Group Design Report
Overton Street Development

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Introduction (617 words, All Contributed)

This report documents TWENZ Engineering's design process for the Overton Street Development (OSD) building. A description of the project and overall objective and scope is provided, followed by existing site conditions and their impact on design. Regulatory compliance and permitting considerations are explored to ensure the safety and environmental responsibility of the design. This report highlights the structural and water resource engineering and design provided by TWENZ engineering and details the process and results for each respective design. Alternative design approaches were explored for each element of design, and subsequent design choices are rationalized and summarized. A summary of design solutions is located at the end of the report, and all applicable calculations are included in Appendix A.

Project Background

Figure 1**.** Building render and vicinity map of the Overton Street Development Project (render courtesy Mackenzie Engineering, vicinity map from Google Maps)

Located in Portland, Oregon, the Overton Street Development project expands the adjacent Dovelewis Animal Hospital with a new three-story building, complete with underground parking, driveways, and minimal space for landscaping. The new building is positioned between NW Pettygrove and NW Overton streets. The 30,179 square foot animal hospital expansion fills the first two floors, while the third floor is 16,237 square feet of dedicated office space. The 14,876 square foot underground parking garage contains two vertical car stackers to optimize capacity.

The client has requested a sustainable design approach including a green roof and structural steel. The green roof consists of a soil medium with planted vegetation and will reduce energy consumption by aiding in building cooling while providing a picturesque landscape for nearby high-rise buildings. The green roof will also act as a natural air filter for the surrounding downtown area and provide stormwater retention and natural filtration before municipal sewer integration.

Objective and Scope

TWENZ Engineering prioritizes public safety, economy, environmental impact, and sustainability for the design of the OSD building. To meet these project goals, the team incorporated a broad range of engineering strategies. To ensure the environmental footprint of the project is mitigated the team constructed a green roof component for the building.

Both the water resource and structural teams will deliver accompanying plans reflecting the design approaches of each respective discipline. The water resources team will analyze and engineer solutions for efficient stormwater management, mitigation of water infiltration, and the interface with municipal systems. A crucial aspect of this project consists of planning for the uncertainties that may result from the impacts of the load capacities on the water drainage system. The load capacity for the roof was based on a worst-case scenario outcome of future storms that also accounts for full saturation loads to ensure longevity. To complement the green roof, 16 drains tied into two subgrade water detention vaults were incorporated into the design which significantly reduces flood events, over saturation, and overload of the roofing drainage infrastructure.

The structural team is responsible for engineering specifically chosen members to resist gravity and lateral loads, in this case, a special concentric braced frame building design was chosen due to strength longevity, and cost-efficiency. The brace frame configuration and shape are multistory X with HSS round because of their economy and efficiency. Especially as components of all steel, girders, beams, and columns that will complement the brace frame design. The building foundation utilized a gravity load analysis component of the project design; used to emphasize building structural integrity and ensure user safety. Also, incorporates a fully concrete developed underground parking garage that will be implemented along with a slab to serve as the foundational piece. Two-way mat slabs were chosen for the OSD as well as reinforcement stirrups at each column face. These components yield a design, meet objectives, and complete the goals for both; the water resource and structural team, who strive to develop a sustainable, economical, and structurally sound building for the downtown Portland area.

Existing Conditions (311 words, Nicota Liesy)

The proposed project site sits between NW Pettygrove St. and NW Overton St., with both streets being used daily by pedestrians and different modes of transportation. Pre-existing buildings sit on either side of the project site to the East and West, meaning staging and material laydown spots will need to be organized in a manner that does not inhibit the use of the public roadways. Based on the topographic map provided by Mackenzie Engineering (see **Figure** 2), the project site is level with a small slope toward the North and the maximum changes in elevation do not exceed 5 feet. The existing site grade requires excavation of 15 feet of material to accommodate the proposed underground parking facility.

The provided geotechnical results indicate there is an undocumented filled stream channel that crosses the site area. The depth of fill is on the order of 15-25 feet in depth, with gravels and cobbles that contain boulders. Additionally, the project site is made up of native silts and sands that are underlain with gravel and cobbles at a depth of 25-30 feet. The undocumented fill on-site poses a concern for the geotechnical engineers, as it will affect the design of the foundation and floor supports of the new development. The soil make-up of the site also poses a concern, as silts and sands are easily disturbed and sensitive to moisture fluctuations. According to the City of Portland stormwater management and provided geotechnical report, on-site stormwater disposal facilities will be considered.

Before beginning excavation and grading, multiple pre-existing items that must be removed and are noted in the provided Mackenzie Engineering demolition plan. These include asphalt, foundations, curbs, fences, stormwater infrastructure, oil tanks, and utilities. The pre-exiting sidewalks on the North and South fronts of the project site will have to be partially removed to allow for the placement of subgrade water mitigation systems.

Site Plan

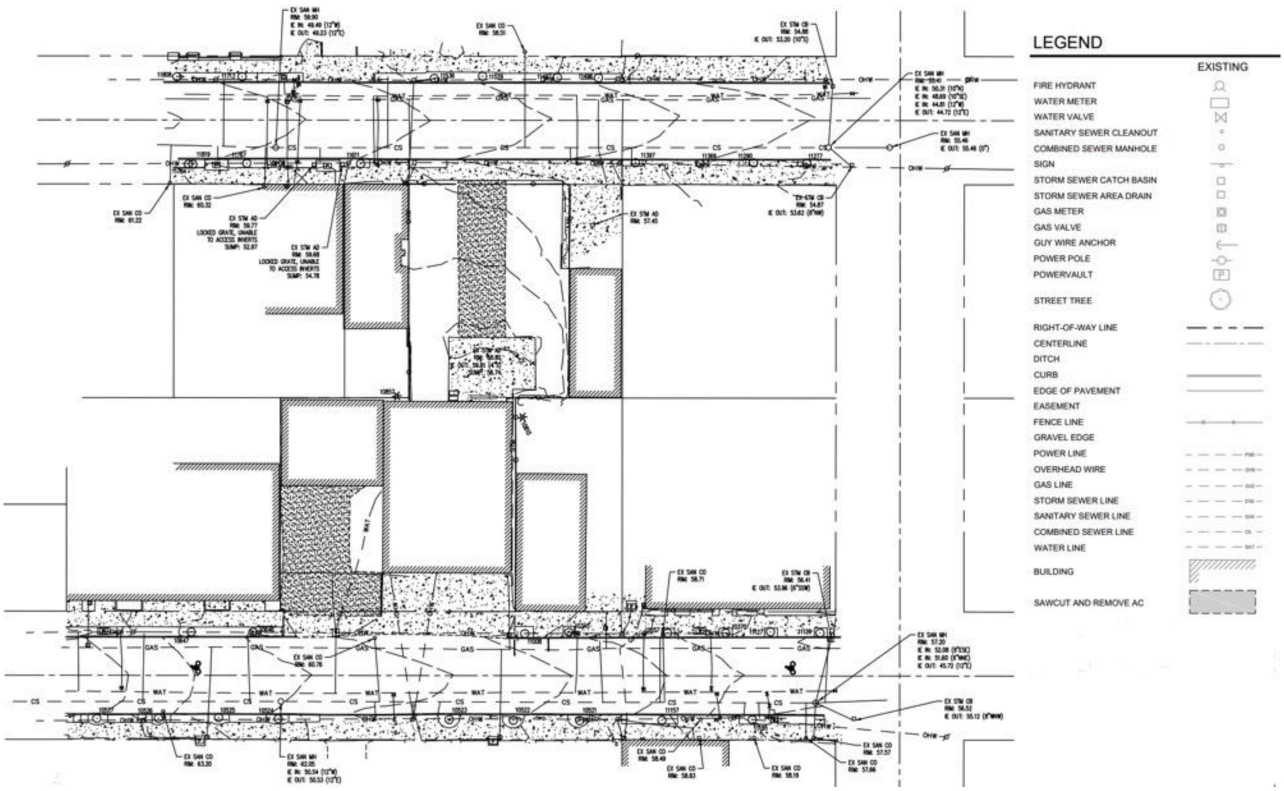

Figure 2**.** Exiting conditions site plan for the Overton Street Veterinary Hospital development.

Evaluation and Selection of Alternatives (1425 words, All Contributed)

Structural

Gravity System

Design alternative 1: Cross-Laminated Timber

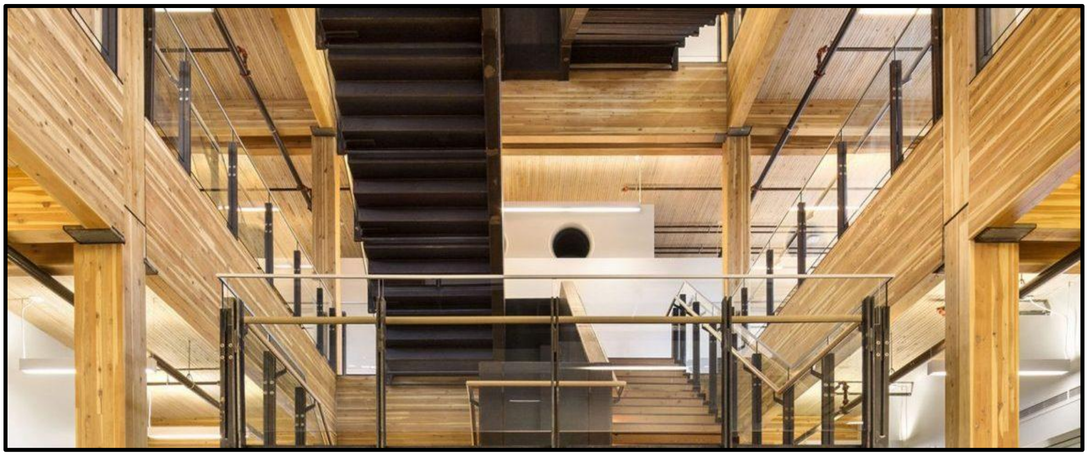

Figure 3. Visual representation of an integrated steel and mass timber design (courtesy of pesengineers.com)

TWENZ Engineering initially expressed interest in exploring cross-laminated timber design. As it would be dedicated to making an aesthetical appearance for the infrastructure of the building. However, the glulam columns were considered but ruled out due to the significant spacing of the existing column layout. Creating an alternative column layout within the architectural floor plan was deemed outside the scope of this course, primarily due to time constraints. Engineered glulam girders were considered but ruled out due to the dropped ceilings removing any aesthetic benefit of using wood members. The use of glulam beams was also ultimately discarded due to their limited aesthetic contribution.

Design alternative 2: Structural Steel

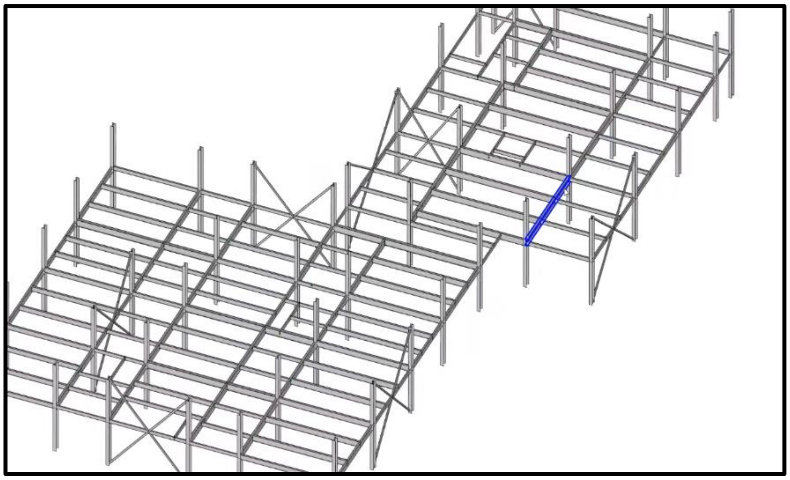

Figure 4. Structural framing member system of girders and beams on the 2nd floor of the veterinary hospital implementing only steel design for framing members

Structural steel will be more economical and is the material of choice for girders in the OSD building. Beam spacing is highly configurable, which allows the use of a multitude of materials. Structural steel beams and steel lattice trusses are realistic options because their spacing can be adjusted to suit depth constraints. Steel beams were selected due to their routine integration with the surrounding steel girders. Structural steel also provides other economic benefits, such as:

- Lower foundation costs as structural steel has higher strength-to-weight ratio than other materials
- Increased revenue since steel framing can be constructed faster
- Future cost savings when project of modification or expansion comes into play

Design alternative 3: Reinforced Concrete

Figure 5**.** Visual representation of entirely concrete infrastructure of building (courtesy of concreteparking.org)

Composite corrugated steel/concrete slab subfloors are common in large commercial structures and were likely used in the original design proposed by the architect. The first floor will be a solid 12-inch thick reinforced concrete slab with steel girder supports as needed. Further expanding on the involvement of reinforced concrete columns, they will be incorporated into the underground parking facility due to their high strength, inexpensive cost, limited aesthetic contribution, and resilience to high moisture levels.

Foundation Design Alternatives

The footing at location C-5.5, highlighted in Figure 6, was chosen for analysis. This location was chosen because it contains all gravity load cases and carries a large tributary area. By designing the foundation using a worst-case-scenario column, the design results could be applied to all other foundation elements while remaining conservative.

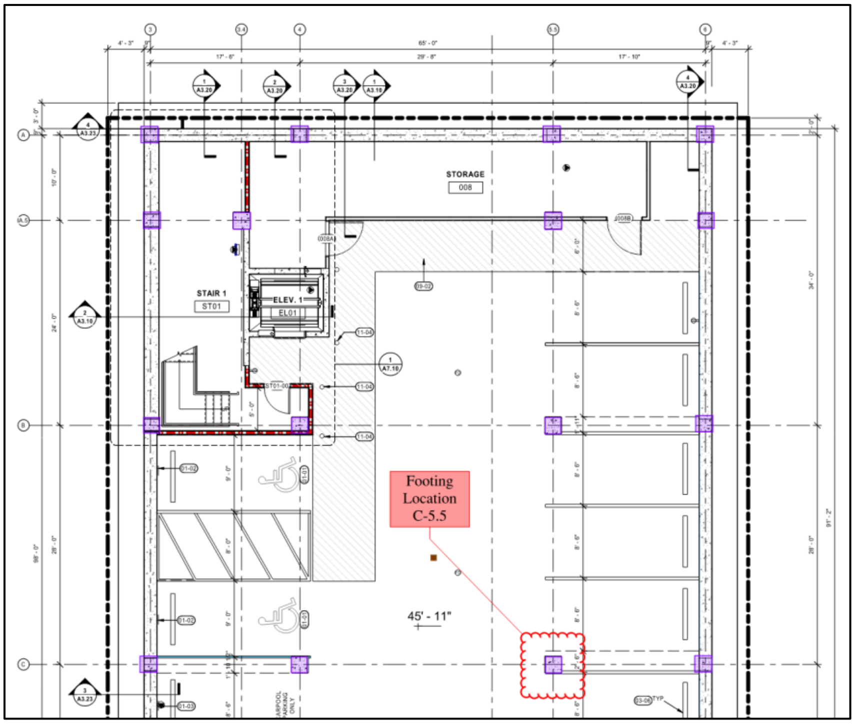

Figure 6. Footing Location of C-5.5 Chosen for Analysis

The maximum factored design load at footing C-5.5, used for concrete design, was calculated to be 699.3 kips (LRFD) (ACI, 2019). The service load, used for calculating bearing pressure, was calculated to be 547.5 kips. The accompanying geotechnical report lacked specifications for soil bearing capacity, so the maximum soil bearing capacity of 1,500 psf will be used for footing design, as specified by Portland Development Services (BDS, 2019)(BDS, 2019). Two foundation design alternatives were explored: (1) a rectangular spread footing and (2) a mat slab (shown in **Figure 7**). Initial spread footing calculations at C-5.5 resulted in a footing greater than 19' x 19'. Spread footings of this size would compose greater than half the building footprint, so a mat slab of reinforced concrete was chosen instead. An additional benefit of the mat slab is that it forms a continuous concrete floor suitable for the underground parking garage.

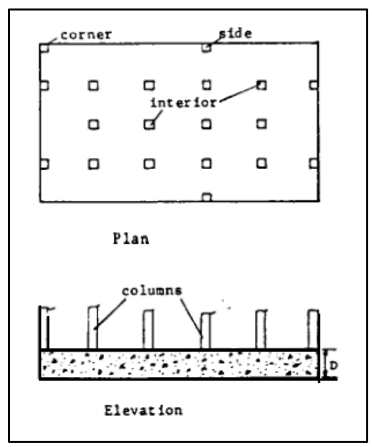

Figure 7. Mat Slab Design (ACI 336.2R)

Brace Frame Evaluation and Selection of Alternatives

There are two main factors considered that influence the LRFS design alternatives: member shape and braced frame configurations. The considered member shape alternatives are W-shape and HSS round. The braced frame configurations including three options: inverted V-braced frame, X- braced frame, and multistory X brace (MXB) frame.

Design alternative for Brace Frame Configuration

This section describes the brace frame configuration alternatives and provides the insight of their functionalities.

Design alternative 1: Inverted V-braced frame

The inverted V-braced frame or Chevron braced frame have braces connected in the midspan of the girder and form an inverted V-shape. Figure 5 provides a clear visual of the inverted V-braced frame. However, for this type of braced frame, the beam must be designed to be continuous between columns and capable of resisting the maximum unbalanced vertical and horizontal loads when the braces below have buckled or yield (Yang et al, 2019). This results in large and bulky beam selection for this structure.

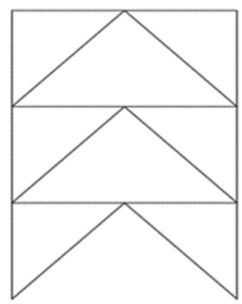

Figure 8**.** Configuration of Inverted V-Brace Frame

Design alternative 2: X-braced frame

The X-bracing is a cross brace connected at each end of the girder. Figure 6 indicates the X-bracing system. This braced frame structural has more advantage than the chevron brace due to its symmetrical geometry, no unbalanced vertical forces in the beams, but the columns are designed to resist large axial forces when the braces have yielded or buckled (Yang et al, 2019). Also, an additional disadvantage is the inelastic deformation capacity is reduced because the inelastic deformation is concentrated in one-half the brace length, and other half cannot fully develop its capacity as the more damaged half deteriorates. (Sabelli et al 2013)

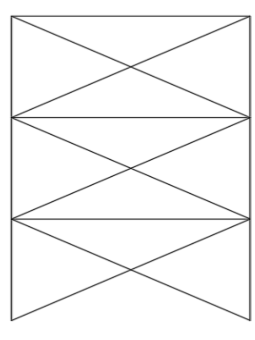

Figure 9**.** Configuration of X-Brace Frame

Design alternative 3: Multistory X-braced frame

The Multistory X-bracing (MXB) over a 3-story frame utilized V-bracing and Chevron Bracing alternately to form a X-shape between 2 stories. Figure 7 shows the proposed MBX for this project. Same as the X-bracing, MBX has a symmetrical geometry so it can prevent unbalanced vertical forces in the beams. It also allows transferring story shear to adjacent stories even after brace buckling and fracture because the remaining tension brace may directly transfer its force to the next story. (Sabella et al., 2013)

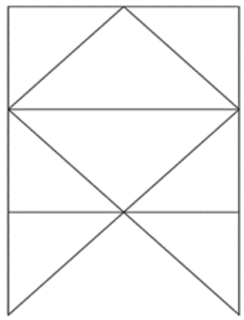

Figure 10**.** Configuration of Multistory X-Brace Frame

Design alternative for Member Shapes

This section identifies the W-shape and HSS round properties and characteristics.

Design alternative 1: W shapes

W- shapes are also called wide-flange shapes and are one of the most popular shapes used in construction. They have two flanges with parallel inner and outer faces and a single web located midway along the flanges (Geschwindner et al., 2017). The W-shape steel are commonly made from A992 steel, a high-strength low alloy steel, with a minimum yield stress, Fy = 50 ksi, and a minimum tensile stress, Fu = 65 ksi. Figure 8 provides a presentation of cross section of wide flange shape (Geschwindner et al., 2017).

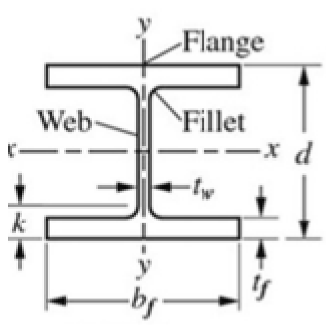

Figure 11**.** W-shaped Cross Section

Design alternative 2: Round HSS shapes

Round HSS shapes, also called round hollow structural sections, are increasing popularity within construction, and building community. They are commonly made from A500 carbon steel and come in Grade C with a minimum yield stress, Fy = 46 ksi and minimum tensile stress, Fu = 62 ksi. The advantages of round HSS shapes over W-shape are attractive appearance, ease of maintenance, and economy (Sabelli et al., 2013). In addition, HSS is effective for longer tension components, when slenderness and related serviceability considerations may be important (Geschwindner et al., 2017). Due to lack of weak axis, round HSS shapes also are superior in compressive strength. However, the connection ends tend to be complicated and expensive. Figure 9 provides a visual aid for round HSS shape.

Figure 12**.** HSS Round Shape

Brace Frame Selection Alternative

According to the research perform by Yang TY., Sheikh H. and Tobber L (2019), the multistory X-brace frame is the lightest system. The second lightest system is X-brace configuration, and the inverted V-brace frame uses the most material. As for the steel shape, HSS round is more economical than W-shape. As the result, the proposed braced frame system is the multistory X-braced frame with HSS round shape. Figure 10 provides a presentation of the proposal brace frame system.

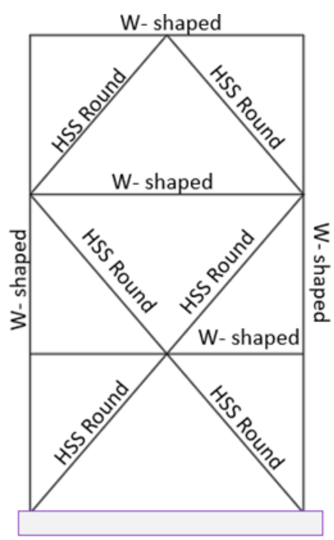

Figure 13**.** Proposed Brace Frame System

Water Resources

Design alternative for Stormwater Mitigation System

Design Alternative #1: Singular Concrete Water Detention Vault

In the initial phase of designing the stormwater mitigation system, the first design alternative was a singular concrete water detention vault. This stormwater vault would either be placed beneath one of the curbs at the front of the proposed building, or somewhere beneath the subgrade parking facility. This design alternative was discarded, as placement beneath the structure would make replacement and maintenance difficult. Additionally, results of the hydrologic study showed that the size of a singular detention vault would be too large to place beneath curb or structure.

Design Alternative #2: Two Concrete Water Detention Vaults

After a singular concrete water detention vault was ruled out as not an option, one of the water resources engineers decided that multiple vaults might solve the problems. Through trial and error, the engineer decided on two concrete water detention vaults that would be placed beneath the North and South curbs of the proposed building. These vaults solved the problem of sizing requirements, as well as providing ease of replacement and future maintenance.

Design alternative for Stormwater Retention System

Design Alternative #1: Green Roof System

The green roof system is a design that focuses on mitigating stormwater runoff by adding a vegetation layer on top of the proposed building. The green roof system itself is made up of 3 layers, a vegetation layer, a soil medium, and a waterproofing layer which is sometimes paired with a rooting protection membrane. The system in the proposed design will be an extensive green roof with to obtain the clients wants of being as sustainable as possible.

The benefits of the green roof design system include the ability to reduce energy consumption for the building in the warm season, ecologically friendly way to deal with stormwater runoff, improve air quality, and increase the aesthetics the building provides to location. Some negatives for this design include the added loads from the green roof which affects the structural loading of the building.

Design Methodology (4998 words, All Contributed)

Structural

Global Design Parameters

The first step of the design process is to establish global design parameters. Building risk category, found in the Oregon Structural Specialty Code (ICC, 2019, Table 1604.5), designates the design criteria according to the importance of the structure. A high-risk category is associated with higher consequences to the building occupants should the structural system fail, while a low-risk category poses little or no risk to human life. Higher risk category structures use load factors that result in a more conservative structural design. The building will contain a veterinary hospital and office space. **Table 1** contains a summary of the design parameters.

Table 1. Basic Structural Engineering Design Parameters (ICC, 2019)

Structural Engineering Parameters	
Building Risk Category	II
Building use-case	Veterinary Hospital, Office

Design Loads - Roof

Roof gravity loads consist of snow loads, live loads (usually during construction or service), and dead loads. This section outlines the process of determining the various roof loads used to design structural members of the OSD building.

Snow loads are informed by computer models of snowfall based on geographical location and elevation. A useful tool for design ground snow load can be found at the Oregon Structural Engineers Associations of Oregon website (SEAO, 2022)(SEAO, 2022). Prior to engineering, the design ground snow load must be adjusted to obtain the design snow load used for calculations. If the real elevation is significantly different than the modeled elevation, it must be adjusted in accordance with guidelines located on the SEA website. In this case, the OSD building site elevation fit within the modeled elevation, so no adjustments were necessary. A rain-on-snow surcharge must be added to the roof design snow load in accordance with ASCE 7-16 (ASCE, 2017, Section 7.10)(ASCE, 2017, Section 7.10). Excessive deflection of low-slope roof members caused by high snow loads can result in a ponding effect, further increasing the worst-case-scenario load. To avoid this, member deflections must be calculated for ponding instability in accordance with ASCE 7-16, Section 8.4. The OSD has a maximum roof slope of 3/8” : 1’-0” and must be designed for both rain-on-snow surcharge and ponding instability. Finally, the snow loads must be increased at all locations where snow drifts exceed the depth of the design snow load. Snow drift adjustments account for elevation disparities such as mechanical equipment and parapet walls, which cause excess snow to accumulate around them, sometimes exceeding the design roof snow load. The OSD building roof is surrounded by parapet walls, thus drift loads must be checked in accordance with ASCE 7-16, Section 7.6, 7.7, and 7.8.

The roof design dead load of the OSD consists of the green roof system, mechanical equipment such as heating and air conditioning units, roofing materials, and the structural members. The dead load of the concrete subfloor was calculated based on a 3-inch concrete slab on a metal deck with 150 pcf concrete density. Green roof and rain loads were furnished by the water resource team, while the remaining dead load assumptions were based on mentor guidance of commonly used values. All roof design loads used to engineer the structural system are summarized in **Table 2**.

Table 2. Summary of Design Roof Loads

Design Roof Loads	
Roof Live Load (global)	20 psf

Design Roof Loads	
Green Roof Dead Load (local)	28 psf
Snow Load, including rain-on-snow surcharge (global)	25 psf
Snow Drift Load (trapezoidal load within 10’ of parapet)	39 psf
Rain Load (global)	13 psf
Roofing Dead Load (global)	5 psf
Insulation Dead Load (global)	2 psf
Mechanical Dead Load (local)	100 psf
Beam/Girder Dead Load (global)	8 psf
Composite Subfloor Dead Load (global)	50 psf

Design Loads - Floor

Floor gravity loads consist of live loads, global dead loads, and local dead loads from mechanical equipment. This section outlines the process of determining the various floor loads used to design structural members of the OSD.

The intended use case of the structure, referenced in Table 1607.1 of the 2019 OSSC, informs the floor design live load. Based on mentor advice, the OSD building will be designed for 100 psf live load throughout all floors. This design results in a slightly conservative analysis and offers flexibility for future occupancies or use-cases, including assembly areas, which require 100 psf design live load.

Section 1606.2 of the OSSC defines design dead loads as the weights of all materials used to build the structure, including mechanical service equipment. Based on mentor guidance, conservative assumptions were made for typical mechanical dead load and structural member dead load. The composite subfloor dead load of floors 2 and 3 assume a 3-inch concrete slab on a metal deck, while the main floor dead load assumes a 12-inch-thick reinforced concrete slab. **Table 3** summarizes the design floor loads used for subsequent structural analysis.

Table 3. Summary of Design Floor Loads

Design Floor Loads	
Live Load (global)	100 psf
Beam/Girder Dead Load (global)	8 psf
Mechanical Dead Load (global)	10 psf

Design Floor Loads	
Composite Subfloor Dead Load (Floors 2 & 3)	50 psf
Slab Dead Load (Ground Floor)	150 psf

Mat Slab Design Methodology

Per mentor guidance, column C-5.5 was tested against the mat shear capacity to resist the shear force "punching through" the mat. The mat slab was designed using ACI 336.2R - Suggested Analysis and Design Procedures for Combined Footings and Mats (ACI, 2002) in combination with ACI 381-19. Mat design is an iterative process because the weight increases with mat thickness, thereby increasing soil pressure, and the shear capacity of the mat changes based on concrete compressive strength, mat thickness, and column location. ACI 336.2R points out mat slabs are commonly designed assuming a mat without shear reinforcement, so this conservative approach was adopted and checked prior to designing shear reinforcement. Shear reinforcement around the column will be calculated and specified to ensure thorough shear transfer through the adjacent mat area. Additionally, mat reinforcement will be specified for temperature and shrinkage as required for nonprestressed slabs based on ACI 318-19 Section 8.6.1.1, with a minimum reinforcement area of $0.0018A_g$, in both directions.

Soil Bearing

To ensure soil bearing capacity was adequate for the building and slab weights, a reasonable slab thickness needed to be chosen. Early design iterations suggested a 36" deep mat would provide suitable shear strength, so this depth was chosen for the mat design. The unfactored service load of the entire structure, including the slab and concrete columns, calculated to be 19,541 kips. This resulted in a soil pressure of 1,242 psf across the 15,735 square-foot footprint of the structure, which is less than the maximum of 1,500 psf. Because the mat slab pressure is less than the soil bearing capacity, the design approach is feasible, and the process could move forward.

Unreinforced Concrete Shear

Concrete design uses LRFD factored loads to determine adequacy (ACI, 2019) and the factored building weight calculated as 15,942 kips. This was calculated without the slab because the slab is continuously supported by the soil and does not need to be considered in mat slab strength. The factored building weight was then divided by the building footprint of 15,735 sf, which resulted in a factored building pressure of 1,010 psf. At this point the building was envisioned to be "inverted" and the factored building pressure applied to column C-5.5 proportional to its tributary area of 672 sf. This resulted in a shear force of 681 kips at column C-5.5.

In accordance with ACI 318-19 Section 22.6.5.2, the concrete mat shear capacity was calculated to be 156 psi when using 3500 psi concrete. To calculate the nominal concrete shear capacity, the critical perimeter around the column was calculated in accordance with ACI 318-19 Section 22.6.4, as illustrated in **Figure 14**. The 24-inch concrete column translated to a critical perimeter of 240-inches, and resulted in a nominal unreinforced concrete shear capacity of 767 kips, which is greater than the applied shear force of 681 kips at column C-5.5.

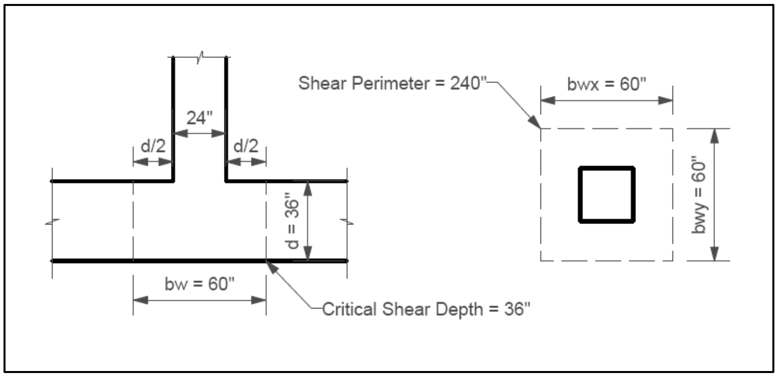

Figure 14. Mat Shear Critical Section Detail

Reinforced Concrete Shear

To prevent abrupt shear failure in the event of the slab cracking, reinforcement stirrups must be placed in the slab orthogonally from each column point load as shown in **Figure 15**. This effect increases with higher strength concrete, as critical failure can occur more abruptly than lower strength concrete. Stirrups must be present where the maximum concrete shear strength is less than the applied shear stress (ACI, 2019, Section 22.6.61).

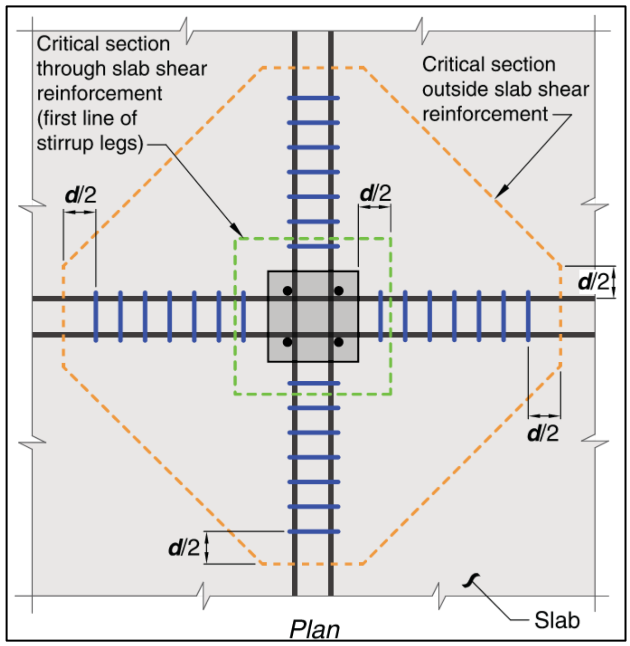

Figure 15. Critical Section Through Slab at Column Location (Figure from ACI 318-19 Section 22.6.4)

A single side of column C-5.5 was analyzed for two-way shear stirrup requirements in the slab using one-quarter of the factored column shear stress, equal to 170.2 kips. This shear stress was concentrated to an area equal to the slab depth (36 in) by the column reinforcement width (assumed to be 20 in), which created an ultimate shear stress of 236.4 psi. The maximum concrete shear strength was calculated to be 78 psi (ACI, 2019, 22.6.6.1)(ACI, 2019, 22.6.6.1), which left 158.4 psi shear stress to be carried by the reinforcement stirrups. Using #8 reinforcement, a maximum stirrup spacing of 9” was calculated (ACI, 2019, 22.6.7.1)(ACI, 2019, 22.6.7.1). By combining the concrete shear capacity with the steel reinforcement capacity, the nominal two-way design shear capacity of the mat was calculated as 190 psi, which is greater than the demand shear stress of 170.2 psi at the column.

The next step is to extend the chosen stirrup reinforcement into the slab, as illustrated in **Figure 15**, far enough to reduce shear forces to the previously calculated maximum two-way concrete shear strength of 78 psi. This step was unable to be completed for this report but would need to be completed before the design is finalized.

Temperature and Shrinkage Reinforcement

Minimum reinforcement requirements for temperature and shrinkage of $0.0018A_g$ are required per ACI 318-19 Section 8.6.1.1. The reinforcement specified in this section will be present in all areas of the slab and will help distribute tension forces due to column loads throughout the slab. An arbitrary cross-section of the slab, 36" deep x 60" wide, was chosen to determine the reinforcement and the process is summarized in **Table 4**.

Table 4. Temperature and Shrinkage Slab Reinforcement (8.6.1.1)

#5 Bar Reinforcement Area (in2)	0.31
Area to Examine, A_g (in^2)	2160
Minimum Reinforcement Factor	0.0018
Required Reinforcement, As,min (in2)	3.89
Rows of Reinforcement (through depth)	3
Reinforcement Spacing (in)	12
Critical Area Reinforcement, As (in^2)	**4.65**
$A_s > A_{s,min}$?	**YES**

Reinforcement size was chosen as #5, which has a cross-section of 5/8". The minimum reinforcement factor of 0.0018 was multiplied by the gross cross-sectional area to find the area of steel needed. An excel sheet was established to iterate various combinations of number of rows and spacing of the reinforcement, from which a conservative option was chosen. The selected slab reinforcement specifies (3) evenly spaced rows of #5 bars @ 12" O.C., each way.

Additional Design Considerations

Before finalizing the mat slab design, an additional flexural analysis is necessary. The column load results in a concentrated tension force near the top of the slab and the reinforcement needs to be checked in accordance with ACI 318-19 Section 22.3. While it is possible the temperature and shrinkage reinforcement present in the top of the slab would satisfy the required flexural strength, it should be checked and adjusted, if needed, prior to constructing the slab. This step was unable to be completed prior to the submission deadline of this report.

Lateral Force-Resisting System (LFRS) Design

The main steps for LFRS design are: analyzing lateral loads, determining brace demands, and sizing the brace members.

Assumptions

Wind load. In wind load analysis, because the building is fully enclosed, the internal pressure evaluation is negligible.

Braced frame location. For braced frame location, going to the level of calculating rigidity of frame, and portion of shear to frame considered to be beyond our scope. The braced locations will be determined based on mentor reference and architectural design.

Brace frame design. There are four braced frames in N-S direction and fours in E-W direction with different width length. Due to the limited time in this project, the brace frame design will be determined using the largest bay in both directions and assumed the others are approximately equal.

Connection. Connection design is not within the scope of this project. Therefore, connection will be determined based on AISC example reference. The connection consists of a 5/8” gusset plate with a weld length of 16 inches. Assume the capacity of gusset plate and welds are adequate.

Story drift. Assume the SCBF system is stiff at the connection point. Hence, the second order effects are negligible.

Lateral Load Analysis

There are two main lateral load considerations: wind load and seismic load.

Wind Load

To investigate wind load, first, refer to ASCE 7-16 table 27.2-1 in chapter 27 Wind Load Analysis. The wind load analysis process is conducted through seven steps and using the main wind force resisting system directional procedure. **Figure 16** provides a summary of the wind pressure process.

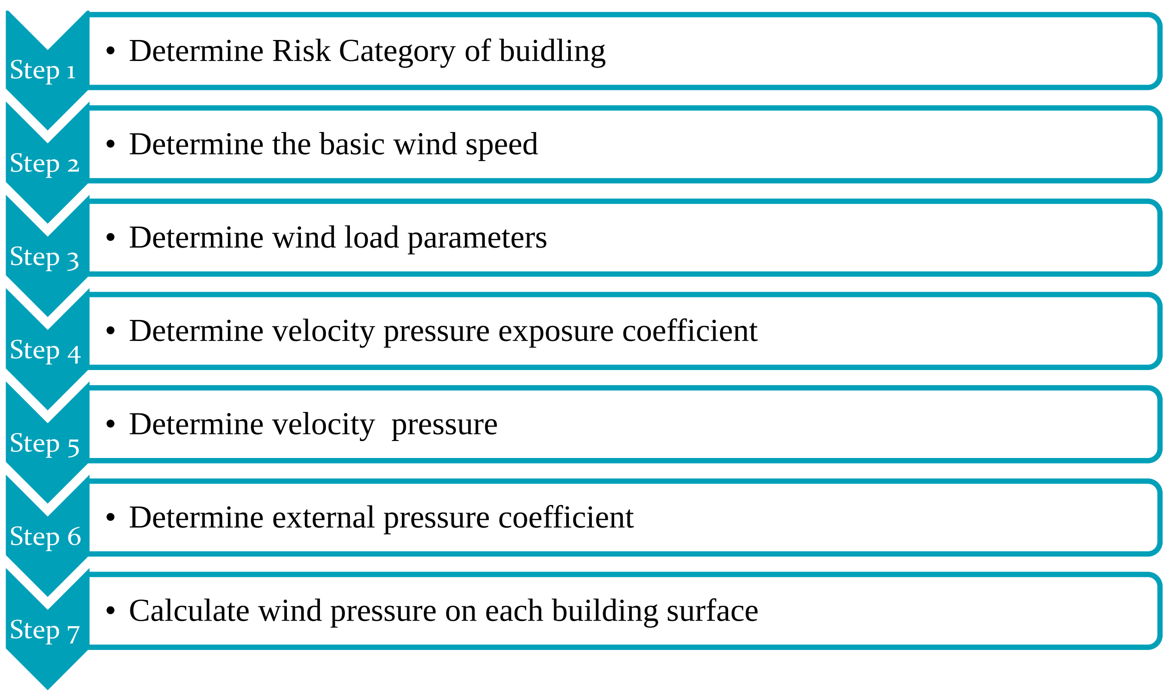

Figure 16. Wind Pressure Calculation Steps

Wind pressure, then, converted into lateral forces by multiplying by the tributary area of each floor. Note that refer to the appendix section of this report for detailed calculations. **Table 5** provides the wind load values at each level of the building.

Table 5. Wind Force Values

Floor	E-W Wind Forces (kips)	N-S Wind Forces (kips)
1st	1.11	1.41
2nd	1.56	1.22
3rd	0.84	0.66

Seismic Load

Using the provided geotechnical report, the site class is determined to be Site Class D. The area has a short period S_S = 0.887 and one-second period S_1= 0.396. The seismic Importance Factor is 1.0, that of the veterinary hospital mixed occupancy (Risk Category II).

Refer to ASCE 7-16 chapter 11-12 to determine the seismic load using equivalent lateral force method. The dead, partitions, mechanical, and exterior wall loads are used to calculate an effective seismic weight and the shear at the base of the building. Then, the lateral loads for each level of the building were estimated based on seismic activity at the specific locations, soil type, the height of the floor or roof above ground, the mass at each level, and the fundamental period of the building. **Table 6** provides the summary of seismic load calculation.

Table 6. Seismic Force Values

Floor	E-W Seismic Forces (kips)	N-S Seismic Forces (kips)
1st	65.49	65.49
2nd	51.13	51.13
3rd	25.57	25.57

Brace Demands

To produce the brace demand, the initial step is evaluating the lateral forces and establishing the governing force for the braced frame system. Using the wind and seismic values from the previous step, the governing force is seismic load. The brace frame demands then are analyzed using method of section. The frame members are cut at each level, which reveal the brace member internal forces; then solve equilibrium equations of governing loads and internal forces to produce the axial demand in brace member. (Liu, 2021). Table 7 shows the value of brace member demands for each story.

Table 7. Brace Demands. The load for E-W and N-S braces at each level, unit in kips

Location	E-W Frame Brace Demand (Kips)	N-S Frame Brace Demand (Kips)
1st story	±116.31	±101.47
2nd story	±95.40	±83.22
3rd story	±53.57	±46.73

Brace Frame Design

SCBFs system design for tension and compression. There are four criteria needed to be considered during the design process: slenderness limitation, compression strength, tensile yielding, and tensile rupture.

Slenderness limitation. According to AISC specification the slenderness ration (L/r) will not exceed 300" for tension member and 200" for compressive member. Using this ratio to identify the control radius of gyration for design elements.

Compression strength. Nominal compression strength can be determined using AISC manual table 4-5 for HSS round members. The selected shape should have a radius of gyration equal to or less than the control radius of gyration. The nominal compressive strength will be equal or greater than the demand values calculated in the previous step.

Tensile Yield. After selecting the achievable compressive member shape, the tensile yield of the same shape can be found in AISC manual table 5-6. The tensile yield strength will be greater than or equal to the tensile demand.

Tensile Rupture. Tensile rupture strength can be determined using AISC manual table 5-6 with assumption that the effective net area is 75% of the gross area. The second method is using calculated effective area and shear lag factor, *U* (AISC manual D3.1 table case 5), to produce the nominal rupture strength.

Note that refer to appendix section for detailed calculations.

Summary of Design

The structural goals are developing a solid framework that is safe, resilient, and sustainable. The brace frame central focuses are to withstand gravity, wind, and seismic loads. By focusing on these primary concerns the structural team aims to create a design with a long-life span that promotes safety and minimal environmental impact. Figure 17 indicates plan view of the building and the location of proposal brace frame.

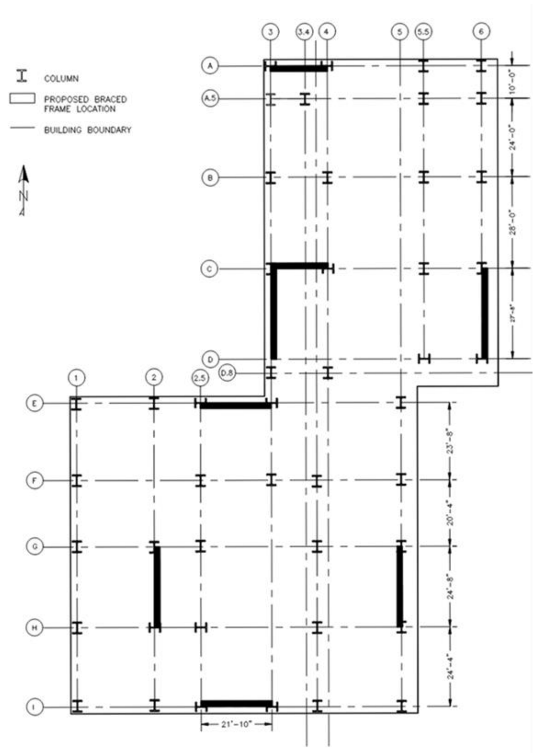

Figure 17. Brace Frame Locations

The proposed brace frame is SCBFs using steel material which functions effectively and safely against high seismic events. The brace frame configuration and shape are multistory X with HSS round because of their economy and efficiency. Brace frame sizing was selected based on their available strength in compression and tensile, and the lightest weight. In short, the proposed brace frame design has achieved the goals of safety, sustainability, and economical requirements. Table 8 provides the presentation of proposed brace frame with sizing members.

Table 8. Proposed Sizing Members.

Story	**N-S**	**E-W**
1st	HSS 5.500 X 0.500	HSS 5.500 X 0.500
2nd	HSS 5.500 X 0.500	HSS 5.500 X 0.500
3rd	HSS 5.000 X 0.312	HSS 5.000 X 0.312

Beam and Girder Member Design

<u>*Steps in determining sizing of beam and girder members:*</u> Selecting a W-shape beam for a continuously braced span at the North end of the vet hospital for member length of 28'-0 1/8" connected to two columns. Loading on member converted to kips/ft, having a W_D = 1.9047 k/ft & W_L = 2.801 k/ft.

Limit State – A degree of loading or other actions imposed on a structure can result in a "limit state," where the structure's condition no longer fulfills the design criteria, such as durability, integrity, and fitness of use. It is important to account for limit states as they are conditions of potential failure, some of the limit states that are explored to determine beam sizing consider: web local buckling (WLB), lateral torsional buckling (LTB), flange local buckling (FLB), and yielding.

- AISC Manual Table 2-4, for acquiring material properties
- Chapter 2 of ASCE/SEI 7, for requiring flexure strength
    - AISC Manual Table 3-23, case 1: $M_u = \frac{w_u L^2}{8}$
    - Moment of inertia; $I_{x(req)} = \frac{5 w_L L^4}{384 E \Delta_{max}}$
    - Select W18x86, as it has moment inertia of 1530 in^4 > 1432.7 in^4 calculated from $I_{x(req)}$ of beam dimensions
- AISC Manual Table 3-2, available flexural strength: $\Phi_b M_n = \Phi_b M_{px}$ (AISC Spec. Section F1)
- Check for Nominal Flexural Strength: $M_n = M_p = F_y Z_x$
- Check limit states with AISC Table B4.1b

Serviceability **–** The conditions under which a building is still considered useful. Should these limit states be exceeded, a structure that may still be structurally sound would still be considered unfit. Serviceability limit state design of structures includes factors such as durability, overall stability, fire resistance, deflection, cracking, and excessive vibration. Deflection will be investigated within the overall design, and if the deflection exceeds the allowable value, then the structure is stiffened to meet requirements.

- Chapter 2 of ASCE/SEI 7
    - live-load deflection: $\Delta_{max} = \frac{L}{360}$

Shear – The force applied along the surface, in opposition to an offset force acting in the opposite direction. Resulting in shear strain. One part of the surface is pushed in one direction, while another part of the surface is pushed in the opposite direction. When a structural member experiences failure by shear, two parts of it are pushed in different directions. For high-rise building shear must be accounted for and always resisted.

- Calculate shear for beam dimension and constraints: $V_u = \frac{w_u l}{2}$
- AISC Table 3-2: Find shear for W18x86 must be > $V_u = \frac{w_u l}{2}$, to prove adequate

With these following steps, the 28'-0 1/8" member will be sized at <u>**W18x86,**</u> being that is has met all specifications, calculations, and limit states to be determined serviceable and fit for design.

Water Resources

Hydrologic Analysis of Proposed Roof

To begin designing the subgrade water detention systems, hydrologic studies of the proposed roof were conducted to provide an estimate of the amount of water that would need to be mitigated in design storm events. The Rational Method was used for all hydrologic studies instead of the SCS TR-55 Method, as the roof area is less than 50 acres. Additionally, the Portland Stormwater Management Manual indicates that the Rational Method is the most used analysis method for designing. To accurately use the Rational Method for analysis, the Oregon Department of Transportation Hydraulics Manual (Hydraulic Engineering) was utilized. To conduct the hydrologic studies, assumptions had to be made prior to analysis, which included estimating the longest flow path on the roof due to the disjointed geometry and idealizing a scenario that all drains fail at once to provide a rain loading value for the structural team.

At the beginning of the design process, the water resource engineers decided to use a 100-year design storm to estimate the volume of water on the roof. The 100-year design storm study yielded a runoff volume 187.97 ft^3, which translates into 11,734.97 pounds of water and a loading value of roughly 0.700 lb/sq-ft (see Appendix A-1 for calculations). In the later stages of the design process, the water resource engineers decided to conduct a 25-year design storm study. This was done to meet all code requirements for reducing the green roof design from 90% roof coverage to 50-60%. The 25-year design storm study yielded a runoff volume 144.75 ft^3 (see Figure 18), which translates into 9,036.74 pounds of water and a loading value of roughly 0.524 lb/sq-ft (see Appendix A-2 for calculations).

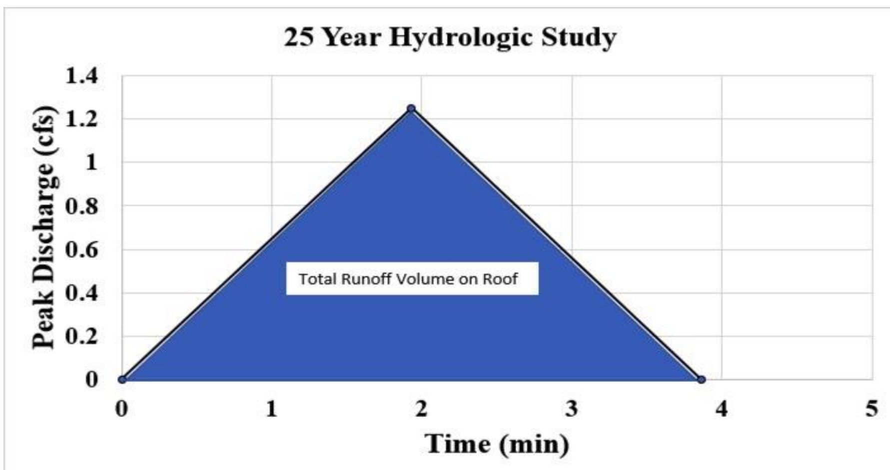

Figure 18. 25-year design storm hydrologic study, with time vs. peak discharge.
Total runoff volume on the roof is 144.75 ft3

The 25-year design storm hydrologic study was completed not only to provide an estimate of the amount of water that would need to be mitigated, but also to aid the other water resource engineer in determining the rain loading value. The volumetric result of the hydrologic study provided the other engineer with enough information to calculate the weight of saturated soil in the green roof system. This is of high importance, as the results of this engineer's work gave the structural engineering team a rain loading value that would help dictate the design of all structural components.

Subgrade Water Detention Vault Sizing

For the design alternative, determination of the dimensions for the subgrade water detention facilities will be based on analysis of the geotechnical report and results of the hydrologic studies. As of now, the design solution consists of two subgrade concrete water detention vaults. At this point through the design process, rough estimates for the dimensioning of both vaults have been completed. This was done by splitting the roof area into two separate parts. Once the roof was broken up into two areas, those areas were compared to the total roof area to be able to yield percentage values. The percentage values are important because they allow for a quick rough estimation of the dimensioning of the two-subgrade concrete water detention vaults. This was done by taking the volumetric results of the 25-year design storm hydrologic study (see Appendix A-2 for calculations) and multiplying them by the percentages obtained for the two areas of the roof (see Appendix A-3 for calculations). The values yielded from these calculations provided a volumetric size that each subgrade detention vault needed to be to successfully mitigate all potential storm water from the roof. Using practical engineering judgements, in addition to trial and error, rough estimations for the dimensioning of both vaults were obtained. The estimated values for both subgrade concrete water detention vaults are 3ft x 3ft x 7ft and 3ft x 3ft x 10ft. Though it is to note that these calculations provide a rough estimate and still need to be cross-checked against all Portland Stormwater Management codes (Environmental Services) and the City of Portland Building codes (Development Services). This will be completed as further progressions into the design process are made.

Subgrade Water Detention Vault Locations

Final location selections of the proposed design will be determined using practical engineering judgment, subsoil conditions of the site, and the property boundary lines of the project. Though final locations have not been fully determined yet, the best areas on the project site for proposed locations seem to be at the north and south fronts of the building. The two-subgrade concrete water detention vaults would be placed beneath the curbs of the north and south fronts (see Figure 19). The south front would have the 3ft x 3ft x 10ft vault, while the north front would have the 3ft x 3ft x 7ft vault. These locations seem to be the most ideal, as the project site does not offer any other usable space for the design solutions. The two vaults could be combined into one larger vault and placed beneath the structure, but there are some concerns that would need to be addressed. One of the largest concerns associated with placing it beneath the building would be maintaining the vault and or replacing it if need be. Therefore, the proposed locations are beneath the curbs of the north and south fronts of the building, as it would provide easier access for maintenance, as well as easier replacement if a vault is damaged.

Figure 19. Shown in red hatching is the proposed roof area for the building, while the blue boxes indicate all areas where the two detention vaults could be placed.

Total Load For Green Roof System

The dry load of the green roof system all depends on the choices of the design, which resulted in a load of 34 psf. These choices consist of the vegetation on the surface of the soil, the soil membrane itself and how thick it is going to be, the drainage system, and the waterproofing to prevent damage to the structure supporting the system. To fit our client's needs, the green roof design will be featuring local fauna to maximize the sustainability aspects of this system. To get an understanding of how to implement this design, inspiration was drawn typical green roof design in a profile view that is from the City of Portland's code which can be seen in Figure 20 which is below.

The total load would be made up of a dry load of the green roof, and the saturated load. To determine the level of saturation, the Stormwater Management Manual was referenced. The infiltration rate, which is stated to be 2 inches per hour, is used to find the level of saturation. This means surface water on the green roof membrane would infiltrate the soil layer till it is fully saturated or into the drainage layer of the proposed design during a long enough downpour. This fully infiltrated/saturated state results in a combined load of 57 psf. Though to determine the dry load, the design of the green roof is what determines it.

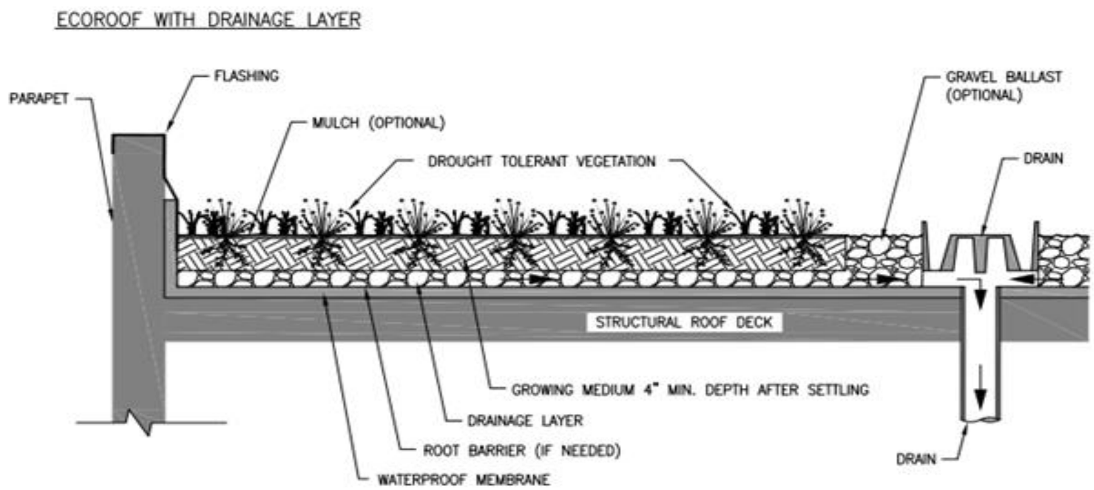

Figure 20. City of Portland's typical details STWM

Vegetation and Membrane selection process

For the green roof system to fulfill its purpose, a growing medium is needed to ensure the vegetation can thrive. The Stormwater Management Manual (add citation) requires the growing medium depth to be equivalent to the vegetation rooting depth. If a larger green roof design was chosen to fit the needs of a public access roof, raising the proposed parapet height in the provided Mackenzie drawings could lead to a larger green roof design if a public access roof was the goal, but this falls outside the scope of work. The OSD building lacks public roof access, and therefore no increase in parapet height is needed. With that knowledge stated, the process of referencing a manufacture design could begin. To select the correct type of green roof, Portland's climate had to be taken into consideration. Portland Oregon is usually a very wet city three out of the four seasons a year. Knowing this, selecting a green roof that would have a high-water retention capability would be key. Though, Oregon has dealt with some recent draughts these past few years which does affect the selection of what materials and vegetation will be used to accomplish the goal of making the green roof a sustainable design that is low maintenance.

To ensure that the green roof is able to fulfill our clients wants, the City of Portland's eco roof Plant list was referenced. Since the climate in Oregon is rainy throughout the year, but deals with a dry season, a succulent was picked to be the sedum. The two succulents for our sedum mixture would be oreganum and spathulifolium, also known as their common names of Oregon Stonecrop and Broad-leaved Stonecrop. These two selections for the green roof system's sedum seemed the most logical due to the native aspect of the species as well as being very low maintenance. The expected potential height of 4 inches will help achieve the goal of our client of providing an aesthetic view to the adjacent neighboring buildings. To continue to achieve the goal of being an aesthetic addition to the building, 2 types of herbaceous plants were added. These additions were fragaria chiloensis and fragaria virginiana, commonly known as Coastal Strawberry and Wild Strawberry. These additions also improve the ability of water infiltration that is capable of the green roof per City of Portland's Soil Specifications for stormwater systems. Since these plants will have a potential of a max of 6 inches in growth height, the minimum thickness to maintain such plants will be 6 inches per green roof technology. Knowing the vegetation selections, the soil membrane is required to meet the needs of said plants. To fit the Soil Specifications for stormwater management listed by the City of Portland, the membrane will consist of topsoil that is made of a blended material that will incorporate loamy soil, sand, and compost. The compost in this membrane must be 30 percent by volume to meet the standards of the specifications that the City of Portland provides in the previously stated documents. The next selection for the membrane after the soil layer will be the drainage layer. To make things more economical, it will be retention board with a waterproofing membrane below. This allows for the water to be retained in a fabricated board which will then drain to one of the 16 pipes located on the roof. The last parameter that has to be considered in selecting the overall design height of this green roof is the total square footage of the roof that will be vegetation covered. Since the green roof will only be 60% of the surface area, per the City of Portland's eco roof manual, the depth of the soil will have to be greater. This is due to the water runoff from gravel-based roof that is not meant to retain water like the green roof.

From these parameters, the design can be based from Green Technology's B3 green roof system. The dry load will be 34 pounds per square foot, while the rain load will be 23 pounds per square foot, which equals a saturated weight of 57 pounds per square foot. The value came out higher than expected due to going with an 8 inch depth for the soil since the green roof system will have to account for 40 percent of the roof's surface are that cannot detain water.

Stormwater Routing

One thing that has yet to be discussed in depth between the water resource engineers is how the water will be routed off the roof and down into the two separate subgrade vaults. Through preliminary discussions, the water resource engineers believe that a piping system with multiple redundancy connections would be optimal. The point of having redundancy connections is to allow multiple routes to get water off the roof if the main piping connections become damaged or clogged with debris. In the team's preliminary discussions, two separate piping systems will wrap the perimeter of the roof, with each system being tied into their respective subgrade detention vault. Each separate piping system and connection points will be tied into the designated drains for the two areas of roof. The number of drains that will be placed on the roof is still being discussed, but the water resources team is leaning towards roughly 16 drains total (see Figure 21). Similar to the piping connections, the number of drains will be more than required, providing redundancies in case of multiple drain failures. The locations of each drain are still being determined, as further City of Portland Building code research is needed so all requirements are sure to be met.

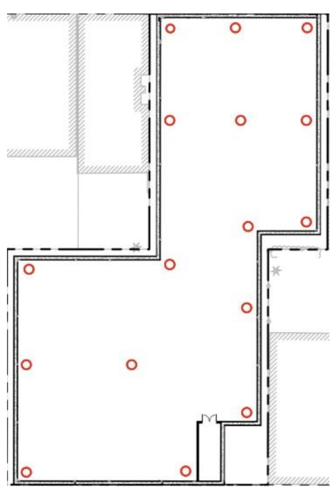

Figure 21. Shown in red circles are the proposed locations for all 16 roof drains

Description of Design (485 words, All Contributed)

This report went into thorough detail of the total scope of work and design process that team V6 underwent for the Overton Street Veterinary Hospital development. The two disciplines associated with the scope of work were water resources and structural. For the structural team's designs, three major alternatives, along with sub alternatives, were looked at.

Foundation design. One of the three designs for the structural team was the foundation design. The criteria that went into shaping the design were the load distribution of the gravity load and the fact that the ground surface could not exceed a load of 1500 psf. These criteria lead to the choice of a two-way mat slab to be used for the OSD that would be designed to meet these requirements. The sizing of the slab was based off a conservative take that the slab was unreinforced which resulted in a design that could fit the stated criteria without reinforcement in the design. When it came to the design of the stir-ups, number 8 bars were chosen at a max spacing of 9 inches off center.

Brace frame. The second of the three structural designs is the structural framework for the building. The criteria that shaped the design was withstand gravity, wind, and seismic loads. These values determined the brace frame design which resulted in a SCBFs which would be made of a steel material. The shapes are multistory X's made with HSS round due to the benefits they provide.

Girders and beams. The third structural design focused on the girders and beams of the structure. The criteria that shaped the design of the girders were the gravity, seismic, and wind loads. For the structure, the design of the beams will be primarily W21x62, while the girders will be W18x86. The flexure that the ASCE manual presented helped in the design of these members.

The two designs related to the water resources discipline were a green roof retention system and two subgrade concrete detention vaults. The data that determined the design for these two independent systems was the amount of rainfall that a 25-year design storm would bring, as well as the climate for the location of the site.

Green Roof. These criteria shaped the roof retention system into an extensive green roof, which would have a dry load of 34 psf and a saturated load of 54 psf. These calculations were done by taking the proposed membrane design and choice of materials to determine the density of the so

Subgrade Detention Basin. The subgrade detention basin features two separate vaults that have a rough dimensioning of 3ft x 3ft x 7ft for the North vault and 3ft x 3ft x 10ft for the South vault. To take the water off the roof and route it to the vaults, a piping system with redundancy connections will wrap the perimeter of the roof and tie it into the 16 roof drains.

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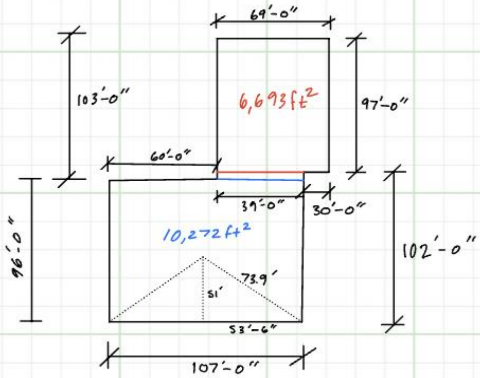
Appendix A - Calculations Package

A-1: Calculations for hydrologic study of the proposed roof using a 100-year design storm.

100 Year Design Storm Hydrologic Analysis



Total Area = 17,241 ft²

- $17{,}241\,ft^2 \times \dfrac{1\ Acre}{43{,}560\,ft^2} = 0.396$ Acres

* Because Total Area = 0.396 Acres < 50 Acres, Rational Method of analysis will be used.

* All equations and values will come from 2014 Oregon Department of Transportation's Hydraulic Manual.

- Portland, Oregon → Zone 7

$$T_{OSF}\text{ (Overland Sheet Flow)} = \frac{0.93\,(L^{0.6}\,n^{0.6})}{(i^{0.4}\,S^{0.3})}$$

$$= \frac{0.93\,(73.9^{0.6}(0.014)^{0.6})}{i^{0.4}(0.02)^{0.3}}$$

- Assuming L = 73.9', essentially longest path that water must travel to reach a drain.
- S = Slope = 2% (≈ Flat)
- n = 0.014

Will need to iterate T_{OSF} equation using Zone 7 Rainfall Intensity – Duration – Recurrence Interval curves to find "i"

Iteration:

- 1st guess T_{OSF} = 10 min
  i = 2.4 in/hr
  T_{OSF} from Calc = 2.16 min
- 2nd guess T_{OSF} = 2.16 min
  i = 3.9 in/hr
  T_{OSF} from Calc = 1.78 min
- 3rd guess T_{OSF} = 1.78 min
  i = 4.0 in/hr Answer
  T_{OSF} from Calc = 1.76 min

A-1 (cont.): Calculations for hydrologic study of the proposed roof using a 100-year design storm.

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April 8, 2022 | 100 Year Hydrologic Analysis | Water Resources Engineer

100 Year Design Storm Hydrologic Analysis Continued

$Q = C_F \times C \times i \times A$

$= 1.25 \times 0.90 \times 4.0 \times .396$

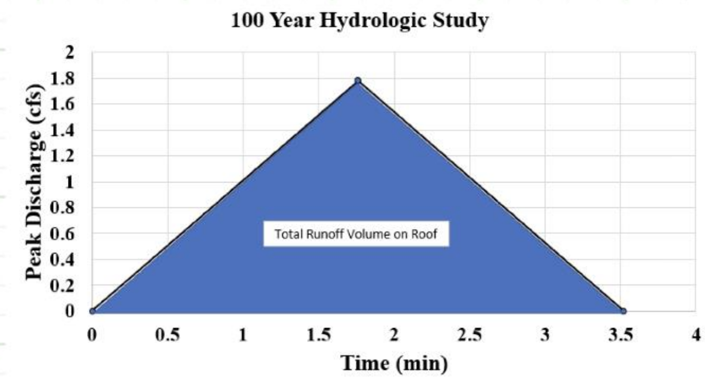
$Q = 1.78\ ft^3/s$, Peak Discharge

Where: $C = 0.90$ (Flat roof)

$C_f = 1.25$ (100 yr recurrence)

$i \approx 4.0\ in/hr$

$A = 0.396$ Acres



Total Runoff Volume $= \left(1.76\,min \times \frac{60\,s}{1\,min}\right) \times 1.78\ ft^3/s$

$\boxed{= 187.97\ ft^3}$

Weight of water $= \gamma_w V = 62.43\ lbf/ft^3 \times 187.97\ ft^3$

$\boxed{= 11{,}734.97\ lb}$

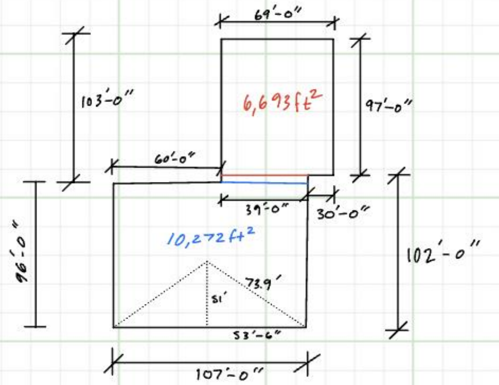
Weight of water per sqft $= \frac{11{,}734.97\ lb}{17{,}241\ ft^2}$

$\boxed{= 0.680\ lb/ft^2}$

A-2: Calculations for hydrologic study of the proposed roof using a 25-year design storm.

ARE 419 — Spring 2022	Veterinary Hospital	Nicota Liesy	1/2
April 8, 2022	25 Year Hydrologic Analysis	Water Resources Engineer	

25 Year Design Storm Hydrologic Analysis



Total Area = 17,241 ft²

- $17{,}241\,ft^2 \times \frac{1\ Acre}{43{,}560\,ft^2} = 0.396$ Acres

* Because Total Area = 0.396 Acres < 50 Acres, Rational Method of analysis will be used.

* All equations and values will come from 2014 Oregon Department of Transportation's Hydraulic Manual.

- Portland, Oregon → Zone 7

$$T_{OSF}\text{(Overland Sheet Flow)} = \frac{0.93\,(L^{0.6}\,n^{0.6})}{(i^{0.4}\,S^{0.3})}$$

$$= \frac{0.93\,(73.9^{0.6}(0.014)^{0.6})}{i^{0.4}(0.02)^{0.3}}$$

- Assuming L = 73.9', essentially longest path that water must travel to reach a drain
- S = Slope = 2% (≈ Flat)
- n = 0.014

Will need to iterate T_{OSF} equation using Zone 7 Rainfall Intensity – Duration – Recurrence Interval curves to find "i"

Iteration:

- 1st guess $T_{OSF} = 10$ min
  $i = 1.9$ in/hr
  T_{OSF} from Calc = 2.37 min

- 2nd guess $T_{OSF} = 2.37$ min
  $i = 3.1$ in/hr
  T_{OSF} from Calc = 1.95 min

- 3rd guess $T_{OSF} = 1.95$ min
  $i = 3.2$ in/hr Answer
  T_{OSF} from Calc = 1.93 min

A-2 (cont.): Calculations for hydrologic study of the proposed roof using a 25-year design storm.

ARE 419 — Spring 2022 April 8, 2022	Veterinary Hospital 25 Year Hydrologic Analysis	Nicota Liesy Water Resources Engineer	2/2

25 Year Design Storm Hydrologic Analysis Continued

$Q = C_F \times C \times i \times A$

$= 1.1 \times 0.90 \times 3.2 \times .396$

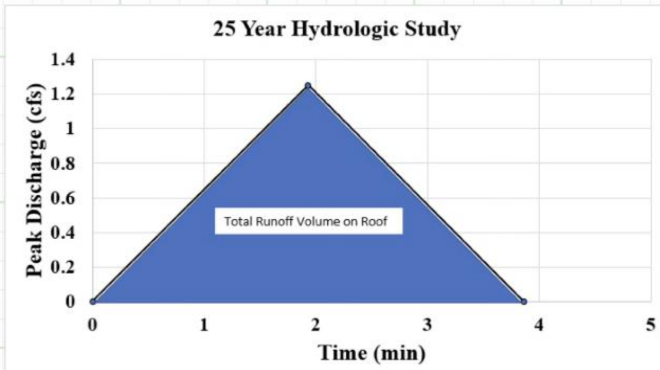
$Q = 1.25\ ft^3/s$, Peak Discharge

Where: $C = 0.90$ (Flat roof)

$C_f = 1.1$ (25 yr recurrence)

$i \approx 3.2\ in/hr$

$A = 0.396$ Acres



Total Runoff Volume $= \left(1.93\ min \times \frac{60 s}{1\ min}\right) \times 1.25\ ft^3/s$

$\boxed{= 144.75\ ft^3}$

Weight of water $= \gamma_w V = 62.43\ lbf/ft^3 \times 144.75\ ft^3$

$\boxed{= 9{,}036.74\ lb}$

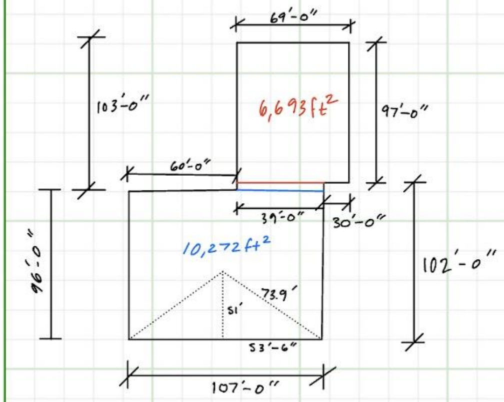
Weight of water per sqft $= \frac{9{,}036.74\ lb}{17{,}241\ ft^2}$

$\boxed{= 0.524\ lb/ft^2}$

A-3: Calculations for determining rough dimensioning of north and south subgrade water detention vaults.

ARE 419 — Spring 2022	Veterinary Hospital	Nicota Liesy	1/1
April 8, 2022	Rough Dimensioning	Water Resources Engineer	

Rough Dimensionin of both Subgrade Detention Vaults



Total Area = 17,241 ft²

North Area = Red
South Area = Blue

North Area Percentage of total Area

$$\frac{6{,}693\ ft^2}{17{,}241\ ft^2} \approx 40\%$$

Dimensioning of North Vault

$144.75\ ft^3 \times 0.40 = 57.9\ ft^3$

Sizing options

$3' \times 3' \times 7' = 63\ ft^3$ → choosing since it will be Smallest

$4' \times 3' \times 5' = 60\ ft^3$

South Area Percentage of total Area

$$\frac{10{,}272\ ft^2}{17{,}241\ ft^2} \approx 60\%$$

Dimensioning of South Vault

$144.75\ ft^3 \times 0.60 = 86.85\ ft^3$

Sizing options

$3 \times 3 \times 10 = 90\ ft^3$ → choosing since it will be Smallest

$4 \times 4 \times 6 = 96\ ft^3$

A2.1 – Summary of Load Cases

Table 9. Summary of Total Building Weight Sources

Roof

Load Case	Loading (psf)	Tributary Area (ft^2)	Load (kips)
Snow Load, S (psf)	25	15735	393.4
Roof Live Load, L_r (psf)	20	15735	314.7
Rain Load, R (psf)	13	15735	204.6
Dead Load, D (psf)	65	15735	1022.8
Mechanical Load, D (psf)	100	3933.75	393.4
Green Roof Load, D (psf)	28	11801.25	330.4

Floor 3

Load Case	Loading (psf)	Tributary Area (ft^2)	Load (kips)
Live Load, L (psf)	100	15735	1573.5
Dead Load, D (psf)	68	15735	1070.0

Floor 2

Load Case	Loading (psf)	Tributary Area (ft^2)	Load (kips)
Live Load, L (psf)	100	15735	1573.5
Dead Load, D (psf)	68	15735	1070.0

Floor 1

Load Case	Loading (psf)	Tributary Area (ft^2)	Load (kips)
Live Load, L (psf)	100	15735	1573.5
Dead Load, D (psf)	168	15735	2643.5

(30) 2' x 2' x 16.5' Concrete Columns

Load Case	Density (pcf)	Concrete Volume(ft^3)	Load (kips)
Dead Load, D (psf)	150	1980	297.0

Mat Slab (36" deep, 15,735 ft^2)

Load Case	Density (pcf)	Concrete Volume(ft^3)	Load (kips)
Dead Load, D (psf)	150	47205	7080.8

A2.2 – Summary of Total Unfactored Service Loads of the Structure

Table 10. Summary of Unfactored Service Loads

Loads	**Weight (kips)**
Dead Load (**D**)	13907.8
Live Load (**L**)	4720.5
Roof Live Load (**L_r**)	314.7

Loads	**Weight (kips)**
Earthquake (**E**)	0.0
Rain (**R**)	204.6
Snow (**S**)	393.4
Wind (**W**)	0.0
Total Unfactored Service Load	**19540.9**

A2.3 – Summary of Load Combinations and Total Factored Building Load

Table 11. Summary of Load Combinations & Total Factored Building Load (without slab)

Combo	**Factored Load (kips)**	**Equation in Use**	**Equation #**
U_1	9557.8	**U = 1.4D**	5.3.1a
U_2	*Controlling: 15941.9*	***U = 1.2D + 1.6L + 0.5(L_r or S or R)***	*5.3.1b*
U_3	13542.3	**U = 1.2D + 1.6(L_r or S or R) + (1.0L or 0.5W)**	5.3.1c
U_4	13109.6	**U = 1.2D + 1.0W + 1.0L + 0.5(L_r or S or R)**	5.3.1d
U_5	12991.6	**U = 1.2D + 1.0E + 1.0L + 0.2S**	5.3.1e
U_6	6144.3	**U = 0.9D + 1.0W**	5.3.1f
U_7	6144.3	**U = 09D + 1.0E**	5.3.1g
Controlling LRFD Factored Building Load (kips)			**15941.9**

A2.4 – Unreinforced Concrete Shear Strength Calculations

Table 12. Two-Way Unreinforced Concrete Shear Strength

Concrete Compressive Strength, f_c' (psi)	3500
Critical Perimeter, b_o (in)	240
Mat Depth, d (in)	36
Size Effect Factor, λ_s	0.659

λ (150 pcf concrete)	1.0
β (ratio of sides)	1.0
α_s (Interior Column)	40
v_c , Eq. (a) (psi)	156
v_c , Eq. (b) (psi)	234
v_c , Eq. (c) (psi)	312
Controlling v_c , Minimum (psi) =	156

v_c		
Least of (a), (b), and (c):	$4\lambda_s\lambda\sqrt{f_c'}$	(a)
	$\left(2+\frac{4}{\beta}\right)\lambda_s\lambda\sqrt{f_c'}$	(b)
	$\left(2+\frac{\alpha_s d}{b_o}\right)\lambda_s\lambda\sqrt{f_c'}$	(c)

A2.5 – Two-way Shear Strength Calculation

Table 13. Two-Way Shear Calculation Summary

Two-Way w/ Shear (22.6.6.1)	
Vu (one side of column) (kips)	170.2
vu (one side of column) (psi)	236.4
$Vu_{,max}$ = Φ6sqrt(fc') (psi) (22.6.6.3)	266.2
Vu < $Vu_{,max}$?	YES
$vc_{,max}$ = 2λsλsqrt(fc') = (psi) (22.6.6.1)	78.0
Stirrups Required?	YES
Stirrup Spacing (22.6.7.2)	
#8 Bar Reinforcement Area (in2)	0.79
Steel Yield Strength, fyt (psi)	60000
Critical Section, bo (in)	60
Critical Section Reinforcement Av (in2)	1.58
Required vs (psi)	158.4

smax (in)	9.976
Stirrup Spacing, s (in)	9
Supplied vs = Avfyt/bos (psi)	175.6

Design Shear Strength (22.6.1)	
Concrete Shear Strength, vc (psi) =	78
Steel Sheer Strength, vs (psi) =	175.6
Nominal Shear Strength, vn = vc + vs (psi)	254
Shear Reduction Factor, Φ	0.75
Design Shear Strength , Φvn (psi)	190.2
Φvn > vu ? (8.5.1.1)	YES

A2.6 – Temperature and Shrinkage Reinforcement Calculations

Table 14. Temperature and Shrinkage Reinforcement Calculation Summary

Temperature and Shrinkage Slab Reinforcement (8.6.1.1)	
#5 Bar Reinforcement Area (in2)	0.31
Area to Examine, Ag (in2)	2160
Minimum Reinforcement Factor	0.0018
Required Reinforcement, $As_{,min}$ (in2)	3.89
Rows of Reinforcement (through depth)	3
Reinforcement Spacing (in)	14
Critical Area Reinforcement, As (in2)	3.99
As > $As_{,min}$?	YES

A3.1 - Wind Load Determination

Use Directional Procedure for wind loads on Main Wind Force Resisting System (MWFRS).

<u>Step 1 through 3:</u> Define input parameters

Table W1 represents the wind parameters

Risk Category	II	ASCE 7-16, table 1.5-1
Basic Wind Speed	V = 97 mph	
Exposure Category	B	ASCE 7-16, section 26.7.3
Topographic factor	Kzt = 1.0	ASCE7-16, section 26.8.2
Gust effected factor	G = 0.85	ASCE 7-16, Section 26.11
Ground elevation factor	Ke = 1.00	ASCE 7-16, Section 26.9
Wind directionality factor	Kd = 0.85	ASCE7-16, table 26.6-1

<u>Step 4:</u> Determine velocity pressure exposure coefficient, Kz, using ASCE 7-16, table 26.10-1

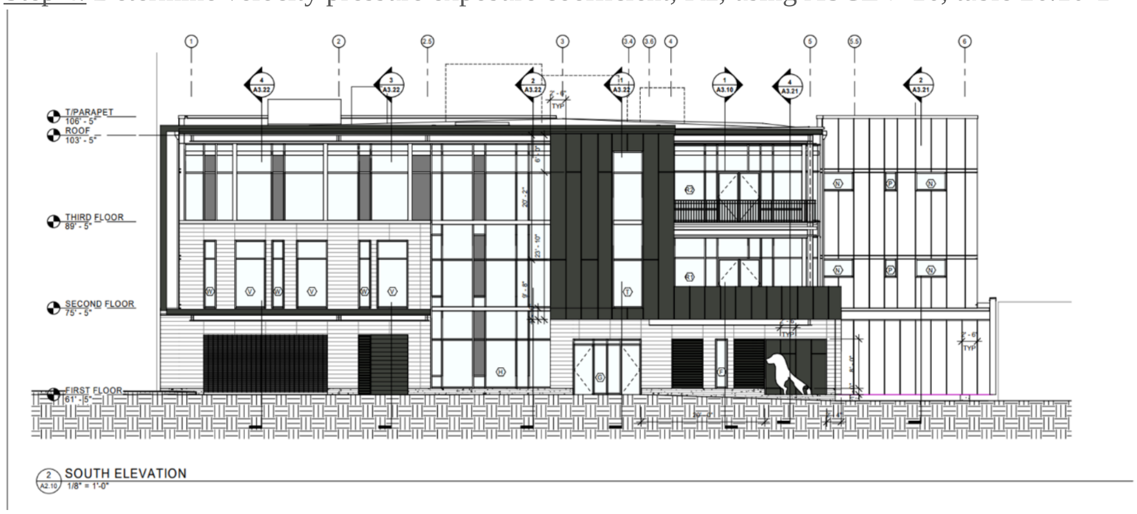


Figure W1 provides the elevations overviews of the project

Table W2 indicates the pressure exposure coefficient Kz according to story height.

Story Height (ft)	Kz
14	0.57
28	0.68
42	0.77
45 (parapet)	0.78

<u>Step 5</u>: Determine velocity pressure, qz, using ASCE 7-16 equation 26.10-1

$$q_z = 0.00256 \times K_z \times K_d \times K_e \times V^2$$

Table W3 provides the values of velocity pressure at each level in windward direction

Height (ft)	Kz	Kd	Ke	V (mph)	qz
45 (parapet)	0.78	0.85	1.0	97	15.97
42	0.77	0.85	1.0	97	15.76
28	0.68	0.85	1.0	97	13.92
14	0.57	0.85	1.0	97	11.67

Step 6: Determine external pressure coefficient, Cp, using ASCE 7-16, Figure 27.3-1

Windward Cp = 0.8 for all L/B value

With B = Horizontal dimension of building in ft, measured normal to wind direction

L = Horizontal dimension of building in ft, measured parallel to wind direction.

Leeward Cp = -0.5 conservatively for N-S wind L/B=200/132 = 1.5; for E-W wind L/B = 132/200=0.66

Step 7: Calculate the wind pressure, pz, for each building surface using ASCE 7-16 equation 27.3-1, assuming internal pressure is equal to 0.

$$p = qGC_p$$

Windward pressure

Table W4 provides the windward pressure for each level

Height (ft)	qz	Cp	G	p (psf)
42	15.76	0.8	0.85	**10.72**
28	13.92	0.8	0.85	**9.47**
14	11.67	0.8	0.85	**7.94**

Leeward Pressure

$$p = 15.76 \times 0.85 \times (-0.5) = -6.70\ (psf)$$

For parapets, the design wind pressure shall be determined by the ASCE 7-16, equation 27.3-3

$$p_p = q_p(GC_{pm})$$

With (GCpm) = combined net pressure coefficient; = +1.5 for wind ward parapet or = -1.0 for leeward parapet.

Windward pressure on parapet

$$p_p = 15.97 * 1.5 = 23.96\ (psf)$$

Leeward pressure on parapet

$$p_p = 15.97 * (-1.0) = 15.97\ (psf)$$

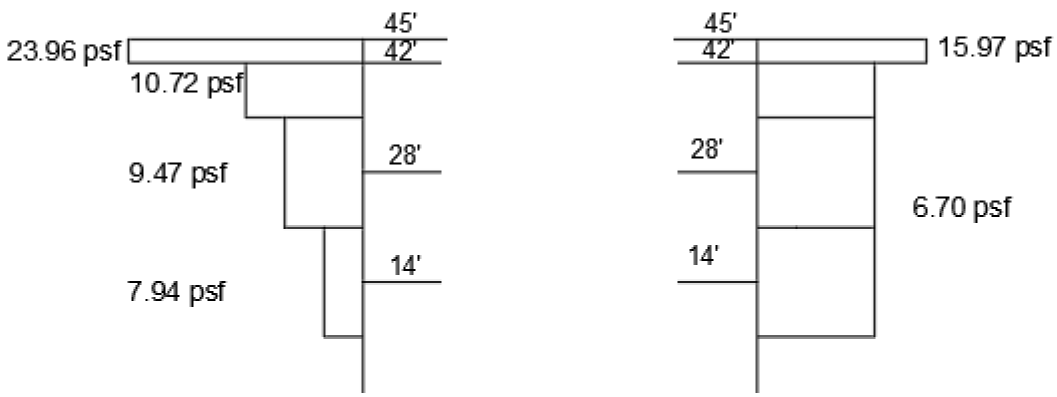


Figure W2 provides label wind pressure values at each story heights.

<u>Calculate total wind load on brace frame system</u>

Wind Load on Braced frames at top of Third Floor

	Pressure			Tributary					
Frame	Windward (psf)	Lee Ward (psf)	Total Pressure (psf) (Eq.2)	Height (ft)	Width (ft)	Area (ft^2)(Eq.1	Forces (K)(Eq.3)	No. of frame	Final Forces
E-W	10.72	6.70	17.42	7	27.5	192.50	3.35	4	0.84
N-S	10.72	6.70	17.42	7	21.625	151.38	2.64	4	0.66

Wind Load on Braced frames at Third Floor

	Pressure			Tributary					
Frame	Windward (psf)	Lee Ward (psf)	Total Pressure (psf) (Eq.2)	Height (ft)	Width (ft)	Area (ft^2)(Eq.1	Forces (K)(Eq.3)	No. of frame	Final Forces
E-W	9.47	6.70	16.17	14	27.5	385.00	6.22	4	1.56
N-S	9.47	6.70	16.17	14	21.625	302.75	4.89	4	1.22

Wind Load on Braced frames at Second Floor

	Pressure			Tributary					
Frame	Windward (psf)	Lee Ward (psf)	Total Pressure (psf) (Eq.2)	Height (ft)	Width (ft)	Area (ft^2)(Eq.1	Forces (K)(Eq.3)	No. of frame	Forces (K)
E-W	7.94	6.70	14.64	14	27.5	385.00	5.63	4	1.41
N-S	7.94	6.70	14.64	14	21.625	302.75	4.43	4	1.11

Equation 1: Tributary Area , A

$H \times W$

Equation 2: Total Wind Pressure, P

$P1 + P2$

Equation 3: Wind Load on brace frames, F

$P \times A$

Equation 4: Wind Load on one brace frames, F1

$F \div N$

A3.2 – Seismic Load Determination

Determine the seismic risk category, soil class and importance factors

- Risk Category II (provided by mentor)
- Site class D (also provided by mentor)
- Seismic importance factor: Ie = 1.0 (ASCE 7-16 section 11.5.1 and chapter 1, table 1.5-2)

Using ATC Hazards tool to identify short periods (Ss and Fa), side-modified spectral acceleration value (SMS), and design spectral response acceleration (SDS). Figure ## provides the outcomes from ATC Hazards tool.

Search by Address   Search by Coordinate

Portland, OR, USA   Search

Coordinates: 45.515232, -122.6783853

Wind   Snow   Tornado   Seismic

Basic Parameters

Name	Value	Description
S_S	0.887	MCE_R ground motion (period=0.2s)
S_1	0.396	MCE_R ground motion (period=1.0s)
S_{MS}	1.016	Site-modified spectral acceleration value
S_{M1}	* null	Site-modified spectral acceleration value
S_{DS}	0.677	Numeric seismic design value at 0.2s SA
S_{D1}	* null	Numeric seismic design value at 1.0s SA

* See Section 11.4.8

Additional Information

Name	Value	Description
SDC	* null	Seismic design category
F_a	1.145	Site amplification factor at 0.2s
F_v	* null	Site amplification factor at 1.0s
CR_S	0.889	Coefficient of risk (0.2s)
CR_1	0.87	Coefficient of risk (1.0s)
PGA	0.4	MCE_G peak ground acceleration
F_{PGA}	1.2	Site amplification factor at PGA
PGA_M	0.48	Site modified peak ground acceleration
T_L	16	Long-period transition period (s)
SsRT	0.887	Probabilistic risk-targeted ground motion (0.2s)
SsUH	0.998	Factored uniform-hazard spectral acceleration (2% probability of exceedance in 50 years)
SsD	1.5	Factored deterministic acceleration value (0.2s)
S1RT	0.396	Probabilistic risk-targeted ground motion (1.0s)
S1UH	0.455	Factored uniform-hazard spectral acceleration (2% probability of exceedance in 50 years)
S1D	0.6	Factored deterministic acceleration value (1.0s)
PGAd	0.5	Factored deterministic acceleration value (PGA)

Determine the response modification coefficient (R) ASCE 7-16 table 12.2-1

- R = 6

Determine the seismic response coefficient (CS), ASCE 7-16,12.8.1

$$C_s = \frac{S_{DS}}{\left(\frac{R}{I_e}\right)} = \frac{0.677}{6} = 0.113$$

CS constraint check:

- CS shall not be taken less than: 0.030

$$C_s = 0.044 S_{DS} I_e \geq 0.01$$

$$C_s = 0.044 \times 0.677 \times 1.0 = 0.030$$

- CS will not greater than: 0.26

$$C_{s-max} = \frac{S_{D1}}{T\left(\frac{R}{I_e}\right)}$$

  - Find approximate fundamental period, T, ASCE 7-16 equation 12.8-7

$$T_a = C_t \times h_n^x$$

    With Ct = 0.02, and x = 0.75 (ASCE 7-16 table 12.8-2), and hn = 42 ft
    Therefore Ta = T = 0.330s

  - Check upper limit on period, T

$$T = T \times C_u$$

    With Cu = 1.4 ASCE 7-16, table 12.8-1
    Therefore T = 1.4 X 0.330 = 0.46 s > 0.030 =>> T = 0.330 s OK!

Then

$$C_{s-max} = \frac{0.51}{0.330\left(\frac{6}{1}\right)} = 0.26$$

Cs = 0.113

Determine typical roof dead load

Roof Deck	3 psf	Table C3. 1-1a ASCE 7-16
Rigid Insulation	3 psf	Table C3. 1-1a ASCE 7-16
Mech./elec./piping and ceiling system	10 psf	(Liu, 2022)
Roofing	6 psf	Table C3. 1-1a ASCE 7-16
Beam	8 psf	Mentor help section
Green roof soil	41 psf	Previous calculation
Columns	3.5 psf	
Total	74.5 psf	

Determine typical floor deadload

Column Dead Load Underneath Typical Floor (LB/FT2)		PSF	
Slab (6" Light WT Concrete) (LWC Density = 96 pcf)		48	
MECH/ELEC/PIPING (Common practice = 10psf)		10	
Ceiling System (Table C3.1-1a, ASCE 7-16)		5	
Joists (Assuming: 11 LB/LF, @ 3' O.C.)		3.7	
Girders (Assuming: 85 LB/LF, @ 29'-8"O.C.)		2.9	
Columns (28'x29'-8" = 830.667FT2) Assuming: (150LB/L.F.*14')/830.667ft2		2.53	
	Total:	72.13	Column Total Dead Load - Typical Floor (PSF)

Determine the partitions loads = 20 psf (Liu, 2020)

Determine the exterior wall load = 15 (Liu, 2020)

Determine the mechanical loads = 100 psf (Liu, 2020)

Determine the effective seismic weight

Calculate Dead Loads

	Load (psf)	South Area	North Area	Floor area (ft^2)	Weight (kips)
W_{roof}	74.5	10379	7416	17795	1325.73
W_3	72.13	10379	7416	17795	1283.55
W_2	72.13	10379	7416	17795	1283.55

Calculate Partition Loads

	Load (psf)	South Area	North Area	Floor area (ft^2)	Weight (kips)
W_{roof}	N.A	N.A	N.A	N.A	N.A
W_3	20	10379	7416	17795	355.90
W_2	20	10379	7416	17795	355.90

Calculate Mechanical Loads

	Load (psf)	South Area	North Area	Floor area (ft^2)	Weight (kips)
W_{roof}	100	594.6875	594.6875	1189.4	118.9
W_3	N.A	N.A	N.A	N.A	N.A
W_2	N.A	N.A	N.A	N.A	N.A

Calculate Exteriors Wall Loads

	Load (psf)	South Area	North Area	Wall Perimeter	Tributary wall	Weight (kips)
W_{roof}	15	301	303	604	7	63.42
W_3	15	301	303	604	14	126.84
W_2	15	301	303	604	14	126.84

Weight	Dead Load	Partitions	Mech. Loa	Exterior w	Total (kips)
W_{roof}	1325.728	0	118.9375	63.42	1508.085
W_3	1283.553	355.9	0	126.84	1766.293
W_2	1283.553	355.9	0	126.84	1766.293
Total weights					5,040.67

Determine the total design lateral force at the base of the structure, ASCE 7-16 equation 12.8-1

$$V = C_s \times W = 0.113 \times 5040.67 = 568.76\,Kips$$

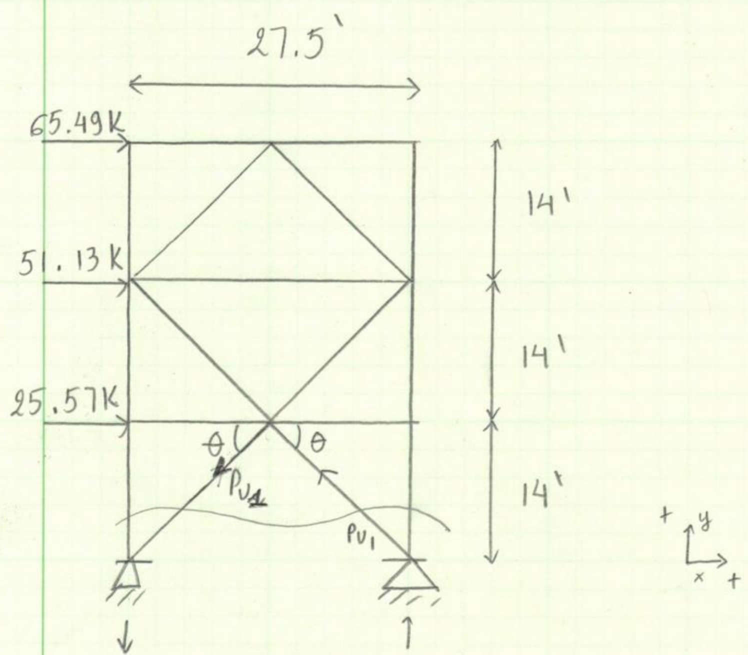
Determine the vertical distribution of seismic force

Level	W_x	H_x	$(H_x)^k$	$W_x*(H_x)^k$	C_{vx}	F_x	No. of Frame	Final Force
Roof	1508.085	42	42	63339.57	0.460571	261.95	4	65.49
3	1766.293	28	28	49456.21	0.359619	204.54	4	51.13
2	1766.293	14	14	24728.11	0.17981	102.27	4	25.57
			TOTAL	137523.9	1	568.76		

A3.3 - Brace Demand CALCULATION

N-S frame demand hand calculation.

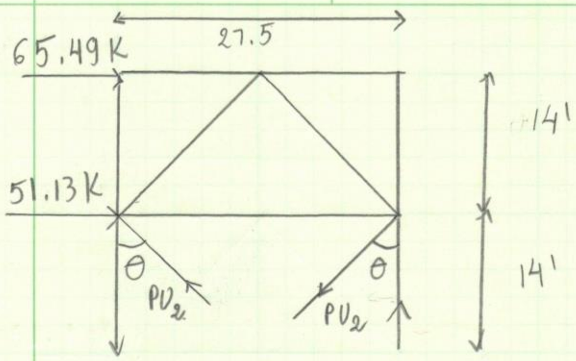
N-S Frame 1/2



$$\theta = \tan^{-1}\left(\frac{14'}{13.75'}\right) = 45.52°$$

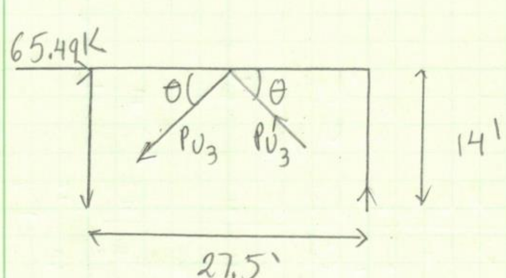
$$\Sigma F_x = 0 = 65.49 + 51.13 + 25.57 - 2 P_{u_1} \cos(45.52°)$$

$$\Rightarrow \boxed{P_{u_1} = 101.47\ K}$$



$$\Sigma F_x = 0 = 65.49 + 51.13 - 2\,PU_2 \sin\left(\tan^{-1}\left(\frac{13.75}{14}\right)\right)$$

$$\Rightarrow \boxed{Pu_2 = 83.22\ K}$$

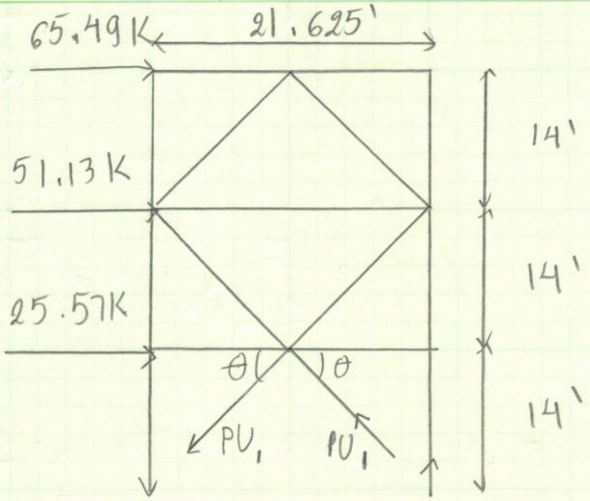


$$\Sigma F_x = 0 = 65.49 - 2PU_3 \cos(45.52°)$$

$$\Rightarrow \boxed{PU_3 = 46.73\ K}$$

E-W frame demands hand calculation

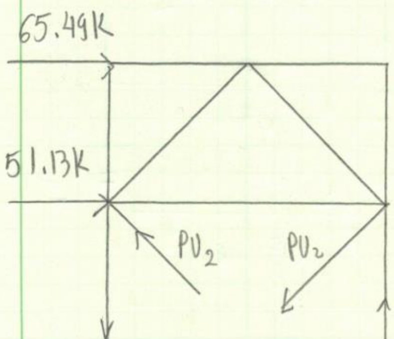
E - W Frame



$$\theta = \tan^{-1}\left(\frac{14'}{10.8125}\right) = 52.32^\circ$$

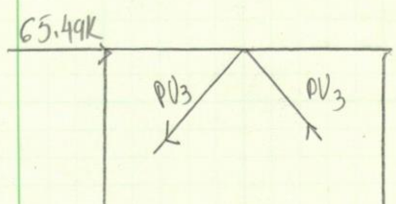
$$\Sigma F_x = 0 = 65.49 + 51.13 + 25.57 - 2PU_1 \cos(52.32^\circ)$$

$$\Rightarrow \boxed{PU_1 = 116.31\ K}$$



$$\Sigma F_x = 0 = 65.49 + 51.13 - 2PU_2 \sin\left(\tan^{-1}\left(\frac{10.8125}{14}\right.\right.$$

$$\Rightarrow \boxed{PU_2 = 95.40\ K}$$



$$\Sigma F_x = 0 = 65.49 - 2PU_3 \cos(52.3$$

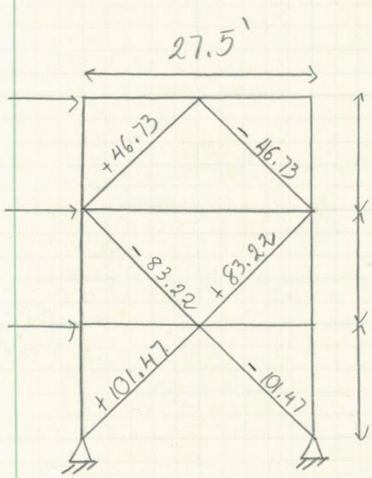
$$\Rightarrow \boxed{PU_3 = 53.57\ K}$$

A3.4 - Brace Design Calculation

N-S Braced Frame Design hand calculation

1/5

Brace Frame Design for N-S Brace



$L = \sqrt{14^2 + 13.75^2}$

$= 19.62\ ft$

- Assume capacity of gusset plate and welds are adequate
- Choose $\frac{5}{8}$" gusset plate to resist the compression load in brace, with the weld length of 16in.

Slenderness limitation:

- Tension member:

$\frac{L}{r} \leq 300" = \frac{(19.62 ft) \times \frac{12 in}{1 ft}}{r} \leq 300"$

$\Rightarrow r \geq 0.785$

- Compressive member

$\frac{L}{r} \leq 200" \Rightarrow r \geq 1.18$ (control)

Brace frame design for N-S Brace - Cont'

- First Floor:

$P_U = \pm\ 101.47\ K$

Trial shape: ROUND HSS 7.000 × 0.250

AISC Table 1-13: $F_y = 46\ Ksi$, $F_u = 62\ Ksi$

$t = 0.233$ $\quad A_g = 4.95\ in^2$

$W_t = 18.04\ lb/ft$ $\quad r' = 2.39\ in > 1.18$

+ Compression: $\phi_c P_n = 106.66\ K > 101.47\ K$

(table 4-5, AISC) $\frac{D}{t} = 30 < \lambda_r = 0.11\frac{E}{F_y} = 0.11\frac{29000}{46} = 69.34$ ⇒ No local Buckling

Width-to-thickness Ratios (Table B4.1a)

+ Tensile Yielding:

$\phi_t P_n = 205\ K > P_u = 101.47K$ OK!

+ Tensile Rupture:

$\phi_t P_u = 173K > 101.47K$ OK

. Check that $\frac{A_e}{A_g} > 0.75$

Determine U from AISC Table D3.1 - Case 5

$\ell = 16 in$

$D = 7 inch$

$\frac{\ell}{D} = \frac{16}{7} = 2.3 > 1.3$ therefore $U = 1.0$

$A_n = A_g - 2\left(\frac{5''}{8}\right) \times t = 4.95 - 2 \times \frac{5''}{8} \times 0.233$

$= 4.66\ in^2$

$A_e = A_n \times 1.0 = 4.66 in^2$

$\frac{A_e}{A_g} = \frac{4.66}{4.95} = 0.941 > 0.75$

Because AISC Mannual Table 5-6 provides an overly conservative estimate of the available tensile rupture strength, calculate P_n using AISC specification section D2

$P_n = F_u A_e = 62\,ksi \times 4.66\,in^2 = 288.92\,k$

$\phi_t P_n = 0.75 \times 288.92\,K = 217\,K > 101.47\,K$

$\Rightarrow$ $\boxed{\text{Choose ROUND HSS 7.000} \times \text{0.250 for Brace frame first floor}}$

- Second floor:

$P_U = \pm 83.22\,K$

- Trial shape: HSS ROUND HSS 5.563×0.5

$t = 0.465$ $\quad$ $r = 1.81\,in > r_{min} = 1.18''$

$W_t = 27.06\,lb/ft$ $\quad$ $A_g = 7.45\,in^2$

Compression strength

$\phi_c P_n = 99.61\,K$ (Table 4-5, AISC)

Width-to-thickness Ratios: Compression elements members subject to axial compression (B4.1a

$\frac{D}{t} = 12 < \lambda_r = 69.34 \Rightarrow O.K!$

- Tensile yielding:

$\phi_t P_u = 308\,K > 83.22\,K \Rightarrow O.K$

+ Tensile rupture:

Determine U from AISC table D3.1 – case 5

$$\frac{\ell}{D} = \frac{16\,in}{5.563\,in} = 2.88 > 1.3 \Rightarrow U = 1.0$$

$$A_n = A_g - \left(2 \times \frac{5}{8}\right) \times t = 7.45 - 2 \times \frac{5}{8} \times 0.465$$

$$= 6.87\ in^2$$

$$A_e = UA_n = 6.87\ in^2$$

$$\phi P_u = 0.75 \times F_u \times A_e$$

$$= 0.75 \times 62KSI \times 6.87 in^2$$

$$= 319.4\ K > 83.22\ K \Rightarrow O.K!$$

$\Rightarrow$ $\boxed{\text{Choose HSS ROUND } 5.563 \times 0.5 \text{ for } 2^{nd} \text{ floor}}$

• Third floor

$P_u = \pm\ 46.73\ K$

Trial shape: HSS ROUND 5.0×0.312

$t = 0.291\ in$ $\quad r = 1.67'' > r_{min} = 1.18''$

$W_t = 15.64\ \frac{lb}{ft}$ $\quad A_g = 4.30\ in^2$

Compression strength:

$$\phi P_c = 48.94\ K > 46.73\ K \Rightarrow O.K!$$

Tensile yielding:

$$\phi P_t = 178\ K > 46.73\ K \Rightarrow O.K!$$

N-S

Rupture

+ Determine U

$$\frac{\ell}{D} = \frac{16}{5} = 3.2 > 1.3 \Rightarrow U = 1.0$$

+ $A_n = 4.30 in^2 - 2 \times \frac{5}{8}" \times 0.291" = 3.94 in^2$

+ $A_e = 1.0 \times 3.94 in^2 = 3.94 in^2$

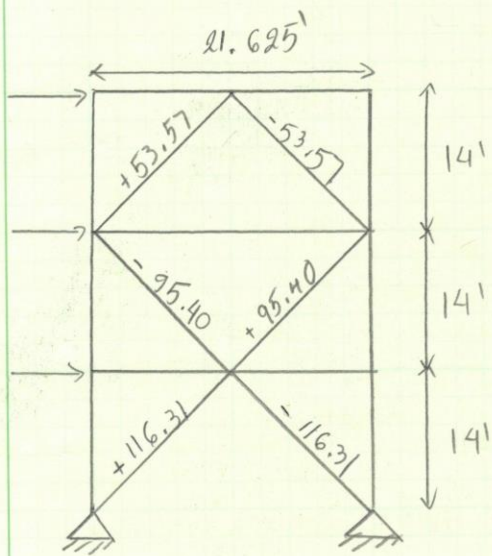
$\phi P_u = 0.75 \times 62 ksi \times 3.94 in^2 = 183.21 K > 46.7$ $\Rightarrow$ O.K

$\Rightarrow$ $\boxed{\text{Choose HSS ROUND } 5.0 \times 0.312 \text{ for third floor}}$

E-W Braced Frame Design hand calculation

E-W 112

Brace Frame Design for E-W Brace



$L = \sqrt{14^2 + \left(\frac{21.625}{2}\right)^2}$

$= 17.69\ ft$

Slenderness limitation.

. Tension member

$\frac{L}{r} \leqslant 300'' = \frac{17.69 \times 12}{r} < 300$

$\Rightarrow r \geqslant 0.71''$

Compression member

$\frac{L}{r} \leqslant 200'' \Rightarrow r \geqslant 1.06''$

$\Rightarrow$ Control

. First floor

$P_u = \pm 116.31$

Trial Shape: HSS ROUND 5.5 × 0.5

$A_g = 7.36\ in^2$ $t = 0.465\ in$

$r = 1.79\ in > 1.06\ in$ $W_t = 26.73\ lb/ft$

Compression:

$\phi P_c = 118.02 K > 116.31 K \Rightarrow O.K$

Tensile yielding:

$\phi_t P_n = 305 K > 116.31 K \Rightarrow O.K$

Rupture

$\frac{\ell}{D} = \frac{16}{5.5} = 2.9 > 1.3 \Rightarrow U = 1.0$

$Ae = An = 7.36 - 2 \times \frac{5}{8} \times 0.465 = 6.78 (in^2)$

$\phi_v P_n = 0.75 \times 62\,ksi \times 6.78\,in^2 = 315\,K > 116K \Rightarrow OK$

$\Rightarrow$ choose HSS 5.5 × 0.5 for first floor

Second floor : $P_u = \pm 95.40K$

Trial Shape : HSS ROUND 5.0 × 0.5

$\Rightarrow$ Similar with previous calculation

$\Rightarrow$ Choose HSS ROUND 5.0 × 0.5 for Second floor

Third floor : $P_u = \pm 53.57K$

Trial Shape HSS ROUND 5.0 × 0.312

$\phi_c P_n = 60.2K > 53.57K \Rightarrow OK$

$\phi_t P_n = 178K > 53.57K \Rightarrow OK$

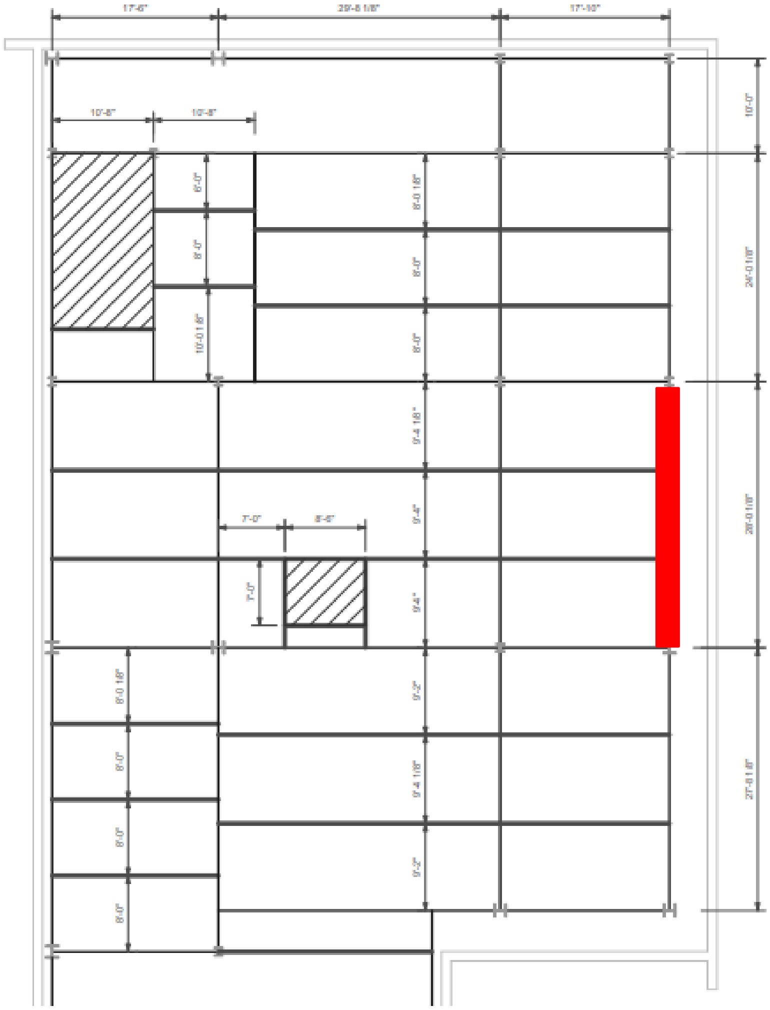
$\phi_v P_n = 150K > 53.5K \Rightarrow OK$

assume 75% effective net Area

$\boxed{\Rightarrow \text{choose HSS ROUND } 5.0 \times 0.312 \text{ for third floor}}$

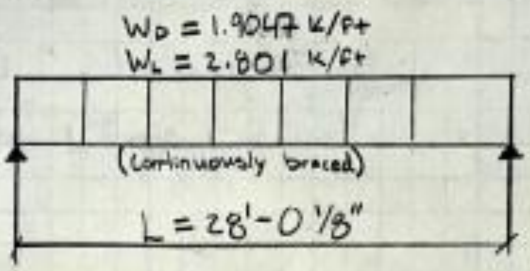
A4.1 – Bending Member Calculations

The image below displays the North-end of the veterinary hospital's 2nd-floor framing layout. The girder highlighted in red will be accounted for through a series of calculative approaches that pertains to flexure member process. This will include calculating for limit states, serviceability (deflection), shear, and other components. All calculative measures have been referenced in accordance with the AISC Manual Standards. To reduce the amount of workload necessary, the work translates for the process needed to discover all member sizes throughout the building, this is the mathematical representation of the student's understanding of the assignment



Selecting W-shape beam for continuously braced span – flexural member

Selecting W-shape beam for span and uniform dead and live loads.



- Typical floor live load = 100 psf $\rightarrow$ $100\ \text{lb/ft}^2 \times 28.01\ \text{ft} = 2801$ plf
- Dead load for Each floor = 68 psf $\rightarrow$ $68\ \text{lb/ft}^2 \times 28.01\ \text{ft} = 1904.7$ plf

$$LL = W_L = 2.801\ \text{k/ft}$$
$$DL = W_D = 1.9047\ \text{k/ft}$$

SOLUTION:

From AISC Manual Table 2-4, the material properties are as follows:

ASTM A992
$F_y = 50$ ksi
$F_u = 65$ ksi

$\rightarrow$ From Chapter 2 of ASCE/SEI 7, required flexural strength is:

* Using Load and Resistance Factor Design (LRFD)

$$W_u = 1.2(1.9047\ \text{kip/ft}) + 1.6(2.801\ \text{k/ft}) = 6.767\ \text{kip/ft}$$

From AISC Manual Table 3-23, Case 1:

$$M_u = \frac{w_u L^2}{8} \rightarrow \frac{(6.767\ \text{kip/ft})(28.01\ \text{ft})^2}{8}$$

$$M_u = 663.6\ \text{kip-ft}$$

Required Moment of Inertia for Live-Load Deflection Criterion of L/360

$$\Delta_{max} = \frac{L}{360} \Rightarrow \frac{(28.01\ \text{ft})(12\ \text{in/ft})}{360} = 0.9337\ \text{in}$$

$$I_{x\,(reqd)} = \frac{5 w_L L^4}{384 E \Delta_{max}} \Rightarrow \frac{5(2.801\ \text{k/ft})(28.01\ \text{ft})^4(12\ \text{in/ft})^3}{384(29{,}000\ \text{ksi})(0.9337\ \text{in})}$$
$$= 1432.7\ \text{in}^4$$

→ Beam Selection

Select a W18×86 from AISC Manual Table 3-3

$I_x = 1530 \text{ in}^4 > 1432.7 \text{ in}^4$ **O.K.** ✓

→ From AISC Manual Table 3-2, available flexural strength: (LRFD)

$\phi_b M_n = \phi_b M_{px}$

$= 698 \text{ kip-ft} > 663.6 \text{ kip-ft}$ **O.K.** ✓

Verifying the available flexural strength of ASTM A992 W18×86 beam by directly applying the requirements of AISC Specifications:

- W18×86 → $Z_x = 186 \text{ in}^3$
  AISC Manual Table 1-1

Nominal Flexural Strength:

- $M_n = M_p = F_y Z_x$
  $= (50 \text{ ksi})(186 \text{ in}^3)$
  $= 9300 \text{ kip-in, or } 775 \text{ kip-ft}$

Available Flexural Strength: (AISC Spec. Section F1)

- $\phi_b = 0.90$
- $\phi_b M_n = 0.90(775 \text{ kip-ft}) = 697.5 \text{ k-ft}$
  $= 697.5 \text{ kip-ft} > 663.6 \text{ kip ft}$
  O.K. ✓

Limits Table B4.1b (W18×86)

$\lambda = b/t$	W-Shape	λ_p	λ_r
$b_f/2t_f$	Flange (Case 10)	$0.38\sqrt{\frac{E}{F_y}}$	$1.0\sqrt{\frac{E}{F_y}}$
$\frac{h}{t_w}$	Web (Case 15)	$3.76\sqrt{\frac{E}{F_y}}$	$5.70\sqrt{\frac{E}{F_y}}$

Flange: AISC Manual Table 1-1

$$\frac{b_f}{2t_f} = \frac{11.1}{2(0.770)} = 7.2 \leftarrow \lambda$$

$$\lambda_p = 0.38\sqrt{\frac{29000\text{ ksi}}{50\text{ ksi}}} = 9.15 > \lambda$$

∴ Compact flange

No FLB

Web: AISC Manual Table 1-1

$$\frac{h}{t_w} = \frac{15.125\text{in}}{0.480\text{in}} = 31 \leftarrow \lambda$$

$$\lambda_p = 3.76\sqrt{\frac{29{,}000\text{ ksi}}{50\text{ ksi}}} = 90.6$$ ∴ Compact flange

NO WLB

Shear: $V_u = \frac{w_u l}{2} = \frac{(6.767\text{ kip/ft})(28.01\text{ft})}{2}$

$V_u = 94.8\text{K}$

$V_u = 94.8\text{K} + \frac{1.2(0.067\text{ K/ft})(28.01\text{ft})}{2}$

$V_u = 95.9\text{ K}$

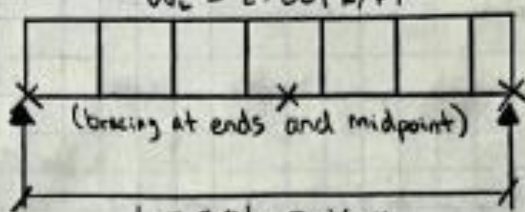
AISC Table 3-2 $\phi V_n = 265$ Kips > 95.9 K o.k.✓

*W18×86 adequate for Shear

Selecting W-shape beam bracing at ends and midpoint braced span – flexural member

Verifying the available flexural strength of the W18×86 beam size. The beam is simply supported and braced at the ends and midpoint.

$W_D = 1.9047$ K/ft
$W_L = 2.801$ K/ft



$L = 28'-0\,1/8''$

→ Required flexural strength @ midspan

(LRFD): $M_u = 663.6$ kip-ft

Unbraced length: $L_b = \dfrac{(28.01\text{ft})}{2} = 14.005$ ft

Available Flexural Strength: AISC Manual Table 3-10

(LRFD): $\phi_b M_n \approx 634$ kip-ft

From AISC Manual Table 3-2:

$\phi_b M_p = 698$ kip-ft (upper limit on $C_b\phi_b M_n$)

Adjust for C_b.

$1.30(634 \text{ kip-ft}) = 824.2$ kip-ft

Check available versus required strength

824.2 k-ft > 663.6 k-ft O.K. ✓

Calculate the adjusted required strength:

$M_u' = \dfrac{663.6 \text{ k-ft}}{1.30} = 510.5$ kip-ft

$\phi_b M_n \approx 634$ kip-ft > 510.5 kip-ft O.K. ✓

$\phi_b M_p = 698$ kip-ft > 663.6 kip-ft O.K. ✓

Verifying the available flexural strength of the W18×86 beam Selected with the beam braced at the ends and center point by directly applying the requirements of the AISC Specs.

W18 × 50

- $r_{ts} = 3.05$ in
- $S_x = 166$ in^3
- $J = 4.10$ in^4
- $h_o = 17.6$ in

→ Required flexural strength: $M_u = 663.6$ kip-ft

Nominal Flexural Strength:

Calculate C_b. Required moments for AISC Specs can be calculated as a percentage of the max midspan moment as: $M_{max} = 1.00$, $M_A = 0.438$, $M_B = 0.750$, and $M_C = 0.938$

$$\cdot\ C_b = \frac{12.5\,M_{max}}{2.5\,M_{max} + 3M_A + 4M_B + 3M_C}$$

$$C_b = 1.30$$

From AISC Manual Table 3-2:

- $L_p = 9.29$ ft
- $L_r = 28.6$ ft
- $L_b = 14.005$ ft

Calculate F_{cr}, where $c = 1.0$ for doubly symmetric I-shapes

$$\cdot\ F_{cr} = \frac{C_b \pi^2 E}{\left(\frac{L_b}{r_{ts}}\right)^2}\sqrt{1 + 0.078\frac{Jc}{S_x h_o}\left(\frac{L_b}{r_{ts}}\right)^2}$$

$$\Rightarrow \frac{1.30\pi^2(29{,}000\text{ ksi})}{\left[\frac{(14.005\text{ft})(12\text{in/ft})}{1.98\text{in}}\right]^2}\sqrt{1 + 0.078\frac{(4.1\text{in}^4)(1.0)}{(166\text{in}^3)(17.6\text{in})}\left[\frac{(14.005)(12\text{in/ft})}{3.05\text{in}}\right]^2}$$

$$F_{cr} = 59.6 \text{ ksi}$$

$M_p = 9300$ kip-in

$M_n = F_{cr} S_x \leq M_p \rightarrow 59.6\text{ksi} \times 166\text{in}^3$

$9845.98 \leq 9300$ k-in → $M_p = 9300$ k-in controls

$\phi_b = 0.9$

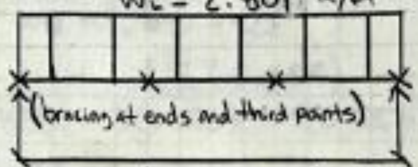
$\phi_b M_n = 0.9(775\text{ k-ft}) = 697.5$ k-ft > 663.6 kip-ft

OK. ✓

Selecting W-shape beam bracing at ends and third points braced span – flexural member

Verifying the available flexural strength of W18×86 beam. The beam is simply supported and braced at the ends and third points. ASTM A992 material

$W_D = 1.9047$ K/ft
$W_L = 2.801$ K/ft



$L = 28'\text{-}0\ 1/8''$

→ Required flexural strength @ midspan

(LRFD): $M_u = 663.6$ kip-ft

Unbraced length: $L_b = \frac{(28.01\text{ft})}{3} = 9.34$ ft

Available Flexural Strength: AISC Manual Table 3-10

(LRFD): $\phi_b M_n \approx 697$ kip-ft > 663.6 kip-ft O.K. ✓

Verifying available flexural strength of W18×86 w/ beam braced at the ends and third points:

- From AISC Manual Table 1-1; geometric properties:

W18×86
- $r_y = 2.63$ in
- $S_x = 166$ in^3
- $J = 4.10$ in^4
- $r_{ts} = 3.05$ in
- $h_o = 17.6$ in

Nominal Flexural Strength:

- Calculate C_b. For the lateral-torsional buckling limit state. The nonuniform moment modification factor (C_b). For the center segment of the beam, the required moments for AISC Specification Equation F1-1 can be calculated as a percentage of the max midspan moment as: $M_{max} = 1.00$, $M_A = 0.972$, $M_B = 1.00$ and $M_C = 0.972$

- $$C_b = \frac{12.5 M_{max}}{2.5M_{max} + 3M_A + 4M_B + 3M_C} \quad \text{(Spec. Eq. F1-1)}$$

$$C_b = \frac{12.5(1.00)}{2.5(1.00)+3(0.972)+4(1.00)+3(0.972)} = 1.01$$

- End-span beam segments, the required moments for AISC Specification Equation F1-1 can be calculated as a percentage of maximum midspan moment as: $M_{max} = 0.889$, $M_A = 0.306$, $M_B = 0.556$, and $M_C = 0.750$

$$C_b = \frac{12.5(0.889)}{2.5(0.889)+3(0.306)+4(0.556)+3(0.750)} = 1.46$$

(Spec. Eq. F1-1)

★ Thus, the centerspan, with the higher required strength and lower C_b, will govern.

→ The limiting laterally unbraced length for the limit state of yielding:

$$L_p = 1.76 r_y \sqrt{\frac{E}{F_y}} \quad \text{(Spec. Eq. F2-5)}$$

$$\Rightarrow 1.76(2.63\text{in})\sqrt{\frac{29{,}000\text{ ksi}}{50\text{ ksi}}} = \underline{\underline{111.5\text{ in or } 9.29\text{ ft}}}$$

→ The limiting unbraced length for the limit state of inelastic lateral-torsional buckling, w/ $c = 1$ from AISC Specification Equation F2-8a for doubly symmetric I-shaped members:

$$L_r = 1.95 r_{ts} \frac{E}{0.7F_y}\sqrt{\frac{Jc}{S_x h_o} + \sqrt{\left(\frac{Jc}{S_x h_o}\right)^2 + 6.76\left(\frac{0.7F_y}{E}\right)^2}}$$

$$\Rightarrow 1.95(3.05\text{in})\left[\frac{29{,}000\text{ ksi}}{0.7(50\text{ ksi})}\right]\sqrt{\frac{(4.1\text{in}^4)(1.0)}{166\text{in}^3 \times 17.6\text{in}} + \sqrt{\left[\frac{(4.1\text{in}^4)(1.0)}{166\text{in}^3 \times 17.6\text{in}}\right]^2 + \longrightarrow}}$$

$$6.76\left[\frac{0.7(50\text{ ksi})}{29{,}000\text{ ksi}}\right]^2$$

$$= \underline{343\text{in or } 28.6\text{ ft}}$$

→ For a compact beam with an unbraced length of $L_p < L_b \leq L_r$, the lesser of either the flexural yielding limit state or the inelastic lateral-torsional buckling limit state controls the nominal strength.

$M_p = 9300 \text{ kip-in}$

$$M_n = C_b\left[M_p - (M_p - 0.7F_yS_x)\left(\frac{L_b - L_p}{L_r - L_p}\right)\right] \leq M_p \quad \text{(Spec. Eq. F2-2)}$$

$$= 1.01\left\{9300 \text{ kip-in} - \left[\left(9300 \text{ kip-in} - 0.7(50 \text{ ksi})(166 \text{ in}^3)\right]\right.\right.$$
$$\left.\times\left(\frac{9.34 \text{ ft} - 9.29 \text{ ft}}{28.6 \text{ ft} - 9.29 \text{ ft}}\right)\right\} \leq 9300 \text{ kip-in}$$

$$= 9383 > 9300 \text{ kip-in}$$

$$= 9300 \text{ kip-in or } 775 \text{ kip-ft} \quad \left(\text{yielding limit state controls}\right)$$

<u>Available Flexural Strength:</u>

(LRFD): $\phi_b = 0.90$

$$\phi_b M_n = 0.9(775 \text{ kip-ft}) = 697.5 \text{ kip-ft}$$

$$697.5 \text{ kip-ft} > 663.6 \text{ kip-ft} \quad \underline{\text{O.K.}} \ \checkmark$$

Table 15**.** Range of cost estimates for mass timber construction

Economic Costs of Mass Timber Construction

Building Assemblies	($US) Cost per square foot (sq. ft)
Lumber, HD system	$28 - $30
Removal of floors and roof	$5 - $8
CLT floors and roofs	$12 - $15
Framing	$8.50 – $11.25
CLT/Wood Frame Hybrid	$36.50 - $38.50
5-ply CLT/Wood Frame Hybrid	$42.50 – 45.50
Beams and Girders	$45.50 - $52.50

Table 16**.** Estimated cost values of structural steel elements

Structural Steel Cost Estimates

Item	($US) Cost – Units Vary
Structural members	$16 - $20.50 per sq. ft
Foundation	$5 - $10 per sq. ft
Construction	$5 - $10 per sq. ft
Load-bearing beam	$3 - $35 linear foot
Steel Fabrication	$1,200 - $1,500 per ton of structural steel
Labor	$6 - $10 per sq. ft

Table 17**.** Preliminary cost-range estimates of the concrete building installation (courtesy

Oregon Concrete Costs & Prices

Building Assemblies	($US) Cost per square foot (sq. ft)
Foundation Installation	$6.39 - $7.11
Slab Leveling or Mud jacking	$4.99 - $6.12
Floor Coating Application	$3.59 - $5.58
Beams and Girders	$100 - $580
Block Wall Installation	$8.40 - $10.45

Delivery	$108.57 - $121.55
Slab	$4.34 - $7.73

A5.1 – Green Roof Load Calculations

Preliminary Calc's [illegible]

- 60% of roof will be [illegible] roof
- 16 drains [illegible] plan
- Slope less than 1% [illegible]
- Clay silty soils
  dry bulk density = 85-115 lb/ft³
- extensive green roof design
  estimate 54 psf saturated, 34 dry.
- 8 inch proposed depth of soil
- 1.75 in [illegible] board
- 1/8" filter fabric
- nominal thickness = 10"
  max i = 3.5 in/[illegible]

- assume nominal weight as
- [illegible] mixture of silty / loamy soil
- density of fabric and boards
  = negligible / [illegible] 2 psf dry
- Soil mix [illegible] loamy and silty
  = 74 pcf bulk / saturated

75.5 lb/ft³ · 0.75 ft
= 56.625 psf ⇒ go
with design based weight
56.625 ⇒ $\boxed{\text{57 psf saturated}}$

Acre = 0.396 ac
Square footage = 17249.9 ft²
green roof = 0.6 (17249.9 ft²)
gr = 10349.88 ft²

C = 0.9 → slope negligible / [illegible]
C_f = 1.1 → 25 year
i = 3.2 in/hr A = 0.396 ac

Q = C_f × C × i × A
= 1.1 × 0.9 × 3.2 × 0.396
= Q = 1.25 ft³/s
⇒ total runoff volume →
→ 1.93 × 60.5 × 1.25
⇒ $\boxed{144.75 \text{ ft}^3 = \text{total [illegible] volume}}$

Weight of water:
⇒ 62.4 lb/ft³ × 144.75 ft³
⇒ 9032.74 lb
⇒ psf of water = 0.524 psf

- Dry unit density
  ⇒ 45 pcf for [illegible] mulch mix
  ⇒ 45 lb/ft³ · .75 ft = 33.75 psf
  ⇒ $\boxed{\text{34 psf dry load}}$