gypsum Drywall)(^1)</td>
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<tr>
<td>Member Self-Weight(^2)</td>
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<td><strong>Total:</strong></td>
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</tr>
<tr>
<td><strong>2nd Floor Columns:</strong></td>
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<td></td>
</tr>
<tr>
<td>Roof Beam(^4)</td>
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<td></td>
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<tr>
<td>Member Self-Weight(^2)</td>
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</tr>
<tr>
<td><strong>Total:</strong></td>
<td>18.0</td>
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</tr>
<tr>
<td><strong>1st Floor Columns:</strong></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Roof Beam(^4)</td>
<td>34.8</td>
<td></td>
</tr>
<tr>
<td>Member Self-Weight(^2)</td>
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<tr>
<td><strong>Total:</strong></td>
<td>36.0</td>
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</tr>
</tbody>
</table>

\(^1\)Appendix B of *Design of Wood Structures ASD/LRFD*

\(^2\)Rosboro X-Beam Technical Guide

\(^3\)Density of Wood Structures - The Engineering Toolbox

\(^4\)Values are the Summation of all Loads acting on Specified Member
VU VILLA STRUCTURAL DESIGN/REHABILITATION
MONTANA TECH

LIVE LOAD
### LIVE LOADS

#### Roof Beams:

<table>
<thead>
<tr>
<th>TA, Tributary Area</th>
<th>295.6 (ft²)</th>
</tr>
</thead>
<tbody>
<tr>
<td>θ, Roof Slope</td>
<td>0.0 (°)</td>
</tr>
<tr>
<td>LW, Initial Live Load</td>
<td>20.0 (psf)</td>
</tr>
<tr>
<td>R, Tributary Area Reduction Factor</td>
<td>0.9 (psf)</td>
</tr>
<tr>
<td>L, Roof Live Load</td>
<td>18.1 (psf)</td>
</tr>
</tbody>
</table>

#### 2nd Floor Beams:

<table>
<thead>
<tr>
<th>TA, Tributary Area</th>
<th>295.6 (ft²)</th>
</tr>
</thead>
<tbody>
<tr>
<td>LW, Initial Live Load</td>
<td>100.0 (psf)</td>
</tr>
<tr>
<td>K, Live Load Element Factor</td>
<td>2.0 (psf)</td>
</tr>
<tr>
<td>L, Floor Live Load</td>
<td>86.7 (psf)</td>
</tr>
</tbody>
</table>

#### 2nd Floor Columns:

<table>
<thead>
<tr>
<th>TA, Tributary Area</th>
<th>295.6 (ft²)</th>
</tr>
</thead>
<tbody>
<tr>
<td>LW, Initial Live Load</td>
<td>20.0 (psf)</td>
</tr>
<tr>
<td>R, Tributary Area Reduction Factor</td>
<td>0.9 (psf)</td>
</tr>
<tr>
<td>L, Roof Live Load</td>
<td>18.1 (psf)</td>
</tr>
</tbody>
</table>

#### 1st Floor Columns:

<table>
<thead>
<tr>
<th>TA, Tributary Area</th>
<th>295.6 (ft²)</th>
</tr>
</thead>
<tbody>
<tr>
<td>LW, Initial Live Load</td>
<td>100.0 (psf)</td>
</tr>
<tr>
<td>K, Live Load Element Factor</td>
<td>4.0 (psf)</td>
</tr>
<tr>
<td>L, Floor Live Load</td>
<td>68.6 (psf)</td>
</tr>
</tbody>
</table>
VU VILLA STRUCTURAL DESIGN/REHABILITATION
MONTANA TECH

SNOW LOAD
## SNOW LOAD

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Value</th>
<th>Unit</th>
<th>Source</th>
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</thead>
<tbody>
<tr>
<td>$p_{gr}$, Ground Snow Load</td>
<td>36.8</td>
<td>psf</td>
<td>(Medeek Design - Montana Snow Load Calculator)</td>
</tr>
<tr>
<td>$C_e$, Exposure Factor</td>
<td>0.9</td>
<td></td>
<td>(ASCE 7-02 Table 7-2)</td>
</tr>
<tr>
<td>$C_t$, Thermal Factor</td>
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<td>(ASCE 7-02 Table 7-3)</td>
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<tr>
<td>$I$, Importance Factor</td>
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<td></td>
<td>(ASCE 7-02 Section 4.9.1)</td>
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<tr>
<td>$S$, Flat Roof Snow Load</td>
<td>28.1</td>
<td>psf</td>
<td>(ASCE 7-02 Equ. 7-1)</td>
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</table>
**Wind Loading Properties:**

<table>
<thead>
<tr>
<th>V, Basic Wind Speed(^1)</th>
<th>110 (mph)</th>
<th>(See Footnote 1)</th>
</tr>
</thead>
<tbody>
<tr>
<td>θ, Roof Angle</td>
<td>0 (°)</td>
<td></td>
</tr>
<tr>
<td>h, Roof Height</td>
<td>28 (ft)</td>
<td></td>
</tr>
<tr>
<td>W, Building Width</td>
<td>60.125 (ft)</td>
<td></td>
</tr>
<tr>
<td>Risk Category</td>
<td>III</td>
<td>(ASCE 7-02 Table 1-1)</td>
</tr>
<tr>
<td>a, Width of Pressure Coefficient Zone</td>
<td>6 (ft)</td>
<td></td>
</tr>
<tr>
<td>2a</td>
<td>12 (ft)</td>
<td></td>
</tr>
<tr>
<td>λ, Adjustment Factor for Building Height/Exposure</td>
<td>1.0</td>
<td>(ASCE 7-02 Figure 6-2)</td>
</tr>
<tr>
<td>I, Importance Factor</td>
<td>1.15</td>
<td>(ASCE 7-02 Table 6-1)</td>
</tr>
</tbody>
</table>

**MWFRS:**

<table>
<thead>
<tr>
<th>Zone</th>
<th>E</th>
<th>F</th>
<th>G</th>
<th>H</th>
<th>E</th>
<th>F</th>
<th>G</th>
<th>H</th>
</tr>
</thead>
<tbody>
<tr>
<td>P(_{n}), Design Wind Pressure (psf)(^3)</td>
<td>0.0</td>
<td>0.0</td>
<td>0.0</td>
<td>0.0</td>
<td>0.0</td>
<td>0.0</td>
<td>0.0</td>
<td>0.0</td>
</tr>
<tr>
<td>Uplift (psf)(^4)</td>
<td>-26.6</td>
<td>-16.0</td>
<td>-18.4</td>
<td>-16.0</td>
<td>-26.6</td>
<td>-16.2</td>
<td>-18.4</td>
<td>-16.0</td>
</tr>
</tbody>
</table>

---

\(^1\) Based on ASCE 7-02 Figure 6-1, basic wind speed was approximately 90 mph. From 2012 IBC Figure 1609B, basic wind speed was 120 mph. Local maximum wind data between 1980 and 2019 was evaluated and determined 110 mph was adequate for the design.

\(^2\) Minimum Wind Pressure was assumed to be -16 psf from IBC 2012 1609.6.3

\(^3\) Wind loading was determined using the Simplified Procedure method as specified in ASCE 7-02 Section 6.4. Building and site met all requirements necessary for the Simplified Procedure as outlined in ASCE 7-02 Section 6.4.1.1.

\(^4\) Values used for connection design only, for conservative design of members assumed no uplift.

\(^5\) Values were assumed to be zero as this provided the most conservative design of gravity load members.
VU VILLA STRUCTURAL DESIGN/REHABILITATION
MONTANA TECH

SEISMIC LOAD
### Inputs

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Value</th>
<th>Source</th>
</tr>
</thead>
<tbody>
<tr>
<td>$S_1$, Max 1-s Spectral Response Acceleration</td>
<td>0.142</td>
<td>ASCE 7-02 Equ 9.4.1.2.4-2</td>
</tr>
<tr>
<td>$S_s$, Max. Short-Period Spectral Response Acceleration</td>
<td>0.449</td>
<td>ASCE 7-02 Equ 9.4.1.2.4-1</td>
</tr>
<tr>
<td>$h$, Total Height Above Base-Level</td>
<td>28 (ft)</td>
<td>(ASCE 7-02 Table 9.5.5.3.2)</td>
</tr>
<tr>
<td>Site Soil Classification</td>
<td>B</td>
<td></td>
</tr>
<tr>
<td>$T_a$, Approximate Fundament Period</td>
<td>0.24 (s)</td>
<td>(ASCE 7-02 Table 9.5.5.3.2)</td>
</tr>
<tr>
<td>$F_a$, Site Coefficient for 1-s Spectra</td>
<td>1.0</td>
<td>(ASCE 7-02 Table 9.5.5.3.2)</td>
</tr>
<tr>
<td>$S_{D1}$, Design 1-s Spectral Acceleration</td>
<td>0.095</td>
<td>(ASCE 7-02 Table 9.5.5.3.2)</td>
</tr>
<tr>
<td>$S_{D2}$, Design Short-Period Spectral Acceleration</td>
<td>0.299</td>
<td>(ASCE 7-02 Table 9.5.5.3.2)</td>
</tr>
<tr>
<td>$S_{M1}$, Adjusted 1-s Spectral Acceleration</td>
<td>0.142</td>
<td>(ASCE 7-02 Table 9.5.5.3.2)</td>
</tr>
<tr>
<td>$S_{MS}$, Adjusted Short-Period Spectral Accelerations</td>
<td>0.449</td>
<td>(ASCE 7-02 Table 9.5.5.3.2)</td>
</tr>
<tr>
<td>$S_{DS}$, Design Short-Period Spectral Acceleration</td>
<td>0.299</td>
<td>(ASCE 7-02 Table 9.5.5.3.2)</td>
</tr>
<tr>
<td>$S_v$, Design Spectral Response Acceleration</td>
<td>0.299</td>
<td>(ASCE 7-02 Table 9.5.5.3.2)</td>
</tr>
<tr>
<td>$C_v$, Building Period Coefficient</td>
<td>0.02</td>
<td>(ASCE 7-02 Table 9.5.5.3.2)</td>
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<tr>
<td>$m$, Exponential Coefficient</td>
<td>0.75</td>
<td>(ASCE 7-02 Table 9.5.5.3.2)</td>
</tr>
<tr>
<td>$C_s$, Seismic Response Coefficient</td>
<td>0.249</td>
<td>(ASCE 7-02 Table 9.5.5.3.2)</td>
</tr>
<tr>
<td>$C_{p}$, Deflection Amplification Factor</td>
<td>1.0</td>
<td>(ASCE 7-02 Table 9.5.5.3.2)</td>
</tr>
<tr>
<td>$I$, Importance Factor</td>
<td>1.25</td>
<td>(ASCE 7-02 Table 9.5.5.3.2)</td>
</tr>
<tr>
<td>$k$, Structure Period Exponent</td>
<td>1.0</td>
<td>(ASCE 7-02 Table 9.5.5.3.2)</td>
</tr>
<tr>
<td>$R$, Response Modification Factor</td>
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<td>(ASCE 7-02 Table 9.5.5.3.2)</td>
</tr>
<tr>
<td>$C_D$, Deflection Amplification Factor</td>
<td>1.25</td>
<td>(ASCE 7-02 Table 9.5.5.3.2)</td>
</tr>
<tr>
<td>$C_{D2}$, Site Coefficient for Short Spectra</td>
<td>1.0</td>
<td>(ASCE 7-02 Table 9.5.5.3.2)</td>
</tr>
<tr>
<td>$C_{S1}$, Seismic Response Coefficient</td>
<td>0.299</td>
<td>(ASCE 7-02 Table 9.5.5.3.2)</td>
</tr>
<tr>
<td>$C_{S2}$, Short-Period Spectral Coefficient</td>
<td>0.299</td>
<td>(ASCE 7-02 Table 9.5.5.3.2)</td>
</tr>
<tr>
<td>$C_{S1}$, Short-Period Spectral Coefficient</td>
<td>0.299</td>
<td>(ASCE 7-02 Table 9.5.5.3.2)</td>
</tr>
<tr>
<td>$C_{S2}$, Short-Period Spectral Coefficient</td>
<td>0.299</td>
<td>(ASCE 7-02 Table 9.5.5.3.2)</td>
</tr>
<tr>
<td>$C_{S1}$, Short-Period Spectral Coefficient</td>
<td>0.299</td>
<td>(ASCE 7-02 Table 9.5.5.3.2)</td>
</tr>
<tr>
<td>$C_{S2}$, Short-Period Spectral Coefficient</td>
<td>0.299</td>
<td>(ASCE 7-02 Table 9.5.5.3.2)</td>
</tr>
<tr>
<td>$C_{S1}$, Short-Period Spectral Coefficient</td>
<td>0.299</td>
<td>(ASCE 7-02 Table 9.5.5.3.2)</td>
</tr>
<tr>
<td>$C_{S2}$, Short-Period Spectral Coefficient</td>
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<td>(ASCE 7-02 Table 9.5.5.3.2)</td>
</tr>
<tr>
<td>$C_{S1}$, Short-Period Spectral Coefficient</td>
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<td>(ASCE 7-02 Table 9.5.5.3.2)</td>
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<tr>
<td>$C_{S2}$, Short-Period Spectral Coefficient</td>
<td>0.299</td>
<td>(ASCE 7-02 Table 9.5.5.3.2)</td>
</tr>
<tr>
<td>$C_{S1}$, Short-Period Spectral Coefficient</td>
<td>0.299</td>
<td>(ASCE 7-02 Table 9.5.5.3.2)</td>
</tr>
<tr>
<td>$C_{S2}$, Short-Period Spectral Coefficient</td>
<td>0.299</td>
<td>(ASCE 7-02 Table 9.5.5.3.2)</td>
</tr>
<tr>
<td>$C_{S1}$, Short-Period Spectral Coefficient</td>
<td>0.299</td>
<td>(ASCE 7-02 Table 9.5.5.3.2)</td>
</tr>
</tbody>
</table>

### Vertical Seismic Forces

<table>
<thead>
<tr>
<th>Source</th>
<th>Roof</th>
<th>2nd Floor</th>
<th>Total</th>
</tr>
</thead>
<tbody>
<tr>
<td>D, Dead Load (psf)</td>
<td>17.5</td>
<td>34.8</td>
<td>52.3</td>
</tr>
<tr>
<td>S_{D1}, Design Short-Period Spectra Response</td>
<td>0.299</td>
<td>0.299</td>
<td>0.299</td>
</tr>
<tr>
<td>E, Earthquake Load (psf)</td>
<td>1.0</td>
<td>2.1</td>
<td>3.1</td>
</tr>
</tbody>
</table>
VU VILLA STRUCTURAL DESIGN/REHABILITATION
MONTANA TECH

LATERAL LOADS
### ROOF DIAPHRAGM:

<table>
<thead>
<tr>
<th>EQUATION NO.</th>
<th>LOAD COMBINATION EQUATIONS</th>
<th>ULTIMATE LOAD (plf)</th>
<th>TIME EFFECT FACTOR (λ)</th>
<th>ULTIMATE LOAD/λ (plf)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>1.4D</td>
<td>0.0</td>
<td>0.6</td>
<td>0.0</td>
</tr>
<tr>
<td>2</td>
<td>1.2D + 1.6L + 0.5(Lr or S)</td>
<td>0.0</td>
<td>0.8</td>
<td>0.0</td>
</tr>
<tr>
<td>3</td>
<td>1.2D + 1.6(Lr or S) + L + 0.5W</td>
<td>66.2</td>
<td>0.8</td>
<td>82.8</td>
</tr>
<tr>
<td>4</td>
<td>1.2D + W + L + 0.5(Lr or S)</td>
<td>132.5</td>
<td>1.0</td>
<td>132.5</td>
</tr>
<tr>
<td>5</td>
<td>1.2D + E + L + 0.2S</td>
<td>566.8</td>
<td>1.0</td>
<td>566.8</td>
</tr>
<tr>
<td>6</td>
<td>0.9D + W</td>
<td>132.5</td>
<td>1.0</td>
<td>132.5</td>
</tr>
<tr>
<td>7</td>
<td>0.9D + E</td>
<td>566.8</td>
<td>1.0</td>
<td>566.8</td>
</tr>
</tbody>
</table>

\[ P_u, \text{ ULTIMATE ROOF DIAPHRAGM LOAD} = 566.8 \text{ (plf)} \]

\[ \lambda, \text{ APPLICABLE TIME EFFECT FACTOR} = 1.0 \]

### 2ND FLOOR DIAPHRAGM:

<table>
<thead>
<tr>
<th>EQUATION NO.</th>
<th>LOAD COMBINATION EQUATIONS</th>
<th>ULTIMATE LOAD (plf)</th>
<th>TIME EFFECT FACTOR (λ)</th>
<th>ULTIMATE LOAD/λ (plf)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>1.4D</td>
<td>0.0</td>
<td>0.6</td>
<td>0.0</td>
</tr>
<tr>
<td>2</td>
<td>1.2D + 1.6L + 0.5(Lr or S)</td>
<td>0.0</td>
<td>0.8</td>
<td>0.0</td>
</tr>
<tr>
<td>3</td>
<td>1.2D + 1.6(Lr or S) + L + 0.5W</td>
<td>154.6</td>
<td>0.8</td>
<td>193.2</td>
</tr>
<tr>
<td>4</td>
<td>1.2D + W + L + 0.5(Lr or S)</td>
<td>309.1</td>
<td>1.0</td>
<td>309.1</td>
</tr>
<tr>
<td>5</td>
<td>1.2D + E + L + 0.2S</td>
<td>1,268.3</td>
<td>1.0</td>
<td>1,268.3</td>
</tr>
<tr>
<td>6</td>
<td>0.9D + W</td>
<td>309.1</td>
<td>1.0</td>
<td>309.1</td>
</tr>
<tr>
<td>7</td>
<td>0.9D + E</td>
<td>1,268.3</td>
<td>1.0</td>
<td>1,268.3</td>
</tr>
</tbody>
</table>

\[ P_u, \text{ ULTIMATE 2ND FLOOR DIAPHRAGM LOAD} = 1,268.3 \text{ (plf)} \]

\[ \lambda, \text{ APPLICABLE TIME EFFECT FACTOR} = 1.0 \]

### ROOF SHEAR WALLS:

<table>
<thead>
<tr>
<th>EQUATION NO.</th>
<th>LOAD COMBINATION EQUATIONS</th>
<th>ULTIMATE LOAD (plf)</th>
<th>TIME EFFECT FACTOR (λ)</th>
<th>ULTIMATE LOAD/λ (plf)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>1.4D</td>
<td>0.0</td>
<td>0.6</td>
<td>0.0</td>
</tr>
<tr>
<td>2</td>
<td>1.2D + 1.6L + 0.5(Lr or S)</td>
<td>0.0</td>
<td>0.8</td>
<td>0.0</td>
</tr>
<tr>
<td>3</td>
<td>1.2D + 1.6(Lr or S) + L + 0.5W</td>
<td>154.6</td>
<td>0.8</td>
<td>193.2</td>
</tr>
<tr>
<td>4</td>
<td>1.2D + W + L + 0.5(Lr or S)</td>
<td>309.1</td>
<td>1.0</td>
<td>309.1</td>
</tr>
<tr>
<td>5</td>
<td>1.2D + E + L + 0.2S</td>
<td>1,342.3</td>
<td>1.0</td>
<td>1,342.3</td>
</tr>
<tr>
<td>6</td>
<td>0.9D + W</td>
<td>1,342.3</td>
<td>1.0</td>
<td>1,342.3</td>
</tr>
<tr>
<td>7</td>
<td>0.9D + E</td>
<td>1,342.3</td>
<td>1.0</td>
<td>1,342.3</td>
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</tbody>
</table>

\[ P_u, \text{ ULTIMATE ROOF SHEAR WALL LOAD} = 1,342.3 \text{ (plf)} \]

\[ \lambda, \text{ APPLICABLE TIME EFFECT FACTOR} = 1.0 \]

### 2ND FLOOR SHEAR WALLS:

<table>
<thead>
<tr>
<th>EQUATION NO.</th>
<th>LOAD COMBINATION EQUATIONS</th>
<th>ULTIMATE LOAD (plf)</th>
<th>TIME EFFECT FACTOR (λ)</th>
<th>ULTIMATE LOAD/λ (plf)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>1.4D</td>
<td>0.0</td>
<td>0.6</td>
<td>0.0</td>
</tr>
<tr>
<td>2</td>
<td>1.2D + 1.6L + 0.5(Lr or S)</td>
<td>0.0</td>
<td>0.8</td>
<td>0.0</td>
</tr>
<tr>
<td>3</td>
<td>1.2D + 1.6(Lr or S) + L + 0.5W</td>
<td>154.6</td>
<td>0.8</td>
<td>193.2</td>
</tr>
<tr>
<td>4</td>
<td>1.2D + W + L + 0.5(Lr or S)</td>
<td>309.1</td>
<td>1.0</td>
<td>309.1</td>
</tr>
<tr>
<td>5</td>
<td>1.2D + E + L + 0.2S</td>
<td>1,716.2</td>
<td>1.0</td>
<td>1,716.2</td>
</tr>
<tr>
<td>6</td>
<td>0.9D + W</td>
<td>309.1</td>
<td>1.0</td>
<td>309.1</td>
</tr>
<tr>
<td>7</td>
<td>0.9D + E</td>
<td>1,716.2</td>
<td>1.0</td>
<td>1,716.2</td>
</tr>
</tbody>
</table>

\[ P_u, \text{ ULTIMATE 2ND FLOOR SHEAR WALL LOAD} = 1,716.2 \text{ (plf)} \]

\[ \lambda, \text{ APPLICABLE TIME EFFECT FACTOR} = 1.0 \]
VU VILLA STRUCTURAL DESIGN/REHABILITATION
MONTANA TECH

CHORD & DRAG ANALYSIS
### CHORD FORCES 2:

<table>
<thead>
<tr>
<th></th>
<th>LONGITUDINAL</th>
<th>TRANSVERSE</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>ROOF LEVEL</td>
<td>2ND FLOOR</td>
</tr>
<tr>
<td>$\omega_u$, Ultimate Uniform Load (lb/ft)</td>
<td>566.8</td>
<td>1,268.3</td>
</tr>
<tr>
<td>$M_u$, Ultimate Diaphragm Moment (k-ft)</td>
<td>256.1</td>
<td>143.3</td>
</tr>
<tr>
<td>$T_u/C_u$, Ultimate Chord Tension/Compression (k)</td>
<td>4.3</td>
<td>2.4</td>
</tr>
<tr>
<td>$t_u/c_u$, Ultimate Uniform Chord Tension/Compression (lb/ft)</td>
<td>70.9</td>
<td>79.3</td>
</tr>
</tbody>
</table>

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### DIAPHRAGM DRAG FORCES 2:

<table>
<thead>
<tr>
<th></th>
<th>LONGITUDINAL</th>
<th>TRANSVERSE</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>ROOF LEVEL</td>
<td>2ND FLOOR</td>
</tr>
<tr>
<td>$\omega_d$, Ultimate Uniform Load (lb/ft)</td>
<td>566.8</td>
<td>1,268.3</td>
</tr>
<tr>
<td>$P_u$, Ultimate Concentrated Load (lb/ft)</td>
<td>34.1</td>
<td>38.1</td>
</tr>
<tr>
<td>$V_u$, Ultimate Drag Shear (k)</td>
<td>17.0</td>
<td>12.7</td>
</tr>
<tr>
<td>$v_u$, Ultimate Unit Drag Shear (lb/ft)</td>
<td>283.4</td>
<td>211.4</td>
</tr>
</tbody>
</table>

---

### SHEAR WALL DRAG FORCES 2:

<table>
<thead>
<tr>
<th></th>
<th>LONGITUDINAL</th>
<th>TRANSVERSE</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>ROOF LEVEL</td>
<td>2ND FLOOR</td>
</tr>
<tr>
<td>$\omega_s$, Ultimate Uniform Load (lb/ft)</td>
<td>1,342.3</td>
<td>1,716.2</td>
</tr>
<tr>
<td>$P_s$, Ultimate Concentrated Load (lb/ft)</td>
<td>80.7</td>
<td>103.2</td>
</tr>
<tr>
<td>$V_s$, Ultimate Drag Shear (k)</td>
<td>40.4</td>
<td>34.4</td>
</tr>
<tr>
<td>$v_s$, Ultimate Unit Drag Shear (lb/ft)</td>
<td>671.2</td>
<td>572.1</td>
</tr>
</tbody>
</table>

---

1See Lateral Load Combination Equation Sheet for Values
2Drag Forces for Shear Walls used the Seismic Force for a Vertical Element ($F_s$) while Drag & Chord Forces for the Diaphragm used the Horizontal Force ($F_{hs}$)
3See Hand Calculations for Specific Determination of Values
Root Level - Transverse

1. See Lateral Load Combination Equation Sheet
2. Forces apply for both Longitudinal & Transverse Directions due to square dimensions of building.

**Shear Wall Drag:**

\[ P_u = W_u \times \text{Dia} \times L \]
\[ = (1342 \times 13.5\% \times 60.125 \text{ ft}) \times \frac{1}{10000} = 80.7 \text{ K} \]

\[ V_u = \frac{P_u}{2} = \frac{80.7 \text{ K}}{2} = 40.4 \text{ K} \]

\[ V_u = \sqrt{V_u} = \sqrt{40.4 \text{ K} \times \frac{1000 \text{ lb}}{1000 \text{ ft}}} \times \frac{1}{60.125 \text{ ft}} = 671.0 \text{ ft/s} \]

**Diaphragm Drag:**

\[ P_u = W_u \times \text{Dia} \times L = (566.8 \times 13.5\% \times 60.125 \text{ ft}) \times \frac{1}{10000} = 34.1 \text{ K} \]

\[ V_u = \frac{P_u}{2} = \frac{34.1 \text{ K}}{2} = 17.0 \text{ K} \]

\[ V_u = \sqrt{V_u} = \sqrt{17.0 \text{ K} \times \frac{1000 \text{ lb}}{1000 \text{ ft}}} \times \frac{1}{60.125 \text{ ft}} = 283.4 \text{ ft/s} \]
Root Level - Transverse (Cont'd)

Diaphragm Chord:

\[ M_u = \frac{W_uD_u}{g} \times L^2 = \left( \frac{566.8 \text{ lb/ft}}{60.125 \text{ lb/ft}^2} \right) \times \left( \frac{1}{14226} \right)^2 \times \left( \frac{1}{256.1 \text{ k-ft}} \right) \]

\[ T_u = C_u = \frac{M_u}{W} = \frac{258.1 \text{ k-ft}}{60.125 \text{ lb/ft}^2} = 4.28 \text{ k-ft} \]

\[ T_u = C_u = \frac{L}{T_u} = \frac{4.781 \text{ lb}}{60.25 \text{ lb/ft}} = 76.9 \text{ lb/ft} \]

Root Level - Longitudinal

\[ W_{u, SW} = 1342.2 \text{ lb/ft} \]

\[ W_{u, Din} = 566.8 \text{ lb/ft} \]
Shear Wall Drag:

\[ P_u = W_u \cdot s_w \cdot W = 80.7 \text{k} \]
\[ V_u = 40.4 \text{k} \]
\[ V_u = 671.2 \text{ lb/ft} \]

Diaphragm Drag:

\[ P_u = W_u \cdot d_{dia} \cdot W = 34.1 \text{k} \]
\[ V_u = 17.0 \text{k} \]
\[ V_u = 283.4 \text{ lb/ft} \]

Diaphragm Chord:

\[ M_u = \frac{W_u \cdot d_{dia} \cdot W^2}{8} = 256.1 \text{k-ft} \]
\[ T_u = C_u = \frac{M_u}{L} = 4.26 \text{k} \]
\[ t_u = C_u = \frac{T_u}{C_u} = 70.9 \text{ lb/ft} \]

*Same as Transverse due to same dimensions*
2nd Floor - Transverse

Shear Wall Drag:

\[ P_u = \omega_{w,sw} \cdot L = (1.716.2 \text{ lb}) \left( 60.125 \text{ ft} \right) = 103.2 \text{ k} \]

\[ V_u = \frac{P_u}{2} = 51.6 \text{ k} \]

\[ V_u = \frac{V}{W} = \left( 51.75 \text{ k} \right) \left( \frac{1000 \text{ lb/ft}}{k} \right) \left( \frac{60.125 \text{ ft}^2}{k} \right) = 858.1 \text{ lb/ft} \]

Diaphragm Drag:

\[ P_u = \omega_{w,Di} \cdot L = (1.268.3 \text{ lb/ft}) \left( 60.125 \text{ ft} \right) = 76.3 \text{ k} \]

\[ V_u = \frac{P_u}{2} = 38.1 \text{ k} \]

\[ V_u = V_u / W = \frac{38.3 \text{ k} (1000 \text{ lb/ft})}{60.125 \text{ ft}^2} = 634.1 \text{ lb/ft} \]

Diaphragm Chord:

\[ M_u = \omega_{w,Di} \cdot \frac{L}{2} = \left( 1.268.3 \text{ lb/ft} \right) \left( 60.125 \text{ ft} \right)^2 \left( \frac{L}{1000 \text{ lb/ft}} \right) = 573.1 \text{ k-ft} \]

\[ T_u = C_u = M_u / W = \left( 573.1 \text{ k-ft} \right) \left( 60.125 \text{ ft} \right) = 9.53 \text{ k} \]
2nd Floor - Longitudinal

Existing Masonry Interior Shearwall

$W_n, sw = 1,716.7 \text{ lb/ft}$

$W_n, Dw = 1,268.3 \text{ lb/ft}$

**Shear Wall Drag:**

$P_n = W_n, sw \cdot W = (1,716.7 \text{ lb/ft}) \left( 60.125 \text{ ft} \cdot \frac{k}{1000 \text{ lb}} \right) = 103.2 \text{ k}$

$V_n = \frac{P_n}{3} = \frac{103.2 \text{ k}}{3} = 34.4 \text{ k}$

$\bar{V_n} = \frac{V_n}{L} = \left( 8143 \text{ PLF} \cdot \frac{1000 \text{ lb}}{k} \right) \left( 60.125 \text{ ft} \right) = 572.1 \text{ lb/ft}$

**Diaphragm Drag:**

$P_n = W_n, Dw \cdot \frac{W}{2} = (1,268.3 \text{ lb/ft}) \left( \frac{60.125 \text{ ft}}{2} \right) \left( \frac{k}{1000 \text{ lb}} \right) = 38.1 \text{ k}$

$V_n = \frac{P_n}{3} = 12.7 \text{ k}$

$\bar{V_n} = \frac{V_n}{L} = \left( 12.7 \text{ k} \cdot \frac{1000 \text{ lb}}{k} \right) \left( 60.125 \text{ ft} \right) = 811.4 \text{ lb/ft}$

**Diaphragm Chord:**

$M_n = (P_n, Dw)(\frac{W}{2})^2 = (1,268.3 \text{ lb/ft}) \left( \frac{60.125 \text{ ft}}{2} \right)^2 \frac{k}{1000 \text{ lb}} = 143.3 \text{ k-ft}$

$T_n = \frac{M_n}{L} = \left( 143.3 \text{ k-ft} \right) \left( 60.125 \text{ ft} \right) = 79.3 \text{ k-ft}$
VU VILLA STRUCTURAL DESIGN/REHABILITATION
MONTANA TECH

WIND LOAD
VU VILLA STRUCTURAL DESIGN & REHABILITATION
LOADING CONDITIONS
LATERAL ANALYSIS

WIND LOADING

**Inputs**

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>V, Basic Wind Speed</td>
<td>110 (mph)</td>
</tr>
<tr>
<td>θ, Roof Angle</td>
<td>0 (⁰)</td>
</tr>
<tr>
<td>h, Roof Height</td>
<td>28 (ft)</td>
</tr>
<tr>
<td>W, Building Width</td>
<td>60.125 (ft)</td>
</tr>
<tr>
<td>Risk Category</td>
<td>III</td>
</tr>
<tr>
<td>a, Width of Pressure Coefficient Zone</td>
<td>6 (ft)</td>
</tr>
<tr>
<td>2a</td>
<td>12 (ft)</td>
</tr>
<tr>
<td>λ, Adjustment Factor for Building Height/Exposure</td>
<td>1.0</td>
</tr>
<tr>
<td>I, Importance Factor</td>
<td>1.15</td>
</tr>
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MWFRS:

<table>
<thead>
<tr>
<th>Zone</th>
<th>Roof Level</th>
<th>2nd Floor</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>A</td>
<td>B</td>
</tr>
<tr>
<td>$P_{s30}$, Net Design Wind Pressure (psf)</td>
<td>19.2</td>
<td>-10.0</td>
</tr>
<tr>
<td>$P_u$, Design Wind Pressure (psf)</td>
<td>22.1</td>
<td>16.0</td>
</tr>
<tr>
<td>h, Tributary Height (ft)</td>
<td>6.0</td>
<td>0.0</td>
</tr>
<tr>
<td>Uniform Wind Load along Diaphragm (lb/ft)</td>
<td>132.5</td>
<td>0.0</td>
</tr>
<tr>
<td>W, Max. Wind Load along Diaphragm (lb/ft)</td>
<td>132.5</td>
<td>309.1</td>
</tr>
</tbody>
</table>

---

1Based on ASCE 7-02 Figure 6-1, basic wind speed was approximately 90 mph. From 2012 IBC, basic wind speed was 120 mph. Local maximum wind data between 1980 and 2019 was evaluated and determined 110 mph was adequate for the design.

2Minimum Wind Pressure was assumed to be 16 psf from IBC 2012 1609.6.3

3Wind loading was determined using the Simplified Procedure method as specified in ASCE 7-02 Section 6.4. Building and site met all requirements necessary for the Simplified Procedure as outlined in ASCE 7-02 Section 6.4.1.1.
VU VILLA STRUCTURAL DESIGN/REHABILITATION
MONTANA TECH

SEISMIC LOAD
### Inputs

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Value</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>$S_a$</td>
<td>0.142</td>
<td>Max. 1-s Spectral Response Acceleration</td>
</tr>
<tr>
<td>$S_s$</td>
<td>0.449</td>
<td>Max. Short-Period Spectral Response Acceleration</td>
</tr>
<tr>
<td>$h_n$</td>
<td>28</td>
<td>Total Height Above Base-Level</td>
</tr>
<tr>
<td>$T_T$</td>
<td>0.06</td>
<td>Initial Period</td>
</tr>
<tr>
<td>$T_T$</td>
<td>0.32</td>
<td>Short Period</td>
</tr>
<tr>
<td>$T_L$</td>
<td>6.00</td>
<td>Long Period</td>
</tr>
<tr>
<td>Site Soil Classification</td>
<td>B</td>
<td></td>
</tr>
<tr>
<td>$C_t$</td>
<td>0.02</td>
<td>Building Period Exponent</td>
</tr>
<tr>
<td>$R$</td>
<td>1.5</td>
<td>Response Modification Factor</td>
</tr>
<tr>
<td>$x$</td>
<td>0.75</td>
<td>Exponential Coefficient</td>
</tr>
<tr>
<td>$C_S$</td>
<td>0.249</td>
<td>Seismic Response Coefficient</td>
</tr>
<tr>
<td>$F_D$</td>
<td>1.25</td>
<td>Deflection Amplification Factor</td>
</tr>
<tr>
<td>$C_M$</td>
<td>0.249</td>
<td>Seismic Coefficient for 1-s Spectra</td>
</tr>
<tr>
<td>$F_s$</td>
<td>1</td>
<td>Site Coefficient for Short Spectra</td>
</tr>
<tr>
<td>$F_T$</td>
<td>0.249</td>
<td>Site Coefficient for Short Spectra</td>
</tr>
<tr>
<td>$F_{DS}$</td>
<td>1.25</td>
<td>Seismic Design Category</td>
</tr>
<tr>
<td>$F_{DS}$</td>
<td>1</td>
<td>Structure Period Exponent</td>
</tr>
<tr>
<td>$R_s$</td>
<td>1</td>
<td>Seismic Force of Story (k)</td>
</tr>
<tr>
<td>$F_{DS}$</td>
<td>509.5</td>
<td>Total Gravity Load of Story (k)</td>
</tr>
<tr>
<td>$W_x$</td>
<td>227.7</td>
<td>Story Height (ft)</td>
</tr>
<tr>
<td>$W_x$</td>
<td>8,474</td>
<td>Total Gravity Load of Story (k)</td>
</tr>
<tr>
<td>$W_x$</td>
<td>1,268.3</td>
<td>Total Gravity Load of Story (k)</td>
</tr>
<tr>
<td>$F_x$</td>
<td>80.7</td>
<td>Seismic Force of Story (k)</td>
</tr>
<tr>
<td>$F_x$</td>
<td>1,716.2</td>
<td>Seismic Force of Story (k)</td>
</tr>
<tr>
<td>$F_x$</td>
<td>34.1</td>
<td>Max Diaphragm Design Force (k)</td>
</tr>
<tr>
<td>$F_x$</td>
<td>17.0</td>
<td>Min Diaphragm Design Force (k)</td>
</tr>
<tr>
<td>$F_x$</td>
<td>606.8</td>
<td>Total Gravity Load of Story (k)</td>
</tr>
</tbody>
</table>

---

**Vu Villa Structural Design and Rehabilitation**

**Loading Conditions**

**Montana Tech**
APPENDIX D – DESIGN CALCULATIONS
VU VILLA STRUCTURAL DESIGN/REHABILITATION

DESIGN CALCULATIONS

Kolby Sukut
General Engineering – Civil Option

Montana Tech
2019
VU VILLA STRUCTURAL DESIGN/REHABILITATION
MONTANA TECH

REPLACEMENT MEMBER DESIGN
VU VILLA STRUCTURAL DESIGN/REHABILITATION
MONTANA TECH

BEAM DESIGN
Roof Beam Design
**VU VILLA STRUCTURAL DESIGN/REHABILITATION**  
**REPLACEMENT MEMBER DESIGN**  

**ROOF BEAMS**

### Beam Type:
5 1/2" x 21" 2F-V4 (DF/DF) GluLam

#### Beam Properties:

<table>
<thead>
<tr>
<th>Property</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>b, breadth</td>
<td>5 1/2 (ft)</td>
</tr>
<tr>
<td>d, depth</td>
<td>21 (in)</td>
</tr>
<tr>
<td>A, Cross-Sectional Area</td>
<td>110.3 (in²)</td>
</tr>
<tr>
<td>S₀, Section Modulus</td>
<td>404.3 (k-in)</td>
</tr>
<tr>
<td>I₀, Moment of Inertia</td>
<td>4,245 (in⁴)</td>
</tr>
<tr>
<td>t₀, Moment of Inertia</td>
<td>291 (in³)</td>
</tr>
</tbody>
</table>

#### Reference Design Values (NDS 2015 Table 5A):

<table>
<thead>
<tr>
<th>Property</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>L₀, Bending</td>
<td>2,400 (psf)</td>
</tr>
<tr>
<td>t₀, Bending</td>
<td>1,450 (psf)</td>
</tr>
<tr>
<td>L₀, Shear</td>
<td>265 (psf)</td>
</tr>
<tr>
<td>t₀, Shear</td>
<td>230 (psf)</td>
</tr>
<tr>
<td>L₀, Compression</td>
<td>1,650 (psf)</td>
</tr>
<tr>
<td>t₀, Shear</td>
<td>1,100 (psf)</td>
</tr>
<tr>
<td>E₀</td>
<td>1,800,000 (psi)</td>
</tr>
<tr>
<td>E₀</td>
<td>1,800,000 (psi)</td>
</tr>
<tr>
<td>E₀</td>
<td>950,000 (psi)</td>
</tr>
<tr>
<td>E₀</td>
<td>850,000 (psi)</td>
</tr>
</tbody>
</table>

#### Gravity Loading

##### Loading Conditions:

- w, Tributary Width 15.00 (ft)
- ω, Ultimate Load 65.87 (psf)
- ω, Ultimate Load 566.8 (plf)

#### Lateral Loading

##### Loading Conditions:

- w, Tributary Width 15.00 (ft)
- ω, Ultimate Load 65.87 (psf)
- ω, Ultimate Load 566.8 (plf)

### Check for Shear (Transverse direction):

- \( \Phi \) = 0.75
- \( \lambda \) = 0.80
- \( \Phi_{V} \) = 30.26 (k-ft) (NDS 2015 Table 5.3.1)
- \( \Phi_{V} = V_{u} \)  

### Check for Bending (Transverse direction):

- \( \lambda = 0.80 \)  
- \( \Phi = 2.88 \) (NDS 2015 Table 3.3.3)
- \( \Phi_{V} = 397.44 \) (NDS 2015 Table 5.3.7)
- \( \Phi_{V} = V_{u} \)  

### Check for Deflection

- \( \gamma_{i} = 0.44 \) (in) (NDS Section 3.5.1)  
- \( \gamma_{i} = 0.44 \) (in) (NDS Section 3.5.1)  

---

**Villa Structural Design and Rehabilitation**  
**Replacement Member Design**

Montana Tech
VU VILLA DESIGN/REHABILITATION
ROOF BEAM - LONGITUDINAL ANALYSIS

CODE: ANSI/AWC NDS-2012 LRFD
ANALYSIS TYPE: Code Group Verification

CODE GROUP: 2 Roof Beams
MEMBER: 40 Beam_40 POINT: 1 COORDINATE: x = 0.00 L = 0.00 ft

LOADS:
Governing Load Case: 44 Longitudinal_Roof Beam (20+22)*1.00+(24+25+26)*1.20+30*0.50

MATERIAL: GL VG SOFTWOOD 24F-V4 DF/DF
Structural Glued Laminated Softwood Timber - Tab.5A
Ft=1.10 ksi Fc=1.65 ksi
Fby=1.85 ksi Fvc=0.27 ksi Fcpc=0.65 ksi Ey=1800.00 ksi
Fb=2.40 ksi Fbvz=0.27 ksi Fcpvz=0.65 ksi Ez=1800.00 ksi

SECTION PARAMETERS: 5.5x21

<table>
<thead>
<tr>
<th>d</th>
<th>21.00 in</th>
</tr>
</thead>
<tbody>
<tr>
<td>b</td>
<td>5.50 in</td>
</tr>
<tr>
<td>Ay</td>
<td>76.867 in²</td>
</tr>
<tr>
<td>Az</td>
<td>76.867 in²</td>
</tr>
<tr>
<td>Iy</td>
<td>4865.000 in⁴</td>
</tr>
<tr>
<td>Iz</td>
<td>252.400 in⁴</td>
</tr>
<tr>
<td>Ix</td>
<td>1864.665 in⁴</td>
</tr>
<tr>
<td>Sy</td>
<td>98.498 in³</td>
</tr>
<tr>
<td>Sz</td>
<td>98.498 in³</td>
</tr>
</tbody>
</table>

MEMBER PARAMETERS:

| FcEy = INF ksi | FeZ = INF ksi | FbE = INF ksi |

INTERNAL FORCES AND ACTUAL STRESSES:

<table>
<thead>
<tr>
<th>N</th>
<th>0.22 kip</th>
</tr>
</thead>
<tbody>
<tr>
<td>fc</td>
<td>0.00 ksi</td>
</tr>
<tr>
<td>Mx</td>
<td>-0.01 kip/ft</td>
</tr>
<tr>
<td>M</td>
<td>-15.90 kip/ft</td>
</tr>
<tr>
<td>Mz</td>
<td>-0.00 kip/ft</td>
</tr>
<tr>
<td>V</td>
<td>10.11 kip</td>
</tr>
<tr>
<td>Vz</td>
<td>-0.00 kip</td>
</tr>
<tr>
<td>Vf</td>
<td>0.13 ksi</td>
</tr>
<tr>
<td>Vfz</td>
<td>-0.00 ksi</td>
</tr>
</tbody>
</table>

DESIGN WOOD STRENGTHS:

Fc' = Fc(1.65)*CM(1.00)*Ct(1.00)*KF(2.40)*Fi(0.90)*Lam(1.00) = 3.56 ksi
Fby' = Fby(1.85)*CM(1.00)*Ct(1.00)*CV(1.00)*KF(2.54)*Fi(0.85)*Lam(1.00) = 3.99 ksi
Fbz' = Fbz(2.40)*CM(1.00)*Ct(1.00)*Cu(1.10)*KF(2.54)*Fi(0.85)*Lam(1.00) = 5.70 ksi
Fvy' = Fvy(0.27)*CM(1.00)*Ct(1.00)*Cvu(1.00)*KF(2.88)*Fi(0.75)*Lam(1.00) = 0.57 ksi
Fvz' = Fvz(0.27)*CM(1.00)*Ct(1.00)*Cvz(1.00)*KF(2.88)*Fi(0.75)*Lam(1.00) = 0.57 ksi

RESULTS:

(FC/FC')² + Fby/(Fby*(1-fc/FcEy) + fbz/(Fbz*(1-fc/FcEz-(Fby/FbE)^2)) = 0.11 < 1.00 [3.9-3] OK!
(Fvy + 3/2*fvty)/Fvy' = 0.23 < 1.00 [3.4.1] OK!, (fvz + 3/2*fvtz)/Fvz' = 0.00 < 1.00 [3.4.1] OK!

Section OK !!!
VU VILLA DESIGN/REHABILITATION
ROOF BEAMS - TRANSVERSE ANALYSIS

CODE: ANSI/AWC NDS-2012 LRFD
ANALYSIS TYPE: Code Group Verification

CODE GROUP:  2  Roof Beams
MEMBER:  40  Beam_40  POINT:  1  COORDINATE:  x = 0.00 L = 0.00 ft

LOADS:
Governing Load Case:  43 Transverse_Roof Beams  (4+22)*1.00+(24+25+26)*1.20+30*1.60

MATERIAL: GL VG SOFTWOOD 24F-V4 DF/DF
Structural Glued Laminated Softwood Timber - Tab.5A
Ft=1.10 ksi  Fc=1.65 ksi
Fby=1.85 ksi  Fvy=0.27 ksi  Fcpy=0.65 ksi  Ey=1800.00 ksi
Eminy=950.00 ksi  Fbz=2.40 ksi  Fvz=0.27 ksi  Fcpz=0.65 ksi  Ez=1800.00 ksi

SECTION PARAMETERS: 5.5x21
d=21.00 in  b=5.50 in
Ay=76.867 in²  Az=76.867 in²  A=115.300 in²
Iy=4865.000 in⁴  Iz=252.400 in⁴  Ix=864.665 in⁴
Sy=432.444 in³  Sz=98.498 in³

MEMBER PARAMETERS:

INTERNAL FORCES AND ACTUAL STRESSES:
N = 0.34 kip  My = -20.47 kip*ft  Mz = -0.00 kip*ft  Vy = 13.02 kip  Vz = -0.00 kip
fc = 0.00 ksi  fby = -0.57 ksi  fbz = -0.00 ksi  fvy = 0.17 ksi  fvz = -0.00 ksi
Mx = -0.01 kip*ft  fvyt = 0.00 ksi  fvtz = 0.00 ksi

DESIGN WOOD STRENGTHS:
Fc' = Fc(1.65)*CM(1.00)*Ct(1.00)*KF(2.40)*Fi(0.90)*Lam(0.70) = 2.49 ksi
Fby' = Fby(1.85)*CM(1.00)*Ct(1.00)*CV(1.00)*KF(2.54)*Fi(0.85)*Lam(0.70) = 2.80 ksi
Fbz' = Fbz(2.40)*CM(1.00)*Ct(1.00)*Cu(1.10)*KF(2.54)*Fi(0.85)*Lam(0.70) = 3.99 ksi
Fvy' = Fvy(0.27)*CM(1.00)*Ct(1.00)*Cvr(1.00)*KF(2.88)*Fi(0.75)*Lam(0.70) = 0.40 ksi
Fvz' = Fvz(0.27)*CM(1.00)*Ct(1.00)*Cvr(1.00)*KF(2.88)*Fi(0.75)*Lam(0.70) = 0.40 ksi

RESULTS:
(fc/Fc')² + fby/(Fby*(1-fc/FcEy)) + fbz/(Fbz*(1-fc/FcEz-(fby/FbEy)^2)) = 0.20 < 1.00 [3.9-3]  OK!
(fvy + 3/2*fvyt)/Fvy' = 0.43 < 1.00 [3.4.1]  OK!,  (fvz + 3/2*fvtz)/Fvz' = 0.00 < 1.00 [3.4.1]  OK!

Section OK !!!!
Root Beam under Gravity Loading:

\[ w_{L}=0.53 (65.87 \text{ lb/ft}) = 9861.6 \text{ lb/ft} \]

Strength:

- **Bending**: \( F_b = F_b \left( C_m C_t C_L C_G C_k C_c C_i K_F \Phi \right) \)
- **Shear**: \( F_v = F_v \left[ C_m C_t C_G \right] \)
- **Modulus of Elasticity**: \( E' = E' \left[ C_m C_t \right] \)
  \[ E_{min} = E_{min} \left[ C_m C_t K_F \Phi \right] \]

Note:
- For \( F_b \): \( K_F = 2.54 \), \( \Phi = 0.85 \)
- For \( F_v \): \( K_F = 2.88 \), \( \Phi = 0.75 \)
- For \( E \): No \( K_F \) or \( \Phi \)
- For \( E_{min} \): \( K_F = 1.74 \), \( \Phi = 0.85 \)
Roof Beam under Lateral Loading:

\[ w_a = 566.8 \text{ lb/ft} \quad \text{(from Chord & Drag Analysis)} \]

**Strength:**

- **Bending:** 
  \[ F_d' = F_d \left[ C_m C_e C_2 C_6 C_{fu} C_6 C_2 K_f \phi \right]^2 \]

- **Shear:** 
  \[ F_v' = F_v \left[ C_m C_e C_{ur} \right]^2 \]

- **Modulus of Elasticity:**
  \[ E' = E \left[ C_m C_e \right]^2 \]
  \[ E_{min}' = E_{min} \left[ C_m C_e K_f \phi \right]^2 \]

1 Adjustment factors based on criteria and calculations from 2015 NDS
2 Time-effect factor based on controlling LRFD load combination equation
3 Reference Design Values from 2015 NDS reference design value tables

*See Roof Beam Design spreadsheet for calculations.
Root Beam acting as a Collector

\[ T_u = \frac{C_u}{C_{u, collector}} = \frac{283.4 \text{ lb/ft}^2 \times 19.71 \text{ ft}}{5.586 \text{ lb}} \]

\[ T_u, \text{ collector} / C_u, \text{ collector} = \frac{5.586 \text{ lb}}{(5/8'' \times 16'' \times 1/8'')} = 66 \text{ psi} \]

\[ F_c' = F_c \left[ \frac{C_m C_t C_p K_f \phi}{A_{beam}} \right]^2 \times \lambda^2 \]

\[ = (1150 \text{ psi}) \left[ 1 \times 1 \times 0.5 \times 2.40 \times 0.95 \right] (0.8) \]

\[ = 1182 \text{ psi} \]

\[ F_T' = F_T \left[ \frac{C_m C_t K_f \phi}{A_{beam}} \right]^2 \times \lambda^2 \]

\[ = (1100 \text{ psi}) \left[ 1 \times 1 \times 2.70 \times 0.85 \right] (0.8) \]

\[ = 950 \text{ psi} \]

\[ F_T' \gg \frac{T_u, \text{ collector}}{C_u, \text{ collector}} \quad \text{OK.} \]

\[ F_c' \gg \frac{T_u, \text{ collector}}{C_u, \text{ collector}} \quad \text{OK.} \]

\[ 1, 2, 3 \text{ See the corresponding footnote on Page 2.} \]

\[ 4 \text{ In the interest of time an extremely conservative value was assumed.} \]
2nd Floor Beam Design
### Beam Type:
6 3/4" x 24" 24F-V4 (DF/DF) GluLam

### Beam Properties:
- **b**, breadth: 6.74 (in)
- **d**, depth: 24 (in)
- **A, Cross-Sectional Area**: 162.0 (in$^2$)
- **b, breadth**: 6.74 (in)
- **d, depth**: 24 (in)
- **fby, Bending 1,450 (psi)**
- **fb, Bending 2,400 (psi)**
- **fvy, Shear 265 (psi)**
- **fv, Shear 230 (psi)**
- **d, depth**: 24 (in)
- **fby, Bending 1,450 (psi)**
- **fb, Bending 2,400 (psi)**
- **fvy, Shear 230 (psi)**
- **fv, Shear 265 (psi)**
- **A, Cross-Sectional Area**: 162.0 (in$^2$)
- **fby, Bending 1,450 (psi)**
- **fb, Bending 2,400 (psi)**
- **fvy, Shear 230 (psi)**
- **fv, Shear 265 (psi)**
- **A, Cross-Sectional Area**: 162.0 (in$^2$)
- **fby, Bending 1,450 (psi)**
- **fb, Bending 2,400 (psi)**
- **fvy, Shear 230 (psi)**
- **fv, Shear 265 (psi)**

### Reference Design Values (NDS 2015 Table 5A):
- **$L_u$, Ultimate Load**: 421.6 (plf)
- **$w$, Tributary Width**: 15.00 (ft)
- **$l$, Tributary Length**: 19.71 (ft)
- **$M_u$, Ultimate Moment**: 20.47 (k-ft)
- **$V_u$, Ultimate Shear**: 4.15 (k)
- **$T_u$, Ultimate Tension**: 19.02 (k)
- **$E'_{x}$, Elastic Modulus**: 1,800,000 (psi)
- **$E'_{y}$, Elastic Modulus**: 1,800,000 (psi)
- **$I_{x}$, Moment of Inertia**: 7,776 (in$^4$)
- **$I_{y}$, Moment of Inertia**: 615 (in$^4$)

### Loading Conditions:
- **$ω$, Ultimate Load**: 194.44 (psf)
- **$w$, Tributary Width**: 15.00 (ft)
- **$l$, Tributary Length**: 19.71 (ft)
- **$V_u$, Ultimate Shear**: 28.74 (k)

**Gravity Loading**

### Check for Bending:
- $M_u$ = 194.44 (psf) (NDS 2015 Section 5.1.4)
- $C_u$ = 1
- $C_t$ = 1
- $C_c$ = 0.91
- $C_v$ = 1
- $C_f$ = 1
- $C_v$ = 1
- $C_f$ = 1
- $K_W$ = 2.54
- $\Phi$ = 0.85 (NDS 2015 Table 5.1.1)

### Check for Shear:
- $V_u$ = 28.74 (k) (NDS 2015 Section 5.1.4)
- $C_u$ = 1
- $C_t$ = 1
- $C_v$ = 1
- $C_f$ = 1
- $C_v$ = 1
- $C_f$ = 1
- $K_W$ = 2.54
- $\Phi$ = 0.85

### Check for Deflection:
- $E'_{x}$ = 1,800,000 (psi) (NDS 2015 Table 5.1.1)
- $E'_{y}$ = 1,800,000 (psi) (NDS 2015 Table 5.1.1)
- $\phi = 0.71$ (NDS Section 3.5.1)

**Lateral Loading**

**Check for Bending (Transverse direction):**
- $M_u$ = 20.47 (k-ft) (NDS 2015 Section 5.1.4)
- $C_u$ = 1
- $C_t$ = 1
- $C_v$ = 1
- $C_f$ = 1
- $C_v$ = 1
- $C_f$ = 1
- $C_{fu}$ = 1.07
- $C_{fu}$ = 1
- $K_W$ = 2.54
- $\Phi$ = 0.85

**Check for Shear (Transverse direction):**
- $V_u$ = 28.74 (k) (NDS 2015 Section 5.1.1)
- $C_u$ = 1
- $C_t$ = 1
- $C_v$ = 1
- $C_f$ = 1
- $C_v$ = 1
- $C_f$ = 1
- $C_{fu}$ = 1.07
- $C_{fu}$ = 1
- $K_W$ = 2.54
- $\Phi$ = 0.85

**Check for Deflection:**
- $E'_{x}$ = 1,800,000 (psi) (NDS 2015 Section 3.4.2)
- $E'_{y}$ = 1,800,000 (psi) (NDS 2015 Section 3.4.2)
- $\phi = 0.71$ (NDS Section 3.5.1)
VU VILLA DESIGN/REHABILITATION
2ND FLOOR BEAM - LONGITUDINAL ANALYSIS

CODE: ANSI/AWC NDS-2012 LRFD
ANALYSIS TYPE: Code Group Verification

CODE GROUP: 1 2nd Beams
MEMBER: 19 Beam_19 POINT: 3 COORDINATE: x = 1.00 L = 20.00 ft

LOADS:
Governing Load Case: 46 Longitudinal_2nd Floor Beam (22+23)*1.00+(24+25+26+27+28+29+34)*1.20+31*1.60

MATERIAL: GL VG SOFTWOOD 24F-V4 DF/DF
Structural Glued Laminated Softwood Timber - Tab.5A
Ft=1.10 ksi Fc=1.65 ksi
Fby=1.85 ksi Fvy=0.27 ksi Fcpy=0.65 ksi Ey=1800.00 ksi
Eminy=950.00 ksi Fvz=0.27 ksi Fcpz=0.65 ksi Ez=1800.00 ksi
Eminz=950.00 ksi

SECTION PARAMETERS: 6.75x24

Ay=108.000 in² Az=108.000 in² A=162.000 in²
Iy=7776.000 in⁴ Iz=615.100 in⁴ Ix=2024.306 in⁴
Sy=648.000 in³ Sz=182.252 in³

MEMBER PARAMETERS:

INTERNAL FORCES AND ACTUAL STRESSES:
N = -0.31 kip My = -27.93 kip*ft Mz = -0.00 kip*ft Vy = -14.99 kip Vz = 0.01 kip
ft = -0.00 ksi fby = -0.52 ksi fbz = -0.00 ksi fvy = -0.14 ksi fvtz = 0.00 ksi
Mx = 0.01 kip*ft fty = 0.00 ksi fvtz = 0.00 ksi

DESIGN WOOD STRENGTHS:
Ft' = Ft(1.10)*CM(1.00)*Ct(1.00)*KF(2.70)*Fi(0.80)*Lam(0.70) = 1.66 ksi
Fby' = Fby(1.85)*CM(1.00)*Ct(1.00)*CV(1.00)*KF(2.54)*Fi(0.85)*Lam(0.70) = 2.80 ksi
Fbz' = Fbz(2.40)*CM(1.00)*Ct(1.00)*Cfu(1.07)*KF(2.54)*Fi(0.85)*Lam(0.70) = 3.87 ksi
Fvy' = Fvy(0.27)*CM(1.00)*Ct(1.00)*Cvr(1.00)*KF(2.88)*Fi(0.75)*Lam(0.70) = 0.40 ksi
Fvz' = Fvz(0.27)*CM(1.00)*Ct(1.00)*Cvr(1.00)*KF(2.88)*Fi(0.75)*Lam(0.70) = 0.40 ksi
Fby* = Fby(1.85)*CM(1.00)*Ct(1.00)*CV(1.00)*KF(2.54)*Fi(0.85)*Lam(0.70) = 2.80 ksi
Fby** = Fby(1.85)*CM(1.00)*Ct(1.00)*KF(2.54)*Fi(0.85)*Lam(0.70) = 2.80 ksi

RESULTS:
ft/Ft' + fby/Fby* + fbz/Fbz' = 0.19 < 1.00 [3.9-1] OK!
(fby-ft)/Fby** + fbz/Fbz = 0.18 < 1.00 [3.9-2] OK!
(fvy + 3/2*fvy)/Fvy' = 0.35 < 1.00 [3.4.1] OK!, (fvtz + 3/2*fvtz)/Fvz' = 0.00 < 1.00 [3.4.1] OK!

Section OK !!!
VU VILLA DESIGN/REHABILITATION

2ND FLOOR BEAM_- TRANSVERSE ANALYSIS

---

**CODE:** ANSI/AWC NDS-2012 LRFD

**ANALYSIS TYPE:** Code Group Verification

---

**CODE GROUP:** 1 2nd Beams

**MEMBER:** 19 Beam_19  **POINT:** 3  **COORDINATE:** x = 1.00 L = 20.00 ft

---

**LOADS:**

*Governing Load Case:* 45 Transverse_2nd Floor Beam (11+22)*1.00+(24+25+26+27+28+29+34)*1.20+31*1.60

---

**MATERIAL:** GL VG SOFTWOOD 24F-V4 DF/DF

Structural Glued Laminated Softwood Timber - Tab.5A

<table>
<thead>
<tr>
<th>Property</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Ft</td>
<td>1.10 ksi</td>
</tr>
<tr>
<td>Fc</td>
<td>1.65 ksi</td>
</tr>
<tr>
<td>Fby</td>
<td>1.85 ksi</td>
</tr>
<tr>
<td>Eminy</td>
<td>950.00 ksi</td>
</tr>
<tr>
<td>Fbz</td>
<td>2.40 ksi</td>
</tr>
<tr>
<td>Eminz</td>
<td>950.00 ksi</td>
</tr>
<tr>
<td>Fvy</td>
<td>0.27 ksi</td>
</tr>
<tr>
<td>Fcpz</td>
<td>0.65 ksi</td>
</tr>
<tr>
<td>Ey</td>
<td>1800.00 ksi</td>
</tr>
<tr>
<td>Ez</td>
<td>1800.00 ksi</td>
</tr>
</tbody>
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**SECTION PARAMETERS:** 6.75x24

<table>
<thead>
<tr>
<th>Property</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>d</td>
<td>24.00 in</td>
</tr>
<tr>
<td>b</td>
<td>6.75 in</td>
</tr>
<tr>
<td>Ay</td>
<td>108.000 in²</td>
</tr>
<tr>
<td>Az</td>
<td>108.000 in²</td>
</tr>
<tr>
<td>Iy</td>
<td>7776.000 in⁴</td>
</tr>
<tr>
<td>Iz</td>
<td>615.100 in⁴</td>
</tr>
<tr>
<td>Sy</td>
<td>648.000 in³</td>
</tr>
<tr>
<td>Sz</td>
<td>182.252 in³</td>
</tr>
</tbody>
</table>

---

**MEMBER PARAMETERS:**

- **BUCKLING Y**
- **BUCKLING Z**
- **LT BUCKLING**

---

**INTERNAL FORCES AND ACTUAL STRESSES:**

<table>
<thead>
<tr>
<th>Force</th>
<th>Value</th>
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<tbody>
<tr>
<td>N</td>
<td>-0.13 kip</td>
</tr>
<tr>
<td>fty</td>
<td>-0.51 ksi</td>
</tr>
<tr>
<td>Mx</td>
<td>0.01 kip*ft</td>
</tr>
<tr>
<td>ft</td>
<td>-0.00 ksi</td>
</tr>
<tr>
<td>fby</td>
<td>-0.51 ksi</td>
</tr>
<tr>
<td>fbz</td>
<td>-0.00 ksi</td>
</tr>
<tr>
<td>My</td>
<td>-27.75 kip*ft</td>
</tr>
<tr>
<td>Mz</td>
<td>-0.00 kip*ft</td>
</tr>
<tr>
<td>Vy</td>
<td>-14.94 kip</td>
</tr>
<tr>
<td>Vz</td>
<td>0.01 kip</td>
</tr>
<tr>
<td>Vfy</td>
<td>-0.14 ksi</td>
</tr>
<tr>
<td>Vfz</td>
<td>0.00 kip</td>
</tr>
</tbody>
</table>

---

**DESIGN WOOD STRENGTHS:**

<table>
<thead>
<tr>
<th>Strength</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Ft'</td>
<td>1.66 ksi</td>
</tr>
<tr>
<td>Fby'</td>
<td>2.80 ksi</td>
</tr>
<tr>
<td>Fbz'</td>
<td>3.87 ksi</td>
</tr>
<tr>
<td>Fvy'</td>
<td>0.40 ksi</td>
</tr>
<tr>
<td>Fvz'</td>
<td>0.00 ksi</td>
</tr>
</tbody>
</table>

---

**RESULTS:**

<table>
<thead>
<tr>
<th>Expression</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>ft/Ft' + fby/Fby' + fbz/Fbz'</td>
<td>0.18 &lt; 1.00 [3.9-1]  OK!</td>
</tr>
<tr>
<td>(fby-ft)/Fby' + fbz/Fbz'</td>
<td>0.18 &lt; 1.00 [3.9-2]  OK!</td>
</tr>
<tr>
<td>(fvy + 3/2*fvty)/Fvy'</td>
<td>0.35 &lt; 1.00 [3.4.1]  OK!</td>
</tr>
<tr>
<td>(fvz + 3/2*fvtz)/Fvz'</td>
<td>0.00 &lt; 1.00 [3.4.1]  OK!</td>
</tr>
</tbody>
</table>

---

*Section OK !!!*
Vu Villa
Structural Design
2nd Floor Beam Design
KBS
8/10/19

Cross-Section

ISO View

Profile

2nd Floor Beam under Gravity Loading:

\[ w_0 = (6') \times 144.411 \times 10^{-6} = 2.916.615 \]

Strength:
- Bending: \[ F_b' = F_b \left[ C_m C_6 C_4 C_v C_p K_f \phi J \right] \]
- Shear: \[ F_v' = F_v \left[ C_m C_6 C_w K_f \phi J \right] \]

Modulus of Elasticity:
- \[ E = E \left[ C_m + J \right] \]
- \[ E'_w = E_w \left[ C_m + K_f \phi J \right] \]
- \[ E_{min} = E_{min} \left[ C_m + K_f \phi J \right] \]
2nd Floor Beam under Lateral Loading:
(In Transverse Direction)

\[ W_u = \frac{12.68.3 \text{ lb/ft}}{[60.125 - 15.1^2]} = 421.6 \text{ lb/ft} \]

**Strength:**

- **Bending:**
  \[ F_b' = F_b\left[ C_m C_1 C_v C_{nl} C_c C_i K_F \phi\right]^2 \]

- **Shear:**
  \[ F_v' = F_v\left[ C_m C_1 C_{nr} K_F \phi\right]^2 \]

**Modulus of Elasticity:**

\[ E' = E\left[ C_m C_1\right]^2 \]

\[ E_{min} = E_{min}\left[ C_m C_1 K_F \phi\right]^2 \]

(In Longitudinal Direction)

\[ W_u = (15^2 \times 12.68.3 \text{ lb/ft})/60.125^2 = 316.4 \text{ lb/ft} \]

\[ T_u = C_u = (316.4 \text{ lb/ft}) (60.125 \text{ ft}) (\frac{k}{1000 \text{ lb}}) = 19k \]

*Continue on next page*
Strength:

Tension: \( F_t' = F_t \left[ C_{MC} + K_F \Phi^{(2)} \right] \lambda \)

Compression: \( F_c' = F_c \left[ C_{MC} + C_p K_F \Phi^{(2)} \right] \lambda \)

\(^{1}\text{Values from 2015 NDS reference design value tables.}\)

\(^{2}\text{Values for adjustment factors from criteria and calculations in 2015 NDS.}\)

\(^{3}\text{Time-effect factor based on LRFD load combo equations; see Appendix N of the 2015 NDS.}\)
VU VILLA STRUCTURAL DESIGN/REHABILITATION
MONTANA TECH

COLUMN DESIGN
2nd Floor Column Design
**Column Type:** 6" x 6" Select Structural Doug Fir - Larch (North)

<table>
<thead>
<tr>
<th>Column Properties</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>b, breadth</td>
<td>5.5</td>
</tr>
<tr>
<td>d, depth</td>
<td>5.5</td>
</tr>
<tr>
<td>A, Cross-Sectional Area</td>
<td>30.25</td>
</tr>
<tr>
<td>S, Section Modulus</td>
<td>27.71</td>
</tr>
<tr>
<td>I, Moment of Inertia</td>
<td>70.20</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Reference Design Values:</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>f_c, Bending</td>
<td>1,900 (psi)</td>
</tr>
<tr>
<td>f_t, shear</td>
<td>170 (psi)</td>
</tr>
<tr>
<td>f_c, Compression</td>
<td>1,150 (psi)</td>
</tr>
<tr>
<td>f_t, Tension</td>
<td>1,000 (psi)</td>
</tr>
<tr>
<td>E</td>
<td>1,852,000 (psi)</td>
</tr>
<tr>
<td>E_w</td>
<td>580,000 (psi)</td>
</tr>
</tbody>
</table>

**Gravity Loading**

<table>
<thead>
<tr>
<th>Loading Condition:</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>u_s, Ultimate Load</td>
<td>96.54 (psf)</td>
</tr>
<tr>
<td>v_s, Tributary Width</td>
<td>15.05 (ft)</td>
</tr>
<tr>
<td>l_s, Tributary Length</td>
<td>19.71 (ft)</td>
</tr>
<tr>
<td>L_{t, Column Length}</td>
<td>12 (ft)</td>
</tr>
<tr>
<td>P_u, Ultimate Compression</td>
<td>19.68 (k)</td>
</tr>
</tbody>
</table>

**Check for Compression:***

| f_u          | 1,150 (psi) |
| f_c^s        | 1           |
| C_{u,s}      | 1           |
| C_{u}        | 1           |
| C_{s}        | 1           |
| K_p          | 2.40        | (NDS 2015 Table 4.3.1) |
| e             | 0.00        |
| K_e          | 0.00        | (Based on Governing Load Combination Equation) |
| K_{w,s}      | 3.06        |
| K_{w}        | 1.00        | (NDS 2015 Section 4.3.8) |
| λ             | 0.00        |
| A             | 1.00        | (Based on Governing Load Combination Equation) |
| K_{w,s}      | 3.06        |
| K_{w}        | 1.00        |
| λ             | 0.00        |
| A             | 1.00        |

**Gravity Loading**

<table>
<thead>
<tr>
<th>Loading Condition:</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>u_s, Ultimate Load</td>
<td>566.4 (psf)</td>
</tr>
<tr>
<td>v_s, Tributary Width</td>
<td>15.05 (ft)</td>
</tr>
<tr>
<td>l_s, Tributary Length</td>
<td>19.71 (ft)</td>
</tr>
<tr>
<td>W, Building Width</td>
<td>60.13 (ft)</td>
</tr>
<tr>
<td>L_s, Building Length</td>
<td>60.13 (ft)</td>
</tr>
<tr>
<td>V_{u,x}, Ultimate Shear (Trans.)</td>
<td>0.56 (k)</td>
</tr>
<tr>
<td>M_{u,x}, Ultimate Moment (Trans.)</td>
<td>5.30 (k-ft)</td>
</tr>
<tr>
<td>V_{u,y}, Ultimate Shear (Long.)</td>
<td>0.83 (k)</td>
</tr>
<tr>
<td>M_{u,y}, Ultimate Moment (Long.)</td>
<td>3.15 (k-ft)</td>
</tr>
</tbody>
</table>

**Check for Bending:**

| l             | 1.00 (psi) |
| f_u          | 1,000 (psi) |
| f_c^s        | 1,150 (psi) |
| C_{u,s}      | 1           |
| C_{u}        | 1           |
| C_{s}        | 1           |
| K_p          | 1.00        |
| e             | 0.00        |
| K_e          | 0.00        |
| A             | 1.00        |
| K_{w,s}      | 1.00        |
| K_{w}        | 1.00        |
| λ             | 0.00        |
| A             | 1.00        |

**Additional 25% of the lateral load. As an added factor of safety, Trans.,columns were designed to resist an additional 25% of the lateral load.**

---

**Notes:**

- Existing Masonry Shear Walls will resist entire lateral load. As an added factor of safety, Trans.,columns were designed to resist an additional 25% of the lateral load.
- O.K. = Ok, no further action is needed.
VU VILLA DESIGN/REHABILITATION
2ND FLOOR COLUMN - LONGITUDINAL ANALYSIS

CODE: ANSI/AWC NDS-2012 LRFD
ANALYSIS TYPE: Code Group Verification

CODE GROUP: 3 2nd Columns
MEMBER: 32 Column_32  POINT: 1  COORDINATE: x = 0.00 L = 0.00 ft

LOADS:
Governing Load Case: 9 Longitudinal_2nd Column  (20+22)*1.00+(24+25+26+34)*1.20+30*0.50

MATERIAL: DOUGLAS FIR-LARCH (N) Select Structural 2+
Visually Graded Dimension Lumber - Tab.4A
Fb=1.35 ksi  Ft=0.83 ksi  Fv=0.18 ksi  Fcp=0.63 ksi
Fc=1.90 ksi  E=1900.00 ksi  Emin=690.00 ksi

SECTION PARAMETERS: 6x6

d=5.50 in  b=5.50 in
Ay=20.167 in²  Az=20.167 in²  A=30.250 in²
Iy=76.260 in⁴  Iz=76.260 in⁴  Ix=128.654 in⁴
Sy=27.731 in³  Sz=27.731 in³

MEMBER PARAMETERS:

BUCKLING Y

Key = 1.00  ley = 12.00 ft  ley/d = 26.18
CPy = 0.28  FcEy = 1.24 ksi

BUCKLING Z

Kez = 1.00  lez = 12.00 ft  lez/b = 26.18
CPz = 0.28  FcEz = 1.24 ksi

LT BUCKLING

lu = 12.00 ft  RB = 6.94

INTERNAL FORCES AND ACTUAL STRESSES:

N = 28.10 kip  My = 0.00 kip*ft  Mz = 0.03 kip*ft  Vy = -0.00 kip  Vz = 0.01 kip
fc = 0.93 ksi  fby = 0.00 ksi  fbz = 0.01 ksi  fvy = -0.00 ksi  fvz = 0.00 ksi
Mx = 0.00 kip*ft
fvty = 0.00 ksi  fvtz = 0.00 ksi

DESIGN WOOD STRENGTHS:

Fc' = Fc(1.90)*CM(1.00)*Ct(1.00)*CF(1.00)*CP(0.28)*KF(2.40)*Fi(0.90)*Lam(1.00) = 1.15 ksi
Fb' = Fb(1.35)*CM(1.00)*Ct(1.00)*CL(0.99)*CF(1.00)*KF(2.54)*Fi(0.85)*Lam(1.00) = 2.90 ksi
Fv' = Fv(0.18)*CM(1.00)*Ct(1.00)*KF(2.88)*Fi(0.75)*Lam(1.00) = 0.39 ksi
Emin' = Emin(690.00)*CM(1.00)*Ct(1.00)*KF(1.76)*Fi(0.85) = 1032.24 ksi

RESULTS:

(fc/Fc')² + fby/(Fb*(1-fc/FcEy)) + fbz/(Fb*(1-fc/FcEz-(fby/FbE))²) = 0.67 < 1.00 [3.9-3] OK!
fC/FcEz + (fby/FbE)/2 = 0.75 < 1.00 [3.9-4] OK!,  fC/FcEy = 0.75 < 1.00 [3.9-2] OK!
(fvy + 3/2*fvt)/f'v = 0.00 < 1.00 [3.4.1] OK!,  (fvz + 3/2*fvtz)/f'v = 0.00 < 1.00 [3.4.1] OK!
ley/d = 26.18 < 50.00 STABLE,  lez/b = 26.18 < 50.00 STABLE,  Rb = 6.94 < 50.00 STABLE

Section OK !!
CODE: ANSI/AWC NDS-2012 LRFD
ANALYSIS TYPE: Code Group Verification

CODE GROUP: 3 2nd Columns
MEMBER: 29 Column_29  POINT: 3  COORDINATE: x = 1.00 L = 12.00 ft

LOADS:
Governining Load Case: 8 Transverse_2nd Column  (4+22)*0.50+(24+25+26+34)*1.20+30*1.60

MATERIAL: DOUGLAS FIR-LARCH (N) SelectStructural 2+
Visually Graded Dimension Lumber - Tab.4A
Fb=1.35 ksi  Ft=0.83 ksi  Fv=0.18 ksi  Fcp=0.63 ksi
Fc=1.90 ksi  E=1900.00 ksi  Emin=690.00 ksi

SECTION PARAMETERS: 6x6
d=5.50 in
b=5.50 in
Ay=20.167 in²  Az=20.167 in²  A=30.250 in²
Iy=76.260 in⁴  Iz=76.260 in⁴  Ix=128.654 in⁴
Sy=27.731 in³  Sz=27.731 in³

MEMBER PARAMETERS:
Key = 1.00
ley = 12.00 ft
ley/d = 26.18
CPy = 0.38
FcEy = 1.24 ksi

BUCKLING Y
Key = 1.00
ley = 12.00 ft
ley/d = 26.18
CPy = 0.38
FcEy = 1.24 ksi

BUCKLING Z
Kez = 1.00
lez = 12.00 ft
lez/b = 26.18
CPz = 0.38
FcEz = 1.24 ksi

LT BUCKLING
lu = 12.00 ft
leb = 22.08 ft
RB = 6.94
CL = 1.00

INTERNAL FORCES AND ACTUAL STRESSES:
N = 31.34 kip  My = 0.00 kip*ft  Mz = 0.07 kip*ft  Vy = -0.00 kip  Vz = -0.01 kip
fc = 1.04 ksi  fby = -0.00 ksi  fbz = 0.03 ksi  fvy = -0.00 ksi  fvz = -0.00 ksi
Mx = 0.00 kip*ft  fvtz = 0.00 ksi

DESIGN WOOD STRENGTHS:
Fc' = Fc(1.90)*CM(1.00)*Ct(1.00)*CF(1.00)*CP(0.38)*KF(2.40)*Fi(0.90)*Lam(0.70) = 1.10 ksi
Fb' = Fb(1.35)*CM(1.00)*Ct(1.00)*CL(1.00)*CF(1.00)*KF(2.54)*Fi(0.85)*Lam(0.70) = 2.03 ksi
Fv' = Fv(0.18)*CM(1.00)*Ct(1.00)*KF(2.88)*Fi(0.75)*Lam(0.70) = 0.27 ksi
Emin' = Emin(690.00)*CM(1.00)*Ct(1.00)*KF(1.76)*Fi(0.85) = 1032.24 ksi

RESULTS:
(fc/Fc')^2 + fby/(Fb'*(1-fc/FcEy) + fbz/(Fb'*1-fc/FcEz-(fby/FbE)^2)) = 0.98 < 1.00 [3.9-3]  OK!
fc/FcEy + (fby/FbE)^2 = 0.84 < 1.00 [3.9-4]  OK!,  fc/FcEy = 0.84 < 1.00 [3.9-2]  OK!,
(fvy + 3/2*fvy)/f'v = 0.00 < 1.00 [3.4.1]  OK!,  (fvz + 3/2*fvtz)/f'v = 0.00 < 1.00 [3.4.1]  OK!
ley/d = 26.18 < 50.00 STABLE,  lez/b = 26.18 < 50.00 STABLE,  Rb = 6.94 < 50.00 STABLE

Section OK !!!
2nd Floor Column under Gravity Loading:

\[ P_u = 66.52 \text{ psf} \times (15\text{in} \times 19.7\text{in}) = 19,674 \text{ K} \]

**Strength:**

Compression:

\[ F_c' = F_c \left( C_m C_t C_i C_p K_F \Phi \right)^2 \]

*See 2nd Floor Columns Design supplement for calculations*

**Modulus of Elasticity:**

\[ E'_{min} = E_{min} \left( C_m C_t C_i C_p K_F \Phi \right)^2 \]

Note: For \( F_c = 2.40 \), \( \Phi = 0.90 \)
For \( E'_{min} \): \( K_F = 1.74 \), \( \Phi = 0.85 \)
2nd Floor Column under Lateral Loading

(Via Villa Structural Design)

2nd Floor Columns
KBS
7/3/07

Assumptions:

- All members (including meshing) have some stiffness (conservative, though this may occur at the middle of the column.
- Frame has rigid connections.
- Frame is fixed at one end.
- Shear walls will resist entire lateral load.

Vn = (0.25)(502 lb) = 125.5 lb
Mn = 531,150 lb-ft

Mn = 531,150 lb-ft

Vn = 125.5 lb

FBD - Shear wall

(Transverse Beams)

Concrete Column

Steel Column

Bearing Wall
FBD

(a) 11,171.6 lb

(b) $V_u = \frac{11,171.6 \text{ lb}}{5} = 558.6 \text{ lb}$

$M_u = (558.6 \text{ lb})(\frac{12\text{ ft}}{2}) = 3,351.5 \text{ lb-ft}$
Strength:
Check for Bending:

\[ F_b' = F_b \left[ C M C_t C_L C_F C_{fu} C_i C_r K_F \phi \right]^2 \lambda \]

\[ E_n' = E_n \left[ C M C_t C_r K_F \phi \right]^2 \]

Check for Shear:

\[ F_v' = F_v \left[ C M C_t C_i K_F \phi \right]^2 \lambda \]

Reference Design Values from 2015 NDS Reference Design Value tables

*Adjustment factors based on criteria and calculations from 2015 NDS

Time-effect factor based on controlling LRFD load combination equation

*See 2nd Floor Column Design spreadsheet for calculation
1st Floor Column Design
### Column Type: 10" x 10" Select Structural Doug Fir - Larch (North)

**Column Properties:**

Sawn Lumber Loading Conditions:

- **b**, breadth: 9.5 (in) ω, Ultimate Load: 1268.3 (psf)
- **d**, depth: 9.5 (in) w, Tributary Width: 15.00 (ft)
- **A**, Cross-Sectional Area: 90.25 (in²)
- **l**, Tributary Length: 19.71 (ft)
- **S**, Section Modulus: 142.90 (in³)
- **W**, Building Width: 60.13 (ft)
- **I**, Moment of Inertia: 678.76 (in⁴)
- **Vu,x**, Ultimate Shear (Trans.): 1.81 (k) (See Hand Calculations)
- **Mu,x**, Ultimate Moment (Trans.): 14.47 (k-ft) (See Hand Calculations)

**Reference Design Values:**

Number 1 Grade

- **Vu,y**, Ultimate Shear (Long.): 1.72 (k) (See Hand Calculations)
- **f₏**, Bending: 1,200 (psi) (NDS 2015 Table 4D)
- **fᵥ**, Shear: 170 (psi) (NDS 2015 Table 4D)
- **fₐ**, Compression: 1,000 (psi) (NDS 2015 Table 4D)

**Check for Bending:**

- **f₄**, Tension: 825 (psi) (NDS 2015 Table 4D)
- **E**, Modulus of Elasticity: 1,600,000 (psi) (NDS 2015 Table 4D)
- **CM**, Unfactored: 1 (NDS 2015 Section 4.1.4)
- **Cₗ**, Factored: 1 (NDS 2015 Section 2.3.3)
- **Cₐ**, Factored: 1 (NDS 2015 Section 4.3.7)
- **Cₔ**, Factored: 1 (NDS 2015 Section 4.3.8)
- **Kₕ**, Factored: 2.40 (NDS 2015 Table 4.3.1)
- **Φₘₐₕ**, factored: 30.77 (k-ft)
- **Φₘₚₜ**, factored: 87.26 (k)

**Check for Compression:**

- **Eₘₚₜ**, factored: 867,680 (psi) (NDS 2015 Table 3.3.3)
- **F₏ₗₚₜ**, factored: 2,584 (psi) (NDS 2015 Table 4.3.1)

**Check for Shear:**

- **Fᵥₗₚₜ**, factored: 2,584 (psi) (NDS 2015 Table 3.7.1.5)
- **Cₐ**, Factored: 1,728 (psi) (NDS 2015 Section 3.7.1.5)
- **Cₔ**, Factored: 1,000 (psi) (NDS 2015 Section 3.7.1.5)
- **Cₗₚₜ**, factored: 0.56 (NDS 2015 Section 3.7.1.5)
- **Fᵥₗₚₜ**, factored: 367.2 (psi) (NDS 2015 Table 4.3.1)
- **F₏ₗₚₜ**, factored: 967 (psi) (NDS 2015 Table 4.3.1)
- **Fₗₚₜ**, factored: 33.1 (k) (NDS 2015 Table 4.3.1)

### Lateral Loading

**Loading Conditions:**

- **G**, Ultimate Load: 1268.3 (psf)
- **w**, Tributary Width: 15.00 (ft)
- **I**, Tributary Length: 19.71 (ft)
- **W**, Building Width: 60.13 (ft)

**Reference Design Values:**

Number 1 Grade

- **Vₓₚₜ**, Ultimate Shear (Trans.): 1.81 (k) (See Hand Calculations)
- **Mₓₚₜ**, Ultimate Moment (Trans.): 14.47 (k-ft) (See Hand Calculations)

**Check for Bending:**

- **f₄**, Tension: 825 (psi) (NDS 2015 Table 4.3.1)
- **Φₘₕ**, factored: 30.77 (k-ft)
- **Φₘₚₜ**, factored: 87.26 (k)

**Check for Compression:**

- **Eₘₚₜ**, factored: 867,680 (psi) (NDS 2015 Table 4.3.1)
- **F₏ₗₚₜ**, factored: 2,584 (psi) (NDS 2015 Table 4.3.1)
- **Φₗₚₜ**, factored: 30.77 (k-ft)

**Check for Shear:**

- **Fᵥₗₚₜ**, factored: 2,584 (psi) (NDS 2015 Table 3.7.1.5)
- **Φₗₚₜ**, factored: 30.77 (k-ft)

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NDS: National Design Specification

**Note:** Existing Masonry Shear Walls will resist entire lateral load. As an added factor of safety, frame/columns were designed to resist an additional 25% of the lateral load.
CODE: ANSI/AWC NDS-2012 LRFD
ANALYSIS TYPE: Code Group Verification

CODE GROUP: 7 1st Columns
MEMBER: 15 Column_15 POINT: 1 COORDINATE: x = 0.00 L = 0.00 ft

LOADS:
Governing Load Case: 41 Transverse_1st Columns 11*1.00+22*0.50+(24+25+26+27+28+29+33+34)*1.20+31*1.60

MATERIAL: DOUGLAS FIR-LARCH (N) Select Structural 2+
Visually Graded Dimension Lumber - Tab.4A
Fb=1.35 ksi Ft=0.83 ksi Fv=0.18 ksi Fcp=0.63 ksi
Fc=1.90 ksi E=1900.00 ksi Emin=690.00 ksi

SECTION PARAMETERS: 10x10

b=9.50 in

Ay=60.167 in² Az=60.167 in² A=90.250 in²
Iy=678.800 in⁴ Iz=678.800 in⁴ Ix=1145.159 in⁴
Sy=142.905 in³ Sz=142.905 in³

BUCKLING Y BUCKLING Z LT BUCKLING

Key = 1.00 Kez = 1.00
ley = 16.00 ft lez = 16.00 ft
ley/d = 20.21 lez/b = 20.21
CPy = 0.57 CPz = 0.57
Fcy = 2.08 ksi Fcz = 2.08 ksi

INTERNAL FORCES AND ACTUAL STRESSES:
N = 48.55 kip Vy = -0.01 kip Vz = -0.01 kip
fc = 0.54 ksi fvy = -0.00 ksi fvz = -0.00 ksi

DESIGN WOOD STRENGTHS:
Fc' = Fc(1.90)*CM(1.00)*Ct(1.00)*CF(1.00)*CP(0.57)*KF(2.40)*Fi(0.90)*Lam(0.70) = 1.64 ksi
Fv' = Fv(0.18)*CM(1.00)*Ct(1.00)*KF(2.88)*Fi(0.75)*Lam(0.70) = 0.27 ksi
Emin' = Emin(690.00)*CM(1.00)*Ct(1.00)*KF(1.76) = 1032.24 ksi

RESULTS:
fc/Fc' = 0.33 < 1.00 [3.6.3] OK!
fvy/Fv' = 0.00 < 1.00 [3.4.1] OK!, fvz/Fv' = 0.00 < 1.00 [3.4.1] OK!
ley/d = 20.21 < 50.00 STABLE, lez/b = 20.21 < 50.00 STABLE,

Section OK !!!!
VU VILLA DESIGN/REHABILITATION
1ST FLOOR COLUMN - LONGITUDINAL ANALYSIS

CODE: ANSI/AWC NDS-2012 LRFD
ANALYSIS TYPE: Code Group Verification

CODE GROUP: 7 1st Columns
MEMBER: 15 Column_15  POINT: 1  COORDINATE: x = 0.00 L = 0.00 ft

LOADS:
Governing Load Case: 42 Longitudinal_1st Columns 22*0.50+23*1.00+(24+25+26+27+28+29+33+34)*1.20+31*1.60

MATERIAL: DOUGLAS FIR-LARCH (N) SelectStructural 2+
Visually Graded Dimension Lumber - Tab.4A
Fb=1.35 ksi  Ft=0.83 ksi  Fv=0.18 ksi  Fcp=0.63 ksi
Fc=1.90 ksi  E=1900.00 ksi  Emin=690.00 ksi

SECTION PARAMETERS:
10x10
d=9.50 in
b=9.50 in
Ay=60.167 in²  Az=60.167 in²  A=90.250 in²
Iy=678.800 in⁴  Iz=678.800 in⁴  Ix=1145.159 in⁴
Sy=142.905 in³  Sz=142.905 in³

MEMBER PARAMETERS:
Key = 1.00  Kez = 1.00
ley = 16.00 ft  lez = 16.00 ft
ley/d = 20.21  lez/b = 20.21
CPy = 0.57  CPz = 0.57
FcEy = 2.08 ksi  FcEz = 2.08 ksi

INTERNAL FORCES AND ACTUAL STRESSES:
N = 48.55 kip  Vy = -0.00 kip  Vz = -0.01 kip
fc = 0.54 ksi  fvy = -0.00 ksi  fvz = -0.00 ksi

DESIGN WOOD STRENGTHS:
Fc' = Fc(1.90)*CM(1.00)*Ct(1.00)*CF(1.00)*CP(0.57)*KF(2.40)*Fi(0.90)*Lam(0.70) = 1.64 ksi
Fv' = Fv(0.18)*CM(1.00)*Ct(1.00)*KF(2.88)*Fi(0.75)*Lam(0.70) = 0.27 ksi
Emin' = Emin(690.00)*CM(1.00)*Ct(1.00)*KF(1.76) = 1032.24 ksi

RESULTS:
fc/Fc' = 0.33 < 1.00 [3.6.3]  OK!
fvy/Fv' = 0.00 < 1.00 [3.4.1]  OK!,  fvz/Fv' = 0.00 < 1.00 [3.4.1]  OK!
ley/d = 20.21 < 50.00 STABLE,  lez/b = 20.21 < 50.00 STABLE,

Section OK !!!
2nd Floor Column under Gravity Loading:

\[ P_u = (157.02 \text{ kips})(156)(9.716) = 49.38 \text{ k} \]

Strength:

Compressing:

\[ F_c = F_e \left[ C_{m} + C_{f} C_{l} \left( \frac{F}{F} \right)^{c} \right] \]

Modulus of Elasticity:

\[ E' = E \left( \frac{C_{m} + C_{f}}{2} \right) \]

\[ E' = \min \left[ \frac{e_{m}}{C_{m} + C_{f} + C_{l} \left( \frac{F}{F} \right)^{c}} \right] \]
2nd Floor Column under Lateral Loading:

(In Longitudinal Direction)

Assumptions:
- Design frame to support 25% of lateral load
- Shear walls resist entire lateral load
- Frame has rigid connections
- Frame is fixed at base
- All members (including masonry) have same stiffness (constant)
- Inflection point occurs in the middle of the column

\[
V_u = 0.25 \left( \frac{8,321.5 + 19,024.516}{2} \right) = 17,201.4115
\]

\[
M_u = (17,201.4115) \left( \frac{1}{2} \right) = 13,781.1755
\]
Vu Villa Structural Design

1st Floor Columns

Exterior Masonry Walls

2nd Floor Surf

2nd Floor Beams

1st Floor Columns

FBD

\[ V_u = \sqrt{\frac{11,171\text{lb} + 24,998\text{lb}}{5}} \]

\[ V_u = 1,808.51\text{lb} \]

\[ M_u = \left( \frac{1,808.51\text{lb}}{2} \right) \left( 16\text{ft} \right) \]

\[ M_u = 14,467.6\text{ lb-ft} \]
Strength:

Bending: \( F'_b = F'_b \left[ C_m C_t C_u C_F C_{f_u} C_{f_r} K_F \phi \right]^{(2)} \times \)  

Shear: \( F'_v = F'_v \left[ C_m C_t C_i C_r K_F \phi \right]^{(2)} \)  

Modulus of Elasticity:

\[ E_{m2} = E_{m1} \left[ C_m C_t C_i C_r K_F \phi \right]^{(2)} \]  

*See 2nd Floor Column Design Spreadsheet for Calculations.

2. Adjustment factors based on criteria and calculations from 2015 NDS.
3. Time-effect factor based on controlling LRFD load Combo equation; see Appendix N of the 2015 NDS.
VU VILLA STRUCTURAL DESIGN/REHABILITATION
MONTANA TECH

CONNECTION DESIGN
Roof Level Connection Design
**Joist to Beam Connection:**

*Note: All calculations are based on the NDS 2015 and Simpson Strong-Tie Catalogs.*

### Loading Conditions:
- **T/C, Tension/Compression Force in Transverse Direction:**
  - 3,181 (lb) (See Hand Calculations)
- **T/C, Tension/Compression Force in Longitudinal Direction:**
  - 5,274 (lb) (See Hand Calculations)
- **Pm,n, Bearing Force on Connection:**
  - 18,673 (lb) (See Hand Calculations)
- **Pm,v, Uplift Force on Connection:**
  - 76 (lb) (See Hand Calculations)

### Check Lateral Load Resistance:
- **Minimum Machine Bolt Yield Strength:**
  - 77,000 (psi) (Simpson Strong-Tie Catalog)
- **Machine Bolt Diameter:**
  - 3/8 (in) (Simpson Strong-Tie Catalog)
- **Vs, V, Shear in Transverse Direction at Connection:**
  - 189 (lb) (See Hand Calculations)
- **Vl, Shear in Longitudinal Direction at Connection:**
  - 189 (lb) (See Hand Calculations)
- **Pm,v, Uplift Force on Connection:**
  - 250 (lb) (See Hand Calculations)

### Check Vertical Load Resistance:
- **Check for Withdrawal:**
  - Simpson CC66 Column Cap with CTS218 Straps
- **Z, Reference Withdrawal Design Value for Nail:**
  - 0.65
  - Simpson Load Tables for CC66 indicate that uplift and lateral forces cannot be resisted by the CC66 Column Cap in the event of a Beam Splice Condition. For constructability purposes, Beam Splice must occur in this location. Therefore, strapping was also designed in order to transfer lateral and uplift forces created from bending conditions created from uplift. The following calculations demonstrate the ability of the specified strapping to resist lateral and uplift generated compression/tension forces at the beam splice location.

### Check Shippin Capacity at Beam Splice:
- **Cv, Cv, Compression/Tension Strip:**
  - 787 (lb)
- **w, Weight HGLBA beams over Beams:**
  - 1,183 (lb-ft) (See Hand Calculations)

### Check Shippin Capacity at Beam Splice:
- **σ, Ultimate Normal Stress from Uplift:**
  - 37.4 (psi) (See Hand Calculations)
- **Cv, Cv, Ultimate Compression/Tension from Uplift:**
  - 1,599 (lb) (See Hand Calculations)

### Check Shear Wall Connection:
- **θ, Connection Type:**
  - Simpson HGLBA Beam Seat with HTSM20 Straps
- **W, Reference Withdrawal Design Value for Nail:**
  - 48 (lb) (See Simpson Strong-Tie Catalog)
- **Fm, Minimum Nail Penetration on Connection:**
  - 0.79 (in) (Simpson Strong-Tie Catalog)
- **Pm,v, Minimum Nail Penetration:**
  - 0.79 (in) (Simpson Strong-Tie Catalog)

### Check Vertical Load Resistance:
- **Check for Rotation:**
  - L, Nail Length
  - 2.12 (in) (Simpson Strong-Tie Catalog)
- **S, Nail Shear Strength:**
  - 2.05 (psi) (Simpson Strong-Tie Catalog)
- **H, Nail Head Diameter:**
  - 3/4 (in) (Simpson Strong-Tie Catalog)
- **P, Minimum Tension/Compression Force in Transverse:**
  - 30 (lb) (Simpson Strong-Tie Catalog)
- **P, Minimum Tension/Compression Force in Longitudinal:**
  - 30 (lb) (Simpson Strong-Tie Catalog)
- **R, Rebar Shear Resistance in Transverse:**
  - 68 (lb) (Simpson Strong-Tie Catalog)
- **R, Rebar Shear Resistance in Longitudinal:**
  - 68 (lb) (Simpson Strong-Tie Catalog)

### Check Shear Wall Connection:
- **θ, Connection Type:**
  - Simpson HGLBA Beam Seat with HTSM20 Straps
- **W, Reference Withdrawal Design Value for Nail:**
  - 48 (lb) (Simpson Strong-Tie Catalog)
- **Fm, Minimum Nail Penetration on Connection:**
  - 0.79 (in) (Simpson Strong-Tie Catalog)
- **Pm,v, Minimum Nail Penetration:**
  - 0.79 (in) (Simpson Strong-Tie Catalog)

### Check Shear Wall Connection:
- **θ, Connection Type:**
  - Simpson HGLBA Beam Seat with HTSM20 Straps
- **W, Reference Withdrawal Design Value for Nail:**
  - 48 (lb) (Simpson Strong-Tie Catalog)
- **Fm, Minimum Nail Penetration on Connection:**
  - 0.79 (in) (Simpson Strong-Tie Catalog)
- **Pm,v, Minimum Nail Penetration:**
  - 0.79 (in) (Simpson Strong-Tie Catalog)
Lateral Loading:

\[(T/C)_x = (185.8 \text{ lb/ft})(45 \text{ ft}) = 8,361 \text{ lb}\]

\[(T/C)_y = (141.4 \text{ lb/ft})(2 \times 19.7 \text{ ft}) = 5,574 \text{ lb}\]

Vertical Loading:

\[P_{bearing} = 2 \times V_i, \text{Roof Beam} = 2(9,736 \text{ lb}) = 19,472 \text{ lb}\]

(for Uplift)

Governing Load Equation: \(0.9D + 1.0W = 0.9(17.5 \text{ psf}) + 1.0(18.4 \text{ psf}) = 26.5 \text{ psf}\)

T.A. for Connection = \((15 \text{ ft})(19.7 \text{ ft}) = 295.7 \text{ ft}^2\)

\[P_{uplift} = (-2.65 \text{ psf})(295.7 \text{ ft}^2) = 757 \text{ lb}\]
Check Connection Lateral Resistance:

\[ \text{Bolt Yield Strength} = (57,000 \text{ psi}) \left( \pi \cdot \left( \frac{5/8}{2} \right)^2 \right) = 17,487 \text{ lb} \]

\[ \text{Bolt Yield Strength} > T/C_x \text{ and } T/C_y \text{ O.K.} \]

Check Connection Vertical Resistance:

\[ \text{CC66 Bearing Capacity} = 33,275 \text{ lb} \]

\[ \text{CC66 Bearing Capacity} > P_{\text{Bearing}} \text{ O.K.} \]

\[ \text{CC66 Uplift Capacity} = 5,545 \text{ lb} \]

\[ \text{CC66 Uplift Capacity} > P_{\text{Uplift}} \text{ O.K.} \]
SEE NOTE ON 'ROOFLEVEL CONNECTION DESIGN' SPREADSHEET FOR FOLLOWING CALCULATION LOGIC/METHODOLOGY

Check Stopping Capacity at Beam Splice:

Lateral Forces generated from Uplift:

\[ W = (2.66 \text{pcf}) (15\text{ft}) = -39.8 \text{ lb/ft} \]

\[ M_u = \frac{W \cdot L^2}{8} \]

\[ = \frac{(-39.8 \text{ lb/ft})(19.71 \text{ ft})^2}{8} \]

\[ = -19301.8 \text{ lb-ft} \]

\[ c_u = \frac{M_u}{S_x} = \frac{-19301.8 \text{ lb-ft}(12 \text{ in/ft})}{404.3 \text{ m}^2} = -57.3 \text{ psi} \]

\[ c_u/T_u = c_u \cdot \frac{A}{2} = \left(57.3 \text{ psi}\right) \left(\frac{115.5 \text{ m}^2}{2}\right) = 3,308 \text{ lb} \]
C_{strap} = 5,470 \text{ lb} \quad (1)
T_{strap} = 10,040 \text{ lb} \quad (2)

\text{For 4-CTS218 Compression/Tension Straps installed 2 each side of Beam.}

\text{C}_{\text{strap}} > \frac{C_w}{T_u} \quad \text{OK.}

\text{T}_{\text{strap}} > \frac{C_w}{T_u} \quad \text{O.K.}

\text{C}_{\text{strap}} > \frac{(I/L_y)}{2} \quad \text{O.K.}

\text{T}_{\text{strap}} > \frac{(I/L_y)}{2} \quad \text{O.K.}

1^\text{Bolt yield strength is for a Grade 2 Macroline Bolt acquired from BoltDepot.com.}

2^\text{Values from Simpson Strong-Tie Catalog Specifications and load tables.}
Loading Calculations:

Shear in Transverse Direction

\[ V_x = \frac{(283.4 \text{ lb/ft})(15\text{ ft})}{60.125\text{ ft}} = 472.5\text{ lb} \]

Shear in Longitudinal Direction

\[ V_y = \frac{(516.8 \text{ lb/ft})(15\text{ ft})}{60.125\text{ ft}} = 141.4\text{ lb} \]

Bearing Force on Connection

\[ P_{\text{bearing}} = V_y \times \text{Root Beam} = 9,736\text{ lb} \]

Uplift Force on Connection:

(Governing Load Combo):

\[ 0.9D + 1.0W = (69)(19.5) - 10(266) = -10,858\text{ lb} \]

\[ P_{\text{uplift}} = (-10,858\text{ lb})(15\text{ ft})\left(\frac{19.71\text{ ft}}{2}\right) = -1,600\text{ lb} \]
Check Lateral Load Resistance:

Rebar Shear (Transverse Direction):
\[ R_x = (60 \text{ ksi}) \pi \left( \frac{750 \text{ in}}{2} \right)^2 = 26,506 \text{ lb} \]
\[ R_x > V_x \quad \text{O.K.} \]

Bolt Tension (Transverse Direction):
\[ B_x = 24,700 \text{ lb} \]
\[ B_x > V_x \quad \text{O.K.} \]

Bolt Shear (Longitudinal Direction):
\[ Z_y = 10,305 \text{ lb} \]
\[ Z_y > V_y \quad \text{O.K.} \]

Check Vertical Load Resistance:

Bearing Strength:
\[ H + L + A \text{ Bearing} = 18,750 \text{ lb} \]
\[ H + L + A \text{ Bearing} > P \text{ Bearing} \quad \text{O.K.} \]

Uplift Strength:
\[ 2 \times H + T \text{SMAC upl} = 2(1,110 \text{ lb}) = 2,220 \text{ lb} \]
\[ 2 \times H + T \text{SMAC upl} > P \text{ upl} \quad \text{O.K.} \]
Shear in Transverse Direction

\[ V_u = \left( \frac{566.8 \text{ lb/ft}}{12 \text{ ft}} \right) \left( \frac{15 \text{ ft}}{12 \text{ ft}} \right) = 12.61 \text{ lb/ft} \]

\[ V_{ux} = (12.61 \text{ lb/ft})(15 \text{ ft}) = 189.15 \text{ lb} \]

Shear in Longitudinal Direction

\[ V_{uy} = \left( \frac{283.4 \text{ lb}}{30 \text{ ft}^3} \right) (16 \text{ ft}) = 141.71 \text{ lb/ft} \]

\[ V_{uy} = (141.71 \text{ lb/ft})(16 \text{ ft}) = 188.43 \text{ lb} \]

Uplift on Connections

(Governing Load Combination:

\[ 0.9 D + 1.0 W = 0.9 (15.7 \text{ psf}) + 1.0 (-26.4 \text{ psf}) = -12.5 \text{ psf} \]

\[ A_{connection} = \left( \frac{16}{12} \text{ ft} \right) (15 \text{ ft}) = 20 \text{ ft}^2 \]

\[ P_{uplift} = (-12.5 \text{ psf})(20 \text{ ft}^2) = 250 \text{ lb} \]

\[ P_{uplift} = 26.6 \text{ psf} \]
Check Lateral Load Resistance:

For shear:  \[ Z' = Z [C_n + C_g C_a C_g C_d; C_m K = \Phi] \]

\[ #. Z' > V_{ux} \quad \text{O.K.} \]

\[ #. Z' > V_{uy} \quad \text{O.K.} \]

Check for penetration:

\[ P_{min} = 6.0 \cdot D = 0.79 \text{ in} \]

\[ \text{Penetration} = L - \frac{H_3}{\cos 30^\circ} \]

\[ = 2\frac{1}{2}'' - \frac{0.833''}{\cos 30^\circ} = 1\frac{1}{2}'' \]

\[ \text{Penetration} > P_{min} \quad \text{O.K.} \]

Check for withdrawal:

\[ W' = W [C_t + C_g C_m K = \Phi] \]

\[ #. W' \cdot \text{Penetration} > P_{pullout} \quad \text{O.K.} \]

1. Reference Design Values from 2015 NDS
2. Adjustment factor based on criteria & calculations from 2015 NDS
3. Time-effect factor based on governing LRFD load center equation.
2nd Floor Connection Design
2nd Floor Column to 1st Floor Column Connection:

O.K. (NDS 2015 Section 12.1.6.5) (lb) (See Hand Calculations)

P19,664 (NDS 2015 Table 11.3.1) (Simpson Strong-Tie Catalog; Bearing on Min. 6 3/4" Wood) (lb) (psi)

O.K. (lb) (in) (psi)

650 (NDS 2015 Section 11.3.6.1) (in) (Simpson Strong-Tie Catalog) (psi)

3.32 (Simpson Strong-Tie Catalog) (in) (See Hand Calculations)

Beam Strength: 26,353 (lb) (Based on a 10x10 Column for Bearing Area)

Beam Strength + Pmax = 6.4:

Check for Uplift: 313 (lb) (Simpson Strong-Tie Catalog) ( erf) Uplift Strength

Uplift strength + Pmax = 6.4:

Check for Lateral Load Resistance (Transverse Direction):

Minimum Bolt Yield Strength 57,800 (lb) (Bolt Depot inc; Grade 2 Bolt) (psi)

Bolt Diameter 1.5 (in) (Simpson Strong-Tie Catalog) (Bolt Diameter)

Vmax, Machine Bolt Yield Resistance 25,366 (lb) (Bolt Area times Strength for 2 Bolts)

Vmax = TcB = 6.4:

Check for Lateral Load Resistance (Longitudinal Direction):

Minimum Bolt Yield Strength 57,800 (lb) (Bolt Depot inc; Grade 2 Bolt) (psi)

Bolt Diameter 1.5 (in) (Simpson Strong-Tie Catalog) (Bolt Diameter)

Vmax, Machine Bolt Yield Resistance 25,366 (lb) (Bolt Area times Strength for 2 Bolts)

Vmax = TcB = 6.4:

Check for Penetration:

Check for Withdrawal:

Check for Bearing:

Check for Penetration:

Check Initial Load Resistance:

L, Nail Length 3 (in) (AIPMPS 2015 Appendix F; Table 1C)

Tn, Minimum Nail Penetration 2.89 (in) (NDS 2015 Section 12.1.4.3) (min)

Pmax, Minimum Nail Penetration = 2.89 (in) (NDS 2015 Section 12.1.4.3) (min)

Penetration = 2.89:

Check for Withdrawal:

W, Withdrawal Design Value for Nail:

12 (lb) (NDS 2015 Section 12.8.2)

Cw1 1 (NDS 2015 Section 12.8.2)

Cw2 1.67 (NDS 2015 Section 12.8.2)

W 10 (lb) (NDS 2015 Section 12.8.2)

FPW = 10 (lb) (NDS 2015 Section 12.8.2) (min)

FPW(Withdrawal) = 2.89:

HGLBD, Simpson HGLBD Bearing Strength 18,150 (lb) (Simpson Strong-Tie Catalog, Bearing on Min. 2 1/4" Wood) (lb) (psi):
Lateral Loading:

\[ T_{ux}/C_{ux} = \left(\frac{415.8 \text{ lb/ft}}{15 \text{ ft}}\right) \times 15 \text{ ft} = 18,708 \text{ lb} \]

\[ T_{uy}/C_{uy} = \left(\frac{318.4 \text{ lb/ft}}{2 \times 19.71 \text{ ft}^2}\right) = 12,472 \text{ lb} \]

Vertical Loading:

\[ P_{\text{bearing}} = \left(6.52 \text{ kips/ft}\right) \times 15 \text{ ft} \times 19.71 \text{ ft}^2 = 19,614 \text{ kips} \]

(for uplift)

**Governing Load Equation:**

\[ 0.9D + 1.0W = 0.9 \left(18 \text{ psf}\right) + 1.0 \left(18.4 \text{ psf}\right) = 36.4 \text{ psf} \]

**T.A. for Conduit:** 295.62 \text{ ft}^2

**Puplift:** -643.1 lb
2nd Floor Column to 2nd Floor Beam Connection

(Vetical Loading)

Check for Bearing:

\[ F_{c1}^\prime = F_{c1}^{(2)} \left[ C_h C_e C_b K_F \phi \right]^{(2)} \]

\[ F_{c1} = 1977 \text{ psi} \]

(See 2nd Floor Connection Spreadsheet)

Bearing Strength = (977 psi)(5.5 in)(0.9 in)

\[ = 29,553 \text{ lb} > P_{Bearing} \text{ O.K.} \]

Check for Uplift:

Uplift Strength = 1 lbf

Uplift Strength > P_{Uplift} \text{ O.K.}

(Lateral Loading)

Check for Lateral Load Resistance (Transverse Direction):

Bolt Yield Strength = 57,000 psi (Grade 2 Bolt)

Bolt Diameter = \( \frac{1}{2} \)"

Bolt Yield Resistance = \( 2(57,000 \text{ psi})(\pi \left( \frac{3/4}{2} \right)^2) = 22,384 \text{ lbf} > T_{ux}/C_{ux} \text{ O.K.} \)
Check for Lateral Resistance (Longitudinal Direction):

Bolt Resistance:
Bolt Yield Strength = 57,000 psi (Grade 2 Bolt)
Bolt Diameter = \frac{3}{4}".

Bolt Yield Resistance = 2\left(57,000\psi\right)\frac{\pi}{4}\left(\frac{3/4}{2}\right)^2 = 57,364 \text{ lb} > T_{uy}, C_{uy} \text{ OK.}

Strap Resistance:
\sigma_y = 33,000 psi
A = t \cdot \frac{L}{2} = \left(\frac{3}{16}\right) \left(\frac{18}{2}\right) = 1.688 \text{ in}^2

V_{strap} = \sigma_y \cdot A = \left(33,000 \psi\right) \left(1.688 \text{ in}^2\right) = 55,688 \text{ lb} \text{ O.K.}

2nd Floor Beam to 1st Floor Loading Conditions
(Lateral Loading)

\frac{T_{ux}}{C_{uy}} = 18,708 \text{ lb}

\frac{T_{uy}}{C_{uy}} = 12,472 \text{ lb}

(Vertical Loading)

P_{bearing} = \left(194.44 \text{ psi}\right) \left(15.87\right) \left(14.71 \text{ ft}\right) = 57,481 \text{ lb}

(For Uplift)

Governing Load Equation: 0.9D + 1.0W = \left(9\right)(348.7\text{ psf}) + 1.0(-18.4\text{ psf})

= 12.92 \text{ psf}

P_{uplift} = 0 \text{ lb}

Note: The factored dead load exceeds the factored uplift from wind; therefore, no uplift occurs on the connection.
(Vertical loading)

Check for Bearing:

\[ F_{c1} = F_{c1}^{(2)} \left[ C_m C_e C_o V_f \phi \right]^{(2)} \]

\[ F_{c1} = 977 \text{ psi} \]

Bearing Strength = \((977 \text{ psi})(9.5 \text{ m})(9.5 \text{ m})\)

= 88,170 lb > Bearing O.K.

Check for Uplift:

No uplift exists on this connection.

Check for Lateral Load Resistance (Transverse Direction):

Bolt Yield Strength = 57,000 psi (Grade 2 Bolt)

Bolt Diameter = 1/2"

Bolt Yield Resistance = 22,384 lb > Tuy/Cuy O.K.

Check for Lateral Load Resistance (Longitudinal Direction):

Bolt Resistance:

Bolt Yield Strength = 57,000 psi (Grade 2 Bolt)

Bolt Diameter = 5/8"

Bolt Yield Resistance = 34,976 lb > Tuy/Cuy O.K.

Strap Resistance:

\[ f_y = 33,000 \text{ psi}, A = \ell \cdot \frac{1}{2} = 1.992 \text{ in}^2 \]

\[ V_{\text{strap}} = f_y A = 65,782 \text{ lb} \quad \text{O.K.} \]

\(^1\text{Value from 2019 NDS reference design value tables.}\)

\(^2\text{Adjustment factors based on criteria and calculations from 2015 NDS.}\)
Loading Conditions:

Shear in Transverse Direction:

\[ V_{tx} = \frac{634.1 \times 15}{12} = 9,511.5 \text{ lb} \]

Shear in Longitudinal Direction:

\[ V_{ty} = \frac{(1288.3 \times 15)}{60.125} = 316.4 \text{ lb} \]

\[ V_{ty} = (316.4 \times 60.125) = 19,024.5 \text{ lb} \]

Beam Force on Connection:

\[ P_{beam} = V_{1st \ floor \ beam} = 28,741 \text{ lb} \]

Uplift on Connection:

\[ (Gravity \ Load \ Center) \]

\[ 0.9D + 1.0W = 0.9 \times 34.8 \text{ psi} + 1.0 \times 26.6 \text{ psi} \]

\[ = 61.72 \]

\[ P_{uplift} = 0 \text{ lb} \]

Note: Factored dead load exceeds the factored uplift from wind. Therefore no uplift will occur on connection.
Check Lateral Load Resistance:

Rebar Shear (Transverse Direction):

\[ R_x = (60 \text{ ksi}) \pi \left( \frac{1.760}{2} \right)^2 = 26,506 \text{ lb} \]

\[ R_x > V_x \quad \text{OK} \]

Wood Beam (Basic Check):

Beam Strength = \( (650 \text{ psi})(1.67 \times 0.9) = 977 \text{ psi} \)

Rebar Lateral Force = \( \frac{19074 \times 0.6}{3(1^2)(12)} \) = 704 psi

Beam Strength > Rebar Lateral Force \( \text{OK} \)

Bolt Tension (Transverse Direction):

\[ B_x = 24,700 \text{ lb} \]

\[ B_x > V_x \quad \text{OK} \]

Bolt Shear (Longitudinal Direction):

\[ Z' = (10,305 \text{ lb/ft})(2 \text{ bolts}) = 20,610 \text{ lb} \]

\[ Z' = V_y \quad \text{OK} \]

Check Vertical Load Resistance:

Bearing:

\[ HGLBD_{\text{Bearing}} = 35,100 \text{ lb} \quad \text{(For 6\% Beam)} \]

\[ HGLBD_{\text{Bearing}} > P_{\text{Bearing}} \quad \text{OK} \]
Vu Villa
Structural Design

2nd Floor Connections - Joint to Beam Connection

KBS
8/11/19

16" O.C.
Existing
2x12 Joists

Plan View  2nd Floor Beam

Cross-Section View
N.T.S.

Profile View
N.T.S.
Villa Structural Design

**Leading Conditions:**

\[ \text{Shear in Transverse Direction:} \]

\[ V_{\text{tx}} = \frac{(1208.3 \text{ lb}) \left( \frac{1}{12} \text{ ft} \right)}{60 \text{ ft}^2} = 28.13 \text{ lb/ft} \]

\[ V_{\text{tx}} = \left( 28.13 \frac{\text{lb}}{\text{ft}} \right) (15 \text{ ft}) = 421.9 \text{ lb} \]

**Shear in Longitudinal Direction:**

\[ V_{\text{ty}} = 0 \]

\[ V_{\text{ty}} = 0 \]

\[ P_{\text{ex}} = 0 \]

**Uplift Connection:**

Note: Uplift from roof is transferred through beams & columns, therefore no uplift occurs on 2nd floor joists.

\[ P_{\text{uplift}} = 0 \]
Check for Lateral Load Resistance:

For shear:

\[ z' = \frac{1}{E'[t]} \left( \sum_{i=1}^{n} C_i u_c G_i \right) \]

\[ \# \cdot z' > V_{y,x} \quad \text{O.K.} \]

\[ \# \cdot z' > V_{y,y} \quad \text{O.K.} \]

Check for Penetration:

\[ \frac{30'}{3} = 1' \]

\[ \cos 30' = \frac{1'}{\text{Hyp.}} \]

\[ \text{Hyp.} = \frac{1'}{\cos 30'} \]

\[ P_{mn} = 6 \cdot 0.89 = 5.34 \text{ in} \]

\[ \text{Penetration} = L - \frac{y_3}{\cos 30'} = 1.85 \]

\[ \text{Penetration} > P_{mn} \quad \text{O.K.} \]

Check for Withdrawal:

\[ W' = W^{1'} \left( E'[t] \sum C_i G_i \right) \]

\[ \# \cdot W' \cdot \text{Penetration} > P_{pull} \quad \text{O.K.} \]

1. Values from 2015 NDS reference design value tables.
2. Adjustment factors based on criteria & calculations from 2015 NDS.
3. Time-effect factor has influence on concrete load control equation.
VU VILLA STRUCTURAL DESIGN/REHABILITATION
MONTANA TECH

EXISTING MEMBER VERIFICATION
VU VILLA STRUCTURAL DESIGN/REHABILITATION
MONTANA TECH

DIAPHRAGM VERIFICATION
Roof Diaphragm Verification
### Vu Villa Structural Design and Rehabilitation

#### Existing Member Verification

**Roof Diaphragm**

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<td>Maximum Depth of Sheathing &amp; 6&quot; Roof Joist</td>
<td>APA Structural Panels Load-Span Tables</td>
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<td><strong>Existing Span Rating:</strong></td>
<td></td>
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<tr>
<td></td>
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</tbody>
</table>

#### Material Properties

- **Nominal Width of Nailed Face:** 2 (Based on Existing Conditions)
- **Shear:** 200 147 54.9 (psf)
- **Bending:** 87.0 81.0 54.9 (psf)
- **DL+LL (L/240):** 83.0 147 54.9 (psf)

#### Loading

- **Nominal Width of Nailed Face:** 2 (Based on Existing Conditions)
- **Controlling Load:** Seismic (From Lateral Load Calculations)

#### Geometry

- **Nominal Width of Nailed Face:** 2 (Based on Existing Conditions)
- **Controlling Load:** Seismic (From Lateral Load Calculations)

#### Analysis

- **Minimum Fastener Penetration:** 1 3/8 (in) (SDPWS 2015 Table 4.2C)
- **Minimum Fastner Penetration:** 1 2/5 (in) (SDPWS 2015 Table 4.2C)

#### Nailing

- **Nail Type:** Common (SDPWS 2015 Table A1)
- **Nail Size:** 8d (SDPWS 2015 Table 4.2C)

#### Case: 1 (SDPWS 2015 Table 4.2C)

- **withdrawal strength:** 181 (psf) (Based on 16" Joist Spacing & Nail Spacing Above)
- **Penetration*W' > Uplift:**
- **Penetration*W' = 121 lb/nail**
- **Penetration*W' = 78 lb/nail**

#### Withdrawal Strength

- **Penetration*W' = 64.7 (lb/in of penetration)** (NDS 2015 Table 11.3.1)

#### Shear

- **Check for Shear:**
- **Penetration*W' > Uplift:**
- **Penetration*W' = 78 lb/nail**

#### Nailing Withdrawal

- **Check for Nailing Withdrawal:**
- **Penetration > Min. Penetration:**

#### Diaphragm Design

- **Δallow > ẟdia**
- **Δ, Maximum Mid-Span Diaphragm Deflection:**

#### Member Categories

- **Axial Diaphragm:**
- **W, Reference Lateral Design Value for Nail:**
- **Δ, Maximum Mid-Span Diaphragm Deflection:**

#### Parameters

- **Cg 1 (NDS 2015 Section 11.3.6)**
- **Ceg 1 (NDS 2015 Section 12.5.2)**
- **CΔ 1 (NDS 2015 Section 12.5.1)**
- **Cfn 1 (NDS 2015 Section 12.5.4)**
- **Cdi 1 (NDS 2015 Section 12.5.3)**
- **Ctn 1 (NDS 2015 Section 12.5.4)**

#### Force Equilibrium

- **A, Cross-Sectional Area of Chords**
- **G, Specific Gravity of Sheathing Material**
- **Ct, Diaphragm Chord Splice Slenderness:**
- **φ, Diaphragm Core Slants:**
- **Sh, Sheathing Material:**
- **R, Reference Lateral Design Value for Nail:**
- **Δ, Maximum Mid-Span Diaphragm Deflection:**
- **E, Modulus of Elasticity of Diaphragm Chords**
- **λ, Boundary Nail Length**
- **ν, Diaphragm Core Slants:**
- **Δ, Maximum Mid-Span Diaphragm Deflection:**

#### Analysis

- **Δ, Allowable Drift:**
- **Z, Reference Lateral Design Value for Nail:**
- **W, Reference Withdrawal Design Value**
- **Z', 166.2 (lb/Nail) (NDS 2015 Table 11.3.1)**
- **W', 64.7 (lb/in of penetration)** (NDS 2015 Table 11.3.1)

#### Design

- **Z, Reference Lateral Design Value for Nail:**
- **W, Reference Withdrawal Design Value**
- **Z', 166.2 (lb/Nail) (NDS 2015 Table 11.3.1)**
- **W', 64.7 (lb/in of penetration)** (NDS 2015 Table 11.3.1)

#### Verification

- **Δallow, Allowable Drift:**
- **Δ, Allowable Drift:**
- **Δ, Maximum Mid-Span Diaphragm Deflection:**
- **Δ, Maximum Mid-Span Diaphragm Deflection:**
- **Δ, Maximum Mid-Span Diaphragm Deflection:**

#### Calculation

- **Δallow, Allowable Drift:**
- **Δ, Maximum Mid-Span Diaphragm Deflection:**
- **Δ, Maximum Mid-Span Diaphragm Deflection:**
- **Δ, Maximum Mid-Span Diaphragm Deflection:**
- **Δ, Maximum Mid-Span Diaphragm Deflection:**

#### Verification

- **Δallow, Allowable Drift:**
- **Δ, Maximum Mid-Span Diaphragm Deflection:**
- **Δ, Maximum Mid-Span Diaphragm Deflection:**
- **Δ, Maximum Mid-Span Diaphragm Deflection:**
- **Δ, Maximum Mid-Span Diaphragm Deflection:**

#### Report

- **Vu Villa Structural Design and Rehabilitation**
- **Existing Member Analysis**
- **Montana Tech**
### Existing Member Analysis

**Vu Villa Structural Design and Rehabilitation**

**Montana Tech 2 of 3**

#### Shear

- **200**  
- **147**  
- **54.9 (psf)**

#### Bending

- **87.0**  
- **81.0**  
- **54.9 (psf)**

**νu, Ultimate Shear Loading**

- **283.4 (plf)** (For Boundary Nailing)

**Nominal Width of Nailed Face**

- **2** (Based on Existing Conditions)

**Controlling Load**

- Seismic (From Lateral Load Calculations)

**Panel Required Nailing**

- **Δdia, Maximum Mid-Span Diaphragm Deflection**
- **24** (Nails)
  
- **Cfn 1 (NDS 2015 Section 12.5.4)**
  
- **Ceg 1 (NDS 2015 Section 12.5.2)**
  
- **Vs, Shear Capacity**
- **360 (plf)** (SDPWS 2015 Table 4.2C)
  
- **Minimum Nominal Panel Thickness**
- **15/32 (in)** (SDPWS 2015 Table 4.2C)
  
- **Minimum Fastner Penetration**
- **1 3/8 (in)** (SDPWS 2015 Table 4.2C)
  
- **Nail Type**
- **Common** (SDPWS 2015 Table A1)
  
- **Nail Size**
- **8d** (SDPWS 2015 Table 4.2C)
  
- **Case**
- **3** (SDPWS 2015 Table 4.2C)

#### Check for Shear

- **W, Diaphragm Width**
- **60 (ft)**
  
- **L, Diaphragm Length**
- **60 (ft)**

**νu,x, Ultimate Shear Loading**

- **200.0 (plf)** (For Interior Nailing)

**x, Distance from Shear Wall to Edge Nailing**

- **9 1/3 (ft)** (Location for Permitted Nailing Change)

**fm’, Modulus of Elasticity of Diaphragm Chords**

- **100 (psi)**

**A, Cross-Sectional Area of Chords**

- **76.5 (in²)** (Assumed Depth of Sheathing & 6" Roof Joist)

**E, Modulus of Elasticity of Diaphragm Chords**

- **70,000 (psi)** (ACI 530-13 Section 1.8.2.2.1)

**Ga, Apparent Diaphragm Shear Stiffness**

- **5.5 (kips/in)** (SDPWS 2015 Table 4.2C; Used Plywood Value for Conservatism)

**Δ Allow, Allowable Drift**

- **3.36 (in)**

**Δ, Ultimate Drift**

- **0.032 (in)**

**Kf 3.32 (NDS 2015 Table 11.3.1)**

**CM 1 (NDS 2015 Table 11.3.3)**

**CΔ 1 (NDS 2015 Section 12.5.1)**

**Cg 1 (NDS 2015 Section 11.3.6)**

**Ct 1 (NDS 2015 Table 11.3.4)**

**λ 1.0 (NDS 2015 Appendix N.3.3)**

**Φ 0.65 (NDS 2015 Table 11.3.1)**

**W', Reference Withdrawal Design Value**

- **30 (lb/in of penetration)** (NDS 2015 Table 12.2c)

**G, Specific Gravity of Sheathing Material**

- **0.49 (NDS 2015 Table 12.3.3.A)**

**Z', Withdrawal Strength**

- **173.3 (lb/Nail)** (NDS 2015 Table 11.3.1)

**W', Penetration*W'**

- **115 lb/nail**

**Penetration*W' > Uplift**

- **Ultimate Uplift**
- **25.8 (psf)**

**W' 64.7 (lb/in of penetration)** (NDS 2015 Table 11.3.1)

**Penetration*W' > Uplift**

- **Penetration**
- **1 7/9 (in)** (SDPWS 2015 Table 4.2C)

**Penetration > Min. Penetration**

- **Penetration**
- **1 1/3 (in)** (SDPWS 2015 Table 4.2C)

**Penetration*W'**

- **73 lb/nail**

**Penetration*W' > Uplift**

- **Ultimate Uplift**
- **25.8 (psf)**

**W' 56.1 (lb/in of penetration)** (NDS 2015 Table 11.3.1)

**Penetration*W' > Uplift**

- **Penetration**
- **1 1/3 (in)** (SDPWS 2015 Table 4.2C)

**Penetration*W'**

- **73 lb/nail**

**Penetration*W' > Uplift**

- **Ultimate Uplift**
- **25.8 (psf)**

**W' 56.1 (lb/in of penetration)** (NDS 2015 Table 11.3.1)

**Penetration*W' > Uplift**

- **Penetration**
- **1 1/3 (in)** (SDPWS 2015 Table 4.2C)

**Penetration*W'**

- **73 lb/nail**

**Penetration*W' > Uplift**

- **Ultimate Uplift**
- **25.8 (psf)**

**W' 56.1 (lb/in of penetration)** (NDS 2015 Table 11.3.1)
SPECIFICATIONS & MINIMUM CRITERIA FOR EXISTING ROOF DIAPHRAGM

Roof Sheathing:
- Transverse Direction permitted 3/8-inch panel thickness, however, Longitudinal direction (and SDPWS 2015 Section 4.4.1.2) required greater thickness
  - Minimum Sheathing Thickness
    + 15/32-inch Standard OSB (Does NOT need to be Structural 1)
    or
    + 19/32-inch Sanded Plywood (Does NOT need to be Structural 1)
  - Minimum 4x8 Sheets Oriented along Longitudinal Direction [North-South] (per SDPWS 2015 Section 4.2.7.1.3)
  - Engineer shall assess material layout and condition; if material is deficient or varies from assumptions, Engineer shall re-evaluate material for compliance
  - If material is inadequate; material shall be replaced according to Specifications

Roof Nailing:
- Minimum Nailing Criteria:
  + 8d Common Nails (Boundary)
  + 6d Common Nails (Edge/Interior)
- Boundary Pattern:
  + 8d Common Nails @ 6" O.C. Spacing (East-West along Joist)
  + 8d Common Nails @ 16" O.C. Spacing (North-South along Joist Spacing - equivalent to Joist Spacing)
- Edge Pattern:
  + Minimum 3/8" from Edge of Panel (SDPWS 2015 Section 4.2.7.1.3)
  + 6d Common Nails @ 6" O.C. Spacing (East-West along Joist
  + X = 9'-4" from Shear Wall for start of Interior Nailing
- Interior Pattern:
  + 6d Common Nails @ 12" O.C. Spacing (SDPWS Section 4.2.7.1.3)
- Engineer shall assess nail layout and condition and re-evaluate if variation from assumptions exists.
  - If inadequate nailing exists; the minimum additional nailing shall be required to bring the diaphragm into compliance in accordance with the Specifications

1See Roof Diaphragm Detail for Minimum Nailing Pattern Example
Transverse Analysis

\[ U_t = 283.4 \text{ lb/ft} \]

\[ C_t = 4,260 \text{ lb} \]

\[ T_u = 1,260 \text{ lb} \]

Wu = 566.8 lb/ft

---

See chord & drag analysis for unit shear & tension/compression values.

### Panel Span Rating & Minimum Thickness:

<table>
<thead>
<tr>
<th>Load Type</th>
<th>Permissible Load Values (^2)</th>
<th>Actual Applied Loads (GsF) (^3)</th>
</tr>
</thead>
<tbody>
<tr>
<td>LL (Y=60)</td>
<td>56.98 (Plywood) 0.83</td>
<td>18</td>
</tr>
<tr>
<td>DL+LL (Y=10)</td>
<td>83.117</td>
<td>55</td>
</tr>
<tr>
<td>Bending</td>
<td>87.81</td>
<td>55</td>
</tr>
<tr>
<td>Shear</td>
<td>200.147</td>
<td>55</td>
</tr>
</tbody>
</table>

Minimum Panel Thickness \(^4\) = \frac{3}{8}\text{-inch Sanded Plywood, or} \frac{3}{16}\text{-inch OSB (Standard Sheathing)}

\(^2\) Values from APA Load-Span Tables for Structural-use Panels

\(^3\) Values from Gravity Load Calculations (Snow/Live, Dead, RUR & Sheathing)

\(^4\) \frac{3}{8}\text{-inch is sufficient for plywood; however, nailing requires } \frac{3}{16}\text{-inch minimum panel.}

Note: Panel thickness is the minimum structural requirement based on the assumption of plywood sub-floor. Engineer is to assess and determine whether existing sub-floor is of adequate condition and structure to be reused. Further analysis may be required in the event...
Panel Required Nailing:

Based on visual inspection assume unblocked rafters.
Assume Case 1: Nails used on 2" Nominal Rafter. Service Force Controls.

$V_u = 283.4 \text{ lb/ft}$ (for Boundary Nails)

<table>
<thead>
<tr>
<th>Nail Type</th>
<th>Minimum Nailer</th>
<th>Minimum Panel Thickness</th>
<th>$V_s$ (PLF)</th>
<th>$\phi V_s$ (PLF)</th>
</tr>
</thead>
<tbody>
<tr>
<td>6d</td>
<td>1 1/4</td>
<td>3/8</td>
<td>340</td>
<td>272</td>
</tr>
<tr>
<td>8d</td>
<td>1 3/8</td>
<td>3/8</td>
<td>430</td>
<td>344</td>
</tr>
<tr>
<td>10d</td>
<td>1 1/2</td>
<td>1 15/32</td>
<td>510</td>
<td>408</td>
</tr>
</tbody>
</table>

Values and Case from 2015 SDPWS Table 4.2.6.

$\phi = 0.9$ adjustment factor for LRFD according to 2015 SDPWS Section 4.2.3

$V_u = 283.4 \text{ lb/ft} \leq \phi V_s = 344 \text{ lb/ft}$

Boundary Nailing: 8d Nails @ 6" O.C.

Nailing Change Location: $X = \left(\frac{283.4 - 272}{283.4 - 344}\right)(32) = 1.21 \text{ ft} \approx 1' - 3''$

Edge Nailing: 6d Nails @ 6" O.C.

This only applies at least one row from 2" rafter trim, Sheathing #1 and #2.
Panel Required Nailing (continued):

Check on Nailing Penetration

Boundary Nails: 8d Common: \( L = 2\frac{1}{2}'' \) \( D = 0.131'' \)

Penetration = \( L - t_{ply} - 2D \)

\[ = 2\frac{1}{2}'' - \frac{3}{8}'' - 2(0.131)'' \]

\[ = 1.863'' > \text{Min. Penetration} = 1.25'' \text{ O.K.} \]

Edge/Interior Nails: 6d Common: \( L = 2'' \) \( D = 0.113'' \)

Penetration = \( L - t_{ply} - 2D \)

\[ = 2'' - \frac{3}{8}'' - 2(0.113)'' \]

\[ = 1.40'' > \text{Min. Penetration} = 1.25'' \text{ O.K.} \]

Number of Boundary Nails along Chord:

\[ I = \left[ Z \cdot C_m \cdot c_i \cdot c_{ej} \cdot c_{e} \cdot c_{di} \cdot c_{m} \cdot k_{F} \cdot F_{O} \right]^{0.91} \cdot \left[ 0.8 \cdot 0.8 \cdot 0.8 \cdot 0.8 \cdot 0.8 \cdot 0.8 \cdot 0.8 \cdot 0.8 \cdot 0.8 \cdot 0.8 \right]^{0.1} = 164.15 \]

\[ n = \frac{T_{e, \text{chord}}}{(I^2)_{F}} = \frac{4.260}{164.15} = 6.14 \text{ Nails} \]

Spacing = \( \frac{60''}{2.5 \text{ Nails}} = 28.8'' \text{ in} \leq 28'' \text{ in O.C.}\)

Note: Spacing requires a minimum of 16 in O.C. due to Rafter spacing constraints, so still ok.

\(^8\) Values from 2015 SDPWS Table A1
\(^9\) Values from 2015 SDC (\( f_{y} \) from Table 12.4 - Values from A. 9 and 12)
Transverse Analysis

Check on Deflection:

\[ \delta_{\text{dia}} = \frac{5v^2L^4}{384EI} + \frac{0.25v^2L}{1000GAE} + \frac{v^2L^2}{2W} \]

\[ = \frac{5(238.4 \text{ in})(L0.125ft)^3}{8(700 \cdot 100psf)(L0.125ft)(L0.125ft)} + \frac{0.25(238.4 \text{ in})(L0.125ft)}{1000 (5.5 \text{ k/ft})} \]

\[ = 0.894 \text{ in} \]

\[ \Delta_{\text{Daphne}} = \frac{L}{600} = \frac{(60.125\text{ ft})(12\text{ in/ft})}{600} = 1.20 \text{ in} \]

\[ \delta_{\text{dia}} \leq \Delta_{\text{Daphne}} \quad \text{O.K.} \]

---

10 Equation from ACI 530, \( f_m = 100\text{ psf} \) from Table 5.3.2 Assuming worst-case (Typical masonry, standard)  
12 Assumed shear wall thickness (6=12\text{ in}) of masonry wall times depth of sheathing + rafter,  
13 No chord splice exists for masonry shear wall.  
14 For \( G \)-value from SPANS Table 4.1.2, sanded plywood value was used for determining deflection as it provided the greater deflection compared to OSB (Sheathing vs. Structural).  
15 Assumed permissible deflection was equivalent to allowable Masonry deflection (\( \frac{L}{500} \)), see ASCE 7-02 Section 9.5.2.5 and 2-2.7: "The deflection in the plane of the diaphragm...shall not exceed the permissible deflection of the attached elements."
Longitudinal Analysis

\[ T_u = 4,260 \text{ lb} \]

\[ W_{1,0,\text{in}} = 566.8 \text{ lb/ft} \]

Panel Span Rating & Minimum Thickness:

<table>
<thead>
<tr>
<th>Load Type</th>
<th>Permissible Load Values (lbs)</th>
<th>Actual Applied Loads (lbs)</th>
<th>O.K.</th>
</tr>
</thead>
<tbody>
<tr>
<td>LL (1/3w)</td>
<td>56.98</td>
<td>18</td>
<td>O.K.</td>
</tr>
<tr>
<td>DL+LL (1/2w)</td>
<td>83.147</td>
<td>55</td>
<td>O.K.</td>
</tr>
<tr>
<td>Bending</td>
<td>87.81</td>
<td>55</td>
<td>O.K.</td>
</tr>
<tr>
<td>Shear</td>
<td>200.147</td>
<td>55</td>
<td>O.K.</td>
</tr>
</tbody>
</table>

Minimum Panel Thickness \(^4\) = 15/32" - inch Sanded Plywood; or 15/32"-inch OSB3 (standard sheathing)

1. See Chord & Drag Analysis for unit shear & tension/compression values
2. Values from APA Load-Span Tables for Structural-Use Panels
3. Values from Gravity Load Calculations (Snow, Live, Dead - BURS sheathing)
4. 11/32" is sufficient for plywood, however, nailing is required. 15/32" minimum panel.

Note: Panel thickness is the minimum structural requirement based on the assumption of plywood subfloor. Engineer is to assess and determine whether existing subfloor is of adequate condition and structure for reuse. Further analysis may be required.
Panel Required Nailing:

- **Existing Rafter Direction**
- **N.T.S.**
- **Based on visual inspection assume unblocked rafters;**
- **Assume Case 3:**
- **Nails used on 2" Nominal Rafters;**
- **Seismic Force Contracts**

\[ u = 283.4 \text{ lb/ft} \] (for Boundary Nails)

<table>
<thead>
<tr>
<th>Nail Type</th>
<th>Minimum Nailer Application</th>
<th>Minimum Nominal Panel Thickness</th>
<th>( V_s (\text{psi}) )</th>
<th>( \phi V_s (\text{psi}) )</th>
</tr>
</thead>
<tbody>
<tr>
<td>6d</td>
<td>1 1/4</td>
<td>3/8</td>
<td>250</td>
<td>200</td>
</tr>
<tr>
<td>8d</td>
<td>1 3/8</td>
<td>15/32</td>
<td>360</td>
<td>288</td>
</tr>
<tr>
<td>10d</td>
<td>1 1/2</td>
<td>15/32</td>
<td>380</td>
<td>304</td>
</tr>
</tbody>
</table>

\[ u = 283.4 \leq \phi V_s = 288 \]

**Boundary Nailing:** 8d Nails @ 6" O.C.

**Nailing Change Location:**

\[ x = \left[ \frac{283.4 - 200}{283.4 - 0} \right] (20ft) = 9\text{ft} \approx 9'4" \]

**Edge & Interior Nailing:** 6d Nails @ 6" O.C.

\[^5\text{Values and case from 2015 SDPWS Table 4.2C}\]
\[^6\phi=0.8 \text{ adjustment factor for LRFD according to 2015 SDPWS Section 4.2.3}\]
\[^7\text{This only applies at locations greater than 9' away from Shear Wall 3 and 5}\]
Longitudinal Analysis

Check on Nailing Penetration:

Boundary Nails: 8d Common: \( L = 2 \frac{3}{8}'' \quad D = 0.131'' \)

\[
\text{Penetration} = L - t_{\text{sheathing}} - 2D \\
= 2 \frac{3}{8}'' - 15/32'' - 2(0.131'') \\
= 1.77'' > Min. Penetration = 1.375'' \quad \text{OK}
\]

Edge/Interior Nails: 6d Common: \( L = 2'' \quad D = 0.113'' \)

\[
\text{Penetration} = L - t_{\text{sheathing}} - 2D \\
= 2'' - 15/32'' - 2(0.113'') \\
= 1.31'' > Min. Penetration = 1.25'' \quad \text{O.K.}
\]

Number of Boundary Nails along Chords:

\[
N = \frac{T/C_{\text{chord}}}{(Z')^q} = \frac{1426016}{99.15\text{in}} = 43 \text{Nails}
\]

\[
Z' = \sqrt{\frac{E' \gamma_{\text{f1}} C_1 + E' \gamma_{\text{f2}} C_2 + E_{y} \gamma_{\text{f3}} C_3 + C_m K_f (X')^{10}}{5}} = 173 \text{ lb/in}
\]

Spacing: \( \frac{600+2485}{63 \text{Nails}} = 30.0 \text{ to } 28 \text{ in O.C.} \)

Values from 2015 SDPWS Table A1.

Values from 2015 NDS C2 from Table 12A, Factors from Chapter 9 and 12.
Longitudinal Analysis

Check on Deflection:

\[ E_m = (700 \cdot f_m') \]

\[ \delta_{dm} = \frac{5V^2L^3}{36EAW} + \frac{0.25V^2L}{10006m} + \frac{E \times AC}{2W} \]

\[ = \frac{5(293.4,134)(60.125\text{ft})^3}{3 \cdot (700 \cdot 000 \cdot 000) \cdot (6 \frac{1}{2} \text{in})(12\text{in})(60.125\text{ft})} + \]

\[ = \frac{0.25(293.4,134)(60.125\text{ft})}{1,000 (5.0 \text{ k.ft/m})} + (6)^13 \]

\[ = 0.970 \text{ in} \]

\[ \Delta_{diaphragm} = \frac{L}{600} = \frac{(60.125\text{ft})}{600} \cdot (12\text{in}) = 1.20 \text{ in} \]

\[ \delta_{dm} \leq \Delta_{diaphragm} \text{ O.K.} \]

11. Equation from ACI 530, \( f_m' = 1000\) psf from Table 5.4.2, assuming worst-case (Type N mortar, Solid Brick)

12. Assumed area, was thickness \((t=12\text{in})\) of masonry wall times depth of sheathing + notches.

13. No chord splice exists for masonry shear wall

14. For \( C_a \)-value from SDPWS Table 4.2C, Sanded Plywood value was used for determining deflection as it provided the greater deflection compared to OSB (Sheathing vs Structural 4).

15. Assumed permissible deflection was equivalent to allowable Masonry deflection \((\delta_{dm})\). See ASCE 7-02 Section 9.5.2.6.7.7: "The deflection, in the plane of the diaphragm... shall not exceed the permissible deflection of..."
Specifications & Minimum Criteria
for Roof Diaphragm

Roof Sheathing:

- Transverse direction permitted 3/8" thickness, however, Longitudinal direction (per SDPWS 2015 Section 4.2.7.1.2) required greater thickness.
- Minimum Sheathing Thickness:
  + 15 38-inch Standard OSB (Does NOT need to Structural)
  + 15 38-inch Sanded Plywood (Does NOT need to be Structural)

- Minimum 4X8 Sheets oriented along Longitudinal Direction (per SDPWS 2015 Section 4.2.7.1.3)

- Engineer shall assess material layout and condition.
- If material is deficient or varies from assumptions, Engineer shall re-evaluate material.
- If material is inadequate; material shall be replaced according to specifications.

Roof Nailing:

- Minimum Nailing Criteria:
  + 8d Common Nails (Boundary)
  + 6d Common Nails (Edge/Interior)

- Boundary Pattern:
  + 8d Common Nails @ 6" O.C. (East-West along Joint)

- Edge Pattern:
  + 9d Common Nails @ 16" O.C. (North-South along Joint Spanning)
  + 3/8" from Edge of Panel (SDPWS 2015 Section 4.2.7.1.3)
  + 6d Common Nails @ 6" O.C. (East-West along Joint)
  + 9-3/4" from Shear Wall; Begin Interior Nailing

- Interior Pattern:
  + 6d Common Nails @ 12" O.C. (SDPWS 2015 Section 4.2.7.1.3)

- Engineer shall assess nail layout and condition and re-evaluate & Variation from assumption charts.
- If inadequate; material shall be replaced according to specifications.
2nd Floor Diaphragm Verification
### 2ND FLOOR DIAPHRAGM

#### Diaphragm Performance

**Observed Deflection**: 0.022 in

**Check for Deflection**: Allowable Deflection 0.60 in > Deflection 0.022 in

**Observed Load**:
- **DL**: 295.0 lb
- **LL**: 1,620 lb
- **DL+LL**: 1,915 lb (Assumed Depth of Sheathing & 12" Joist)

**Load-Span Tables for APA Structural Panels**

**Deflection/Drift**:
- **Δ**: 0.0
- **Φ**: 0.65
- **KF**: 3.32

**Nail Type**: Common

**Penetration**: 1 8/13 in

**Number of Nails/(Board*Support)**
- **At End Bearing Locations**: 6 (ea)
- **At Other Locations**: 23 (Nails)

**Load Type**:
- **Existing Span Rating**: 16 in O.C.
- **Controlling Load**: Seismic (From Lateral Load Calculations)

**Parameters**
- **DL**: 295.0 lb
- **LL**: 1,620 lb
- **DL+LL**: 1,915 lb (Assumed Depth of Sheathing & 12" Joist)

#### Diaphragm Analysis

**Load Type**:
- **DL**: 295.0 lb
- **LL**: 1,620 lb
- **DL+LL**: 1,915 lb (Assumed Depth of Sheathing & 12" Joist)

**Load-Span Tables for APA Structural Panels**

**Deflection/Drift**:
- **Δ**: 0.0
- **Φ**: 0.65
- **KF**: 3.32

**Nail Type**: Common

**Penetration**: 1 8/13 in

**Number of Nails/(Board*Support)**
- **At End Bearing Locations**: 6 (ea)
- **At Other Locations**: 23 (Nails)

**Load Type**:
- **Existing Span Rating**: 16 in O.C.
- **Controlling Load**: Seismic (From Lateral Load Calculations)

**Parameters**
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- **LL**: 1,620 lb
- **DL+LL**: 1,915 lb (Assumed Depth of Sheathing & 12" Joist)

#### Diaphragm Deflection/Drift

**Deflection/Drift**:
- **Δ**: 0.0
- **Φ**: 0.65
- **KF**: 3.32

**Nail Type**: Common

**Penetration**: 1 8/13 in

**Number of Nails/(Board*Support)**
- **At End Bearing Locations**: 6 (ea)
- **At Other Locations**: 23 (Nails)

**Load Type**:
- **Existing Span Rating**: 16 in O.C.
- **Controlling Load**: Seismic (From Lateral Load Calculations)

**Parameters**
- **DL**: 295.0 lb
- **LL**: 1,620 lb
- **DL+LL**: 1,915 lb (Assumed Depth of Sheathing & 12" Joist)

#### Diaphragm Analysis

**Load Type**:
- **DL**: 295.0 lb
- **LL**: 1,620 lb
- **DL+LL**: 1,915 lb (Assumed Depth of Sheathing & 12" Joist)

**Load-Span Tables for APA Structural Panels**

**Deflection/Drift**:
- **Δ**: 0.0
- **Φ**: 0.65
- **KF**: 3.32

**Nail Type**: Common

**Penetration**: 1 8/13 in

**Number of Nails/(Board*Support)**
- **At End Bearing Locations**: 6 (ea)
- **At Other Locations**: 23 (Nails)

**Load Type**:
- **Existing Span Rating**: 16 in O.C.
- **Controlling Load**: Seismic (From Lateral Load Calculations)

**Parameters**
- **DL**: 295.0 lb
- **LL**: 1,620 lb
- **DL+LL**: 1,915 lb (Assumed Depth of Sheathing & 12" Joist)
### 2ND FLOOR DIAPHRAGM

#### Panel Nailing/Span Data Sheet:

<table>
<thead>
<tr>
<th>Span Type</th>
<th>Parameters</th>
<th>Load Type</th>
<th>Panel Type</th>
<th>Nail Type</th>
<th>Nail Size</th>
<th>Nail Quantity</th>
<th>Nail Spacing</th>
<th>Connection Type</th>
</tr>
</thead>
<tbody>
<tr>
<td>24 (in O.C.)</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>15/32 (in)</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>211.4 (plf)</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

#### Parameters:

- **Load Type:**
  - Nails per foot
  - Penetration
- **Panel Type:**
  - Load
  - Panel Type
- **Nail Type:**
  - Penetration
  - Nails per board
  - Nail Spacing
- **Connection Type:**
  - Shear
  - Panel Type
- **Nail Spacing:**
  - Panel Type
  - Load

#### Notes:

- **Load:**
  - Nails per foot
  - Penetration
- **Panel Type:**
  - Load
  - Panel Type
- **Nail Type:**
  - Penetration
  - Nails per board
  - Nail Spacing
- **Connection Type:**
  - Shear
  - Panel Type
- **Nail Spacing:**
  - Panel Type
  - Load

---

**Montana Tech 2 of 3**

VU VILLA STRUCTURAL DESIGN/REHABILITATION

EXISTING MEMBER VERIFICATION

Vu Villa Structural Design and Rehabilitation
Existing Member Analysis

Montana Tech
SPECIFICATIONS & MINIMUM CRITERIA FOR EXISTING 2ND FLOOR DIAPHRAGM

2nd Floor Sheathing: - Transverse Direction Controlled Permissible Sheathing

- Minimum Sheathing Thickness
  - 1x6 Double Diagonal Lumber Planks
  - 19/32-inch Sanded Plywood (Does NOT need to be Structural 1)

- Minimum 4x8 Sheets Oriented along Longitudinal Direction [North-South] (per SDPWS 2015 Section 4.2.7.1.3)

- Engineer shall assess material layout and condition; if material is deficient or varies from assumptions, Engineer shall re-evaluate material for compliance

- If material is inadequate; material shall be replaced according to Specifications.

2nd Floor Nailing: - Transverse Direction Controlled Nailing Requirements

- Blocking is required for 2nd Floor Diaphragm

- Minimum Nailing Criteria:
  - 20d Common Nails for Lumber Plank Sheathing
  - 10d Common Nails for Sanded Plywood Sheathing

- Nailing Pattern for Lumber Sheathing:
  - 3 each - 20d Common Nails at Boundaries (i.e. where each plank ties into shear wall)
  - 2 each - 20d Common Nails at Supports (i.e. where each plank crosses a joist)

- Nailing Pattern for Plywood Sheathing:
  - 10d Common Nails @ 4" O.C. for Boundaries and Panel Edges Parallel to Load
  - 10d Common Nails @ 6" O.C. for Panel Edges Perpendicular to Load
  - 10d Common Nails @ 12" O.C. for Panel Interior Nailing (SDPWS Section 4.2.7.1.3)

- Engineer shall assess existing nail pattern and condition and re-evaluate if variation from assumptions exists.

- If inadequate nailing exists; the minimum additional nailing shall be required to bring the diaphragm into compliance in accordance with the Specifications.
Panel Span Rating & Minimum Thickness:

<table>
<thead>
<tr>
<th>Load Type</th>
<th>Permissible Load Values (psf)</th>
<th>Actual Applied Loads (psf)</th>
</tr>
</thead>
<tbody>
<tr>
<td>LL (1/360)</td>
<td>196, 1620 (Plywood, Wood Planks)</td>
<td>86.7 O.K.</td>
</tr>
<tr>
<td>DL+LL (1/240)</td>
<td>295, 1620</td>
<td>118.9 O.K.</td>
</tr>
<tr>
<td>Bending</td>
<td>199, 4275</td>
<td>118.9 O.K.</td>
</tr>
<tr>
<td>Shear</td>
<td>303, 1620</td>
<td>118.9 O.K.</td>
</tr>
</tbody>
</table>

Minimum Panel Thickness = \( \frac{1}{4} \)"-inch Sanded Plywood or 1"x6" #3 DF(LN) Planks

Panel Required Nailing:

- Existing Joint Direction
- Nailing Pattern
Vu Villa
Structural Design

2nd Floor Diaphragm Design | KBS

*Assume blocked joints (unblocked does not have capacity)*

Case 1: Nails used on 2" Nominal Joints; Seismic Controls

\[ \eta_0 = 634.1 \text{ lb/ft} \]  (For Boundary/Panel Edge Nails)

For Sanded Plywood Sheathing:

<table>
<thead>
<tr>
<th>Nail Type</th>
<th>Minimum Fastening Penetration</th>
<th>Min. Nominal Panel Thickness</th>
<th>( \eta_0 ) (15&quot;)</th>
<th>( \phi \eta_0 ) (5, 6)</th>
</tr>
</thead>
<tbody>
<tr>
<td>6d</td>
<td>1/2&quot;</td>
<td>3/8&quot;</td>
<td>500</td>
<td>400</td>
</tr>
<tr>
<td>8d</td>
<td>1 3/8&quot;</td>
<td>15/32&quot;</td>
<td>720</td>
<td>576</td>
</tr>
<tr>
<td>10d</td>
<td>1 1/2&quot;</td>
<td>19/32&quot;</td>
<td>850</td>
<td>680</td>
</tr>
</tbody>
</table>

\[ \phi \eta_0 = 680 \text{ lb/ft} > \eta_0 = 634.1 \text{ lb/ft} \]  O.K.

Boundary Nailing / Edge Nailing (Parallel to Load): 10d Common Nails @ 4" o.c.

Edge Nailing (at other panel edges): 10d Common Nails @ 6" o.c.

For Wood Plank Sheathing:

<table>
<thead>
<tr>
<th>Sheathing Material</th>
<th>( V_s ) (5)</th>
<th>( \phi \eta_0 ) (6, 7)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Horizontal Lumber</td>
<td>100</td>
<td>80</td>
</tr>
<tr>
<td>Diagonal Lumber</td>
<td>600</td>
<td>490</td>
</tr>
<tr>
<td>Double Diagonal Lumber</td>
<td>1200</td>
<td>960</td>
</tr>
</tbody>
</table>

\[ \phi \eta_0 = 960 \text{ lb/ft} > \eta_0 = 634.1 \text{ lb/ft} \]  O.K.
Sheathing Type: 1"x6" Double Diagonal Lumber

Nailing at Supports: 2ea - 8d Common Nails (8)

Nailing at Boundaries: 3ea - 8d Common Nails (8)

Check on Nailing Penetration

Plywood: 10d Common (9): \( L = 3'' \), \( D = 0.148'' \)

\[
\text{Penetration} = L - 2D - t_{ply} = 3 - 2(0.148) = 1.952''
\]

\[
= 2.11'' > \text{Min. penetration} = 1.5'' \quad \text{O.K.}
\]

Lumber Sheathing: 20d Common (8): \( L = 4'' \), \( D = 0.192'' \)

\[
\text{Penetration} = L - 2D - t_{ply} = 4 - 2(0.192) = 2''
\]

\[
= 1.616'' > \text{Min. penetration} = 1.5'' \quad \text{O.K.}
\]

Check on Nail Withdrawal:

Uplift = 0; Rod uplift forces distributed through Beams to Columns. No uplift occurs on 2nd Floor diaphragm.

\[
W' = W \left( C_{m} + C_{y} C_{m} K_{F} \phi \right)^{\left( g \right)}
\]

\[
W' > \text{Uplift} \quad \text{O.K.}
\]

* Applies for both Plywood & Lumber Sheathing.
**Check on Deflection:**

\[ E_m = 700 \cdot f_m' \]

Lumber Sheathing:

\[ d_{\text{dim}} = \frac{5L^2}{384E_{\text{w}}A_w} + \frac{0.25WL}{1000 C_{\text{w}}} + \frac{E_{\text{w}}A_w}{2L} \]

\[
= \frac{5(634,110 \times 14.125\text{ft}^2)}{8(700 \times 100)(12\times 14.1\text{in})(60.125\text{ft})} + \frac{0.25(634,110 \times (60.125\text{ft})^2)}{1000 (9.5\frac{\text{in.}}{\text{ft}})} + 0.5 \]

\[ = 0.517 \text{ in} \]

Plywood Sheathing:

\[ d_{\text{dim}} = \frac{5(634,110 \times 14.125\text{ft}^2)}{3(700 \times 100)(12\times 12\text{in})(10.125\text{ft})} + \frac{0.25(634,110 \times (10.125\text{ft})^2)}{1000 (8\frac{\text{in.}}{\text{ft}})} \]

\[ = 0.612 \text{ in} \]

\[ d_{\text{allow}} = \frac{L}{600} = \frac{60.125\text{ft}}{600} = 0.100 \text{ in} \]

\[ d_{\text{dim}}, \text{Lumber} \leq d_{\text{allow}} \quad \text{O.K.} \]

\[ d_{\text{dim}}, \text{Plywood} \geq d_{\text{allow}} \quad 	ext{Failed; deflection within 2 percent of criteria. If drift is satisfied will proceed with specified diaphragm criteria.} \]
Check on Drift:

\[ \Delta_{\text{dim, Lumber}} = \frac{d_{\text{dim, Lumber}}}{h_{\text{2nd Floor}}} = \frac{0.517\text{m}}{16\text{ft}} = 0.032\text{ in} \]

\[ \Delta_{\text{dim, Plywood}} = \frac{d_{\text{dim, Plywood}}}{h_{\text{2nd Floor}}} = \frac{0.612\text{m}}{16\text{ft}} = 0.038\text{ in} \]

(8)

\[ \Delta_{\text{allow}} = (0.01)h_{\text{2nd Story}} = (0.01)(108\text{ ft}) = 1.08\text{ in} \]

\[ \Delta_{\text{dim, Lumber}} \ll \Delta_{\text{allow}}, \quad \text{O.K.} \]

\[ \Delta_{\text{dim, Plywood}} \ll \Delta_{\text{allow}}, \quad \text{O.K.} \]
**Longitudinal Analysis**

\[ W_{u} = 1,268.3 \text{ lb/ft} \]

### Panel Span Rating & Minimum Thickness:

<table>
<thead>
<tr>
<th>Load Type</th>
<th>Permissible Load Values</th>
<th>Actual Applied Loads</th>
<th>Actual Applied Loads</th>
</tr>
</thead>
<tbody>
<tr>
<td>LL ((\frac{1}{2})w)</td>
<td>196, 1120 (Plywood, WoodPlank)</td>
<td>86.7</td>
<td>O.K.</td>
</tr>
<tr>
<td>DL + LL ((\frac{1}{2})w)</td>
<td>295, 1620</td>
<td>118.9</td>
<td>O.K.</td>
</tr>
<tr>
<td>Bending</td>
<td>199, 4275</td>
<td>118.9</td>
<td>O.K.</td>
</tr>
<tr>
<td>Shear</td>
<td>303, 1620</td>
<td>118.9</td>
<td>O.K.</td>
</tr>
</tbody>
</table>

Minimum Panel Thickness = \[ \frac{15}{32} \text{-in. Sanded Plywood or} \]
\[ 1'' \times 6'' \#3 DFLWD Planks \]
Panel Required Nailing:

Assume blocked joists (already required from transverse analysis)

Case 3: Nails used on 2" Nominal joists, seismic controls

\[ V_u = 211.4 \text{ lbf} \]

For Sanded Plywood Sheathing: (6" O.C. Spacing for Boundary & Panel Edge)

<table>
<thead>
<tr>
<th>Nail Type</th>
<th>Minimum Fastener</th>
<th>Panel Thickness</th>
<th>Minimum Nominal Panel Thickness</th>
<th>Vs</th>
<th>φV_s</th>
</tr>
</thead>
<tbody>
<tr>
<td>1bd</td>
<td>1 3/4&quot;</td>
<td>5/16</td>
<td>5/16</td>
<td>310</td>
<td>272</td>
</tr>
<tr>
<td>2bd</td>
<td>1 3/8&quot;</td>
<td>1 1/2&quot;</td>
<td>1 1/2&quot;</td>
<td>540</td>
<td>432</td>
</tr>
<tr>
<td>3bd</td>
<td>1 1/2&quot;</td>
<td>1 1/2&quot;</td>
<td>1 1/2&quot;</td>
<td>580</td>
<td>464</td>
</tr>
</tbody>
</table>

*Use 3bd Nails due to Min Panel Thickness

\[ V_u = 211.4 \text{ lbf} \rightarrow \phi V_s = 432 \text{ lbf} \quad O.K. \]

Boundary & All Edge Nailing: 8d Common Nails @ 6" O.C.

For Wood Plank Sheathing:

<table>
<thead>
<tr>
<th>Sheathing Material</th>
<th>Vs</th>
<th>φV_s</th>
</tr>
</thead>
<tbody>
<tr>
<td>Horizontal Lumber</td>
<td>100</td>
<td>80</td>
</tr>
<tr>
<td>Diagonal Lumber</td>
<td>600</td>
<td>480</td>
</tr>
<tr>
<td>Double Diagonal Lumber</td>
<td>1200</td>
<td>960</td>
</tr>
</tbody>
</table>

\[ \phi V_s = 480 > 77 \quad \text{and} \quad V_u = 211.4 \text{ lbf} \]

O.K.
Shelving Type: 1" x 6" Diagonal Lumber Sheathing

Nailing at Supports: 2ea - 8d Common Nails

Nailing at Boundaries: 3ea - 8d Common Nails

NOTE: Transverse Direction Governs for Both Shelving Types.

Check on Penetration:

Plywood:

8d Common \( L = 2.5" \quad D = 0.131" \)

\[
\text{Penetration} = L - 2D - t_{\text{ply}} = 2.5" - 2(0.131" - 15/2) \\
= 1.77" > 1.375" \quad \text{OK.}
\]

Lumber:

10d Common \( L = 3" \quad D = 0.148" \)

\[
\text{Penetration} = L - 2D - t_{\text{ply}} = 3" - 2(0.148") - 1" \\
= 1.7" > 1.375" \quad \text{OK.}
\]

Check on Nail Withdrawal

Uplift = 0; Root uplift transferred through beams to columns. No uplift occurs on 2nd Floor Diaphragm.

\[
W' = W (1 + \left[ C_{\text{m}} + C_{\text{e}} + C_{\text{n}} \right] V_F P_{\text{f}}) \quad (12)
\]

\[
W' > Uplift \quad \text{OK.}
\]

* Applies for both Plywood & Lumber Sheathing.
Boundary Nails story Chord:

Plywood:

\[ z' = z^{(10)} \left[ c_m c_t c_g c_d \right] \phi \frac{f_t}{f_y} \lambda \]

\[ z' = 173 \text{ lb/ft} \quad \text{(see spreadsheet for calcs)} \]

Number of Nails needed \( n \) = \[ \frac{2,38316}{173 \text{ lb/ft}} = 13 \text{ nails} \]

Required Spacing between Nails = \[ \frac{W}{n} = \frac{30 \text{ ft}}{13 \text{ nails}} \cdot \frac{12 \text{ in}}{4} = 24 \text{ in O.C.} \]

Lumber:

\[ z' = 275 \text{ lb/ft} \quad \text{(see spreadsheet for calcs)} \]

\[ n = \frac{2,38316}{275 \text{ lb/ft}} = 8 \text{ nails} \]

Spacing = \[ \frac{W}{n} = \frac{30 \text{ ft}}{8 \text{ nails}} \cdot \frac{12 \text{ in}}{4} = 44 \text{ in O.C.} \]
Check on Deflection Drift:

\[ \delta_{\text{dim}} = \frac{5vL^3}{8EA} + \frac{0.25aL}{1000 G_m} + \frac{\Sigma x A_c}{2W} \]

Plywood:

\[ \delta_{\text{dim}} = \frac{5(211.4 \text{ kips})(60125 \text{ ksi})^3}{8(700 \cdot 1000 \text{ ksi})(2800)(12+2)20^2} + \frac{0.25(211.4 \text{ kips})(60125 \text{ ksi})}{1000 (9.5 \text{ kips/ft})} + (0) \]

\[ = 0.143 \text{ in} \]

Lumber:

\[ \delta_{\text{dim}} = \frac{5(211.4 \text{ kips})(160125 \text{ ksi})^3}{8(700 \cdot 1000 \text{ ksi})(12)(12+2)20^2} + \frac{0.25(211.4 \text{ kips})(160125 \text{ ksi})}{1000 (6 \text{ kips/ft})} + (0) \]

\[ = 0.61 \text{ in} \]

\[ \delta_{\text{allow}} = \frac{L}{600} = \frac{(60 \text{ in})(12 \text{ in})}{600} = 1.20 \text{ in} \]

\[ \delta_{\text{dim, lumber}} \leq \delta_{\text{allow}}, \quad \text{O.K.} \]

\[ \delta_{\text{dim, plywood}} \leq \delta_{\text{allow}}, \quad \text{O.K.} \]

For drift:

\[ \Delta_{\text{lumber}} = \frac{0.61 \text{ in}}{16\text{ ft}} = 0.038 \text{ in}, \quad \text{O.K.} \]

\[ \Delta_{\text{plywood}} = \frac{0.61 \text{ in}}{16\text{ ft}} = 0.038 \text{ in}, \quad \text{O.K.} \]

\[ \Delta_{\text{allow}} = (0.01)(12\text{ in})(\frac{12\text{ in}}{4}) = 1.92 \text{ in} \]
Note: For Specifications & Minimum Criteria for Existing 2nd Floor Diaphragm see Diaphragm Design Spreadsheet. See 2nd Floor Diaphragm Details for minimum permissible nailing & sheathing examples.

Footnotes:

1 See Chord and Drag Force Spreadsheet for values.
2 Values for plywood from APA Load-Span Tables for Structural Use Panels.
3 Plain values based on true 10G DFLW; reference design values.
4 Live load is for 2nd Floor live loading and dead load was assumed to be the 2nd floor beam dead load minus the self-weight of the beam.
5 Values from 2015 SDPWs Table 4.2A.
6 $\phi = 0.95$ adjustment factor for LRFD from 2015 SDPWs Section 4.2.3.
7.8 Values from 2015 SDPWs Table 4.2D.
9 Values from 2015 SDPWs Table A.1.
10 Values from Ch. 11 and 12 of 2015 NDS.
11 Applicable factors according to Table 11.3.1 of the 2015 NDS. Values determined from various sections of 2015 NDS Chs. 11 and 12.
12 $\lambda = 1.0$ from load combination Equations for Lateral forces.
13 Chord area is based on thickness of sheathing and 12" joists and the 12" thickness of Masonry walls.
14 Values from Tables 4.2A & 4.2D.
15 Equation from ACI 530 and assuming the worst-case masonry strength of 10psi/linear inch.
16 No chord space exists for Masonry walls/Chords.
17 Assumed allowable deflection was equivalent to the allowable masonry deflection 4/1000.
18 Allowable drift is from ASCE 7-02 seismic drift table.
VU VILLA STRUCTURAL DESIGN/REHABILITATION
MONTANA TECH

JOIST VERIFICATION
Roof Joist Verification
Joint Properties: 2” x 6” Doug-Fir Larch (R)

<table>
<thead>
<tr>
<th>Property</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>b, breadth</td>
<td>2 (in)</td>
</tr>
<tr>
<td>d, depth</td>
<td>6 (in)</td>
</tr>
<tr>
<td>A, Cross-Sectional Area</td>
<td>12.00 (in²)</td>
</tr>
<tr>
<td>Sx, Section Modulus</td>
<td>12.0</td>
</tr>
<tr>
<td>Sy, Section Modulus</td>
<td>4.0</td>
</tr>
<tr>
<td>�, Moment of Inertia</td>
<td>36 (in²)</td>
</tr>
<tr>
<td>∆, Moment of Inertia</td>
<td>4 (in²)</td>
</tr>
</tbody>
</table>

Joint Type: Reference Design Values (NDS 2015 Table 4A)

<table>
<thead>
<tr>
<th>Load Case</th>
<th>Property</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Mu, Ultimate Moment</td>
<td>2.47 (k-ft)</td>
<td></td>
</tr>
<tr>
<td>Tu, Ultimate Tension</td>
<td>0.75 (k)</td>
<td></td>
</tr>
<tr>
<td>Fb'</td>
<td>4,022 (psi)</td>
<td></td>
</tr>
<tr>
<td>ΦMn &gt; Mu</td>
<td>4.02</td>
<td></td>
</tr>
<tr>
<td>Fb' &gt; Mu</td>
<td>4.02</td>
<td></td>
</tr>
<tr>
<td>E'x</td>
<td>1,900,000 (psi)</td>
<td></td>
</tr>
<tr>
<td>Fby'</td>
<td>1,350 (psi)</td>
<td></td>
</tr>
<tr>
<td>Φ &gt; 0.25 (NDS 2015 Section 3.3.3)</td>
<td>0.80</td>
<td></td>
</tr>
<tr>
<td>ω, Ultimate Load</td>
<td>65.87 (psf)</td>
<td></td>
</tr>
<tr>
<td>ω, Ultimate Load</td>
<td>12.6 (plf)</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Loading Conditions:</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Gravity Loading</td>
<td></td>
<td></td>
</tr>
<tr>
<td>w, Tributary Weight</td>
<td>1.33 (ft)</td>
<td></td>
</tr>
<tr>
<td>l, Tributary Length</td>
<td>15.00 (ft)</td>
<td></td>
</tr>
<tr>
<td>Mu, Ultimate Moment</td>
<td>2.47 (k-ft)</td>
<td></td>
</tr>
<tr>
<td>∆, Permissible Deflection</td>
<td>0.75 (in)</td>
<td></td>
</tr>
<tr>
<td>Check for Deflection:</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Check for Bending (Transverse):</td>
<td></td>
<td></td>
</tr>
<tr>
<td>fn</td>
<td>1,350 (psi)</td>
<td></td>
</tr>
<tr>
<td>c1</td>
<td>1</td>
<td></td>
</tr>
<tr>
<td>NDS 2015 Section 3.3.3</td>
<td></td>
<td></td>
</tr>
<tr>
<td>c2</td>
<td>1</td>
<td></td>
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<tr>
<td>NDS 2015 Section 3.3.3</td>
<td></td>
<td></td>
</tr>
<tr>
<td>c3</td>
<td>1</td>
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<td>NDS 2015 Section 3.3.3</td>
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<td></td>
</tr>
<tr>
<td>NDS 2015 Section 4.3.3</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Kv</td>
<td>2.94 (k-ft)</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Check for Shear (Transverse):</td>
<td></td>
<td></td>
</tr>
<tr>
<td>fn</td>
<td>100 (psi)</td>
<td></td>
</tr>
<tr>
<td>c1</td>
<td>1</td>
<td></td>
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<tr>
<td>NDS 2015 Section 3.3.3</td>
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<td></td>
</tr>
<tr>
<td>c2</td>
<td>1</td>
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<tr>
<td>NDS 2015 Section 3.3.3</td>
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</tr>
<tr>
<td>c3</td>
<td>1</td>
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<tr>
<td>NDS 2015 Section 3.3.3</td>
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<td></td>
</tr>
<tr>
<td>NDS 2015 Section 4.3.3</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Kv</td>
<td>2.88 (k-ft)</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Check for Deflection:</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Check for Tension/Compression:</td>
<td></td>
<td></td>
</tr>
<tr>
<td>fn</td>
<td>1,900,000 (psi)</td>
<td></td>
</tr>
<tr>
<td>E'x</td>
<td>1,900,000 (psi)</td>
<td></td>
</tr>
<tr>
<td>Service Load</td>
<td>32.74 (psf)</td>
<td></td>
</tr>
<tr>
<td>d</td>
<td>10</td>
<td></td>
</tr>
<tr>
<td>f</td>
<td>0.75 (k)</td>
<td></td>
</tr>
<tr>
<td>∆</td>
<td>4.02</td>
<td></td>
</tr>
</tbody>
</table>

Lateral Loading:

<table>
<thead>
<tr>
<th>Load Case</th>
<th>Property</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Mu, Ultimate Moment</td>
<td>0.35 (k-ft)</td>
<td></td>
</tr>
<tr>
<td>Tu, Ultimate Tension</td>
<td>0.09 (k)</td>
<td></td>
</tr>
<tr>
<td>Fb'</td>
<td>4,022 (psi)</td>
<td></td>
</tr>
<tr>
<td>ΦMn &gt; Mu</td>
<td>4.02</td>
<td></td>
</tr>
<tr>
<td>Fb' &gt; Mu</td>
<td>4.02</td>
<td></td>
</tr>
<tr>
<td>E'x</td>
<td>1,900,000 (psi)</td>
<td></td>
</tr>
<tr>
<td>Fby'</td>
<td>1,350 (psi)</td>
<td></td>
</tr>
<tr>
<td>Φ &gt; 0.25 (NDS 2015 Section 3.3.3)</td>
<td>0.80</td>
<td></td>
</tr>
</tbody>
</table>

Check for Shear (Longitudinal): 0.01 (k-ft) [NDS 2015 Section 3.3.3]

Roof Joist's nominal size was equivalent to the actual size, which is common with lumber dimensioning from the 19th century.
Roof Joists under Gravity Loading:

Bending: \[ F'_b = F'_b \left[ C_m C_l C_F C_t C_i C_r \frac{K_F}{\phi} \right] \]

Shear: \[ F'_v = F'_v \left[ C_m C_l C_i K_F \phi \right] \]

Modulus of Elasticity:
\[ E' = E \left[ C_m C_l \right] \]

*For calculations see Roof Joist Verification Spreadsheet.

Roof Joists under Lateral Loading:

(In Longitudinal Direction)

\[ W = \frac{(563.8 \text{ lb})}{[60.125 \text{ ft} / (14 \text{ lb/ft})]} = 12.6 \text{ lb/ft} \]

Bending: \[ F'_b = F'_b \left[ C_m C_l C_F C_t C_i C_r \frac{K_F}{\phi} \right] \]

Shear: \[ F'_v = F'_v \left[ C_m C_l C_i K_F \phi \right] \]

(In Transverse Direction - acting as a collector)

\[ T_u = \frac{(563.8 \text{ lb/ft})(1.334)}{[60.125 \text{ ft}]} = 12.54 \text{ lb/ft} \]

\[ T_u = C_u = \frac{12.54 \text{ lb/ft} \cdot 60.125 \text{ ft}}{10000} = 0.75 \text{k} \]

*Calculations continue on next page.
Tension: \[ F_t^* = F_c \left[ C_m C_e C_f C_i K_F \Phi \right]^{(2)(3)} \]

Compression: \[ F_c^* = F_c \left[ C_m C_e C_f C_i C_p \right]^{(2)(3)} \]

*For calculations, see Roof Joint Verification Spreadsheet.

1. Reference Design Values from 2015 NDS reference design value tables
2. Adjustment factors based on testing and calculations from 2015 NDS
3. Time effect factor based on controlling LRFD load combo equation
2nd Floor Joist Verification
### Joist Type:
- **2" x 12" Doug-Fir Larch (N)**

### Joist Properties:
- **fb, Bending:** 1,150 (psi) (Bending about x-axis)
- **fby, Bending:** 1,150 (psi) (Bending about y-axis)
- **fv, Shear:** 180 (psi) (Shear about x-axis)
- **fvy, Shear:** 180 (psi) (Shear about y-axis)
- **d, Moment of Inertia:** 288 (in²)
- **I, Moment of Inertia:** 8 (in²)
- **b, breadth:** 2 (in) (See Note 1)
- **A, Cross-Sectional Area:** 24.00 (in²)
- **fc, Compression:** 1,800 (psi) (Axial Compressive Strength)
- **ft, Tension:** 750 (psi) (Axial Tensile Strength)
- **Ex, Modulus of Elasticity:** 1,800,000 (psi)
- **Emin,x, Min. Modulus of Elasticity:** 660,000 (psi)
- **Emin,y, Min. Modulus of Elasticity:** 660,000 (psi)

### Section Moduli:
- **Sx, Section Modulus:** 48.0 (in³)
- **Sy, Section Modulus:** 8.0 (in³)

### Gravity Loading:
- **Loading Conditions:**
  - **ω, Ultimate Load:** 194.44 (psf)
  - **w, Tributary Width:** 1.33 (ft)
- **Check for Bending:**
  - **fb**
    - 1,150 (psi) (NDS 2015 Section 4.1.4)
    - 1 (NDS 2015 Section 4.1.4)
  - **Ct**
    - 1 (NDS 2015 Section 2.3.3)
    - 1 (NDS 2015 Section 2.3.3)
  - **Cf**
    - 1 (NDS 2015 Table 4A)
    - 1 (NDS 2015 Table 4A)
  - **Cfu**
    - 1 (NDS 2015 Section 4.3.7)
    - 1.2 (NDS 2015 Table 4A)
  - **Cf'**
    - 1.15 (NDS 2015 Section 4.3.9)
    - 1.15 (NDS 2015 Section 5.3.9)
  - **Kf**
    - 2.54 (NDS 2015 Table 4.3.1)
    - 2.54 (NDS 2015 Table 5.3.1)
  - **Φ**
    - 0.85 (NDS 2015 Table 4.3.1)
    - 0.85 (NDS 2015 Table 5.3.1)
  - **λ**
    - 0.80 (NDS 2015 Appendix N.3.3)
    - 0.80 (NDS 2015 Appendix N.3.3)
  - **cil**
    - 1 (NDS 2015 Section 3.3.3)
    - 1 (NDS 2015 Section 3.3.3)
  - **Fv**
    - 3,426 (psi) (NDS 2015 Table 4.3.1)
    - 3,426 (psi) (NDS 2015 Table 4.3.1)
  - **Fv'**
    - 388.8 (psi) (NDS 2015 Table 4.3.1)
    - 388.8 (psi) (NDS 2015 Table 4.3.1)
  - **Fvu**
    - 4.98 (k) (NDS 2015 Equ. 3.4.2)
    - 4.98 (k) (NDS 2015 Equ. 3.4.2)

### Shear:
- **fv, Shear:** 180 (psi)
- **Fv**
  - 3,426 (psi) (NDS 2015 Table 4.3.1)
  - 3,426 (psi) (NDS 2015 Table 4.3.1)
- **Fvu**
  - 4.98 (k) (NDS 2015 Equ. 3.4.2)
  - 4.98 (k) (NDS 2015 Equ. 3.4.2)

### Deflection:
- **s, Permissible Deflection:** 0.75 (in)

### Check for Deflection:
- **s, Service Load:** 121.45 (psf)
  - (Dead Load & Live Load)
  - (Dead Load & Live Load)
  - (Dead Load & Live Load)
- **δ, Deflection:**
  - 0.36 (in) (NDS Section 3.5.1)
  - 0.36 (in) (NDS Section 3.5.1)

---

1. Roof Joist's nominal size was equivalent to the actual size, which is common with lumber dimensioning from the 19th century.
2nd Floor Joists under Gravity Loading:

Bending: \[ F_b' = F_b \left[ C_m + C_c + C_f C_m C_c \frac{K_F}{K_i} \right]^{(2)} \]

Shear: \[ F_v' = F_v \left[ C_m + C_c + C_f C_m C_c \frac{K_F}{K_i} \right]^{(2)} \]

Modulus of Elasticity: \[ E' = E \left[ C_m + C_c \right]^{(2)} \]

*For canvas see 2nd Floor Joist verification Spreadsheet

2nd Floor Joists under Lateral Loading
(In Longitudinal Direction)

Bending: \[ F_b' = F_b \left[ C_m + C_c + C_f C_m C_c \frac{K_F}{K_i} \right]^{(2)} \]

Shear: \[ F_v' = F_v \left[ C_m + C_c + C_f C_m C_c \frac{K_F}{K_i} \right]^{(2)} \]

\[ w = \left( 1.2 \text{ kips} \cdot 13.3 \text{ fps}^2 \right) \left( 60.125 \text{ ft} \right)^2 = 28.1 \text{ lb/ft} \]

(In Transverse Direction - acting as a collector)

\[ h = \left( 12.68 \text{ in} \cdot 1.33 \text{ ft} \right) \left( 60.125 \text{ ft} \right)^2 = 28.1 \text{ in} \]

\[ T_n = C_n = \left( 28.1 \text{ in} \cdot 1.33 \text{ ft} \right) \left( 60.125 \text{ ft} \cdot 1.33 \text{ ft} \right) = 1.69 K \]

*Calculations continue on next page
Tension: \( F_t' = F_t \left[ C_m + C_f + C_i \cdot K_F \cdot (\phi) \right]^{\frac{1}{(3)}} \)

Compression: \( F_c' = F_c \left[ C_m \cdot C_f \cdot C_i \cdot K_f \cdot (\phi) \right]^{\frac{1}{(3)}} \)

*For calculations see 2nd Floor Joint Verification Spreadsheet*

1 Reference Design Values from 2015 NDS reference design value table
2 Adjustment factors based on criteria and calculations from 2015 NDS
3 Time-effect factor based on controlling LRFD load combo equation; see Appendix N of the 2015 NDS.
VU VILLA STRUCTURAL DESIGN/REHABILITATION
MONTANA TECH

SHEAR WALL VERIFICATION
Shear Wall 1 Verification
**SHEAR WALL 1**

---

**Montana Tech**

**Vu Villa Structural Design and Rehabilitation**

---

**Existing Member Verification**

---

### Check for Axial Strength

<table>
<thead>
<tr>
<th>Component</th>
<th>Formula</th>
<th>Value</th>
<th>Reference</th>
</tr>
</thead>
<tbody>
<tr>
<td>( \phi P_n &gt; R_4 )</td>
<td> </td>
<td> </td>
<td>(NDS 2015 Table 4.3.1)</td>
</tr>
<tr>
<td>( \phi P_n &gt; R_3 )</td>
<td> </td>
<td> </td>
<td>(ACI 530 Equ. 3-11 or 3-12)</td>
</tr>
<tr>
<td>( \phi P_n &gt; R_2 )</td>
<td> </td>
<td> </td>
<td>(ACI 530 Section 3.1.4.3)</td>
</tr>
<tr>
<td>( \phi P_n &gt; R_1 )</td>
<td> </td>
<td> </td>
<td>(ACI 530 Section 3.1.4.3)</td>
</tr>
</tbody>
</table>

---

### Check for Eccentric Moment

<table>
<thead>
<tr>
<th>Component</th>
<th>Formula</th>
<th>Value</th>
<th>Reference</th>
</tr>
</thead>
<tbody>
<tr>
<td>( \phi M_n &gt; M_u )</td>
<td> </td>
<td> </td>
<td>(ACI 530 Section 3.1.4.3)</td>
</tr>
</tbody>
</table>

---

### Shear Wall Designation

- **Wall Type:** Shear Wall
- **Length:** 30 (ft)
- **Height:** 12 (ft)
- **Thickness:** 12 (in)
- **Section Modulus:** 8,640
- **Moment of Inertia:** 46,656,000
- **Radius of Gyration:** 104 (in)
- **Cross-Sectional Area:** 4,320 (in²)
- **Strength:**
  - **Ultimate Load:** 61 (k)
  - **Interior Loads:** 29 (k)
  - **Self-weight:** 32 (k)

---

### Member Check

<table>
<thead>
<tr>
<th>Component</th>
<th>Formula</th>
<th>Value</th>
<th>Reference</th>
</tr>
</thead>
<tbody>
<tr>
<td>( \phi M_n &gt; M_e )</td>
<td> </td>
<td> </td>
<td>(See Hand Calculations)</td>
</tr>
<tr>
<td>( \phi V_n &gt; V_u )</td>
<td> </td>
<td> </td>
<td>(See Hand Calculations)</td>
</tr>
</tbody>
</table>

---

### Wall Stability

- **Ultimate Vertical Load on Wall:** 61 (k)
- **Eccentricity of Load:** 4 (in)
- **Net Cross-Sectional Area:** 4,320 (in²)
- **Section Modulus:** 8,640
- **Moment of Inertia:** 46,656,000
- **Radius of Gyration:** 104 (in)

---

### Wall Sections

- **West Bay:** Shear Wall
- **East Bay:** 1st Floor Wood-Framing Wall

---

### Check for Compression of Wall

- **Compressive Strength of Masonry:** 1,200
- **Compressive Strength of Wood:** 1,300

---

### Check for Bending/Shear

- **Wood Beam Shear Strength:** 170
- **Wood Beam Modulus of Rupture:** 1,300
- **Wood Beam Head Breadth:** 12
- **Wood Beam Depth:** 6
- **Header Cross-Sectional Area:** 144 (in²)
- **Header Section Modulus:** 288 (in³)

---

### Wood & Masonry

- **Shear Wall Type:** Wood & Masonry
- **Wall Type:** Shear Wall
- **Length:** 30 (ft)
- **Height:** 12 (ft)
- **Thickness:** 12 (in)

---

### Loading Conditions

- **Assumed Standard Brick Size & Weight:**
  - **Compression:** 1,200
  - **Modulus of Rupture:** 1,300
  - **Number of Supports:** 23 (ea)
  - **Depth:** 12 (in)
  - **Breadth:** 12 (in)
  - **Slenderness Ratio:** 1 < 99
  - **Self-weight of Wall:** 32 (k)

---

### Check for Axial Strength

- **Ultimate Axial Strength:** 553 (k)
- **Nominal Axial Strength:** 144 (k)

---

### Check for Eccentric Moment

- **Ultimate Bending Strength:** 54
- **Nominal Bending Strength:** 54
- **Moment Magnification Factor:** 1
- **Reduction Factor:** 1

---

### Summary

- **Eccentricity of Load:** 4 (in)
- **Eccentricity of PInt:** 4 (in)
- **Wall Safety Factor:**
  - **Availabe:** O.K.
  - **Available:** O.K.

---

### Appendix

- **ACI 530 Section 3.1.4.3**
- **NDS 2015 Table 4.3.1**
- **See Hand Calculations**

---

### References

- **ACI 530 Section 3.2.2.4.3**
- **NDS 2015 Table 4D**
- **See Hand Calculations**

---

**Vu Villa Structural Design and Rehabilitation**

**Existing Member Verification**

**Montana Tech**

1 of 2
### 2nd Floor Masonry Wall - West Bay

#### Loading Conditions:
- **Mu, Ultimate Seismic Lateral Bending Moment**: 1,971 (lb-ft)  
  (See Hand Calculations)
- **Mu, Ultimate Wind Lateral Bending Moment**: 354 (lb-ft/ft)  
  (See Hand Calculations)
- **Mu, Ultimate Wind Lateral Bending Moment**: 398 (lb-ft/ft)  
  (See Hand Calculations)

#### Check for Bending:
- **fr, Masonry Modulus of Rupture**: 75 (psi)  
  (ACI 530 Table 3.1.8.2)
- **Ф, Strength-Reduction Factor**: 0.6  
  (ACI 530 Section 3.1.4.3)
- **ẟФMn**: 1,080 (lb-ft/ft)
- **ФMn > Mu (Seismic)**: Failed
- **ФMn > Mu (Wind)**: O.K.

### 1st Floor Wood Columns

#### Loading Conditions:
- **Mu, Ultimate Seismic Lateral Bending Moment**: 1,752 (lb-ft)  
  (See Hand Calculations)
- **Mu, Ultimate Seismic Lateral Bending Moment**: 4.67 (k-ft)  
  (See Hand Calculations)
- **Mu, Ultimate Wind Lateral Bending Moment**: 398 (lb-ft)  
  (See Hand Calculations)
- **Mu, Ultimate Wind Lateral Bending Moment**: 0.94 (k-ft)  
  (See Hand Calculations)

#### Check for Bending:
- **fb, Bending Strength of DFL (Ref. Design Value)**: 725 (psi)  
  (NDS 2015 Table 4D)
- **fb, Bending Strength of DFL (Ref. Design Value)**: 900 (psi)  
  (NDS 2015 Table 4A)
- **Ф, Strength-Reduction Factor**: 0.85  
  (NDS 2015 Table 4.3.1)
- **KF**: 2.54  
  (NDS 2015 Table 4.3.1)
- **ФM**: 4,696 (lb-ft)
- **ФM > Mu (Seismic)**: O.K.
- **ФM > Mu (Wind)**: O.K.

#### In-Plane Loading

- **Vu, Ultimate Lateral Shear**: 40.4 (k)  
  (See Hand Calculations)
- **Vu, Ultimate Lateral Shear**: 92.0 (k)  
  (See Hand Calculations)
- **νu, Ultimate Lateral Unit Shear**: 3,067 (lb/ft)  
  (See Hand Calculations)
- **Vn, Nominal Shear Strength**: 328 (k)  
  (ACI 530 Section 3.2.4)
- **Фνn, Columns, Column's Nominal Shear Strength**: 2,722 (lb/ft)  
  (See Hand Calculations; Assumes 8x8 & 2- 6x6 Columns)
- **ФVn**: 263 (k)
- **ФVn > Vu**: O.K.
- **ФVn > Vn**: O.K.

#### Check for Overturning:
- **Mo, Overturning Moment**: 485 (k-ft)  
  (See Hand Calculations)
- **Mr, Resisting Moment**: 1,911 (k-ft)  
  (See Hand Calculations)
- **Mr > Mo**: O.K.

#### 1st Floor Wood-Beam Header

- **Pu, Ultimate Lateral Axial Load on Header**: 92.0 (k)
- **b, Breadth of Header**: 12 (in)
- **d, Depth of Header**: 12 (in)
- **fc, Compression Strength of DFL (Ref. Design Value)**: 1,200 (psi)  
  (ACI 530 Section 3.2.4)
- **Ф, Strength-Reduction Factor**: 0.90  
  (NDS 2015 Table 4.3.1)
- **KF**: 2.40  
  (NDS 2015 Table 4.3.1)
- **f'c, Compression Strength of DFL**: 2,592
- **ФPn, Axial Header Strength**: 373 (k)
- **ФPn > Pu**: O.K.

---

Note: Ultimate information is for wind loading only. Wall will not support the ultimate seismic load. It is indicated that the wall will support approximately 40% of the ultimate seismic load.

It is well documented that Unreinforced Masonry (URM) structures do NOT perform well under loading from earthquakes. Though beyond the scope of this project, seismic retrofitting is highly recommended for any actual work performed on the Vu Villa. Appendix A1 of the 2012 International Existing Building Code (IEBC) lays out the expectations and requirements of seismic retrofitting of URM Walls.

For this assessment, as seismic loading of this scale has a low probability and strong winds are a more likely threat to the building, the bending strength of the shear walls was assessed for wind loading.

---

Website values based on dimensions: Guide from The Belden Brick Company Website

---

Vu Villa Structural Design and Rehabilitation
Existing Member Verification
Montana Tech
Vu Villa
Structural Design

Plan View N.T.S.

Sheer Wall 1
Sheer Wall 2
Sheer Wall 3
Sheer Wall 5
Sheer Wall 4
### Loading Calculations:

\[ W_u = 1061 \text{kip} \]

### Assumptions:
- Each span behaves as a Simply Supported Beam (Conservative Approach)
- Cumulative effect at each reaction
- The beam has the same stiffness (EI) all the way across.

### Results:
- \( R_1 = 2.37k \)
- \( R_2 = 23.7 + 4 = 6.37k \)
- \( R_3 = 4 + 18.2 = 22.2k \)
- \( R_4 = 18.2 + 6 = 24.2k \)
- \( R_5 = 6k \)
Loading Conditions: (continued)

East Bay:

![Diagram of shear wall with labels: P_{roof}, P_{wall}, P_{2nd Floor}, and P_{wood}]

\[ P_{\text{wall}} = P_{\text{roof}} + P_{\text{wall}} = 9.74 \text{ k} + \left( \frac{4.5 \text{ ksf}}{\text{brf}} \right) \times 3 \text{ brf} \times \left( \frac{12 \text{ ft}}{\text{brf}} \right) \times \left( \frac{13}{3} \right) \]
\[ = 41.6 \text{ k} \]

Loading on Wall:
\[ P_{\text{wall}} = P_{\text{wall}} + P_{\text{2nd Floor}} + P_{\text{wood}} = 41.6 + 28.7 \text{ k} + \left( 1.4 \text{ ksf} + 3.0 \text{ ksf} \right) \times \left( 3 \text{ brf} \times \frac{12 \text{ ft}}{\text{brf}} \right) \times 1000 \text{ ksf} \]
\[ = 71.9 \text{ k} \]
Check for Vertical Loading:

West Bay: (Masonry Wall Check)

\[ P_n = 28.7k + (1061.1 \frac{k}{l_e})(30x4) = 60.5k \]

\[ M_e = (28.7k)(4m)(\frac{6k}{12k}) = 9.6k-ft \]

\[ \phi P_n = 0.65(71.5k)(1-\left(\frac{7}{40}\right)^2) = 9.6k \]

\[ \phi P_n > P_n \quad \text{O.K.} \]

\[ M_n = Sy \cdot ft \]

\[ \phi M_n > M_e \quad \text{O.K.} \]

(Wood Beam Header Check)

\[ \phi V_n = 367k > V_u \quad \text{O.K.} \]

\[ \phi M_n = 67k > V_u \quad \text{O.K.} \]

East Bay: (Masonry Wall Check)

*Same as West Bay

\[ P_{\text{2nd floor}} = P_{\text{wall}} + P_{\text{int}} \]

\[ P_n = 71.9k \quad \text{(See Sheet 2)} \]

\[ \phi P_n = 71.5k > P_n \quad \text{O.K.} \]
Check for Lateral Loading: (Out of Plane)

West Bay:

\[ W_u (\text{Seismic}) = 109.5 \text{ psf} \]
\[ W_u (\text{Wind}) = 22.1 \text{ psf} \]

Check for Masonry Wall Bending:
\[ M_u (\text{Seismic}) = \frac{(109.5 \times 1.20)(24)^2}{8} = 1,971 \text{ lb-ft} \]
\[ M_u (\text{Wind}) = \frac{(22.1 \times 1.20)(24)^2}{8} = 398 \text{ lb-ft} \]

\[ \phi M_n = 0.6 \left[ \frac{(12.4)^2(12.4)}{6} \right] (179) \]
\[ = 1,080 \text{ lb-ft} \]

\[ \phi M_n < M_u (\text{Seismic}) \] Failed
\[ \phi M_n > M_u (\text{Wind}) \] O.K.

(See Note on Masonry Wall 1 Verification Spreadsheet)

East Bay:

\[ W_u (\text{Seismic}) = 109.5 \text{ psf} \]
\[ W_u (\text{Wind}) = 22.1 \text{ psf} \]

Check for Masonry Wall Bending:
\[ M_u (\text{Seismic}) = 1,971 \text{ lb-ft} \]
\[ M_u (\text{Wind}) = 398 \text{ lb-ft} \]

\[ \phi M_n = 1,080 \text{ lb-ft} \]

\[ \phi M_n < M_u (\text{Seismic}) \] Failed
\[ \phi M_n > M_u (\text{Wind}) \] O.K.

(See Note on Masonry Wall 1 Verification Spreadsheet)
Check for Latera Loading: (Cont'd - Out-of-Plane)

West Bay:

Check for Column/Wall Bearing:

\[
\begin{align*}
\text{West Bay:} \\
\phi M_n &= \phi_D \cdot S \\
&= \left[ (1.565 \, \text{psi}) \left( \frac{6 \times (60)^2}{2} \right) \right] \\
&= 352.16 \, \text{lb-ft} \\
\phi M_n &> Mu (Suecinc) \quad \text{O.K.} \\
\phi M_n &> Mu (Ward) \quad \text{O.K.}
\end{align*}
\]

\[
\begin{align*}
\text{East Bay:} \\
\phi M_n &= \left[ \frac{(1443 \, \text{psi}) \left( 27.3 \times (60)^2 \right)}{6} \right] \\
&= 11.7 \, \text{K-ft} \\
\phi M_n &> Mu (Suecinc) \quad \text{O.K.} \\
\phi M_n &> Mu (Ward) \quad \text{O.K.}
\end{align*}
\]
Check for Lateral Loading: (In-Plane)

40.4k

51.6k

Check 2nd Floor Masonry Shear & overturning:

\[ V_u = 40.4k \]

\[ V_n = \min \left\{ 3.8 \cdot \frac{A_n}{100}, \frac{300}{A_n} \right\} = 328K \]

\[ \phi V_n = (0.8 \cdot 328k) = 263k \text{ O.K.} \]

\[ M_o = (40.4k)(12 \text{ ft}) = 485 \text{ k-ft} \]

\[ M_r = (63.7k)(30 \text{ ft}) = 1,910 \text{ k-ft} \]

\[ M_r > M_o \text{ O.K.} \]
Check 1st Floor Wood Framed Wall Shear & Torsion:

\[ V_n = 40.4K + 51.6K = 92K \]

\[ \phi V_n = f_v \cdot A_n \]

\[ = \left[ 140 \text{psi} \cdot \left( 2.58 \times 0.75 \times 2'' 	imes 6'' \right) \right] \]

\[ \quad \div 16 \text{in} \left( \frac{1}{12} \right) \]

\[ = 7,761.6 \text{ kips} \]

\[ \phi V_n = 7,761.6 \text{ kips} \]

\[ = 9,860.8 \text{ kips} > V_n \quad \text{O.K.} \]

\[ M_o = (40.4K)(28.8) + (51.6K)(16.75) = 19,568 \text{ kips-ft} \]

\[ M_r = \left[ 45.5 \left( \frac{1}{30} \right)^{1/4} \right] \\
\left[ \left( 6.36 \right)^{1/4} \times 0.93 \times 30 \times \frac{30}{2} \times \frac{30}{2} \right] + \left[ 1.5 \left( \frac{30}{2} \right) \left( \frac{30}{2} \right) \left( \frac{30}{2} \right) \right] \]

\[ = 21,061 \text{ kips-ft} \]

\[ M_r > M_o \quad \text{O.K.} \]
Check Wood Beam Header for Compression:

\[ P_u = 40.4k + 51.6k = 92k \]

\[ \phi_{c} = 700\text{ psi} \quad \text{(2015 NDS Table 4D)} \]

\[ f'_{c} = (700\text{ psi})(2.4)(0.90) = 1,512\text{ psi} \]

\[ \phi_{p_y} = (1.512\text{ psi})(12\times12) = 218k \]

\[ \phi_{p_y} > P_u \quad \text{OK} \]
Shear Wall 2 Verification
<table>
<thead>
<tr>
<th>Wall Criteria</th>
<th>Shear Wall 2</th>
<th>Masonry</th>
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<tbody>
<tr>
<td>f'm, Masonry Compressive Strength</td>
<td>100 (psi)</td>
<td>(ACI 530 Table 5.4.2)</td>
</tr>
<tr>
<td>f'r, Masonry Modulus of Rupture</td>
<td>75 (psi)</td>
<td>(ACI 530 Table 3.1.8.2)</td>
</tr>
<tr>
<td>t, Wall Thickness</td>
<td>12 (in)</td>
<td></td>
</tr>
<tr>
<td>h, Wall Height</td>
<td>20 (ft)</td>
<td></td>
</tr>
<tr>
<td>L, Wall Length</td>
<td>90 (ft)</td>
<td></td>
</tr>
<tr>
<td>Lw, Effective Length of Wall</td>
<td>14 (ft)</td>
<td>(ACI 530 Section 1.9.7)</td>
</tr>
<tr>
<td>Aw, Wall Net Cross-Sectional Area</td>
<td>2,016 (in²)</td>
<td>(For Vertical Wall Cross Section)</td>
</tr>
<tr>
<td>Sw, Wall Section Modulus</td>
<td>225,792 (in³)</td>
<td>(For Vertical Wall Cross Section)</td>
</tr>
<tr>
<td>Jaw, Wall Moment of Inertia</td>
<td>37,933,056 (in⁴)</td>
<td>(For Vertical Wall Cross Section)</td>
</tr>
<tr>
<td>An,z, Wall Net Cross-Sectional Area</td>
<td>8,640 (in²)</td>
<td>(For Lateral Wall Cross Section)</td>
</tr>
<tr>
<td>Iz, Wall Moment of Inertia</td>
<td>17,280 (in³)</td>
<td>(For Lateral Wall Cross Section)</td>
</tr>
<tr>
<td>Sy, Wall Section Modulus</td>
<td>17,280 (in³)</td>
<td>(For Lateral Wall Cross Section)</td>
</tr>
<tr>
<td>Iy, Wall Moment of Inertia</td>
<td>103,680 (in⁴/ft)</td>
<td>(For Lateral Wall Cross Section)</td>
</tr>
<tr>
<td>r, Wall Radius of Gyration</td>
<td>137 (in)</td>
<td></td>
</tr>
</tbody>
</table>

**Check Wall for Vertical Load Resistance:**

- **Check for Axial Load Resistance:**
  - Pn, Nominal Axial Strength: 128 (k)
  - φp, Strength-Reduction Factor: 0.6
  - $\Phi P_n = P_n$

- **Check for Eccentric Moment:**
  - Mn, Nominal Bending Strength: 128 (k-ft)
  - φm, Strength-Reduction Factor: 0.6
  - $\Phi M_n = M_n$

**Check Wall for Lateral Load Resistance:**

- **Check for Axial Load Resistance:**
  - Pn, Nominal Axial Strength: 128 (k)
  - φp, Strength-Reduction Factor: 0.6
  - $\Phi P_n = P_n$

- **Check for Eccentric Moment:**
  - Mn, Nominal Bending Strength: 128 (k-ft)
  - φm, Strength-Reduction Factor: 0.6
  - $\Phi M_n = M_n$

**Check for Shear Strength:**

- Vn, Nominal Shear Strength: 128 (k)
- φ, Strength-Reduction Factor: 0.8
- $\Phi V_n = V_n$

**Check for Overturning:**

- Mr, Resisting Moment: 4,457 (k-ft)
- $\Phi M_r > M_r$

- Mr > Mo
  - (No hold-down restraint required)

**Load Conditions:**

- **Vertical Loading:**
  - Pn, Ultimate Vertical Loads: 29 (k)
  - E, Eccentricity of Pn: 4 (in)
  - Mn, Eccentric Moment: 10 (k-ft)
  - W, Self-Weight of Wall: 37 (k)
  - (Assumed Standard Brick Size & Weight)

- **Lateral Loading:**
  - W, Ultimate Lateral Loading: 22 (k)
  - M, Ultimate Lateral Bending Moment: 42 (k-ft)
  - V, Ultimate Lateral Shear: 42 (k)

**Check Wall for Lateral Load Resistance:**

- $\Phi M_n > M_n$

**Note:**

- Ultimate Moment is for wind loading only. Wall will not support the ultimate seismic load. It is estimated that the wall will support approximately 30% of the ultimate seismic load.

- Unreinforced Masonry (URM) Structures do NOT perform well under loading from earthquakes. Though beyond the scope of this project, seismic retrofitting is highly recommended for any actual work performed on the Vu Villa. Appendix A1 of the 2012 International Existing Building Code (IEBC) lays out the expectations and requirements of seismic retrofitting of URM Walls.

- For this assessment, as seismic loading of this scale has a low probability and strong winds are a more likely threat to the building, the bending strength of the shear walls was assessed for wind loading.
Check for Vertical Local Resistance:

\[ P_n = P_{int} + P_{wall} \]

\[ P_{int} = \left( 19.4 \, \text{psi} \right) \left( 15 \, \text{ft} \right) \left( 19.7 \, \text{ft/k} \right) = 29 \, \text{k} \]

\[ \frac{2 \left( 1000 \, \text{lbf/k} \right)}{2 \left( 1000 \, \text{lbf/k} \right)} \]

\[ P_{wall} = \left( 4.5 \, \text{lbf/ft} \right) \left( 3 \, \text{layers of brick} \right) \left( 6.5 \, \text{lbft/ft} \right) \left( 23.5 \, \text{ft} \right) \left( 15 \, \text{ft/k} \right) \]

\[ = 37 \, \text{k} \]

\[ P_n = 66 \, \text{k} \]

\[ \frac{1}{4} < 0.9 \quad \text{so} \quad P_n = 0.8 \left( 0.8 \left( 0.8 \right) \left( 1.0 \right) \right) \]

Note:

\[ An = t \cdot Le; \text{ where} \]

\[ Le - \text{effective wall length, see Section 1.9.7 of ACI 530 for logic.} \]

\[ \phi P_n > P_n \quad \text{O.K.} \]

(Check for Eccentric Moment)

\[ M_e = \left( \frac{4 \text{in.}}{12 \text{in.}} \right) \left( 29 \, \text{k} \right) = 9.67 \, \text{k-ft} \]

\[ \phi M_n > M_e \quad \text{O.K.} \]
Check for Lateral Load Resistance:

\[ W_u (\text{Seismic}) = 109.5 \text{ kips} \quad M_u = \frac{109.5 \times 12}{8} = 1692 \text{ kips-ft} \]

\[ W_u (\text{Luw}) = 22.1 \text{ kips} \quad M_u = \frac{22.1 \times 12}{8} = 331.5 \text{ kips-ft} \]

\[ \phi M_u = (0.6) \left[ \frac{(12 \times 10^3)(12)^2}{6} \right] (7500) = 1080 \text{ kips-ft} \]

\[ \phi M_u < M_u (\text{Seismic}) \quad \text{Failed} \]

\[ \phi M_u > M_u (\text{Luw}) \quad \text{O.K.} \]

Check for In-Plane Shear & Bending:

\[ V_u = 40.4 (\text{kips}) + 51.6 (\text{kips}) = 92.0 \text{ kips} \]

\[ \phi V_u = 0.8 (92.0 \text{ kips}) = 73.6 \text{ kips} > V_u \quad \text{O.K.} \]

\[ M_o = (40.4 \times 12) + (51.6 \times 10) = 1310.4 \text{ kips-ft} \]

\[ M_r = (444 \times 30) = 4457 \text{ kips-ft} > M_o \quad \text{O.K.} \]

\[ M_r = 4457 \text{ kips-ft} > M_o \quad \text{No weld design required.} \]
Shear Wall 3 & 4 Verification
Shear Wall Designation: SHEAR 3 AND 4

**Wall Criteria:**
- Shear Wall Type: Masonry

**Shear Wall Designation:**
- Wall Criteria: Loading Conditions:
  - Masonry Compressive Strength: 100 (psi) (ACI 530 Table 5.4.2)
  - Masonry Modulus of Rupture: 75 (psi) (ACI 530 Table 3.1.8.2)
  - Ultimate Interior Loads: 86 (k) (Loads from Interior Members)
  - Ultimate Vertical Load on Wall: 235 (k)
  - Ultimate Lateral Loading: 22 (lb/ft²) (See Hand Calculations)

**Check Wall for Vertical Load Resistance:**
- Masonry Axial Strength: 552 (k) (ACI 530 Eqs. 3-11 or 3-12)
- Strength-Reduction Factor: 0.6 (ACI 530 Section 3.1.4.3)
  - $\phi P_v = P_v$ O.K.

**Check for Bending Strength:**
- Nominal Lateral Bending Strength: 108 (k-ft) (Sy x fr)
- Strength-Reduction Factor: 0.6 (ACI 530 Section 3.1.4.3)
  - $\phi V_n > V_u$ O.K.

**Check for Overturning:**
- Overturning Moment: 1955 (k-ft) (See Hand Calculations)
  - $M_o > M_r$ O.K. (No hold-down restraint required)

**Note:** Ultimate Moment is for wind loading only. Wall will not support the ultimate seismic load. It is estimated that the wall will support approximately 30% of the ultimate seismic load.

It is well documented that Unreinforced Masonry (URM) Structures do NOT perform well under loading from earthquakes. Though beyond the scope of this project, seismic retrofitting is highly recommended for any actual work performed on the Vu Villa. Appendix A1 of the 2012 International Existing Building Code (IEBC) lays out the expectations and requirements of seismic retrofitting of URM Walls.

For this assessment, as seismic loading of this scale has a low probability and strong winds are a more likely threat to the building, the bending strength of the shear walls was assessed for wind loading.

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**Montana Tech 1 of 1**

Vu Villa Structural Design and Rehabilitation
Existing Member Verification
Check for Vertical Load Resistance:

\[ P_n = P_{\text{wall}} + P_{\text{int}} \]

2nd Floor Beam-loading

\[ P_{\text{int}} = \left( \frac{194.44 \times 10^6 \text{ kips}}{12000 \times 12 \text{ in}} \right) - (1.2 \times 26 \times 10^5) \times (15 \times \text{60.125 ft}) = 172.5 \text{ k} \]

\[ P_{\text{wall}} = (4.5 \times 12 \times 10^3 \text{ lb/ft}) \times \left( \frac{6.55 \times 12 \times 10^3 \text{ lb/ft}}{1000} \right) \times (28 + \text{60.125 ft}) / (1000 \%) = 148.9 \text{ k} \]

\[ P_n = 321 \text{ k} \]

Note: Drawings provide lateral boundary/support for buckling in east-west direction. Only need to address buckling in North-South.

\[ \frac{b}{h} < 99 \text{ so;} \]

\[ P_n = 0.8 \times 0.8 \times A_n \times f_n \left( 1 - \left( \frac{b}{h} \right)^2 \right) \]

\[ \varphi = 0.6 \text{ (ACI 530 Section 3.1.4.3)} \]

\[ \varphi P_n > P_n \text{ O.K.} \]

(Check for Eccentric Moment)

\[ M_e \left( \frac{h_0}{12} \right) \times 321 \text{ k} = 58 \text{ k-ft} \]

\[ A_n = S \cdot f_n \]

\[ \varphi = 0.6 \]

\[ \varphi M_n > M_e \text{ O.K.} \]
Check for Lateral Load Resistance:

\[ Wu = \left( \frac{193.89 \text{ kips}}{2600 \text{ ft}^2} \right)^2 = 109.5 \text{ kips/ft}^2 \]

\[ Mu = \frac{Wu \times h_3^2}{8} = \frac{(109.5 \times 10^4 \text{ kips})(16 \text{ ft})^2}{8} \]

\[ = 3,504 \text{ kips/ft} \]

\[ Mn = S \times fr \]

\[ = \left( \frac{12 \text{ kips}}{3 \text{ ft}} \right) \left( 75 \text{ psi} \right) \left( \frac{14}{12 \text{ ft}} \right) \]

\[ = 1,800 \text{ lb-ft/ft} \]

\[ \phi Mn < Mu \quad \text{Failed for Seismic} \]

Note: It is well documented that Unreinforced Masonry (URM) structures do not perform well under loading from earthquakes. Though beyond the scope of this project, seismic retrofitting is highly recommended as part of any actual work performed on the Vu Villa.

As seismic loading of this scale has a low probability and strong winds are a greater threat to the building, the bending strength of the shear walls was assessed for wind loading as well. See next page for analysis.
(Check for Bonding Strength - Wind)

\[ W_n = 22.1 \text{ psf} \]

\[ M_n = \frac{(22.1 \text{ psf})(160 \text{ ft})^2}{8} = 707.2 \text{ ft-lb} \]

\[ M_n = 1900 \text{ ft-lb} \]

\[ \varnothing M_n > M_n \quad \text{O.K.} \]

Note: Bonding along the length of the wall was not addressed as the wall is supported the entire length by the foundation & roof/floor diaphragms.

(Check for Shear Strength)

\[ V_{\text{net}} = 40.4 \text{ K} \]

\[ V_{\text{end}} = 51.6 \text{ K} \]

\[ V_n = V_{\text{net}} + V_{\text{end}} = 91.9 \text{ K} \]

\[ C = 3.8 A_n \sqrt{\frac{\text{Rm}}{A_n}} = 3.8 \left[ \frac{12 \text{ in} \cdot (160 \text{ ft} \times 12 \text{ in})}{1 \text{ in} \cdot (60 \text{ ft} \times 12 \text{ in})} \right] = 328.3 \text{ K} \]

\[ V_n = \text{Minimum} \left( \frac{300 A_n}{12 \text{ in} \cdot (60 \text{ ft} \times 12 \text{ in})} \right) = 300 \left[ \frac{12 \text{ in} \cdot (60 \text{ ft} \times 12 \text{ in})}{1 \text{ in} \cdot (60 \text{ ft} \times 12 \text{ in})} \right] = 328.3 \text{ K} \]

\[ V_n = 328.3 \text{ K} \]

\[ \varnothing V_n = 6.8(328.3 \text{ K}) = 2.627 \text{ K} > V_n \quad \text{O.K.} \]
(Check for Overturning)

Overturning Moment:
\[ M_o = (40.4K \times 36') + (51.6K \times 16') = 1,956.8 \text{ K-ft} \]

Resisting Moment:
\[ P_{wall} = 149K \]
\[ M_r = (149K \times 30\text{ft}) = 4,470 \text{ K-ft} \]
\[ M_r > M_o \text{ O.K.} \]

\(^1\) Value is from seismic loading cases, for total force/shear.
Shear Wall 5 Verification
Shear Wall Designation: Shear Wall 5

<table>
<thead>
<tr>
<th>Wall Criteria</th>
<th>Masonry</th>
</tr>
</thead>
<tbody>
<tr>
<td>f_m, Masonry Compressive Strength</td>
<td>100 (psi) (ACI 530 Table 5.4.2)</td>
</tr>
<tr>
<td>f_r, Masonry Modulus of Rupture</td>
<td>75 (psi) (ACI 530 Table 3.1.8.2)</td>
</tr>
<tr>
<td>t, Wall Thickness</td>
<td>12 (in)</td>
</tr>
<tr>
<td>h, Wall Height</td>
<td>90 (ft)</td>
</tr>
<tr>
<td>L, Wall Length</td>
<td>80 (ft)</td>
</tr>
<tr>
<td>A_w, Wall Net Cross-Sectional Area</td>
<td>8,640 in² (For Vertical Wall Cross Section)</td>
</tr>
<tr>
<td>S_w, Wall Section Modulus</td>
<td>73,728 in³ (For Vertical Wall Cross Section)</td>
</tr>
<tr>
<td>l_w, Wall Moment of Inertia</td>
<td>7,077,888 in⁴ (For Vertical Wall Cross Section)</td>
</tr>
<tr>
<td>l_s, Wall Section Modulus</td>
<td>17,280 in³ (For Lateral Wall Cross Section)</td>
</tr>
<tr>
<td>l_t, Wall Moment of Inertia</td>
<td>103,680 in⁴ (For Lateral Wall Cross Section)</td>
</tr>
<tr>
<td>r, Wall Radius of Gyration</td>
<td>29 (in)</td>
</tr>
<tr>
<td>e, Eccentricity</td>
<td>0 (in)</td>
</tr>
<tr>
<td>h, Wall Height</td>
<td>16 (ft)</td>
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<tr>
<td>M_e, Eccentric Moment</td>
<td>0 (k-ft)</td>
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<tr>
<td>P_n, Nominal Bending Moment</td>
<td>173 (k) ( Loads from Interior Members)</td>
</tr>
<tr>
<td>P_u, Ultimate Vertical Load on Wall</td>
<td>237 (k)</td>
</tr>
<tr>
<td>S_n, Wall Section Modulus</td>
<td>73,728 in³ (For Vertical Wall Cross Section)</td>
</tr>
<tr>
<td>I_w, Wall Moment of Inertia</td>
<td>7,077,888 in⁴ (For Vertical Wall Cross Section)</td>
</tr>
<tr>
<td>ω_u, Ultimate Lateral Loading</td>
<td>22 (lb/ft²) (See Hand Calculations)</td>
</tr>
<tr>
<td>M_u, Ultimate Lateral Bending Moment</td>
<td>21 (k-ft) (See Hand Calculations)</td>
</tr>
<tr>
<td>V_u, Ultimate Lateral Shear</td>
<td>24 (k) (See Hand Calculations)</td>
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<tr>
<td>I_y, Wall Moment of Inertia</td>
<td>103,680 in⁴ (For Lateral Wall Cross Section)</td>
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<tr>
<td>r, Wall Radius of Gyration</td>
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<tr>
<td>M_r, Resisting Moment</td>
<td>2,552 (k-ft)</td>
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<tr>
<td>V_r, Resisting Shear</td>
<td>34 (k)</td>
</tr>
</tbody>
</table>

Based on values from brick dimensions guide from The Belden Brick Company Website

Note: Ultimate Moment is for wind loading only. Wall will not support the ultimate seismic load. It is estimated that the wall will support approximately 60% of the ultimate seismic load.

It is well documented that Unreinforced Masonry (URM) Structures do NOT perform well under loading from earthquakes. Though beyond the scope of this project, seismic retrofitting is highly recommended for any actual work performed on the Vu Villa. Appendix A1 of the 2012 International Existing Building Code (IEBC) lays out the expectations and requirements of seismic retrofitting of URM Walls.

For this assessment, as seismic loading of this scale has a low probability and strong winds are a more likely threat to the building, the bending strength of the shear walls was assessed for wind loading.

VU VILLA STRUCTURAL DESIGN/REHABILITATION
EXISTING MEMBER VERIFICATION

Note: Ultimate Moment is for wind loading only. Wall will not support the ultimate seismic load. It is estimated that the wall will support approximately 60% of the ultimate seismic load.
Check for Vertical Load Resistance:

\( P_n = P_{int} + P_{wall} \)

\( \frac{b}{r} < 99 \text{ so:} \)

\( P_n = 0.8 \geq 0.8 \text{ A}_n \)

Check for Lateral Load Resistance:

(Bending Strength)

\( M_u \)

\( W_u (\text{ seismic}) = 109.5 \text{ lb/ft}^2 \)

\( W_u (\text{ wind}) = 22.1 \text{ lb/ft}^2 \)

\( M_u = (109.5 \text{ lb/ft}^2)(16\text{ ft}) = 1,752 \frac{\text{ lb-ft}}{\text{ ft}} \)

\( M_u = (22.1 \text{ lb/ft}^2)(16\text{ ft}) = 353.6 \frac{\text{ lb-ft}}{\text{ ft}} \)

\( \phi M_u = (\frac{6}{\text{16}}) \left[ \frac{(12^2)(12^2)}{6} \right] (75\% \text{ of}) = 1,080 \frac{\text{ lb-ft}}{\text{ ft}} \)

\( \phi M_n < M_u \) (scarecrow) Failed (See Note on Spreadsheet)

\( \phi M_n > M_u \) (wind) O.K.
\[ V_n = 34.4 \times 16' = 550.4 \text{ kips} \]

\[ V_n = \text{Minimum} \left( \frac{3.8A_n f_m'}{300 A_n} \right) = 328 \text{ kips} \]

\[ \phi V_n > V_n \quad \text{O.K.} \]

\[ M_o = (34.4 \times 16') (16') = 550.4 \text{ k-ft} \]

\[ M_r = (85\phi \times 30') = 2,550 \text{ k-ft} \]

\[ M_r > M_o \quad \text{O.K.} \]

No hold-down required
### Task: Administration, Planning, & Engineering

<table>
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<tr>
<th>Description</th>
<th>Rate</th>
<th>Units</th>
<th>LC</th>
<th>MLC</th>
<th>HLC</th>
<th>Comments</th>
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### Task: Mobilization & Project Preparation

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### Task: Mobilization/Deconstruction

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### Task: Demolition/Removal of Replacement Members

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<td>4.1.14 Simpson HBLD30 Beam Caps</td>
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<td>EA</td>
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### Task: Replacing Member Installation

<table>
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<tr>
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<th>LC</th>
<th>MLC</th>
<th>HLC</th>
<th>Comments</th>
</tr>
</thead>
<tbody>
<tr>
<td>5.1 Labor(^4)</td>
<td>N/A</td>
<td>N/A</td>
<td>N/A</td>
<td>N/A</td>
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<tr>
<td>5.1.1 Carpenter</td>
<td>$55.00</td>
<td>HRS</td>
<td>320</td>
<td>HRS</td>
<td>$17,400.00</td>
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<td>5.1.2 Carpenter</td>
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<td>HRS</td>
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### Task: Mobilization/Removal of Replacement Members

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<tbody>
<tr>
<td>6.0 Existing Member Repair/Replacement</td>
<td>N/A</td>
<td>N/A</td>
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<tr>
<td>6.1 Materials</td>
<td>N/A</td>
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<td>N/A</td>
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<tr>
<td>6.1.1 Roof Joist (6-3/4&quot; x 24&quot;)</td>
<td>$11,000.00</td>
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### Task: Local Lowes\(^6\)

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</thead>
<tbody>
<tr>
<td>7.0 Administration, Planning, &amp; Engineering</td>
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### Task: Contractor\(^5\)

<table>
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<th>LC</th>
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<tr>
<td>8.0 Administration, Planning, &amp; Engineering</td>
<td>$4.90</td>
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### Task: Contractor\(^5\)

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<th>Description</th>
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<th>HLC</th>
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<tr>
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### Task: Contractor\(^5\)

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<th>Description</th>
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<th>HLC</th>
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</thead>
<tbody>
<tr>
<td>10.0 Administration, Planning, &amp; Engineering</td>
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<td>EA</td>
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## PROJECT PHASES ANTICIPATED DURATION

<table>
<thead>
<tr>
<th>TASK NO.</th>
<th>DESCRIPTION DURATION (HRS)</th>
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<tbody>
<tr>
<td>1.0</td>
<td>ADMINISTRATION, PLANNING, &amp; ENGINEERING</td>
</tr>
<tr>
<td>2.0</td>
<td>MOBILIZATION &amp; PROJECT PREPARATION</td>
</tr>
<tr>
<td>3.0</td>
<td>DEMOLITION/REMOVAL OF REPLACEMENT MEMBERS</td>
</tr>
<tr>
<td>4.0</td>
<td>REPLACEMENT MEMBER INSTALLATION</td>
</tr>
<tr>
<td>5.0</td>
<td>EXISTING MEMBER REPAIR/REPLACEMENT</td>
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</table>

**PROJECT DURATION (LESS TASK 1.0)** 720.00

**TOTAL PROJECT DURATION** 1,040.00

## ESTIMATED LABOR RATES

<table>
<thead>
<tr>
<th>POSITION</th>
<th>RATE ($/HR)</th>
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<tbody>
<tr>
<td>SUPERVISOR</td>
<td>$115.00</td>
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<td>SAFETY ENGINEER</td>
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<td>CARPENTER</td>
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<tr>
<td>LABORER</td>
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*RATES BASED ON 250% OF BUREAU OF LABOR STATICS MEDIAN ESTIMATES FOR 2018*

## ESTIMATED EQUIPMENT RATES

<table>
<thead>
<tr>
<th>EQUIPMENT TYPE</th>
<th>RATE ($/HR)</th>
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<tbody>
<tr>
<td>PICKUP</td>
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<tr>
<td>SCISSOR LIFT</td>
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<tr>
<td>FORKLIFT</td>
<td>$35.00</td>
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<td>DUMP TRAILER (W/PICKUP)</td>
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<table>
<thead>
<tr>
<th>Item</th>
<th>Description</th>
<th>U/M</th>
<th>Price</th>
<th>Per</th>
<th>Amount</th>
</tr>
</thead>
<tbody>
<tr>
<td>SPOLF</td>
<td>5 1/2&quot;x21&quot; 24F-V4 GLULAM BEAM</td>
<td>LF</td>
<td>26.2500</td>
<td>LF</td>
<td>525.00</td>
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<td>SPOLF</td>
<td>6 3/4&quot;x24&quot; 24F-V4 GLULAM BEAM</td>
<td>LF</td>
<td>42.3000</td>
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<td>846.00</td>
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<td>SPOEA</td>
<td>6&quot;x6&quot;x10' SELECT STRUCT DOUG/FIR COLUMN</td>
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<td>478.0000</td>
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Quoted prices are good for 7 days.

Subtotal
Sales Tax
Total

Buyer: ____________________________

Signature: ____________________________
**QUOTE**

1911-934229  R1  PAGE 2 OF 3

<table>
<thead>
<tr>
<th>Item</th>
<th>Description</th>
<th>D</th>
<th>Quantity</th>
<th>U/M</th>
<th>Price</th>
<th>Per</th>
<th>Amount</th>
</tr>
</thead>
<tbody>
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<td>CC66</td>
<td>CC66 6&quot;x6&quot; COLUMN CAP</td>
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<td>EA</td>
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<tr>
<td>SPOEA</td>
<td>SIMPSON HGLBA BEAM SEAT</td>
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<td>EA</td>
<td>106.950</td>
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<td>SIMPSON HTSM20 STRAP</td>
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<td>EA</td>
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<td>SIMPSON HL53 ANGLE BRACKET</td>
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<td>SPOEA</td>
<td>SIMPSON HST5PC STRAP</td>
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<td>57.750</td>
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Quoted prices are good for 7 days.

<table>
<thead>
<tr>
<th></th>
<th>Subtotal</th>
<th>Sales Tax</th>
<th>Total</th>
</tr>
</thead>
</table>

Thank you we appreciate your business!!

Buyer: ____________________________

Signature: ____________________________
Thank you we appreciate your business!!

<table>
<thead>
<tr>
<th>Item</th>
<th>Description</th>
<th>D</th>
<th>Quantity</th>
<th>U/M</th>
<th>Price</th>
<th>Per</th>
</tr>
</thead>
<tbody>
<tr>
<td>SPOEA</td>
<td>NON-REFUNDABLE. PLEASE DOUBLE CHECK YOUR QUANTITIES BEFORE ORDERING. SIMPSON HGLBD BEAM SEAT w1= 6 3/4&quot; 7 TO 10 DAY LEAD TIME TO OUR YARD UPS CHARGES WILL BE ADDED WHEN ORDERING SPECIAL ORDERS ARE NON-RETURNABLE AND NON-REFUNDABLE. PLEASE DOUBLE CHECK YOUR QUANTITIES BEFORE ORDERING.</td>
<td>1</td>
<td>EA</td>
<td>139.9500</td>
<td>EA</td>
<td>139.95</td>
</tr>
</tbody>
</table>

**PRICE**

****IMPORTANT PRICING NOTE***

ALL PRICING GOOD FOR TWO WEEKS FROM DATE OF QUOTE. PLEASE CALL FOR UPDATED PRICING.

In order for the prices on this quote to be secured, quantities must be verified and purchasing arrangements made before the quote expiration date shown above. Any material purchased after this quote expiration date will be subject to current market price.

Quoted prices are good for 7 days.

| Subtotal | 2,336.80 |
| Sales Tax | 0.00 |
| Total | 2,336.80 |

Signature
APPENDIX F – VU VILLA SAG REPORT
July 29, 2013

Elizabeth Larson
521 West Park Street
Butte, Montana 59701

RE: Vu Villa Bar and Pizza Investigation
521 West Park Street
Butte, Montana 59701

Elizabeth:

As requested, an investigation was conducted to assess the structure at the above noted address. Bruce Haroldson performed the initial investigation on July 18, 2013. A secondary inspection was completed by Maria Chesnut, PE and Claire Rogan, EIT on July 23, 2013. The top of the stairs located in the bar area has visibly deflected and prompted the concern.

The Vu Villa Bar and Pizza building consists of two identical sections—the bar is in the west section, and the pizza parlor is in the east section. The visible structural systems in both sections suggest that the original footprint was the same for both sides of the building. Anecdotal evidence indicates that the stairs were removed from the east section long ago and supports were added more recently to re-support the main and upper floors. However, the main structural systems remain much the same.

The west section is approximately 29’ – 0” x 61’ – 7” in plan, and the east is approximately 25’ – 0” x 61’ – 7” in plan. The exterior walls and separation wall between the two sections are all original brick walls. The west section sits atop concrete slab-on-grade, and the east section has an unoccupied basement area with rubblestone foundation walls. The main floor framing in the east section consists of 2x12 @ 18” on-center, supported by the foundation walls and a beam line in the center. We were unable to access the upper floor framing but assume that it is similar to the main floor framing.

Photo 1, above, illustrates the deflection at the top of the stairs, as visible from the bar area at Vu Villa. From the inside of the stairwell, the stairs visibly slope left to right (west to east) and a large crack is visible in wall covering at the top of the stairs, see Photo 3 below. On the second level, it was obvious that the floor framing has deflected, and the low point occurs at the top of the stairs, where the
deflection can be seen from below. The deflection is most likely due to the removal of the bearing wall below the stairs. See Photo 2, at right.

The entire second floor of the building is not currently in use, so the main concern is stabilization of the stairs and associated upper floor framing. There are several options for stabilization and repair, dependent on scope of current and future plans for the building.

Because the second floor is unoccupied, a beam above the floor framing spanning between the brick walls of the west bay, with a beam along the inside stairwell wall to support the stairs would provide stabilization and mitigate further settlement. It is important to note that this option is for stabilization only. The pre-existing settlement would still be visible, but this solution would prevent progression.

If future plans, for a second level renovation or otherwise, warrant a more permanent solution, the second option is to demo the floors and walls, as noted on attached plan, jack up the stairs to their original location, and put a new steel beam in the joist cavity and a new steel beam along the inside wall of the stairs to re-support the stairs and floor framing. Both steel beams would be contained within the floor framing, and would not be visible upon refinishing the floors and ceiling. Existing floor joists would need to be reconnected to the new steel beams, and floor sheathing would need to be replaced in the areas of demolition. Wood finishes could be salvaged throughout the process.

Upon investigation of the structure at Vu Villa, we recommend that the stairs and associated floor framing be stabilized in one of the aforementioned manners. This report is preliminary in nature. Further analysis and design will be required for stabilization and repair of the structure at the stairs.

Please feel free to contact us at any time for questions of concerns.

Sincerely,

BEAUDETTE CONSULTING ENGINEERS

Maria Chesnut, PE Claire Rogan, EIT Bruce Haroldson