Clinton, Iowa, YMCA Building Renovation and New Construction Design

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The University of Iowa
IOWA INITIATIVE
FOR SUSTAINABLE COMMUNITIES

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Section I: Executive Summary

The following proposal outlines the rehabilitation and redesign of a multi-use facility at the corner of 5th Avenue S and S 3rd Street in Clinton, Iowa. The site has been designed to provide luxury apartments and a new restaurant in Clinton's downtown district. This proposal has been prepared by a team of civil engineering students from the University of Iowa. The four-member team consists of Drew Hambly, Steven Susmarski, Trevor Thornburgh, and David Wu. The team members all have in-depth experience in structural and civil engineering design projects during their time at the University of Iowa. Additionally, each member has had experience as engineering interns training under professional civil, structural and construction engineers.

The existing facility was a former YMCA complex that had been abandoned for a considerable amount of time. The facility consisted of two main buildings that were connected. The site was broken down into two facilities as shown in the images below.

Figure 1.1: Existing site layout

Figure 1.2: Designed site layout

The first building, which in has been referred to as "Building A," was built in 1905 and is considered an unofficial historic landmark by the City of Clinton. This three-story building with a basement sits on the northwest corner of 5th Ave. S and S 3rd St. and was formerly temporarily used as low-income housing and commercial space. Due to this building being an unofficial historical landmark, demolition of the structure is not an option. The facility had uninhabitable conditions due to extensive water damage and exposed utilities. The existing elevator was non-functionable and needed to be replaced.

There were no existing structural/architectural drawings for the existing facilities. Dimensions for Building A were determined by hand measurement. The structural layout of the building was decided based on on-site inspections and engineering judgement. Both shall be confirmed by the contractor on site.

Building A has been designed for interior renovation. The renovation consists of 16 luxury apartment units with in-unit laundry. The apartment units vary from one to two bedrooms with an approximate size of 1,000 SF on average. This building received special zoning to allow for residential apartments on all four floors. The basement includes two units, a fitness center, a recreation lounge, and mechanical rooms for the elevator and general equipment. The first floor has been renovated to include three apartment units with one ADA accessible unit. A mail station has also been added on the first floor. The second and third floors are identical in their floor plan layout containing five units each. The rooftop has been designed to include a synthetic greenspace and areas for grilling. The existing east stairwell was redesigned to reach the roof patio and updated to present standards; a second stairwell was designed into the north-west corner of the building that also extends to the rooftop. A new elevator shaft and footing was designed to house a Schumacker elevator that is to be installed.

The second building, which has been labeled as "Building B," was connected to the western wall of Building A and had been used as a two-story recreational facility. Building B was constructed several years after Building A and was not considered a historical landmark. Building B has been demolished per client request.

The proposed building has taken the place of Building B, and has been labelled as "Building C." This building was designed as a restaurant with a size of 5,685 SF. It is on the same site as the other building but is now separated by a parking lot. The restaurant includes a commercial kitchen, a long bar, a dining area, and an outdoor patio for dining. The restaurant has a total of 236 seats based on the recommended seating arrangement. A 1,770 SF basement has been designed on the northern side of the building to provide storage. The new building has a stone cladding finish, and a dark-metallic parapet to match the surrounding architecture.

The site has been remodeled to provide tenant and customer parking on the east and west sides of the restaurant to go along with the existing street parking. Grading has been done to provide better drainage of the site by directing stormwater runoff to 5th Ave S and the back alley. Lastly, some of the existing sidewalks at the south entrance of the building have been removed to provide more green space for tenants.

The total project cost for design, administration, and construction of the project have been estimated as $13,339,000. The cost estimate has been broken into engineering costs, demolition, apartment renovations, restaurant construction, and general sitework. A contingency of 20% was used when performing the cost estimate due to many of the existing conditions being unknown. An engineering design and administration rate of 15% was used. Consumer Price Index rates for construction were used to adjust the cost to the present-day value. Multiple material alternatives were provided for the parking lot, but we recommend the HMA overlay since it is most cost effective. Fire suppression, HVAC, electrical work, and utilities were included as a lump sum and shall be designed separately by a licensed engineer in that field.

We have designed a modern restaurant/bar to be paired with luxury apartments that can serve as an entertainment hub in Clinton. Surveys from the US Census Bureau show a declining population in the last decade. With the average demographic of 41 years old, we feel that this design will target this demographic well, and it can serve as a forefront of liveliness and modernity while still preserving important city history.

Section II: Organization Qualifications and Experience

The project team consists of civil and environmental engineering students from the University of Iowa in the senior capstone design course. The team members assigned to the City of Clinton's YMCA Building Redesign were Drew Hambly, Steven Susmarski, Trevor Thornburgh, and David Wu. All members are in their last semester of study as civil engineering students with a focus area in structures, mechanics, and materials.

Drew Hambly served as the technology service manager and managed all documents related to the project. Drew worked for the City of Cedar Rapids as a civil engineering intern within their construction department. He assisted in project inspection by conducting topographic surveying, performed concrete testing for new roads in accordance with Iowa Department of Transportation standards, and performed daily site visits to ensure contractors were meeting Iowa Statewide Urban Design and Specifications during construction. Drew led the interior demolition planning, existing building structural layout, structural design for the elevator shaft and stairwell.

Steven Susmarski served as the report production manager for written deliverables. Steven worked at Alfred Benesch and Company in Chicago, Illinois as a civil engineering intern. He performed site visits to update topography files, proposed quality plans for a six-mile-long rehabilitation of U.S. Route 41, aided in crosswalk design and quantity take-offs for Americans with Disabilities Act improvements, and assisted in developing land use and drainage plans for highway reconstruction. Steven led the design of the restaurant superstructure design, foundation design, and finalized all project deliverables.

Trevor Thornburgh served as the project manager and was lead contact for the project. Trevor worked at Shive-Hattery Architecture & Engineering in Iowa City, as a civil/structural design intern, and materials testing technician. He assisted the Government/Higher Education team with civil design projects, Structural team with industrial framing design and modeling, conducted laboratory soil proctor and aggregate gradation tests on field specimens in accordance with specifications, and performed on-site inspections to ensure compliance with construction documents. Trevor coordinated all project tasks between team members and led site design and structural assessment.

David Wu served as the report production manager for graphic design deliverables. David has worked at Knutson Construction as a project engineer and pre-construction estimator intern. He performed quantity takeoffs for new buildings, developed project schedules to maintain timely task completion, communicated with the architect and the contractor over construction design challenges, and coordinated weekly meetings with architects, structural engineers, and contractors. David led the architectural design for the interior and exterior of all buildings.

All team members have experience in design software such as Autodesk Civil3D, Revit, Robot, Sketchup, and Lumion. Models and project deliverables were provided using the above software. All members have completed or are currently enrolled in relevant courses related to this project. These courses include Structural Systems for Buildings, Foundations of Structures, Design of Concrete Structures, Design of Steel Structures, Civil Engineering Tools, and Construction Management.

Section III: Design Services

Project Scope

The project is in the downtown district of Clinton, IA. The goals of the project were to provide additional housing, a congregation space for social events, and an open recreation space within the downtown district. The exterior and interior of building A was designed to be renovated into luxury apartments and a basement fitness space. It was determined after the initial proposal phase that building A will be re-zoned to allow for 100% of the space to be apartments. To complete this renovation, several tasks were completed including the creation of existing condition plans, demolition design of all nonstructural load-bearing systems, exterior façade and window renovation design, elevator shaft design, stair design, and interior architectural design. Following this, Building B was designed to be demolished while protecting the integrity of Building A. To complete this, full demolition plans were produced. Building C was designed to partially fill the space occupied by Building B. Building C was designed to house a first-floor restaurant space with an outdoor patio seating area and a basement storage area. To complete this design, a series of tasks were completed including substructure design, superstructure design, and architectural design. Finally, an off-street parking lot, sidewalk system, and accessible entrances and exits were provided for Buildings A and C to be used by consumers in the restaurant and tenants of the luxury apartments. To complete this, the team produced a site/parking layout design, site grading design, and stormwater drainage scheme.

Work Plan

To complete the project, the team followed a work plan to ensure all project deadlines were met. Figure 3.1 shows a Gantt chart for the design phase of this project. This chart includes start dates and duration of the individual project tasks. A task manager took the responsibility of leading each individual task as specified in the chart. Each group member was responsible for contributing time and effort to most of the design tasks.

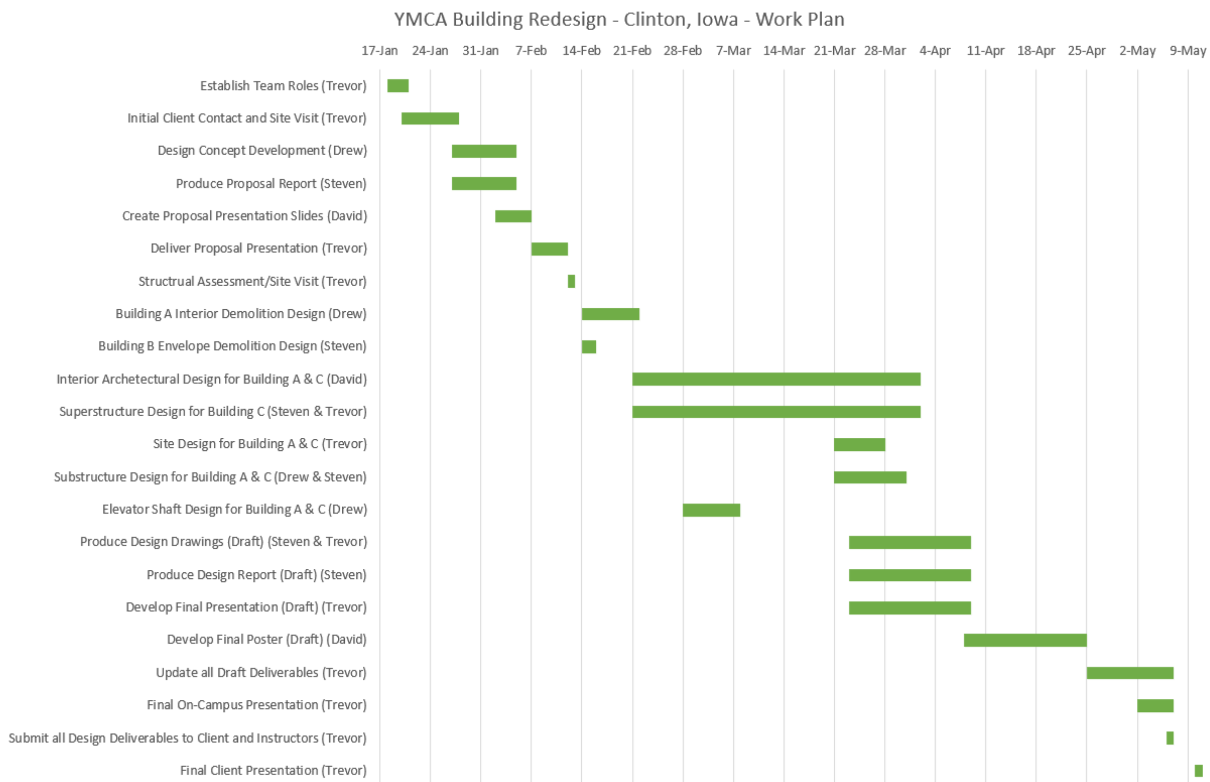

Figure 3.1: Gantt chart showing proposed work plan.

Section IV: Constraints, Challenges, and Impacts

Constraints

The client did not state a set monetary constraint when discussing this project; however, this project needed to appeal to developers and be marketable to attract tenants and customers. With labor being the most expensive component of construction projects, ease-of-construction was considered when designing this project. Structural framing members were repetitive and only ranged in a few sizes to allow for easier installation. Additionally, the demolition of Building B was generalized to allow for the contractor to demolish the building efficiently while maintaining structural stability of Building A. The contractor is also able to salvage any construction materials they find during demolition, making the project more enticing.

Building A is unofficially recognized as a building on the city historic registry. As a result, the precautions for any exterior façade adjustment must adhere to city codes and City of Clinton's Downtown Master Plan, maintaining cohesive aesthetics with adjacent buildings. Specifically for the renovation of Building A, the exterior must be rehabilitated and maintained as closely as possible to the present aesthetic. The ADA ramp was set on the west side of the building to maintain the historic look with the large steps at the front entrance.

The client expressed the importance of Building C "fitting in" with the downtown Clinton area. Like Building A, it must follow the City of Clinton's Downtown Master Plan by maintaining cohesive aesthetics with adjacent buildings. This created a constraint on the types of materials to be used for the construction of the building. Stone cladding with varying shades of grey was determined to be the best viable option for the exterior finish. A parapet was also included to imitate geometric features of Building A.

Challenges

The greatest challenge presented in this project was the lack of existing plans and specifications. These documents would have been critical to accurately rehabilitate the historic building and strategically demolish the attached building. To overcome this, all dimensions of Building A were measured by hand. The structural layout of the building was also determined during on-site inspections with the team's engineering judgment. Both dimensions and existing structural layout must be verified by the contractor on site for the renovation of Building A.

The poor structural integrity of Building B provided the challenge of accurately determining the layout of the building and preparing demolition plans. Ceiling systems have collapsed in certain areas due to water damage which made the mapping of this building difficult. As previously stated, drawings and specifications were unavailable for reference. This was resolved by lumping the demolition of Building B into one phase. This provides simplicity for the contractor; with the specific requirement of maintaining the structural integrity of Building A. Structural analysis must be performed throughout the demolition to prevent unwarranted structural failure.

The existing parking infrastructure was in critical condition and needed to be redesigned. The site itself was flat with no existing structure for stormwater runoff connection. Topographic data on the existing grading was gathered using Autodesk Infraworks. Grading was designed to route the runoff to the north and south ends of the site. This was done to prevent pooling on the lot without relying on storm water structure.

Societal Impact

Referencing Clinton's master plans, the community would like to see modern day land uses. This ensures any new infrastructure is designed with children in mind regarding both safety and appeal. The public would like to see an event center or similar designated large public gathering space, food and beverage options, local brewery, housing, and rooftop development to further visual connection to the Mississippi river. Along with new infrastructure, improvements of existing aesthetics to create a cohesive "theme" with connection to the city's history is emphasized.

A community survey to gauge community sentiment, perceptions, and habits highlights a majority of negative/neutral attitude towards the present downtown. The survey respondents' demographic fell within the 25-64 age group with a majority wanting a greater variety of stores/establishments and more places to eat.

According to the United States Census Bureau, the City of Clinton has been on a steady decline in population between 2010 with a recorded 26,885 to 24,469 in 2020. Clinton's priority is to maintain the local community while introducing assets to further help the community grow. The addition of more food and beverages, a greater variety of establishments, residential units and community space is necessary to maintain the growth of the community.

The renovation and repurpose of the old YMCA building is important to maintain its historical presence in the downtown area while also creating space for new memories in the community. The addition of a restaurant encompasses the same purpose but instead creates a new chapter and brings forth ideas of what the city can attain in the future.

Section V: Alternative Solutions That Were Considered

Multiple schematic design options were considered for the client to choose from. The client had expressed that they were set on having Building A renovated into apartments, as well as the demolition of Building B. Therefore, all alternatives provided include this criterion.

An alternative that had been considered was using the first floor of Building A as commercial space. A constraint tied to zoning usage stated that 75% of the square footage of the front of the building for the first floor must be designated for retail space per City of Clinton, Iowa Code of Ordinances (159.027 SP SPECIAL PURPOSE COMMERCIAL AND HISTORICAL OVERLAY DISTRICTS). This would have provided space for another business in their downtown district. However, the client wanted to maximize the number of apartments in the building and elected to use the first floor as residential space instead of commercial. A special zoning change was granted to make this possible.

Figure 5.1 Example view of first floor commercial space considered for Building A

A community center had been considered when deciding the purpose of Building C. When discussing uses for the new building, the client had mentioned it as a possibility. A community center would have been a good space for locals to hold parties and gatherings. However, when researching the surrounding area of the site, there are already multiple community centers in the area. It was agreed that a restaurant would be the best use of the space to bring new business.

Having a shared greenspace for the apartment building and community center was also considered. The client expressed interest in having an outdoor recreation area for people and pets to enjoy. When Building C was determined to be a restaurant, the shared greenspace was removed. The outdoor greenspace would have restricted the number of on-site parking spaces for tenants and customers. To meet the client's request for an outdoor recreation space, some existing sidewalk pavement near the south entrance of Building A was removed to allow for green space and benches. The rooftop of Building A was also designed to be a rooftop patio including a turf area and grilling station for tenant use.

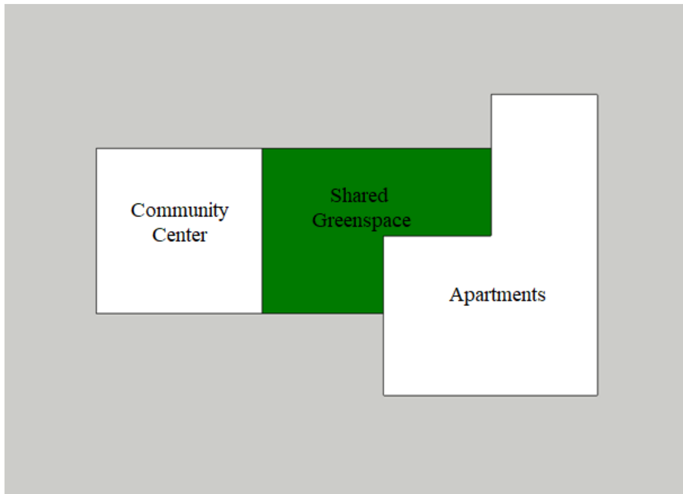

Figure 5.2 Sketch of alternative site layout

Section VI: Final Design Details

Building Information and Elevations

Building A will remain a three-story complex with a basement. Each floor consists of approximately 6,430 SF in area, resulting in a total usable building area of approximately 25,700 SF. The floor to ceiling height of the basement, first, second and third floors are approximately: 12 feet, 14 feet, 13 feet and 11 feet, respectively, resulting in a rooftop elevation of 49 feet above the ground. The redesign of Building A is a luxury apartment complex. For elevations of Building A, see Design Sheets A-8 through A-11.

The proposed Building C features a one-story space of approximately 6,050 SF in area. There is an additional basement space, accessible via stairs, of approximately 1,770 SF of area. The basement's intended use is for a mechanical access room and storage area. The floor to ceiling height of the main floor is 18 feet. Roof decking and insulation along with a roof parapet of approximately four feet brings the rooftop elevation to approximately 23 feet. The intended design of the building is a medium to large restaurant/bar with indoor and outdoor seating. For building dimensions and elevations, see Design Sheets A-6, A-14, and A-15, respectively.

Building A: Renovation Design and Layout

The renovation of the exterior of Building A will feature cosmetic improvements to the façade. The façade will be cleaned and polished to look new, while the historic integrity of the façade will remain intact. Some structural improvements will be made to patch cracking in the existing masonry and prevent cracking in the future. For a visual of the refurbished façade, see Figure C.5 in *Appendix C* below.

The renovation of Building A will include 16 luxury apartments across the three floors and basement. The apartments include in-unit laundry and central heating/cooling. In the basement of Building A, a fitness center and sauna are included for tenant use. The basement also includes a lounge/recreational area where tenants can congregate and relax. The facility was designed to include two stairways and a newly designed elevator and shaft. Both stairways and the elevator reach to the top of the roof, where a rooftop patio with greenspace is located. The greenspace includes a patch of faux grass, such as AstroTurf, and the rooftop patio includes space for lounge chairs, picnic tables and grills. An additional stairway has been designed in the northwest corner of the building to allow traffic through the north end of the building and provide a second means of egress for the rooftop patio. For the architectural layouts of Building A, see Design Sheets A-1 through A-5.

Building A: Structural Elements

The structural system of Building A will mostly stay in existing condition. During site inspections, the structural layout was determined to be over-designed, which was common in 1905. The existing elevator is nonfunctional. The elevator will be replaced with a hydraulic, in-ground, center parting elevator from Schumacher Elevators Company Inc with a capacity of 3,500 lbs. The size of the elevator was chosen due to it being able to fit a stretcher/gurney. The elevator shaft and foundation were redesigned to support the Schumacher elevator which is larger than the existing elevator. An A992 W6x25 hoist beam for the elevator was designed following American Institute of Steel Construction (AISC) Manual 15th Edition. An A36 steel bearing plate of 8"x8"x1/4" is to be used to transfer loads from the hoist beam to the shaft wall. The shaft wall is to be constructed with 8"x8"x16" full square concrete masonry units with a compressive strength of 2 ksi. Vertical reinforcement was provided in the CMU walls as #4 bars. Compressive strength of the CMU wall was checked following Building Code Requirements for Structural Concrete American Concrete Institute, June 2019 (ACI 318-19) – see *Appendix D* for the plan/section drawings of the elevator provided by Schumacher Elevators Company Inc. The elevator is to travel from the basement to the rooftop, giving the shaft wall a total height of 62'-2". The footing for the elevator is 13'x13'x16" of normal weight concrete and contains #5 rebar at 10" spacing for reinforcement. To reinforce the interface between the shaft wall and footing, #3 dowel bars were selected - see *Appendix F* for supporting elevator shaft and footing design calculations.

The new stairway in the northwest corner of the building travels from the basement to the rooftop for a total height of 58'-2". The stairs have a consistent run of 11" for each step and range from 7.25-7.75" (varies per story) following ADA standards. The stairs are designed as A992 MC12x10.6 double stringers with A36 ¾" steel plates as stair treads. The landings were designed as structural systems made up of MC12x10.6 and MC12x14.3 stringers with A36 ¾" steel plates as treads. The elements were checked for deflection, yielding, and lateral torsional buckling when applicable in accordance with the AISC Manual. To connect the and support the stairwells to the landings, 6"x4"x3/8" angles with Dewalt Power-stud SD1's was used – see *Appendix E* for supporting design calculations.

The new stairwell shaft wall is to be constructed with 8"x8"x16" full square concrete masonry units with a compressive strength of 2 ksi. Vertical reinforcement was provided in the CMU walls as #4 bars. Compressive strength of the CMU wall was checked following Building Code Requirements for Structural Concrete American Concrete Institute (ACI 318-19). The footing for the stairwell is 16'x23'x16" of normal weight concrete and contains #5 rebar at 7" spacing for reinforcement. To reinforce the interface between the shaft wall and footing, #3 dowel bars were selected - see *Appendix E* for supporting elevator shaft and footing design calculations.

Structural analysis was performed on the existing roof framing layout to ensure the system could support the increased rooftop patio live load. Roof joists were assumed to be 3x16 Grade 1 Douglas Fir joists spaced at 16”. The joists were checked for bending, bearing, shear, and deflection following the National Design Specification for Wood Construction manual. Simpson Strong Tie joist hangars were used to connect the roof joists to the designed elevator and stairwell shaft walls – see *Appendix W* for supporting design calculations

Building C: Design and Layout

The goal of the exterior design of the proposed Building C was to design a state-of-the-art facility without looking out of place within the historic surrounding area of Clinton. The façade is grey-tone stone cladding featuring mostly light color schemes with dark undertones. The toe of the building and the parapet both use dark colors to offset the stone cladding. The building features many large windows to both increase the aesthetic look from the outside as well as to allow for plenty of natural light to pass through the inside. Above the main entrance doors, an aluminum awning is suspended for aesthetic purposes. The outdoor patio pavement is a cross hatch pattern of masonry brick pavers. For a visual depiction of the proposed building exterior, see Figure C.1 in *Appendix C* below.

The interior layout consists of a centrally located bar with approximately 24 seats/stools. The perimeter of the bar is filled with booths for dining. The total proposed number of seats available from the booths is 108. Across the rest of the facility, table seating is available. The proposed number of tables is nine, with the ability to seat four guests per table, consisting of 36 total seats. The total number of guests that can be seated at one time within the indoor facility (bar seating included) is 168 guests.

On the southeast side of the building, an outdoor patio area has been designed. The patio has a brick paver floor design to provide a natural look. The patio allows space for up to 17 tables, which can accommodate 68 guests at maximum capacity. The patio was also designed to feature up to two fireplaces to keep guests warm during cold nights as well as for an enhanced aesthetic experience. For the proposed table and booth sizes, a typical size was chosen for design – see *Appendix C* for renderings of restaurant layout. The total number of indoor and outdoor seating allows for a maximum accommodation of 236 guests. For a seating floor plan, see Design Sheet A-6.

The selection of long spanning beams and girders has allowed the usable space of the facility to be maximized. All structural columns and walls have been placed as close to the perimeter of the building as possible to allow guests and workers to move freely throughout the building. A kitchen space of approximately 1,270 square feet has been placed in the northwest corner of the building near the alleyway to allow for easy deliveries and dumpster access. With the previously mentioned eighteen-foot ceiling height, HVAC ductwork will be able to be suspended from the roof and span throughout the building, keeping the users of the building at a comfortable temperature as well keeping the air clean and pure. For a visual of the interior of the proposed building, see Figure C.2 through Figure C.4 in *Appendix C* below.

Building C: Structural Elements

The structural system of the proposed building was designed for gravity and lateral loading. Using ASCE 7-16, appropriate LRFD load combinations were used to find the total loading acting on the roof and first floor. Based on the factored loading, the structural system was designed. Selection of loading and factored load calculations can be found in *Appendices G and L* below. The gravity systems consist of W-Shape columns and girders that were selected from Table 1-1 of the American Institute of Steel Construction (AISC) Manual 15th Edition. The main roof support is open web joists spaced at five feet on center, selected using the Nucor Vulcraft Steel Joist Catalog. The roof system is comprised of a 1-1/2” steel deck, selected from the Nucor Vulcraft Steel Deck Catalog, with ½” plywood sheathing, six inches of rigid insulation and a roll-on waterproofing. On the first floor, above the basement is a precast 14” hollow core slab, selected from the PCI Hollow Core Slab Catalog. The slab transfers load from the first floor to the basement walls without the need for structural framing beneath the slab. A detailed cross section of the precast hollow core slab can be found in Design Sheet D-2. For the area of the first floor that is not directly above the basement, a standard, 5” PCC cast in place slab on grade with a 6” x 6” welded steel mesh was selected. The slab transfers load directly to the soil below it. The basement is enclosed by 8” PCC cast in place bearing walls with several square 18” pilasters embedded in the walls which receive load from the W-shape columns directly above them. Between the steel columns and concrete pilasters, bearing plates have been designed to prevent the concrete from cracking. Bearing plate details can be found in Design Sheets D-7 and D-8. The basement slab is also a 5” PCC cast in place slab with a 6” x 6” welded steel mesh was selected.

To size the open web joists, LRFD load combinations were used along with the equations provided within the Vulcraft Steel Joist Catalog to calculate the deflection, in inches, of each joist. The criteria for allowable deflection were determined by dividing the total span of the joist by 360, in accordance with the Vulcraft Steel Joist Catalog. 26K8 (twenty-six-inch depth) and 18K3 (eighteen-inch depth) joist were selected.

The lateral system was designed to resist the considerable amount of loading on the building due to average wind speeds for Clinton. The lateral system is composed of a two-fold system, a roof diaphragm and fixed girder-column moment frame connections throughout the structure. The lateral load is first accepted by the roof diaphragm, and then resisted by the moment frame connections. To design these moment fame connections, initial member sizes were selected and placed into an Autodesk Robot model. The loading within the Robot model reflected the serviceability load case included within ASCE 7-16. An iterative process was conducted to determine the minimum member sizes in each moment frame system while meeting the H/500 story drift requirement found in ASCE 7-16. Detailed lateral analysis calculations can be found in *Appendix O*.

The girders and columns that were part of the moment frame systems were first sized for lateral loading, since the magnitude of the lateral loading was larger than the gravity loading. After the lateral analysis, a gravity analysis was performed. For columns, Chapter E of the AISC Manual was used and for girders, Chapter F and Chapter G were used. The elements were sized to prevent large deflection, yielding, lateral torsional buckling, flange buckling, web buckling due to shear, flexural torsional buckling, and torsional buckling, in accordance with Chapters E and F of the AISC Manual. The gravity analysis calculations are shown in *Appendix P* below. After the members had been preliminarily sized for both gravity and lateral loads, a final analysis was run to ensure that the moment frames had enough strength to resist the combined loading of flexure, torsion, and axial compression simultaneously. Chapter H of the AISC manual was used for this analysis, and the results were used to determine that three column sizes and three girder sizes would be used. The sizes are W14x48, W14x132 and W14x159 for columns and W18x60, W30x132 and W33x141 for girders. *Appendix Q* below shows the details of the combined loading analysis. A framing plan can be found in Design Sheet S-4.

Once final member sizes were selected, moment connections for the moment frames and shear connections were designed following the simplified procedure in the AISC design manual. It was found that all connections will utilize ¾" diameter bolts and 3/8" thick plates. Details of the connections can be found in Design Sheet D-3 below, and detailed calculations for the design of the connections are shown in *Appendix R.*

The roof deck was selected based on LRFD factored loading and the deflection requirement of total span length divided by 240, in accordance with the Nucor Vulcraft Steel Deck Catalog. The selected deck is a 1.5B Grade 50 19 Gage eight-foot double span. For a cross section view of the roof deck, see Design Sheet D-2. The design calculations for the deck can be found in *Appendix S* below.

The foundations of Building C were designed according to the Foundation Design Principles and Practices textbook by Donald Coduto et al. and Foundation Analysis and Design textbook by Joseph Bowles. The foundations were designed as a mixture of continuous footings for the basement bearing walls and frost walls and square footings for the pilasters. Designed according to the ASD method, the unfactored loads from the roof and first floor were used to find the design loads for the pilasters. For ASD load combinations and column loads, see *Appendices L and M* below. Then, the loading on the basement walls was determined to design the continuous footings. The square and continuous footings were then designed to prevent bearing failure and immediate settlement failure. After this, a final check of differential settlement was performed to ensure that the foundations were not settling at different rates. After this geotechnical limit state analysis had been conducted, a structural analysis was performed. The footings were designed to provide adequate one-way and punching shear strength along with adequate flexural strength. To provide the flexural strength, hooked rebars were designed in the tension face of the footings. The purpose for the hooked rebars was to provide the required development length without the need to increase the footing area. In the pilasters and walls, hooked dowel bars were used along with straight rebars to allow the load to be transferred to the footings below. Stirrups were also designed within the pilasters and walls to provide additional shear resistance.

Four final footing sizes were selected (width x length x thickness): 52” x 52” x 24” (square footing), 44” x 44” x 24” (square footing), 36” x varying length (north/south basement wall footings) x 18”, 38” x varying length x 18” (east/west basement wall footings) and 12” x varying length x 12” (frost wall footings). A detailed foundation guide and cross section views can be viewed in Design Sheet S-5 through S-7 and D-5 through D-8, respectively. Detailed foundation design calculations can be found in *Appendices T and U.*

Site Plan and Drainage

The site plan for the combined use of Building A and Building C features two driveways that connect to 5th Ave S on the south side of the site and two driveways that connect to the alleyway on the north side of the site. The driveways are connected with two 22-foot-wide aisles for cars, service vehicles, and emergency vehicles to travel through. Both sides of the aisles are surrounded by parking stalls for the shared use of Building A and Building C. On the north side of building C, perpendicular to the alleyway, additional parking stalls are provided. Two trash/dumpster areas on the north end of the site have been designated for use by Building A and Building C.

Two alternatives for pavement were considered for the parking lot: six inches of Hot Mix Asphalt and five inches of Portland Cement Concrete. Detailed pavement thickness design can be viewed in *Appendix V*. Based on the cost estimation which can be found in Section VII of this report, the asphalt alternative is the recommended option. The designed site features 57 parking stalls, including four ADA accessible parking stalls. Using the guidelines specified by Chapter 8 Section 8C-1 of the Statewide Urban Design and Specifications (SUDAS) Design Manual, it was determined that based on the number of parking stalls within the site, a minimum of three ADA accessible parking stalls were required. One additional ADA accessible parking stall was added to the site as a conservative assumption. For a site plan and parking lot view, see Design Sheet C-1.

As discussed in Section IV of this report, there are no existing drainage structures located on the site. The nearest intakes are in the alley on the west side of the site and on 5th Ave S near the northwest corner of the 5th Ave S and S 3rd Street intersection. Based on the location of these existing intakes, the drainage scheme of the proposed site has been developed to drain toward them. The intake in the alleyway is much further away from the site than the intake on 5th Ave S. Therefore, to avoid pooling in the alleyway, the drainage scheme is crafted to drain most of the water to the 5th Ave S intake. The south driveways and the majority of both aisles drain to the intake on 5th Ave S, leaving the north driveways and the north parking stalls to drain toward the alley intake.

The slopes of the parking lot were designed based on ADA standards. Therefore, the maximum design running slope and cross slope for sidewalks are 1:20 (5% grade) and 1:48 (2.1% grade), respectively. The parking lot was also designed to adhere to these ADA standards; therefore, the maximum design slope of the parking is 1:20 (5%). The ADA access ramp connected to Building A was designed according to ADA standard specifications and SUDAS standard specification 9072. The maximum design running slope was 1:12 (%) and the maximum design cross slope was 1:48 (%). A minimum of three feet of clear width is required, but the designed ramp features five feet of clear width for additional ease of access. Due to the site grading and the required excavation for the basement of Building C, the site is in a state of net cut (445 CY of cut). For the drainage plan, see Design Sheet C-2. For parking lot, sidewalk, and ramp details, see Design Sheet D-1 through D-2.

Section VII: Engineer's Cost Estimate

Using Autodesk Revit and Civil3D to generate material takeoffs, as shown in *Appendix C*, a quantity was assigned to each unit. In accordance with the Gordian 2018 Edition of Heavy Construction Costs with RSMeans Data, prices were then assigned to each quantity. The cost estimate was divided into four main sections: demolition cost, site cost, Building A renovation cost and Building C construction cost. Specific material costs were estimated for each section, and can be found below in *Appendix C. Figure 7.1* below shows the item cost breakdown for the renovation of Building A.

Note that some items within *Figure 7.1* do not have a quantity associated with them because those items were priced using a percentage of the total square footage of a three-story building, rather than by the actual quantity of the material, in accordance with the guidelines found within the Gordian 2019 Edition of Square Foot Costs with RSMeans Data. Once the total cost of each material was calculated, the total construction cost of the project was calculated.

Figure 7.2 contains a service breakdown with its respective cost in U.S. dollars. In addition to the construction cost, a 20% contingency and 15% construction and administration estimate were included in the total project cost to account for the structural uncertainties in the existing Building A. The unit prices were taken from the 2018 edition of the Gordian Construction Costs book, and Consumer Price Index rates for construction was used to provide a more accurate estimate of the cost in today's dollar. The total project cost was rounded according to the RSMeans rounding standards, which can be found in *Figure C.12* in *Appendix C*. The estimated construction cost for this project came to $13,339,000.

Item	Quantity Type	Quantity	Unit Price	Cost w/ OH	w/ Inflation Rate	
Substructure						
Elevator Foundation	CY	9	$ 126.00	$ 1,134.00	$ 1,134.00	2022 Price
Shell						
Superstructure						
Floor Construction	SF Building	32420	$ 14.26	$ 462,309.20	$ 584,520.85	Excluded
Roof Construction	SF Roof	6484	$ 2.61	$ 16,923.24	$ 21,396.91	
Masonry	EA	2387	$ 13.78	$ 32,892.86	$ 32,892.86	2022 Price
Exterior Enclosure						
Exterior Windows	EA	88	$ 529.00	$ 46,552.00	$ 58,858.04	
Exterior Doors	EA	9	$ 3,170.00	$ 28,530.00	$ 36,071.92	
Roofing						
Roof Covers	SF Roof	6484	$ 0.73	$ 4,733.32	$ 5,984.58	
Interiors						
Partitions	SF Wall	38247.5	$ 6.96	$ 266,202.60	$ 336,573.38	
Interior Doors	EA	107	$ 1,398.00	$ 149,586.00	$ 189,129.13	
Interior Windows	EA	12	$ 529.00	$ 6,348.00	$ 8,026.10	
Stair Construction	Flight	9	$ 2,920.00	$ 26,280.00	$ 33,227.13	
Wall Finishes	SF Wall	38247.5	$ 2.62	$ 100,208.45	$ 126,698.60	
Floor Finishes	SF Floor	32420	$ 5.32	$ 172,474.40	$ 218,068.09	
Ceiling Finishes	SF Ceiling	25936	$ 4.59	$ 119,046.24	$ 150,516.17	
Tube Railing	LF	320	$ 76.50	$ 24,480.00	$ 30,951.30	
Services						
Conveying						
Elevators & Lifts	EA	1	$ 117,675.00	$ 117,675.00	$ 148,782.44	
			Rate			
Plumbing						
Lump Sum	SF Building		12%	$ 719,334.96	$ 909,491.48	
HVAC						
Lump Sum	SF Building		13%	$ 779,279.54	$ 985,282.44	
Fire Protection						
Lump Sum	SF Building		3%	$ 179,833.74	$ 227,372.87	
Electrical						
Lump Sum	SF Building		9%	$ 539,501.22	$ 682,118.61	
Building Site Work						
ADA Ramp	CY	14	$ 126.00	$ 1,764.00	$ 1,764.00	2022 Price
Special Constuction						
Turf	SF Turf	2070	$ 18.75	$ 38,812.50	$ 38,812.50	2022 Price
Concrete Curb	CY	4	$ 126.00	$ 504.00	$ 504.00	2022 Price
				Total:	$4,243,656.53	

Figure 7.1: Sample Material Cost Sheet for Renovation of Building A

Discipline	Cost (USD)
Demolition	$ 671,874
Site	$ 554,221
Structural	$ 2,252,547
Architectural	$ 2,169,650
MEP	$ 4,017,530
Materials and Labor Subtotal	$ 9,665,822
Construction and Administration (20%)	$ 1,933,165
Contingency (15%)	$ 1,739,849
Total Construction	**$ 13,339,000**

Figure 7.2 Final Construction Cost Estimate

Section VIII: Reference Attachments

Appendix A: Bibliography

ACI 318 –19 Building Code Requirements for Structural Concrete. American Concrete Institute, June 2019

ADA Accessibility Details: Based on 2010 ADA. Builders Book, Inc., 2014.

Heavy Construction Costs with RSMeans Data 2018 32nd Annual edition.

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International Building Code® 2018. International Code Council, Inc., 2018.

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

"2018 National Design Specification (NDS) for Wood Construction." *American Wood Council*, 14 Mar. 2022

Nucor Vulcraft. *Vulcraft Joist & Joist Girder ASD-K-Series Load Table Manual*, 2010.

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

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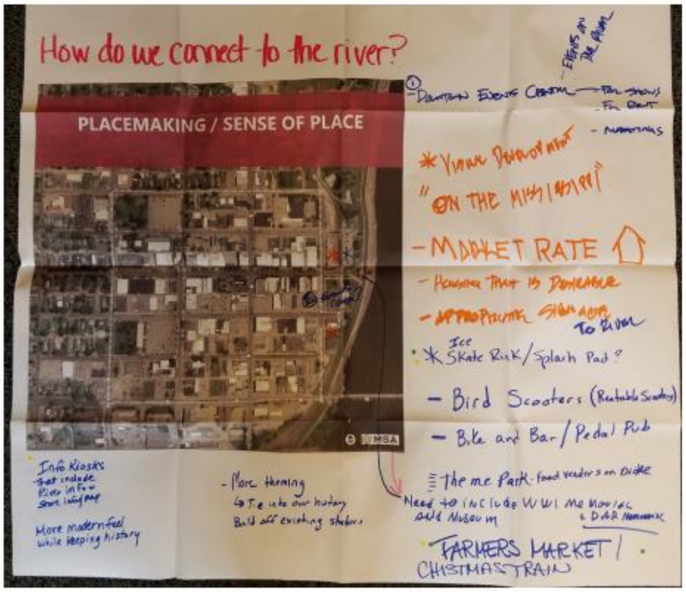
Square Foot Costs with RSMeans Data 2019 40th Annual Edition. Gordian, 2019.

Steel Construction Manual. American Institute of Steel Construction, 2017.

Appendix B: City of Clinton's Downtown Master Plan

Placemaking / Sense of Place



Overall, respondents would like to see a "theme" or "cohesive" treatment for the downtown. Something that makes Downtown Clinton unique and sets it apart from nearby river towns.

» Make a stronger connection to the river
» Improves aesthetics
» Create an environment for all ages, including families
» Continue and create new events to bring a variety of people and interests downtown
» Connect to history
» Continue to create a 24-hour living environment (residential, entertainment, employment, recreation, retail, etc)
» Wayfinding and storytelling (history, logging, riverfront, art displays, etc)
» Better theme the public spaces and right-of-way to create unique environment and reinforce messaging
» Visually tie the Mississippi River to the Downtown
» Create more market rate housing for all ages and household types

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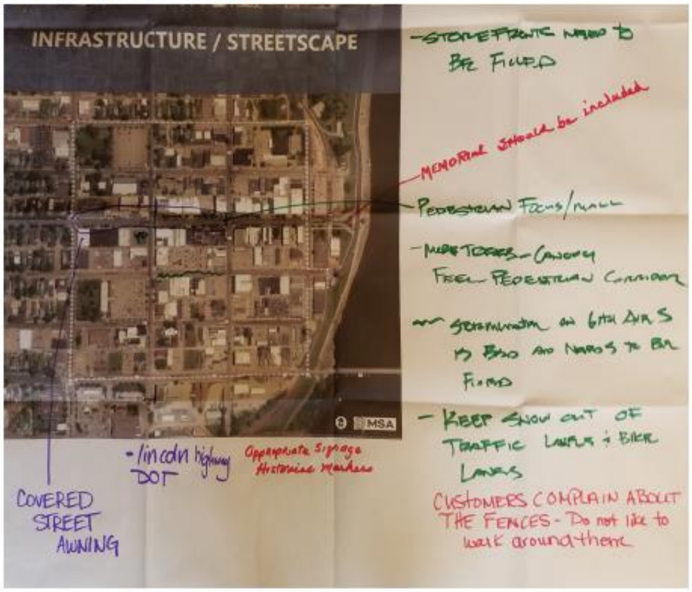
Figure B.1: Community response to the city questionnaire on placemaking/sense of place

Infrastructure / Streetscape



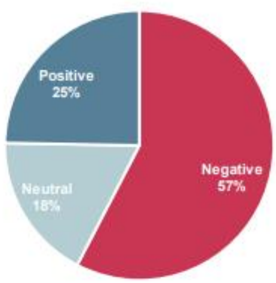
Overall, respondents would like to see a "theme" or "cohesive" treatment for the downtown. Something that makes Downtown Clinton unique and sets it apart from nearby river towns.

» Make a stronger connection to the river through streetscape elements.
» Include Memorial as part of the downtown.
» Become more pedestrian oriented.
» Improve snow removal practices.
» Incorporate wayfinding.
» Provide shade.
» Improve stormwater conditions

Community Survey

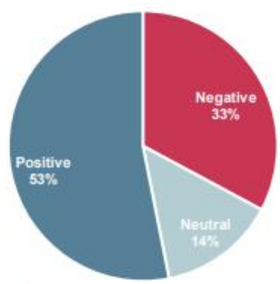
A short survey was created to gauge community sentiment, perceptions, and habits. The following is a summary of key findings from the nearly 100 surveys collected. Other key findings can are noted throughout the document.

Please offer a word or phrase that you use to describe downtown Clinton today.



Negative words: Dilapidated, empty, rundown, struggling, under-utilized, etc.
Neutral words: Limited attraction, trying, underwhelming, stagnant, potential, etc.
Positive words: Beautiful, progressive, reviving, eclectic, local, etc.

What will downtown Clinton look like about 25 years from now? Use your own word or phrase to describe what you envision for the area.

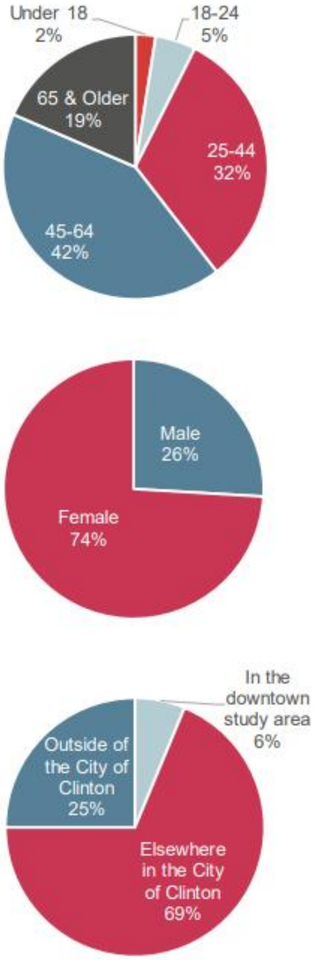


Negative words: Empty, same, abandoned, vacant, etc.
Neutral words: "if" phrases, great IF we can bring in new companies, the same IF we don't we can modernize it, etc.
Positive words: Funky cool urban zone, vibrant, lively, updated nostalgia, full of life, a meeting place, clean and well kept, full store fronts, etc.

Downtown **Clinton**. 17

Figure B.2: Community response to the city questionnaire on public opinion of the downtown Clinton area.

Survey Respondent's Demographics:



SURVEY QUICK FACTS

Top 2 Attractants:
- **16% | Shopping after 5pm on weekdays**
- **12% | Selection of Goods/Services**

Top 3 Disadvantages to Downtown:
- **67% Poor selection of goods/services**
- **61% Limited hours**
- **53% Poor appearance**

4 Changes for More Business Downtown:
- **83% Greater variety of stores/establishments**
- **71% More places to eat**
- **55% Better selection of merchandise/services**
- **34% Better atmosphere/aesthetics**


Participants at the After Hours Open House.

Figure B.3: Community response demographic statistics to the city questionnaire on public opinion of the downtown Clinton area.

One-Way Pair(3rd/4th)

work in tandem to provide a north/south alternative to SR 67 (2nd Street). The overall character is less urban than the other character areas in the downtown, providing a mix of residential and commercial properties.

Existing characteristics of this area include:

- 4-lane Roads with on-street parking in places
- Vacant lots
- Minimal character in the streetscape
- Good landscaping and tree installation
- Limited Downtown Character

Characteristics needed to elevate the "One-Way Pair" to a higher level:

- Murals reflecting history and vision of community
- Screening of parking lots and vacant lots
- Facade improvements: install windows, colorful awnings, and pedestrian scale signage
- Streetscaping elements, but to a lesser degree of 5th Avenue should complement, but not overwhelm the corridor
- Entrance features to 5th Avenue
- Wayfinding signage to direct traffic to parking and destinations throughout Downtown
- Mid-block bumpouts to reduce traffic speed and increase pedestrian safety and use
- Intersection treatments to promote place-making and community identity

94%
PARK ON STREET
49%
PARK WITHIN 1/2 BLOCK
30%
PARK NEAR ENTRY

Figure B.4: Page 24 of the City of Clinton, IA Downtown Master Plan

REDEVELOPMENT RECOMMENDATION:

» Work with owner of 522 S. 1st Street to redevelop as multi-story building, or consider a pocket park with potential utilizing the side of 516 S. 1st Street to view movies or placing a large display of art/ mural as background to the park space.

522 S. 1st Street (vacant lot)

Makeshift Outdoor Movie Theater (Example Image)

» If the upper stories of 516 S. 1st Street and 101 S. 5th Avenue are vacant, work with property owners to re-purpose those floors for high-end residential (lofts) or short-term vacation rentals. Location suggest high potential for this type of use, which can bring additional expandable incomes to the downtown.

516 S. 1st Street and 101 S. 5th Avenue (underutilized)

» Develop the 1.3-acre City-owned property on Riverview Drive between 4th and 5th Avenues. The views and location within the downtown (i.e., near 5th Avenue, Riverview Park, and Discovery Trail) makes this property highly marketable. This site would be ideal for high-end housing with or without a commercial component (e.g., restaurant and/or office) or a hotel with or without a restaurant. The design can mitigate concerns with the railroad (with ground floor parking or commercial use and a decorative solid wall along railroad) and develop around existing utility structure (if required to remain). This development can bring expandable income to the downtown and increase the increment in the Tax Increment Finance district.

City-owned Lot on Riverview Drive (underutilized)

Signature Hotel (Example Image)

» The 0.72-acre City lot on 1st Street between 4th and 5th Avenues could remain as a parking lot, or redevelop as a commercial use – ideally a restaurant or another destination business.

City-owned Lot on S. 1st Street (underutilized)

Restaurant w/ Rooftop Seating (Example Image)

Downtown **Clinton.** 37

Figure B.5: Page 37 of the City of Clinton, IA Downtown Master Plan

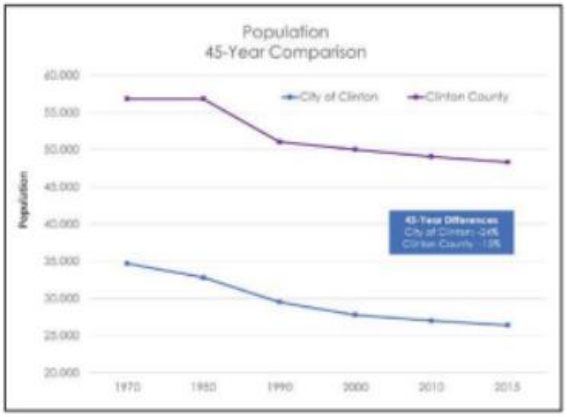


Figure 2: 45-Year Population Change. Data source: ACS.

AGE

Along with the decline of population, the City of Clinton has grown older. From 2011 to 2015, the number of individuals 50 years and older in Clinton grew by 15.1%, while those 19 years and younger decreased by 12.9%. The largest decrease in population in the age group of individuals between 40 and 49 years old, with a decrease of 17.9%. These findings present both challenges and opportunities for the community, and must be taken into account for when considering different economic development strategies.

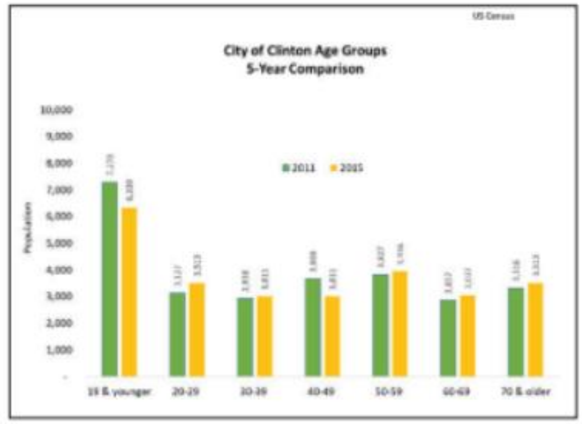


Figure 3: Clinton Age Groups Comparison. Data source: US Census.

POVERTY

Based on 2015 Census data, the poverty rate for the City of Clinton was 17.3%, while the County was 13.9%. For 2015, the poverty rate for the State of Iowa was 12.5%.

EDUCATIONAL ATTAINMENT

Overall, the City of Clinton has proportionately fewer individuals who hold four-year, graduate, and professional degrees, when compared to the State of Iowa and U.S. The percentage of individuals in Clinton who have attained a bachelor's degree is 13.9%. Clinton's percentage of individuals who have completed a graduate or professional degree is 5.6%. The national average

Figure B.6: Population demographic statistics for Clinton, IA

Appendix C: Design Renderings and Models

Figure C.1 Rendering of Building C (Restaurant)

Figure C.2 Model of Building C interior

Figure C.3 Model of bar in Building C

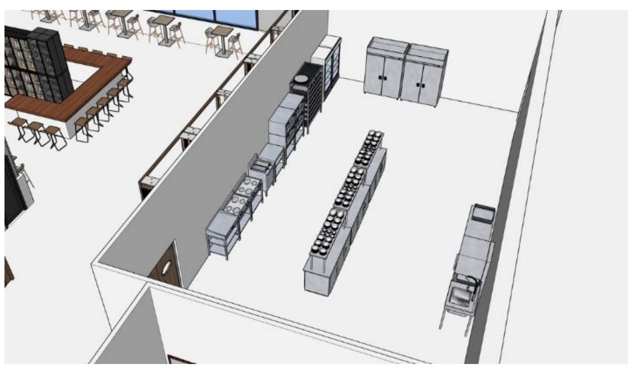


Figure C.4 Model of kitchen in Building C

Figure C.5 Rendering of Building A (Apartments)

Figure C.6 Model of example layout for a unit in Building A

Floor Material Takeoff			
Family and Type	Material: Volume (Cu Yd)	Material: Area	Material: Name
Floor: 5" Concrete	63.08	4087	Concrete
Floor: 14" Hollow Core Slab	83.49	1932	Concrete Precast
Floor: Metal Deck and Roofing	28.02	6053	Metal Deck
Floor: Metal Deck and Roofing	9.34	6053	Plywood Sheathing
Floor: Metal Deck and Roofing	112.09	6053	Rigid insulation
Floor: Metal Deck and Roofing	0	6053	Roofing Membrane

Figure C.7 Material Takeoff Generated Through Revit for Structural Flooring and Roofing

Structural Framing Material Takeoff		
Family and Type	Material: Volume (Cu Ft)	Material: Unit weight
W-Wide Flange: W18X60	3.572	490.00 lb/ft³
W-Wide Flange: W18X60	2.952	490.00 lb/ft³
W-Wide Flange: W18X60	2.942	490.00 lb/ft³
W-Wide Flange: W18X60	2.404	490.00 lb/ft³
W-Wide Flange: W30X132	7.78	490.00 lb/ft³
W-Wide Flange: W30X132	6.718	490.00 lb/ft³
W-Wide Flange: W30X132	5.194	490.00 lb/ft³
W-Wide Flange: W30X132	5.229	490.00 lb/ft³
W-Wide Flange: W30X132	12.086	490.00 lb/ft³
W-Wide Flange: W30X132	12.086	490.00 lb/ft³
W-Wide Flange: W30X132	9.494	490.00 lb/ft³
W-Wide Flange: W30X132	6.698	490.00 lb/ft³
W-Wide Flange: W30X132	6.427	490.00 lb/ft³
W-Wide Flange: W30X132	9.225	490.00 lb/ft³
W-Wide Flange: W30X132	6.437	490.00 lb/ft³
W-Wide Flange: W33X141	5.525	490.00 lb/ft³

Figure C.8 Material Takeoff Generated Through Revit for Structural Framing

Wall Material Takeoff		
Family and Type	Material: Volume (Cu Yd)	Material: Name
Basic Wall: Bearing Wall 8" Concrete	1.82	Concrete
Basic Wall: Bearing Wall 8" Concrete	1.21	Concrete
Basic Wall: Bearing Wall 8" Concrete	2.14	Concrete
Basic Wall: Bearing Wall 8" Concrete	1.21	Concrete
Basic Wall: Bearing Wall 8" Concrete	1.82	Concrete
Basic Wall: Bearing Wall 8" Concrete	1.56	Concrete
Basic Wall: Bearing Wall 8" Concrete	1.5	Concrete
Basic Wall: Bearing Wall 8" Concrete	1.5	Concrete
Basic Wall: Bearing Wall 8" Concrete	1.5	Concrete
Basic Wall: Foundation - 8" Concrete	26.65	Concrete
Basic Wall: Foundation - 8" Concrete	6.06	Concrete
Basic Wall: Foundation - 8" Concrete	7.45	Concrete
Basic Wall: Foundation - 8" Concrete	7.77	Concrete
Basic Wall: Foundation - 8" Concrete	10.6	Concrete
Basic Wall: Foundation - 8" Concrete	6.06	Concrete

Figure C.9 Material Takeoff Generated Through Revit for Structural Walls

Structural Foundation Material Takeoff			
Family and Type	Material: Volume (Cu Ft)	Material: Unit weight	Assembly Description
Footing-Rectangular: 44" x 44" x 24" Sq Footing	26.89	150.28 lb/ft³	Footings & Pile Caps
Footing-Rectangular: 44" x 44" x 24" Sq Footing	26.89	150.28 lb/ft³	Footings & Pile Caps
Footing-Rectangular: 44" x 44" x 24" Sq Footing	26.89	150.28 lb/ft³	Footings & Pile Caps
Footing-Rectangular: 44" x 44" x 24" Sq Footing	26.89	150.28 lb/ft³	Footings & Pile Caps
Footing-Rectangular: 44" x 44" x 24" Sq Footing	26.89	150.28 lb/ft³	Footings & Pile Caps
Footing-Rectangular: 44" x 44" x 24" Sq Footing	26.89	150.28 lb/ft³	Footings & Pile Caps
Footing-Rectangular: 44" x 44" x 24" Sq Footing	26.89	150.28 lb/ft³	Footings & Pile Caps
Footing-Rectangular: 44" x 44" x 24" Sq Footing	26.89	150.28 lb/ft³	Footings & Pile Caps
Footing-Rectangular: 44" x 44" x 24" Sq Footing	26.89	150.28 lb/ft³	Footings & Pile Caps
Footing-Rectangular: 44" x 44" x 24" Sq Footing	26.89	150.28 lb/ft³	Footings & Pile Caps
Footing-Rectangular: 44" x 44" x 24" Sq Footing	26.89	150.28 lb/ft³	Footings & Pile Caps
Footing-Rectangular: 44" x 44" x 24" Sq Footing	26.89	150.28 lb/ft³	Footings & Pile Caps
Footing-Rectangular: 44" x 44" x 24" Sq Footing	26.89	150.28 lb/ft³	Footings & Pile Caps
Footing-Rectangular: 52" x 52" x 24" Sq Footing	37.56	150.28 lb/ft³	Footings & Pile Caps
Foundation Slab: 5" Foundation Slab With 4" of Backfill	1327.63	150.28 lb/ft³	Slab
Wall Foundation: Wall Foundation 1' x 21'-4" x 1'	21.33	150.28 lb/ft³	Strip Footings
Wall Foundation: Wall Foundation 1' x 22'-5" x 1'	22.42	150.28 lb/ft³	Strip Footings
Wall Foundation: Wall Foundation 1' x 22'-5" x 1'	22.42	150.28 lb/ft³	Strip Footings
Wall Foundation: Wall Foundation 1' x 31'-4" x 1'	31.33	150.28 lb/ft³	Strip Footings
Wall Foundation: Wall Foundation 1.5' x 23'-8" x 1' 2	106.67	150.28 lb/ft³	Strip Footings
Wall Foundation: Wall Foundation 3' x 24'-6" x 1'-8"	111.67	150.28 lb/ft³	Strip Footings
Wall Foundation: Wall Foundation 3' x 33'-6" x 1'-8"	156.67	150.28 lb/ft³	Strip Footings
Wall Foundation: Wall Foundation 3' x 84'-6" x 1'-8"	411.67	150.28 lb/ft³	Strip Footings
Wall Foundation: Wall Foundation 3'-2" x 19'-3" x 1'-8"	89.28	150.28 lb/ft³	Strip Footings
Wall Foundation: Wall Foundation 3'-2" x 19'-3" x 1'-8"	89.28	150.28 lb/ft³	Strip Footings
Wall Foundation: Wall Foundation - 1' x 16'-9" x 1'	16.75	150.28 lb/ft³	Strip Footings
Wall Foundation: Wall Foundation - 1' x 16'-9" x 1'	16.75	150.28 lb/ft³	Strip Footings
Wall Foundation: Wall Foundation - 1' x 22'-0" x 1'	22	150.28 lb/ft³	Strip Footings
Wall Foundation: Wall Foundation - 1' x 26'-0" 1'	26.33	150.28 lb/ft³	Strip Footings
Wall Foundation: Wall Foundation - 1' x 26'-0" 1'	26	150.28 lb/ft³	Strip Footings

Figure C.10 Material Takeoff Generated Through Revit for Structural Foundations

Structural Column Material Takeoff				
Family and Type	Material: Volume (Cu Yd)	Material: Unit weight	Count	Material: Name
Concrete-Rectangular-Column: 18" x 18" Pilaster	1.01	150.28 lb/ft³	1	Concrete
Concrete-Rectangular-Column: 18" x 18" Pilaster	1.01	150.28 lb/ft³	1	Concrete
Concrete-Rectangular-Column: 18" x 18" Pilaster	1.01	150.28 lb/ft³	1	Concrete
Concrete-Rectangular-Column: 18" x 18" Pilaster	1.01	150.28 lb/ft³	1	Concrete
Concrete-Rectangular-Column: 18" x 18" Pilaster	1.01	150.28 lb/ft³	1	Concrete
Concrete-Rectangular-Column: 18" x 18" Pilaster	1.01	150.28 lb/ft³	1	Concrete
Concrete-Rectangular-Column: 18" x 18" Pilaster	0.215	150.28 lb/ft³	1	Concrete
Concrete-Rectangular-Column: 18" x 18" Pilaster	0.215	150.28 lb/ft³	1	Concrete
Concrete-Rectangular-Column: 18" x 18" Pilaster	0.215	150.28 lb/ft³	1	Concrete
Concrete-Rectangular-Column: 18" x 18" Pilaster	0.215	150.28 lb/ft³	1	Concrete
Concrete-Rectangular-Column: 18" x 18" Pilaster	0.215	150.28 lb/ft³	1	Concrete
Concrete-Rectangular-Column: 18" x 18" Pilaster	0.215	150.28 lb/ft³	1	Concrete
Concrete-Rectangular-Column: 18" x 18" Pilaster	0.215	150.28 lb/ft³	1	Concrete
Concrete-Rectangular-Column: 18" x 18" Pilaster	0.215	150.28 lb/ft³	1	Concrete
W Shapes-Column: W14X48	0.065	490.00 lb/ft³	1	Steel ASTM A992
W Shapes-Column: W14X48	0.065	490.00 lb/ft³	1	Steel ASTM A992
W Shapes-Column: W14X48	0.065	490.00 lb/ft³	1	Steel ASTM A992
W Shapes-Column: W14X48	0.065	490.00 lb/ft³	1	Steel ASTM A992
W Shapes-Column: W14X48	0.065	490.00 lb/ft³	1	Steel ASTM A992
W Shapes-Column: W14X48	0.065	490.00 lb/ft³	1	Steel ASTM A992
W Shapes-Column: W14X132	0.179	490.00 lb/ft³	1	Steel ASTM A992
W Shapes-Column: W14X132	0.179	490.00 lb/ft³	1	Steel ASTM A992
W Shapes-Column: W14X132	0.179	490.00 lb/ft³	1	Steel ASTM A992
W Shapes-Column: W14X132	0.179	490.00 lb/ft³	1	Steel ASTM A992
W Shapes-Column: W14X132	0.179	490.00 lb/ft³	1	Steel ASTM A992
W Shapes-Column: W14X132	0.179	490.00 lb/ft³	1	Steel ASTM A992
W Shapes-Column: W14X159	0.217	490.00 lb/ft³	1	Steel ASTM A992
W Shapes-Column: W14X159	0.217	490.00 lb/ft³	1	Steel ASTM A992

Figure C.11 Material Takeoff Generated Through Revit for Structural Columns

Prices From	To	Rounded to Nearest
$0.01	$5.00	$0.01
5.01	20.00	0.05
20.01	100.00	1.00
100.01	1,000.00	5.00
1,000.01	10,000.00	25.00
10,000.01	50,000.00	100.00
50,000.01	Up	500.00

Figure C.12 RSMeans Rounding Standards

Total SF (Building B)	19500
Total SF (Building A)	32420
Building B Height (LF)	35

Item	Quantity Type	Quantity	Unit Price	Cost w/ OH	w/ Inflation
Selective Structural Demoliton	CF	682500	$ 0.38	$ 259,350.00	$ 346,766.69
Selective Interior Demolition	SF Floor	32420	$ 7.50	$ 243,150.00	$ 325,106.31

Total:	$ 671,873.00

Figure C.13 Demolition Cost Estimate

Item	Quantity Type	Quantity	Unit Cost	Cost w/ OH	w/ Inflation Rate	
Earth Work	CY	444	$ 9.85	$ 4,373.40	$ 5,847.50	
Markers	LF	1220	$ 1.00	$ 1,220.00	$ 1,220.00	2021 Price
Concrete Work (Site)	SF	3812	$ 4.56	$ 17,382.72	$ 23,241.75	
Asphalt Work	SY	2613	$ 124.00	$ 324,012.00	$ 433,223.71	
Landscaping	SF	2724	$ 12.00	$ 32,688.00	$ 32,688.00	2022 Price

Total:	$ 496,220.97

Figure C.14 Civil Site Cost Estimate

Item	Quantity Type	Quantity	Unit Price	Cost w/ OH	w/ Inflation Rate	
Substructure						
Elevator Foundation	CY	9	$ 126.00	$ 1,134.00	$ 1,134.00	2022 Price
Shell						
Superstructure						
Floor Construction	SF Building	32420	$ 14.26	$ 462,309.20	$ 584,520.85	Excluded
Roof Construction	SF Roof	6484	$ 2.61	$ 16,923.24	$ 21,396.91	
Masonry	EA	2387	$ 13.78	$ 32,892.86	$ 32,892.86	2022 Price
Exterior Enclosure						
Exterior Windows	EA	88	$ 529.00	$ 46,552.00	$ 58,858.04	
Exterior Doors	EA	9	$ 3,170.00	$ 28,530.00	$ 36,071.92	
Roofing						
Roof Covers	SF Roof	6484	$ 0.73	$ 4,733.32	$ 5,984.58	
Interiors						
Partitions	SF Wall	38247.5	$ 6.96	$ 266,202.60	$ 336,573.38	
Interior Doors	EA	107	$ 1,398.00	$ 149,586.00	$ 189,129.13	
Interior Windows	EA	12	$ 529.00	$ 6,348.00	$ 8,026.10	
Stair Construction	Flight	9	$ 2,920.00	$ 26,280.00	$ 33,227.13	
Wall Finishes	SF Wall	38247.5	$ 2.62	$ 100,208.45	$ 126,698.60	
Floor Finishes	SF Floor	32420	$ 5.32	$ 172,474.40	$ 218,068.09	
Ceiling Finishes	SF Ceiling	25936	$ 4.59	$ 119,046.24	$ 150,516.17	
Tube Railing	LF	320	$ 76.50	$ 24,480.00	$ 30,951.30	
Services						
Conveying						
Elevators & Lifts	EA	1	$ 117,675.00	$ 117,675.00	$ 148,782.44	
			Rate			
Plumbing						
Lump Sum	SF Building		12%	$ 719,334.96	$ 909,491.48	
HVAC						
Lump Sum	SF Building		13%	$ 779,279.54	$ 985,282.44	
Fire Protection						
Lump Sum	SF Building		3%	$ 179,833.74	$ 227,372.87	
Electrical						
Lump Sum	SF Building		9%	$ 539,501.22	$ 682,118.61	
Building Site Work						
ADA Ramp	CY	14	$ 126.00	$ 1,764.00	$ 1,764.00	2022 Price
Special Constuction						
Turf	SF Turf	2070	$ 18.75	$ 38,812.50	$ 38,812.50	2022 Price
Concrete Curb	CY	4	$ 126.00	$ 504.00	$ 504.00	2022 Price

Total:	$4,243,656.53

Figure C.15 Building A Renovation Cost Estimate

Item	Quantity Type	Quantity	Unit Price	Cost w/ OH	w/ Inflation Rate	
Earth Work	CY	1000	$ 58.00	$ 58,000.00	$ 58,000.00	2022 Price
Substructure						
Footing	CY	25	$ 160.00	$ 4,000.00	$ 4,000.00	2022 Price
Foundation Wall	CY	125	$ 160.00	$ 20,000.00	$ 20,000.00	2022 Price
Superstructure						
Columns	LF	252	$ 4,200.00	$ 1,058,400.00	$ 1,058,400.00	2022 Price
Girders	LF	238	$ 4,200.00	$ 999,600.00	$ 999,600.00	2022 Price
O.W. Joists	Ton	5.2	$ 6,000.00	$ 31,200.00	$ 31,200.00	2022 Price
Hollow Core	CY	85	$ 450.00	$ 38,250.00	$ 38,250.00	2022 Price
Slab	CY	50	$ 160.00	$ 8,000.00	$ 8,000.00	2022 Price
Grid Reinforcing	CSF	41	$ 64.50	$ 2,644.50	$ 3,535.86	
Finishing						
Concrete	SF	4090	$ 0.81	$ 3,312.90	$ 4,429.55	
Roofing						
Decking	SF	6053	$ 4.00	$ 24,212.00	$ 32,372.91	
Plywood	SF	6053	$ 2.00	$ 12,106.00	$ 12,106.00	2022 Price
Insulation	SF	6053	$ 4.50	$ 27,238.50	$ 27,238.50	2022 Price
Roof Shield	SF	6053	$ 0.23	$ 1,392.19	$ 1,861.44	
Exterior Wall						
Stone Cladding	SF Wall Ext.	3666	$ 33.50	$ 122,811.00	$ 164,205.76	
Weather Barrier	SF Wall Ext.	4043	$ 0.75	$ 3,032.25	$ 3,032.25	2022 Price
Rigid Insulation Board	SF Wall Ext.	4043	$ 2.75	$ 11,118.25	$ 11,118.25	2022 Price
Plywood	SF Wall Ext.	4043	$ 4.50	$ 18,193.50	$ 18,193.50	2022 Price
C-Channel Stud	SF Wall Ext.	4043	$ 10.50	$ 42,451.50	$ 42,451.50	2022 Price
Fire Retardent	SF Wall Ext.	4043	$ 2.00	$ 8,086.00	$ 8,086.00	2022 Price
Vapor Barrier	SF Wall Ext.	4043	$ 1.00	$ 4,043.00	$ 4,043.00	2022 Price
Gypsum Board	SF Wall Ext.	4043	$ 2.29	$ 9,258.47	$ 9,258.47	2022 Price
Exterior Wood Trim	LF Building Perimeter	377	$ 22.00	$ 8,294.00	$ 8,294.00	2022 Price
Exterior Doors						
Glass	EA	2	$ 4,800.00	$ 9,600.00	$ 9,600.00	2022 Price
Industrial Door	EA	4	$ 3,170.00	$ 12,680.00	$ 16,953.93	
Exterior Windows	EA	32	$ 3,500.00	$ 112,000.00	$ 112,000.00	2022 Price
Partition Wall	SF Wall Int.	333	$ 7.15	$ 2,380.95	$ 3,183.47	
Interior Doors	EA	4	$ 1,398.00	$ 5,592.00	$ 7,476.84	
Outdoor Patio Pavers	SF	2600	$ 8.00	$ 20,800.00	$ 20,800.00	2022 Price
Fence	LF	85	$ 60.00	$ 5,100.00	$ 5,100.00	2022 Price
			Rate			
Plumbing						
Lump Sum			10%		$ 274,279.12	
HVAC						
Lump Sum			25%		$ 688,440.60	
Fire Protection						
Lump Sum			8%		$ 219,423.30	
Electrical						
Lump Sum			12%		$ 329,134.95	

Total:	$ 4,254,069.22

Figure C.16 Building C Construction Cost Estimate

Appendix D: Building A: Elevator Design

Elevator Design

Manufacturer: Schumacher
Model: In-Ground Hydraulic Elevator
Capacity: 3,500 lb (stretcher/gurney access)
Hoistway Requirements: 8'4" x 6'11"
Platform Requirements: 7'0" x 6'2.5"
Interior: 6'8" x 5'5"
Cab Design: 700 Series - Laminate Cab

Hoist Beam (A992 W6x25):

$E := 29000\ ksi$ $\quad F_y := 50\ ksi$ $\quad I_x := 53.4\ in^4$ $\quad S_x := 16.7\ in^3$ $\quad Z_x := 18.9\ in^3$

$A_g := 7.34\ in^2$ $\quad r := 2.7\ in$ $\quad d := 6.38\ in$ $\quad b_f := 6.08\ in$ $\quad t_f := 0.455\ in$

$t_w := 0.32\ in$ $\quad r_y := 1.52\ in$ $\quad I_y := 17.1\ in^4$ $\quad C_w := 150\ in^6$ $\quad J := 0.461\ in^4$

$c := 1$ $\quad h_o := 5.93\ in$ $\quad r_{ts} := 1.74\ in$

$P := 10\ kip$

$$L := 8\ ft + 4\ in + 2 \cdot \left(7\ in + \frac{5}{8}\ in\right) = 9.604\ ft$$

$$R := \frac{P}{2} = 5\ kip$$

$$V_{max} := R = 5\ kip \qquad M_{max} := \frac{P \cdot L}{4} = 24.01\ kip \cdot ft$$

$$\Delta_{allowable} := \frac{L}{360} = 0.32\ in \qquad \Delta_{max} := \frac{P \cdot L^3}{48 \cdot E \cdot I_x} = 0.206\ in$$

$$\Delta_{max} \le \Delta_{allowable}$$

Flexure:

TABLE USER NOTE E1.1
Selection Table for the Application of Chapter E Sections

	Without Slender Elements		With Slender Elements	
Cross Section	Sections in Chapter E	Limit States	Sections in Chapter E	Limit States
	E3 E4	FB TB	E7	LB FB TB

$$C_b := 1.32$$

$$M_p := F_y \cdot Z_x = 78.75\ kip \cdot ft$$

$$\lambda_f := \frac{b_f}{(2 \cdot t_f)} = 6.681$$

$$\lambda_{pf} := 0.38 \cdot \sqrt{\frac{E}{F_y}} = 9.152 \qquad \lambda_{rf} := 1 \cdot \sqrt{\frac{E}{F_y}} = 24.083$$

For compact flanges ($\lambda_f \le \lambda_{pf}$): $\phi M_n = 0.9\, M_p$.

$$\phi M_{n.FLB} := 0.9 \cdot M_p = 70.875\ kip \cdot ft$$

$$L_b := L = 9.604\ ft$$

$$L_p := 1.76 \cdot r_y \cdot \sqrt{\frac{E}{F_y}} = 5.369\ ft$$

$$L_r := 1.95 \cdot r_{ts} \cdot \frac{E}{0.7 \cdot F_y} \cdot \sqrt{\frac{J \cdot c}{S_x \cdot h_o} + \sqrt{\left(\frac{J \cdot c}{S_x \cdot h_o}\right)^2 + 6.76 \cdot \left(\frac{0.7 \cdot F_y}{E}\right)^2}} = 23.741\ ft$$

$$F_{cr} := \frac{C_b \cdot \pi^2 \cdot E}{\left(\frac{L_b}{r_{ts}}\right)^2} \cdot \sqrt{1 + 0.078 \cdot \frac{J \cdot c}{S_x \cdot h_o} \cdot \left(\frac{L_b}{r_{ts}}\right)^2} = 138.671\ ksi$$

For $L_p < L_b \le L_r$

$$\phi M_n = 0.9 \times \mathrm{Min}\left[M_p,\ C_b\left[M_p - (M_p - 0.7\, F_y\, S_x)\left(\frac{L_b - L_p}{L_r - L_p}\right)\right]\right]$$

$$\phi M_{n.LTB} := 0.9 \cdot C_b \cdot \left(M_p - \left(M_p - 0.7 \cdot F_y \cdot S_x\right) \cdot \left(\frac{L_b - L_p}{L_r - L_p}\right)\right) = 85.328\ kip \cdot ft$$

$$\phi M_n := min\left(\phi M_{n.FLB}, \phi M_{n.LTB}\right) = 70.875\ kip \cdot ft$$

$$M_u := M_{max} = 24.01\ kip \cdot ft \qquad M_u \le \phi M_n$$

Web Local Yielding:

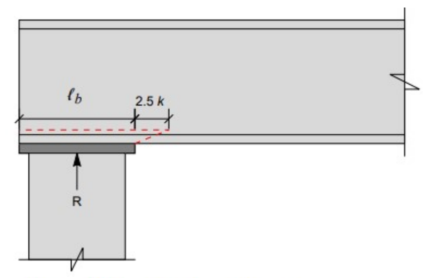
$R = 5\ kip$

$l_b := 7\ in + \frac{5}{8}\ in = 7.625\ in$

$k := 0.705\ in$

$\phi R_n := 1 \cdot (2.5 \cdot k + l_b) \cdot F_y \cdot t_w = 150.2\ kip$

$R \le \phi R_n$



$\phi R_n = 1.00 \times (2.5\,k + \ell_b)\,F_{yw}\,t_w$

Steel Bearing Plate (A36 8"x8"x1/8"):

$R_u := 5\ kip$ $\quad f_c' := 2000\ psi$

$b_f = 6.08\ in$ $\quad F_y := 36\ ksi$

$B := 8\ in$

$N := 8\ in$

$l_b := N$

$\phi_c := 0.65$

(a) On the full area of a concrete support

$$P_p = 0.85 f_c' A_1 \qquad \text{(J8-1)}$$

(b) On less than the full area of a concrete support

$$P_p = 0.85 f_c' A_1 \sqrt{A_2/A_1} \le 1.7 f_c' A_1 \qquad \text{(J8-2)}$$

where

A_1 = area of steel concentrically bearing on a concrete support, in.^{2} (mm^2)

A_2 = maximum area of the portion of the supporting surface that is geometrically similar to and concentric with the loaded area, in.^{2} (mm^2)

f_c' = specified compressive strength of concrete, ksi (MPa)

$A_1 := B \cdot N = 64\ in^2$

$a_1 := 1\ in \cdot \left(\left(7\ in + \frac{5}{8}\ in\right) - 2 \cdot (1.25\ in)\right) = 5.125\ in^2$

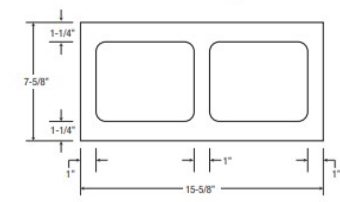
$a_2 := 2 \cdot (8\ in \cdot 1.25\ in) = 20\ in^2$

$A_2 := a_1 + a_2 = 25.125\ in^2$

$P_p := 0.85 \cdot f_c' \cdot A_1 \cdot \sqrt{\frac{A_2}{A_1}} = 68.17\ kip$

$check := 1.7 \cdot f_c' \cdot A_1 = 217.6\ kip$

8-Inch Unit Configurations



$X := \left(\frac{4 \cdot d \cdot b_f}{(d + b_f)^2}\right) \cdot \frac{R_u}{\phi_c \cdot P_p} = 0.113$

$m := \frac{N - 0.95 \cdot d}{2} = 0.97\ in$

$n := \frac{B - 0.8\ b_f}{2} = 1.568\ in$

$n' := \frac{\sqrt{d \cdot b_f}}{4} = 1.557\ in$

$\lambda := \frac{2 \cdot \sqrt{X}}{1 + \sqrt{1 - X}} = 0.346$

$l := \max(m, n, \lambda \cdot n') = 1.568\ in$

$t_{min} := l \cdot \sqrt{\frac{2 \cdot R_u}{0.9 \cdot F_y \cdot B \cdot N}} = 0.109\ in$

$t := \frac{1}{8}\ in = 0.125\ in$

$\phi R_n := 0.75 \cdot 1.8 \cdot F_y \cdot A_2 = 1221.075\ kip$ $\qquad R_u \le \phi R_n$

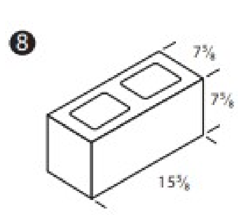
Final Steel Plate Dimensions:

$B = 8\ in$

$N = 8\ in$

$t = 0.125\ in$

Full Square 8"x8"x16" CMU Unit:



FULL SQUARE
200812700000

$b := 7\ in + \frac{5}{8}\ in$

$d := 15\ in + \frac{5}{8}\ in$

$h := 7\ in + \frac{5}{8}\ in$

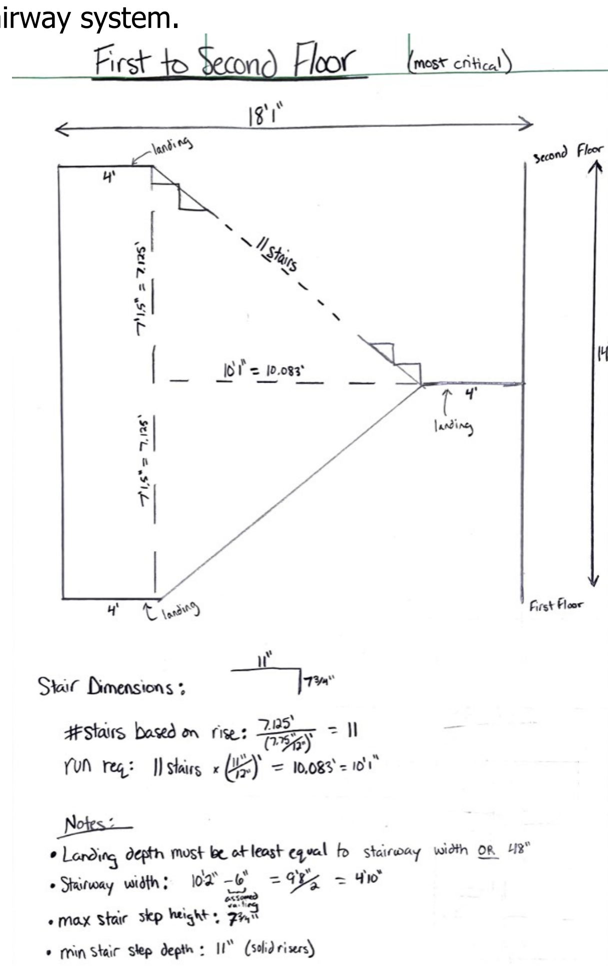
$R_u := 5\ kip$

$f_c' := 2000\ psi$

$\sigma := \frac{R_u}{A_2} = 199.005\ psi$ $\qquad \sigma \le f_c'$

Appendix E: Building A: Stairway Design

Most critical scenario for designing the members is the stair system from first floor to second floor. Stair components will be designed based on this case and will be uniform for the rest of the stairway system.



$$X := 10\ ft + 1\ in = 10.083\ ft$$
$$Y := 7\ ft + 1.5\ in = 7.125\ ft$$

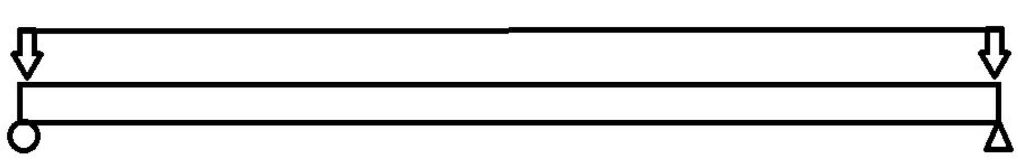
$$Z := \sqrt{X^2 + Y^2} = 12.347\ ft$$

$$9\ ft + 11\ in + \frac{3}{8}\ in = 9.948\ ft$$

Stair Stringer (Double Stringer - MC12x10.6):

$$w_{live} := 100\ psf$$
$$stairway.width := \frac{9\ ft + 11\ in + \frac{3}{8}\ in - 4\ in}{2} = 4.807\ ft$$

To treat the stair stringer as a horizontal beam, the distributed load is increased to 120 psf.



$$w := 120\ psf \cdot \frac{stairway.width}{2} = 0.288\ \frac{kip}{ft}$$
$$L := 12.347\ ft$$

$E := 29000\ ksi$	$F_y := 50\ ksi$	$I_x := 55.3\ in^4$	$S_x := 9.22\ in^3$	$Z_x := 11.6\ in^3$
$A_g := 3.1\ in^2$	$r_x := 4.22\ in$	$d := 12\ in$	$b_f := 1.5\ in$	$t_f := 0.309\ in$
$t_w := 0.19\ in$	$r_y := 0.349\ in$	$I_y := 0.378\ in^4$	$C_w := 11.7\ in^6$	$J := 0.0596\ in^4$
$c := 1$	$h_o := 11.7\ in$	$r_{ts} := 0.478\ in$	$k := 0.75\ in$	$h := d - 2\ k$

$$R := \frac{w \cdot L}{2} = 1.781\ kip$$

$$V_{max} := R = 1.781\ kip$$

$$M_{max} := \frac{w \cdot L^2}{8} = 5.496\ kip \cdot ft$$

$$\Delta_{allowable} := \frac{L}{360} = 0.412\ in \qquad \Delta_{max} := \frac{5 \cdot w \cdot L^4}{384 \cdot E \cdot I_x} = 0.094\ in$$

$$\Delta_{max} \le \Delta_{allowable}$$

$$\phi V_n := 0.9 \cdot 0.6 \cdot F_y \cdot d \cdot t_w = 61.56\ kip$$

$$\phi V_n = \frac{W \cdot L}{2} \xrightarrow{solve, W} \frac{9.9716530331254547663 \cdot kip}{ft}$$

$$w_{yield} := 9.97\ kip \cdot ft$$

$$w \le w_{yield}$$

TABLE USER NOTE F1.1 Selection Table for the Application of Chapter F Sections				
Section in Chapter F	Cross Section	Flange Slenderness	Web Slenderness	Limit States
F2		C	C	Y, LTB

$C_b := 1$

$L_b := \dfrac{L}{10} = 1.235\ ft$

$M_p := F_y \cdot Z_x = 48.333\ kip \cdot ft$

$\phi M_{yield} := 0.9 \cdot F_y \cdot S_x = 34.575\ kip \cdot ft$

$L_p := 1.76 \cdot r_y \cdot \sqrt{\dfrac{E}{F_y}} = 1.233\ ft$

$L_r := 1.95 \cdot r_{ts} \cdot \dfrac{E}{0.7 \cdot F_y} \cdot \sqrt{\dfrac{J \cdot c}{S_x \cdot h_o} + \sqrt{\left(\dfrac{J \cdot c}{S_x \cdot h_o}\right)^2 + 6.76 \cdot \left(\dfrac{0.7 \cdot F_y}{E}\right)^2}} = 3.935\ ft$

For $L_p < L_b \le L_r$

$\phi M_n = 0.9 \times \mathrm{Min}\left[M_p,\ C_b\left[M_p - (M_p - 0.7\,F_y\,S_x)\left(\frac{L_b - L_p}{L_r - L_p}\right)\right]\right]$

$\phi M_{LTB} := 0.9 \cdot min\left(M_p, C_b \cdot \left(M_p - \left(M_p - 0.7 \cdot F_y \cdot S_x\right) \cdot \left(\dfrac{L_b - L_p}{L_r - L_p}\right)\right)\right) = 43.486\ kip \cdot ft$

$\phi M_n := min\left(\phi M_{yield}, \phi M_{LTB}\right) = 34.575\ kip \cdot ft$

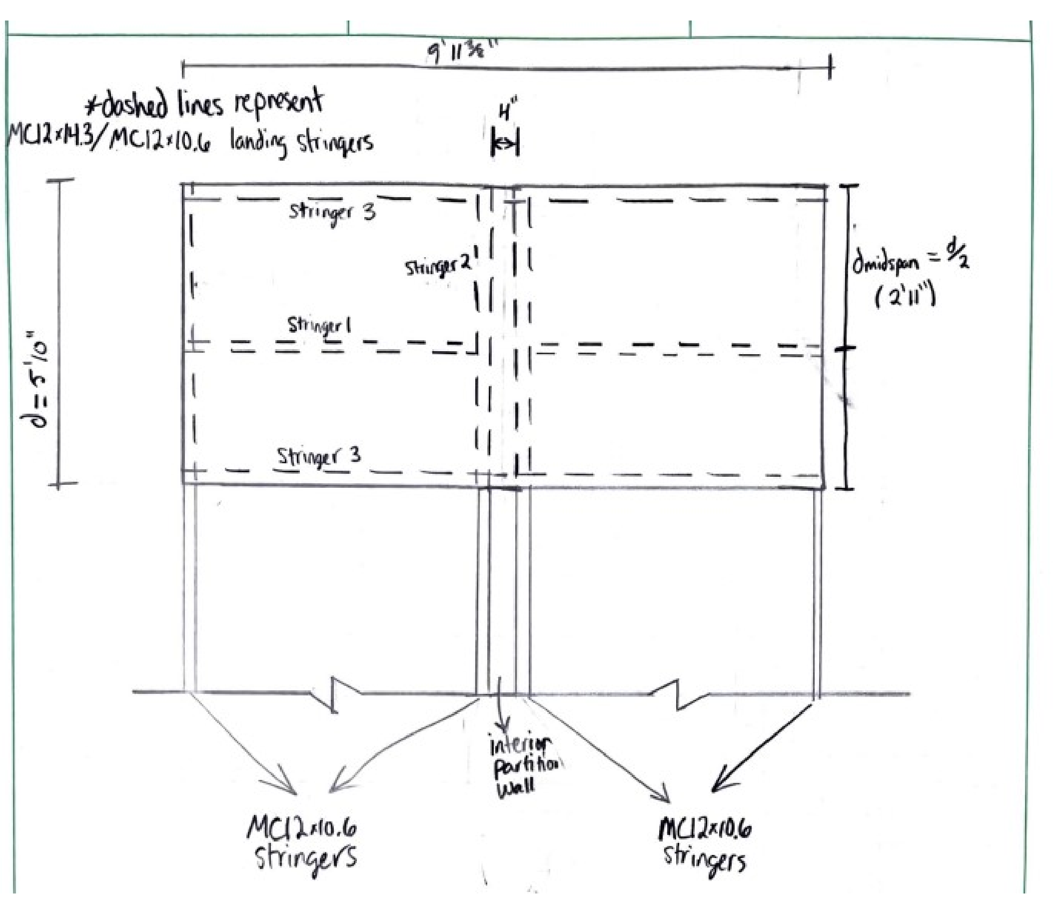
$M_u := M_{max} = 5.496\ kip \cdot ft$

$M_u \le \phi M_n$

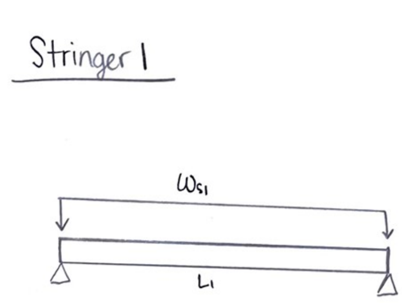
Stair Landing (Most Critical Case):

Using MC12x14.3 stringer to connect stair stringer to landing(stringer 3, as shown in figure below) and MC12x10.6 stringers as support members. A supporting stringer is to span horizontally at midway through the landing.

$d := 5\ ft + 10\ in \qquad d_{mid} := \dfrac{d}{2} = 2.917\ ft \qquad L := 9\ ft + \left(11 + \dfrac{3}{8}\right)\ in$



Stringer 1 (MC12x10.6):



$L_1 := stairway.width = 4.807\ ft$

$d_1 := d_{mid} = 2.917\ ft$

$w_{s1} := w_{live} \cdot d_1 = 0.292\ \dfrac{kip}{ft}$

$R_1 := \dfrac{w_{s1} \cdot L_1}{2} = 0.701\ kip$

$$V_{max1} := R_1 = 0.701 \; kip$$

$$M_{max1} := \frac{w_{s1} \cdot L_1^{\,2}}{8} = 0.843 \; kip \cdot ft$$

$$\Delta_{allowable} := \frac{L_1}{360} = 0.16 \; in \qquad \Delta_{max} := \frac{5 \cdot w_{s1} \cdot L_1^{\,4}}{384 \cdot E \cdot I_x} = 0.002 \; in$$

$$\Delta_{max} \le \Delta_{allowable}$$

$$\phi V_n := 0.9 \cdot 0.6 \cdot F_y \cdot d \cdot t_w = 359.1 \; kip$$

$$\phi V_n = \frac{W \cdot L_1}{2} \xrightarrow{solve, W} \frac{149.3980498374864572 \cdot kip}{ft}$$

$$w_{yield} := 146 \; kip \cdot ft$$

$$w_{s1} \le w_{yield}$$

TABLE USER NOTE F1.1
Selection Table for the Application of Chapter F Sections

Section in Chapter F	Cross Section	Flange Slenderness	Web Slenderness	Limit States
F2		C	C	Y, LTB

$$C_b := 1$$

$$L_b := L_1 = 4.807 \; ft$$

$$M_p := F_y \cdot Z_x = 48.333 \; kip \cdot ft$$

$$\phi M_{yield} := 0.9 \cdot F_y \cdot S_x = 34.575 \; kip \cdot ft$$

$$L_p := 1.76 \cdot r_y \cdot \sqrt{\frac{E}{F_y}} = 1.233 \; ft$$

$$L_r := 1.95 \cdot r_{ts} \cdot \frac{E}{0.7 \cdot F_y} \cdot \sqrt{\frac{J \cdot c}{S_x \cdot h_o} + \sqrt{\left(\frac{J \cdot c}{S_x \cdot h_o}\right)^2 + 6.76 \cdot \left(\frac{0.7 \cdot F_y}{E}\right)^2}} = 3.935 \; ft$$

For $L_b > L_r$:

$$\phi M_n = 0.9 \times \text{Min}\{M_p, F_{cr} S_x\}$$

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

$$F_{cr} := \frac{C_b \cdot \pi^2 \cdot E}{\left(\frac{L_b}{r_{ts}}\right)^2 \cdot \sqrt{1 + 0.078 \cdot \frac{J \cdot c}{S_x \cdot h_o} \cdot \left(\frac{L_b}{r_{ts}}\right)^2}} = 15.403 \; ksi$$

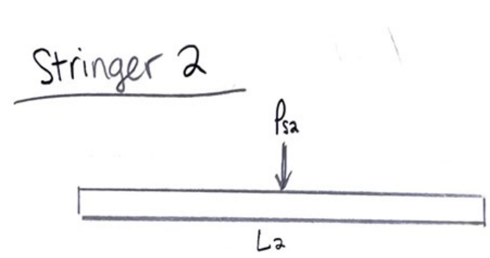
$$\phi M_{LTB} := 0.9 \cdot min\left(M_p, F_{cr} \cdot S_x\right) = 10.651 \; kip \cdot ft$$

$$\phi M_n := min\left(\phi M_{yield}, \phi M_{LTB}\right) = 10.651 \; kip \cdot ft$$

$$M_u := M_{max} = 5.496 \; kip \cdot ft$$

$$M_u \le \phi M_n$$

Stringer 2 (MC12x10.6):



$$L_2 := d = 5.833 \; ft$$

$$P_{s2} := R_1 = 0.701 \; kip$$
$$V_{max2} := P_{s2} = 0.701 \; kip$$

$$M_{max2} := \frac{P_{s2} \cdot L_2}{4} = 1.022 \; kip \cdot ft$$

$$\Delta_{allowable} := \frac{L_2}{360} = 0.194 \; in \qquad \Delta_{max} := \frac{P_{s2} \cdot L_2^{\,3}}{48 \cdot E \cdot I_x} = 0.003 \; in$$

$$\Delta_{max} \le \Delta_{allowable}$$

clear (P)

$$\phi V_n := 0.9 \cdot 0.6 \cdot F_y \cdot d \cdot t_w = 359.1 \; kip$$

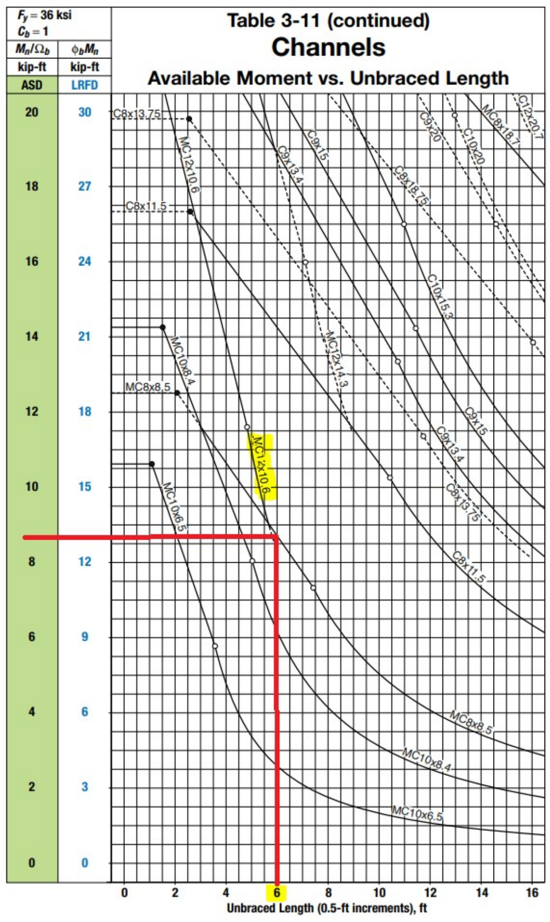
$$\phi V_n = \frac{P}{2} \xrightarrow{solve, P} 718.2 \cdot kip$$

$$P_{yield} := 718.2 \; kip \cdot ft$$

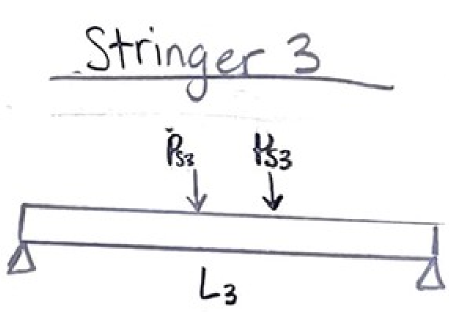
$$V_{max2} \le P_{yield}$$

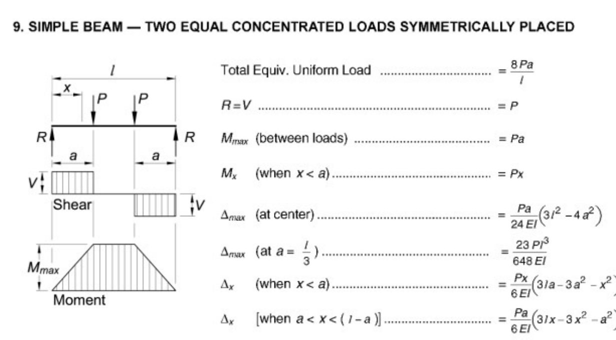
$L_b := L_2 = 5.833\ ft$

Member has enough flexural strength based on unbraced length as seen in Table 3-11 from AISC Manual below.



Stringer 3 (MC12x14.3):





$L_3 := L = 9.948\ ft \qquad a := L_1$

$P_{s3} := P_{s2} + R = 2.482\ kip$
$V_{max3} := P_{s3} = 2.482\ kip$

$M_{max3} := P_{s3} \cdot a = 11.93\ kip \cdot ft$

$I_x := 55.3\ in^4 \qquad d := 12\ in$
$t_w := 0.25\ in$

$\Delta_{allowable} := \dfrac{L_3}{360} = 0.332\ in \qquad \Delta_{max} := \dfrac{23 \cdot P_{s3} \cdot {L_3}^3}{648 \cdot E \cdot I_x} = 0.093\ in$

$\Delta_{max} \le \Delta_{allowable}$

$\text{clear}(P)$

$\phi V_n := 0.9 \cdot 0.6 \cdot F_y \cdot d \cdot t_w = 81\ kip$

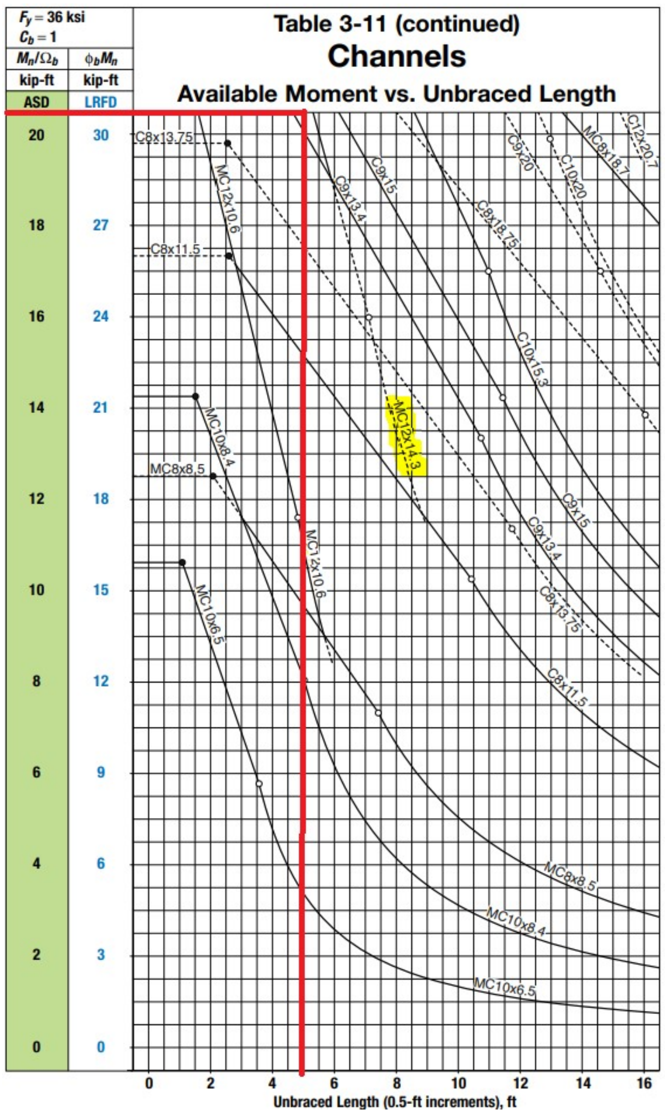
$\phi V_n = P \xrightarrow{solve, P} 80.999999999999986 \cdot kip$

$P_{yield} := 81\ kip \cdot ft$

$V_{max3} \le P_{yield}$

$L_b := L_1 = 4.807\ ft$

Member has enough flexural strength based on unbraced length as seen in Table 3-11 from AISC Manual below.



Stair Tread (A36 3/4" metal plate pan):

$F_y := 36\ ksi$
$F_u := 58\ ksi$

$b := stairway.width = 4.807\ ft$
$d := 11\ in$ $\qquad L := b$
$t := 0.75\ in$

Modeling as simply supported beam

$w := 100\ psf \cdot d = 91.667\ \frac{lbf}{ft}$

$I_x := \frac{1}{12} \cdot d \cdot t^3 \qquad S_x := \frac{d \cdot t^2}{6} = 1.031\ in^3$

$M_{max} := \frac{w \cdot L^2}{8} = 0.265\ kip \cdot ft$

$\phi M_{yield} := 0.9 \cdot F_y \cdot S_x = 2.784\ kip \cdot ft \quad M_{max} \le \phi M_{yield}$

$\Delta_{allowable} := \frac{b}{360} = 0.16\ in \qquad \Delta_{max} := \frac{5 \cdot w \cdot b^4}{384 \cdot E \cdot I_x} = 0.098\ in$

$\Delta_{max} \le \Delta_{allowable}$

Landing Tread (A36 3/4" metal plate pan):

$b := stairway.width = 4.807\ ft$
$d := d_{mid} = 2.917\ ft$ $\qquad L := b$
$t := 0.75\ in$

Modeling as simply supported beam

$w := 100\ psf \cdot d = 291.667\ \frac{lbf}{ft}$

$I_x := \frac{1}{12} \cdot d \cdot t^3 \qquad S_x := \frac{d \cdot t^2}{6} = 3.281\ in^3$

$M_{max} := \frac{w \cdot L^2}{8} = 0.843\ kip \cdot ft$

$\phi M_{yield} := 0.9 \cdot F_y \cdot S_x = 8.859\ kip \cdot ft \quad M_{max} \le \phi M_{yield}$

$\Delta_{max} := \frac{5 \cdot w \cdot b^4}{384 \cdot E \cdot I_x} = 0.098\ in \qquad \Delta_{allowable} := \frac{b}{360} = 0.16\ in$

$\Delta_{max} \le \Delta_{allowable}$

Stringer and Landing Bolts (Dewalt Power-Stud SD1 (3/8"x5.5")):

$V_u := \max\left(V_{max}, V_{max1}, V_{max2}, V_{max3}\right) = 2.482\ kip$

Ultimate and Allowable Load Capacities in Shear for Power-Stud+ SD1 in Grout Filled Concrete Masonry Wall Faces[1,2,3,4,5]

CODE LISTED ICC-ES ESR-2966

Nominal Anchor Diameter in.	Nominal Drill Bit Diameter in.	Min. Embed. Depth in. (mm)	Min. Edge Distance in. (mm)	Min. End Distance in. (mm)	Direction of Loading	Installation Torque T_{inst} ft-lbf (N-m)	Grout-Filled Concrete Masonry: f'm = 1,500 psi: Ultimate Load Shear lbs. (kN)	Grout-Filled Concrete Masonry: f'm = 1,500 psi: Allowable Load Shear lbs. (kN)	Grout-Filled Concrete Masonry: f'm = 2,000 psi: Ultimate Load Shear lbs. (kN)	Grout-Filled Concrete Masonry: f'm = 2,000 psi: Allowable Load Shear lbs. (kN)
3/8	3/8 ANSI	2-3/8 (60.3)	4 (101.6)	4 (101.6)	Perpendicular or parallel to wall edge or end	20 (27)	2,975 (13.4)	595 (2.7)	3,570 (16.1)	715 (3.2)
1/2	1/2 ANSI	2-1/2 (63.5)	4 (101.6)	12 (304.8)	Perpendicular or parallel to wall edge or end	40 (54)	2,800 (12.6)	560 (2.5)	3,360 (15.1)	670 (3.0)
			12 (304.8)	4 (101.6)	Parallel to wall end		4,025 (18.1)	805 (3.6)	4,830 (21.7)	965 (4.3)
			4 (101.6)	12 (304.8)	Parallel to wall edge					
5/8	5/8 ANSI	3-3/8 (85.7)	4 (101.6)	4 (101.6)	Perpendicular or parallel to wall edge or end	50 (68)	3,425 (15.4)	685 (3.1)	4,110 (18.5)	820 (3.7)
			12 (304.8)	4 (101.6)	Parallel to wall end		5,325 (24.0)	1,065 (4.8)	6,390 (28.8)	1,280 (5.8)
			4 (101.6)	12 (304.8)	Parallel to wall edge					
3/4	3/4 ANSI	3-3/8 (85.7)	12 (304.8)	12 (304.8)	Perpendicular or parallel to wall edge or end	80 (108)	8,850 (39.4)	1,770 (7.9)	9,375 (41.7)	1,875 (8.3)
			20 (508.0)	20 (508.0)			10,200 (45.4)	2,040 (9.1)	10,800 (48.0)	2,160 (9.6)
		4-3/4 (120.7)	12 (304.8)	12 (304.8)			12,735 (56.7)	2,545 (11.3)	12,735 (56.7)	2,545 (11.3)

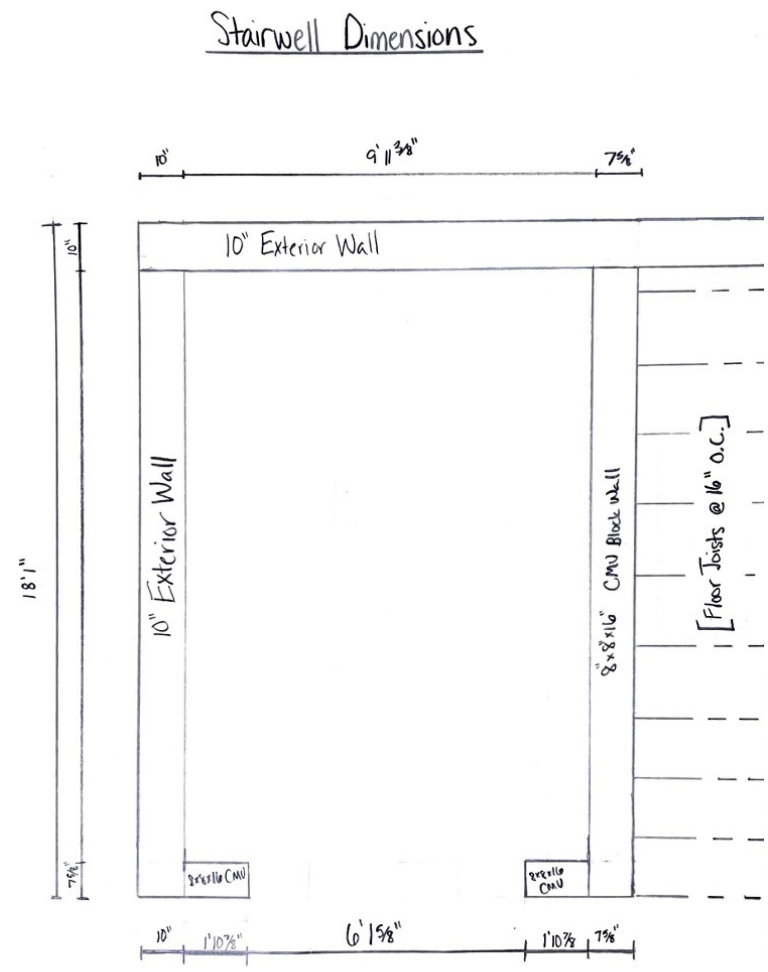
1. Tabulated load values for 3/8", 1/2" and 5/8" diameter anchors are installed in minimum 6" wide, Grade N, Type II, lightweight, medium-weight or normal-weight concrete masonry units conforming to ASTM C 90. Mortar must be minimum Type N. Masonry compressive strength must be at specified minimum at the time of installation.
2. Tabulated load values for 3/4" diameter anchors are installed in minimum 8" wide, Grade N, Type II, lightweight, medium-weight or normal-weight concrete masonry units conforming to ASTM C 90. Mortar must be minimum Type N. Masonry compressive strength must be at specified minimum at the time of installation.
3. Allowable load capacities listed are calculated using an applied safety factor of 5.0.
4. The tabulated values are applicable for anchors installed into grouted masonry wall faces at a critical spacing distance, s_{cr}, between anchors of 16 times the anchor diameter. The spacing distance between two anchors may be reduced to minimum distance, s_{min}, of 8 times the anchor diameter provided the allowable tension loads are multiplied by a reduction factor 0.80 and allowable shear loads are multiplied by a reduction factor of 0.90. Linear interpolation for calculation of allowable loads may be used for intermediate anchor spacing distances.
5. Anchors may be installed in the grouted cells and in cell webs and bed joints not closer than 1-3/8" from head joints. The minimum edge and end distances must also be maintained.

$V_{all} := 595\ kN = 133.761\ kip$

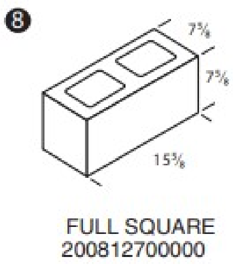
$V_u \le V_{all}$

Source: https://www.buildsite.com/pdf/dewaltanchors/Power-Stud-SD1-Product-Data-2051725.pdf

Stairwell Design



Designing right wall to handle stair and floor joist loads. Using 8"x8"x16" CMU blocks



$b_{CMU} := 7\ in + \frac{5}{8}\ in$

$d_{CMU} := 15\ in + \frac{5}{8}\ in$

$h_{CMU} := 7\ in + \frac{5}{8}\ in$

$w_{CMU} := 115\ pcf$

$f_c' := 2000\ psi$

$L_{WALL} := 18\ ft + 1\ in - 10\ in$

$A_{CMU.WALL} := b_{CMU} \cdot L_{WALL} = 10.961\ ft^2$

$H := (11\ ft + 10\ in) + (14\ ft + 3\ in) + (13\ ft + 4\ in) + (10\ ft + 11\ in) + (8\ ft + 6\ in)$

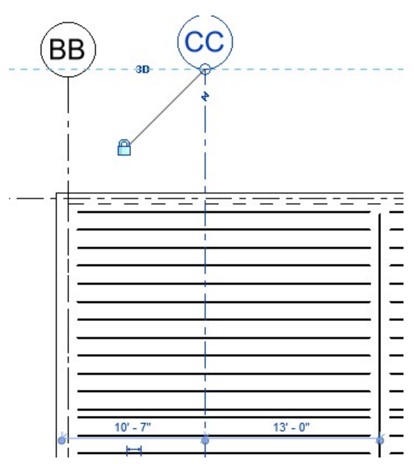
$V_{CMU.WALL} := A_{CMU.WALL} \cdot H = 644.868\ ft^3$

$W_{CMU} := V_{CMU.WALL} \cdot w_{CMU} = 74.16\ kip$

$TW := \frac{13\ ft}{2}$

$TH := L_{WALL} = 17.25\ ft$

$TA := TW \cdot TH = 112.125\ ft^2$



$w_{live} := 100\ psf$
$w_{dead.floor} := 20\ psf$
$w_{dead.roof} := 25\ psf$
$w_{snow} := 16\ psf$

$w_{floor} := 1.2 \cdot (w_{dead.floor}) + 1.6 \cdot (w_{live}) = 184\ psf$
$w_{roof} := 1 \cdot (w_{dead.roof}) + 0.75\ (w_{live}) + 0.75 \cdot (w_{snow}) = 112\ psf$

$w_{joists} := 3 \cdot (w_{floor}) + 1 \cdot (w_{roof}) = 664\ psf$

$W_{JOISTS} := w_{joists} \cdot (TA) = 74.451\ kip$

$W := W_{CMU} + W_{JOISTS} = 148.611\ kip$

$\sigma := \dfrac{W}{A_{CMU.WALL}} = 94.154\ psi$

$f_c{}' = 2000\ psi \qquad \sigma \le f_c{}'$

Soil Properties:

Assumptions:
- water table = 5ft below surface
- angle of friction = 30 deg
- Gs = 2.7
- non-cohesive soil
- Es = 750 tsf

$\gamma_{conc} := 150\ pcf \qquad \gamma_{backfill} := 120\ pcf \qquad \gamma_w := 62.4\ pcf \qquad \gamma_d := 130\ pcf$
$\phi' := 30\ deg \qquad c' := 0\ psf \qquad y_{watertable} := 5\ ft \qquad Es := (750 \cdot 2000)\ psf = 1500000\ psf$

$Gs := 2.7 \qquad e := 0.4 \qquad u_s := 0.3$

$\gamma_{sat} := \dfrac{(Gs + e) \cdot \gamma_w}{1 + e} = 138.171\ pcf \qquad \gamma' := \gamma_{sat} - \gamma_w = 75.771\ pcf$

$D := 5\ ft \qquad D_f := D$

$u := (D - y_{watertable}) \cdot \gamma_w = 0\ psf$
$\sigma'_{zo} := \gamma_d \cdot y_{watertable} + ((D - y_{watertable}) \cdot \gamma') - u = 650\ psf$
$\Delta\sigma'_{ZD} := \gamma_{conc} \cdot D_f = 750\ psf$

$t_f := 16\ in \qquad B := 15\ ft \qquad L := 22\ ft$

Bearing Capacity:

$P := W \qquad A := B \cdot L \qquad FS_q := 3$

$q := (\gamma_{conc} \cdot t_f) + \left(\dfrac{P}{A}\right) = 0.65\ ksf$

$q_{net} := q - \sigma'_{zo} = 0\ ksf$

$Nq := \exp(3.14 \cdot \tan(\phi')) \cdot \left(\tan\left(45\ deg + \dfrac{\phi'}{2}\right)^2\right) = 18.384$

$Nc := \dfrac{Nq - 1}{\tan(\phi')} = 30.11 \qquad i_c := 1$

$i_q := 1$

$Ny := 2 \cdot (Nq + 1) \cdot \tan(\phi') = 22.383 \qquad \dfrac{D_f}{B} = 0.333 \qquad k := \operatorname{atan}\left(\dfrac{D_f}{B}\ rad\right) = 0.322 \qquad i_y := 1$

$d_c := 1 + 0.4\ k = 1.129 \qquad d_y := 1 \qquad d_q := 1 + (2 \cdot k \cdot \tan(\phi')) \cdot (1 - \sin(\phi'))^2 = 1.093$

$s_c := 1 + \left(\dfrac{B}{L}\right) \cdot \left(\dfrac{Nq}{Nc}\right) = 1.416 \qquad s_q := 1 + \left(\dfrac{B}{L}\right) \cdot \tan(\phi') = 1.394 \qquad s_y := 1 - 0.4 \cdot \left(\dfrac{B}{L}\right) = 0.727$

$q_N := c' \cdot Nc \cdot s_c \cdot d_c \cdot i_c + (\Delta\sigma'_{ZD} \cdot Nq \cdot s_q \cdot d_q \cdot i_q) + \left(\dfrac{1}{2} \cdot B \cdot \gamma' \cdot Ny \cdot s_y \cdot d_y \cdot i_y\right) = 30.251\ ksf$

$q_{all} := \dfrac{q_N}{FS_q} = 10.084\ ksf \qquad \dfrac{q_N}{q_{net}} = 90037.162 \qquad FS_q := 3$

Settlement:

$P := W \qquad B' := \dfrac{B}{2} \qquad L' := \dfrac{L}{2} \qquad A_f := B \cdot L \qquad \alpha := 4 \qquad H := 5\ B$

$q := (\gamma_{conc} \cdot t_f) + \left(\dfrac{P}{A_f}\right) = 0.65\ ksf$

$q_{net} := q - \sigma'_{zo} = 0\ ksf \qquad r := 2 \cdot D_f = 10\ ft$

$\beta_1 := 3 - 4 \cdot u_s = 1.8 \qquad \beta_2 := 5 - (12 \cdot u_s) + (8 \cdot u_s{}^2) = 2.12 \qquad \beta_3 := -4 \cdot u_s \cdot (1 - 2\ u_s) = -0.48$

$\beta_4 := -1 + (4 \cdot u_s) - (8\ u_s{}^2) = -0.52 \qquad \beta_5 := -4 \cdot (1 - 2\ u_s)^2 = -0.64$

$r_1 := \sqrt{L^2 + r^2} = 24.166\ ft \qquad r_2 := \sqrt{B^2 + r^2} = 18.028\ ft$

$r_3 := \sqrt{L^2 + B^2 + r^2} = 28.443\ ft \qquad r_4 := \sqrt{L^2 + B^2} = 26.627\ ft$

$Y_1 := L \cdot \ln\left(\dfrac{r_4 + B}{L}\right) + B \cdot \ln\left(\dfrac{r_4 + L}{B}\right) - \dfrac{r_4{}^3 - L^3 - B^3}{3 \cdot L \cdot B} = 26.767\ ft$

$$Y_2 := L \cdot \ln\left(\frac{r_3+B}{r_1}\right) + B \cdot \ln\left(\frac{r_3+L}{r_2}\right) - \frac{r_3^{\,3} - r_2^{\,3} - r_1^{\,3} + r^3}{3 \cdot L \cdot B} = 24.258\ ft$$

$$Y_3 := \frac{r^2}{L} \cdot \ln\left(\frac{(B+r_2) \cdot r_1}{(B+r_3) \cdot r}\right) + \frac{r^2}{B} \cdot \ln\left(\frac{(L+r_1) \cdot r_2}{(L+r_3) \cdot r}\right) = 6.103\ ft$$

$$Y_4 := \frac{r^2 \cdot (r_1 + r_2 - r_3 - r)}{L \cdot B} = 1.137\ ft$$

$$Y_5 := r \cdot \operatorname{atan}\left(\frac{L \cdot B}{r \cdot r_3}\right) = 8.594\ ft$$

$$I_f := \frac{(\beta_1 \cdot Y_1) + (\beta_2 \cdot Y_2) + (\beta_3 \cdot Y_3) + (\beta_4 \cdot Y_4) + (\beta_5 \cdot Y_5)}{(\beta_1 + \beta_2) \cdot Y_1} = 0.863$$

$$M := \frac{L'}{B'} = 1.467 \qquad N := \frac{H}{B'} = 10$$

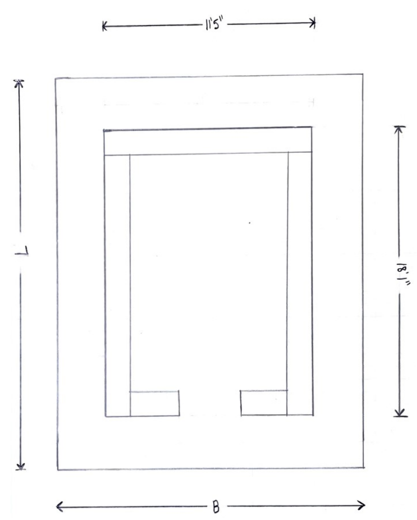
$$I_1 := \frac{1}{3.14} \cdot \left(\left(M \cdot \ln\left(\frac{\left(1+\sqrt{M^2+1}\right) \cdot \left(\sqrt{M^2+N^2}\right)}{M \cdot \left(1+\sqrt{M^2+N^2+1}\right)}\right)\right) + \ln\left(\frac{\left(M+\sqrt{M^2+1}\right) \cdot \left(\sqrt{1+N^2}\right)}{M+\sqrt{M^2+N^2+1}}\right)\right) = 0.58$$

$$I_2 := \frac{N}{2 \cdot 3.14} \cdot \operatorname{atan}\left(\frac{M}{N \cdot \sqrt{M^2+N^2+1}}\right) = 0.023$$

$$I_s := I_1 + \left(\frac{1-2\,u_s}{1-u_s}\right) \cdot I_2 = 0.593$$

$$\delta := \alpha \cdot q_{net} \cdot B' \cdot \left(\frac{1-u_s^{\,2}}{Es}\right) \cdot I_s \cdot I_f \cdot 0.93 = 0\ in \qquad \delta_{all} := 0.25\ in$$

Design for Shear



Section view of Stairwell shaft on foundation

Assume using #4 rebar

$d_{bar} := 0.625\ in \qquad c_{clear} := 3\ in \qquad t_f = 16\ in$

$d_{eff} := t_f - (d_{bar} + c_{clear}) = 12.375\ in$

Area of footing = $\frac{\text{Total load}}{\text{Allowable soil pressure}}$

$A_{req} := \frac{W}{q_{all}} = 14.738\ ft^2$ Need to slightly increase area of footing

$B := 16\ ft \qquad L := 23\ ft$

$X_{shaft} := 11\ ft + 5\ in = 11.417\ ft$

$Y_{shaft} := 18\ ft + 1\ in = 18.083\ ft$

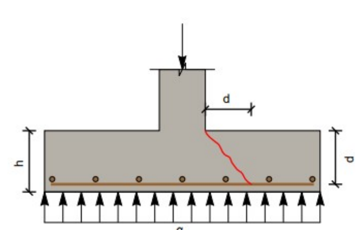
$X_{TA} := \frac{B - X_{shaft}}{2} = 2.292\ ft \qquad check := \frac{B}{X_{TA}} = 6.982$ one way shear

$Y_{TA} := \frac{L - Y_{shaft}}{2} = 2.458\ ft \qquad check := \frac{L}{Y_{TA}} = 9.356$ one way shear

$A := B \cdot L = 368\ ft^2$

$h := t_f$

$q_{upward} := \frac{W}{A} = 403.834\ psf$

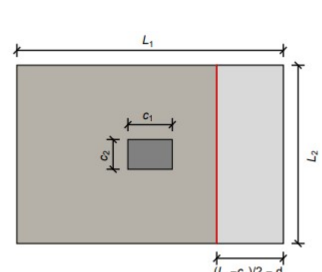


One-way shear

$$V_{u,\text{oneWay}} = q_u \times L_2\left(\frac{L_1 - c_1}{2} - d\right)$$

One-way shear resistance of concrete

$$\phi V_c = 0.75 \times \left(2\,\lambda\,\sqrt{f_c'}\ L_2\,d\right)$$



$V_{u.OneWay} := q_{upward} \cdot L \cdot (X_{TA} - d_{eff}) = 11.707\ kip$

$\lambda := 1 \qquad f_c{}' := 2000\ psi$

$\phi V_c := 0.75 \cdot 2 \cdot \lambda \cdot \sqrt{2000} \cdot psi \cdot B \cdot d_{eff} = 159.387\ kip$

$V_{u.OneWay} \le \phi V_c$ Footing thickness of 16" is adequate

Design for Flexural Reinforcement

$M_u = q_u L_2 \left(\frac{L_1 - c_1}{2}\right)\left(\frac{L_1 - c_1}{4}\right)$

$M_u := q_{upward} \cdot L \cdot X_{TA} \cdot \frac{X_{TA}}{2} = 24.39\ kip \cdot ft$

$A_{s.ruleofthumb} := \left(\frac{\left(\frac{M_u}{kip \cdot ft}\right)}{4 \cdot \left(\frac{d_{eff}}{in}\right)}\right) in^2 = 0.493\ in^2$

$A_{s.min} := 0.0018 \cdot h \cdot L = 7.949\ in^2$

$A_{bar.5} := 0.31\ in^2$

try 26 #5 bars

$A_s := 26 \cdot A_{bar.5} = 8.06\ in^2$ $A_s \ge A_{s.min}$

$f_y := 60\ ksi$

$s_{bar.max} := min(3 \cdot h, 18\ in) = 18\ in$

$s_{bar.provided} := \frac{B}{26} = 7.385\ in$

Bar spacing is less than the maximum allowed thus 26 #5 bars is ok based on flexural strength and shrinkage & temperature

$j := 0.95$

$\phi M_n := 0.9 \cdot A_s \cdot f_y \cdot j \cdot d_{eff} = 426.399\ kip \cdot ft$
$M_u \le \phi M_n$

Rebar amount is adequate for flexure.

Development Length:

For simplicity and to be conservative:
$\psi_t := 1.0$ $\psi_e := 1.0$ $\psi_s := 1.0$ $\lambda := 1.0$ $K_{tr} := 0$ $c_b := 3$ $\alpha_{exs} := 1.0$

$$l_d := \max\left(12\ in, \frac{3}{40} \cdot \alpha_{exs} \cdot \frac{\psi_s \cdot min(\psi_t \cdot \psi_e, 1.7)}{min\left(2.5, \frac{c_b + K_{tr}}{\frac{d_{bar}}{in}}\right)} \cdot \frac{f_y \cdot d_{bar}}{\lambda \cdot min\left(100\ psi, \sqrt{2000}\ psi\right)}\right) = 25.156\ in$$

$X_{TA} = 27.5\ in$ $Y_{TA} = 29.5\ in$

Footing size was increased to 16'x23' to provide enough development length for bars from critical bending section.

Dowel Bars:

Dowel bars are used to transfer load from the column to the footing. If the bearing strength is larger than the column load P_u only the minimum dowel bar area is needed. If the bearing strength is smaller than the column load then the dowel bars must transfer this excess load. Thus area of dowel bars is determined as follows.

Span of wall:
$d_{CMU} := 15\ in + \frac{5}{8}\ in$

$A_1 := (18\ ft + 1\ in - 10\ in) \cdot b_{CMU} = 10.961\ ft^2$

$A_2 := B \cdot L = 368\ ft^2$

$N_1 := 0.65 \cdot (0.85 \cdot f_c{}' \cdot A_1) = 1744.104\ kip$

$N_2 := 0.65 \cdot (0.85 \cdot f_c{}' \cdot A_1) \cdot min\left(2, \sqrt{\frac{A_2}{A_1}}\right) = 3488.209\ kip$

$\phi P_{nb} := min(N_1, N_2) = 1744.104\ kip$

$p := 0.005 \cdot A_1 = 7.892\ in^2$

$A_{s.Dowel.min} := p = 7.892\ in^2$

Use 6#3 dowel bars in each block of CMU wall to footing at 90 degree hooks

$A_{bar.3} := 0.11\ in^2$

$$Number.blocks := \frac{(18\ ft + 1\ in - 10\ in)}{d_{CMU}} = 13.248 \qquad Number.blocks := 13$$

$A_{s.Dowel} := 6 \cdot Number.blocks \cdot A_{bar.3} = 8.58\ in^2$

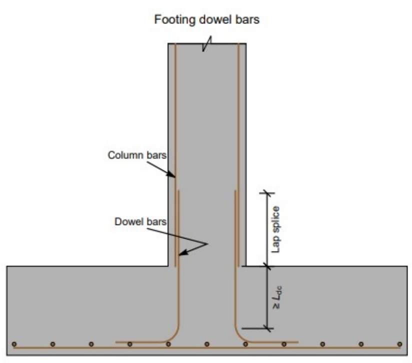
Development length for bars in compression

$$\ell_{dc} = \mathrm{Max}\left[8\ \mathrm{in},\ \frac{0.02\ f_y\ d_b}{\lambda\ \mathrm{Min}[100,\ \sqrt{f_c'}]},\ 0.0003\ f_y\ d_b\right]$$

$d_{bar} := 0.375\ in$

$$l_{dc} := \max\left(8\ in, \frac{0.02 \cdot f_y \cdot d_{bar}}{\lambda \cdot min\left(100\ psi, \sqrt{2000}\ psi\right)}, \frac{0.0003}{psi} \cdot f_y \cdot d_{bar}\right) = 10.062\ in$$

$d_{eff} = 12.375\ in \qquad d_{eff} \ge l_{dc}$



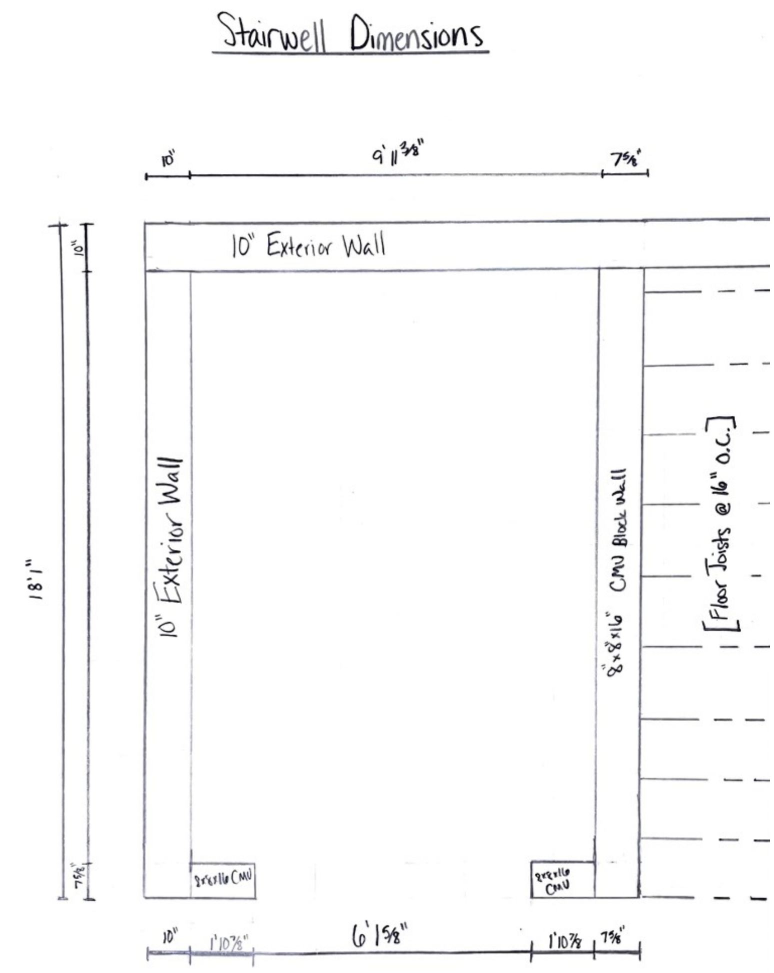
The restraint here is that Ldc was too large for our footing depth when using larger dowel bars. We have elected to use smaller bars more often to keep the footing depth at 16in.

$\alpha_s := 1$

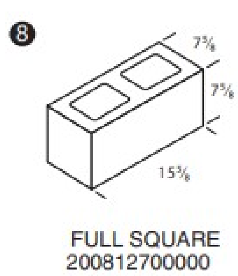
$$l_{splice} := \max\left(12\ in, l_{dc}, \frac{0.0005}{psi} \cdot f_y \cdot d_{bar} \cdot \alpha_s\right) = 12\ in$$

Dowel bars must extend into CMU wall 12" from top of footing.

Appendix F: Building A: Elevator Foundation Design



Designing right wall to handle stair and floor joist loads. Using 8"x8"x16" CMU blocks



$$b_{CMU} := 7\ in + \frac{5}{8}\ in$$

$$d_{CMU} := 15\ in + \frac{5}{8}\ in$$

$$h_{CMU} := 7\ in + \frac{5}{8}\ in$$

$$w_{CMU} := 115\ pcf$$

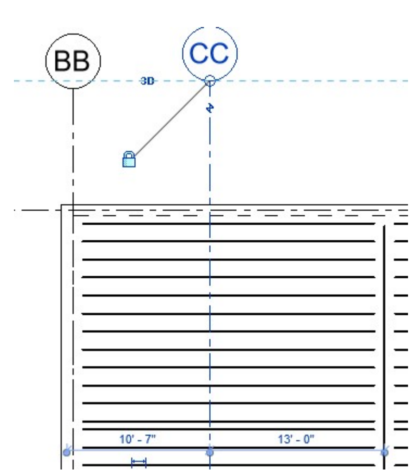
$$f_c' := 2000\ psi$$

$$L_{WALL} := 18\ ft + 1\ in - 10\ in$$

$$A_{CMU.WALL} := b_{CMU} \cdot L_{WALL} = 10.961\ ft^2$$

$$H := (11\ ft + 10\ in) + (14\ ft + 3\ in) + (13\ ft + 4\ in) + (10\ ft + 11\ in) + (8\ ft + 6\ in)$$
$$V_{CMU.WALL} := A_{CMU.WALL} \cdot H = 644.868\ ft^3$$

$$W_{CMU} := V_{CMU.WALL} \cdot w_{CMU} = 74.16\ kip$$



$$TW := \frac{13\ ft}{2}$$

$$TH := L_{WALL} = 17.25\ ft$$

$$TA := TW \cdot TH = 112.125\ ft^2$$

$$w_{live} := 100\ psf$$
$$w_{dead.floor} := 20\ psf$$
$$w_{dead.roof} := 25\ psf$$
$$w_{snow} := 16\ psf$$

$$w_{floor} := 1.2 \cdot (w_{dead.floor}) + 1.6 \cdot (w_{live}) = 184\ psf$$
$$w_{roof} := 1 \cdot (w_{dead.roof}) + 0.75\ (w_{live}) + 0.75 \cdot (w_{snow}) = 112\ psf$$

$$w_{joists} := 3 \cdot (w_{floor}) + 1 \cdot (w_{roof}) = 664\ psf$$

$$W_{JOISTS} := w_{joists} \cdot (TA) = 74.451\ kip$$

$$W := W_{CMU} + W_{JOISTS} = 148.611\ kip$$

$$\sigma := \frac{W}{A_{CMU.WALL}} = 94.154\ psi$$

$$f_c' = 2000\ psi \qquad \sigma \le f_c'$$

Soil Properties:
Assumptions:
- water table = 5ft below surface
- angle of friction = 30 deg
- Gs = 2.7
- non-cohesive soil
- Es = 750 tsf

$\gamma_{conc} := 150\ pcf \quad \gamma_{backfill} := 120\ pcf \quad \gamma_w := 62.4\ pcf \quad \gamma_d := 130\ pcf$
$\phi' := 30\ deg \quad c' := 0\ psf \quad y_{watertable} := 5\ ft \quad Es := (750 \cdot 2000)\ psf = 1500000\ psf$

$Gs := 2.7 \quad e := 0.4 \quad u_s := 0.3$

$\gamma_{sat} := \dfrac{(Gs + e) \cdot \gamma_w}{1 + e} = 138.171\ pcf \qquad \gamma' := \gamma_{sat} - \gamma_w = 75.771\ pcf$

$D := 5\ ft \qquad D_f := D$

$u := (D - y_{watertable}) \cdot \gamma_w = 0\ psf$
$\sigma'_{zo} := \gamma_d \cdot y_{watertable} + ((D - y_{watertable}) \cdot \gamma') - u = 650\ psf$
$\Delta\sigma'_{ZD} := \gamma_{conc} \cdot D_f = 750\ psf$

$t_f := 16\ in \quad B := 15\ ft \quad L := 22\ ft$

Bearing Capacity:

$P := W \quad A := B \cdot L \quad FS_q := 3$

$q := (\gamma_{conc} \cdot t_f) + \left(\dfrac{P}{A}\right) = 0.65\ ksf$

$q_{net} := q - \sigma'_{zo} = 0\ ksf$

$Nq := \exp(3.14 \cdot \tan(\phi')) \cdot \left(\tan\left(45\ deg + \dfrac{\phi'}{2}\right)^2\right) = 18.384$

$Nc := \dfrac{Nq - 1}{\tan(\phi')} = 30.11 \qquad i_c := 1$

$i_q := 1$

$Ny := 2 \cdot (Nq + 1) \cdot \tan(\phi') = 22.383 \quad \dfrac{D_f}{B} = 0.333 \quad k := \operatorname{atan}\left(\dfrac{D_f}{B}\ rad\right) = 0.322 \quad i_y := 1$

$d_c := 1 + 0.4\ k = 1.129 \quad d_y := 1 \quad d_q := 1 + (2 \cdot k \cdot \tan(\phi')) \cdot (1 - \sin(\phi'))^2 = 1.093$

$s_c := 1 + \left(\dfrac{B}{L}\right) \cdot \left(\dfrac{Nq}{Nc}\right) = 1.416 \quad s_q := 1 + \left(\dfrac{B}{L}\right) \cdot \tan(\phi') = 1.394 \quad s_y := 1 - 0.4 \cdot \left(\dfrac{B}{L}\right) = 0.727$

$q_N := c' \cdot Nc \cdot s_c \cdot d_c \cdot i_c + (\Delta\sigma'_{ZD} \cdot Nq \cdot s_q \cdot d_q \cdot i_q) + \left(\dfrac{1}{2} \cdot B \cdot \gamma' \cdot Ny \cdot s_y \cdot d_y \cdot i_y\right) = 30.251\ ksf$

$q_{all} := \dfrac{q_N}{FS_q} = 10.084\ ksf \quad \dfrac{q_N}{q_{net}} = 90037.162 \quad FS_q := 3$

Settlement:

$P := W \quad B' := \dfrac{B}{2} \quad L' := \dfrac{L}{2} \quad A_f := B \cdot L \quad \alpha := 4 \quad H := 5\ B$

$q := (\gamma_{conc} \cdot t_f) + \left(\dfrac{P}{A_f}\right) = 0.65\ ksf$

$q_{net} := q - \sigma'_{zo} = 0\ ksf \qquad r := 2 \cdot D_f = 10\ ft$

$\beta_1 := 3 - 4 \cdot u_s = 1.8 \quad \beta_2 := 5 - (12 \cdot u_s) + (8 \cdot u_s^2) = 2.12 \quad \beta_3 := -4 \cdot u_s \cdot (1 - 2\ u_s) = -0.48$

$\beta_4 := -1 + (4 \cdot u_s) - (8\ u_s^2) = -0.52 \quad \beta_5 := -4 \cdot (1 - 2\ u_s)^2 = -0.64$

$r_1 := \sqrt{L^2 + r^2} = 24.166\ ft \qquad r_2 := \sqrt{B^2 + r^2} = 18.028\ ft$

$r_3 := \sqrt{L^2 + B^2 + r^2} = 28.443\ ft \qquad r_4 := \sqrt{L^2 + B^2} = 26.627\ ft$

$Y_1 := L \cdot \ln\left(\dfrac{r_4 + B}{L}\right) + B \cdot \ln\left(\dfrac{r_4 + L}{B}\right) - \dfrac{r_4^3 - L^3 - B^3}{3 \cdot L \cdot B} = 26.767\ ft$

$Y_2 := L \cdot \ln\left(\dfrac{r_3 + B}{r_1}\right) + B \cdot \ln\left(\dfrac{r_3 + L}{r_2}\right) - \dfrac{r_3^3 - r_2^3 - r_1^3 + r^3}{3 \cdot L \cdot B} = 24.258\ ft$

$Y_3 := \dfrac{r^2}{L} \cdot \ln\left(\dfrac{(B + r_2) \cdot r_1}{(B + r_3) \cdot r}\right) + \dfrac{r^2}{B} \cdot \ln\left(\dfrac{(L + r_1) \cdot r_2}{(L + r_3) \cdot r}\right) = 6.103\ ft$

$Y_4 := \dfrac{r^2 \cdot (r_1 + r_2 - r_3 - r)}{L \cdot B} = 1.137\ ft$

$Y_5 := r \cdot \operatorname{atan}\left(\dfrac{L \cdot B}{r \cdot r_3}\right) = 8.594\ ft$

$I_f := \dfrac{(\beta_1 \cdot Y_1) + (\beta_2 \cdot Y_2) + (\beta_3 \cdot Y_3) + (\beta_4 \cdot Y_4) + (\beta_5 \cdot Y_5)}{(\beta_1 + \beta_2) \cdot Y_1} = 0.863$

$M := \dfrac{L'}{B'} = 1.467 \qquad N := \dfrac{H}{B'} = 10$

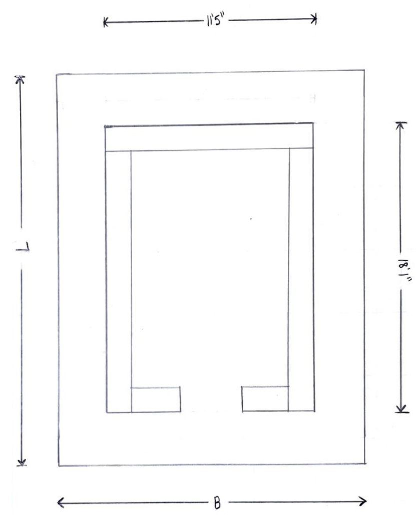
$I_1 := \dfrac{1}{3.14} \cdot \left(\left(M \cdot \ln\left(\dfrac{\left(1 + \sqrt{M^2 + 1}\right) \cdot \left(\sqrt{M^2 + N^2}\right)}{M \cdot \left(1 + \sqrt{M^2 + N^2 + 1}\right)}\right)\right) + \ln\left(\dfrac{\left(M + \sqrt{M^2 + 1}\right) \cdot \left(\sqrt{1 + N^2}\right)}{M + \sqrt{M^2 + N^2 + 1}}\right)\right) = 0.58$

$I_2 := \dfrac{N}{2 \cdot 3.14} \cdot \operatorname{atan}\left(\dfrac{M}{N \cdot \sqrt{M^2 + N^2 + 1}}\right) = 0.023$

$I_s := I_1 + \left(\dfrac{1 - 2\ u_s}{1 - u_s}\right) \cdot I_2 = 0.593$

$$\delta := \alpha \cdot q_{net} \cdot B' \cdot \left(\frac{1-u_s^2}{Es}\right) \cdot I_s \cdot I_f \cdot 0.93 = 0\ in \qquad \delta_{all} := 0.25\ in$$

Design for Shear



Section view of Stairwell shaft on foundation

Assume using #4 rebar

$$d_{bar} := 0.625\ in \qquad c_{clear} := 3\ in \qquad t_f = 16\ in$$

$$d_{eff} := t_f - (d_{bar} + c_{clear}) = 12.375\ in$$

$$\text{Area of footing} = \frac{\text{Total load}}{\text{Allowable soil pressure}}$$

$$A_{req} := \frac{W}{q_{all}} = 14.738\ ft^2$$ Need to slightly increase area of footing

$$B := 16\ ft \qquad L := 23\ ft$$

$$X_{shaft} := 11\ ft + 5\ in = 11.417\ ft$$

$$Y_{shaft} := 18\ ft + 1\ in = 18.083\ ft$$

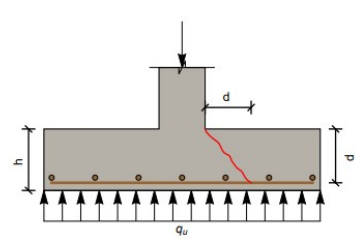
$$X_{TA} := \frac{B - X_{shaft}}{2} = 2.292\ ft \qquad check := \frac{B}{X_{TA}} = 6.982$$ one way shear

$$Y_{TA} := \frac{L - Y_{shaft}}{2} = 2.458\ ft \qquad check := \frac{L}{Y_{TA}} = 9.356$$ one way shear

$$A := B \cdot L = 368\ ft^2$$

$$h := t_f$$

$$q_{upward} := \frac{W}{A} = 403.834\ psf$$

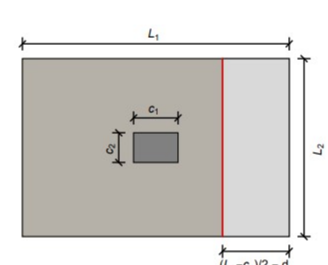


One-way shear

$$V_{u,\text{oneWay}} = q_u \times L_2\left(\frac{L_1 - c_1}{2} - d\right)$$

One-way shear resistance of concrete

$$\phi V_c = 0.75 \times \left(2\lambda \sqrt{f_c'}\ L_2\, d\right)$$



$$V_{u.OneWay} := q_{upward} \cdot L \cdot (X_{TA} - d_{eff}) = 11.707\ kip$$

$$\lambda := 1 \qquad f_c' := 2000\ psi$$

$$\phi V_c := 0.75 \cdot 2 \cdot \lambda \cdot \sqrt{2000} \cdot psi \cdot B \cdot d_{eff} = 159.387\ kip$$

$$V_{u.OneWay} \le \phi V_c$$ Footing thickness of 16" is adequate

Design for Flexural Reinforcement

$$M_u = q_u\, L_2 \left(\frac{L_1 - c_1}{2}\right)\left(\frac{L_1 - c_1}{4}\right)$$

$$M_u := q_{upward} \cdot L \cdot X_{TA} \cdot \frac{X_{TA}}{2} = 24.39\ kip \cdot ft$$

$$A_{s.ruleofthumb} := \left(\frac{\left(\frac{M_u}{kip \cdot ft}\right)}{4 \cdot \left(\frac{d_{eff}}{in}\right)}\right) in^2 = 0.493\ in^2$$

$$A_{s.min} := 0.0018 \cdot h \cdot L = 7.949\ in^2$$

$$A_{bar.5} := 0.31\ in^2$$

try 26 #5 bars

$A_s := 26 \cdot A_{bar.5} = 8.06\ in^2 \qquad A_s \geq A_{s.min}$

$f_y := 60\ ksi$

$s_{bar.max} := min(3 \cdot h, 18\ in) = 18\ in$

$s_{bar.provided} := \dfrac{B}{26} = 7.385\ in$

Bar spacing is less than the maximum allowed thus 26 #5 bars is ok based on flexural strength and shrinkage & temperature

$j := 0.95$

$\phi M_n := 0.9 \cdot A_s \cdot f_y \cdot j \cdot d_{eff} = 426.399\ kip \cdot ft$

$M_u \leq \phi M_n$

Rebar amount is adequate for flexure.

Development Length:

For simplicity and to be conservative:

$\psi_t := 1.0 \quad \psi_e := 1.0 \quad \psi_s := 1.0 \quad \lambda := 1.0 \quad K_{tr} := 0 \quad c_b := 3 \quad \alpha_{exs} := 1.0$

$$l_d := \max\left(12\ in, \frac{3}{40} \cdot \alpha_{exs} \cdot \frac{\psi_s \cdot min(\psi_t \cdot \psi_e, 1.7)}{min\left(2.5, \dfrac{c_b + K_{tr}}{\dfrac{d_{bar}}{in}}\right)} \cdot \frac{f_y \cdot d_{bar}}{\lambda \cdot min\left(100\ psi, \sqrt{2000}\ psi\right)}\right) = 25.156\ in$$

$X_{TA} = 27.5\ in \qquad Y_{TA} = 29.5\ in$

Footing size was increased to 16'x23' to provide enough development length for bars from critical bending section.

Dowel Bars:

Dowel bars are used to transfer load from the column to the footing. If the bearing strength is larger than the column load P_u only the minimum dowel bar area is needed. If the bearing strength is smaller than the column load then the dowel bars must transfer this excess load. Thus area of dowel bars is determined as follows.

Span of wall:

$d_{CMU} := 15\ in + \dfrac{5}{8}\ in$

$A_1 := (18\ ft + 1\ in - 10\ in) \cdot b_{CMU} = 10.961\ ft^2$

$A_2 := B \cdot L = 368\ ft^2$

$N_1 := 0.65 \cdot (0.85 \cdot f_c' \cdot A_1) = 1744.104\ kip$

$N_2 := 0.65 \cdot (0.85 \cdot f_c' \cdot A_1) \cdot min\left(2, \sqrt{\dfrac{A_2}{A_1}}\right) = 3488.209\ kip$

$\phi P_{nb} := min(N_1, N_2) = 1744.104\ kip$

$p := 0.005 \cdot A_1 = 7.892\ in^2$

$A_{s.Dowel.min} := p = 7.892\ in^2$

Use 6#3 dowel bars in each block of CMU wall to footing at 90 degree hooks

$A_{bar.3} := 0.11\ in^2$

$Number.blocks := \dfrac{(18\ ft + 1\ in - 10\ in)}{d_{CMU}} = 13.248 \qquad Number.blocks := 13$

$A_{s.Dowel} := 6 \cdot Number.blocks \cdot A_{bar.3} = 8.58\ in^2$

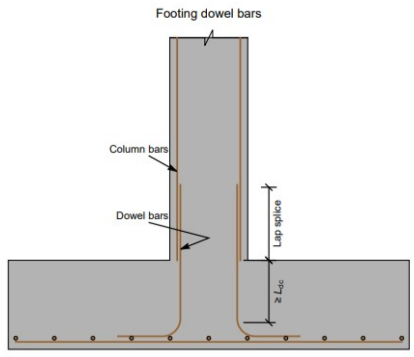
Development length for bars in compression

$$\ell_{dc} = \text{Max}\left[8 \text{ in}, \frac{0.02\ f_y\ d_b}{\lambda\ \text{Min}[100, \sqrt{f_c'}]}, 0.0003\ f_y\ d_b\right]$$

$d_{bar} := 0.375\ in$

$$l_{dc} := \max\left(8\ in, \frac{0.02 \cdot f_y \cdot d_{bar}}{\lambda \cdot min\left(100\ psi, \sqrt{2000}\ psi\right)}, \frac{0.0003}{psi} \cdot f_y \cdot d_{bar}\right) = 10.062\ in$$

$d_{eff} = 12.375\ in \qquad d_{eff} \geq l_{dc}$



The restraint here is that Ldc was too large for our footing depth when using larger dowel bars. We have elected to use smaller bars more often to keep the footing depth at 16in.

$\alpha_s := 1$

$l_{splice} := \max\left(12\ in, l_{dc}, \frac{0.0005}{psi} \cdot f_y \cdot d_{bar} \cdot \alpha_s\right) = 12\ in$

Dowel bars must extend into CMU wall 12" from top of footing.

Appendix G: Building C: Selection of Loading

DEAD LOADS:

Roof:

$MEP := 6\ psf$

$Metal\_Deck := 3\ psf$

$Plywood\_Sheathing := 2\ psf$ (1/2 in)

$Rigid\_Insulation := 9\ psf$ (XPS 6in R30 1.5psf/in thick)

$Waterproofing\_membrane := 0.7\ psf$

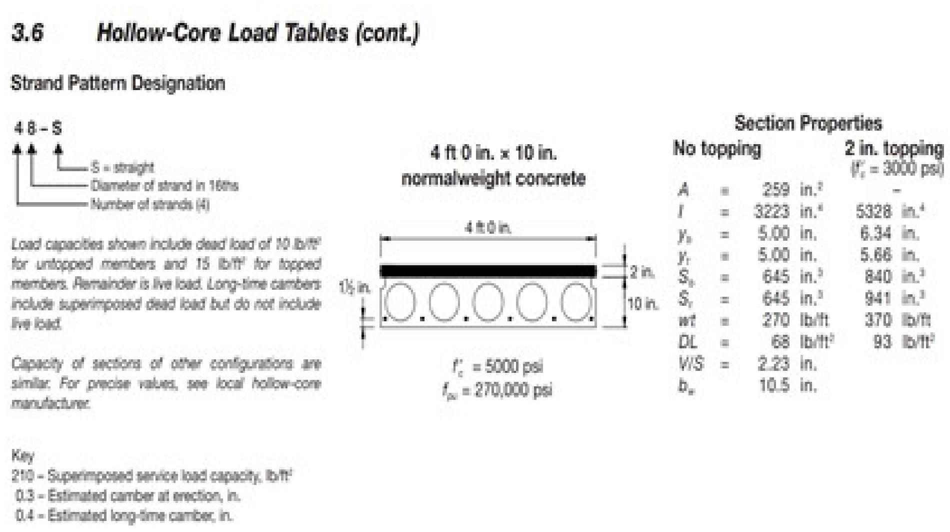
Extruded polystyrene (XPS)

R-VALUE: R-5 per in.
COST: 47¢ per sq. ft. at 1-in. thickness (material only)
APPLICATION: Under slabs; below-grade walls; above-grade walls; ceilings; and roofs

$$Roof_{DL\_Total} := MEP + Metal\_Deck + Plywood\_Sheathing + Rigid\_Insulation + Waterproofing\_membrane = 20.7\ psf$$

Slab above Basement:

$Slab := 93\ psf$ Hollow core Load Tables includes 2 in topping.



3.6 Hollow-Core Load Tables (cont.)

Strand Pattern Designation

4 8 – S
S = straight
Diameter of strand in 16ths
Number of strands (4)

Load capacities shown include dead load of 10 lb/ft² for untopped members and 15 lb/ft² for topped members. Remainder is live load. Long-time cambers include superimposed dead load but do not include live load.

Capacity of sections of other configurations are similar. For precise values, see local hollow-core manufacturer.

Key
210 – Superimposed service load capacity, lb/ft²
0.3 – Estimated camber at erection, in.
0.4 – Estimated long-time camber, in.

Section Properties

		No topping	2 in. topping (f'_c = 3000 psi)
A	=	259 in.²	–
I	=	3223 in.⁴	5328 in.⁴
y_b	=	5.00 in.	6.34 in.
y_t	=	5.00 in.	5.66 in.
S_b	=	645 in.³	840 in.³
S_t	=	645 in.³	941 in.³
wt	=	270 lb/ft	370 lb/ft
DL	=	68 lb/ft²	93 lb/ft²
V/S	=	2.23 in.	
b_w	=	10.5 in.	

$Tile\_Flooring := 16\ psf$ (3/4 in ceramic tile on 1/2 inch mortar bed)
$MEP := 6\ psf$
$Total := Slab + MEP + Tile\_Flooring = 115\ psf$

Live Loads:

Uniform Distributed Live Load for Restaurant:

$L_0 := 100\ psf$

Uniform Distributed Live Load for Restaurant Roof:

$L_0 := 20\ psf$

Appendix H: Building C: Snow Loading

Step 1. Select Risk Category:

Risk Category = I (Very low risk to human life)

Risk Category = II (Not a substantial risk to human life)

(Table 1.5-1)

Risk Category = III (Failure could pose a substantial risk to human life)

Risk Category = IV (Buildings and other structures designated as essential facilities)

$risk := \text{“II”}$

$$I_s := \begin{array}{l} \text{if } risk = \text{“I”} \\ \quad \| \; 0.80 \\ \text{else if } risk = \text{“II”} \\ \quad \| \; 1.00 \\ \text{else if } risk = \text{“III”} \\ \quad \| \; 1.10 \\ \text{else} \\ \quad \| \; 1.20 \end{array}$$

(Table 1.5-2)

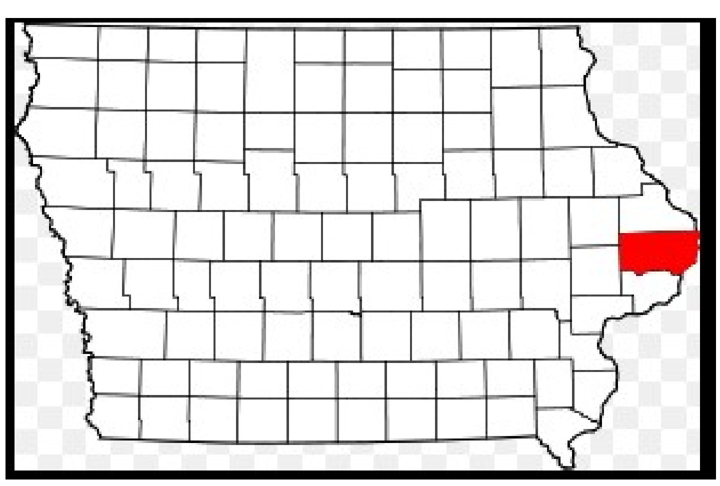
$I_s = 1$

Step 2. Select Ground Snow Load:

This depends on the location of the structure you are designing. See the map below:

$P_g := 25 \; psf$ (Clinton, IA)

Location of Clinton Iowa



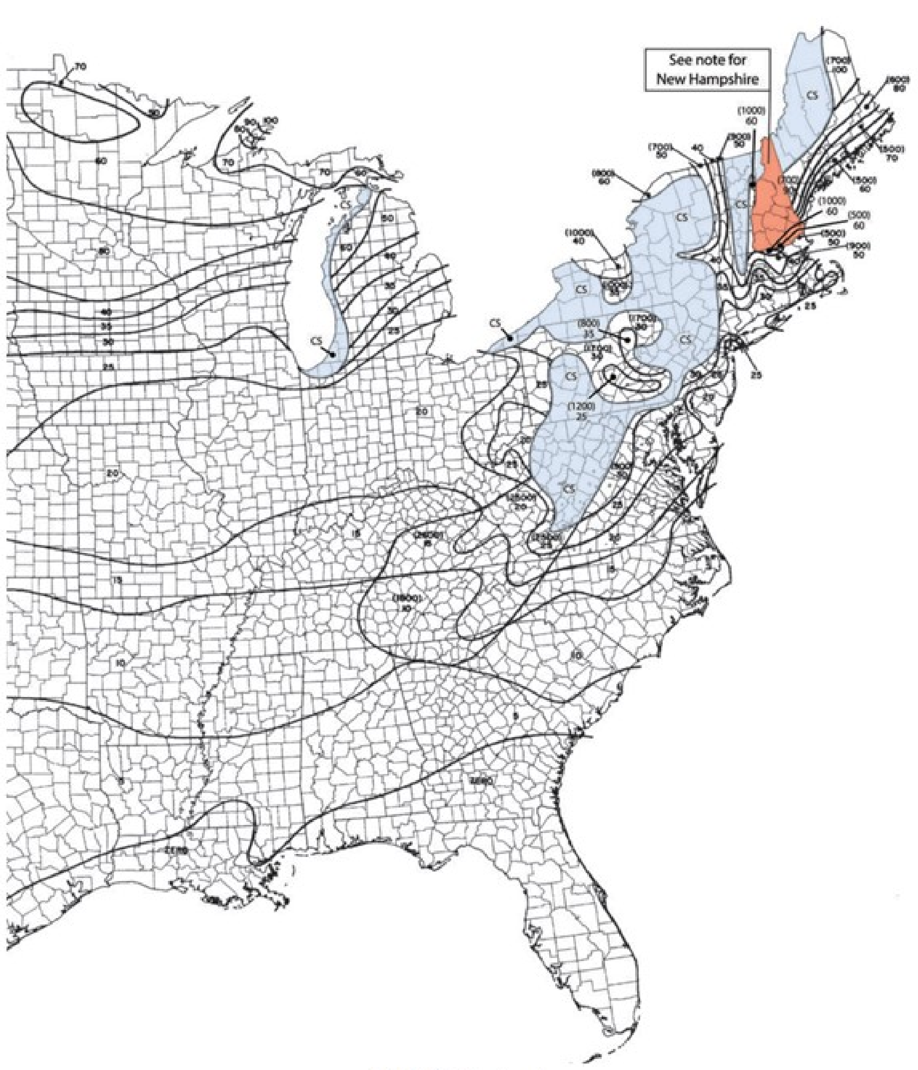


FIGURE 7.2-1 (*Continued*)

Step 3. Determine Roof Snow Load Factors:

First, choose a surface roughness

26.7.2 Surface Roughness Categories. A ground surface roughness within each 45° sector shall be determined for a distance upwind of the site, as defined in Section 26.7.3, from the categories defined in the following text, for the purpose of assigning an exposure category as defined in Section 26.7.3.

Surface Roughness B: Urban and suburban areas, wooded areas, or other terrain with numerous, closely spaced obstructions that have the size of single-family dwellings or larger.

Surface Roughness C: Open terrain with scattered obstructions that have heights generally less than 30 ft (9.1 m). This category includes flat, open country and grasslands.

Surface Roughness D: Flat, unobstructed areas and water surfaces. This category includes smooth mud flats, salt flats, and unbroken ice.

$$SurfaceRoughness := \text{“C”}$$

Next, choose an exposure category for the roof:

$$Exposure := \text{“FullyExposed”}$$

Define Ce:

Table 7.3-1 Exposure Factor, C_e

Surface Roughness Category	Exposure of Roof[a] Fully Exposed	Partially Exposed	Sheltered
B (see Section 26.7)	0.9	1.0	1.2
C (see Section 26.7)	0.9	1.0	1.1
D (see Section 26.7)	0.8	0.9	1.0
Above the tree line in windswept mountainous areas	0.7	0.8	NA
In Alaska, in areas where trees do not exist within a 2-mi (3-km) radius of the site	0.7	0.8	NA

$$C_e := 0.9$$

Define Ct:

Table 7.3-2 Thermal Factor, C_t

Thermal Condition[a]	C_t
All structures except as indicated below	1.0
Structures kept just above freezing and others with cold, ventilated roofs in which the thermal resistance (R-value) between the ventilated space and the heated space exceeds 25°F × h × ft^2/Btu (4.4 K × m^2/W)	1.1
Unheated and open air structures	1.2
Freezer building	1.3
Continuously heated greenhouses[b] with a roof having a thermal resistance (R-value) less than 2.0°F × h × ft^2/Btu (0.4 K × m^2/W)	0.85

$$C_t := 1.0$$

Step 4. Calculate Balanced Snow Load:

$$P_f := 0.7 \cdot C_e \cdot C_t \cdot I_s \cdot P_g = 15.75 \ psf$$ (Flat Roof Snow Load)

Appendix I: Building C: Wind Loading Main Wind Force Resisting System – Directional Procedure

Step 1. Select Risk Category:

Risk Category = II (Table 1.5-1)

$I_w := 1.0$ (Table 1.5-2)

Step 2. Select Basic Wind Speed:

$V := 107\ mph$ (Figure 26.5-1B)

Step 3. Determine k_d:

$K_d := 0.85$ (Table 26.6-1 > Buildings > Main Wind Force Resisting System

Step 4. Determine Exposure Category:

$surfaceRoughness := \text{“C”}$

$exposure := \text{“C”}$

Step 5. Determine Topographic Factor:

$K_{zt} := 1.00$ (Conservative assumption)

Step 6. Determine K_e:

$z_{ground} := 600\ ft$ (Average elevation in Clinton IA)

$$K_e := \exp\left(-0.0000362 \cdot \frac{z_{ground}}{ft}\right)$$

$$K_e = 0.979$$

Step 7. Calculate Gust Factor, G:

$G := 0.85$ (Default factor for rigid buildings)

Step 8. Determine Enclosure Classification

$enclosure := \text{“partially enclosed”}$

Step 9. Determine Internal Pressure Coefficient

$$GC_{pi} := \begin{aligned} &\text{if } enclosure = \text{“enclosed”} \\ &\quad \| \; 0.18 \\ &\text{else if } enclosure = \text{“partially enclosed”} \\ &\quad \| \; 0.55 \\ &\text{else if } enclosure = \text{“partially open”} \\ &\quad \| \; 0.18 \\ &\text{else} \\ &\quad \| \; 0 \end{aligned}$$

$GC_{pi} = 0.55$ + or -

Step 10. Define Variables:

$z_{15} := 15\ ft$ $z_{18} := 18\ ft$ $z_p := 22\ ft$

$h := z_{18}$ (Roof Height)

$B := 96\ ft$ (Building Width -- perpendicular to wind direction)

$L := 86\ ft$ (Building Length -- parallel to wind direction)

Step 11. Calculate K_z:

$$\alpha := \begin{array}{l} \text{if } exposure = \text{"B"} \\ \quad \| 7.0 \\ \text{else if } exposure = \text{"C"} \\ \quad \| 9.5 \\ \text{else} \\ \quad \| 11.5 \end{array}$$

(Table 26.11-1)

$\alpha = 9.5$

$$z_g := \begin{array}{l} \text{if } exposure = \text{"B"} \\ \quad \| 1200\ ft \\ \text{else if } exposure = \text{"C"} \\ \quad \| 900\ ft \\ \text{else} \\ \quad \| 700\ ft \end{array}$$

(Table 26.11-1)

$z_g = 900\ ft$

$$K_{15} := 2.01 \cdot \left(\frac{z_{15}}{z_g}\right)^{\frac{2}{\alpha}} = 0.849$$

$$K_{18} := 2.01 \cdot \left(\frac{z_{18}}{z_g}\right)^{\frac{2}{\alpha}} = 0.882$$

$$K_p := 2.01 \cdot \left(\frac{z_p}{z_g}\right)^{\frac{2}{\alpha}} = 0.92$$

Step 12. Calculate q_z:

q_t = (0.00256 Kz Kzt Kd Ke v^2)

$$q_{15} := \left(\frac{0.00256\ psf}{mph^2}\right) \cdot K_{15} \cdot K_{zt} \cdot K_d \cdot K_e \cdot V^2 = 20.694\ psf$$

$$q_{18} := \left(\frac{0.00256\ psf}{mph^2}\right) \cdot K_{18} \cdot K_{zt} \cdot K_d \cdot K_e \cdot V^2 = 21.504\ psf$$

$$q_p := \left(\frac{0.00256\ psf}{mph^2}\right) \cdot K_p \cdot K_{zt} \cdot K_d \cdot K_e \cdot V^2 = 22.432\ psf$$

$$q_h := q_{18} = 21.504\ psf$$

Step 13. Determine External Pressure Coefficients

Test L/B: $\frac{L}{B} = 0.896$

Windward wall: $C_{p\_windward} := 0.8$ (Figure 27.3-1)

Leeward wall: $C_{p\_leeward} := -0.5$ (Figure 27.3-1<L/B<1)

Windward parapet: $GC_{pn\_windward} := 1.5$ (Section 27.3.4)

Leeward parapet: $GC_{pn\_leeward} := -1.0$ (Section 27.3.4)

Step 13. Calculate Design Wind Pressures:

Positive Internal Pressure:

Windward Wall:

$$p_{15\_pos} := q_{15} \cdot G \cdot C_{p\_windward} - q_h \cdot \left(GC_{pi}\right) = 2.245 \; psf$$

$$p_{18\_pos} := q_{18} \cdot G \cdot C_{p\_windward} - q_h \cdot \left(GC_{pi}\right) = 2.795 \; psf$$

Leeward Wall:

$$p_{leeward\_pos} := q_h \cdot G \cdot C_{p\_leeward} - q_h \cdot \left(GC_{pi}\right) = -20.966 \; psf$$

Net Pressure:

$$p_{15\_netPos} := p_{15\_pos} - p_{leeward\_pos} = 23.211 \; psf$$

$$p_{18\_netPos} := p_{18\_pos} - p_{leeward\_pos} = 23.762 \; psf$$

Negative Internal Pressure:

Windward Wall:

$$p_{15\_neg} := q_{15} \cdot G \cdot C_{p\_windward} - q_h \cdot \left(-GC_{pi}\right) = 25.899 \; psf$$

$$p_{18\_neg} := q_{18} \cdot G \cdot C_{p\_windward} - q_h \cdot \left(-GC_{pi}\right) = 26.449 \; psf$$

Leeward Wall:

$$p_{leeward\_neg} := q_h \cdot G \cdot C_{p\_leeward} - q_h \cdot \left(-GC_{pi}\right) = 2.688 \; psf$$

Net Pressure:

$$p_{15\_netNeg} := p_{15\_neg} - p_{leeward\_neg} = 23.211 \; psf$$

$$p_{18\_netNeg} := p_{18\_neg} - p_{leeward\_neg} = 23.762 \; psf$$

Calculate Parapet Design Pressures:

Windward Wall:

$$p_{p\_windward} := q_p \cdot GC_{pn\_windward} = 33.647 \; psf$$

Leeward Wall:

$$p_{p\_leeward} := q_p \cdot GC_{pn\_leeward} = -22.432 \; psf$$

Net Parapet Pressure:

$$p_{p\_net} := p_{p\_windward} - p_{p\_leeward} = 56.079 \; psf$$

Appendix J: Building C: Wind Loading Main Wind Force Resisting System – Serviceability Procedure

Step 1. Select Risk Category:

Risk Category = II (Table 1.5-1)

$I_w := 1.0$ (Table 1.5-2)

Step 2. Select Basic Wind Speed:

$V := 82\ mph$ (Figure 26.5-1B)

Step 3. Determine k_d:

$K_d := 0.85$ (Table 26.6-1 > Buildings > Main Wind Force Resisting System

Step 4. Determine Exposure Category:

$surfaceRoughness := \text{"C"}$

$exposure := \text{"C"}$

Step 5. Determine Topographic Factor:

$K_{zt} := 1.00$ (Conservative assumption)

Step 6. Determine K_e:

$z_{ground} := 600\ ft$ (Average elevation in Clinton IA)

$$K_e := \exp\left(-0.0000362 \cdot \frac{z_{ground}}{ft}\right)$$

$$K_e = 0.979$$

Step 7. Calculate Gust Factor, G:

$G := 0.85$ (Default factor for rigid buildings)

Step 8. Determine Enclosure Classification

$enclosure := \text{"partially enclosed"}$

Step 9. Determine Internal Pressure Coefficient

$$GC_{pi} := \begin{array}{l} \text{if } enclosure = \text{"enclosed"} \\ \quad \| \ 0.18 \\ \text{else if } enclosure = \text{"partially enclosed"} \\ \quad \| \ 0.55 \\ \text{else if } enclosure = \text{"partially open"} \\ \quad \| \ 0.18 \\ \text{else} \\ \quad \| \ 0 \end{array}$$

$GC_{pi} = 0.55$ + or -

Step 10. Define Variables:

$z_{15} := 15\ ft$ $z_{18} := 25\ ft$ $z_p := 22\ ft$

$h := z_{18}$ (Roof Height)

$B := 96\ ft$ (Building Width -- perpendicular to wind direction)

$L := 86\ ft$ (Building Length -- parallel to wind direction)

Step 11. Calculate K_z:

$$\alpha := \begin{array}{l} \text{if } exposure = \text{"B"} \\ \quad \| \ 7.0 \\ \text{else if } exposure = \text{"C"} \\ \quad \| \ 9.5 \\ \text{else} \\ \quad \| \ 11.5 \end{array}$$

(Table 26.11-1)

$\alpha = 9.5$

$$z_g := \begin{array}{l} \text{if } exposure = \text{"B"} \\ \quad \| \ 1200 \ ft \\ \text{else if } exposure = \text{"C"} \\ \quad \| \ 900 \ ft \\ \text{else} \\ \quad \| \ 700 \ ft \end{array}$$

(Table 26.11-1)

$z_g = 900 \ ft$

$$K_{15} := 2.01 \cdot \left(\frac{z_{15}}{z_g}\right)^{\frac{2}{\alpha}} = 0.849$$

$$K_{18} := 2.01 \cdot \left(\frac{z_{18}}{z_g}\right)^{\frac{2}{\alpha}} = 0.945$$

$$K_p := 2.01 \cdot \left(\frac{z_p}{z_g}\right)^{\frac{2}{\alpha}} = 0.92$$

Step 12. Calculate q_z:

q_t = (0.00256 Kz Kzt Kd Ke v^2)

$$q_{15} := \left(\frac{0.00256 \ psf}{mph^2}\right) \cdot K_{15} \cdot K_{zt} \cdot K_d \cdot K_e \cdot V^2 = 12.154 \ psf$$

$$q_{18} := \left(\frac{0.00256 \ psf}{mph^2}\right) \cdot K_{18} \cdot K_{zt} \cdot K_d \cdot K_e \cdot V^2 = 13.533 \ psf$$

$$q_p := \left(\frac{0.00256 \ psf}{mph^2}\right) \cdot K_p \cdot K_{zt} \cdot K_d \cdot K_e \cdot V^2 = 13.174 \ psf$$

$q_h := q_{18} = 13.533 \ psf$

Step 13. Determine External Pressure Coefficients

Test L/B: $\frac{L}{B} = 0.896$

Windward wall: $C_{p\_windward} := 0.8$ (Figure 27.3-1)

Leeward wall: $C_{p\_leeward} := -0.5$ (Figure 27.3-1<L/B<1)

Windward parapet: $GC_{pn\_windward} := 1.5$ (Section 27.3.4)

Leeward parapet: $GC_{pn\_leeward} := -1.0$ (Section 27.3.4)

Step 13. Calculate Design Wind Pressures:

Positive Internal Pressure:

Windward Wall:

$$p_{15\_pos} := q_{15} \cdot G \cdot C_{p\_windward} - q_h \cdot \left(GC_{pi}\right) = 0.821 \; psf$$

$$p_{18\_pos} := q_{18} \cdot G \cdot C_{p\_windward} - q_h \cdot \left(GC_{pi}\right) = 1.759 \; psf$$

Leeward Wall:

$$p_{leeward\_pos} := q_h \cdot G \cdot C_{p\_leeward} - q_h \cdot \left(GC_{pi}\right) = -13.195 \; psf$$

Net Pressure:

$$p_{15\_netPos} := p_{15\_pos} - p_{leeward\_pos} = 14.016 \; psf$$

$$p_{25\_netPos} := p_{18\_pos} - p_{leeward\_pos} = 14.954 \; psf$$

Negative Internal Pressure:

Windward Wall:

$$p_{15\_neg} := q_{15} \cdot G \cdot C_{p\_windward} - q_h \cdot \left(-GC_{pi}\right) = 15.708 \; psf$$

$$p_{18\_neg} := q_{18} \cdot G \cdot C_{p\_windward} - q_h \cdot \left(-GC_{pi}\right) = 16.646 \; psf$$

Leeward Wall:

$$p_{leeward\_neg} := q_h \cdot G \cdot C_{p\_leeward} - q_h \cdot \left(-GC_{pi}\right) = 1.692 \; psf$$

Net Pressure:

$$p_{15\_netNeg} := p_{15\_neg} - p_{leeward\_neg} = 14.016 \; psf$$

$$p_{18\_netNeg} := p_{18\_neg} - p_{leeward\_neg} = 14.954 \; psf$$

Calculate Parapet Design Pressures:

Windward Wall:

$$p_{p\_windward} := q_p \cdot GC_{pn\_windward} = 19.761 \; psf$$

Leeward Wall:

$$p_{p\_leeward} := q_p \cdot GC_{pn\_leeward} = -13.174 \; psf$$

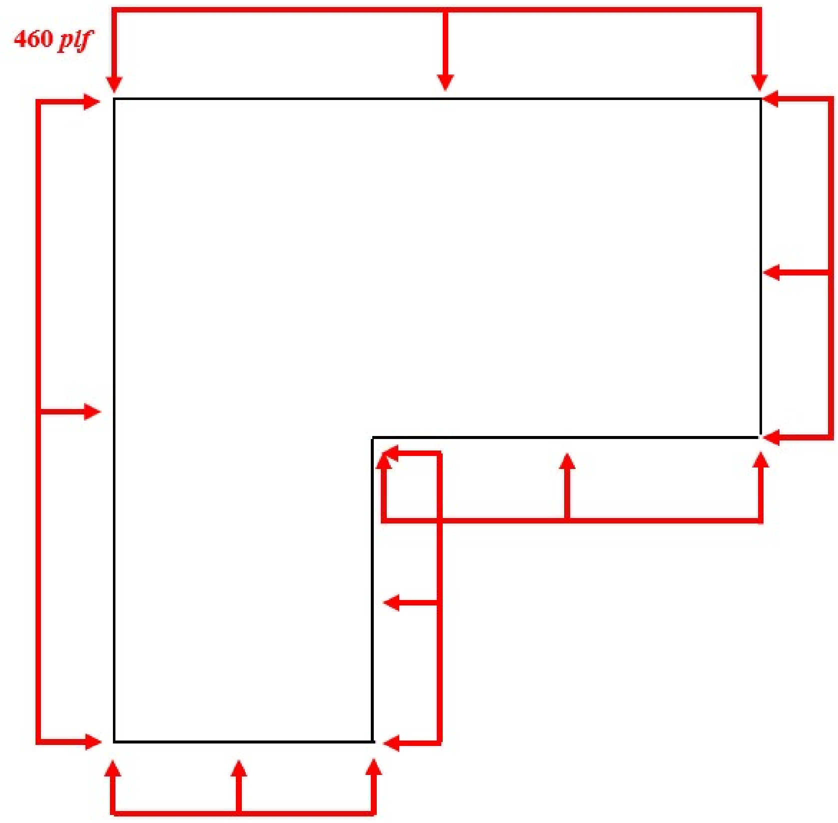
Net Parapet Pressure:

$$p_{p\_net} := p_{p\_windward} - p_{p\_leeward} = 32.935 \; psf$$

Appendix K: Building C: Wind Load Reactions Due to Ultimate and Serviceability Load

ULTIMATE LOADING:

Lateral Structural System: Diaphragm



Building Geometry:

$h' := 18\ ft$ (height without the parapet)
$h_{15} := 15\ ft$ (height at 15 ft)
$h := 22\ ft$ (height with the parapet)
$h_p := 4\ ft$ (height of parapet)

$B := 96\ ft$ (length N-S)
$L := 86\ ft$ (length E-W)

Wind Pressure: all pressures below are net pressures.

$q_{15} := 23.211\ psf$ (pressure up until the parapet at h' = 18')

$q_{18} := 23.762\ psf$

$q_p := 56.079\ psf$ (pressure on just the parapet)

Tributary Method for Lateral Load Path: Flexible Diaphragm

$$w_{roof} := \frac{1}{h'} \cdot \left(q_{15} \cdot h_{15} \cdot \frac{h_{15}}{2} + q_{18} \cdot (h' - h_{15}) \cdot \left(\frac{h_{15} + h'}{2} \right) + q_p \cdot h_p \cdot \left(h' + \frac{h_p}{2} \right) \right) = 459.654\ plf$$

$w_{roof} := 460\ plf$

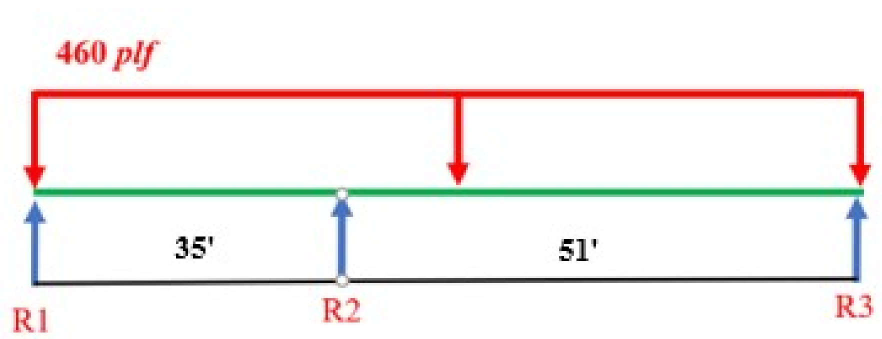
N-S Direction

$$R_{1\_left} := \frac{35\ ft}{2} \cdot w_{roof} = 8.05\ kip$$
$$R_{2\_middle} := \frac{35\ ft}{2} \cdot w_{roof} + \frac{51\ ft}{2} \cdot w_{roof} = 19.78\ kip$$
$$R_{3\_right} := \frac{51\ ft}{2} \cdot w_{roof} = 11.73\ kip$$

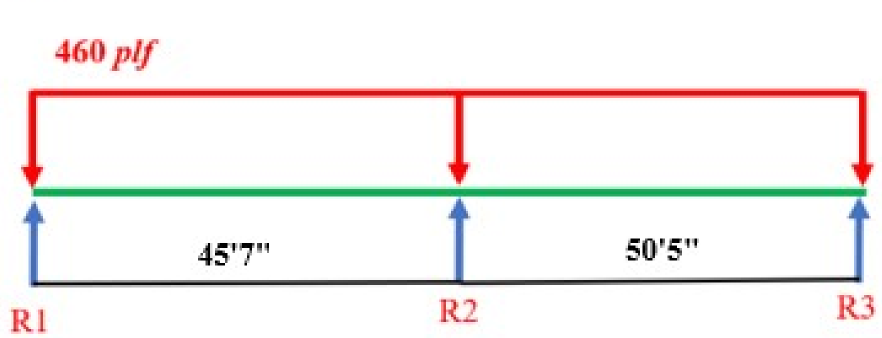
E-W Direction

$$R_{1\_top} := \frac{FIF(\text{“45'7”})}{2} \cdot w_{roof} = 10.484\ kip$$
$$R_{2\_middle} := \frac{FIF(\text{“45'7”})}{2} \cdot w_{roof} + \frac{FIF(\text{“50'5”})}{2} \cdot w_{roof} = 22.08\ kip$$
$$R_{3\_bottotm} := \frac{FIF(\text{“50'5”})}{2} \cdot w_{roof} = 11.596\ kip$$

N-S

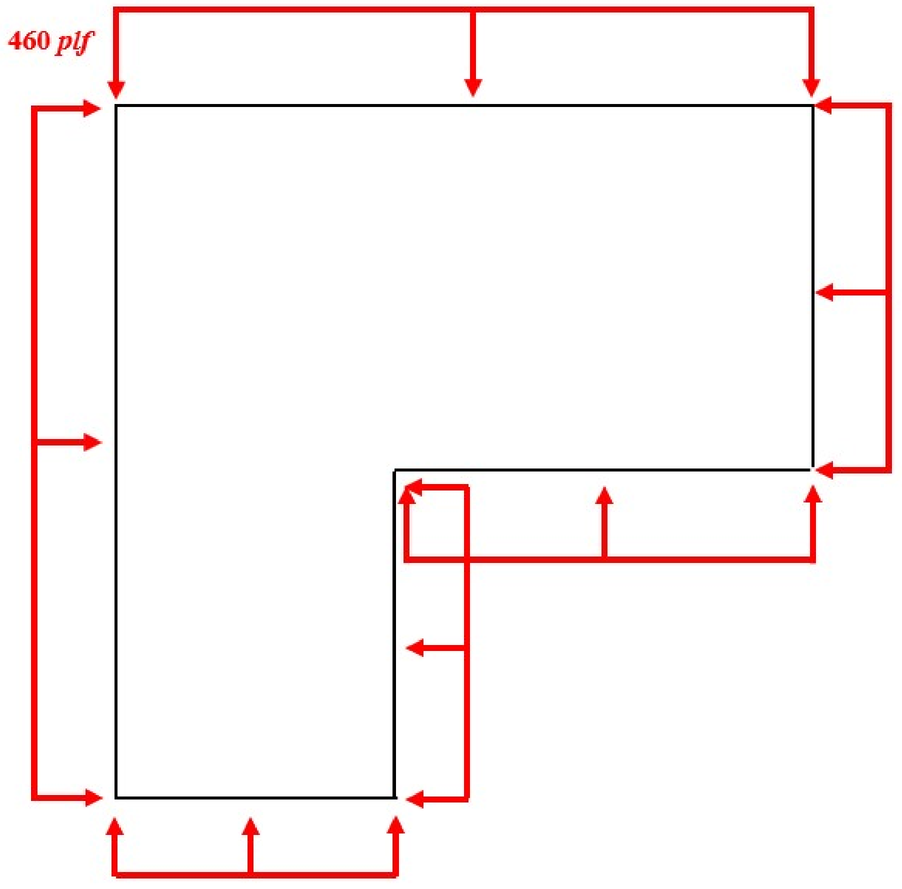


E-W



SERVICABILITY LOADING:

Lateral Structural System: Diaphragm



Building Geometry:

$h' := 18\ ft$ (height without the parapet)
$h_{15} := 15\ ft$ (height at 15 ft)
$h := 22\ ft$ (height with the parapet)
$h_p := 4\ ft$ (height of parapet)

$B := 96\ ft$ (length N-S)
$L := 86\ ft$ (length E-W)

Wind Pressure: all pressures below are net pressures.

$q_{15} := 14.016\ psf$ (pressure up until the parapet at h' = 18')

$q_{18} := 14.954\ psf$

$q_p := 32.935\ psf$ (pressure on just the parapet)

Tributary Method for Lateral Load Path: Flexible Diaphragm

$$w_{roof} := \frac{1}{h'} \cdot \left(q_{15} \cdot h_{15} \cdot \frac{h_{15}}{2} + q_{18} \cdot (h' - h_{15}) \cdot \left(\frac{h_{15} + h'}{2} \right) + q_p \cdot h_p \cdot \left(h' + \frac{h_p}{2} \right) \right) = 275.101\ plf$$

$w_{roof} := 276\ plf$

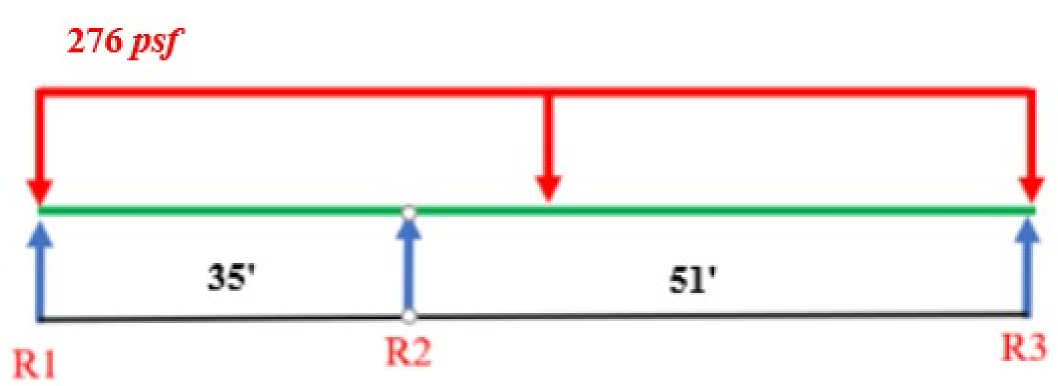
N-S Direction

$$R_{1\_left} := \frac{35\ ft}{2} \cdot w_{roof} = 4.83\ kip$$
$$R_{2\_middle} := \frac{35\ ft}{2} \cdot w_{roof} + \frac{51\ ft}{2} \cdot w_{roof} = 11.868\ kip$$
$$R_{3\_right} := \frac{51\ ft}{2} \cdot w_{roof} = 7.038\ kip$$

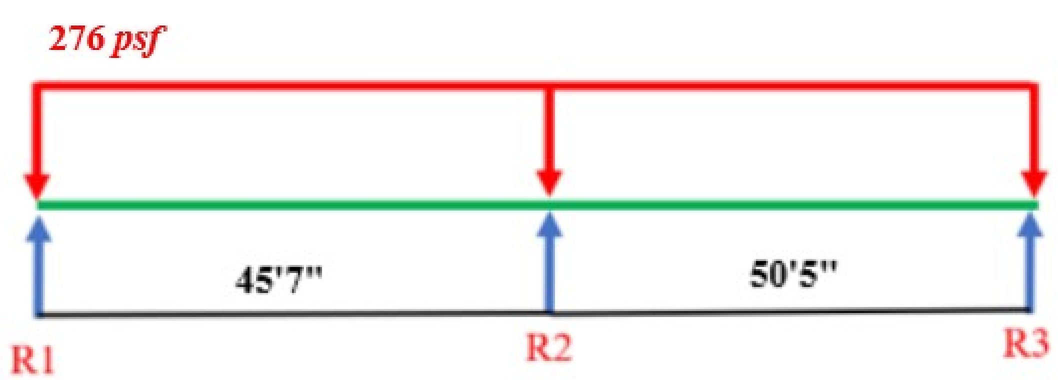
E-W Direction

$$R_{1\_top} := \frac{FIF(\text{“45'7”})}{2} \cdot w_{roof} = 6.291\ kip$$
$$R_{2\_middle} := \frac{FIF(\text{“45'7”})}{2} \cdot w_{roof} + \frac{FIF(\text{“50'5”})}{2} \cdot w_{roof} = 13.248\ kip$$
$$R_{3\_bottotm} := \frac{FIF(\text{“50'5”})}{2} \cdot w_{roof} = 6.958\ kip$$

N-S



E-W



Appendix L: Building C: LRFD and ASD Factored Design

LRFD:

Roof Loading:

Dead Load: $D_{Lroof} := 20\ psf$

Live Load: $L_{Lroof} := 20\ psf$

Snow Load: $S_{roof} := 16\ psf$

Applicable LRFD Load Combinations:

Table C2.3-1 Principal Loads for Strength Design Load Combinations

	Load Combination	Principal Load
1	$1.4D$	D
2	$1.2D + 1.6L + 0.5(L_r$ or S or $R)$	L
3	$1.2D + 1.6(L_r$ or S or $R) + (1.0L$ or $0.5W)$	L_r or S or R
4	$1.2D + 1.0W + 1.0L + 0.5(L_r$ or S or $R)$	W
5	$0.9D + 1.0W$	W
6	$1.2D + E_v + E_h + L + 0.2S$	E
7	$0.9D - E_v + E_h$	E

$$q_1 := 1.4 \cdot D_{Lroof} = 28\ psf$$

$$q_2 := 1.2 \cdot D_{Lroof} + 1.6 \cdot L_{Lroof} + 0.5 \cdot S_{roof} = 64\ psf$$

$$q_3 := 1.2 \cdot D_{Lroof} + 1.6 \cdot S_{roof} + 1.0 \cdot L_{Lroof} = 69.6\ psf$$

Factored Roof Uniform Area Load:

$$q_{roof} := \max\left(q_1, q_2, q_3\right) = 69.6\ psf$$

ASD:

Applicable ASD Load Combinations:

$$q_1 := D_{Lroof} = 20\ psf$$

$$q_2 := D_{Lroof} + L_{Lroof} = 40\ psf$$

$$q_3 := D_{Lroof} + S_{roof} = 36\ psf$$

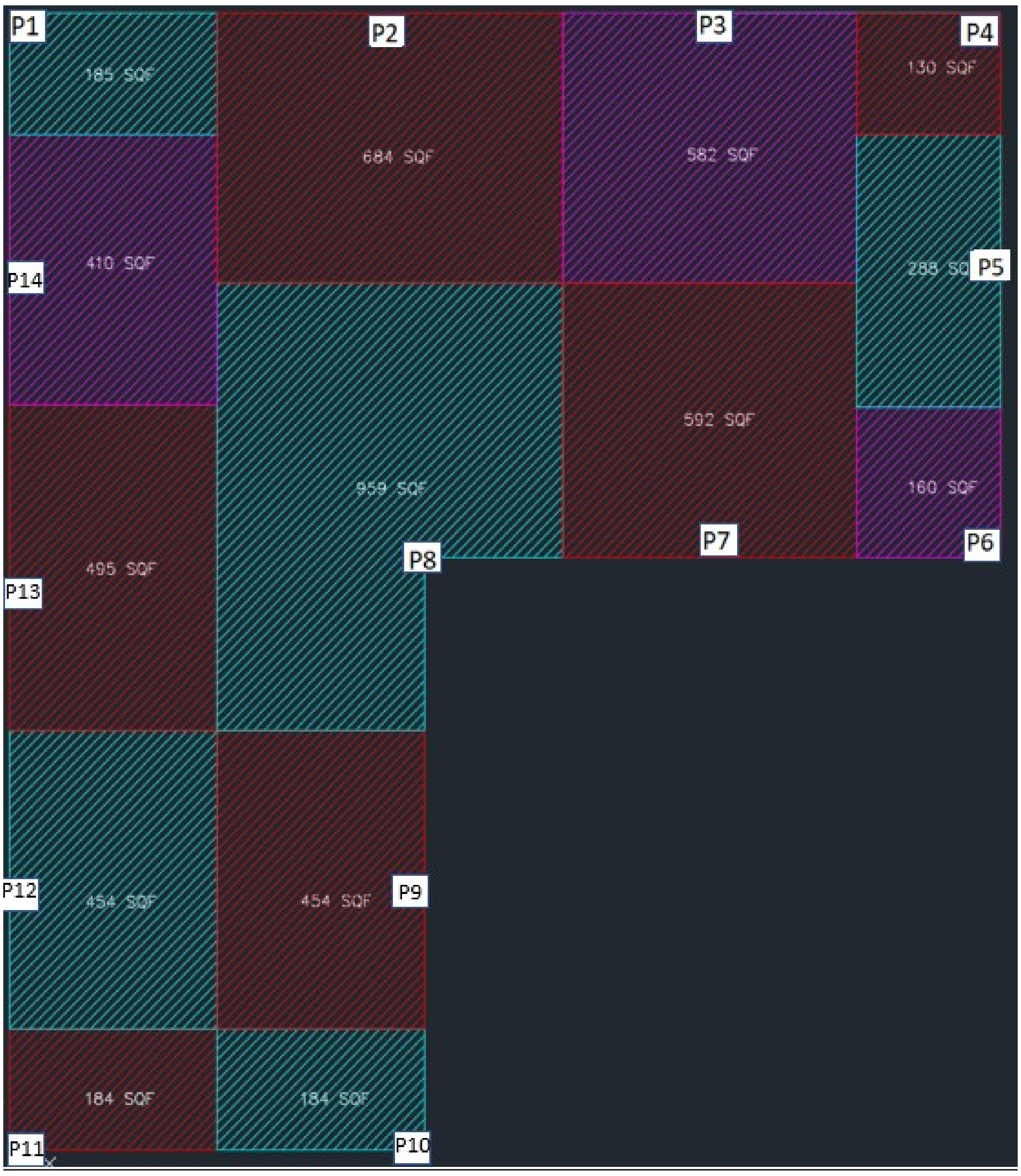
$$q_4 := D_{Lroof} + 0.75 \cdot L_{Lroof} + 0.75 \cdot S_{roof} = 47\ psf$$

Factored Roof Uniform Area Load:

$$q_{roof} := \max\left(q_1, q_2, q_3, q_4\right) = 47\ psf$$

Appendix M: Building C: Tributary Areas and Column Axial Loads

Column Tributary Areas based on Dimensions:



Tributary Area:

$A_{trib1} := 185\ ft^2$ $\quad A_{trib4} := 130\ ft^2$ $\quad A_{trib7} := 592\ ft^2$ $\quad A_{trib10} := 184\ ft^2$

$A_{trib2} := 684\ ft^2$ $\quad A_{trib5} := 288\ ft^2$ $\quad A_{trib8} := 959\ ft^2$ $\quad A_{trib11} := 184\ ft^2$ $\quad A_{trib13} := 495\ ft^2$

$A_{trib3} := 582\ ft^2$ $\quad A_{trib6} := 160\ ft^2$ $\quad A_{trib9} := 454\ ft^2$ $\quad A_{trib12} := 454\ ft^2$ $\quad A_{trib14} := 410\ ft^2$

LRFD Column Loading:

$$q_{roofLRFD} := 69.6\ psf$$

$P_1 := q_{roofLRFD} \cdot A_{trib1} = 12.876\ kip$ $\quad P_8 := q_{roofLRFD} \cdot A_{trib8} = 66.746\ kip$

$P_2 := q_{roofLRFD} \cdot A_{trib2} = 47.606\ kip$ $\quad P_9 := q_{roofLRFD} \cdot A_{trib9} = 31.598\ kip$

$P_3 := q_{roofLRFD} \cdot A_{trib3} = 40.507\ kip$ $\quad P_{10} := q_{roofLRFD} \cdot A_{trib10} = 12.806\ kip$

$P_4 := q_{roofLRFD} \cdot A_{trib4} = 9.048\ kip$ $\quad P_{11} := q_{roofLRFD} \cdot A_{trib11} = 12.806\ kip$

$P_5 := q_{roofLRFD} \cdot A_{trib5} = 20.045\ kip$ $\quad P_{12} := q_{roofLRFD} \cdot A_{trib12} = 31.598\ kip$

$P_6 := q_{roofLRFD} \cdot A_{trib6} = 11.136\ kip$ $\quad P_{13} := q_{roofLRFD} \cdot A_{trib13} = 34.452\ kip$

$P_7 := q_{roofLRFD} \cdot A_{trib7} = 41.203\ kip$ $\quad P_{14} := q_{roofLRFD} \cdot A_{trib14} = 28.536\ kip$

ASD Column Loading:

$$q_{roofASD} := 47\ psf$$

$P_1 := q_{roofASD} \cdot A_{trib1} = 8.695\ kip$ $\quad P_8 := q_{roofASD} \cdot A_{trib8} = 45.073\ kip$

$P_2 := q_{roofASD} \cdot A_{trib2} = 32.148\ kip$ $\quad P_9 := q_{roofASD} \cdot A_{trib9} = 21.338\ kip$

$P_3 := q_{roofASD} \cdot A_{trib3} = 27.354\ kip$ $\quad P_{10} := q_{roofASD} \cdot A_{trib10} = 8.648\ kip$

$P_4 := q_{roofASD} \cdot A_{trib4} = 6.11\ kip$ $\quad P_{11} := q_{roofASD} \cdot A_{trib11} = 8.648\ kip$

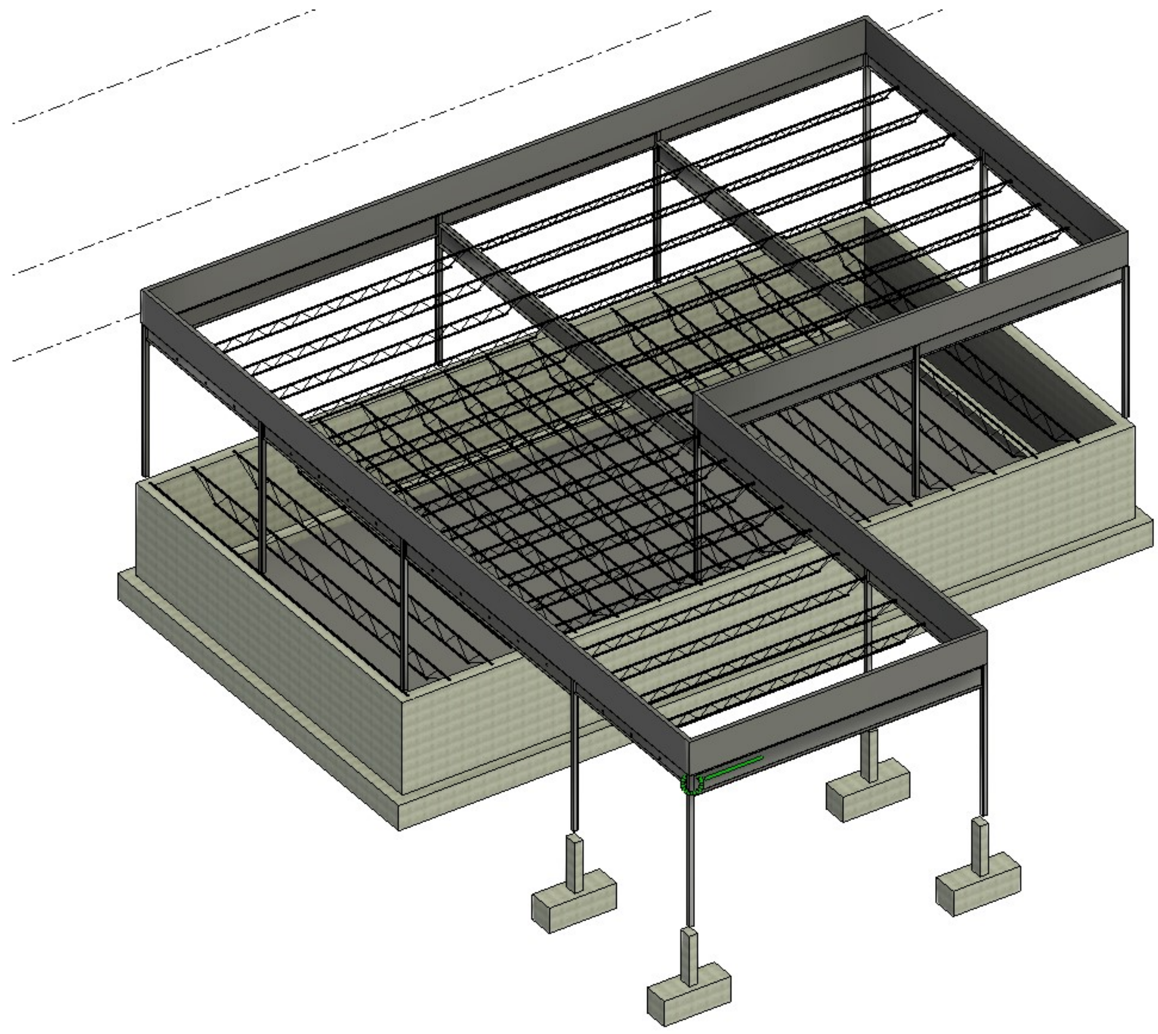
$P_5 := q_{roofASD} \cdot A_{trib5} = 13.536\ kip$ $\quad P_{12} := q_{roofASD} \cdot A_{trib12} = 21.338\ kip$

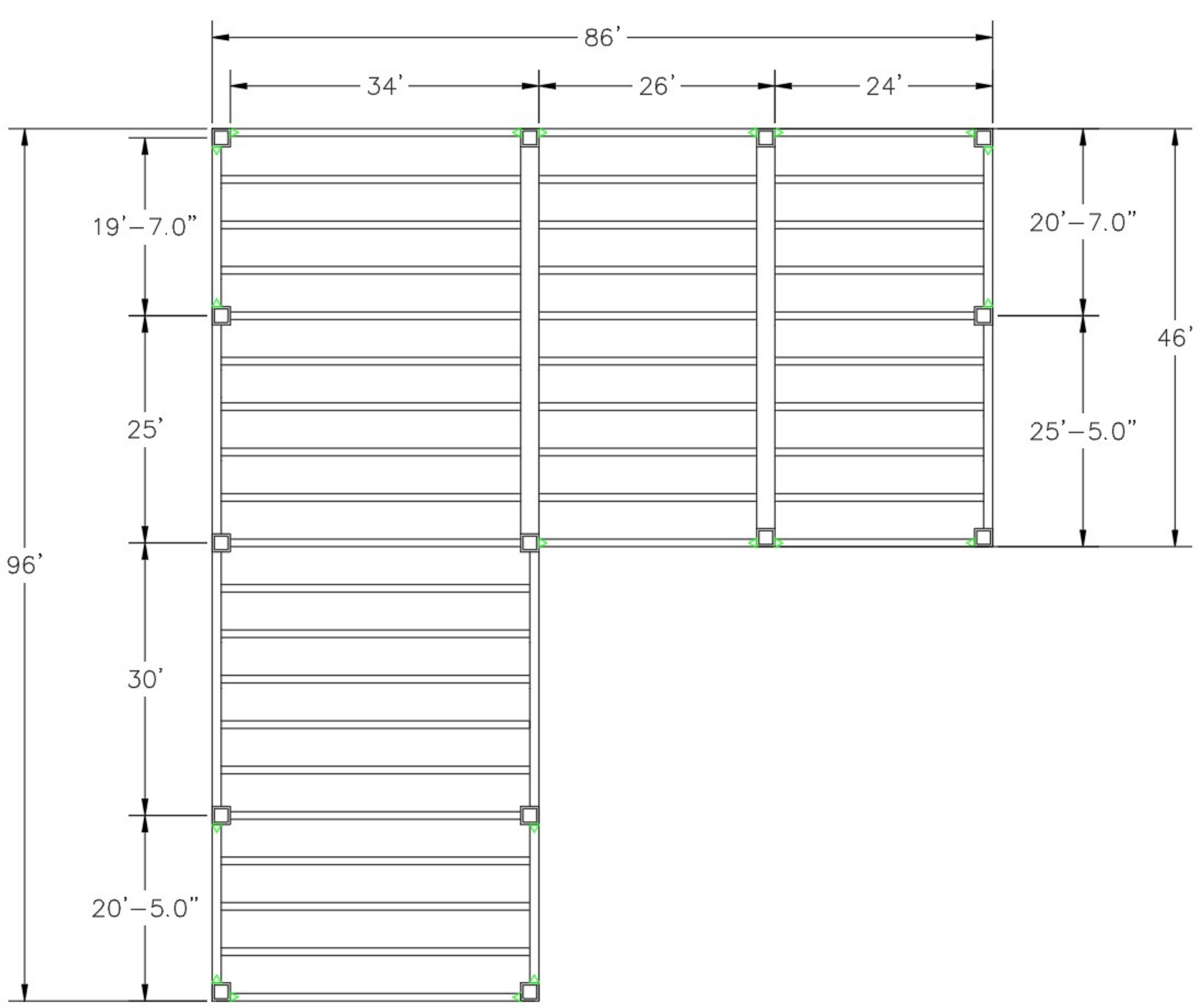
$P_6 := q_{roofASD} \cdot A_{trib6} = 7.52\ kip$ $\quad P_{13} := q_{roofASD} \cdot A_{trib13} = 23.265\ kip$

$P_7 := q_{roofASD} \cdot A_{trib7} = 27.824\ kip$ $\quad P_{14} := q_{roofASD} \cdot A_{trib14} = 19.27\ kip$

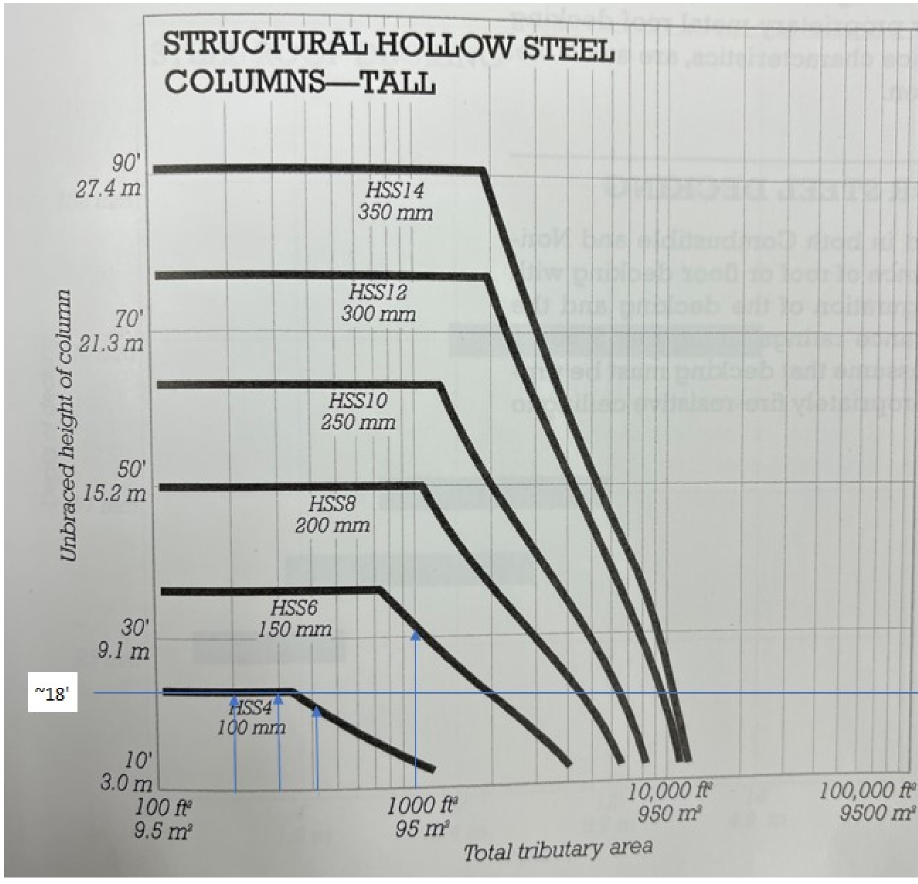
Appendix N: Building C: Preliminary Member Sizes and Rules of Thumb

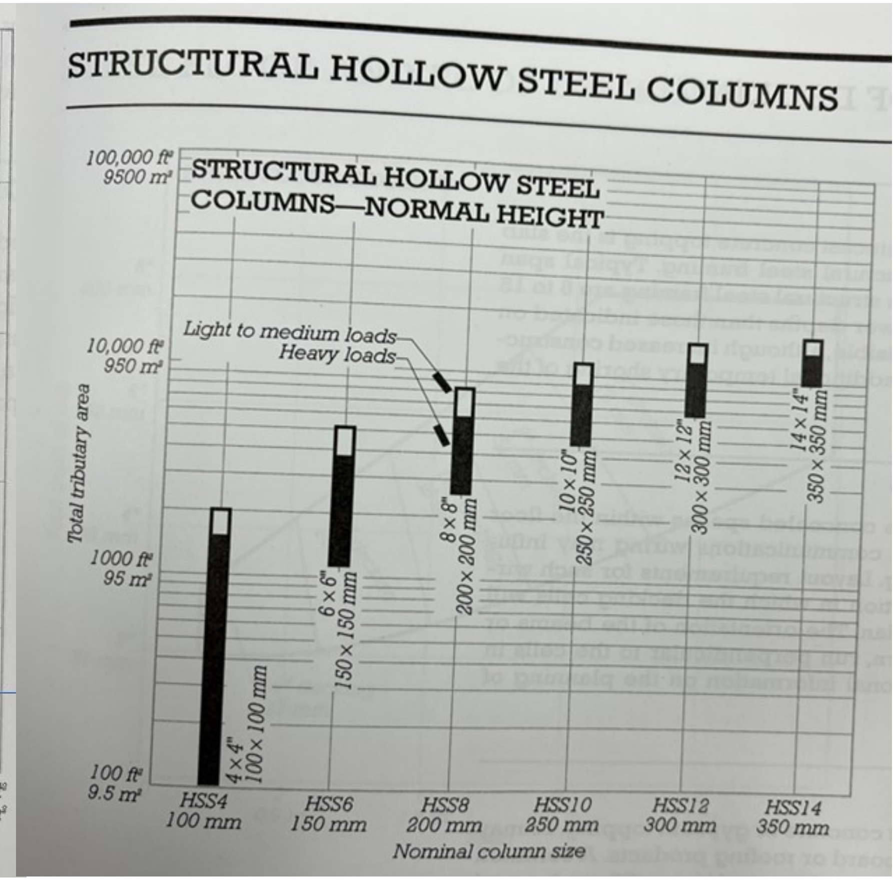
Framing Plan. Rigid Connections are indicated by green triangles:





Preliminary Column Sizing:





The top chart is for hollow steel section columns up to 12 ft (3.7 m) tall between floors. Read in the top open areas for light and medium loads. Read in the lower solid areas for heavy loads. *Total tributary area* is the summed area of the roof and all floors supported by the column.

■ Actual column size is equal to nominal size.

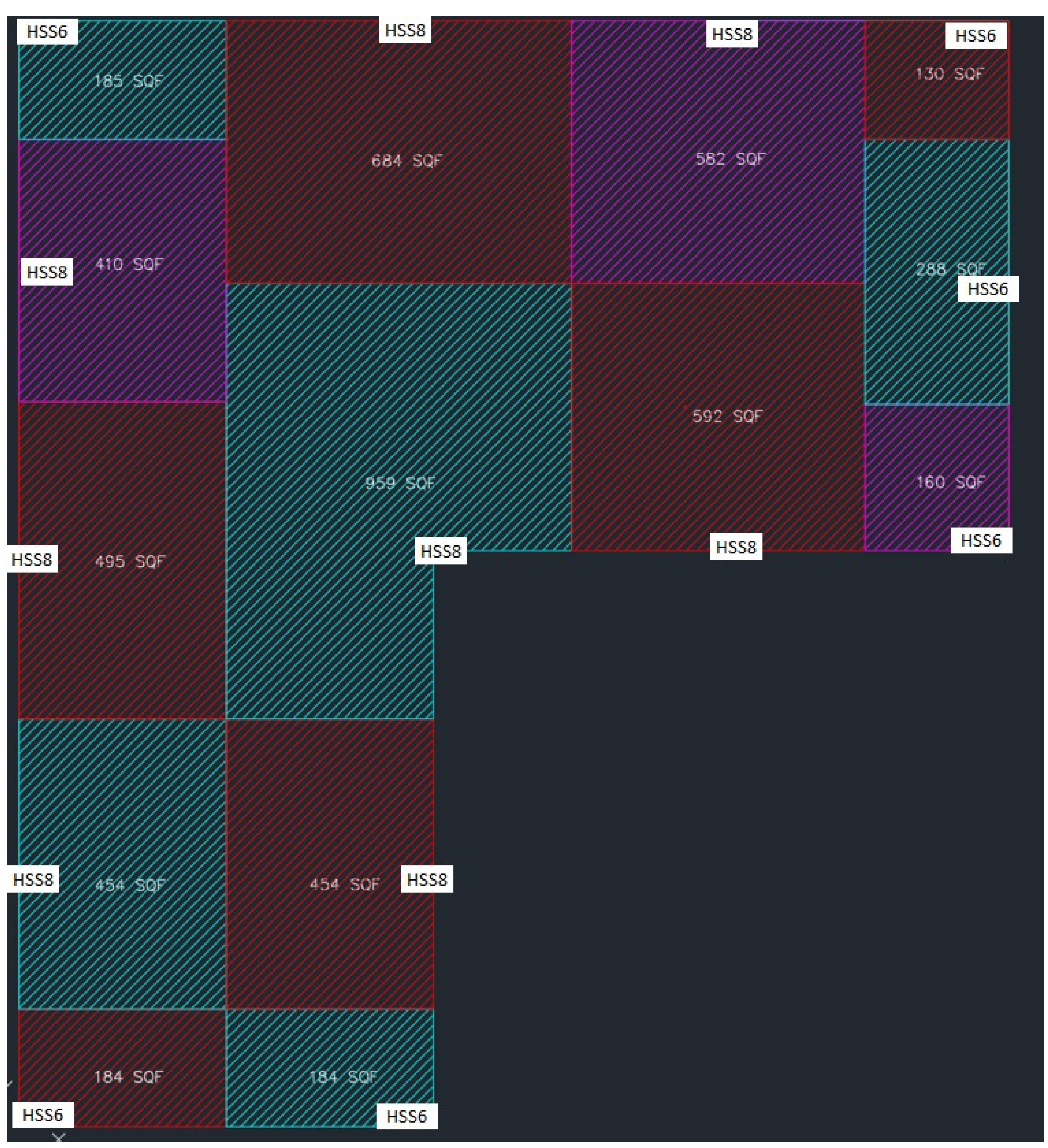
■ For columns located at the perimeter of a building, or ones that are part of a rigid frame system, select one nominal size larger than the size indicated by this chart.

For columns taller than 12 ft (3.7 m), read from both charts on this page, using the larger column size indicated by either one. *Unbraced height of column* is the vertical distance between floors or other supports that brace the column laterally against buckling.

■ Minimum column size will be larger for heavily loaded columns or columns that are part of rigid frame systems.

SIZING THE STRUCTURAL SYSTEM 101

Tributary Areas For Each Column with Member Sizing:

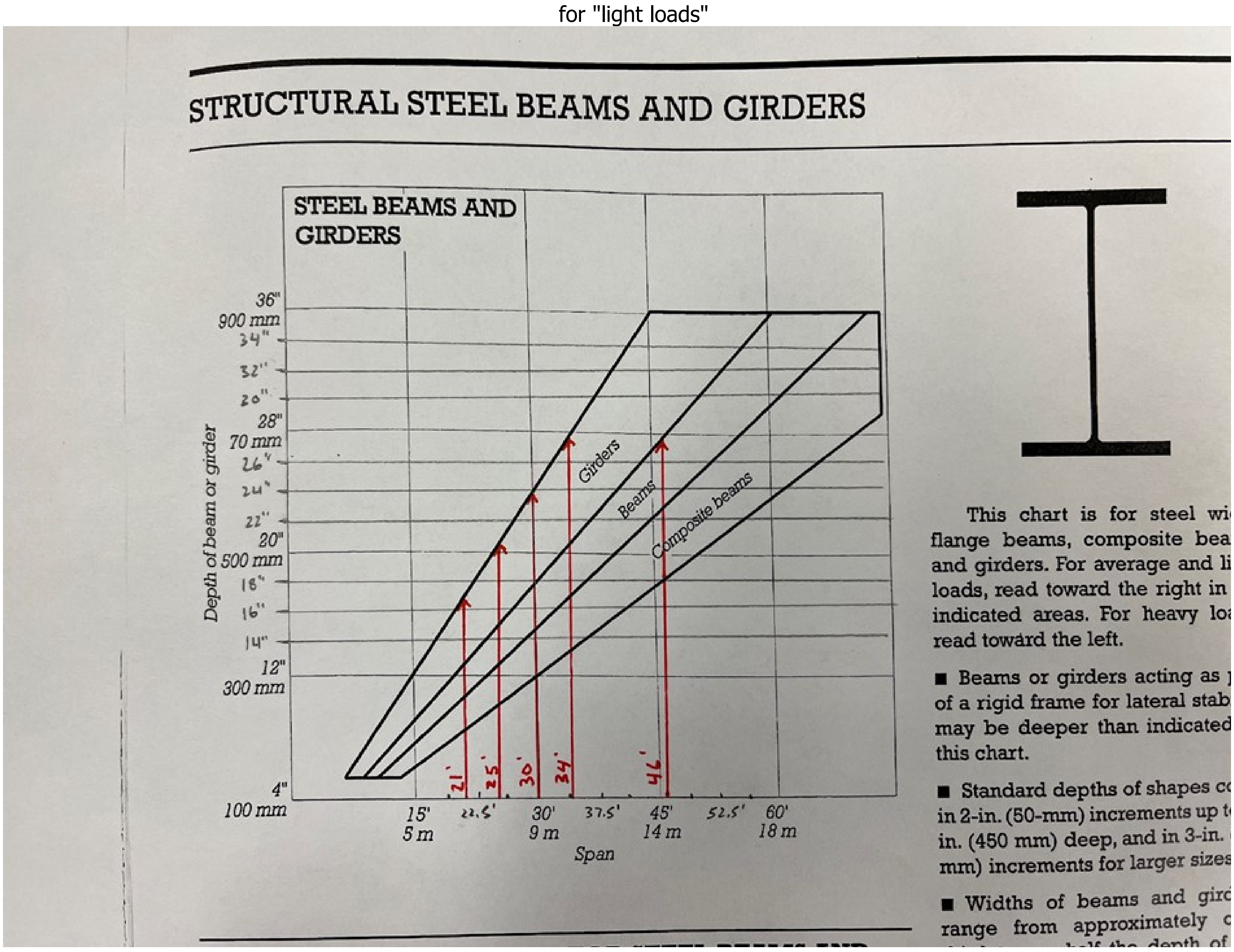


List of Columns Selected:
-HSS6
-HSS8

Column sizes have been increased by one nominal size for either being part of a the rigid frame system or for being a perimeter column.

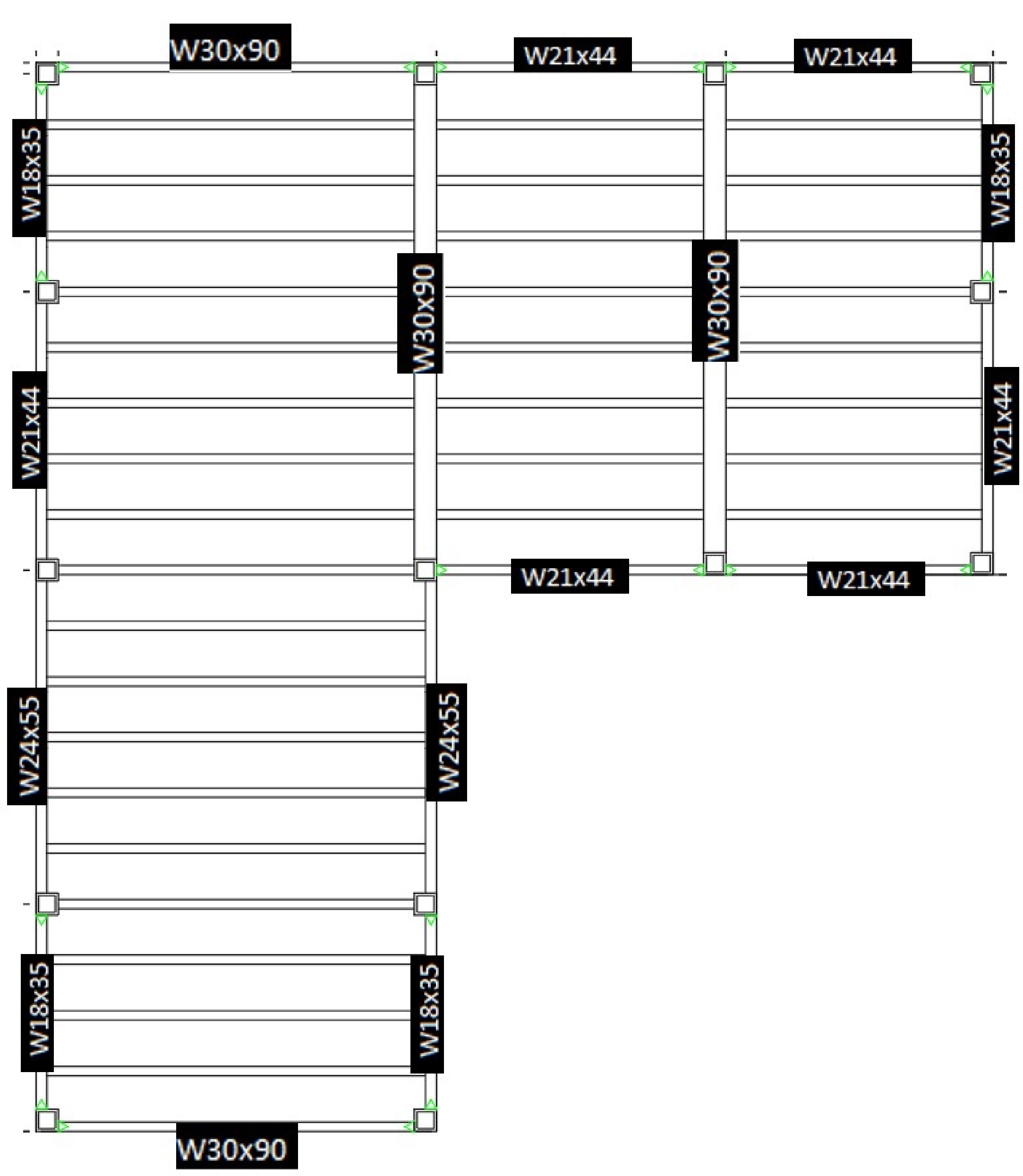
Preliminary Girder Sizing:

Girder sizing for different spans. Girders that are acting as part of a rigid frame system or are perimeter girders have been preliminarily sized for "heavy loads" using the chart below. Interior girders that are not part of a rigid frame system have been sized for "light loads"



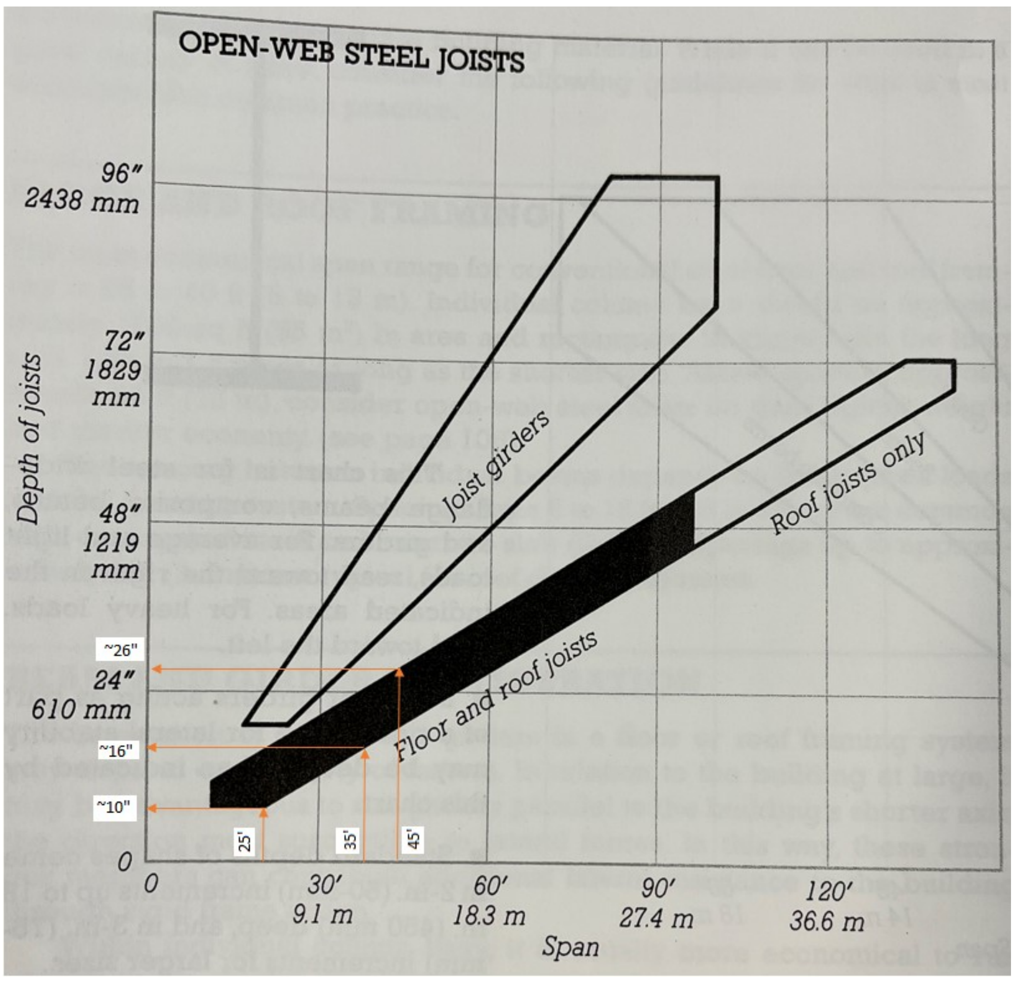
Preliminary Girder sizes:

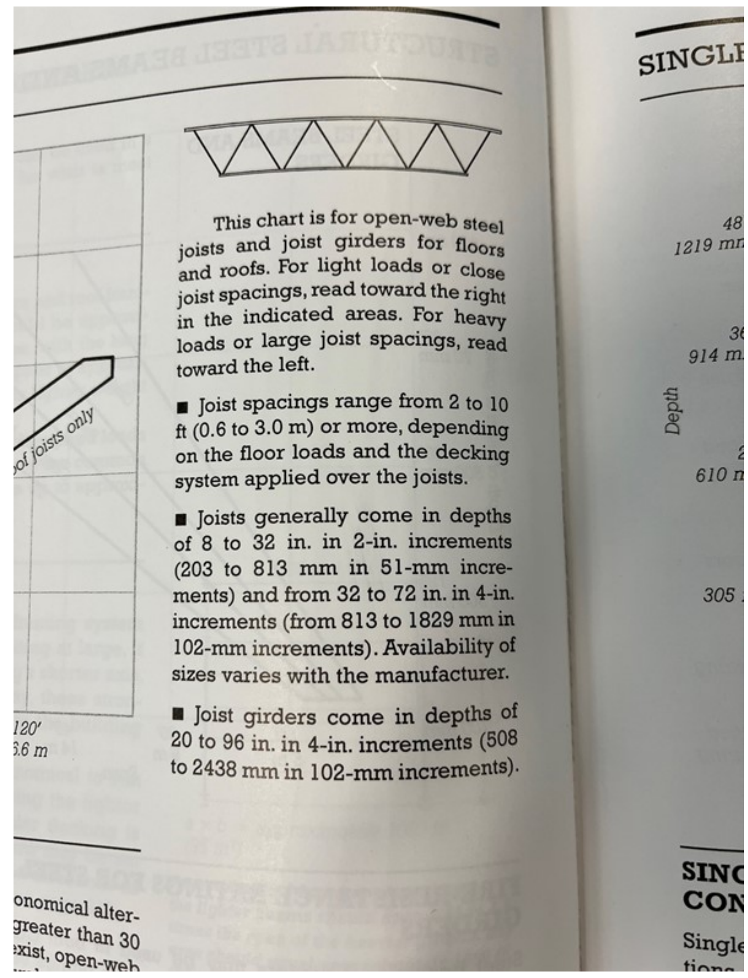
List of Girders Selected:
- W30x90
- W24x55
- W21x44
- W18x35



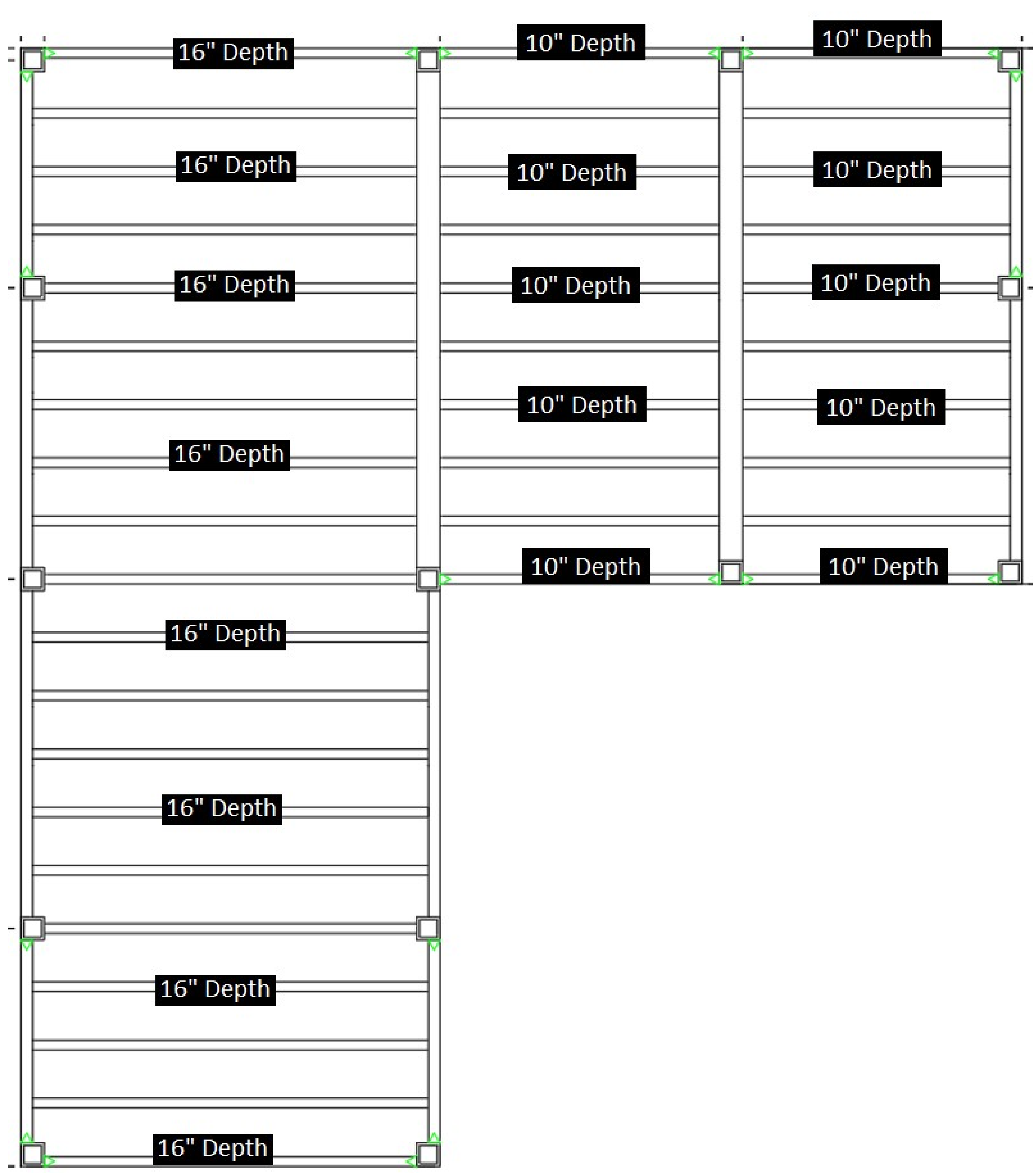
Preliminary Joist Sizing:

Open Web Joist sizing for different spans for first floor and basement. Joists that are supporting the first floor have been preliminarily sized for "heavy loads" using the chart below. Joists supporting the roof have been sized for "light loads"





First Floor Framing Plan:



List of Joists Selected:
-25' span, 10" Depth
-35' span, 16" Depth
-45' span, 26" Depth

Appendix O: Building C: Moment Frame Design

Moment Frame Calculations:

$DL := 20\ psf$ $SL := 16\ psf$ $LL := 20\ psf$ $L_1 := 35\ ft$ $L_2 := 26\ ft$ $L_3 := 25\ ft$

N-S West Wall:

$x_{f1\_1} := \frac{FIF(``20'7")}{4} = 5.146\ ft$ $x_{f1\_2} := \frac{2 \cdot FIF(``20'7")}{4} = 10.292\ ft$ $x_{f1\_3} := \frac{3 \cdot FIF(``20'7")}{4} = 15.438\ ft$

$x_{f1\_4} := \frac{4 \cdot FIF(``20'7")}{4} = 20.583\ ft$ $x_{f2\_1} := \frac{4 \cdot FIF(``20'7")}{4} + \frac{FIF(``25'0")}{5} = 25.583\ ft$ $x_{f2\_2} := \frac{4 \cdot FIF(``20'7")}{4} + \frac{2 \cdot FIF(``25'0")}{5} = 30.583\ ft$

$x_{f2\_3} := \frac{4 \cdot FIF(``20'7")}{4} + \frac{3 \cdot FIF(``25'0")}{5} = 35.583\ ft$ $x_{f2\_4} := \frac{4 \cdot FIF(``20'7")}{4} + \frac{4 \cdot FIF(``25'0")}{5} = 40.583\ ft$

$x_{f2\_5} := \frac{4 \cdot FIF(``20'7")}{4} + \frac{5 \cdot FIF(``25'0")}{5} = 45.583\ ft$ $x_{f3\_1} := 45.583\ ft + \frac{1 \cdot FIF(``30'0")}{6} = 50.583\ ft$

$x_{f3\_2} := 45.583\ ft + \frac{2 \cdot FIF(``30'0")}{6} = 55.583\ ft$ $x_{f3\_3} := 45.583\ ft + \frac{3 \cdot FIF(``30'0")}{6} = 60.583\ ft$

$x_{f3\_4} := 45.583\ ft + \frac{4 \cdot FIF(``30'0")}{6} = 65.583\ ft$ $x_{f3\_5} := 45.583\ ft + \frac{5 \cdot FIF(``30'0")}{6} = 70.583\ ft$

$x_{f3\_6} := 45.583\ ft + \frac{6 \cdot FIF(``30'0")}{6} = 75.583\ ft$ $x_{f4\_1} := 75.583\ ft + \frac{FIF(``20'5")}{4} = 80.687\ ft$

$x_{f4\_2} := 75.583\ ft + \frac{2 \cdot FIF(``20'5")}{4} = 85.791\ ft$ $x_{f4\_3} := 75.583\ ft + \frac{3 \cdot FIF(``20'5")}{4} = 90.896\ ft$

$$x_{f4\_3} := 75.583\ ft + \frac{4 \cdot FIF(``20'5")}{4} = 96\ ft$$

Joist Spacings:

$J_{f1\_c\_ext} := \frac{\frac{FIF(``20'7")}{4}}{2} = 2.573\ ft$ $J_{f1\_int} := \frac{\frac{2 \cdot FIF(``20'7")}{4}}{2} = 5.146\ ft$

$J_{f1\&f2\_c\_int} := \frac{\frac{FIF(``20'7")}{4} + \frac{FIF(``25'0")}{5}}{2} = 5.073\ ft$ $J_{f2\_int} := \frac{\frac{2 \cdot FIF(``25'0")}{5}}{2} = 5\ ft$

$J_{f2\&f3\_c\_int} := \frac{\frac{FIF(``25'0")}{5} + \frac{FIF(``30'0")}{6}}{2} = 5\ ft$ $J_{f3\_int} := \frac{\frac{2 \cdot FIF(``30'0")}{6}}{2} = 5\ ft$

$J_{f3\&f4\_c\_int} := \frac{\frac{FIF(``30'0")}{6} + \frac{FIF(``20'5")}{4}}{2} = 5.052\ ft$ $J_{f4\_int} := \frac{\frac{2 \cdot FIF(``20'5")}{4}}{2} = 5.104\ ft$

$J_{f4\_c\_ext} := \frac{\frac{FIF(``20'5")}{4}}{2} = 2.552\ ft$

DL:

$P_{DL\_ext\_f1} := \frac{DL \cdot J_{f1\_c\_ext} \cdot L_1}{2} = 0.901\ kip$ $P_{DL\_int\_f1} := \frac{DL \cdot J_{f1\_int} \cdot L_1}{2} = 1.801\ kip$

$P_{DL\_int\_f1\&f2\_c} := \frac{DL \cdot J_{f1\&f2\_c\_int} \cdot L_1}{2} = 1.776\ kip$ $P_{DL\_int\_f2} := \frac{DL \cdot J_{f2\_int} \cdot L_1}{2} = 1.75\ kip$

$P_{DL\_int\_f2\&f3\_c} := \frac{DL \cdot J_{f2\&f3\_c\_int} \cdot L_1}{2} = 1.75\ kip$ $P_{DL\_int\_f3} := \frac{DL \cdot J_{f3\_int} \cdot L_1}{2} = 1.75\ kip$

$P_{DL\_int\_f3\&f4\_c} := \frac{DL \cdot J_{f3\&f4\_c\_int} \cdot L_1}{2} = 1.768\ kip$ $P_{DL\_int\_f4} := \frac{DL \cdot J_{f4\_int} \cdot L_1}{2} = 1.786\ kip$

$$P_{DL\_ext\_f4\_c} := \frac{DL \cdot J_{f4\_c\_ext} \cdot L_1}{2} = 0.893\ kip$$

LL:

$$P_{LL\_ext\_f1} := \frac{LL \cdot J_{f1\_c\_ext} \cdot L_1}{2} = 0.901\ kip$$

$$P_{LL\_int\_f1} := \frac{LL \cdot J_{f1\_int} \cdot L_1}{2} = 1.801\ kip$$

$$P_{LL\_int\_f1\&f2\_c} := \frac{LL \cdot J_{f1\&f2\_c\_int} \cdot L_1}{2} = 1.776\ kip$$

$$P_{LL\_int\_f2} := \frac{LL \cdot J_{f2\_int} \cdot L_1}{2} = 1.75\ kip$$

$$P_{LL\_int\_f2\&f3\_c} := \frac{LL \cdot J_{f2\&f3\_c\_int} \cdot L_1}{2} = 1.75\ kip$$

$$P_{LL\_int\_f3} := \frac{LL \cdot J_{f3\_int} \cdot L_1}{2} = 1.75\ kip$$

$$P_{LL\_int\_f3\&f4\_c} := \frac{LL \cdot J_{f3\&f4\_c\_int} \cdot L_1}{2} = 1.768\ kip$$

$$P_{LL\_int\_f4} := \frac{LL \cdot J_{f4\_int} \cdot L_1}{2} = 1.786\ kip$$

$$P_{LL\_ext\_f4\_c} := \frac{LL \cdot J_{f4\_c\_ext} \cdot L_1}{2} = 0.893\ kip$$

SL:

$$P_{SL\_ext\_f1} := \frac{SL \cdot J_{f1\_c\_ext} \cdot L_1}{2} = 0.72\ kip$$

$$P_{SL\_int\_f1} := \frac{SL \cdot J_{f1\_int} \cdot L_1}{2} = 1.441\ kip$$

$$P_{SL\_int\_f1\&f2\_c} := \frac{SL \cdot J_{f1\&f2\_c\_int} \cdot L_1}{2} = 1.42\ kip$$

$$P_{SL\_int\_f2} := \frac{SL \cdot J_{f2\_int} \cdot L_1}{2} = 1.4\ kip$$

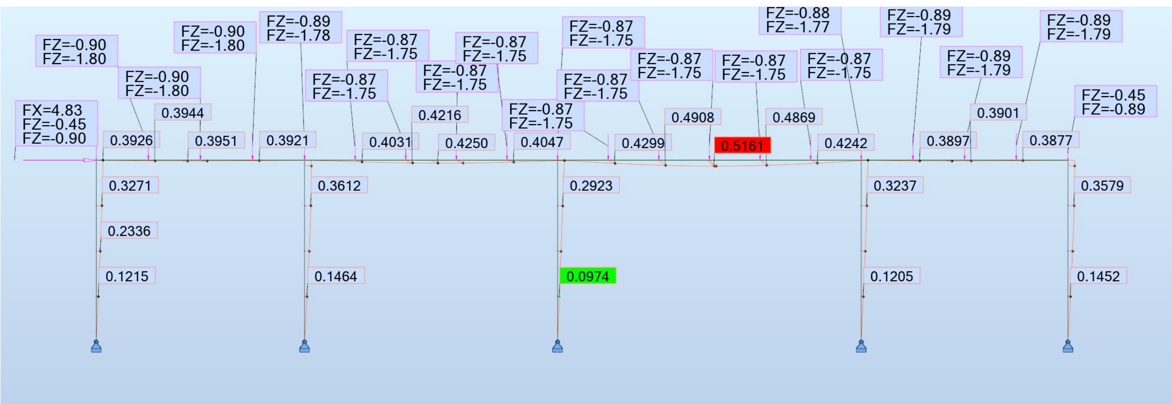
$$P_{SL\_int\_f2\&f3\_c} := \frac{SL \cdot J_{f2\&f3\_c\_int} \cdot L_1}{2} = 1.4\ kip$$

$$P_{SL\_int\_f3} := \frac{SL \cdot J_{f3\_int} \cdot L_1}{2} = 1.4\ kip$$

$$P_{SL\_int\_f3\&f4\_c} := \frac{SL \cdot J_{f3\&f4\_c\_int} \cdot L_1}{2} = 1.415\ kip$$

$$P_{SL\_int\_f4} := \frac{SL \cdot J_{f4\_int} \cdot L_1}{2} = 1.429\ kip$$

$$P_{SL\_ext\_f4\_c} := \frac{SL \cdot J_{f4\_c\_ext} \cdot L_1}{2} = 0.715\ kip$$



Member Sizes Utilized:

- MF-column: W 10x77
- MF-beam: W18x60
- GF-column: W 8x18
- GF-beam: W 18x60

$$\Delta_{max\_lateral} := \frac{18 \cdot 12\ in}{500} = 0.432\ in$$

N-S East Wall:

$$x_{f1\_1} := \frac{FIF(\text{“20'7”})}{4} = 5.146\ ft \qquad x_{f1\_2} := \frac{2 \cdot FIF(\text{“20'7”})}{4} = 10.292\ ft \qquad x_{f1\_3} := \frac{3 \cdot FIF(\text{“20'7”})}{4} = 15.438\ ft$$

$$x_{f1\_4} := \frac{4 \cdot FIF(\text{“20'7”})}{4} = 20.583\ ft \qquad x_{f2\_1} := \frac{4 \cdot FIF(\text{“20'7”})}{4} + \frac{FIF(\text{“25'0”})}{5} = 25.583\ ft$$

$$x_{f2\_2} := \frac{4 \cdot FIF(\text{“20'7”})}{4} + \frac{2 \cdot FIF(\text{“25'0”})}{5} = 30.583\ ft \qquad x_{f2\_3} := \frac{4 \cdot FIF(\text{“20'7”})}{4} + \frac{3 \cdot FIF(\text{“25'0”})}{5} = 35.583\ ft$$

$$x_{f2\_4} := \frac{4 \cdot FIF(\text{“20'7”})}{4} + \frac{4 \cdot FIF(\text{“25'0”})}{5} = 40.583\ ft \qquad x_{f2\_5} := \frac{4 \cdot FIF(\text{“20'7”})}{4} + \frac{5 \cdot FIF(\text{“25'0”})}{5} = 45.583\ ft$$

Joist Spacings:

$$J_{f1\_c\_ext} := \frac{\frac{FIF(\text{“20'7”})}{4}}{2} = 2.573\ ft \qquad J_{f1\_int} := \frac{\frac{2 \cdot FIF(\text{“20'7”})}{4}}{2} = 5.146\ ft \qquad J_{f1\&f2\_c\_int} := \frac{\frac{FIF(\text{“20'7”})}{4} + \frac{FIF(\text{“25'0”})}{5}}{2} = 5.073\ ft$$

$$J_{f2\_int} := \frac{\frac{2 \cdot FIF(\text{“25'0”})}{5}}{2} = 5\ ft \qquad J_{f2\_c\_ext} := \frac{\frac{FIF(\text{“25'0”})}{5}}{2} = 2.5\ ft$$

DL:

$$P_{DL\_ext\_f1} := \frac{DL \cdot J_{f1\_c\_ext} \cdot L_3}{2} = 0.643\ kip \qquad P_{DL\_int\_f1} := \frac{DL \cdot J_{f1\_int} \cdot L_3}{2} = 1.286\ kip \qquad P_{DL\_int\_f1\&f2\_c} := \frac{DL \cdot J_{f1\&f2\_c\_int} \cdot L_3}{2} = 1.268\ kip$$

$$P_{DL\_int\_f2} := \frac{DL \cdot J_{f2\_int} \cdot L_3}{2} = 1.25\ kip \qquad P_{DL\_ext\_f2\_c} := \frac{DL \cdot J_{f2\_c\_ext} \cdot L_3}{2} = 0.625\ kip$$

LL:

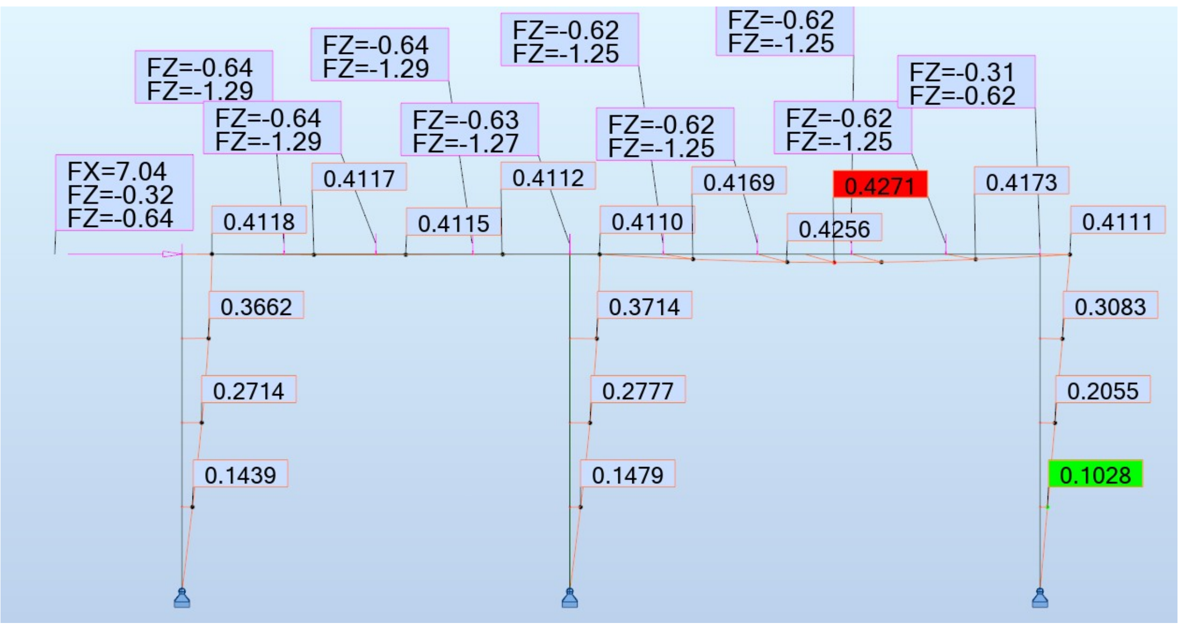
$$P_{LL\_ext\_f1} := \frac{LL \cdot J_{f1\_c\_ext} \cdot L_3}{2} = 0.643\ kip \qquad P_{LL\_int\_f1} := \frac{LL \cdot J_{f1\_int} \cdot L_3}{2} = 1.286\ kip \qquad P_{LL\_int\_f1\&f2\_c} := \frac{LL \cdot J_{f1\&f2\_c\_int} \cdot L_3}{2} = 1.268\ kip$$

$$P_{LL\_int\_f2} := \frac{LL \cdot J_{f2\_int} \cdot L_3}{2} = 1.25\ kip \qquad P_{LL\_ext\_f2\_c} := \frac{LL \cdot J_{f2\_c\_ext} \cdot L_3}{2} = 0.625\ kip$$

SL:

$$P_{SL\_ext\_f1} := \frac{SL \cdot J_{f1\_c\_ext} \cdot L_3}{2} = 0.515\ kip \qquad P_{SL\_int\_f1} := \frac{SL \cdot J_{f1\_int} \cdot L_3}{2} = 1.029\ kip \qquad P_{SL\_int\_f1\&f2\_c} := \frac{SL \cdot J_{f1\&f2\_c\_int} \cdot L_3}{2} = 1.015\ kip$$

$$P_{SL\_int\_f2} := \frac{SL \cdot J_{f2\_int} \cdot L_3}{2} = 1\ kip \qquad P_{SL\_ext\_f2\_c} := \frac{SL \cdot J_{f2\_c\_ext} \cdot L_3}{2} = 0.5\ kip$$



Member Sizes Utilized:

- MF-column: W 14 x 99
- MF-beam: W30x132
- GF-column: W 8x18
- GF-beam: W 18x60

$$\Delta_{max\_lateral} := \frac{18 \cdot 12\ in}{500} = 0.432\ in$$

N-S interior Wall Line:

$x_{f1\_1} := \frac{FIF(\text{“20'7”})}{4} = 5.146\ ft$ $x_{f1\_2} := \frac{2 \cdot FIF(\text{“20'7”})}{4} = 10.292\ ft$ $x_{f1\_3} := \frac{3 \cdot FIF(\text{“20'7”})}{4} = 15.438\ ft$

$x_{f1\_4} := \frac{4 \cdot FIF(\text{“20'7”})}{4} = 20.583\ ft$ $x_{f1\_5} := \frac{4 \cdot FIF(\text{“20'7”})}{4} + \frac{FIF(\text{“25'0”})}{5} = 25.583\ ft$

$x_{f1\_6} := \frac{4 \cdot FIF(\text{“20'7”})}{4} + \frac{2 \cdot FIF(\text{“25'0”})}{5} = 30.583\ ft$ $x_{f1\_7} := \frac{4 \cdot FIF(\text{“20'7”})}{4} + \frac{3 \cdot FIF(\text{“25'0”})}{5} = 35.583\ ft$

$x_{f1\_8} := \frac{4 \cdot FIF(\text{“20'7”})}{4} + \frac{4 \cdot FIF(\text{“25'0”})}{5} = 40.583\ ft$ $x_{f1\_9} := \frac{4 \cdot FIF(\text{“20'7”})}{4} + \frac{5 \cdot FIF(\text{“25'0”})}{5} = 45.583\ ft$

$x_{f2\_1} := 45.583\ ft + \frac{1 \cdot FIF(\text{“30'0”})}{6} = 50.583\ ft$ $x_{f2\_2} := 45.583\ ft + \frac{2 \cdot FIF(\text{“30'0”})}{6} = 55.583\ ft$

$x_{f2\_3} := 45.583\ ft + \frac{3 \cdot FIF(\text{“30'0”})}{6} = 60.583\ ft$ $x_{f2\_4} := 45.583\ ft + \frac{4 \cdot FIF(\text{“30'0”})}{6} = 65.583\ ft$

$x_{f2\_5} := 45.583\ ft + \frac{5 \cdot FIF(\text{“30'0”})}{6} = 70.583\ ft$ $x_{f2\_6} := 45.583\ ft + \frac{6 \cdot FIF(\text{“30'0”})}{6} = 75.583\ ft$

$x_{f3\_1} := 75.583\ ft + \frac{FIF(\text{“20'5”})}{4} = 80.687\ ft$ $x_{f3\_2} := 75.583\ ft + \frac{2 \cdot FIF(\text{“20'5”})}{4} = 85.791\ ft$

$x_{f3\_3} := 75.583\ ft + \frac{3 \cdot FIF(\text{“20'5”})}{4} = 90.896\ ft$ $x_{f3\_3} := 75.583\ ft + \frac{4 \cdot FIF(\text{“20'5”})}{4} = 96\ ft$

Joist Spacings:

$J_{f1\_c\_ext} := \frac{\frac{FIF(\text{“20'7”})}{4}}{2} = 2.573\ ft$ $J_{f1\_int} := \frac{\frac{2 \cdot FIF(\text{“20'7”})}{4}}{2} = 5.146\ ft$

$J_{f1\&f2\_c\_int} := \frac{\frac{FIF(\text{“20'7”})}{4} + \frac{FIF(\text{“25'0”})}{5}}{2} = 5.073\ ft$ $J_{f2\_int} := \frac{\frac{2 \cdot FIF(\text{“25'0”})}{5}}{2} = 5\ ft$

$J_{f2\&f3\_c\_int} := \frac{\frac{FIF(\text{“25'0”})}{5} + \frac{FIF(\text{“30'0”})}{6}}{2} = 5\ ft$ $J_{f3\_int} := \frac{\frac{2 \cdot FIF(\text{“30'0”})}{6}}{2} = 5\ ft$

$J_{f3\&f4\_c\_int} := \frac{\frac{FIF(\text{“30'0”})}{6} + \frac{FIF(\text{“20'5”})}{4}}{2} = 5.052\ ft$ $J_{f4\_int} := \frac{\frac{2 \cdot FIF(\text{“20'5”})}{4}}{2} = 5.104\ ft$

$J_{f4\_c\_ext} := \frac{\frac{FIF(\text{“20'5”})}{4}}{2} = 2.552\ ft$

DL:

$P_{DL\_ext\_f1} := \frac{DL \cdot J_{f1\_c\_ext} \cdot L_1}{2} + \frac{DL \cdot J_{f1\_c\_ext} \cdot L_2}{2} = 1.569\ kip$ $P_{DL\_int\_f1} := \frac{DL \cdot J_{f1\_int} \cdot L_1}{2} + \frac{DL \cdot J_{f1\_int} \cdot L_2}{2} = 3.139\ kip$

$P_{DL\_int\_f1\&f2\_c} := \frac{DL \cdot J_{f1\&f2\_c\_int} \cdot L_1}{2} + \frac{DL \cdot J_{f1\&f2\_c\_int} \cdot L_2}{2} = 3.094\ kip$ $P_{DL\_int\_f2} := \frac{DL \cdot J_{f2\_int} \cdot L_1}{2} + \frac{DL \cdot J_{f2\_int} \cdot L_2}{2} = 3.05\ kip$

$P_{DL\_int\_f2\&f3\_c} := \frac{DL \cdot J_{f2\&f3\_c\_int} \cdot L_1}{2} + \frac{DL \cdot J_{f2\&f3\_c\_int} \cdot L_2}{2} = 3.05\ kip$ $P_{DL\_int\_f3} := \frac{DL \cdot J_{f3\_int} \cdot L_1}{2} = 1.75\ kip$

$P_{DL\_int\_f3\&f4\_c} := \frac{DL \cdot J_{f3\&f4\_c\_int} \cdot L_1}{2} = 1.768\ kip$ $P_{DL\_int\_f4} := \frac{DL \cdot J_{f4\_int} \cdot L_1}{2} = 1.786\ kip$

$P_{DL\_ext\_f4\_c} := \frac{DL \cdot J_{f4\_c\_ext} \cdot L_1}{2} = 0.893\ kip$

LL:

$P_{LL\_ext\_f1} := \frac{LL \cdot J_{f1\_c\_ext} \cdot L_1}{2} + \frac{LL \cdot J_{f1\_c\_ext} \cdot L_2}{2} = 1.569\ kip$ $P_{LL\_int\_f1} := \frac{LL \cdot J_{f1\_int} \cdot L_1}{2} + \frac{LL \cdot J_{f1\_int} \cdot L_2}{2} = 3.139\ kip$

$P_{LL\_int\_f1\&f2\_c} := \frac{LL \cdot J_{f1\&f2\_c\_int} \cdot L_1}{2} + \frac{LL \cdot J_{f1\&f2\_c\_int} \cdot L_2}{2} = 3.094\ kip$ $P_{LL\_int\_f2} := \frac{LL \cdot J_{f2\_int} \cdot L_1}{2} + \frac{LL \cdot J_{f2\_int} \cdot L_2}{2} = 3.05\ kip$

$P_{LL\_int\_f2\&f3\_c} := \frac{LL \cdot J_{f2\&f3\_c\_int} \cdot L_1}{2} + \frac{LL \cdot J_{f2\&f3\_c\_int} \cdot L_2}{2} = 3.05\ kip$ $P_{LL\_int\_f3} := \frac{LL \cdot J_{f3\_int} \cdot L_1}{2} = 1.75\ kip$

$$P_{LL\_int\_f3\&f4\_c} := \frac{LL \cdot J_{f3\&f4\_c\_int} \cdot L_1}{2} = 1.768\ kip \qquad P_{LL\_int\_f4} := \frac{LL \cdot J_{f4\_int} \cdot L_1}{2} = 1.786\ kip$$

$$P_{LL\_ext\_f4\_c} := \frac{LL \cdot J_{f4\_c\_ext} \cdot L_1}{2} = 0.893\ kip$$

SL:

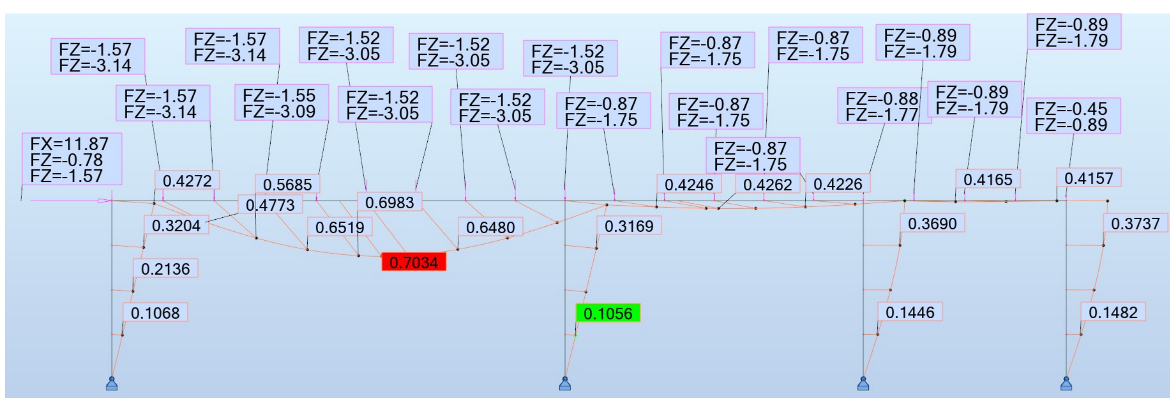
$$P_{SL\_ext\_f1} := \frac{SL \cdot J_{f1\_c\_ext} \cdot L_1}{2} + \frac{SL \cdot J_{f1\_c\_ext} \cdot L_2}{2} = 1.256\ kip \qquad P_{SL\_int\_f1} := \frac{SL \cdot J_{f1\_int} \cdot L_1}{2} + \frac{SL \cdot J_{f1\_int} \cdot L_2}{2} = 2.511\ kip$$

$$P_{SL\_int\_f1\&f2\_c} := \frac{SL \cdot J_{f1\&f2\_c\_int} \cdot L_1}{2} + \frac{SL \cdot J_{f1\&f2\_c\_int} \cdot L_2}{2} = 2.476\ kip \qquad P_{SL\_int\_f2} := \frac{SL \cdot J_{f2\_int} \cdot L_1}{2} + \frac{SL \cdot J_{f2\_int} \cdot L_2}{2} = 2.44\ kip$$

$$P_{SL\_int\_f2\&f3\_c} := \frac{SL \cdot J_{f2\&f3\_c\_int} \cdot L_1}{2} + \frac{SL \cdot J_{f2\&f3\_c\_int} \cdot L_2}{2} = 2.44\ kip \qquad P_{SL\_int\_f3} := \frac{SL \cdot J_{f3\_int} \cdot L_1}{2} = 1.4\ kip$$

$$P_{SL\_int\_f3\&f4\_c} := \frac{SL \cdot J_{f3\&f4\_c\_int} \cdot L_1}{2} = 1.415\ kip \qquad P_{SL\_int\_f4} := \frac{SL \cdot J_{f4\_int} \cdot L_1}{2} = 1.429\ kip$$

$$P_{SL\_ext\_f4\_c} := \frac{SL \cdot J_{f4\_c\_ext} \cdot L_1}{2} = 0.715\ kip$$



Member Sizes Utilized:

- MF-column: W 14x159
- MF-beam: W 33x141
- GF-column: W 8x18
- GF-beam: W 30x132

$$\Delta_{max\_lateral} := \frac{18 \cdot 12\ in}{500} = 0.432\ in$$

E-W North Wall Line:

Joist Spacings:

$$J_{f1\_c\_ext} := \frac{\frac{FIF(\text{“}20'7\text{”})}{4}}{2} = 2.573\ ft$$

DL:

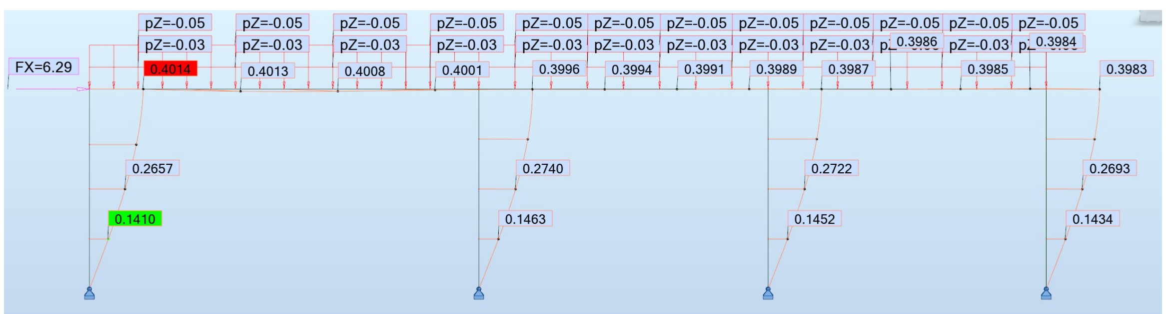
$$w_{DL} := DL \cdot J_{f1\_c\_ext} = 0.051\ \frac{kip}{ft}$$

LL:

$$w_{LL} := LL \cdot J_{f1\_c\_ext} = 0.051\ \frac{kip}{ft}$$

SL:

$$w_{SL} := SL \cdot J_{f1\_c\_ext} = 0.041\ \frac{kip}{ft}$$



Member Sizes Utilized:

- MF-column exterior: W 14x48
- MF-beam: W 30x132

$$\Delta_{max\_lateral} := \frac{18 \cdot 12\ in}{500} = 0.432\ in$$

E-W South Wall Line:

Joist Spacings:

$$J_{f1\_c\_ext} := \frac{\frac{FIF(\text{“}20'5\text{”})}{4}}{2} = 2.552\ ft$$

DL:

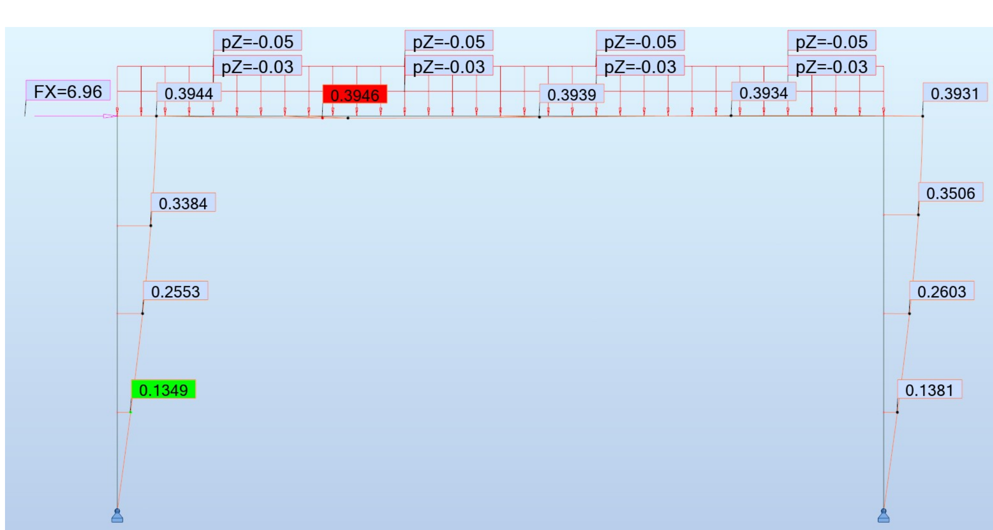
$$w_{DL} := DL \cdot J_{f1\_c\_ext} = 0.051\ \frac{kip}{ft}$$

LL:

$$w_{LL} := LL \cdot J_{f1\_c\_ext} = 0.051\ \frac{kip}{ft}$$

SL:

$$w_{SL} := SL \cdot J_{f1\_c\_ext} = 0.041\ \frac{kip}{ft}$$



Member Sizes Utilized:

- MF-column exterior: W 14x109
- MF-beam: W 30x132

$$\Delta_{max\_lateral} := \frac{18 \cdot 12\ in}{500} = 0.432\ in$$

E-W Interior Wall Line:

Joist Spacings:

$$J_{f1\_c\_ext} := \frac{\frac{FIF(\text{“}25'0\text{”})}{5}}{2} = 2.5\ ft \qquad J_{f2\&f3\_c\_int} := \frac{\frac{FIF(\text{“}25'0\text{”})}{5} + \frac{FIF(\text{“}30'0\text{”})}{6}}{2} = 5\ ft$$

DL:

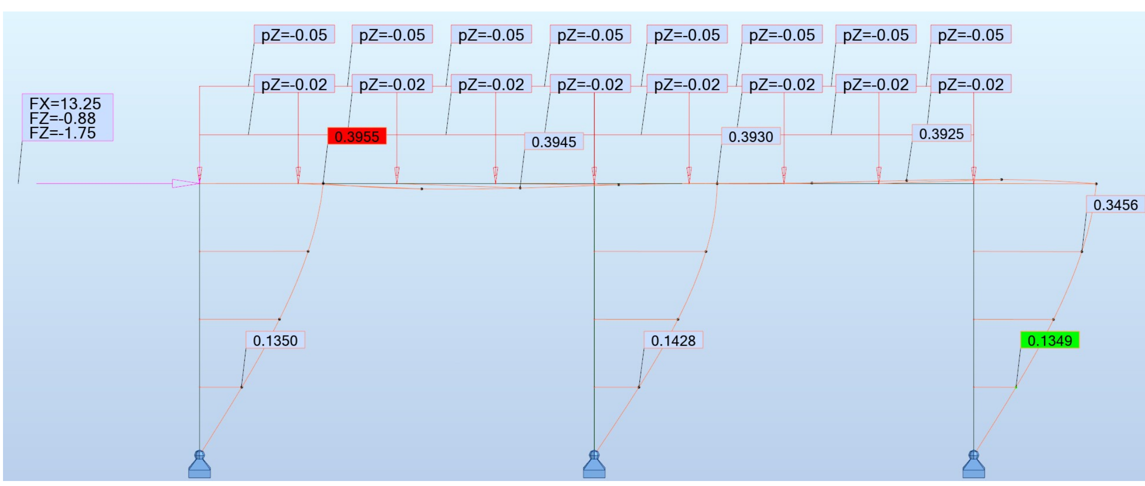
$$w_{DL} := DL \cdot J_{f1\_c\_ext} = 0.05\ \frac{kip}{ft} \qquad P_{DL} := \frac{DL \cdot J_{f2\&f3\_c\_int} \cdot L_1}{2} = 1.75\ kip$$

LL:

$$w_{LL} := LL \cdot J_{f1\_c\_ext} = 0.05\ \frac{kip}{ft} \qquad P_{LL} := \frac{LL \cdot J_{f2\&f3\_c\_int} \cdot L_1}{2} = 1.75\ kip$$

SL:

$$w_{SL} := SL \cdot J_{f1\_c\_ext} = 0.04\ \frac{kip}{ft} \qquad P_{SL} := \frac{SL \cdot J_{f1\&f2\_c\_int} \cdot L_1}{2} = 1.42\ kip$$



Member Sizes Utilized:

- MF-column exterior: W 14x132
- MF-beam: W 30x132

$$\Delta_{max\_lateral} := \frac{18 \cdot 12\ in}{500} = 0.432\ in$$

Moment Frame Calculation Summary

N-S West Wall:

Member Sizes Utilized:

- MF-column: W 10x77
- MF-beam: W18x60
- GF-column: W 8x18
- GF-beam: W 18x60

N-S East Wall:

Member Sizes Utilized:

- MF-column: W 14x99
- MF-beam: W30x132
- GF-column: W 8x18
- GF-beam: W 18x60

N-S interior Wall Line:

Member Sizes Utilized:

- MF-column: W 14x159
- MF-beam: W 33x141
- GF-column: W 8x18
- GF-beam: W 30x132

E-W North Wall Line:

Member Sizes Utilized:

- MF-column exterior: W 14x48
- MF-beam: W 30x132

E-W South Wall Line:

Member Sizes Utilized:

- MF-column: W 14x109
- MF-beam: W 30x132

E-W North Wall Line:

Member Sizes Utilized:

- MF-column: W 14x132
- MF-beam: W 30x132

Final Sizes: Below (in blue)

N-S West Wall:

Member Sizes Utilized:

- MF-column: W 14x48
- MF-beam: W18x60
- GF-column: W 8x18
- GF-beam: W 18x60

N-S East Wall:

Member Sizes Utilized:

- MF-column: W 14x132
- MF-beam: W30x132
- GF-column: W 8x18
- GF-beam: W 18x60

N-S interior Wall Line:

Member Sizes Utilized:

- MF-column: W 14x159
- MF-beam: W 33x141
- GF-column: W 8x18
- GF-beam: W 30x132

E-W North Wall Line:

Member Sizes Utilized:

- MF-column exterior: W 14x48
- MF-beam: W 30x132

E-W South Wall Line:

Member Sizes Utilized:

- MF-column: W 14x132
- MF-beam: W 30x132

E-W Interior Wall Line:

Member Sizes Utilized:

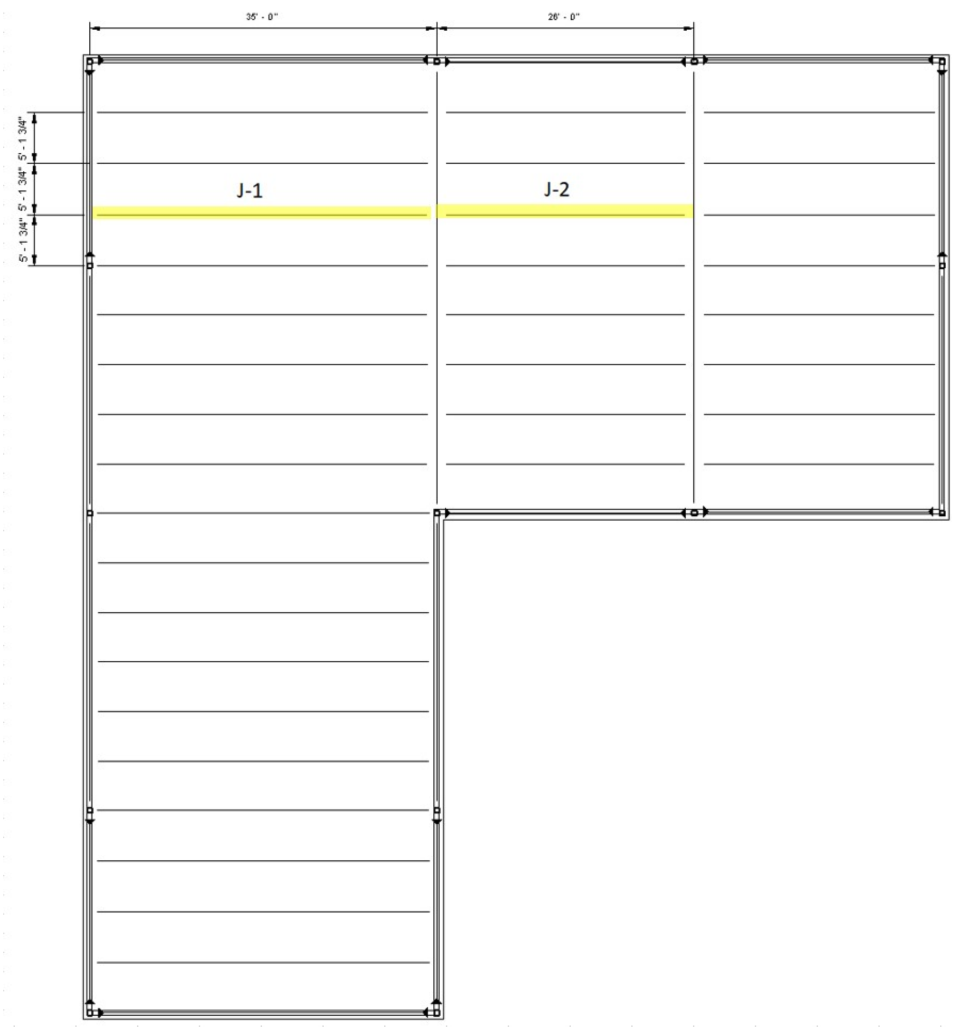
- MF-column: W 14x132
- MF-beam: W 30x132

Appendix P: Building C: Column, Girder and Joist Gravity Analysis

Factored Roof Uniform Area Load:

$$q_{roof} := 69.6\ psf \qquad \text{(LRFD)}$$

Select Joists with most critical loading



Joist J-1 Analysis:

Using Vulcraft Steel Joist Catalog, Select a K-Series 26K8 (26 in. depth):

$$w_{trib\_J1} := \frac{FIF(\text{“}5'\ 1-3/4\text{”}) + FIF(\text{“}5'\ 1-3/4\text{”})}{2} = 5.146\ ft \qquad w_{self} := 9.7\ plf \qquad L := 35\ ft \qquad E := 29000\ ksi$$

$$DL_{uf} := (D_{Lroof} \cdot w_{trib\_J1}) + (S_{roof} \cdot w_{trib\_J1}) + w_{self} = 194.95\ plf$$

$$LL_{uf} := L_{Lroof} \cdot w_{trib\_J1} = 102.917\ plf$$

$$TotalLoad_{UF} := DL_{uf} + LL_{uf} = 297.867\ plf$$

$$TotalLoad_{F} := 1.2 \cdot (DL_{uf}) + 1.6 \cdot (LL_{uf}) = 398.607\ plf$$

The approximate gross moment of inertia (not adjusted for shear deformation) of a standard joist listed in the Load Table may be determined as follows:

$I_j = 26.767(W)(L^3)(10^{-6})$ in^4 or $2.6953(W)(L^3)(10^{-5})$ mm^4, where W= **RED** figure in the Load Table, and
L = (span – 0.33) in feet or (span – 102) in millimeters

$$I_g := 26.767 \cdot 286 \cdot (35 - 0.33)^3 \cdot (10^{-6}) = 319.027$$

$$I_g := 319.027\ in^4$$

Allowable Deflection due to unfactored total load:

$$\Delta_{allowable} := \frac{L}{360} = 1.167\ in$$

Actual deflection experienced:

$$\Delta_{max} := \frac{5 \cdot TotalLoad_{UF} \cdot L^4}{384 \cdot E \cdot I_g} = 1.087\ in$$

Based on the deflection criteria, a K-26K8 joist is acceptable.

Joist J-2 Analysis:

Using Vulcraft Steel Joist Catalog, Select a K-Series 18K3 (18 in. depth):

$$w_{trib\_J1} := \frac{\boldsymbol{FIF}\left(\text{“5' 1−3/4”}\right) + \boldsymbol{FIF}\left(\text{“5' 1−3/4”}\right)}{2} = 5.146\ \boldsymbol{ft} \qquad w_{self} := 6.4\ \boldsymbol{plf} \qquad L := 26\ \boldsymbol{ft} \qquad E := 29000\ \boldsymbol{ksi}$$

$$DL_{uf} := \left(D_{Lroof} \cdot w_{trib\_J1}\right) + \left(S_{roof} \cdot w_{trib\_J1}\right) + w_{self} = 191.65\ \boldsymbol{plf}$$

$$LL_{uf} := L_{Lroof} \cdot w_{trib\_J1} = 102.917\ \boldsymbol{plf}$$

$$TotalLoad_{UF} := DL_{uf} + LL_{uf} = 294.567\ \boldsymbol{plf}$$

$$TotalLoad_{F} := 1.2 \cdot \left(DL_{uf}\right) + 1.6 \cdot \left(LL_{uf}\right) = 394.647\ \boldsymbol{plf}$$

The approximate gross moment of inertia (not adjusted for shear deformation) of a standard joist listed in the Load Table may be determined as follows:

I_j = 26.767(W)(L^3)(10^{-6}) in^4 or 2.6953(W)(L^3)(10^{-5}) mm^4, where W= **RED** figure in the Load Table, and
L = (span – 0.33) in feet or (span – 102) in millimeters

$$I_g := 26.767 \cdot 190 \cdot \left(35 - 0.33\right)^3 \cdot \left(10^{-6}\right) = 211.941$$

$$I_g := 211.941\ \boldsymbol{in}^4$$

Allowable Deflection due to unfactored total load:

$$\Delta_{allowable} := \frac{L}{360} = 0.867\ \boldsymbol{in}$$

Actual deflection experienced:

$$\Delta_{max} := \frac{5 \cdot TotalLoad_{UF} \cdot L^4}{384 \cdot E \cdot I_g} = 0.493\ \boldsymbol{in}$$

Based on the deflection criteria, a K-18K3 joist is acceptable.

Girder G-1 Analysis:

Section: W30x132

$$W_{TributaryG1} := 30.5\ ft \qquad w_{self} := 132\ \frac{lbf}{ft} = 0.132\ klf$$

$$w_{SelfFactored} := 1.2 \cdot w_{self} = 0.158\ klf$$

$$w_2 := \left(q_{roof} \cdot W_{TributaryG1}\right) + w_{SelfFactored} = 2.281\ klf$$ (Factored load with self weight)

Check Deflection:

$$L := 46\ ft \qquad E := 29000\ ksi \qquad I_x := 5770\ in^4$$

$$\Delta_{allowable} := \frac{L}{360} = 1.533\ in \qquad \Delta_{max} := \frac{5 \cdot w_2 \cdot L^4}{384 \cdot E \cdot I_x} = 1.373\ in$$

Section has adequate deflection requirements

FLEXURE: Check Compact vs Slender:

$b_f := 10.5\ in \qquad t_f := 1\ in \qquad F_y := 50\ ksi$

Width to thickness ratio for flanges of doubly symmetric I-section:

$$\frac{b_f}{2\ t_f} = 5.25$$

Limiting Width to Thickness Ratios:

$$\lambda_p := 0.38 \cdot \sqrt{\frac{E}{F_y}} = 9.152 \qquad \lambda_r := 1.0 \cdot \sqrt{\frac{E}{F_y}} = 24.083$$

$$\frac{b_f}{2\ t_f} < \lambda_p < \lambda_r$$

therefore the flanges are compact

Width to thickness ratio for web of doubly symmetric I-section:

$$\frac{h}{t_w} = \ h_{div} t_w := 43.9$$

Limiting Width to Thickness Ratios:

$$\lambda_p := 3.76 \cdot \sqrt{\frac{E}{F_y}} = 90.553 \qquad \lambda_r := 5.70 \cdot \sqrt{\frac{E}{F_y}} = 137.274$$

$$\frac{h}{t_w} < \lambda_p < \lambda_r$$

therefore the web is compact

Flange local buckling will not occur.

Doubly Symmetric Compact I-Shape Subjected to Bending About the Major Axis:

Check Yielding:

$Z_x := 437\ in^3$

$$M_n := F_y \cdot Z_x = 1820.833\ kip \cdot ft$$

Check Lateral Torsional Buckling:

From AISC Section Tables:

$$I_y := 196\ in^4 \quad C_w := 42100\ in^6 \quad r_y := 2.25\ in \quad S_x := 380\ in^3 \quad h_o := 29.3\ in \quad J := 9.72\ in^4$$

$c := 1$ (for doubly symmetric I-shapes)

$$r_{ts} := \sqrt{\frac{\left(I_y \cdot C_w\right)^{.5}}{S_x}} = 2.749\ in$$

$$L_p := 1.76 \cdot r_y \cdot \sqrt{\frac{E}{F_y}} = 7.947\ ft$$

$$L_r := 1.95 \cdot r_{ts} \cdot \frac{E}{0.7 \cdot F_y} \cdot \sqrt{\frac{J \cdot c}{S_x \cdot h_o} + \sqrt{\left(\frac{J \cdot c}{S_x \cdot h_o}\right)^2 + 6.76 \cdot \left(\frac{0.7 \cdot F_y}{E}\right)^2}} = 23.791\ ft$$

$$L_b := 5\ ft$$

Lb < Lp, therefore the limit state of lateral torsional buckling does not apply.

Flexural strength:

$$\phi M_n := 0.9 \cdot M_n = 1638.75\ kip \cdot ft$$

Maximum Moment:

$$M_{max} := \frac{w_2 \cdot L^2}{8} = 603.377\ kip \cdot ft$$

The section has adequate flexural strength.

Check Web Buckling Due to Shear:

$d := 30.3\ in$ $\quad t_w := 0.615\ in$ $\quad A_w := d \cdot t_w$ $\quad$ Recall: $\frac{h}{t_w} = 43.9$

$\frac{h}{t_w} < 2.24 \cdot \sqrt{\frac{E}{F_y}} = 53.946$ $\quad$ therefore, $\quad \phi_v := 1.00 \quad C_{v1} := 1.00$

Shear strength:

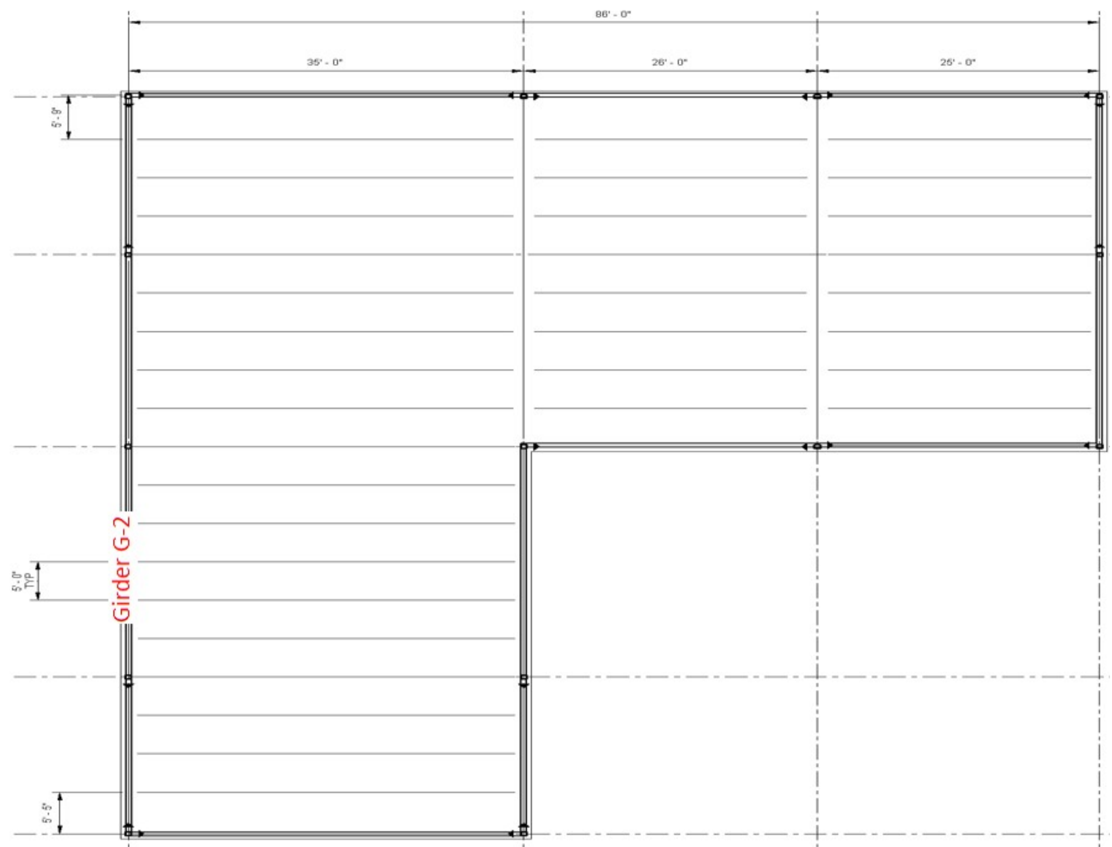
$$\phi V_n := \phi_v \cdot \left(0.6 \cdot F_y \cdot A_w \cdot C_{v1}\right) = 559.035\ kip$$

Maximum Shear:

$$V_{max} := \frac{w_2 \cdot L}{2} = 52.468\ kip$$

The section has adequate shear strength. Stiffeners not required.

Girder G-2 Analysis:
Section: W18x60



$W_{TributaryG2} := \dfrac{35\ ft}{2} = 17.5\ ft$ $\qquad w_{self} := 60\ \dfrac{lbf}{ft} = 0.06\ klf$

$$w_{SelfFactored} := 1.2 \cdot w_{self} = 0.072\ klf$$

$w_2 := \left(q_{roof} \cdot W_{TributaryG2}\right) + w_{SelfFactored} = 1.29\ klf$ (Factored load with self weight)

Check Deflection:

$$L := 30\ ft \qquad E := 29000\ ksi \qquad I_x := 984\ in^4$$

$$\Delta_{allowable} := \frac{L}{360} = 1\ in \qquad \Delta_{max} := \frac{5 \cdot w_2 \cdot L^4}{384 \cdot E \cdot I_x} = 0.824\ in$$

Section has adequate deflection requirements

FLEXURE: Check Compact vs Slender:

$b_f := 7.56\ in \qquad t_f := 0.695\ in \qquad F_y := 50\ ksi$

Width to thickness ratio for flanges of doubly symmetric I-section:

$$\frac{b_f}{2\ t_f} = 5.439$$

Limiting Width to Thickness Ratios:

$$\lambda_p := 0.38 \cdot \sqrt{\frac{E}{F_y}} = 9.152 \qquad \lambda_r := 1.0 \cdot \sqrt{\frac{E}{F_y}} = 24.083$$

$\dfrac{b_f}{2\ t_f} < \lambda_p < \lambda_r$

therefore the flanges are compact

Width to thickness ratio for web of doubly symmetric I-section:

$$\frac{h}{t_w} = \ h_{div}t_w := 38.7$$

Limiting Width to Thickness Ratios:

$$\lambda_p := 3.76 \cdot \sqrt{\frac{E}{F_y}} = 90.553 \qquad \lambda_r := 5.70 \cdot \sqrt{\frac{E}{F_y}} = 137.274$$

$\dfrac{h}{t_w} < \lambda_p < \lambda_r$

therefore the web is compact

Flange local buckling will not occur.

Doubly Symmetric Compact I-Shape Subjected to Bending About the Major Axis:

Check Yielding:

$Z_x := 123\ in^3$

$$M_n := F_y \cdot Z_x = 512.5\ kip \cdot ft$$

Check Lateral Torsional Buckling:

From AISC Section Tables:

$$I_y := 50.1\ in^4 \quad C_w := 3850\ in^6 \qquad r_y := 1.68\ in \quad S_x := 108\ in^3 \quad h_o := 17.5\ in \quad J := 2.17\ in^4$$

$c := 1$ (for doubly symmetric I-shapes)

$$r_{ts} := \sqrt{\frac{\left(I_y \cdot C_w\right)^{.5}}{S_x}} = 2.017 \ in$$

$$L_p := 1.76 \cdot r_y \cdot \sqrt{\frac{E}{F_y}} = 5.934 \ ft$$

$$L_r := 1.95 \cdot r_{ts} \cdot \frac{E}{0.7 \cdot F_y} \cdot \sqrt{\frac{J \cdot c}{S_x \cdot h_o} + \sqrt{\left(\frac{J \cdot c}{S_x \cdot h_o}\right)^2 + 6.76 \cdot \left(\frac{0.7 \cdot F_y}{E}\right)^2}} = 18.193 \ ft$$

$$L_b := 5 \ ft$$

Lb < Lp, therefore the limit state of lateral torsional buckling does not apply.

Flexural strength:

$$\phi M_n := 0.9 \cdot M_n = 461.25 \ kip \cdot ft$$

Maximum Moment:

$$M_{max} := \frac{w_2 \cdot L^2}{8} = 145.125 \ kip \cdot ft$$

The section has adequate flexural strength.

Check Web Buckling Due to Shear:

$d := 18.2 \ in$ $\quad t_w := 0.415 \ in$ $\quad A_w := d \cdot t_w$ $\quad$ Recall: $\frac{h}{t_w} = 38.7$

$\frac{h}{t_w} < 2.24 \cdot \sqrt{\frac{E}{F_y}} = 53.946$ $\quad$ therefore, $\quad \phi_v := 1.00 \quad C_{v1} := 1.00$

Shear strength:

$$\phi V_n := \phi_v \cdot \left(0.6 \cdot F_y \cdot A_w \cdot C_{v1}\right) = 226.59 \ kip$$

Maximum Shear:

$$V_{max} := \frac{w_2 \cdot L}{2} = 19.35 \ kip$$

The section has adequate shear strength. Stiffeners not required.

Girder G-3 Analysis:
Section: W33x141

$$W_{TributaryG3} := \frac{35\ ft}{2} = 17.5\ ft \qquad w_{self} := 141\ \frac{lbf}{ft} = 0.141\ klf$$

$$w_{SelfFactored} := 1.2 \cdot w_{self} = 0.169\ klf$$

$$w_3 := \left(q_{roof} \cdot W_{TributaryG3}\right) + w_{SelfFactored} = 1.387\ klf$$ (Factored load with self weight)

Check Deflection:

$$L := 20.5\ ft \qquad E := 29000\ ksi \qquad I_x := 7450\ in^4$$

$$\Delta_{allowable} := \frac{L}{360} = 0.683\ in \qquad \Delta_{max} := \frac{5 \cdot w_2 \cdot L^4}{384 \cdot E \cdot I_x} = 0.024\ in$$

Section has adequate deflection requirements

FLEXURE: Check Compact vs Slender:

$$b_f := 11.5\ in \qquad t_f := 0.960\ in \quad F_y := 50\ ksi$$

Width to thickness ratio for flanges of doubly symmetric I-section:

$$\frac{b_f}{2\ t_f} = 5.99$$

Limiting Width to Thickness Ratios:

$$\lambda_p := 0.38 \cdot \sqrt{\frac{E}{F_y}} = 9.152 \qquad \lambda_r := 1.0 \cdot \sqrt{\frac{E}{F_y}} = 24.083$$

$$\frac{b_f}{2\ t_f} < \lambda_p < \lambda_r$$

therefore the flanges are compact

Width to thickness ratio for web of doubly symmetric I-section:

$$\frac{h}{t_w} = \ h_{div} t_w := 49.6$$

Limiting Width to Thickness Ratios:

$$\lambda_p := 3.76 \cdot \sqrt{\frac{E}{F_y}} = 90.553 \qquad \lambda_r := 5.70 \cdot \sqrt{\frac{E}{F_y}} = 137.274$$

$$\frac{h}{t_w} < \lambda_p < \lambda_r$$

therefore the web is compact

Flange local buckling will not occur.

Doubly Symmetric Compact I-Shape Subjected to Bending About the Major Axis:

Check Yielding:

$$Z_x := 514\ in^3$$

$$M_n := F_y \cdot Z_x = 2141.667\ kip \cdot ft$$

Check Lateral Torsional Buckling:

From AISC Section Tables:

$$I_y := 246\ in^4 \quad C_w := 64400\ in^6 \quad r_y := 2.43\ in \quad S_x := 448\ in^3 \quad h_o := 32.3\ in \quad J := 9.70\ in^4$$

$$c := 1$$ (for doubly symmetric I-shapes)

$$r_{ts} := \sqrt{\frac{\left(I_y \cdot C_w\right)^{.5}}{S_x}} = 2.981\ in$$

$$L_p := 1.76 \cdot r_y \cdot \sqrt{\frac{E}{F_y}} = 8.583\ ft$$

$$L_r := 1.95 \cdot r_{ts} \cdot \frac{E}{0.7 \cdot F_y} \cdot \sqrt{\frac{J \cdot c}{S_x \cdot h_o} + \sqrt{\left(\frac{J \cdot c}{S_x \cdot h_o}\right)^2 + 6.76 \cdot \left(\frac{0.7 \cdot F_y}{E}\right)^2}} = 24.996\ ft$$

$$L_b := FIF\left(\text{“5' 1–1/4”}\right) = 5.104\ ft$$

Lb < Lp, therefore the limit state of lateral torsional buckling does not apply.

Flexural strength:

$$\phi M_n := 0.9 \cdot M_n = 1927.5\ kip \cdot ft$$

Maximum Moment:

$$M_{max} := \frac{w_3 \cdot L^2}{12} = 48.581\ kip \cdot ft$$

The section has adequate flexural strength.

Check Web Buckling Due to Shear:

$d := 33.3\ in$ $t_w := 0.605\ in$ $A_w := d \cdot t_w$ Recall: $\frac{h}{t_w} = 38.7$

$\frac{h}{t_w} < 2.24 \cdot \sqrt{\frac{E}{F_y}} = 53.946$ therefore, $\phi_v := 1.00$ $C_{v1} := 1.00$

Shear strength:

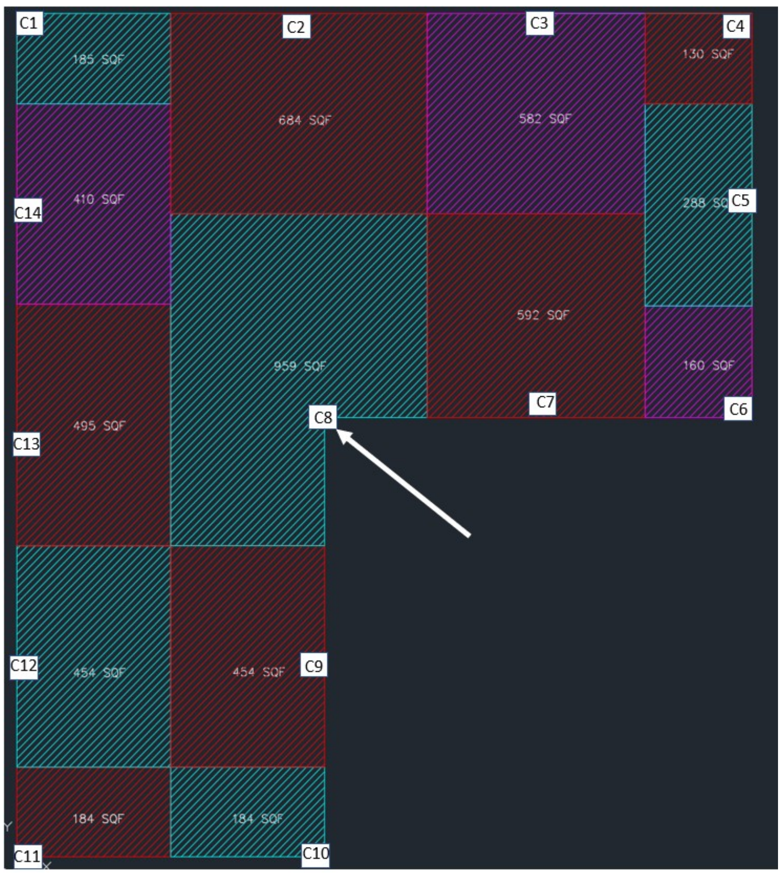
$$\phi V_n := \phi_v \cdot \left(0.6 \cdot F_y \cdot A_w \cdot C_{v1}\right) = 604.395\ kip$$

Maximum Shear:

$$V_{max} := \frac{w_2 \cdot L}{2} = 13.223\ kip$$

The section has adequate shear strength. Stiffeners not required.

Column C-8 (Moment Frame Column i.e. Pinned-Fixed) Analysis:
Section: W14x132



Check Nonslender vs Slender:

$b_f := 14.7\ in$ $\quad t_f := 1.03\ in$ $\quad F_y := 50\ ksi$

Width to thickness ratio for flanges of doubly symmetric I-section:

$$\frac{b_f}{2\ t_f} = 7.136$$

Limiting Width to Thickness Ratios:

$$\lambda_r := 0.56 \cdot \sqrt{\frac{E}{F_y}} = 13.487$$

$\frac{b_f}{2\ t_f} < \lambda_r$

therefore the flanges are nonslender

Width to thickness ratio for web of doubly symmetric I-section:

$$\frac{h}{t_w} = \ h_{div} t_w := 17.7$$

Limiting Width to Thickness Ratios:

$$\lambda_r := 1.49 \cdot \sqrt{\frac{E}{F_y}} = 35.884$$

$\frac{h}{t_w} < \lambda_r$

therefore the web is nonslender

Flexural Buckling of Doubly Symmetric I-Shape Members Without Slender Elements:

Column is Pinned-Fixed, therefore:

$$L := 18\ ft \qquad L_c := 0.80 \cdot L = 14.4\ ft$$

From AISC Tables:

$r_x := 6.28\ in$ $\quad I_x := 1530\ in^4$ $\quad A_g := 38.8\ in^2$ $\quad L_{cx} := L_c$ $\quad L_{cy} := L_c$ $\quad L_{cz} := L_c$

$r_y := 3.76\ in$ $\quad I_y := 548\ in^4$ $\quad J := 12.3\ in^4$ $\quad G := 11200\ ksi$ $\quad C_w := 25500\ in^6$

$$F_{ex} := \frac{\left(\pi^2 \cdot E\right)}{\left(\left(\frac{L_{cx}}{r_x}\right)^2\right)} = 378.033\ ksi \qquad F_{ey} := \frac{\left(\pi^2 \cdot E\right)}{\left(\left(\frac{L_{cy}}{r_y}\right)^2\right)} = 135.515\ ksi$$

Torsional and Flexural Torsional Buckling Consideration:

$$F_{ez} := \left(\frac{\left(\pi^2 \cdot E \cdot C_w\right)}{\left(L_{cz}^{\ 2}\right)} + G \cdot J\right) \cdot \frac{1}{\left(I_x + I_y\right)} = 183.921\ ksi$$

$$F_e := min\left(F_{ex}, F_{ey}, F_{ez}\right) = 135.515\ ksi$$ (Elastic Buckling Stress)

$$\frac{F_y}{F_e} = 0.369$$ < 2.25, therefore $F_{cr} = \left(0.658^{\frac{F_y}{F_e}}\right) \cdot F_y$

$$F_{cr} := \left(0.658^{\frac{F_y}{F_e}}\right) \cdot F_y = 42.845\ ksi$$ (Critical Buckling Stress)

Design Axial Compressive Strength:

$$P_n := F_{cr} \cdot A_g = 1662.393\ kip$$

$$\phi P_n := 0.9 \cdot P_n = 1496.153\ kip$$

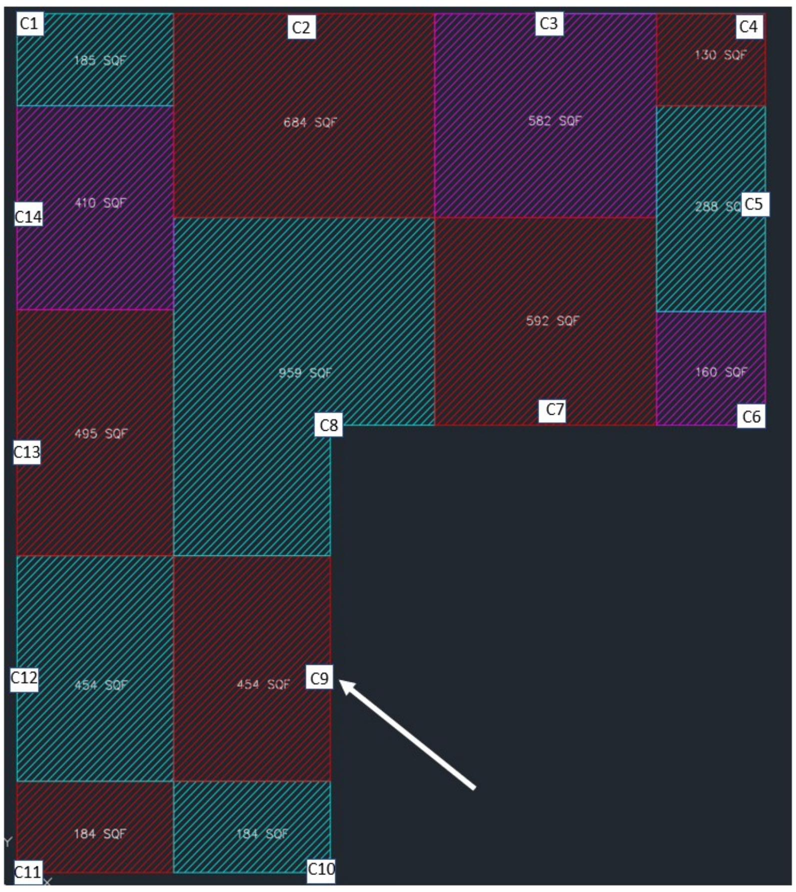
Maximum Axial Load:

$$A_{TributaryC8} := 960\ ft^2$$

$$P_u := \left(q_{roof} \cdot A_{TributaryC8}\right) = 66.816\ kip$$

The section has adequate axial strength.

<u>Column C-9 (Moment Frame Column i.e. Pinned-Fixed) Analysis:</u>
Section: W14x159



Check Nonslender vs Slender:

$b_f := 15.6\ in$ $\quad t_f := 1.19\ in$ $\quad F_y := 50\ ksi$

Width to thickness ratio for flanges of doubly symmetric I-section:

$$\frac{b_f}{2\ t_f} = 6.555$$

Limiting Width to Thickness Ratios:

$$\lambda_r := 0.56 \cdot \sqrt{\frac{E}{F_y}} = 13.487$$

$\frac{b_f}{2\ t_f} < \lambda_r$

therefore the flanges are nonslender

Width to thickness ratio for web of doubly symmetric I-section:

$$\frac{h}{t_w} = \ h_{div}t_w := 15.3$$

Limiting Width to Thickness Ratios:

$$\lambda_r := 1.49 \cdot \sqrt{\frac{E}{F_y}} = 35.884$$

$\frac{h}{t_w} < \lambda_r$

therefore the web is nonslender

Flexural Buckling of Doubly Symmetric I-Shape Members Without Slender Elements:

Column is Pinned-Fixed, therefore:

$$L := 18\ ft \qquad L_c := 0.80 \cdot L = 14.4\ ft$$

From AISC Tables:

$r_x := 6.38\ in$ $\quad I_x := 1900\ in^4$ $\quad A_g := 46.7\ in^2$ $\quad L_{cx} := L_c$ $\quad L_{cy} := L_c$ $\quad L_{cz} := L_c$

$r_y := 4.00\ in$ $\quad I_y := 748\ in^4$ $\quad J := 19.7\ in^4$ $\quad G := 11200\ ksi$ $\quad C_w := 35600\ in^6$

$$F_{ex} := \frac{\left(\pi^2 \cdot E\right)}{\left(\left(\frac{L_{cx}}{r_x}\right)^2\right)} = 390.168\ ksi \qquad F_{ey} := \frac{\left(\pi^2 \cdot E\right)}{\left(\left(\frac{L_{cy}}{r_y}\right)^2\right)} = 153.366\ ksi$$

Torsional and Flexural Torsional Buckling Consideration:

$$F_{ez} := \left(\frac{\left(\pi^2 \cdot E \cdot C_w\right)}{\left(L_{cz}^{\ 2}\right)} + G \cdot J\right) \cdot \frac{1}{\left(I_x + I_y\right)} = 212.19\ ksi$$

$$F_e := min\left(F_{ex}, F_{ey}, F_{ez}\right) = 153.366\ ksi$$ (Elastic Buckling Stress)

$\frac{F_y}{F_e} = 0.326$ < 2.25, therefore $F_{cr} = \left(0.658^{\frac{F_y}{F_e}}\right) \cdot F_y$

$$F_{cr} := \left(0.658^{\frac{F_y}{F_e}}\right) \cdot F_y = 43.622\ ksi$$ (Critical Buckling Stress)

Design Axial Compressive Strength:

$$P_n := F_{cr} \cdot A_g = 2037.162\ kip$$

$$\phi P_n := 0.9 \cdot P_n = 1833.446\ kip$$

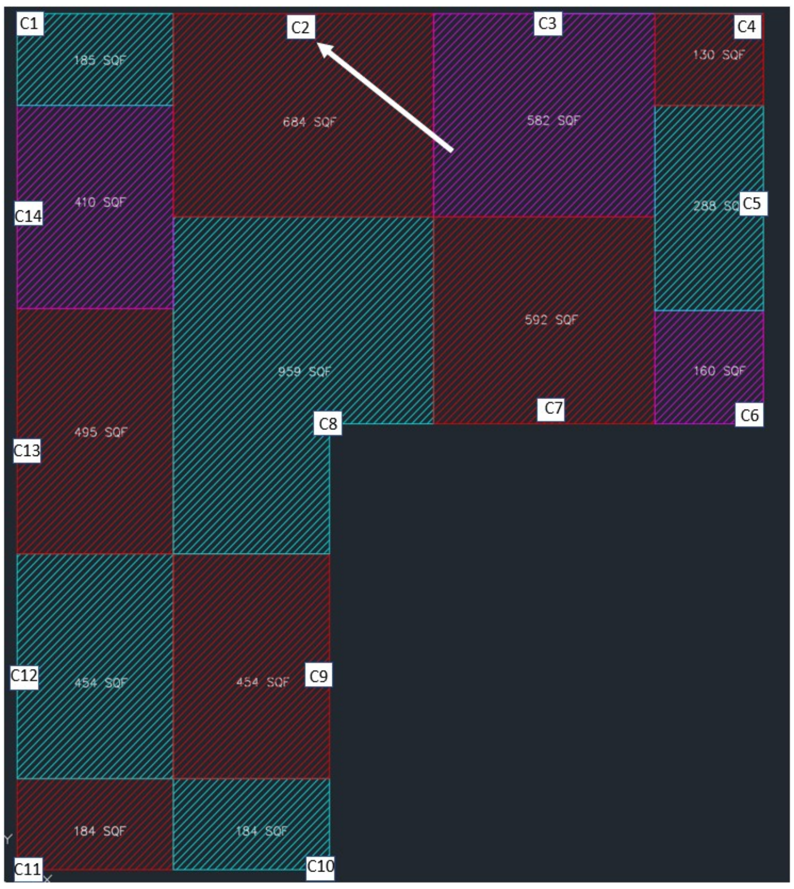
Maximum Axial Load:

$$A_{TributaryC9} := 454\ ft^2$$

$$P_u := \left(q_{roof} \cdot A_{TributaryC9}\right) = 31.598\ kip$$

The section has adequate axial strength.

Column C-2 (Moment Frame Column i.e. Pinned-Fixed) Analysis:
Section: W14x48



Check Nonslender vs Slender:

$b_f := 8.03\ in$ $\quad t_f := 0.595\ in$ $\quad F_y := 50\ ksi$

Width to thickness ratio for flanges of doubly symmetric I-section:

$$\frac{b_f}{2\ t_f} = 6.748$$

Limiting Width to Thickness Ratios:

$$\lambda_r := 0.56 \cdot \sqrt{\frac{E}{F_y}} = 13.487$$

$\frac{b_f}{2\ t_f} < \lambda_r$

therefore the flanges are nonslender

Width to thickness ratio for web of doubly symmetric I-section:

$$\frac{h}{t_w} = \ h_{div}t_w := 33.6$$

Limiting Width to Thickness Ratios:

$$\lambda_r := 1.49 \cdot \sqrt{\frac{E}{F_y}} = 35.884$$

$\frac{h}{t_w} < \lambda_r$

therefore the web is nonslender

Flexural Buckling of Doubly Symmetric I-Shape Members Without Slender Elements:

Column is Pinned-Fixed, therefore:

$$L := 18\ ft \qquad L_c := 0.8 \cdot L = 14.4\ ft$$

From AISC Tables:

$r_x := 5.85\ in$ $\quad I_x := 484\ in^4$ $\quad A_g := 14.1\ in^2$ $\quad L_{cx} := L_c$ $\quad L_{cy} := L_c$ $\quad L_{cz} := L_c$

$r_y := 1.91\ in$ $\quad I_y := 51.4\ in^4$ $\quad J := 1.45\ in^4$ $\quad G := 11200\ ksi$ $\quad C_w := 2240\ in^6$

$$F_{ex} := \frac{(\pi^2 \cdot E)}{\left(\left(\frac{L_{cx}}{r_x}\right)^2\right)} = 328.036\ ksi \qquad F_{ey} := \frac{(\pi^2 \cdot E)}{\left(\left(\frac{L_{cy}}{r_y}\right)^2\right)} = 34.968\ ksi$$

Torsional and Flexural Torsional Buckling Consideration:

$$F_{ez} := \left(\frac{(\pi^2 \cdot E \cdot C_w)}{(L_{cz}^{\ 2})} + G \cdot J\right) \cdot \frac{1}{(I_x + I_y)} = 70.436\ ksi$$

$$F_e := min(F_{ex}, F_{ey}, F_{ez}) = 34.968\ ksi$$ (Elastic Buckling Stress)

$\frac{F_y}{F_e} = 1.43$ < 2.25, therefore $F_{cr} = \left(0.658^{\frac{F_y}{F_e}}\right) \cdot F_y$

$$F_{cr} := \left(0.658^{\frac{F_y}{F_e}}\right) \cdot F_y = 27.483\ ksi$$ (Critical Buckling Stress)

Design Axial Compressive Strength:

$$P_n := F_{cr} \cdot A_g = 387.506\ kip$$

$$\phi P_n := 0.9 \cdot P_n = 348.755\ kip$$

Maximum Axial Load:

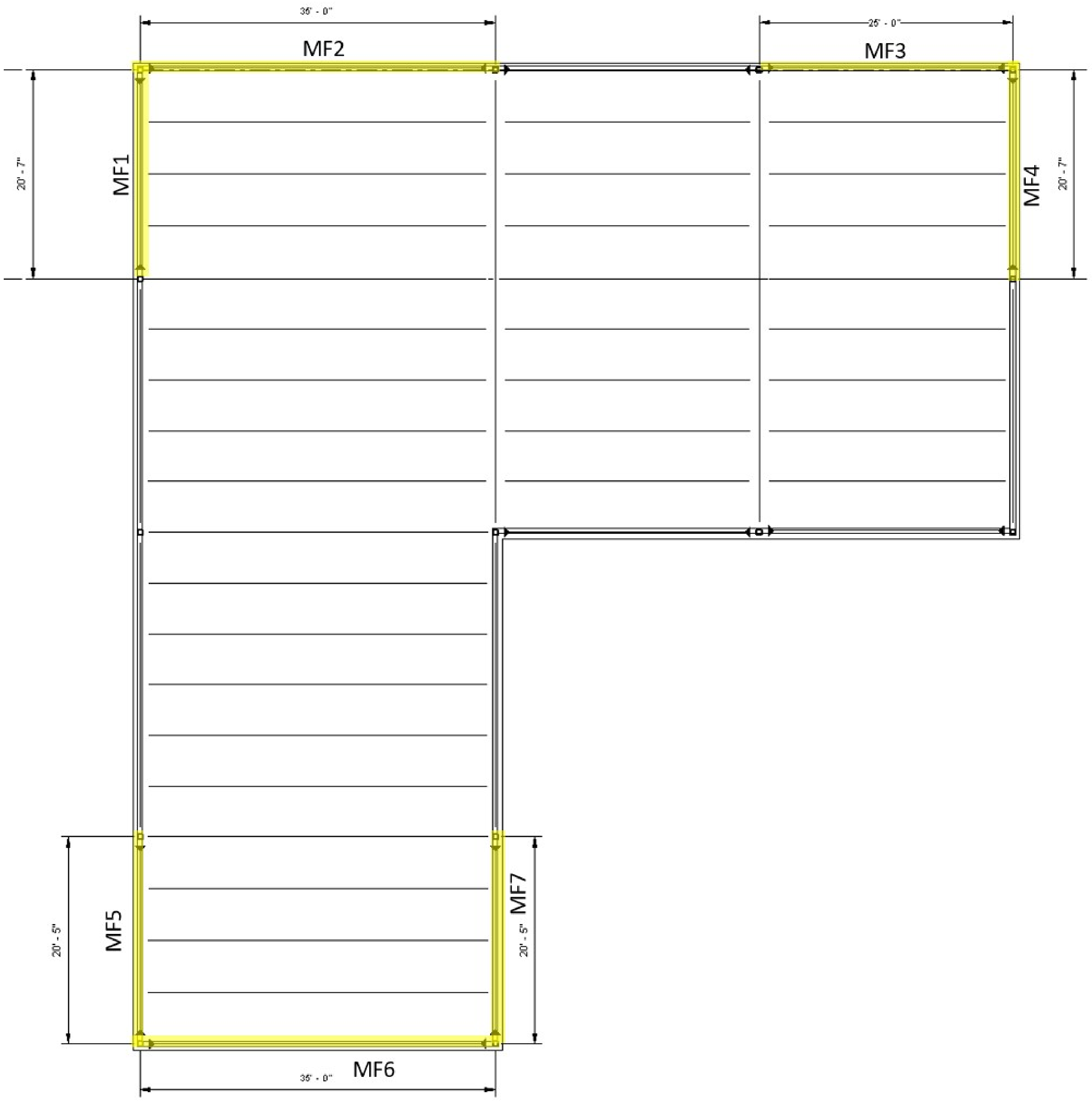
$$A_{TributaryC2} := 684\ ft^2$$

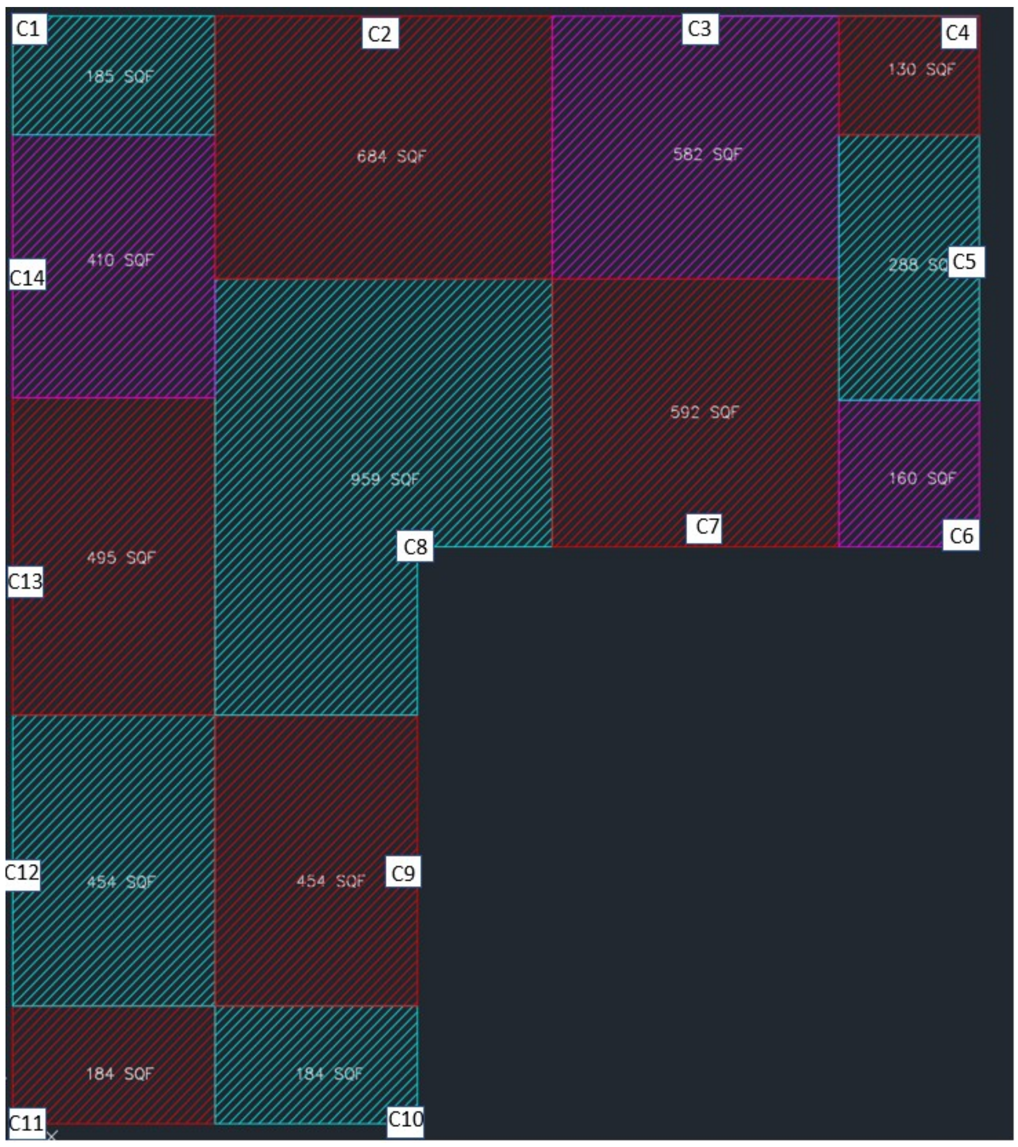
$$P_u := (q_{roof} \cdot A_{TributaryC2}) = 47.606\ kip$$

The section has adequate axial strength.

Appendix Q: Building C: Combined Loading and Final Member Sizes

Moment Frames Combined Forces - Flexure and Axial Force Analysis:





MF1 - W18x60 Beam, W14x48 Column:

Combined Forces - Flexure and Axial Force:

Axial Load (Column):

Check Nonslender vs Slender:

$b_f := 8.03\ in \qquad t_f := 0.595\ in \qquad F_y := 50\ ksi$

Width to thickness ratio for flanges of doubly symmetric I-section:

$$\frac{b_f}{2\ t_f} = 6.748$$

Limiting Width to Thickness Ratios:

$$\lambda_r := 0.56 \cdot \sqrt{\frac{E}{F_y}} = 13.487$$

$$\frac{b_f}{2\ t_f} < \lambda_r$$

therefore the flanges are nonslender

Width to thickness ratio for web of doubly symmetric I-section:

$$\frac{h}{t_w} = \ h_{div}t_w := 33.6$$

Limiting Width to Thickness Ratios:

$$\lambda_r := 1.49 \cdot \sqrt{\frac{E}{F_y}} = 35.884$$

$$\frac{h}{t_w} < \lambda_r$$

therefore the web is nonslender

Flexural Buckling of Doubly Symmetric I-Shape Members Without Slender Elements:

Column is Fixed-Fixed, therefore:

$$L := 18\ ft \qquad L_c := 0.8 \cdot L = 14.4\ ft$$

From AISC Tables:

$r_x := 5.85\ in \qquad I_x := 484\ in^4 \qquad A_g := 14.1\ in^2 \qquad L_{cx} := L_c \qquad L_{cy} := L_c \qquad L_{cz} := L_c$

$r_y := 1.91\ in \qquad I_y := 51.4\ in^4 \qquad J := 1.45\ in^4 \qquad G := 11200\ ksi \qquad C_w := 2240\ in^6$

$$F_{ex} := \frac{\left(\pi^2 \cdot E\right)}{\left(\left(\frac{L_{cx}}{r_x}\right)^2\right)} = 328.036\ ksi \qquad F_{ey} := \frac{\left(\pi^2 \cdot E\right)}{\left(\left(\frac{L_{cy}}{r_y}\right)^2\right)} = 34.968\ ksi$$

Torsional and Flexural Torsional Buckling Consideration:

$$F_{ez} := \left(\frac{\left(\pi^2 \cdot E \cdot C_w\right)}{\left(L_{cz}^{\ 2}\right)} + G \cdot J\right) \cdot \frac{1}{\left(I_x + I_y\right)} = 70.436\ ksi$$

$$F_e := min\left(F_{ex}, F_{ey}, F_{ez}\right) = 34.968\ ksi \qquad \text{(Elastic Buckling Stress)}$$

$$\frac{F_y}{F_e} = 1.43 \qquad < 2.25\text{, therefore} \qquad F_{cr} = \left(0.658^{\frac{F_y}{F_e}}\right) \cdot F_y$$

$$F_{cr} := \left(0.658^{\frac{F_y}{F_e}}\right) \cdot F_y = 27.483\ ksi \qquad \text{(Critical Buckling Stress)}$$

Design Axial Compressive Strength:

$$P_n := F_{cr} \cdot A_g = 387.506\ kip$$

$$\phi P_n := 0.9 \cdot P_n = 348.755\ kip$$

Maximum Axial Load:

$$A_{TributaryC1} := 185\ ft^2$$

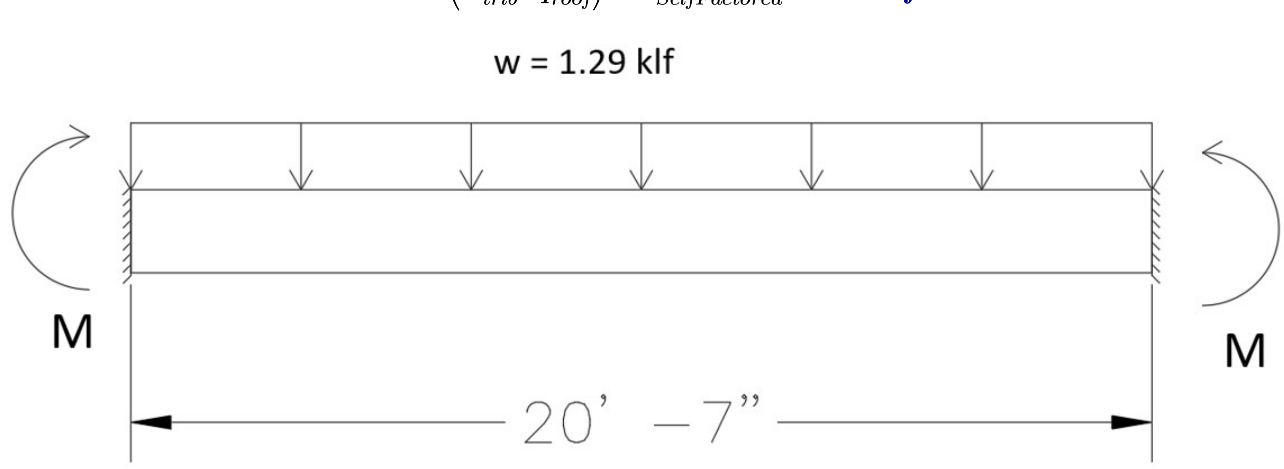
$$P_u := \left(q_{roof} \cdot A_{TributaryC1}\right) = 12.876\ kip$$

$$f_{ra} := \frac{P_u}{A_g} = 0.913\ ksi \qquad F_{ca} := \frac{\phi P_n}{A_g} = 24.734\ ksi$$

Bending about the x-axis (W18x60):

$$w_{trib} := \frac{35\ ft}{2} \qquad w_{self} := 60\ plf \qquad w_{SelfFactored} := 1.2 \cdot w_{self} = 0.072\ klf \qquad L := FIF\left(\text{“}20'\,7\text{”}\right)$$

$$w := \left(w_{trib} \cdot q_{roof}\right) + w_{SelfFactored} = 1.29\ klf$$



FLEXURE: Check Compact vs Slender:

$b_f := 7.56\ in$ $\quad t_f := 0.695\ in$ $\quad F_y := 50\ ksi$

Width to thickness ratio for flanges of doubly symmetric I-section:

$$\frac{b_f}{2\ t_f} = 5.439$$

Limiting Width to Thickness Ratios:

$$\lambda_p := 0.38 \cdot \sqrt{\frac{E}{F_y}} = 9.152 \qquad \lambda_r := 1.0 \cdot \sqrt{\frac{E}{F_y}} = 24.083$$

$$\frac{b_f}{2\ t_f} < \lambda_p < \lambda_r$$

therefore the flanges are compact

Width to thickness ratio for web of doubly symmetric I-section:

$$\frac{h}{t_w} = \ h_{div}t_w := 38.7$$

Limiting Width to Thickness Ratios:

$$\lambda_p := 3.76 \cdot \sqrt{\frac{E}{F_y}} = 90.553 \qquad \lambda_r := 5.70 \cdot \sqrt{\frac{E}{F_y}} = 137.274$$

$$\frac{h}{t_w} < \lambda_p < \lambda_r$$

therefore the web is compact

Flange local buckling will not occur.

Doubly Symmetric Compact I-Shape Subjected to Bending About the Major Axis:

Check Yielding:

$Z_x := 123\ in^3$

$$M_n := F_y \cdot Z_x = 512.5\ kip \cdot ft$$

Check Lateral Torsional Buckling:

From AISC Section Tables:

$I_y := 50.1\ in^4$ $\quad C_w := 3850\ in^6$ $\quad r_y := 1.68\ in$ $\quad S_x := 108\ in^3$ $\quad h_o := 17.5\ in$ $\quad J := 2.17\ in^4$

$c := 1$ (for doubly symmetric I-shapes)

$$r_{ts} := \sqrt{\frac{\left(I_y \cdot C_w\right)^{.5}}{S_x}} = 2.017\ in$$

$$L_p := 1.76 \cdot r_y \cdot \sqrt{\frac{E}{F_y}} = 5.934\ ft$$

$$L_r := 1.95 \cdot r_{ts} \cdot \frac{E}{0.7 \cdot F_y} \cdot \sqrt{\frac{J \cdot c}{S_x \cdot h_o} + \sqrt{\left(\frac{J \cdot c}{S_x \cdot h_o}\right)^2 + 6.76 \cdot \left(\frac{0.7 \cdot F_y}{E}\right)^2}} = 18.193\ ft$$

$$L_b := FIF\left(\text{“}5'\ 1\text{–}3/4\text{”}\right) = 5.146\ ft$$

Lb < Lp, therefore the limit state of lateral torsional buckling does not apply.

Flexural strength:

$$\phi M_n := 0.9 \cdot M_n = 461.25\ kip \cdot ft$$

Maximum Moment:

$$M_{ux} := \frac{w \cdot L^2}{12} = 45.545\ kip \cdot ft$$

$$f_{rbx} := \frac{M_{ux}}{S_x} = 5.061\ ksi \qquad F_{cbx} := \frac{\phi M_n}{S_x} = 51.25\ ksi$$

Bending about the y-axis:

$Z_y := 20.6\ in^3$ $\quad S_y := 13.3\ in^3$

$$M_y := F_y \cdot S_y = 55.417\ kip \cdot ft$$

$$M_{np} := F_y \cdot Z_y = 85.833\ kip \cdot ft \qquad M_{ny} := 1.6\ M_y = 88.667\ kip \cdot ft \qquad M_{nFLB} := \min\left(M_{np}, 1.6\ M_y\right) = 85.833\ kip \cdot ft$$

$$\phi M_n := \min\left(M_{np}, M_{ny}, M_{nFLB}\right) = 85.833\ kip \cdot ft$$

$M_{uy} := 31.64\ kip \cdot ft$ (analysis done by robot to find Muy)

$$f_{rby} := \frac{M_{uy}}{S_y} = 28.547\ ksi \qquad F_{cby} := \frac{\phi M_n}{S_y} = 77.444\ ksi$$

$$\left(\frac{f_{ra}}{F_{ca}} + \frac{f_{rbx}}{F_{cbx}} + \frac{f_{rby}}{F_{cby}}\right) = 0.504$$ < 1, therefore the design is adequate

MF2 - W30x132 Beam, W14x48 Column:
Combined Forces - Flexure and Axial Force:

Axial Load (Column):

Check Nonslender vs Slender:

$b_f := 8.03\ in$ $\quad t_f := 0.595\ in$ $\quad F_y := 50\ ksi$

Width to thickness ratio for flanges of doubly symmetric I-section:

$$\frac{b_f}{2\ t_f} = 6.748$$

Limiting Width to Thickness Ratios:

$$\lambda_r := 0.56 \cdot \sqrt{\frac{E}{F_y}} = 13.487$$

$\frac{b_f}{2\ t_f} < \lambda_r$

therefore the flanges are nonslender

Width to thickness ratio for web of doubly symmetric I-section:

$$\frac{h}{t_w} = \ h_{div}t_w := 33.6$$

Limiting Width to Thickness Ratios:

$$\lambda_r := 1.49 \cdot \sqrt{\frac{E}{F_y}} = 35.884$$

$\frac{h}{t_w} < \lambda_r$

therefore the web is nonslender

Flexural Buckling of Doubly Symmetric I-Shape Members Without Slender Elements:

Column is Fixed-Fixed, therefore:

$$L := 18\ ft \qquad L_c := 0.8 \cdot L = 14.4\ ft$$

From AISC Tables:

$r_x := 5.85\ in$ $\quad I_x := 484\ in^4$ $\quad A_g := 14.1\ in^2$ $\quad L_{cx} := L_c$ $\quad L_{cy} := L_c$ $\quad L_{cz} := L_c$

$r_y := 1.91\ in$ $\quad I_y := 51.4\ in^4$ $\quad J := 1.45\ in^4$ $\quad G := 11200\ ksi$ $\quad C_w := 2240\ in^6$

$$F_{ex} := \frac{\left(\pi^2 \cdot E\right)}{\left(\left(\frac{L_{cx}}{r_x}\right)^2\right)} = 328.036\ ksi \qquad F_{ey} := \frac{\left(\pi^2 \cdot E\right)}{\left(\left(\frac{L_{cy}}{r_y}\right)^2\right)} = 34.968\ ksi$$

Torsional and Flexural Torsional Buckling Consideration:

$$F_{ez} := \left(\frac{\left(\pi^2 \cdot E \cdot C_w\right)}{\left(L_{cz}^{\ 2}\right)} + G \cdot J\right) \cdot \frac{1}{\left(I_x + I_y\right)} = 70.436\ ksi$$

$$F_e := min\left(F_{ex}, F_{ey}, F_{ez}\right) = 34.968\ ksi \qquad \text{(Elastic Buckling Stress)}$$

$$\frac{F_y}{F_e} = 1.43 \qquad < 2.25\text{, therefore} \qquad F_{cr} = \left(0.658^{\frac{F_y}{F_e}}\right) \cdot F_y$$

$$F_{cr} := \left(0.658^{\frac{F_y}{F_e}}\right) \cdot F_y = 27.483\ ksi \qquad \text{(Critical Buckling Stress)}$$

Design Axial Compressive Strength:

$$P_n := F_{cr} \cdot A_g = 387.506\ kip$$

$$\phi P_n := 0.9 \cdot P_n = 348.755\ kip$$

Maximum Axial Load:

$$A_{TributaryC1} := 185\ ft^2$$

$$P_u := \left(q_{roof} \cdot A_{TributaryC1}\right) = 12.876\ kip$$

$$f_{ra} := \frac{P_u}{A_g} = 0.913\ ksi \qquad F_{ca} := \frac{\phi P_n}{A_g} = 24.734\ ksi$$

Bending about the x-axis (W30x132):

$$w_{trib} := \frac{FIF\left(\text{“}5'\,1\text{−}3/4\text{”}\right)}{2} \qquad w_{self} := 132\ plf \qquad w_{SelfFactored} := 1.2 \cdot w_{self} = 0.158\ klf \qquad L := 35\ ft$$

$$w := \left(w_{trib} \cdot q_{roof}\right) + w_{SelfFactored} = 0.337\ klf$$

FLEXURE: Check Compact vs Slender:

$b_f := 10.5\ in$ $\quad t_f := 1\ in$ $\quad F_y := 50\ ksi$

Width to thickness ratio for flanges of doubly symmetric I-section:

$$\frac{b_f}{2\ t_f} = 5.25$$

Limiting Width to Thickness Ratios:

$$\lambda_p := 0.38 \cdot \sqrt{\frac{E}{F_y}} = 9.152 \qquad \lambda_r := 1.0 \cdot \sqrt{\frac{E}{F_y}} = 24.083$$

$$\frac{b_f}{2\ t_f} < \lambda_p < \lambda_r$$

therefore the flanges are compact

Width to thickness ratio for web of doubly symmetric I-section:

$$\frac{h}{t_w} = \ h_{div}t_w := 43.9$$

Limiting Width to Thickness Ratios:

$$\lambda_p := 3.76 \cdot \sqrt{\frac{E}{F_y}} = 90.553 \qquad \lambda_r := 5.70 \cdot \sqrt{\frac{E}{F_y}} = 137.274$$

$$\frac{h}{t_w} < \lambda_p < \lambda_r$$

therefore the web is compact

Flange local buckling will not occur.

Doubly Symmetric Compact I-Shape Subjected to Bending About the Major Axis:

Check Yielding:

$Z_x := 437\ in^3$

$$M_n := F_y \cdot Z_x = 1820.833\ kip \cdot ft$$

Check Lateral Torsional Buckling:

From AISC Section Tables:

$$I_y := 196\ in^4 \quad C_w := 42100\ in^6 \quad r_y := 2.25\ in \quad S_x := 380\ in^3 \quad h_o := 29.3\ in \quad J := 9.72\ in^4$$

$c := 1$ (for doubly symmetric I-shapes)

$$r_{ts} := \sqrt{\frac{\left(I_y \cdot C_w\right)^{.5}}{S_x}} = 2.749\ in$$

$$L_p := 1.76 \cdot r_y \cdot \sqrt{\frac{E}{F_y}} = 7.947\ ft$$

$$L_r := 1.95 \cdot r_{ts} \cdot \frac{E}{0.7 \cdot F_y} \cdot \sqrt{\frac{J \cdot c}{S_x \cdot h_o} + \sqrt{\left(\frac{J \cdot c}{S_x \cdot h_o}\right)^2 + 6.76 \cdot \left(\frac{0.7 \cdot F_y}{E}\right)^2}} = 23.791\ ft$$

$$L_b := 35\ ft$$

Lb > Lr, therefore,

$$M_{nLTB} = F_{cr} \cdot S_x$$

$$M_{max} := \frac{w \cdot L^2}{12} = 34.451\ kip \cdot ft$$

$$M_A := \frac{w}{12} \cdot \left(6 \cdot L \cdot \frac{L}{4} - L^2 - 6 \cdot \left(\frac{L}{4}\right)^2\right) = 4.306\ kip \cdot ft \qquad \text{(quarter point)}$$

$$M_B := \frac{w}{12} \cdot \left(6 \cdot L \cdot \frac{L}{2} - L^2 - 6 \cdot \left(\frac{L}{2}\right)^2\right) = 17.225\ kip \cdot ft \qquad \text{(halfway point)}$$

$$M_C := \frac{w}{12} \cdot \left(6 \cdot L \cdot \frac{3\ L}{4} - L^2 - 6 \cdot \left(\frac{3\ L}{4}\right)^2\right) = 4.306\ kip \cdot ft \qquad \text{(three-quarter point)}$$

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

$$F_{cr} := \frac{C_b \cdot \pi^2 \cdot E}{\left(\frac{L_b}{r_{ts}}\right)^2} \cdot \sqrt{1 + 0.078 \cdot \frac{J \cdot c}{S_x \cdot h_o} \left(\frac{L_b}{r_{ts}}\right)^2} = 49.339\ ksi$$

$$M_{nLTB} := F_{cr} \cdot S_x = 1562.402\ kip \cdot ft$$

Flexural strength:

$$\phi M_n := 0.9 \cdot min\left(M_n, M_{nLTB}\right) = 1406.161\ kip \cdot ft$$

Maximum Moment:

$$M_{ux} := \frac{w_2 \cdot L^2}{12} = 131.688\ kip \cdot ft$$

$$f_{rbx} := \frac{M_{ux}}{S_x} = 4.159\ ksi \qquad F_{cbx} := \frac{\phi M_n}{S_x} = 44.405\ ksi$$

Bending about the y-axis:

$Z_y := 58.4\ in^3 \qquad S_y := 37.2\ in^3$

$$M_y := F_y \cdot S_y = 155\ kip \cdot ft$$

$$M_{np} := F_y \cdot Z_y = 243.333\ kip \cdot ft \qquad M_{ny} := 1.6\ M_y = 248\ kip \cdot ft \qquad M_{nFLB} := min\left(M_{np}, 1.6\ M_y\right) = 243.333\ kip \cdot ft$$

$$\phi M_n := min\left(M_{np}, M_{ny}, M_{nFLB}\right) = 243.333\ kip \cdot ft$$

$M_{uy} := 22.59\ kip \cdot ft$ (analysis done by robot to find Muy)

$$f_{rby} := \frac{M_{uy}}{S_y} = 7.287\ ksi \qquad F_{cby} := \frac{\phi M_n}{S_y} = 78.495\ ksi$$

$\left(\frac{f_{ra}}{F_{ca}} + \frac{f_{rbx}}{F_{cbx}} + \frac{f_{rby}}{F_{cby}}\right) = 0.223$ < 1, therefore the design is adequate

MF3 - W30x132 Beam, W14x132 Column:
Combined Forces - Flexure and Axial Force:

Axial Load (Column):

Check Nonslender vs Slender:

$b_f := 14.7\ in$ $\quad t_f := 1.03\ in$ $\quad F_y := 50\ ksi$

Width to thickness ratio for flanges of doubly symmetric I-section:

$$\frac{b_f}{2\ t_f} = 7.136$$

Limiting Width to Thickness Ratios:

$$\lambda_r := 0.56 \cdot \sqrt{\frac{E}{F_y}} = 13.487$$

$\frac{b_f}{2\ t_f} < \lambda_r$

therefore the flanges are nonslender

Width to thickness ratio for web of doubly symmetric I-section:

$$\frac{h}{t_w} = \ h_{div}t_w := 17.7$$

Limiting Width to Thickness Ratios:

$$\lambda_r := 1.49 \cdot \sqrt{\frac{E}{F_y}} = 35.884$$

$\frac{h}{t_w} < \lambda_r$

therefore the web is nonslender

Flexural Buckling of Doubly Symmetric I-Shape Members Without Slender Elements:

Column is Pinned-Fixed, therefore:

$$L := 18\ ft \qquad L_c := 0.80 \cdot L = 14.4\ ft$$

From AISC Tables:

$r_x := 6.28\ in$ $\quad I_x := 1530\ in^4$ $\quad A_g := 38.8\ in^2$ $\quad L_{cx} := L_c$ $\quad L_{cy} := L_c$ $\quad L_{cz} := L_c$

$r_y := 3.76\ in$ $\quad I_y := 548\ in^4$ $\quad J := 12.3\ in^4$ $\quad G := 11200\ ksi$ $\quad C_w := 25500\ in^6$

$$F_{ex} := \frac{\left(\pi^2 \cdot E\right)}{\left(\left(\frac{L_{cx}}{r_x}\right)^2\right)} = 378.033\ ksi \qquad F_{ey} := \frac{\left(\pi^2 \cdot E\right)}{\left(\left(\frac{L_{cy}}{r_y}\right)^2\right)} = 135.515\ ksi$$

Torsional and Flexural Torsional Buckling Consideration:

$$F_{ez} := \left(\frac{\left(\pi^2 \cdot E \cdot C_w\right)}{\left(L_{cz}^{\ 2}\right)} + G \cdot J\right) \cdot \frac{1}{\left(I_x + I_y\right)} = 183.921\ ksi$$

$$F_e := min\left(F_{ex}, F_{ey}, F_{ez}\right) = 135.515\ ksi \qquad \text{(Elastic Buckling Stress)}$$

$$\frac{F_y}{F_e} = 0.369 \qquad < 2.25\text{, therefore} \qquad F_{cr} = \left(0.658^{\frac{F_y}{F_e}}\right) \cdot F_y$$

$$F_{cr} := \left(0.658^{\frac{F_y}{F_e}}\right) \cdot F_y = 42.845\ ksi \qquad \text{(Critical Buckling Stress)}$$

Design Axial Compressive Strength:

$$P_n := F_{cr} \cdot A_g = 1662.393\ kip$$

$$\phi P_n := 0.9 \cdot P_n = 1496.153\ kip$$

Maximum Axial Load:

$$A_{TributaryC4} := 130\ ft^2$$

$$P_u := \left(q_{roof} \cdot A_{TributaryC4}\right) = 9.048\ kip$$

$$f_{ra} := \frac{P_u}{A_g} = 0.233\ ksi \qquad F_{ca} := \frac{\phi P_n}{A_g} = 38.561\ ksi$$

Bending about the x-axis (W30x132):

$$w_{trib} := \frac{FIF\left(\text{“}5'\,1\text{–}3/4\text{”}\right)}{2} \qquad w_{self} := 132\ plf \qquad w_{SelfFactored} := 1.2 \cdot w_{self} = 0.158\ klf \qquad L := 25\ ft$$

$$w := \left(w_{trib} \cdot q_{roof}\right) + w_{SelfFactored} = 0.337\ klf$$

FLEXURE: Check Compact vs Slender:

$b_f := 10.5\ in \qquad t_f := 1\ in \qquad F_y := 50\ ksi$

Width to thickness ratio for flanges of doubly symmetric I-section:

$$\frac{b_f}{2\ t_f} = 5.25$$

Limiting Width to Thickness Ratios:

$$\lambda_p := 0.38 \cdot \sqrt{\frac{E}{F_y}} = 9.152 \qquad \lambda_r := 1.0 \cdot \sqrt{\frac{E}{F_y}} = 24.083$$

$$\frac{b_f}{2\ t_f} < \lambda_p < \lambda_r$$

therefore the flanges are compact

Width to thickness ratio for web of doubly symmetric I-section:

$$\frac{h}{t_w} = \ h_{div}t_w := 43.9$$

Limiting Width to Thickness Ratios:

$$\lambda_p := 3.76 \cdot \sqrt{\frac{E}{F_y}} = 90.553 \qquad \lambda_r := 5.70 \cdot \sqrt{\frac{E}{F_y}} = 137.274$$

$$\frac{h}{t_w} < \lambda_p < \lambda_r$$

therefore the web is compact

Flange local buckling will not occur.

Doubly Symmetric Compact I-Shape Subjected to Bending About the Major Axis:

Check Yielding:

$Z_x := 437\ in^3$

$$M_n := F_y \cdot Z_x = 1820.833\ kip \cdot ft$$

Check Lateral Torsional Buckling:

From AISC Section Tables:

$I_y := 196\ in^4 \quad C_w := 42100\ in^6 \quad r_y := 2.25\ in \quad S_x := 380\ in^3 \quad h_o := 29.3\ in \quad J := 9.72\ in^4$

$c := 1$ (for doubly symmetric I-shapes)

$$r_{ts} := \sqrt{\frac{\left(I_y \cdot C_w\right)^{.5}}{S_x}} = 2.749\ in$$

$$L_p := 1.76 \cdot r_y \cdot \sqrt{\frac{E}{F_y}} = 7.947\ ft$$

$$L_r := 1.95 \cdot r_{ts} \cdot \frac{E}{0.7 \cdot F_y} \cdot \sqrt{\frac{J \cdot c}{S_x \cdot h_o} + \sqrt{\left(\frac{J \cdot c}{S_x \cdot h_o}\right)^2 + 6.76 \cdot \left(\frac{0.7 \cdot F_y}{E}\right)^2}} = 23.791\ ft$$

$$L_b := 25\ ft$$

Lb > Lr, therefore,

$$M_{nLTB} = F_{cr} \cdot S_x$$

$$M_{max} := \frac{w \cdot L^2}{12} = 17.577\ kip \cdot ft$$

$$M_A := \frac{w}{12} \cdot \left(6 \cdot L \cdot \frac{L}{4} - L^2 - 6 \cdot \left(\frac{L}{4}\right)^2\right) = 2.197\ kip \cdot ft \qquad \text{(quarter point)}$$

$$M_B := \frac{w}{12} \cdot \left(6 \cdot L \cdot \frac{L}{2} - L^2 - 6 \cdot \left(\frac{L}{2}\right)^2\right) = 8.788\ kip \cdot ft \qquad \text{(halfway point)}$$

$$M_C := \frac{w}{12} \cdot \left(6 \cdot L \cdot \frac{3\ L}{4} - L^2 - 6 \cdot \left(\frac{3\ L}{4}\right)^2\right) = 2.197\ kip \cdot ft \qquad \text{(three-quarter point)}$$

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

$$F_{cr} := \frac{C_b \cdot \pi^2 \cdot E}{\left(\frac{L_b}{r_{ts}}\right)^2} \cdot \sqrt{1 + 0.078 \cdot \frac{J \cdot c}{S_x \cdot h_o} \left(\frac{L_b}{r_{ts}}\right)^2} = 80.873\ ksi$$

$$M_{nLTB} := F_{cr} \cdot S_x = 2560.989\ kip \cdot ft$$

Flexural strength:

$$\phi M_n := 0.9 \cdot min\left(M_n, M_{nLTB}\right) = 1638.75\ kip \cdot ft$$

Maximum Moment:

$$M_{ux} := \frac{w_2 \cdot L^2}{12} = 67.188\ kip \cdot ft$$

$$f_{rbx} := \frac{M_{ux}}{S_x} = 2.122\ ksi \qquad F_{cbx} := \frac{\phi M_n}{S_x} = 51.75\ ksi$$

Bending about the y-axis:

$Z_y := 58.4\ in^3 \qquad S_y := 37.2\ in^3$

$$M_y := F_y \cdot S_y = 155\ kip \cdot ft$$

$$M_{np} := F_y \cdot Z_y = 243.333\ kip \cdot ft \qquad M_{ny} := 1.6\ M_y = 248\ kip \cdot ft \qquad M_{nFLB} := min\left(M_{np}, 1.6\ M_y\right) = 243.333\ kip \cdot ft$$

$$\phi M_n := min\left(M_{np}, M_{ny}, M_{nFLB}\right) = 243.333\ kip \cdot ft$$

$M_{uy} := 103.33\ kip \cdot ft$ (analysis done by robot to find Muy)

$$f_{rby} := \frac{M_{uy}}{S_y} = 33.332\ ksi \qquad F_{cby} := \frac{\phi M_n}{S_y} = 78.495\ ksi$$

$\left(\frac{f_{ra}}{F_{ca}} + \frac{f_{rbx}}{F_{cbx}} + \frac{f_{rby}}{F_{cby}}\right) = 0.472$ < 1, therefore the design is adequate

MF4 - W30x132 Beam, W14x132 Column:
Combined Forces - Flexure and Axial Force:

Axial Load (Column):
Check Nonslender vs Slender:

$b_f := 14.7\ in$ $\quad t_f := 1.03\ in$ $\quad F_y := 50\ ksi$

Width to thickness ratio for flanges of doubly symmetric I-section:

$$\frac{b_f}{2\ t_f} = 7.136$$

Limiting Width to Thickness Ratios:

$$\lambda_r := 0.56 \cdot \sqrt{\frac{E}{F_y}} = 13.487$$

$\frac{b_f}{2\ t_f} < \lambda_r$

therefore the flanges are nonslender

Width to thickness ratio for web of doubly symmetric I-section:

$$\frac{h}{t_w} = \ h_{div}t_w := 17.7$$

Limiting Width to Thickness Ratios:

$$\lambda_r := 1.49 \cdot \sqrt{\frac{E}{F_y}} = 35.884$$

$\frac{h}{t_w} < \lambda_r$

therefore the web is nonslender

Flexural Buckling of Doubly Symmetric I-Shape Members Without Slender Elements:

Column is Pinned-Fixed, therefore:

$$L := 18\ ft \qquad L_c := 0.80 \cdot L = 14.4\ ft$$

From AISC Tables:

$r_x := 6.28\ in$ $\quad I_x := 1530\ in^4$ $\quad A_g := 38.8\ in^2$ $\quad L_{cx} := L_c$ $\quad L_{cy} := L_c$ $\quad L_{cz} := L_c$

$r_y := 3.76\ in$ $\quad I_y := 548\ in^4$ $\quad J := 12.3\ in^4$ $\quad G := 11200\ ksi$ $\quad C_w := 25500\ in^6$

$$F_{ex} := \frac{\left(\pi^2 \cdot E\right)}{\left(\left(\frac{L_{cx}}{r_x}\right)^2\right)} = 378.033\ ksi \qquad F_{ey} := \frac{\left(\pi^2 \cdot E\right)}{\left(\left(\frac{L_{cy}}{r_y}\right)^2\right)} = 135.515\ ksi$$

Torsional and Flexural Torsional Buckling Consideration:

$$F_{ez} := \left(\frac{\left(\pi^2 \cdot E \cdot C_w\right)}{\left(L_{cz}^{\ 2}\right)} + G \cdot J\right) \cdot \frac{1}{\left(I_x + I_y\right)} = 183.921\ ksi$$

$$F_e := min\left(F_{ex}, F_{ey}, F_{ez}\right) = 135.515\ ksi \qquad \text{(Elastic Buckling Stress)}$$

$$\frac{F_y}{F_e} = 0.369 \qquad < 2.25\text{, therefore} \qquad F_{cr} = \left(0.658^{\frac{F_y}{F_e}}\right) \cdot F_y$$

$$F_{cr} := \left(0.658^{\frac{F_y}{F_e}}\right) \cdot F_y = 42.845\ ksi \qquad \text{(Critical Buckling Stress)}$$

Design Axial Compressive Strength:

$$P_n := F_{cr} \cdot A_g = 1662.393\ kip$$

$$\phi P_n := 0.9 \cdot P_n = 1496.153\ kip$$

Maximum Axial Load:

$$A_{TributaryC4} := 130\ ft^2$$

$$P_u := \left(q_{roof} \cdot A_{TributaryC4}\right) = 9.048\ kip$$

$$f_{ra} := \frac{P_u}{A_g} = 0.233\ ksi \qquad F_{ca} := \frac{\phi P_n}{A_g} = 38.561\ ksi$$

Bending about the x-axis (W30x132):

$$w_{trib} := \frac{25\ ft}{2} \qquad w_{self} := 132\ plf \qquad w_{SelfFactored} := 1.2 \cdot w_{self} = 0.158\ klf \qquad L := FIF\left(\text{“20' 7”}\right)$$

$$w := \left(w_{trib} \cdot q_{roof}\right) + w_{SelfFactored} = 1.028\ klf$$

FLEXURE: Check Compact vs Slender:

$b_f := 10.5\ in \qquad t_f := 1\ in \qquad F_y := 50\ ksi$

Width to thickness ratio for flanges of doubly symmetric I-section:

$$\frac{b_f}{2\ t_f} = 5.25$$

Limiting Width to Thickness Ratios:

$$\lambda_p := 0.38 \cdot \sqrt{\frac{E}{F_y}} = 9.152 \qquad \lambda_r := 1.0 \cdot \sqrt{\frac{E}{F_y}} = 24.083$$

$$\frac{b_f}{2\ t_f} < \lambda_p < \lambda_r$$

therefore the flanges are compact

Width to thickness ratio for web of doubly symmetric I-section:

$$\frac{h}{t_w} = \ h_{div} t_w := 43.9$$

Limiting Width to Thickness Ratios:

$$\lambda_p := 3.76 \cdot \sqrt{\frac{E}{F_y}} = 90.553 \qquad \lambda_r := 5.70 \cdot \sqrt{\frac{E}{F_y}} = 137.274$$

$$\frac{h}{t_w} < \lambda_p < \lambda_r$$

therefore the web is compact

Flange local buckling will not occur.

Doubly Symmetric Compact I-Shape Subjected to Bending About the Major Axis:

Check Yielding:

$Z_x := 437\ in^3$

$$M_n := F_y \cdot Z_x = 1820.833\ kip \cdot ft$$

Check Lateral Torsional Buckling:

From AISC Section Tables:

$$I_y := 196\ in^4 \quad C_w := 42100\ in^6 \quad r_y := 2.25\ in \quad S_x := 380\ in^3 \quad h_o := 29.3\ in \quad J := 9.72\ in^4$$

$c := 1$ (for doubly symmetric I-shapes)

$$r_{ts} := \sqrt{\frac{\left(I_y \cdot C_w\right)^{.5}}{S_x}} = 2.749\ in$$

$$L_p := 1.76 \cdot r_y \cdot \sqrt{\frac{E}{F_y}} = 7.947\ ft$$

$$L_r := 1.95 \cdot r_{ts} \cdot \frac{E}{0.7 \cdot F_y} \cdot \sqrt{\frac{J \cdot c}{S_x \cdot h_o} + \sqrt{\left(\frac{J \cdot c}{S_x \cdot h_o}\right)^2 + 6.76 \cdot \left(\frac{0.7 \cdot F_y}{E}\right)^2}} = 23.791\ ft$$

$$L_b := 5\ ft$$

Lb < Lp, therefore the limit state of lateral torsional buckling does not apply.

Flexural strength:

$$\phi M_n := 0.9 \cdot M_n = 1638.75\ kip \cdot ft$$

Maximum Moment:

$$M_{ux} := \frac{w \cdot L^2}{12} = 36.309\ kip \cdot ft$$

$$f_{rbx} := \frac{M_{ux}}{S_x} = 1.147\ ksi \qquad F_{cbx} := \frac{\phi M_n}{S_x} = 51.75\ ksi$$

Bending about the y-axis:

$Z_y := 58.4\ in^3 \qquad S_y := 37.2\ in^3$

$$M_y := F_y \cdot S_y = 155\ kip \cdot ft$$

$$M_{np} := F_y \cdot Z_y = 243.333\ kip \cdot ft \qquad M_{ny} := 1.6\ M_y = 248\ kip \cdot ft \qquad M_{nFLB} := min\left(M_{np}, 1.6\ M_y\right) = 243.333\ kip \cdot ft$$

$$\phi M_n := min\left(M_{np}, M_{ny}, M_{nFLB}\right) = 243.333\ kip \cdot ft$$

$M_{uy} := 87.63\ kip \cdot ft$ (analysis done by robot to find Muy)

$$f_{rby} := \frac{M_{uy}}{S_y} = 28.268\ ksi \qquad F_{cby} := \frac{\phi M_n}{S_y} = 78.495\ ksi$$

$\left(\frac{f_{ra}}{F_{ca}} + \frac{f_{rbx}}{F_{cbx}} + \frac{f_{rby}}{F_{cby}}\right) = 0.388$ < 1, therefore the design is adequate

MF5 - W30x132 Beam, W14x132 Column:
Combined Forces - Flexure and Axial Force:

Axial Load (Column):

Check Nonslender vs Slender:

$b_f := 14.7\ in$ $\quad t_f := 1.03\ in$ $\quad F_y := 50\ ksi$

Width to thickness ratio for flanges of doubly symmetric I-section:

$$\frac{b_f}{2\ t_f} = 7.136$$

Limiting Width to Thickness Ratios:

$$\lambda_r := 0.56 \cdot \sqrt{\frac{E}{F_y}} = 13.487$$

$\frac{b_f}{2\ t_f} < \lambda_r$

therefore the flanges are nonslender

Width to thickness ratio for web of doubly symmetric I-section:

$$\frac{h}{t_w} = \ h_{div}t_w := 17.7$$

Limiting Width to Thickness Ratios:

$$\lambda_r := 1.49 \cdot \sqrt{\frac{E}{F_y}} = 35.884$$

$\frac{h}{t_w} < \lambda_r$

therefore the web is nonslender

Flexural Buckling of Doubly Symmetric I-Shape Members Without Slender Elements:

Column is Pinned-Fixed, therefore:

$$L := 18\ ft \qquad L_c := 0.80 \cdot L = 14.4\ ft$$

From AISC Tables:

$r_x := 6.28\ in$ $\quad I_x := 1530\ in^4$ $\quad A_g := 38.8\ in^2$ $\quad L_{cx} := L_c$ $\quad L_{cy} := L_c$ $\quad L_{cz} := L_c$

$r_y := 3.76\ in$ $\quad I_y := 548\ in^4$ $\quad J := 12.3\ in^4$ $\quad G := 11200\ ksi$ $\quad C_w := 25500\ in^6$

$$F_{ex} := \frac{\left(\pi^2 \cdot E\right)}{\left(\left(\frac{L_{cx}}{r_x}\right)^2\right)} = 378.033\ ksi \qquad F_{ey} := \frac{\left(\pi^2 \cdot E\right)}{\left(\left(\frac{L_{cy}}{r_y}\right)^2\right)} = 135.515\ ksi$$

Torsional and Flexural Torsional Buckling Consideration:

$$F_{ez} := \left(\frac{\left(\pi^2 \cdot E \cdot C_w\right)}{\left(L_{cz}^{\ 2}\right)} + G \cdot J\right) \cdot \frac{1}{\left(I_x + I_y\right)} = 183.921\ ksi$$

$$F_e := min\left(F_{ex}, F_{ey}, F_{ez}\right) = 135.515\ ksi \qquad \text{(Elastic Buckling Stress)}$$

$$\frac{F_y}{F_e} = 0.369 \qquad < 2.25\text{, therefore} \qquad F_{cr} = \left(0.658^{\frac{F_y}{F_e}}\right) \cdot F_y$$

$$F_{cr} := \left(0.658^{\frac{F_y}{F_e}}\right) \cdot F_y = 42.845\ ksi \qquad \text{(Critical Buckling Stress)}$$

Design Axial Compressive Strength:

$$P_n := F_{cr} \cdot A_g = 1662.393\ kip$$

$$\phi P_n := 0.9 \cdot P_n = 1496.153\ kip$$

Maximum Axial Load:

$$A_{TributaryC11} := 184\ ft^2$$

$$P_u := \left(q_{roof} \cdot A_{TributaryC11}\right) = 12.806\ kip$$

$$f_{ra} := \frac{P_u}{A_g} = 0.33\ ksi \qquad F_{ca} := \frac{\phi P_n}{A_g} = 38.561\ ksi$$

Bending about the x-axis (W30x132):

$$w_{trib} := \frac{35\ ft}{2} \qquad w_{self} := 132\ plf \qquad w_{SelfFactored} := 1.2 \cdot w_{self} = 0.158\ klf \qquad L := FIF\left(\text{“20' 5”}\right)$$

$$w := \left(w_{trib} \cdot q_{roof}\right) + w_{SelfFactored} = 1.376\ klf$$

FLEXURE: Check Compact vs Slender:

$b_f := 10.5\ in$ $\quad t_f := 1\ in$ $\quad F_y := 50\ ksi$

Width to thickness ratio for flanges of doubly symmetric I-section:

$$\frac{b_f}{2\ t_f} = 5.25$$

Limiting Width to Thickness Ratios:

$$\lambda_p := 0.38 \cdot \sqrt{\frac{E}{F_y}} = 9.152 \qquad \lambda_r := 1.0 \cdot \sqrt{\frac{E}{F_y}} = 24.083$$

$$\frac{b_f}{2\ t_f} < \lambda_p < \lambda_r$$

therefore the flanges are compact

Width to thickness ratio for web of doubly symmetric I-section:

$$\frac{h}{t_w} = h_{div}t_w := 43.9$$

Limiting Width to Thickness Ratios:

$$\lambda_p := 3.76 \cdot \sqrt{\frac{E}{F_y}} = 90.553 \qquad \lambda_r := 5.70 \cdot \sqrt{\frac{E}{F_y}} = 137.274$$

$$\frac{h}{t_w} < \lambda_p < \lambda_r$$

therefore the web is compact

Flange local buckling will not occur.

Doubly Symmetric Compact I-Shape Subjected to Bending About the Major Axis:

Check Yielding:

$Z_x := 437\ in^3$

$$M_n := F_y \cdot Z_x = 1820.833\ kip \cdot ft$$

Check Lateral Torsional Buckling:

From AISC Section Tables:

$I_y := 196\ in^4$ $\quad C_w := 42100\ in^6$ $\quad r_y := 2.25\ in$ $\quad S_x := 380\ in^3$ $\quad h_o := 29.3\ in$ $\quad J := 9.72\ in^4$

$c := 1$ (for doubly symmetric I-shapes)

$$r_{ts} := \sqrt{\frac{\left(I_y \cdot C_w\right)^{.5}}{S_x}} = 2.749\ in$$

$$L_p := 1.76 \cdot r_y \cdot \sqrt{\frac{E}{F_y}} = 7.947\ ft$$

$$L_r := 1.95 \cdot r_{ts} \cdot \frac{E}{0.7 \cdot F_y} \cdot \sqrt{\frac{J \cdot c}{S_x \cdot h_o} + \sqrt{\left(\frac{J \cdot c}{S_x \cdot h_o}\right)^2 + 6.76 \cdot \left(\frac{0.7 \cdot F_y}{E}\right)^2}} = 23.791\ ft$$

$$L_b := FIF\left(\text{“5' 1-1/4”}\right)$$

Lb < Lp, therefore the limit state of lateral torsional buckling does not apply.

Flexural strength:

$$\phi M_n := 0.9 \cdot M_n = 1638.75\ kip \cdot ft$$

Maximum Moment:

$$M_{ux} := \frac{w \cdot L^2}{12} = 47.812\ kip \cdot ft$$

$$f_{rbx} := \frac{M_{ux}}{S_x} = 1.51\ ksi \qquad F_{cbx} := \frac{\phi M_n}{S_x} = 51.75\ ksi$$

Bending about the y-axis:

$Z_y := 58.4\ in^3$ $\quad S_y := 37.2\ in^3$

$$M_y := F_y \cdot S_y = 155\ kip \cdot ft$$

$$M_{np} := F_y \cdot Z_y = 243.333\ kip \cdot ft \qquad M_{ny} := 1.6\ M_y = 248\ kip \cdot ft \qquad M_{nFLB} := min\left(M_{np}, 1.6\ M_y\right) = 243.333\ kip \cdot ft$$

$$\phi M_n := min\left(M_{np}, M_{ny}, M_{nFLB}\right) = 243.333\ kip \cdot ft$$

$M_{uy} := 102.47\ kip \cdot ft$ (analysis done by robot to find Muy)

$$f_{rby} := \frac{M_{uy}}{S_y} = 33.055\ ksi \qquad F_{cby} := \frac{\phi M_n}{S_y} = 78.495\ ksi$$

$$\left(\frac{f_{ra}}{F_{ca}} + \frac{f_{rbx}}{F_{cbx}} + \frac{f_{rby}}{F_{cby}}\right) = 0.459$$ < 1, therefore the design is adequate

MF5+MF6 - W30x132 Beam, W14x132 Column:
Combined Forces - Flexure and Axial Force:

Axial Load (Column):

Check Nonslender vs Slender:

$b_f := 14.7\ in$ $\quad t_f := 1.03\ in$ $\quad F_y := 50\ ksi$

Width to thickness ratio for flanges of doubly symmetric I-section:

$$\frac{b_f}{2\ t_f} = 7.136$$

Limiting Width to Thickness Ratios:

$$\lambda_r := 0.56 \cdot \sqrt{\frac{E}{F_y}} = 13.487$$

$\frac{b_f}{2\ t_f} < \lambda_r$

therefore the flanges are nonslender

Width to thickness ratio for web of doubly symmetric I-section:

$$\frac{h}{t_w} = \ h_{div}t_w := 17.7$$

Limiting Width to Thickness Ratios:

$$\lambda_r := 1.49 \cdot \sqrt{\frac{E}{F_y}} = 35.884$$

$\frac{h}{t_w} < \lambda_r$

therefore the web is nonslender

Flexural Buckling of Doubly Symmetric I-Shape Members Without Slender Elements:

Column is Pinned-Fixed, therefore:

$$L := 18\ ft \qquad L_c := 0.80 \cdot L = 14.4\ ft$$

From AISC Tables:

$r_x := 6.28\ in$ $\quad I_x := 1530\ in^4$ $\quad A_g := 38.8\ in^2$ $\quad L_{cx} := L_c$ $\quad L_{cy} := L_c$ $\quad L_{cz} := L_c$

$r_y := 3.76\ in$ $\quad I_y := 548\ in^4$ $\quad J := 12.3\ in^4$ $\quad G := 11200\ ksi$ $\quad C_w := 25500\ in^6$

$$F_{ex} := \frac{\left(\pi^2 \cdot E\right)}{\left(\left(\frac{L_{cx}}{r_x}\right)^2\right)} = 378.033\ ksi \qquad F_{ey} := \frac{\left(\pi^2 \cdot E\right)}{\left(\left(\frac{L_{cy}}{r_y}\right)^2\right)} = 135.515\ ksi$$

Torsional and Flexural Torsional Buckling Consideration:

$$F_{ez} := \left(\frac{\left(\pi^2 \cdot E \cdot C_w\right)}{\left(L_{cz}^{\ 2}\right)} + G \cdot J\right) \cdot \frac{1}{\left(I_x + I_y\right)} = 183.921\ ksi$$

$$F_e := min\left(F_{ex}, F_{ey}, F_{ez}\right) = 135.515\ ksi$$ (Elastic Buckling Stress)

$\frac{F_y}{F_e} = 0.369$ < 2.25, therefore $F_{cr} = \left(0.658^{\frac{F_y}{F_e}}\right) \cdot F_y$

$$F_{cr} := \left(0.658^{\frac{F_y}{F_e}}\right) \cdot F_y = 42.845\ ksi$$ (Critical Buckling Stress)

Design Axial Compressive Strength:

$$P_n := F_{cr} \cdot A_g = 1662.393\ kip$$

$$\phi P_n := 0.9 \cdot P_n = 1496.153\ kip$$

Maximum Axial Load:

$$A_{TributaryC11} := 184\ ft^2$$

$$P_u := \left(q_{roof} \cdot A_{TributaryC11}\right) = 12.806\ kip$$

$$f_{ra} := \frac{P_u}{A_g} = 0.33\ ksi \qquad F_{ca} := \frac{\phi P_n}{A_g} = 38.561\ ksi$$

Bending about the x-axis (W30x132):

$w_{trib} := \frac{FIF\left(\text{“5' 1-1/4”}\right)}{2}$ $\quad w_{self} := 132\ plf$ $\quad w_{SelfFactored} := 1.2 \cdot w_{self} = 0.158\ klf$ $\quad L := 35\ ft$

$$w := \left(w_{trib} \cdot q_{roof}\right) + w_{SelfFactored} = 0.336\ klf$$

FLEXURE: Check Compact vs Slender:

$b_f := 10.5\ in \qquad t_f := 1\ in \qquad F_y := 50\ ksi$

Width to thickness ratio for flanges of doubly symmetric I-section:

$$\frac{b_f}{2\ t_f} = 5.25$$

Limiting Width to Thickness Ratios:

$$\lambda_p := 0.38 \cdot \sqrt{\frac{E}{F_y}} = 9.152 \qquad \lambda_r := 1.0 \cdot \sqrt{\frac{E}{F_y}} = 24.083$$

$$\frac{b_f}{2\ t_f} < \lambda_p < \lambda_r$$

therefore the flanges are compact

Width to thickness ratio for web of doubly symmetric I-section:

$$\frac{h}{t_w} = \ h_{div}t_w := 43.9$$

Limiting Width to Thickness Ratios:

$$\lambda_p := 3.76 \cdot \sqrt{\frac{E}{F_y}} = 90.553 \qquad \lambda_r := 5.70 \cdot \sqrt{\frac{E}{F_y}} = 137.274$$

$$\frac{h}{t_w} < \lambda_p < \lambda_r$$

therefore the web is compact

Flange local buckling will not occur.

Doubly Symmetric Compact I-Shape Subjected to Bending About the Major Axis:

Check Yielding:

$Z_x := 437\ in^3$

$$M_n := F_y \cdot Z_x = 1820.833\ kip \cdot ft$$

Check Lateral Torsional Buckling:

From AISC Section Tables:

$I_y := 196\ in^4 \quad C_w := 42100\ in^6 \quad r_y := 2.25\ in \quad S_x := 380\ in^3 \quad h_o := 29.3\ in \quad J := 9.72\ in^4$

$c := 1$ (for doubly symmetric I-shapes)

$$r_{ts} := \sqrt{\frac{\left(I_y \cdot C_w\right)^{.5}}{S_x}} = 2.749\ in$$

$$L_p := 1.76 \cdot r_y \cdot \sqrt{\frac{E}{F_y}} = 7.947\ ft$$

$$L_r := 1.95 \cdot r_{ts} \cdot \frac{E}{0.7 \cdot F_y} \cdot \sqrt{\frac{J \cdot c}{S_x \cdot h_o} + \sqrt{\left(\frac{J \cdot c}{S_x \cdot h_o}\right)^2 + 6.76 \cdot \left(\frac{0.7 \cdot F_y}{E}\right)^2}} = 23.791\ ft$$

$$L_b := 35\ ft$$

Lb > Lr, therefore,

$$M_{nLTB} = F_{cr} \cdot S_x$$

$$M_{max} := \frac{w \cdot L^2}{12} = 34.303\ kip \cdot ft$$

$$M_A := \frac{w}{12} \cdot \left(6 \cdot L \cdot \frac{L}{4} - L^2 - 6 \cdot \left(\frac{L}{4}\right)^2\right) = 4.288\ kip \cdot ft \qquad \text{(quarter point)}$$

$$M_B := \frac{w}{12} \cdot \left(6 \cdot L \cdot \frac{L}{2} - L^2 - 6 \cdot \left(\frac{L}{2}\right)^2\right) = 17.151\ kip \cdot ft \qquad \text{(halfway point)}$$

$$M_C := \frac{w}{12} \cdot \left(6 \cdot L \cdot \frac{3\ L}{4} - L^2 - 6 \cdot \left(\frac{3\ L}{4}\right)^2\right) = 4.288\ kip \cdot ft \qquad \text{(three-quarter point)}$$

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

$$F_{cr} := \frac{C_b \cdot \pi^2 \cdot E}{\left(\frac{L_b}{r_{ts}}\right)^2} \cdot \sqrt{1 + 0.078 \cdot \frac{J \cdot c}{S_x \cdot h_o} \left(\frac{L_b}{r_{ts}}\right)^2} = 49.339\ ksi$$

$$M_{nLTB} := F_{cr} \cdot S_x = 1562.402\ kip \cdot ft$$

Flexural strength:

$$\phi M_n := 0.9 \cdot min\left(M_n, M_{nLTB}\right) = 1406.161\ kip \cdot ft$$

Maximum Moment:

$$M_{ux} := \frac{w_2 \cdot L^2}{12} = 131.688\ kip \cdot ft$$

$$f_{rbx} := \frac{M_{ux}}{S_x} = 4.159\ ksi \qquad F_{cbx} := \frac{\phi M_n}{S_x} = 44.405\ ksi$$

Bending about the y-axis:

$$Z_y := 58.4\ in^3 \qquad S_y := 37.2\ in^3$$

$$M_y := F_y \cdot S_y = 155\ kip \cdot ft$$

$$M_{np} := F_y \cdot Z_y = 243.333\ kip \cdot ft \qquad M_{ny} := 1.6\ M_y = 248\ kip \cdot ft \qquad M_{nFLB} := min\left(M_{np}, 1.6\ M_y\right) = 243.333\ kip \cdot ft$$

$$\phi M_n := min\left(M_{np}, M_{ny}, M_{nFLB}\right) = 243.333\ kip \cdot ft$$

$M_{uy} := 78.75\ kip \cdot ft$ (analysis done by robot to find Muy)

$$f_{rby} := \frac{M_{uy}}{S_y} = 25.403\ ksi \qquad F_{cby} := \frac{\phi M_n}{S_y} = 78.495\ ksi$$

$\left(\frac{f_{ra}}{F_{ca}} + \frac{f_{rbx}}{F_{cbx}} + \frac{f_{rby}}{F_{cby}}\right) = 0.426$ < 1, therefore the design is adequate

MF6+MF7 - W30x132 Beam, W14x159 Column:
Combined Forces - Flexure and Axial Force:

Axial Load (Column):

Check Nonslender vs Slender:

$b_f := 15.6\ in$ $\quad t_f := 1.19\ in$ $\quad F_y := 50\ ksi$

Width to thickness ratio for flanges of doubly symmetric I-section:

$$\frac{b_f}{2\ t_f} = 6.555$$

Limiting Width to Thickness Ratios:

$$\lambda_r := 0.56 \cdot \sqrt{\frac{E}{F_y}} = 13.487$$

$\frac{b_f}{2\ t_f} < \lambda_r$

therefore the flanges are nonslender

Width to thickness ratio for web of doubly symmetric I-section:

$$\frac{h}{t_w} = \ h_{div}t_w := 15.3$$

Limiting Width to Thickness Ratios:

$$\lambda_r := 1.49 \cdot \sqrt{\frac{E}{F_y}} = 35.884$$

$\frac{h}{t_w} < \lambda_r$

therefore the web is nonslender

Flexural Buckling of Doubly Symmetric I-Shape Members Without Slender Elements:

Column is Pinned-Fixed, therefore:

$$L := 18\ ft \qquad L_c := 0.80 \cdot L = 14.4\ ft$$

From AISC Tables:

$r_x := 6.38\ in$ $\quad I_x := 1900\ in^4$ $\quad A_g := 46.7\ in^2$ $\quad L_{cx} := L_c$ $\quad L_{cy} := L_c$ $\quad L_{cz} := L_c$

$r_y := 4.00\ in$ $\quad I_y := 748\ in^4$ $\quad J := 19.7\ in^4$ $\quad G := 11200\ ksi$ $\quad C_w := 35600\ in^6$

$$F_{ex} := \frac{\left(\pi^2 \cdot E\right)}{\left(\left(\frac{L_{cx}}{r_x}\right)^2\right)} = 390.168\ ksi \qquad F_{ey} := \frac{\left(\pi^2 \cdot E\right)}{\left(\left(\frac{L_{cy}}{r_y}\right)^2\right)} = 153.366\ ksi$$

Torsional and Flexural Torsional Buckling Consideration:

$$F_{ez} := \left(\frac{\left(\pi^2 \cdot E \cdot C_w\right)}{\left(L_{cz}^{\ 2}\right)} + G \cdot J\right) \cdot \frac{1}{\left(I_x + I_y\right)} = 212.19\ ksi$$

$$F_e := min\left(F_{ex}, F_{ey}, F_{ez}\right) = 153.366\ ksi$$ (Elastic Buckling Stress)

$$\frac{F_y}{F_e} = 0.326$$ < 2.25, therefore $F_{cr} = \left(0.658^{\frac{F_y}{F_e}}\right) \cdot F_y$

$$F_{cr} := \left(0.658^{\frac{F_y}{F_e}}\right) \cdot F_y = 43.622\ ksi$$ (Critical Buckling Stress)

Design Axial Compressive Strength:

$$P_n := F_{cr} \cdot A_g = 2037.162\ kip$$

$$\phi P_n := 0.9 \cdot P_n = 1833.446\ kip$$

Maximum Axial Load:

$$A_{TributaryC10} := 184\ ft^2$$

$$P_u := \left(q_{roof} \cdot A_{TributaryC10}\right) = 12.806\ kip$$

$$f_{ra} := \frac{P_u}{A_g} = 0.274\ ksi \qquad F_{ca} := \frac{\phi P_n}{A_g} = 39.26\ ksi$$

Bending about the x-axis (W30x132):

$w_{trib} := \frac{FIF\left(\text{“5' 1-1/4”}\right)}{2}$ $\quad w_{self} := 132\ plf$ $\quad w_{SelfFactored} := 1.2 \cdot w_{self} = 0.158\ klf$ $\quad L := 35\ ft$

$$w := \left(w_{trib} \cdot q_{roof}\right) + w_{SelfFactored} = 0.336\ klf$$

FLEXURE: Check Compact vs Slender:

$b_f := 10.5\ in \qquad t_f := 1\ in \qquad F_y := 50\ ksi$

Width to thickness ratio for flanges of doubly symmetric I-section:

$$\frac{b_f}{2\ t_f} = 5.25$$

Limiting Width to Thickness Ratios:

$$\lambda_p := 0.38 \cdot \sqrt{\frac{E}{F_y}} = 9.152 \qquad \lambda_r := 1.0 \cdot \sqrt{\frac{E}{F_y}} = 24.083$$

$$\frac{b_f}{2\ t_f} < \lambda_p < \lambda_r$$

therefore the flanges are compact

Width to thickness ratio for web of doubly symmetric I-section:

$$\frac{h}{t_w} = \ h_{div} t_w := 43.9$$

Limiting Width to Thickness Ratios:

$$\lambda_p := 3.76 \cdot \sqrt{\frac{E}{F_y}} = 90.553 \qquad \lambda_r := 5.70 \cdot \sqrt{\frac{E}{F_y}} = 137.274$$

$$\frac{h}{t_w} < \lambda_p < \lambda_r$$

therefore the web is compact

Flange local buckling will not occur.

Doubly Symmetric Compact I-Shape Subjected to Bending About the Major Axis:

Check Yielding:

$Z_x := 437\ in^3$

$$M_n := F_y \cdot Z_x = 1820.833\ kip \cdot ft$$

Check Lateral Torsional Buckling:

From AISC Section Tables:

$I_y := 196\ in^4 \quad C_w := 42100\ in^6 \quad r_y := 2.25\ in \quad S_x := 380\ in^3 \quad h_o := 29.3\ in \quad J := 9.72\ in^4$

$c := 1$ (for doubly symmetric I-shapes)

$$r_{ts} := \sqrt{\frac{\left(I_y \cdot C_w\right)^{.5}}{S_x}} = 2.749\ in$$

$$L_p := 1.76 \cdot r_y \cdot \sqrt{\frac{E}{F_y}} = 7.947\ ft$$

$$L_r := 1.95 \cdot r_{ts} \cdot \frac{E}{0.7 \cdot F_y} \cdot \sqrt{\frac{J \cdot c}{S_x \cdot h_o} + \sqrt{\left(\frac{J \cdot c}{S_x \cdot h_o}\right)^2 + 6.76 \cdot \left(\frac{0.7 \cdot F_y}{E}\right)^2}} = 23.791\ ft$$

$$L_b := 35\ ft$$

Lb > Lr, therefore,

$$M_{nLTB} = F_{cr} \cdot S_x$$

$$M_{max} := \frac{w \cdot L^2}{12} = 34.303\ kip \cdot ft$$

$$M_A := \frac{w}{12} \cdot \left(6 \cdot L \cdot \frac{L}{4} - L^2 - 6 \cdot \left(\frac{L}{4}\right)^2\right) = 4.288\ kip \cdot ft$$ (quarter point)

$$M_B := \frac{w}{12} \cdot \left(6 \cdot L \cdot \frac{L}{2} - L^2 - 6 \cdot \left(\frac{L}{2}\right)^2\right) = 17.151\ kip \cdot ft$$ (halfway point)

$$M_C := \frac{w}{12} \cdot \left(6 \cdot L \cdot \frac{3\ L}{4} - L^2 - 6 \cdot \left(\frac{3\ L}{4}\right)^2\right) = 4.288\ kip \cdot ft$$ (three-quarter point)

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

$$F_{cr} := \frac{C_b \cdot \pi^2 \cdot E}{\left(\frac{L_b}{r_{ts}}\right)^2} \cdot \sqrt{1 + 0.078 \cdot \frac{J \cdot c}{S_x \cdot h_o} \left(\frac{L_b}{r_{ts}}\right)^2} = 49.339\ ksi$$

$$M_{nLTB} := F_{cr} \cdot S_x = 1562.402\ kip \cdot ft$$

Flexural strength:

$$\phi M_n := 0.9 \cdot min\left(M_n, M_{nLTB}\right) = 1406.161\ \boldsymbol{kip \cdot ft}$$

Maximum Moment:

$$M_{ux} := \frac{w \cdot L^2}{12} = 34.303\ \boldsymbol{kip \cdot ft}$$

$$f_{rbx} := \frac{M_{ux}}{S_x} = 1.083\ \boldsymbol{ksi} \qquad F_{cbx} := \frac{\phi M_n}{S_x} = 44.405\ \boldsymbol{ksi}$$

Bending about the y-axis:

$Z_y := 58.4\ \boldsymbol{in}^3 \qquad S_y := 37.2\ \boldsymbol{in}^3$

$$M_y := F_y \cdot S_y = 155\ \boldsymbol{kip \cdot ft}$$

$$M_{np} := F_y \cdot Z_y = 243.333\ \boldsymbol{kip \cdot ft} \qquad M_{ny} := 1.6\ M_y = 248\ \boldsymbol{kip \cdot ft} \qquad M_{nFLB} := min\left(M_{np}, 1.6\ M_y\right) = 243.333\ \boldsymbol{kip \cdot ft}$$

$$\phi M_n := min\left(M_{np}, M_{ny}, M_{nFLB}\right) = 243.333\ \boldsymbol{kip \cdot ft}$$

$M_{uy} := 186.77\ \boldsymbol{kip \cdot ft}$ (analysis done by robot to find Muy)

$$f_{rby} := \frac{M_{uy}}{S_y} = 60.248\ \boldsymbol{ksi} \qquad F_{cby} := \frac{\phi M_n}{S_y} = 78.495\ \boldsymbol{ksi}$$

$\left(\frac{f_{ra}}{F_{ca}} + \frac{f_{rbx}}{F_{cbx}} + \frac{f_{rby}}{F_{cby}}\right) = 0.799$ < 1, therefore the design is adequate

MF7 - W33x141 Beam, W14x159 Column:
Combined Forces - Flexure and Axial Force:

Axial Load (Column):

Check Nonslender vs Slender:

$b_f := 15.6\ in$ $\quad t_f := 1.19\ in$ $\quad F_y := 50\ ksi$

Width to thickness ratio for flanges of doubly symmetric I-section:

$$\frac{b_f}{2\ t_f} = 6.555$$

Limiting Width to Thickness Ratios:

$$\lambda_r := 0.56 \cdot \sqrt{\frac{E}{F_y}} = 13.487$$

$\frac{b_f}{2\ t_f} < \lambda_r$

therefore the flanges are nonslender

Width to thickness ratio for web of doubly symmetric I-section:

$$\frac{h}{t_w} = h_{div}t_w := 15.3$$

Limiting Width to Thickness Ratios:

$$\lambda_r := 1.49 \cdot \sqrt{\frac{E}{F_y}} = 35.884$$

$\frac{h}{t_w} < \lambda_r$

therefore the web is nonslender

Flexural Buckling of Doubly Symmetric I-Shape Members Without Slender Elements:

Column is Pinned-Fixed, therefore:

$$L := 18\ ft \qquad L_c := 0.80 \cdot L = 14.4\ ft$$

From AISC Tables:

$r_x := 6.38\ in$ $\quad I_x := 1900\ in^4$ $\quad A_g := 46.7\ in^2$ $\quad L_{cx} := L_c$ $\quad L_{cy} := L_c$ $\quad L_{cz} := L_c$

$r_y := 4.00\ in$ $\quad I_y := 748\ in^4$ $\quad J := 19.7\ in^4$ $\quad G := 11200\ ksi$ $\quad C_w := 35600\ in^6$

$$F_{ex} := \frac{\left(\pi^2 \cdot E\right)}{\left(\left(\frac{L_{cx}}{r_x}\right)^2\right)} = 390.168\ ksi \qquad F_{ey} := \frac{\left(\pi^2 \cdot E\right)}{\left(\left(\frac{L_{cy}}{r_y}\right)^2\right)} = 153.366\ ksi$$

Torsional and Flexural Torsional Buckling Consideration:

$$F_{ez} := \left(\frac{\left(\pi^2 \cdot E \cdot C_w\right)}{\left(L_{cz}^{\ 2}\right)} + G \cdot J\right) \cdot \frac{1}{\left(I_x + I_y\right)} = 212.19\ ksi$$

$$F_e := min\left(F_{ex}, F_{ey}, F_{ez}\right) = 153.366\ ksi \qquad \text{(Elastic Buckling Stress)}$$

$$\frac{F_y}{F_e} = 0.326 \qquad < 2.25\text{, therefore} \qquad F_{cr} = \left(0.658^{\frac{F_y}{F_e}}\right) \cdot F_y$$

$$F_{cr} := \left(0.658^{\frac{F_y}{F_e}}\right) \cdot F_y = 43.622\ ksi \qquad \text{(Critical Buckling Stress)}$$

Design Axial Compressive Strength:

$$P_n := F_{cr} \cdot A_g = 2037.162\ kip$$

$$\phi P_n := 0.9 \cdot P_n = 1833.446\ kip$$

Maximum Axial Load:

$$A_{TributaryC10} := 184\ ft^2$$

$$P_u := \left(q_{roof} \cdot A_{TributaryC10}\right) = 12.806\ kip$$

$$f_{ra} := \frac{P_u}{A_g} = 0.274\ ksi \qquad F_{ca} := \frac{\phi P_n}{A_g} = 39.26\ ksi$$

Bending about the x-axis (W33x141):

$w_{trib} := \frac{35\ ft}{2}$ $\quad w_{self} := 141\ plf$ $\quad w_{SelfFactored} := 1.2 \cdot w_{self} = 0.169\ klf$ $\quad L := FIF\left(\text{“20' 5”}\right)$

$$w := \left(w_{trib} \cdot q_{roof}\right) + w_{SelfFactored} = 1.387\ klf$$

FLEXURE: Check Compact vs Slender:

$b_f := 11.5\ in \qquad t_f := 0.960\ in \quad F_y := 50\ ksi$

Width to thickness ratio for flanges of doubly symmetric I-section:

$$\frac{b_f}{2\ t_f} = 5.99$$

Limiting Width to Thickness Ratios:

$$\lambda_p := 0.38 \cdot \sqrt{\frac{E}{F_y}} = 9.152 \qquad \lambda_r := 1.0 \cdot \sqrt{\frac{E}{F_y}} = 24.083$$

$$\frac{b_f}{2\ t_f} < \lambda_p < \lambda_r$$

therefore the flanges are compact

Width to thickness ratio for web of doubly symmetric I-section:

$$\frac{h}{t_w} = \ h_{div}t_w := 49.6$$

Limiting Width to Thickness Ratios:

$$\lambda_p := 3.76 \cdot \sqrt{\frac{E}{F_y}} = 90.553 \qquad \lambda_r := 5.70 \cdot \sqrt{\frac{E}{F_y}} = 137.274$$

$$\frac{h}{t_w} < \lambda_p < \lambda_r$$

therefore the web is compact

Flange local buckling will not occur.

Doubly Symmetric Compact I-Shape Subjected to Bending About the Major Axis:

Check Yielding:

$Z_x := 514\ in^3$

$$M_n := F_y \cdot Z_x = 2141.667\ kip \cdot ft$$

Check Lateral Torsional Buckling:

From AISC Section Tables:

$$I_y := 246\ in^4 \quad C_w := 64400\ in^6 \qquad r_y := 2.43\ in \quad S_x := 448\ in^3 \quad h_o := 32.3\ in \quad J := 9.70\ in^4$$

$c := 1$ (for doubly symmetric I-shapes)

$$r_{ts} := \sqrt{\frac{\left(I_y \cdot C_w\right)^{.5}}{S_x}} = 2.981\ in$$

$$L_p := 1.76 \cdot r_y \cdot \sqrt{\frac{E}{F_y}} = 8.583\ ft$$

$$L_r := 1.95 \cdot r_{ts} \cdot \frac{E}{0.7 \cdot F_y} \cdot \sqrt{\frac{J \cdot c}{S_x \cdot h_o} + \sqrt{\left(\frac{J \cdot c}{S_x \cdot h_o}\right)^2 + 6.76 \cdot \left(\frac{0.7 \cdot F_y}{E}\right)^2}} = 24.996\ ft$$

$$L_b := FIF\left(\text{“}5'\ 1\text{–}1/4\text{”}\right) = 5.104\ ft$$

Lb < Lp, therefore the limit state of lateral torsional buckling does not apply.

Flexural strength:

$$\phi M_n := 0.9 \cdot M_n = 1927.5\ kip \cdot ft$$

Maximum Moment:

$$M_{max} := \frac{w \cdot L^2}{12} = 48.187\ kip \cdot ft$$

$$f_{rbx} := \frac{M_{ux}}{S_x} = 0.919\ ksi \qquad F_{cbx} := \frac{\phi M_n}{S_x} = 51.629\ ksi$$

Bending about the y-axis:

$Z_y := 66.9\ in^3 \qquad S_y := 42.7\ in^3$

$$M_y := F_y \cdot S_y = 177.917\ kip \cdot ft$$

$$M_{np} := F_y \cdot Z_y = 278.75\ kip \cdot ft \qquad M_{ny} := 1.6\ M_y = 284.667\ kip \cdot ft \qquad M_{nFLB} := min\left(M_{np}, 1.6\ M_y\right) = 278.75\ kip \cdot ft$$

$$\phi M_n := min\left(M_{np}, M_{ny}, M_{nFLB}\right) = 278.75\ kip \cdot ft$$

$M_{uy} := 110.12\ kip \cdot ft$ (analysis done by robot to find Muy)

$$f_{rby} := \frac{M_{uy}}{S_y} = 30.947\ ksi \qquad F_{cby} := \frac{\phi M_n}{S_y} = 78.337\ ksi$$

$$\left(\frac{f_{ra}}{F_{ca}} + \frac{f_{rbx}}{F_{cbx}} + \frac{f_{rby}}{F_{cby}}\right) = 0.42$$

< 1, therefore the design is adequate

Appendix R: Building C: Shear and Moment Connection Design

Shear Connection Design:

Shear Tab Connection to column web:

$V_u := 43.08\ kip$ $F_{y\_plate} := 36\ ksi$ $F_{u\_bolt} := 120\ ksi$ $Z_x := 437\ in^3$

$F_y := 50\ ksi$ $F_{u\_plate} := 58\ ksi$

$E := 29000\ ksi$ $F_u := 65\ ksi$

$A_{beam} := 38.8\ in^2$ $t_{w\_beam} := 0.615\ in$ $A_{col} := 38.8\ in^2$ $t_{w\_col} := 0.645\ in$

$d_{beam} := 30.3\ in$ $t_{f\_beam} := 1\ in$ $d_{col} := 14.7\ in$ $t_{f\_col} := 1.03\ in$

$b_{f\_beam} := 10.5\ in$ $S_{beam} := 380\ in^3$ $b_{f\_col} := 14.7\ in$ $S_{col} := 209\ in^3$

$t_{plate\_shear} := \frac{3}{8}\ in$

$d_{b\_shear} := \frac{3}{4}\ in = 0.75\ in$

$\frac{d_{b\_shear}}{2} + \frac{1}{16}\ in = 0.438\ in$ > $t_{plate\_shear} = 0.375\ in$ OR $t_{w\_beam} = 0.615\ in$

both sides of the Plate are welded to the column web using Fillet welds.
Provide 1/4" weld on both sides.

$w := \frac{5}{8} \cdot t_{plate\_shear} = 0.234\ in$ $w := 0.25\ in$

Bolted connection of the plate to the web of the beam:

$A_b := \left(\frac{\pi}{4}\right) \cdot d_{b\_shear}{}^2 = 0.442\ in^2$

$F_{nv} := 0.4 \cdot F_{u\_bolt} = 48\ ksi$

$\phi R_{n\_shear} := 0.75 \cdot F_{nv} \cdot A_b = 15.904\ kip$

Bolt bearing strength in the beam web:

$\phi R_{n\_bearing\_web} := 0.75 \cdot 2.4 \cdot d_{b\_shear} \cdot t_{w\_beam} \cdot F_u = 53.966\ kip$

Bolt bearing strength in the shear plate:

$\phi R_{n\_shearplate} := 0.75 \cdot 2.4 \cdot d_{b\_shear} \cdot t_{plate\_shear} \cdot F_{u\_plate} = 29.363\ kip$

Bolt shear governs:

$N_{bolts\_shear} := \frac{V_u}{\phi R_{n\_shearplate}} = 1.467$ try 3 bolts: $N_{bolts} := 3$

$s := 3\ in$ $L_e := 1.5\ in$

$L_p := 2 \cdot L_e + 2 \cdot s = 9\ in$

$d_{beam} - 2 \cdot t_{f\_beam} = 28.3\ in$

$b_p := s + L_e = 4.5\ in$

Shear failure mode of shear tab:

$d_h := d_{b\_shear} + \frac{1}{16}\ in + \frac{1}{16}\ in = 0.875\ in$

$A_{gv} := t_{plate\_shear} \cdot L_p = 3.375\ in^2$

$A_{nv} := A_{gv} - 2 \cdot d_h \cdot t_{plate\_shear} = 2.719\ in^2$

Plate strength in shear:

$\phi R_{n\_plate\_shear} := min\left(1.0 \cdot 0.6 \cdot F_{y\_plate} \cdot A_{gv}, 0.75 \cdot 0.6 \cdot F_{u\_plate} \cdot A_{nv}\right) = 70.959\ kip$

> $V_u = 43.08\ kip$ OK

Block shear strength:

$$A_{gv} := t_{plate\_shear} \cdot \left(L_e + \left(N_{bolts} - 1\right) \cdot s\right) = 2.813\ in^2$$

$$A_{nv} := A_{gv} - \left(\left(N_{bolts} - 1\right) + 0.5\right) \cdot d_h \cdot t_{plate\_shear} = 1.992\ in^2$$

$$A_{nt} := \left(L_e - \frac{d_h}{2}\right) \cdot t_{plate\_shear} = 0.398\ in^2$$

$$U := 1$$

$$\phi R_{n\_blockshear} := 0.75 \cdot \left(min\left(0.6 \cdot F_{u\_plate} \cdot A_{nv}, 0.6 \cdot F_{y\_plate} \cdot A_{gv}\right) + U \cdot F_{u\_plate} \cdot A_{nt}\right) = 62.895\ kip$$

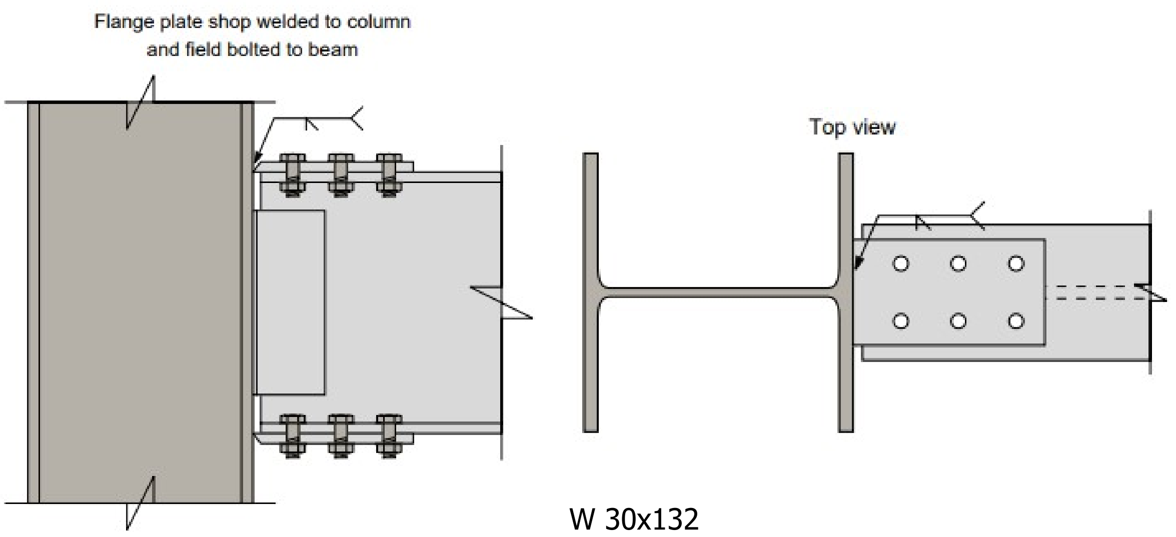
$> \quad V_u = 43.08\ kip$ OK

Final Shear Plate dimensions:

$s := 3\ in$ $L_e := 1.5\ in$ $b_p = 4.5\ in$

$N_{bolts} = 3$ $L_p = 9\ in$

Moment Connection:



W 30x132

$V_u := 7.16\ kip$ $F_{y\_plate} := 36\ ksi$ $F_{u\_bolt} := 120\ ksi$ $Z_x := 437\ in^3$

$M_u := 122.99\ kip \cdot ft$ $F_y := 50\ ksi$ $F_{u\_plate} := 58\ ksi$

$E := 29000\ ksi$ $F_u := 65\ ksi$

$A_{beam} := 38.8\ in^2$ $t_{w\_beam} := 0.615\ in$ $A_{col} := 38.8\ in^2$ $t_{w\_col} := 0.645\ in$

$d_{beam} := 30.3\ in$ $t_{f\_beam} := 1\ in$ $d_{col} := 14.7\ in$ $t_{f\_col} := 1.03\ in$

$b_{f\_beam} := 10.5\ in$ $S_{beam} := 380\ in^3$ $b_{f\_col} := 14.7\ in$ $S_{col} := 209\ in^3$

$$d_b := \frac{3}{4}\ in$$

$$A_b := \left(\frac{\pi}{4}\right) \cdot {d_b}^2 = 0.442\ in^2$$ assuming standard punched holes

$$d_h := d_b + \frac{1}{16}\ in + \frac{1}{16}\ in = 0.875\ in$$

Selection of flange plate dimensions:

$$P_u := \frac{M_u}{d_{beam}} = 48.709\ kip$$

$L_e := 1.75\ in$ $s := 3\ in$

$g := 6\ in$

$b_p \quad > \quad g + 2 \cdot L_e = 9.5\ in$

$b_p := 10\ in$ OK

$$A_p \quad > \quad \frac{P_u}{0.9 \cdot F_{y\_plate}} = 1.503\ in^2$$

$$t_{plate} := \frac{3}{8}\ in = 0.375\ in$$

$A_p := b_p \cdot t_{plate} = 3.75\ in^2$ OK

$F_{nv} := 0.4 \cdot F_{u\_bolt} = 48\ ksi$

$\phi R_{n\_shear} := 0.75 \cdot F_{nv} \cdot A_b = 15.904\ kip$

$N_{bolts\_req} := \dfrac{P_u}{\phi R_{n\_shear}} = 3.063$

will use 2 rows of 2 bolts with at total of 4 to be conservative. at 6 in gauge

$L_{plate\_min} := 2 \cdot L_e + s = 6.5\ in$

Final Flange Plate Dimensions:
Tension

$L_{plate} := L_{plate\_min} + 0.5\ in = 7\ in$ add an additional half for setback.

$t_{plate} = 0.375\ in$

$b_{plate} := b_p = 10\ in$

Bolt Bearing in Beam Flange:

$\begin{bmatrix} t_{plate} \cdot F_{u\_plate} \\ t_{f\_beam} \cdot F_u \end{bmatrix} = \begin{bmatrix} 261 \\ 780 \end{bmatrix} \dfrac{kip}{ft}$

The plate will be critical in this case.

$t := t_{plate} = 0.375\ in$

$L_{c1} := L_e - \dfrac{d_h}{2} = 1.313\ in$

$\phi R_{n\_end\_bearing} := 0.75 \cdot min\left(1.2 \cdot L_{c1} \cdot t \cdot F_{u\_plate}, 2.4 \cdot d_b \cdot t \cdot F_{u\_plate}\right) = 25.692\ kip$

Both bolts are end bolts so this is the only calc needed.

$\phi R_{n\_bolts} := 4 \cdot \phi R_{n\_end\_bearing} = 102.769\ kip$ > $P_u = 48.709\ kip$ OK

Tensile strength of flange plate:

$A_p = 3.75\ in^2$

$\phi R_{n\_tension\_yield} := 0.9 \cdot F_{y\_plate} \cdot A_p = 121.5\ kip$

$A_n := A_p - 2 \cdot d_h \cdot t_{plate} = 3.094\ in^2$

$\phi R_{n\_Tension\_Fracture} := 0.75 \cdot F_{u\_plate} \cdot A_n = 134.578\ kip$

$\phi R_{n\_plate} := min\left(\phi R_{n\_tension\_yield}, \phi R_{n\_Tension\_Fracture}\right) = 121.5\ kip$ > $P_u = 48.709\ kip$

OK

Block shear Failure of the plate between the two lines of bolts:

$A_{gv} := 2 \cdot t_{plate} \cdot \left(s + L_e\right) = 3.563\ in^2$

$A_{nv} := A_{gv} - 2 \cdot 1.5 \cdot d_h \cdot t_{plate} = 2.578\ in^2$

$A_{gt} := g \cdot t_{plate} = 2.25\ in^2$

$A_{nt} := A_{gt} - d_h \cdot t_{plate} = 1.922\ in^2$

$U := 1$

$\phi R_{n\_block\_shearangle} := 0.75 \cdot \left(min\left(0.6 \cdot F_{u\_plate} \cdot A_{nv}, 0.6 \cdot F_{y\_plate} \cdot A_{gv}\right) + U \cdot F_{u\_plate} \cdot A_{nt}\right) = 169.181\ kip$

> $P_u = 48.709\ kip$ OK

Block shear failure of the plate outside of the bolt lines:

$A_{gv} := 2 \cdot t_{plate} \cdot \left(s + L_e\right) = 3.563\ in^2$

$A_{nv} := A_{gv} - 2 \cdot 1.5 \cdot d_h \cdot t_{plate} = 2.578\ in^2$

$A_{nt} := \left(b_{plate} - g - d_h\right) \cdot t_{plate} = 1.172\ in^2$

$U := 1$

$$\phi R_{n\_block\_shearangle} := 0.75 \cdot \left(\min\left(0.6 \cdot F_{u\_plate} \cdot A_{nv}, 0.6 \cdot F_{y\_plate} \cdot A_{gv}\right) + U \cdot F_{u\_plate} \cdot A_{nt}\right) = 125.681\ kip$$

$>$ $P_u = 48.709\ kip$ OK

Block shear failure of the beam flange outside of the bolt lines:

$$A_{gv} := 2 \cdot t_{f\_beam} \cdot \left(s + L_e\right) = 9.5\ in^2$$

$$A_{nv} := A_{gv} - 2 \cdot 3.5 \cdot d_h \cdot t_{f\_beam} = 3.375\ in^2$$

$$A_{nt} := \left(b_{f\_beam} - g - d_h\right) \cdot t_{f\_beam} = 3.625\ in^2$$

$$\phi R_{n\_block\_shearangle} := 0.75 \cdot \left(\min\left(0.6 \cdot F_{u\_plate} \cdot A_{nv}, 0.6 \cdot F_{y\_plate} \cdot A_{gv}\right) + U \cdot F_{u\_plate} \cdot A_{nt}\right) = 298.338\ kip$$

$>>$ $P_u = 48.709\ kip$ OK

Final Plate Dimensions: Compression

$L_{plate} := L_{plate\_min} + 0.5\ in = 7\ in$

$t_{plate} = 0.375\ in$

$b_{plate} := b_p = 10\ in$

Local plate buckling:

$$\lambda_r := 0.45 \cdot \sqrt{\frac{E}{F_{y\_plate}}} = 12.772$$ unstiffened

$$\lambda_r := 1.49 \cdot \sqrt{\frac{E}{F_{y\_plate}}} = 42.29$$ stiffened

$$\begin{bmatrix} \dfrac{L_e}{t_{plate}} \\ \dfrac{g}{t_{plate}} \end{bmatrix} = \begin{bmatrix} 4.667 \\ 16 \end{bmatrix}$$

these are less than the above so
we can consider these non slender.

$Q := 1$

Plate buckling over its length:

$$L := \max\left(s, L_e + 0.5\ in\right) = 3\ in$$

$$I_p := \left(\frac{1}{12}\right) \cdot b_{plate} \cdot t_{plate}^{\ 3} = 0.044\ in^4$$

$$r_p := \sqrt{\frac{I_p}{A_p}} = 0.108\ in$$

$$F_e := \frac{\pi^2 \cdot E}{\left(\dfrac{L}{r_p}\right)^2} = 372.68\ ksi$$

$$F_{cr} := \begin{cases} 0.658^{\left(\frac{F_{y\_plate}}{F_e}\right)} \cdot F_{y\_plate} & \text{if } \dfrac{F_{y\_plate}}{F_e} \\ 0.877 \cdot F_e & \text{else} \end{cases} = 34.574\ ksi$$

$$\frac{L}{r_p} = 27.713$$

$\phi P_n := 0.9 \cdot F_{cr} \cdot A_p = 116.686\ kip$ $>$ $P_u = 48.709\ kip$ OK

Flange plate welded connection to the column flange:

$F_{exx} := 70\ ksi$

$\text{clear}(w)$

$$P_u = 0.75 \cdot 0.707 \cdot w \cdot L \cdot (0.9 \cdot F_{exx}) \xrightarrow{solve, w} \frac{0.48603320178002407265 \cdot kip}{in \cdot ksi}$$

$$w := \frac{0.29073471546431716 98 \cdot kip}{in \cdot ksi} = 0.291\ in$$

This means we will need a larger plate to fit this weld increase plate thickness to 3/8. This will allow the whole plate to be welded.

$$\frac{3}{8}\ in = 0.375\ in$$

Check reduced beam strength due to holes in the flange:

$$A_{fg} := b_{f\_beam} \cdot t_{f\_beam} = 10.5\ in^2$$

$$A_{fn} := A_{fg} - 2 \cdot d_h \cdot t_{f\_beam} = 8.75\ in^2$$

$$\frac{F_y}{F_u} = 0.769$$

< 0.8 so

$$Y_t := 1$$

$$\begin{bmatrix} F_u \cdot A_{fn} \\ Y_t \cdot F_y \cdot A_{fg} \end{bmatrix} = \begin{bmatrix} 568.75 \\ 525 \end{bmatrix} kip$$

since the first is greater than the second the bolts can be ignored.

Flange Local Buckling

$$\lambda_f := \frac{b_{f\_beam}}{2 \cdot t_{f\_beam}} = 5.25$$

$$\lambda_{pf} := 0.38 \cdot \sqrt{\frac{E}{F_y}} = 9.152$$

$$\lambda_{rf} := 1.0 \cdot \sqrt{\frac{E}{F_y}} = 24.083$$

$$M_p := F_y \cdot Z_x = 1820.833\ kip \cdot ft$$

For compact flanges ($\lambda_f \leq \lambda_{pf}$): $\phi M_n = 0.9\, M_p$.

$$\phi M_n := 0.9 \cdot M_p = 1638.75\ kip \cdot ft \quad >> \quad M_u = 122.99\ kip \cdot ft$$

OK

Shear Plate Connection:

$$t_{plate\_shear} := \frac{3}{8}\ in$$

$$d_{b\_shear} := \frac{3}{4}\ in = 0.75\ in$$

$$\frac{d_{b\_shear}}{2} + \frac{1}{16}\ in = 0.438\ in \quad > \quad t_{plate\_shear} = 0.375\ in \quad \text{OR} \quad t_{w\_beam} = 0.615\ in$$

both sides of the Plate are welded to the column flange using Fillet welds. Provide 1/4" weld on both sides.

$$w := \frac{5}{8} \cdot t_{plate\_shear} = 0.234\ in \qquad w := 0.25\ in$$

Bolted connection of the plate to the web of the beam:

$$A_b := \left(\frac{\pi}{4}\right) \cdot {d_{b\_shear}}^2 = 0.442\ in^2$$

$$F_{nv} := 0.4 \cdot F_{u\_bolt} = 48\ ksi$$

$$\phi R_{n\_shear} := 0.75 \cdot F_{nv} \cdot A_b = 15.904\ kip$$

Bolt bearing strength in the beam web:

$$\phi R_{n\_bearing\_web} := 0.75 \cdot 2.4 \cdot d_{b\_shear} \cdot t_{w\_beam} \cdot F_u = 53.966\ kip$$

Bolt bearing strength in the shear plate:

$$\phi R_{n\_shearplate} := 0.75 \cdot 2.4 \cdot d_{b\_shear} \cdot t_{plate\_shear} \cdot F_{u\_plate} = 29.363\ kip$$

Bolt shear governs:

$N_{bolts\_shear} := \dfrac{V_u}{\phi R_{n\_shearplate}} = 0.244$ try 2 bolts:

$s := 3\ in$ $L_e := 1.5\ in$

$L_p := 2 \cdot L_e + s = 6\ in$

$d_{beam} - 2 \cdot t_{f\_beam} = 28.3\ in$

$b_p := s + L_e = 4.5\ in$

Shear failure mode of shear tab:

$d_h := d_{b\_shear} + \dfrac{1}{16}\ in + \dfrac{1}{16}\ in = 0.875\ in$

$A_{gv} := t_{plate\_shear} \cdot L_p = 2.25\ in^2$

$A_{nv} := A_{gv} - 2 \cdot d_h \cdot t_{plate\_shear} = 1.594\ in^2$

Plate strength in shear:

$\phi R_{n\_plate\_shear} := min\left(1.0 \cdot 0.6 \cdot F_{y\_plate} \cdot A_{gv}, 0.75 \cdot 0.6 \cdot F_{u\_plate} \cdot A_{nv}\right) = 41.597\ kip$

> $V_u = 7.16\ kip$ OK

Block shear strength:

$A_{gv} := t_{plate\_shear} \cdot \left(L_e + (2-1) \cdot s\right) = 1.688\ in^2$

$A_{nv} := A_{gv} - ((2-1) + 0.5) \cdot d_h \cdot t_{plate\_shear} = 1.195\ in^2$

$A_{nt} := \left(L_e - \dfrac{d_h}{2}\right) \cdot t_{plate\_shear} = 0.398\ in^2$

$U := 1$

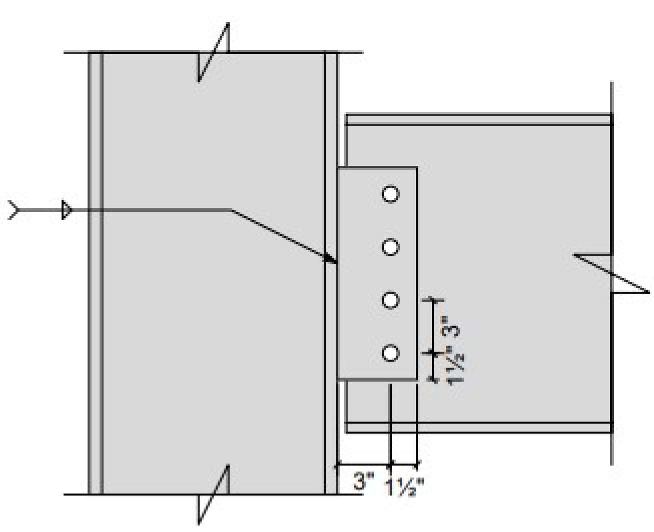
$\phi R_{n\_blockshear} := 0.75 \cdot \left(min\left(0.6 \cdot F_{u\_plate} \cdot A_{nv}, 0.6 \cdot F_{y\_plate} \cdot A_{gv}\right) + U \cdot F_{u\_plate} \cdot A_{nt}\right) = 44.67\ kip$

> $V_u = 7.16\ kip$ OK

Final Shear Plate dimensions:

$s := 3\ in$ $L_e := 1.5\ in$ $b_p = 4.5\ in$

$N_{bolts} := 2$ $L_p = 6\ in$



Appendix S: Building C: Roof Deck Design

Roof Loading:

Dead Load:	Live Load:	Snow Load:
$D_{Lroof} := 20\ psf$	$L_{Lroof} := 20\ psf$	$S_{roof} := 16\ psf$

Applicable LRFD Load Combinations:

$q_1 := 1.4 \cdot D_{Lroof} = 28\ psf$ $\quad$ $q_2 := 1.2 \cdot D_{Lroof} + 1.6 \cdot L_{Lroof} + 0.5 \cdot S_{roof} = 64\ psf$ $\quad$ $q_3 := 1.2 \cdot D_{Lroof} + 1.6 \cdot S_{roof} + 1.0 \cdot L_{Lroof} = 69.6\ psf$

$$q_{roof} := \max\left(q_1, q_2, q_3\right) = 69.6\ psf$$

Maximum factored roof load is 70 psf, so must select a deck to support this load.

The building is 96 feet long. Using double span decks, it would take 6 sheets at an 8'-0" span (16'-0" for double span) to cover the roof.

1.5B-36/1.5BI-36/1.5PLB-36 ROOF DECKS GRADE 50 STEEL

LRFD

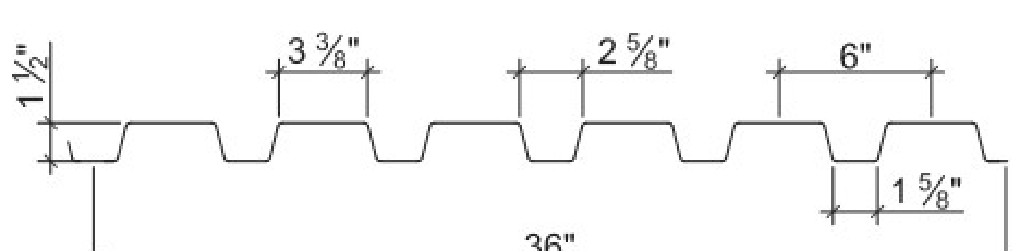
Inward Uniform Design Loads, LRFD (psf)

Deck Gage	Spans	Criteria	2'-0"	3'-0"	4'-0"	5'-0"	6'-0"	7'-0"	8'-0"	9'-0"	10'-0"	11'-0"	12'-0"
			Span (ft-in.)										
22	Single	$øW_n$	1267	563	317	203	141	103	79	63	51	42	35
		L/240	1270	376	159	81	47	30	20	14	10	8	6
	Double	$øW_n$	1240	575	329	212	148	109	83	66	54	44	37
		L/240	3514	1041	439	225	130	82	55	39	28	21	16
	Triple	$øW_n$	1502	708	407	263	184	136	104	82	67	55	46
		L/240	2754	816	344	176	102	64	43	30	22	17	13
20	Single	$øW_n$	1679	746	420	269	187	137	105	83	67	56	47
		L/240	1614	478	202	103	60	38	25	18	13	10	7
	Double	$øW_n$	1572	732	419	271	189	139	107	84	68	57	48
		L/240	4283	1269	535	274	159	100	67	47	34	26	20
	Triple	$øW_n$	1898	900	519	336	235	173	133	105	85	71	59
		L/240	3357	995	420	215	124	78	52	37	27	20	16
19	Single	$øW_n$	1994	886	499	319	222	163	125	98	80	66	55
		L/240	1958	580	245	125	73	46	31	21	16	12	9
	Double	$øW_n$	1894	886	508	328	229	169	129	102	83	69	58
		L/240	5073	1503	634	325	188	118	79	56	41	30	23
	Triple	$øW_n$	2281	1087	628	407	285	210	161	128	104	86	72
		L/240	3976	1178	497	254	147	93	62	44	32	24	18
18	Single	$øW_n$	2295	1020	574	367	255	187	143	113	92	76	64
		L/240	2270	673	284	145	84	53	35	25	18	14	11
	Double	$øW_n$	2162	1012	581	375	262	193	148	117	95	79	66
		L/240	5724	1696	716	366	212	134	89	63	46	34	27
	Triple	$øW_n$	2602	1242	718	465	326	240	185	146	119	98	82
		L/240	4487	1329	561	287	166	105	70	49	36	27	21
16	Single	$øW_n$	2948	1310	737	472	328	241	184	146	118	97	82
		L/240	2983	884	373	191	110	70	47	33	24	18	14
	Double	$øW_n$	2727	1278	734	474	331	244	187	148	120	99	83
		L/240	7244	2146	906	464	268	169	113	79	58	44	34
	Triple	$øW_n$	3280	1567	907	588	412	304	233	185	150	124	104
		L/240	5678	1682	710	363	210	132	89	62	45	34	26

Note:
1. Table does not account for web crippling. Required bearing should be determined based on specific span conditions.

From the tables for 1.5B deck, at 8'-0" double span, deflection is the controlling criteria. A 19 gage Grade 50 Double 8'-0" Span works.
Deck Weight = 2.3 psf

Nominal Dimensions



Section Properties

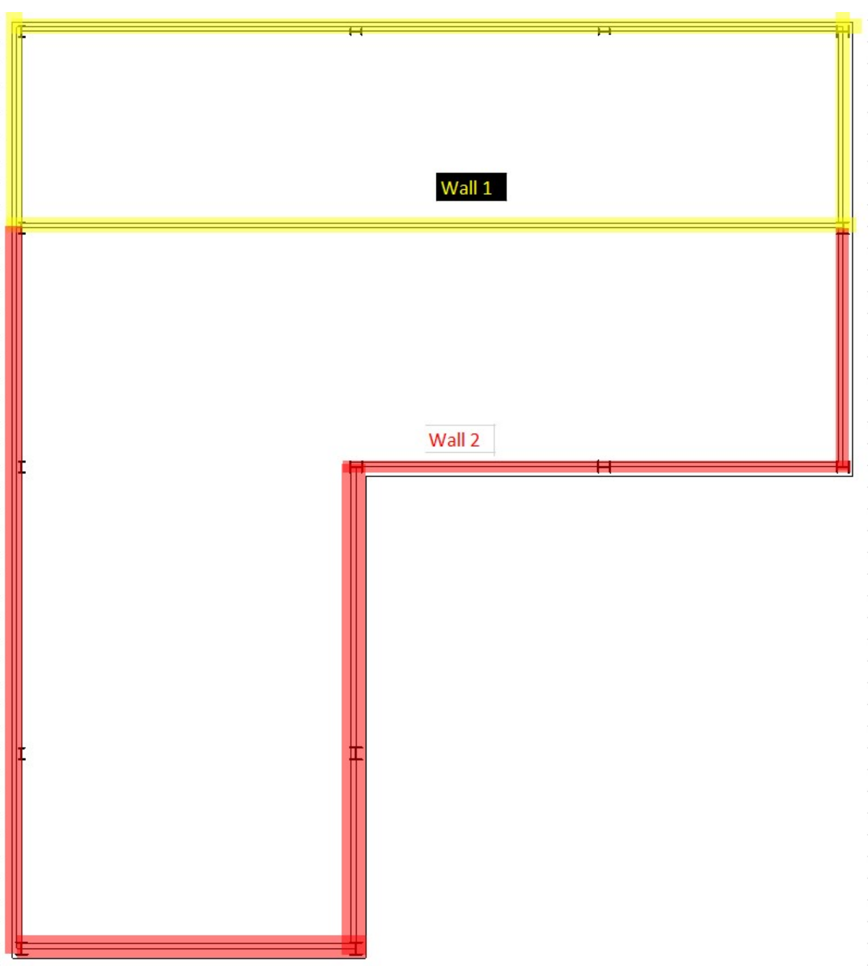
Deck Gage	Deck Weight w_{dd} (psf)	Base Metal Thickness t (in.)	Yield Strength F_y (ksi)	Effective Moment of Inertia at Service Load $I_d = (2I_e+I_g)/3$ I_d+ (in⁴/ft)	I_d- (in⁴/ft)	Effective Section Modulus at F_y = 50 ksi S_e+ (in³/ft)	S_e- (in³/ft)	Design Moment $øM_n$+ (lb-ft/ft)	$øM_n$- (lb-ft/ft)	Vertical Web Shear $øV_n$ (lb/ft)
22	1.6	0.0295	50	0.155	0.178	0.169	0.179	634	671	4035
20	2.0	0.0358	50	0.197	0.217	0.224	0.229	840	859	4874
19	2.3	0.0418	50	0.239	0.257	0.266	0.278	997	1042	5666
18	2.6	0.0474	50	0.277	0.290	0.306	0.318	1148	1193	6398
16	3.3	0.0598	50	0.364	0.367	0.393	0.402	1474	1508	7996

https://vulcraft.com/catalogs/Deck/RoofDeck/LRFD-1.5B-36-1.5BI-36-1.5PLB-36_GR50_Roof_Deck.pdf

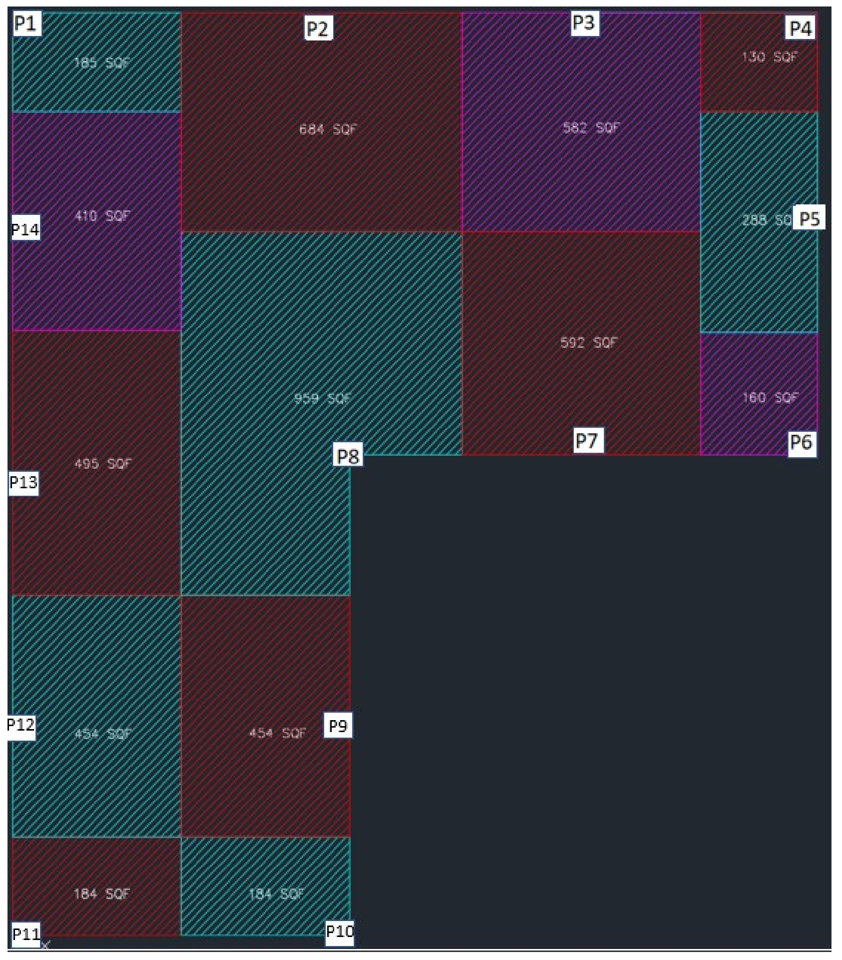
Appendix T: Building C: Foundation Geotechnical Limit State Design

Assumptions:
- water table = 4 ft below surface
- angle of friction = 30 deg
- Gs = 2.7
- non-cohesive soil
- Es = 750 tsf

Walls 1 and 2 to be designed:



Tributary Areas for Columns:



$$q_{roofASD} := 47 \; psf$$

$$q_{roof} := q_{roofASD}$$

Column Loading:

$P_1 := q_{roof} \cdot A_{trib1} = 8.695 \; kip$

$P_2 := q_{roof} \cdot A_{trib2} = 32.148 \; kip$

$P_3 := q_{roof} \cdot A_{trib3} = 27.354 \; kip$

$P_4 := q_{roof} \cdot A_{trib4} = 6.11 \; kip$

$P_5 := q_{roof} \cdot A_{trib5} = 13.536 \; kip$

$P_6 := q_{roof} \cdot A_{trib6} = 7.52 \; kip$

$P_7 := q_{roof} \cdot A_{trib7} = 27.824 \; kip$

$P_8 := q_{roof} \cdot A_{trib8} = 45.073 \; kip$

$P_9 := q_{roof} \cdot A_{trib9} = 21.338 \; kip$

$P_{10} := q_{roof} \cdot A_{trib10} = 8.648 \; kip$

$P_{11} := q_{roof} \cdot A_{trib11} = 8.648 \; kip$

$P_{12} := q_{roof} \cdot A_{trib12} = 21.338 \; kip$

$P_{13} := q_{roof} \cdot A_{trib13} = 23.265 \; kip$

$P_{14} := q_{roof} \cdot A_{trib14} = 19.27 \; kip$

Wall 1: Pilaster Design:

Soil properties from soil reports:

$$Gs := 2.7 \qquad e := 0.4 \qquad E_s := 750\ \frac{tonf}{ft^2} \qquad D_w := 4\ ft \qquad \phi' := 30\ deg \qquad \gamma_w := 62.4\ pcf \quad \gamma_{conc} := 150\ pcf$$

$$\gamma_{sat} := \frac{(Gs + e) \cdot \gamma_w}{1 + e} = 138.171\ pcf \quad \gamma' := \gamma_{sat} - \gamma_w = 75.771\ pcf \quad \gamma_{dry} := 130\ pcf \qquad c' := 0\ psf \qquad \gamma_{backfill} := 120\ pcf$$

For wall 1, Square footings for the Pilasters will need to be designed for columns P1, P2, P3, P4, P5 and P14. For ease of construction, one footing size can be selected based on the most critical loading.

$$P_{des} := \max\left(P_1, P_2, P_3, P_4, P_5, P_{14}\right) = 32.148\ kip$$

Define Design Parameters:

$$B := FIF(\text{“}3'\,2\text{”}) \quad H_w := FIF(\text{“}12'\,0\text{”}) \quad B_{col} := FIF(\text{“}1'\,6\text{”}) \quad t_f := 2\ ft$$

$D_f := 9\ in + t_f = 2.75\ ft$ (5 inch slab, 4 inch of backfill between slab and foundation)

$$H := \left(H_w + \left(D_f - t_f\right)\right) = 12.75\ ft$$

$$H_{total} := H_w + D_f = 14.75\ ft$$

Check for Bearing Failure:

$$FS_q := 3$$

Vesic's Equation for Bearing Capacity:

$$q_{ult} = c' \cdot N_c \cdot s_c \cdot d_c \cdot i_c \cdot b_c \cdot g_c + \sigma'_z \cdot N_q \cdot s_q \cdot d_q \cdot i_q \cdot b_q \cdot g_q + 0.5 \cdot \gamma_{backfill} \cdot B' \cdot N_\gamma \cdot s_\gamma \cdot d_\gamma \cdot i_\gamma \cdot b_\gamma \cdot g_\gamma$$

Bearing Capacity Factors:
From table of bearing capacity factors for ϕ' = 30 degrees and using Vesic's Equation:

$$N_q := 18.4 \quad N_\gamma := 22.4$$

Ground Inclination Factors:
Assume level ground, therefore, ground inclination factors are equal to 1:

$$g_q := 1 \qquad g_\gamma := 1$$

Load Inclination Factors:

$$i_q := 1 \qquad i_\gamma := 1$$

Base Inclination Factors:
The base is not inclined, therefore the base inclination factors are equal to 1:

$$b_q := 1 \qquad b_\gamma := 1$$

For continuous footings, $B/L \rightarrow 0$, so s_c, s_q, and s_γ become equal to 1. This means the s factors may be ignored when analyzing continuous footings.

Shape Factors:

$$s_q := 1 \qquad s_\gamma := 1$$

Depth Factors:

$$d_q := 1 + 2 \cdot \frac{D_f}{B} \cdot \tan(\phi') \cdot \left(1 - \sin(\phi')\right)^2 = 1.251 \qquad d_\gamma := 1$$

Effective Unit Weight:

$$\gamma' := \gamma_{sat} - \gamma_w = 75.771\ pcf$$

Vertical Effective Stress:

$$u := \gamma_w \cdot \left(H_{total} - D_w\right) = 670.8\ psf$$

$$\sigma'_{zD} := \left(D_w \cdot \gamma_{backfill}\right) + \left(\left(H_{total} \cdot \gamma'\right) - u\right) = 926.829\ psf$$

Since c' is 0, the first part of the equation is cancelled, left with:

$$q_{ult} := \sigma'_{zD} \cdot N_q \cdot s_q \cdot d_q \cdot i_q \cdot b_q \cdot g_q + 0.5 \cdot \gamma' \cdot B \cdot N_\gamma \cdot s_\gamma \cdot d_\gamma \cdot i_\gamma \cdot b_\gamma \cdot g_\gamma = 24016.211\ psf$$

Bearing Pressure:

$$q_{gross} := \frac{P_{des}}{(B \cdot B)} + \left(\gamma_{conc} \cdot t_f\right) + \left(\gamma_{backfill} \cdot \left(D_f - t_f\right)\right) + \left(\gamma_{backfill} \cdot H\right) = 5125.895\ psf$$

$$check := \text{if } \frac{q_{ult}}{q_{gross}} > FS_q \;\; \| \; \text{“The design is acceptable”} \;\; \text{else} \;\; \| \; \text{“The design is not acceptable”}$$

$$\frac{q_{ult}}{q_{gross}} = 4.685$$

$$check = \text{“The design is acceptable”}$$

Check for Settlement Failure:

$L := B \qquad B = 3.167\ ft \qquad H := 5 \cdot B \qquad \mu_s := 0.3 \qquad \alpha := 4$ (Footing Center) $\delta_{all} := 0.5\ in$

$$L' := \frac{L}{2} = 1.583\ ft \qquad B' := \frac{B}{2} = 1.583\ ft$$

$$M := \frac{L'}{B'} = 1 \qquad N := \frac{H}{B'}$$

Influence Factors:

$$I_1 := \frac{1}{\pi} \cdot \left(M \cdot \ln\left(\frac{\left(1 + \sqrt{M^2 + 1}\right) \cdot \sqrt{M^2 + N^2}}{M \cdot \left(1 + \sqrt{M^2 + N^2 + 1}\right)} \right) + \ln\left(\frac{\left(M + \sqrt{M^2 + 1}\right) \cdot \sqrt{1 + N^2}}{M + \sqrt{M^2 + N^2 + 1}} \right) \right) = 0.498$$

$$I_2 := \frac{N}{2 \cdot \pi} \cdot \operatorname{atan}\left(\frac{M}{N \cdot \sqrt{M^2 + N^2 + 1}} \right) = 0.016$$

Shape Correction Factor:

$$I_s := I_1 + \left(\frac{1 - 2 \cdot \mu_s}{1 - \mu_s} \right) \cdot I_2 = 0.507$$

Fox Depth Correction Factor, If:

$$\beta_1 := 3 - 4 \cdot \mu_s \qquad \beta_2 := 5 - 12 \cdot \mu_s + 8 \cdot {\mu_s}^2 \qquad \beta_3 := -4 \cdot \mu_s \cdot \left(1 - 2 \cdot \mu_s\right)$$

$$\beta_4 := -1 + 4 \cdot \mu_s - 8 \cdot {\mu_s}^2 \qquad \beta_5 := -4 \cdot \left(1 - 2 \cdot \mu_s\right)^2$$

$$r := 2 \cdot D_f = 5.5\ ft$$

$$r_1 := \sqrt{L^2 + r^2} = 6.346\ ft$$

$$r_2 := \sqrt{B^2 + r^2} = 6.346\ ft \qquad r_3 := \sqrt{L^2 + B^2 + r^2} = 7.093\ ft \qquad r_4 := \sqrt{L^2 + B^2} = 4.478\ ft$$

$$Y_1 := L \cdot \ln\left(\frac{r_4 + B}{L} \right) + B \cdot \ln\left(\frac{r_4 + L}{B} \right) - \frac{{r_4}^3 - L^3 - B^3}{3 \cdot L \cdot B} = 4.708\ ft$$

$$Y_2 := L \cdot \ln\left(\frac{r_3 + B}{r_1} \right) + B \cdot \ln\left(\frac{r_3 + L}{r_2} \right) - \frac{{r_3}^3 - {r_2}^3 - {r_1}^3 + r^3}{3 \cdot L \cdot B} = 2.645\ ft$$

$$Y_3 := \frac{r^2}{L} \cdot \ln\left(\frac{\left(B + r_2\right) \cdot r_1}{\left(B + r_3\right) \cdot r} \right) + \frac{r^2}{B} \cdot \ln\left(\frac{\left(L + r_1\right) \cdot r_2}{\left(L + r_3\right) \cdot r} \right) = 1.292\ ft$$

$$Y_4 := \frac{r^2 \cdot \left(r_1 + r_2 - r_3 - r\right)}{L \cdot B} = 0.303\ ft$$

$$Y_5 := r \cdot \operatorname{atan}\left(\frac{L \cdot B}{r \cdot r_3} \right) = 1.384\ ft$$

$$I_f := \frac{\beta_1 \cdot Y_1 + \beta_2 \cdot Y_2 + \beta_3 \cdot Y_3 + \beta_4 \cdot Y_4 + \beta_5 \cdot Y_5}{\left(\beta_1 + \beta_2\right) \cdot Y_1} = 0.673$$

Bearing Pressure:

$$q_{gross} := \frac{P_{des}}{(B \cdot B)} + \left(\gamma_{conc} \cdot t_f\right) + \left(\gamma_{backfill} \cdot \left(D_f - t_f\right)\right) + \left(\gamma_{backfill} \cdot H\right) = 5495.895\ psf$$

$$\sigma'_{zo} := \left(D_w \cdot \gamma_{backfill}\right) + \left(\left(H_{total} \cdot \gamma'\right) - u\right) = 926.829\ psf$$

$$q_{net} := q_{gross} - \sigma'_{zo} = 4569.066\ psf$$

Foundation Settlement:

$$\delta_{flexible} := \alpha \cdot I_s \cdot I_f \cdot \left(\frac{q_{net} \cdot \left(1 - {\mu_s}^2\right)}{E_s} \right) \cdot B'$$

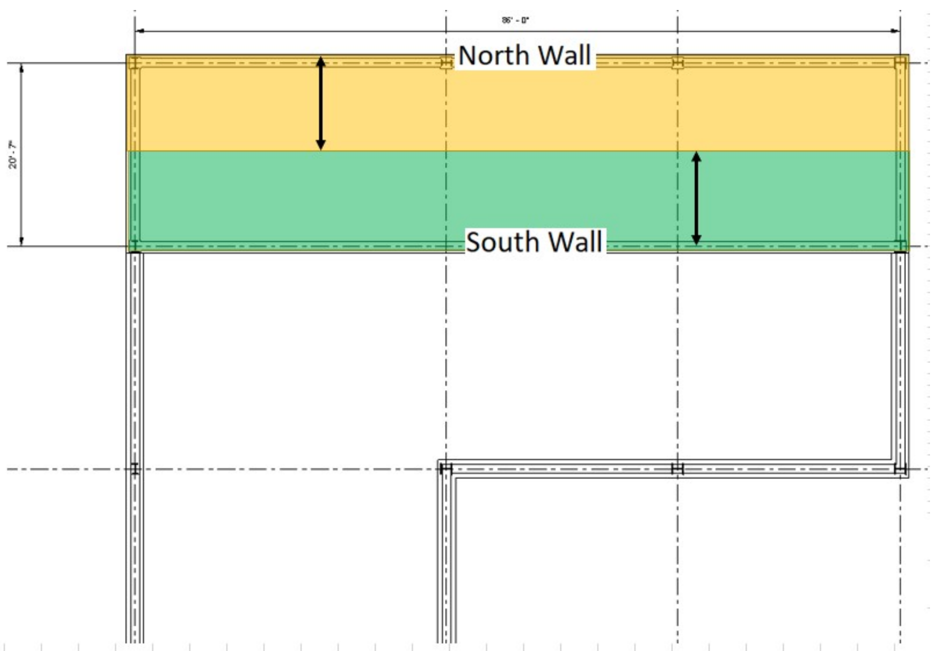
$$\delta_{flexible} = 0.072\ in$$

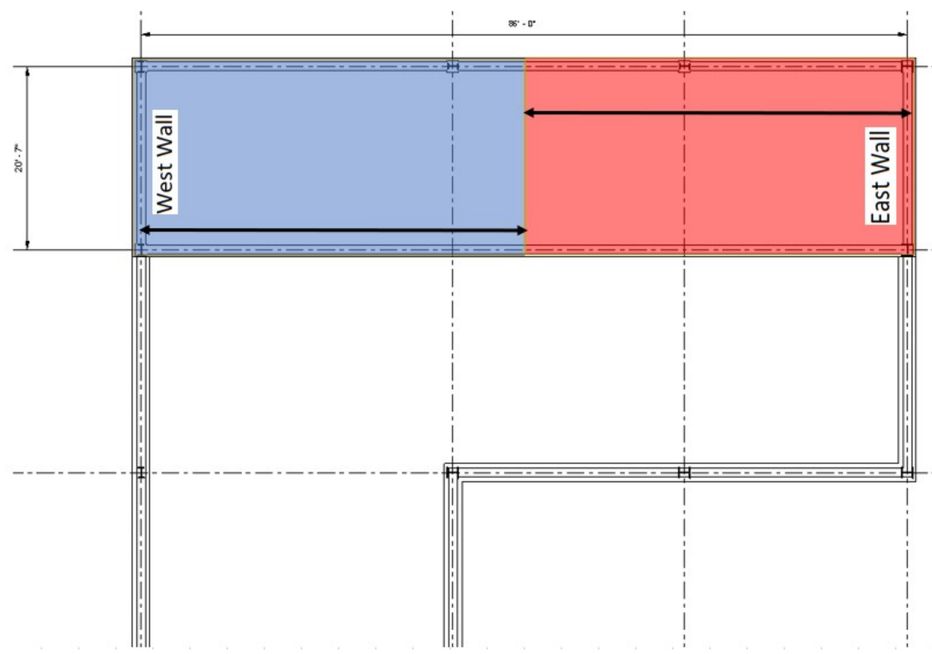
$$\delta_{rigid1} := 0.93 \cdot \delta_{flexible} = 0.067\ in \qquad \delta_{all} = 0.5\ in$$

$check :=$ if $\delta_{rigid1} < \delta_{all}$
  "The design is acceptable against settlement"
else
  "The design is not acceptable against settlement. Redesign the wall"

$check =$ "The design is acceptable against settlement"

Tributary Widths for Foundation walls:





First Floor Loading:

$$q_{FirstFloor} := 115\ psf$$

$$width_{TribNorthWall} := \frac{FIF(\text{“20' 7”})}{2} = 10.292\ ft \qquad width_{TribEastWall} := \frac{FIF(\text{“86' 0”})}{2} = 43\ ft$$

$$width_{TribSouthWall} := \frac{FIF(\text{“20' 7”})}{2} = 10.292\ ft \qquad width_{TribWestWall} := \frac{FIF(\text{“86' 0”})}{2} = 43\ ft$$

$$w_{NorthWall} := q_{FirstFloor} \cdot width_{TribNorthWall} = 1.184\ klf \qquad w_{EastWall} := q_{FirstFloor} \cdot width_{TribEastWall} = 4.945\ klf$$

$$w_{SouthWall} := q_{FirstFloor} \cdot width_{TribSouthWall} = 1.184\ klf \qquad w_{WestWall} := q_{FirstFloor} \cdot width_{TribWestWall} = 4.945\ klf$$

Wall 1: Foundation Design for North and South Walls:

Define Design Parameters:

$$B := FIF(\text{“1' 6”}) \quad H_w := FIF(\text{“12' 0”}) \quad B_{col} := FIF(\text{“0' 8”}) \quad t_f := 2\ ft$$

$D_f := 9\ in + t_f$ (5 inch slab, 4 inch of backfill between slab and foundation)

$$H := \left(H_w + \left(D_f - t_f\right)\right) = 12.75\ ft$$

$$H_{total} := H_w + D_f = 14.75\ ft$$

Check for Bearing Failure:

$$FS_q := 3$$

Vesic's Equation for Bearing Capacity:

$$q_{ult} = c' \cdot N_c \cdot s_c \cdot d_c \cdot i_c \cdot b_c \cdot g_c + \sigma'_z \cdot N_q \cdot s_q \cdot d_q \cdot i_q \cdot b_q \cdot g_q + 0.5 \cdot \gamma_{backfill} \cdot B' \cdot N_\gamma \cdot s_\gamma \cdot d_\gamma \cdot i_\gamma \cdot b_\gamma \cdot g_\gamma$$

Bearing Capacity Factors:
From table of bearing capacity factors for ϕ' = 30 degrees and using Vesic's Equation:

$$N_q := 18.4 \quad N_\gamma := 22.4$$

Ground Inclination Factors:
Assume level ground, therefore, ground inclination factors are equal to 1:

$$g_q := 1 \quad g_\gamma := 1$$

Load Inclination Factors:

$$i_q := 1 \quad i_\gamma := 1$$

Base Inclination Factors:
The base is not inclined, therefore the base inclination factors are equal to 1:

$$b_q := 1 \quad b_\gamma := 1$$

For continuous footings, $B/L \rightarrow 0$, so s_c, s_q, and s_γ become equal to 1. This means the s factors may be ignored when analyzing continuous footings.

Shape Factors:

$$s_q := 1 \quad s_\gamma := 1$$

Depth Factors:

$$d_q := 1 + 2 \cdot \frac{D_f}{B} \cdot \tan(\phi') \cdot (1 - \sin(\phi'))^2 = 1.529 \qquad d_\gamma := 1$$

Effective Unit Weight:

$$\gamma' := \gamma_{sat} - \gamma_w = 75.771\ pcf$$

Vertical Effective Stress:

$$u := \gamma_w \cdot (H_{total} - D_w) = 670.8\ psf$$

$$\sigma'_{zD} := (D_w \cdot \gamma_{backfill}) + ((H_{total} \cdot \gamma') - u) = 926.829\ psf$$

Since c' is 0, the first part of the equation is cancelled, left with:

$$q'_n := \sigma'_{zD} \cdot N_q \cdot s_q \cdot d_q \cdot i_q \cdot b_q \cdot g_q + 0.5 \cdot \gamma' \cdot B \cdot N_\gamma \cdot s_\gamma \cdot d_\gamma \cdot i_\gamma \cdot b_\gamma \cdot g_\gamma = 27352.039\ psf$$

Bearing Pressure:

$$W_f := (\gamma_{conc} \cdot t_f \cdot B) + \left(\gamma_{backfill} \cdot 4\ in \cdot \left(\frac{B}{2} - \frac{B_{col}}{2}\right)\right) + \left(\gamma_{backfill} \cdot \left(H \cdot \left(\frac{B}{2} - \frac{B_{col}}{2}\right)\right)\right) = 1.104\ klf$$

$$q_{gross} := \frac{w_{NorthWall} + W_f}{B} - u = 854.339\ psf$$

$$q_{allow} = \frac{q'_n}{FS_q} \quad \text{---->} \quad w_{allow} = \frac{w'_n}{FS_q}$$

$$w'_{nNorthWall} := q'_n \cdot B = 41.028\ klf$$

$$w_a := \frac{w'_{nNorthWall}}{FS_q} = 13.676\ klf$$

$$check := \text{if } w_{NorthWall} \le w_a$$
$$\quad \text{“The design is acceptable”}$$
$$\text{else}$$
$$\quad \text{“The design is not acceptable”}$$

$$check = \text{“The design is acceptable”}$$

Check for Settlement Failure:

$L := FIF(\text{“86' 0”})$ $\quad B = 1.5\ ft$ $\quad H := 5 \cdot B$ $\quad \mu_s := 0.3$ $\quad \alpha := 4$ (Footing Center) $\quad \delta_{all} := 0.5\ in$

$$L' := \frac{L}{2} = 43\ ft \qquad B' := \frac{B}{2} = 0.75\ ft$$

$$M := \frac{L'}{B'} = 57.333 \qquad N := \frac{H}{B'}$$

Influence Factors:

$$I_1 := \frac{1}{\pi} \cdot \left(M \cdot \ln\left(\frac{\left(1 + \sqrt{M^2 + 1}\right) \cdot \sqrt{M^2 + N^2}}{M \cdot \left(1 + \sqrt{M^2 + N^2 + 1}\right)}\right) + \ln\left(\frac{\left(M + \sqrt{M^2 + 1}\right) \cdot \sqrt{1 + N^2}}{M + \sqrt{M^2 + N^2 + 1}}\right)\right) = 0.737$$

$$I_2 := \frac{N}{2 \cdot \pi} \cdot \operatorname{atan}\left(\frac{M}{N \cdot \sqrt{M^2 + N^2 + 1}}\right) = 0.156$$

Shape Correction Factor:

$$I_s := I_1 + \left(\frac{1 - 2 \cdot \mu_s}{1 - \mu_s}\right) \cdot I_2 = 0.826$$

Fox Depth Correction Factor, If:

$$\beta_1 := 3 - 4 \cdot \mu_s \qquad \beta_2 := 5 - 12 \cdot \mu_s + 8 \cdot \mu_s^{\ 2} \qquad \beta_3 := -4 \cdot \mu_s \cdot (1 - 2 \cdot \mu_s)$$

$$\beta_4 := -1 + 4 \cdot \mu_s - 8 \cdot \mu_s^{\ 2} \qquad \beta_5 := -4 \cdot (1 - 2 \cdot \mu_s)^2$$

$$r := 2 \cdot D_f = 5.5\ ft$$

$$r_1 := \sqrt{L^2 + r^2} = 86.176\ ft$$

$$r_2 := \sqrt{B^2 + r^2} = 5.701\ ft \qquad r_3 := \sqrt{L^2 + B^2 + r^2} = 86.189\ ft \qquad r_4 := \sqrt{L^2 + B^2} = 86.013\ ft$$

$$Y_1 := L \cdot \ln\left(\frac{r_4 + B}{L}\right) + B \cdot \ln\left(\frac{r_4 + L}{B}\right) - \frac{r_4^{\ 3} - L^3 - B^3}{3 \cdot L \cdot B} = 7.872\ ft$$

$$Y_2 := L \cdot \ln\left(\frac{r_3 + B}{r_1}\right) + B \cdot \ln\left(\frac{r_3 + L}{r_2}\right) - \frac{r_3^{\ 3} - r_2^{\ 3} - r_1^{\ 3} + r^3}{3 \cdot L \cdot B} = 5.906\ ft$$

$$Y_3 := \frac{r^2}{L} \cdot \ln\left(\frac{(B + r_2) \cdot r_1}{(B + r_3) \cdot r}\right) + \frac{r^2}{B} \cdot \ln\left(\frac{(L + r_1) \cdot r_2}{(L + r_3) \cdot r}\right) = 0.811\ ft$$

$$Y_4 := \frac{r^2 \cdot (r_1 + r_2 - r_3 - r)}{L \cdot B} = 0.044\ ft$$

$$Y_5 := r \cdot \operatorname{atan}\left(\frac{L \cdot B}{r \cdot r_3}\right) = 1.461\ ft$$

$$I_f := \frac{\beta_1 \cdot Y_1 + \beta_2 \cdot Y_2 + \beta_3 \cdot Y_3 + \beta_4 \cdot Y_4 + \beta_5 \cdot Y_5}{(\beta_1 + \beta_2) \cdot Y_1} = 0.821$$

Bearing Pressure:

$$q_{gross} := \frac{w_{NorthWall} + W_f}{B} = 1525.139\ psf$$

$$\sigma'_{zo} := \left(D_w \cdot \gamma_{backfill}\right) + \left(\left(H_{total} \cdot \gamma'\right) - u\right) = 926.829\ psf$$

$$q_{net} := q_{gross} - \sigma'_{zo} = 598.31\ psf$$

Foundation Settlement:

$$\delta_{flexible} := \alpha \cdot I_s \cdot I_f \cdot \left(\frac{q_{net} \cdot \left(1 - {\mu_s}^2\right)}{E_s}\right) \cdot B'$$

$$\delta_{flexible} = 0.009\ in$$

$$\delta_{rigid2} := 0.93 \cdot \delta_{flexible} = 0.008\ in \qquad \delta_{all} = 0.5\ in$$

$check :=$ if $\delta_{rigid2} < \delta_{all}$
‖ "The design is acceptable against settlement"
else
‖ "The design is not acceptable against settlement. Redesign the wall"

$check =$ "The design is acceptable against settlement"

Wall 1: Foundation Design for East and West Walls:

Define Design Parameters:

$B := FIF(\text{"1' 6"}) \quad H_w := FIF(\text{"12' 0"}) \quad B_{col} := FIF(\text{"0' 8"}) \quad t_f := 1\ ft$

$D_f := 9\ in + t_f$ (5 inch slab, 4 inch of backfill between slab and foundation)

$$H := \left(H_w + \left(D_f - t_f\right)\right) = 12.75\ ft$$

$$H_{total} := H_w + D_f = 13.75\ ft$$

Check for Bearing Failure:

$$FS_q := 3$$

Vesic's Equation for Bearing Capacity:

$$q_{ult} = c' \cdot N_c \cdot s_c \cdot d_c \cdot i_c \cdot b_c \cdot g_c + \sigma'_z \cdot N_q \cdot s_q \cdot d_q \cdot i_q \cdot b_q \cdot g_q + 0.5 \cdot \gamma_{backfill} \cdot B' \cdot N_\gamma \cdot s_\gamma \cdot d_\gamma \cdot i_\gamma \cdot b_\gamma \cdot g_\gamma$$

Bearing Capacity Factors:
From table of bearing capacity factors for ϕ' = 30 degrees and using Vesic's Equation:

$$N_q := 18.4 \quad N_\gamma := 22.4$$

Ground Inclination Factors:
Assume level ground, therefore, ground inclination factors are equal to 1:

$$g_q := 1 \quad g_\gamma := 1$$

Load Inclination Factors:

$$i_q := 1 \quad i_\gamma := 1$$

Base Inclination Factors:
The base is not inclined, therefore the base inclination factors are equal to 1:

$$b_q := 1 \quad b_\gamma := 1$$

For continuous footings, $B/L \rightarrow 0$, so s_c, s_q, and s_γ become equal to 1. This means the s factors may be ignored when analyzing continuous footings.

Shape Factors:

$$s_q := 1 \quad s_\gamma := 1$$

Depth Factors:

$$d_q := 1 + 2 \cdot \frac{D_f}{B} \cdot \tan\left(\phi'\right) \cdot \left(1 - \sin\left(\phi'\right)\right)^2 = 1.337 \qquad d_\gamma := 1$$

Effective Unit Weight:

$$\gamma' := \gamma_{sat} - \gamma_w = 75.771\ pcf$$

Vertical Effective Stress:

$$u := \gamma_w \cdot \left(H_{total} - D_w\right) = 608.4\ psf$$

$$\sigma'_{zD} := \left(D_w \cdot \gamma_{backfill}\right) + \left(\left(H_{total} \cdot \gamma'\right) - u\right) = 913.457\ psf$$

Since c' is 0, the first part of the equation is cancelled, left with:

$$q'_n := \sigma'_{zD} \cdot N_q \cdot s_q \cdot d_q \cdot i_q \cdot b_q \cdot g_q + 0.5 \cdot \gamma' \cdot B \cdot N_\gamma \cdot s_\gamma \cdot d_\gamma \cdot i_\gamma \cdot b_\gamma \cdot g_\gamma = 23741.168 \ psf$$

Bearing Pressure:

$$W_f := \left(\gamma_{conc} \cdot t_f \cdot B\right) + \left(\gamma_{backfill} \cdot 4 \ in \cdot \left(\frac{B}{2} - \frac{B_{col}}{2}\right)\right) + \left(\gamma_{backfill} \cdot \left(H \cdot \left(\frac{B}{2} - \frac{B_{col}}{2}\right)\right)\right) = 0.879 \ klf$$

$$q_{gross} := \frac{w_{EastWall} + W_f}{B} - u = 3274.378 \ psf$$

$$q_{allow} = \frac{q'_n}{FS_q} \quad \text{---->} \quad w_{allow} = \frac{w'_n}{FS_q}$$

$$w'_{nEastWall} := q'_n \cdot B = 35.612 \ klf$$

$$w_a := \frac{w'_{nEastWall}}{FS_q} = 11.871 \ klf$$

$$check := \text{if } w_{EastWall} \le w_a$$
"The design is acceptable"
else
"The design is not acceptable"

$$check = \text{“The design is acceptable”}$$

Check for Settlement Failure:

$L := FIF(\text{“20' 7”})$ $\quad B = 1.5 \ ft$ $\quad H := 5 \cdot B$ $\quad \mu_s := 0.3$ $\quad \alpha := 4$ (Footing Center) $\quad \delta_{all} := 0.5 \ in$

$$L' := \frac{L}{2} = 10.292 \ ft \qquad B' := \frac{B}{2} = 0.75 \ ft$$

$$M := \frac{L'}{B'} = 13.722 \qquad N := \frac{H}{B'}$$

Influence Factors:

$$I_1 := \frac{1}{\pi} \cdot \left(M \cdot \ln\left(\frac{\left(1 + \sqrt{M^2 + 1}\right) \cdot \sqrt{M^2 + N^2}}{M \cdot \left(1 + \sqrt{M^2 + N^2 + 1}\right)}\right) + \ln\left(\frac{\left(M + \sqrt{M^2 + 1}\right) \cdot \sqrt{1 + N^2}}{M + \sqrt{M^2 + N^2 + 1}}\right)\right) = 0.76$$

$$I_2 := \frac{N}{2 \cdot \pi} \cdot \operatorname{atan}\left(\frac{M}{N \cdot \sqrt{M^2 + N^2 + 1}}\right) = 0.128$$

Shape Correction Factor:

$$I_s := I_1 + \left(\frac{1 - 2 \cdot \mu_s}{1 - \mu_s}\right) \cdot I_2 = 0.833$$

Fox Depth Correction Factor, If:

$$\beta_1 := 3 - 4 \cdot \mu_s \qquad \beta_2 := 5 - 12 \cdot \mu_s + 8 \cdot \mu_s^{\ 2} \qquad \beta_3 := -4 \cdot \mu_s \cdot \left(1 - 2 \cdot \mu_s\right)$$

$$\beta_4 := -1 + 4 \cdot \mu_s - 8 \cdot \mu_s^{\ 2} \qquad \beta_5 := -4 \cdot \left(1 - 2 \cdot \mu_s\right)^2$$

$$r := 2 \cdot D_f = 3.5 \ ft$$

$$r_1 := \sqrt{L^2 + r^2} = 20.879 \ ft$$

$$r_2 := \sqrt{B^2 + r^2} = 3.808 \ ft \qquad r_3 := \sqrt{L^2 + B^2 + r^2} = 20.933 \ ft \qquad r_4 := \sqrt{L^2 + B^2} = 20.638 \ ft$$

$$Y_1 := L \cdot \ln\left(\frac{r_4 + B}{L}\right) + B \cdot \ln\left(\frac{r_4 + L}{B}\right) - \frac{r_4^{\ 3} - L^3 - B^3}{3 \cdot L \cdot B} = 5.754 \ ft$$

$$Y_2 := L \cdot \ln\left(\frac{r_3 + B}{r_1}\right) + B \cdot \ln\left(\frac{r_3 + L}{r_2}\right) - \frac{r_3^{\ 3} - r_2^{\ 3} - r_1^{\ 3} + r^3}{3 \cdot L \cdot B} = 4.432 \ ft$$

$$Y_3 := \frac{r^2}{L} \cdot \ln\left(\frac{\left(B + r_2\right) \cdot r_1}{\left(B + r_3\right) \cdot r}\right) + \frac{r^2}{B} \cdot \ln\left(\frac{\left(L + r_1\right) \cdot r_2}{\left(L + r_3\right) \cdot r}\right) = 0.883 \ ft$$

$$Y_4 := \frac{r^2 \cdot \left(r_1 + r_2 - r_3 - r\right)}{L \cdot B} = 0.101 \ ft$$

$$Y_5 := r \cdot \operatorname{atan}\left(\frac{L \cdot B}{r \cdot r_3}\right) = 1.396 \ ft$$

$$I_f := \frac{\beta_1 \cdot Y_1 + \beta_2 \cdot Y_2 + \beta_3 \cdot Y_3 + \beta_4 \cdot Y_4 + \beta_5 \cdot Y_5}{\left(\beta_1 + \beta_2\right) \cdot Y_1} = 0.815$$

Bearing Pressure:

$$q_{gross} := \frac{w_{NorthWall} + W_f}{B} = 1375.139 \ psf$$

$$\sigma'_{zo} := \left(D_w \cdot \gamma_{backfill}\right) + \left(\left(H_{total} \cdot \gamma'\right) - u\right) = 913.457 \ psf$$

$$q_{net} := q_{gross} - \sigma'_{zo} = 461.682\ psf$$

Foundation Settlement:

$$\delta_{flexible} := \alpha \cdot I_s \cdot I_f \cdot \left(\frac{q_{net} \cdot \left(1 - {\mu_s}^2\right)}{E_s} \right) \cdot B'$$

$$\delta_{flexible} = 0.007\ in$$

$$\delta_{rigid3} := 0.93 \cdot \delta_{flexible} = 0.006\ in \qquad \delta_{all} = 0.5\ in$$

$check :=$ if $\delta_{rigid3} < \delta_{all}$
  "The design is acceptable against settlement"
else
  "The design is not acceptable against settlement. Redesign the wall"

$check =$ "The design is acceptable against settlement"

Check for Differential Settlement:

Pilaster Square Foundations vs North and South Bearing Walls:

$$\delta_D := \left| \delta_{rigid1} - \delta_{rigid2} \right| = 0.0586\ in$$

$check :=$ if $\delta_D \le 0.25\ in$
  "Differential settlement is not a concern"
else
  "Differential settlement is too large"

$check =$ "Differential settlement is not a concern"

Pilaster Square Foundations vs East and West Bearing Walls:

$$\delta_D := \left| \delta_{rigid1} - \delta_{rigid3} \right| = 0.060458\ in$$

$check :=$ if $\delta_D \le 0.25\ in$
  "Differential settlement is not a concern"
else
  "Differential settlement is too large"

$check =$ "Differential settlement is not a concern"

North and South Bearing Walls vs East and West Bearing Walls:

$$\delta_D := \left| \delta_{rigid2} - \delta_{rigid3} \right| = 0.0019\ in$$

$check :=$ if $\delta_D \le 0.25\ in$
  "Differential settlement is not a concern"
else
  "Differential settlement is too large"

$check =$ "Differential settlement is not a concern"

Wall 2: Pilaster Design:

For wall 2, Square footings for the Pilasters will need to be designed for columns P6-P13. For ease of construction, two footing sizes shall be selected, one for P8, which is an outlier due to it's large tributary area/loading and one for the rest of the pilasters.

$$P_{des1} := \max\left(P_6, P_7, P_9, P_{10}, P_{11}, P_{12}, P_{13}\right) = 27.824\ \mathit{kip}$$

$$P_{des2} := P_8 = 45.073\ \mathit{kip}$$

First Square Footing Design:

Define Design Parameters:

$B := \mathit{FIF}(\text{“3′ 8”})$ $\quad H := 3\ \mathit{ft}$ $\quad B_{col} := \mathit{FIF}(\text{“1′ 6”})$ $\quad t_f := 2\ \mathit{ft}$

$D_f := 3\ \mathit{ft} + t_f$ (3 foot frost wall between slab and foundation)

$$H_{total} := H + t_f = 5\ \mathit{ft}$$

Check for Bearing Failure:

$$FS_q := 3$$

Vesic's Equation for Bearing Capacity:

$$q_{ult} = c' \cdot N_c \cdot s_c \cdot d_c \cdot i_c \cdot b_c \cdot g_c + \sigma'_z \cdot N_q \cdot s_q \cdot d_q \cdot i_q \cdot b_q \cdot g_q + 0.5 \cdot \gamma_{backfill} \cdot B' \cdot N_\gamma \cdot s_\gamma \cdot d_\gamma \cdot i_\gamma \cdot b_\gamma \cdot g_\gamma$$

Bearing Capacity Factors:
From table of bearing capacity factors for ϕ' = 30 degrees and using Vesic's Equation:

$$N_q := 18.4 \quad N_\gamma := 22.4$$

Ground Inclination Factors:
Assume level ground, therefore, ground inclination factors are equal to 1:

$$g_q := 1 \quad g_\gamma := 1$$

Load Inclination Factors:

$$i_q := 1 \quad i_\gamma := 1$$

Base Inclination Factors:
The base is not inclined, therefore the base inclination factors are equal to 1:

$$b_q := 1 \quad b_\gamma := 1$$

For continuous footings, $B/L \rightarrow 0$, so s_c, s_q, and s_γ become equal to 1. This means the s factors may be ignored when analyzing continuous footings.

Shape Factors:

$$s_q := 1 \quad s_\gamma := 1$$

Depth Factors:

$$d_q := 1 + 2 \cdot \frac{D_f}{B} \cdot \tan(\phi') \cdot \left(1 - \sin(\phi')\right)^2 = 1.394 \qquad d_\gamma := 1$$

Effective Unit Weight:

$$\gamma' := \begin{cases} \gamma_{sat} - \gamma_w & \text{if } D_w \le D_f \\ \gamma_{sat} - \gamma_w \cdot \left(1 - \left(\dfrac{D_w - D_f}{B}\right)\right) & \text{else if } D_f < D_w < D_f + B \\ \gamma_{sat} & \text{else if } \left(D_f + B\right) \le D_w \end{cases}$$

$$\gamma' = 75.771\ \mathit{pcf}$$

Vertical Effective Stress:

$$u := \gamma_w \cdot \left(H_{total} - D_w\right) = 62.4\ \mathit{psf}$$

$$\sigma'_{zD} := \left(\left(H_{total} \cdot \gamma'\right) - u\right) = 316.457\ \mathit{psf}$$

Since c' is 0, the first part of the equation is cancelled, left with:

$$q_{ult} := \sigma'_{zD} \cdot N_q \cdot s_q \cdot d_q \cdot i_q \cdot b_q \cdot g_q + 0.5 \cdot \gamma' \cdot B \cdot N_\gamma \cdot s_\gamma \cdot d_\gamma \cdot i_\gamma \cdot b_\gamma \cdot g_\gamma = 11226.629\ \mathit{psf}$$

Bearing Pressure:

$$q_{gross} := \frac{P_{des1}}{(B \cdot B)} + \left(\gamma_{conc} \cdot t_f\right) + 2 \cdot \left(\gamma_{backfill} \cdot H\right) = 3089.554\ \mathit{psf}$$

$$check := \text{if } \frac{q_{ult}}{q_{gross}} > FS_q \quad \| \text{ "The design is acceptable" } \quad \text{else} \quad \| \text{ "The design is not acceptable"}$$

$$\frac{q_{ult}}{q_{gross}} = 3.634$$

$$check = \text{"The design is acceptable"}$$

Check for Settlement Failure:

$L := B$ $\quad B = 3.667\ ft$ $\quad H := 5 \cdot B$ $\quad \mu_s := 0.3$ $\quad \alpha := 4$ (Footing Center) $\quad \delta_{all} := 0.5\ in$

$$L' := \frac{L}{2} = 1.833\ ft \qquad B' := \frac{B}{2} = 1.833\ ft$$

$$M := \frac{L'}{B'} = 1 \qquad N := \frac{H}{B'}$$

Influence Factors:

$$I_1 := \frac{1}{\pi} \cdot \left(M \cdot \ln\left(\frac{\left(1 + \sqrt{M^2 + 1}\right) \cdot \sqrt{M^2 + N^2}}{M \cdot \left(1 + \sqrt{M^2 + N^2 + 1}\right)} \right) + \ln\left(\frac{\left(M + \sqrt{M^2 + 1}\right) \cdot \sqrt{1 + N^2}}{M + \sqrt{M^2 + N^2 + 1}} \right) \right) = 0.498$$

$$I_2 := \frac{N}{2 \cdot \pi} \cdot \operatorname{atan}\left(\frac{M}{N \cdot \sqrt{M^2 + N^2 + 1}} \right) = 0.016$$

Shape Correction Factor:

$$I_s := I_1 + \left(\frac{1 - 2 \cdot \mu_s}{1 - \mu_s} \right) \cdot I_2 = 0.507$$

Fox Depth Correction Factor, If:

$$\beta_1 := 3 - 4 \cdot \mu_s \qquad \beta_2 := 5 - 12 \cdot \mu_s + 8 \cdot \mu_s^{\ 2} \qquad \beta_3 := -4 \cdot \mu_s \cdot \left(1 - 2 \cdot \mu_s\right)$$

$$\beta_4 := -1 + 4 \cdot \mu_s - 8 \cdot \mu_s^{\ 2} \qquad \beta_5 := -4 \cdot \left(1 - 2 \cdot \mu_s\right)^2$$

$$r := 2 \cdot D_f = 10\ ft$$

$$r_1 := \sqrt{L^2 + r^2} = 10.651\ ft$$

$$r_2 := \sqrt{B^2 + r^2} = 10.651\ ft \qquad r_3 := \sqrt{L^2 + B^2 + r^2} = 11.264\ ft \qquad r_4 := \sqrt{L^2 + B^2} = 5.185\ ft$$

$$Y_1 := L \cdot \ln\left(\frac{r_4 + B}{L} \right) + B \cdot \ln\left(\frac{r_4 + L}{B} \right) - \frac{r_4^{\ 3} - L^3 - B^3}{3 \cdot L \cdot B} = 5.451\ ft$$

$$Y_2 := L \cdot \ln\left(\frac{r_3 + B}{r_1} \right) + B \cdot \ln\left(\frac{r_3 + L}{r_2} \right) - \frac{r_3^{\ 3} - r_2^{\ 3} - r_1^{\ 3} + r^3}{3 \cdot L \cdot B} = 2.161\ ft$$

$$Y_3 := \frac{r^2}{L} \cdot \ln\left(\frac{\left(B + r_2\right) \cdot r_1}{\left(B + r_3\right) \cdot r} \right) + \frac{r^2}{B} \cdot \ln\left(\frac{\left(L + r_1\right) \cdot r_2}{\left(L + r_3\right) \cdot r} \right) = 1.152\ ft$$

$$Y_4 := \frac{r^2 \cdot \left(r_1 + r_2 - r_3 - r\right)}{L \cdot B} = 0.279\ ft$$

$$Y_5 := r \cdot \operatorname{atan}\left(\frac{L \cdot B}{r \cdot r_3} \right) = 1.188\ ft$$

$$I_f := \frac{\beta_1 \cdot Y_1 + \beta_2 \cdot Y_2 + \beta_3 \cdot Y_3 + \beta_4 \cdot Y_4 + \beta_5 \cdot Y_5}{\left(\beta_1 + \beta_2\right) \cdot Y_1} = 0.605$$

Bearing Pressure:

$$q_{gross} := \frac{P_{des1}}{(B \cdot B)} + \left(\gamma_{conc} \cdot t_f\right) + 2 \cdot \left(\gamma_{backfill} \cdot H\right) = 6769.554\ psf$$

$$\sigma'_{zo} := \left(\left(H_{total} \cdot \gamma'\right) - u\right) = 316.457\ psf$$

$$q_{net} := q_{gross} - \sigma'_{zo} = 6453.097\ psf$$

Foundation Settlement:

$$\delta_{flexible} := \alpha \cdot I_s \cdot I_f \cdot \left(\frac{q_{net} \cdot \left(1 - \mu_s^{\ 2}\right)}{E_s} \right) \cdot B'$$

$$\delta_{flexible} = 0.106\ in$$

$$\delta_{rigid4} := 0.93 \cdot \delta_{flexible} = 0.098\ in \qquad \delta_{all} = 0.5\ in$$

$$check := \text{if } \delta_{rigid4} < \delta_{all} \quad \| \text{ "The design is acceptable against settlement" } \quad \text{else} \quad \| \text{ "The design is not acceptable against settlement. Redesign the wall"}$$

$$check = \text{"The design is acceptable against settlement"}$$

Second Square Footing Design:

Define Design Parameters:

$$B:=FIF(\text{“4' 4”})\quad H:=FIF(\text{“2' 8”})\qquad B_{col}:=FIF(\text{“1' 6”})\quad t_f:=2\ ft$$

$$D_f:=3\ ft+t_f$$ (3 foot frost wall between slab and foundation)

$$H_{total}:=H+t_f=4.667\ ft$$

Check for Bearing Failure:

$$FS_q:=3$$

Vesic's Equation for Bearing Capacity:

$$q_{ult}=c'\cdot N_c\cdot s_c\cdot d_c\cdot i_c\cdot b_c\cdot g_c+\sigma'_z\cdot N_q\cdot s_q\cdot d_q\cdot i_q\cdot b_q\cdot g_q+0.5\cdot\gamma_{backfill}\cdot B'\cdot N_\gamma\cdot s_\gamma\cdot d_\gamma\cdot i_\gamma\cdot b_\gamma\cdot g_\gamma$$

$$q_{ult}:=\sigma'_{zD}\cdot N_q\cdot s_q\cdot d_q\cdot i_q\cdot b_q\cdot g_q+0.5\cdot\gamma'\cdot B\cdot N_\gamma\cdot s_\gamma\cdot d_\gamma\cdot i_\gamma\cdot b_\gamma\cdot g_\gamma=11792.389\ psf$$

Bearing Pressure:

$$q_{gross}:=\frac{P_{des2}}{(B\cdot B)}+(\gamma_{conc}\cdot t_f)+2\cdot(\gamma_{backfill}\cdot H)=3340.337\ psf$$

$$check:=\text{if }\frac{q_{ult}}{q_{gross}}>FS_q$$
"The design is acceptable"
else
"The design is not acceptable"

$$\frac{q_{ult}}{q_{gross}}=3.53$$

$check=$ "The design is acceptable"

Check for Settlement Failure:

$$L:=B\qquad B=4.333\ ft\qquad H:=5\cdot B\qquad \mu_s:=0.3\qquad \alpha:=4$$ (Footing Center) $$\delta_{all}:=0.5\ in$$

$$L':=\frac{L}{2}=2.167\ ft\qquad B':=\frac{B}{2}=2.167\ ft$$

$$M:=\frac{L'}{B'}=1\qquad N:=\frac{H}{B'}$$

Influence Factors:

$$I_1:=\frac{1}{\pi}\cdot\left(M\cdot\ln\left(\frac{\left(1+\sqrt{M^2+1}\right)\cdot\sqrt{M^2+N^2}}{M\cdot\left(1+\sqrt{M^2+N^2+1}\right)}\right)+\ln\left(\frac{\left(M+\sqrt{M^2+1}\right)\cdot\sqrt{1+N^2}}{M+\sqrt{M^2+N^2+1}}\right)\right)=0.498$$

$$I_2:=\frac{N}{2\cdot\pi}\cdot\operatorname{atan}\left(\frac{M}{N\cdot\sqrt{M^2+N^2+1}}\right)=0.016$$

Shape Correction Factor:

$$I_s:=I_1+\left(\frac{1-2\cdot\mu_s}{1-\mu_s}\right)\cdot I_2=0.507$$

Fox Depth Correction Factor, If:

$$\beta_1:=3-4\cdot\mu_s\qquad \beta_2:=5-12\cdot\mu_s+8\cdot{\mu_s}^2\qquad \beta_3:=-4\cdot\mu_s\cdot(1-2\cdot\mu_s)$$

$$\beta_4:=-1+4\cdot\mu_s-8\cdot{\mu_s}^2\qquad \beta_5:=-4\cdot(1-2\cdot\mu_s)^2$$

$$r:=2\cdot D_f=10\ ft$$

$$r_1:=\sqrt{L^2+r^2}=10.899\ ft$$

$$r_2:=\sqrt{B^2+r^2}=10.899\ ft\qquad r_3:=\sqrt{L^2+B^2+r^2}=11.728\ ft\qquad r_4:=\sqrt{L^2+B^2}=6.128\ ft$$

$$Y_1:=L\cdot\ln\left(\frac{r_4+B}{L}\right)+B\cdot\ln\left(\frac{r_4+L}{B}\right)-\frac{{r_4}^3-L^3-B^3}{3\cdot L\cdot B}=6.442\ ft$$

$$Y_2:=L\cdot\ln\left(\frac{r_3+B}{r_1}\right)+B\cdot\ln\left(\frac{r_3+L}{r_2}\right)-\frac{{r_3}^3-{r_2}^3-{r_1}^3+r^3}{3\cdot L\cdot B}=2.93\ ft$$

$$Y_3:=\frac{r^2}{L}\cdot\ln\left(\frac{(B+r_2)\cdot r_1}{(B+r_3)\cdot r}\right)+\frac{r^2}{B}\cdot\ln\left(\frac{(L+r_1)\cdot r_2}{(L+r_3)\cdot r}\right)=1.523\ ft$$

$$Y_4:=\frac{r^2\cdot(r_1+r_2-r_3-r)}{L\cdot B}=0.366\ ft$$

$$Y_5:=r\cdot\operatorname{atan}\left(\frac{L\cdot B}{r\cdot r_3}\right)=1.588\ ft$$

$$I_f:=\frac{\beta_1\cdot Y_1+\beta_2\cdot Y_2+\beta_3\cdot Y_3+\beta_4\cdot Y_4+\beta_5\cdot Y_5}{(\beta_1+\beta_2)\cdot Y_1}=0.628$$

Bearing Pressure:

$$q_{gross} := \frac{P_{des2}}{(B \cdot B)} + (\gamma_{conc} \cdot t_f) + 2 \cdot (\gamma_{backfill} \cdot H) = 7900.337\ psf$$

$$\sigma'_{zo} := ((H_{total} \cdot \gamma') - u) = 291.2\ psf$$

$$q_{net} := q_{gross} - \sigma'_{zo} = 7609.137\ psf$$

Foundation Settlement:

$$\delta_{flexible} := \alpha \cdot I_s \cdot I_f \cdot \left(\frac{q_{net} \cdot (1 - {\mu_s}^2)}{E_s}\right) \cdot B'$$

$$\delta_{flexible} = 0.153\ in$$

$$\delta_{rigid5} := 0.93 \cdot \delta_{flexible} = 0.142\ in \qquad \delta_{all} = 0.5\ in$$

$check :=$ if $\delta_{rigid5} < \delta_{all}$
  "The design is acceptable against settlement"
else
  "The design is not acceptable against settlement. Redesign the wall"

$check =$ "The design is acceptable against settlement"

Wall 2: Foundation Design for Frost Wall:

Define Design Parameters:

$B := FIF(\text{"1' 0"}) \quad H := FIF(\text{"2' 7"}) \qquad B_{col} := FIF(\text{"0' 8"}) \quad t_f := 1\ ft$

$D_f := H + t_f = 3.583\ ft$ (3 foot frost wall between slab and foundation)

$H_{total} := H + t_f = 3.583\ ft$

Check for Bearing Failure:

$$FS_q := 3$$

Vesic's Equation for Bearing Capacity:

$$q_{ult} = c' \cdot N_c \cdot s_c \cdot d_c \cdot i_c \cdot b_c \cdot g_c + \sigma'_z \cdot N_q \cdot s_q \cdot d_q \cdot i_q \cdot b_q \cdot g_q + 0.5 \cdot \gamma_{backfill} \cdot B' \cdot N_\gamma \cdot s_\gamma \cdot d_\gamma \cdot i_\gamma \cdot b_\gamma \cdot g_\gamma$$

Bearing Capacity Factors:
From table of bearing capacity factors for ϕ' = 30 degrees and using Vesic's Equation:

$$N_q := 18.4 \quad N_\gamma := 22.4$$

Ground Inclination Factors:
Assume level ground, therefore, ground inclination factors are equal to 1:

$$g_q := 1 \quad g_\gamma := 1$$

Load Inclination Factors:

$$i_q := 1 \quad i_\gamma := 1$$

Base Inclination Factors:
The base is not inclined, therefore the base inclination factors are equal to 1:

$$b_q := 1 \quad b_\gamma := 1$$

For continuous footings, $B/L \rightarrow 0$, so s_c, s_q, and s_γ become equal to 1. This means the s factors may be ignored when analyzing continuous footings.

Shape Factors:

$$s_q := 1 \quad s_\gamma := 1$$

Depth Factors:

$$d_q := 1 + 2 \cdot \frac{D_f}{B} \cdot \tan(\phi') \cdot (1 - \sin(\phi'))^2 = 2.034 \qquad d_\gamma := 1$$

Effective Unit Weight:

$$\gamma' := \gamma_{sat} - \gamma_w = 75.771\ pcf$$

Vertical Effective Stress:

$$u := \gamma_w \cdot (H_{total} - D_w) = -26\ psf$$

$$\sigma'_{zD} := ((H_{total} \cdot \gamma') - u) = 297.514\ psf$$

Since c' is 0, the first part of the equation is cancelled, left with:

$$q'_n := \sigma'_{zD} \cdot N_q \cdot s_q \cdot d_q \cdot i_q \cdot b_q \cdot g_q + 0.5 \cdot \gamma' \cdot B \cdot N_\gamma \cdot s_\gamma \cdot d_\gamma \cdot i_\gamma \cdot b_\gamma \cdot g_\gamma = 11985.586\ psf$$

Bearing Pressure:

$$W_f := \left(\gamma_{conc} \cdot t_f \cdot B\right) + 2 \cdot \left(\gamma_{backfill} \cdot \left(H \cdot \left(B - B_{col}\right)\right)\right) = 0.357\ klf$$

$$q_{gross} := \frac{W_f}{B} - u = 382.667\ psf$$

$$q_{allow} = \frac{q'_n}{FS_q} \quad \text{---->} \quad w_{allow} = \frac{w'_n}{FS_q}$$

$$w'_n := q'_n \cdot B = 11.986\ klf$$

$$w_a := \frac{w'_n}{FS_q} = 3.995\ klf$$

$$check := \text{if } W_f \le w_a$$
$$\quad \text{"The design is acceptable"}$$
$$\text{else}$$
$$\quad \text{"The design is not acceptable"}$$

$$check = \text{"The design is acceptable"}$$

Check for Settlement Failure:

$L := FIF(\text{"51' 0"}")$ $\quad B = 1\ ft \quad H := 5 \cdot B \quad \mu_s := 0.3 \quad \alpha := 4$ (Footing Center) $\quad \delta_{all} := 0.5\ in$

$$L' := \frac{L}{2} = 25.5\ ft \qquad B' := \frac{B}{2} = 0.5\ ft$$

$$M := \frac{L'}{B'} = 51 \qquad N := \frac{H}{B'}$$

Influence Factors:

$$I_1 := \frac{1}{\pi} \cdot \left(M \cdot \ln\left(\frac{\left(1 + \sqrt{M^2 + 1}\right) \cdot \sqrt{M^2 + N^2}}{M \cdot \left(1 + \sqrt{M^2 + N^2 + 1}\right)}\right) + \ln\left(\frac{\left(M + \sqrt{M^2 + 1}\right) \cdot \sqrt{1 + N^2}}{M + \sqrt{M^2 + N^2 + 1}}\right)\right) = 0.737$$

$$I_2 := \frac{N}{2 \cdot \pi} \cdot \operatorname{atan}\left(\frac{M}{N \cdot \sqrt{M^2 + N^2 + 1}}\right) = 0.156$$

Shape Correction Factor:

$$I_s := I_1 + \left(\frac{1 - 2 \cdot \mu_s}{1 - \mu_s}\right) \cdot I_2 = 0.826$$

Fox Depth Correction Factor, If:

$$\beta_1 := 3 - 4 \cdot \mu_s \qquad \beta_2 := 5 - 12 \cdot \mu_s + 8 \cdot \mu_s^{\ 2} \qquad \beta_3 := -4 \cdot \mu_s \cdot \left(1 - 2 \cdot \mu_s\right)$$

$$\beta_4 := -1 + 4 \cdot \mu_s - 8 \cdot \mu_s^{\ 2} \qquad \beta_5 := -4 \cdot \left(1 - 2 \cdot \mu_s\right)^2$$

$$r := 2 \cdot D_f = 7.167\ ft$$

$$r_1 := \sqrt{L^2 + r^2} = 51.501\ ft$$

$$r_2 := \sqrt{B^2 + r^2} = 7.236\ ft \qquad r_3 := \sqrt{L^2 + B^2 + r^2} = 51.511\ ft \qquad r_4 := \sqrt{L^2 + B^2} = 51.01\ ft$$

$$Y_1 := L \cdot \ln\left(\frac{r_4 + B}{L}\right) + B \cdot \ln\left(\frac{r_4 + L}{B}\right) - \frac{r_4^{\ 3} - L^3 - B^3}{3 \cdot L \cdot B} = 5.131\ ft$$

$$Y_2 := L \cdot \ln\left(\frac{r_3 + B}{r_1}\right) + B \cdot \ln\left(\frac{r_3 + L}{r_2}\right) - \frac{r_3^{\ 3} - r_2^{\ 3} - r_1^{\ 3} + r^3}{3 \cdot L \cdot B} = 3.207\ ft$$

$$Y_3 := \frac{r^2}{L} \cdot \ln\left(\frac{\left(B + r_2\right) \cdot r_1}{\left(B + r_3\right) \cdot r}\right) + \frac{r^2}{B} \cdot \ln\left(\frac{\left(L + r_1\right) \cdot r_2}{\left(L + r_3\right) \cdot r}\right) = 0.611\ ft$$

$$Y_4 := \frac{r^2 \cdot \left(r_1 + r_2 - r_3 - r\right)}{L \cdot B} = 0.06\ ft$$

$$Y_5 := r \cdot \operatorname{atan}\left(\frac{L \cdot B}{r \cdot r_3}\right) = 0.984\ ft$$

$$I_f := \frac{\beta_1 \cdot Y_1 + \beta_2 \cdot Y_2 + \beta_3 \cdot Y_3 + \beta_4 \cdot Y_4 + \beta_5 \cdot Y_5}{\left(\beta_1 + \beta_2\right) \cdot Y_1} = 0.75$$

Bearing Pressure:

$$q_{gross} := \frac{W_f}{B} = 356.667\ psf$$

$$\sigma'_{zo} := \left(\left(H_{total} \cdot \gamma'\right) - u\right) = 297.514\ psf$$

$$q_{net} := q_{gross} - \sigma'_{zo} = 59.152\ psf$$

Foundation Settlement:

$$\delta_{flexible} := \alpha \cdot I_s \cdot I_f \cdot \left(\frac{q_{net} \cdot \left(1 - {\mu_s}^2\right)}{E_s} \right) \cdot B'$$

$\delta_{flexible} = 0.001\ in$

$\delta_{rigid6} := 0.93 \cdot \delta_{flexible} = 0\ in$ $\delta_{all} = 0.5\ in$

$check :=$ if $\delta_{rigid6} < \delta_{all}$
  "The design is acceptable against settlement"
else
  "The design is not acceptable against settlement. Redesign the wall"

$check =$ "The design is acceptable against settlement"

<u>Check for Differential Settlement:</u>

Pilaster Square Foundation 4 vs Pilaster Square Foundation 5

$\delta_D := \left| \delta_{rigid4} - \delta_{rigid5} \right| = 0.0439\ in$

$check :=$ if $\delta_D \le 0.25\ in$
  "Differential settlement is not a concern"
else
  "Differential settlement is too large"

$check =$ "Differential settlement is not a concern"

Pilaster Square Foundation 4 vs Wall Foundation 2

$\delta_D := \left| \delta_{rigid4} - \delta_{rigid6} \right| = 0.0978\ in$

$check :=$ if $\delta_D \le 0.25\ in$
  "Differential settlement is not a concern"
else
  "Differential settlement is too large"

$check =$ "Differential settlement is not a concern"

Pilaster Square Foundation 5 vs Wall Foundation 2

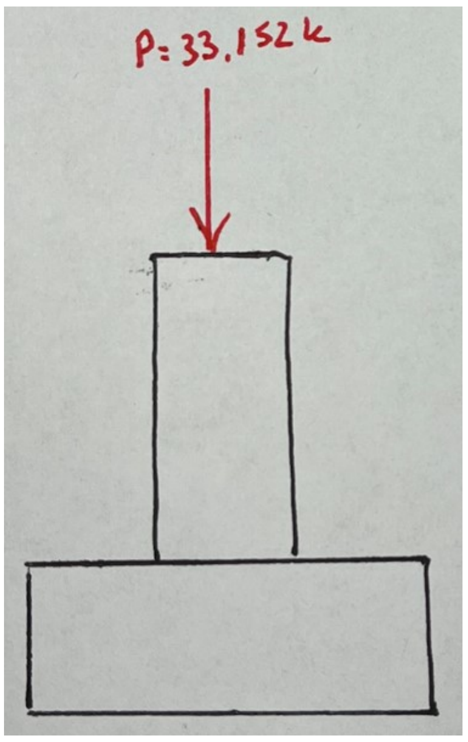
$\delta_D := \left| \delta_{rigid5} - \delta_{rigid6} \right| = 0.1417\ in$

$check :=$ if $\delta_D \le 0.25\ in$
  "Differential settlement is not a concern"
else
  "Differential settlement is too large"

$check =$ "Differential settlement is not a concern"

Appendix U: Building C: Foundation Structural Design

First Square Footing Design:



Check One-Way Shear Strength:

$B := FIF(\text{"3' 8"})$ $\quad H := FIF(\text{"2' 8"})$ $\quad B_{col} := FIF(\text{"1' 6"})$ $\quad t_f := 2\ ft$ $\quad h := t_f$

$P_{des1} := 33.152\ kip$ $\quad \gamma_{conc} := 150\ pcf$ $\quad \gamma_{backfill} := 120\ pcf$ $\quad f'_c := 4000\ psi$ $\quad f_y := 60\ ksi$

Effective depth of footing:

Assume a 3in clear cover, #6 rebars and bars in both directions:

$c_c := 3\ in \qquad D_{\#6} := 0.750\ in$

$$cover_1 := c_c + \frac{D_{\#6}}{2} = 3.375\ in \qquad cover_2 := c_c + D_{\#6} + \frac{D_{\#6}}{2} = 4.125\ in$$

$$cover_{Avg} := \frac{cover_1 + cover_2}{2} = 3.75\ in$$

$$d := h - cover_{Avg} = 20.25\ in$$

Bearing Pressure:

$$q_u := \frac{P_{des1}}{(B \cdot B)} + (\gamma_{conc} \cdot t_f) + 2 \cdot (\gamma_{backfill} \cdot H) = 3405.851\ psf$$

$L_1 := B \qquad L_2 := B \qquad c_1 := B_{col} \qquad c_2 := B_{col} \qquad \lambda := 1$

$$V_{uOneWay} := q_u \cdot L_2 \cdot \left(\frac{L_1 - c_1}{2} - d\right) = -7.545\ kip$$

$$\phi V_{cOneWay} := 0.75 \cdot \left(2 \cdot \lambda \cdot \sqrt{f'_c \cdot psi} \cdot L_2 \cdot d\right) = 84.528\ kip$$

$check :=$ if $V_{uOneWay} < \phi V_{cOneWay}$
  "The footing has adequate shear strength"
else
  "The footing has inadequate shear strength"

$check =$ "The footing has adequate shear strength"

Check Punching Shear Strength:

$$V_{uPunching} := q_u \cdot \left(L_1 \cdot L_2 - (c_1 + d) \cdot (c_2 + d)\right) = 11.186\ kip$$

$$\beta := \frac{B_{col}}{B_{col}} = 1 \qquad \alpha_s := 20 \qquad b_o := 2\ (c_1 + d) + 2 \cdot (c_2 + d) = 12.75\ ft$$

$$\phi V_{cPunching} := 0.75 \cdot min\left(4, \left(2 + \frac{4}{\beta}\right), \left(2 + \frac{\alpha_s \cdot d}{b_o}\right)\right) \cdot \lambda \cdot \sqrt{f'_c \cdot psi} \cdot b_o \cdot d = 587.852\ kip$$

$$check := \text{if } V_{uPunching} < \phi V_{cPunching} \;\; \| \text{"The footing has adequate punching shear strength"} \;\; \text{else} \;\; \| \text{"The footing has inadequate punching shear strength"}$$

$check =$ "The footing has adequate punching shear strength"

Check Flexural Strength:

$$M_u := q_u \cdot L_2 \cdot \left(\frac{L_1 - c_1}{2}\right) \cdot \left(\frac{L_1 - c_1}{4}\right) = 7.328 \; kip \cdot ft$$ (required flexural resistance)

$\rho_{min} := 0.0018$

$A_{sMin} := \rho_{min} \cdot B \cdot h = 1.901 \; in^2$

Try 5 #6 Rebars: $A_{\#6} := 0.440 \; in^2$

$A_s := 5 \cdot A_{\#6} = 2.2 \; in^2$ 5 #6 bars is adequate

Bar spacing:

$BarSpace_{max} := min(3 \cdot h, 18 \; in) = 18 \; in$

$BarSpace := \frac{B}{4} = 0.917 \; ft$ actual bar spacing is less than max, so design is okay. Use 1' spacing

Compute flexural strength of a singly reinforced rectangular section:

$depth := h$ $y_{s1} := cover_1$ $A_s = 2.2 \; in^2$ $b := B$

$d_t := depth - y_{s1} = 20.625 \; in$

$$\beta_1 := \text{if } f'_c \le 4000 \; psi \;\; \| 0.85 \;\; \text{else} \;\; \| \max\left(0.65, 0.85 - 0.05 \cdot \frac{f'_c - 4000 \; psi}{1000 \; psi}\right)$$

$\beta_1 = 0.85$

$$A_{sTensionControlled} := \frac{\beta_1 \cdot 0.85 \cdot f'_c \cdot b}{f_y} \cdot \frac{3 \cdot d_t}{8} = 16.392 \; in^2$$

The design is tension controlled

Depth of Concrete Compression block:

$$a := \frac{A_s \cdot f_y}{0.85 \cdot f'_c \cdot b} = 0.882 \; in$$

$$M_n := A_s \cdot f_y \cdot \left(d - \frac{a}{2}\right) = 217.897 \; kip \cdot ft$$

$\phi M_n := 0.9 \cdot M_n = 196.107 \; kip \cdot ft$ (flexural strength)

$$check := \text{if } M_u < \phi M_n \;\; \| \text{"The footing has adequate flexural strength"} \;\; \text{else} \;\; \| \text{"The footing has inadequate flexural strength"}$$

$check =$ "The footing has adequate flexural strength"

Check Development Length of Flexural 180 degree Hooked Rebars:

As conservative assumptions:

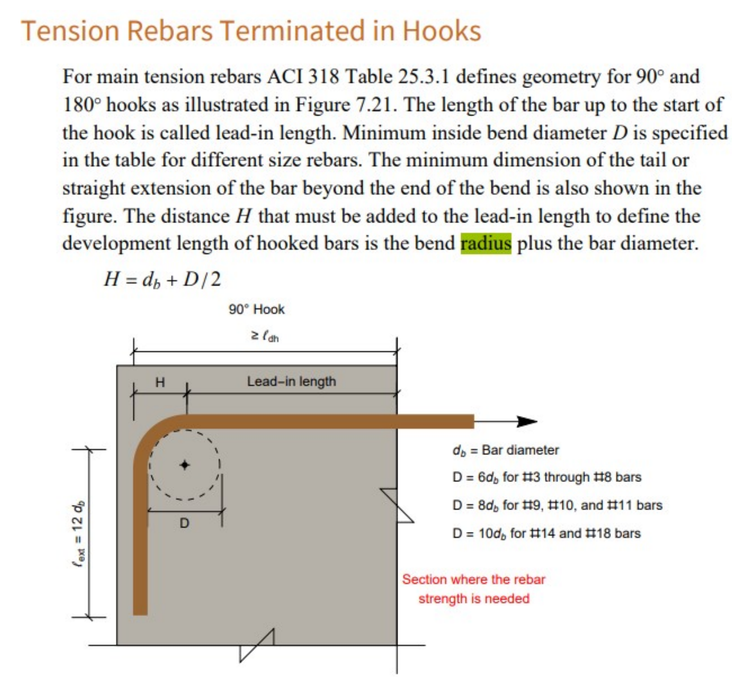
$\psi_e := 1.0$ $\psi_r := 1.0$ $\psi_o := 1.0$ $\psi_c := 0.6 + \frac{f'_c}{15000 \; psi} = 0.867$ $d_{bar} := D_{\#6}$

$$l_{dhook} := \max\left(6\ in, 8 \cdot d_{bar}, \left(\frac{\psi_e \cdot \psi_r \cdot \psi_o \cdot \psi_c \cdot f_y}{55 \cdot \lambda \cdot min\left(100\ psi, \sqrt{f'_c \cdot psi}\right)}\right) \cdot 1\ in \cdot \left(\frac{d_{bar}}{in}\right)^{1.5}\right) = 9.71\ in$$

Length of bars from the critical bending section: $\frac{\left(B - B_{col}\right)}{2} = 13\ in$

$$check := \text{if } l_{dhook} < \frac{\left(B - B_{col}\right)}{2}$$
$$\quad \| \text{“There is adequate room to develop the hooked bars”}$$
$$\text{else}$$
$$\quad \| \text{“There is inadequate room to develop the hooked bars”}$$

$$check = \text{“There is adequate room to develop the hooked bars”}$$


Tension Rebars Terminated in Hooks

For main tension rebars ACI 318 Table 25.3.1 defines geometry for 90° and 180° hooks as illustrated in Figure 7.21. The length of the bar up to the start of the hook is called lead-in length. Minimum inside bend diameter D is specified in the table for different size rebars. The minimum dimension of the tail or straight extension of the bar beyond the end of the bend is also shown in the figure. The distance H that must be added to the lead-in length to define the development length of hooked bars is the bend radius plus the bar diameter.

$$H = d_b + D/2$$

Check to make sure footing is thick enough to accommodate development length:

$$r := \frac{d_{bar} \cdot 6}{2} = 2.25\ in \qquad \text{(radius of dowel bar bend)}$$

$$depth_{FlexuralHooks} := \left(\left(d_{bar} \cdot 6\right) + \left(2 \cdot d_{bar}\right)\right) + d_{bar} = 6.75\ in$$

$$L_{extension} := \max\left(4 \cdot d_{bar}, 2.5\ in\right) = 3\ in$$

$$H := d_{bar} + r = 3\ in \qquad \text{(distance required for hook)}$$

$$h_{min} := depth_{FlexuralHooks} = 6.75\ in$$

$$check := \text{if } h_{min} < h$$
$$\quad \| \text{“The footing thickness is adequate to accomodate the hooked bar”}$$
$$\text{else}$$
$$\quad \| \text{“The footing thickness is not adequate to accomodate the hooked bar”}$$

$$check = \text{“The footing thickness is adequate to accomodate the hooked bar”}$$

Check Bearing Capacity of Column at Base:

$$A_1 := B_{col} \cdot B_{col} = 324\ in^2 \qquad L := B = 3.667\ ft \qquad l := min\left(L, \left(\left(2 \cdot h\right) + B_{col} + \left(2 \cdot h\right)\right)\right) = 3.667\ ft$$

$$A_2 := l^2 = 1936\ in^2$$

$$N_1 := 0.65 \cdot \left(0.85 \cdot f'_c \cdot A_1\right) = 716.04\ kip \qquad N_2 := 0.65 \cdot min\left(\left(\left(0.85 \cdot f'_c \cdot A_1\right) \cdot \sqrt{\frac{A_2}{A_1}}\right), \left(2 \cdot 0.85 \cdot f'_c \cdot A_1\right)\right) = 1432.08\ kip$$

$$\phi P_{BaseBearing} := min\left(N_1, N_2\right) = 716.04\ kip$$

$$check := \text{if } P_{des1} < \phi P_{BaseBearing}$$
$$\quad \| \text{“The footing has adequate bearing strength at the base”}$$
$$\text{else}$$
$$\quad \| \text{“The footing has inadequate bearing strength at the base”}$$

$$check = \text{“The footing has adequate bearing strength at the base”}$$

Check 90 Degree Hooked Dowel Bars in Column:

Minimum Steel Ratio:

$$\rho_{min} := 0.005 \cdot A_1 = 1.62\ in^2$$

Use 4 #6 dowels $\quad A_{\#6} := 0.440\ in^2 \qquad D_{\#6} := 0.750\ in$

$$A_{sDowel} := 4 \cdot A_{\#6} = 1.76\ in^2$$

Development Length:

$$d_{dowel} := D_{\#6}$$

$$l_{dc} := \max\left(8\ in, \frac{0.02 \cdot f_y \cdot d_{dowel}}{\lambda \cdot min\left(100\ psi, \sqrt{f'_c \cdot psi}\right)}, \frac{0.0003}{psi} \cdot f_y \cdot d_{dowel}\right) = 14.23\ in$$

$l_{dc} := 14.25\ in$ (round up to get an appropriate constructible dimension)

Check to make sure footing is thick enough to accommodate development length:

$r := \frac{d_{dowel} \cdot 6}{2} = 2.25\ in$ (radius of dowel bar bend)

$L_{extension} := 12 \cdot d_{dowel} = 9\ in$

$H := d_{dowel} + r = 3\ in$ (distance required for hook)

$h_{min} := l_{dc} + depth_{FlexuralHooks} + c_c = 24\ in$

$$check := \text{if } h_{min} \le h \\ \quad \text{"There footing thickness is adequate"} \\ \text{else} \\ \quad \text{"There footing thickness is inadequate"}$$

$check = \text{"There footing thickness is adequate"}$

Check Rebars in Column:

$d_{bar} := D_{\#6}$ column bars are #6

Development Length:

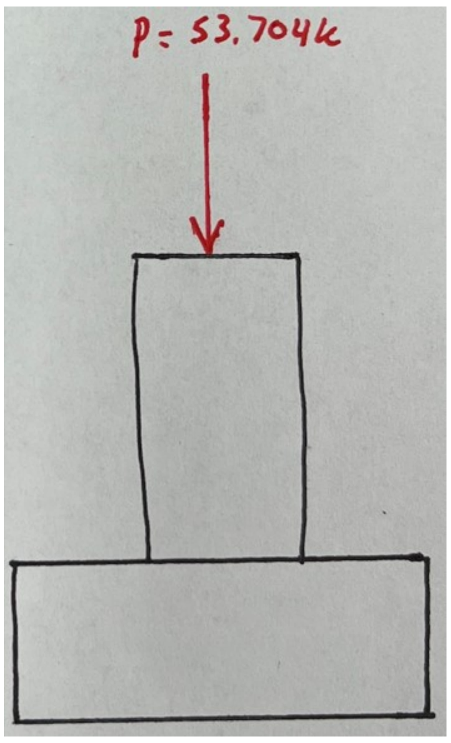
$$l_{dcCol} := \max\left(8\ in, \frac{0.02 \cdot f_y \cdot d_{bar}}{\lambda \cdot min\left(100\ psi, \sqrt{f'_c \cdot psi}\right)}, \frac{0.0003}{psi} \cdot f_y \cdot d_{bar}\right) = 14.23\ in$$

Splice length for rebars in compression:

$\alpha_s := 1$

$l_{splice} := \max\left(12\ in, l_{dcCol}, \frac{0.0005}{psi} \cdot f_y \cdot d_{bar} \cdot \alpha_s\right) = 22.5\ in$ (round up to a 24in (2ft) splice)

Second Square Footing Design:



Check One-Way Shear Strength:

$B := FIF(\text{"4' 4"})$ $H := FIF(\text{"2' 8"})$ $B_{col} := FIF(\text{"1' 6"})$ $t_f := 2\ ft$ $h := t_f$

$P_{des2} := 53.704\ kip$ $\gamma_{conc} := 150\ pcf$ $\gamma_{backfill} := 120\ pcf$ $f'_c := 4000\ psi$ $f_y := 60\ ksi$

Effective depth of footing:

Assume a 3in clear cover, #6 rebars and bars in both directions:

$c_c := 3\ in$ $D_{\#6} := 0.750\ in$

$$cover_1 := c_c + \frac{D_{\#6}}{2} = 3.375\ in \qquad cover_2 := c_c + D_{\#6} + \frac{D_{\#6}}{2} = 4.125\ in$$

$$cover_{Avg} := \frac{cover_1 + cover_2}{2} = 3.75\ in$$

$$d := h - cover_{Avg} = 20.25\ in$$

Bearing Pressure:

$$q_u := \frac{P_{des2}}{(B \cdot B)} + (\gamma_{conc} \cdot t_f) + 2 \cdot (\gamma_{backfill} \cdot H) = 3799.976\ psf$$

$L_1 := B$ $L_2 := B$ $c_1 := B_{col}$ $c_2 := B_{col}$ $\lambda := 1$

$$V_{uOneWay} := -q_u \cdot L_2 \cdot \left(\frac{L_1 - c_1}{2} - d\right) = 4.46\ kip$$

$$\phi V_{cOneWay} := 0.75 \cdot \left(2 \cdot \lambda \cdot \sqrt{f'_c \cdot psi} \cdot L_2 \cdot d\right) = 99.896\ kip$$

$check :=$ if $V_{uOneWay} < \phi V_{cOneWay}$
  "The footing has adequate shear strength"
else
  "The footing has inadequate shear strength"

$check =$ "The footing has adequate shear strength"

Check Punching Shear Strength:

$$V_{uPunching} := q_u \cdot \left(L_1 \cdot L_2 - (c_1 + d) \cdot (c_2 + d)\right) = 32.747\ kip$$

$\beta := \frac{B_{col}}{B_{col}} = 1$ $\alpha_s := 20$ $b_o := 2\ (c_1 + d) + 2 \cdot (c_2 + d) = 12.75\ ft$

$$\phi V_{cPunching} := 0.75 \cdot min\left(4, \left(2 + \frac{4}{\beta}\right), \left(2 + \frac{\alpha_s \cdot d}{b_o}\right)\right) \cdot \lambda \cdot \sqrt{f'_c \cdot psi} \cdot b_o \cdot d = 587.852\ kip$$

$$check := \text{if } V_{uPunching} < \phi V_{cPunching}$$
$$\quad \text{"The footing has adequate punching shear strength"}$$
$$\text{else}$$
$$\quad \text{"The footing has inadequate punching shear strength"}$$

$$check = \text{"The footing has adequate punching shear strength"}$$

Check Flexural Strength:

$$M_u := q_u \cdot L_2 \cdot \left(\frac{L_1 - c_1}{2}\right) \cdot \left(\frac{L_1 - c_1}{4}\right) = 16.524 \; kip \cdot ft$$ (required flexural resistance)

$$\rho_{min} := 0.0018$$

$$A_{sMin} := \rho_{min} \cdot B \cdot h = 2.246 \; in^2$$

Try 6 #6 Rebars: $A_{\#6} := 0.440 \; in^2$

$A_s := 6 \cdot A_{\#6} = 2.64 \; in^2$ 6 #6 bars is adequate

Bar spacing:

$$BarSpace_{max} := min(3 \cdot h, 18 \; in) = 18 \; in$$

$BarSpace := \dfrac{B}{5} = 0.867 \; ft$ actual bar spacing is less than max, so design is okay. Use 1' spacing

Compute flexural strength of a singly reinforced rectangular section:

$depth := h \qquad y_{s1} := cover_1 \qquad A_s = 2.64 \; in^2 \qquad b := B$

$$d_t := depth - y_{s1} = 20.625 \; in$$

$$\beta_1 := \text{if } f'_c \le 4000 \; psi$$
$$\quad 0.85$$
$$\text{else}$$
$$\quad \max\left(0.65, 0.85 - 0.05 \cdot \frac{f'_c - 4000 \; psi}{1000 \; psi}\right)$$

$$\beta_1 = 0.85$$

$$A_{sTensionControlled} := \frac{\beta_1 \cdot 0.85 \cdot f'_c \cdot b}{f_y} \cdot \frac{3 \cdot d_t}{8} = 19.372 \; in^2$$

The design is tension controlled

Depth of Concrete Compression block:

$$a := \frac{A_s \cdot f_y}{0.85 \cdot f'_c \cdot b} = 0.896 \; in$$

$$M_n := A_s \cdot f_y \cdot \left(d - \frac{a}{2}\right) = 261.387 \; kip \cdot ft$$

$\phi M_n := 0.9 \cdot M_n = 235.248 \; kip \cdot ft$ (flexural strength)

$$check := \text{if } M_u < \phi M_n$$
$$\quad \text{"The footing has adequate flexural strength"}$$
$$\text{else}$$
$$\quad \text{"The footing has inadequate flexural strength"}$$

$$check = \text{"The footing has adequate flexural strength"}$$

Check Development Length of Flexural 180 degree Hooked Rebars:

As conservative assumptions:

$\psi_e := 1.0 \qquad \psi_r := 1.0 \qquad \psi_o := 1.0 \qquad \psi_c := 0.6 + \dfrac{f'_c}{15000 \; psi} = 0.867 \qquad d_{bar} := D_{\#6}$

$$l_{dhook} := \max\left(6\ in, 8 \cdot d_{bar}, \left(\frac{\psi_e \cdot \psi_r \cdot \psi_o \cdot \psi_c \cdot f_y}{55 \cdot \lambda \cdot min\left(100\ psi, \sqrt{f'_c \cdot psi}\right)}\right) \cdot 1\ in \cdot \left(\frac{d_{bar}}{in}\right)^{1.5}\right) = 9.71\ in$$

Length of bars from the critical bending section: $\frac{(B - B_{col})}{2} = 17\ in$

$check :=$ if $l_{dhook} < \frac{(B - B_{col})}{2}$
  "There is adequate room to develop the hooked bars"
else
  "There is inadequate room to develop the hooked bars"

$check =$ "There is adequate room to develop the hooked bars"

Check to make sure footing is thick enough to accommodate development length:

$r := \frac{d_{bar} \cdot 6}{2} = 2.25\ in$ (radius of dowel bar bend)

$depth_{FlexuralHooks} := \left(\left(d_{bar} \cdot 6\right) + \left(2 \cdot d_{bar}\right)\right) + d_{bar} = 6.75\ in$

$H := d_{bar} + r = 3\ in$ (distance required for hook)

$h_{min} := depth_{FlexuralHooks} = 6.75\ in$

$check :=$ if $h_{min} < h$
  "The footing thickness is adequate to accomodate the hooked bar"
else
  "The footing thickness is not adequate to accomodate the hooked bar"

$check =$ "The footing thickness is adequate to accomodate the hooked bar"

Check Bearing Capacity of Column at Base:

$A_1 := B_{col} \cdot B_{col} = 324\ in^2$ $L := B = 4.333\ ft$

$l := min\left(L, \left((2 \cdot h) + B_{col} + (2 \cdot h)\right)\right) = 4.333\ ft$

$A_2 := l^2 = 2704\ in^2$

$N_1 := 0.65 \cdot \left(0.85 \cdot f'_c \cdot A_1\right) = 716.04\ kip$ $N_2 := 0.65 \cdot min\left(\left(\left(0.85 \cdot f'_c \cdot A_1\right) \cdot \sqrt{\frac{A_2}{A_1}}\right), \left(2 \cdot 0.85 \cdot f'_c \cdot A_1\right)\right) = 1432.08\ kip$

$\phi P_{BaseBearing} := min\left(N_1, N_2\right) = 716.04\ kip$

$check :=$ if $P_{des2} < \phi P_{BaseBearing}$
  "The footing has adequate bearing strength at the base"
else
  "The footing has inadequate bearing strength at the base"

$check =$ "The footing has adequate bearing strength at the base"

Check 90 Degree Hooked Dowel Bars in Column:

Minimum Steel Ratio:

$\rho_{min} := 0.005 \cdot A_1 = 1.62\ in^2$

Use 4 #6 dowels $A_{\#6} := 0.440\ in^2$ $D_{\#6} := 0.750\ in$

$A_{sDowel} := 4 \cdot A_{\#6} = 1.76\ in^2$

Development Length:

$d_{dowel} := D_{\#6}$

$$l_{dc} := \max\left(8\ in, \frac{0.02 \cdot f_y \cdot d_{dowel}}{\lambda \cdot min\left(100\ psi, \sqrt{f'_c \cdot psi}\right)}, \frac{0.0003}{psi} \cdot f_y \cdot d_{dowel}\right) = 14.23\ in$$

$l_{dc} := 14.25\ in$ (round up to get an appropriate constructible dimension)

Check to make sure footing is thick enough to accommodate development length:

$$r := \frac{d_{dowel} \cdot 6}{2} = 2.25\ in$$ (radius of dowel bar bend)

$$L_{extension} := 12 \cdot d_{dowel} = 9\ in$$

$$H := d_{dowel} + r = 3\ in$$ (distance required for hook)

$$h_{min} := l_{dc} + depth_{FlexuralHooks} + c_c = 24\ in$$

$$check := \text{if } h_{min} \le h \;\Big\|\; \text{“There footing thickness is adequate”} \;\; \text{else} \;\Big\|\; \text{“There footing thickness is inadequate”}$$

$$check = \text{“There footing thickness is adequate”}$$

Check Rebars in Column:

$$d_{bar} := D_{\#6}$$ column bars are #6

Development Length:

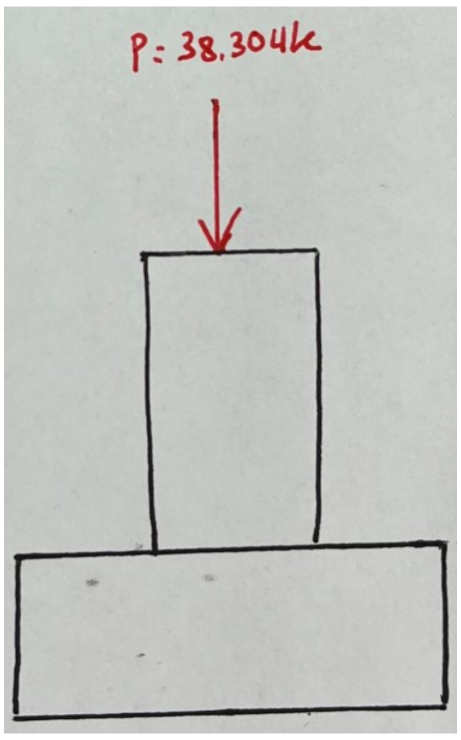
$$l_{dcCol} := \max\left(8\ in, \frac{0.02 \cdot f_y \cdot d_{bar}}{\lambda \cdot min\left(100\ psi, \sqrt{f'_c \cdot psi}\right)}, \frac{0.0003}{psi} \cdot f_y \cdot d_{bar}\right) = 14.23\ in$$

Splice length for rebars in compression:

$$\alpha_s := 1$$

$$l_{splice} := \max\left(12\ in, l_{dcCol}, \frac{0.0005}{psi} \cdot f_y \cdot d_{bar} \cdot \alpha_s\right) = 22.5\ in$$ (round up to a 24in (2ft) splice)

Third Square Footing Design:



Check One-Way Shear Strength:

$B := FIF(\text{“3' 8”})$ $H := FIF(\text{“12' 9”})$ $B_{col} := FIF(\text{“1' 6”})$ $t_f := 2\ ft$ $h := t_f$

$P_{des3} := 38.304\ kip$ $\gamma_{conc} := 150\ pcf$ $\gamma_{backfill} := 120\ pcf$ $f'_c := 4000\ psi$ $f_y := 60\ ksi$ $D_f := 2.75\ ft$

Effective depth of footing:

Assume a 3in clear cover, #6 rebars and bars in both directions:

$c_c := 3\ in$ $D_{\#6} := 0.750\ in$

$$cover_1 := c_c + \frac{D_{\#6}}{2} = 3.375\ in \qquad cover_2 := c_c + D_{\#6} + \frac{D_{\#6}}{2} = 4.125\ in$$

$$cover_{Avg} := \frac{cover_1 + cover_2}{2} = 3.75\ in$$

$$d := h - cover_{Avg} = 20.25\ in$$

Bearing Pressure:

$$q_u := \frac{P_{des3}}{(B \cdot B)} + (\gamma_{conc} \cdot t_f) + (\gamma_{backfill} \cdot (D_f - t_f)) + (\gamma_{backfill} \cdot H) = 4769.058\ psf$$

$L_1 := B$ $L_2 := B$ $c_1 := B_{col}$ $c_2 := B_{col}$ $\lambda := 1$

$$V_{uOneWay} := -q_u \cdot L_2 \cdot \left(\frac{L_1 - c_1}{2} - d\right) = 10.565\ kip$$

$$\phi V_{cOneWay} := 0.75 \cdot \left(2 \cdot \lambda \cdot \sqrt{f'_c \cdot psi} \cdot L_2 \cdot d\right) = 84.528\ kip$$

$check :=$ if $V_{uOneWay} < \phi V_{cOneWay}$
 “The footing has adequate shear strength”
else
 “The footing has inadequate shear strength”

$check =$ “The footing has adequate shear strength”

Check Punching Shear Strength:

$$V_{uPunching} := -q_u \cdot (L_1 \cdot L_2 - (c_1 + d) \cdot (c_2 + d)) = -15.663\ kip$$

$$\beta := \frac{B_{col}}{B_{col}} = 1 \qquad \alpha_s := 20 \qquad b_o := 2\ (c_1 + d) + 2 \cdot (c_2 + d) = 12.75\ ft$$

$$\phi V_{cPunching} := 0.75 \cdot min\left(4, \left(2 + \frac{4}{\beta}\right), \left(2 + \frac{\alpha_s \cdot d}{b_o}\right)\right) \cdot \lambda \cdot \sqrt{f'_c \cdot psi} \cdot b_o \cdot d = 587.852\ kip$$

$check :=$ if $V_{uPunching} < \phi V_{cPunching}$
 “The footing has adequate punching shear strength”
else
 “The footing has inadequate punching shear strength”

$check =$ “The footing has adequate punching shear strength”

Check Flexural Strength:

$$M_u := q_u \cdot L_2 \cdot \left(\frac{L_1 - c_1}{2}\right) \cdot \left(\frac{L_1 - c_1}{4}\right) = 10.261 \ kip \cdot ft$$ (required flexural resistance)

$$\rho_{min} := 0.0018$$

$$A_{sMin} := \rho_{min} \cdot B \cdot h = 1.901 \ in^2$$

Try 5 #6 Rebars: $A_{\#6} := 0.440 \ in^2$

$A_s := 5 \cdot A_{\#6} = 2.2 \ in^2$ 5 #6 bars is adequate

Bar spacing:

$$BarSpace_{max} := min(3 \cdot h, 18 \ in) = 18 \ in$$

$BarSpace := \frac{B}{3} = 14.667 \ in$ actual bar spacing is less than max, so design is okay. Use 12" spacing

Compute flexural strength of a singly reinforced rectangular section:

$depth := h$ $y_{s1} := cover_1$ $A_s = 2.2 \ in^2$ $b := B$

$$d_t := depth - y_{s1} = 20.625 \ in$$

$$\beta_1 := \text{if } f'_c \le 4000 \ psi \ \| \ 0.85 \ \text{else} \ \| \ \max\left(0.65, 0.85 - 0.05 \cdot \frac{f'_c - 4000 \ psi}{1000 \ psi}\right)$$

$$\beta_1 = 0.85$$

$$A_{sTensionControlled} := \frac{\beta_1 \cdot 0.85 \cdot f'_c \cdot b}{f_y} \cdot \frac{3 \cdot d_t}{8} = 16.392 \ in^2$$

The design is tension controlled

Depth of Concrete Compression block:

$$a := \frac{A_s \cdot f_y}{0.85 \cdot f'_c \cdot b} = 0.882 \ in$$

$$M_n := A_s \cdot f_y \cdot \left(d - \frac{a}{2}\right) = 217.897 \ kip \cdot ft$$

$\phi M_n := 0.9 \cdot M_n = 196.107 \ kip \cdot ft$ (flexural strength)

check := if $M_u < \phi M_n$
‖ "The footing has adequate flexural strength"
else
‖ "The footing has inadequate flexural strength"

check = "The footing has adequate flexural strength"

Check Development Length of Flexural 180 degree Hooked Rebars:

As conservative assumptions:

$\psi_e := 1.0$ $\psi_r := 1.0$ $\psi_o := 1.0$ $\psi_c := 0.6 + \frac{f'_c}{15000 \ psi} = 0.867$ $d_{bar} := D_{\#6}$

$$l_{dhook} := \max\left(6 \ in, 8 \cdot d_{bar}, \left(\frac{\psi_e \cdot \psi_r \cdot \psi_o \cdot \psi_c \cdot f_y}{55 \cdot \lambda \cdot min\left(100 \ psi, \sqrt{f'_c \cdot psi}\right)}\right) \cdot 1 \ in \cdot \left(\frac{d_{bar}}{in}\right)^{1.5}\right) = 9.71 \ in$$

Length of bars from the critical bending section: $\frac{(B - B_{col})}{2} = 13 \ in$

$$check := \text{if } l_{dhook} < \frac{(B - B_{col})}{2}$$
$$\quad \| \text{“There is adequate room to develop the hooked bars”}$$
$$\text{else}$$
$$\quad \| \text{“There is inadequate room to develop the hooked bars”}$$

$$check = \text{“There is adequate room to develop the hooked bars”}$$

Check to make sure footing is thick enough to accommodate development length:

$$r := \frac{d_{bar} \cdot 6}{2} = 2.25\ in$$ (radius of dowel bar bend)

$$depth_{FlexuralHooks} := ((d_{bar} \cdot 6) + (2 \cdot d_{bar})) + d_{bar} = 6.75\ in$$

$$H := d_{bar} + r = 3\ in$$ (distance required for hook)

$$h_{min} := depth_{FlexuralHooks} = 6.75\ in$$

$$check := \text{if } h_{min} < h$$
$$\quad \| \text{“The footing thickness is adequate to accomodate the hooked bar”}$$
$$\text{else}$$
$$\quad \| \text{“The footing thickness is not adequate to accomodate the hooked bar”}$$
$$check = \text{“The footing thickness is adequate to accomodate the hooked bar”}$$

Check Bearing Capacity of Column at Base:

$$A_1 := B_{col} \cdot B_{col} = 324\ in^2 \qquad L := B = 3.667\ ft$$

$$l := min(L, ((2 \cdot h) + B_{col} + (2 \cdot h))) = 3.667\ ft$$

$$A_2 := l^2 = 1936\ in^2$$

$$N_1 := 0.65 \cdot (0.85 \cdot f'_c \cdot A_1) = 716.04\ kip \qquad N_2 := 0.65 \cdot min\left(\left((0.85 \cdot f'_c \cdot A_1) \cdot \sqrt{\frac{A_2}{A_1}}\right), (2 \cdot 0.85 \cdot f'_c \cdot A_1)\right) = 1432.08\ kip$$

$$\phi P_{BaseBearing} := min(N_1, N_2) = 716.04\ kip$$

$$check := \text{if } P_{des3} < \phi P_{BaseBearing}$$
$$\quad \| \text{“The footing has adequate bearing strength at the base”}$$
$$\text{else}$$
$$\quad \| \text{“The footing has inadequate bearing strength at the base”}$$
$$check = \text{“The footing has adequate bearing strength at the base”}$$

Check 90 Degree Hooked Dowel Bars in Column:

Minimum Steel Ratio:

$$\rho_{min} := 0.005 \cdot A_1 = 1.62\ in^2$$

Use 4 #6 dowels $\quad A_{\#6} := 0.440\ in^2 \qquad D_{\#6} := 0.750\ in$

$$A_{sDowel} := 4 \cdot A_{\#6} = 1.76\ in^2$$

Development Length:

$$d_{dowel} := D_{\#6}$$

$$l_{dc} := \max\left(8\ in, \frac{0.02 \cdot f_y \cdot d_{dowel}}{\lambda \cdot min\left(100\ psi, \sqrt{f'_c \cdot psi}\right)}, \frac{0.0003}{psi} \cdot f_y \cdot d_{dowel}\right) = 14.23\ in$$

$$l_{dc} := 14.25\ in$$ (round up to get an appropriate constructible dimension)

Check to make sure footing is thick enough to accommodate development length:

$$r := \frac{d_{dowel} \cdot 6}{2} = 2.25\ in$$ (radius of dowel bar bend)

$$L_{extension} := 12 \cdot d_{dowel} = 9\ in$$

$$H := d_{dowel} + r = 3\ in$$ (distance required for hook)

$$h_{min} := l_{dc} + depth_{FlexuralHooks} + c_c = 24\ in$$

$$check := \text{if } h_{min} \le h \;\; \|\text{“The footing thickness is adequate”} \;\; \text{else} \;\; \|\text{“The footing thickness is inadequate”}$$

$$check = \text{“The footing thickness is adequate”}$$

Check Rebars in Column:

$d_{bar} := D_{\#6}$ column bars are #6

Development Length:

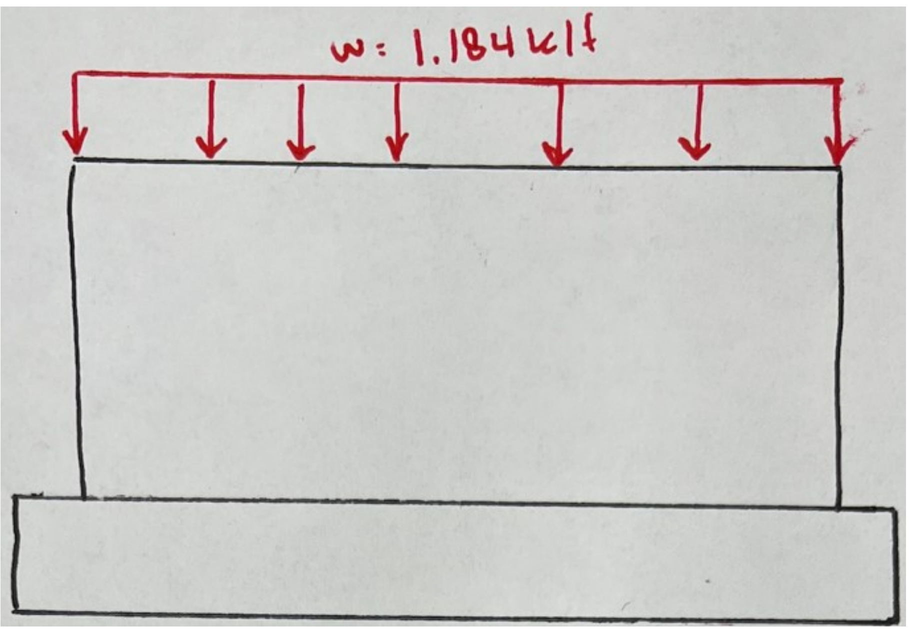
$$l_{dcCol} := \max\left(8\ in, \frac{0.02 \cdot f_y \cdot d_{bar}}{\lambda \cdot min\left(100\ psi, \sqrt{f'_c \cdot psi}\right)}, \frac{0.0003}{psi} \cdot f_y \cdot d_{bar}\right) = 14.23\ in$$

Splice length for rebars in compression:

$$\alpha_s := 1$$

$$l_{splice} := \max\left(12\ in, l_{dcCol}, \frac{0.0005}{psi} \cdot f_y \cdot d_{bar} \cdot \alpha_s\right) = 22.5\ in$$ (round up to a 24in (2ft) splice)

First Wall Footing Design:



Check One-Way Shear Strength:

$B := FIF(\text{“2' 10”})$ $\quad H := FIF(\text{“12' 9”})$ $\quad B_{wall} := FIF(\text{“0' 8”})$ $\quad t_f := FIF(\text{“1' 8”})$ $\quad h := t_f$

$\gamma_{conc} := 150\ pcf$ $\quad \gamma_{backfill} := 120\ pcf$ $\quad f'_c := 4000\ psi$ $\quad f_y := 60\ ksi$

$w_{NorthWall} := 1.184\ klf$ $\quad u := 670.8\ psf$ (from geotechnical limit state analysis)

Effective depth of footing:

For continuous footings, effective depth d is measured from the top of the footing to the center of the lateral bars. Longitudinal bars are designed separately:

Assume a 3in clear cover, #3 rebars:

$c_c := 3\ in$ $\quad D_{\#6} := 0.750\ in$

$$cover := c_c + \frac{D_{\#6}}{2} = 3.375\ in$$

$$d := h - cover = 16.625\ in$$

Bearing Pressure:

$$W_f := (\gamma_{conc} \cdot t_f \cdot B) + \left(\gamma_{backfill} \cdot 4\ in \cdot \left(\frac{B}{2} - \frac{B_{wall}}{2}\right)\right) + \left(\gamma_{backfill} \cdot \left(H \cdot \left(\frac{B}{2} - \frac{B_{wall}}{2}\right)\right)\right) = 2.409\ klf$$

$$q_u := \frac{w_{NorthWall} + W_f}{B} - u = 597.376\ psf$$

$L_1 := 1\ ft$ (Long dimension. Use 1 ft analysis strip)

$L_2 := B$ (short dimension)

$c := B_{wall}$ (width of wall)

$$P_u := w_{NorthWall} = 1.184\ klf$$

$$V_{uOneWay} := P_u \cdot \left(\frac{B - c - 2 \cdot d}{B}\right) = -0.252\ klf$$

$$\phi V_{cOneWay} := \frac{0.75 \cdot \left(2 \cdot \lambda \cdot \sqrt{f'_c \cdot psi} \cdot L_2 \cdot d\right)}{1\ ft} = 53.624\ klf$$

check := if $V_{uOneWay} < \phi V_{cOneWay}$
  “The footing has adequate shear strength”
else
  “The footing has inadequate shear strength”

check = “The footing has adequate shear strength”

Check Flexural Strength:

$$l := \frac{B - \frac{c}{2}}{2} = 1.25\ ft$$

$$M_u := \frac{P_u \cdot l^2}{2 \cdot B} = 0.326\ \frac{kip \cdot ft}{ft}$$ (required flexural resistance)

$$\rho_{min} := 0.0018$$

$$A_{sMin} := \rho_{min} \cdot d \cdot \frac{12\ in}{1 \cdot ft} = 0.359\ \frac{in^2}{ft}$$

Try 1 #6 Rebars: $A_{\#6} := 0.440\ in^2$

$A_s := 1 \cdot A_{\#6} = 0.44\ in^2$ 1 #6 bar is adequate

Bar spacing:

$$BarSpace_{max} := min(3 \cdot h, 18\ in) = 18\ in$$

$BarSpace := \frac{1\ ft}{1} = 12\ in$ Need one bar every foot

Compute flexural strength of a singly reinforced rectangular section:

$depth := h$ $y_{s1} := cover_1$ $A_s = 0.44\ in^2$ $b := B$

$$d_t := depth - y_{s1} = 16.625\ in$$

$$\beta_1 := \text{if } f'_c \le 4000\ psi \;\|\; 0.85 \;\text{else}\; \| \max\left(0.65, 0.85 - 0.05 \cdot \frac{f'_c - 4000\ psi}{1000\ psi}\right)$$

$$\beta_1 = 0.85$$

$$A_{sTensionControlled} := \frac{\beta_1 \cdot 0.85 \cdot f'_c \cdot b}{f_y} \cdot \frac{3 \cdot d_t}{8} = 10.21\ in^2$$

The design is tension controlled

Depth of Concrete Compression block:

$$a := \frac{A_s \cdot f_y}{0.85 \cdot f'_c \cdot b} = 0.228\ in$$

$$M_n := A_s \cdot f_y \cdot \left(d - \frac{a}{2}\right) = 36.324\ kip \cdot ft$$

$\phi M_n := \frac{0.9 \cdot M_n}{1\ ft} = 32.691\ \frac{kip \cdot ft}{ft}$ (flexural strength)

$check :=$ if $M_u < \phi M_n$
‖ "The footing has adequate flexural strength"
else
‖ "The footing has inadequate flexural strength"

$check =$ "The footing has adequate flexural strength"

Check Development Length of Flexural 180 degree Hooked Rebars:

As conservative assumptions:
$\psi_e := 1.0$ $\psi_r := 1.0$ $\psi_o := 1.0$ $\psi_c := 0.6 + \frac{f'_c}{15000\ psi} = 0.867$ $d_{bar} := D_{\#6}$

$$l_{dhook} := \max\left(6\ in, 8 \cdot d_{bar}, \left(\frac{\psi_e \cdot \psi_r \cdot \psi_o \cdot \psi_c \cdot f_y}{55 \cdot \lambda \cdot min\left(100\ psi, \sqrt{f'_c \cdot psi}\right)}\right) \cdot 1\ in \cdot \left(\frac{d_{bar}}{in}\right)^{1.5}\right) = 9.71\ in$$

Length of bars from the critical bending section: $\frac{(B - B_{wall})}{2} - c_c = 10\ in$

$check :=$ if $l_{dhook} < \frac{(B - B_{wall})}{2}$
‖ "There is adequate room to develop the hooked bars"
else
‖ "There is inadequate room to develop the hooked bars"

$check =$ "There is adequate room to develop the hooked bars"

Design the Longitudinal Steel:

$$\rho_{min} := 0.0018$$

$$A_{sMin} := \rho_{min} \cdot B \cdot d = 1.017\ in^2$$

Try 3 #6 Rebars:

$$A_s := 3 \cdot A_{\#6} = 1.32\ in^2$$ 3 #6 bars is adequate

Bar spacing:

$$BarSpace_{max} := min(3 \cdot h, 18\ in) = 18\ in$$

$$BarSpace := \frac{B - 2\ c_c}{2} = 14\ in$$ use 12 in spacing to be conservative

Check to make sure footing is thick enough to accommodate development length:

$$r := \frac{d_{bar} \cdot 6}{2} = 2.25\ in$$ (radius of dowel bar bend)

$$depth_{FlexuralHooks} := ((D_{\#6} \cdot 6) + (2 \cdot D_{\#6})) + D_{\#6} = 6.75\ in$$

$$H := d_{bar} + r = 3\ in$$ (distance required for hook)

$$h_{min} := depth_{FlexuralHooks} = 6.75\ in$$

$check :=$ if $h_{min} < h$
  "The footing thickness is adequate to accomodate the hooked bar"
else
  "The footing thickness is not adequate to accomodate the hooked bar"

$check =$ "The footing thickness is adequate to accomodate the hooked bar"

Check Bearing Capacity of Column at Base:

$$L_{wall} := 1\ ft$$

$$A_1 := B_{wall} \cdot L_{wall} = 96\ in^2$$

$$l := min(L, ((2 \cdot h) + B_{wall} + (2 \cdot h))) = 3.667\ ft$$

$$A_2 := l^2 = 1936\ in^2$$

$$N_1 := 0.65 \cdot (0.85 \cdot f'_c \cdot A_1) = 212.16\ kip$$

$$N_2 := 0.65 \cdot min\left(\left((0.85 \cdot f'_c \cdot A_1) \cdot \sqrt{\frac{A_2}{A_1}}\right), (2 \cdot 0.85 \cdot f'_c \cdot A_1)\right) = 424.32\ kip$$

$$\phi P_{BaseBearing} := \frac{min(N_1, N_2)}{1\ ft} = 212.16\ klf$$

$check :=$ if $P_u < \phi P_{BaseBearing}$
  "The footing has adequate bearing strength at the base"
else
  "The footing has inadequate bearing strength at the base"

$check =$ "The footing has adequate bearing strength at the base"

Check 90 Degree Hooked Dowel Bars in Column:

Minimum Steel Ratio:

$$\rho_{min} := 0.005 \cdot A_1 \cdot \frac{1}{1 \cdot ft} = 0.48\ \frac{in^2}{ft}$$

Try 2 #5 dowels $A_{\#5} := 0.310\ in^2$

$$A_{sDowel} := 2 \cdot A_{\#5} = 0.62\ in^2$$ (two dowels per 1 ft is adequate)

Development Length:

$$D_{\#5} := 0.625\ in$$

$$d_{dowel} := D_{\#5}$$

$$l_{dc} := \max\left(8\ in, \frac{0.02 \cdot f_y \cdot d_{dowel}}{\lambda \cdot min\left(100\ psi, \sqrt{f'_c \cdot psi}\right)}, \frac{0.0003}{psi} \cdot f_y \cdot d_{dowel}\right) = 11.859\ in$$

$$l_{dc} := 12\ in$$ (round up to get an appropriate constructible dimension)

Check to make sure footing is thick enough to accommodate development length:

$$r := \frac{d_{dowel} \cdot 6}{2} = 1.875\ in \quad \text{(radius of dowel bar bend)}$$

$$L_{extension} := 12 \cdot d_{dowel} = 7.5\ in$$

$$H := d_{dowel} + r = 2.5\ in \quad \text{(distance required for hook)}$$

$$h_{min} := l_{dc} + depth_{FlexuralHooks} + c_c = 21.75\ in$$

$$check := \begin{array}{l} \text{if } h_{min} \le h \\ \quad \| \text{“The footing thickness is adequate”} \\ \text{else} \\ \quad \| \text{“The footing thickness is inadequate”} \end{array}$$

$$check = \text{“The footing thickness is inadequate”}$$

Check Rebars in Column:

$$d_{bar} := D_{\#6} \quad \text{column bars are \#6}$$

Development Length:

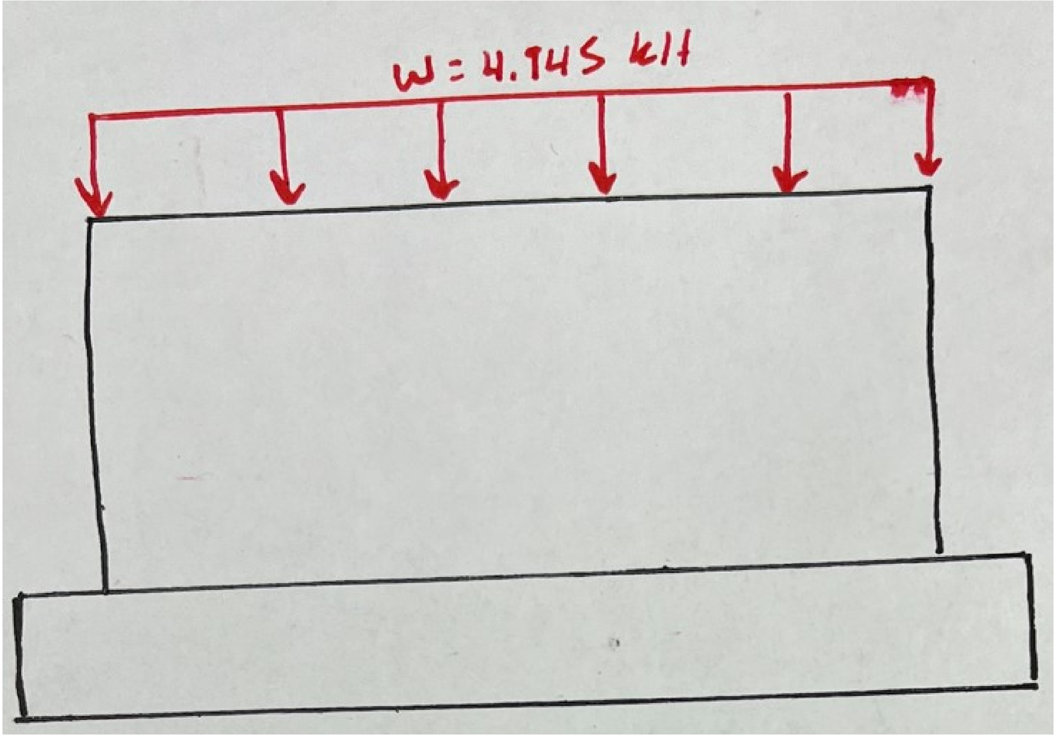
$$l_{dcCol} := \max\left(8\ in, \frac{0.02 \cdot f_y \cdot d_{bar}}{\lambda \cdot min\left(100\ psi, \sqrt{f'_c \cdot psi}\right)}, \frac{0.0003}{psi} \cdot f_y \cdot d_{bar}\right) = 14.23\ in$$

Splice length for rebars in compression:

$$\alpha_s := 1$$

$$l_{splice} := \max\left(12\ in, l_{dcCol}, \frac{0.0005}{psi} \cdot f_y \cdot d_{bar} \cdot \alpha_s\right) = 22.5\ in \quad \text{(round up to a 24in (2ft) splice)}$$

Second Wall Footing Design:



Check One-Way Shear Strength:

$B := FIF(\text{“2' 10”})$ $\quad H := FIF(\text{“12' 9”})$ $\quad B_{wall} := FIF(\text{“0' 8”})$ $\quad t_f := FIF(\text{“1' 8”})$ $\quad h := t_f$

$\gamma_{conc} := 150\ pcf$ $\quad \gamma_{backfill} := 120\ pcf$ $\quad f'_c := 4000\ psi$ $\quad f_y := 60\ ksi$

$w_{EastWall} := 4.945\ klf$ $\quad u := 670.8\ psf$ (from geotechnical limit state analysis)

Effective depth of footing:

For continuous footings, effective depth d is measured from the top of the footing to the center of the lateral bars. Longitudinal bars are designed separately:

Assume a 3in clear cover, #3 rebars:

$c_c := 3\ in$ $\quad D_{\#6} := 0.750\ in$

$$cover := c_c + \frac{D_{\#6}}{2} = 3.375\ in$$

$$d := h - cover = 16.625\ in$$

Bearing Pressure:

$$W_f := (\gamma_{conc} \cdot t_f \cdot B) + \left(\gamma_{backfill} \cdot 4\ in \cdot \left(\frac{B}{2} - \frac{B_{wall}}{2}\right)\right) + \left(\gamma_{backfill} \cdot \left(H \cdot \left(\frac{B}{2} - \frac{B_{wall}}{2}\right)\right)\right) = 2.409\ klf$$

$$q_u := \frac{w_{EastWall} + W_f}{B} - u = 1924.788\ psf$$

$L_1 := 1\ ft$ (Long dimension. Use 1 ft analysis strip)

$L_2 := B$ (short dimension)

$c := B_{wall}$ (width of wall)

$$P_u := w_{EastWall} = 4.945\ klf$$

$$V_{uOneWay} := P_u \cdot \left(\frac{B - c - 2 \cdot d}{B}\right) = -1.054\ klf$$

$$\phi V_{cOneWay} := \frac{0.75 \cdot \left(2 \cdot \lambda \cdot \sqrt{f'_c \cdot psi} \cdot L_2 \cdot d\right)}{1\ ft} = 53.624\ klf$$

$check :=$ if $V_{uOneWay} < \phi V_{cOneWay}$
  “The footing has adequate shear strength”
else
  “The footing has inadequate shear strength”

$check =$ “The footing has adequate shear strength”

Check Flexural Strength:

$$l := \frac{B - \frac{c}{2}}{2} = 1.25\ ft$$

$$M_u := \frac{P_u \cdot l^2}{2 \cdot B} = 1.364\ \frac{kip \cdot ft}{ft}$$ (required flexural resistance)

$$\rho_{min} := 0.0018$$

$$A_{sMin} := \rho_{min} \cdot d \cdot \frac{12\ in}{1 \cdot ft} = 0.359\ \frac{in^2}{ft}$$

Try 1 #6 Rebars: $A_{\#6} := 0.440\ in^2$

$A_s := 1 \cdot A_{\#6} = 0.44\ in^2$ 1 #6 bar is adequate

Bar spacing:

$$BarSpace_{max} := min(3 \cdot h, 18\ in) = 18\ in$$

$BarSpace := \frac{1\ ft}{1} = 12\ in$ Need one bar every foot

Compute flexural strength of a singly reinforced rectangular section:

$depth := h \qquad y_{s1} := cover_1 \qquad A_s = 0.44\ in^2 \qquad b := B$

$$d_t := depth - y_{s1} = 16.625\ in$$

$$\beta_1 := \begin{cases} 0.85 & \text{if } f'_c \le 4000\ psi \\ \max\left(0.65, 0.85 - 0.05 \cdot \frac{f'_c - 4000\ psi}{1000\ psi}\right) & \text{else} \end{cases}$$

$$\beta_1 = 0.85$$

$$A_{sTensionControlled} := \frac{\beta_1 \cdot 0.85 \cdot f'_c \cdot b}{f_y} \cdot \frac{3 \cdot d_t}{8} = 10.21\ in^2$$

The design is tension controlled

Depth of Concrete Compression block:

$$a := \frac{A_s \cdot f_y}{0.85 \cdot f'_c \cdot b} = 0.228\ in$$

$$M_n := A_s \cdot f_y \cdot \left(d - \frac{a}{2}\right) = 36.324\ kip \cdot ft$$

$\phi M_n := \frac{0.9 \cdot M_n}{1\ ft} = 32.691\ \frac{kip \cdot ft}{ft}$ (flexural strength)

$$check := \begin{cases} \text{"The footing has adequate flexural strength"} & \text{if } M_u < \phi M_n \\ \text{"The footing has inadequate flexural strength"} & \text{else} \end{cases}$$

$check =$ "The footing has adequate flexural strength"

Check Development Length of Flexural 180 degree Hooked Rebars:

As conservative assumptions:

$\psi_e := 1.0 \qquad \psi_r := 1.0 \qquad \psi_o := 1.0 \qquad \psi_c := 0.6 + \frac{f'_c}{15000\ psi} = 0.867 \qquad d_{bar} := D_{\#6}$

$$l_{dhook} := \max\left(6\ in, 8 \cdot d_{bar}, \left(\frac{\psi_e \cdot \psi_r \cdot \psi_o \cdot \psi_c \cdot f_y}{55 \cdot \lambda \cdot min\left(100\ psi, \sqrt{f'_c \cdot psi}\right)}\right) \cdot 1\ in \cdot \left(\frac{d_{bar}}{in}\right)^{1.5}\right) = 9.71\ in$$

Length of bars from the critical bending section: $\frac{(B - B_{wall})}{2} - c_c = 10\ in$

$$check := \text{if } l_{dhook} < \frac{(B - B_{wall})}{2}$$
$$\quad \text{“There is adequate room to develop the hooked bars”}$$
$$\text{else}$$
$$\quad \text{“There is inadequate room to develop the hooked bars”}$$

$$check = \text{“There is adequate room to develop the hooked bars”}$$

Design the Longitudinal Steel:

$$\rho_{min} := 0.0018$$

$$A_{sMin} := \rho_{min} \cdot B \cdot d = 1.017 \; in^2$$

Try 3 #6 Rebars:

$A_s := 3 \cdot A_{\#6} = 1.32 \; in^2$ 3 #6 bars is adequate

Bar spacing:

$$BarSpace_{max} := min(3 \cdot h, 18 \; in) = 18 \; in$$

$BarSpace := \frac{B - 2 \; c_c}{2} = 14 \; in$ use 12 in spacing to be conservative

Check to make sure footing is thick enough to accommodate development length:

$r := \frac{d_{bar} \cdot 6}{2} = 2.25 \; in$ (radius of dowel bar bend)

$$depth_{FlexuralHooks} := ((D_{\#6} \cdot 6) + (2 \cdot D_{\#6})) + D_{\#6} = 6.75 \; in$$

$H := d_{bar} + r = 3 \; in$ (distance required for hook)

$$h_{min} := depth_{FlexuralHooks} = 6.75 \; in$$

$$check := \text{if } h_{min} < h$$
$$\quad \text{“The footing thickness is adequate to accomodate the hooked bar”}$$
$$\text{else}$$
$$\quad \text{“The footing thickness is not adequate to accomodate the hooked bar”}$$
$$check = \text{“The footing thickness is adequate to accomodate the hooked bar”}$$

Check Bearing Capacity of Column at Base:

$$L_{wall} := 1 \; ft$$

$$A_1 := B_{wall} \cdot L_{wall} = 96 \; in^2$$

$$l := min(L, ((2 \cdot h) + B_{wall} + (2 \cdot h))) = 3.667 \; ft$$

$$A_2 := l^2 = 1936 \; in^2$$

$N_1 := 0.65 \cdot (0.85 \cdot f'_c \cdot A_1) = 212.16 \; kip$ $N_2 := 0.65 \cdot min\left(\left((0.85 \cdot f'_c \cdot A_1) \cdot \sqrt{\frac{A_2}{A_1}}\right), (2 \cdot 0.85 \cdot f'_c \cdot A_1)\right) = 424.32 \; kip$

$$\phi P_{BaseBearing} := \frac{min(N_1, N_2)}{1 \; ft} = 212.16 \; klf$$

$$check := \text{if } P_u < \phi P_{BaseBearing}$$
$$\quad \text{“The footing has adequate bearing strength at the base”}$$
$$\text{else}$$
$$\quad \text{“The footing has inadequate bearing strength at the base”}$$
$$check = \text{“The footing has adequate bearing strength at the base”}$$

Check 90 Degree Hooked Dowel Bars in Column:

Minimum Steel Ratio:

$$\rho_{min} := 0.005 \cdot A_1 \cdot \frac{1}{1 \cdot ft} = 0.48 \; \frac{in^2}{ft}$$

Try 2 #5 dowels $A_{\#5} := 0.310 \; in^2$

$A_{sDowel} := 2 \cdot A_{\#5} = 0.62 \; in^2$ (two dowels per 1 ft is adequate)

Development Length:

$$D_{\#5} := 0.625\ in$$

$$d_{dowel} := D_{\#5}$$

$$l_{dc} := \max\left(8\ in, \frac{0.02 \cdot f_y \cdot d_{dowel}}{\lambda \cdot min\left(100\ psi, \sqrt{f'_c \cdot psi}\right)}, \frac{0.0003}{psi} \cdot f_y \cdot d_{dowel}\right) = 11.859\ in$$

$l_{dc} := 12\ in$ (round up to get an appropriate constructible dimension)

Check to make sure footing is thick enough to accommodate development length:

$r := \dfrac{d_{dowel} \cdot 6}{2} = 1.875\ in$ (radius of dowel bar bend)

$$L_{extension} := 12 \cdot d_{dowel} = 7.5\ in$$

$H := d_{dowel} + r = 2.5\ in$ (distance required for hook)

$$h_{min} := l_{dc} + depth_{FlexuralHooks} + c_c = 21.75\ in$$

$check :=$ if $h_{min} \le h$
  "The footing thickness is adequate"
else
  "The footing thickness is inadequate"

$check =$ "The footing thickness is inadequate"

Check Rebars in Column:

$d_{bar} := D_{\#6}$ column bars are #6

Development Length:

$$l_{dcCol} := \max\left(8\ in, \frac{0.02 \cdot f_y \cdot d_{bar}}{\lambda \cdot min\left(100\ psi, \sqrt{f'_c \cdot psi}\right)}, \frac{0.0003}{psi} \cdot f_y \cdot d_{bar}\right) = 14.23\ in$$

Splice length for rebars in compression:

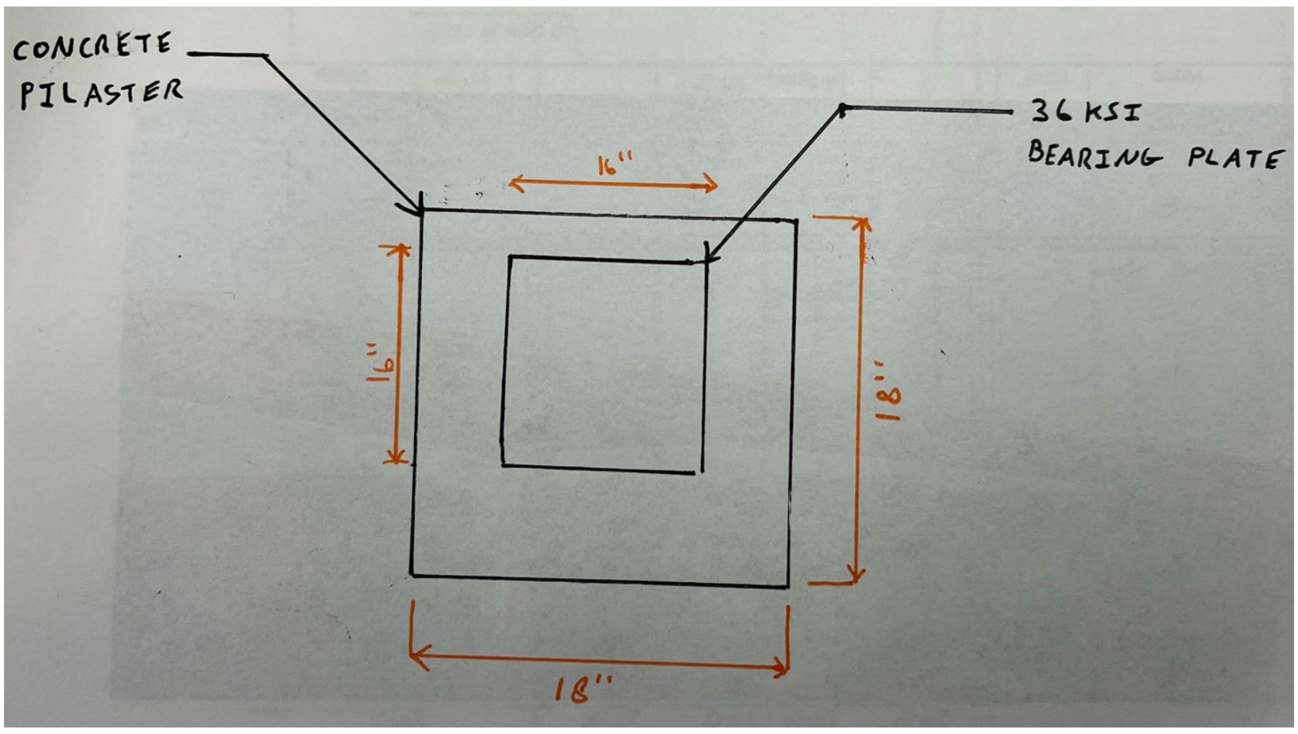
$$\alpha_s := 1$$

$l_{splice} := \max\left(12\ in, l_{dcCol}, \dfrac{0.0005}{psi} \cdot f_y \cdot d_{bar} \cdot \alpha_s\right) = 22.5\ in$ (round up to a 24in (2ft) splice)

Bearing Plate Design:

Largest column size is 14x159, will design all base plates for these dimensions for construction ease and as a conservative approach:

Try a 16"x16" plate



$N := 16\ in$ $B := 16\ in$ $P_{des1} := 33.152\ kip$ $P_{des2} := 53.704\ kip$ $P_{des3} := 38.304\ kip$ $f'_c := 4000\ psi$ $F_{Yplate} := 36\ ksi$

$A_1 := N \cdot B = 256\ in^2$ Area of the base plate

$e := 18\ in - 16\ in = 2\ in$

$A_2 := (N + 2 \cdot e) \cdot (B + 2 \cdot e) = 400\ in^2$ Area of the base plate support

Design bearing strength of concrete:

$$\phi_c P_p := 0.65 \cdot 0.85 \cdot f'_c \cdot A_1 \cdot min\left(2, \sqrt{\frac{A_2}{A_1}}\right) = 707.2\ kip$$

$P_u \le \phi_c P_p$, therefore a 16"x16" plate is sufficient

For W-14 x 159:

$d := 15\ in$ $b_f := 15.6\ in$

$m := \dfrac{N - 0.95 \cdot d}{2}$ $n := \dfrac{B - 0.80 \cdot b_f}{2}$

$$l := \max\left(m, n, \frac{1}{4} \cdot \sqrt{d \cdot b_f}\right) = 3.824\ in$$

Plate Thickness for Axial Design Load P1:

$t_p := l \cdot \sqrt{\dfrac{2 \cdot P_{des1}}{0.9 \cdot N \cdot B \cdot F_{Yplate}}} = 0.342\ in$ increase to the next eighth of an inch

$t_p := 0.375\ in$ Provide a 3/8 in thick plate

Plate Thickness for Axial Design Load P2:

$t_p := l \cdot \sqrt{\dfrac{2 \cdot P_{des2}}{0.9 \cdot N \cdot B \cdot F_{Yplate}}} = 0.435\ in$ increase to the next eighth of an inch

$t_p := 0.5\ in$ Provide a 1/2 in thick plate

Plate Thickness for Axial Design Load P3:

$t_p := l \cdot \sqrt{\dfrac{2 \cdot P_{des3}}{0.9 \cdot N \cdot B \cdot F_{Yplate}}} = 0.368\ in$ increase to the next eighth of an inch

$t_p := 0.375\ in$ Provide a 3/8 in thick plate

Appendix V: Building A & C: Parking Lot and Sidewalk Pavement Design

Table 8B-1.04: Pavement Thickness for Moderate Loads
(Parking areas, entrances, perimeter travel lanes, and frontage roads subject to 201 to 700 cars/day and/or 3 to 50 trucks/day or equivalent axle loads)

Subgrade CBR	Surface Material	On 12"of Prepared Subgrade		On 12" of Prepared Subgrade with Granular Subbase		
		Minimum	*Desirable*	*Thickness of Granular Subbase*	*Minimum*	*Desirable*
9	Rigid	5"	6"	4"	4"	5"
	Flexible	5"	6"	6"	4"	5"
6	Rigid	5"	6"	6"	4.5"	5"
	Flexible	6"	6"	8"	5"	5"
3	Rigid	5.5"	6"	6"	5"	5"
	Flexible	6"	7"	8"	6"	6"

The portions of the parking facility serving truck traffic such as entrances, perimeter travel lanes, trash dumpster sites, and delivery truck routes must be designed to accommodate heavier loads. The number, type, and weight of delivery vehicles can usually be predicted with a fair level of accuracy. With this information, ESAL values and pavement thicknesses can be determined using the methodology described in Chapter 5 - Roadway Design.

If the parking lot is to service an industrial area, such as a truck stop or manufacturing facility, the volume of truck traffic and the associated ESALs should be determined and an independent pavement thickness determination completed to ensure meeting the 20 year design life needs of the project.

The subgrade should be designed according to Section 6E-1. If soils tests are not available to determine the CBR value and uniformity of the soil (before and after construction), a CBR value of 3 and a non-uniform subgrade should be assumed.

PCC: 12" prepared subgrade, 6" subbase, and 5" pavement

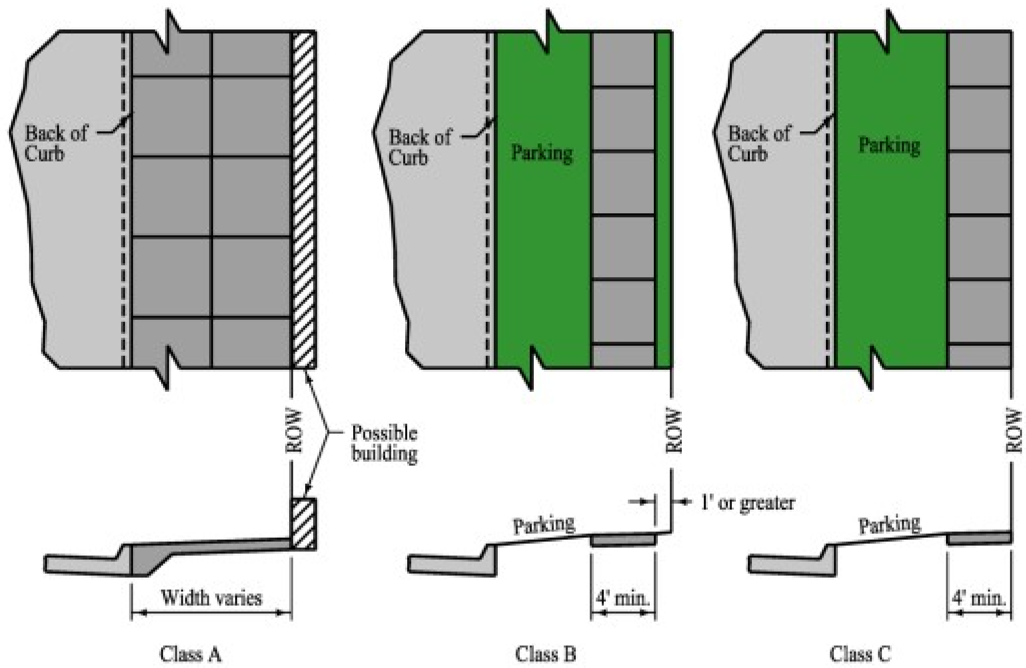
HMA: 12" prepared subgrade, 8"subbase, 6" pavement

Sidewalk:

1. **Sidewalk Thickness:** Sidewalks should be constructed of PCC with a minimum thickness of 4 inches. Where sidewalks cross driveways, the minimum thickness is 6 inches, or the thickness of the driveway, whichever is greater.

PCC: 6in due to sidewalk Class A assumption:

Figure 12A-1.01: Classes of Sidewalk



Appendix W: Building A: Wood Joist and Connection Design

Existing Floor Joist Check:

$$LL := 100\ psf \qquad DL := 25\ psf \qquad SL := 16\ psf$$

Assumed existing DL on roof as 20psf. Include additional 5 PSF for synthetic astro turf, curb surrounding astro turf, and furnishings.

3x16 Timber

$L := FIF(\text{“23'7”})$ $\qquad s := 16\ in$ $\qquad b := 2.5\ in$ $\qquad d := FIF(\text{“1'3–1/4”}) = 15.25\ in$

WOOD Design Use ASD

1. D
2. $D + L$
3. $D + (L_r \text{ or } S \text{ or } R)$
4. $D + 0.75L + 0.75(L_r \text{ or } S \text{ or } R)$
5. $D + (0.6W)$
6. $D + 0.75L + 0.75(0.6W) + 0.75(L_r \text{ or } S \text{ or } R)$
7. $0.6D + 0.6W$

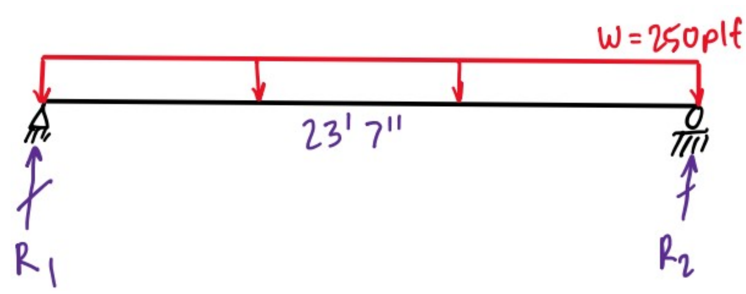
$$w_1 := DL \cdot s = 33.333\ plf$$

$$w_2 := (DL + LL) \cdot s = 166.667\ plf$$

$$w_3 := (DL + SL) \cdot s = 54.667\ plf$$

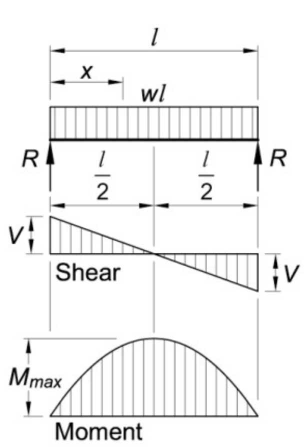
$$w_4 := (DL + 0.75 \cdot SL + 0.75 \cdot LL) \cdot s = 149.333\ plf$$

$$w := \max(w_1, w_2, w_3, w_4) = 166.667\ plf$$



Use AISC Beam tables

1. SIMPLE BEAM — UNIFORMLY DISTRIBUTED LOAD



Total Equiv. Uniform Load .... $= wl$

$R = V$ .... $= \frac{wl}{2}$

V_x .... $= w\left(\frac{l}{2} - x\right)$

M_{max} (at center) .... $= \frac{wl^2}{8}$

M_x .... $= \frac{wx}{2}(l - x)$

Δ_{max} (at center) .... $= \frac{5wl^4}{384EI}$

Δ_x .... $= \frac{wx}{24EI}(l^3 - 2lx^2 + x^3)$

$$R_1 := \frac{w \cdot L}{2} = 1.965\ kip \qquad R_2 := \frac{w \cdot L}{2} = 1.965\ kip$$

$$I_x := 738.9\ in^4 \qquad S_x := 96.9\ in^3 \qquad A := 38.13\ in^2 \quad E := 1800\ ksi$$

DOUGLAS FIR-LARCH										
Select Structural		1,500	1,000	180	625	1,700	1,900,000	690,000		
No. 1 & Btr		1,200	800	180	625	1,550	1,800,000	660,000		
No. 1	2" & wider	1,000	675	180	625	1,500	1,700,000	620,000		
No. 2		900	575	180	625	1,350	1,600,000	580,000		
No. 3		525	325	180	625	775	1,400,000	510,000	0.50	WCLIB WWPA
Stud	2" & wider	700	450	180	625	850	1,400,000	510,000		
Construction		1,000	650	180	625	1,650	1,500,000	550,000		
Standard	2" - 4" wide	575	375	180	625	1,400	1,400,000	510,000		
Utility		275	175	180	625	900	1,300,000	470,000		

Table 1B Section Properties of Standard Dressed (S4S) Sawn Lumber

Nominal Size b × d	Standard Dressed Size (S4S) b × d inches × inches	Area of Section A in.²	X-X Axis Section Modulus S_{xx} in.³	X-X Axis Moment of Inertia I_{xx} in.⁴	Y-Y Axis Section Modulus S_{yy} in.³	Y-Y Axis Moment of inertia I_{yy} in.⁴	Approximate weight in pounds per linear foot (lb./ft.) of piece when density of wood equals: 25 lb./ft.³	30 lb./ft.³	35 lb./ft.³	40 lb./ft.³	45 lb./ft.³	50 lb./ft.³
1 × 3	3/4 × 2-1/2	1.875	0.781	0.977	0.234	0.088	0.326	0.391	0.456	0.521	0.586	0.651
1 × 4	3/4 × 3-1/2	2.625	1.531	2.680	0.328	0.123	0.456	0.547	0.638	0.729	0.820	0.911
1 × 6	3/4 × 5-1/2	4.125	3.781	10.40	0.516	0.193	0.716	0.859	1.003	1.146	1.289	1.432
1 × 8	3/4 × 7-1/4	5.438	6.570	23.82	0.680	0.255	0.944	1.133	1.322	1.510	1.699	1.888
1 × 10	3/4 × 9-1/4	6.938	10.70	49.47	0.867	0.325	1.204	1.445	1.686	1.927	2.168	2.409
1 × 12	3/4 × 11-1/4	8.438	15.82	88.99	1.055	0.396	1.465	1.758	2.051	2.344	2.637	2.930
2 × 3	1-1/2 × 2-1/2	3.750	1.563	1.953	0.938	0.703	0.651	0.781	0.911	1.042	1.172	1.302
2 × 4	1-1/2 × 3-1/2	5.250	3.063	5.359	1.313	0.984	0.911	1.094	1.276	1.458	1.641	1.823
2 × 5	1-1/2 × 4-1/2	6.750	5.063	11.39	1.688	1.266	1.172	1.406	1.641	1.875	2.109	2.344
2 × 6	1-1/2 × 5-1/2	8.250	7.563	20.80	2.063	1.547	1.432	1.719	2.005	2.292	2.578	2.865
2 × 8	1-1/2 × 7-1/4	10.88	13.14	47.63	2.719	2.039	1.888	2.266	2.643	3.021	3.398	3.776
2 × 10	1-1/2 × 9-1/4	13.88	21.39	98.93	3.469	2.602	2.409	2.891	3.372	3.854	4.336	4.818
2 × 12	1-1/2 × 11-1/4	16.88	31.64	178.0	4.219	3.164	2.930	3.516	4.102	4.688	5.273	5.859
2 × 14	1-1/2 × 13-1/4	19.88	43.89	290.8	4.969	3.727	3.451	4.141	4.831	5.521	6.211	6.901
3 × 4	2-1/2 × 3-1/2	8.750	5.104	8.932	3.646	4.557	1.519	1.823	2.127	2.431	2.734	3.038
3 × 5	2-1/2 × 4-1/2	11.25	8.438	18.98	4.688	5.859	1.953	2.344	2.734	3.125	3.516	3.906
3 × 6	2-1/2 × 5-1/2	13.75	12.60	34.66	5.729	7.161	2.387	2.865	3.342	3.819	4.297	4.774
3 × 8	2-1/2 × 7-1/4	18.13	21.90	79.39	7.552	9.440	3.147	3.776	4.405	5.035	5.664	6.293
3 × 10	2-1/2 × 9-1/4	23.13	35.65	164.9	9.635	12.04	4.015	4.818	5.621	6.424	7.227	8.030
3 × 12	2-1/2 × 11-1/4	28.13	52.73	296.6	11.72	14.65	4.883	5.859	6.836	7.813	8.789	9.766
3 × 14	2-1/2 × 13-1/4	33.13	73.15	484.6	13.80	17.25	5.751	6.901	8.051	9.201	10.35	11.50
3 × 16	2-1/2 × 15-1/4	38.13	96.90	738.9	15.89	19.86	6.619	7.943	9.266	10.59	11.91	13.24
4 × 4	3-1/2 × 3-1/2	12.25	7.146	12.51	7.146	12.51	2.127	2.552	2.977	3.403	3.828	4.253
4 × 5	3-1/2 × 4-1/2	15.75	11.81	26.58	9.188	16.08	2.734	3.281	3.828	4.375	4.922	5.469
4 × 6	3-1/2 × 5-1/2	19.25	17.65	48.53	11.23	19.65	3.342	4.010	4.679	5.347	6.016	6.684
4 × 8	3-1/2 × 7-1/4	25.38	30.66	111.1	14.80	25.90	4.405	5.286	6.168	7.049	7.930	8.811
4 × 10	3-1/2 × 9-1/4	32.38	49.91	230.8	18.89	33.05	5.621	6.745	7.869	8.993	10.12	11.24
4 × 12	3-1/2 × 11-1/4	39.38	73.83	415.3	22.97	40.20	6.836	8.203	9.570	10.94	12.30	13.67
4 × 14	3-1/2 × 13-1/4	46.38	102.4	678.5	27.05	47.34	8.051	9.661	11.27	12.88	14.49	16.10
4 × 16	3-1/2 × 15-1/4	53.38	135.7	1034	31.14	54.49	9.266	11.12	12.97	14.83	16.68	18.53
5 × 5	4-1/2 × 4-1/2	20.25	15.19	34.17	15.19	34.17	3.516	4.219	4.922	5.625	6.328	7.031
6 × 6	5-1/2 × 5-1/2	30.25	27.73	76.26	27.73	76.26	5.252	6.302	7.352	8.403	9.453	10.50
6 × 8	5-1/2 × 7-1/2	41.25	51.56	193.4	37.81	104.0	7.161	8.594	10.03	11.46	12.89	14.32
6 × 10	5-1/2 × 9-1/2	52.25	82.73	393.0	47.90	131.7	9.071	10.89	12.70	14.51	16.33	18.14
6 × 12	5-1/2 × 11-1/2	63.25	121.2	697.1	57.98	159.4	10.98	13.18	15.37	17.57	19.77	21.96
6 × 14	5-1/2 × 13-1/2	74.25	167.1	1128	68.06	187.2	12.89	15.47	18.05	20.63	23.20	25.78
6 × 16	5-1/2 × 15-1/2	85.25	220.2	1707	78.15	214.9	14.80	17.76	20.72	23.68	26.64	29.60
6 × 18	5-1/2 × 17-1/2	96.25	280.7	2456	88.23	242.6	16.71	20.05	23.39	26.74	30.08	33.42
6 × 20	5-1/2 × 19-1/2	107.3	348.6	3398	98.31	270.4	18.62	22.34	26.07	29.79	33.52	37.24
6 × 22	5-1/2 × 21-1/2	118.3	423.7	4555	108.4	298.1	20.53	24.64	28.74	32.85	36.95	41.06
6 × 24	5-1/2 × 23-1/2	129.3	506.2	5948	118.5	325.8	22.44	26.93	31.41	35.90	40.39	44.88
8 × 8	7-1/2 × 7-1/2	56.25	70.31	263.7	70.31	263.7	9.766	11.72	13.67	15.63	17.58	19.53
8 × 10	7-1/2 × 9-1/2	71.25	112.8	535.9	89.06	334.0	12.37	14.84	17.32	19.79	22.27	24.74
8 × 12	7-1/2 × 11-1/2	86.25	165.3	950.5	107.8	404.3	14.97	17.97	20.96	23.96	26.95	29.95
8 × 14	7-1/2 × 13-1/2	101.3	227.8	1538	126.6	474.6	17.58	21.09	24.61	28.13	31.64	35.16
8 × 16	7-1/2 × 15-1/2	116.3	300.3	2327	145.3	544.9	20.18	24.22	28.26	32.29	36.33	40.36
8 × 18	7-1/2 × 17-1/2	131.3	382.8	3350	164.1	615.2	22.79	27.34	31.90	36.46	41.02	45.57
8 × 20	7-1/2 × 19-1/2	146.3	475.3	4634	182.8	685.5	25.39	30.47	35.55	40.63	45.70	50.78
8 × 22	7-1/2 × 21-1/2	161.3	577.8	6211	201.6	755.9	27.99	33.59	39.19	44.79	50.39	55.99
8 × 24	7-1/2 × 23-1/2	176.3	690.3	8111	220.3	826.2	30.60	36.72	42.84	48.96	55.08	61.20
10 × 10	9-1/2 × 9-1/2	90.25	142.9	678.8	142.9	678.8	15.67	18.80	21.94	25.07	28.20	31.34
10 × 12	9-1/2 × 11-1/2	109.3	209.4	1204	173.0	821.7	18.97	22.76	26.55	30.35	34.14	37.93
10 × 14	9-1/2 × 13-1/2	128.3	288.6	1948	203.1	964.5	22.27	26.72	31.17	35.63	40.08	44.53
10 × 16	9-1/2 × 15-1/2	147.3	380.4	2948	233.1	1107	25.56	30.68	35.79	40.90	46.02	51.13
10 × 18	9-1/2 × 17-1/2	166.3	484.9	4243	263.2	1250	28.86	34.64	40.41	46.18	51.95	57.73
10 × 20	9-1/2 × 19-1/2	185.3	602.1	5870	293.3	1393	32.16	38.59	45.03	51.46	57.89	64.32
10 × 22	9-1/2 × 21-1/2	204.3	731.9	7868	323.4	1536	35.46	42.55	49.64	56.74	63.83	70.92
10 × 24	9-1/2 × 23-1/2	223.3	874.4	10270	353.5	1679	38.76	46.51	54.26	62.01	69.77	77.52

Assume Douglas Fir-Larch-No. 1 and better grade at 3x16

$$M_u := \frac{w \cdot L^2}{8} = 11.587\ kip \cdot ft \qquad \Delta_u := \frac{5 \cdot w \cdot L^4}{384 \cdot E \cdot I_x} = 0.872\ in \qquad V_u := R_1 = 1.965\ kip$$

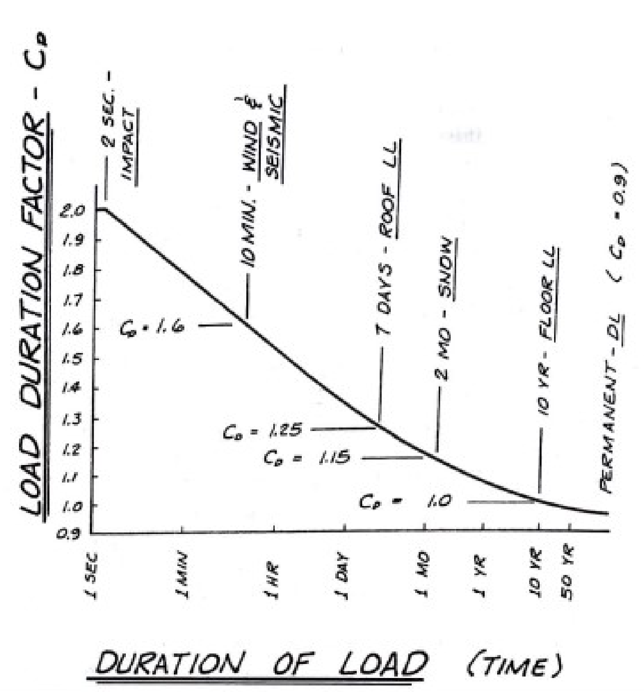
Bending:

Size Factors, C_F

Grades	Width (depth)	F_b Thickness (breadth) 2" & 3"	F_b Thickness (breadth) 4"	F_t	F_c
Select Structural, No.1 & Btr, No.1, No.2, No.3	2", 3", & 4"	1.5	1.5	1.5	1.15
	5"	1.4	1.4	1.4	1.1
	6"	1.3	1.3	1.3	1.1
	8"	1.2	1.3	1.2	1.05
	10"	1.1	1.2	1.1	1.0
	12"	1.0	1.1	1.0	1.0
	14" & wider	0.9	1.0	0.9	0.9
Stud	2", 3", & 4"	1.1	1.1	1.1	1.05
	5" & 6"	1.0	1.0	1.0	1.0
	8" & wider	Use No.3 Grade tabulated design values and size factors			
Construction, Standard	2", 3", & 4"	1.0	1.0	1.0	1.0
Utility	4"	1.0	1.0	1.0	1.0
	2" & 3"	0.4	—	0.4	0.6

$$f_b := \frac{M_u}{S_x} = 1434.916\ psi$$

$$F_b := 1200\ psi$$



$C_D := 1.25$ Roof Live Load rated at 7 days so use Roof LL 1.25 sect 4.15

$C_M := 1.0$

$C_F := 0.9$

$C_t := 1.0$

$C_L := 1.0$ Assume Continuous Lateral Bracing With Sheathing

$C_i := 1.0$ for Timbers

$C_r := 1.15$ Considered Repetitive Member configuration

$$F_b' := F_b \cdot C_D \cdot C_M \cdot C_t \cdot C_L \cdot C_F \cdot C_i \cdot C_r = 1552.5 \ psi$$

$f_b \le F_b' = 1$ OK

Shear:

$$F_v := 180 \ psi$$

$$f_v := 1.5 \cdot \frac{V_u}{A} = 77.312 \ psi$$

$$F_v' := F_v \cdot C_D \cdot C_M \cdot C_t \cdot C_i = 225 \ psi$$

$f_v \le F_v' = 1$ OK

Bearing:
Assume roof joists are connected using strong tie joist hangers in plane with the connecting beam. Therefore there is no bearing from the beam to joist connection.

Deflection:

$$w_{LL} := LL \cdot s = 133.333 \ plf \qquad w_{DL} := DL \cdot s = 33.333 \ plf$$

$$\delta_{ST} := \frac{5 \cdot (0.5 \cdot w_{LL}) \cdot L^4}{384 \cdot E \cdot I_x} = 0.349 \ in \qquad \delta_{LT} := \frac{5 \cdot (0.5 \cdot w_{LL} + w_{DL}) \cdot L^4}{384 \cdot E \cdot I_x} = 0.523 \ in$$

$$\delta_{Total} := 1.5 \cdot \delta_{LT} + \delta_{ST} = 1.134 \ in$$

$$\Delta_{Total} := \frac{L}{240} = 1.179 \ in$$

$\delta_{Total} \le \Delta_{Total} = 1$ OK

$$\frac{L}{360} = 0.786 \ in \qquad \Delta_u := \frac{5 \cdot w_1 \cdot L^4}{384 \cdot E \cdot I_x} = 0.174 \ in$$ DL only

OK

$$\frac{L}{240} = 1.179 \ in \qquad \Delta_u := \frac{5 \cdot w_2 \cdot L^4}{384 \cdot E \cdot I_x} = 0.872 \ in$$ DL+LL

Connection Design:

For 3x16 wood timbers

$W := FIF(\text{“0'2-9/16”}) = 2.563\ in$

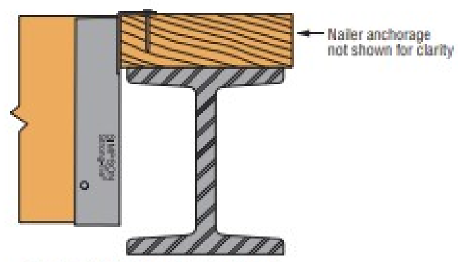
Purlin Top-Flange Hangers (cont.)

The table indicates the maximum allowable loads for WP, HWP and HWPH hangers used on wood nailers. Nailers are wood members attached to the top of a steel I-beam, concrete or masonry wall.

Nailer Table

Model	Nailer	Top Flange Nailing (in.)	Uplift[1] (160)	Allowable Down Loads		
				DF/SP	SPF/HF	LSL
WP	2x	(2) 0.148 x 1½	—	2,525	2,500	3,375
	(2) 2x	(2) 0.148 x 3	—	3,255	3,255	—
	3x	(2) 0.162 x 2½	—	3,000	2,510	3,375
	4x	(2) 0.148 x 3	—	3,255	3,255	—
HWP	(2) 2x	(3) 0.148 x 3	710	4,615	—	—
	3x	(3) 0.162 x 2½	970	4,615	—	—
	4x	(3) 0.162 x 2½	1,535	5,145	—	—
HWPH	(2) 2x	(4) 0.162 x 2½	710	6,400	—	—
	3x	(4) 0.162 x 2½	970	6,470	—	—
	4x	(4) 0.162 x 3½	1,550	6,470	—	—

1. Attachment of nailer to supporting member is the responsibility of the Designer.



Installation on Wood Nailer

Various Header Applications

Model	Joist (in.)		Fasteners (in.)			Allowable Loads Header Type							Code Ref.
	Width	Height	Top	Face	Joist	Uplift (160)	LVL	PSL	LSL	DF/SP	SPF/HF	I-Joist	
WP	1½ to 5⅝	5⅜ to 30	(2) 0.148 x 1½	—	(2) 0.148 x 1½	—	2,865	3,250	—	2,500	2,000	2,030	—
	2½ to 5⅝	5⅜ to 30	(2) 0.148 x 3	—	(2) 0.148 x 1½	—	2,525	3,250	3,650	3,255	2,525	—	IBC, FL, LA
	3½ to 5⅝	5⅜ to 30	(2) 0.162 x 3½	—	(2) 0.148 x 1½	—	3,635	3,320	3,650	3,255	2,600	—	
HWP	1½ to 7	6 to 15⅝	(3) 0.162 x 3½	(6) 0.162 x 3½	(10) 0.148 x 1½	1,535	3,995	4,500	4,350	3,955	3,955	—	
	1½ to 7	15¾ to 28	(3) 0.162 x 3½	(6) 0.162 x 3½	(12) 0.148 x 1½	1,570	3,995	4,500	4,350	3,955	3,955	—	
HWPH	2½ to 7	6 to 15⅝	(4) 0.162 x 3½	(8) 0.162 x 3½	(10) 0.148 x 1½	1,685	6,595	7,025	5,450	5,920	4,740	—	
	2½ to 7	15¾ to 32	(4) 0.162 x 3½	(8) 0.162 x 3½	(12) 0.148 x 1½	2,075	6,595	7,025	5,450	5,920	4,740	—	

1. Code values are based on DF/SP header species.
2. Uplift loads have been increased for wind or earthquake loading with no further increase allowed. Reduce where other loads govern.
3. For hanger heights exceeding the joist height, the allowable load is 0.50 of the table load.
4. HWP widths greater than 5⅝" are not included in the code report.
5. **Fasteners:** Nail dimensions in the table are listed diameter by length. See p. 21 for fastener information.

Include width of 3" and height/depth of 16"

$P_{allow} := 2500\ lbf$

$R_1 = 1.965\ kip$

R1 < both of the above P so OK