THE UNIVERSITY OF IOWA
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FINAL DELIVERABLE

Title	Lansing Old Stone School
Completed By	Chad Johnson, Ben Amelon, Devon Liebe, Yusef Igram
Date Completed	December 2022
UI Department	Civil and Environmental Engineering
Course Name	CEE: 4850:0001 Project Design & Management
Instructor	Paul Hanley and Richard Fosse
Community Partners	City of Lansing

This project was supported by the Iowa Initiative for Sustainable Communities (IISC), a community engagement program at the University of Iowa. IISC partners with rural and urban communities across the state to develop projects that university students and IISC pursues a dual mission of enhancing quality of life in Iowa while transforming teaching and learning at the University of Iowa.

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Lansing Old Stone School Renovation and Site Development

CEE:4850 – Project Design & Management

Ben Amelon, Chad Johnson, Devon Liebe, Yusef Igram

12-9-22

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Section I: Executive Summary

This proposal is written to present the design services for the restoration and renovation of the Old Stone School (OSS) in Lansing, Iowa. The building has been designed as a mixed-use building. A team of four civil engineering students from the University of Iowa prepared this design:
Chad Johnson, Ben Amelon, Devon Liebe, and Yusef Igram. They used their combined experiences from different internships and classes at the University to prepare this proposal, drawings, and presentations.

The OSS was built in 1864 and has had many different uses throughout its history, most recently as a school. It has been unoccupied since 1972. There is a lot of historical value behind this building, not only in the city of Lansing, Iowa, but throughout Allamakee County. The building is on the National Historic Registry, which means the exterior of the building must remain as it is. The figure below shows the current state of the building.

Figure 1: Existing structure pictured during site evaluation.

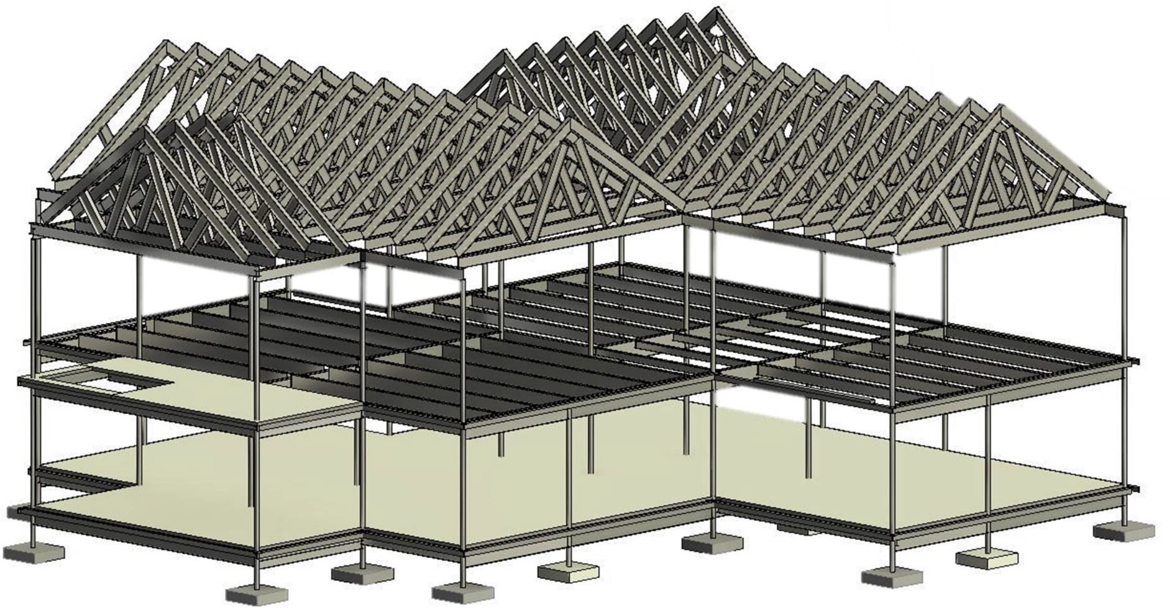
The scope of the work to be completed consists of structural evaluation of the existing structure, engineering design of structural improvements, architectural layout of the proposed uses, engineering design of the site, and cost estimation.

The architectural layout of the building consists of a 2-story building. The proposed first-floor layout consists of a City Hall section used for offices for the City of Lansing; a police station; a council chamber for city council meetings; and a community center that can be used as a multi-purpose space.The second-floor layout is a residential area that consists of 2 two bedrooms, 1 one bedroom, and 1 studio apartment. Each floor has on average 4300 square feet of usable space, which allows for many different uses. The building consists of 2 different sets of stairs to allow multiple access points to the second floor.

To accommodate occupancy, parking has been provided along the east and west sides of the building. Along the east face of the property, a limestone retaining wall is designed to accommodate street parking. Additional sidewalk paths were designed to provide ADA accessibility to the building.

Very little information exists about the construction of the building. Therefore, the least risky and most durable option for the building is to replace the existing interior structure with a steel skeleton. Additionally, the foundation of the building supporting the existing bearing walls will need to be replaced. This can be done by temporarily supporting the building while constructing new foundation walls to support the walls. Furthermore, the existing limestone walls will be connected to the new steel structure via an angle and epoxy bolt system.

Figure 2: Steel skeleton structural improvements.



The overall cost of this structural and site renovation is estimated to be $2,862,000.00. This cost includes four phases of building construction and one phase of site work. In addition to these phases, a $300,000 contingency was also included in this cost estimate.

<u>Section II:</u> Organization Qualifications and Experience

Organizational Location and Contact information
Department of Civil and Environmental Engineering
4105 Seamans Center for the Engineering Arts and Sciences
Iowa City, Iowa, 52242

Organization and Design Team Description
Our team consists of four senior civil engineering students at the University of Iowa enrolled in CEE:4850 Project Design & Management: Ben Amelon, Chad Johnson, Yusef Igram, and Devon Liebe. Chad Johnson is the acting project manager and specializes in structural engineering. Ben is the team's technical services manager and is pursuing a minor in business administration. Yusef leads graphics report production and specializes in general practice. Devon leads text report production and specializes in structural engineering.

Ben has worked in the land development field since 2018 via internships. During his work in the private sector, he prepared construction sheets for residential and commercial developments. He is proficient in all aspects of Civil3D, specializing in stormwater management and site grading. Ben led the design of the site plan and the architectural layout/programming of the building.

Chad has worked predominately in the field ensuring proper construction documentation as well as construction inspection. He has also worked on an estimating team preparing construction cost estimates for owners and architects. Chad led the structural evaluation of the existing structure and design of the structural skeleton.

Yusef has worked in both public and private sector spending a summer with the City of Cedar Rapids working with multiple professional engineers and dealing with design issues, derecho recovery, and working with homeowners. Yusef also worked with a large general contractor and was on the project management team of the construction of a data center. Yusef led coordination of the deliverables to the client.

Devon has worked at a structural engineering firm for 4 months for an internship. He is proficient in Revit, specializing in structural engineering. Devon has also taken design of wood, steel and concrete structures that have examples and small projects in those areas respectively. Devon led the project deliverables of the architectural and structural design.

In addition to work experience, team members have completed relevant coursework necessary for this project. These classes include but are not limited to, Principles of Structural Engineering, Design of Concrete Structures, Design of Wood Structures, and Design of Steel Structures, Design of Water Resources, and Civil Engineering Tools.

<u>Section III:</u> Design Services

Project Scope

The team was tasked with the design services for the restoration and renovation of the Lansing Old Stone School (OSS). It was our job as a team to perform a structural analysis of the building to determine its current structural health and we offered possible solutions with problems being found. After we confirmed the current state of the building, the group drew up interior design for modern use. Such plans included a new city hall/police station/community center. Another possible option was a potential apartment complex. After these plans were created the client selected an option, and the team performed the necessary final design for the preferred option. This included drawing up a new interior layout for the building and performing the necessary structural changes needed to comply with the updated building codes. After the structural analysis was completed, we concluded that there were concerns that relate to the age of structure and uncertainty behind certain walls and the foundation. The group and client decided on demolition of the interior and creation of an interior steel skeleton as the best solution. From the RFP to now, our scope of work has changed with the team also doing architectural work. The team has also done walk throughs with explanations for the client by the means of presentation versus our initial plan of proving a basic idea of what building could be used for rather than the extensive amount of work we did.

Work Plan

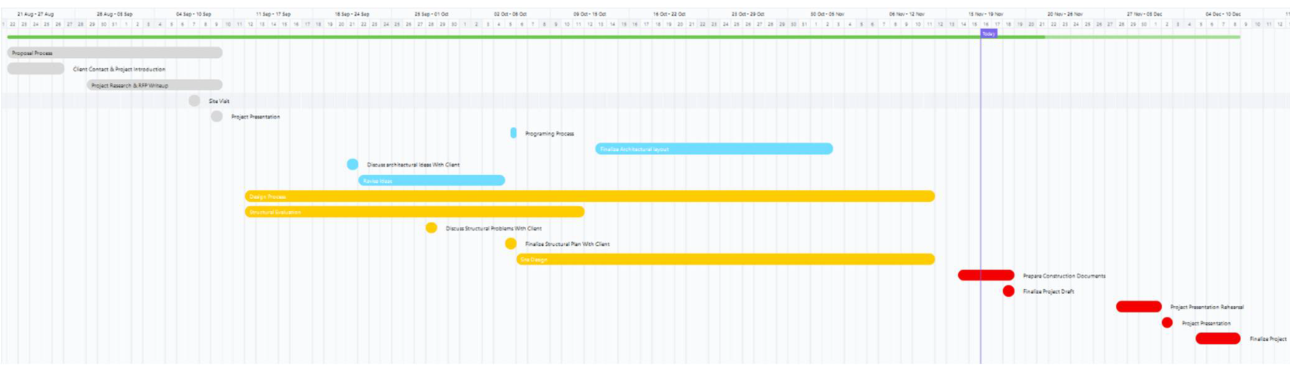


Figure 3: Work schedule for project lifetime. (Readable version in Appendix J)

Section IV: Constraints, Challenges, and Impacts

Constraints

The size of the building and scope of the work will likely result in an expensive renovation, and the project may need to be phased in a way that makes the necessary improvements to the building a possibility for the community over several years as opposed to one single project. This is a constraint specific to the timeline of work. Secondly, as the OSS is certified on the National Registry any design alternatives or upgrades cannot impact its appearance. The building has a very simple architectural layout, and not many changes can be made as most interior walls appear to be load-bearing. If walls need to be moved or removed, expensive structural members (steel) will need to be utilized to meet loading requirements. Because the final design includes a police station, this necessitates that the building is designed to a higher ASCE risk category 4. Risk category 4 is the highest level of the risk categories, meaning it must meet very high and often expensive standards for disaster preparedness. The structure must be designed to remain in operation under any circumstances 24/7.

Challenges

Due to the age of the building, there are likely issues that will be discovered during the construction process. Bids for the construction of this project will include a high contingency cost to account for this uncertainty. During the site inspection, asbestos was discovered in the crawl space on some mechanical piping. Once the demolition process begins, the construction team should be prepared for asbestos abatement. Additionally, with the structural analysis the building requires significant updates, this could create a financial challenge to the community. As stated, price is also a challenge to this project. There should be opportunities for grants and other government funding (see cost analysis). The goal is to make sure any design and recommendation is cost effective. Secondly, the building has not been maintained properly. Upon inspection the team discovered large quantities of animal feces, warped wood panels, and other messes. This will likely require the building to be cleaned and gutted before construction occurs. The parking initially provided does not meet Iowa SUDAS requirements for the planned future use. The addition and change of parking spaces requires a retaining wall to help enforce the foundation of the building and soil around the building. The current layout of the site does not meet ADA requirements. It will be a challenge to make sure the building is up to these standards; adding ramps and multiple access points will resolve this issue.

Societal Impact within the Community and/or State of Iowa

Lansing, Iowa, currently has a population of 962, which has decreased from 1,000 since 2010. This population has decreased linearly since 2010. Lansing is also a majority white population with 96.75% white. The rest of Lansing demographics can be shown on the figure below.

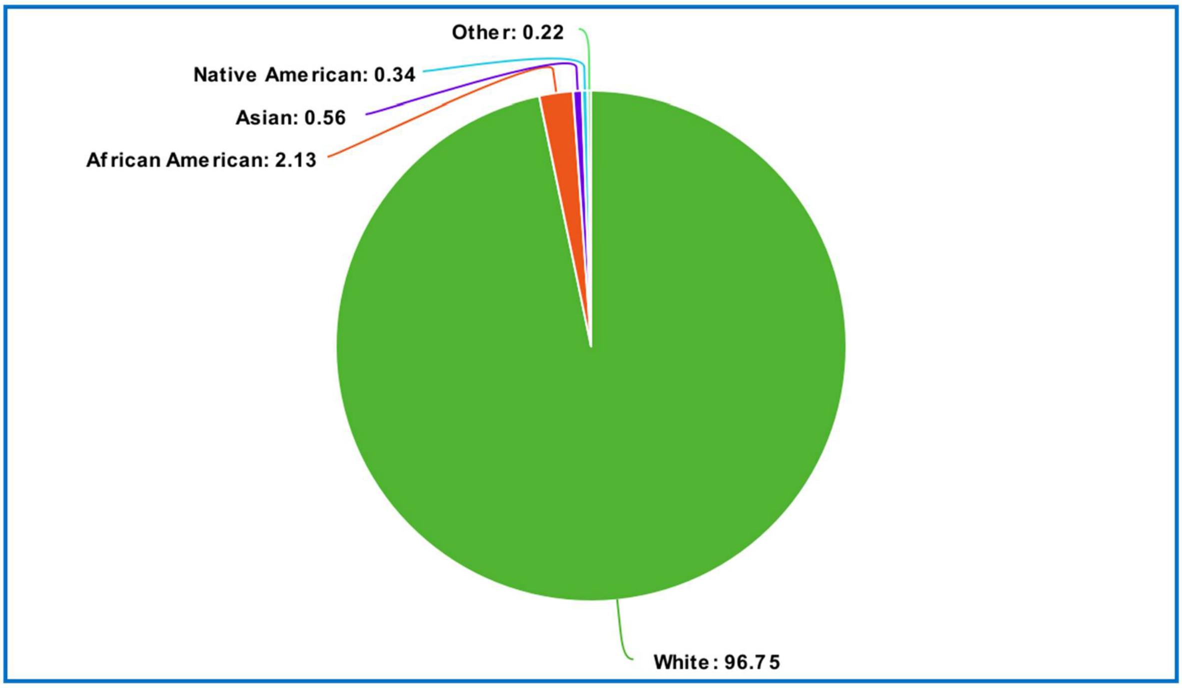


Figure 4: Lansing citizen demographics

Around 68% of families own the houses they live in and the remaining 32% rent. Homes in Lansing are typically heated with gas and a small minority uses electricity. Under 5% of Lansing's citizens have less than a high school degree, 40% have a high school degree, and 35% have more than a high school degree. The average annual earnings for an individual in Lansing is $30k, which is around $6k short of the median earnings for the nation. The restoration of this building would have a huge impact on this small community. The OSS is an important visual within Lansing. It is the longest standing school building west of the Mississippi. Restoring the school would give the community new opportunities to meet and gather, as well as becoming a place of pride. With the OSS second floor repurposed as a residential, it will bring in new community members. The units are planned to accommodate young families. Children of the residents will enjoy short walks to their schooling. Furthermore, the city hall will allow for a larger public workforce as the city grows into its future.

Section V: Alternative Solutions That Were Considered

Commercial market space

We considered a commercial market space similar to NewBo City Market in Cedar Rapids, Iowa. A commercial market space is a great alternative when it comes to helping a city grow because it allows for many uses, including concerts and other public events, farmers markets, and even small pop-up shops. This brings people to the city and adds more economic uses. This alternative, however, isn't sufficient for the needs of Lansing. The city needs a new space for the city hall and upgrades to police station and force.

Daycare center

Another need presented by the client was for a daycare center. This idea was not selected was because placement on the second floor would introduce many difficulties, including noise and movement challenges. A requirement for a daycare center is for it to be inclusive of all bodies, and must meet ADA requirements. T he team would need to also design an elevator, and even though there is space for this, it comes as a large expense. Lastly, with a daycare center there needs to be an outside area with a playground. With the current site layout, this wouldn't be possible.

City Hall & Police Station

The alternative of a combined police station and city hall is one that was requested by the client and a priority for the team. The current city hall and police station are inadequately sized and do not allow for any expansion for employees or rooms. Allowing a bigger area for these two allows the city to provide more functions. One downside to this alternative is that OSS is too large to hold only a police station on one floor and city hall on the other. Also, inclusion of a police station requires meeting stricter and more expensive building code regulations.

Apartment Complex

Another alternative discussed is an apartment complex on one or both floors. The city has a need for affordable housing for young families. A con for this alternative is that with affordable housing when you look at the total price of the project and repurposing in the short term it isn't cost effective in terms of return on investment.

Structural remedies: Foundation Improvements

Fiber Reinforced Plastic (FRP)

The first solution discussed with the client was the use of fiber-reinforced plastic. Fiber-reinforced plastic is made with fiber glass composites that reinforce the structural member. Some of the pros to FRP includes that it provides more flexural resistance, it is relatively inexpensive, and easier to construct than alternatives. Some of the cons to FRP include, that it is not as long lasting as a full foundation replacement, and that it is short term fix versus a long term replacement.

Cast-in-place Foundation

Cast-in-place foundation was the foundation remedy that the client decided was best for them. Cast-in-place foundation is a concrete foundation in which the building is lifted, and the foundation is constructed and can be properly fixed. Some of the positives to the cast-in-place foundation are that it is more durable and very long-lasting. As stated, the foundation can be properly fixed. The difficulties of this solution are that it is much more expensive than other alternatives and that the building would require temporary shoring.

Structural remedies: Building Improvements

Refurbish existing structural walls

In terms of building improvements, refurbishing existing structural walls was an option introduced to the client. This option maintains the existing feel of the building, but it will require intensive labor to complete this process as it is a brick-by-brick refurbishment process. This option would be very expensive for the city.

Steel Skeleton Structure

The most durable option is a steel skeleton structure. This option was selected by the city for this reason. This option allows the building to keep its natural feel while ensuring the safety and longevity necessary for this building. The steel skeleton structure is done by removing most of the existing interior and rebuilding the structure with steel framing. This option is the least risky because the engineering of the building is entirely new and does not require extensive analysis of the existing building.

Cold-Roll steel wall bracing

Another alternative presented was a cold-roll steel wall bracing option. This is done by adding steel framing on the interior of the building along existing bearing walls. This option would be cheaper than the steel skeleton alternative but would not provide as long of lasting option for the city.

<u>Section VI:</u> Final Design Details

Building Information and Elevations

In the OSS, the first floor consists of 4,300 sq ft and the second floor is also 4,300 sq ft, for a total usable space of 8,600 sq ft. In terms of height, from the floor of the first floor to the ceiling it is 12 ft, and from floor of second floor to roof it is 14 ft; from bottom of roof to top of roof it is 11 ft. The total elevation of the building from first floor to top of roof is equivalent to 37 ft. The first floor will be used as a new city hall, including a council chamber, community gathering area, and a police station. The second floor will be repurposed with 4 apartments, 2 of them will be 2 bedrooms, 1 of them being a one bedroom, and the last being a studio. The 3 roomed apartments will have a usable square footage of 900 sq ft, and the studio will be roughly 600 sq ft. See sheet S301 for further detailed building elevations.

The exterior of the building is to remain the same to keep the historical integrity of the building. However, the interior of the building both architecturally and structurally is to be gutted and remade into a structural steel skeleton. The reason for this is because the foundation and bearing walls are failing and rather than fix those certain areas it is best to replace the load bearing portions. Another benefit of updating the structural elements is that the structure would be known throughout the building. The new skeleton allows for properties such as strength to be known. There were uncertainties of the load bearing capacities of the walls. Thus, many conservative assumptions were made while evaluating the current structural state. Replacing the structure with the current code allows for easier renovations in future projects.

Building Structural Elements

The building is to receive a new structural system throughout the building. This choice was made due to the uncertainty of the current structural system. Due to the addition of the police station, a design of ASCE risk category 4 is necessary for use of design loads. ASCE risk categories determine the magnitude of wind and snow loads a building must be designed to. Risk category 4 is the highest level. ASCE 7-16 and LRFD load combinations were used to determine such loads. These loads can be found in Appendix C. To support the gravity loads throughout this building, HSS columns and W shaped beams and joists were used to support the floors on level 1 and level 2. As for the roof, a cold rolled steel truss frame is to be used to keep the current slope of the roof. These trusses are then resting on more W shaped beams that are then supported by the HSS columns. These columns and beams are selected with the use of AISC Steel Construction Manual Fifteenth Edition. The floors used for this building consist of 1 ½" metal deck with 2 ½" of concrete slab on top. The W beams and HSS columns are then used to support this floor. A detailed view of this can be seen on sheets S311 and S312.

When sizing the joists, we used smaller W shapes spaced at 5’ O.C Using LRFD the design checks that were made were bending moment, shear, and deflection. When running through the different sizes we found that the deflection was the main control of how these beams were sized.

The deflection criteria that were used was L/240 for dead load and L/360 for the total load using the steel manual for the first floor the largest joist member size was a W18x35 and for the second floor a W16x31 was used. The team sized these joist members based on different tributary areas. See Appendix D for these areas. Also, to view the other joist sizes that were determined see sheets S101 and S102.

The lateral system designed in this building will follow a typical lateral load bracing system. This analysis has yet to be performed, but space has been provided in areas that are appropriate for this kind of bracing.

The beams were designed following the design of the joists. Using the reactions for the joist that were calculated we were able to determine the different point loads that the beams would be supporting. The beams and joists will be connected via a double angle bolt connection. The top of the joist will be coped for an easier connection to the beam. The largest beam was a W21x68 designed to support the largest tributary areas in the middle of the building. Each beam for the OSS was designed using Robot Structural Analysis. See appendix E for further details and calculations.

The columns were then designed based on all the reactions from the roof trusses, the three levels of beams and any joists that were framed into the columns as well. Using the reactions from all these elements we were able to determine the total load going into the columns. Once the total load was calculated using the AISC Steel Manual we were then able to size the columns for the total load based on the overall height of the column. See appendix F for full design of the columns. Also see sheet S101 for the sizes of the columns.

The floor deck and slab were designed using Vulcraft standards tables. A 1 ½" deck is a standard size for a building with this type of steel framing. The decking and concrete provide the needed structural resistance for the high live loads on the first floor. Additionally, the 1.5” deck supports a 2.5” cover slab which provides the appropriate 2-hr fire rating needed for this building.

The foundations were then designed after the calculations of the columns. Since there is no soils report, we used a rather conservative bearing pressure of 1.5 ksf in order to properly size the foundations. Then by using the total load on each column and dividing by the bearing pressure we were then able to develop a reasonable area for each of the footings. Then by taking the square root of that area we were then able to come up with a reasonable square footing for each of the columns. We have also run a calculation to design for uplift, based on the calculation it was determined that the weight coming down on the columns is enough to properly combat the uplift. Our largest square footing was determined to be an 11-foot square footing. For the rest of the footing sizes see appendix F. Further calculations will be made on the reinforcement of all foundations.

New foundation walls will need to be built to support the existing limestone walls. This will be done by temporarily supporting the building and replacing sections of the existing foundation wall. Furthermore, the limestone walls will be connected to the new steel structure via an angle and epoxy bolt system. See S501 and S502 for further details of this system.

Building Design and Layout

Because of the OSS's age, there are a lot of structural changes that need to be made, both to theinterior and exterior. The exterior walls require slight improvements and need to be cleaned. The main purpose of the slight improvements and cleaning them versus other options is to maintain the historical integrity of the building. The OSS is recognized on the National Registry of Historic Places, which requires maintaining a structure's outside appearance. Some of the structural improvements that will be made include repointing the limestone and patching of the existing cracks. The layout of the first floor includes the city hall which has four separate offices for the City of Lansing: a council chamber; a police station that includes an evidence room, interrogation room, separate bathroom, four police cubicles, police chief office, weapons storage, and a receptions desk; and a community center for multi-purpose use. For more details on the building design and layout see architectural sheets A101 and A102.

Drainage analysis

The addition of parking and sidewalks creates impervious areas on site, which will affect how water runs off the site during storm events. The rational method as specified in the Iowa Stormwater Management Manual (ISWMM) was used to estimate the peak discharge during a storm event. It should be noted that all on-site areas drain to the existing inlet structure shown on sheet C105. Existing conditions produce a peak discharge of 10.56 cfs during a 10-year storm event. Proposed conditions produce a peak discharge of 11.01 cfs during a 10-year storm event. See appendix H for supporting calculations. The development produces a 4.3% increase in discharge. This increase in flow is not significant enough to warrant the addition of an inlet structure.

Addition of parking

The site plan was designed to accommodate the parking requirements of the uses of the renovated building. Currently, the building has eight street parking spots. The site plan features 20 regular-sized stalls with three ADA accessible stalls, one being van accessible. The amount of parking required was determined by Chapter 8C of the Statewide Urban Design and Specification Manual (SUDAS). From the total spaces provided, two ADA stalls are required. Three spots were designed to accommodate all members of the community as the building will have public functions. Parking dimensions are designed to follow Chapter 8B of SUDAS. Parking blocks are specified in sheet C103 to ensure that sidewalk paths always remain accessible. The east side of the site is specified to be 45-degree angle parking. See sheet C103 for a full layout.

Pavement selection

The sidewalk entrances and stairs are specified to be 5” thick with a 6” gravel sub-base underneath. SUDAS requires a minimum of 4” of pavement, the extra depth is to ensure heavier equipment can be operated on the pavement. Hot mix asphalt (HMA) is specified for the proposed parking areas. This choice is to match the existing streets. Asphalt is also a more cost-effective option than concrete. See sheet C106 for details of pavement cross sections.

ADA accessibility

The proposed sidewalks are designed to the standards of accessible sidewalk requirements in Chapter 12A-2 of SUDAS. Sidewalk cross slopes are less than the maximum of 2% and are designed to be 1.5%. Sidewalks have a minimum width of 4’, the designed sidewalks are to be 5’ to give additional maneuverability to users. Longitudinal slopes shall not exceed 8.3%. The sidewalk along 5^{th} Street is class A and designed to match the existing grade of the street. The site is designed to provide two ADA accessible access routes. One path runs towards the handicap parking stalls on the east side of the building. An additional path is specified to connect to the sidewalk of Center Street, matching the cross slope. See sheet C104 for a grading plan of the site.

Section VII: Engineer’s Cost Estimate

The engineer’s cost estimate was calculated by gathering building and site quantities while referencing the 2022 National Construction Estimator manual. As one can see in Appendix B, each item includes material, labor, and any associated machinery costs. The building material quantities were calculated from a material takeoff generated by Autodesk Revit. The cost of this project was divided into four phases of building construction and one phase of site improvements. The first phase of the project includes all necessary work that needs to be done to prepare the building for initial construction. Asbestos was discovered during a site evaluation; additionally, the building requires an extensive amount of cleaning and interior demolition before improvements can commence. The second phase of the project includes all the critical improvements needed to be made to the building. This includes all necessary work to improve the foundation wall underneath the North wall, as well as the improvement of the building’s insulation and moisture protection. Phase two also includes all work needed to be done to prevent animals and insects from infiltrating the building. After phase two is completed the City of Lansing could elect to stop investing money into the building. The building will be in a condition that will maintain the health of the building for years to come. Phase three includes all necessary improvements of the building's structure to allow for the proposed use of a city hall and police station. This includes the transport, material, construction, and material cost of all structural members needed for phase three. Furthermore, phase four includes all the costs associated with the architectural and mechanical use of the building. These costs were calculated using lump sum estimates as well as square footage costs. The material quantities for the site improvements were calculated by areas, lengths, and quantities generated by Civil3D. The site improvements cost is inclusive of demolition, earthwork, landscaping, and paving.

In addition to these phases a $300,000 contingency was also included in this estimate. A contingency of this percentage is common on structural renovation projects like this project. Additionally, for every phase in this project a 20% markup was added for contractor overhead and profit. The final construction cost of the project is estimated to be **$2,862,000.00**.

Project Phasing and Total Cost Estimate	
Phase 1	$ 63,000
Phase 2	$ 373,000.00
Phase 3	$ 743,000.00
Phase 4	$ 1,253,000.00
Site	$ 130,000.00
Contingency	$ 300,000.00
Total	$ 2,862,000.00

Table 1: Engineer’s cost estimate for all phases of construction

For funding and tax incentives, OSS has a lot of options because of its historic distinction. The team has researched some grants and government funding that the City of Lansing can choose to use for the OSS renovation. These include but are not limited to: The Historic Preservation Fund, Save America's Treasures Grant, Kinsman Foundation Historic Preservation Grant, Semi Quincentennial Grant Program, and Housing Preservation Grants.

Appendix A: Bibliography

Design manual. Iowa Statewide Urban Design and Specifications. (n.d.). Retrieved November 17, 2022, from https://iowasudas.org/manuals/design-manual/

American Concrete Institute. (2020). *Building code requirements for structural concrete (Aci 318-19): An Aci Standard ; commentary on building code requirements for structural concrete (Aci 318R-19)*.

International Code Council, Inc. (2020). *2021 International Code Council Performance Code for buildings and facilities*.

American Institute of Steel Construction. (2017). *Steel construction manual*.

Minimum Design Loads and Associated Criteria for Buildings and Other Structures: ASCE/SEI 7-16. American Society of Civil Engineers, 2017.

Nucor Vulcraft. *Vulcraft Steel Roof & Floor Deck Manual*, 2018.

Pray, R. (2022). *National Construction Estimator*. Craftsman Book Company.

Appendix B: Cost Estimation

Division	Section	Item	Unit	Amount	Material	Labor	Equipment	Total	Cost
1 - General Requirements	NOT APPLICABLE TO JOB								
2 - Existing Conditions									
	Gut Building								
		Commercial Building	SF	9000	$ -	$ 4.33	$ -	$ 4.33	$ 38,970.00
	Clean Building								
		Commercial Building	LS	1	$ -	$ -	$ -	$ 5,000.00	$ 5,000.00
	Asbestos Abatement								
		Asbestos Survey	HR	4	$ -	$ -	$ -	$ 91.80	$ 367.20
		Sample Analysis	EA	10	$ -	$ -	$ -	$ 56.10	$ 561.00
		Report Writing	LS	1	$ -	$ -	$ -	$ 200.00	$ 200.00
	Asbestos Removal								
		Pipe Insulation	LF	100	$ -	$ -	$ -	$ 72.30	$ 7,230.00
2 - Total Exisiting Conditions Cost									$ 52,328.20
Phase 1 Total									$ 52,328.20
								Total Phases	$ 2,025,325.03

Phase 1 Estimated Costs

Division	Section	Item	Unit	Amount	Material	Labor	Equipment	Total	Total	
1 - General Requirements	NOT APPLICABLE TO JOB									
2 - Existing Conditions										
	Temporary Structural Support									
		Masonry Structure	SF	434	$ -	$ 7.17	$ 13.60	$ 20.77	$ 9,014.18	
	Foundation and Footing Removal									
		Masonry Foundation	CY	5	$ -	$ 77.00	$ 82.50	$ 159.50	$ 797.50	
2 - Total Exisiting Conditions Cost									$ 9,811.68	
3 - Concrete										
	Excavation for Concrete Work									
		Trench 24 x 36 Depth	LF	58	$ -	$ 1.44	$ 0.66	$ 2.10	$ 121.80	
		Backfill	CY	9.7	$ -	$ 2.74	$ 1.24	$ 3.98	$ 38.47	
	Formwork for Concrete									
		Footing, 1 use	LF	20	$ 1.50	$ 0.98	$ -	$ 2.48	$ 49.60	
		Wall Forms, 1 use 4-6'	SFCA	120	$ 5.37	$ 6.83	$ -	$ 12.20	$ 1,464.00	
	Reinforcement									
		#5 Rebar	LF	280	$ 0.98	$ 0.59	$ -	$ 1.57	$ 439.60	
		Stirrups 6" x 24"	EA	15	$ 3.45	$ 0.92	$ -	$ 4.37	$ 65.55	
	Concrete									
		CIP Concrete Footing	CY	2.6	$ 133.00	$ 23.50	$ -	$ 156.50	$ 405.74	
		CIP Concrete Wall	CY	3.9	$ 133.00	$ 23.50	$ -	$ 156.50	$ 608.61	
3 - Total Concrete Cost									$ 3,193.38	
4 - Masonry										
	Stone Work									
		Limestone Replacement	CF	375	$ 42.10	$ 19.80	$ 61.90	$ 123.80	$ 46,425.00	
	Repointing Masonry									
		Repointing Stone	SF	500	$ 50.08	$ 26.59	$ -	$ 76.67	$ 38,335.00	
	Masonry Reinforcing									
		Masonry Anchors, 1/4" x 18"	EA	100	$ 6.71	$ 10.40	$ 17.11	$ 34.22	$ 3,422.00	
4 - Total Masonry Cost									$ 88,182.00	

Phase 2 Estimated Costs (Part 1)

5 - Metals									
6 - Wood and Composites									
	Structural Wood Inspection								
			LS	1	$ -	$ -	$ -	$ 10,000.00	$ 10,000.00
6 - Total Wood and Composites Cost									$ 10,000.00
7 - Thermal and Moisture Protection									
	Vents, Louvers, and Screens								
		Bird Screens With Frame	SF	963	$ 1.93	$ 1.72	$ -	$ 3.65	$ 3,513.13
		Attic Vents With Louvers	EA	5	$ 27.90	$ 17.90	$ -	$ 45.80	$ 229.00
	Exterior Wall Insulation and Finish System								
		Adhesive Mixture, 2 coats	SF	8988	$ 1.05	$ 1.10	$ -	$ 2.15	$ 19,324.20
		Glass Fiber Mesh	SF	8988	$ 0.39	$ 0.52	$ -	$ 0.91	$ 8,179.08
		Insulation Board, 1" Thick	SF	8988	$ 0.65	$ 1.10	$ -	$ 1.75	$ 15,729.00
		Textured Finish Coat	SF	8988	$ 0.83	$ 1.98	$ -	$ 2.81	$ 25,256.28
7 - Total Thermal and Moisture Protection Cost									$ 68,488.56
8 - Openings									
	Hollow Metal Door and Frame								
		Total Cost	EA	37	$ 1,552.04	$ 362.70	$ -	$ 1,914.74	$ 70,845.38
		Hardware	EA	37	$ 100.00	$ 50.00	$ -	$ 150.00	$ 5,550.00
	Windows								
		Custom Windows	EA	45	$ 973.00	$ 56.10	$ -	$ 1,029.10	$ 46,309.50
		Misc. Hardware	EA	45	$ 50.00	$ 25.00	$ -	$ 75.00	$ 3,375.00
		Glazing	SF	810	$ 2.23	$ 2.79	$ -	$ 5.02	$ 4,066.20
8 - Total Openings Cost									$ 130,146.08
Phase 2 Total									$ 309,821.70

Phase 2 Estimated Costs (Part 2)

Division	Section	Item	Unit	Amount	Material	Labor	Equipment	Total	Total
1 - General Requirements	NOT APPLICABLE TO JOB								
2 - Existing Conditions									
	Roof Demolition								
		Built-Up Roofing	SF	4500	$ -	$ 1.21	$ -	$ 1.21	$ 5,445.00
	Building Demolition								
		Interior Masonry Stcuture	SF	4500	$ -	$ 4.40	$ 1.49	$ 5.89	$ 26,505.00
	Temporary Structural Support								
		Masonry Structure	SF	4500	$ -	$ 7.17	$ 13.60	$ 20.77	$ 93,465.00
	Foundation and Footing Removal								
		Masonry Foundation	CY	11.9	$ -	$ 77.00	$ 82.50	$ 159.50	$ 1,896.28
2 - Total Exisiting Conditions Cost									$ 127,311.28
3 - Concrete									
	Excavation for Concrete Founation Work								
		Trench 24 x 36 Depth	LF	263	$ -	$ 1.44	$ 0.66	$ 2.10	$ 552.30
		Backfill	CY	43.8	$ -	$ 2.74	$ 1.24	$ 3.98	$ 174.46
	Formwork for Foundation Footing								
		Footing, 1 use	LF	321	$ 1.50	$ 0.98	$ -	$ 2.48	$ 796.08
		Wall Forms, 1 use 4-6'	SFCA	3852	$ 5.37	$ 6.83	$ -	$ 12.20	$ 46,994.40
	Reinforcement								
		#5 Rebar	LF	4494	$ 0.98	$ 0.59	$ -	$ 1.57	$ 7,055.58
		Stirrups 6" x 24"	EA	321	$ 3.45	$ 0.92	$ -	$ 4.37	$ 1,402.77
	Concrete								
		CIP Concrete Footing	CY	17.8	$ 133.00	$ 23.50	$ -	$ 156.50	$ 2,790.92
		CIP Concrete Wall	CY	35.7	$ 133.00	$ 23.50	$ -	$ 156.50	$ 5,581.83
	Elevated Slabs								
		CIP Concrete by Pump	CY	83.3	$ 145.00	$ 2.46	$ 5.69	$ 153.15	$ 12,762.50
		Reinforcement, #4 Rebar	LF	1000	$ 0.59	$ 0.39	$ -	$ 0.98	$ 980.00
	Stairs								
		Stairs Formwork	SFCA	136	$ 3.55	$ 5.85	$ -	$ 9.40	$ 1,278.40
		CIP Concrete by Pump	CY	71.3	$ 145.00	$ 49.60	$ 28.60	$ 223.20	$ 15,921.60
	Column Footings								
		Hand Dug Excavations	CY	58.3	$ -	$ 60.00	$ -	$ 60.00	$ 3,500.00
		Formwork	SFCA	1575	$ 4.61	$ 4.39	$ 9.00	$ 18.00	$ 28,350.00
		CIP Concrete Wheelbarrow	CY	52.5	$ 145.00	$ 29.20	$ 10.10	$ 184.30	$ 9,675.75
		#5 Rebar	LF	120	$ 0.98	$ 0.59	$ -	$ 1.57	$ 188.40
		Stirrups 6" x 24"	EA	120	$ 3.45	$ 0.92	$ -	$ 4.37	$ 524.40
3 - Total Concrete Cost									$ 138,529.39

Phase 3 Estimated Costs (Part 1)

4 - Masonry									
5 - Metals									
	Floor Deck System								
		18 Gauge Decking	SF	6000	$ 2.40	$ 0.71	$ 0.23	$ 3.34	$ 20,040.00
		Welded Wire Mesh	SF	6000	$ 0.29	$ 0.26	$ -	$ 0.55	$ 3,300.00
	Girder/Beam	L6x6x3/4	LF	640	28.7	9.184			
		W8X18	LF	107	18	0.963			
		W10x22	LF	107	22	1.177			
		W12x26	LF	367	26	4.771			
		W16x26	LF	534	26	6.942			
		W18X35	LF	85	35	1.4875			
		W21X44	LF	346	44	7.612			
		W21X68	LF	96	68	3.264			
		W16X31	LF	253	31	3.9215			
		W16X36	LF	383	36	6.894			
		W18X35	LF	10	35	0.175			
		TOTAL WEIGHT GIRDER	TON	46.4	$ 3,430.00	$ 681.00	$ 370.00	$ 4,481.00	$ 207,878.07
	Column								
		HSS6x6x5/16	LF	85	23.34	0.99195			
		HSS7X7X5/16	LF	142	27.59	1.95889			
		HSS8X8X1/4	LF	85	25.82	1.09735			
		HSS5X5X1/4	LF	155	15.62	1.21055			
		HSS7X7X1/4	LF	85	22.42	0.95285			
		HSS4X4X5/16	LF	28	14.83	0.20762			
		HSS9X9X1/4	LF	29	29.23	0.423835			
		TOTAL COLUMN WEIGHT	TON	6.8	$ 3,280.00	$ 619.00	$ 336.00	$ 4,235.00	$ 28,980.30
	Truss System								
		Cold Roll Steel Trusses	LS	1	$ -	$ -	$ -	$ 80,000.00	$ 80,000.00
5 - Total Metals Cost									$ 340,198.37
6 - Wood and Composites									
7 - Thermal and Moisture Protection									
	Fireproofing								
		Floor and Roof Fireproofing	LF	2897	$ 3.00	$ 1.50	$ -	$ 4.50	$ 13,036.50
7 - Total Thermal and Moisture Protection Cost									$ 13,036.50
Phase 3 Total									$ 619,075.53

Phase 3 Estimated Costs (Part 2)

Division	Section	Item	Unit	Amount	Material	Labor	Equipment	Total	Total
1 - General Requirements	NOT APPLICABLE TO JOB								
9 - Finishes									
	Flooring								
	First Floor	Resilient Flooring	SF	4500	$ 1.65	$ 0.75	$ -	$ 2.40	$ 10,800.00
	Second Floor	Wood Flooring	SF	4500	$ 6.15	$ 5.10	$ -	$ 11.25	$ 50,625.00
	Ceiling								
	First Floor	Acoustical Ceiling	SF	4500	$ 1.04	$ 0.48	$ -	$ 1.52	$ 6,840.00
	Second Floor	Acousticcal Ceiling	SF	4500	$ 1.78	$ 0.57	$ -	$ 2.35	$ 10,575.00
	Wall Coverings								
		Painting	SF	32060	$ 1.62	$ 1.08	$ -	$ 2.70	$ 86,562.00
	Partition Wall (14')								
	First Floor		SF	5838	$ 0.43	$ 0.37	$ -	$ 0.80	$ 4,670.40
	Second Floor		SF	5698	$ 0.43	$ 0.37	$ -	$ 0.80	$ 4,558.40
9 - Total Finishes Cost									$ 174,630.80
10 - Specialties									
	Fire Extiniguisher								
		Fire Extinguisher and Cabinet	EA	10	$ 250.00	$ 100.00		$ 350.00	$ 3,500.00
	Bathroom								
		Toilet	EA	7	$ 425.00	$ 138.00	$ -	$ 563.00	$ 3,941.00
		ADA	EA	21	$ 107.00	$ 100.00	$ -	$ 207.00	$ 4,347.00
		Sink	EA	8	$ 425.00	$ 138.00	$ -	$ 563.00	$ 4,504.00
		Toilet Paper	EA	8	$ 68.70	$ 104.00	$ -	$ 172.70	$ 1,381.60
		Hand Towel	EA	8	$ 284.00	$ 100.00	$ -	$ 384.00	$ 3,072.00
		Soap	EA	8	$ 51.40	$ 100.00	$ -	$ 151.40	$ 1,211.20
		Mirror	EA	7	$ 137.00	$ 100.00	$ -	$ 237.00	$ 1,659.00
		Waste Receptacles	EA	8	$ 223.00	$ 100.00	$ -	$ 323.00	$ 2,584.00
	Identifiying Devices								
		Entire Project	LS	1	$ -	$ -	$ -	$ 10,000.00	$ 10,000.00
10 - Total Specialites Cost									$ 36,199.80
11 - Equipment									
	Kitchen								
		Kithen Equipment	LS	1	$ -	$ -	$ -	$ 23,505.00	$ 23,505.00
11 - Total Equipment Cost									$ 23,505.00

Phase 4 Estimated Costs (Part 1)

12 - Furnishings									
	Desks								
		Desk	EA	10	$ 400.00	$ 100.00	-	$ 500.00	$ 5,000.00
		Coucil Desk	EA	1	$ 5,000.00	$ 1,000.00	$ -	$ 6,000.00	$ 6,000.00
		Community Table	EA	4	$ 400.00	$ 100.00	$ -	$ 500.00	$ 2,000.00
	Chairs								
		Chair	EA	75	$ 156.00	$ 20.40	$ -	$ 176.40	$ 13,230.00
	Cabinetry								
		Counter Top	LF	60	$ 36.10	$ 8.90	$ -	$ 45.00	$ 2,700.00
		Cabinents	LF	60	$ 369.00	$ 29.90	$ -	$ 398.90	$ 23,934.00
12 - Total Furnishings Cost									$ 52,864.00
22 - Plumbing									
	Plumbing								
		Entire Project	SF	9000	$ 6.00	$ 3.00	$ -	$ 9.00	$ 81,000.00
22 - Total Plumbing Cost									$ 81,000.00
23 - HVAC									
	HVAC								
		Entire Project	SF	9000	$ 40.00	$ 20.00	$ -	$ 60.00	$ 540,000.00
23 - Total HVAC Cost									$ 540,000.00
26 - Electrical									
	Electrical								
		Entire Project	SF	9000	$ 8.00	$ 4.00	$ -	$ 12.00	$ 108,000.00
26 - Total Electricaal Cost									$ 108,000.00
27 - Communications									
	Communications								
		Entire Project	SF	9000	$ 2.10	$ 1.00	$ -	$ 3.10	$ 27,900.00
26 - Total Communications Cost									$ 27,900.00
Phase 4 Total									$ 1,044,099.60

Phase 4 Estimated Costs (Part 2)

Section	Item	Unit	Quantity	Material	Labor	Equipment	Total	Cost
Curbs	6" x 24" Straight vertical curb	LF	125.00	$ 13.10	$ 6.03	$ 1.84	$ 20.97	$ 2,621.25
	6" x 24" Curved vertical curb	LF	20.00	$ 13.20	$ 8.77	$ 2.68	$ 24.65	$ 493.00
	6" x 24" Straight rolled curb	LF	112.00	$ 16.50	$ 8.23	$ 2.52	$ 27.25	$ 3,052.00
	6" x 24" Curved rolled curb	LF	5.00	$ 16.70	$ 11.60	$ 3.55	$ 31.85	$ 159.25
Bollards	Pipe bollards, 6" diameter	Ea	3.00	$ 183.00	$ 37.30	$ 13.20	$ 233.50	$ 700.50
Pavement Striping and Marking	Parking lot spaces, single line	Ea	15.00	$ 3.39	$ 6.18	$ 1.04	$ 10.61	$ 159.15
	Handicapped symbol, one color	Ea	3.00	$ 7.59	$ 13.40	$ 2.25	$ 23.24	$ 69.72
	Single line striping, 4" wide solid	LF	30.00	$ 0.14	$ 0.27	$ 0.05	$ 0.46	$ 13.80
	Tactile warning	Ea	4.00	$ -	$ -	$ -	$ 300.00	$ 1,200.00
Traffic Signs	Handicapped parking, 12" x 18"	Ea	3.00	$ 108.00	$ 52.10	$ -	$ 160.10	$ 480.30
Parking blocks	Parking blocks, 6' x 4" x 6"	Ea	15.00	$ 99.00	$ 31.20	$ -	$ 130.20	$ 1,953.00
Seeding and planting	Hydroseeding, 10,001 SF to 1 Acre job	MSF	6.20	$ -	$ -	$ -	$ 150.00	$ 930.00
Concrete work	Handicapped access ramp, railing both sides, 5' wide	LF	29.00	$ 456.00	$ 237.60	$ 2.59	$ 696.19	$ 20,189.57
	Stairs, Direct from chute	Ea	4.00	$ -	$ -	$ -	$ 1,150.00	$ 4,600.00
	Sidewalk, 5" thick	SF	1,569.00	$ -	$ -	$ -	$ 2.72	$ 4,267.68
	Sidewalk formwork, 3 uses	SF	1,569.00	$ 1.80	$ 4.06	$ -	$ 5.86	$ 9,194.34
	Subbase and aggregate, 6" thick	SF	58.11	$ 7.46	$ 1.51	$ 1.38	$ 10.35	$ 601.45
Asphalt Work	Parking lot paving	SF	2,767.00	$ 4.56	$ 1.21	$ 1.14	$ 6.91	$ 19,119.97
Retaining wall	Limestone, large blocks	CF	515.00	$ 42.10	$ 19.80	$ -	$ 61.90	$ 31,878.50
	Underdrain, 6" Corrugated Polyethylene	LF	103.00	$ 2.18	$ 0.58	$ -	$ 2.76	$ 284.28
Misc	Aluminum railing	LF	53.00	$ 44.30	$ 12.50	$ -	$ 56.80	$ 3,010.40
	Site grading	Ea	1.00	$ -	$ -	$ -	$ 5,000.00	$ 5,000.00
Site survey	Topographic survey	Ea.	1.00	$ -	$ -	$ -	$ 3,000.00	$ 3,000.00
								Total
								$112,978.16

Sitework Improvements Estimated Costs

Item	Unit	Quantity	Material	Labor	Equipment	Total	Cost
Remove existing stair	Ea	1	$ -	$ -	$ -	$ 800.00	$ 800.00
Sawcut, 5" deep, remove pavement full depth	LF	251	$ -	$ -	$ -	$ 2.16	$ 542.16
Sidewalk removal, 5" thick	SF	698.75	$ -	$ 2.19	$ 1.59	$ 3.78	$ 2,641.28
Curb removal	LF	16	$ -	$ 4.87	$ 3.54	$ 8.41	$ 134.56
Wooden ramp removal	Ea	1	$ -	$ -	$ -	$ 500.00	$ 500.00
							Total
							$ 4,618.00

Sitework Demolition Estimated Costs

Project Phasing and Total Cost Estimate						
Phase 1	Phase 2	Phase 3	Phase 4	Site	Contingency	Total
$ 63,000.00	$ 373,000.00	$ 743,000.00	$ 1,253,000.00	$ 130,000.00	$ 300,000.00	$ 2,862,000.00

Cost Summary

Table of contents:

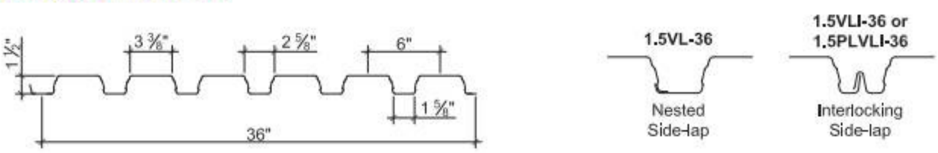
Floor dead loads

Slab on Deck, Vulcraft Composite Deck Slab 1.5VL

$w_c = 145\ pcf$ $\quad f_{c'} = 4000\ psi$ $\quad E_c = 33 \cdot \left(\dfrac{w_c}{\dfrac{lbf}{ft^3}}\right)^{1.5} \cdot \sqrt{\dfrac{f_{c'}}{psi}}\ psi = \left(3.644 \cdot 10^3\right)\ ksi$

$h_s = 2.5\ in$ 3.5" Slab $\quad f_{yDeck} = 50\ ksi$ 50 steel grade, 20 gauge

Nominal Dimensions



Slab Depth Total	Slab Depth Topping	Deck Gage	Maximum Unshored Spans: Maximum Unshored Construction Clear Span 1	2	3	Composite Deck-Slab Properties: Concrete + Deck (psf)	Deflection $I_d = (I_{cr}+I_u)/2$ (in⁴/ft)	Moment $ØM_{no}$ (kip-ft/ft)	Shear $ØV_{no}$ (kip/ft)
3½"	2"	22	6'-5"	7'-5"	7'-8"	37.5	3.43	3.86	5.03
		20	7'-7"	8'-6"	8'-9"	37.9	3.68	4.58	5.03
		19	8'-2"	9'-3"	9'-7"	38.2	3.91	5.25	5.03
		18	8'-6"	9'-11"	10'-3"	38.5	4.11	5.85	5.03
		16	9'-2"	11'-2"	11'-3"	39.2	4.50	7.12	5.03
5"	3½"	22	5'-8"	6'-5"	6'-8"	55.6	9.34	5.65	7.51
		20	6'-7"	7'-5"	7'-8"	56.0	9.97	6.74	7.81
		19	7'-3"	8'-1"	8'-4"	56.3	10.55	7.77	7.81
		18	7'-6"	8'-8"	8'-11"	56.6	11.05	8.69	7.81
		16	8'-1"	9'-9"	10'-0"	57.3	12.09	10.68	7.81
6"	4½"	22	5'-3"	6'-0"	6'-2"	67.7	15.62	7.20	8.36
		20	6'-2"	6'-11"	7'-1"	68.1	16.63	8.60	9.11
		19	6'-10"	7'-6"	7'-9"	68.4	17.55	9.92	9.49
		18	7'-1"	8'-0"	8'-4"	68.7	18.36	11.12	9.49
		16	7'-8"	9'-1"	9'-5"	69.4	20.03	13.72	9.49

$q_c = \left(37.5 + (55.6 - 37.5) \cdot \dfrac{1}{3}\right)\ psf = 43.533\ psf$ Concrete and 22 gauge deck dead weight

$q_f = 2.5\ psf$ Vinyl flooring dead weight

$MEP = 5\ psf$ $\quad F = 1\ psf$ Fireproofing and MEP loads

$q_{ceil} = 1\ psf$ Ceiling coverings

$D_f = \mathrm{ceil}\left(\dfrac{\left(q_c + q_f + q_{ceil}\right) + MEP + F}{psf}\right)\ psf = 54\ psf$ Floor Dead Load

Roof dead loads

Cold roll steel system supporting roof sheathing and insulation
Manufacturer to design truss

$q_r = 2\ psf$ Roof Coverings

$q_i = 1.5\ psf$ Insulation

$w_t = 120\ plf$ Assumed Truss Dead Load

$t_b = 4\ ft$ Tributary width for roof truss

$$D_r = \text{ceil}\left(\dfrac{\left(q_r + q_i\right) + \dfrac{w_t}{t_b}}{psf}\right) psf = 34\ psf$$ Roof dead load

Snow Loads

Define Variables

Define all known variables to be used in design calculations

$p_g = 58\ psf$ Lansing, IA, from Figure 7.2-1 in Chp. 4

$C_e = 1.0$ Based on partially exposed assumption and exposure category C from Table 7.3-1 in Chp. 4

$C_t = 1.1$ Cold ventilated roof, table 7.3-2

$C_s = 1.0$ Ct is 1 and slope is 6 on 12, not smooth.

$I_s = 1.5$ Risk factor of 2 table 1.5-2 Chp. 1

Balanced Snow Load Design Calculations

$$\cancel{S_b = \text{ceil}\left(\frac{0.7 \cdot C_e \cdot C_t \cdot I_s \cdot p_g}{psf}\right) \cdot psf = 67\ psf}$$

Unbalanced Snow Load Design Calculations

$$S_u = \text{ceil}\left(\frac{I_s \cdot p_g}{psf}\right) \cdot psf = 87\ psf$$

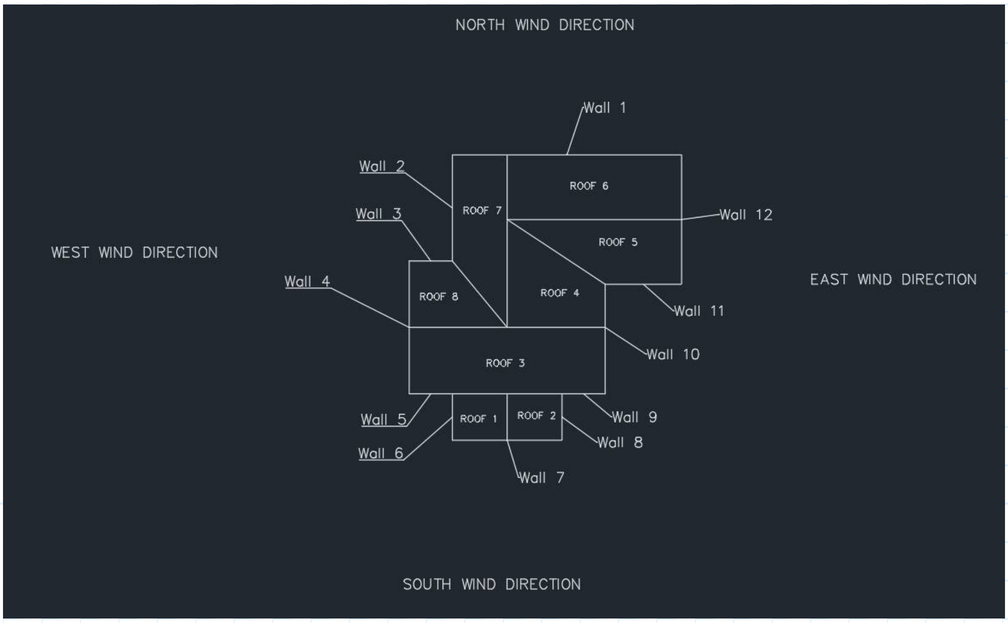
Live Load Summary

Define all known variables to be used in design calculations

$L = 100$ ***psf*** This was found in chp. 4 table 4.3-1 under the residential - all other residential occupancies - private rooms and corridors serving them.

$L_r = 20$ ***psf*** This was found in chp. 4 table 4.3-1 under the roofs - ordinary flat, pitched, and curved roofs.

Wind Load Calculations



All wind load calculations are based on the diagram above

Mean Roof Height

$h_e = 26\ \mathbf{ft}$ $\quad h_r = 37\ \mathbf{ft}$

$$h = \frac{(h_e + h_r)}{2} = 31.5\ \mathbf{ft}$$

Risk Category = IV

Wind Speed

$V = 120\ \mathbf{mph}$

Determine Wind Load Parameters

Wind Directionally Factor

$K_d = 0.85$

Exposure Category = C

Topographic Factor

$K_{zt} = 1.0$

Ground Elevation Factor

$K_e = 1.0$

Gust Effect Factor

$G = 0.85$

Enclosure Classification = Enclosed

$GC_{pip} = 0.18$

$GC_{pin} = -0.18$

Velocity pressure exposure

Because Exposure C

$\alpha = 9.8$

$z_g = 900\ ft$

$$K_z = 2.01 \cdot \left(\frac{15\ ft}{z_g} \right)^{\frac{2}{\alpha}} = 0.872$$

Velocity Pressure

$$q_z = \left(0.00256\ \frac{psf}{mph^2} \right) \cdot K_z \cdot K_{zt} \cdot K_d \cdot K_e \cdot V^2 = 27.31\ psf$$

North Wind Pressure $B = 82\ ft$ $L = 86\ ft$

$$\frac{L}{B} = 1.049$$

$C_{pw} = 0.8$ Windward Pressure

$C_{pl} = -0.5$ Leeward Pressure

$C_{ps} = -0.7$ Sidewall Pressure

Design Wind Pressure for Walls

Wall 1:

$p_{zw1p} = q_z \cdot G \cdot C_{pw} - q_z \cdot GC_{pip} = 13.655\ psf$

$p_{zw1n} = q_z \cdot G \cdot C_{pw} - q_z \cdot GC_{pin} = 23.487\ psf$

Wall 2:

$p_{zw2p} = q_z \cdot G \cdot C_{ps} - q_z \cdot GC_{pip} = -21.166\ psf$

$p_{zw2n} = q_z \cdot G \cdot C_{ps} - q_z \cdot GC_{pin} = -11.334\ psf$

Wall 3:

$p_{zw1p} = q_z \cdot G \cdot C_{pw} - q_z \cdot GC_{pip} = 13.655\ psf$

$p_{zw1n} = q_z \cdot G \cdot C_{pw} - q_z \cdot GC_{pin} = 23.487\ psf$

Wall 4:

$p_{zw2p} = q_z \cdot G \cdot C_{ps} - q_z \cdot GC_{pip} = -21.166\ psf$

$p_{zw2n} = q_z \cdot G \cdot C_{ps} - q_z \cdot GC_{pin} = -11.334\ psf$

Wall 5:

$p_{zw3p} = q_z \cdot G \cdot C_{pl} - q_z \cdot GC_{pip} = -16.523\ psf$

$p_{zw3n} = q_z \cdot G \cdot C_{pl} - q_z \cdot GC_{pin} = -6.691\ psf$

Wall 6:

$p_{zw2p} = q_z \cdot G \cdot C_{ps} - q_z \cdot GC_{pip} = -21.166\ psf$

$p_{zw2n} = q_z \cdot G \cdot C_{ps} - q_z \cdot GC_{pin} = -11.334\ psf$

Wall 7:

$p_{zw3p} = q_z \cdot G \cdot C_{pl} - q_z \cdot GC_{pip} = -16.523\ psf$

$p_{zw3n} = q_z \cdot G \cdot C_{pl} - q_z \cdot GC_{pin} = -6.691\ psf$

Wall 8:

$p_{zw2p} = q_z \cdot G \cdot C_{ps} - q_z \cdot GC_{pip} = -21.166\ psf$

$p_{zw2n} = q_z \cdot G \cdot C_{ps} - q_z \cdot GC_{pin} = -11.334\ psf$

Wall 9:

$p_{zw3p} = q_z \cdot G \cdot C_{pl} - q_z \cdot GC_{pip} = -16.523\ psf$

$p_{zw3n} = q_z \cdot G \cdot C_{pl} - q_z \cdot GC_{pin} = -6.691\ psf$

Wall 10:

$p_{zw2p} = q_z \cdot G \cdot C_{ps} - q_z \cdot GC_{pip} = -21.166\ psf$

$p_{zw2n} = q_z \cdot G \cdot C_{ps} - q_z \cdot GC_{pin} = -11.334\ psf$

Wall 11:

$p_{zw3p} = q_z \cdot G \cdot C_{pl} - q_z \cdot GC_{pip} = -16.523\ psf$

$p_{zw3n} = q_z \cdot G \cdot C_{pl} - q_z \cdot GC_{pin} = -6.691\ psf$

Wall 12:

$p_{zw2p} = q_z \cdot G \cdot C_{ps} - q_z \cdot GC_{pip} = -21.166\ psf$

$p_{zw2n} = q_z \cdot G \cdot C_{ps} - q_z \cdot GC_{pin} = -11.334\ psf$

Lengths of Roofs:

$L_1 = 53$ **ft** $L_2 = 52$ **ft** $L_3 = 53$ **ft** $L_4 = 33$ **ft**

$L_5 = 30$ **ft** $L_6 = 59$ **ft** $L_7 = 14$ **ft** $L_8 = 14$ **ft**

Roof Pressure Coefficients

Normal to Ridge
(Roof 1,3,5,6)

$\frac{h}{L_1} = 0.594$ $\frac{h}{L_3} = 0.594$ $\frac{h}{L_6} = 0.534$ $0 = 25$ **deg**

$C_{pwn1} = -0.3$ $C_{pln1} = 0$
$C_{pwn2} = 0.2$ $C_{pln2} = -0.6$

Parallel to Ridge
(Roof 2,4,7,8)

$C_{pwp1} = -0.9$ $C_{plp1} = -0.18$
$C_{pwp2} = -0.9$ $C_{plp2} = C_{plp1} = -0.18$
$C_{pwp3} = -0.5$ $C_{plp3} = C_{plp1} = -0.18$
$C_{pwp4} = -0.3$ $C_{plp4} = C_{plp1} = -0.18$

Design Wind Pressure for Roof

Roof 1: (normal to ridge, windward)

$p_{zw1} = q_z \cdot G \cdot C_{pwn1} - q_z \cdot GC_{pip} = -11.88$ **psf** $p_{zw3} = q_z \cdot G \cdot C_{pwn1} - q_z \cdot GC_{pin} = -2.048$ **psf**

$p_{zw2} = q_z \cdot G \cdot C_{pwn2} - q_z \cdot GC_{pip} = -0.273$ **psf** $p_{zw4} = q_z \cdot G \cdot C_{pwn2} - q_z \cdot GC_{pin} = 9.559$ **psf**

Roof 2: (parallel to ridge, windward)

$p_{zw1} = q_z \cdot G \cdot C_{pwp1} - q_z \cdot GC_{pip} = -25.808$ **psf** $p_{zw5} = q_z \cdot G \cdot C_{pwp1} - q_z \cdot GC_{pin} = -15.977$ **psf**

$p_{zw2} = q_z \cdot G \cdot C_{pwp2} - q_z \cdot GC_{pip} = -25.808$ **psf** $p_{zw6} = q_z \cdot G \cdot C_{pwp2} - q_z \cdot GC_{pin} = -15.977$ **psf**

$p_{zw3} = q_z \cdot G \cdot C_{pwp3} - q_z \cdot GC_{pip} = -16.523$ **psf** $p_{zw7} = q_z \cdot G \cdot C_{pwp3} - q_z \cdot GC_{pin} = -6.691$ **psf**

$p_{zw4} = q_z \cdot G \cdot C_{pwp4} - q_z \cdot GC_{pip} = -11.88$ **psf** $p_{zw8} = q_z \cdot G \cdot C_{pwp4} - q_z \cdot GC_{pin} = -2.048$ **psf**

Roof 3: (normal to ridge, leeward)

$p_{zl1} = q_z \cdot G \cdot C_{pln1} - q_z \cdot GC_{pip} = -4.916$ **psf**

$p_{zl3} = q_z \cdot G \cdot C_{pln1} - q_z \cdot GC_{pin} = 4.916$ **psf**

$p_{zl2} = q_z \cdot G \cdot C_{pln2} - q_z \cdot GC_{pip} = -18.844$ **psf**

$p_{zl4} = q_z \cdot G \cdot C_{pln2} - q_z \cdot GC_{pin} = -9.012$ **psf**

Roof 4: (parallel to ridge, leeward)

$p_{zl1} = q_z \cdot G \cdot C_{plp1} - q_z \cdot GC_{pip} = -9.094$ **psf** $p_{zl5} = q_z \cdot G \cdot C_{plp1} - q_z \cdot GC_{pin} = 0.737$ **psf**

$p_{zl2} = q_z \cdot G \cdot C_{plp2} - q_z \cdot GC_{pip} = -9.094$ **psf** $p_{zl6} = q_z \cdot G \cdot C_{plp2} - q_z \cdot GC_{pin} = 0.737$ **psf**

$p_{zl3} = q_z \cdot G \cdot C_{plp3} - q_z \cdot GC_{pip} = -9.094$ **psf** $p_{zl7} = q_z \cdot G \cdot C_{plp3} - q_z \cdot GC_{pin} = 0.737$ **psf**

$p_{zl4} = q_z \cdot G \cdot C_{plp4} - q_z \cdot GC_{pip} = -9.094$ **psf** $p_{zl8} = q_z \cdot G \cdot C_{plp4} - q_z \cdot GC_{pin} = 0.737$ **psf**

Roof 5: (normal to ridge, windward)

Note: Since Roof 5 has a significant difference in length roof pressures will be used for this roof

$\frac{h}{L_5} = 1.05$ $C_{pwn15} = -0.5$ $C_{pln1} = 0$

$C_{pwn25} = 0$ $C_{pln2} = -0.6$

$p_{zw1} = q_z \cdot G \cdot C_{pwn15} - q_z \cdot GC_{pip} = -16.523$ **psf** $p_{zw3} = q_z \cdot G \cdot C_{pwn15} - q_z \cdot GC_{pin} = -6.691$ **psf**

$p_{zw2} = q_z \cdot G \cdot C_{pwn25} - q_z \cdot GC_{pip} = -4.916$ **psf** $p_{zw4} = q_z \cdot G \cdot C_{pwn25} - q_z \cdot GC_{pin} = 4.916$ **psf**

Roof 6: (normal to ridge, leeward)

$p_{zl1} = q_z \cdot G \cdot C_{pln1} - q_z \cdot GC_{pip} = -4.916$ **psf** $p_{zl3} = q_z \cdot G \cdot C_{pln1} - q_z \cdot GC_{pin} = 4.916$ **psf**

$p_{zl2} = q_z \cdot G \cdot C_{pln2} - q_z \cdot GC_{pip} = -18.844$ **psf** $p_{zl4} = q_z \cdot G \cdot C_{pln2} - q_z \cdot GC_{pin} = -9.012$ **psf**

Roof 7: (parallel to ridge, leeward)

$p_{zl1} = q_z \cdot G \cdot C_{plp1} - q_z \cdot GC_{pip} = -9.094$ **psf** $p_{zl5} = q_z \cdot G \cdot C_{plp1} - q_z \cdot GC_{pin} = 0.737$ **psf**

$p_{zl2} = q_z \cdot G \cdot C_{plp2} - q_z \cdot GC_{pip} = -9.094$ **psf** $p_{zl6} = q_z \cdot G \cdot C_{plp2} - q_z \cdot GC_{pin} = 0.737$ **psf**

$p_{zl3} = q_z \cdot G \cdot C_{plp3} - q_z \cdot GC_{pip} = -9.094$ **psf** $p_{zl7} = q_z \cdot G \cdot C_{plp3} - q_z \cdot GC_{pin} = 0.737$ **psf**

$p_{zl4} = q_z \cdot G \cdot C_{plp4} - q_z \cdot GC_{pip} = -9.094$ **psf** $p_{zl8} = q_z \cdot G \cdot C_{plp4} - q_z \cdot GC_{pin} = 0.737$ **psf**

Roof 8: (parallel to ridge, leeward)

$p_{zl1} = q_z \cdot G \cdot C_{plp1} - q_z \cdot GC_{pip} = -9.094$ **psf** $p_{zl5} = q_z \cdot G \cdot C_{plp1} - q_z \cdot GC_{pin} = 0.737$ **psf**

$p_{zl2} = q_z \cdot G \cdot C_{plp2} - q_z \cdot GC_{pip} = -9.094$ **psf**

$p_{zl3} = q_z \cdot G \cdot C_{plp3} - q_z \cdot GC_{pip} = -9.094$ **psf**

$p_{zl4} = q_z \cdot G \cdot C_{plp4} - q_z \cdot GC_{pip} = -9.094$ **psf**

$p_{zl6} = q_z \cdot G \cdot C_{plp2} - q_z \cdot GC_{pin} = 0.737$ **psf**

$p_{zl7} = q_z \cdot G \cdot C_{plp3} - q_z \cdot GC_{pin} = 0.737$ **psf**

$p_{zl8} = q_z \cdot G \cdot C_{plp4} - q_z \cdot GC_{pin} = 0.737$ **psf**

South Wind Pressure $B = 82$ **ft** $L = 86$ **ft**

$\frac{L}{B} = 1.049$

$C_{pw} = 0.8$ Windward Pressure

$C_{pl} = -0.5$ Leeward Pressure

$C_{ps} = -0.7$ Sidewall Pressure

Design Wind Pressure for Walls

Wall 1:

$p_{zw1p} = q_z \cdot G \cdot C_{pl} - q_z \cdot GC_{pip} = -16.523$ **psf**

$p_{zw1n} = q_z \cdot G \cdot C_{pl} - q_z \cdot GC_{pin} = -6.691$ **psf**

Wall 2:

$p_{zw2p} = q_z \cdot G \cdot C_{ps} - q_z \cdot GC_{pip} = -21.166$ **psf**

$p_{zw2n} = q_z \cdot G \cdot C_{ps} - q_z \cdot GC_{pin} = -11.334$ **psf**

Wall 3:

$p_{zw1p} = q_z \cdot G \cdot C_{pl} - q_z \cdot GC_{pip} = -16.523$ **psf**

$p_{zw1n} = q_z \cdot G \cdot C_{pl} - q_z \cdot GC_{pin} = -6.691$ **psf**

Wall 4:

$p_{zw2p} = q_z \cdot G \cdot C_{ps} - q_z \cdot GC_{pip} = -21.166$ **psf**

$p_{zw2n} = q_z \cdot G \cdot C_{ps} - q_z \cdot GC_{pin} = -11.334$ **psf**

Wall 5:

$p_{zw3p} = q_z \cdot G \cdot C_{pw} - q_z \cdot GC_{pip} = 13.655$ **psf**

$p_{zw3n} = q_z \cdot G \cdot C_{pw} - q_z \cdot GC_{pin} = 23.487$ **psf**

Wall 6:

$p_{zw2p} = q_z \cdot G \cdot C_{ps} - q_z \cdot GC_{pip} = -21.166$ **psf**

$p_{zw2n} = q_z \cdot G \cdot C_{ps} - q_z \cdot GC_{pin} = -11.334$ **psf**

Wall 7:

$p_{zw3p} = q_z \cdot G \cdot C_{pw} - q_z \cdot GC_{pip} = 13.655$ **psf**

$p_{zw3n} = q_z \cdot G \cdot C_{pw} - q_z \cdot GC_{pin} = 23.487$ **psf**

Wall 8:

$p_{zw2p} = q_z \cdot G \cdot C_{ps} - q_z \cdot GC_{pip} = -21.166$ **psf**

$p_{zw2n} = q_z \cdot G \cdot C_{ps} - q_z \cdot GC_{pin} = -11.334$ **psf**

Wall 9:

$p_{zw3p} = q_z \cdot G \cdot C_{pw} - q_z \cdot GC_{pip} = 13.655$ **psf**

$p_{zw3n} = q_z \cdot G \cdot C_{pw} - q_z \cdot GC_{pin} = 23.487$ **psf**

Wall 10:

$p_{zw2p} = q_z \cdot G \cdot C_{ps} - q_z \cdot GC_{pip} = -21.166$ **psf**

$p_{zw2n} = q_z \cdot G \cdot C_{ps} - q_z \cdot GC_{pin} = -11.334$ **psf**

Wall 11:

$p_{zw3p} = q_z \cdot G \cdot C_{pw} - q_z \cdot GC_{pip} = 13.655$ **psf**

$p_{zw3n} = q_z \cdot G \cdot C_{pw} - q_z \cdot GC_{pin} = 23.487$ **psf**

Wall 12:

$p_{zw2p} = q_z \cdot G \cdot C_{ps} - q_z \cdot GC_{pip} = -21.166$ **psf**

$p_{zw2n} = q_z \cdot G \cdot C_{ps} - q_z \cdot GC_{pin} = -11.334$ **psf**

Lengths of Roofs:

$L_1 = 53$ **ft** $L_2 = 52$ **ft** $L_3 = 53$ **ft** $L_4 = 33$ **ft**

$L_5 = 30$ **ft** $L_6 = 59$ **ft** $L_7 = 14$ **ft** $L_8 = 14$ **ft**

Roof Pressure Coefficients

Normal to Ridge
(Roof 1,3,5,6)

$\frac{h}{L_1} = 0.594$ $\frac{h}{L_3} = 0.594$ $\frac{h}{L_6} = 0.534$ $\theta = 25$ **deg**

$C_{pwn1} = -0.3$ $C_{pwn2} = 0.2$

$C_{pln1} = 0$
$C_{pln2} = -0.6$

Parallel to Ridge
(Roof 2,4,7,8)

$C_{pwp1} = -0.9$ $C_{plp1} = -0.18$
$C_{pwp2} = -0.9$ $C_{plp2} = C_{plp1} = -0.18$
$C_{pwp3} = -0.5$ $C_{plp3} = C_{plp1} = -0.18$
$C_{pwp4} = -0.3$ $C_{plp4} = C_{plp1} = -0.18$

Design Wind Pressure for Roof
Roof 1: (normal to ridge, leeward)

$p_{zl1} = q_z \cdot G \cdot C_{pln1} - q_z \cdot GC_{pip} = -4.916$ ***psf*** $p_{zl3} = q_z \cdot G \cdot C_{pln1} - q_z \cdot GC_{pin} = 4.916$ ***psf***

$p_{zl2} = q_z \cdot G \cdot C_{pln2} - q_z \cdot GC_{pip} = -18.844$ ***psf*** $p_{zl4} = q_z \cdot G \cdot C_{pln2} - q_z \cdot GC_{pin} = -9.012$ ***psf***

Roof 2: (parallel to ridge, leeward)

$p_{zl1} = q_z \cdot G \cdot C_{plp1} - q_z \cdot GC_{pip} = -9.094$ ***psf*** $p_{zl5} = q_z \cdot G \cdot C_{plp1} - q_z \cdot GC_{pin} = 0.737$ ***psf***

$p_{zl2} = q_z \cdot G \cdot C_{plp2} - q_z \cdot GC_{pip} = -9.094$ ***psf*** $p_{zl6} = q_z \cdot G \cdot C_{plp2} - q_z \cdot GC_{pin} = 0.737$ ***psf***

$p_{zl3} = q_z \cdot G \cdot C_{plp3} - q_z \cdot GC_{pip} = -9.094$ ***psf*** $p_{zl7} = q_z \cdot G \cdot C_{plp3} - q_z \cdot GC_{pin} = 0.737$ ***psf***

$p_{zl4} = q_z \cdot G \cdot C_{plp4} - q_z \cdot GC_{pip} = -9.094$ ***psf*** $p_{zl8} = q_z \cdot G \cdot C_{plp4} - q_z \cdot GC_{pin} = 0.737$ ***psf***

Roof 3: (normal to ridge, windward)

$p_{zw1} = q_z \cdot G \cdot C_{pwn1} - q_z \cdot GC_{pip} = -11.88$ ***psf*** $p_{zw3} = q_z \cdot G \cdot C_{pwn1} - q_z \cdot GC_{pin} = -2.048$ ***psf***

$p_{zw2} = q_z \cdot G \cdot C_{pwn2} - q_z \cdot GC_{pip} = -0.273$ ***psf*** $p_{zw4} = q_z \cdot G \cdot C_{pwn2} - q_z \cdot GC_{pin} = 9.559$ ***psf***

Roof 4: (parallel to ridge, windward)

$p_{zw1} = q_z \cdot G \cdot C_{pwp1} - q_z \cdot GC_{pip} = -25.808$ ***psf*** $p_{zw5} = q_z \cdot G \cdot C_{pwp1} - q_z \cdot GC_{pin} = -15.977$ ***psf***

$p_{zw2} = q_z \cdot G \cdot C_{pwp2} - q_z \cdot GC_{pip} = -25.808$ ***psf*** $p_{zw6} = q_z \cdot G \cdot C_{pwp2} - q_z \cdot GC_{pin} = -15.977$ ***psf***

$p_{zw3} = q_z \cdot G \cdot C_{pwp3} - q_z \cdot GC_{pip} = -16.523$ ***psf*** $p_{zw7} = q_z \cdot G \cdot C_{pwp3} - q_z \cdot GC_{pin} = -6.691$ ***psf***

$p_{zw4} = q_z \cdot G \cdot C_{pwp4} - q_z \cdot GC_{pip} = -11.88$ ***psf*** $p_{zw8} = q_z \cdot G \cdot C_{pwp4} - q_z \cdot GC_{pin} = -2.048$ ***psf***

Roof 5: (normal to ridge, leeward)

Note: Since Roof 5 has a significant difference in length roof pressures will be used for this roof

$\frac{h}{L_5} = 1.05$ $C_{pwn15} = -0.5$ $C_{pln15} = 0$

$C_{pwn25} = 0$ $C_{pln25} = -0.6$

$p_{zw1} = q_z \cdot G \cdot C_{pln15} - q_z \cdot GC_{pip} = -4.916$ ***psf*** $p_{zw3} = q_z \cdot G \cdot C_{pln15} - q_z \cdot GC_{pin} = 4.916$ ***psf***

$p_{zw2} = q_z \cdot G \cdot C_{pln25} - q_z \cdot GC_{pip} = -18.844$ ***psf*** $p_{zw4} = q_z \cdot G \cdot C_{pln25} - q_z \cdot GC_{pin} = -9.012$ ***psf***

Roof 6: (normal to ridge, windward)

$p_{zw1} = q_z \cdot G \cdot C_{pwn1} - q_z \cdot GC_{pip} = -11.88$ ***psf*** $p_{zw3} = q_z \cdot G \cdot C_{pwn1} - q_z \cdot GC_{pin} = -2.048$ ***psf***

$p_{zw2} = q_z \cdot G \cdot C_{pwn2} - q_z \cdot GC_{pip} = -0.273$ ***psf*** $p_{zw4} = q_z \cdot G \cdot C_{pwn2} - q_z \cdot GC_{pin} = 9.559$ ***psf***

Roof 7: (parallel to ridge, windward)

$p_{zw1} = q_z \cdot G \cdot C_{pwp1} - q_z \cdot GC_{pip} = -25.808$ ***psf*** $p_{zw5} = q_z \cdot G \cdot C_{pwp1} - q_z \cdot GC_{pin} = -15.977$ ***psf***

$p_{zw2} = q_z \cdot G \cdot C_{pwp2} - q_z \cdot GC_{pip} = -25.808$ ***psf*** $p_{zw6} = q_z \cdot G \cdot C_{pwp2} - q_z \cdot GC_{pin} = -15.977$ ***psf***

$p_{zw3} = q_z \cdot G \cdot C_{pwp3} - q_z \cdot GC_{pip} = -16.523$ ***psf*** $p_{zw7} = q_z \cdot G \cdot C_{pwp3} - q_z \cdot GC_{pin} = -6.691$ ***psf***

$p_{zw4} = q_z \cdot G \cdot C_{pwp4} - q_z \cdot GC_{pip} = -11.88$ ***psf*** $p_{zw8} = q_z \cdot G \cdot C_{pwp4} - q_z \cdot GC_{pin} = -2.048$ ***psf***

Roof 8: (parallel to ridge, windward)

$p_{zw1} = q_z \cdot G \cdot C_{pwp1} - q_z \cdot GC_{pip} = -25.808$ ***psf*** $p_{zw5} = q_z \cdot G \cdot C_{pwp1} - q_z \cdot GC_{pin} = -15.977$ ***psf***

$p_{zw2} = q_z \cdot G \cdot C_{pwp2} - q_z \cdot GC_{pip} = -25.808$ ***psf*** $p_{zw6} = q_z \cdot G \cdot C_{pwp2} - q_z \cdot GC_{pin} = -15.977$ ***psf***

$p_{zw3} = q_z \cdot G \cdot C_{pwp3} - q_z \cdot GC_{pip} = -16.523$ ***psf*** $p_{zw7} = q_z \cdot G \cdot C_{pwp3} - q_z \cdot GC_{pin} = -6.691$ ***psf***

$p_{zw4} = q_z \cdot G \cdot C_{pwp4} - q_z \cdot GC_{pip} = -11.88$ ***psf*** $p_{zw8} = q_z \cdot G \cdot C_{pwp4} - q_z \cdot GC_{pin} = -2.048$ ***psf***

East Wind Pressure $B = 86\ ft$ $L = 82\ ft$

$\frac{L}{B} = 0.953$

$C_{pw} = 0.8$ Windward Pressure
$C_{pl} = -0.5$ Leeward Pressure
$C_{ps} = -0.7$ Sidewall Pressure

Design Wind Pressure for Walls

Wall 1:
$p_{zw1p} = q_z \cdot G \cdot C_{ps} - q_z \cdot GC_{pip} = -21.166\ psf$
$p_{zw1n} = q_z \cdot G \cdot C_{ps} - q_z \cdot GC_{pin} = -11.334\ psf$

Wall 2:
$p_{zw2p} = q_z \cdot G \cdot C_{pl} - q_z \cdot GC_{pip} = -16.523\ psf$
$p_{zw2n} = q_z \cdot G \cdot C_{pl} - q_z \cdot GC_{pin} = -6.691\ psf$

Wall 3:
$p_{zw1p} = q_z \cdot G \cdot C_{ps} - q_z \cdot GC_{pip} = -21.166\ psf$
$p_{zw1n} = q_z \cdot G \cdot C_{ps} - q_z \cdot GC_{pin} = -11.334\ psf$

Wall 4:
$p_{zw2p} = q_z \cdot G \cdot C_{pl} - q_z \cdot GC_{pip} = -16.523\ psf$
$p_{zw2n} = q_z \cdot G \cdot C_{pl} - q_z \cdot GC_{pin} = -6.691\ psf$

Wall 5:
$p_{zw3p} = q_z \cdot G \cdot C_{ps} - q_z \cdot GC_{pip} = -21.166\ psf$
$p_{zw3n} = q_z \cdot G \cdot C_{ps} - q_z \cdot GC_{pin} = -11.334\ psf$

Wall 6:
$p_{zw2p} = q_z \cdot G \cdot C_{pl} - q_z \cdot GC_{pip} = -16.523\ psf$
$p_{zw2n} = q_z \cdot G \cdot C_{pl} - q_z \cdot GC_{pin} = -6.691\ psf$

Wall 7:
$p_{zw3p} = q_z \cdot G \cdot C_{ps} - q_z \cdot GC_{pip} = -21.166\ psf$
$p_{zw3n} = q_z \cdot G \cdot C_{ps} - q_z \cdot GC_{pin} = -11.334\ psf$

Wall 8:
$p_{zw2p} = q_z \cdot G \cdot C_{pw} - q_z \cdot GC_{pip} = 13.655\ psf$
$p_{zw2n} = q_z \cdot G \cdot C_{pw} - q_z \cdot GC_{pin} = 23.487\ psf$

Wall 9:
$p_{zw3p} = q_z \cdot G \cdot C_{ps} - q_z \cdot GC_{pip} = -21.166\ psf$
$p_{zw3n} = q_z \cdot G \cdot C_{ps} - q_z \cdot GC_{pin} = -11.334\ psf$

Wall 10:
$p_{zw2p} = q_z \cdot G \cdot C_{pw} - q_z \cdot GC_{pip} = 13.655\ psf$
$p_{zw2n} = q_z \cdot G \cdot C_{pw} - q_z \cdot GC_{pin} = 23.487\ psf$

Wall 11:
$p_{zw3p} = q_z \cdot G \cdot C_{ps} - q_z \cdot GC_{pip} = -21.166\ psf$
$p_{zw3n} = q_z \cdot G \cdot C_{ps} - q_z \cdot GC_{pin} = -11.334\ psf$

Wall 12:
$p_{zw2p} = q_z \cdot G \cdot C_{pw} - q_z \cdot GC_{pip} = 13.655\ psf$
$p_{zw2n} = q_z \cdot G \cdot C_{pw} - q_z \cdot GC_{pin} = 23.487\ psf$

Lengths of Roofs:

$L_1 = 53\ ft$ $L_2 = 52\ ft$ $L_3 = 53\ ft$ $L_4 = 33\ ft$

$L_5 = 30\ ft$ $L_6 = 59\ ft$ $L_7 = 14\ ft$ $L_8 = 14\ ft$

Roof Pressure Coefficients
Normal to Ridge
(Roof 2,4,7,8)

$\frac{h}{L_4} = 0.955$ $\frac{h}{L_7} = 2.25$ $\frac{h}{L_8} = 2.25$ $\theta = 25\ deg$

$C_{pwn1} = -0.5$ $C_{pwn2} = 0$

$C_{pln1} = 0$
$C_{pln2} = -0.6$

Parallel to Ridge
(Roof 1,3,5,6)

$C_{pwp1} = -0.9$ $C_{plp1} = -0.18$
$C_{pwp2} = -0.9$ $C_{plp2} = C_{plp1} = -0.18$
$C_{pwp3} = -0.5$ $C_{plp3} = C_{plp1} = -0.18$
$C_{pwp4} = -0.3$ $C_{plp4} = C_{plp1} = -0.18$

Design Wind Pressure for Roof
Roof 1: (parallel to ridge, windward)

$p_{zw1} = q_z \cdot G \cdot C_{pwp1} - q_z \cdot GC_{pip} = -25.808$ ***psf*** $p_{zw5} = q_z \cdot G \cdot C_{pwp1} - q_z \cdot GC_{pin} = -15.977$ ***psf***

$p_{zw2} = q_z \cdot G \cdot C_{pwp2} - q_z \cdot GC_{pip} = -25.808$ ***psf*** $p_{zw6} = q_z \cdot G \cdot C_{pwp2} - q_z \cdot GC_{pin} = -15.977$ ***psf***

$p_{zw3} = q_z \cdot G \cdot C_{pwp3} - q_z \cdot GC_{pip} = -16.523$ ***psf*** $p_{zw7} = q_z \cdot G \cdot C_{pwp3} - q_z \cdot GC_{pin} = -6.691$ ***psf***

$p_{zw4} = q_z \cdot G \cdot C_{pwp4} - q_z \cdot GC_{pip} = -11.88$ ***psf*** $p_{zw8} = q_z \cdot G \cdot C_{pwp4} - q_z \cdot GC_{pin} = -2.048$ ***psf***

Roof 2: (normal to ridge, leeward)
Note: Since Roof 2 has a significant difference in length roof pressures will be used for this roof

$\frac{h}{L_2} = 0.606$ $C_{pwn12} = -0.3$ $C_{pln12} = 0$

$C_{pwn22} = 0.2$ $C_{pln22} = -0.6$

$p_{zw1} = q_z \cdot G \cdot C_{pln12} - q_z \cdot GC_{pip} = -4.916$ ***psf*** $p_{zw3} = q_z \cdot G \cdot C_{pln12} - q_z \cdot GC_{pin} = 4.916$ ***psf***

$p_{zw2} = q_z \cdot G \cdot C_{pln22} - q_z \cdot GC_{pip} = -18.844$ ***psf*** $p_{zw4} = q_z \cdot G \cdot C_{pln22} - q_z \cdot GC_{pin} = -9.012$ ***psf***

Roof 3: (parallel to ridge, windward)

$p_{zw1} = q_z \cdot G \cdot C_{pwp1} - q_z \cdot GC_{pip} = -25.808$ ***psf*** $p_{zw5} = q_z \cdot G \cdot C_{pwp1} - q_z \cdot GC_{pin} = -15.977$ ***psf***

$p_{zw2} = q_z \cdot G \cdot C_{pwp2} - q_z \cdot GC_{pip} = -25.808$ ***psf*** $p_{zw6} = q_z \cdot G \cdot C_{pwp2} - q_z \cdot GC_{pin} = -15.977$ ***psf***

$p_{zw3} = q_z \cdot G \cdot C_{pwp3} - q_z \cdot GC_{pip} = -16.523$ ***psf*** $p_{zw7} = q_z \cdot G \cdot C_{pwp3} - q_z \cdot GC_{pin} = -6.691$ ***psf***

$p_{zw4} = q_z \cdot G \cdot C_{pwp4} - q_z \cdot GC_{pip} = -11.88$ ***psf*** $p_{zw8} = q_z \cdot G \cdot C_{pwp4} - q_z \cdot GC_{pin} = -2.048$ ***psf***

Roof 4: (normal to ridge, windward)

$p_{zw1} = q_z \cdot G \cdot C_{pwn1} - q_z \cdot GC_{pip} = -16.523$ ***psf***

$p_{zw3} = q_z \cdot G \cdot C_{pwn1} - q_z \cdot GC_{pin} = -6.691$ ***psf***

$p_{zw2} = q_z \cdot G \cdot C_{pwn2} - q_z \cdot GC_{pip} = -4.916$ ***psf***

$p_{zw4} = q_z \cdot G \cdot C_{pwn2} - q_z \cdot GC_{pin} = 4.916$ ***psf***

Roof 5: (parallel to ridge, leeward)

$p_{zl1} = q_z \cdot G \cdot C_{plp1} - q_z \cdot GC_{pip} = -9.094$ **psf** $p_{zl5} = q_z \cdot G \cdot C_{plp1} - q_z \cdot GC_{pin} = 0.737$ **psf**

$p_{zl2} = q_z \cdot G \cdot C_{plp2} - q_z \cdot GC_{pip} = -9.094$ **psf** $p_{zl6} = q_z \cdot G \cdot C_{plp2} - q_z \cdot GC_{pin} = 0.737$ **psf**

$p_{zl3} = q_z \cdot G \cdot C_{plp3} - q_z \cdot GC_{pip} = -9.094$ **psf** $p_{zl7} = q_z \cdot G \cdot C_{plp3} - q_z \cdot GC_{pin} = 0.737$ **psf**

$p_{zl4} = q_z \cdot G \cdot C_{plp4} - q_z \cdot GC_{pip} = -9.094$ **psf** $p_{zl8} = q_z \cdot G \cdot C_{plp4} - q_z \cdot GC_{pin} = 0.737$ **psf**

Roof 6: (parallel to ridge, windward)

$p_{zw1} = q_z \cdot G \cdot C_{pwp1} - q_z \cdot GC_{pip} = -25.808$ **psf** $p_{zw5} = q_z \cdot G \cdot C_{pwp1} - q_z \cdot GC_{pin} = -15.977$ **psf**

$p_{zw2} = q_z \cdot G \cdot C_{pwp2} - q_z \cdot GC_{pip} = -25.808$ **psf** $p_{zw6} = q_z \cdot G \cdot C_{pwp2} - q_z \cdot GC_{pin} = -15.977$ **psf**

$p_{zw3} = q_z \cdot G \cdot C_{pwp3} - q_z \cdot GC_{pip} = -16.523$ **psf** $p_{zw7} = q_z \cdot G \cdot C_{pwp3} - q_z \cdot GC_{pin} = -6.691$ **psf**

$p_{zw4} = q_z \cdot G \cdot C_{pwp4} - q_z \cdot GC_{pip} = -11.88$ **psf** $p_{zw8} = q_z \cdot G \cdot C_{pwp4} - q_z \cdot GC_{pin} = -2.048$ **psf**

Roof 7: (normal to ridge, leeward)

$p_{zl1} = q_z \cdot G \cdot C_{pln1} - q_z \cdot GC_{pip} = -4.916$ **psf** $p_{zl3} = q_z \cdot G \cdot C_{pln1} - q_z \cdot GC_{pin} = 4.916$ **psf**

$p_{zl2} = q_z \cdot G \cdot C_{pln2} - q_z \cdot GC_{pip} = -18.844$ **psf** $p_{zl4} = q_z \cdot G \cdot C_{pln2} - q_z \cdot GC_{pin} = -9.012$ **psf**

Roof 8: (normal to ridge, windward)

$p_{zw1} = q_z \cdot G \cdot C_{pwn1} - q_z \cdot GC_{pip} = -16.523$ **psf** $p_{zw3} = q_z \cdot G \cdot C_{pwn1} - q_z \cdot GC_{pin} = -6.691$ **psf**

$p_{zw2} = q_z \cdot G \cdot C_{pwn2} - q_z \cdot GC_{pip} = -4.916$ **psf** $p_{zw4} = q_z \cdot G \cdot C_{pwn2} - q_z \cdot GC_{pin} = 4.916$ **psf**

West Wind Pressure $B=86\ ft$ $L=82\ ft$

$\frac{L}{B}=0.953$

$C_{pw}=0.8$ Windward Pressure
$C_{pl}=-0.5$ Leeward Pressure
$C_{ps}=-0.7$ Sidewall Pressure

Design Wind Pressure for Walls

Wall 1:
$p_{zw1p}=q_z \cdot G \cdot C_{ps}-q_z \cdot GC_{pip}=-21.166\ psf$
$p_{zw1n}=q_z \cdot G \cdot C_{ps}-q_z \cdot GC_{pin}=-11.334\ psf$

Wall 2:
$p_{zw2p}=q_z \cdot G \cdot C_{pw}-q_z \cdot GC_{pip}=13.655\ psf$
$p_{zw2n}=q_z \cdot G \cdot C_{pw}-q_z \cdot GC_{pin}=23.487\ psf$

Wall 3:
$p_{zw1p}=q_z \cdot G \cdot C_{ps}-q_z \cdot GC_{pip}=-21.166\ psf$
$p_{zw1n}=q_z \cdot G \cdot C_{ps}-q_z \cdot GC_{pin}=-11.334\ psf$

Wall 4:
$p_{zw2p}=q_z \cdot G \cdot C_{pw}-q_z \cdot GC_{pip}=13.655\ psf$
$p_{zw2n}=q_z \cdot G \cdot C_{pw}-q_z \cdot GC_{pin}=23.487\ psf$

Wall 5:
$p_{zw3p}=q_z \cdot G \cdot C_{ps}-q_z \cdot GC_{pip}=-21.166\ psf$
$p_{zw3n}=q_z \cdot G \cdot C_{ps}-q_z \cdot GC_{pin}=-11.334\ psf$

Wall 6:
$p_{zw2p}=q_z \cdot G \cdot C_{pw}-q_z \cdot GC_{pip}=13.655\ psf$
$p_{zw2n}=q_z \cdot G \cdot C_{pw}-q_z \cdot GC_{pin}=23.487\ psf$

Wall 7:
$p_{zw3p}=q_z \cdot G \cdot C_{ps}-q_z \cdot GC_{pip}=-21.166\ psf$
$p_{zw3n}=q_z \cdot G \cdot C_{ps}-q_z \cdot GC_{pin}=-11.334\ psf$

Wall 8:
$p_{zw2p}=q_z \cdot G \cdot C_{pl}-q_z \cdot GC_{pip}=-16.523\ psf$
$p_{zw2n}=q_z \cdot G \cdot C_{pl}-q_z \cdot GC_{pin}=-6.691\ psf$

Wall 9:
$p_{zw3p}=q_z \cdot G \cdot C_{ps}-q_z \cdot GC_{pip}=-21.166\ psf$
$p_{zw3n}=q_z \cdot G \cdot C_{ps}-q_z \cdot GC_{pin}=-11.334\ psf$

Wall 10:
$p_{zw2p}=q_z \cdot G \cdot C_{pl}-q_z \cdot GC_{pip}=-16.523\ psf$
$p_{zw2n}=q_z \cdot G \cdot C_{pl}-q_z \cdot GC_{pin}=-6.691\ psf$

Wall 11:
$p_{zw3p}=q_z \cdot G \cdot C_{ps}-q_z \cdot GC_{pip}=-21.166\ psf$
$p_{zw3n}=q_z \cdot G \cdot C_{ps}-q_z \cdot GC_{pin}=-11.334\ psf$

Wall 12:
$p_{zw2p}=q_z \cdot G \cdot C_{pl}-q_z \cdot GC_{pip}=-16.523\ psf$
$p_{zw2n}=q_z \cdot G \cdot C_{pl}-q_z \cdot GC_{pin}=-6.691\ psf$

Lengths of Roofs:
$L_1=53\ ft$ $L_2=52\ ft$ $L_3=53\ ft$ $L_4=33\ ft$

$L_5=30\ ft$ $L_6=59\ ft$ $L_7=14\ ft$ $L_8=14\ ft$

Roof Pressure Coefficients
Normal to Ridge
(Roof 2,4,7,8)

$\frac{h}{L_4}=0.955$ $\frac{h}{L_7}=2.25$ $\frac{h}{L_8}=2.25$ $\theta=25\ deg$

$C_{pwn1}=-0.5$ $C_{pwn2}=0$

$C_{pln1} = 0$
$C_{pln2} = -0.6$

Parallel to Ridge
(Roof 1,3,5,6)

$C_{pwp1} = -0.9$ $C_{plp1} = -0.18$
$C_{pwp2} = -0.9$ $C_{plp2} = C_{plp1} = -0.18$
$C_{pwp3} = -0.5$ $C_{plp3} = C_{plp1} = -0.18$
$C_{pwp4} = -0.3$ $C_{plp4} = C_{plp1} = -0.18$

Roof 1: (parallel to ridge, leeward)

$p_{zl1} = q_z \cdot G \cdot C_{plp1} - q_z \cdot GC_{pip} = -9.094$ **psf** $p_{zl5} = q_z \cdot G \cdot C_{plp1} - q_z \cdot GC_{pin} = 0.737$ **psf**

$p_{zl2} = q_z \cdot G \cdot C_{plp2} - q_z \cdot GC_{pip} = -9.094$ **psf** $p_{zl6} = q_z \cdot G \cdot C_{plp2} - q_z \cdot GC_{pin} = 0.737$ **psf**

$p_{zl3} = q_z \cdot G \cdot C_{plp3} - q_z \cdot GC_{pip} = -9.094$ **psf** $p_{zl7} = q_z \cdot G \cdot C_{plp3} - q_z \cdot GC_{pin} = 0.737$ **psf**

$p_{zl4} = q_z \cdot G \cdot C_{plp4} - q_z \cdot GC_{pip} = -9.094$ **psf** $p_{zl8} = q_z \cdot G \cdot C_{plp4} - q_z \cdot GC_{pin} = 0.737$ **psf**

Roof 2: (normal to ridge, windward)
Note: Since Roof 2 has a significant difference in length roof pressures will be used for this roof

$\frac{h}{L_2} = 0.606$ $C_{pwn12} = -0.3$ $C_{pln12} = 0$

$C_{pwn22} = 0.2$ $C_{pln22} = -0.6$

$p_{zw1} = q_z \cdot G \cdot C_{pwn12} - q_z \cdot GC_{pip} = -11.88$ **psf** $p_{zw3} = q_z \cdot G \cdot C_{pwn12} - q_z \cdot GC_{pin} = -2.048$ **psf**

$p_{zw2} = q_z \cdot G \cdot C_{pwn22} - q_z \cdot GC_{pip} = -0.273$ **psf** $p_{zw4} = q_z \cdot G \cdot C_{pwn22} - q_z \cdot GC_{pin} = 9.559$ **psf**

Roof 3: (parallel to ridge, leeward)

$p_{zl1} = q_z \cdot G \cdot C_{plp1} - q_z \cdot GC_{pip} = -9.094$ **psf** $p_{zl5} = q_z \cdot G \cdot C_{plp1} - q_z \cdot GC_{pin} = 0.737$ **psf**

$p_{zl2} = q_z \cdot G \cdot C_{plp2} - q_z \cdot GC_{pip} = -9.094$ **psf** $p_{zl6} = q_z \cdot G \cdot C_{plp2} - q_z \cdot GC_{pin} = 0.737$ **psf**

$p_{zl3} = q_z \cdot G \cdot C_{plp3} - q_z \cdot GC_{pip} = -9.094$ **psf** $p_{zl7} = q_z \cdot G \cdot C_{plp3} - q_z \cdot GC_{pin} = 0.737$ **psf**

$p_{zl4} = q_z \cdot G \cdot C_{plp4} - q_z \cdot GC_{pip} = -9.094$ **psf** $p_{zl8} = q_z \cdot G \cdot C_{plp4} - q_z \cdot GC_{pin} = 0.737$ **psf**

Roof 4: (normal to ridge, leeward)

$p_{zl1} = q_z \cdot G \cdot C_{pln1} - q_z \cdot GC_{pip} = -4.916$ **psf**

$p_{zl3} = q_z \cdot G \cdot C_{pln1} - q_z \cdot GC_{pin} = 4.916$ **psf**

$p_{zl2} = q_z \cdot G \cdot C_{pln2} - q_z \cdot GC_{pip} = -18.844$ **psf**

$p_{zl4} = q_z \cdot G \cdot C_{pln2} - q_z \cdot GC_{pin} = -9.012$ **psf**

Roof 5: (parallel to ridge, windward)

$p_{zw1} = q_z \cdot G \cdot C_{pwp1} - q_z \cdot GC_{pip} = -25.808$ **psf** $p_{zw5} = q_z \cdot G \cdot C_{pwp1} - q_z \cdot GC_{pin} = -15.977$ **psf**

$p_{zw2} = q_z \cdot G \cdot C_{pwp2} - q_z \cdot GC_{pip} = -25.808$ **psf** $p_{zw6} = q_z \cdot G \cdot C_{pwp2} - q_z \cdot GC_{pin} = -15.977$ **psf**

$p_{zw3} = q_z \cdot G \cdot C_{pwp3} - q_z \cdot GC_{pip} = -16.523$ **psf** $p_{zw7} = q_z \cdot G \cdot C_{pwp3} - q_z \cdot GC_{pin} = -6.691$ **psf**

$p_{zw4} = q_z \cdot G \cdot C_{pwp4} - q_z \cdot GC_{pip} = -11.88$ **psf** $p_{zw8} = q_z \cdot G \cdot C_{pwp4} - q_z \cdot GC_{pin} = -2.048$ **psf**

Roof 6: (parallel to ridge, windward)

$p_{zw1} = q_z \cdot G \cdot C_{pwp1} - q_z \cdot GC_{pip} = -25.808$ **psf** $p_{zw5} = q_z \cdot G \cdot C_{pwp1} - q_z \cdot GC_{pin} = -15.977$ **psf**

$p_{zw2} = q_z \cdot G \cdot C_{pwp2} - q_z \cdot GC_{pip} = -25.808$ **psf** $p_{zw6} = q_z \cdot G \cdot C_{pwp2} - q_z \cdot GC_{pin} = -15.977$ **psf**

$p_{zw3} = q_z \cdot G \cdot C_{pwp3} - q_z \cdot GC_{pip} = -16.523$ **psf** $p_{zw7} = q_z \cdot G \cdot C_{pwp3} - q_z \cdot GC_{pin} = -6.691$ **psf**

$p_{zw4} = q_z \cdot G \cdot C_{pwp4} - q_z \cdot GC_{pip} = -11.88$ **psf** $p_{zw8} = q_z \cdot G \cdot C_{pwp4} - q_z \cdot GC_{pin} = -2.048$ **psf**

Roof 7: (normal to ridge, windward)

$p_{zw1} = q_z \cdot G \cdot C_{pwn1} - q_z \cdot GC_{pip} = -16.523$ **psf** $p_{zw3} = q_z \cdot G \cdot C_{pwn1} - q_z \cdot GC_{pin} = -6.691$ **psf**

$p_{zw2} = q_z \cdot G \cdot C_{pwn2} - q_z \cdot GC_{pip} = -4.916$ **psf** $p_{zw4} = q_z \cdot G \cdot C_{pwn2} - q_z \cdot GC_{pin} = 4.916$ **psf**

Roof 8: (normal to ridge, leeward)

$p_{zl1} = q_z \cdot G \cdot C_{pln1} - q_z \cdot GC_{pip} = -4.916$ **psf** $p_{zl3} = q_z \cdot G \cdot C_{pln1} - q_z \cdot GC_{pin} = 4.916$ **psf**

$p_{zl2} = q_z \cdot G \cdot C_{pln2} - q_z \cdot GC_{pip} = -18.844$ **psf** $p_{zl4} = q_z \cdot G \cdot C_{pln2} - q_z \cdot GC_{pin} = -9.012$ **psf**

Components and Cladding Wind Pressure:

Mean Roof Height

$h_e = 26\ ft \qquad h_r = 37\ ft$

$$h = \frac{(h_e + h_r)}{2} = 31.5\ ft$$

Risk Category = IV

Wind Speed

$V = 120\ mph$

Determine Wind Load Parameters

Wind Directionally Factor

$K_d = 0.85$

Exposure Category = C

Topographic Factor

$K_{zt} = 1.0$

Ground Elevation Factor

$K_e = 1.0$

Gust Effect Factor

$G = 0.85$

Enclosure Classification = Enclosed

$GC_{pip} = 0.18$

$GC_{pin} = -0.18$

Velocity pressure exposure

Because Exposure C

$\alpha := 9.8$

$z_g = 900\ ft$

$$K_z = 2.01 \cdot \left(\frac{15\ ft}{z_g}\right)^{\frac{2}{\alpha}} = 0.872$$

Velocity Pressure

$$q_z = \left(\frac{0.00256\ psf}{mph^2}\right) \cdot K_z \cdot K_{zt} \cdot K_d \cdot K_e \cdot V^2 = 27.31\ psf$$

GCp: (ASCE 7-16 Figure 30.3-1)

Positive: $GC_{pp} = 0.7$

Negative: $GC_{pn} = -0.8$

Design C&C Wind Pressures for Walls

$$p_1 = q_z \cdot (GC_{pp} - GC_{pip}) = 14.201 \ \mathbf{psf}$$

$$p_2 = q_z \cdot (GC_{pp} - GC_{pin}) = 24.033 \ \mathbf{psf}$$

$$p_3 = q_z \cdot (GC_{pn} - GC_{pip}) = -26.764 \ \mathbf{psf}$$

$$p_4 = q_z \cdot (GC_{pn} - GC_{pin}) = -16.933 \ \mathbf{psf}$$

Use Worst Cases:

Positive Pressure: $p_p = p_2 = 24.033 \ \mathbf{psf}$

Negative Pressure: $p_n = p_3 = -26.764 \ \mathbf{psf}$

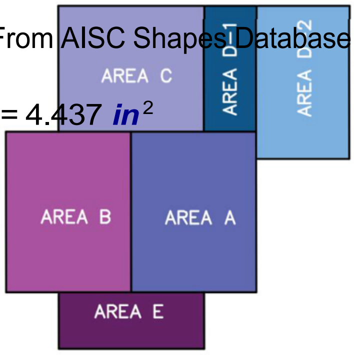
First Floor Joist Area A Analysis

Assumptions (if needed)

- Risk factor IV
- Cold ventilated roof
- LRFD Load combination 2, 1.2D + 1.6L governs floor gravity loads
- Assume W16x36

Define Variables

$L_A = 28\ ft$	All member dimensions are for a W16x36
$D_l = 54\ psf$	See load calculations
$w_b = 36\ plf$	From AISC Shapes Database
$L_l = 100\ psf$	First floor live load for lobby/gathering areas
$t_b = 5\ ft$	Assumed tributary width for joists
$E = 29000\ ksi$	Assumed Modulus of Elasticity of Steel
$f_y = 50\ ksi$	Yield stress
$I = 448\ in^4$	From AISC Shapes Database
$Z_x = 64.0\ in^3$	From AISC Shapes Database
$t_w = 0.295\ in$	From AISC Shapes Database
$t_f = 0.430\ in$	From AISC Shapes Database
$b_f = 6.99\ in$	From AISC Shapes Database



$d = 15.9\ in$ From AISC Shapes Database

$A_w = (d - t_f \cdot 2) \cdot t_w = 4.437\ in^2$

Member Strength

$$M_{all} = 0.9 \cdot f_y \cdot Z_x = 240 \ \textbf{kip} \cdot \textbf{ft}$$

$$V_{all} = 0.6 \cdot f_y \cdot A_w = 133.104 \ \textbf{kip}$$

Strength Requirements

$$w_d = t_b \cdot D_l + w_b = 0.306 \ \textbf{klf}$$

$$w_l = t_b \cdot L_l = 0.5 \ \textbf{klf}$$

$$w_u = \max(1.4 \cdot w_d, 1.2 \cdot w_d + 1.6 \cdot w_l) = (1.167 \cdot 10^3) \ \textbf{plf}$$

Maximum moment

$$M_{max} = w_u \cdot \frac{L_A^{\ 2}}{8} = 114.386 \ \textbf{kip} \cdot \textbf{ft}$$

$Moment$ = “Okay”

Maximum shear

$$V_{max} = w_u \cdot \frac{L_A}{2} = 16.341 \ \textbf{kip}$$

$Shear$ = “Okay”

Allowable Serviceability Deflections

$$8_{Sall} = \frac{L_A}{240} = 1.4 \ \textbf{in}$$ Short Term Allowable Deflection

$$8_{Tall} = \frac{L_A}{360} = 0.933 \ \textbf{in}$$ Total Allowable Deflection

Serviceability Calculations

$$\delta_{short} = \frac{5 \cdot \frac{w_l}{2} \cdot L_A^{\,4}}{384 \cdot E \cdot I} = 0.266 \; \boldsymbol{in}$$

Short = “Okay”

$$\delta_{long} = \frac{5 \cdot \left(\frac{w_l}{2} + w_d \right) \cdot L_A^{\,4}}{384 \cdot E \cdot I} = 0.592 \; \boldsymbol{in}$$

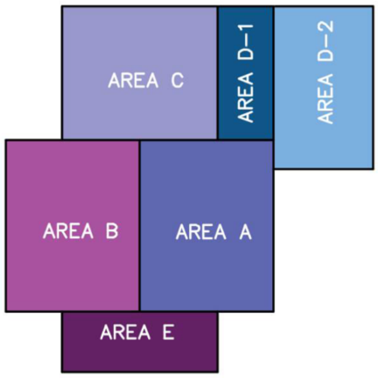
$$\delta_{tot} = \delta_{short} + \delta_{long} = 0.858 \; \boldsymbol{in}$$

Total = “Okay”

Design Summary

A W16x36 satisfies all requirements for this preliminary design.

First Floor Joist Area B Analysis



Assumptions (if needed)

- Assume W16x36
- LRFD Load combination 2, 1.2D + 1.6L governs floor gravity loads

Define Variables

$L_B = 28\ \boldsymbol{ft}$ All member dimensions are for a W16x36

$D_l = 54\ \boldsymbol{psf}$ See load calculations

$w_b = 36\ \boldsymbol{plf}$ From AISC Shapes Database

$L_l = 100\ \boldsymbol{psf}$ First floor live load for lobby/gathering areas

$t_b = 5\ \boldsymbol{ft}$ Assumed tributary width for joists

$E = 29000\ \boldsymbol{ksi}$ Assumed Modulus of Elasticity of Steel

$I = 448\ \boldsymbol{in}^4$ From AISC Shapes Database

$f_y = 50\ \boldsymbol{ksi}$ Yield stress

$Z_x = 64.0\ \boldsymbol{in}^3$ From AISC Shapes Database

$t_w = 0.295\ \boldsymbol{in}$ From AISC Shapes Database

$t_f = 0.430\ \boldsymbol{in}$ From AISC Shapes Database

$b_f = 6.99\ \boldsymbol{in}$ From AISC Shapes Database

$d = 15.9\ \boldsymbol{in}$ From AISC Shapes Database

$$A_w = \left(d - t_f \cdot 2\right) \cdot t_w = 4.437\ \boldsymbol{in}^2$$

Member Strength

$$M_{all} = 0.9 \cdot f_y \cdot Z_x = 240\ \boldsymbol{kip \cdot ft}$$

$$V_{all} = 0.6 \cdot f_y \cdot A_w = 133.104\ \boldsymbol{kip}$$

Strength Requirements

$$w_d = t_b \cdot D_l + w_b = 0.306\ \boldsymbol{klf}$$

$$w_l = t_b \cdot L_l = 0.5\ \boldsymbol{klf}$$

$$w_u = \max\left(, 1.4 \cdot w_d, 1.2 \cdot w_d + 1.6 \cdot w_l\right) = \left(, 1.167 \cdot 10^3\right)\ \boldsymbol{plf}$$

Maximum moment

$$M_{max} = w_u \cdot \frac{L_B^{\ 2}}{8} = 114.386\ \boldsymbol{kip \cdot ft}$$

$Moment$ = "Okay"

Maximum shear

$$V_{max} = w_u \cdot \frac{L_B}{2} = 16.341\ \boldsymbol{kip}$$

$Shear$ = "Okay"

Allowable Serviceability Deflections

$$8_{Sall} = \frac{L_B}{240} = 1.4\ \boldsymbol{in}$$ Short Term Allowable Deflection

$$8_{Tall} = \frac{L_B}{360} = 0.933\ \boldsymbol{in}$$ Total Allowable Deflection

Serviceability Calculations

$$\delta_{short} = \frac{5 \cdot \frac{w_l}{2} \cdot L_B^{\ 4}}{384 \cdot E \cdot I} = 0.266\ in$$

$Short$ = "Okay"

$$\delta_{long} = \frac{5 \cdot \left(\frac{w_l}{2} + w_d \right) \cdot L_B^{\ 4}}{384 \cdot E \cdot I} = 0.592\ in$$

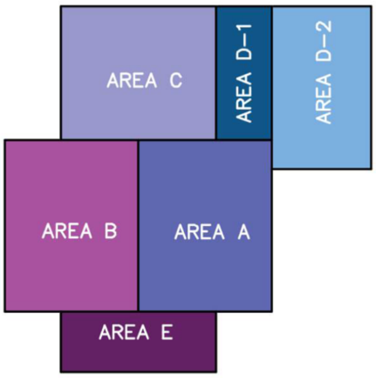
$$\delta_{tot} = \delta_{short} + \delta_{long} = 0.858\ in$$

$Total$ = "Okay"

Design Summary

A W16x36 satisfies all requirements for this preliminary design.

First Floor Joist Area C Analysis



Assumptions (if needed)

- Assume W16x31
- LRFD Load combination 2, 1.2D + 1.6L governs floor gravity loads

Define Variables

$L_C = 26\ ft$ All member dimensions are for a W16x31

$D_l = 54\ psf$ See load calculations

$w_b = 31\ plf$ From AISC Shapes Database

$L_l = 100\ psf$ First floor live load for lobby/gathering areas

$t_b = 5\ ft$ Assumed tributary width for joists

$E = 29000\ ksi$ Assumed Modulus of Elasticity of Steel

$I = 375\ in^4$ From AISC Shapes Database

$f_y = 50\ ksi$ Yield stress

$Z_x = 54.0\ in^3$ From AISC Shapes Database

$t_w = 0.275\ in$ From AISC Shapes Database

$t_f = 0.440\ in$ From AISC Shapes Database

$b_f = 5.53\ in$ From AISC Shapes Database

$d = 15.9\ in$ From AISC Shapes Database

$A_w = \left(d - t_f \cdot 2\right) \cdot t_w = 4.131 \; \boldsymbol{in}^2$

Member Strength

$M_{all} = 0.9 \cdot f_y \cdot Z_x = 202.5 \ \textbf{kip} \cdot \textbf{ft}$

$V_{all} = 0.6 \cdot f_y \cdot A_w = 123.915 \ \textbf{kip}$

Strength Requirements

$w_d = t_b \cdot D_l + w_b = 0.301 \ \textbf{klf}$

$w_l = t_b \cdot L_l = 0.5 \ \textbf{klf}$

$w_u = \max\left(, 1.4 \cdot w_d, 1.2 \cdot w_d + 1.6 \cdot w_l\right) = \left(, 1.161 \cdot 10^3\right) \ \textbf{plf}$

Maximum moment

$$M_{max} = w_u \cdot \frac{L_C^{\,2}}{8} = 98.121 \ \textbf{kip} \cdot \textbf{ft}$$

$Moment$ = “Okay”

Maximum shear

$$V_{max} = w_u \cdot \frac{L_C}{2} = 15.096 \ \textbf{kip}$$

$Shear$ = “Okay”

Allowable Serviceability Deflections

$$8_{Sall} = \frac{L_C}{240} = 1.3 \ \textbf{in}$$ Short Term Allowable Deflection

$$8_{Tall} = \frac{L_C}{360} = 0.867 \ \textbf{in}$$ Total Allowable Deflection

Serviceability Calculations

$$\delta_{short} = \frac{5 \cdot \frac{w_l}{2} \cdot L_C^{\ 4}}{384 \cdot E \cdot I} = 0.236\ in$$

$Short$ = “Okay”

$$\delta_{long} = \frac{5 \cdot \left(\frac{w_l}{2} + w_d\right) \cdot L_C^{\ 4}}{384 \cdot E \cdot I} = 0.521\ in$$

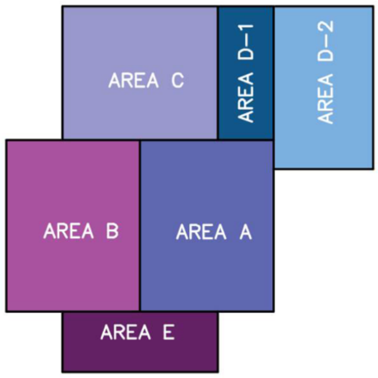
$$\delta_{tot} = \delta_{short} + \delta_{long} = 0.757\ in$$

$Total$ = “Okay”

Design Summary

A W16x31 satisfies all requirements for this preliminary design.

First Floor Joist Area D1 Analysis



Assumptions (if needed)

- Assume W10x22
- LRFD Load combination 2, 1.2D + 1.6L governs floor gravity loads

Define Variables

$L_{D1} = 16\ ft$ All member dimensions are for a W10x22

$D_l = 54\ psf$ See load calculations

$L_l = 100\ psf$ First floor live load for lobby/gathering areas

$t_b = 5\ ft$ Assumed tributary width for joists

$E = 29000\ ksi$ Assumed Modulus of Elasticity of Steel

$f_y = 50\ ksi$ Yield stress

$w_b = 22\ plf$ From AISC Shapes Database

$I = 118\ in^4$ From AISC Shapes Database

$Z_x = 26.0\ in^3$ From AISC Shapes Database

$t_w = 0.240\ in$ From AISC Shapes Database

$t_f = 0.360\ in$ From AISC Shapes Database

$b_f = 5.75\ in$ From AISC Shapes Database

$d = 10.2\ in$ From AISC Shapes Database

$$A_w = \left(d - t_f \cdot 2\right) \cdot t_w = 2.275 \ \boldsymbol{in}^2$$

Member Strength

$M_{all} = 0.9 \cdot f_y \cdot Z_x = 97.5\ \textbf{kip} \cdot \textbf{ft}$

$V_{all} = 0.6 \cdot f_y \cdot A_w = 68.256\ \textbf{kip}$

Strength Requirements

$w_d = t_b \cdot D_l + w_b = 0.292\ \textbf{klf}$

$w_l = t_b \cdot L_l = 0.5\ \textbf{klf}$

$w_u = \max\left(1.4 \cdot w_d, 1.2 \cdot w_d + 1.6 \cdot w_l\right) = \left(1.15 \cdot 10^3\right)\ \textbf{plf}$

Maximum moment

$$M_{max} = w_u \cdot \frac{L_{D1}^2}{8} = 36.813\ \textbf{kip} \cdot \textbf{ft}$$

$Moment$ = "Okay"

Maximum shear

$$V_{max} = w_u \cdot \frac{L_{D1}}{2} = 9.203\ \textbf{kip}$$

$Shear$ = "Okay"

~~Allowable Serviceability Deflections~~

$$8_{Sall} = \frac{L_{D1}}{240} = 0.8\ \textbf{in}$$ Short Term Allowable Deflection

$$8_{Tall} = \frac{L_{D1}}{360} = 0.533\ \textbf{in}$$ Total Allowable Deflection

Serviceability Calculations

$$\delta_{short} = \frac{5 \cdot \frac{w_l}{2} \cdot L_{D1}^{4}}{384 \cdot E \cdot I} = 0.108\ in$$

$Short$ = "Okay"

$$\delta_{long} = \frac{5 \cdot \left(\frac{w_l}{2} + w_d\right) \cdot L_{D1}^{4}}{384 \cdot E \cdot I} = 0.234\ in$$

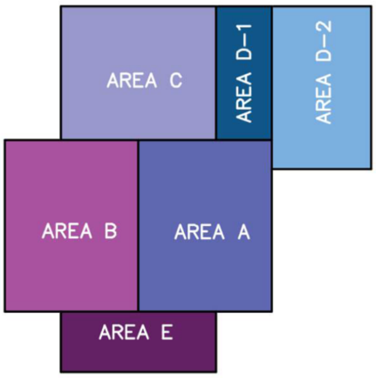
$$\delta_{tot} = \delta_{short} + \delta_{long} = 0.341\ in$$

$Total$ = "Okay"

Design Summary

A W10x22 satisfies all requirements for this preliminary design.

First Floor Joist Area D2 Analysis



Assumptions (if needed)

- Assume W16x26
- LRFD Load combination 2, 1.2D + 1.6L governs floor gravity loads

Define Variables

$L_{D2} = 24\ ft$ All member dimensions are for a W16x26

$D_l = 54\ psf$ See load calculations

$L_l = 100\ psf$ First floor live load for lobby/gathering areas

$t_b = 5\ ft$ Assumed tributary width for joists

$E = 29000\ ksi$ Assumed Modulus of Elasticity of Steel

$f_y = 50\ ksi$ Yield stress

$w_b = 26\ plf$ From AISC Shapes Database

$I = 301\ in^4$ From AISC Shapes Database

$Z_x = 44.2\ in^3$ From AISC Shapes Database

$t_w = 0.250\ in$ From AISC Shapes Database

$t_f = 0.345\ in$ From AISC Shapes Database

$b_f = 5.50\ in$ From AISC Shapes Database

$d = 15.7\ in$ From AISC Shapes Database

$$A_w = \left(d - t_f \cdot 2\right) \cdot t_w = 3.753\ \boldsymbol{in}^2$$

Member Strength

$$M_{all} = 0.9 \cdot f_y \cdot Z_x = 165.75\ \boldsymbol{kip \cdot ft}$$

$$V_{all} = 0.6 \cdot f_y \cdot A_w = 112.575\ \boldsymbol{kip}$$

Strength Requirements

$$w_d = t_b \cdot D_l + w_b = 0.296\ \boldsymbol{klf}$$

$$w_l = t_b \cdot L_l = 0.5\ \boldsymbol{klf}$$

$$w_u = \max\left(1.4 \cdot w_d, 1.2 \cdot w_d + 1.6 \cdot w_l\right) = \left(1.155 \cdot 10^3\right)\ \boldsymbol{plf}$$

Maximum moment

$$M_{max} = w_u \cdot \frac{{L_{D2}}^2}{8} = 83.174\ \boldsymbol{kip \cdot ft}$$

$$Moment = \text{“Okay”}$$

Maximum shear

$$V_{max} = w_u \cdot \frac{L_{D2}}{2} = 13.862\ \boldsymbol{kip}$$

$$Shear = \text{“Okay”}$$

Allowable Serviceability Deflections

$$8_{Sall} = \frac{L_{D2}}{240} = 1.2\ \boldsymbol{in}$$ Short Term Allowable Deflection

$$8_{Tall} = \frac{L_{D2}}{360} = 0.8\ \boldsymbol{in}$$ Total Allowable Deflection

Serviceability Calculations

$$8_{short} = \frac{5 \cdot \frac{w_l}{2} \cdot L_{D2}^{4}}{384 \cdot E \cdot I} = 0.214\ in$$

$Short$ = “Okay”

$$8_{long} = \frac{5 \cdot \left(\frac{w_l}{2} + w_d\right) \cdot L_{D2}^{4}}{384 \cdot E \cdot I} = 0.467\ in$$

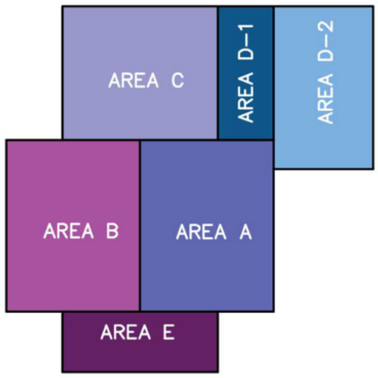
$$8_{tot} = 8_{short} + 8_{long} = 0.681\ in$$

$Total$ = “Okay”

Design Summary

A W16x26 satisfies all requirements for this preliminary design.

First Floor Joist Area E Analysis



Assumptions (if needed)

- Assume W18x35
- LRFD Load combination 2, 1.2D + 1.6L governs floor gravity loads

Define Variables

$L_E = 29\ ft$ All member dimensions are for a W18x35

$D_l = 54\ psf$ See load calculations

$L_l = 100\ psf$ First floor live load for lobby/gathering areas

$t_b = 5\ ft$ Assumed tributary width for joists

$E = 29000\ ksi$ Assumed Modulus of Elasticity of Steel

$f_y = 50\ ksi$ Yield stress

$w_b = 35\ plf$ From AISC Shapes Database

$I = 510\ in^4$ From AISC Shapes Database

$Z_x = 66.5\ in^3$ From AISC Shapes Database

$t_w = 0.300\ in$ From AISC Shapes Database

$t_f = 0.425\ in$ From AISC Shapes Database

$b_f = 6.00\ in$ From AISC Shapes Database

$d = 17.7\ in$ From AISC Shapes Database

$$A_w = \left(d - t_f \cdot 2\right) \cdot t_w = 5.055 \ \mathbf{in}^2$$

Member Strength

$M_{all} = 0.9 \cdot f_y \cdot Z_x = 249.375 \; \mathbf{kip \cdot ft}$

$V_{all} = 0.6 \cdot f_y \cdot A_w = 151.65 \; \mathbf{kip}$

Strength Requirements

$w_d = t_b \cdot D_l + w_b = 0.305 \; \mathbf{klf}$

$w_l = t_b \cdot L_l = 0.5 \; \mathbf{klf}$

$w_u = \max\left(1.4 \cdot w_d , 1.2 \cdot w_d + 1.6 \cdot w_l\right) = \left(1.166 \cdot 10^3\right) \; \mathbf{plf}$

Maximum moment

$$M_{max} = w_u \cdot \frac{L_E^{\,2}}{8} = 122.576 \; \mathbf{kip \cdot ft}$$

Moment = “Okay”

Maximum shear

$$V_{max} = w_u \cdot \frac{L_E}{2} = 16.907 \; \mathbf{kip}$$

Shear = “Okay”

Allowable Serviceability Deflections

$$8_{Sall} = \frac{L_E}{240} = 1.45 \; \mathbf{in}$$ Short Term Allowable Deflection

$$8_{Tall} = \frac{L_E}{360} = 0.967 \; \mathbf{in}$$ Total Allowable Deflection

Serviceability Calculations

$$\delta_{short} = \frac{5 \cdot \frac{w_l}{2} \cdot L_E^{\ 4}}{384 \cdot E \cdot I} = 0.269 \ \mathbf{in}$$

$Short$ = "Okay"

$$\delta_{long} = \frac{5 \cdot \left(\frac{w_l}{2} + w_d \right) \cdot L_E^{\ 4}}{384 \cdot E \cdot I} = 0.597 \ \mathbf{in}$$

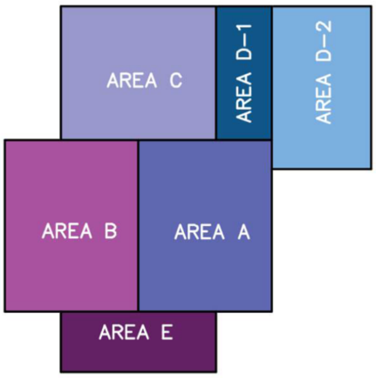
$$\delta_{tot} = \delta_{short} + \delta_{long} = 0.866 \ \mathbf{in}$$

$Total$ = "Okay"

Design Summary

A W18x35 satisfies all requirements for this preliminary design.

Second Floor Joist Area A Analysis



Assumptions (if needed)

- Assume W16x26
- LRFD Load combination 2, 1.2D + 1.6L governs floor gravity loads

Define Variables

$L_A = 28\ ft$ All member dimensions are for a W16x26

$D_l = 54\ psf$ See load calculations

$L_l = 40\ psf$ Proposed use is residential

$t_b = 5\ ft$ Assumed tributary width for joists

$E = 29000\ ksi$ Assumed Modulus of Elasticity of Steel

$f_y = 50\ ksi$ Yield stress

$w_b = 26\ plf$ From AISC Shapes Database

$I = 301\ in^4$ From AISC Shapes Database

$Z_x = 44.2\ in^3$ From AISC Shapes Database

$t_w = 0.250\ in$ From AISC Shapes Database

$t_f = 0.345\ in$ From AISC Shapes Database

$b_f = 5.50\ in$ From AISC Shapes Database

$d = 15.7\ in$ From AISC Shapes Database

$$A_w = \left(d - t_f \cdot 2\right) \cdot t_w = 3.753 \; \boldsymbol{in}^2$$

Member Strength

$$M_{all} = 0.9 \cdot f_y \cdot Z_x = 165.75\ \textbf{kip} \cdot \textbf{ft}$$

$$V_{all} = 0.6 \cdot f_y \cdot A_w = 112.575\ \textbf{kip}$$

Strength Requirements

$$w_d = t_b \cdot D_l + w_b = 0.296\ \textbf{klf}$$

$$w_l = t_b \cdot L_l = 0.2\ \textbf{klf}$$

$$w_u = \max\left(, 1.4 \cdot w_d , 1.2 \cdot w_d + 1.6 \cdot w_l\right) = 675.2\ \textbf{plf}$$

Maximum moment

$$M_{max} = w_u \cdot \frac{L_A{}^2}{8} = 66.17\ \textbf{kip} \cdot \textbf{ft}$$

$Moment$ = “Okay”

Maximum shear

$$V_{max} = w_u \cdot \frac{L_A}{2} = 9.453\ \textbf{kip}$$

$Shear$ = “Okay”

Allowable Serviceability Deflections

$$8_{Sall} = \frac{L_A}{240} = 1.4\ \textbf{in}$$ Short Term Allowable Deflection

$$8_{Tall} = \frac{L_A}{360} = 0.933\ \textbf{in}$$ Total Allowable Deflection

Serviceability Calculations

$$\delta_{short} = \frac{5 \cdot \frac{w_l}{2} \cdot L_A^{\,4}}{384 \cdot E \cdot I} = 0.158\ in$$

$Short$ = “Okay”

$$\delta_{long} = \frac{5 \cdot \left(\frac{w_l}{2} + w_d\right) \cdot L_A^{\,4}}{384 \cdot E \cdot I} = 0.627\ in$$

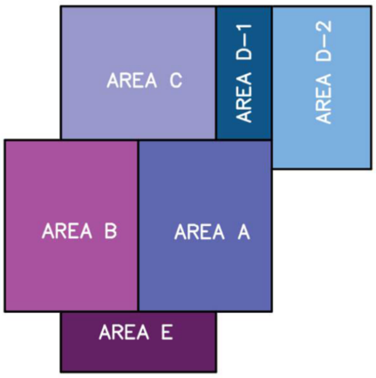
$$\delta_{tot} = \delta_{short} + \delta_{long} = 0.786\ in$$

$Total$ = “Okay”

Design Summary

A W16x26 satisfies all requirements for this preliminary design.

Second Floor Joist Area B Analysis



Assumptions (if needed)

- Assume W16x26
- LRFD Load combination 2, 1.2D + 1.6L governs floor gravity loads

Define Variables

$L_B = 28\ ft$ All member dimensions are for a W16x26

$D_l = 54\ psf$ See load calculations

$L_l = 40\ psf$ Proposed use is residential

$t_b = 5\ ft$ Assumed tributary width for joists

$E = 29000\ ksi$ Assumed Modulus of Elasticity of Steel

$f_y = 50\ ksi$ Yield stress

$w_b = 26\ plf$ From AISC Shapes Database

$I = 301\ in^4$ From AISC Shapes Database

$Z_x = 44.2\ in^3$ From AISC Shapes Database

$t_w = 0.250\ in$ From AISC Shapes Database

$t_f = 0.345\ in$ From AISC Shapes Database

$b_f = 5.50\ in$ From AISC Shapes Database

$d = 15.7\ in$ From AISC Shapes Database

$A_w = \left(d - t_f \cdot 2\right) \cdot t_w = 3.753\ \boldsymbol{in}^2$

Member Strength

$M_{all} = 0.9 \cdot f_y \cdot Z_x = 165.75\ \textbf{kip} \cdot \textbf{ft}$

$V_{all} = 0.6 \cdot f_y \cdot A_w = 112.575\ \textbf{kip}$

Strength Requirements

$w_d = t_b \cdot D_l + w_b = 0.296\ \textbf{klf}$

$w_l = t_b \cdot L_l = 0.2\ \textbf{klf}$

$w_u = \max\left(, 1.4 \cdot w_d, 1.2 \cdot w_d + 1.6 \cdot w_l\right) = 675.2\ \textbf{plf}$

Maximum moment

$M_{max} = w_u \cdot \dfrac{{L_B}^2}{8} = 66.17\ \textbf{kip} \cdot \textbf{ft}$

$Moment$ = "Okay"

Maximum shear

$V_{max} = w_u \cdot \dfrac{L_B}{2} = 9.453\ \textbf{kip}$

$Shear$ = "Okay"

Allowable Serviceability Deflections

$8_{Sall} = \dfrac{L_B}{240} = 1.4\ \textbf{in}$ Short Term Allowable Deflection

$8_{Tall} = \dfrac{L_B}{360} = 0.933\ \textbf{in}$ Total Allowable Deflection

Serviceability Calculations

$$\delta_{short} = \frac{5 \cdot \frac{w_l}{2} \cdot L_B^{\,4}}{384 \cdot E \cdot I} = 0.158\ \textit{in}$$

Short = "Okay"

$$\delta_{long} = \frac{5 \cdot \left(\frac{w_l}{2} + w_d\right) \cdot L_B^{\,4}}{384 \cdot E \cdot I} = 0.627\ \textit{in}$$

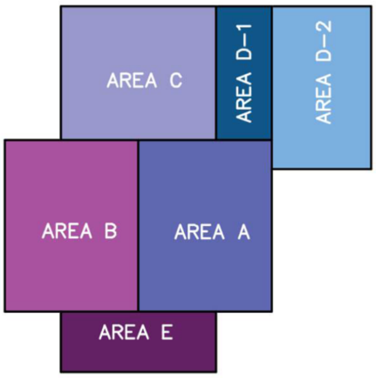
$$\delta_{tot} = \delta_{short} + \delta_{long} = 0.786\ \textit{in}$$

Total = "Okay"

Design Summary

A W16x26 satisfies all requirements for this preliminary design.

Second Floor Joist Area C Analysis



Assumptions (if needed)

- Assume W12x26
- LRFD Load combination 2, 1.2D + 1.6L governs floor gravity loads

Define Variables

$L_C = 26\ \mathbf{ft}$ All member dimensions are for a W12x26

$D_l = 54\ \mathbf{psf}$ See load calculations

$L_l = 40\ \mathbf{psf}$ Proposed use is residential

$t_b = 5\ \mathbf{ft}$ Assumed tributary width for joists

$E = 29000\ \mathbf{ksi}$ Assumed Modulus of Elasticity of Steel

$f_y = 50\ \mathbf{ksi}$ Yield stress

$w_b = 26\ \mathbf{plf}$ From AISC Shapes Database

$I = 204\ \mathbf{in}^4$ From AISC Shapes Database

$Z_x = 37.2\ \mathbf{in}^3$ From AISC Shapes Database

$t_w = 0.230\ \mathbf{in}$ From AISC Shapes Database

$t_f = 0.380\ \mathbf{in}$ From AISC Shapes Database

$b_f = 6.49\ \mathbf{in}$ From AISC Shapes Database

$d = 12.2\ \mathbf{in}$ From AISC Shapes Database

$A_w = \left(d - t_f \cdot 2\right) \cdot t_w = 2.631 \; \boldsymbol{in}^2$

Member Strength

$M_{all} = 0.9 \cdot f_y \cdot Z_x = 139.5\ \mathbf{kip \cdot ft}$

$V_{all} = 0.6 \cdot f_y \cdot A_w = 78.936\ \mathbf{kip}$

Strength Requirements

$w_d = t_b \cdot D_l + w_b = 0.296\ \mathbf{klf}$

$w_l = t_b \cdot L_l = 0.2\ \mathbf{klf}$

$w_u = \max(, 1.4 \cdot w_d, 1.2 \cdot w_d + 1.6 \cdot w_l) = 675.2\ \mathbf{plf}$

Maximum moment

$$M_{max} = w_u \cdot \frac{L_C^{\ 2}}{8} = 57.054\ \mathbf{kip \cdot ft}$$

$Moment$ = “Okay”

Maximum shear

$$V_{max} = w_u \cdot \frac{L_C}{2} = 8.778\ \mathbf{kip}$$

$Shear$ = “Okay”

Allowable Serviceability Deflections

$$8_{Sall} = \frac{L_C}{240} = 1.3\ \mathbf{in}$$ Short Term Allowable Deflection

$$8_{Tall} = \frac{L_C}{360} = 0.867\ \mathbf{in}$$ Total Allowable Deflection

Serviceability Calculations

$$\delta_{short} = \frac{5 \cdot \frac{w_l}{2} \cdot L_C^{\ 4}}{384 \cdot E \cdot I} = 0.174\ in$$

$Short$ = “Okay”

$$\delta_{long} = \frac{5 \cdot \left(\frac{w_l}{2} + w_d\right) \cdot L_C^{\ 4}}{384 \cdot E \cdot I} = 0.688\ in$$

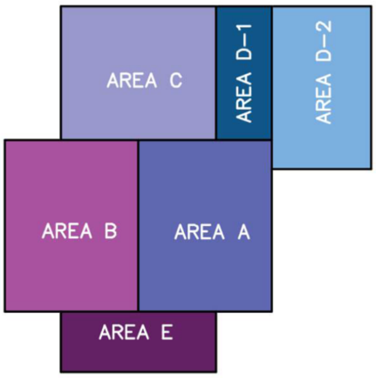
$$\delta_{tot} = \delta_{short} + \delta_{long} = 0.862\ in$$

$Total$ = “Okay”

Design Summary

A W12x26 satisfies all requirements for this preliminary design.

Second Floor Joist Area D1 Analysis



Assumptions (if needed)

- Assume W8x18
- LRFD Load combination 2, 1.2D + 1.6L governs floor gravity loads

Define Variables

$L_{D1} = 16\ \mathbf{ft}$ All member dimensions are for a W8x18

$D_l = 54\ \mathbf{psf}$ See load calculations

$L_l = 40\ \mathbf{psf}$ Proposed use is residential

$t_b = 5\ \mathbf{ft}$ Assumed tributary width for joists

$E = 29000\ \mathbf{ksi}$ Assumed Modulus of Elasticity of Steel

$f_y = 50\ \mathbf{ksi}$ Yield stress

$w_b = 18\ \mathbf{plf}$ From AISC Shapes Database

$I = 61.9\ \mathbf{in}^4$ From AISC Shapes Database

$Z_x = 17.0\ \mathbf{in}^3$ From AISC Shapes Database

$t_w = 0.230\ \mathbf{in}$ From AISC Shapes Database

$t_f = 0.330\ \mathbf{in}$ From AISC Shapes Database

$b_f = 5.25\ \mathbf{in}$ From AISC Shapes Database

$d = 8.14\ \mathbf{in}$ From AISC Shapes Database

$A_w = \left(d - t_f \cdot 2\right) \cdot t_w = 1.72\ \boldsymbol{in}^2$

Member Strength

$M_{all} = 0.9 \cdot f_y \cdot Z_x = 63.75\ \mathbf{kip \cdot ft}$

$V_{all} = 0.6 \cdot f_y \cdot A_w = 51.612\ \mathbf{kip}$

Strength Requirements

$w_d = t_b \cdot D_l + w_b = 0.288\ \mathbf{klf}$

$w_l = t_b \cdot L_l = 0.2\ \mathbf{klf}$

$w_u = \max\left(,1.4 \cdot w_d, 1.2 \cdot w_d + 1.6 \cdot w_l\right) = 665.6\ \mathbf{plf}$

Maximum moment

$$M_{max} = w_u \cdot \frac{{L_{D1}}^2}{8} = 21.299\ \mathbf{kip \cdot ft}$$

$Moment$ = "Okay"

Maximum shear

$$V_{max} = w_u \cdot \frac{L_{D1}}{2} = 5.325\ \mathbf{kip}$$

$Shear$ = "Okay"

~~Allowable Serviceability Deflections~~

$$8_{Sall} = \frac{L_{D1}}{240} = 0.8\ \mathbf{in}$$ Short Term Allowable Deflection

$$8_{Tall} = \frac{L_{D1}}{360} = 0.533\ \mathbf{in}$$ Total Allowable Deflection

Serviceability Calculations

$$\delta_{short} = \frac{5 \cdot \frac{w_l}{2} \cdot L_{D1}^{4}}{384 \cdot E \cdot I} = 0.082\ in$$

$Short$ = "Okay"

$$\delta_{long} = \frac{5 \cdot \left(\frac{w_l}{2} + w_d\right) \cdot L_{D1}^{4}}{384 \cdot E \cdot I} = 0.319\ in$$

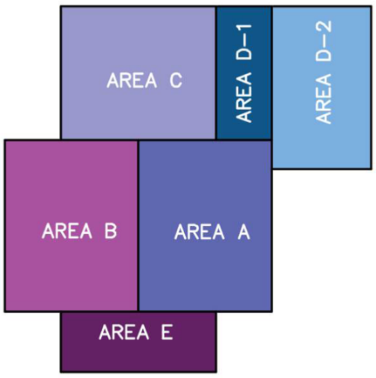
$$\delta_{tot} = \delta_{short} + \delta_{long} = 0.401\ in$$

$Total$ = "Okay"

Design Summary

A W8x18 satisfies all requirements for this preliminary design.

Second Floor Joist Area D2 Analysis



Assumptions (if needed)

- Assume W12x26
- LRFD Load combination 2, 1.2D + 1.6L governs floor gravity loads

Define Variables

$L_{D2} = 24\ ft$ All member dimensions are for a W12x26

$D_l = 54\ psf$ See load calculations

$L_l = 40\ psf$ Proposed use is residential

$t_b = 5\ ft$ Assumed tributary width for joists

$E = 29000\ ksi$ Assumed Modulus of Elasticity of Steel

$f_y = 50\ ksi$ Yield stress

$w_b = 26\ plf$ From AISC Shapes Database

$I = 204\ in^4$ From AISC Shapes Database

$Z_x = 37.2\ in^3$ From AISC Shapes Database

$t_w = 0.230\ in$ From AISC Shapes Database

$t_f = 0.380\ in$ From AISC Shapes Database

$b_f = 6.49\ in$ From AISC Shapes Database

$d = 12.2\ in$ From AISC Shapes Database

$$A_w = \left(d - t_f \cdot 2\right) \cdot t_w = 2.631 \ \boldsymbol{in}^2$$

Member Strength

$M_{all} = 0.9 \cdot f_y \cdot Z_x = 139.5\ \mathbf{kip \cdot ft}$

$V_{all} = 0.6 \cdot f_y \cdot A_w = 78.936\ \mathbf{kip}$

Strength Requirements

$w_d = t_b \cdot D_l + w_b = 0.296\ \mathbf{klf}$

$w_l = t_b \cdot L_l = 0.2\ \mathbf{klf}$

$w_u = \max\left(, 1.4 \cdot w_d, 1.2 \cdot w_d + 1.6 \cdot w_l\right) = 675.2\ \mathbf{plf}$

Maximum moment

$$M_{max} = w_u \cdot \frac{L_{D2}^{\ 2}}{8} = 48.614\ \mathbf{kip \cdot ft}$$

$Moment$ = “Okay”

Maximum shear

$$V_{max} = w_u \cdot \frac{L_{D2}}{2} = 8.102\ \mathbf{kip}$$

$Shear$ = “Okay”

~~Allowable Serviceability Deflections~~

$8_{Sall} = \frac{L_{D2}}{240} = 1.2\ \mathbf{in}$ Short Term Allowable Deflection

$8_{Tall} = \frac{L_{D2}}{360} = 0.8\ \mathbf{in}$ Total Allowable Deflection

Serviceability Calculations

$$\delta_{short} = \frac{5 \cdot \frac{w_l}{2} \cdot L_{D2}^{\ 4}}{384 \cdot E \cdot I} = 0.126\ in$$

$Short$ = "Okay"

$$\delta_{long} = \frac{5 \cdot \left(\frac{w_l}{2} + w_d\right) \cdot L_{D2}^{\ 4}}{384 \cdot E \cdot I} = 0.5\ in$$

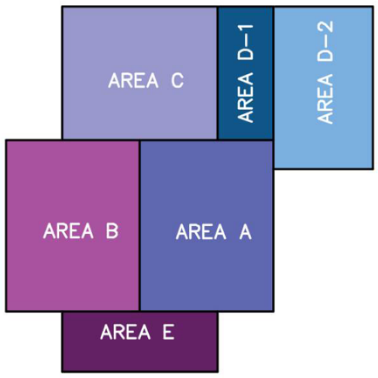
$$\delta_{tot} = \delta_{short} + \delta_{long} = 0.626\ in$$

$Total$ = "Okay"

Design Summary

A W12x26 satisfies all requirements for this preliminary design.

Second Floor Joist Area E Analysis



Assumptions (if needed)

- Assume W18x35
- LRFD Load combination 2, 1.2D + 1.6L governs floor gravity loads

Define Variables

$L_E = 29\ ft$ All member dimensions are for a W18x35

$D_l = 54\ psf$ See load calculations

$L_l = 40\ psf$ Proposed use is residential

$t_b = 5\ ft$ Assumed tributary width for joists

$E = 29000\ ksi$ Assumed Modulus of Elasticity of Steel

$f_y = 50\ ksi$ Yield stress

$w_b = 35\ plf$ From AISC Shapes Database

$I = 510\ in^4$ From AISC Shapes Database

$Z_x = 66.5\ in^3$ From AISC Shapes Database

$t_w = 0.300\ in$ From AISC Shapes Database

$t_f = 0.425\ in$ From AISC Shapes Database

$b_f = 6.00\ in$ From AISC Shapes Database

$d = 17.7\ in$ From AISC Shapes Database

$$A_w = \left(d - t_f \cdot 2\right) \cdot t_w = 5.055 \ \mathbf{in}^2$$

Member Strength

$M_{all} = 0.9 \cdot f_y \cdot Z_x = 249.375\ \mathbf{kip \cdot ft}$

$V_{all} = 0.6 \cdot f_y \cdot A_w = 151.65\ \mathbf{kip}$

Strength Requirements

$w_d = t_b \cdot D_l + w_b = 0.305\ \mathbf{klf}$

$w_l = t_b \cdot L_l = 0.2\ \mathbf{klf}$

$w_u = \max\left(,1.4 \cdot w_d, 1.2 \cdot w_d + 1.6 \cdot w_l\right) = 686\ \mathbf{plf}$

Maximum moment

$M_{max} = w_u \cdot \frac{{L_E}^2}{8} = 72.116\ \mathbf{kip \cdot ft}$

Moment = “Okay”

Maximum shear

$V_{max} = w_u \cdot \frac{L_E}{2} = 9.947\ \mathbf{kip}$

Shear = “Okay”

Allowable Serviceability Deflections

$8_{Sall} = \frac{L_E}{240} = 1.45\ \mathbf{in}$ Short Term Allowable Deflection

$8_{Tall} = \frac{L_E}{360} = 0.967\ \mathbf{in}$ Total Allowable Deflection

Serviceability Calculations

$$8_{short} = \frac{5 \cdot \frac{w_l}{2} \cdot L_E^{4}}{384 \cdot E \cdot I} = 0.108 \ \boldsymbol{in}$$

$Short$ = “Okay”

$$8_{long} = \frac{5 \cdot \left(\frac{w_l}{2} + w_d \right) \cdot L_E^{4}}{384 \cdot E \cdot I} = 0.436 \ \boldsymbol{in}$$

$$8_{tot} = 8_{short} + 8_{long} = 0.543 \ \boldsymbol{in}$$

$Total$ = “Okay”

Design Summary

A W18x35 satisfies all requirements for this preliminary design.

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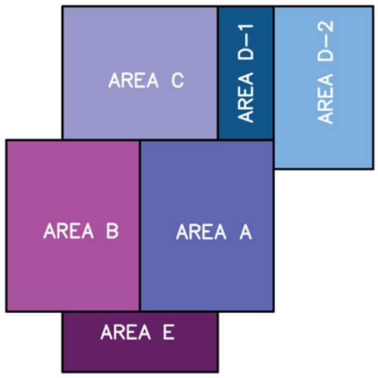
First Floor Beam Area A/B Analysis



Assumptions (if needed)

- Risk factor IV
- Cold ventilated roof
- LRFD Load combination 2, 1.2D + 1.6L governs floor gravity loads
- Assume W21x68

Define Variables

$L = 19\ ft$ All member dimensions are for a W21x68

$D_l = 54\ psf$ See load calculations

$L_l = 100\ psf$ First floor live load for lobby/gathering areas

$L_A = 28\ ft$ Joist length

$w_b = 36\ plf$ Joist weight from AISC Shapes Database

$t_b = 5\ ft$ Assumed tributary width for joists

$E = 29000\ ksi$ Assumed Modulus of Elasticity of Steel

$f_y = 50\ ksi$ Yield stress

$I = 1480\ in^4$ From AISC Shapes Database

$Z_x = 160\ in^3$ From AISC Shapes Database

$t_w = 0.430\ in$ From AISC Shapes Database

$t_f = 0.685\ in$ From AISC Shapes Database

$b_f = 8.27\ in$ From AISC Shapes Database

$d = 21.1\ in$ From AISC Shapes Database

$A_w = \left(d - t_f \cdot 2\right) \cdot t_w = 8.484\ in^2$

Calculation of Point Loads

$P_d = L_A \cdot (\backslash t_b \cdot D_l + w_b) = 8.568$ ***kip***

$P_l = t_b \cdot L_l \cdot L_A = 14$ ***kip***

$P_u = \max(\backslash 1.4 \cdot P_d, 1.2 \cdot P_d + 1.6 \cdot P_l) = 32.682$ ***kip***

Moment, Shear, Deflection analysis performed in ROBOT. Results are summarized below.

Member Strength

$M_{all} = 0.9 \cdot f_y \cdot Z_x = 600$ ***kip•ft***

$V_{all} = 0.6 \cdot f_y \cdot A_w = 254.517$ ***kip***

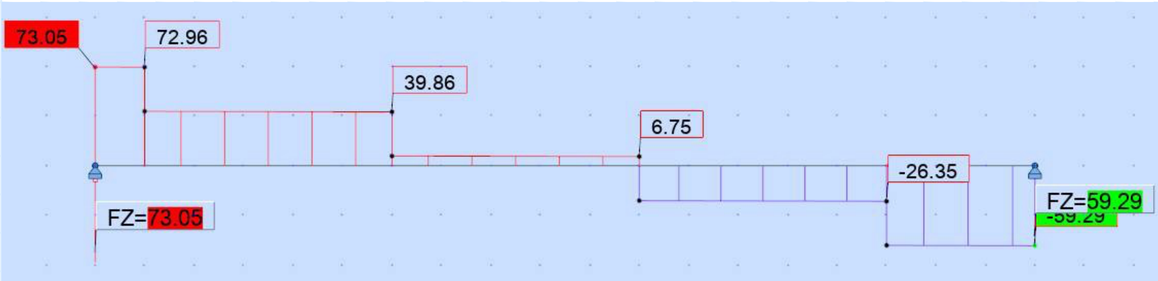
Strength Requirements

Maximum moment

$M_{max} = 308.17$ ***kip•ft***

Moment = "Okay"

Maximum shear



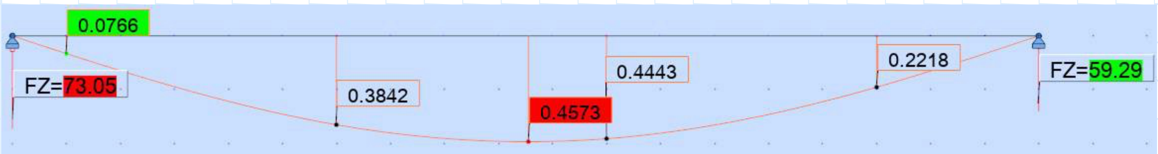
$V_{max} = 73.05$ ***kip***

Shear = "Okay"

Allowable Serviceability Deflections

${}_{Tall} = \frac{L}{360} = 0.633$ ***in*** Total allowable deflection

Serviceability Calculations



${}_{tot} = 0.4573$ ***in***

Total = "Okay"

Design Summary

A W21x68 satisfies all requirements for this preliminary design.

First Floor Beam Area C Analysis

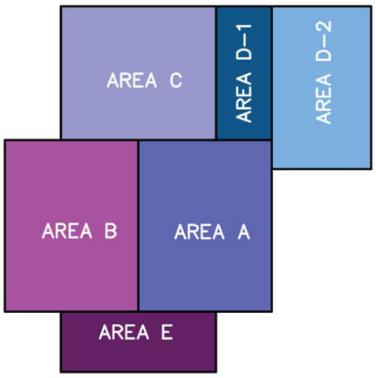
Assumptions (if needed)

- Risk factor IV
- Cold ventilated roof
- LRFD Load combination 2, 1.2D + 1.6L governs floor gravity loads
- Assume W21x44

Define Variables

$L = 18\ ft$	All member dimensions are for a W21x44
$D_l = 54\ psf$	See load calculations
$L_l = 100\ psf$	First floor live load for lobby/gathering areas
$L_C = 26\ ft$	Joist length
$w_b = 31\ plf$	Joist weight from AISC Shapes Database
$t_b = 5\ ft$	Assumed tributary width for joists
$E = 29000\ ksi$	Assumed Modulus of Elasticity of Steel
$f_y = 50\ ksi$	Yield stress
$I = 843\ in^4$	From AISC Shapes Database
$Z_x = 95.4\ in^3$	From AISC Shapes Database
$t_w = 0.350\ in$	From AISC Shapes Database
$t_f = 0.450\ in$	From AISC Shapes Database

$b_f = 6.50$ ***in*** From AISC Shapes Database



$d = 20.7$ ***in*** From AISC Shapes Database

$A_w = \left(d - t_f \cdot 2\right) \cdot t_w = 6.93$ ***in***^{2}

Calculation of Point Loads

$P_d = L_C \cdot (t_b \cdot D_l + w_b) = 7.826$ ***kip***

$P_l = t_b \cdot L_l \cdot L_C = 13$ ***kip***

$P_u = \max(1.4 \cdot P_d, 1.2 \cdot P_d + 1.6 \cdot P_l) = 30.191$ ***kip***

Moment, Shear, Deflection analysis performed in ROBOT. Results are summarized below.

Member Strength

$M_{all} = 0.9 \cdot f_y \cdot Z_x = 357.75$ ***kip • ft***

$V_{all} = 0.6 \cdot f_y \cdot A_w = 207.9$ ***kip***

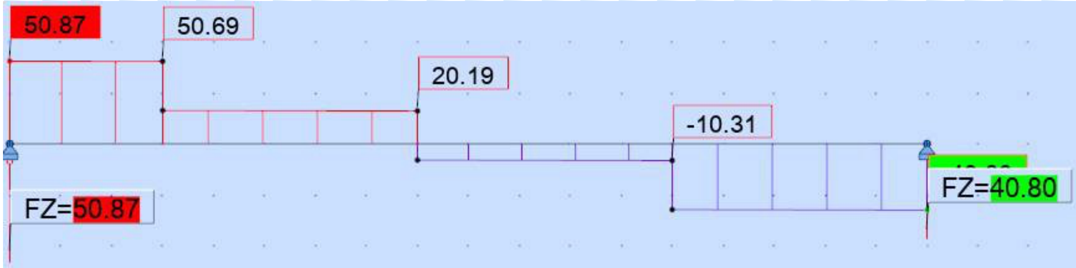
Strength Requirements

Maximum moment

$M_{max} = 254.03$ ***kip • ft***

Moment = "Okay"

Maximum shear



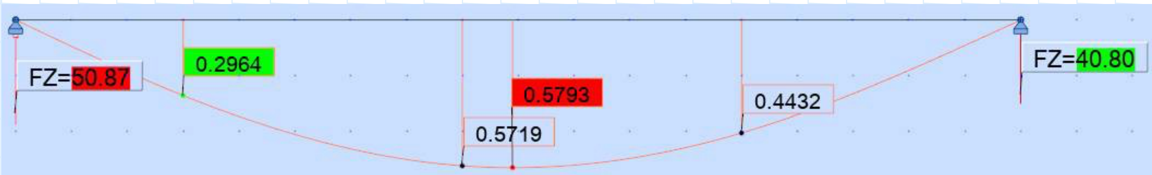
$V_{max} = 50.87$ ***kip***

Shear = "Okay"

Allowable Serviceability Deflections

$_{Tall} = \frac{L}{360} = 0.6$ ***in*** Total allowable deflection

Serviceability Calculations



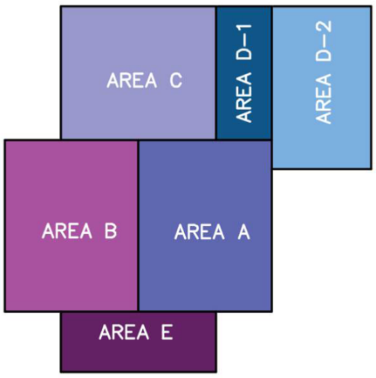
$_{tot} = 0.5793$ ***in***

Total = "Okay"

Design Summary

A W21x44 satisfies all requirements for this preliminary design.

First Floor Beam Area D Analysis



Assumptions (if needed)

- Risk factor IV
- Cold ventilated roof
- LRFD Load combination 2, 1.2D + 1.6L governs floor gravity loads
- Assume W21x44

Define Variables

$L=18\ ft$ All member dimensions are for a W21x44

$D_l=54\ psf$ See load calculations

$L_l=100\ psf$ First floor live load for lobby/gathering areas

$L_{D1}=24\ ft$ Joist length

$w_b=26\ plf$ Joist weight from AISC Shapes Database

$t_b=5\ ft$ Assumed tributary width for joists

$E=29000\ ksi$ Assumed Modulus of Elasticity of Steel

$f_y=50\ ksi$ Yield stress

$I=843\ in^4$ From AISC Shapes Database

$Z_x=95.4\ in^3$ From AISC Shapes Database

$t_w=0.350\ in$ From AISC Shapes Database

$t_f=0.450\ in$ From AISC Shapes Database

$b_f=6.50\ in$ From AISC Shapes Database

$d = 20.7\ \textbf{in}$ From AISC Shapes Database

$A_w = \left(d - t_f \cdot 2\right) \cdot t_w = 6.93\ \textbf{in}^2$

Calculation of Point Loads

$P_d = L_{D1} \cdot (t_b \cdot D_l + w_b) = 7.104$ ***kip***

$P_l = t_b \cdot L_l \cdot L_{D1} = 12$ ***kip***

$P_u = \max(1.4 \cdot P_d, 1.2 \cdot P_d + 1.6 \cdot P_l) = 27.725$ ***kip*** $P_u \cdot 2 = 55.45$ ***kip***

Moment, Shear, Deflection analysis performed in ROBOT. Results are summarized below.

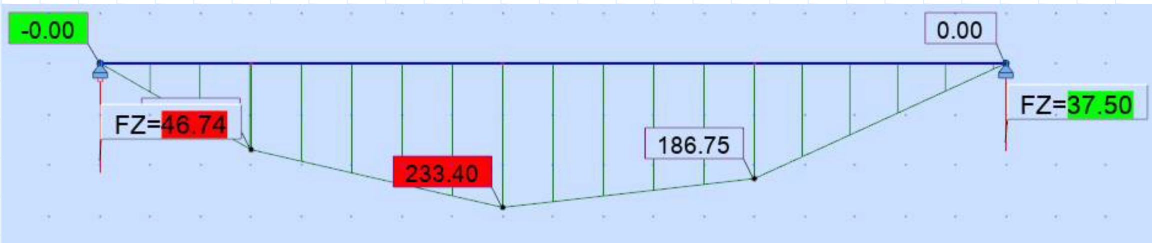
Member Strength

$M_{all} = 0.9 \cdot f_y \cdot Z_x = 357.75$ ***kip • ft***

$V_{all} = 0.6 \cdot f_y \cdot A_w = 207.9$ ***kip***

Strength Requirements

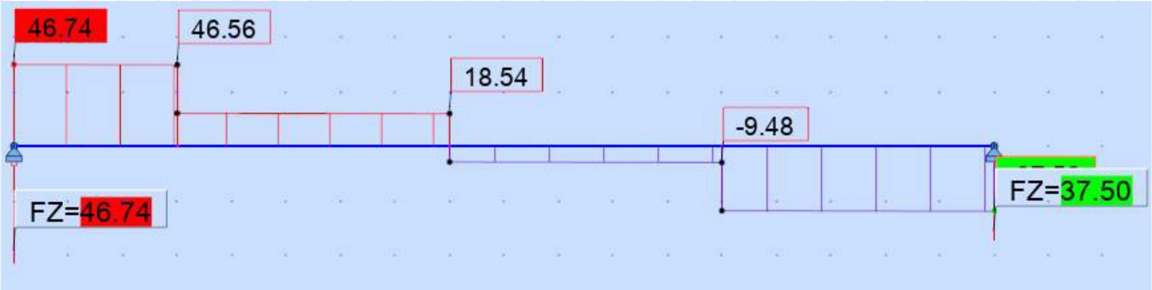
Maximum moment



$M_{max} = 233.40$ ***kip • ft***

$Moment$ = "Okay"

Maximum shear



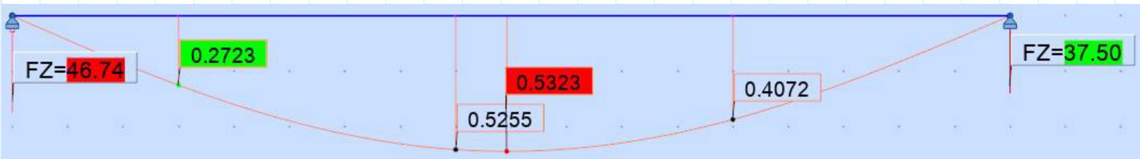
$V_{max} = 46.74$ ***kip***

Shear = "Okay"

Allowable Serviceability Deflections

$_{Tall} = \frac{L}{360} = 0.6$ ***in*** Total allowable deflection

Serviceability Analysis



$_{tot} = 0.5325$ ***in***

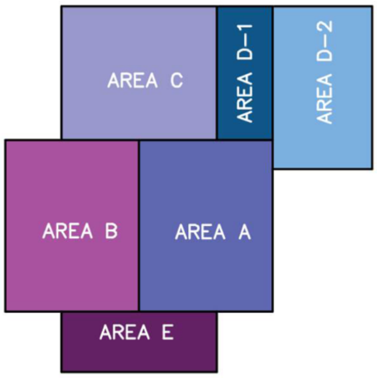
Total = "Okay"

Design Summary

A W21x44 satisfies all requirements for this preliminary design.

First Floor Beam Area E Analysis

Assumptions (if needed)



- Risk factor IV
- Cold ventilated roof
- LRFD Load combination 2, 1.2D + 1.6L governs floor gravity loads
- Assume W21x44

Define Variables

$L=14\ ft$	All member dimensions are for a W21x44
$D_l=54\ psf$	See load calculations
$L_l=100\ psf$	First floor live load for lobby/gathering areas
$L_E=24\ ft$	Joist length
$w_b=35\ plf$	Joist weight from AISC Shapes Database
$t_b=5\ ft$	Assumed tributary width for joists
$E=29000\ ksi$	Assumed Modulus of Elasticity of Steel
$f_y=50\ ksi$	Yield stress
$I=843\ in^4$	From AISC Shapes Database
$Z_x=95.4\ in^3$	From AISC Shapes Database
$t_w=0.350\ in$	From AISC Shapes Database
$t_f=0.450\ in$	From AISC Shapes Database
$b_f=6.50\ in$	From AISC Shapes Database

$d = 20.7\ \textit{in}$ From AISC Shapes Database

$A_w = \left(d - t_f \cdot 2\right) \cdot t_w = 6.93\ \textit{in}^2$

Calculation of Point Loads

$P_d = L_E \cdot (t_b \cdot D_l + w_b) = 7.32$ ***kip***

$P_l = t_b \cdot L_l \cdot L_E = 12$ ***kip***

$P_u = \max(1.4 \cdot P_d, 1.2 \cdot P_d + 1.6 \cdot P_l) = 27.984$ ***kip***

Moment, Shear, Deflection analysis performed in ROBOT. Results are summarized below.

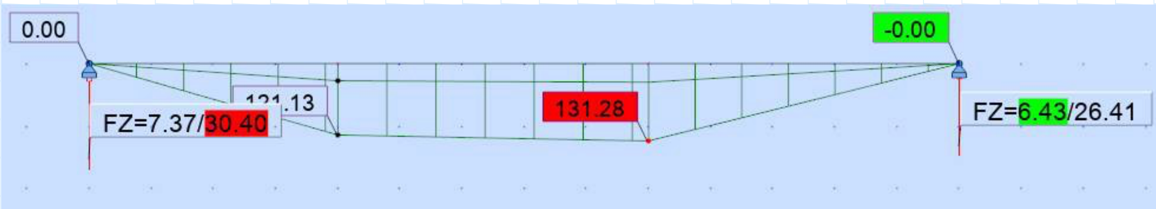
Member Strength

$M_{all} = 0.9 \cdot f_y \cdot Z_x = 357.75$ ***kip • ft***

$V_{all} = 0.6 \cdot f_y \cdot A_w = 207.9$ ***kip***

Strength Requirements

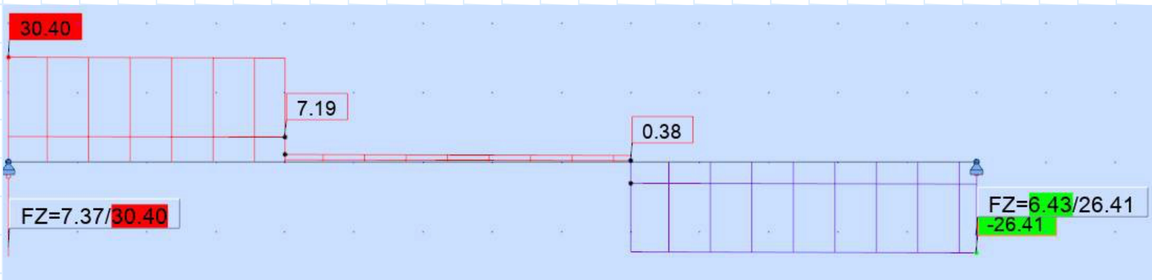
Maximum moment



$M_{max} = 131.28$ ***kip • ft***

Moment = “Okay”

Maximum shear



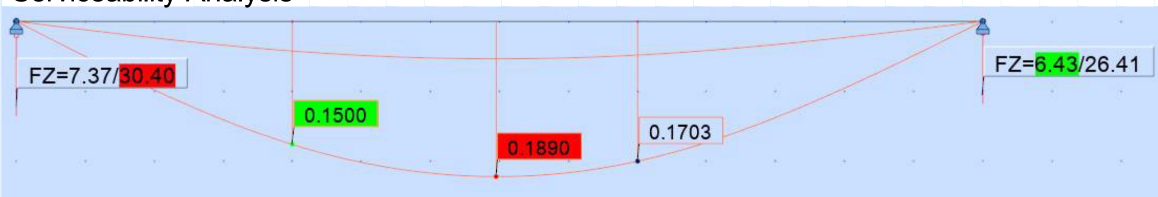
$V_{max} = 30.40$ ***kip***

Shear = “Okay”

Allowable Serviceability Deflections

$_{Tall} = \frac{L}{360} = 0.467\ \boldsymbol{in}$ Total allowable deflection

Serviceability Analysis



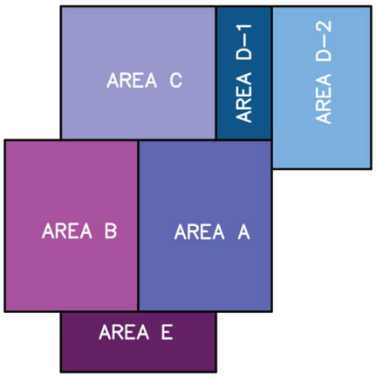
$_{tot} = 0.1890\ \boldsymbol{in}$

Total = “Okay”

Design Summary

A W21x44 satisfies all requirements for this preliminary design.

Second Floor Beam Area A/B Analysis



Assumptions (if needed)

- Risk factor IV
- Cold ventilated roof
- LRFD Load combination 2, 1.2D + 1.6L governs floor gravity loads
- Assume W21x44

Define Variables

$L = 19\ \textbf{ft}$ All member dimensions are for a W21x44

$D_l = 54\ \textbf{psf}$ See load calculations

$L_l = 40\ \textbf{psf}$ First floor live load for lobby/gathering areas

$L_A = 28\ \textbf{ft}$ Joist length

$w_b = 26\ \textbf{plf}$ Joist weight from AISC Shapes Database

$t_b = 5\ \textbf{ft}$ Assumed tributary width for joists

$E = 29000\ \textbf{ksi}$ Assumed Modulus of Elasticity of Steel

$f_y = 50\ \textbf{ksi}$ Yield stress

$I = 843\ \textbf{in}^4$ From AISC Shapes Database

$Z_x = 95.4\ \textbf{in}^3$ From AISC Shapes Database

$t_w = 0.350\ \textbf{in}$ From AISC Shapes Database

$t_f = 0.450\ \textbf{in}$ From AISC Shapes Database

$b_f = 6.50\ \textbf{in}$ From AISC Shapes Database

$d = 20.7\ in$ From AISC Shapes Database

$A_w = \left(d - t_f \cdot 2\right) \cdot t_w = 6.93\ in^2$

Calculation of Point Loads

$P_d = L_A \cdot (t_b \cdot D_l + w_b) = 8.288$ ***kip***

$P_l = t_b \cdot L_l \cdot L_A = 5.6$ ***kip***

$P_u = \max(1.4 \cdot P_d, 1.2 \cdot P_d + 1.6 \cdot P_l) = 18.906$ ***kip***

Moment, Shear, Deflection analysis performed in ROBOT. Results are summarized below.

Member Strength

$M_{all} = 0.9 \cdot f_y \cdot Z_x = 357.75$ ***kip • ft***

$V_{all} = 0.6 \cdot f_y \cdot A_w = 207.9$ ***kip***

Strength Requirements

Maximum moment

$M_{max} = 178.78$ ***kip • ft***

Moment = “Okay”

Maximum shear

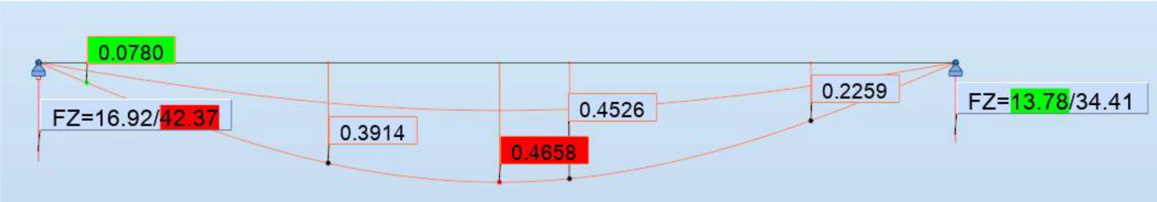
$V_{max} = 42.37$ ***kip***

Shear = “Okay”

Allowable Serviceability Deflections

$_{Tall} = \dfrac{L}{360} = 0.633$ ***in*** Total allowable deflection

Serviceability Analysis



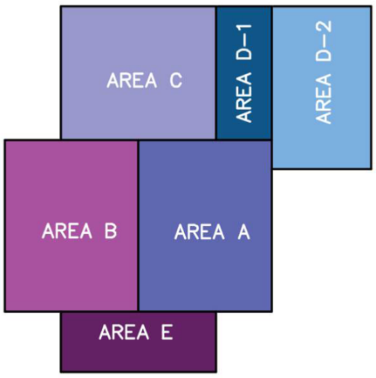
$_{tot} = 0.4658$ ***in***

Total = “Okay”

Design Summary

A W21x44 satisfies all requirements for this preliminary design.

Second Floor Beam Area C Analysis



Assumptions (if needed)

- Risk factor IV
- Cold ventilated roof
- LRFD Load combination 2, 1.2D + 1.6L governs floor gravity loads
- Assume W21x44

Define Variables

$L = 18\ ft$	All member dimensions are for a W21x44
$D_l = 54\ psf$	See load calculations
$L_l = 40\ psf$	First floor live load for lobby/gathering areas
$L_C = 26\ ft$	Joist length
$w_b = 31\ plf$	Joist weight from AISC Shapes Database
$t_b = 5\ ft$	Assumed tributary width for joists
$E = 29000\ ksi$	Assumed Modulus of Elasticity of Steel
$f_y = 50\ ksi$	Yield stress
$I = 843\ in^4$	From AISC Shapes Database
$Z_x = 95.4\ in^3$	From AISC Shapes Database
$t_w = 0.350\ in$	From AISC Shapes Database
$t_f = 0.450\ in$	From AISC Shapes Database
$b_f = 6.50\ in$	From AISC Shapes Database

$d = 20.7\ \boldsymbol{in}$ From AISC Shapes Database

$A_w = \left(d - t_f \cdot 2\right) \cdot t_w = 6.93\ \boldsymbol{in}^2$

Calculation of Point Loads

$P_d = L_C \cdot \left(t_b \cdot D_l + w_b \right) = 7.826$ ***kip***

$P_l = t_b \cdot L_l \cdot L_C = 5.2$ ***kip***

$P_u = \max\left(1.4 \cdot P_d , 1.2 \cdot P_d + 1.6 \cdot P_l \right) = 17.711$ ***kip***

Moment, Shear, Deflection analysis performed in ROBOT. Results are summarized below.

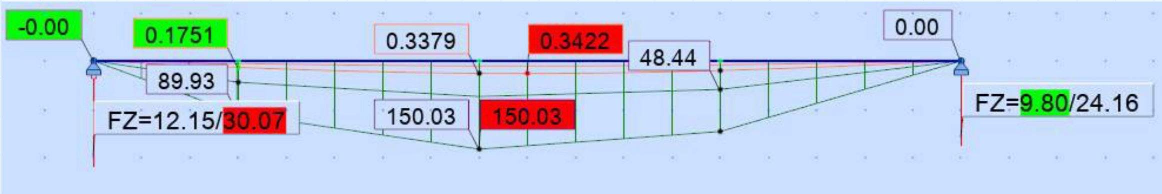
Member Strength

$M_{all} = 0.9 \cdot f_y \cdot Z_x = 357.75$ ***kip • ft***

$V_{all} = 0.6 \cdot f_y \cdot A_w = 207.9$ ***kip***

Strength Requirements

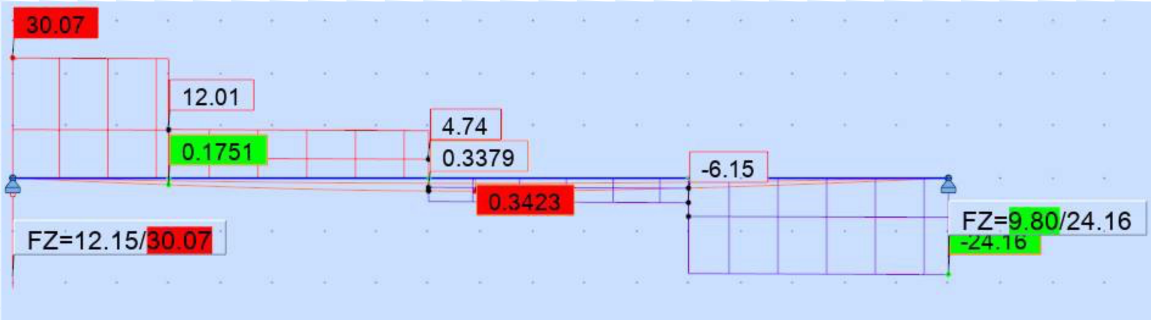
Maximum moment



$M_{max} = 150.03$ ***kip • ft***

Moment = "Okay"

Maximum shear



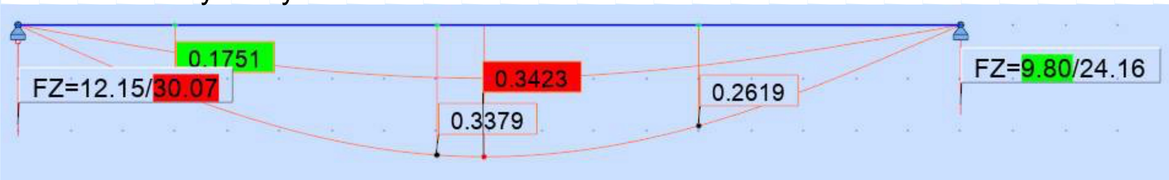
$V_{max} = 30.07$ ***kip***

Shear = "Okay"

Allowable Serviceability Deflections

$$_{Tall} = \frac{L}{360} = 0.6 \; \boldsymbol{in}$$ Total allowable deflection

Serviceability Analysis



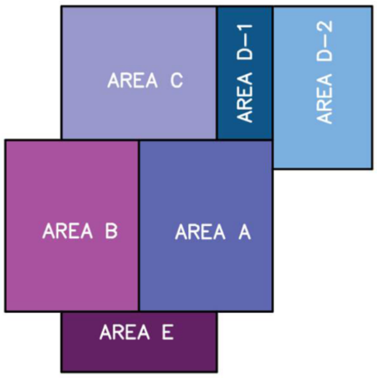
$$_{tot} = 0.3423 \; \boldsymbol{in}$$

Total = “Okay”

Design Summary

A W21x44 satisfies all requirements for this preliminary design.

Second Floor Beam Area D Analysis



Assumptions (if needed)

- Risk factor IV
- Cold ventilated roof
- LRFD Load combination 2, 1.2D + 1.6L governs floor gravity loads
- Assume W21x44

Define Variables

$L = 18\ ft$ All member dimensions are for a W21x44

$D_l = 54\ psf$ See load calculations

$L_l = 40\ psf$ First floor live load for lobby/gathering areas

$L_{D1} = 24\ ft$ Joist length

$w_b = 26\ plf$ Joist weight from AISC Shapes Database

$t_b = 5\ ft$ Assumed tributary width for joists

$E = 29000\ ksi$ Assumed Modulus of Elasticity of Steel

$f_y = 50\ ksi$ Yield stress

$I = 843\ in^4$ From AISC Shapes Database

$Z_x = 95.4\ in^3$ From AISC Shapes Database

$t_w = 0.350\ in$ From AISC Shapes Database

$t_f = 0.450\ in$ From AISC Shapes Database

$b_f = 6.50\ in$ From AISC Shapes Database

$d = 20.7$ ***in*** From AISC Shapes Database

$A_w = (d - t_f \cdot 2) \cdot t_w = 6.93$ $\boldsymbol{in}^2$

Calculation of Point Loads

$P_d = L_{D1} \cdot (t_b \cdot D_l + w_b) = 7.104$ ***kip***

$P_l = t_b \cdot L_l \cdot L_{D1} = 4.8$ ***kip***

$P_u = \max(1.4 \cdot P_d, 1.2 \cdot P_d + 1.6 \cdot P_l) = 16.205$ ***kip***

Moment, Shear, Deflection analysis performed in ROBOT. Results are summarized below.

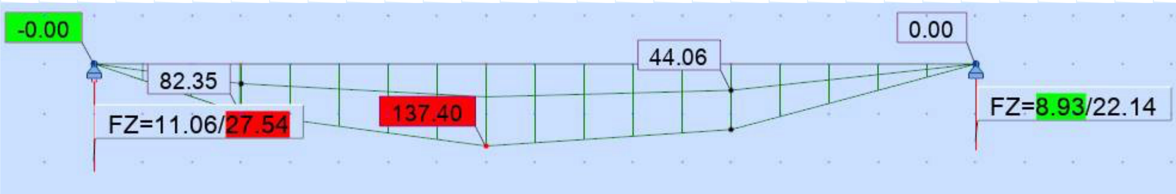
Member Strength

$M_{all} = 0.9 \cdot f_y \cdot Z_x = 357.75$ ***kip • ft***

$V_{all} = 0.6 \cdot f_y \cdot A_w = 207.9$ ***kip***

Strength Requirements

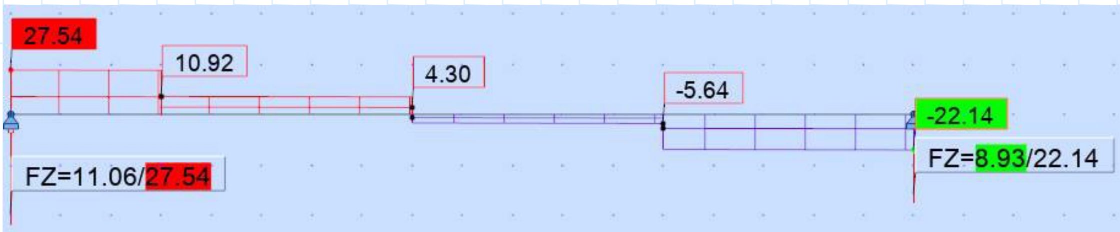
Maximum moment



$M_{max} = 137.40$ ***kip • ft***

Moment = “Okay”

Maximum shear



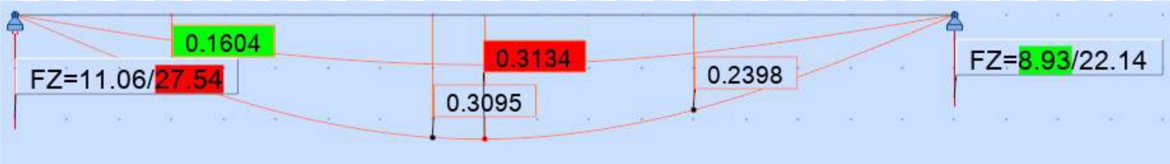
$V_{max} = 27.54$ ***kip***

Shear = “Okay”

Allowable Serviceability Deflections

$$_{Tall} = \frac{L}{360} = 0.6\ in$$ Total allowable deflection

Serviceability Analysis



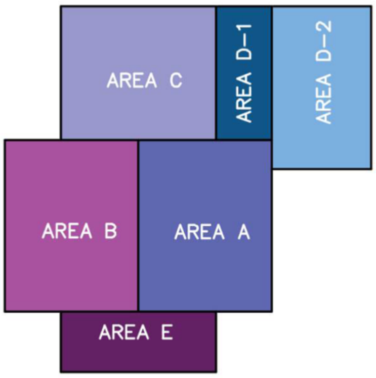
$$_{tot} = 0.3134\ in$$

Total = "Okay"

Design Summary

A W21x44 satisfies all requirements for this preliminary design.

Second Floor Beam Area E Analysis



Assumptions (if needed)

- Risk factor IV
- Cold ventilated roof
- LRFD Load combination 2, 1.2D + 1.6L governs floor gravity loads
- Assume W21x44

Define Variables

$L = 14\ ft$	All member dimensions are for a W21x44
$D_l = 54\ psf$	See load calculations
$L_l = 40\ psf$	First floor live load for lobby/gathering areas
$L_E = 24\ ft$	Joist length
$w_b = 35\ plf$	Joist weight from AISC Shapes Database
$t_b = 5\ ft$	Assumed tributary width for joists
$E = 29000\ ksi$	Assumed Modulus of Elasticity of Steel
$f_y = 50\ ksi$	Yield stress
$I = 843\ in^4$	From AISC Shapes Database
$Z_x = 95.4\ in^3$	From AISC Shapes Database
$t_w = 0.350\ in$	From AISC Shapes Database
$t_f = 0.450\ in$	From AISC Shapes Database
$b_f = 6.50\ in$	From AISC Shapes Database

$d = 20.7$ ***in*** From AISC Shapes Database

$A_w = \left(d - t_f \cdot 2\right) \cdot t_w = 6.93$ $\boldsymbol{in}^2$

Calculation of Point Loads

$P_d = L_E \cdot (t_b \cdot D_l + w_b) = 7.32$ ***kip***

$P_l = t_b \cdot L_l \cdot L_E = 4.8$ ***kip***

$P_u = \max(1.4 \cdot P_d, 1.2 \cdot P_d + 1.6 \cdot P_l) = 16.464$ ***kip***

Moment, Shear, Deflection analysis performed in ROBOT. Results are summarized below.

Member Strength

$M_{all} = 0.9 \cdot f_y \cdot Z_x = 357.75$ ***kip•ft***

$V_{all} = 0.6 \cdot f_y \cdot A_w = 207.9$ ***kip***

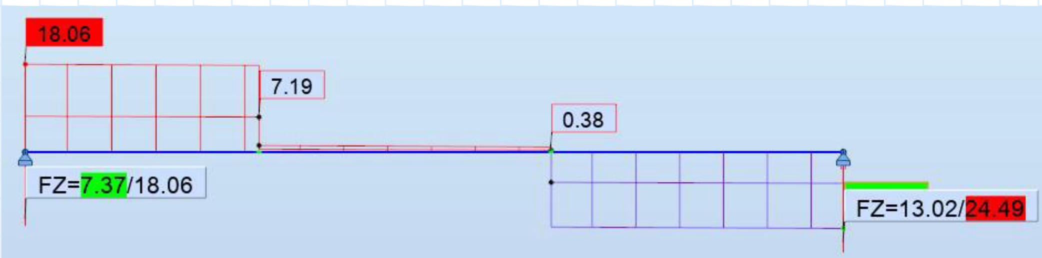
Strength Requirements

Maximum moment

$M_{max} = 77.79$ ***kip•ft***

Moment = "Okay"

Maximum shear



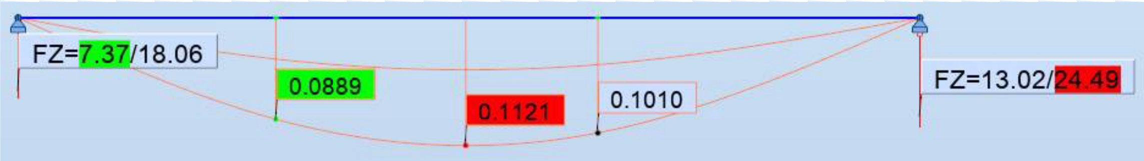
$V_{max} = 24.49$ ***kip***

Shear = "Okay"

Allowable Serviceability Deflections

$\Delta_{Tall} = \frac{L}{360} = 0.467\ in$ Total allowable deflection

Serviceability Analysis



$\Delta_{tot} = 0.1121\ in$

Total = "Okay"

Design Summary

A W21x44 satisfies all requirements for this preliminary design.

Column Analysis Overview

Assumptions (if needed)

- Risk factor IV
- Foundation to Roof 26'
- Foundation to 2nd floor 16'

Define Variables

$H_r = 26\ ft$ Height of column from foundation to roof

$H_f = 16\ ft$ Height of column from foundation to second floor

$S = 87\ psf$ Peak snow load due to unbalanced loading, see load calculations

$D_r = 34\ psf$ Roof dead load from trusses and roof coverings

$D_f = 60\ psf$ Floor dead load from slab, joists, beams, and misc.

$L_r = 20\ psf$ Roof live load, see load calculations

$L_{l1} = 100\ psf$ First floor live load

$L_{l2} = 40\ psf$ Second floor live load

$W = 9.6\ psf$ Worst case wind load on roof

$W_u = -26\ psf$ Worst case wind uplift on roof

Note: Column loads will be calculated from each columns respective tributary area.

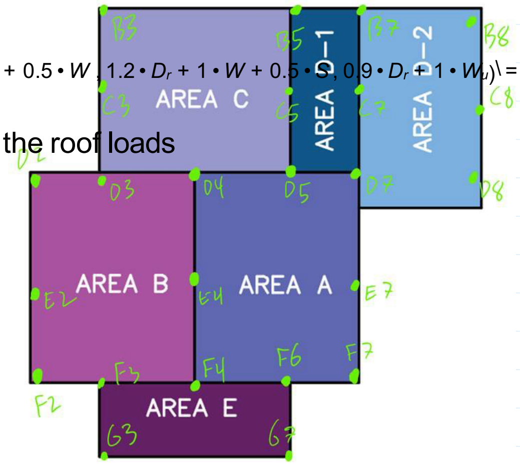
$$F_1 = \max(1.4 \cdot D_f, 1.2 \cdot D_f + 1.6 \cdot L_{l1}) = 232\ psf$$

$$F_2 = \max(1.4 \cdot D_f, 1.2 \cdot D_f + 1.6 \cdot L_{l2}) = 136\ psf$$

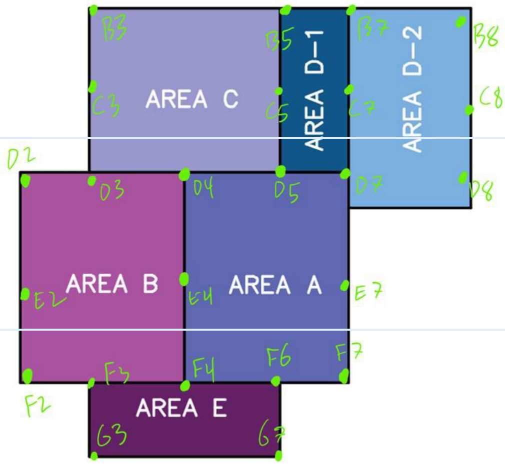
$$F_r = \max(0.9 \cdot D_r + 1 \cdot W_u) = 4.6\ psf$$

$$F_r = \max\left(1.4 \cdot D_r , 1.2 \cdot D_r + 0.5 \cdot S , 1.2 \cdot D_r + 1.6 \cdot S + 0.5 \cdot W , 1.2 \cdot D_r + 1 \cdot W + 0.5 \cdot S , 0.9 \cdot D_r + 1 \cdot W_u\right) = 184.8 \textbf{ psf}$$

LRFD Load combination 3 governs for the roof loads



Column B3 Analysis



Assumptions (if needed)

- Risk factor IV
- Foundation to Roof 26'
- Foundation to 2nd floor 16'

Define Variables

$T_r = 1$ 1 for roof bearing, 0 for non-roof bearing

$H = \begin{cases} H_f & \text{if } T_r = 0 \\ H_r & \text{else} \end{cases} = 26\ ft$

$q = 1.5\ ksf$ Assumed Bearing Pressure

First Floor Load

$d_{rf} = 26\ ft$ Distance away from next column to the right in plan view

$d_{lf} = 0\ ft$ Distance away from next column to the left in plan view

$d_{uf} = 0\ ft$ Distance away from next column above in plan view

$d_{df} = 18\ ft$ Distance away from next column below in plan view

$$T_{Af} = \left(\frac{d_{rf}}{2} + \frac{d_{lf}}{2}\right) \cdot \left(\frac{d_{uf}}{2} + \frac{d_{df}}{2}\right) = 117\ ft^2$$

$P_{u1} = F_1 \cdot T_{Af} = 27.144\ kip$

Second Floor Load

$P_{u2} = F_2 \cdot T_{Af} = 15.912\ kip$

Roof Load

$d_{rr} = 26\ ft$ Distance away from next column to the right in plan view

$d_{lr} = 0\ ft$ Distance away from next column to the left in plan view

$d_{ur} = 0\ ft$ Distance away from next column above in plan view

$d_{dr} = 18\ ft$ Distance away from next column below in plan view

$$T_{Ar} = \left(\frac{d_{rr}}{2} + \frac{d_{lr}}{2}\right) \cdot \left(\frac{d_{ur}}{2} + \frac{d_{dr}}{2}\right) = 117\ ft^2$$

$P_{ur} = F_r \cdot T_{Ar} = 21.622\ kip$

Strength Requirements

$$P_u = \mathrm{ceil}\left(\frac{P_{u1} + P_{u2} + P_{ur}}{kip}\right) kip = 65\ kip$$

Member Strength

$H = 26\ ft$ HSS 6 x 6 x 5/16

$\phi_c P_n = 79.6\ kip$

$U = \frac{P_u}{\phi_c P_n} \cdot 100 = 81.658$ Percent strength of column utilized

Design Summary

An HSS 6 x 6 x 5/16 for column B3 satisfies this preliminary design

Footing Design:

$A = \frac{P_u}{q} = 43.333\ ft^2$

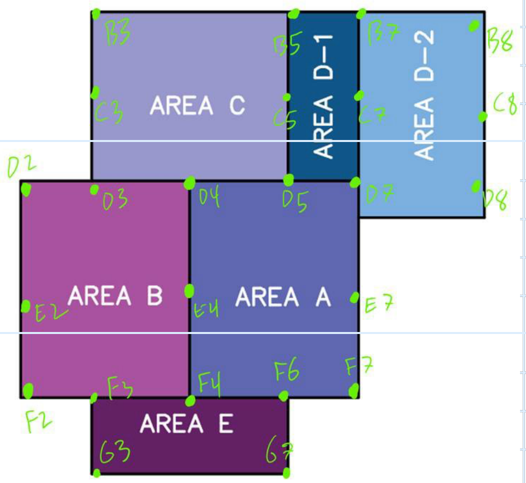
$L = \sqrt{A} = 6.583\ ft$

Use 7 ft square footing

Column B5 Analysis

Assumptions (if needed)

- Risk factor IV
- Foundation to Roof 26'
- Foundation to 2nd floor 16'



Define Variables

$T_r = 1$ 1 for roof bearing, 0 for non-roof bearing

$$H = \begin{cases} \text{if } T_r = 0 \\ \quad H_f \\ \text{else} \\ \quad H_r \end{cases} = 26\ ft$$

First Floor Load

$d_{rf} = 16\ ft$ Distance away from next column to the right in plan view

$d_{lf} = 26\ ft$ Distance away from next column to the left in plan view

$d_{uf} = 0\ ft$ Distance away from next column above in plan view

$d_{df} = 18\ ft$ Distance away from next column below in plan view

$$T_{Af} = \left(\frac{d_{rf}}{2} + \frac{d_{lf}}{2}\right) \cdot \left(\frac{d_{uf}}{2} + \frac{d_{df}}{2}\right) = 189\ ft^2$$

$$P_{u1} = F_1 \cdot T_{Af} = 43.848\ kip$$

Second Floor Load

$$P_{u2} = F_2 \cdot T_{Af} = 25.704\ kip$$

Roof Load

$d_{rr} = \begin{cases} d_{rf} & \text{if } T_r = 1 \\ 0\ ft & \text{else} \end{cases} = 16\ ft$ Distance away from next column to the right in plan view

$d_{lr} = \begin{cases} d_{lf} & \text{if } T_r = 1 \\ 0\ ft & \text{else} \end{cases} = 26\ ft$ Distance away from next column to the left in plan view

$d_{ur} = \begin{cases} d_{uf} & \text{if } T_r = 1 \\ 0\ ft & \text{else} \end{cases} = 0\ ft$ Distance away from next column above in plan view

$d_{dr} = \begin{cases} d_{df} & \text{if } T_r = 1 \\ 0\ ft & \text{else} \end{cases} = 18\ ft$ Distance away from next column below in plan view

$$T_{Ar} = \left(\frac{d_{rr}}{2} + \frac{d_{lr}}{2}\right) \cdot \left(\frac{d_{ur}}{2} + \frac{d_{dr}}{2}\right) = 189\ ft^2$$

$$P_{ur} = F_r \cdot T_{Ar} = 34.927\ kip$$

Strength Requirements $P_{ur} = 34.927\ kip$

$$P_u = \text{ceil}\left(\frac{P_{u1} + P_{u2} + P_{ur}}{kip}\right) kip = 105\ kip$$

Member Strength

$H = 26\ ft$ HSS 7x 7 x 5/16

$\phi_c P_n = 130\ kip$

$U = \frac{P_u}{\phi_c P_n} \cdot 100 = 80.769$ Percent strength of column utilized

Design Summary

An HSS 7 x 7 x 5/16 for column B5 satisfies this preliminary design

Footing Design:

$$A = \frac{P_u}{q} = 70\ ft^2$$

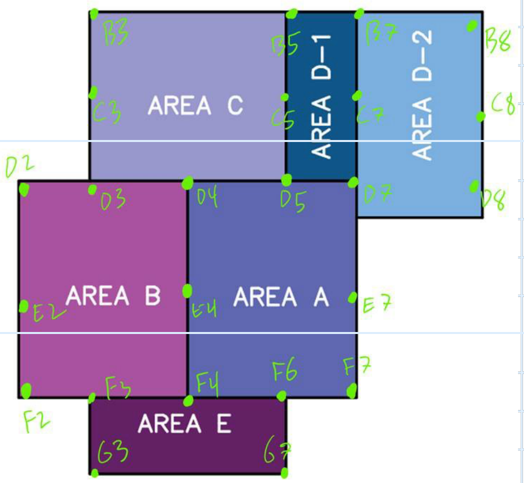
$$L = \sqrt{A} = 8.367\ ft$$

Use 8.5 ft square footing

Column B7 Analysis

Assumptions (if needed)

- Risk factor IV
- Foundation to Roof 26'
- Foundation to 2nd floor 16'



Define Variables

$T_r = 1$ 1 for roof bearing, 0 for non-roof bearing

$$H = \begin{cases} \text{if } T_r = 0 \\ \quad H_f \\ \text{else} \\ \quad H_r \end{cases} = 26\ ft$$

First Floor Load

$d_{rf} = 23\ ft$ Distance away from next column to the right in plan view

$d_{lf} = 16\ ft$ Distance away from next column to the left in plan view

$d_{uf} = 0\ ft$ Distance away from next column above in plan view

$d_{df} = 18\ ft$ Distance away from next column below in plan view

$$T_{Af} = \left(\frac{d_{rf}}{2} + \frac{d_{lf}}{2}\right) \cdot \left(\frac{d_{uf}}{2} + \frac{d_{df}}{2}\right) = 175.5\ ft^2$$

$P_{u1} = F_1 \cdot T_{Af} = 40.716\ kip$

Second Floor Load

$P_{u2} = F_2 \cdot T_{Af} = 23.868\ kip$

Roof Load

$K_r = 0$ 1 for roof bearing with no floor columns around, 0 for non-roof bearing

$d_{rr} = \begin{cases} \text{if } K_r = 1 \\ \quad d_{rf} \\ \text{else} \\ \quad 24\ ft \end{cases} = 24\ ft$ Distance away from next column to the right in plan view

$d_{lr} = \begin{cases} \text{if } K_r = 1 \\ \quad d_{lf} \\ \text{else} \\ \quad 16\ ft \end{cases} = 16\ ft$ Distance away from next column to the left in plan view

$d_{ur} = \begin{cases} \text{if } K_r = 1 \\ \quad d_{uf} \\ \text{else} \\ \quad 0\ ft \end{cases} = 0\ ft$ Distance away from next column above in plan view

$d_{dr} = \begin{cases} \text{if } K_r = 1 \\ \quad d_{df} \\ \text{else} \\ \quad 31\ ft \end{cases} = 31\ ft$ Distance away from next column below in plan view

$$T_{Ar} = \left(\frac{d_{rr}}{2} + \frac{d_{lr}}{2}\right) \cdot \left(\frac{d_{ur}}{2} + \frac{d_{dr}}{2}\right) = 310\ ft^2$$

$$P_{ur} = F_r \cdot T_{Ar} = 57.288\ kip$$

Strength Requirements

$P_{ur} = 57.288\ kip$

$$P_u = \text{ceil}\left(\frac{P_{u1} + P_{u2} + P_{ur}}{kip}\right) kip = 122\ kip$$

Member Strength

$H = 26\ ft$

$\phi_c P_n = 156\ kip$

$U = \frac{P_u}{\phi_c P_n} \cdot 100 = 78.205$ Percent strength of column utilized

Design Summary

An HSS 8 x 8 x 1/4 for column B7 satisfies this preliminary design

Footing Design:

$$A = \frac{P_u}{q} = 81.333 \ ft^2$$

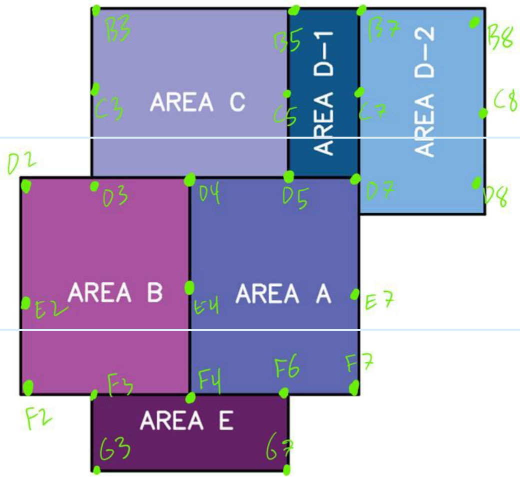
$$L = \sqrt{A} = 9.018 \ ft$$

Use 9.5 ft square footing

Column B8 Analysis

Assumptions (if needed)

- Risk factor IV
- Foundation to Roof 26'
- Foundation to 2nd floor 16'



Define Variables

$T_r = 1$ 1 for roof bearing, 0 for non-roof bearing

$$H = \begin{cases} \text{if } T_r = 0 \\ \quad H_f \\ \text{else} \\ \quad H_r \end{cases} = 26\ ft$$

First Floor Load

$d_{rf} = 0\ ft$ Distance away from next column to the right in plan view

$d_{lf} = 23\ ft$ Distance away from next column to the left in plan view

$d_{uf} = 0\ ft$ Distance away from next column above in plan view

$d_{df} = 18\ ft$ Distance away from next column below in plan view

$$T_{Af} = \left(\frac{d_{rf}}{2} + \frac{d_{lf}}{2}\right) \cdot \left(\frac{d_{uf}}{2} + \frac{d_{df}}{2}\right) = 103.5\ ft^2$$

$P_{u1} = F_1 \cdot T_{Af} = 24.012\ kip$

Second Floor Load

$P_{u2} = F_2 \cdot T_{Af} = 14.076\ kip$

Roof Load

$K_r = 0$ 1 for roof bearing with no floor columns around,
0 for roof bearing with floor columns around

$d_{rr} = \begin{cases} \text{if } K_r = 1 \\ \quad d_{rf} \\ \text{else} \\ \quad 0\ ft \end{cases} = 0\ ft$ Distance away from next column to the right in plan view

$d_{lr} = \begin{cases} \text{if } K_r = 1 \\ \quad d_{lf} \\ \text{else} \\ \quad 23\ ft \end{cases} = 23\ ft$ Distance away from next column to the left in plan view

$d_{ur} = \begin{cases} \text{if } K_r = 1 \\ \quad d_{uf} \\ \text{else} \\ \quad 0\ ft \end{cases} = 0\ ft$ Distance away from next column above in plan view

$d_{dr} = \begin{cases} \text{if } K_r = 1 \\ \quad d_{df} \\ \text{else} \\ \quad 31\ ft \end{cases} = 31\ ft$ Distance away from next column below in plan view

$T_{Ar} = \left(\frac{d_{rr}}{2} + \frac{d_{lr}}{2}\right) \cdot \left(\frac{d_{ur}}{2} + \frac{d_{dr}}{2}\right) = 178.25\ ft^2$

$P_{ur} = F_r \cdot T_{Ar} = 32.941\ kip$

Strength Requirements $P_{ur} = 32.941\ kip$

$P_u = \mathrm{ceil}\left(\frac{P_{u1} + P_{u2} + P_{ur}}{kip}\right) kip = 72\ kip$

Member Strength

$H = 26\ ft$ HSS 7 x 7 x 5/16

$\phi_c P_n = 130\ kip$

$U = \frac{P_u}{\phi_c P_n} \cdot 100 = 55.385$ Percent strength of column utilized

Design Summary

An HSS 7 x 7 x 5/16 for column B8 satisfies this preliminary design

Footing Design:

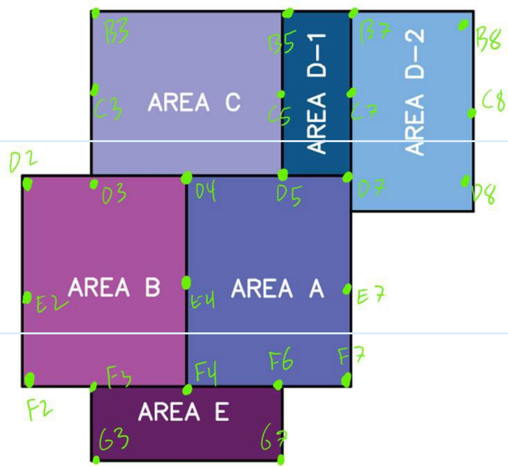
$$A = \frac{P_u}{q} = 48\ ft^2$$

$$L = \sqrt{A} = 6.928\ ft$$

Use 7 ft square column

Column C3 Analysis

Assumptions (if needed)



- Risk factor IV
- Foundation to Roof 26'
- Foundation to 2nd floor 16'

Define Variables

$T_r = 1$ — 1 for roof bearing, 0 for non-roof bearing

$$H = \begin{cases} \text{if } T_r = 0 \\ \quad H_f \\ \text{else} \\ \quad H_r \end{cases} = 26\ ft$$

First Floor Load

$d_{rf} = 26\ ft$ — Distance away from next column to the right in plan view

$d_{lf} = 0\ ft$ — Distance away from next column to the left in plan view

$d_{uf} = 13\ ft$ — Distance away from next column above in plan view

$d_{df} = 18\ ft$ — Distance away from next column below in plan view

$$T_{Af} = \left(\frac{d_{rf}}{2} + \frac{d_{lf}}{2}\right) \cdot \left(\frac{d_{uf}}{2} + \frac{d_{df}}{2}\right) = 201.5\ ft^2$$

$$P_{u1} = F_1 \cdot T_{Af} = 46.748\ kip$$

Second Floor Load

$$P_{u2} = F_2 \cdot T_{Af} = 27.404\ kip$$

Roof Load

$K_r = 1$ 1 for roof bearing with no floor columns around, 0 for roof bearing with floor columns around

$d_{rr} = \text{if } K_r = 1 \;\; d_{rf} \;\; \text{else} \;\; 0\ ft = 26\ ft$ Distance away from next column to the right in plan view

$d_{lr} = \text{if } K_r = 1 \;\; d_{lf} \;\; \text{else} \;\; 0\ ft = 0\ ft$ Distance away from next column to the left in plan view

$d_{ur} = \text{if } K_r = 1 \;\; d_{uf} \;\; \text{else} \;\; 0\ ft = 13\ ft$ Distance away from next column above in plan view

$d_{dr} = \text{if } K_r = 1 \;\; d_{df} \;\; \text{else} \;\; 0\ ft = 18\ ft$ Distance away from next column below in plan view

$$T_{Ar} = \left(\frac{d_{rr}}{2} + \frac{d_{lr}}{2}\right) \cdot \left(\frac{d_{ur}}{2} + \frac{d_{dr}}{2}\right) = 201.5\ ft^2$$

$$P_{ur} = F_r \cdot T_{Ar} = 37.237\ kip$$

Strength Requirements $P_{ur} = 37.237\ kip$

$$P_u = \text{ceil}\left(\frac{P_{u1} + P_{u2} + P_{ur}}{kip}\right) kip = 112\ kip$$

Member Strength

$H = 26\ ft$ HSS 7 x 7 x 5/16

$\phi_c P_n = 130\ kip$

$$U = \frac{P_u}{\phi_c P_n} \cdot 100 = 86.154$$ Percent strength of column utilized

Design Summary

An HSS 7 x 7 x 5/16 for column C3 satisfies this preliminary design

Footing Design:

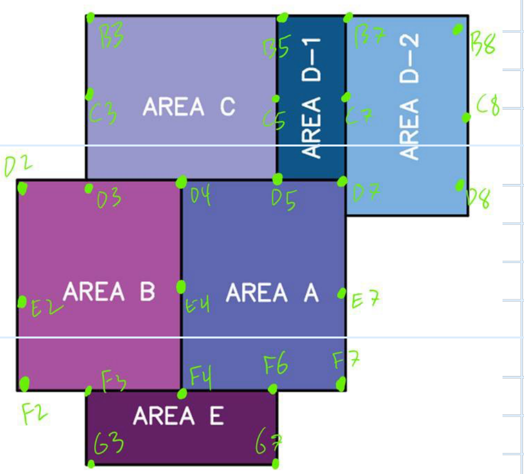
$$A = \frac{P_u}{q} = 74.667\ ft^2$$

$$L = \sqrt{A} = 8.641\ ft$$

Use 9 ft square footing

Column C5 Analysis

Assumptions (if needed)



- Risk factor IV
- Foundation to Roof 26'
- Foundation to 2nd floor 16'

Define Variables

$T_r = 1$ 1 for roof bearing, 0 for non-roof bearing

$H = \begin{cases} H_f & \text{if } T_r = 0 \\ H_r & \text{else} \end{cases} = 26\ ft$

First Floor Load

$d_{rf} = 16\ ft$ Distance away from next column to the right in plan view

$d_{lf} = 26\ ft$ Distance away from next column to the left in plan view

$d_{uf} = 18\ ft$ Distance away from next column above in plan view

$d_{df} = 13\ ft$ Distance away from next column below in plan view

$$T_{Af} = \left(\frac{d_{rf}}{2} + \frac{d_{lf}}{2}\right) \cdot \left(\frac{d_{uf}}{2} + \frac{d_{df}}{2}\right) = 325.5\ ft^2$$

$P_{u1} = F_1 \cdot T_{Af} = 75.516\ kip$

Second Floor Load

$P_{u2} = F_2 \cdot T_{Af} = 44.268\ kip$

Roof Load

$K_r = 0$ 1 for roof bearing with no floor columns around, 0 for roof bearing with floor columns around

$d_{rr} = \text{if } K_r = 1 \;\; \| \; d_{rf} \;\; \text{else} \;\; \| \; 16\ ft = 16\ ft$ Distance away from next column to the right in plan view

$d_{lr} = \text{if } K_r = 1 \;\; \| \; d_{lf} \;\; \text{else} \;\; \| \; 26\ ft = 26\ ft$ Distance away from next column to the left in plan view

$d_{ur} = \text{if } K_r = 1 \;\; \| \; d_{uf} \;\; \text{else} \;\; \| \; 18\ ft = 18\ ft$ Distance away from next column above in plan view

$d_{dr} = \text{if } K_r = 1 \;\; \| \; d_{df} \;\; \text{else} \;\; \| \; 13\ ft = 13\ ft$ Distance away from next column below in plan view

$$T_{Ar} = \left(\frac{d_{rr}}{2} + \frac{d_{lr}}{2}\right) \cdot \left(\frac{d_{ur}}{2} + \frac{d_{dr}}{2}\right) = 325.5\ ft^2$$

$$P_{ur} = F_r \cdot T_{Ar} = 60.152\ kip$$

Strength Requirements

$P_{ur} = 60.152\ kip$

$$P_u = \text{ceil}\left(\frac{P_{u1} + P_{u2} + P_{ur}}{kip}\right) kip = 180\ kip$$

Member Strength

$H = 26\ ft$ HSS 9 x 9 x 1/4

$\phi_c P_n = 206\ kip$

$$U = \frac{P_u}{\phi_c P_n} \cdot 100 = 87.379$$ Percent strength of column utilized

Design Summary

An HSS 9 x 9 x 1/4 for column C5 satisfies this preliminary design

Footing Design:

$$A = \frac{P_u}{q} = 120\ ft^2$$

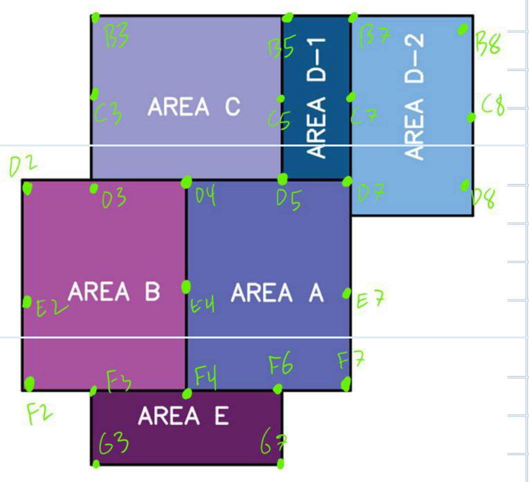
$$L = \sqrt{A} = 10.954\ ft$$

Use 11 ft square footing

Column C7 Analysis

Assumptions (if needed)

- Risk factor IV
- Foundation to Roof 26'
- Foundation to 2nd floor 16'



Define Variables

$T_r = 0$ 1 for roof bearing, 0 for non-roof bearing

$H = \begin{cases} H_f & \text{if } T_r = 0 \\ H_r & \text{else} \end{cases} = 16\ ft$

First Floor Load

$d_{rf} = 23\ ft$ Distance away from next column to the right in plan view

$d_{lf} = 0\ ft$ Distance away from next column to the left in plan view

$d_{uf} = 18\ ft$ Distance away from next column above in plan view

$d_{df} = 13\ ft$ Distance away from next column below in plan view

$$T_{Af} = \left(\frac{d_{rf}}{2} + \frac{d_{lf}}{2}\right) \cdot \left(\frac{d_{uf}}{2} + \frac{d_{df}}{2}\right) = 178.25\ ft^2$$

$P_{u1} = F_1 \cdot T_{Af} = 41.354\ kip$

Second Floor Load

$P_{u2} = F_2 \cdot T_{Af} = 24.242\ kip$

Roof Load

$K_r = 0$ 1 for roof bearing with no floor columns around, 0 for roof bearing with floor columns around

$d_{rr} = \text{if } K_r = 1 \; \| \; d_{rf} \; \| \; \text{else} \; \| \; 0\ ft = 0\ ft$ Distance away from next column to the right in plan view

$d_{lr} = \text{if } K_r = 1 \; \| \; d_{lf} \; \| \; \text{else} \; \| \; 0\ ft = 0\ ft$ Distance away from next column to the left in plan view

$d_{ur} = \text{if } K_r = 1 \; \| \; d_{uf} \; \| \; \text{else} \; \| \; 0\ ft = 0\ ft$ Distance away from next column above in plan view

$d_{dr} = \text{if } K_r = 1 \; \| \; d_{df} \; \| \; \text{else} \; \| \; 0\ ft = 0\ ft$ Distance away from next column below in plan view

$$T_{Ar} = \left(\frac{d_{rr}}{2} + \frac{d_{lr}}{2}\right) \cdot \left(\frac{d_{ur}}{2} + \frac{d_{dr}}{2}\right) = 0\ ft^2$$

$$P_{ur} = F_r \cdot T_{Ar} = 0\ kip$$

Strength Requirements

$P_{ur} = 0\ kip$

$$P_u = \text{ceil}\left(\frac{P_{u1} + P_{u2} + P_{ur}}{kip}\right) kip = 66\ kip$$

Member Strength

$H = 16\ ft$ HSS 5 x 5 x 1/4

$$\phi_c P_n = 93.8\ kip$$

$$U = \frac{P_u}{\phi_c P_n} \cdot 100 = 70.362$$ Percent strength of column utilized

Design Summary

An HSS 5 x 5 x 1/4 for column C7 satisfies this preliminary design

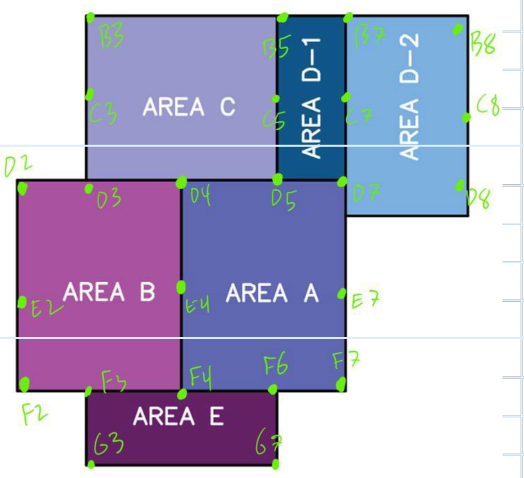
Footing Design:

$$A = \frac{P_u}{q} = 44\ ft^2$$

$$L = \sqrt{A} = 6.633\ ft$$

Use 7 ft square footing

Column C8 Analysis



Assumptions (if needed)

- Risk factor IV
- Foundation to Roof 26'
- Foundation to 2nd floor 16'

Define Variables

$T_r = 0$ 1 for roof bearing, 0 for non-roof bearing

$H = \begin{cases} \text{if } T_r = 0 \\ \quad H_f \\ \text{else} \\ \quad H_r \end{cases} = 16\ ft$

First Floor Load

$d_{rf} = 0\ ft$ Distance away from next column to the right in plan view

$d_{lf} = 24\ ft$ Distance away from next column to the left in plan view

$d_{uf} = 18\ ft$ Distance away from next column above in plan view

$d_{df} = 13\ ft$ Distance away from next column below in plan view

$$T_{Af} = \left(\frac{d_{rf}}{2} + \frac{d_{lf}}{2}\right) \cdot \left(\frac{d_{uf}}{2} + \frac{d_{df}}{2}\right) = 186\ ft^2$$

$P_{u1} = F_1 \cdot T_{Af} = 43.152\ kip$

Second Floor Load

$P_{u2} = F_2 \cdot T_{Af} = 25.296\ kip$

Roof Load

$K_r = 0$ 1 for roof bearing with no floor columns around, 0 for roof bearing with floor columns around

$d_{rr} = \text{if } K_r = 1 \; \| \; d_{rf} \; \text{else} \; \| \; 0\ ft$ $= 0\ ft$ Distance away from next column to the right in plan view

$d_{lr} = \text{if } K_r = 1 \; \| \; d_{lf} \; \text{else} \; \| \; 0\ ft$ $= 0\ ft$ Distance away from next column to the left in plan view

$d_{ur} = \text{if } K_r = 1 \; \| \; d_{uf} \; \text{else} \; \| \; 0\ ft$ $= 0\ ft$ Distance away from next column above in plan view

$d_{dr} = \text{if } K_r = 1 \; \| \; d_{df} \; \text{else} \; \| \; 0\ ft$ $= 0\ ft$ Distance away from next column below in plan view

$$T_{Ar} = \left(\frac{d_{rr}}{2} + \frac{d_{lr}}{2}\right) \cdot \left(\frac{d_{ur}}{2} + \frac{d_{dr}}{2}\right) = 0\ ft^2$$

$$P_{ur} = F_r \cdot T_{Ar} = 0\ kip$$

Strength Requirements

$P_{ur} = 0\ kip$

$$P_u = \text{ceil}\left(\frac{P_{u1} + P_{u2} + P_{ur}}{kip}\right) kip = 69\ kip$$

Member Strength

$H = 16\ ft$ HSS 5 x 5 x 1/4

$$\phi_c P_n = 93.8\ kip$$

$$U = \frac{P_u}{\phi_c P_n} \cdot 100 = 73.561$$ Percent strength of column utilized

Design Summary

An HSS 5 x 5 x 1/4 for column C8 satisfies this preliminary design

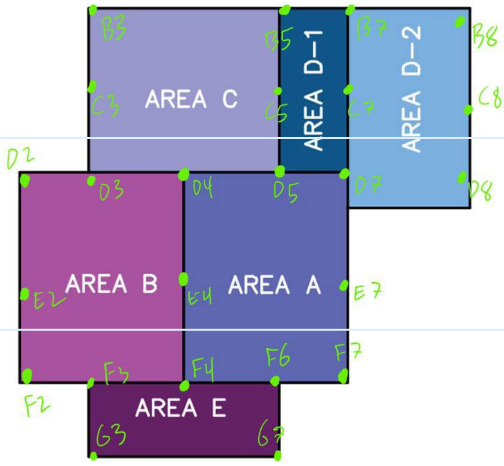
Footing Design:

$$A = \frac{P_u}{q} = 46 \; ft^2$$

$$L = \sqrt{A} = 6.782 \; ft$$

Use 7 ft square footing

Column D2 Analysis



Assumptions (if needed)

- Risk factor IV
- Foundation to Roof 26'
- Foundation to 2nd floor 16'

Define Variables

$T_r = 1$ 1 for roof bearing, 0 for non-roof bearing

$$H = \begin{cases} \text{if } T_r = 0 \\ \quad H_f \\ \text{else} \\ \quad H_r \end{cases} = 26\ ft$$

First Floor Load

$d_{rf} = 13\ ft$ Distance away from next column to the right in plan view

$d_{lf} = 0\ ft$ Distance away from next column to the left in plan view

$d_{uf} = 0\ ft$ Distance away from next column above in plan view

$d_{df} = 20\ ft$ Distance away from next column below in plan view

$$T_{Af} = \left(\frac{d_{rf}}{2} + \frac{d_{lf}}{2}\right) \cdot \left(\frac{d_{uf}}{2} + \frac{d_{df}}{2}\right) = 65\ ft^2$$

$P_{u1} = F_1 \cdot T_{Af} = 15.08\ kip$

Second Floor Load

$P_{u2} = F_2 \cdot T_{Af} = 8.84\ kip$

Roof Load $K_r = 0$ 1 matching, 0 for non-matching

$d_{rr} = \text{if } K_r = 1 \; d_{rf} \text{ else } 13\ ft = 13\ ft$ Distance away from next column to the right in plan view

$d_{lr} = \text{if } K_r = 1 \; d_{lf} \text{ else } 0\ ft = 0\ ft$ Distance away from next column to the left in plan view

$d_{ur} = \text{if } K_r = 1 \; d_{uf} \text{ else } 0\ ft = 0\ ft$ Distance away from next column above in plan view

$d_{dr} = \text{if } K_r = 1 \; d_{df} \text{ else } 33\ ft = 33\ ft$ Distance away from next column below in plan view

$$T_{Ar} = \left(\frac{d_{rr}}{2} + \frac{d_{lr}}{2}\right) \cdot \left(\frac{d_{ur}}{2} + \frac{d_{dr}}{2}\right) = 107.25\ ft^2$$

$$P_{ur} = F_r \cdot T_{Ar} = 19.82\ kip$$

Strength Requirements $P_{ur} = 19.82\ kip$

$$P_u = \text{ceil}\left(\frac{P_{u1} + P_{u2} + P_{ur}}{kip}\right) kip = 44\ kip$$

Member Strength

$H = 26\ ft$ HSS 6 x 6 x 5/16

$$\phi_c P_n = 79.6\ kip$$

$$U = \frac{P_u}{\phi_c P_n} \cdot 100 = 55.276$$ Percent strength of column utilized

Design Summary

An HSS 6 x 6 x 5/16 for column D2 satisfies this preliminary design

Footing Design:

$$A = \frac{P_u}{q} = 29.333\ ft^2$$

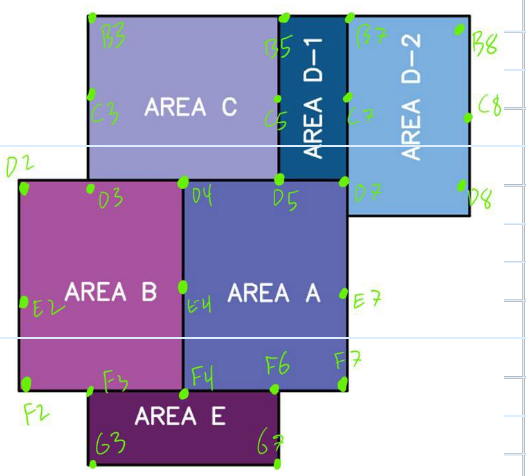
$$L = \sqrt{A} = 5.416\ ft$$

Use 5.5 ft square footing

Column D3 Analysis

Assumptions (if needed)

- Risk factor IV
- Foundation to Roof 26'
- Foundation to 2nd floor 16'



Define Variables

$T_r = 1$ 1 for roof bearing, 0 for non-roof bearing

$H = \begin{cases} H_f & \text{if } T_r = 0 \\ H_r & \text{else} \end{cases} = 26\ ft$

First Floor Load

$d_{rf} = 15\ ft$ Distance away from next column to the right in plan view

$d_{lf} = 13\ ft$ Distance away from next column to the left in plan view

$d_{uf} = 13\ ft$ Distance away from next column above in plan view

$d_{df} = 19\ ft$ Distance away from next column below in plan view

$$T_{Af} = \left(\frac{d_{rf}}{2} + \frac{d_{lf}}{2}\right) \cdot \left(\frac{d_{uf}}{2} + \frac{d_{df}}{2}\right) - \left(\frac{d_{uf}}{2} \cdot \frac{d_{lf}}{2}\right) = 181.75\ ft^2$$

$P_{u1} = F_1 \cdot T_{Af} = 42.166\ kip$

Second Floor Load

$P_{u2} = F_2 \cdot T_{Af} = 24.718\ kip$

Roof Load $K_r = 0$ 1 matching, 0 for non-matching

$d_{rr} = \text{if } K_r = 1 \;\; d_{rf} \;\; \text{else} \;\; 15\ ft = 15\ ft$ Distance away from next column to the right in plan view

$d_{lr} = \text{if } K_r = 1 \;\; d_{lf} \;\; \text{else} \;\; 0\ ft = 0\ ft$ Distance away from next column to the left in plan view

$d_{ur} = \text{if } K_r = 1 \;\; d_{uf} \;\; \text{else} \;\; 13\ ft = 13\ ft$ Distance away from next column above in plan view

$d_{dr} = \text{if } K_r = 1 \;\; d_{df} \;\; \text{else} \;\; 0\ ft = 0\ ft$ Distance away from next column below in plan view

$$T_{Ar} = \left(\frac{d_{rr}}{2} + \frac{d_{lr}}{2}\right) \cdot \left(\frac{d_{ur}}{2} + \frac{d_{dr}}{2}\right) - \left(\frac{d_{dr}}{2} \cdot \frac{d_{lr}}{2}\right) = 48.75\ ft^2$$

$$P_{ur} = F_r \cdot T_{Ar} = 9.009\ kip$$

Strength Requirements $P_{ur} = 9.009\ kip$

$$P_u = \text{ceil}\left(\frac{P_{u1} + P_{u2} + P_{ur}}{kip}\right) kip = 76\ kip$$

Member Strength

$H = 26\ ft$ HSS 7 x 7 x 5/16

$\phi_c P_n = 130\ kip$

$U = \frac{P_u}{\phi_c P_n} \cdot 100 = 58.462$ Percent strength of column utilized

Design Summary

An HSS 7 x 7 x 5/16 for column D3 satisfies this preliminary design

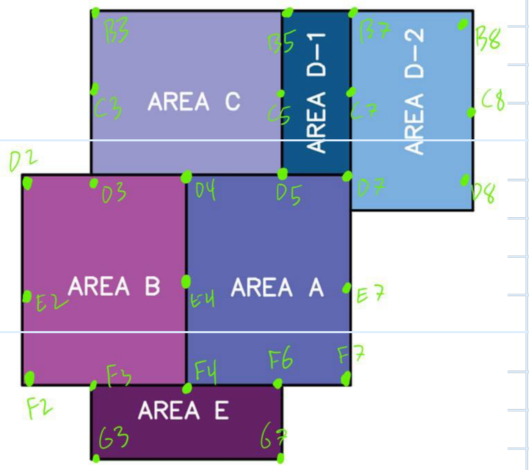
Footing Design:

$$A = \frac{P_u}{q} = 50.667\ ft^2$$

$$L = \sqrt{A} = 7.118\ ft$$

Use 7.5 ft square footing

Column D4 Analysis



Assumptions (if needed)

- Risk factor IV
- Foundation to Roof 26'
- Foundation to 2nd floor 16'

Define Variables

$T_r = 1$ 1 for roof bearing, 0 for non-roof bearing

$$H = \begin{cases} \text{if } T_r = 0 \\ \quad H_f \\ \text{else} \\ \quad H_r \end{cases} = 26\ ft$$

First Floor Load

$d_{rf} = 12\ ft$ Distance away from next column to the right in plan view

$d_{lf} = 15\ ft$ Distance away from next column to the left in plan view

$d_{uf} = 13\ ft$ Distance away from next column above in plan view

$d_{df} = 19\ ft$ Distance away from next column below in plan view

$$T_{Af} = \left(\frac{d_{rf}}{2} + \frac{d_{lf}}{2}\right) \cdot \left(\frac{d_{uf}}{2} + \frac{d_{df}}{2}\right) = 216\ ft^2$$

$P_{u1} = F_1 \cdot T_{Af} = 50.112\ kip$

Second Floor Load

$P_{u2} = F_2 \cdot T_{Af} = 29.376\ kip$

Roof Load $K_r = 0$ 1 matching, 0 for non-matching

$$d_{rr} = \begin{cases} \text{if } K_r = 1 \\ \quad d_{rf} \\ \text{else} \\ \quad 12\ ft \end{cases} = 12\ ft$$

Distance away from next column to the right in plan view

$$d_{lr} = \begin{cases} \text{if } K_r = 1 \\ \quad d_{lf} \\ \text{else} \\ \quad 15\ ft \end{cases} = 15\ ft$$

Distance away from next column to the left in plan view

$$d_{ur} = \begin{cases} \text{if } K_r = 1 \\ \quad d_{uf} \\ \text{else} \\ \quad 0\ ft \end{cases} = 0\ ft$$

Distance away from next column above in plan view

$$d_{dr} = \begin{cases} \text{if } K_r = 1 \\ \quad d_{df} \\ \text{else} \\ \quad 19\ ft \end{cases} = 19\ ft$$

Distance away from next column below in plan view

$$T_{Ar} = \left(\frac{d_{rr}}{2} + \frac{d_{lr}}{2}\right) \cdot \left(\frac{d_{ur}}{2} + \frac{d_{dr}}{2}\right) = 128.25\ ft^2$$

$$P_{ur} = F_r \cdot T_{Ar} = 23.701\ kip$$

Strength Requirements $P_{ur} = 23.701\ kip$

$$P_u = \text{ceil}\left(\frac{P_{u1} + P_{u2} + P_{ur}}{kip}\right) kip = 104\ kip$$

Member Strength

$H = 26\ ft$ HSS 7 x 7 x 5/16

$\phi_c P_n = 130\ kip$

$$U = \frac{P_u}{\phi_c P_n} \cdot 100 = 80$$

Percent strength of column utilized

Design Summary

An HSS 7 x 7 x 5/16 for column D4 satisfies this preliminary design

Footing Design:

$$A = \frac{P_u}{q} = 69.333\ ft^2$$

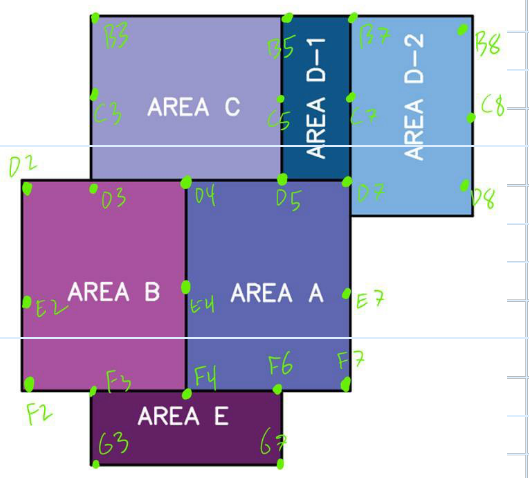
$$L = \sqrt{A} = 8.327\ ft$$

Use 8.5 ft square footing

Column D5 Analysis

Assumptions (if needed)

- Risk factor IV
- Foundation to Roof 26'
- Foundation to 2nd floor 16'



Define Variables

$T_r = 1$ 1 for roof bearing, 0 for non-roof bearing

$$H = \begin{cases} \text{if } T_r = 0 \\ \quad H_f \\ \text{else} \\ \quad H_r \end{cases} = 26\ ft$$

First Floor Load

$d_{rf} = 16\ ft$ Distance away from next column to the right in plan view

$d_{lf} = 12\ ft$ Distance away from next column to the left in plan view

$d_{uf} = 13\ ft$ Distance away from next column above in plan view

$d_{df} = 19\ ft$ Distance away from next column below in plan view

$$T_{Af} = \left(\frac{d_{rf}}{2} + \frac{d_{lf}}{2}\right) \cdot \left(\frac{d_{uf}}{2} + \frac{d_{df}}{2}\right) = 224\ ft^2$$

$P_{u1} = F_1 \cdot T_{Af} = 51.968\ kip$

Second Floor Load

$P_{u2} = F_2 \cdot T_{Af} = 30.464\ kip$

Roof Load $K_r = 1$ 1 matching, 0 for non-matching

$d_{rr} = \text{if } K_r = 1 \;\; d_{rf} \;\; \text{else} \;\; 16\ ft = 16\ ft$ Distance away from next column to the right in plan view

$d_{lr} = \text{if } K_r = 1 \;\; d_{lf} \;\; \text{else} \;\; 12\ ft = 12\ ft$ Distance away from next column to the left in plan view

$d_{ur} = \text{if } K_r = 1 \;\; d_{uf} \;\; \text{else} \;\; 13\ ft = 13\ ft$ Distance away from next column above in plan view

$d_{dr} = \text{if } K_r = 1 \;\; d_{df} \;\; \text{else} \;\; 19\ ft = 19\ ft$ Distance away from next column below in plan view

$$T_{Ar} = \left(\frac{d_{rr}}{2} + \frac{d_{lr}}{2}\right) \cdot \left(\frac{d_{ur}}{2} + \frac{d_{dr}}{2}\right) = 224\ ft^2$$

$$P_{ur} = F_r \cdot T_{Ar} = 41.395\ kip$$

Strength Requirements $P_{ur} = 41.395\ kip$

$$P_u = \text{ceil}\left(\frac{P_{u1} + P_{u2} + P_{ur}}{kip}\right) kip = 124\ kip$$

Member Strength

$H = 26\ ft$ HSS 8 x 8 x 1/4

$\phi_c P_n = 156\ kip$

$U = \frac{P_u}{\phi_c P_n} \cdot 100 = 79.487$ Percent strength of column utilized

Design Summary

An HSS 8 x 8 x 1/4 for column D5 satisfies this preliminary design

Footing Design:

$$A = \frac{P_u}{q} = 82.667 \ ft^2$$

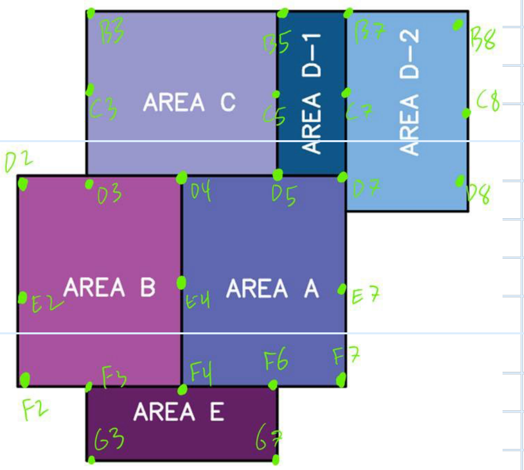
$$L = \sqrt{A} = 9.092 \ ft$$

Use 9.5 ft square footing

Column D7 Analysis

Assumptions (if needed)

- Risk factor IV
- Foundation to Roof 26'
- Foundation to 2nd floor 16'



Define Variables

$T_r = 1$ 1 for roof bearing, 0 for non-roof bearing

$$H = \begin{cases} \text{if } T_r = 0 \\ \quad H_f \\ \text{else} \\ \quad H_r \end{cases} = 26\ ft$$

First Floor Load

$d_{rf} = 24\ ft$ Distance away from next column to the right in plan view

$d_{lf} = 16\ ft$ Distance away from next column to the left in plan view

$d_{uf} = 13\ ft$ Distance away from next column above in plan view

$d_{df} = 19\ ft$ Distance away from next column below in plan view

$$T_{Af} = \left(\frac{d_{rf}}{2} + \frac{d_{lf}}{2}\right) \cdot \left(\frac{d_{uf}}{2} + \frac{d_{df}}{2}\right) - \left(\frac{d_{df}}{2} \cdot \frac{d_{rf}}{2}\right) = 206\ ft^2$$

$P_{u1} = F_1 \cdot T_{Af} = 47.792\ kip$

Second Floor Load

$P_{u2} = F_2 \cdot T_{Af} = 28.016\ kip$

Roof Load

$K_r = 1$ 1 matching, 0 for non-matching

$d_{rr} = \begin{cases} \text{if } K_r = 1 \\ \quad d_{rf} \\ \text{else} \\ \quad 16\ ft \end{cases} = 24\ ft$ Distance away from next column to the right in plan view

$d_{lr} = \begin{cases} \text{if } K_r = 1 \\ \quad d_{lf} \\ \text{else} \\ \quad 12\ ft \end{cases} = 16\ ft$ Distance away from next column to the left in plan view

$d_{ur} = \begin{cases} \text{if } K_r = 1 \\ \quad d_{uf} \\ \text{else} \\ \quad 13\ ft \end{cases} = 13\ ft$ Distance away from next column above in plan view

$d_{dr} = \begin{cases} \text{if } K_r = 1 \\ \quad d_{df} \\ \text{else} \\ \quad 0\ ft \end{cases} = 19\ ft$ Distance away from next column below in plan view

$$T_{Ar} = \left(\frac{d_{rr}}{2} + \frac{d_{lr}}{2}\right) \cdot \left(\frac{d_{ur}}{2} + \frac{d_{dr}}{2}\right) - \left(\frac{d_{dr}}{2} \cdot \frac{d_{rr}}{2}\right) = 206\ ft^2$$

$$P_{ur} = F_r \cdot T_{Ar} = 38.069\ kip$$

Strength Requirements

$P_{ur} = 38.069\ kip$

$$P_u = \text{ceil}\left(\frac{P_{u1} + P_{u2} + P_{ur}}{kip}\right) kip = 114\ kip$$

Member Strength

$H = 26\ ft$ HSS 8 x 8 x 1/4

$\phi_c P_n = 156\ kip$

$U = \frac{P_u}{\phi_c P_n} \cdot 100 = 73.077$ Percent strength of column utilized

Design Summary

An HSS 8 x 8 x 1/4 for column D7 satisfies this preliminary design

Footing Design:

$$A = \frac{P_u}{q} = 76\ ft^2$$

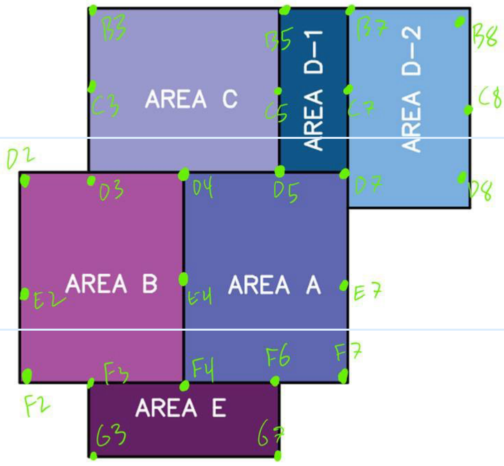
$$L = \sqrt{A} = 8.718\ ft$$

Use 9 ft square footing

Column D8 Analysis

Assumptions (if needed)

- Risk factor IV
- Foundation to Roof 26'
- Foundation to 2nd floor 16'



Define Variables

$T_r = 1$ 1 for roof bearing, 0 for non-roof bearing

$$H = \begin{cases} \text{if } T_r = 0 \\ \quad H_f \\ \text{else} \\ \quad H_r \end{cases} = 26\ ft$$

First Floor Load

$d_{rf} = 0\ ft$ Distance away from next column to the right in plan view

$d_{lf} = 24\ ft$ Distance away from next column to the left in plan view

$d_{uf} = 13\ ft$ Distance away from next column above in plan view

$d_{df} = 0\ ft$ Distance away from next column below in plan view

$$T_{Af} = \left(\frac{d_{rf}}{2} + \frac{d_{lf}}{2}\right) \cdot \left(\frac{d_{uf}}{2} + \frac{d_{df}}{2}\right) = 78\ ft^2$$

$P_{u1} = F_1 \cdot T_{Af} = 18.096\ kip$

Second Floor Load

$P_{u2} = F_2 \cdot T_{Af} = 10.608\ kip$

Roof Load $K_r = 1$ 1 matching, 0 for non-matching

$d_{rr} = \text{if } K_r = 1 \; \| \; d_{rf} \; \text{else} \; \| \; 16 \; ft$ $= 0 \; ft$ Distance away from next column to the right in plan view

$d_{lr} = \text{if } K_r = 1 \; \| \; d_{lf} \; \text{else} \; \| \; 12 \; ft$ $= 24 \; ft$ Distance away from next column to the left in plan view

$d_{ur} = \text{if } K_r = 1 \; \| \; d_{uf} \; \text{else} \; \| \; 13 \; ft$ $= 13 \; ft$ Distance away from next column above in plan view

$d_{dr} = \text{if } K_r = 1 \; \| \; d_{df} \; \text{else} \; \| \; 0 \; ft$ $= 0 \; ft$ Distance away from next column below in plan view

$$T_{Ar} = \left(\frac{d_{rr}}{2} + \frac{d_{lr}}{2}\right) \cdot \left(\frac{d_{ur}}{2} + \frac{d_{dr}}{2}\right) = 78 \; ft^2$$

$$P_{ur} = F_r \cdot T_{Ar} = 14.414 \; kip$$

Strength Requirements $P_{ur} = 14.414 \; kip$

$$P_u = \text{ceil}\left(\frac{P_{u1} + P_{u2} + P_{ur}}{kip}\right) kip = 44 \; kip$$

Member Strength

$H = 26 \; ft$ HSS 6 x 6 x 5/16

$$\phi_c P_n = 79.6 \; kip$$

$$U = \frac{P_u}{\phi_c P_n} \cdot 100 = 55.276$$ Percent strength of column utilized

Design Summary

An HSS 6 x 6 x 5/16 for column D8 satisfies this preliminary design

Footing Design:

$$A = \frac{P_u}{q} = 29.333\ ft^2$$

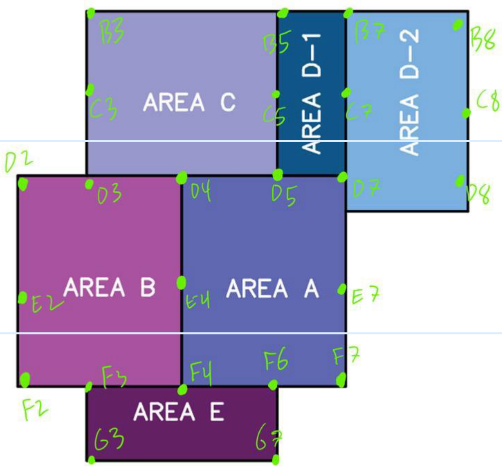
$$L = \sqrt{A} = 5.416\ ft$$

Use 5.5 ft square footing

Column E2 Analysis

Assumptions (if needed)

- Risk factor IV
- Foundation to Roof 26'
- Foundation to 2nd floor 16'



Define Variables

$T_r = 0$ 1 for roof bearing, 0 for non-roof bearing

$$H = \begin{cases} H_f & \text{if } T_r = 0 \\ H_r & \text{else} \end{cases} = 16\ ft$$

First Floor Load

$d_{rf} = 13\ ft$ Distance away from next column to the right in plan view

$d_{lf} = 0\ ft$ Distance away from next column to the left in plan view

$d_{uf} = 19\ ft$ Distance away from next column above in plan view

$d_{df} = 14\ ft$ Distance away from next column below in plan view

$$T_{Af} = \left(\frac{d_{rf}}{2} + \frac{d_{lf}}{2}\right) \cdot \left(\frac{d_{uf}}{2} + \frac{d_{df}}{2}\right) = 107.25\ ft^2$$

$P_{u1} = F_1 \cdot T_{Af} = 24.882\ kip$

Second Floor Load

$P_{u2} = F_2 \cdot T_{Af} = 14.586\ kip$

Roof Load

$K_r = 0$ 1 matching, 0 for non-matching

$d_{rr} = \begin{cases} \text{if } K_r = 1 \\ \quad d_{rf} \\ \text{else} \\ \quad 0\ ft \end{cases} = 0\ ft$ Distance away from next column to the right in plan view

$d_{lr} = \begin{cases} \text{if } K_r = 1 \\ \quad d_{lf} \\ \text{else} \\ \quad 0\ ft \end{cases} = 0\ ft$ Distance away from next column to the left in plan view

$d_{ur} = \begin{cases} \text{if } K_r = 1 \\ \quad d_{uf} \\ \text{else} \\ \quad 0\ ft \end{cases} = 0\ ft$ Distance away from next column above in plan view

$d_{dr} = \begin{cases} \text{if } K_r = 1 \\ \quad d_{df} \\ \text{else} \\ \quad 0\ ft \end{cases} = 0\ ft$ Distance away from next column below in plan view

$$T_{Ar} = \left(\frac{d_{rr}}{2} + \frac{d_{lr}}{2}\right) \cdot \left(\frac{d_{ur}}{2} + \frac{d_{dr}}{2}\right) = 0\ ft^2$$

$$P_{ur} = F_r \cdot T_{Ar} = 0\ kip$$

Strength Requirements

$P_{ur} = 0\ kip$

$$P_u = \text{ceil}\left(\frac{P_{u1} + P_{u2} + P_{ur}}{kip}\right) kip = 40\ kip$$

Member Strength

$H = 16\ ft$ HSS 4 x 4 x 5/16

$$\phi_c P_n = 55.8\ kip$$

$$U = \frac{P_u}{\phi_c P_n} \cdot 100 = 71.685$$ Percent strength of column utilized

Design Summary

An HSS 4 x 4 x 5/16 for column E2 satisfies this preliminary design

Footing Design:

$$A = \frac{P_u}{q} = 26.667 \; ft^2$$

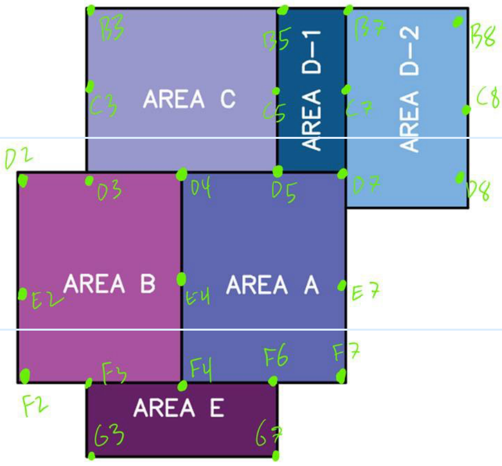
$$L = \sqrt{A} = 5.164 \; ft$$

Use 5.5 ft square footing

Column E4 Analysis

Assumptions (if needed)

- Risk factor IV
- Foundation to Roof 26'
- Foundation to 2nd floor 16'



Define Variables

$T_r = 0$ 1 for roof bearing, 0 for non-roof bearing

$$H = \begin{cases} \text{if } T_r = 0 \\ \quad H_f \\ \text{else} \\ \quad H_r \end{cases} = 16 \ ft$$

First Floor Load

$d_{rf} = 12 \ ft$ Distance away from next column to the right in plan view

$d_{lf} = 15 \ ft$ Distance away from next column to the left in plan view

$d_{uf} = 19 \ ft$ Distance away from next column above in plan view

$d_{df} = 14 \ ft$ Distance away from next column below in plan view

$$T_{Af} = \left(\frac{d_{rf}}{2} + \frac{d_{lf}}{2}\right) \cdot \left(\frac{d_{uf}}{2} + \frac{d_{df}}{2}\right) = 222.75 \ ft^2$$

$P_{u1} = F_1 \cdot T_{Af} = 51.678 \ kip$

Second Floor Load

$P_{u2} = F_2 \cdot T_{Af} = 30.294 \ kip$

Roof Load $K_r = 0$ 1 matching, 0 for non-matching

$d_{rr} = \begin{cases} d_{rf} & \text{if } K_r = 1 \\ 0\ ft & \text{else} \end{cases} = 0\ ft$ Distance away from next column to the right in plan view

$d_{lr} = \begin{cases} d_{lf} & \text{if } K_r = 1 \\ 0\ ft & \text{else} \end{cases} = 0\ ft$ Distance away from next column to the left in plan view

$d_{ur} = \begin{cases} d_{uf} & \text{if } K_r = 1 \\ 0\ ft & \text{else} \end{cases} = 0\ ft$ Distance away from next column above in plan view

$d_{dr} = \begin{cases} d_{df} & \text{if } K_r = 1 \\ 0\ ft & \text{else} \end{cases} = 0\ ft$ Distance away from next column below in plan view

$$T_{Ar} = \left(\frac{d_{rr}}{2} + \frac{d_{lr}}{2}\right) \cdot \left(\frac{d_{ur}}{2} + \frac{d_{dr}}{2}\right) = 0\ ft^2$$

$$P_{ur} = F_r \cdot T_{Ar} = 0\ kip$$

Strength Requirements $P_{ur} = 0\ kip$

$$P_u = \text{ceil}\left(\frac{P_{u1} + P_{u2} + P_{ur}}{kip}\right) kip = 82\ kip$$

Member Strength

$H = 16\ ft$ HSS 5 x 5 x 1/4

$$\phi_c P_n = 93.8\ kip$$

$$U = \frac{P_u}{\phi_c P_n} \cdot 100 = 87.42$$ Percent strength of column utilized

Design Summary

An HSS 5 x 5 x 1/4 for column E4 satisfies this preliminary design

Footing Design:

$$A = \frac{P_u}{q} = 54.667\ \boldsymbol{ft}^2$$

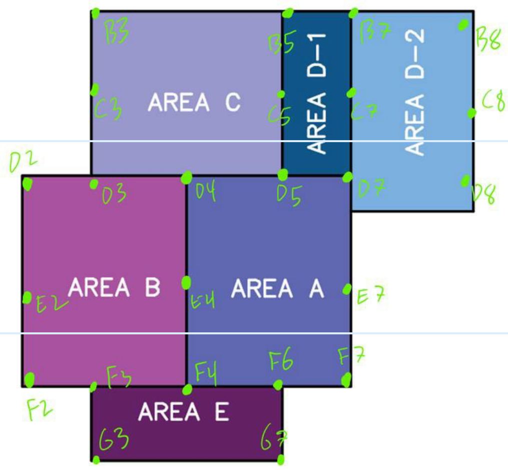
$$L = \sqrt{A} = 7.394\ \boldsymbol{ft}$$

Use 7.5 ft square footing

Column E7 Analysis

Assumptions (if needed)

- Risk factor IV
- Foundation to Roof 26'
- Foundation to 2nd floor 16'



Define Variables

$T_r = 0$ 1 for roof bearing, 0 for non-roof bearing

$$H = \begin{cases} H_f & \text{if } T_r = 0 \\ H_r & \text{else} \end{cases} = 16\ ft$$

First Floor Load

$d_{rf} = 0\ ft$ Distance away from next column to the right in plan view

$d_{lf} = 16\ ft$ Distance away from next column to the left in plan view

$d_{uf} = 19\ ft$ Distance away from next column above in plan view

$d_{df} = 14\ ft$ Distance away from next column below in plan view

$$T_{Af} = \left(\frac{d_{rf}}{2} + \frac{d_{lf}}{2}\right) \cdot \left(\frac{d_{uf}}{2} + \frac{d_{df}}{2}\right) = 132\ ft^2$$

$$P_{u1} = F_1 \cdot T_{Af} = 30.624\ kip$$

Second Floor Load

$$P_{u2} = F_2 \cdot T_{Af} = 17.952\ kip$$

Roof Load

$K_r = 0$ 1 matching, 0 for non-matching

$$d_{rr} = \begin{cases} d_{rf} & \text{if } K_r = 1 \\ 0\ ft & \text{else} \end{cases} = 0\ ft$$

Distance away from next column to the right in plan view

$$d_{lr} = \begin{cases} d_{lf} & \text{if } K_r = 1 \\ 0\ ft & \text{else} \end{cases} = 0\ ft$$

Distance away from next column to the left in plan view

$$d_{ur} = \begin{cases} d_{uf} & \text{if } K_r = 1 \\ 0\ ft & \text{else} \end{cases} = 0\ ft$$

Distance away from next column above in plan view

$$d_{dr} = \begin{cases} d_{df} & \text{if } K_r = 1 \\ 0\ ft & \text{else} \end{cases} = 0\ ft$$

Distance away from next column below in plan view

$$T_{Ar} = \left(\frac{d_{rr}}{2} + \frac{d_{lr}}{2}\right) \cdot \left(\frac{d_{ur}}{2} + \frac{d_{dr}}{2}\right) = 0\ ft^2$$

$$P_{ur} = F_r \cdot T_{Ar} = 0\ kip$$

Strength Requirements

$P_{ur} = 0\ kip$

$$P_u = \text{ceil}\left(\frac{P_{u1} + P_{u2} + P_{ur}}{kip}\right) kip = 49\ kip$$

Member Strength

$H = 16\ ft$ HSS 4 x 4 x 5/16

$$\phi_c P_n = 55.8\ kip$$

$$U = \frac{P_u}{\phi_c P_n} \cdot 100 = 87.814$$

Percent strength of column utilized

Design Summary

An HSS 4 x 4 x 5/16 for column E7 satisfies this preliminary design

Footing Design:

$$A = \frac{P_u}{q} = 32.667 \ ft^2$$

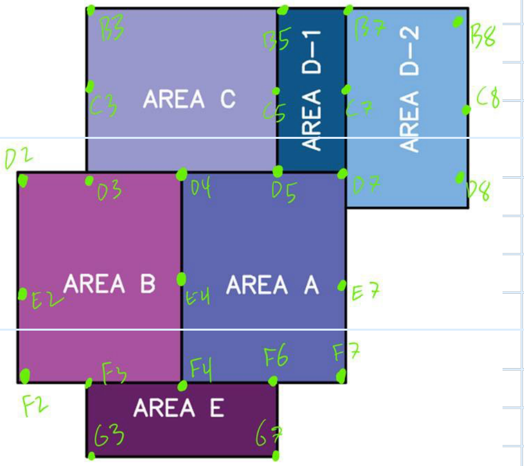
$$L = \sqrt{A} = 5.715 \ ft$$

Use 6 ft square footing

Column F2 Analysis

Assumptions (if needed)

- Risk factor IV
- Foundation to Roof 26'
- Foundation to 2nd floor 16'



Define Variables

$T_r = 1$ 1 for roof bearing, 0 for non-roof bearing

$H = \begin{cases} \text{if } T_r = 0 \\ \quad H_f \\ \text{else} \\ \quad H_r \end{cases} = 26\ ft$

First Floor Load

$d_{rf} = 13\ ft$ Distance away from next column to the right in plan view

$d_{lf} = 0\ ft$ Distance away from next column to the left in plan view

$d_{uf} = 14\ ft$ Distance away from next column above in plan view

$d_{df} = 0\ ft$ Distance away from next column below in plan view

$$T_{Af} = \left(\frac{d_{rf}}{2} + \frac{d_{lf}}{2}\right) \cdot \left(\frac{d_{uf}}{2} + \frac{d_{df}}{2}\right) = 45.5\ ft^2$$

$P_{u1} = F_1 \cdot T_{Af} = 10.556\ kip$

Second Floor Load

$P_{u2} = F_2 \cdot T_{Af} = 6.188\ kip$

Roof Load $K_r = 1$ 1 matching, 0 for non-matching

$d_{rr} = \text{if } K_r = 1 \; \| \; d_{rf} \; \text{else} \; \| \; 0\ ft = 13\ ft$ Distance away from next column to the right in plan view

$d_{lr} = \text{if } K_r = 1 \; \| \; d_{lf} \; \text{else} \; \| \; 0\ ft = 0\ ft$ Distance away from next column to the left in plan view

$d_{ur} = \text{if } K_r = 1 \; \| \; d_{uf} \; \text{else} \; \| \; 0\ ft = 14\ ft$ Distance away from next column above in plan view

$d_{dr} = \text{if } K_r = 1 \; \| \; d_{df} \; \text{else} \; \| \; 0\ ft = 0\ ft$ Distance away from next column below in plan view

$$T_{Ar} = \left(\frac{d_{rr}}{2} + \frac{d_{lr}}{2}\right) \cdot \left(\frac{d_{ur}}{2} + \frac{d_{dr}}{2}\right) = 45.5\ ft^2$$

$$P_{ur} = F_r \cdot T_{Ar} = 8.408\ kip$$

Strength Requirements $P_{ur} = 8.408\ kip$

$$P_u = \text{ceil}\left(\frac{P_{u1} + P_{u2} + P_{ur}}{kip}\right) kip = 26\ kip$$

Member Strength

$H = 26\ ft$ HSS 5 x 5 x 1/4

$\phi_c P_n = 37.2\ kip$

$U = \frac{P_u}{\phi_c P_n} \cdot 100 = 69.892$ Percent strength of column utilized

Design Summary

An HSS 5 x 5 x 1/4 for column F2 satisfies this preliminary design

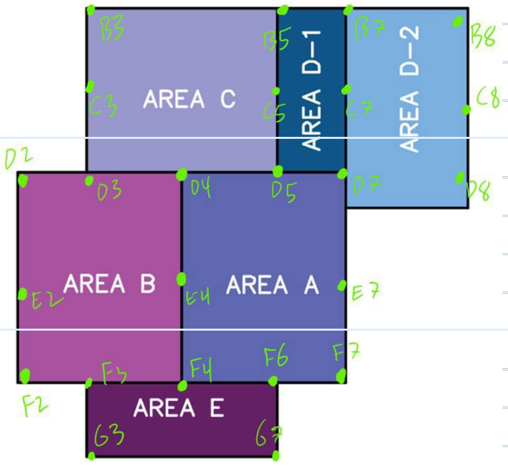
Footing Design:

$$A = \frac{P_u}{q} = 17.333\ \boldsymbol{ft}^2$$

$$L = \sqrt{A} = 4.163\ \boldsymbol{ft}$$

Use 4.5 ft square footing

Column F3 Analysis



Assumptions (if needed)

- Risk factor IV
- Foundation to Roof 26'
- Foundation to 2nd floor 16'

Define Variables

$T_r = 1$ 1 for roof bearing, 0 for non-roof bearing

$$H = \begin{cases} \text{if } T_r = 0 \\ \quad H_f \\ \text{else} \\ \quad H_r \end{cases} = 26\ ft$$

First Floor Load

$d_{rf} = 15\ ft$ Distance away from next column to the right in plan view

$d_{lf} = 13\ ft$ Distance away from next column to the left in plan view

$d_{uf} = 14\ ft$ Distance away from next column above in plan view

$d_{df} = 14\ ft$ Distance away from next column below in plan view

$$T_{Af} = \left(\frac{d_{rf}}{2} + \frac{d_{lf}}{2}\right) \cdot \left(\frac{d_{uf}}{2} + \frac{d_{df}}{2}\right) - \left(\frac{d_{df}}{2} \cdot \frac{d_{lf}}{2}\right) = 150.5\ ft^2$$

$$P_{u1} = F_1 \cdot T_{Af} = 34.916\ kip$$

Second Floor Load

$$P_{u2} = F_2 \cdot T_{Af} = 20.468\ kip$$

Roof Load $K_r = 1$ 1 matching, 0 for non-matching

$d_{rr} = \begin{cases} \text{if } K_r = 1 & d_{rf} \\ \text{else} & 15\ ft \end{cases} = 15\ ft$ Distance away from next column to the right in plan view

$d_{lr} = \begin{cases} \text{if } K_r = 1 & d_{lf} \\ \text{else} & 0\ ft \end{cases} = 13\ ft$ Distance away from next column to the left in plan view

$d_{ur} = \begin{cases} \text{if } K_r = 1 & d_{uf} \\ \text{else} & 13\ ft \end{cases} = 14\ ft$ Distance away from next column above in plan view

$d_{dr} = \begin{cases} \text{if } K_r = 1 & d_{df} \\ \text{else} & 0\ ft \end{cases} = 14\ ft$ Distance away from next column below in plan view

$$T_{Ar} = \left(\frac{d_{rr}}{2} + \frac{d_{lr}}{2}\right) \cdot \left(\frac{d_{ur}}{2} + \frac{d_{dr}}{2}\right) - \left(\frac{d_{dr}}{2} \cdot \frac{d_{lr}}{2}\right) = 150.5\ ft^2$$

$P_{ur} = F_r \cdot T_{Ar} = 27.812\ kip$

Strength Requirements $P_{ur} = 27.812\ kip$

$$P_u = \text{ceil}\left(\frac{P_{u1} + P_{u2} + P_{ur}}{kip}\right) kip = 84\ kip$$

Member Strength

$H = 26\ ft$ HSS 7 x 7 x 1/4

$\phi_c P_n = 108\ kip$

$U = \frac{P_u}{\phi_c P_n} \cdot 100 = 77.778$ Percent strength of column utilized

Design Summary

An HSS 7 x 7 x 1/4 for column F3 satisfies this preliminary design

Footing Design:

$$A = \frac{P_u}{q} = 56\ ft^2$$

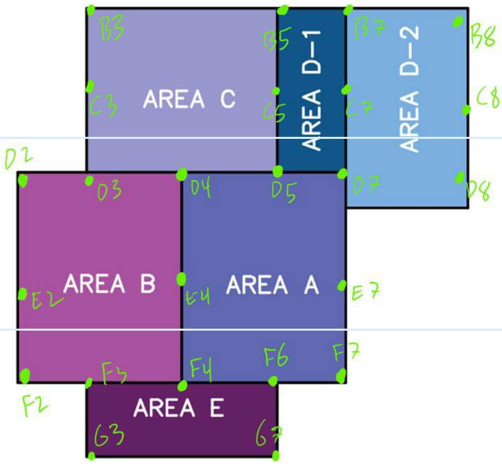
$$L = \sqrt{A} = 7.483\ ft$$

Use 7.5 ft square footing

Column F4 Analysis

Assumptions (if needed)

- Risk factor IV
- Foundation to Roof 26'
- Foundation to 2nd floor 16'



Define Variables

$T_r = 1$ 1 for roof bearing, 0 for non-roof bearing

$$H = \begin{cases} H_f & \text{if } T_r = 0 \\ H_r & \text{else} \end{cases} = 26\ ft$$

First Floor Load

$d_{rf} = 15\ ft$ Distance away from next column to the right in plan view

$d_{lf} = 15\ ft$ Distance away from next column to the left in plan view

$d_{uf} = 14\ ft$ Distance away from next column above in plan view

$d_{df} = 14\ ft$ Distance away from next column below in plan view

$$T_{Af} = \left(\frac{d_{rf}}{2} + \frac{d_{lf}}{2}\right) \cdot \left(\frac{d_{uf}}{2} + \frac{d_{df}}{2}\right) = 210\ ft^2$$

$P_{u1} = F_1 \cdot T_{Af} = 48.72\ kip$

Second Floor Load

$P_{u2} = F_2 \cdot T_{Af} = 28.56\ kip$

Roof Load $K_r = 0$ 1 matching, 0 for non-matching

$d_{rr} = \begin{cases} \text{if } K_r = 1 & d_{rf} \\ \text{else} & 15\ ft \end{cases} = 15\ ft$ Distance away from next column to the right in plan view

$d_{lr} = \begin{cases} \text{if } K_r = 1 & d_{lf} \\ \text{else} & 15\ ft \end{cases} = 15\ ft$ Distance away from next column to the left in plan view

$d_{ur} = \begin{cases} \text{if } K_r = 1 & d_{uf} \\ \text{else} & 14\ ft \end{cases} = 14\ ft$ Distance away from next column above in plan view

$d_{dr} = \begin{cases} \text{if } K_r = 1 & d_{df} \\ \text{else} & 0\ ft \end{cases} = 0\ ft$ Distance away from next column below in plan view

$T_{Ar} = \left(\frac{d_{rr}}{2} + \frac{d_{lr}}{2}\right) \cdot \left(\frac{d_{ur}}{2} + \frac{d_{dr}}{2}\right) = 105\ ft^2$

$P_{ur} = F_r \cdot T_{Ar} = 19.404\ kip$

Strength Requirements $P_{ur} = 19.404\ kip$

$P_u = \text{ceil}\left(\frac{P_{u1} + P_{u2} + P_{ur}}{kip}\right) kip = 97\ kip$

Member Strength

$H = 26\ ft$ HSS 7 x 7 x 1/4

$\phi_c P_n = 108\ kip$

$U = \frac{P_u}{\phi_c P_n} \cdot 100 = 89.815$ Percent strength of column utilized

Design Summary

An HSS 7 x 7 x 1/4 for column F4 satisfies this preliminary design

Footing Design:

$$A = \frac{P_u}{q} = 64.667\ ft^2$$

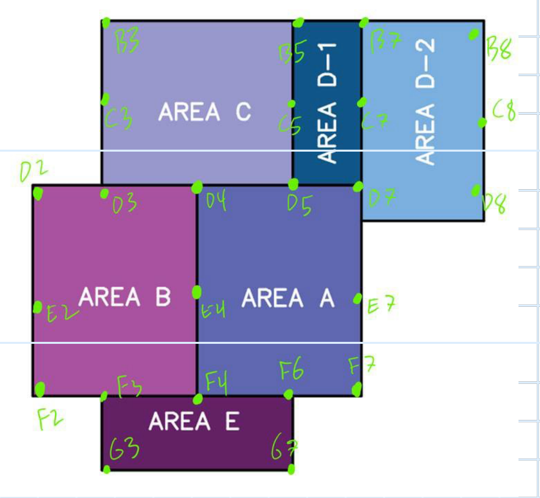
$$L = \sqrt{A} = 8.042\ ft$$

Use 8.5 ft square footing

Column F6 Analysis

Assumptions (if needed)

- Risk factor IV
- Foundation to Roof 26'
- Foundation to 2nd floor 16'



Define Variables

$T_r = 1$ 1 for roof bearing, 0 for non-roof bearing

$$H = \begin{cases} \text{if } T_r = 0 \\ \quad H_f \\ \text{else} \\ \quad H_r \end{cases} = 26 \ \mathbf{ft}$$

First Floor Load

$d_{rf} = 13 \ \mathbf{ft}$ Distance away from next column to the right in plan view

$d_{lf} = 16 \ \mathbf{ft}$ Distance away from next column to the left in plan view

$d_{uf} = 14 \ \mathbf{ft}$ Distance away from next column above in plan view

$d_{df} = 14 \ \mathbf{ft}$ Distance away from next column below in plan view

$$T_{Af} = \left(\frac{d_{rf}}{2} + \frac{d_{lf}}{2}\right) \cdot \left(\frac{d_{uf}}{2} + \frac{d_{df}}{2}\right) - \left(\frac{d_{df}}{2} \cdot \frac{d_{rf}}{2}\right) = 157.5 \ \mathbf{ft}^2$$

$P_{u1} = F_1 \cdot T_{Af} = 36.54 \ \mathbf{kip}$

Second Floor Load

$P_{u2} = F_2 \cdot T_{Af} = 21.42 \ \mathbf{kip}$

Roof Load $K_r = 1$ 1 matching, 0 for non-matching

$d_{rr} = \text{if } K_r = 1 \;\; d_{rf} \;\; \text{else} \;\; 16\ ft = 13\ ft$ Distance away from next column to the right in plan view

$d_{lr} = \text{if } K_r = 1 \;\; d_{lf} \;\; \text{else} \;\; 12\ ft = 16\ ft$ Distance away from next column to the left in plan view

$d_{ur} = \text{if } K_r = 1 \;\; d_{uf} \;\; \text{else} \;\; 13\ ft = 14\ ft$ Distance away from next column above in plan view

$d_{dr} = \text{if } K_r = 1 \;\; d_{df} \;\; \text{else} \;\; 0\ ft = 14\ ft$ Distance away from next column below in plan view

$$T_{Ar} = \left(\frac{d_{rr}}{2} + \frac{d_{lr}}{2}\right) \cdot \left(\frac{d_{ur}}{2} + \frac{d_{dr}}{2}\right) - \left(\frac{d_{dr}}{2} \cdot \frac{d_{rr}}{2}\right) = 157.5\ ft^2$$

$$P_{ur} = F_r \cdot T_{Ar} = 29.106\ kip$$

Strength Requirements $P_{ur} = 29.106\ kip$

$$P_u = \text{ceil}\left(\frac{P_{u1} + P_{u2} + P_{ur}}{kip}\right) kip = 88\ kip$$

Member Strength

$H = 26\ ft$ HSS 7 x 7 x 1/4

$$\phi_c P_n = 108\ kip$$

$$U = \frac{P_u}{\phi_c P_n} \cdot 100 = 81.481$$ Percent strength of column utilized

Design Summary

An HSS 7 x 7 x 1/4 for column F6 satisfies this preliminary design

Footing Design:

$$A = \frac{P_u}{q} = 58.667\ ft^2$$

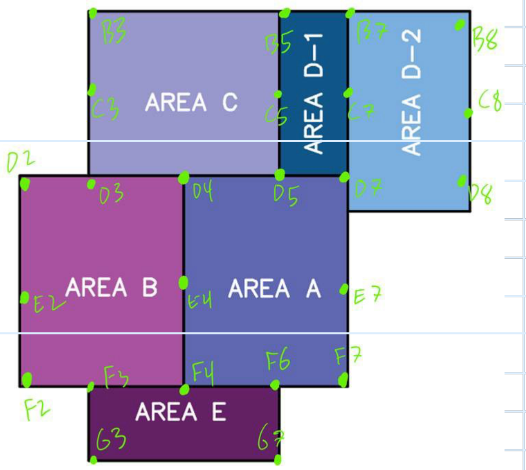
$$L = \sqrt{A} = 7.659\ ft$$

Use 8 ft square footing

Column F7 Analysis

Assumptions (if needed)

- Risk factor IV
- Foundation to Roof 26'
- Foundation to 2nd floor 16'



Define Variables

$T_r = 1$ 1 for roof bearing, 0 for non-roof bearing

$$H = \begin{cases} \text{if } T_r = 0 \\ \quad H_f \\ \text{else} \\ \quad H_r \end{cases} = 26\ ft$$

First Floor Load

$d_{rf} = 0\ ft$ Distance away from next column to the right in plan view

$d_{lf} = 13\ ft$ Distance away from next column to the left in plan view

$d_{uf} = 14\ ft$ Distance away from next column above in plan view

$d_{df} = 0\ ft$ Distance away from next column below in plan view

$$T_{Af} = \left(\frac{d_{rf}}{2} + \frac{d_{lf}}{2}\right) \cdot \left(\frac{d_{uf}}{2} + \frac{d_{df}}{2}\right) = 45.5\ ft^2$$

$P_{u1} = F_1 \cdot T_{Af} = 10.556\ kip$

Second Floor Load

$P_{u2} = F_2 \cdot T_{Af} = 6.188\ kip$

Roof Load $\quad K_r = 1 \quad$ 1 matching, 0 for non-matching

$$d_{rr} = \begin{cases} \text{if } K_r = 1 \\ \quad d_{rf} \\ \text{else} \\ \quad 16\ ft \end{cases} = 0\ ft$$

Distance away from next column to the right in plan view

$$d_{lr} = \begin{cases} \text{if } K_r = 1 \\ \quad d_{lf} \\ \text{else} \\ \quad 12\ ft \end{cases} = 13\ ft$$

Distance away from next column to the left in plan view

$$d_{ur} = \begin{cases} \text{if } K_r = 1 \\ \quad d_{uf} \\ \text{else} \\ \quad 13\ ft \end{cases} = 14\ ft$$

Distance away from next column above in plan view

$$d_{dr} = \begin{cases} \text{if } K_r = 1 \\ \quad d_{df} \\ \text{else} \\ \quad 0\ ft \end{cases} = 0\ ft$$

Distance away from next column below in plan view

$$T_{Ar} = \left(\frac{d_{rr}}{2} + \frac{d_{lr}}{2}\right) \cdot \left(\frac{d_{ur}}{2} + \frac{d_{dr}}{2}\right) = 45.5\ ft^2$$

$$P_{ur} = F_r \cdot T_{Ar} = 8.408\ kip$$

Strength Requirements $\quad P_{ur} = 8.408\ kip$

$$P_u = \text{ceil}\left(\frac{P_{u1} + P_{u2} + P_{ur}}{kip}\right) kip = 26\ kip$$

Member Strength

$H = 26\ ft \quad$ HSS 5 x 5 x 1/4

$$\phi_c P_n = 37.2\ kip$$

$$U = \frac{P_u}{\phi_c P_n} \cdot 100 = 69.892$$ Percent strength of column utilized

Design Summary

An HSS 5 x 5 x 1/4 for column F7 satisfies this preliminary design

Footing Design:

$$A = \frac{P_u}{q} = 17.333\ ft^2$$

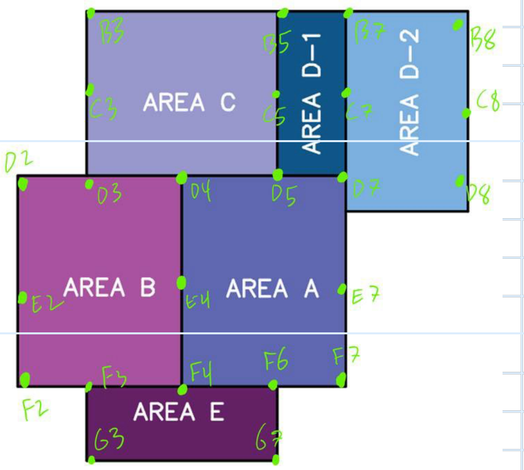
$$L = \sqrt{A} = 4.163\ ft$$

Use 4.5 ft square footing

Column G3 Analysis

Assumptions (if needed)

- Risk factor IV
- Foundation to Roof 26'
- Foundation to 2nd floor 16'



Define Variables

$T_r = 1$ 1 for roof bearing, 0 for non-roof bearing

$$H = \begin{cases} \text{if } T_r = 0 \\ \quad H_f \\ \text{else} \\ \quad H_r \end{cases} = 26\ ft$$

First Floor Load

$d_{rf} = 15\ ft$ Distance away from next column to the right in plan view

$d_{lf} = 0\ ft$ Distance away from next column to the left in plan view

$d_{uf} = 14\ ft$ Distance away from next column above in plan view

$d_{df} = 0\ ft$ Distance away from next column below in plan view

$$T_{Af} = \left(\frac{d_{rf}}{2} + \frac{d_{lf}}{2}\right) \cdot \left(\frac{d_{uf}}{2} + \frac{d_{df}}{2}\right) = 52.5\ ft^2$$

$P_{u1} = F_1 \cdot T_{Af} = 12.18\ kip$

Second Floor Load

$P_{u2} = F_2 \cdot T_{Af} = 7.14\ kip$

Roof Load

$K_r = 1$ 1 matching, 0 for non-matching

$d_{rr} = \begin{cases} \text{if } K_r = 1 & d_{rf} \\ \text{else} & 0\ ft \end{cases} = 15\ ft$ Distance away from next column to the right in plan view

$d_{lr} = \begin{cases} \text{if } K_r = 1 & d_{lf} \\ \text{else} & 0\ ft \end{cases} = 0\ ft$ Distance away from next column to the left in plan view

$d_{ur} = \begin{cases} \text{if } K_r = 1 & d_{uf} \\ \text{else} & 0\ ft \end{cases} = 14\ ft$ Distance away from next column above in plan view

$d_{dr} = \begin{cases} \text{if } K_r = 1 & d_{df} \\ \text{else} & 0\ ft \end{cases} = 0\ ft$ Distance away from next column below in plan view

$T_{Ar} = \left(\frac{d_{rr}}{2} + \frac{d_{lr}}{2}\right) \cdot \left(\frac{d_{ur}}{2} + \frac{d_{dr}}{2}\right) = 52.5\ ft^2$

$P_{ur} = F_r \cdot T_{Ar} = 9.702\ kip$

Strength Requirements

$P_{ur} = 9.702\ kip$

$P_u = \text{ceil}\left(\frac{P_{u1} + P_{u2} + P_{ur}}{kip}\right) kip = 30\ kip$

Member Strength

$H = 26\ ft$ HSS 5 x 5 x 1/4

$\phi_c P_n = 37.2\ kip$

$U = \frac{P_u}{\phi_c P_n} \cdot 100 = 80.645$ Percent strength of column utilized

Design Summary

An HSS 5 x 5 x 1/4 for column G3 satisfies this preliminary design

Footing Design:

$$A = \frac{P_u}{q} = 20\ ft^2$$

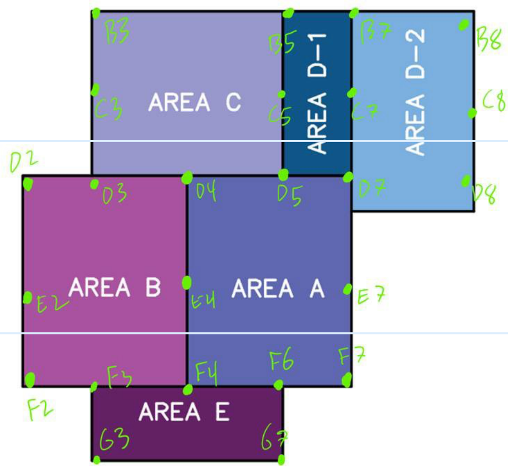
$$L = \sqrt{A} = 4.472\ ft$$

Use 4.5 ft square footing

Column G6 Analysis

Assumptions (if needed)

- Risk factor IV
- Foundation to Roof 26'
- Foundation to 2nd floor 16'



Define Variables

$T_r = 1$ 1 for roof bearing, 0 for non-roof bearing

$$H = \begin{cases} \text{if } T_r = 0 \\ \quad H_f \\ \text{else} \\ \quad H_r \end{cases} = 26\ ft$$

First Floor Load

$d_{rf} = 0\ ft$ Distance away from next column to the right in plan view

$d_{lf} = 15\ ft$ Distance away from next column to the left in plan view

$d_{uf} = 0\ ft$ Distance away from next column above in plan view

$d_{df} = 14\ ft$ Distance away from next column below in plan view

$$T_{Af} = \left(\frac{d_{rf}}{2} + \frac{d_{lf}}{2}\right) \cdot \left(\frac{d_{uf}}{2} + \frac{d_{df}}{2}\right) = 52.5\ ft^2$$

$P_{u1} = F_1 \cdot T_{Af} = 12.18\ kip$

Second Floor Load

$P_{u2} = F_2 \cdot T_{Af} = 7.14\ kip$

Roof Load $K_r = 1$ 1 matching, 0 for non-matching

$d_{rr} = \text{if } K_r = 1 \;\; d_{rf} \;\; \text{else} \;\; 0\ ft = 0\ ft$ Distance away from next column to the right in plan view

$d_{lr} = \text{if } K_r = 1 \;\; d_{lf} \;\; \text{else} \;\; 0\ ft = 15\ ft$ Distance away from next column to the left in plan view

$d_{ur} = \text{if } K_r = 1 \;\; d_{uf} \;\; \text{else} \;\; 0\ ft = 0\ ft$ Distance away from next column above in plan view

$d_{dr} = \text{if } K_r = 1 \;\; d_{df} \;\; \text{else} \;\; 0\ ft = 14\ ft$ Distance away from next column below in plan view

$$T_{Ar} = \left(\frac{d_{rr}}{2} + \frac{d_{lr}}{2}\right) \cdot \left(\frac{d_{ur}}{2} + \frac{d_{dr}}{2}\right) = 52.5\ ft^2$$

$$P_{ur} = F_r \cdot T_{Ar} = 9.702\ kip$$

Strength Requirements $P_{ur} = 9.702\ kip$

$$P_u = \text{ceil}\left(\frac{P_{u1} + P_{u2} + P_{ur}}{kip}\right) kip = 30\ kip$$

Member Strength

$H = 26\ ft$ HSS 5 x 5 x 1/4

$$\phi_c P_n = 37.2\ kip$$

$$U = \frac{P_u}{\phi_c P_n} \cdot 100 = 80.645$$ Percent strength of column utilized

Design Summary

An HSS 5 x 5 x 1/4 for column G6 satisfies this preliminary design

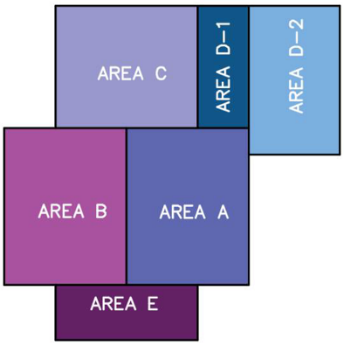
Footing Design:

$$A = \frac{P_u}{q} = 20\ ft^2$$

$$L = \sqrt{A} = 4.472\ ft$$

Use 4.5 ft square footing

Truss Manufacturing Design



Truss design to be performed by cold roll steel truss manufacturer. Designs should be completed for a risk category IV building with the following load, span, and truss dimensions.

Area A Truss Requirements

$H = 11\ ft$ Truss height is 11 ft.

$Length = 37\ ft$ Truss 2 be designed for a 33 ft. span including a 2 ft. overhang on either side.

$S = 87\ psf$ Truss to be designed for this snow loading.

$D = 34\ psf$ Truss to be designed for this superimposed dead load.

$W = 10\ psf$ Truss to be designed for this wind load.

$W_u = -\ 26\ psf$ Truss to be designed for this wind uplift.

Area B Truss Requirements

$H = 11\ ft$ Truss height is 11 ft.

$Length = 37\ ft$ Truss 2 be designed for a 33 ft. span including a 2 ft. overhang on either side.

$S = 87\ psf$ Truss to be designed for this snow loading.

$D = 34\ psf$ Truss to be designed for this superimposed dead load.

$W = 10\ psf$ Truss to be designed for this wind load.

$W_u = -\ 26\ psf$ Truss to be designed for this wind uplift.

Area C Truss Requirements

$H = 11$ ***ft*** Truss height is 11 ft.

$Length = 26$ ***ft*** Truss 2 be designed for a 30 ft. span including a 2 ft. overhang on either side.

$S = 87$ ***psf*** Truss to be designed for this snow loading.

$D = 34$ ***psf*** Truss to be designed for this superimposed dead load.

$W = 10$ ***psf*** Truss to be designed for this wind load.

$W_u = -26$ ***psf*** Truss to be designed for this wind uplift.

Area D Truss Requirements

$H = 11$ ***ft*** Truss height is 11 ft.

$Length = 35$ ***ft*** Truss 2 be designed for a 31 ft. span including a 2 ft. overhang on either side.

$S = 87$ ***psf*** Truss to be designed for this snow loading.

$D = 34$ ***psf*** Truss to be designed for this superimposed dead load.

$W = 10$ ***psf*** Truss to be designed for this wind load.

$W_u = -26$ ***psf*** Truss to be designed for this wind uplift.

Area E Truss Requirements

$H = 11$ ***ft*** Truss height is 11 ft.

$Length = 33$ ***ft*** Truss 2 be designed for a 29 ft. span including a 2 ft. overhang on either side.

$S = 87$ ***psf*** Truss to be designed for this snow loading.

$D = 34$ ***psf*** Truss to be designed for this superimposed dead load.

$W = 10$ ***psf*** Truss to be designed for this wind load.

$W_u = -26$ ***psf*** Truss to be designed for this wind uplift.

Isolated Footings Check: (Check for Largest Footing Size)

Footing Size:

$B = 11\ \mathbf{ft}$

$H = 11\ \mathbf{ft}$

$t_f = 2.5\ \mathbf{ft}$

$h = t_f$

Column Size

$B_{col} = 9\ \mathbf{in}$

Column Load:

$P_c = 180\ \mathbf{kip}$

Assumptions:

$q_u = 1.5\ \mathbf{ksf}$

$\gamma'_{con} = 150\ \mathbf{pcf}$

$f'_c = 4000\ \mathbf{psi}$

$f_y = 60\ \mathbf{ksi}$

$c_c = 3\ \mathbf{in}$

$D_b = 1.128\ \mathbf{in}$

Compute Effective Depth:

$d = h - c_c - D_b = 25.872\ \mathbf{in}$

Bearing Pressure:

$q_u = 1.5\ \mathbf{ksf}$

Check One Way Shear:

$$V_{uOneWay} = q_u \cdot B \cdot \left(\frac{B - B_{col}}{2} - d\right) = 48.989\ \mathbf{kip}$$

$$\phi V_{cOneWay} = 0.75 \cdot \left(2 \cdot 1 \cdot \sqrt{f'_c \cdot \mathbf{psi}} \cdot B \cdot d\right) = 323.985\ \mathbf{kip}$$

$Check =$ if $\phi V_{cOneWay} > V_{uOneWay}$
    return "Okay"
else
    return "Fail"

$Check = \text{"Okay"}$

Check Punching Shear:

$$V_{uPunching} = q_u \cdot \left(B^2 - \left(B_{col} + d\right)^2\right) = 168.833\ \mathbf{kip}$$

$\beta = 1$ $\alpha_s = 20$ $b_o = 2 \cdot \left(B_{col} + d\right) + 2 \cdot \left(B_{col} + d\right) = 11.624\ \mathbf{ft}$

$$\phi V_{cPunching} = 0.75 \cdot min\left(4, \left(2 + \frac{4}{\beta}\right), \left(2 + \frac{\alpha_s \cdot d}{b_o}\right)\right) \cdot 1 \cdot \sqrt{f'_c \cdot \mathbf{psi}} \cdot b_o \cdot d = 684.728\ \mathbf{kip}$$

$Check :=$ if $\phi V_{cPunching} > V_{uPunching}$
  return "Okay"
else
  return "Fail"

$Check =$ "Okay"

Check Bending Moment:

$$M_u = q_u \cdot B \cdot \left(\frac{B - B_{col}}{2}\right) \cdot \left(\frac{B - B_{col}}{4}\right) = 216.691\ \mathbf{kip} \cdot \mathbf{ft}$$ Moment Required

Steel Required:

$p_{min} = 0.0018$

$A_{sreq} = p_{min} \cdot B \cdot h = 7.128\ \mathbf{in}^2$

Provided Steel: Provide 8 #9 Rebar

$A_s = 8 \cdot 1\ \mathbf{in}^2 = 8\ \mathbf{in}^2$

Bar Spacing:

$BarSpace_{max} = min\left(3 \cdot h, 18\ \mathbf{in}\right) = 18\ \mathbf{in}$

$BarSpace = \frac{B}{4} = 33\ \mathbf{in}$ Space Rebar the max 18in

Flexural strength of single reinforced rectangular section:

$depth = h = 2.5\ \mathbf{ft}$ $A_s = 8\ \mathbf{in}^2$

$y_{sl} = c_c + \frac{D_b}{2} = 3.564\ \mathbf{in}$ $b = B = 11\ \mathbf{ft}$

$d_t = depth - y_{s1} = 0.671\ \mathbf{m}$ $\beta_1 = 0.85$

$$A_s = \frac{\beta_1 \cdot 0.85 \cdot f'_c \cdot b}{f_y} \cdot \frac{3 \cdot d_t}{8} = 63.03\ in^2$$

Depth of Concrete Compression Block:

$$a = \frac{A_s \cdot f_y}{0.85 \cdot f'_c \cdot b} = 8.426\ in$$

$$\phi M_n = 0.9 \cdot \left(A_s \cdot f_y \cdot \left(d - \frac{a}{2}\right)\right) = \left(6.143 \cdot 10^3\right)\ kip \cdot ft$$

$Check =$ if $\phi M_n > M_u$
  return "Okay"
else
  return "Fail"

$Check =$ "Okay"

$Area = 76177 \cdot ft^2 = 1.749\ acre$

To determine the rainfall intensity i, time of concentration must be found

The method to determine the time of concentration is the NRCS velocity method is used.

Table C3-S4- 1: Runoff coefficients for the Rational method

Hydrologic Soil Group	A			B			C			D		
Recurrence Interval	**5**	**10**	**100**	**5**	**10**	**100**	**5**	**10**	**100**	**5**	**10**	**100**
Land Use Or Surface Characteristics Business:												
A. Commercial Area	.75	.80	.95	.80	.85	.95	.80	.85	.95	.85	.90	.95
B. Neighborhood Area	.50	.55	.65	.55	.60	.70	.60	.65	.75	.65	.70	.80
Residential:												
A. Single Family	.25	.25	.30	.30	.35	.40	.40	.45	.50	.45	.50	.55
B. Multi-Unit (Detached)	.35	.40	.45	.40	.45	.50	.45	.50	.55	.50	.55	.65
C. Multi-Unit (Attached)	.45	.50	.55	.50	.55	.65	.55	.60	.70	.60	.65	.75
D. ½ Lot Or Larger	.20	.20	.25	.25	.25	.30	.35	.40	.45	.40	.45	.50
E. Apartments	.50	.55	.60	.55	.60	.70	.60	.65	.75	.65	.70	.80
Industrial												
A. Light Areas	.55	.60	.70	.60	.65	.75	.65	.70	.80	.70	.75	.90
B. Heavy Areas	.75	.80	.95	.80	.85	.95	.80	.85	.95	.80	.85	.95
Parks, Cemeteries Playgrounds	.10	.10	.15	.20	.20	.25	.30	.35	.40	.35	.40	.45
Schools	.30	.35	.40	.40	.45	.50	.45	.50	.55	.50	.55	.65
Railroad Yard Areas	.20	.20	.25	.30	.35	.40	.40	.45	.45	.45	.50	.55
Streets												
A. Paved	.85	.90	.95	.85	.90	.95	.85	.90	.95	.85	.90	.95
B. Gravel	.25	.25	.30	.35	.40	.45	.40	.45	.50	.40	.45	.50
Drives, Walks, & Roofs	.85	.90	.95	.85	.90	.95	.85	.90	.95	.85	.90	.95
Lawns												
A. 50%-75% Grass (Fair Condition)	.10	.10	.15	.20	.20	.25	.30	.35	.40	.30	.35	.40
B. 75% Or More Grass (Good Condition)	.05	.05	.10	.15	.15	.20	.25	.25	.30	.30	.35	.40
Undeveloped Surface[1] (By Slope)[2]												
A. Flat (0-1%)	0.04-0.09			0.07-0.12			0.11-0.16			0.15-0.20		
B. Average (2-6%)	0.09-0.14			0.12-0.17			0.16-0.21			0.20-0.25		
C. Steep	0.13-0.18			0.18-0.24			0.23-0.31			0.28-0.38		

$A_{pre.dev.impervious} = 41892\ ft^2 = 0.962\ acre$
$A_{pre.dev.pervious} = Area - A_{pre.dev.impervious} = 0.787\ acre$

Allamakee County, Iowa

249C—Zwingle silt loam, 1 to 9 percent slopes

Map Unit Setting

National map unit symbol: fhr2
Elevation: 600 to 1,020 feet
Mean annual precipitation: 31 to 39 inches
Mean annual air temperature: 41 to 50 degrees F
Frost-free period: 120 to 190 days
Farmland classification: Farmland of statewide importance

Map Unit Composition

Zwingle and similar soils: 95 percent
Minor components: 5 percent
Estimates are based on observations, descriptions, and transects of the mapunit.

Description of Zwingle

Setting

Landform: Terraces
Landform position (two-dimensional): Toeslope
Landform position (three-dimensional): Tread
Down-slope shape: Linear
Across-slope shape: Linear
Parent material: Clayey lacustrine deposits over loamy and sandy alluvium

Typical profile

H1 - 0 to 11 inches: silt loam
H2 - 11 to 41 inches: clay
H3 - 41 to 60 inches: stratified loamy sand to loam

Properties and qualities

Slope: 1 to 9 percent
Depth to restrictive feature: More than 80 inches
Drainage class: Poorly drained
Runoff class: Very high
Capacity of the most limiting layer to transmit water (Ksat): Very low to moderately low (0.00 to 0.01 in/hr)
Depth to water table: About 0 to 12 inches
Frequency of flooding: None
Frequency of ponding: None
Available water supply, 0 to 60 inches: Moderate (about 8.2 inches)

Interpretive groups

Land capability classification (irrigated): None specified
Land capability classification (nonirrigated): 3e
Hydrologic Soil Group: D
Ecological site: F105XY005WI - Wet Loamy-Clayey Lowland

The recurrence interval selected for calculation is a 10 year storm event

$C_{open.space} = .35$ 10 year recurrence interval, hydrologic soil group D
$C_{impervious} = .90$ 10 year recurrence interval, hydrologic soil group D

$$C_{pre.dev} = \frac{(A_{pre.dev.impervious} \cdot C_{impervious}) + (A_{pre.dev.pervious} \cdot C_{open.space})}{Area} = 0.652$$

$A_{post.dev.impervious} = 45782\ ft^2 = 1.051\ acre$ $\quad A_{post.dev.pervious} = Area - A_{post.dev.impervious}$

$$C_{post.dev} = \frac{(A_{post.dev.impervious} \cdot C_{impervious}) + (A_{post.dev.pervious} \cdot C_{open.space})}{Area} = 0.681$$

Shallow concentrated flow (along gutters)

$L = 501 \cdot ft$ Length of longest flow path

$S = .0245 \cdot \frac{ft}{ft}$

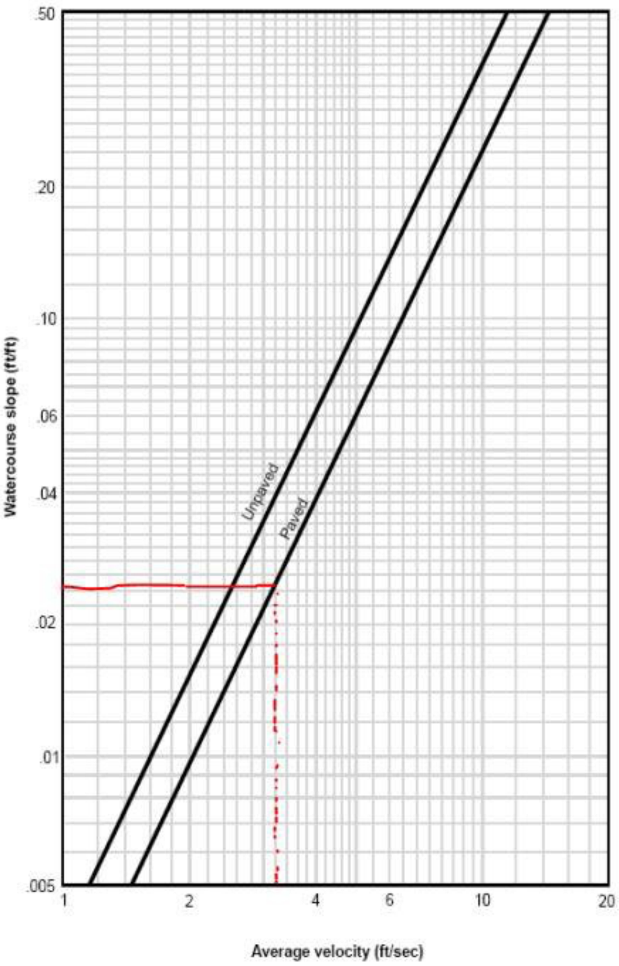


Figure C3-S3-1 indicates that the average velovity is 3.2 ft/s

$V = 3.2 \ \frac{ft}{s}$

$T_t = \frac{L}{V} = 0.043 \ hr$ $\quad T_c = T_t = 0.043 \ hr$ $\quad T_c = 2.609 \ min$ $\quad T_c = 156.563 \ s$

Watershed time of concentration remains the same between post development and pre development

01 - Northwest
02 - North Central
03 - Northeast
04 - West Central
05 – Central
06 - East Central
07 - Southwest
08 - South Central
09 - Southeast

Figure C3-S2- 1: Climatic Sectional Codes for Iowa*

Table C3-S2- 4: Sectional mean rainfall intensity for storm periods of 5 minutes to 6 hours and return period (recurrence interval) of 1 to 500 years in Iowa (see Figure C3-S2- 1, Iowa Map)
Rainfall intensity (inches/hour) for given duration and return period

		Return Period							
	Duration	1-yr	2-yr	5-yr	10-yr	25-yr	50-yr	100-yr	500-yr
Section 1 – Northwest Iowa	6-hr	0.32	0.38	0.48	0.57	0.72	0.84	0.96	1.30
	3-hr	0.56	0.66	0.83	0.99	1.22	1.40	1.60	2.11
	2-hr	0.76	0.90	1.13	1.34	1.63	1.87	2.11	2.72
	1-hr	1.25	1.48	1.86	2.18	2.64	3.01	3.38	4.30
	30-min	1.94	2.30	2.89	3.38	4.05	4.56	5.08	6.30
	15-min	2.78	3.29	4.12	4.82	5.77	6.50	7.24	8.98
	10-min	3.43	4.06	5.07	5.92	7.09	8.00	8.91	11.0
	5-min	4.69	5.53	6.92	8.11	9.69	10.9	12.1	15.0
Section 2 – North Central Iowa	6-hr	0.34	0.40	0.51	0.61	0.76	0.89	1.03	1.40
	3-hr	0.58	0.69	0.88	1.05	1.30	1.52	1.74	2.34
	2-hr	0.79	0.93	1.18	1.41	1.74	2.02	2.31	3.07
	1-hr	1.28	1.52	1.92	2.27	2.80	3.23	3.69	4.85
	30-min	1.98	2.33	2.94	3.47	4.23	4.85	5.50	7.13
	15-min	2.79	3.28	4.12	4.87	5.92	6.79	7.68	9.93
	10-min	3.44	4.04	5.07	5.98	7.29	8.35	9.45	12.2
	5-min	4.69	5.53	6.93	8.18	9.96	11.4	12.9	16.6
Section 3 – Northeast Iowa	6-hr	0.33	0.39	0.49	0.59	0.73	0.86	0.99	1.34
	3-hr	0.57	0.67	0.85	1.01	1.24	1.44	1.64	2.18
	2-hr	0.76	0.90	1.14	1.35	1.65	1.89	2.15	2.79
	1-hr	1.25	1.47	1.85	2.17	2.64	3.01	3.39	4.34
	30-min	1.93	2.28	2.83	3.31	3.96	4.47	4.98	6.20
	15-min	2.77	3.24	4.02	4.68	5.60	6.31	7.03	8.77
	10-min	3.40	4.00	4.94	5.76	6.89	7.75	8.64	10.7
	5-min	4.66	5.47	6.76	7.86	9.42	10.5	11.8	14.7

$i_{5min} = 7.86 \frac{in}{hr}$ $\quad i_{10min} = 4.94 \frac{in}{hr}$

$$i_{2.61min} = i_{10min} + (2.61\ min - 10\ min) \cdot \frac{\left(i_{5min} - i_{10min}\right)}{5\ min - 10\ min} = 9.256 \frac{in}{hr}$$

$$i_{pre.dev} = i_{2.61min} = 9.256 \frac{in}{hr}$$

$$i_{post.dev} = i_{2.61min} = 9.256 \frac{in}{hr}$$

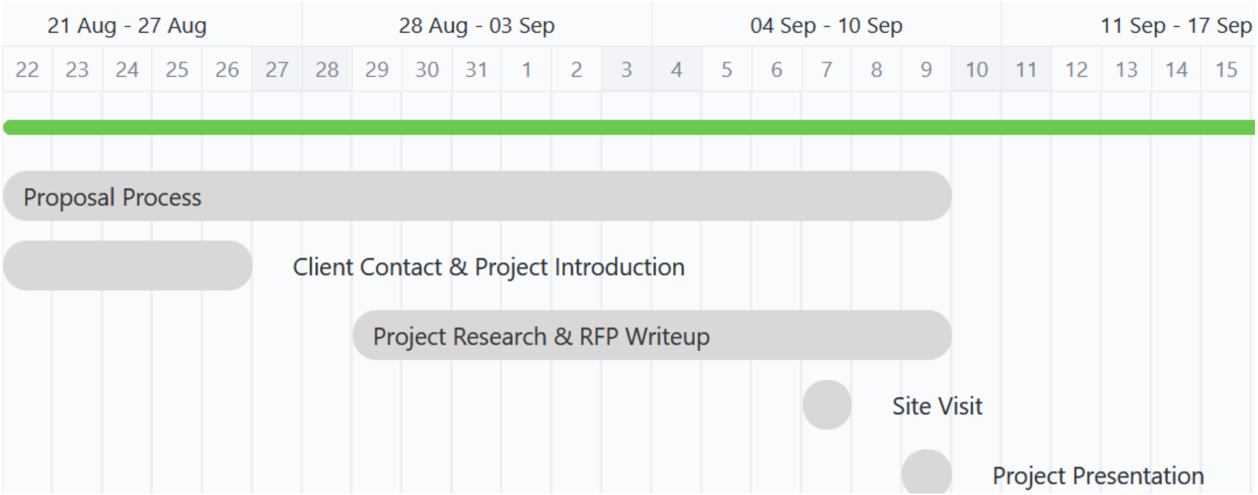
$$Q10yr_{pre.dev} = \left(C_{pre.dev} \cdot \frac{i_{pre.dev}}{\frac{in}{hr}} \cdot \frac{Area}{acre}\right) \cdot \frac{ft^3}{s} = 10.561 \frac{ft^3}{s}$$

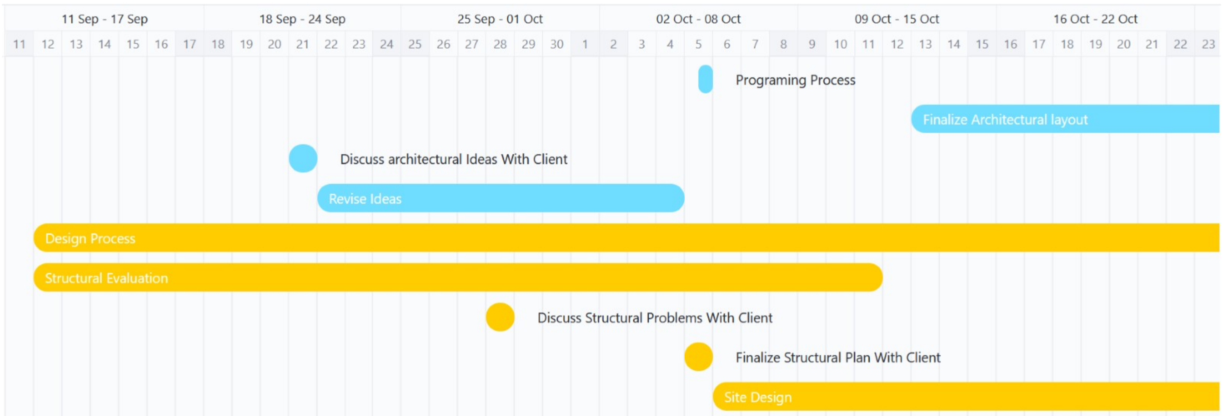
$$Q10yr_{post.dev} = \left(C_{post.dev} \cdot \frac{i_{post.dev}}{\frac{in}{hr}} \cdot \frac{Area}{acre}\right) \cdot \frac{ft^3}{s} = 11.016 \frac{ft^3}{s}$$

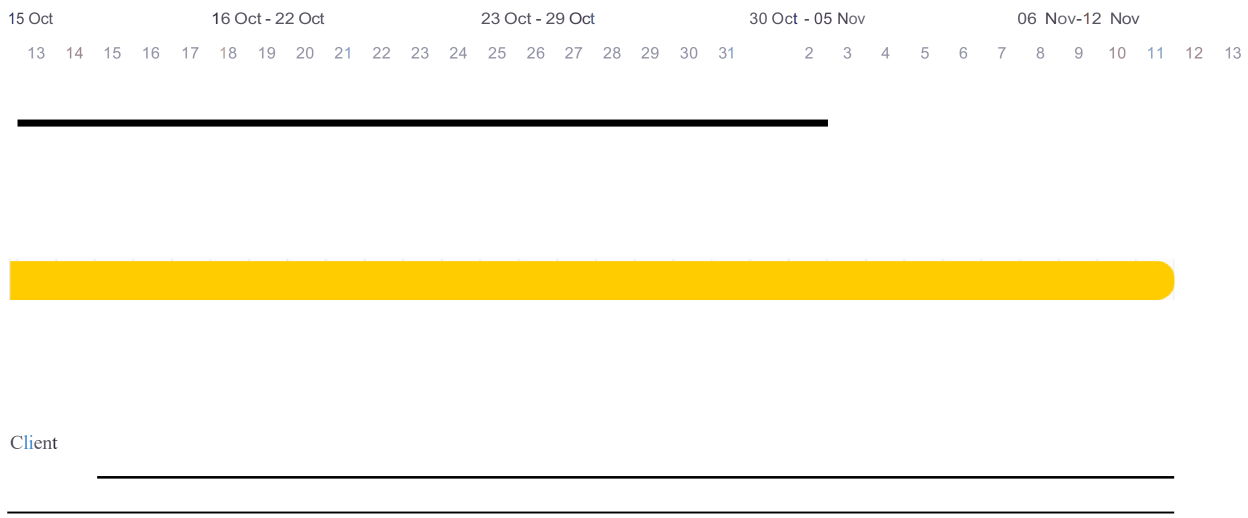
$$PercentChange = \frac{Q10yr_{post.dev} - Q10yr_{pre.dev}}{Q10yr_{pre.dev}} = 4.305\%$$

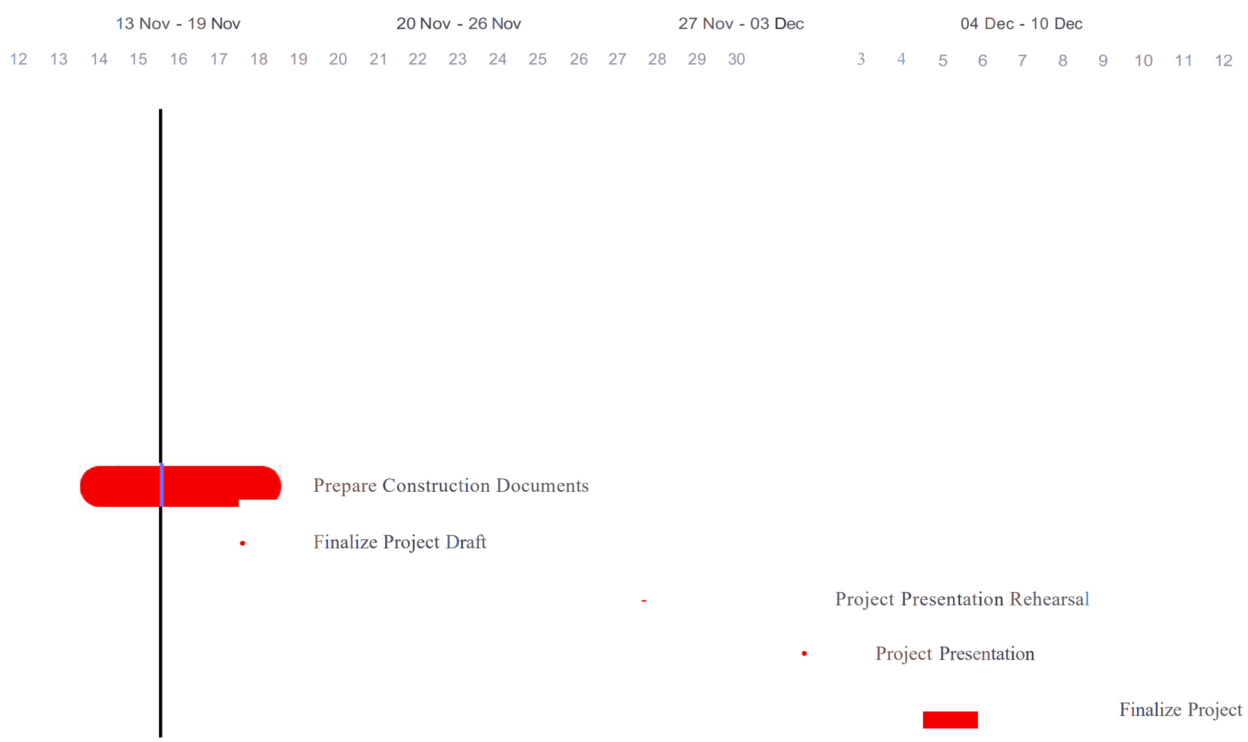
Development does not have significant effect on the existing storm structure, no additional storm structures are needed during site development.

Appendix J: Gantt Chart






15 Oct
16 Oct - 22 Oct
23 Oct - 29 Oct
30 Oct - 05 Nov
06 Nov-12 Nov
13 14 15 16 17 18 19 20 21 22 23 24 25 26 27 28 29 30 31 2 3 4 5 6 7 8 9 10 11 12 13
Client


13 Nov - 19 Nov
20 Nov - 26 Nov
27 Nov - 03 Dec
04 Dec - 10 Dec
12 13 14 15 16 17 18 19 20 21 22 23 24 25 26 27 28 29 30 3 4 5 6 7 8 9 10 11 12
Prepare Construction Documents
Finalize Project Draft
Project Presentation Rehearsal
Project Presentation
Finalize Project