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FINAL DELIVERABLE

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MAQUOKETA
GREEN SPACE REDEVELOPMENT

DESIGN REPORT | May 5, 2023

Prepared for: The city of Maquoketa

Prepared By:

Jamie Trentz, Jackson Stephens, Jacob Murphy, & Amanda Guerra

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<u>SECTION I</u>

Executive Summary

The following design report was submitted for the City of Maquoketa's Green Space Redevelopment project. This project aimed to transform the northeast corner of South Main Street and East Pleasant Street in Maquoketa, IA, into a place for the community to gather for concerts and other events. Working on behalf of the University of Iowa Department of Civil and Environmental Engineering are fourth year civil engineering students Jamie Trentz, Jackson Stephens, Jacob Murphy, and Amanda Guerra. Throughout our time at the University, we have studied both the structural and architectural aspects of civil engineering. In addition to our studies, our team has gained an assortment of experience working with major engineering companies like Shive-Hattery Architecture and Engineering, Kimley-Horn, and Sevan Multi-Site Solutions. We have combined our experiences and worked alongside professional civil, environmental, and structural engineers to design a new green space for the city of Maquoketa that will bring more activity to its downtown.

Figure 1.1. Southwest view of the existing site during a typical event
Taken by Sarah Jones, 2022. Concert Showing sun and shade from west side Buildings.
Used with permission.

The existing green space is already home to a variety of events with turnouts reaching anywhere from 50 to 2,000 people. During these events, a temporary stage platform is set up and the community gathers around the performer, sitting on blankets, lawn chairs, and surrounding picnic benches.

For larger events at this space, the city must order portable restrooms. Due to the frequency of events held in the space, this rental is an unnecessary reoccurring cost.

Maquoketa aims to take this empty plot of land and transform it into a more defined space. We have implemented a restroom structure and a permanent stage into our design that will match the downtown's red brick aesthetic while also bringing in a contemporary aesthetic through the incorporation of exposed steel beams.

Summer performances are typically scheduled in the space during late afternoons and evenings, which subjects the audience to harsh summer sun from the west. Consequently, crowds tend to gather in the small strip of shade provided from the buildings across South Main Street. This is a major issue as it creates a lot of empty space between the audience and performer, making the overall experience for both parties less enjoyable. It is also important to note that the space exists due to a fire that burned down several storefronts located on the site. Therefore, the city would like to have this history recognized in the design. Our team has proposed painting a mural along the south side of the restroom structure that commemorates the storefronts that used to belong on this corner. Not only does this satisfy the need for historic remembrance, but it creates an opportunity for more community engagement. We strongly suggest Maquoketa turns to its own residents for mural ideas; creating a historical mural contest would get the whole community invested, informed, and involved in the project.

Storage is another substantial issue for the site and items related to the entertainment events held here. We have designated 450-square feet of the restroom building to storage.

Having a permanent stage, restroom structure, adequate shade, and storage are all high priority requests. Just as important is incorporating lighting and a sidewalk network to provide more accessibility and safety at night. Thus, we have incorporated lighting bollards into our design to provide adequate lighting for the sidewalk connecting the existing parking lot to the stage, restroom, and surrounding sidewalks. More features requested but not considered top priority were solar panels, a concession stand, and a gathering space.

There was not a budget given for the Maquoketa Green Space Redevelopment. Instead, the city of Maquoketa wanted to take a rendering-based approach where they could present the designs and gather appropriate funding. For that reason, we split the project into three phases in order to spread the overall cost into smaller, more manageable increments. This also provides down time between construction to apply for more funding to continue construction of the subsequent phase. We have designed the phasing so that Phase 1 contains all top priority features and can stand alone if more funding isn't available for the project. In the case that there is sufficient funding, Phases 2 and 3 can be implemented as additions to the previous phases. These phases integrate the more expensive secondary features into the design, which will in turn elevate the overall atmosphere of the site.

Phase 1 is the foundation for both phases that follow. This initial design contains the essential items requested by the client: a permanent stage on the east side of the parcel, a concrete pad in front of the stage, a public restroom at the south end of the parcel with built in storage, shade from the west, lighting for the site, a sidewalk that connects the parking lot to the restrooms, water fountains, an informational kiosk, and landscaping. This phase was designed to accommodate all future phases, meaning all structural elements will support the loads of all future phases to avoid having to resize members or perform unnecessary and costly demolitions.

Phase 2 contains all elements from its previous phase, with the addition of an ADA compliant ramp and railing system to transform the roof of the restroom structure into an usable space.

Figure 1.2. Proposed green space for Phase Three

Phase 3 is comprised of all previously mentioned elements, along with the addition of a permanent steel roof covering, shading the entirety of the roof. This covering is designed to be able to withstand the load of an array of solar panels that span the length of the roof covering. Additionally, a concession stand is integrated into the storage room through the addition of a partition wall dividing the spaces.

The overall total cost for design, administration, and construction of the project if built straight to completion (Phase 3) has been estimated as $1,153,500. The estimated cost break-down for Phases 1, 2, and 3 are $774,500, $276,000, and $103,000, respectively.

The deliverables of the Maquoketa Green Space Redevelopment project include a written report, construction drawings of the proposed site and structures, project poster with renderings of the site, as well as a phasing plan and a construction cost estimate breakdown. Although we have designed Phase 1 to be able to exist on its own, our team strongly encourages the city of Maquoketa to follow through with the construction of both subsequent phases. In the end, the proposed structures will not only complement the aesthetic of downtown, but they will also complement the community-based nature of the city of Maquoketa. The roof overlooking the green space will likely bring new interest in the space that might attract families to gather even on days where there are no events planned for the space. We believe that by generating this excitement, more people will be drawn into the green space. This will further build Maquoketa's community feel, educate people on the site's history, and increase overall event attendance.

SECTION II

Organization Qualifications and Experience

Organization Location and Contact Information

103 S. Capitol St. Iowa City, IA 52242

Project Manager: Jamie Trentz

Organization and Design Team Description

The team is a group of students who participated in the senior design class required of all final semester students within the Department of Civil and Environmental Engineering at the University of Iowa. Jamie Trentz is a Civil Engineering major with a focus in structures. He was the project manager and ensured all deadlines were met and that we were working effectively during all portions of the project. Jacob Murphy is a Civil Engineering major with a focus in structures. He was the text editor and revised documents so they could be read with ease. Amanda Guerra is a Civil Engineering major with a focus in architecture. She was the graphics editor and performed renderings for designs. She also assisted in the creation of the graphics used in reports and presentations. Jackson Stephens is a Civil Engineering major with a focus in structures. His role was technology support and he created and edited presentations that can be easily followed and communicated the proper information. All members contributed to the calculation methods needed for all structures. All members also used knowledge learned from previous design classes to ensure all building and city codes were followed and were documented throughout the process.

- **Description of Experience with Similar Projects**

Jamie Trentz has taken courses at the University of Iowa such as Principles of Structures, Civil Engineering Materials, Design of Wood Structures, Foundations of Structures, Design of Steel Structures, and Design Optimization. Jamie has also worked as a structural engineering intern with Shive-Hattery Engineering and Architecture for two summers. During his time there, Jamie has helped produce structural construction documents, using Revit for industrial platforms, new buildings, building additions, and a pedestrian bridge. Jamie also worked in the structural reference detail library, refining the details used across all the Shive-Hattery locations. He also has experience in structural analysis programs and designing steel connections.

Jacob Murphy has had internships with the Iowa DOT and Shive-Hattery Engineering and Architecture. The Iowa DOT internship gave Jacob the skills to navigate through specification books, interpret engineering plans, and document progress throughout a project. The Shive-Hattery internship gave him the experience of performing structural calculations for buildings and platform systems in RISA 3D. Jacob also evaluated results by performing hand calculations and produced structural drawings in Revit. Many courses at the University of Iowa prepared Jacob for those internships and this project. Some of those courses were Design of Steel Structures, Design of Concrete Structures, Foundations of Structures, Civil Engineering Materials, and Principles of Transportation Engineering.

Amanda Guerra has taken structural courses as well as design-intensive courses at the University of Iowa. Amanda's design courses allowed her to familiarize herself with the software package Autodesk 3dsMax. Using this program, she was able to design and produce high quality renderings of a library, pavilion, and a bus stop shelter. This past summer, Amanda worked as a solar engineering intern with Kimley-Horn. Through this internship, she gained experience drafting solar field layouts and preparing construction documents using AutoCAD Civil3D. To accomplish these tasks, Amanda had to read and follow county the solar zoning ordinances, and import utility line, road network, property line, FEMA, and wetland data into Civil 3D drawings.

Jackson Stephens has taken various structural engineering courses at the University of Iowa to prepare him for this project. Some of those include Design of Steel Structures, Design of Wood Structures, Design of Concrete Structures, Foundations of Structures, Civil Engineering Materials, and Resilient Infrastructure. These classes helped Jackson to learn how to use design manuals and software like Revit and Civil3D to assist him in the project. For the latter half of 2022, Jackson worked as a project management intern with Sevan Multi-Site Solutions. He worked with the Walgreens team and assisted in the construction process of the parking lot, roof, and LED efficiency teams. This exposed him to the bidding process and overall budgeting of projects. He became familiar with Smartsheet, PowerPoint, and Excel during his time.

SECTION III

Design Services

1. Project Scope

Our design will redevelop the Maquoketa Green Space into a place that can host a variety of events in addition to daily use. Some of these events include the Summer Concert Series, Maqtoberfest, Fireball, Art in the Park, and the Farmer's Market. The city of Maquoketa wishes to use this space to further expand community involvement. With the inclusion of the following aspects, the city hopes the structures provide a strong foundation for overall growth in the community. At the utmost importance, these gatherings require a permanent stage that accommodates lighting and sound equipment, public restrooms, storage, shade, site lighting, sidewalks, as well as landscaping. Additional features to be included in budget permitting design renders include a water fountain with bottle filling ability, a dog-level water fountain, an informational kiosk, a historical marker, and low maintenance plantings. All high priority and inexpensive features were incorporated in Phase One while the expensive secondary features were incorporated into the other two phases accordingly.

For the three phases, the following items will be presented: project report, construction drawings, 3D rendering of the site, and a design summary poster.

2. Work Plan

The schedule for our design team is shown in Figures 3.1-3.7.

Maquoketa Greenspace Redevelopment

Jan 18 - Jan 21				Jan 22 - Jan 27						
18	19	20	21	22	23	24	25	26	27	28
W	T	F	S	S	M	T	W	T	F	S

Tasks	Start	End
Task 1: Project Kickoff and Data Collection	18-Jan	27-Jan
Design Teams' Role Designations and Understanding of Project (All)	18-Jan	20-Jan
Initial Client Contact and Meeting Setup (Jamie)	19-Jan	24-Jan
Data Collection of Similar Previous Projects (All)	20-Jan	25-Jan
Introductory Meeting and Gathering Client Input (All)	25-Jan	27-Jan

Figure 3.1. Project kickoff and data collection Gantt chart

Jan 22 - Jan 27							Jan 29 - Feb 4							Feb 5 - Feb 11						
22	23	24	25	26	27	28	29	30	31	1	2	3	4	5	6	7	8	9	10	11
S	M	T	W	T	F	S	S	M	T	W	T	F	S	S	M	T	W	T	F	S

Tasks	Start	End
Task 2: Field Assessment and Initial Concept Development	28-Jan	10-Feb
Site Visit and Data Collection of Site (All)	28-Jan	28-Jan
Draft Initial Proposal Report (All)	28-Jan	30-Jan
Finalize Initial Proposal Report (All)	31-Jan	1-Feb
Draft and Finalize Initial Proposal Slides (All)	1-Feb	3-Feb
Rehearse Initial Proposal Presentation (All)	4-Feb	5-Feb
Initial Proposal Presentation (All)	6-Feb	10-Feb

Figure 3.2. Field assessment and initial concept development Gantt chart

Feb 5 - Feb 11							Feb 12 - Feb 18							Feb 19 - Feb 25						
5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	20	21	22	23	24	25
S	M	T	W	T	F	S	S	M	T	W	T	F	S	S	M	T	W	T	F	S

Tasks	Start	End
Task 3: Design Concept Development	11-Feb	22-Feb
Generate Guiding Conceptual Designs (All)	11-Feb	17-Feb
Narrow Down Conceptual Designs Based on Field and Client Criteria (All)	14-Feb	18-Feb
Develop Drawings/Sketches to Illustrate Design Options (All)	17-Feb	21-Feb
Discuss Advantages and Disadvantages of Each Design (All)	19-Feb	21-Feb
Client Meeting to Discuss and Choose 3 Levels of Design Options (All)	22-Feb	22-Feb

Figure 3.3. Design concept development Gantt chart

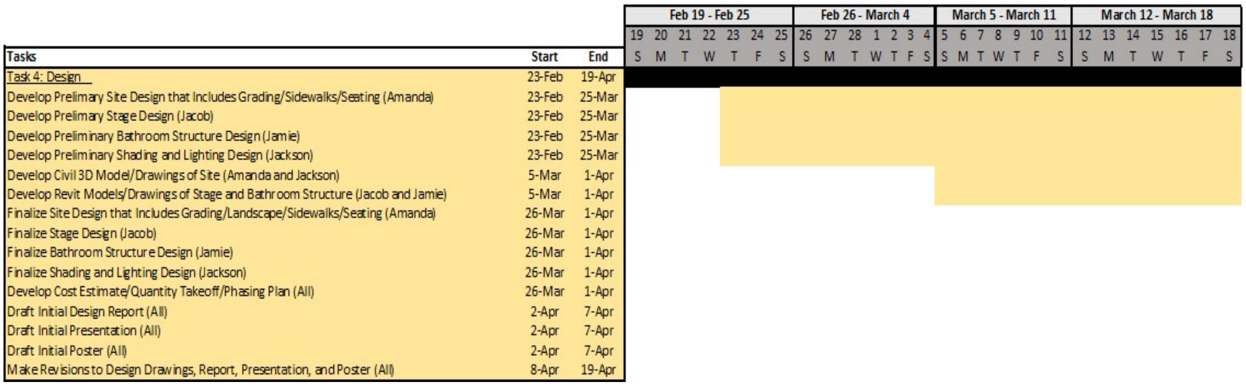

Figure 3.4. Design Gantt chart part 1

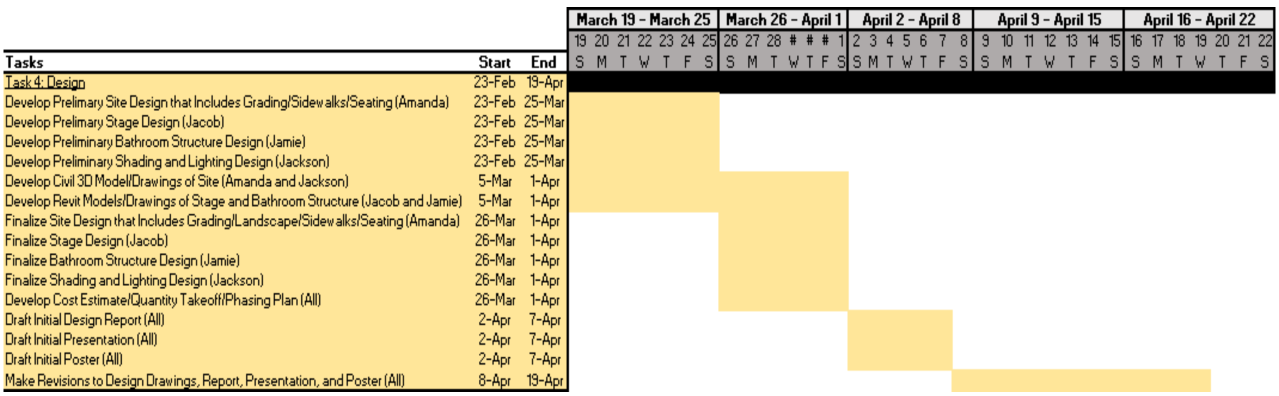

Figure 3.5. Design Gantt chart part 2

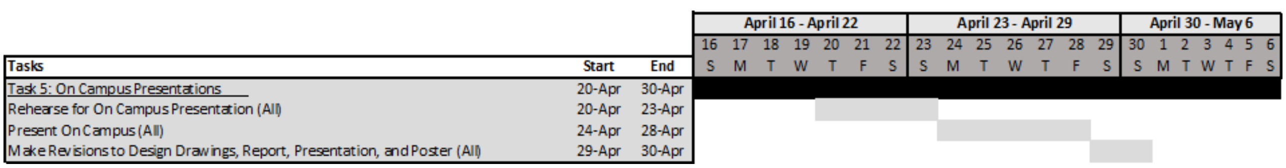

Figure 3.6. On campus presentation Gantt chart

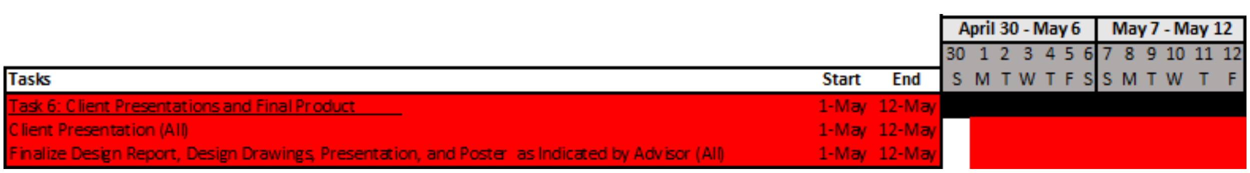

Figure 3.7. Client presentation and final product Gantt chart

SECTION IV

Constraints, Challenges, and Impacts

1. Constraints

The Maquoketa Green Space Redevelopment needed to match the existing aesthetic of the downtown area. The buildings in the area are built out of a mixture of brick, concrete, and limestone. Wood was avoided for construction of the stage or bathroom in order to decrease the amount of eventual maintenance. The stage needed to stand three-feet or higher off the ground to allow for a good line of sight from the audience. The stage was designed to maintain a line of sight from Main Street due to the events that flow over into the downtown area. This restricted stage placements to either the north or east side of the parcel. The existing north sidewalk must remain. Shade must be provided from the west due to events occurring mostly in the late afternoon. If any seating was provided, it must also be removable. Lastly, the project was limited to the current parcel area; no land acquisitions were possible. Budget was not a constraint for this project.

2. Challenges

The Maquoketa Green Space only has five inches of topsoil due to existing basements being filled with limestone. This creates challenges to either plantings or foundation installation. Foundations need to be installed by excavating existing limestone and replacing it with soil fill. Another challenge was creating a line of sight for the crowd that allows people in the back to see the stage with ease but also allows the people towards the front to not strain their necks.

3. Societal Impact within the Community and/or State of Iowa

The societal impact of this project was notable. Activities that will be held on this site will greatly impact the City of Maquoketa for the better. The overall look of the site will be enhanced and will allow for the history and aesthetic of the town to be improved. Events will bring more people from the area to Maquoketa. This influx of visitors will allow local businesses to prosper from more people. The new structures and new layout of the site will allow for more daily activities to be held on the site. This allows people to be outside more during the warm weather, which in turn leads to better health, wellness, and happiness. The additions will provide greater accessibility, lowering the risk of health and safety concerns. New bathrooms will allows residents to have cleaner and more accessible locations for their use, improving community needs and sanitation. This addition means there is no need for public portable restrooms, lowering community costs for each event and allowing for the downtown to have a more natural feel and appeal.

SECTION V

Phasing of Alternatives

We are proposing three designs for the Green Space Redevelopment Project in the form of three phases. Phase One incorporates all important baseline features the City of Maquoketa expects to be included in every design alternative. The subsequent two phases are meant to be constructed as additions to the previous phase. Phases Two and Three both incorporate secondary features the city requested but noted were not as high priority as the features in Phase One.

Using phasing reduces the initial cost by not immediately incorporating expensive lower priority features. All phases were designed to be able to withstand the load of the subsequent phase so there will be no major demolition or replacement of structural members needed when construction of the next phase begins. Every phase can be constructed at Maquoketa's discretion, which provides extra time during the funding process for each update.

1. Phase One - Site Layout and Proposed Structures

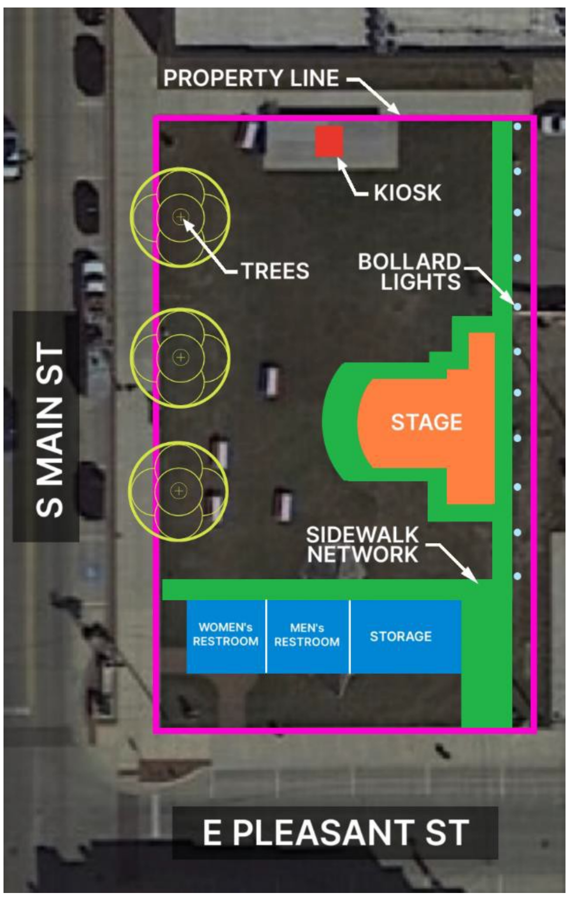

Figure 5.1. Phase One schematic design

SITE LAYOUT:

Phase One of the Green Space Redevelopment is displayed in Figure 5.1. This schematic outlines the location of the proposed structures and sidewalk network on the parcel. The green path is the proposed sidewalk that connects the restroom entrance, storage entrance, stage, and northeast parking lot to the existing surrounding sidewalks.

The permanent stage, shown in orange, has been placed on the east side of the parcel and faces west. This stage orientation proved to be the most optimal arrangement as it keeps the audience in closer proximity to the performers, while simultaneously providing more lines of sight along the length of the parcel. Occasionally, South Main Street gets blocked off for larger events, so having the stage face west results in optimal performance visibility.

In contrast, East Pleasant Street does not get shut down; therefore, we believe it was most appropriate to locate the restroom structure (in blue) on the south end, closer to existing water hookups.

Also depicted in the graphic above is the relocation of the informational kiosk as well as the locations of proposed trees that will provide shade on the site. Additionally, the picnic benches and sculpture shown in the aerial are subject to relocation prior to construction.

Figure 5.2. Stage design for all phases

STAGE DESIGN:

As shown in Figure 5.2, our team has proposed a permanent stage composed mainly of CMU blocks with a red brick façade. The brick in combination with the exposed steel beams, metal deck roof, and concrete slab performance area all work together to compliment the aesthetic of Maquoketa's downtown architecture. Additionally, this model features an ADA compliant ramp, as well as a staircase leading up to the stage. Both entrance points have north access locations. This configuration provides the most direct route to the parking lot, which is essential for loading and unloading heavy equipment. Other features of the design include approximately 735 square-feet of performance space, three box trusses for lighting, and sound equipment. Due to the green space's vital need for a permanent performance area, the stage was designed to its fullest potential in Phase One and will remain unchanged in the subsequent phases.

RESTROOM STRUCTURE DESIGN:

Figure 5.3. Phase One restroom building design – view from S Main St

Figure 5.4. Phase One restroom building floor plan

Like our proposed stage design, the restroom (pictured in Figure 13) is comprised of a masonry interior with a brick façade exterior. The front and back wall feature six columns that aesthetically add a sense of depth to the building. More importantly, they are required to be able to structurally withstand the loads of the following phases. The garage door entrance to the storage room, as well as the entrance to the men and women's restrooms, are located on the north facing wall to allow for the most direct access to people occupying the green space.
The east wall, (left-most wall in Figure 5.4), features a secondary entrance to the storage area and a concession garage window. Although these features are not essential for Phase One, it is more cost efficient to incorporate them prematurely rather than performing a demolition in later phases when these features become a necessity. Furthermore, assuming there will be longer bathroom lines during bigger events, the design features a four-foot front overhang to protect from rain and provide additional shading.

From left to right, the floorplan in Figure 5.4 exhibits the 450-square foot storage area, men's restroom, and women's restroom. Both restrooms are equipped with two sinks, two hand dryers, and two ADA compliant stalls. Additionally, the women's restroom features four standard stalls, while the men's features one standard stall and four urinals. Stall amounts were calculated assuming an average event attendance of 390 people.

2. Phase Two - Site Layout and Proposed Structures

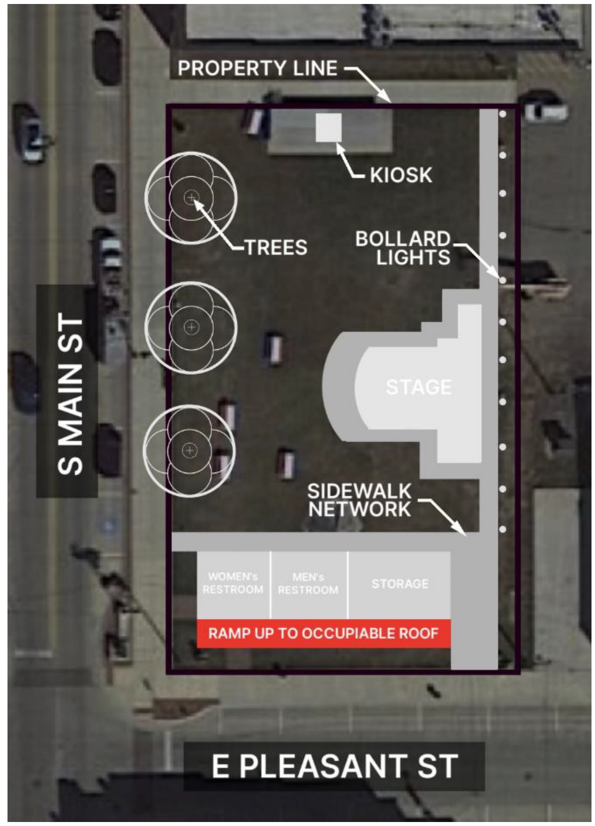

Figure 5.5. Phase two schematic design

SITE LAYOUT:
For this phase, the sidewalk network and permanent stage location remain the same as Phase One. The restroom structure, however, increases in width by 13-feet to accommodate a ramp into the design.

STAGE DESIGN:
Remains the same as shown in Phase One (See Figure 5.2).

RESTROOM STRUCTURE DESIGN:
To further create more lines of sight to the stage, we have proposed that Phase Two will include an occupiable roof. This is possible through the addition of an ADA compliant ramp and railing system. The ramp entrance was strategically placed facing east where food trucks tend to park during major green space events. This allows for fast access to rooftop picnic bench seating, subsequently minimizing overall foot traffic on the site. Since shading is a major issue for the site, our team strongly encourages the use of picnic table umbrellas as a cost efficient and temporary solution to provide shade during the transition period between Phase Two and Three.

Figure 5.6. Phase Two ramp and occupiable roof design

3. Phase Three - Site Layout and Proposed Structures

SITE LAYOUT:
All major changes in this phase happen within the bounds of the previous phase's site layout. Therefore, the Phase Three layout remains the same with the only exception being the division of storage area into two separate spaces. Figure 5.7 exhibits this division.

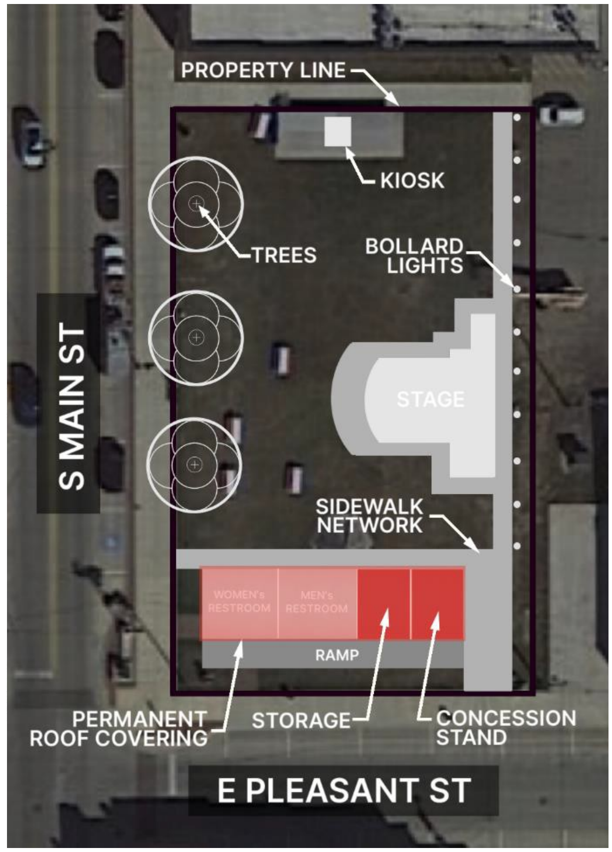

Figure 5.7. Phase Three schematic design

STAGE DESIGN:
Remains the same as shown in Phase One (See Figure 5.2).

RESTROOM STRUCTURE DESIGN:
The main upgrade for this phase is the construction of a permanent metal deck roof for the occupiable roof that is held up by exposed steel beams. Overtop, we are proposing solar panels along the length of the permanent roof. The east to west panel placement allows them to experience the longest duration of direct sunlight possible.

On the interior, a partition wall is proposed to divide the storage room to accommodate a concession stand on the far east end of the building. Of course, this room will also have water and electricity hookups readily available for all necessary appliances.


Figure 5.8. Phase Three permanent roof restroom structure

All things considered, although each phase can stand without its subsequent phases, our team encourages following through to the construction of Phase Three. Due to this phase's incorporation of solar energy, the design will require a significant initial capital expenditure but will result in reduced costs in terms of energy and maintenance expenses over time.

SECTION VI
Final Design Details

SITE DESIGN

The design was based off the site's most up to date Lidar scans and was done using Civil3D. Both the bathroom and stage structures were set at an even elevation of 707.5 ft. This allowed for the most effective grading and allowed the sidewalk network to best meet ADA requirements. Grading goes from west to east, with the stage being used as a divider, grading away from the stage to reduce ponding. This follows the previous grading of the site, with most of the flow heading to the northeast and southeast corners of the parcel. The sidewalk network follows the perimeter of the stage and restroom structures, with a larger pad being inserted at the front and east of the stage and another at the west end of the bathroom structure. Connections were made to existing sidewalk networks on the southwest, southeast, and northeast ends of the parcel. Fill will be needed for the site to accommodate the raised elevation of the stage. Cut will be needed for the bathroom structure. The existing structure and pavement on the south side of the parcel will be removed, along with the existing kiosk being moved to the north end concrete pad. An overall site layout plan is located on sheet C2.01.

STAGE DESIGN

Design Loads:
Wind and roof snow loads were calculated using ASCE 7-22. The wind loads were calculated assuming the Main Wind Force Resisting System (MWFRS) and using the Directional Procedure (Minimum Designs, Chapter 26 and 27). The snow loads were using the horizontal projection of the roof. The ground snow load was found to be 25 psf. and the sloped roof snow load on the roof was found (Minimum Designs, Chapter 7). All final design loads were calculated using Load and Resistance Factor equations (Minimum Designs, Chapter 2).

Layout:
The layout of the stage was thoroughly analyzed. Shading, line-of-sight, dimension constraints, and access were all factors for the layout of the stage. The stage is placed on the east side of the parcel so that the audience does not get blinded or overheated by the setting sun. The stage was elevated about 4 feet off grade due to no elevation of seating. The stage was also placed on the east side so people can watch performances from Main St. The required dimensions of the stage were 24 feet wide and 20 feet deep. An ADA ramp was required for access to the stage, and it provides an easy way to roll equipment onto the stage. The ramp has a slope of 1/12 which is the maximum allowed and the width is 5 feet (ADA). The ramp is orientated to the north for easy access from the parking lot.

Roof:
The roof was designed using a Nucor Vulcraft metal deck. The use of wood and shingle materials were not considered in the roof design due to the required maintenance over the years. Concrete on top of the metal deck was not needed for strength or added weight to

resist uplift, but this alternative was considered. The metal deck was required to hold a gravity load of 29.21 psf., an uplift load of 9.605 psf., and a span length of 6 ft. The metal deck chosen was a 22 Gage 1.5B-36 Grade 50 with a 36/4 connection pattern using #12 screws. The metal deck has a fire rating of 1.5 hours per International Building Code (IBC). The metal deck is topped with a metal standing seam sheet that provides drainage of water to the east. The quantity of these two roofing materials is 880 sf. The roof can be seen on the elevation views of the drawing set. These views are on sheet S1.04.

Box Truss:
The box trusses span horizontally between beams in the roof framing. This means the box trusses span 6 ft. The desired box truss size was an 8 in. x 8 in. truss. The truss was designed to support a load of lights, speakers, banners, etc. A 10 ft, 8 in. x 8 in. box truss can support a distributed load of 90 lb./ft. or a point load of 600 lb. at mid span per Applied Truss and Electronics (8 in. x 8 in.). Our span is only 6 ft, so these numbers are conservative. This truss provides more than enough strength for the intended use. The stage design requires 12 8 in. x 8 in. x 6 ft. box trusses.

Beams:
Standard steel ASTM A992 grade 50 wide-flange hot-rolled shapes were used for the beams. Loads from the dead load of the roof and box trusses, wind load, and snow load were all accounted for in the design of the roof beam. The roof beam was designed against shear, flexure, and deflection. Capacities were found in the AISC Steel Construction Manual as well as the International Building Code. The beam shape selected for the roof was a W10x12. Five W10x12 beams with a length of 33 ft. and 9 in. are required for the stage design. These beams are cantilevered over the front of the stage to allow for a good line-of-sight from all directions around the front of the stage. All analysis was done in Robot Structural Analysis Professional 2023. The beam connecting the two masonry walls above the entrance of the stage was designed to be a W8x10 that is 24 ft long. This beam is not load-bearing and is purely for aesthetics. Therefore, no calculations were performed. The beams are shown on the stage framing plan, located on sheet S1.03.

Moment Frame:
Standard steel ASTM A992 grade 50 wide-flange hot-rolled shapes were used for the girder and columns in the moment frame. The moment frame was designed to resist lateral loads as well as all the loads coming down through the roof beams. The span of the girder is 24 ft. and was sized as a W12x40. The columns are W12x40 and are at a height of 14 ft. and 6 in. The girder was designed based on strength and deflection and the column was designed based on strength and story drift. Combined axial load and moment was also checked. Capacities were found in the AISC Steel Construction Manual as well as the International Building Code. One girder and two columns are needed for the stage design. A design detail of the moment frame is located on sheet S1.03 as well as a general elevation view located on sheet S1.04.

Slab on Grade:
The slab covers the entire stage surface as well as the ramp. It was designed to be the same throughout since there is a 150 psf live load for stage floors (Minimum Designs, Chapter 4). The slab was designed using a resource called Industrial Slabs on Grade. After using a load factor, the design load was 240 psf. The stage and ramp slab are both 6 inches. Portland Cement Concrete (PCC) slabs that have one layer of reinforcement 2 inches from the top of the slab. The layer of reinforcement is a 6x6 6/6 welded wire fabric. The quantity of reinforced PCC is 20 cy. The concrete slab on grade detail is located on sheet S1.01.

Cast-in-Place Stairs:
This component of the stage is placed on the north side of the stage. The stairs are designed to access the stage while maintaining a 5 ft clearance. The stairs will be designed for strength and deflection. A detailed section of the stairs is located on sheet S1.05.

Railing:
The stage utilizes a railing called a pipe guardrail. The length of the railing on the ramp is 74 lf. This aluminum railing is located on sheet S1.05.

Brick Veneer:
The brick veneer is a typical 4 in. nominal red brick masonry unit. These units are purely architectural and supported wall ties that are spaced at 16 in. on center. A detail of a typical wall section is located on page S1.02.

Concrete Masonry Unit (CMU) Wall Design for Cantilever Retaining Wall:
The masonry wall was designed with 8 in. CMU blocks. This wall retains 7 ft. 1 in. in the most critical location of the stage boundary. Based off table 2 in section 15-7 B of National Concrete Masonry Association TEK, the wall needs vertical reinforcement of size No. 4 rebar @ 16 in. on center. The CMU reinforcement design detail is on sheet S1.05.

Concrete Masonry Unit (CMU) Wall Design for East Stage Walls:
These walls were designed to resist combined axial load and moment. The critical location is at the stage level right before the soil pressure aid to support the wall laterally. The reinforcement was designed using the National Concrete Masonry Association TEK. The details of the wall section are located on sheet S1.05.

Strip Footing Design for all CMU Walls:
The retaining wall footing was designed as a continuous footing with a width of 3 ft. 6 in. The footing was checked to satisfy a factor of safety (F.S.) of 1.5 for sliding and overturning, a F.S of 3 for bearing, and a maximum settlement of 0.5 in. Terzaghi's Bearing Capacity equation was used for bearing failure and Boussinesq's Simple Elastic Settlement Method was used for settlement failure. The quantity of reinforced PCC for the footings is 24 cy. Sheets S1.01 and S1.05 of the drawing set contain all strip footings with various foundation walls.

CMU Column:
A CMU column was required for axial support underneath the W-shape steel columns. The column is concentrically loaded and is required to be a 12 in x 16 in column to fit the size of a W12x26. The strength is more than adequate which is shown in Table 2 of 17-03A of National Concrete Masonry Association TEK. The required vertical reinforcement is four No. 4 bars, and the horizontal reinforcement is shown in Figure 2 of the same resource. The CMU column is shown on sheet S1.05.

Isolated Footing for Columns:
The CMU column footing was designed as an isolated footing with a width of 3 ft. 6 in. and a length of 3 ft. 6 in. The footing was checked to satisfy a factor of safety (F.S.) of 1.5 for sliding and overturning, a F.S of 3 for bearing, and a maximum settlement of 0.5 in. Vesic's Bearing Capacity equation was used for bearing failure and Boussinesq's Simple Elastic Settlement Method was used for settlement failure. There are two isolated footings required for the design of the stage and the detailed drawing of the footing is located on sheet S1.05.

PHASE ONE RESTROOM STRUCTURE DESIGN

Overall Structure Dimensions:
The first assumption made when designing the Phase One restroom structure was assuming the client is going to implement all three phases of this project. This means specifying a building length that will completely contain Phase Two length of the ramp within the building walls. This is further discussed in the section: Phase Two Restroom Design. A building length of 78 ft. was determined for a ten ft tall building. We decided on a concrete masonry unit (CMU) structure, to minimize cost, with 8 in. x 8 in. x 16 in. blocks. This is a typical CMU block size. The length of the wall needed to be in a multiple of 8 in., and 78 ft. satisfies this constraint. The minimum clear height needed for a bathroom, in accordance with the 2012 International Building Code (IBC) section 1207.2, is 7 ft. For the occupiable rooftop, we set our clear height at 10 ft. to enhance the lines of to the stage and account for the thickness of the roof slab, insulation, and mechanical, electrical, and plumbing (MEP) equipment. Ten ft. is also a multiple of 8 in., which is the height of the CMU blocks. A plan view of the Phase One restroom is located on sheet A1.01. Interior elevations and exterior elevations are located on sheets A1.01 and A1.02 of the drawing set, respectively.

Room Partitions:
We categorized the site as an auditorium without permanent seating and designed the restroom for 390 people in accordance with IBC table 2902.1. Traditional events at this green space host between 300 and 400 people. For 390 people, these are the conditions that need to be met for a female restroom: 6 water closets and 2 lavatories. These conditions need to be met for a male restroom: 3 water closets and 2 lavatories. We decided on 4 urinals in the men's restroom as well for convenience. Two ADA accessible water closets are provided in each restroom to exceed the ADA water closet requirements.

ADA required water closet dimensions and typical water closet dimensions dictated the length of each restroom. We determined the dimensions of each restroom to be roughly 16'-8" from outside wall face to outside wall face and roughly 21 ft. in length from inside face of wall to inside face of wall. This left roughly 30 ft. for Phase One storage. We designed the structure to contain all openings which included two restroom doors, a door to the storage, a garage door to the storage, and a window to the Phase Three concession stand. All openings were included in Phase One to minimize future phase costs. A plan view of the Phase One restroom and interior elevations are located on sheet A1.01.

Design Loads:
Wind and roof snow loads were calculated using ASCE 7-16 and ASCE 7-22. The wind loads were calculated assuming the Main Wind Force Resisting System (MWFRS) and using the Directional Procedure (Chapter 26 and 27). We assumed the roof as flat, so the snow load found did not have to be projected (Chapter 7). All final design loads were calculated using Load and Resistance Factor equations (LRFD) (Chapter 2). The load calculations can be found in Appendix D.

Occupiable Roof:
The roof was designed using a Nucor Vulcraft composite deck. This style of deck was chosen because of its ability to perform under large loads, such as the live load. The composite deck was required to hold a superimposed dead load of 18.5 psf., a live load of 100 psf., a balanced snow load of 36 psf., a negligible positive wind load, and a wind uplift force of 14 psf., and accommodate a span length of 16 ft. The slab acts as a one-way slab, spanning between North and South walls, because the length of the slab is much larger than the width. The slab was modeled as a simply supported beam in Robot and load combinations were applied. Max shear, positive moment, negative moment, and reaction forces were checked. A special load case of live load only applied to the four-foot overhang was checked to make sure the negative moment did not exceed the capacity. All checks were satisfied, and there was no net uplift at the reactions. Deflection was not a concern because the span table conditions were satisfied. The metal deck chosen was a 16 Gage 3VLI-36 Grade 50 and the concrete used was a normal weight (145 pcf.) with 4D 65/60BG Bekaert Dramix Fibers, 15 pcy. There was an additional layer of concrete on top of the composite deck for sloping purposes. This layer was also a normal weight concrete (145 psf.). The concrete layer is 1 in. thick at the South edge of the roof and 0 in. thick at the north edge of the roof, resulting in a 0.4% slope. In addition, a clear waterproofing membrane adhesive was applied to the top surface of the concrete.

The high-performance deck slab diaphragm strength tool was used to determine how many shear studs were required along the walls of the structure. The calculations for the roof composite deck are shown in Appendix D. Details of the occupiable roof are located on sheet A1.03.

Occupiable Roof/Wall Connection:
A continuous A36 plate was placed along the entire length of the walls to weld the shear studs to the wall. This plate was not treated as a bearing plate because the floor load is

distributed along the entire length of the plate. Thus, there was not a significant load for the plate to act as a bearing plate. The compressive strength of the CMU block and the A36 steel plate are far greater than the distributive reaction force. An anchor screw, to connect the base plate to the wall, was chosen from the Simpson Strong Tie website. The TNTW25134TF Anchor Screw was picked. The screws don't resist tension since there was no net uplift. In plane and out of plane shear were checked for. The calculations for this connection are shown in Appendix D. Details of this connection are located on sheets A1.03 and S1.08.

Concrete Masonry Unit (CMU) Wall Design for Simply Supported Condition:
The masonry wall was designed with 8 in. CMU blocks, which are a standard size. The 10 ft. wall resists a 155 ft-lb/ft moment and a 735-kip axial force. The walls for the restroom structure are treated as simply supported at the slab and at the roof diaphragm. Based on figure 1 in section 14-11B of National Concrete Masonry Association (NCMA) TEK, the wall needed vertical reinforcement of size No. 4 @ 16 in. and lateral reinforcement was provided for the serviceability limit state. This was to prevent cracking of the CMU. The lateral reinforcement selected was a 220 Ladder-Mesh from Hohmann and Barnard Inc. website. It is recommended to be spaced between each CMU brick vertically and specified as Galvanized Steel. The calculations for the walls are shown in Appendix D. Details of the wall are located on sheets A1.03 and S1.08.

Partition Walls:
The partition walls are non-loadbearing. Expansion layers are provided at the ends of the walls to prevent transfer of force, and a gap was left at the top of the wall to enable MEP to be run from room to room. Sections of the partition walls are located on sheet A1.03.

Lintel:
A precast lintel size 8 in. x 16 in. was chosen to adequately support the moment and shear experienced due to the loading above the lintels. The lintel was designed for the longest span, and the same one was used for each opening. One #6 reinforcement bar was placed at the bottom of the lintel to resist moment. This lintel was sized using table 4 in NCMA Tek 17-02A. Lintel calculations are shown in Appendix D. A section cut of the lintel will be located on sheet S1.08.

CMU Column:
A CMU column is required for axial support underneath the W-shape steel columns from the future Phase Three roof. The column is concentrically loaded, and the column was sized using Table 2 of 17-03A of the NCMA TEK. The required vertical reinforcement is 8 No.4 bars, and the horizontal reinforcement is 0.25 in. diameter ties every 8 in. The calculations for the column are shown in Appendix D. A section view of the CMU column is shown on sheet S1.08.

Brick Veneer:
The brick veneer is a typical 4 in. red brick masonry unit. These units are purely architectural and are supported by wall ties 16 in. x 16 in. on center. The Hohmann and Barnard Inc. website recommends an embedment of 3 in. for a Concrete 2-Seal Tie for an

insulation thickness of 3 in. NCMA Tek 16-01A figure 1 recommends a maximum vertical and horizontal tie spacing of 16 in. on center for adjustable ties, which is what we chose. Our cavity width is 4 in. and the maximum cavity width for this condition is 4.5 in. Control joints are spaced every 15 ft. and under stress concentrations in the brick veneer to prevent cracking of the veneer. This is specified in NCMA Tek 10-04. The calculations for the walls are shown in Appendix D. Brick veneer details are located on sheets A1.03 and A2.02.

Insulation:
Closed cell spray foam insulation was chosen for the roof insulation because it must adhere to the roof. The roof slab was treated as a mass floor slab in climate zone 5, so an R value of 14.6 was needed, as specified in the 2018 International Energy Conservation Code in table C402.1. A thickness of 2.5 in. closed cell insulation was determined. For the wall insulation, a mass wall was determined and an R value of 11.4 was needed. An insulation board was chosen, and a thickness of 3 in. was determined. The calculations for the insulation are shown in Appendix D. Cross sections of the insulation are located on sheet A1.03.

Slab on Grade:
The slab covers the area of the restroom structure from outside face of wall to outside face of wall. It was designed to be the same throughout since there is a 60 psf live load for public restrooms (Chapter 4). The slab was designed using a resource called Industrial Slabs on Grade. After using a load factor, the design load was 96 psf. The slab is 5 in. Portland Cement Concrete (PCC) slab and has one layer of reinforcement 2 in from the bottom of the slab. The layer of reinforcement is a 6 in. x 6 in. 6/6 welded wire fabric. A slab on grade detail is located on sheet S1.07.

Strip Footing Design for all CMU Walls:
The wall footing was designed as a continuous footing with a width of 3 ft. 0 in. The footing was checked to satisfy a factor of safety (F.S.) of 1.5 for sliding and overturning, a F.S of 3 for bearing, and a maximum settlement of 0.5 in. Terzaghi's Bearing Capacity equation was used for bearing failure and Boussinesq's Simple Elastic Settlement Method was used for settlement failure. Sheet S1.07 of the drawing set contains the detail for this footing. The calculations for the wall footing are shown in Appendix D.

Isolated Footing for Columns:
The CMU column footing was designed as an isolated footing with a width of 4 ft. 4 in. and a length of 5 ft. The footing was checked to satisfy a factor of safety (F.S.) of 1.5 for sliding and overturning, a F.S of 3 for bearing, and a maximum settlement of 0.5 in. Vesic's Bearing Capacity equation was used for bearing failure and Boussinesq's Simple Elastic Settlement Method was used for settlement failure. There are 12 isolated footings required for the design of the restroom structure and the detailed drawing of the footing is located on sheet S1.07. The calculations for the isolated column footings are shown in Appendix D.

PHASE TWO RESTROOM STRUCTURE DESIGN

Ramp Dimensions:
The maximum grade of a ramp, in accordance with the 2010 Americans with Disabilities Act (ADA) section 405.6, to have a maximum ramp rise of 30 in. is 1:12. Once there is a rise of 30 in. and a run of 30 ft., there is a need for a 6 ft. long landing according to ADA section 405.7.3. We set the clear height of the building at 10 ft from the top of floor slab to the top of the roof slab. To climb a vertical distance of 10 ft., four runs of ramp and four landings are required. This results in a ramp of length of 144 ft. This exceeds the width of the site, so we decided upon a double-back ramp resulting in a ramp length of 72 ft. Therefore, the 72 ft. Phase Two ramp is contained within the entire 78 ft. length of the building. A plan view, elevations views, and a section cut are located on sheet A2.02.

Design Loads:
Wind and roof snow loads were calculated using ASCE 7-16 and ASCE 7-22. The wind loads on the ramp walls were calculated assuming the Main Wind Force Resisting System (MWFRS) and using the Directional Procedure (Chapter 26 and 27). The snow load was projected onto the horizontal (Chapter 7). All final design loads were calculated using Load and Resistance Factor equations (LRFD) (Chapter 2). The load calculations can be found in Appendix E.

Concrete Masonry Unit (CMU) Wall Design for Cantilever Wall:
The masonry wall was designed with 8 in. CMU blocks. Based off table 2 in section 15-7B of NCMA TEK, the wall needs vertical reinforcement of size No. 5 @ 16 in. The calculations for the walls will be shown in Appendix E. A plan view of the walls and a section cut are located on sheets S2.01 and A2.02, respectively.

Strip Footing Design for CMU Walls:
The wall footing was designed as a continuous footing with a width of 3 ft. 8 in. The footing was checked to satisfy a factor of safety (F.S.) of 1.5 for sliding and overturning, a F.S of 3 for bearing, and a maximum settlement of 0.5 in. Terzaghi's Bearing Capacity equation was used for bearing failure and Boussinesq's Simple Elastic Settlement Method was used for settlement failure. Sheet S2.01 of the drawing set contains the detail for this footing. The calculations for the wall footing are shown in Appendix E.

Composite Floor Slab:
The roof was designed using a Nucor Vulcraft composite deck. This style of deck was chosen because of its ability to perform under large loads, such as the live load. The composite deck was required to accommodate a span length of 5 ft. The slab acts as a one-way slab, spanning between north and south ramp walls, because the length of the slab is much larger than the width. The metal deck chosen was a 3VLI-36 Grade 50 and the concrete used was a normal weight (145 pcf.) with 4D 65/60BG Bekaert Dramix Fibers, 45 pcy. The calculations for the composite floor slab will be shown in Appendix E. A detail of the composite floor slab and connections is located on sheet S2.01.

Railing:
The stage utilizes a railing called a pipe guardrail. The length of the railing is 135 ft. An elevation and section cut of the railing is located on sheet A2.01. A plan view of the railing is located on sheet A2.01 as well.

PHASE THREE RESTROOM STRUCTURE DESIGN

Design Loads:
Wind and roof snow loads were calculated using ASCE 7-16 and ASCE 7-22. The wind loads were calculated assuming the MWFRS and Directional Procedure (Chapter 26 and 27). The snow loads were using the horizontal projection on the roof. The ground snow load was found to be 25 psf. and the snow load acting on the roof was found to be 19 psf. Design loads were calculated using LRFD equations (Chapter 2). The calculations for the design loads are shown in Appendix F.

Metal Deck:
The metal decking on the roof was designed using Nucor Vulcraft metal deck. The use of other materials was not considered in this design. The metal deck was designed to support metal sheets, to provide water runoff, and solar panels. Uplift was calculated ignoring the weight of solar panels and was found to be 4.369 psf. The metal deck must also support a gravity load of 45.45 psf. An 18 gage 1.5B-36 Grade 50 deck with 36/4 connection pattern of #12 screws was chosen for a span of 8 ft. The metal deck connects to the joists. The metal deck in total covers an area of 1575 sf. The calculations for the metal deck are shown in Appendix F. Metal deck details are shown on sheet S3.01.

Joists:
Standard ASTM A992 grade 50 wide-flange hot-rolled shapes were used for the joists. The only load the joists will be supporting is that of the metal deck, so they were designed using 45.45 psf. The joist was designed against flexure and deflection. Capacitates were found using AISC Steel Manual and the IBC. The size of the joist was decided at W10x12 with a length of 14 ft 9 in. The joists are located at each column, as well as halfway between column spans, with the largest being 8 feet. The joists that are spaced halfway between columns are connected to the girders and are singly coped at the top side of the joist. A cope of 4.5 in. long and 2 in. deep was used during calculations and was the coped size that was used. The calculations for the joists are shown in Appendix F. Joist details and elevations are shown on sheet S3.01.

Girders:
Standard ASTM A992 grade 50 wide-flange hot-rolled shapers were used for the girders. The girders support the single coped joists and were designed against flexure and deflection. Capacitates were found using AISC Steel Manual and the IBC. The loads that the girder supports come from the ends of the joists at the midspan of the girder. The size of the girder was determined to be W12x14. The size was decided to have adequate connections within the web. Girders span between columns, with the longest being 16 ft. The calculations for the girders are shown in Appendix F. Girder details and elevations are shown on sheet S3.01.

Columns:
Standard ASTM A992 grade 50 wide-flange hot-rolled shapes were used for the columns. The columns support the girders within the flanges and the joists within the web. Exterior columns support one girder and one joist, while interior columns support two girders and one joist. The columns were designed against flexure and deflection. Capacitates were found using AISC Steel Manual and the IBC. The column size was found to be W12x14. The columns were spaced throughout the length of the occupiable roof, with the longest span being 16 ft. The north side columns have a length of 9.5 ft, while the south side columns have a length of 8.5 ft. The columns are anchored to the deck, with an 8 by 12-inch section being cut from the bathroom structures composite roof deck and filled with normal weight concrete. The calculations for the columns are shown in Appendix F. Girder details and elevations are shown on sheet S3.01.

SECTION VII

Engineer's Cost Estimate

Line Number	Item	Quantity	Unit	Unit Price	Material	Labor	Total	Contingency	O & P	Total
STAGE										
B2010 130 5200	Brick Veneer/Metal Stud Backup	1100	SF	$ 7.30	$ 8,030.00	$ 21,285.00	$ 29,315.00	$ 2,931.50	$ 5,863.00	$ 38,109.50
B2010 132 1200	Brick Face Composite Wall - Double Wythe	1310	SF	$ 10.90	$ 14,279.00	$ 32,750.00	$ 47,029.00	$ 4,702.90	$ 9,405.80	$ 61,137.70
	A992 Steel	2.17	ton	$ 1,500.00	$ 3,255.00	$ 7,595.00	$ 10,850.00	$ 1,085.00	$ 2,170.00	$ 14,105.00
	A36 Steel Plate (8x18x0.5)	5	each	$ 50.00	$ 250.00	$ 125.00	$ 375.00	$ 37.50	$ 75.00	$ 487.50
	A36 Steel Plate (4x8x0.5)	5	each	$ 19.00	$ 95.00	$ 125.00	$ 220.00	$ 22.00	$ 44.00	$ 286.00
	Box Trusses	12	each	$ 300.00	$ 3,600.00	$ 3,600.00	$ 7,200.00	$ 720.00	$ 1,440.00	$ 9,360.00
C2010 110 0470	Cast-in-place stairs	0.58	flight	$ 1,500.00	$ 875.00	$ 1,472.92	$ 2,347.92	$ 234.79	$ 469.58	$ 3,052.29
A1030 120 4480	Concrete Slab on Grade	1080	SF	$ 3.59	$ 3,877.20	$ 3,769.20	$ 7,646.40	$ 764.64	$ 1,529.28	$ 9,940.32
A1010 105 1780	Foundation Walls	190	LF	$ 35.00	$ 6,650.00	$ 12,255.00	$ 18,905.00	$ 1,890.50	$ 3,781.00	$ 24,576.50
A1010 210 7200	Isolated Footing	2	each	$ 68.00	$ 136.00	$ 266.00	$ 402.00	$ 40.20	$ 80.40	$ 522.60
	Metal Deck	880	SF	$ 12.00	$ 10,560.00	$ 3,520.00	$ 14,080.00	$ 1,408.00	$ 2,816.00	$ 18,304.00
	Metal Seam Sheet	880	SF	$ 11.60	$ 10,208.00	$ 5,764.00	$ 15,972.00	$ 1,597.20	$ 3,194.40	$ 20,763.60
	Aluminum Railing	74	LF	$ 110.00	$ 8,140.00	$ 5,920.00	$ 14,060.00	$ 1,406.00	$ 2,812.00	$ 18,278.00
A1010 110 4700	Strip Footing	200	LF	$ 39.00	$ 7,800.00	$ 7,600.00	$ 15,400.00	$ 1,540.00	$ 3,080.00	$ 20,020.00
	Subtotal						$ 183,802.32	$ 18,380.23	$ 36,760.46	$ 238,943.01
RESTROOM PHASE 1										
	Foundation Walls	240	LF	$ 35.00	$ 8,400.00	$ 15,480.00	$ 23,880.00	$ 2,388.00	$ 4,776.00	$ 31,044.00
A1010 110 3300	Strip Footing	240	LF	$ 39.00	$ 9,360.00	$ 9,120.00	$ 18,480.00	$ 1,848.00	$ 3,696.00	$ 24,024.00
D2010 110 2120	Water Closet Systems, Floor Mount	9	each	$ 885.00	$ 7,965.00	$ 7,830.00	$ 15,795.00	$ 1,579.50	$ 3,159.00	$ 20,533.50
D2010 210 2000	Urinals	4	each	$ 600.00	$ 2,400.00	$ 3,620.00	$ 6,020.00	$ 602.00	$ 1,204.00	$ 7,826.00
D2010 310 2040	Sinks	4	each	$ 915.00	$ 3,660.00	$ 3,560.00	$ 7,220.00	$ 722.00	$ 1,444.00	$ 9,386.00
D2010 810 1920	Drinking Fountains	2	each	$ 1,350.00	$ 2,700.00	$ 1,060.00	$ 3,760.00	$ 376.00	$ 752.00	$ 4,888.00
C1030 110 400	Toilet Partitions, Painted Metal	9	each	$ 615.00	$ 5,535.00	$ 2,835.00	$ 8,370.00	$ 837.00	$ 1,674.00	$ 10,881.00
C1030 110 1330	Toilet Partitions, Plastic Laminate	3	each	$ 227.00	$ 681.00	$ 468.00	$ 1,149.00	$ 114.90	$ 229.80	$ 1,493.70
	Hand Dryer	4	each	$ 250.00	$ 1,000.00	$ 624.00	$ 1,624.00	$ 162.40	$ 324.80	$ 2,111.20
B2030 220 3350	Exterior Doors	3	each	$ 2,500.00	$ 7,500.00	$ 1,050.00	$ 8,550.00	$ 855.00	$ 1,710.00	$ 11,115.00
C1020 102 5000	Garage Door	1	each	$ 790.00	$ 790.00	$ 895.00	$ 1,685.00	$ 168.50	$ 337.00	$ 2,190.50
B2020 106 6700	Exterior Windows	1	each	$ 395.00	$ 395.00	$ 238.00	$ 633.00	$ 63.30	$ 126.60	$ 822.90
B2010 132 1200	Brick Face Composite Wall - Double Wythe	1581	SF	$ 10.90	$ 17,232.90	$ 39,525.00	$ 56,757.90	$ 5,675.79	$ 11,351.58	$ 73,785.27
	Rigid Insulation, 3"	99	each	$ 50.00	$ 4,940.63	$ 988.13	$ 5,928.75	$ 592.88	$ 1,185.75	$ 7,707.38
	Spray Foam Insulation, 2.5"	1362	LF	$ 2.00	$ 2,724.00	$ 13,620.00	$ 16,344.00	$ 1,634.40	$ 3,268.80	$ 21,247.20
	A36(Length of wall, 10'x8"x0.5")	18	each	$ 30.00	$ 540.00	$ 450.00	$ 990.00	$ 99.00	$ 198.00	$ 1,287.00
	0.75" Screws(200 count)	1	pack	$ 240.00	$ 240.00	$ 100.00	$ 340.00	$ 34.00	$ 68.00	$ 442.00
	Liquid Applied Waterproofing Membrane	4	each	$ 132.00	$ 528.00	$ 100.00	$ 628.00	$ 62.80	$ 125.60	$ 816.40
B3010 610 0050	Gutters	80	LF	$ 3.31	$ 264.80	$ 488.00	$ 752.80	$ 75.28	$ 150.56	$ 978.64
B3010 620 0100	Downspouts	20	LF	$ 1.20	$ 24.00	$ 78.00	$ 102.00	$ 10.20	$ 20.40	$ 132.60
B3010 420 2800	Roof Edge, Sheet Metal	202	LF	$ 18.50	$ 3,737.00	$ 2,383.60	$ 6,120.60	$ 612.06	$ 1,224.12	$ 7,956.78
B1010 217 3700	Cast in Place Slab, One Way	1613	SF	$ 6.90	$ 11,129.70	$ 16,694.55	$ 27,824.25	$ 2,782.43	$ 5,564.85	$ 36,171.53
C1010 102 2000	Cocrete Block Paritions - Regular Weight	545	SF	$ 3.13	$ 1,705.85	$ 4,305.50	$ 6,011.35	$ 601.14	$ 1,202.27	$ 7,814.76
2111-8174100	Subgrade	1362	SF	$ 11.02	$ 15,009.24	$ 4,753.38	$ 19,762.62	$ 1,976.26	$ 3,952.52	$ 25,691.41
50 17 11 0500	MEP (Mixed Use)	1362	SF	$ 30.00	$ 40,860.00	$ 27,240.00	$ 68,100.00	$ 6,810.00	$ 13,620.00	$ 88,530.00
A1030 120 4480	Concrete Slab on Grade	1362	SF	$ 3.59	$ 4,889.58	$ 4,753.38	$ 9,642.96	$ 964.30	$ 1,928.59	$ 12,535.85
	Subtotal						$ 292,591.23	$ 29,259.12	$ 58,518.25	$ 411,412.60

RESTROOM PHASE 2										
A1010 110 3300	Strip Footing	175	LF	$ 39.00	$ 6,825.00	$ 6,650.00	$ 6,825.00	$ 3,962.62	$ 7,925.24	$ 18,712.86
	Foundation Walls	167	LF	$ 35.00	$ 5,845.00	$ 10,771.50	$ 16,616.50	$ 1,661.65	$ 3,323.30	$ 21,601.45
	Railing	319	LF	$ 110.00	$ 35,090.00	$ 25,520.00	$ 60,610.00	$ 6,061.00	$ 12,122.00	$ 78,793.00
B1010 217 2800	Cast in Place Slab, One Way	770	SF	$ 5.30	$ 4,081.00	$ 7,815.50	$ 11,896.50	$ 1,189.65	$ 2,379.30	$ 15,465.45
B2010 130 5200	Brick Veneer/Metal Stud Backup	1740	SF	$ 7.30	$ 12,702.00	$ 33,669.00	$ 46,371.00	$ 4,637.10	$ 9,274.20	$ 60,282.30
B2010 132 1200	3rick Face Composite Wall - Double Wyth	1740	SF	$ 10.90	$ 18,966.00	$ 43,500.00	$ 62,466.00	$ 6,246.60	$ 12,493.20	$ 81,205.80
	Subtotal						$ 204,785.00	$ 23,758.62	$ 47,517.24	$ 276,060.86
RESTROOM PHASE 3										
	Metal Deck	1576	SF	$ 12.00	$ 18,912.00	$ 48.00	$ 18,960.00	$ 1,896.00	$ 3,792.00	$ 24,648.00
	Solar Panels	40	each	$ 350.00	$ 14,000.00	$ 8,750.00	$ 22,750.00	$ 2,275.00	$ 4,550.00	$ 29,575.00
C1010 102 2000	Cocrete Block Paritions - Regular Weight	272.5	SF	$ 3.13	$ 852.93	$ 2,152.75	$ 3,005.68	$ 300.57	$ 601.14	$ 3,907.38
B3010 135 5000	Standing Seam Formed Metal	1576	SF	$ 11.10	$ 17,493.60	$ 64.38	$ 17,557.98	$ 1,755.80	$ 3,511.60	$ 22,825.37
	Connections(Plate and Screw)	40	each	$ 30.00	$ 1,200.00	$ 750.00	$ 1,950.00	$ 195.00	$ 390.00	$ 2,535.00
	Refridgerator	1	each	$ 3,000.00	$ 3,000.00	$ 890.00	$ 3,890.00	$ 389.00	$ 778.00	$ 5,057.00
D2010 310 2040	Sink	1	each	$ 915.00	$ 915.00	$ 890.00	$ 1,805.00	$ 180.50	$ 361.00	$ 2,346.50
	A992 Steel	1.83	ton	$ 1,500.00	$ 2,746.88	$ 6,409.38	$ 9,156.25	$ 915.63	$ 1,831.25	$ 11,903.13
	Subtotal						$ 79,074.91	$ 7,907.49	$ 15,814.98	$ 102,797.38
SITE										
	Lighting	7	each	$ 200.00	$ 1,400.00	$ 24.43	$ 1,424.43	$ 142.44	$ 284.89	$ 1,851.76
2610-0000120	Trees	3	each	$ 470.00	$ 1,410.00	$ 300.00	$ 1,710.00	$ 171.00	$ 342.00	$ 2,223.00
2552-0000140	Excavation for Tree Growth	75	CY	$ 35.00	$ 2,625.00	$ 1,500.00	$ 4,125.00	$ 412.50	$ 825.00	$ 5,362.50
2107-0425020	Fill	375	CY	$ 23.00	$ 8,625.00	$ 1,308.75	$ 9,933.75	$ 993.38	$ 1,986.75	$ 12,913.88
2552-0000140	Cut	82	CY	$ 35.00	$ 2,870.00	$ 286.18	$ 3,156.18	$ 315.62	$ 631.24	$ 4,103.03
	New Benches	3	each	$ 1,000.00	$ 3,000.00	$ 600.00	$ 3,600.00	$ 360.00	$ 720.00	$ 4,680.00
	Picnic Tables	6	each	$ 1,000.00	$ 6,000.00	$ 1,200.00	$ 7,200.00	$ 720.00	$ 1,440.00	$ 9,360.00
2401-6745650	tructure Removal(Sculpture, tables, kiosk	2	each	$ 350.00	$ 700.00	$ 500.00	$ 1,200.00	$ 120.00	$ 240.00	$ 1,560.00
2510-6745850	Pavement Removal	47	SY	$ 11.00	$ 517.00	$ 164.03	$ 681.03	$ 68.10	$ 136.21	$ 885.34
2111-8174100	Subgrade	2888	SF	$ 11.00	$ 31,768.00	$ 10,079.12	$ 41,847.12	$ 4,184.71	$ 8,369.42	$ 54,401.26
A1030 120 4480	Concrete Slab on Grade	2888	SF	$ 3.59	$ 10,367.92	$ 10,079.12	$ 20,447.04	$ 2,044.70	$ 4,089.41	$ 26,581.15
	Subtotal						$ 20,447.04	$ 2,044.70	$ 34,879.89	$ 123,921.92

STAGE, PHASE 1, AND SITE TOTAL	$ 774,500.00
STAGE, PHASE 1 AND 2, AND SITE TOTAL	$ 1,050,500.00
ALL PHASES AND SITE TOTAL	$ 1,153,500.00

SECTION VIII
Appendix A: Bibliography

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Appendix B: Design Renderings and Models

<u>PHASE ONE</u>

Figure 8B.1. Phase One overall site design – S Main St view

Figure 8B.2. Phase One overall site design – Person point of view

Figure 8B.3. Phase One overall site design with trees omitted.

Figure 8B.4. Stage design for all phases – Angled view

Figure 8B.5. Stage design for all phases – Front view

Figure 8B.6. Stage design for all phases – Side view

Figure 8B.7. Phase One restroom structure – Front view

Figure 8B.8. Phase One restroom structure – Side view

Figure 8B.9. Phase One restroom structure – Back view

PHASE TWO

Figure 8B.10. Phase Two overall site design with trees omitted

Figure 8B.11. Phase Two restroom structure – Front angled view

Figure 8B.12. Phase Two restroom structure – Back side view

PHASE THREE

Figure 8B.13. Phase Three overall site design – S Main St and E Pleasant St intersection view

Figure 8B.14. Phase Three overall site design with trees omitted

Figure 8B.15. Phase Three overall site design – Back aerial view from E Pleasant St

Figure 8B.16. Phase Three restroom structure – Front view

Figure 8B.17. Phase Three restroom structure – Front angled view

Figure 8B.18. Phase Three restroom structure – Back angled view

Figure 8B.19. Phase Three restroom structure – Occupiable roof view

Figure 8B.20. Phase Three restroom structure – Concession stand view

PHASE THREE AT NIGHT

Figure 8B.21. Nighttime Phase Three overall site design – S Main St view

Figure 8B.22. Nighttime Phase Three overall site design – From north looking south view

Figure 8B.23. Nighttime Phase Three overall site design – Northeast parking lot view

Figure 8B.24. Nighttime Phase Three restroom structure – Occupiable roof view

FLOORPLAN MODELS

Figure 8B.25. Phase One floorplan

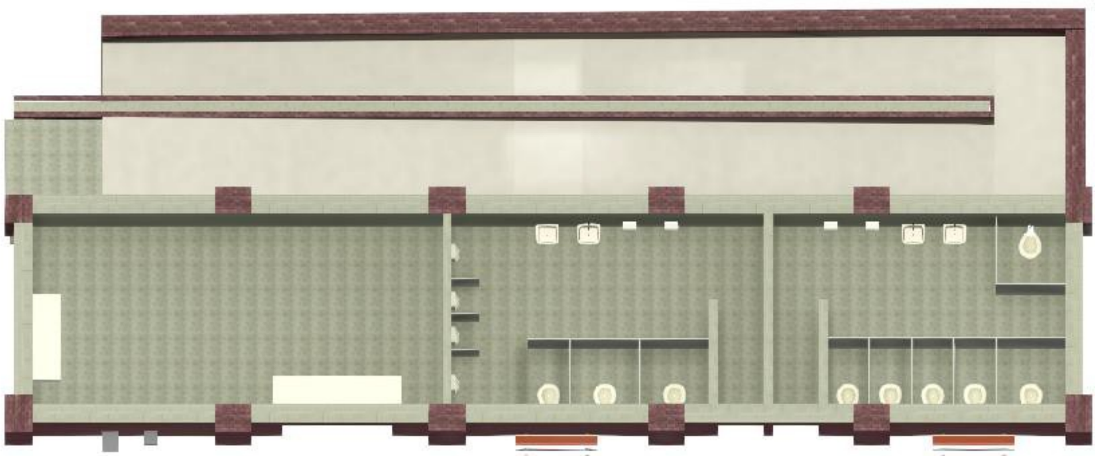


Figure 8B.26. Phase Two floorplan

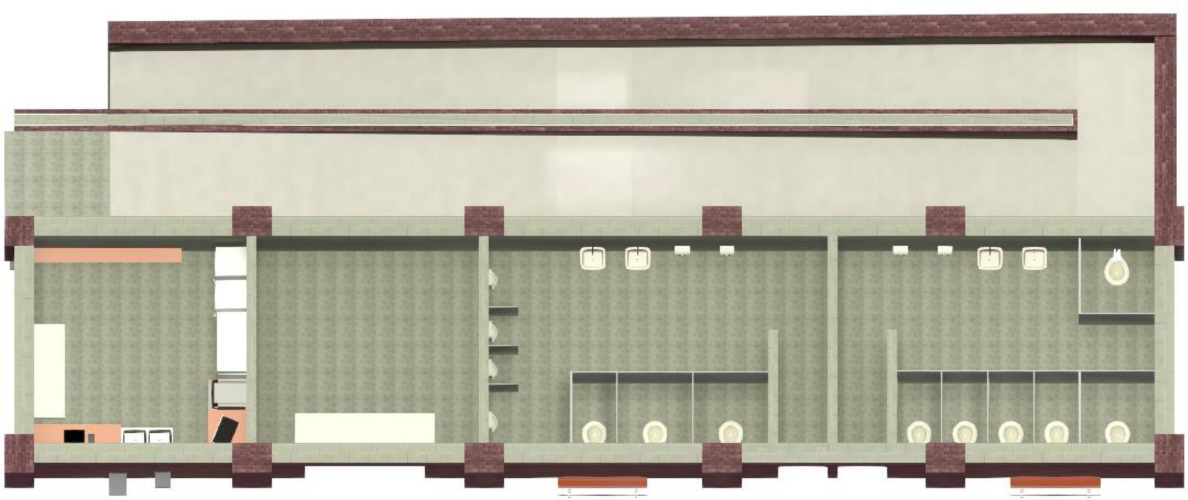


Figure 8B.27. Phase Three floorplan

UOKETA GREENSPACE REDEVELOPMENT STAGE CALCULATION REPORT:

LOAD CALCULATIONS:

oad Parameters:

osure Category: B (ASCE 7-22: Section 26.7.3)

Category: II (ASCE 7-22: Table 1.5-1)

ortance Factor: $I := 1$ (ASCE 7-22: Table 1.5-2)

c Wind Speed: $V := 108\ mph$ (ASCE 7-22: Figure 26.5-1B)

d Directionality Factor: $K_d := 0.85$ (ASCE 7-22: Figure 26.6-1)

ographic Factor: $K_{zt} := 1$ (ASCE 7-22: Section 26.8.1)

und Elevation Factor: $K_e := 1$ (ASCE 7-22: Table 26.9-1)

Effect Factor: $G := 0.85$ (ASCE 7-22: Section 26.11.1)

osure Classification: Building Open (ASCE 7-22: Section 26.2)

S DESIGN WIND PRESSURES FOR THE ROOF

oof Height:

$$\frac{1}{2} \cdot \left(20\ ft + \frac{3}{4}\ in + 17\ ft + 2\ in + \frac{9}{16}\ in\right) = 18.638\ ft$$ (Height Modeled on Revit)

Pressure Exposure Coefficient: $K_h := 0.61$ (ASCE 7-22: Table 26.10-1)

Pressure:

$$0.00256\ \frac{psf}{mph^2} \cdot K_h \cdot K_{zt} \cdot K_d \cdot V^2 \cdot I = 15.4823\ psf$$ (ASCE 7-22: Equation 26.10-1)

from the east:

180

angle is <7.5%, use values for 0%... which means wind loads will be a horizontal projection

$= 1.2$ Net pressure coefficient (ASCE 7-22: Figure 27.3-4)

$q_h \cdot K_d \cdot G \cdot C_N = 13.4232\ psf$

S DESIGN WIND PRESSURES FOR THE WALLS:

F "16'1–1/2" $= 16.125\ ft$

y Pressure Exposure Coefficient: $K_z := 0.58$ (ASCE 7-22: Table 26.10-1)

y Pressure:

$0.00256\ \frac{psf}{mph^2} \cdot K_z \cdot K_{zt} \cdot K_d \cdot V^2 \cdot I = 14.7209\ psf$ (ASCE 7-22: Equation 26.10-1)

q_z

Loads EW:

Pressure Coefficients $GC_{pi\_1} := 0.18$ $GC_{pi\_2} := -0.18$ (ASCE 7-22: Section 26.13; Table 26.13-1)

ssure Coefficients $C_{p\_WW} := 0.8$ $C_{p\_SW} := -0.7$

$L := 3.25\ ft$ $B := 32.5\ ft$ $\frac{L}{B} = 0.1$

$C_{p\_LW} := -0.5$

Wall Pressure Coefficients, C_p

Surface	L / B	C_p	Use with
Windward wall	All values	0.8	q_z
Leeward wall	0–1	–0.5	q_h
	2	–0.3	q_h
	≥ 4	–0.2	q_h
Sidewall	All values	–0.7	q_h
Parapet	All values	See Section 27.3.4 for GC_{pn}	q_p

~~(ASCE 7-22: Table 27.3-1)~~

$K_d \cdot G \cdot C_p - q_i \cdot K_d \cdot \left(\left(GC_{pi}\right)\right)$

$q_z \cdot K_d \cdot G \cdot C_{p\_WW} - q_i \cdot K_d \cdot \left(\left(GC_{pi\_1}\right)\right) = 6.2564\ psf$

$q_h \cdot K_d \cdot G \cdot C_{p\_LW} - q_i \cdot K_d \cdot \left(\left(GC_{pi\_1}\right)\right) = -7.8453\ psf$

$q_h \cdot K_d \cdot G \cdot C_{p\_SW} - q_i \cdot K_d \cdot \left(\left(GC_{pi\_1}\right)\right) = -10.0825\ psf$

$q_z \cdot K_d \cdot G \cdot C_{p\_WW} - q_i \cdot K_d \cdot \left(\left(GC_{pi\_2}\right)\right) = 10.761\ psf$

$q_h \cdot K_d \cdot G \cdot C_{p\_LW} - q_i \cdot K_d \cdot \left(\left(GC_{pi\_2}\right)\right) = -3.3407\ psf$

$q_h \cdot K_d \cdot G \cdot C_{p\_SW} - q_i \cdot K_d \cdot \left(\left(GC_{pi\_2}\right)\right) = -5.5779\ psf$

$:= p_{WW2} = 10.761\ psf$ $p_{negative} := p_{SW1} = -10.0825\ psf$

Loads NS:

Pressure Coefficients $GC_{pi\_1} := 0.18$ $GC_{pi\_2} := -0.18$

ssure Coefficients $C_{p\_WW} := 0.8$ $C_{p\_SW} := -0.7$

$L := 32.5\ ft$ $B := 3.25\ ft$ $\frac{L}{B} = 10$

NOW LOAD CALCULATIONS:

load: use the horizontal projection

al area: $L := 33.9271\ ft$ $W := 26\ ft$ $A := L \cdot W = 882.1046\ ft^2$

psf ground snow load

oad Parameters:

re Category:	Fully Exposed	(ASCE 7-22: Table 7.3-1 footnotes)
Roughness:	B	(ASCE 7-22: Section 26.7.2)
re Factor:	$C_e := 0.9$	(ASCE 7-22: Table 7.3-1)
l Factor:	$C_t := 1.2$	(ASCE 7-22: Table 7.3-2)
ctor	$C_s := 1$	(ASCE 7-22: Table 7.4-1c)
tegory:	II	(ASCE 7-22: Table 1.5-1)
n Snow Load:	$p_{m\_max} := 30\ psf$	(ASCE 7-22: Table 7.3-4)

$in\left(p_g, p_{m\_max}\right) = 25\ psf$ (ASCE 7-22: Section 7.3.3)

$\cdot C_e \cdot C_t \cdot p_m = 18.9\ psf$ (flat roof snow load) (ASCE 7-22: EQ 7.3-1)

$C_s \cdot p_f = 18.9\ psf$ (sloped roof snow load) (ASCE 7-22: EQ 7.4-1)

DECK:

Uniform Load:

psf

$_{now} = 18.9\ psf$

$= 13.4232\ psf$

$(1.6L_r \text{ or } 1.0S \text{ or } 1.6R) + (L \text{ or } 0.5W)$ Load Combinations

22 Gage 1.5B-36 Grade 50

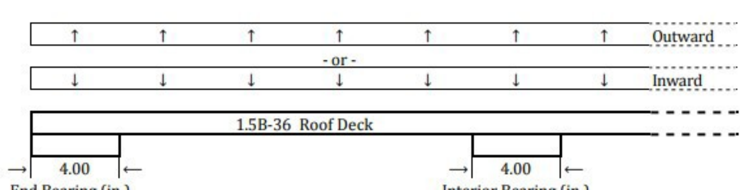
Uniform Design Load Table, LRFD (psf)

For End Lapped Deck

NUC VULCRAFT

36 / 4 Connection Pattern to Supports with #12 Screw

Support Member A992 GR50
$0.21 \le t_2$ (in.) ≤ 0.5



N GRADE:

Occupancy **	Min. Slab Thickness	Reinforcement ‡
abs under other slabs	2″	None
tic or light commercial d less than 100 psf)	4″	One layer 6 x 6 10/10 welded wire fabric, minimum for ideal conditions; 6 x 6 8/8 for average conditions.
ercial—institutional—barns d 100-200 psf)	5″	One layer 6 x 6 8/8 welded wire fabric or one layer 6 x 6 6/6.
rial (loaded not over 400-500 d pavements for industrial , gas stations, and garages	6″	One layer 6 x 6 6/6 welded wire fabric or one layer 6 x 6 4/4.

$_{oors} := 150\ psf$ (ASCE 7-22: Table 4.3-1)

$\cdot LL_{stagefloors} = 240\ psf$

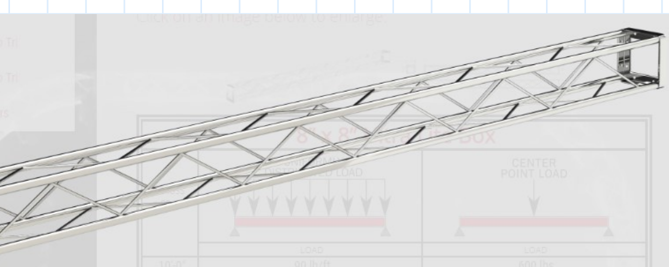
C slab is required for the stage floor as well as the ramp. One layer of 6x6 6/6 welded wire fabric will be used for reinforcement " from the top of the slab.

ATING FOR ROOF:

ck fire rating: 1.5 hours

l fire rating: none

RUSS:



ww.appliednn.com/
/8x8-ultra-lite-box/

DESIGN:

or flexure, deflection, and shear at supports)

ft $\qquad q_{roof} := 3\ psf$

ection: W10x12

$\frac{lbf}{ft^3}$ $\qquad A := 3.54\ in^2$ $\qquad I := 53.8\ in^4$ $\qquad E := 29000\ ksi$ $\qquad F_y := 50\ ksi$ $\qquad Z_x := 12.6\ in^3$

$A \cdot W = 0.295\ \frac{lbf}{ft}$

$_{oof} \cdot b_{trib} + w_{beam} = 18.295\ \frac{lbf}{ft}$

$18.9\ psf$ $\qquad w_s := p_{snow} \cdot b_{trib} = 113.4\ \frac{lbf}{ft}$

$15.8\ psf$ $\qquad w_w := q_{wind} \cdot b_{trib} = 94.8\ \frac{lbf}{ft}$

$_{Truss} := 15\ lbf + 500\ lbf = 515\ ft \cdot \frac{lbf}{ft}$

oad Combinations:

D
$D + 1.6L + (0.5L_r \text{ or } 0.3S \text{ or } 0.5R)$
$D + (1.6L_r \text{ or } 1.0S \text{ or } 1.6R) + (L \text{ or } 0.5W)$
$+ 1.0(W \text{ or } W_T) + L + (0.5L_r \text{ or } 0.3S \text{ or } 0.5R)$
$+ 1.0(W \text{ or } W_T)$

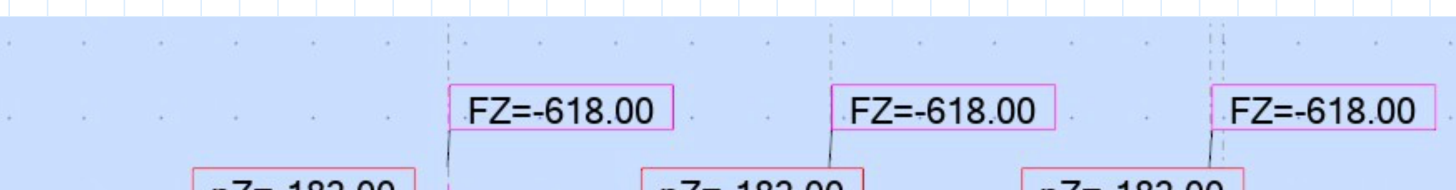
$4 \cdot w_d = 25.613\ \frac{lbf}{ft}$

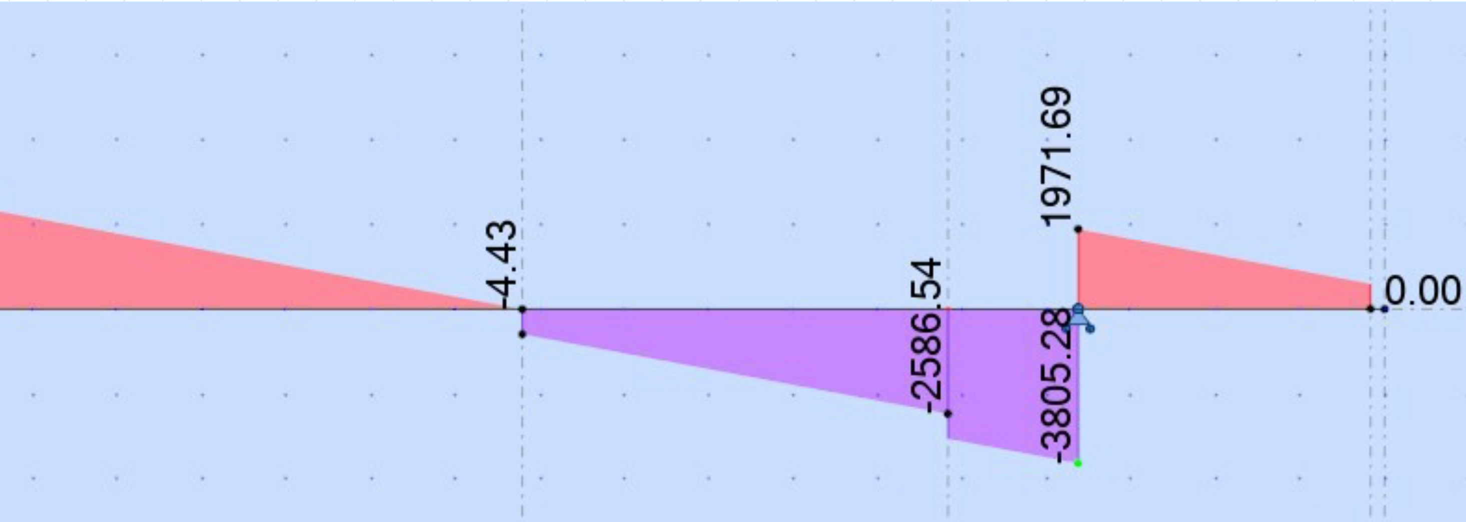
$2 \cdot w_d + w_s + 0.5 \cdot w_w = 182.754\ \frac{lbf}{ft}$

$2 \cdot w_d + w_w + 0.3 \cdot w_s = 150.774\ \frac{lbf}{ft}$

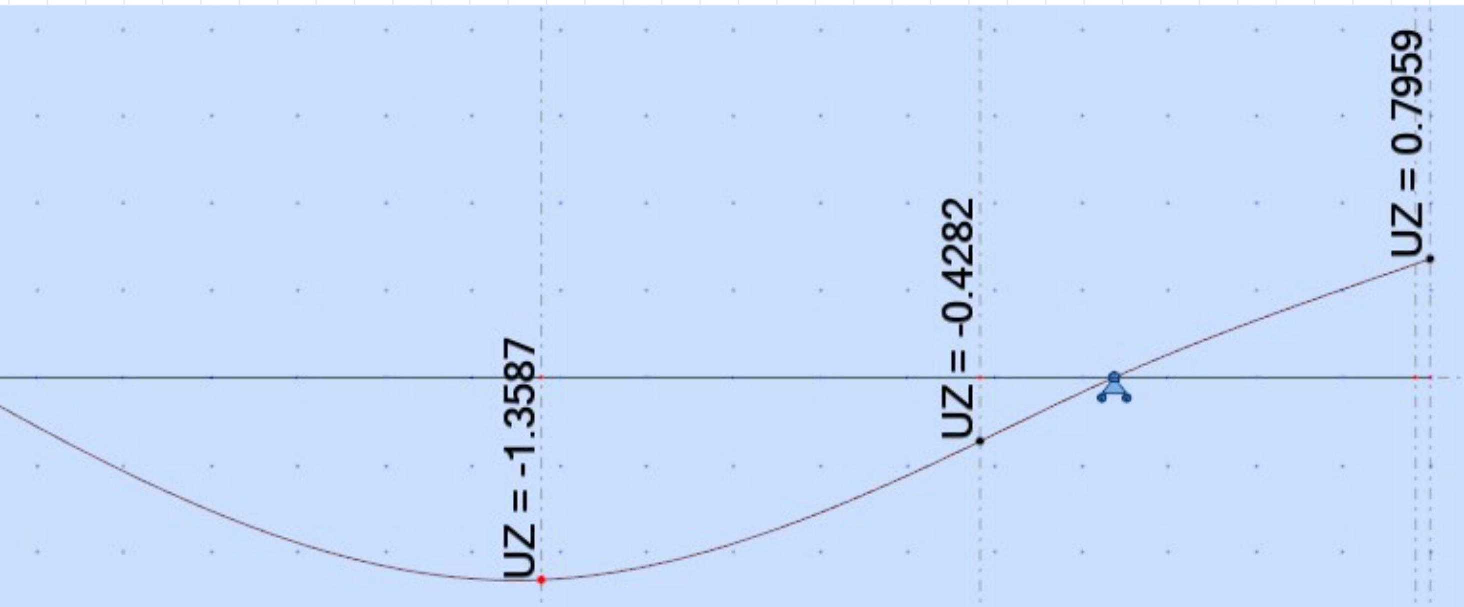
$(\left(w_1, w_2, w_3\right)) = 182.754\ \frac{lbf}{ft}$

$\cdot P_{LoadedTruss} = 618\ lbf$





agram for an interior roof beam (lbf.)



on diagram for an interior roof beam (in.)

parameters for a W10x12 beam:

Strength due to yielding: LTB doesn't apply because $L_b \leq L_p$

$_{wable} \coloneqq \phi_b \cdot F_y \cdot Z_x = 47.25\ kip \cdot ft$ $M_{max} \coloneqq 18\ kip \cdot ft$ OK

le deflection:

$_{le} \coloneqq \frac{L}{240} = 1.6964\ in$ $\Delta_{max} \coloneqq 1.36\ in$ OK

ength:

NT FRAME DESIGN:

n moment frame:

oads:

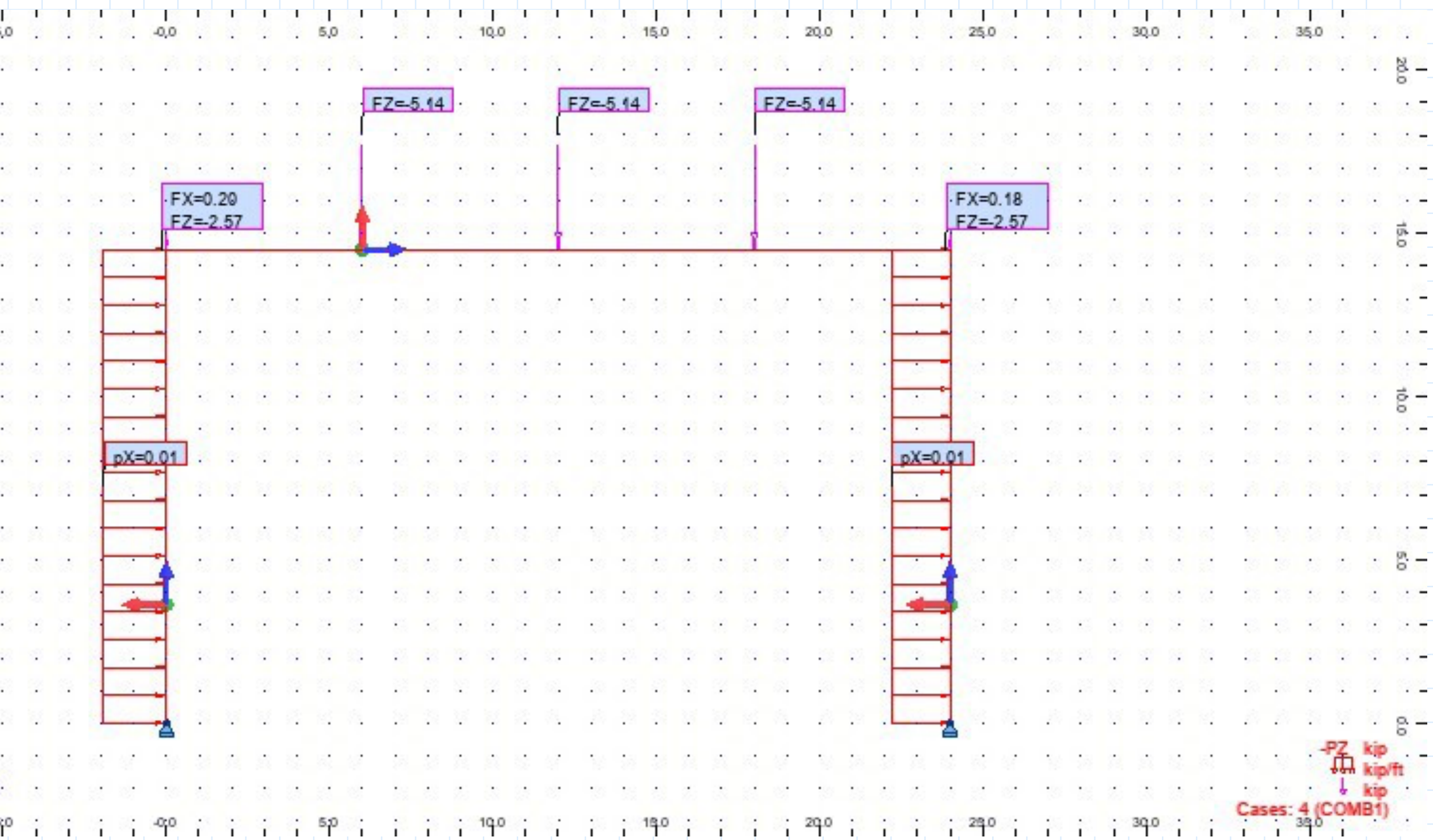
$V_{max} = 5.8\ kip$ $\qquad P_{ext} := \frac{V_{max}}{2} = 2.9\ kip$

oads NS: $\qquad b_f := 6.49\ in \qquad t_{roof} := \frac{13}{12}\ ft$

$:= p_{WW2} = 10.761\ psf \qquad p_{negative} := p_{SW1} = -10.0825\ psf$

$p_{positive} \cdot b_f = 0.0058\ \frac{kip}{ft} \qquad w_{neg} := p_{negative} \cdot b_f = -0.0055\ \frac{kip}{ft}$

$_s := \frac{1}{2} \cdot t_{roof} \cdot L \cdot p_{positive} = 0.1978\ kip \qquad P_{roof\_neg} := \frac{1}{2} \cdot t_{roof} \cdot L \cdot p_{negative} = -0.1853\ kip$



R DESIGN CAPACITIES:

00 $ksi \qquad F_y := 50\ ksi \qquad L := 24\ ft \qquad \phi_b := 0.9 \qquad L_b := 6\ ft$

Strength:

$$:= \text{if } L_b \le L_p$$
$$\phi_b \cdot F_y \cdot Z_x$$
$$\text{else if } L_p < L_b \le L_r$$
$$C_b \cdot \left(F_y \cdot Z_x - \left(F_y \cdot Z_x - 0.7 \cdot F_y \cdot S_x\right) \cdot \left(\frac{L_b - L_p}{L_r - L_p}\right)\right)$$
$$\text{else}$$
$$\frac{C_b \cdot \pi^2 \cdot E}{\left(\frac{L_b}{r_{ts}}\right)^2} \cdot \sqrt{1 + 0.078 \cdot \frac{J \cdot c}{S_x \cdot h_o} \cdot \left(\frac{L_b}{r_{ts}}\right)^2}$$

$= 213.75\ kip \cdot ft$

$= 213.75\ kip \cdot ft$

m deflection:

$$_{le\_g} := \frac{L}{240} = 1.2\ in$$

Table 5-1 (continued) Available Strength in Axial Tension W-Shapes

$F_y = 50$ ksi
$F_u = 65$ ksi

2

	Gross Area, A_g	$A_e = 0.75A_g$	Yielding kips P_n/Ω_t	Yielding kips $\phi_t P_n$	Rupture kips P_n/Ω_t	Rupture kips $\phi_t P_n$
	in.²	in.²	ASD	LRFD	ASD	LRFD
	11.2	8.40	335	504	273	410
	10.0	7.50	299	450	244	366
	8.85	6.64	265	398	216	324
	7.69	5.77	230	346	188	281
	6.49	4.87	194	292	158	237
h	98.9	74.2	2960	4450	2410	3620
h	89.5	67.1	2680	4030	2180	3270
h	81.9	61.4	2450	3690	2000	2990
h	74.1	55.6	2220	3330	1810	2710
h	67.7	50.8	2030	3050	1650	2480
	61.8	46.4	1850	2780	1510	2260
	56.0	42.0	1680	2520	1370	2050
	50.0	37.5	1500	2250	1220	1830
	44.7	33.5	1340	2010	1090	1630
	39.9	29.9	1190	1800	972	1460
	35.2	26.4	1050	1580	858	1290
	31.2	23.4	934	1400	761	1140
	28.2	21.2	844	1270	689	1030
	25.6	19.2	766	1150	624	936
	23.2	17.4	695	1040	566	848
	21.1	15.8	632	950	514	770
	19.1	14.3	572	860	465	697
	17.0	12.8	509	765	416	624
	15.6	11.7	467	702	380	570
	14.6	11.0	437	657	358	536
	13.1	9.83	392	590	319	479
	11.7	8.78	350	527	285	428
	10.3	7.73	308	464	251	377
	8.79	6.59	263	396	214	321
	7.65	5.74	229	344	187	280
	6.48	4.86	194	292	158	237
	5.57	4.18	167	251	136	204
	4.71	3.53	141	212	115	172
	4.16	3.12	125	187	101	152

ASD	LRFD
$\Omega_t = 1.67$	$\phi_t = 0.90$

[h] Flange thickness is greater than 2 in. Special requirements may apply per AISC *Specification* Section A3.1c.
Note: Tensile rupture on the effective net area will control over tensile yielding on the gross area unless the tension member is selected so that an end connection can be

Z_x Table 3-2 (continued) W-Shapes Selection by Z_x

$F_y = 50$ ksi

Shape	Z_x	M_{px}/Ω_b kip-ft	$\phi_b M_{px}$ kip-ft	M_{rx}/Ω_b kip-ft	$\phi_b M_{rx}$ kip-ft	BF/Ω_b kips	$\phi_b BF$ kips	L_p	L_r	I_x	V_{nx}/Ω_v kips	$\phi_v V_{nx}$ kips
	in.³	ASD	LRFD	ASD	LRFD	ASD	LRFD	ft	ft	in.⁴	ASD	LRFD
W21×44	**95.4**	**238**	**358**	**143**	**214**	**11.1**	**16.8**	**4.45**	**13.0**	**843**	**145**	**217**
W16×50	92.0	230	345	141	213	7.69	11.4	5.62	17.2	659	124	186
W18×46	90.7	226	340	138	207	9.63	14.6	4.56	13.7	712	130	195
W14×53	87.1	217	327	136	204	5.22	7.93	6.78	22.3	541	103	154
W12×58	86.4	216	324	136	205	3.82	5.69	8.87	29.8	475	87.8	132
W10×68	85.3	213	320	132	199	2.58	3.85	9.15	40.6	394	97.8	147
W16×45	82.3	205	309	127	191	7.12	10.8	5.55	16.5	586	111	167
W18×40	**78.4**	**196**	**294**	**119**	**180**	**8.94**	**13.2**	**4.49**	**13.1**	**612**	**113**	**169**
W14×48	78.4	196	294	123	184	5.09	7.67	6.75	21.1	484	93.8	141
W12×53	77.9	194	292	123	185	3.65	5.50	8.76	28.2	425	83.5	125
W10×60	74.6	186	280	116	175	2.54	3.82	9.08	36.6	341	85.7	129
W16×40	**73.0**	**182**	**274**	**113**	**170**	**6.67**	**10.0**	**5.55**	**15.9**	**518**	**97.6**	**146**
W12×50	71.9	179	270	112	169	3.97	5.98	6.92	23.8	391	90.3	135
W8×67	70.1	175	263	105	159	1.75	2.59	7.49	47.6	272	103	154
W14×43	69.6	174	261	109	164	4.88	7.28	6.68	20.0	428	83.6	125
W10×54	66.6	166	250	105	158	2.48	3.75	9.04	33.6	303	74.7	112
W18×35	**66.5**	**166**	**249**	**101**	**151**	**8.14**	**12.3**	**4.31**	**12.3**	**510**	**106**	**159**
W12×45	64.2	160	241	101	151	3.80	5.80	6.89	22.4	348	81.1	122
W16×36	64.0	160	240	98.7	148	6.24	9.36	5.37	15.2	448	93.8	141
W14×38	61.5	153	231	95.4	143	5.37	8.20	5.47	16.2	385	87.4	131
W10×49	60.4	151	227	95.4	143	2.46	3.71	8.97	31.6	272	68.0	102
W8×58	59.8	149	224	90.8	137	1.70	2.55	7.42	41.6	228	89.3	134
W12×40	57.0	142	214	89.9	135	3.66	5.54	6.85	21.1	307	70.2	105
W10×45	54.9	137	206	85.8	129	2.59	3.89	7.10	26.9	248	70.7	106
W14×34	**54.6**	**136**	**205**	**84.9**	**128**	**5.01**	**7.55**	**5.40**	**15.6**	**340**	**79.8**	**120**
W16×31	**54.0**	**135**	**203**	**82.4**	**124**	**6.86**	**10.3**	**4.13**	**11.8**	**375**	**87.5**	**131**
W12×35	51.2	128	192	79.6	120	4.34	6.45	5.44	16.6	285	75.0	113
W8×48	49.0	122	184	75.4	113	1.67	2.55	7.35	35.2	184	68.0	102
W14×30	**47.3**	**118**	**177**	**73.4**	**110**	**4.63**	**6.95**	**5.26**	**14.9**	**291**	**74.5**	**112**
W10×39	46.8	117	176	73.5	111	2.53	3.78	6.99	24.2	209	62.5	93.7
W16×26[v]	**44.2**	**110**	**166**	**67.1**	**101**	**5.93**	**8.98**	**3.96**	**11.2**	**301**	**70.5**	**106**
W12×30	43.1	108	162	67.4	101	3.97	5.96	5.37	15.6	238	64.0	95.9

ASD	LRFD
$\Omega_b = 1.67$	$\phi_b = 0.90$
$\Omega_v = 1.50$	$\phi_v = 1.00$

[v] Shape does not meet the h/t_w limit for shear in AISC *Specification* Section G2.1(a) with $F_y = 50$ ksi; therefore, $\phi_v = 0.90$ and $\Omega_v = 1.67$.

COLUMN DESIGN CAPACITIES (A3 and C3):

00 ksi $\quad F_y := 50\ ksi$ $\quad G := 11200\ ksi$ $\quad L := 14.5\ ft$ $\quad \phi_b := 0.9$ $\quad \phi := 0.9$

y drift

.348 in

N SIZE: W12x26

7 in^2 $\quad b_f := 8.01\ in$ $\quad t_w := 0.295\ in$ $\quad t_f := 0.515\ in$ $\quad d := 11.9\ in$ $\quad y_{bar} := \dfrac{d}{2}$

7 in^4 $\quad I_y := 44.1\ in^4$ $\quad S_x := 51.5\ in^3$ $\quad Z_x := 57\ in^3$ $\quad r_x := 5.13\ in$ $\quad r_y := 1.94\ in$

06 in^4 $\quad C_w := 1440\ in^6$

strength:

8 L

$$\frac{\pi^2 \cdot E}{\left(\frac{L_{ex}}{r_x}\right)^2} = 388.7353\ ksi$$

8 L

$$\frac{\pi^2 \cdot E}{\left(\frac{L_{ey}}{r_y}\right)^2} = 55.5933\ ksi$$

$3 \cdot L$

in $\quad y_o := y_{bar}$

$$\frac{I_x + I_y}{A} + x_o^2 + y_o^2 = 8.0877\ in$$

$$\frac{1}{\cdot r_o^2} \cdot \left(\frac{\pi^2 \cdot E \cdot C_w}{L_{ez}^2} + G \cdot J\right) = 41.0526\ ksi$$

$$n\left(F_{ex}, F_{ey}, F_{ez}\right) = 41.0526\ ksi$$

Strength of column:

$= 14.5\ ft$

$76 \cdot r_y \cdot \sqrt{\dfrac{E}{F_y}} = 6.8525\ ft$

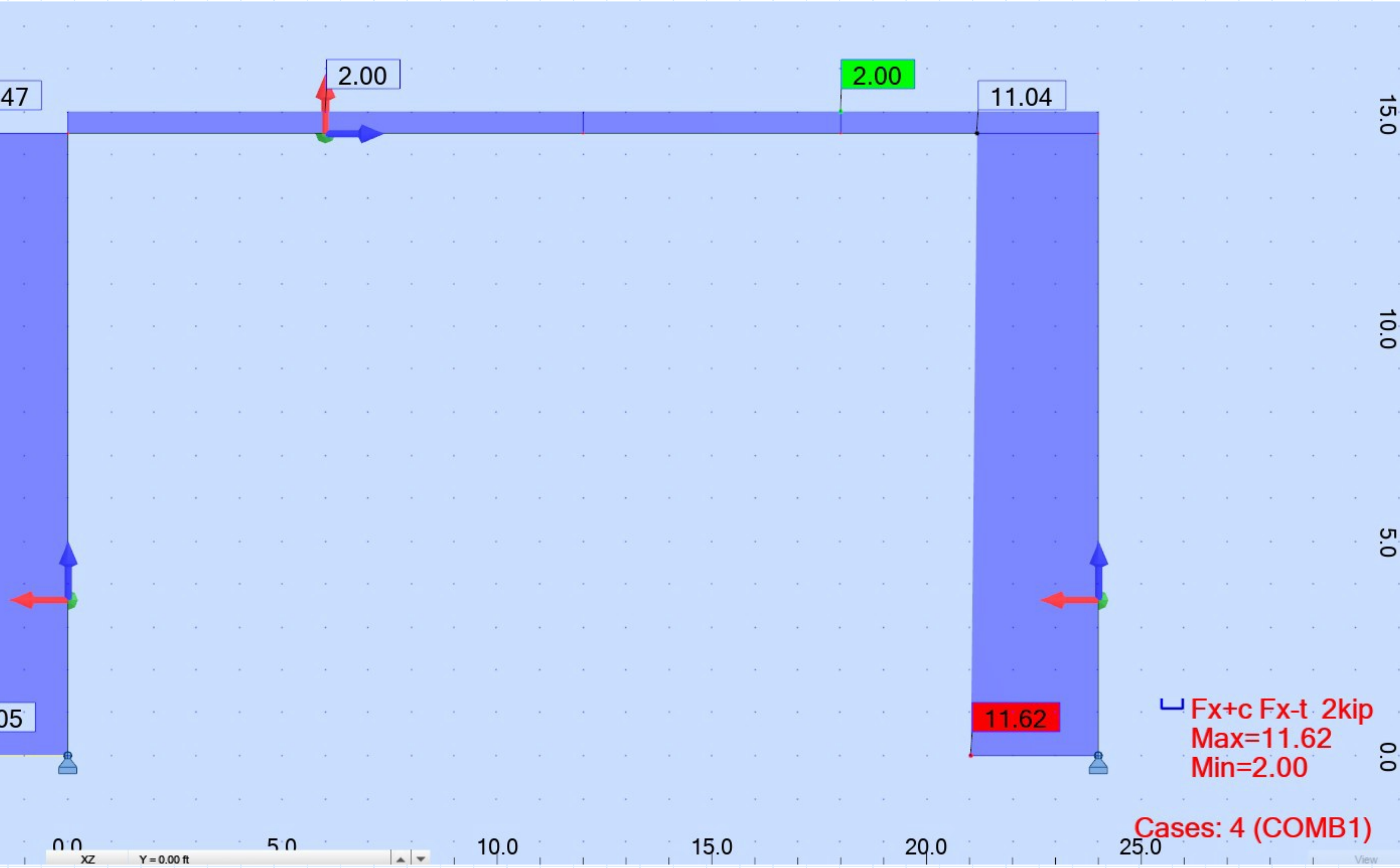
$\dfrac{\sqrt{I_y \cdot C_w}}{S_x} = 2.2121\ in$

$5 \cdot r_{ts} \cdot \dfrac{E}{0.7 \cdot F_y} \cdot \sqrt{\dfrac{J \cdot c}{S_x \cdot h_o} + \sqrt{\left(\dfrac{J \cdot c}{S_x \cdot h_o}\right)^2 + 6.76 \cdot \left(\dfrac{0.7 \cdot F_y}{E}\right)^2}} = 21.1445\ ft$

$$
= \left| \begin{array}{l}
\text{if } L_b \le L_p \\
\quad \phi_b \cdot F_y \cdot Z_x \\
\text{else if } L_p < L_b \le L_r \\
\quad C_b \cdot \left(F_y \cdot Z_x - \left(F_y \cdot Z_x - 0.7 \cdot F_y \cdot S_x\right) \cdot \left(\dfrac{L_b - L_p}{L_r - L_p}\right)\right) \\
\text{else} \\
\quad \dfrac{C_b \cdot \pi^2 \cdot E}{\left(\dfrac{L_b}{r_{ts}}\right)^2} \cdot \sqrt{1 + 0.078 \cdot \dfrac{J \cdot c}{S_x \cdot h_o} \cdot \left(\dfrac{L_b}{r_{ts}}\right)^2}
\end{array} \right| = 190.791\ kip \cdot ft
$$

$= 190.791\ kip \cdot ft$

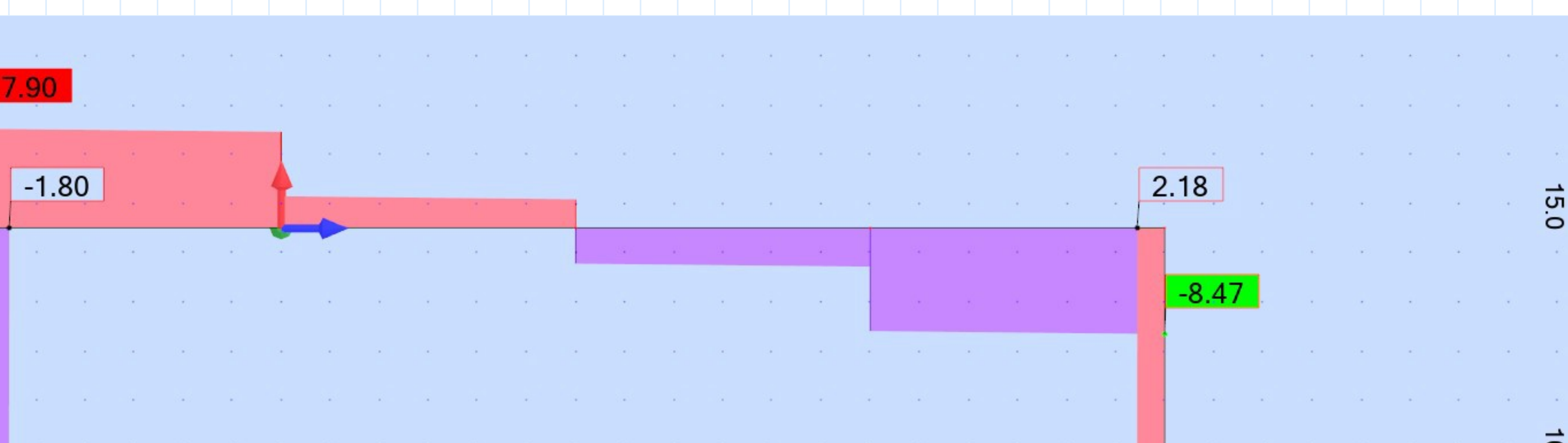
l Strengths:

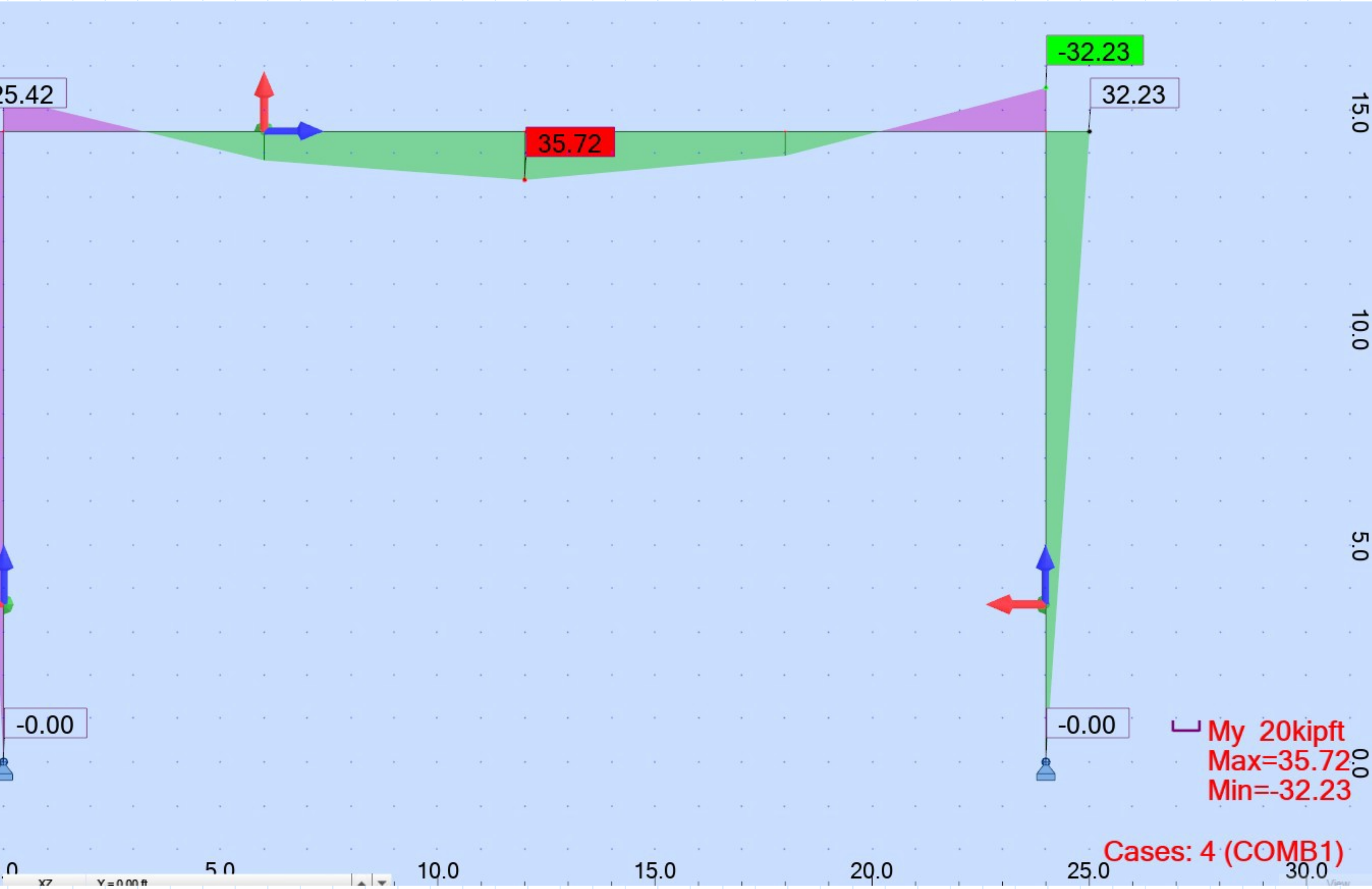


rce diagram for the moment frame

$= 527\ kip$ OK

$_{e\_c} = 480.3149\ kip$ OK

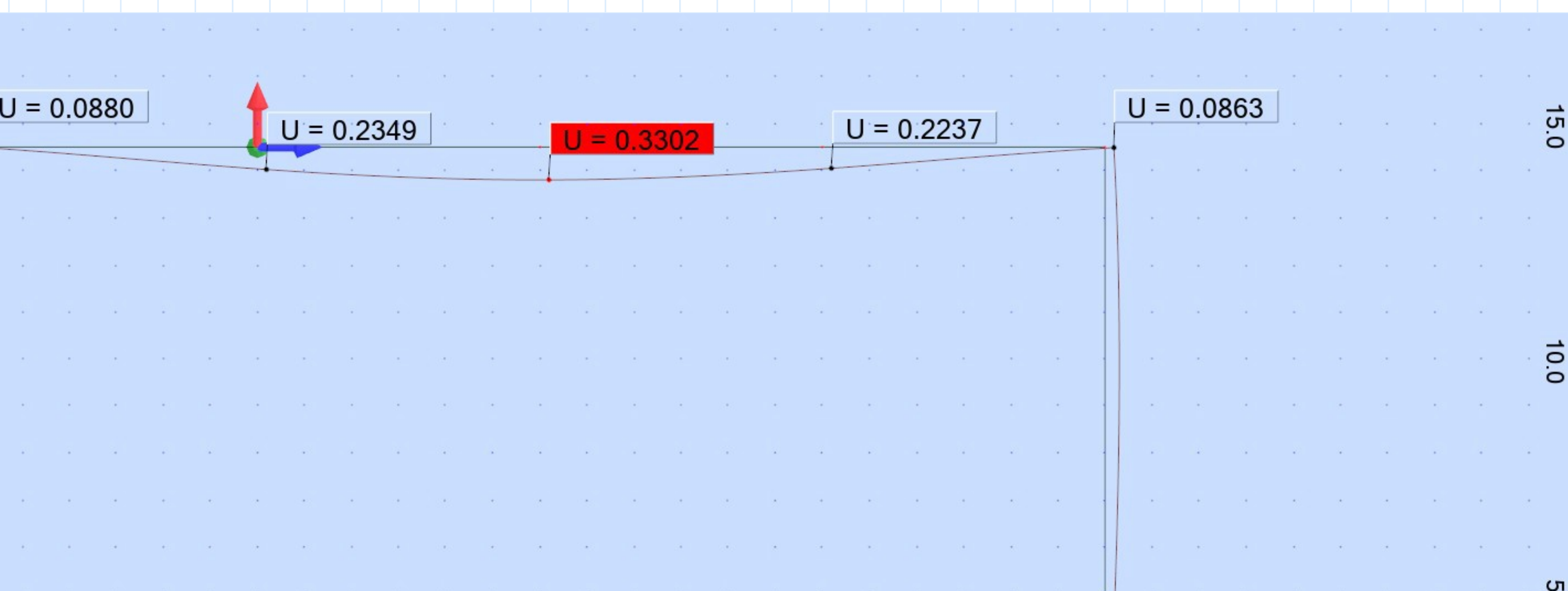




moment diagram for the moment frame

$= 213.75\ kip \cdot ft$ OK

$= 190.791\ kip \cdot ft$ OK



ed axial load and moment check (NOT INCLUDING OUT-OF-PLAIN MOMENT):

en $\frac{P_r}{P_c} \ge 0.2$

$$\frac{P_r}{P_c} + \frac{8}{9}\left(\frac{M_{rx}}{M_{cx}} + \frac{M_{ry}}{M_{cy}}\right) \le 1.0$$

en $\frac{P_r}{P_c} < 0.2$

$$\frac{P_r}{2P_c} + \left(\frac{M_{rx}}{M_{cx}} + \frac{M_{ry}}{M_{cy}}\right) \le 1.0$$

llowable_c $\qquad P_r := 11.67\ kip$

$_bM_{n\_c} \qquad M_r := 33\ kip \cdot ft$

0243 Use (b)

$\left(\frac{M_r}{M_c}\right) = 0.1851$ OK

For design according to Section B3.1 (LRFD):

P_r = required axial strength, determined in accordance with Chapter C, usi LRFD load combinations, kips (N)

$P_c = \phi_c P_n$ = design axial strength, determined in accordance with Chapter F kips (N)

M_r = required flexural strength, determined in accordance with Chapter using LRFD load combinations, kip-in. (N-mm)

$M_c = \phi_b M_n$ = design flexural strength determined in accordance with Chapte kip-in. (N-mm)

ϕ_c = resistance factor for compression = 0.90

ϕ_b = resistance factor for flexure = 0.90

$P_{n\_g} \qquad P_r := 2\ kip$

$_bM_{n\_g} \qquad M_r := 36\ kip \cdot ft$

0038 Use (b)

$\left(\frac{M_r}{M_c}\right) = 0.1703$ OK

For design according to Section B3.1 (LRFD):

P_r = required axial strength, determined in accordance with Chapter C, us LRFD load combinations, kips (N)

$P_c = \phi_t P_n$ = design axial strength, determined in accordance with Section I kips (N)

M_r = required flexural strength, determined in accordance with Chapter C, us LRFD load combinations, kip-in. (N-mm)

$M_c = \phi_b M_n$ = design flexural strength, determined in accordance with Chapte kip-in. (N-mm)

ϕ_t = resistance factor for tension (see Section D2)

ϕ_b = resistance factor for flexure = 0.90

<u>er Retaining Wall Footing Design in Exterior Stage Walls/Ramp Walls:</u>

riteria:

ckness: $t_{cmu} := FIF$ “8” $t_{brick} := FIF$ “4” $t_{air} := FIF$ “1”

USE SOLID GROUT

ight: $\gamma_{fill} := 120\ pcf$ $\gamma_{brick} := 120\ pcf$ $\gamma_{cmu} := 130\ pcf$ $\gamma_{conc} := 150\ pcf$

f safety: $FS_{overturning} := 1.5$ $FS_{sliding} := 1.5$ $FS_{bearing} := 3$

of wall: $H_{total} := FIF$ “7' ” $H_{wall} := FIF$ “4' ”   Need an extra 3 ft of height due to frost line

ss of footing/ $t_f := FIF$ “1' 8"” $t_s := 6\ in$

f footing: $B := FIF$ “3' 8”

f Footing: $D_f := 3\ ft + t_f$ DUE TO FROST LINE AT 4 ft.

arth Pressure:

$$K_a := \tan\left(\frac{\pi}{4} - \frac{\phi'}{2}\right)^2 = 0.3333$$

$$\cdot \gamma_{fill} \cdot \left(\left(H_{total} + t_f\right)\right)^2 \cdot K_a = 1.5022\ \frac{kip}{ft}$$

ing Moment:

$${}_{rning} := P_a \cdot \frac{\left(\left(H_{total} + t_f\right)\right)}{3} = 4.3398\ \frac{kip \cdot ft}{ft}$$

g Moment:

ent weights: moment arms:

$$W_m := \left(\left(t_{cmu}\right)\right) \cdot \left(\left(H_{wall}\right)\right) \cdot \gamma_{cmu} + \left(\left(t_{brick} \cdot \left(\left(H_{wall}\right)\right) \cdot \gamma_{brick}\right)\right) = 0.5067\ \frac{kip}{ft} \qquad r_m := \frac{B}{2} = 1.8333\ ft$$

$$W_e := \left(\frac{B}{2} - \left(0.5 \cdot \left(t_{cmu} + t_{brick}\right)\right)\right) \cdot H_{total} \cdot \gamma_{fill} = 1.12\ \frac{kip}{ft} \qquad r_e := B - 0.5\left(\frac{B}{2} - 0.5\left(t_{cmu} + t_{brick}\right)\right) =$$

on wall: $$W_{wall} := \left(\left(t_{brick} + t_{cmu} + t_{air} + 1\ in\right)\right) \cdot \left(\left(H_{total} - H_{wall}\right)\right) \cdot \gamma_{conc} = 0.525\ \frac{kip}{ft} \qquad r_{wall} := \frac{B}{2} = 1.8333\ ft$$

$$W_f := t_f \cdot B \cdot \gamma_{conc} = 0.9167\ \frac{kip}{ft} \qquad r_f := \frac{B}{2} = 1.8333\ ft$$

Pressure:

$50\ psf$

$\dfrac{\Sigma W + LL \cdot \left(\dfrac{B}{2} - \dfrac{t_{cmu}}{2}\right)}{B} = 939.0909\ psf$

capacity:

's continuous foundation equation:

psf $\qquad \phi \cdot \dfrac{180}{\pi} = 51.5662$ $B = 3.6667\ ft$ $\qquad \gamma' := \gamma_{fill} = 120\ pcf$

$.2$ $\qquad N_q := 22.5$ $\qquad N_\gamma := 20.1$

$D_f \cdot \gamma_{fill} = 560\ psf$

$' \cdot N_c + \sigma'_{zD} \cdot N_q + 0.5 \cdot \gamma' \cdot B \cdot N_\gamma = 17022\ psf$

$\dfrac{q_{ult}}{_{gross}} = 18.126$

$> FS_{bearing}$ $\qquad$ OK

or Settlement Failure: Boussinesq's Simple Elastic Settlement method

FIF "20' 0" $\qquad B = 3.6667\ ft$ $\quad H := 5 \cdot B$ $\quad \mu_s := 0.3$ $\quad \alpha := 4$ $\quad$ (Footing Center) $\qquad E_s := 750\ \dfrac{tonf}{ft^2}$ $\qquad \delta_{all} := 0.5\ in$

$\dfrac{L}{2} = 10\ ft$ $\qquad B' := \dfrac{B}{2} = 1.8333\ ft$

$\dfrac{L'}{B'} = 5.4545$ $\qquad N := \dfrac{H}{B'}$

ence Factors:

$\dfrac{1}{\pi} \cdot \left(M \cdot \ln\left(\dfrac{\left(1 + \sqrt{M^2 + 1}\right) \cdot \sqrt{M^2 + N^2}}{M \cdot \left(1 + \sqrt{M^2 + N^2 + 1}\right)}\right) + \ln\left(\dfrac{\left(M + \sqrt{M^2 + 1}\right) \cdot \sqrt{1 + N^2}}{M + \sqrt{M^2 + N^2 + 1}}\right)\right) = 0.7624$

$\dfrac{N}{2 \cdot \pi} \cdot \operatorname{atan}\left(\dfrac{M}{N \cdot \sqrt{M^2 + N^2 + 1}}\right) = 0.0759$

e Correction Factor:

$= r \cdot \text{atan}\left(\frac{L \cdot B}{r \cdot r_3}\right) = 3.1522\ ft$

$\beta_1 \cdot Y_1 + \beta_2 \cdot Y_2 + \beta_3 \cdot Y_3 + \beta_4 \cdot Y_4 + \beta_5 \cdot Y_5$

$\left(\left(\beta_1 + \beta_2\right)\right) \cdot Y_1$

ing Pressure:

$_s = 939.0909\ psf$

$= \left(\left(D_f\right)\right) \cdot \gamma_{fill} = 560\ psf$

$= q_{gross} - \sigma'_{zo} = 379.091\ psf$

dation Settlement:

$_{ble} := \alpha \cdot I_s \cdot I_f \cdot \left(\frac{q_{net} \cdot \left(\left(1 - \mu_s^{2}\right)\right)}{E_s}\right) \cdot B'$

$_{ble} = 0.012\ in$

$:= 0.93 \cdot \delta_{flexible} = 0.011\ in$ OK

$= 0.5\ in$

ilever Retaining Wall Vertical Reinforcement Design in Exterior Stage Walls/Ramp Walls:

Reinforcement:

$:= K_a \cdot \gamma_{fill} = 40\ pcf$

forcement: No. 4 @ 16in

Table 2—Strength Design: Vertical Reinforcement for Cantilever Retaining Walls[a, b]

Wall thickness, in. (mm)	Wall height, H, ft (m)	Reinforcement size & spacing for equivalent fluid weight of soil, lb/ft²/ft (kN/m²/m), of: 30 (4.7)	45 (7.1)	60 (9.4)
8 (203)	4.0 (1.2)	No.4 @ 120 in.	No.4 @ 96 in.	No.4 @ 64 in.
	4.7 (1.4)	No.4 @ 88 in.	No.4 @ 56 in.	No.4 @ 40 in.
	5.3 (1.6)	No.4 @ 56 in.	No.4 @ 32 in.	No.4 @ 24 in.
	6.0 (1.8)	No.4 @ 32 in.	No.4 @ 24 in.	No.4 @ 16 in.
	6.7 (2.0)	No.4 @ 24 in.	No.4 @ 16 in.	No.5 @ 16 in.

er Retaining Wall Footing Design for Stage Solid Backwall (East side - grid 5):

riteria:

ckness: $t_{cmu} := FIF$ “8” $t_{brick} := FIF$ “8” $t_{air} := FIF$ “1”

USE SOLID GROUT

ight: $\gamma_{fill} := 120\ pcf$ $\gamma_{brick} := 120\ pcf$ $\gamma_{cmu} := 130\ pcf$ $\gamma_{conc} := 150\ pcf$

f safety: $FS_{overturning} := 1.5$ $FS_{sliding} := 1.5$ $FS_{bearing} := 3$

f wall: $H_{wall} := FIF$ “16' ” $H_{total} := FIF$ “3'” $+ H_{wall}$ $H_s := FIF$ “6' 4”

Need an extra 3 ft of height due to frost lin

ss of footing/ $t_f := FIF$ “1' 8”” $t_s := 6\ in$

f footing: $B := FIF$ “3' 8”

f Footing: $D_f := 3\ ft + t_f$ DUE TO FROST LINE AT 4 ft.

oads: $P := 2.65\ kip$ $P_{beam} := \dfrac{P}{b_{trib}} = 0.4417\ \dfrac{kip}{ft}$

oads:

arth Pressure:

$$K_a := \tan\left(\frac{\pi}{4} - \frac{\phi'}{2}\right)^2 = 0.3333$$

$$\cdot \gamma_{fill} \cdot \left(\left(H_s + t_f\right)\right)^2 \cdot K_a = 1.28\ \frac{kip}{ft}$$

$$\left(\left(p_{positive} - p_{negative}\right)\right) \cdot \left(\left(H_{total} - H_s - t_s\right)\right) = 0.2536\ \frac{kip}{ft}$$

ing Moment:

$$_{rning} := P_a \cdot \frac{\left(\left(H_s + t_f\right)\right)}{3} + P_{wind} \cdot \left(\frac{H_{total} - H_s - t_s}{2} + t_s + H_s + t_f\right) = 7.1116\ \frac{kip \cdot ft}{ft}$$

g Moment:

ent weights: moment arms:

$$W_m := \left(\left(t_{cmu}\right)\right) \cdot \left(\left(H_{wall}\right)\right) \cdot \gamma_{cmu} + \left(\left(2 \cdot t_{brick} \cdot \left(\left(H_{wall}\right)\right) \cdot \gamma_{brick}\right)\right) = 3.9467\ \frac{kip}{ft}$$

$$r_m := \frac{B}{2} = 1.8333\ ft$$

$W_m + W_e + W_f + W_b + W_{wall} = 6.9133\ \frac{kip}{ft}$

$:= \frac{\Sigma W \cdot \tan\left(\phi'\right)}{P_a} = 3.1183$ $\qquad FS_{sliding} = 1.5$

$> FS_{sliding}$ OK

Pressure:

$50\ psf$

$\frac{\Sigma W + LL \cdot \left(B - t_{cmu} - 2\ t_{brick} - 2 \cdot t_{air}\right) + \left(H_{total} - H_{wall}\right) \cdot \left(\frac{B}{2} - \frac{t_{cmu}}{2} - t_{brick} - t_{air}\right) \cdot \gamma_{fill}}{B} = 2061.3636\ psf$

capacity:

's continuous foundation equation:

$psf \qquad \phi \cdot \frac{180}{\pi} = 51.5662 \quad B = 3.6667\ ft \qquad \gamma' := \gamma_{fill} = 120\ pcf$

$.2 \qquad N_q := 22.5 \qquad N_\gamma := 20.1$

$D_f \cdot \gamma_{fill} = 560\ psf$

$\cdot N_c + \sigma'_{zD} \cdot N_q + 0.5 \cdot \gamma' \cdot B \cdot N_\gamma = 17022\ psf$

$\frac{q_{ult}}{_{gross}} = 8.2576$

$> FS_{bearing}$ OK

or Settlement Failure:

"32' 5" $\qquad B = 3.6667\ ft \qquad H := 5 \cdot B \qquad \mu_s := 0.3 \qquad \alpha := 4$ (Footing Center) $\qquad E_s := 750\ \frac{tonf}{ft^2} \qquad \delta_{all} := 0.5\ in$

$= 16.2083\ ft \qquad B' := \frac{B}{2} = 1.8333\ ft$

$' = 8.8409 \qquad N := \frac{H}{B'}$

e Factors:

$$\ln\left(\frac{r_3+B}{r_1}\right)+B\cdot\ln\left(\frac{r_3+L}{r_2}\right)-\frac{r_3^3-r_2^3-r_1^3+r^3}{3\cdot L\cdot B}=9.0793\ ft$$

$$\cdot\ln\left(\frac{\left(B+r_2\right)\cdot r_1}{\left(B+r_3\right)\cdot r}\right)+\frac{r^2}{B}\cdot\ln\left(\frac{\left(L+r_1\right)\cdot r_2}{\left(L+r_3\right)\cdot r}\right)=2.3724\ ft$$

$$\frac{\cdot\left(r_1+r_2-r_3-r\right)}{L\cdot B}=0.3633\ ft$$

$$\text{atan}\left(\frac{L\cdot B}{r\cdot r_3}\right)=3.3511\ ft$$

$$\frac{\cdot Y_1+\beta_2\cdot Y_2+\beta_3\cdot Y_3+\beta_4\cdot Y_4+\beta_5\cdot Y_5}{\left(\beta_1+\beta_2\right)\cdot Y_1}=0.7811$$

Pressure:

$2061.3636\ psf$

$D_f)) \cdot \gamma_{fill}=560\ psf$

${}'_{gross}-\sigma'_{zo}=1501.364\ psf$

ion Settlement:

$$=\alpha\cdot I_s\cdot I_f\cdot\left(\frac{q_{net}\cdot\left(1-\mu_s^2\right)}{E_s}\right)\cdot B'$$

$=0.052\ in$

$0.93\cdot\delta_{flexible}=0.048\ in$ OK

$5\ in$

er Retaining Wall Vertical Reinforcement Design for Stage Solid Backwall (East side - grid 5):

$=10.761\ psf$

$$\left(p_{positive}-p_{negative}\right)\cdot\frac{FIF\ \text{“12' 8”}^2}{2}\cdot 8\ in=13376.8717\ lbf\cdot in$$

Table 4—Allowable Shear and Moment Capacities for Concrete Masonry Lintels (width x height) (continued)[a]

		Bottom cover, in. (mm), of:							
	No.	1.5(38)		2(51)		2.5(64)		3(76)	
el	of	V_{all}	M_{all}	V_{all}	M_{all}	V_{all}	M_{all}	V_{all}	M_{all}
ze	bars	lb	in.-lb	lb	in.-lb	lb	in.-lb	lb	in.-lb

er Retaining Wall Footing Design for Stage Split Backwalls (East side - grid 4):

riteria:

kness: $t_{cmu} := FIF$ "8" $t_{brick} := FIF$ "4" $t_{air} := FIF$ "1"

USE SOLID GROUT

ght: $\gamma_{fill} := 120\ pcf$ $\gamma_{brick} := 120\ pcf$ $\gamma_{cmu} := 130\ pcf$ $\gamma_{conc} := 150\ pcf$

safety: $FS_{overturning} := 1.5$ $FS_{sliding} := 1.5$ $FS_{bearing} := 3$

f wall: $H_{wall} := FIF$ "16' " $H_{total} := FIF$ "3'" $+ H_{wall}$ $H_s := FIF$ "6' 4"

Need an extra 3 ft of height due to frost line

s of footing/ $t_f := FIF$ "1' 10""" $t_s := 6\ in$

footing: $B := FIF$ "3' 8" Overdesigned for consistent footing size

Footing: $D_f := 3\ ft + t_f$ DUE TO FROST LINE AT 4 ft.

oads:

arth Pressure:

$$K_a := \tan\left(\frac{\pi}{4} - \frac{\phi'}{2}\right)^2 = 0.3333$$

$$\cdot \gamma_{fill} \cdot \left(H_s + t_f\right)^2 \cdot K_a = 1.3339\ \frac{kip}{ft}$$

$$\left(\left(p_{positive} - p_{negative}\right)\right) \cdot \left(\left(H_{total} - H_s - t_s\right)\right) = 0.2536\ \frac{kip}{ft}$$

ing Moment:

$$_{ning} := P_a \cdot \frac{\left(\left(H_s + t_f\right)\right)}{3} + P_{wind} \cdot \left(\frac{H_{total} - H_s - t_s}{2} + H_s + t_f\right) = 7.2449\ \frac{kip \cdot ft}{ft}$$

Moment:

nt weights:

moment arms:

$$W_m := \left(\left(t_{cmu}\right)\right) \cdot \left(\left(H_{wall}\right)\right) \cdot \gamma_{cmu} + \left(\left(2 \cdot t_{brick} \cdot \left(\left(H_{wall}\right)\right) \cdot \gamma_{brick}\right)\right) = 2.6667\ \frac{kip}{ft}$$

$$r_m := \frac{B}{2} = 1.8333\ ft$$

$$W_e := \left(\left(B - t_{cmu} - 2 \cdot t_{brick} - 2 \cdot t_{air}\right)\right) \cdot H_s \cdot \gamma_{fill} = 1.6467\ \frac{kip}{ft}$$

$$r_e := \frac{B}{2} = 1.8333\ ft$$

on wall: $$W_{wall} := \left(\left(2 \cdot t_{brick} + t_{cmu} + 2 \cdot t_{air}\right)\right) \cdot \left(\left(H_{total} - H_{wall}\right)\right) \cdot \gamma_{conc} = 0.675\ \frac{kip}{ft}$$

$$r_{wall} := \frac{B}{2} = 1.8333\ ft$$

$W_m + W_e + W_f + W_{wall} = 5.9967\ \frac{kip}{ft}$

$:= \frac{\Sigma W \cdot \tan \phi' + P_p}{P_a} = 11.5956$ $FS_{sliding} = 1.5$

$> FS_{sliding}$ OK

Pressure:

$50\ psf$

$\frac{\Sigma W + LL \cdot ((B - t_{cmu} + 2\ t_{brick}))}{B} = 1885.4545\ psf$

capacity:

's continuous foundation equation:

psf $\phi \cdot \frac{180}{\pi} = 51.5662$ $B = 3.6667\ ft$ $\gamma' := \gamma_{fill} = 120\ pcf$

$.2$ $N_q := 22.5$ $N_\gamma := 20.1$

$D_f \cdot \gamma_{fill} = 580\ psf$

$' \cdot N_c + \sigma'_{zD} \cdot N_q + 0.5 \cdot \gamma' \cdot B \cdot N_\gamma = 17472\ psf$

$\frac{q_{ult}}{_{gross}} = 9.2667$

$> FS_{bearing}$ OK

or Settlement Failure:

FIF “8'” $B = 3.6667\ ft$ $H := 5 \cdot B$ $\mu_s := 0.3$ $\alpha := 4$ (Footing Center) $E_s := 750\ \frac{tonf}{ft^2}$ $\delta_{all} := 0.5\ in$

$\frac{L}{2} = 4\ ft$ $B' := \frac{B}{2} = 1.8333\ ft$

Depth Correction Factor, If:

$3-4\cdot\mu_s$ $\quad \beta_2 := 5-12\cdot\mu_s+8\cdot\mu_s^2$ $\quad \beta_3 := -4\cdot\mu_s\cdot(1-2\cdot\mu_s)$ $\quad \beta_4 := -1+4\cdot\mu_s-8\cdot\mu_s^2$ $\quad \beta_5 := -4\cdot(1-2\cdot\mu_s)^2$

$\cdot D_f = 9.6667\ ft$

$\sqrt{L^2+r^2} = 12.5477\ ft$ $\quad r_2 := \sqrt{B^2+r^2} = 10.3387\ ft$ $\quad r_3 := \sqrt{L^2+B^2+r^2} = 13.0724\ ft$ $\quad r_4 := \sqrt{L^2+B^2} = 8.$

$L\cdot\ln\left(\frac{r_4+B}{L}\right)+B\cdot\ln\left(\frac{r_4+L}{B}\right)-\frac{r_4^3-L^3-B^3}{3\cdot L\cdot B} = 7.7639\ ft$

$L\cdot\ln\left(\frac{r_3+B}{r_1}\right)+B\cdot\ln\left(\frac{r_3+L}{r_2}\right)-\frac{r_3^3-r_2^3-r_1^3+r^3}{3\cdot L\cdot B} = 4.2737\ ft$

$\frac{r^2}{L}\cdot\ln\left(\frac{(B+r_2)\cdot r_1}{(B+r_3)\cdot r}\right)+\frac{r^2}{B}\cdot\ln\left(\frac{(L+r_1)\cdot r_2}{(L+r_3)\cdot r}\right) = 2.0344\ ft$

$\frac{r^2\cdot(r_1+r_2-r_3-r)}{L\cdot B} = 0.4692\ ft$

$r\cdot\operatorname{atan}\left(\frac{L\cdot B}{r\cdot r_3}\right) = 2.2049\ ft$

$\frac{\beta_1\cdot Y_1+\beta_2\cdot Y_2+\beta_3\cdot Y_3+\beta_4\cdot Y_4+\beta_5\cdot Y_5}{(\beta_1+\beta_2)\cdot Y_1} = 0.6704$

ing Pressure:

$_s = 1885.4545\ psf$

$= 560\ psf$

$= q_{gross} - \sigma'_{zo} = 1325.455\ psf$

dation Settlement:

$_{ble} := \alpha\cdot I_s\cdot I_f\cdot\left(\frac{q_{net}\cdot(1-\mu_s^2)}{E_s}\right)\cdot B'$

$_{ble} = 0.032\ in$

$:= 0.93\cdot\delta_{flexible} = 0.03\ in$ OK

$0.5\ in$

ilever Retaining Wall Vertical Reinforcement Design for Stage Split Backwalls (East side - grid 4):

f Masonry Columns for Steel Columns to Transfer Load into foundation:

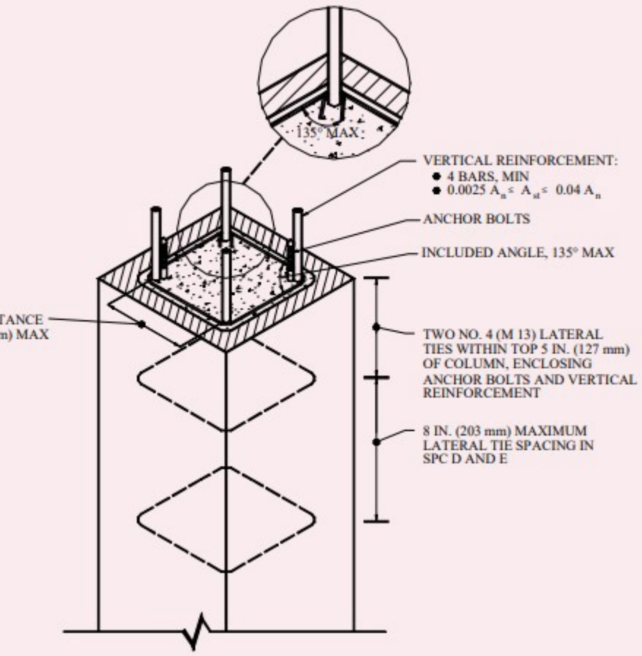


Figure 2—Additional Requirements for Column
forcement in Buildings Assigned to SPC C, D and E

$P_{axial} := 11.6\ kip$

Due to beam depth being 12 in., 12 x 16 masonry column is required.

$H_{wall} := H_s = 6.3333\ ft$

Number of bars: 4

Bar Size: No. 4

2—Allowable Column Compressive Force for
ntrically Loaded Concrete Masonry Columns
up to 20 ft (6.1 m) High[1]

Column ze, in. (mm)	Allowable column compressive force, kip (kN)
8 (203x203)	18[2] (80)
16 (203x406)	37[2,3] (165)
24 (203x610)	56[2,4] (249)
16 (254x406)	46[5] (205)
24 (254x610)	71[6] (316)
12 (305x305)	42 (186)
16 (305x406)	56 (249)
24 (305x610)	85 (378)
32 (305x813)	114 (507)
16 (406x406)	76 (338)
24 (406x610)	115 (511)
32 (406x813)	154 (685)
24 (610x610)	174 (773)
32 (610x813)	233 (1030)
40 (610x1016)	292 (1300)

le assumes the element is in pure compression, i.e., that the
ad falls within the center one-third of the section, under a
um design eccentricity of 0.1t for each axis as required by the
The designer must ensure the section is in compression prior
g the table. f'_m = 1500 psi (10.3 MPa). F_s = 24,000 psi (165

SLENDERNESS LIMITATIONS:

[2] The maximum allowable height for 8 in. columns is 15.9 ft (

	Height ft (m)	Number of bars	Bar size	Maximum kips (k
3	15.1-15.9 (4.6-4.8)	4	No. 4 (M13)	34 (151
4	14.0-15.9 (4.3-4.8)	4	No. 4 (M13)	48 (213
	15.1-15.9 (4.6-4.8)	6	"	52 (231
	15.3-15.9 (4.6-4.8)	4	No. 5 (M16)	53 (236
5	18.6-20 (5.6-6.1)	4	No. 4 (M13)	42 (186
6	16.9-18 (5.1-5.5)	4	No. 4 (M13)	67 (298
	18.0-20 (5.5-6.1)	4	"	60 (266
	18.2-20 (5.5-6.1)	6	"	64 (284
	19.3-20 (5.9-6.1)	8	"	68 (302
	18.3-20 (5.6-6.1)	4	No. 5 (M16)	64 (284
	19.7-20 (6.0-6.1)	6	"	70 (311
	19.7-20 (6.0-6.1)	4	No. 6 (M19)	69 (307

Isolated Footing Design (B3 and C3):

esign Parameters:

F “3' 8” $H := 3\ ft$ $B_{col} := FIF$ “1' 4” $t_f := FIF$ “1' 8” $P_{des} := 11\ kip$

$ft + t_f$

$H + t_f = 4.6667\ ft$

or Bearing Failure:

Equation for Bearing Capacity:

$\cdot N_c \cdot s_c \cdot d_c \cdot i_c \cdot b_c \cdot g_c + \sigma'_z \cdot N_q \cdot s_q \cdot d_q \cdot i_q \cdot b_q \cdot g_q + 0.5 \cdot \gamma_{backfill} \cdot B' \cdot N_\gamma \cdot s_\gamma \cdot d_\gamma \cdot i_\gamma \cdot b_\gamma \cdot g_\gamma$

Capacity Factors:

le of bearing capacity factors for ϕ' = 30 degrees and using Vesic's Equation:

.4 $N_\gamma := 22.4$

Inclination Factors:

level ground, therefore, ground inclination factors are equal to 1:

$g_\gamma := 1$

clination Factors:

$i_\gamma := 1$

lination Factors:

is not inclined, therefore the base inclination factors are equal to 1:

$b_\gamma := 1$

For continuous footings, $B/L \rightarrow 0$, so s_c, s_q, and s_γ become equal to 1. This means factors may be ignored when analyzing continuous footings.

In line with continuous footing

actors:

$s_\gamma := 1$

actors:

$-2 \cdot \frac{D_f}{B} \cdot \tan\phi' \cdot (1 - \sin\phi')^2 = 1.3674$ $d_\gamma := 1$

or Settlement Failure:

$B = 3.6667\ ft$ $\quad H := 5 \cdot B$ $\quad \mu_s := 0.3$ $\quad \alpha := 4$ (Footing Center) $\quad \delta_{all} := 0.5\ in$

$= 1.8333\ ft$ $\quad B' := \dfrac{B}{2} = 1.8333\ ft$ $\quad M := \dfrac{L'}{B'} = 1$ $\quad N := \dfrac{H}{B'}$

e Factors:

$$\cdot \left(M \cdot \ln\left(\frac{\left(1 + \sqrt{M^2 + 1}\right) \cdot \sqrt{M^2 + N^2}}{M \cdot \left(1 + \sqrt{M^2 + N^2 + 1}\right)} \right) + \ln\left(\frac{\left(M + \sqrt{M^2 + 1}\right) \cdot \sqrt{1 + N^2}}{M + \sqrt{M^2 + N^2 + 1}} \right) \right) = 0.4979$$

$$\frac{N}{\pi} \cdot \operatorname{atan}\left(\frac{M}{N \cdot \sqrt{M^2 + N^2 + 1}} \right) = 0.0158$$

orrection Factor:

$$\left(\frac{1 - 2 \cdot \mu_s}{1 - \mu_s} \right) \cdot I_2 = 0.5069$$

th Correction Factor, If:

$4 \cdot \mu_s$ $\quad \beta_2 := 5 - 12 \cdot \mu_s + 8 \cdot \mu_s^2$ $\quad \beta_3 := -4 \cdot \mu_s \cdot (1 - 2 \cdot \mu_s)$ $\quad \beta_4 := -1 + 4 \cdot \mu_s - 8 \cdot \mu_s^2$ $\quad \beta_5 := -4 \cdot (1 - 2 \cdot \mu_s)^2$

$D_f = 9.3333\ ft$

$L^2 + r^2 = 10.0277\ ft$

$B^2 + r^2 = 10.0277\ ft$

$L^2 + B^2 + r^2 = 10.6771\ ft$

$L^2 + B^2 = 5.1854\ ft$

$$\ln\left(\frac{r_4 + B}{L} \right) + B \cdot \ln\left(\frac{r_4 + L}{B} \right) - \frac{r_4^3 - L^3 - B^3}{3 \cdot L \cdot B} = 5.4509\ ft$$

$$\ln\left(\frac{r_3 + B}{r_1} \right) + B \cdot \ln\left(\frac{r_3 + L}{r_2} \right) - \frac{r_3^3 - r_2^3 - r_1^3 + r^3}{3 \cdot L \cdot B} = 2.2895\ ft$$

$$\cdot \ln\left(\frac{(B + r_2) \cdot r_1}{(B + r_3) \cdot r} \right) + \frac{r^2}{B} \cdot \ln\left(\frac{(L + r_1) \cdot r_2}{(L + r_3) \cdot r} \right) = 1.2086\ ft$$

of Rebar in Continuous Foundation For Stage and Ramp Retaining Walls:

riteria:

ckness: $t_{cmu} := FIF$ "8" $t_{brick} := FIF$ "4" $t_{air} := FIF$ "1" $B_{wall} := 8\ in$

USE SOLID GROUT

ight: $\gamma_{fill} := 120\ pcf$ $\gamma_{brick} := 120\ pcf$ $\gamma_{cmu} := 130\ pcf$ $\gamma_{conc} := 150\ pcf$

f safety: $FS_{overturning} := 1.5$ $FS_{sliding} := 1.5$ $FS_{bearing} := 3$

of wall: $H_{total} := FIF$ "7' 1" $H_{wall} := FIF$ "4' 1" Need an extra 3 ft of height due to frost line

ss of footing/ $t_f := FIF$ "1' 8""

f footing: $B := FIF$ "3' 8"

f Footing: $D_f := 3\ ft + t_f$ DUE TO FROST LINE AT 4 ft.

of and steel: $f'_c := 4000\ psi$ $f_y := 60\ ksi$

APPENDIX B—STEEL REINFORCEMENT INFORMATION

aid to users of the ACI Building Code, information on sizes, areas, and weights of various steel reinforcement is .

TANDARD REINFORCING BARS

Bar size, no.	Nominal diameter, in.	Nominal area, in.^{2}	Nominal weight, lb/ft
3	0.375	0.11	0.376
4	0.500	0.20	0.668
5	0.625	0.31	1.043
6	0.750	0.44	1.502
7	0.875	0.60	2.044
8	1.000	0.79	2.670
9	1.128	1.00	3.400
10	1.270	1.27	4.303
11	1.410	1.56	5.313
14	1.693	2.25	7.65
18	2.257	4.00	13.60

One-Way Shear Strength:

e depth of footing:

inuous footings, effective depth d is measured from the top of the footing to the center of the lateral bars. linal bars are designed separately:

a 3in clear cover, #6 rebars:

n $D_{\#6} := 0.75\ in$

$= c_c + \dfrac{D_{\#6}}{2} = 3.375\ in$

ft (Long dimension. Use 1 ft analysis strip)

(short dimension)

l (width of wall)

$_m + W_{wall} = 1.0047\ klf$

$_{ay} := P_u \cdot \left(\frac{B - c - 2 \cdot d}{B}\right) = 0.0628\ klf$

$_{Way} := \frac{0.75 \cdot \left(2 \cdot \lambda \cdot \sqrt{f'_c \cdot psi} \cdot L_2 \cdot d\right)}{1\ ft} = 69.3962\ klf$

= if $V_{uOneWay} < \phi V_{cOneWay}$
‖ "The footing has adequate shear strength"
else
‖ "The footing has inadequate shear strength "

= "The footing has adequate shear strength"

lexural Strength:

$\frac{c}{2} = 1.6667\ ft$

$\frac{_u \cdot l^2}{2 \cdot B} = 0.381\ \frac{kip \cdot ft}{ft}$ (required flexural resistance)

0.0018

$\rho_{min} \cdot d \cdot \frac{12\ in}{1 \cdot ft} = 0.3591\ \frac{in^2}{ft}$

Rebars: $A_{\#6} := 0.44\ in^2$

$A_{\#6} = 0.44\ in^2$ 1 #6 bar is adequate

35

$${}_{nControlled} := \frac{\beta_1 \cdot 0.85 \cdot f'_c \cdot b}{f_y} \cdot \frac{3 \cdot d_t}{8} = 13.2127\ in^2$$

The design is tension controlled $A_s \le A_{s\_TC}$

Concrete Compression block:

$$\frac{A_s \cdot f_y}{.85 \cdot f'_c \cdot b} = 0.1765\ in$$

$$3806\ \frac{kip \cdot ft}{ft}$$

$$_s \cdot f_y \cdot \left(d - \frac{a}{2}\right) = 36.3809\ kip \cdot ft$$

$$\frac{0.9 \cdot M_n}{1\ ft} = 32.7428\ \frac{kip \cdot ft}{ft}$$ (flexural strength)

$$= \text{if } M_u < \phi M_n$$
$$\quad \text{"The footing has adequate flexural strength"}$$
$$\text{else}$$
$$\quad \text{"The footing has inadequate flexural strength "}$$

= "The footing has adequate flexural strength"

Development Length of Flexural 180 degree Hooked Rebars:

rvative assumptions:

$$0 \quad \psi_r := 1.0 \quad \psi_o := 1.0 \quad \psi_c := 0.6 + \frac{f'_c}{15000\ psi} = 0.8667 \quad d_{bar} := D_{\#6}$$

$$\max\left(6\ in, 8 \cdot d_{bar}, \left(\frac{\psi_e \cdot \psi_r \cdot \psi_o \cdot \psi_c \cdot f_y}{55 \cdot \lambda \cdot min\left(100\ psi, \sqrt{f'_c \cdot psi}\right)}\right) \cdot 1\ in \cdot \left(\frac{d_{bar}}{in}\right)^{1.5}\right) = 9.7096\ in$$

f bars from the critical section:

$$\frac{(B - B_{wall})}{2} - c_c = 15\ in$$

$$= \text{if } l_{dhook} < \frac{(B - B_{wall})}{2}$$
$$\quad \text{"There is adequate room to develop the hooked bars"}$$
$$\text{else}$$
$$\quad \text{"There is inadequate room to develop the hooked bars"}$$

= "There is adequate room to develop the hooked bars"

$_{xuralHooks} := \left(\left(\left(D_{\#6} \cdot 6\right)\right) + \left(\left(2 \cdot D_{\#6}\right)\right)\right) + D_{\#6} = 6.75\ in$

$_{ar} + r = 3\ in$ (distance required for hook)

$depth_{FlexuralHooks} = 6.75\ in$

$=$ if $h_{min} < t_f$
"The footing thickness is adequate to accomodate the hooked bar"
else
"The footing thickness is not adequate to accomodate the hooked bar"

$=$"The footing thickness is adequate to accomodate the hooked bar"

earing Capacity of Column at Base:

$1\ ft$ $\qquad h := t_f$

$_{wall} \cdot L_{wall} = 96\ in^2$

$\left(\left(L_{wall}, \left(\left(2 \cdot h + B_{wall} + 2 \cdot h\right)\right)\right)\right) = 1\ ft$

$= 144\ in^2$

$65 \cdot \left(\left(0.85 \cdot f'_c \cdot A_1\right)\right) = 212.16\ kip$

$65 \cdot min\left(\left(\left(\left(0.85 \cdot f'_c \cdot A_1\right)\right) \cdot \sqrt{\frac{A_2}{A_1}}\right), \left(\left(2 \cdot 0.85 \cdot f'_c \cdot A_1\right)\right)\right) = 259.8419\ kip$

$_{earing} := \frac{min\left(\left(N_1, N_2\right)\right)}{1\ ft} = 212.16\ klf$

$=$ 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"

$=$"The footing has adequate bearing strength at the base"

0 Degree Hooked Dowel Bars in Column:

$1 \qquad in^2$

make sure footing is thick enough to accommodate development length:

$$\frac{{}_{vel} \cdot 6}{2} = 1.875\ in$$ (radius of dowel bar bend)

$${}_{owel} + r = 2.5\ in$$ (distance required for hook)

$${}_{dc} + 2 \cdot H + c_c = 20\ in$$

= if $h_{min} \le h$
"The footing thickness is adequate"
else
"The footing thickness is inadequate"

= "The footing thickness is adequate"

ebars in Wall:

$.5\ in$ $d_{bar} := D_{\#4}$ column bars are #4

ment Length:

$$\max\left(8\ in, \frac{0.02 \cdot f_y \cdot d_{bar}}{\lambda \cdot min\left(100\ psi, \sqrt{f'_c \cdot psi}\right)}, \frac{0.0003}{psi} \cdot f_y \cdot d_{bar}\right) = 9.4868\ in$$

ngth for rebars in compression:

$$\max\left(12\ in, l_{dcCol}, \frac{0.0005}{psi} \cdot f_y \cdot d_{bar} \cdot \alpha_s\right) = 15\ in$$ (round up to a 24in (2ft) splice)

of Rebar in Continuous Foundation For Stage Back Walls:

riteria:

kness: $t_{cmu} := FIF$ "8" $t_{brick} := FIF$ "8" $t_{air} := FIF$ "1" $B_{wall} := 8\ in$

USE SOLID GROUT

ght: $\gamma_{fill} := 120\ pcf$ $\gamma_{brick} := 120\ pcf$ $\gamma_{cmu} := 130\ pcf$ $\gamma_{conc} := 150\ pcf$

f wall: $H_{wall} := FIF$ "16' " $H_{total} := FIF$ "3'" $+ H_{wall}$ $H_s := FIF$ "6' 4"

Need an extra 3 ft of height due to frost line

ss of footing/ $t_f := FIF$ "1' 8"" $t_s := 6\ in$

footing: $B := FIF$ "3' 8"

Footing: $D_f := 3\ ft + t_f$ DUE TO FROST LINE AT 4 ft.

oads: $P := 2.65\ kip$ $b_{trib} := 6\ ft$ $P_{beam} := \dfrac{P}{b_{trib}} = 0.4417\ \dfrac{kip}{ft}$

ne-Way Shear Strength:

depth of footing:

nuous footings, effective depth d is measured from the top of the footing to the center of the lateral bars.
inal bars are designed separately:

a 3in clear cover, #6 rebars:

in $D_{\#6} := 0.75\ in$

$c_c + \dfrac{D_{\#6}}{2} = 3.375\ in$

$cover = 16.625\ in$

Pressure:

$$W_m := \left(t_{cmu}\right) \cdot \left(H_{wall}\right) \cdot \gamma_{cmu} + \left(t_{brick} \cdot \left(H_{wall}\right) \cdot \gamma_{brick}\right) = 2.6667\ \frac{kip}{ft}$$

$$W_e := \left(\frac{B}{2} - \left(0.5 \cdot \left(t_{cmu} + t_{brick}\right)\right)\right) \cdot H_{total} \cdot \gamma_{fill} = 2.66\ \frac{kip}{ft}$$

on wall: $$W_{wall} := \left(t_{brick} + t_{cmu} + t_{air}\right) \cdot \left(H_{total} - H_{wall}\right) \cdot \gamma_{conc} = 0.6375\ \frac{kip}{ft}$$

$$W_f := t_f \cdot B \cdot \gamma_{conc} = 0.9167\ \frac{kip}{ft}$$

= if $V_{uOneWay} < \phi V_{cOneWay}$
‖ "The footing has adequate shear strength"
else
‖ "The footing has inadequate shear strength "

= "The footing has adequate shear strength"

lexural Strength:

c

$2 = 1.6667\ ft$

$\frac{{}_u \cdot l^2}{2 \cdot B} = 1.419\ \frac{kip \cdot ft}{ft}$ (required flexural resistance)

.0018

$\rho_{min} \cdot d \cdot \frac{12\ in}{1 \cdot ft} = 0.3591\ \frac{in^2}{ft}$

Rebars: $A_{\#6} := 0.44\ in^2$

$A_{\#6} = 0.44\ in^2$ 1 #6 bar is adequate

ing:

$ce_{max} := min(3 \cdot t_f, 18\ in) = 18\ in$

$ce := \frac{1\ ft}{1} = 12\ in$ Need one bar every foot

flexural strength of a singly reinforced rectangular section:

$:= c_c + \frac{D_{\#6}}{2} = 3.375\ in$

$= t_f$ $y_{s1} := cover_1$ $A_s = 0.44\ in^2$ $b := B$

$pth - y_{s1} = 16.625\ in$

$= \text{if } M_u < \phi M_n$
"The footing has adequate flexural strength"
else
"The footing has inadequate flexural strength "

$=$"The footing has adequate flexural strength"

evelopment Length of Flexural 180 degree Hooked Rebars:

rvative assumptions:

$\psi_r := 1.0$ $\psi_o := 1.0$ $\psi_c := 0.6 + \frac{f'_c}{15000\ psi} = 0.8667$ $d_{bar} := D_{\#6}$

$\max\left(6\ in, 8 \cdot d_{bar}, \left(\frac{\psi_e \cdot \psi_r \cdot \psi_o \cdot \psi_c \cdot f_y}{55 \cdot \lambda \cdot min\left(100\ psi, \sqrt{f'_c \cdot psi}\right)}\right) \cdot 1\ in \cdot \left(\frac{d_{bar}}{in}\right)^{1.5}\right) = 9.7096\ in$

f bars from the critical section: $\frac{(B - B_{wall})}{2} - c_c = 15\ in$

$= \text{if } l_{dhook} < \frac{(B - B_{wall})}{2}$
"There is adequate room to develop the hooked bars"
else
"There is inadequate room to develop the hooked bars"

$=$"There is adequate room to develop the hooked bars"

he Longitudinal Steel:

.0018 $A_{\#6} := 0.44\ in^2$ $D_{\#6} := 0.75\ in$

$\rho_{min} \cdot B \cdot d = 1.3167\ in^2$

Rebars:

$A_{\#6} = 1.32\ in^2$ 3 #6 bars is adequate

cing:

$ce_{max} := min\left(3 \cdot t_f, 18\ in\right) = 18\ in$

$ce := \frac{B - 2\ c_c}{2} = 19\ in$ use 12 in spacing to be conservative

make sure footing is thick enough to accommodate development length:

earing Capacity of Column at Base:

$1\ ft$ $\qquad h := t_f$

$_{wall} \cdot L_{wall} = 96\ in^2$

$\left(L_{wall}, \left(2 \cdot h + B_{wall} + 2 \cdot h\right)\right) = 1\ ft$

$= 144\ in^2$

$5 \cdot \left(0.85 \cdot f'_c \cdot A_1\right) = 212.16\ kip$

$5 \cdot min\left(\left(\left(0.85 \cdot f'_c \cdot A_1\right) \cdot \sqrt{\frac{A_2}{A_1}}\right), \left(2 \cdot 0.85 \cdot f'_c \cdot A_1\right)\right) = 259.8419\ kip$

$_{earing} := \dfrac{min\left(N_1, N_2\right)}{1\ ft} = 212.16\ klf$

$=$ 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"

$=$ "The footing has adequate bearing strength at the base"

0 Degree Hooked Dowel Bars in Column:

$.005 \cdot A_1 \cdot \dfrac{1}{1 \cdot ft} = 0.48\ \dfrac{in^2}{ft}$

dowels $\qquad A_{\#5} := 0.310\ in^2$

$:= 2 \cdot A_{\#5} = 0.62\ in^2$ (two dowels per 1 ft is adequate)

ment Length:

$.625\ in$

$D_{\#5}$

$ax\left(8\ in, \lambda \cdot min\left(\dfrac{0.02 \cdot f_y \cdot d_{dowel}}{100\ psi, \sqrt{f'_c \cdot psi}}\right), \dfrac{0.0003}{psi} \cdot f_y \cdot d_{dowel}\right) = 11.8585\ in$

in (round up to get an appropriate constructible dimension) ebars in Wall:

$.5\ in$ $\qquad d_{bar} := D_{\#4}$ column bars are #4

ment Length:

$$\max\left(8\ in,\ \lambda \cdot min\left(\frac{0.02 \cdot f_y \cdot d_{bar}}{100\ psi,\ \sqrt{f'_c} \cdot psi}\right),\ \frac{0.0003}{psi} \cdot f_y \cdot d_{bar}\right) = 9.4868\ in$$

ngth for rebars in compression:

$$\max\left(12\ in,\ l_{dcCol},\ \frac{0.0005}{psi} \cdot f_y \cdot d_{bar} \cdot \alpha_s\right) = 15\ in$$

(round up to a 24in (2ft) splice)

of Rebar in Isolated Footing:

One-Way Shear Strength:

F "3' 8" $\qquad H := FIF$ "3'" $\qquad B_{col1} := FIF$ "1' 4" $\qquad B_{col2} := FIF$ "1'" $\qquad t_f := FIF$ "1' 10" $\qquad h := t_f$

$1.6\ kip$ $\qquad \gamma_{conc} := 150\ pcf$ $\qquad \gamma_{backfill} := 120\ pcf$ $\qquad f'_c := 4000\ psi$ $\qquad f_y := 60\ ksi$

e depth of footing:

a 3in clear cover, #6 rebars and bars in both directions:

n $\qquad D_{\#6} := 0.750\ in$

$= c_c + \frac{D_{\#6}}{2} = 3.375\ in$ $\qquad cover_2 := c_c + D_{\#6} + \frac{D_{\#6}}{2} = 4.125\ in$

$_g := \frac{cover_1 + cover_2}{2} = 3.75\ in$

$cover_{Avg} = 18.25\ in$

Punching Shear Strength:

$_{g} := q_u \cdot ((L_1 \cdot L_2 - ((c_1 + d)) \cdot ((c_2 + d)))) = 12.3775\ kip$

$\frac{_{11}}{_{12}} = 1.3333$ $\alpha_s := 30$ $b_o := 2\ ((c_1 + d)) + 2 \cdot ((c_2 + d)) = 10.75\ ft$

$_{hing} := 0.75 \cdot min\left(4, \left(2 + \frac{4}{\beta}\right), \left(2 + \frac{\alpha_s \cdot d}{b_o}\right)\right) \cdot \lambda \cdot \sqrt{f'_c \cdot psi} \cdot b_o \cdot d = 446.6875\ kip$

= if $V_{uPunching} < \phi V_{cPunching}$
  "The footing has adequate punching shear strength"
else
  "The footing has inadequate punching shear strength"

= "The footing has adequate punching shear strength"

Flexural Strength:

$\cdot L_2 \cdot \left(\frac{L_1 - c_1}{2}\right) \cdot \left(\frac{L_1 - c_1}{4}\right) = 4.9422\ kip \cdot ft$ (required flexural resistance)

.0018

$\rho_{min} \cdot B \cdot h = 1.7424\ in^2$

Rebars: $A_{\#6} := 0.440\ in^2$

$A_{\#6} = 1.76\ in^2$ 4 #6 bars is adequate

cing:

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

$ace := \frac{B - 2 \cdot c_c}{3} = 12.6667\ in$

e flexural strength of a singly reinforced rectangular section:

$= h$ $y_{s1} := cover_1$ $A_s = 1.76\ in^2$ $b := B$

$.9 \cdot M_n = 141.7447\ kip \cdot ft$ (flexural strength)

$= \text{if } M_u < \phi M_n$
‖ "The footing has adequate flexural strength"
else
‖ "The footing has inadequate flexural strength"

= "The footing has adequate flexural strength"

Development Length of Flexural 180 degree Hooked Rebars:

ervative assumptions:

$\psi_r := 1.0 \quad \psi_o := 1.0 \quad \psi_c := 0.6 + \dfrac{f'_c}{15000\ psi} = 0.8667 \quad d_{bar} := D_{\#6}$

$\max\left(6\ in, 8 \cdot d_{bar}, \left(\dfrac{\psi_e \cdot \psi_r \cdot \psi_o \cdot \psi_c \cdot f_y}{55 \cdot \lambda \cdot min\left(100\ psi, \sqrt{f'_c \cdot psi}\right)}\right) \cdot 1\ in \cdot \left(\dfrac{d_{bar}}{in}\right)^{1.5}\right) = 9.7096\ in$

$10\ in$

of bars from the critical section:

$_{oll})) = 14\ in$

$= \text{if } l_{dhook} < \dfrac{(B - B_{coll})}{2}$
‖ "There is adequate room to develop the hooked bars"
else
‖ "There is inadequate room to develop the hooked bars"

$heck$ = "There is adequate room to develop the hooked bars"

o make sure footing is thick enough to accommodate development length:

$\cdot 6 = 2.25\ in$ (radius of dowel bar bend)

$_{uralHooks} := ((d_{bar} \cdot 6) + (2 \cdot d_{bar})) = 6\ in$

$_n := \max(4 \cdot d_{bar}, 2.5\ in) = 3\ in$

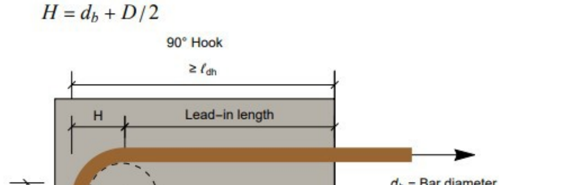
$+ r = 3\ in$ (distance required for hook)

$depth_{FlexuralHooks} = 6\ in$

Tension Rebars Terminated in Hooks

For main tension rebars ACI 318 Table 25.3.1 defines geometry for 9
180° hooks as illustrated in Figure 7.21. The length of the bar up to th
the hook is called lead-in length. Minimum inside bend diameter D is
in the table for different size rebars. The minimum dimension of the t
straight extension of the bar beyond the end of the bend is also shown
figure. The distance H that must be added to the lead-in length to def
development length of hooked bars is the bend radius plus the bar dia

$H = d_b + D/2$



90 Degree Hooked Dowel Bars in Column:

m Steel Ratio:

$.005 \cdot A_1 = 0.96\ in^2$

5 dowels $\quad A_{\#5} := 0.310\ in^2 \quad D_{\#5} := 0.625\ in$

$:= 4 \cdot A_{\#5} = 1.24\ in^2$

pment Length:

$:= D_{\#5}$

$ax\left(8\ in, \dfrac{0.02 \cdot f_y \cdot d_{dowel}}{\lambda \cdot min\left(100\ psi, \sqrt{f'_c \cdot psi}\right)}, \dfrac{0.0003}{psi} \cdot f_y \cdot d_{dowel}\right) = 11.8585\ in$

in (round up to get an appropriate constructible dimension)

o make sure footing is thick enough to accommodate development length:

$\dfrac{{}_{wel} \cdot 6}{2} = 1.875\ in$ (radius of dowel bar bend)

${}_n := 12 \cdot d_{dowel} = 7.5\ in$

${}_{wel} + r = 2.5\ in$ (distance required for hook)

${}_{dc} + 2 \cdot H + c_c + D_{\#6} = 20.75\ in$

= if $h_{min} \le h$
‖ "There footing thickness is adequate"
else
‖ "There footing thickness is inadequate"

= "There footing thickness is adequate"

Rebars in Column:

${}_{\#4}$ column bars are #4

pment Length:

$max\left(8\ in, \dfrac{0.02 \cdot f_y \cdot d_{bar}}{\ldots}, \dfrac{0.0003}{\ldots} \cdot f_y \cdot d_{bar}\right) = 9.4868\ in$

f Beam Bearing on the Back Stage Wall (Masonry):

f Anchor Bolt in Back wall:

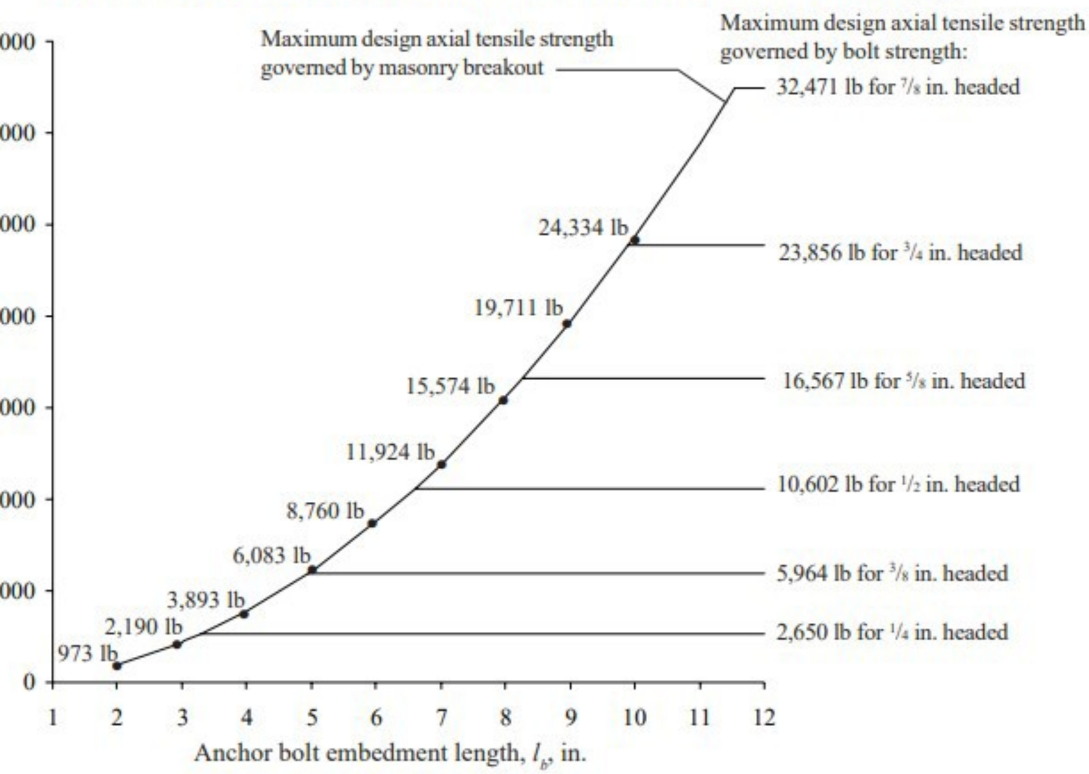
$psf \qquad p_2 := -12.3\ psf$

$9 \cdot q_d + p_2| = 9.6\ psf$

$\cdot\ 6\ ft = 57.6\ \frac{lbf}{ft} \qquad L := FIF\ \text{"33' 9"}$

$_{max} := 0.5\ W \cdot 6\ ft \cdot L = 972\ lbf$

Figure 8—Strength Design, Design Axial Tensile Strength of Headed Anchor Bolts, ϕB_{an} [a]



.500 psi (10.34 MPa), f_y = 60,000 psi (413.6 MPa). Strength reduction factors: 0.50 for masonry breakout; 0.90 chor bolt steel. Values assume that projected areas of adjacent anchor bolts do not overlap (anchor spacing greater than l to $2l_b$). lb x 4.448222 = N; in. x 25.4 = mm.

embedment of a 1/4 in. headed bolt is sufficient

f Bearing plate on Back wall:

$50\ lbf \qquad B := 6\ in \qquad N := 7\ in \qquad F_y := 36\ ksi \qquad f'_c := 1.5\ ksi \qquad e := 0\ in \qquad d := 10\ in$

masonry

$\cdot\ B = 42\ in^2 \qquad A_2 := (B + 2\ e) \cdot (N + 2\ e) = 42\ in^2$

eb Crippling:

7

$$.75 \cdot 0.4 \cdot t_w{}^2 \cdot \left(1 + \left(\frac{4 \cdot N}{d} - 0.2\right) \cdot \left(\frac{t_w}{t_f}\right)^{1.5}\right) \cdot \sqrt{\frac{E \cdot F_{yw} \cdot t_f}{t_w}} = 44.3878\ kip$$ OK

f Weld:

$min\left(\left(t_p, t_f\right)\right) = 0.21\ in$ Try $w := \frac{3}{16}\ in = 0.1875\ in$

$4 \cdot w = 0.75\ in$

n

$.6667$

n the base metal yielding and fracture along the weld base:

$F_y := 50\ ksi$ $F_u := 65\ ksi$

$:= min\left(\left(1 \cdot 0.6 \cdot F_y \cdot t \cdot L, 0.75 \cdot 0.6 \cdot F_u \cdot t \cdot L\right)\right) = 30.7125\ kip$ OK

n weld fracture along effective throat dimension:

$F_{EXX} := 70\ ksi$

$6 \cdot F_{EXX} \cdot \left(1 + 0.5 \cdot \sin\theta^{1.5}\right)$

$n := 0.75 \cdot 0.707 \cdot w \cdot L_e \cdot F_w = 20.8786\ kip$ OK

of Column Base Plate and Anchor Bolts:

$1.6\ kip$ $F_y := 36\ ksi$ A36 Steel

in $N := 14\ in$

ksi $P_u := P_{col} = 11.6\ kip$

$\cdot B = 140\ in^2$

$+2\ e\ \cdot\ N+2\ e\ = 192\ in^2$

$0.65 \cdot 0.85 \cdot f'_c \cdot A_2 \cdot min\left(2, \sqrt{\frac{A_2}{A_1}}\right) = 496.9127\ kip$

ckness:

$\frac{u}{\cdot B} = 82.8571\ psi$

in $b_f := 8\ in$

$\frac{-0.95 \cdot d}{2} = 1.3\ in$ $n := \frac{B-0.8 \cdot b_f}{2} = 1.8\ in$

$\left(m, \frac{B-0.8 \cdot b_f}{2}, \frac{1}{4} \cdot \sqrt{d \cdot b_f}\right) = 2.4495\ in$

$\cdot \sqrt{\frac{2 \cdot f_p}{0.9 \cdot F_y}} = 0.1752\ in$ Round up to 1/2 in. thickness grout underneath plate = 1 in.

Bolts:

th 3/4 in.
rs

$.6\ kip$ $d_b := \frac{3}{4}\ in$ $L_e := 12 \cdot d_b = 9\ in$ $t_{roof} := 3\ in + 10\ in$ $p_{positive} := 10.761\ psf$ $h := 14.5\ ft$

$L_{edge} := \max(4\ in, 5 \cdot d_b) = 4\ in$

$_{ositive} \cdot b_f = 0.0072\ \frac{kip}{ft}$

1

$\cdot t\ \ \ f \cdot L \cdot p\ \ \ = 0.1399\ kip$

strength:

$d_b = 2.25\ in$ $\psi_4 := 1.4$

$.7 \cdot \psi_4 \cdot \left(\left(0.9 \cdot f'_c \cdot e_h \cdot d_b\right)\right) = 5.9535\ kip$ greater than $T_u = 1.3241\ kip$

rods have enough strength, design is OK

of Girder Moment Connection into Column:

oads and properties:

A325N bolts

$3\ kip \cdot ft$ $V := 8.5\ kip$ $E := 29000\ ksi$ $F_{u\_Bolt} := 120\ ksi$

$:= 36\ ksi$ $F_{u\_plate} := 58\ ksi$ $F_y := 50\ ksi$ $F_u := 65\ ksi$

in $A_b := \frac{\pi}{4} \cdot d_b^2 = 0.6013\ in^2$ $d_h := d_b + \frac{2}{16}\ in = 1\ in$

W12x40 A992 steel

$.9\ in$ $b_{fg} := 8.01\ in$ $t_{wg} := 0.295\ in$ $t_{fg} := 0.515\ in$ $S_{xg} := 51.5\ in^3$

W12x40 A992 steel

$.9\ in$ $b_{fc} := 8.01\ in$ $t_{wc} := 0.295\ in$ $t_{fc} := 0.515\ in$

n of flange plate dimensions:

$= 33.2773\ kip$ force carried by each flange plate

of bolts needed:

$= \dfrac{P_u}{\phi R_{nShear}} = 1.5372$ need two bolts Provide two rows of two bolts to be conservative

$= 2 \cdot L_e + s = 6.5\ in$ add 1/2 in setback

$_{\_min} + 0.5\ in = 7\ in$

aring in Beam Flange:

$F_{u\_plate} = 29\ \dfrac{kip}{in}$ $t_{fg} \cdot F_u = 33.475\ \dfrac{kip}{in}$

$_{te}$ bearing critical in flange plate

$_e - \dfrac{d_h}{2} = 1.25\ in$

strength for end bolt:

$_{earing} := 0.75 \cdot min(1.2 \cdot L_{c1} \cdot t \cdot F_u, 2.4 \cdot d_b \cdot t \cdot F_u) = 36.5625\ kip$

$_{Bolt} := min(\phi R_{nShear}, \phi R_{nEndBearing}) = 21.6475\ kip$

$- d_h = 2\ in$

$_{aring} := 0.75 \cdot min(1.2 \cdot L_{c2} \cdot t \cdot F_u, 2.4 \cdot d_b \cdot t \cdot F_u) = 51.1875\ kip$

$_{olt} := min(\phi R_{nShear}, \phi R_{nIntBearing}) = 21.6475\ kip$

$:= 4 \cdot \phi R_{nEndBolt} = 86.5901\ kip$ > $P_u = 33.2773\ kip$ OK

Strength in Flange:

$5\ in^2$

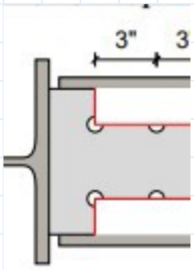
$\cdot t_{f\_plate} = 1.75\ in^2$

$_{gt} - d_h \cdot t_{f\_plate} = 1.25\ in^2$

ear strength of angle:

$_{ShearAngle} := 0.75 \cdot \left(min\left(0.6 \cdot F_{u\_plate} \cdot A_{nv}, 0.6 \cdot F_{y\_plate} \cdot A_{gv}\right) + U \cdot F_{u\_plate} \cdot A_{nt}\right) = 131.325\ kip$ $>$ $P_u = 33.2773\ kip$

hear failure of the plate Outside lines of bolts:



$\cdot t_{f\_plate} \cdot \left(s + L_e\right) = 4.75\ in^2$

$_{gv} - 2 \cdot 1.5 \cdot d_h \cdot t_{f\_plate} = 3.25\ in^2$

$\left(b_p - g - d_h\right)\right)\ t_{f\_plate} = 1.5\ in^2$

ear strength of angle:

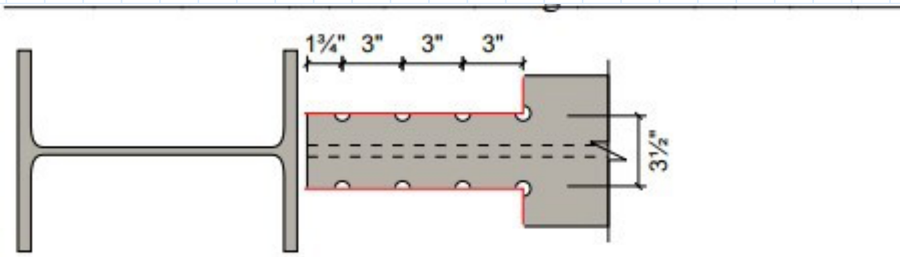
$_{ShearAngle} := 0.75 \cdot \left(min\left(0.6 \cdot F_{u\_plate} \cdot A_{nv}, 0.6 \cdot F_{y\_plate} \cdot A_{gv}\right) + U \cdot F_{u\_plate} \cdot A_{nt}\right) = 142.2\ kip$ $>$ $P_u = 33.2773\ kip$

hear failure of the beam flange Outside lines of bolts:

$t_{fg} \cdot \left(s + L_e\right) = 4.8925\ in^2$

$_{gv} - 2 \cdot 1.5 \cdot d_h \cdot t_{f\_plate} = 3.3925\ in^2$

$\left(b_{fg} - g - d_h\right)\right)\ t_{fg} = 1.8077\ in^2$



ear strength of angle:

$= 0.0543$

$.658^{0.024149} \cdot F_{y\_plate} = 35.638\ ksi$

$.7846 \quad < \quad 25 \quad$ OK

$.9 \cdot F_{cr} \cdot A_p = 120.2781\ kip \quad > \quad P_u = 33.2773\ kip \quad$ OK

Plate welded connection to the column flange:

$70\ ksi \qquad L := b_p \qquad \text{clear}\ \ w$

$.75 \cdot 0.707\ w \cdot L \cdot \left(\left(0.9 \cdot F_{EXX}\right)\right) \cdot in$

$75 \cdot 0.707\ w \cdot L \cdot \left(\left(0.9 \cdot F_{EXX}\right)\right) \cdot in \xrightarrow{solve,w} \dfrac{0.13282069074683148061 \cdot kip}{in^2 \cdot ksi} \qquad w := 0.133\ in$

n weld on each side for a total weld thickness of 0.5 in. $\quad w := 0.25 \quad$ This is adequate

late Connection:

$t_{w\_plate} := 0.25\ in$

$in = 0.5\ in \quad > \quad t_{w\_plate} = 0.25\ in \qquad$ OK

6

$t_{wg} = 0.295\ in$

$\dfrac{5}{8} \cdot t_{w\_plate} = 0.1563\ in \qquad$ provide $\quad w := \dfrac{3}{16}$

ilure mode of shear tab:**

$_{v\_plate} \cdot L_p = 1.5\ in^2$

$_{gv} - 2 \cdot d_h \cdot t_{w\_plate} = 1\ in^2$

$_{Shear} := min\left(1 \cdot 0.6 \cdot F_{y\_plate} \cdot A_{gv}, 0.75 \cdot 0.6 \cdot F_{u\_plate} \cdot A_{nv}\right) = 26.1\ kip$ > $V = 8.5\ kip$ OK

ear strength:

$_{plate} \cdot \left(L_e + \left(n - 1\right) \cdot s\right) = 1.125\ in^2$

$_{gv} - \left(\left(n - 1\right) + 0.5\right) \cdot d_h \cdot t_{w\_plate} = 0.75\ in^2$

$_{kShear} := 0.75 \cdot \left(min\left(0.6 \cdot F_{u\_plate} \cdot A_{nv}, 0.6 \cdot F_{y\_plate} \cdot A_{gv}\right) + U \cdot F_{u\_plate} \cdot A_{nt}\right) = 96.8578\ kip$ > $V = 8.5\ kip$ OK

Phase One Restroom Roof Dead Loads

From top down:

Liquid Applied Waterproofing Membrane

Clear Waterproofing Membrane

Link to Website: https://waterstop.com.au/product/clear-waterproofing-membrane-10l/

Link to Datasheet: https://waterstop.com.au/wp-content/uploads/2021/06/Data-Sheet-Clear-Waterproofing-Membrane-RV2-2.pdf

Waterproofing membranes:	
Bituminous, gravel-covered	5.5
Bituminous, smooth surface	1.5
Liquid applied	1
Single-ply, sheet	0.7
Wood sheathing (per in. thickness)	3
Wood shakes and shingles	3

ASCE 7-22 Table C3.1-1a Minimum Design Dead Loads (psf)

$$Dead_{Membrane} := 1\ psf$$

Composite Roof Deck

1.5VLR-36 COMPOSITE DECK-SLAB Light Weight Concrete - 22 gage - 3.5" total depth - 2" topping

Link to Website: https://vulcraft.com/Products/Deck#composite-deck

Link to Datasheet:https://vulcraft.com/catalogs/Deck/CompositeDeck/ASD-1.5VLR-36_Composite_Deck-Slab.pdf

$$Dead_{CompositeDeck} := 63.9\ psf + 0.5\ in \cdot 110\ pcf = 68.483\ psf$$

Additional 1 inch of concrete for roof sloping averaged over the entire span

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Roof Insulation

HANDI-FOAM® E84 CLASS 1(A) LOW PRESSURE SPRAY FOAM

Link to Wesbite: https://www.energyefficientsolutions.com/Fire_Rated.asp?item=FOAM605E84&gclid=EAIaIQobChMI3LfOk-j1_QIVyv_jBx19aQAPEAQYAyABEgK6K_D_BwE

Link to Datasheet: https://www.energyefficientsolutions.com/data%20sheets/SprayFoamE84-105-205-605.pdf

$$Density := 1.75\ \frac{lbf}{ft^3}$$

$$Dead_{RoofInsulation} := Density \cdot 2.5\ in = 0.365\ psf$$

Mechanical/Electrical/Plumbing $Dead_{MEP} := 5\ psf$

Typical

Total Roof Dead Load

$$Dead_{Roof} := Dead_{Membrane} + Dead_{CompositeDeck} + Dead_{RoofInsulation} + Dead_{MEP} = 74.848\ psf$$

The dead load for the roof is approximately 75 psf

Phase One Restroom Snow Load Calcs

Balanced Snow Load

Used ASCE 7-16 because it is simpler

Exposure Coefficient

Table 7.3-1. Exposure Factor, C_e.

	Exposure of Roof[a]		
Surface Roughness Category	Fully Exposed[b]	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	N/A
In Alaska, in areas where trees do not exist within a 2 mi (3 km) radius of the site	0.7	0.8	N/A

The terrain category and roof exposure condition chosen shall be representative of the anticipated conditions during the life of the structure. An exposure factor shall be determined for each roof of a structure.

[a] Partially Exposed: All roofs not Fully Exposed or Sheltered. Fully Exposed: Roofs exposed on all sides with no shelter[b] afforded by terrain, higher structures, or trees. Roofs that contain several large pieces of mechanical equipment, parapets that extend above the height of the balanced snow load (h_b), or other obstructions are not in this category. Sheltered: Roofs located tight in among conifers that qualify as obstructions.

[b] Obstructions within a distance of $10h_o$ provide "shelter," where h_o is the height of the obstruction above the roof level. If the only obstructions are a few deciduous trees that are leafless in winter, the "fully exposed" category shall be used. Note that these are heights above the roof. Heights used to establish the Exposure Category in Section 26.7 are heights above the ground.

$C_e := 1.0$

Surface Roughness B/Partially Exposed...
because we're in downtown Maquoketa

ASCE 7-16 Table 7.3-1 Exposure Coefficient, Ce

Thermal Factor

Heated closed structure
without a ventilated roof

$C_t := 1.0$

All structures except as
indicated below

ASCE 7-16 Table 7.3-2 Thermal Factor, Ct

Didn't use ASCE 7-22
because C_t is requires
known material properties

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 $25°F \times h \times ft^2/Btu$ ($4.4\ K \times m^2/W$)	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 $2.0°F \times h \times ft^2/Btu$ ($0.4\ K \times m^2/W$)	0.85

[a]These conditions shall be representative of the anticipated conditions during winters for the life of the structure.
[b]Greenhouses with a constantly maintained interior temperature of 50°F (10°C) or more at any point 3 ft (0.9 m) above the floor level during winters and having either a maintenance attendant on duty at all times or a temperature alarm system to provide warning in the event of a heating failure.

Importance Factor

Table 1.5-2 Importance Factors by Risk Category of Buildings and Other Structures for Snow, Ice, and Earthquake Loads

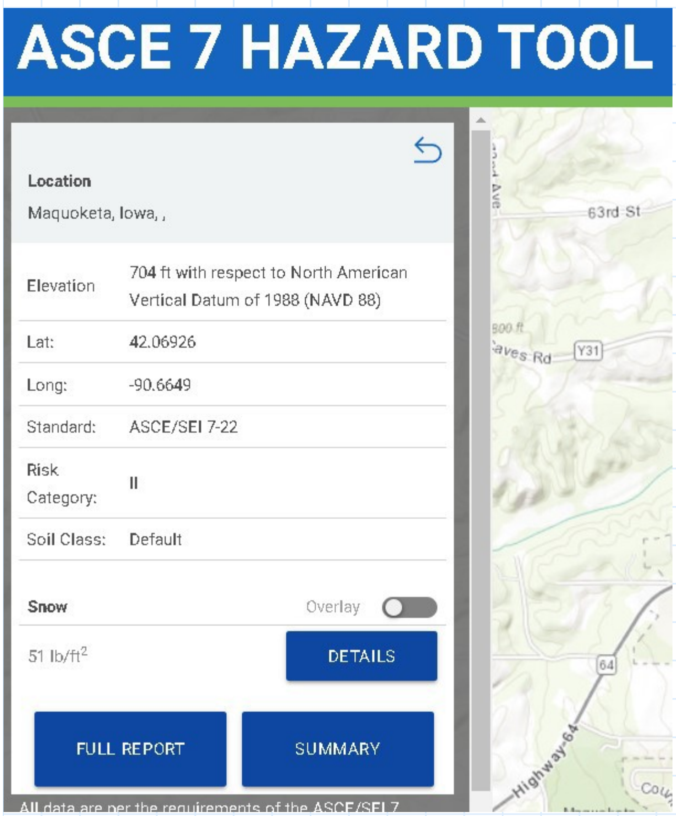
Risk Category from Table 1.5-1	Snow Importance Factor, I_s	Ice Importance Factor—Thickness, I_i	Ice Importance Factor—Wind, I_w	Seismic Importance Factor, I_e
I	0.80	0.80	1.00	1.00
II	1.00	1.00	1.00	1.00
III	1.10	1.15	1.00	1.25
IV	1.20	1.25	1.00	1.50

Note: The component importance factor, I_p, applicable to earthquake loads, is not included in this table because it depends on the importance of the individual component rather than that of the building as a whole, or its occupancy. Refer to Section 13.1.3.

$I_s := 1.0$

Risk Category 2
ASCE 7-16 Table 1.5-2

Ground Snow Load



ASCE Hazard Tool

$p_g := 51\ psf$

No other snow load indicated in the Maquoketa Code of Ordinances

Sloped Roof Factor $C_s := 1$ Essentially a flat roof

Sloped Roof Snow Load $p_s := 0.7 \cdot C_s \cdot C_e \cdot C_t \cdot I_s \cdot p_g = 35.7\ psf$

$BalancedSnow_{Load} := p_s = 35.7\ psf$

The balanced snow load is 35.7 psf

Phase One Restroom Live Load Calculations

Used ASCE 7-22

Table 4.3-1. ***(Continued)*** **Minimum Uniformly Distributed Live Loads, L_o, and Minimum Concentrated Live Loads**

Occupancy or Use	Uniform, L_o psf (kN/m²)	Live Load Reduction Permitted? (Sec. No.)	Multiple-Story Live Load Reduction Permitted? (Sec. No.)	Concentrated lb (kN)	Also See Section
Penal institutions					
Cell blocks	40 (1.92)	Yes (4.7.2)	Yes (4.7.2)		
Corridors	100 (4.79)	Yes (4.7.2)	Yes (4.7.2)		
Recreational uses					
Bowling alleys, poolrooms, and similar uses	75 (3.59)	No (4.7.5)	No (4.7.5)		
Dance halls and ballrooms	100 (4.79)	No (4.7.5)	No (4.7.5)		
Gymnasiums	100 (4.79)	No (4.7.5)	No (4.7.5)		
Residential					
One- and two-family dwellings					
Uninhabitable attics without storage	10 (0.48)	Yes (4.7.2)	Yes (4.7.2)		4.12.1
Uninhabitable attics with storage	20 (0.96)	Yes (4.7.2)	Yes (4.7.2)		4.12.2
Habitable attics and sleeping areas	30 (1.44)	Yes (4.7.2)	Yes (4.7.2)		
All other areas except stairs	40 (1.92)	Yes (4.7.2)	Yes (4.7.2)		
All other residential occupancies					
Private rooms and corridors serving them	40 (1.92)	Yes (4.7.2)	Yes (4.7.2)		
Public rooms	100 (4.79)	No (4.7.5)	No (4.7.5)		
Corridors serving public rooms	100 (4.79)	Yes (4.7.2)	Yes (4.7.2)		
Roofs					
Ordinary flat, pitched, and curved roofs	20 (0.96)	Yes (4.8.2)	—		4.8.1
Roof areas used for occupants	Same as occupancy served	Yes (4.8.3)	—		
Roof areas used for assembly purposes	100 (4.70)	Yes (4.8.3)			
Vegetative and landscaped roofs					
Roof areas not intended for occupancy	20 (0.96)	Yes (4.8.2)	—		
Roof areas used for assembly purposes	100 (4.70)	Yes (4.8.3)	—		
Roof areas used for other occupancies	Same as occupancy served	Yes (4.8.3)	—		
Awnings and canopies					
Fabric construction supported by a skeleton structure	5 (0.24)	No (4.8.2)	—		
Screen enclosure support frame	5 (0.24) based on the tributary area of the roof supported by the frame member	No (4.8.2)	—	200 (0.89)	

$Live_{Load} := 100\ psf$ For roof area used for assembly purposes

Since the roof won't be occupied at the same time as snowstorm, use the larger number load (snow or live load).

$Live_{Load} = 100\ psf$ >> $BalancedSnow_{Load} = 35.7\ psf$

Regardless, we will test each load combination for sufficient strength of members

MWFRS DESIGN WIND PRESSURES FOR WALLS AND ROOF - PHASE 1

Determining Wind Load Parameters:

Exposure Category:	B		(ASCE 7-22: Section 26.7.3)
Risk Category:	II		(ASCE 7-22: Table 1.5-1)
Importance Factor:	$I := 1.0$		(ASCE 7-22: Table 1.5-2)
Basic Wind Speed:	$V := 108\ mph$		(ASCE 7-22: Figure 26.5-1B)
Wind Directionality Factor:	$K_d := 0.85$	MWFRS system	(ASCE 7-22: Figure 26.6-1)
Topographic Factor:	$K_{zt} := 1$	No ridges or hills near this site	(ASCE 7-22: Section 26.8.1)
Ground Elevation Factor:	$K_e := 1$	Conservative approximation	(ASCE 7-22: Table 26.9-1)
Gust Effect Factor:	$G := 0.85$	Rigid Building	(ASCE 7-22: Section 26.11.1)
Enclosure Classification:	Building Enclosed		(ASCE 7-22: Section 26.2)
Internal Pressure Coefficient:	$GC_{pi} := 0.18$		(ASCE 7-22: Table 26.13-1)

Velocity Pressure Exposure Coefficient: $K_z := 0.57$ (ASCE 7-22: Table 26.10-1)

Using directional procedure so not 0.7

Velocity Pressure:

$$q_z := 0.00256 \frac{psf}{mph^2} \cdot K_z \cdot K_{zt} \cdot K_d \cdot V^2 \cdot I = 14.467\ psf \qquad q_i := q_z$$

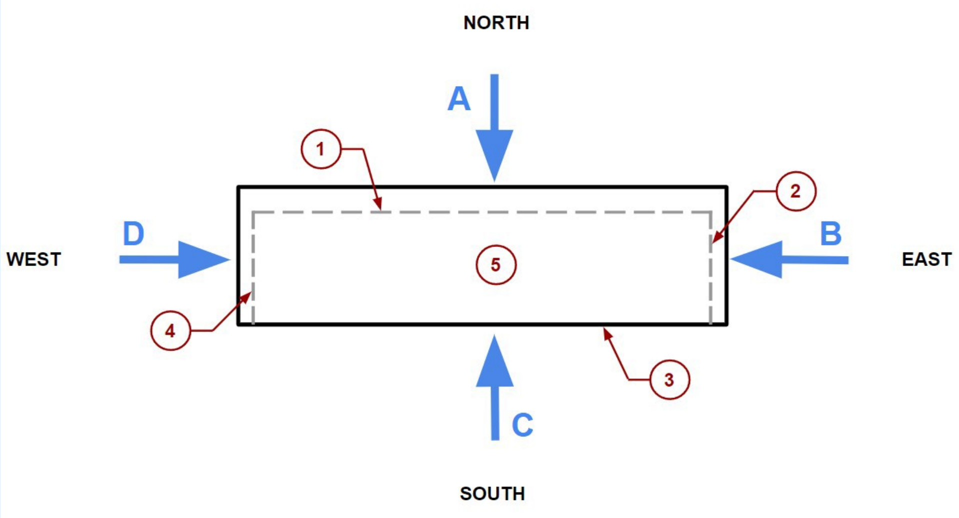
(ASCE 7-22: Equation 26.10-1)

Mean Roof Height: $h := \dfrac{10\ ft + 9.583333\ ft}{2} = 9.792\ ft$ (Height Modeled on Revit)

Based on 5" roof deck .. 3.5" concrete, 1.5" metal decking

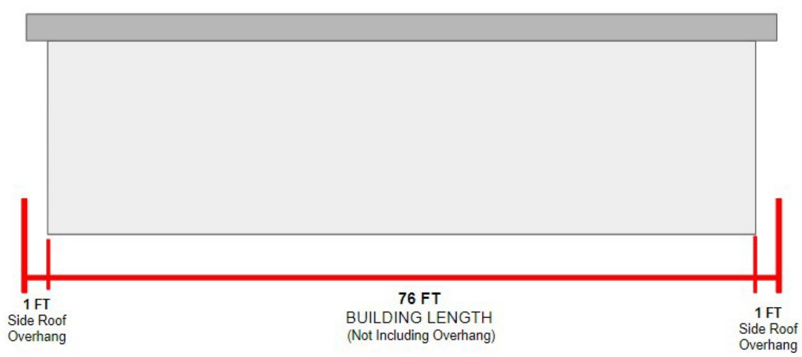
DIAGRAMS FOR WIND PRESSURE CALCULATIONS:

<u>TOP VIEW:</u> <u>OF PHASE 1 RESTROOM STRUCTURE</u>



<u>PROFILE VIEWS:</u> <u>OF PHASE 1 RESTROOM STRUCTURE</u>

VIEW FROM SIDE A:



Building Dimensions:

$L_A := 76\ ft$

$B_A := 16\ ft + 8\ in$

Overhang Dimensions:

$l_{EW.OH} := 2\ ft$

$l_{NS.OH} := 4\ ft$

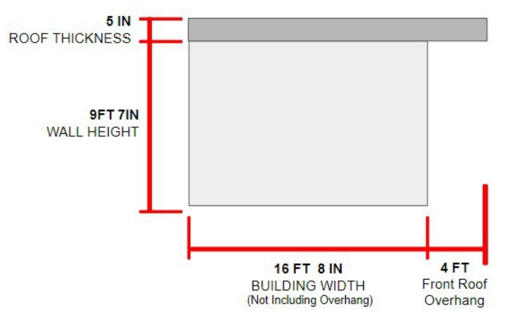
Equivalent Dimensions:

$L_C := L_A \qquad B_C := B_A$

$L_B := B_A \qquad B_B := L_A$

$L_D := L_B \qquad B_D := B_B$

VIEW FROM SIDE B:



NORTH WIND CALCULATIONS (WIND DIRECTION A)

Wall Pressures Coefficients:

$L := L_A = 76\ ft$ $B := B_A = 16.667\ ft$

Variables Needed for Table: $\frac{L}{B} = 4.56$

External Pressure Coefficients: $C_{pWW} := 0.8$ $C_{pLW} := -0.2$ $C_{pS} := -0.7$ (ASCE 7-22: Figure 27.3-1)

Wall Pressures: $p := p_{iext}$ +/- p_{iint}

Surface 1: (windward)

$p_{N1ext} := q_z \cdot G \cdot C_{pWW} = 9.84\ psf$

$p_{N1int} := q_i \cdot ((GC_{pi})) = 2.6\ psf$

$p_{N1plus} := p_{N1ext} + p_{N1int} = 12.442\ psf$

$p_{N1minus} := p_{N1ext} - p_{N1int} = 7.234\ psf$

Surface 2: (side)

$p_{N2ext} := q_z \cdot G \cdot C_{pS} = -8.61\ psf$

$p_{N2int} := q_i \cdot ((GC_{pi})) = 2.6\ psf$

$p_{N2plus} := p_{N2ext} + p_{N2int} = -6.004\ psf$

$p_{N2minus} := p_{N2ext} - p_{N2int} = -11.212\ psf$

Surface 3: (leeward)

$p_{N3ext} := q_z \cdot G \cdot C_{pLW} = -2.46\ psf$

$p_{N3int} := q_i \cdot ((GC_{pi})) = 2.6\ psf$

$p_{N3plus} := p_{N3ext} + p_{N3int} = 0.145\ psf$

$p_{N3minus} := p_{N3ext} - p_{N3int} = -5.063\ psf$

Surface 4: (side)

$p_{N4ext} := q_z \cdot G \cdot C_{pS} = -8.61\ psf$

$p_{N4int} := q_i \cdot ((GC_{pi})) = 2.6\ psf$

$p_{N4plus} := p_{N4ext} + p_{N4int} = -6.004\ psf$

$p_{N4minus} := p_{N4ext} - p_{N4int} = -11.212\ psf$

Roof Pressures Coefficients:

Included Overhang

$L := L_A + l_{EW.OH} = 78\ ft$ $B := B_A + l_{NS.OH} = 20.667\ ft$

Variables Needed for Table: $\frac{h}{L} = 0.1255$ $\frac{h}{2} = 4.9\ ft$ $h = 9.8\ ft$ $2\ h = 19.6\ ft$ $\theta := 0$

External Pressure Coefficient: $C_{p1} := -0.9$ $C_{p2} := -0.18$ (ASCE 7-22: Figure 27.3-1)

Using Conservative Pressure Coefficients

Roof Pressures: $p := p_{iext}$ +/- p_{iint}

Surface 5:

$p_{N5ext1} := q_z \cdot G \cdot C_{p1} = -11.07\ psf$ $p_{N5ext2} := q_z \cdot G \cdot C_{p2} = -2.21\ psf$

$p_{N5int1} := q_i \cdot ((GC_{pi})) = 2.6\ psf$ $p_{N5int2} := q_i \cdot ((GC_{pi})) = 2.6\ psf$

$p_{N5plus1} := p_{N5ext1} + p_{N5int1} = -8.463\ psf$ $p_{N5plus2} := p_{N5ext2} + p_{N5int2} = 0.391\ psf$

$p_{N5minus1} := p_{N5ext1} - p_{N5int1} = -13.671\ psf$ $p_{N5minus2} := p_{N5ext2} - p_{N5int2} = -4.818\ psf$

Overhang Pressures:

$p_{Noh} := q_z \cdot G \cdot C_{p1} = -11.067\ psf$

IMPORTANT NOTE: p_{oh} Must be added to the positive external pressure on windward faces with overhang (surface 1))

EAST WIND CALCULATIONS (WIND DIRECTION B)

Wall Pressures Coefficients:

$L := L_B = 16.667\ ft$ $B := B_B = 76\ ft$

Variables Needed for Table: $\frac{L}{B} = 0.219$

External Pressure Coefficients: $C_{pWW} := 0.8$ $C_{pLW} := -0.5$ $C_{pS} := -0.7$ (ASCE 7-22: Figure 27.3-1)

Wall Pressures: $p := p_{iext}$ +/- p_{iint}

Surface 1: (side)

$p_{E1ext} := q_z \cdot G \cdot C_{pS} = -8.61\ psf$

$p_{E1int} := q_i \cdot \left(\left(GC_{pi}\right)\right) = 2.6\ psf$

$p_{E1plus} := p_{E1ext} + p_{E1int} = -6.004\ psf$

$p_{E1minus} := p_{E1ext} - p_{E1int} = -11.212\ psf$

Surface 2: (windward)

$p_{E2ext} := q_z \cdot G \cdot C_{pWW} = 9.84\ psf$

$p_{E2int} := q_i \cdot \left(\left(GC_{pi}\right)\right) = 2.6\ psf$

$p_{E2plus} := p_{E2ext} + p_{E2int} = 12.442\ psf$

$p_{E2minus} := p_{E2ext} - p_{E2int} = 7.234\ psf$

Surface 3: (side)

$p_{E3ext} := q_z \cdot G \cdot C_{pS} = -8.61\ psf$

$p_{E3int} := q_i \cdot \left(\left(GC_{pi}\right)\right) = 2.6\ psf$

$p_{E3plus} := p_{E3ext} + p_{E3int} = -6.004\ psf$

$p_{E3minus} := p_{E3ext} - p_{E3int} = -11.212\ psf$

Surface 4: (leeward)

$p_{E4ext} := q_z \cdot G \cdot C_{pLW} = -6.15\ psf$

$p_{E4int} := q_i \cdot \left(\left(GC_{pi}\right)\right) = 2.6\ psf$

$p_{E4plus} := p_{E4ext} + p_{E4int} = -3.544\ psf$

$p_{E4minus} := p_{E4ext} - p_{E4int} = -8.753\ psf$

Roof Pressures Coefficients:

$L := L_B + l_{NS.OH} = 20.667\ ft$ $B := B_B + l_{EW.OH} = 78\ ft$

Variables Needed for Table: $\frac{h}{L} = 0.4738$ $\frac{h}{2} = 4.9\ ft$ $h = 9.8\ ft$ $2\ h = 19.6\ ft$ $\theta := 0$

External Pressure Coefficient: $C_{p1} := -0.9$ $C_{p2} := -0.18$ (ASCE 7-22: Figure 27.3-1)

Roof Pressures: $p := p_{iext}$ +/- p_{iint}

Surface 5:

$p_{E5ext1} := q_z \cdot G \cdot C_{p1} = -11.07\ psf$ $p_{E5ext2} := q_z \cdot G \cdot C_{p2} = -2.21\ psf$

$p_{E5int1} := q_i \cdot \left(\left(GC_{pi}\right)\right) = 2.6\ psf$ $p_{E5int2} := q_i \cdot \left(\left(GC_{pi}\right)\right) = 2.6\ psf$

$p_{E5plus1} := p_{E5ext1} + p_{E5int1} = -8.463\ psf$ $p_{E5plus2} := p_{E5ext2} + p_{E5int2} = 0.391\ psf$

$p_{E5minus1} := p_{E5ext1} - p_{E5int1} = -13.671\ psf$ $p_{E5minus2} := p_{E5ext2} - p_{E5int2} = -4.818\ psf$

Overhang Pressures: $p_{Eoh} := q_z \cdot G \cdot C_{p1} = -11.067\ psf$

IMPORTANT NOTE: p_{oh} Must be added to the positive external pressure on windward faces with overhang (surface 2))

SOUTH WIND CALCULATIONS (WIND DIRECTION C)

Wall Pressures Coefficients:

$L := L_C = 76\ ft$ $B := B_C = 16.667\ ft$

Variables Needed for Table: $\frac{L}{B} = 4.56$

External Pressure Coefficients: $C_{pWW} := 0.8$ $C_{pLW} := -0.2$ $C_{pS} := -0.7$ (ASCE 7-22: Figure 27.3-1)

Wall Pressures:

$p := p_{iext}$ +/- p_{iint}

Surface 1: (leeward)

$p_{S1ext} := q_z \cdot G \cdot C_{pLW} = -2.46\ psf$

$p_{S1int} := q_i \cdot (GC_{pi}) = 2.6\ psf$

$p_{S1plus} := p_{S1ext} + p_{S1int} = 0.145\ psf$

$p_{S1minus} := p_{S1ext} - p_{S1int} = -5.063\ psf$

Surface 2: (side)

$p_{S2ext} := q_z \cdot G \cdot C_{pS} = -8.61\ psf$

$p_{S2int} := q_i \cdot (GC_{pi}) = 2.6\ psf$

$p_{S2plus} := p_{S2ext} + p_{S2int} = -6.004\ psf$

$p_{S2minus} := p_{S2ext} - p_{S2int} = -11.212\ psf$

Surface 3: (windward)

$p_{S3ext} := q_z \cdot G \cdot C_{pWW} = 9.84\ psf$

$p_{S3int} := q_i \cdot (GC_{pi}) = 2.6\ psf$

$p_{S3plus} := p_{S3ext} + p_{S3int} = 12.442\ psf$

$p_{S3minus} := p_{S3ext} - p_{S3int} = 7.234\ psf$

Surface 4: (side)

$p_{S4ext} := q_z \cdot G \cdot C_{pS} = -8.61\ psf$

$p_{S4int} := q_i \cdot (GC_{pi}) = 2.6\ psf$

$p_{S4plus} := p_{S4ext} + p_{S4int} = -6.004\ psf$

$p_{S4minus} := p_{S4ext} - p_{S4int} = -11.212\ psf$

Roof Pressures Coefficients:

$L := L_C + l_{EW.OH} = 78\ ft$ $B := B_C + l_{NS.OH} = 20.667\ ft$

Variables Needed for Table: $\frac{h}{L} = 0.1255$ $\frac{h}{2} = 4.9\ ft$ $h = 9.8\ ft$ $2\ h = 19.6\ ft$ $\theta := 0$

External Pressure Coefficient: $C_{p1} := -0.9$ $C_{p2} := -0.18$ (ASCE 7-22: Figure 27.3-1)

Roof Pressures:

$p := p_{iext}$ +/- p_{iint}

Surface 5:

$p_{S5ext1} := q_z \cdot G \cdot C_{p1} = -11.07\ psf$ $p_{S5ext2} := q_z \cdot G \cdot C_{p2} = -2.21\ psf$

$p_{S5int1} := q_i \cdot (GC_{pi}) = 2.6\ psf$ $p_{S5int2} := q_i \cdot (GC_{pi}) = 2.6\ psf$

$p_{S5plus1} := p_{S5ext1} + p_{S5int1} = -8.463\ psf$ $p_{S5plus2} := p_{S5ext2} + p_{S5int2} = 0.391\ psf$

$p_{S5minus1} := p_{S5ext1} - p_{S5int1} = -13.671\ psf$ $p_{S5minus2} := p_{S5ext2} - p_{S5int2} = -4.818\ psf$

Overhang Pressures:

Note: No overhang on windward facing wall so overhang pressure does not need to be added.

WEST WIND CALCULATIONS (WIND DIRECTION D)

Wall Pressures Coefficients:

$L := L_D = 16.667\ ft$ $\quad B := B_D = 76\ ft$

Variables Needed for Table: $\frac{L}{B} = 0.219$

External Pressure Coefficients: $C_{pWW} := 0.8$ $\quad C_{pLW} := -0.5$ $\quad C_{pS} := -0.7$ (ASCE 7-22: Figure 27.3-1)

Wall Pressures: $p := p_{iext}$ +/- p_{iint}

Surface 1: (side)

$p_{W1ext} := q_z \cdot G \cdot C_{pS} = -8.61\ psf$

$p_{W1int} := q_i \cdot (GC_{pi}) = 2.6\ psf$

$p_{W1plus} := p_{W1ext} + p_{W1int} = -6.004\ psf$

$p_{W1minus} := p_{W1ext} - p_{W1int} = -11.212\ psf$

Surface 2: (leeward)

$p_{W2ext} := q_z \cdot G \cdot C_{pLW} = -6.15\ psf$

$p_{W2int} := q_i \cdot (GC_{pi}) = 2.6\ psf$

$p_{W2plus} := p_{W2ext} + p_{W2int} = -3.544\ psf$

$p_{W2minus} := p_{W2ext} - p_{W2int} = -8.753\ psf$

Surface 3: (side)

$p_{W3ext} := q_z \cdot G \cdot C_{pS} = -8.61\ psf$

$p_{W3int} := q_i \cdot (GC_{pi}) = 2.6\ psf$

$p_{W3plus} := p_{W3ext} + p_{W3int} = -6.004\ psf$

$p_{W3minus} := p_{W3ext} - p_{W3int} = -11.212\ psf$

Surface 4: (windward)

$p_{W4ext} := q_z \cdot G \cdot C_{pWW} = 9.84\ psf$

$p_{W4int} := q_i \cdot (GC_{pi}) = 2.6\ psf$

$p_{W4plus} := p_{W4ext} + p_{W4int} = 12.442\ psf$

$p_{W4minus} := p_{W4ext} - p_{W4int} = 7.234\ psf$

Roof Pressures Coefficients:

$L := L_D + l_{NS.OH} = 20.667\ ft$ $\quad B := B_D + l_{EW.OH} = 78\ ft$

Variables Needed for Table: $\frac{h}{L} = 0.4738$ $\quad \frac{h}{2} = 4.9\ ft$ $\quad h = 9.8\ ft$ $\quad 2\ h = 19.6\ ft$ $\quad \theta := 0$

External Pressure Coefficient: $C_{p1} := -0.9$ $\quad C_{p2} := -0.18$ (ASCE 7-22: Figure 27.3-1)

Roof Pressures: $p := p_{iext}$ +/- p_{iint}

Surface 5:

$p_{W5ext1} := q_z \cdot G \cdot C_{p1} = -11.07\ psf$ $\quad p_{W5ext2} := q_z \cdot G \cdot C_{p2} = -2.21\ psf$

$p_{W5int1} := q_i \cdot (GC_{pi}) = 2.6\ psf$ $\quad p_{W5int2} := q_i \cdot (GC_{pi}) = 2.6\ psf$

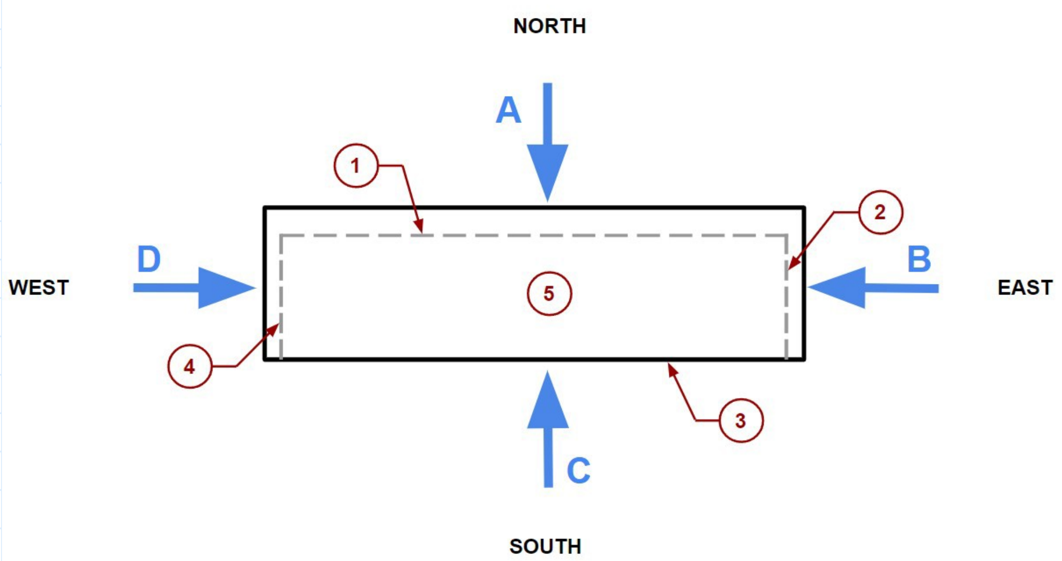
$p_{W5plus1} := p_{W5ext1} + p_{W5int1} = -8.463\ psf$ $\quad p_{W5plus2} := p_{W5ext2} + p_{W5int2} = 0.391\ psf$

$p_{W5minus1} := p_{W5ext1} - p_{W5int1} = -13.671\ psf$ $\quad p_{W5minus2} := p_{W5ext2} - p_{W5int2} = -4.818\ psf$

Overhang Pressures: $p_{Noh} := q_z \cdot G \cdot C_{p1} = -11.067\ psf$

IMPORTANT NOTE: p_{oh} Must be added to the positive external pressure on windward faces with overhang (surface 4))

SUMMARY CALCULATIONS



Maximum Positive Pressure on each wall and roof surface:

Surface 1: $\max\left(p_{N1plus}, p_{E1plus}, p_{S1plus}, p_{W1plus}, p_{N1minus}, p_{E1minus}, p_{S1minus}, p_{W1minus}\right) = 12.442\ psf$

Surface 2: $\max\left(p_{N2plus}, p_{E2plus}, p_{S2plus}, p_{W2plus}, p_{N2minus}, p_{E2minus}, p_{S2minus}, p_{W2minus}\right) = 12.442\ psf$

Surface 3: $\max\left(p_{N3plus}, p_{E3plus}, p_{S3plus}, p_{W3plus}, p_{N3minus}, p_{E3minus}, p_{S3minus}, p_{W3minus}\right) = 12.442\ psf$

Surface 4: $\max\left(p_{N4plus}, p_{E4plus}, p_{S4plus}, p_{W4plus}, p_{N4minus}, p_{E4minus}, p_{S4minus}, p_{W4minus}\right) = 12.442\ psf$

Surface 5: $\max\left(p_{N5plus1}, p_{N5plus2}, p_{E5plus1}, p_{E5plus2}, p_{S5plus1}, p_{S5plus2}, p_{W5plus1}, p_{W5plus2}\right) = 0.391\ psf$

Maximum Negative Pressure on each wall and roof surface:

Surface 1: $\min\left(p_{N1plus}, p_{E1plus}, p_{S1plus}, p_{W1plus}, p_{N1minus}, p_{E1minus}, p_{S1minus}, p_{W1minus}\right) = -11.212\ psf$

Surface 2: $\min\left(p_{N2plus}, p_{E2plus}, p_{S2plus}, p_{W2plus}, p_{N2minus}, p_{E2minus}, p_{S2minus}, p_{W2minus}\right) = -11.212\ psf$

Surface 3: $\min\left(p_{N3plus}, p_{E3plus}, p_{S3plus}, p_{W3plus}, p_{N3minus}, p_{E3minus}, p_{S3minus}, p_{W3minus}\right) = -11.212\ psf$

Surface 4: $\min\left(p_{N4plus}, p_{E4plus}, p_{S4plus}, p_{W4plus}, p_{N4minus}, p_{E4minus}, p_{S4minus}, p_{W4minus}\right) = -11.212\ psf$

Surface 5: $\min\left(p_{N5minus1}, p_{N5minus2}, p_{E5minus1}, p_{E5minus2}, p_{S5minus1}, p_{S5minus2}, p_{W5minus1}, p_{W5minus2}\right) = -13.671\ psf$

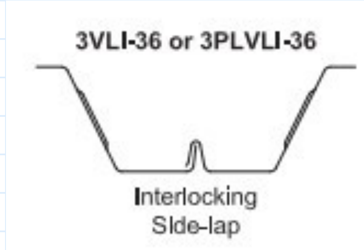
Overhang Pressures: $p_{oh} := q_z \cdot G \cdot C_{p1} = -11.067\ psf$

Phase 1 Restroom Composite Deck Slab

Selected: 3VLI-36 COMPOSITE DECK GRADE 50 STEEL

Link to Website: https://vulcraft.com/Products/Deck#composite-deck

Link to Datasheet: https://vulcraft.com/catalogs/Deck/CompositeDeck/LRFD-3VLI-36-3VLJ-36-3PLVLI-36_Composite_Deck-Slab.pdf



Slab Makeup:

Total - 6.5" + 1" concrete sloped

Topping - 3.5" of normal weight weight concrete (145 pcf) + 1" of normal concrete sloped from South to North

Deck Gauge - 16

Superimposed Load: Based on LRFD Load Combinations

LC	LRFD	ASD
1	$1.4D$	D
2	$1.2D+1.6L+0.5(L_r \text{ or } S \text{ or } R)$	$D+L$
3	$1.2D+1.6(L_r \text{ or } S \text{ or } R)+(L \text{ or } 0.5W)$	$D+(L_r \text{ or } S \text{ or } R)$
4	$1.2D+W+L+0.5(L_r \text{ or } S \text{ or } R)$	$D+0.75L+0.75(L_r \text{ or } S \text{ or } R)$
5	$0.9D+W$	$D+0.6W$
6	$1.2D+E_v+E_h+L+0.2S$	$D+0.75L+0.75(0.6W)+0.75(L_r \text{ or } S \text{ or } R)$
7	$0.9D-E_v+E_h$	$0.6D+0.6W$
8		$D+0.7E_v+0.7E_h$
9		$D+0.525E_v+0.525E_h+0.75L+0.75S$
10		$0.6D-0.7E_v+0.7E_h$

Where:
D= Dead Load
L= Live Load
Lr= Roof Live Load
S= Snow Load
W= Wind Load
R= Rain Load

<u>Loads:</u>

$Dead_{Membrane} := 1\ psf$

$Dead_{MEP} := 5\ psf$

$Dead_{RoofInsulation} := 0.365\ psf$

$Dead_{CompositeDeck} := 63.9\ psf$

$D := Dead_{Membrane} + (1\ in \cdot 145\ pcf) + Dead_{RoofInsulation} + Dead_{MEP} = 18.448\ psf$ **Superimposed**

Additional 1 inch of concrete for roof sloping, specified to installer

$L := 100\ psf$

$S := 35.7\ psf$

$W := 0.391 \ psf$

$W_{uplift} := -13.671 \ psf$

LRFD Load Combos:

$1.4 \ D = 25.828 \ psf$ **(1)**

$1.2 \ D + 1.6 \ L + 0.5 \ S = 199.988 \ psf$ **(2)**

$1.2 \ D + 1.6 \ S + L = 179.258 \ psf$ **(3)** Neglect wind load in combo 3 because it is smaller than L

$1.2 \ D + W + L + 0.5 \ S = 140.379 \ psf$ **(4)**

$0.9 \ \left(\left(D + Dead_{CompositeDeck} \right) \right) + W_{uplift} = 60.443 \ psf$ **(5)** Uplift is not an issue

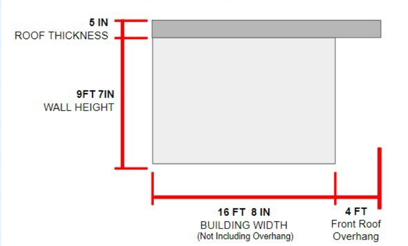
$MaxLoad := 199.988 \ psf$

Allowable Superimposed

Span:

At a preliminary span of 15'-4"

VIEW FROM SIDE B:



$Allowable := 216 \ psf$

$MaxLoad = 199.988 \ psf$ < $Allowable = 216 \ psf$

Sufficient for Allowable Deflection... which is the limiting factor!

Superimposed Design Load, ϕW_n, / Deflection at L/360 (psf) NWC (145 pcf), f'_c = 3000 psi

Total Slab Depth	Deck Gage	Span (ft-in.) 8'-0"	9'-0"	10'-0"	11'-0"	12'-0"	13'-0"	14'-0"	16'-0"
5"	22	592/643	456/451	360/329	288/247	233/190	191/149	157/120	108/80
	20	713/686	552/481	437/351	352/263	287/203	237/159	197/128	138/85
	19	825/724	641/508	509/370	411/278	337/214	279/168	233/135	166/90
	18	927/758	721/533	574/388	465/291	382/224	317/176	266/141	191/94
	16	1144/829	893/582	713/424	579/319	478/245	399/193	336/154	245/103
6½"	22	775/1360	597/955	469/696	375/523	303/403	247/317	203/253	138/170
	20	938/1445	725/1014	573/739	461/555	375/428	308/336	255/269	178/180
	19	1088/1521	843/1068	669/778	540/585	441/450	365/354	304/283	215/190
	18	1225/1589	952/1116	756/813	612/611	502/470	416/370	349/296	249/198
	16	1517/1729	1182/1214	943/885	766/665	631/512	527/403	443/322	321/216
7½"	22	907/2059	698/1446	548/1054	438/792	354/610	288/479	236/384	160/257
	20	1100/2182	850/1533	672/1117	540/839	439/646	361/508	299/407	208/272
	19	1278/2293	991/1611	785/1174	633/882	518/679	428/534	356/428	252/286
	18	1441/2393	1120/1681	890/1225	720/920	590/709	489/557	410/446	292/299
	16	1791/2600	1396/1826	1113/1331	904/1000	745/770	621/606	523/485	379/325

Notes:
1. For high loads long term concrete creep should be considered.
2. Use Composite Deck-Slab Strength Web Based Solutions for alternate slabs or ASD design.

Allowable Reaction at Supports (Based on LRFD Load Combinations) - for web crippling

Treat as a one way slab since the length is much much longer than the width

16 Deck Gauge
One Flange Loading
End bearing is 4"+

$Allowable := 2771\ plf$

Design Reactions at Supports Based on Web Crippling, ϕR_n (lb/ft)

	Bearing Length of Webs											
	One-Flange Loading						Two-Flange Loading					
Deck	End Bearing				Interior Bearing		End Bearing				Interior Bearing	
Gage	1½"	2"	3"	4"	4"	8"	1½"	2"	3"	4"	4"	8"
22	540	593	683	759	1164	1354	510	549	615	671	1353	1588
20	780	855	980	1085	1668	2065	792	851	948	1031	1975	2481
19	1046	1143	1305	1443	2221	2795	1119	1198	1330	1442	2665	3407
18	1324	1444	1645	1814	2798	3504	1473	1573	1742	1883	3389	4314
16	2049	2226	2521	2771	4291	5324	2430	2585	2845	3065	5275	6656

$$Actual := MaxLoad \cdot 8.3333333\ ft = \left(1.667 \cdot 10^{3}\right)\ plf$$

$Actual = \left(1.667 \cdot 10^{3}\right)\ plf$ $\quad < \quad$ $Allowable := 2771\ plf$

Sufficient, web crippling will not be an issue

Shear Stud Spacing

Link: https://vulcraft.com/DesignTools/HighPerformanceDeckSlabDiaphragmStrength

Required Shear

$$WindLoad_{Max} := 12.442\ psf$$

$$ShearDistrib_{Required} := 12.442\ psf \cdot \underbrace{5\ ft}_{\text{Half of building height}} = 62.21\ plf$$

Round up to 65 plf to be conservative

Input

Summary | Detailed Calc | Stud Strength | Perp. Conn. Pattern | Reinforcing Table | Technical Revisions

Keep up with Vulcraft/Verco by following us at https://www.linkedin.com/company/vulcraft--division-of-nucor-corp-/

High Performance Deck-Slab Diaphragm Strength
Input Design Criteria

NUCOR VULCRAFT

Print

Design Basis

Unit System	Imperial
Design Method	LRFD
Required Factored Diaphragm Shear, $\bar{v}$ (plf)	65

Deck Selection

Deck Option	Composite Deck
Deck Type	3VLI-36
Deck Gage	16
Deck Grade	Grade 50

Concrete Slab

Total Slab Thickness (in.)	5 ≤ 6.50 ≤ 9
Structural Concrete Unit Weight (pcf)	90 ≤ 145 ≤ 160
Structural Concrete Strength (psi)	2500 ≤ 3000 ≤ 5000

Structural Concrete Reinforcement

Concrete Reinforcement Type	Dramix® Steel Fibers
Fiber Type	Please Select
Fiber Dosage (pcy)	15 ≤ 15.00 ≤ 66

Support Connections

Deck to Support Connection Type	3/4" Steel Headed Stud Anchor
Support Member Thickness (in.)	0.3 ≤ 0.500
Support Member	A36

3VLI-36/3VLJ-36/3PLVLI-36 Composite Deck-Slab Information $f'_c = 3000$ psi

Total Slab Depth (in.)	Cover Depth (in.)	Theoretical Concrete Volume (yd³/100 ft²)	Min. A_s for T&S (in.²)	Recommended Reinforcing for Temperature and Shrinkage: WWR	(OR)	Bekaert Dramix® Steel Fiber Alternate to WWR (lb/yd³): 4D 65/60BG
Normal Weight Concrete (145 pcf)						
5	2	1.08	0.028	6x6-W1.4xW1.4		23
5½	2½	1.23	0.028	6x6-W1.4xW1.4		18
6	3	1.39	0.028	6x6-W1.4xW1.4		15
6½	3½	1.54	0.032	6x6-W2.1xW2.1		15
7	4	1.70	0.036	6x6-W2.1xW2.1		15
7½	4½	1.85	0.041	6x6-W2.1xW2.1		15
Light Weight Concrete (110 pcf)						
5	2	1.08	0.028	6x6-W1.4xW1.4		33
5½	2½	1.23	0.028	6x6-W1.4xW1.4		25
6	3	1.39	0.028	6x6-W1.4xW1.4		20
6¼	3¼	1.47	0.029	6x6-W2.1xW2.1		20
6½	3½	1.54	0.032	6x6-W2.1xW2.1		20
7¼	4¼	1.77	0.038	6x6-W2.1xW2.1		20

Notes:
1. FRC reinforcement is based on IAPMO UES ER-465.
2. Dramix® fibers may be used in UL or ULC fire rated assemblies in lieu of WWR. See UL file R19307 for additional information.

Output

Deck-Slab Diaphragm Shear Strength

16 ga 3VLI-36 Grade 50 Composite Deck

6.5 in. total slab depth, f'c = 3000 psi, 145 pcf NWC

15 pcy of Bekaert® Dramix® Please Select Fiber Reinforcing [4]

3/4" Steel Headed Stud Anchor at Chords & Collectors for Shear Transfer

Perpendicular Connection Pattern[1]			1 every 3rd rib	
Parallel Connection Attachment (maximum)	1	row at	16	in. o.c.

Minimum Connections to Supporting Members[2]

Minimum connections to all supports may be any of the following: arc spot welds, fillet welds, PAF's, screws, Shearflex® anchors, welded studs or other mechanical connections[3].

Perpendicular Connection Pattern[1]		1 per rib	
Parallel Perimeter Connections for deck spans greater than 5 ft		36	in. o.c.

Governing Deck-Slab Strength and Stiffness

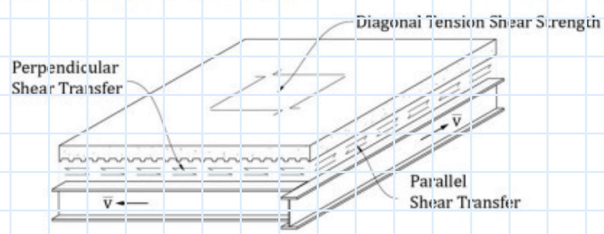
Available Diaphragm Design Strength	**Va = ΦQN =**	**3160**	**plf**
Controlled by Connections to Chords and Collectors			
Deck-Slab Diaphragm Design Shear Stiffness	G' =	1425	kip/in

Deck-Slab Shear Strength

Deck-Slab Diagonal Tension Design Shear Strength	ΦSn =	9118	plf

Chords & Collector Shear Transfer Strength

Chord & Collectors Design Shear Transfer Strength	ΦQN =	3160	plf



Notes:

1. For UL Fire rated assemblies, refer to UL Design Number for support and sidelap connection requirements.
2. Minimum connections to supporting members do not contribute to the diaphragm shear strength.
3. Support welds at interlocking sidelaps may be 3/8" x 1 1/4" arc seam welds in lieu of arc spot welds.
4. Dramix Steel Fibers up to 66 lb/cy are approved in lieu of welded wire fabric in all UL D700, D800 , and 900 Series Designs, and G229.
5. Sidelap connections between steel deck panels may be VSC2, button punch, screw, 1-1/2 in. arc seam weld or 1-1/2 in. top arc seam weld. The maximum sidelap connection spacing shall not exceed 36 in. o.c.

Calculations generated Per 2018 IBC & IAPMO ER-0652 using calculator V1.1 Date: 3/30/2023

Overhang Analysis – treat as beam with two simple supports and an overhang

Loads

$TribWidth := 3\ ft$

1 bearing plate/screw anchor every 3 ribs (3 feet)

Extra Concrete

$$Dead_{Overhang} := \left(\left(Dead_{Membrane} + Dead_{CompositeDeck} + 1\ in \cdot 145\ pcf\right)\right) \cdot TribWidth = 0.231\ klf$$

$$Dead_{Roof} := \left(\left(Dead_{MEP} + Dead_{RoofInsulation} + Dead_{Membrane} + Dead_{CompositeDeck} + 1\ in \cdot 145\ pcf\right)\right) \cdot TribWidth = 0.247\ klf$$

$$Live := L \cdot TribWidth = 0.3\ klf$$

Analyze it for the live over the total roof, just the overhang, and just over the span between the walls

$$Wind_{uplift} := W_{uplift} \cdot TribWidth = -0.041\ klf$$

Neglect Positive Wind load because it is so small

$$Snow := S \cdot TribWidth = 0.107\ klf$$

Load Definitions

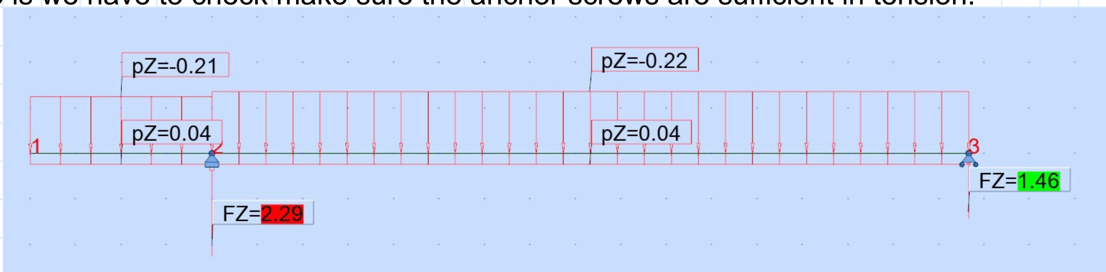
Case	Load type	List								
1:Dead Overh	uniform load	1	PX=0.0	PZ=-0.23	global	not project.	absolute	BE=0.0	DZ=0.0	MEMO:
2:Dead Roof	uniform load	2	PX=0.0	PZ=-0.25	global	not project.	absolute	BE=0.0	DZ=0.0	MEMO:
3:Live Overha	uniform load	1	PX=0.0	PZ=-0.30	global	not project.	absolute	BE=0.0	DZ=0.0	MEMO:
4:Live Roof	uniform load	2	PX=0.0	PZ=-0.30	global	not project.	absolute	BE=0.0	DZ=0.0	MEMO:
5:Live Total	uniform load	1 2	PX=0.0	PZ=-0.30	global	not project.	absolute	BE=0.0	DZ=0.0	MEMO:
6:Uplift	uniform load	1 2	PX=0.0	PZ=0.04	global	not project.	absolute	BE=0.0	DZ=0.0	MEMO:
7:Snow	uniform load	1 2	PX=0.0	PZ=-0.11	global	not project.	absolute	BE=0.0	DZ=0.0	MEMO:

Applied all LRFD Load Combos

Combinations	Name	Analysis type	Combination	Case nature	Definition
8 (C)	1.4D	Linear Combinati	ULS	dead	(1+2)*1.40
9 (C)	1.2D+1.6L(Overhang)+0.5S	Linear Combinati	ULS	dead	(1+2)*1.20+3*1.60+7*0.50
10 (C)	1.2D+1.6L(Roof)+0.5S	Linear Combinati	ULS	dead	(1+2)*1.20+4*1.60+7*0.50
11 (C)	1.2D+1.6L(Total)+0.5S	Linear Combinati	ULS	dead	(1+2)*1.20+5*1.60+7*0.50
12 (C)	1.2D+1.6S+L(Overhang)	Linear Combinati	ULS	dead	(1+2)*1.20+7*1.60+3*1.00
13 (C)	1.2D+1.6S+L(Roof)	Linear Combinati	ULS	dead	(1+2)*1.20+4*1.00+7*1.60
14 (C)	1.2D+1.6S+L(Total)	Linear Combinati	ULS	dead	(1+2)*1.20+5*1.00+7*1.60
15 (C)	1.2D+1.6S+0.5W	Linear Combinati	ULS	dead	(1+2)*1.20+7*1.60+6*1.00
16 (C)	1.2D+W+L(Overhang)+0.5S	Linear Combinati	ULS	dead	(1+2)*1.20+(3+6)*1.00+7*0.50
17 (C)	1.2D+W+L(Roof)+0.5S	Linear Combinati	ULS	dead	(1+2)*1.20+(6+4)*1.00+7*0.50
18 (C)	1.2D+W+L(Total)+0.5S	Linear Combinati	ULS	dead	(1+2)*1.20+(5+6)*1.00+7*0.50
19 (C)	0.9D+W	Linear Combinati	ULS	dead	(1+2)*0.90+6*1.00

Uplift

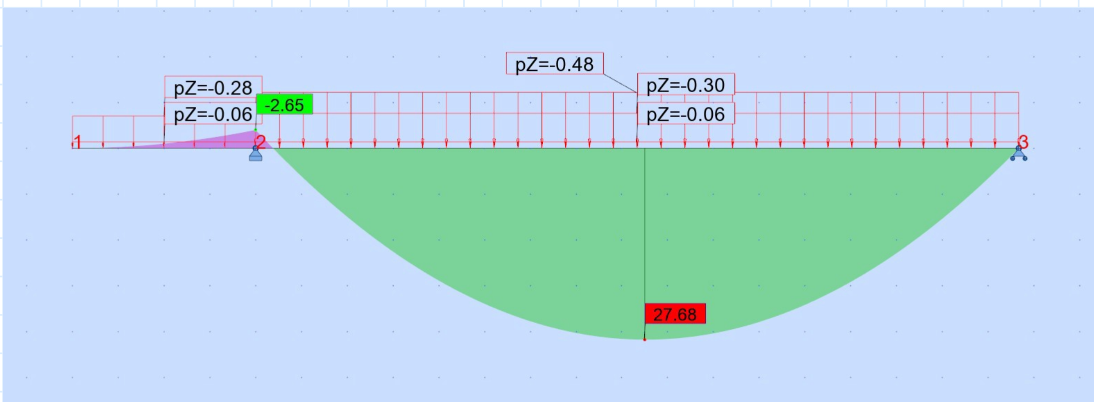
Check all load combos to make sure there is no negative load reaction at supports... if there is we have to check make sure the anchor screws are sufficient in tension.



0.9D+W produces the lowest reaction force, but it is still positive, so there is no need to solve for net tension in anchor screws.

Largest Positive Moment

Check all load combos to solve for the highest moment... this must be lower than the largest moment allowed for the roof deck.



1.2 D+1.6L(Roof)+0.5S produces the highest moment

Slab Depth		Deck	Maximum Unshored Spans: Maximum Unshored Construction Clear Span			Composite Deck-Slab Properties: Concrete + Deck	Deflection $I_d = (I_{cr}+I_u)/2$	Moment $ØM_{no}$	Shear $ØV_{no}$
Total	Topping	Gage	1	2	3	(psf)	(in⁴/ft)	(kip-ft/ft)	(kip/ft)
5"	2"	22	10'-0"	10'-7"	11'-0"	44.0	7.54	5.16	4.72
		20	11'-9"	12'-5"	12'-10"	44.4	8.04	6.14	5.61
		19	12'-3"	13'-10"	14'-3"	44.7	8.49	7.03	5.61
		18	12'-8"	15'-1"	14'-10"	45.0	8.89	7.86	5.61
		16	13'-4"	16'-8"	15'-8"	45.8	9.72	9.60	5.61
6½"	3½"	22	8'-9"	8'-3"	9'-4"	62.1	15.94	6.80	5.90
		20	10'-4"	10'-11"	11'-3"	62.5	16.93	8.11	7.36
		19	11'-4"	12'-2"	12'-7"	62.8	17.82	9.31	7.97
		18	11'-8"	13'-4"	13'-8"	63.1	18.62	10.41	7.97
		16	12'-4"	15'-0"	14'-6"	63.9	20.27	12.75	7.97
7½"	4½"	22	8'-2"	7'-2"	8'-2"	74.2	24.12	7.98	6.77
		20	9'-7"	10'-2"	10'-6"	74.6	25.57	9.52	8.23
		19	10'-9"	11'-5"	11'-9"	74.9	26.87	10.95	9.46
		18	11'-2"	12'-5"	12'-10"	75.2	28.04	12.26	9.70
		16	11'-10"	14'-0"	13'-11"	76.0	30.47	15.06	9.70

Note:
1. Maximum unshored spans do not consider web-crippling. Required bearing should be determined based on specific span conditions.

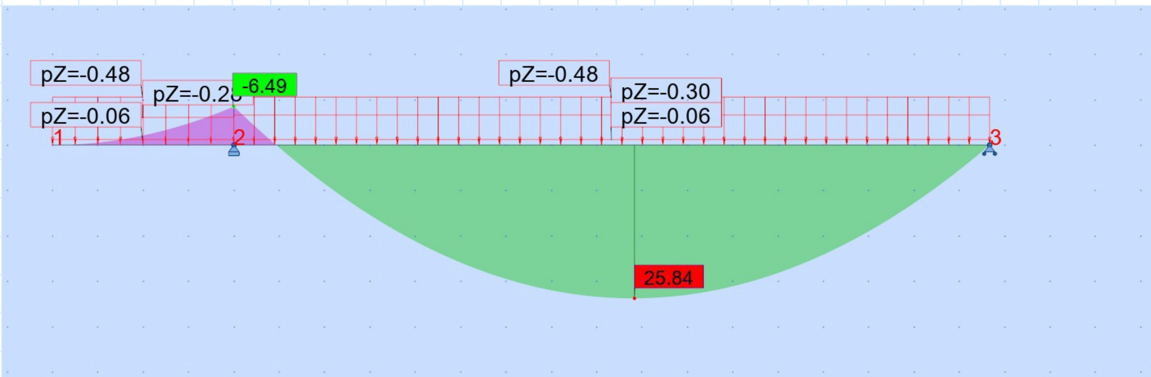
$$Moment_{Allow} := 12.75\ kip \cdot \frac{ft}{ft} \cdot TribWidth = 38.25\ kip \cdot ft$$

Sufficient

Largest Negative Moment

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+ (in^4/ft)	I_d- (in^4/ft)	Effective Section Modulus at F_y = 50 ksi S_e+ (in^3/ft)	S_e- (in^3/ft)	Design Moment $øM_n+$ (lb-ft/ft)	$øM_n-$ (lb-ft/ft)	Vertical Web Shear $øV_n$ (lb/ft)
22	1.7	0.0295	50	0.732	0.737	0.387	0.410	1452	1537	2138
20	2.1	0.0358	50	0.919	0.921	0.512	0.539	1920	2021	3777
19	2.4	0.0418	50	1.099	1.101	0.639	0.669	2397	2509	5152
18	2.7	0.0474	50	1.253	1.253	0.761	0.794	2854	2977	6628
16	3.5	0.0598	50	1.580	1.580	1.013	1.013	3799	3799	9312

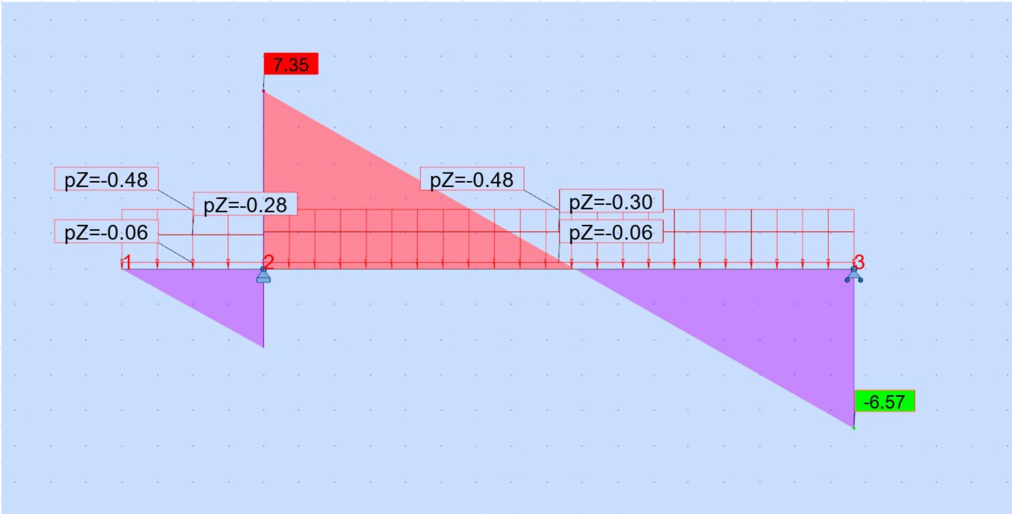
Max negative moment is not given for the deck section, so check to see if metal deck is sufficient in negative moment



1.2 D+1.6L(Total)+0.5S produces the highest negative moment

$$NegativeMoment_{Allow} = -3799\ lbf \cdot \frac{ft}{ft} \cdot TribWidth = -11.397\ kip \cdot ft$$ **Sufficient**

Largest Shear



1.2 D+1.6L(Total)+0.5S produces the highest shear

Slab Depth		Maximum Unshored Spans				Composite Deck-Slab Properties			
		Deck	Maximum Unshored Construction Clear Span			Concrete + Deck	Deflection $I_d = (I_{cr}+I_u)/2$	Moment $\varnothing M_{no}$	Shear $\varnothing V_{no}$
Total	Topping	Gage	1	2	3	(psf)	(in⁴/ft)	(kip-ft/ft)	(kip/ft)
5"	2"	22	10'-0"	10'-7"	11'-0"	44.0	7.54	5.16	4.72
		20	11'-9"	12'-5"	12'-10"	44.4	8.04	6.14	5.61
		19	12'-3"	13'-10"	14'-3"	44.7	8.49	7.03	5.61
		18	12'-8"	15'-1"	14'-10"	45.0	8.89	7.86	5.61
		16	13'-4"	16'-8"	15'-8"	45.8	9.72	9.60	5.61
6½"	3½"	22	8'-9"	8'-3"	9'-4"	62.1	15.94	6.80	5.90
		20	10'-4"	10'-11"	11'-3"	62.5	16.93	8.11	7.36
		19	11'-4"	12'-2"	12'-7"	62.8	17.82	9.31	7.97
		18	11'-8"	13'-4"	13'-8"	63.1	18.62	10.41	7.97
		16	12'-4"	15'-0"	14'-6"	63.9	20.27	12.75	7.97
7½"	4½"	22	8'-2"	7'-2"	8'-2"	74.2	24.12	7.98	6.77
		20	9'-7"	10'-2"	10'-6"	74.6	25.57	9.52	8.23
		19	10'-9"	11'-5"	11'-9"	74.9	26.87	10.95	9.46
		18	11'-2"	12'-5"	12'-10"	75.2	28.04	12.26	9.70
		16	11'-10"	14'-0"	13'-11"	76.0	30.47	15.06	9.70

$NegativeMoment_{Allow} := 7.97 \frac{kip}{ft} \cdot TribWidth = 23.91\ kip$ **Sufficient**

Deflection will not control with the 4 foot overhang

Therefore the overhang is okay to have and is structurally sound

Selection of Expansion Anchor Connecting Base Plate to CMU

Link: https://www.strongtie.com/mechanicalanchors_mechanicalanchoringproducts/tnt_screw/p/titen-turbo

Datasheet: http://embed.widencdn.net/pdf/plus/ssttoolbox/5lzzrblyap/C-A-2021-p147-149.pdf

Designing for the less long wall would result in a higher in plane shear force, so design for the East wall

$WallLength := 78\ ft$

Shear Parallel to Edge

$NumberofScrews := \left(\dfrac{16.666\ ft}{16\ in}\right) = 12.5$ Round $\boxed{NumberofScrews} := 13$

Parallel Connection Attachment (maximum)	1	row at	16	in. o.c.

$$ShearLoad_{Parallel} := \frac{\dfrac{WallLength}{2} \cdot ShearDistrib_{Required}}{NumberofScrews} = 186.63\ lbf$$

Shear Perp. to Edge

$ScrewSpacing := 16\ in$

Parallel Connection Attachment (maximum)	1	row at	16	in. o.c.

$$ShearLoad_{Perp} := ScrewSpacing \cdot ShearDistrib_{Required} = 82.947\ lbf$$

Tension is not required to be accounted for since there is no net uplift

Allowable Tension Load for Titen Turbo Screw Anchor Installed in Hollow CMU Wall Faces[1,2,3]

IBC *

Anchor Diameter (in.)	Embedment Depth (in.)	Minimum Dimensions (in.)			Allowable Load (lb.)[4]
		Spacing	Edge	End	
3⁄16	1 1⁄4	3	3 7⁄8	3 7⁄8	117
1⁄4	1 1⁄4	4	3 7⁄8	3 7⁄8	117

1. The tabulates values are for screw anchors installed in minimum 8"-wide grouted concrete masonry walls having reached a minimum f'm of 1,500 psi at time of installation.
2. Embedment is the thickness of the face shell.
3. Screw anchors may be installed at any location in the wall face provided the minimum edge and end distances are maintained.
4. Allowable loads are based on a safety factor of 5.0 for installations under the IBC and IRC.

Allowable Shear Load for Titen Turbo Screw Anchor Installed in Hollow CMU Wall Faces[1,2,3]

IBC *

Anchor Diameter (in.)	Embedment Depth (in.)	Minimum Dimensions (in.)			Direction of Loading	Allowable Load (lb.)[4]
		Spacing	Edge	End		
3⁄16	1 1⁄4	3	3 7⁄8	3 7⁄8	Toward edge, parallel to wall end	164
1⁄4	1 1⁄4	4	3 7⁄8	3 7⁄8	Toward edge, parallel to wall end	190

1. The tabulates values are for screw anchors installed in minimum 8"-wide grouted concrete masonry walls having reached a minimum f'm of 1,500 psi at time of installation.
2. Embedment is the thickness of the face shell.
3. Screw anchors may be installed at any location in the wall face provided the minimum edge and end distances are maintained.
4. Allowable loads are based on a safety factor of 5.0 for installations under the IBC and IRC.

* See p. 12 for an explanation of the load table icons.

Pick 1 1/4" Screw Anchor Diameter

$\max\left(\left(ShearLoad_{Parallel}, ShearLoad_{Perp}\right)\right) = 186.63\ lbf$ $\quad$ $Shear_{allow} := 190\ lbf$

Required $\quad$ Allowable

Sufficient

Length Required

$EmbedDepth := 1.25\ in$

$BasePlateDepth := 0.5\ in$

$LengthRequired := EmbedDepth + BasePlateDepth = 1.75\ in$

Model No.	Head Type	Coating	Material	Size (in.)	Drill Bit Dia. (in.)	Drive Type	Product Includes	Color	Packaging Qty.
TNT25114H	Hex Head	Zinc Plated with Ceramic Coating	Carbon Steel	1/4 x 1 1/4	3/16	5/16 in. Hex	-	Standard Blue	100
TNT25114TF	Flat Head	Zinc Plated with Ceramic Coating	Carbon Steel	1/4 x 1 1/4	3/16	T30 6-Lobe	(1) T30 drive bit	Standard Blue	100
TNTW25114TF	Flat Head	Zinc Plated with Ceramic Coating	Carbon Steel	1/4 x 1 1/4	3/16	T30 6-Lobe	(1) T30 drive bit	White	100
TNT25134H	Hex Head	Zinc Plated with Ceramic Coating	Carbon Steel	1/4 x 1 3/4	3/16	5/16 in. Hex	-	Standard Blue	100
TNT25134TF	Flat Head	Zinc Plated with Ceramic Coating	Carbon Steel	1/4 x 1 3/4	3/16	T30 6-Lobe	(1) T30 drive bit	Standard Blue	100
TNTW25134TF	Flat Head	Zinc Plated with Ceramic Coating	Carbon Steel	1/4 x 1 3/4	3/16	T30 6-Lobe	(1) T30 drive bit	White	100

Select TNTW25134TF Anchor Screw

$$NumberofScrews := \frac{16.66666\ ft \cdot 2}{16\ in} + \frac{78\ ft \cdot 2}{36\ in} = 77$$

Walls Parallel to Deck Span Every 16"	Walls Perp to Deck Span Every Third Rib

(SAME NUMBER OF SHEAR STUDS AS ANCHOR SCREWS - 77 EACH)

Phase One Restroom Wall Calculations

Link: https://ncma.org/resource/strength-design-of-concrete-masonry-walls-for-axial-flexure/

NCMA TEK

National Concrete Masonry Association
an information series from the national authority on concrete masonry technology

STRENGTH DESIGN OF CONCRETE MASONRY WALLS FOR AXIAL LOAD & FLEXURE

TEK 14-11B
Structural (2003)

Figures 1 through 8 apply to fully or partially grouted reinforced concrete masonry walls with a specified compressive strength f'_m of 1,500 psi (10.34 MPa), and a maximum wall height of 20 ft (6.10 m), Grade 60 (414 MPa) vertical reinforcement, with reinforcing bars positioned in the center of the wall and reinforcing bar spacing s from 8 in. to 120 in. (203 to 3,048 mm). The following discussion applies to simply supported walls and is limited to uniform lateral loads. Other support and loading conditions should comply with applicable engineering procedures. Each figure applies to one specific wall thickness and one reinforcing bar size.

Conditions are met:
-Walls are fully grouted
-CMU blocks have a compressive strength of 1500 psi - standard
-The wall height is 10 feet
-Grade 60 reinforcement will be used
-Reinforcement bars will be positioned in the center of the wall
-Spacing will be between the specified amounts
-Walls are simply supported... supported by the roof diaphragm at the top and the foundations at the bottom
-Wind load is a uniform lateral load (Uniform wall pressure)

****We can use this document****

In strength design, two different deflections are calculated; one for service level loads (δ_s) and another for factored loads (δ_u). For a uniformly loaded simply supported wall, the resulting bending moment is as follows:

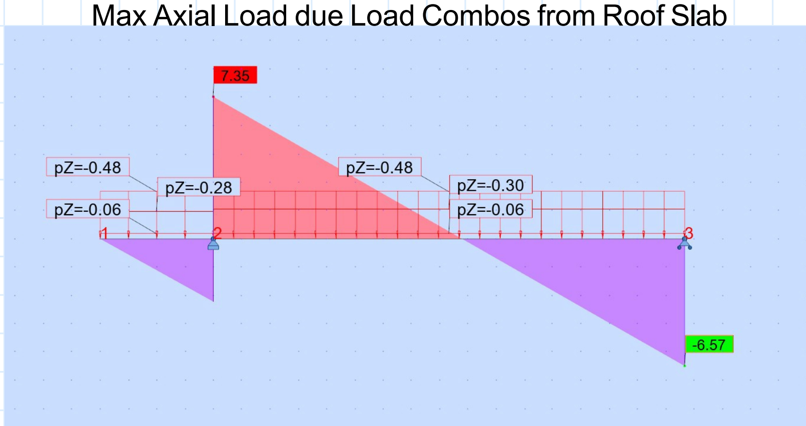
$$M_x = W_x h^2/8 + P_{xf}(e/2) + P_x \delta_x \qquad \text{(Eqn. 1)}$$

In the above equation, notations with "x" are replaced with factored or service level values as appropriate. The first term on the right side of Equation 1 represents the maximum moment of a uniform load at the mid-height of the wall (normally wind or earthquake loads). The second term represents the moment induced by eccentrically applied floor or roof loads. The third term is the P-delta effect, which is the moment induced by vertical axial loads and lateral deflection of the wall.

D	dead load, lb/ft (kN/m)
E_m	modulus of elasticity of masonry in compression, psi (MPa)
e	eccentricity of axial load - measured from centroid of wall, in. (mm)
f'_m	specified masonry compressive strength, psi (MPa)
f_r	modulus of rupture, psi (MPa)
f_1	factor for floor load: = 1.0 for floors in places of public assembly, for live loads in excess of 100 psf (4.8 kPa) and for parking garage live loads; = 0.5 otherwise
h	height of wall, in. (mm)
I_{cr}	moment of inertia of cracked cross-sectional area of a member, in.^{4}/ft (mm^4/m)
I_g	moment of inertia of gross cross-sectional area of a member, taken here as equal to I_{avg}, in.^{4}/ft (mm^4/m)
L	live load, lb/ft (kN/m)
L_r	roof live load, lb/ft (kN/m)
M_{cr}	nominal cracking moment strength, in.-lb/ft (kN·m/m)
M_{ser}	service moment at midheight of a member, including P-delta effects, in.-lb/ft (kN·m/m)
M_u	factored moment, in.-lb/ft or ft-lb/ft (kN·m/m)
P_u	factored axial load, lb/ft (kN/m)
P_{uf}	factored load from tributary floor or roof areas, lb/ft (kN/m)
P_w	load due to wall weight, lb/ft (kN/m)
S_n	section modulus of the net cross-sectional area of a member, in.^{3}/ft (mm^3/m)
s	spacing of vertical reinforcement, in. (mm)
W	wind load, psf (kN/m^2)
δ_s	horizontal deflection at midheight under service loads, in. (mm)
δ_u	deflection due to factored loads, in. (mm)

Solving for Moment:

Max Axial Load due Load Combos from Roof Slab



$W_x := 12.442\ psf$

$h := 10\ ft = 120\ in$

$P_{xf} := 7350\ plf$

$e := 0\ ft$

Assume eccentricity is 0

$P_x := 0\ \dfrac{lbf}{ft}$

No axial force acts directly on wall, there will be an axial force acting on the masonry column

$\delta_x := 0\ in$ The last term is neglected due to P_f being 0

$$M := \left(\frac{W_x \cdot h^2}{8}\right) + \left(\left(P_{xf}\right) \cdot \left(\frac{e}{2}\right)\right) + \left(\left(P_x \cdot \delta_x\right)\right) = 155.525\ \frac{ft \cdot lbf}{ft}$$

$P := P_{xf} = \left(\left(7.35 \cdot 10^3\right)\right)\ plf$

To size vertical rebar...

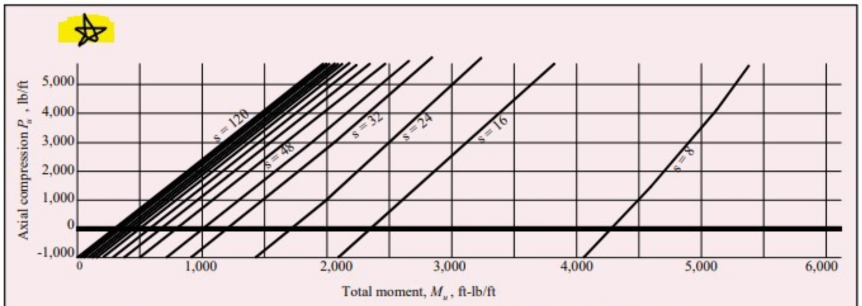


Figure 1—8-Inch (203-mm) Concrete Masonry Wall With No. 4 (M # 13) Reinforcing Bars

Our wall is an 8" CMU section... the moment is very low and the structure could qualify to not be reinforced, but we are going to use No.4 (M13) vertical reinforcing bars at 36 in spacing (3 feet spacing) to reinforce the walls (just being conservative). Professor Stoakes said that is the minimum spacing.

Phase One CMU Column Calculations

Link: https://ncma.org/resource/allowable-stress-design-of-concrete-masonry-columns/

INTRODUCTION

Masonry elements typically support both axial and lateral loads. For structural elements that resist primarily lateral forces, axial load can increase the element's flexural resistance. In this case, axial load is often neglected as a conservative assumption which simplifies the analysis. However, for elements carrying significant axial loads, such as columns, the additional moment due to lateral loads or eccentric axial loads typically reduces the element's axial capacity. In this case, the design must consider the interaction between axial load and moment.

The walls will need to support major axial point loads from the Phase 3 Roof, so columns are needed.

By definition, a column is an isolated vertical member whose horizontal dimension measured at right angles to its thickness does not exceed three times its thickness and whose height is greater than four times its thickness (ref. 1). Columns function primarily as compression members when supporting beams, girders, trusses or similar elements.

Our columns will meet this criteria.

The capacity of columns may be reduced due to either buckling or to additional bending moment caused by deflection (*P–D* effects). In Building Code Requirements for Masonry Structures (ref. 1, referred to hereafter as the Code), slenderness effects are included in the calculation of allowable compressive stress for reinforced masonry. For columns, the *Code* also limits the effective height to thickness ratio to 25, and requires a minimum nominal side dimension of 8 in. (203 mm).

The effective height of a column is typically taken as the clear height between supports. If the designer can demonstrate that there is reliable restraint against both translation and rotation at the supports, the effective height may be reduced in accordance with conventional design principles.

Eccentricity also affects the structural capacity of masonry columns. Eccentricity may be introduced by eccentric axial loads, lateral loads, or a column that is out of plumb. As a minimum, the *Code* requires that the design consider an eccentricity of 0.1 times each side dimension, with each axis considered independently. This minimum eccentricity is intended to account for construction tolerances. If the actual eccentricity exceeds this minimum, the actual eccentricity should be used in the design.

Th effective height of the column will be roughly 10 feet between the roof diaphragm and the floor slab. Our column will be at least 8 inches per side, so 10 ft/8 in = 15 so the ratio is for sure satisfied. In addition, we will be sure to concentrically load our columns, so eccentricity will not be an issue.

For Column Size.....

Section in Compression

An eccentricity located within the kern (center one-third) of the column places the entire section in compression. In this case, capacity is determined by the equations for P_a listed a above, and Table 2 can be used for design for columns up to 20 ft (6.1 m) high. The table assumes the element is in pure compression under a minimum design eccentricity of $0.1t$ for each axis, as required by the *Code*. The designer is responsible for confirming this.

Our columns will be concentrically loaded so we can use table 2 in this guide to size the columns based purely on the axial force

Table 2—Allowable Column Compressive Force for Concentrically Loaded Concrete Masonry Columns up to 20 ft (6.1 m) High[1]

Column size, in. (mm)	Allowable column compressive force, kip (kN)
8 x 8 (203x203)	18[2] (80)
8 x 16 (203x406)	37[2,3] (165)
8 x 24 (203x610)	56[2,4] (249)
10 x 16 (254x406)	46[5] (205)
10 x 24 (254x610)	71[6] (316)
12 x 12 (305x305)	42 (186)
12 x 16 (305x406)	56 (249)
12 x 24 (305x610)	85 (378)
12 x 32 (305x813)	114 (507)
16 x 16 (406x406)	76 (338)
16 x 24 (406x610)	115 (511)
16 x 32 (406x813)	154 (685)
24 x 24 (610x610)	174 (773)
24 x 32 (610x813)	233 (1030)
24 x 40 (610x1016)	292 (1300)

Notes:

[1] The table assumes the element is in pure compression, i.e., that the axial load falls within the center one-third of the section, under a minimum design eccentricity of $0.1t$ for each axis as required by the *Code*. The designer must ensure the section is in compression prior to using the table. f'_m = 1500 psi (10.3 MPa). F_s = 24,000 psi (165 MPa) (Grade 60 steel). One kip = 1,000 lb (4.4 kN).

SLENDERNESS LIMITATIONS:

[2] The maximum allowable height for 8 in. columns is 15.9 ft (4.8 m).

	Height ft (m)	Number of bars	Bar size	Maximum load kips (kN)
[3]	15.1-15.9 (4.6-4.8)	4	No. 4 (M13)	34 (151)
[4]	14.0-15.9 (4.3-4.8)	4	No. 4 (M13)	48 (213)
	15.1-15.9 (4.6-4.8)	6	"	52 (231)
	15.3-15.9 (4.6-4.8)	4	No. 5 (M16)	53 (236)
[5]	18.6-20 (5.6-6.1)	4	No. 4 (M13)	42 (186)
[6]	16.9-18 (5.1-5.5)	4	No. 4 (M13)	67 (298)
	18.0-20 (5.5-6.1)	4	"	60 (266)
	18.2-20 (5.5-6.1)	6	"	64 (284)
	19.3-20 (5.9-6.1)	8	"	68 (302)
	18.3-20 (5.6-6.1)	4	No. 5 (M16)	64 (284)
	19.7-20 (6.0-6.1)	6	"	70 (311)
	19.7-20 (6.0-6.1)	4	No. 6 (M19)	69 (307)

Table 2-Allowable Column Compressive Force for Concentrically Loaded Concrete Masonry Columns up to 20 ft (6.1 m) High

The maximum axial load from the column is 70 kip. Therefore, using a column size of 16"x16" would satisfy this axial load (due to its capacity of 76 kip) and conform to the geometry of the wall pretty well.

For Reinforcement.....

The *Code* allows lateral ties to be placed in either mortar or grout, although placement in grout more effectively prevents buckling and results in more ductile behavior. For this reason, the *Code* requires ties to be embedded in grout in Seismic Performance Categories D and E.

Vertical reinforcement and lateral ties are needed... for the lateral ties, they will be placed in the grout because that results in better performance.

Table 1—Allowable Column Reinforcement

Column size, in. (mm)	Number of reinforcing bars permitted, based on $0.0025A_n \leq A_u \leq 0.04A_n$, for bar sizes: No. 4 (M13)	No. 5 (M16)	No. 6 (M19)	No. 7 (M22)	No. 8 (M25)	No. 9 (M29)	No. 10 (M32)	No. 11 (M36)
8 x 8 (203x203)	4 - 10	4	4	N/A	N/A	N/A	N/A	N/A
8 x 16 (203x406)	4 - 12	4 - 12	4 - 10	4 - 8	4 - 6	4	N/A	N/A
8 x 24 (203x610)	4 - 12	4 - 12	4 - 12	4 - 12	4 - 8	4 - 6	4	4
10 x 16 (254x406)	4 - 12	4 - 12	4 - 12	4 - 10	4 - 6	4 - 6	4	N/A
10 x 24 (254x610)	4 - 12	4 - 12	4 - 12	4 - 12	4 - 10	4 - 8	4 - 6	4
12 x 12 (305x305)	4 - 12	4 - 12	4 - 12	4 - 8	4 - 6	4	4	N/A
12 x 16 (305x406)	4 - 12	4 - 12	4 - 12	4 - 12	4 - 8	4 - 6	4	4
12 x 24 (305x610)	4 - 12	4 - 12	4 - 12	4 - 12	4 - 12	4 - 10	4 - 8	4 - 6
12 x 32 (305x813)	6 - 12	4 - 12	4 - 12	4 - 12	4 - 12	4 - 12	4 - 10	4 - 8
16 x 16 (406x406)	4 - 12	4 - 12	4 - 12	4 - 12	4 - 12	4 - 8	4 - 6	4 - 8
16 x 24 (406x610)	6 - 12	4 - 12	4 - 12	4 - 12	4 - 12	4 - 12	4 - 10	4 - 8
16 x 32 (406x813)	8 - 12	4 - 12	4 - 12	4 - 12	4 - 12	4 - 12	4 - 12	4 - 12
24 x 24 (610x610)	8 - 12	6 - 12	4 - 12	4 - 12	4 - 12	4 - 12	4 - 12	4 - 12
24 x 32 (610x813)	10 - 12	8 - 12	6 - 12	4 - 12	4 - 12	4 - 12	4 - 12	4 - 12
24 x 40 (610x1016)	12	8 - 12	6 - 12	4 - 12	4 - 12	4 - 12	4 - 12	4 - 12

Table 1–Allowable Column Reinforcement

Since Grade 60 No. 4 (M13) bars are used for the walls, we are sticking with this type of reinforcement. 8 vertical bars are required for a 16"x16" column due to a max of 6" clear distance. Additional reinforcement requirements are shown below.

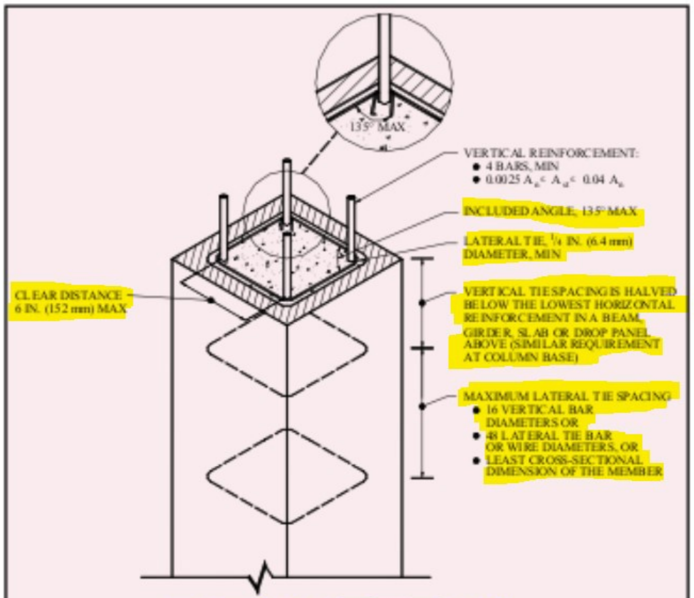


Figure 1—Column Reinforcement and Lateral Tie Requirements

Figure 1—Column Reinforcement and Lateral Tie Requirements

The lateral tie spacing will be 16 vertical bar diameters. The vertical bars are No. 4 bars. No. 4 bars have a diameter of 0.5". Therefore, the vertical spacing of the ties is 8 inches. The ties are 0.25" in diameter.

Phase Three Restroom Wall Calculations

Link: https://ncma.org/resource/loadbearing-concrete-masonry-wall-design/

The design aids in this TEK cover combined axial compression or axial tension and flexure, as determined using the allowable stress design provisions of Building Code Requirements for Masonry Structures (ref. 1). The data in this TEK applies to 8 in. (203 mm) thick reinforced concrete masonry walls with a specified compressive strength, f'_m, of 1500 psi (10.3 MPa), and a maximum wall height of 20 ft (6.1 m) (taller walls can be evaluated using the NCMA computer software (ref. 3) or other design tools). Reinforcing bars are assumed to be located at the center of the wall, and bar sizes 4, 5, 6, 7, and 8 are included.

The maximum wall height of the phase 3 ramp is 15 feet, so we can use this guide.

There will be no axial load on this wall. The moment is as follows

$h := 15\ ft$ Max height of ramp wall

$W_x := 12.442\ psf$ load from wind

$$M := \left(\frac{W_x \cdot h^2}{2}\right) = \left(1.4 \cdot 10^3\right) \frac{ft \cdot lbf}{ft}$$

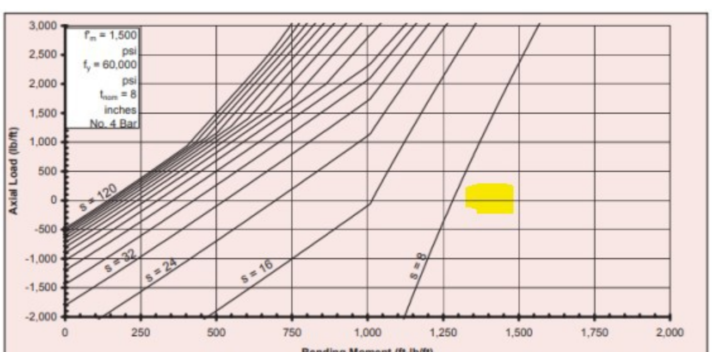


Figure 3—Interaction Diagram of 8 in. (203 mm) Concrete Masonry Wall With No. 4 Reinforcing Bars

There is no No. 4 reinforcement spacing that is sufficient for the walls, so the reinforcement must be sized up to a No. 5 bar

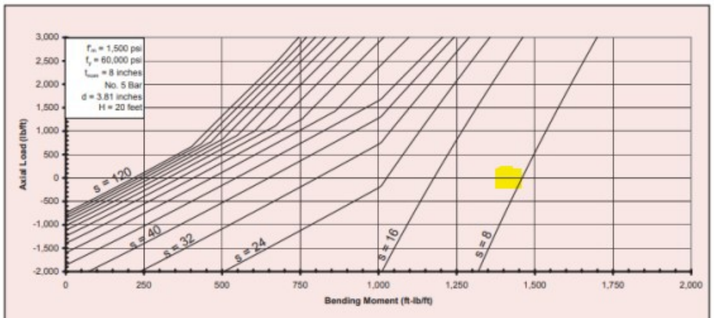


Figure 4—Interaction Diagram of 8 in. (203 mm) Concrete Masonry Wall With No. 5 Reinforcing Bars

Figure 4–Interaction Diagram of 8 in. (203 mm) Concrete Masonry Wall With No. 5 Reinforcing Bars

No. 5 vertical reinforcement bars at 8 inch spacing is sufficient for the moment

Horizontal reinforcement will be the same as specified for the phase one restroom walls

Anchoring of Veneer

Link: https://ncma.org/resource/concrete-masonry-veneers/

A 1 in. (25 mm) minimum air space must be maintained between the anchored veneer and backing to facilitate drainage. A 1 in. (25 mm) air space is considered appropriate if special precautions are taken to keep the air space clean (such as beveling the mortar bed away from the cavity). Otherwise, a 2 in. (51 mm) air space is preferred. As an alternative, proprietary insulating drainage products can be used.

A 1 inch gap between backing and veneer is appropriate

Selecting Wall Ties

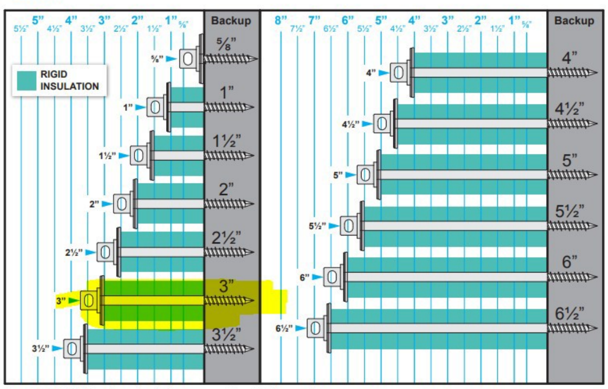
Link: https://www.blok-lok.com/index.php/product/2-seal-concrete-seal-tie/

Spec Sheet: https://www.blok-lok.com/wp-content/uploads/2020/04/13CON2SEALTIE.pdf

Our cavity is 4" (3" insulation and 1" air gap)...
was tested with a 4.5" gap so it is sufficient

HOHMANN & BARNARD, INC.
a MiTek company

Concrete 2-Seal™ Tie
CONCRETE, CMU, or MASONRY BACKUP
SIZE & SELECTION CHART



3" of insulation for restroom structure, so use the highlighted tie... for the stage walls and the ramp walls, use the 5/8" embedded wall tie

Concrete Seal Tie™ is an innovative single screw veneer tie suitable for use with concrete, CMU, wood stud, or brick backup. (for wood stud refer to wood stud size and selection chart)

- Screw has alternating threads and two factory-installed EPDM sealing washers
- Available for 5/8" - 6½" insulation and/or wall board thickness
- **Pre-drill a 7/32" (4 mm) hole into concrete, CMU, or masonry backup to a depth of 2" (51 mm)**

Barrel (Zamac Zinc): **ASTM B86** (92% Zinc Alloy) with protective polymer coating for corrosion resistance

Screw (Carbon Steel): **ASTM A510** (Carbon Steel)
ASTM C954 (1000-hour polymer coating)

2-SEAL WITH 2-SEAL WIRE TIE (WORKING LOAD*)

CAVITY	0" OFFSET	5/8" OFFSET	1¼" OFFSET	TEST
6½"	573#	N/A	206#	TENSION
6½"	402#	N/A	166#	COMPRESSION

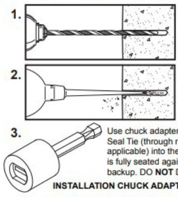
SCREW PULL-OUT (1 ¼" minimum embed)

CONCRETE	C-90 Hollow Block
810# (average ultimate load)	700# (average ultimate load)

* WORKING LOAD DETERMINED AT .05" DEFLECTION

Tests were completed for 4 ½" insulation with 2" air cavity.

Pullout values assume wire 2-Seal Byna-Lok Wire Tie is fully engaged into 2-Seal Tie with "0" eccentricity.



1. With a hammer drill, pre-drill 7/32" (4 mm) hole to a depth of 2" (51 mm).
2. Remove dust and loose particles from drilling using hand pump, compressed air, or vacuum.
3. Use chuck adapter to drive the Concrete Seal Tie (through rigid insulation if applicable) into the pre-drilled hole until it is fully seated against the face of the backup. DO **NOT** DRIVE in hammer mode.

INSTALLATION CHUCK ADAPTER

NOTE: In some areas extremely hard aggregate is mixed into concrete. Test installation into concrete over 5000 psi.
A slightly larger drill bit may be required.

HOHMANN & BARNARD, Inc. | 30 Rasons Court | Hauppauge, NY 11788
T: 800.645.0616 F: 631.234.0683 | www.h-b.com

HOHMANN & BARNARD, INC.
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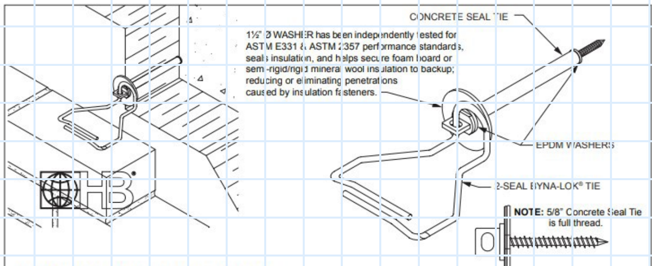
Veneer Anchors

Concrete Seal Tie™



2-SEAL WITH 2-SEAL WIRE TIE (WORKING LOAD*)

CAVITY	0" OFFSET	5/8" OFFSET	¼" OFFSET	TEST
6 ½"	573#	N/A	206#	TENSION
6 ½"	402#	N/A	166#	COMPRESSION

* WORKING LOAD DETERMINED AT .05" DEFLECTION

Tests were completed for 4 ½" insulation with 2" air cavity.

Pullout values assume wire 2-Seal Byna-Lok Wire Tie is fully engaged into 2-Seal Tie with "0" eccentricity.

SCREW PULL-OUT (1 ¼" minimum embed)

CONCRETE	C-90 Hollow Block
810# (average ultimate load)	700# (average ultimate load)

DRAWINGS FOR ILLUSTRATIVE PURPOSES ONLY

H&B RECOMMENDS 16" X 16" SPACING

Concrete Seal Tie™ is an innovative single screw veneer tie suitable for use with concrete, CMU, wood stud or brick backup.

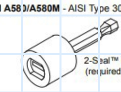
- Screw with alternating threads and two factory-installed EPDM sealing washers
- Available for 5/8" - 4 ½" insulation and/or wall board thickness
- **Pre-drill a 7/32" (4 mm) hole into concrete, CMU, or masonry backup to a depth of 2" (51 mm)**

Barrel (Zamac Zinc): **ASTM B86** (92% Zinc Alloy) with protective polymer coating for corrosion resistance

Screw (Carbon Steel): **ASTM A510** (Carbon Steel) **ASTM C954** (1000-hour polymer coating)

Wire (Carbon Steel): Prefabricated from cold-drawn steel wire conforming to **ASTM A1064/A1064M**
Tensile Strength - 80,000 p.s.i. | Yield Point - 70,000 p.s.i. minimum
Zinc Coating:
Hot-Dip Galvanized after fabrication: **ASTM A153/A153M-B** (1.5 oz/ft²)

Wire (Stainless Steel): **ASTM A580/A580M** - AISI Type 304 or Type 316



2-Seal™ Chuck Adapter (required)

Concrete Seal Tie Finish*: *For Stainless Steel, refer to Thermal Concrete 2-Seal™ or Thermal Concrete 2-Seal™ Wing Nut Anchors.
☑ Polymer Coated

Concrete Seal Tie (Equal to thickness of insulation):
☐ ⅝" ☐ 1" ☐ 1 ½" ☐ 2" ☐ 2 ½" ☑ 3" ☐ 3 ½" ☐ 4" ☐ 4 ½" ☐ 5"

2-Seal Byna-Lok™ Wire Tie Finish:
☑ Hot-Dip Galvanized | Stainless Steel ☐ Type 304 ☐ Type 316

2-Seal Byna-Lok™ Wire Tie Length: (3/16"Ø)
☐ 3" ☑ 4" ☐ 5" ☐ Custom ____

Continuous Wire Finish:
☑ Hot-Dip Galvanized | Stainless Steel ☐ Type 304 ☐ Type 316

Continuous Wire Diameter: ☐ 9 gauge ☑ 3/16"Ø

☑ **2-Seal™ Chuck Adapter (required)**

Note: Hohmann & Barnard recommends Stainless Steel for maximum protection against corrosion.

IMPORTANT: Since each construction project is unique, the appropriate selection and use of any product contained herein must be determined by competent architects, engineers and other appropriate professionals who are familiar with the specific requirements of the project in question.

HOHMANN & BARNARD, Inc.
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Branch/Subsidiary Locations:
ALABAMA - ARIZONA - ILLINOIS
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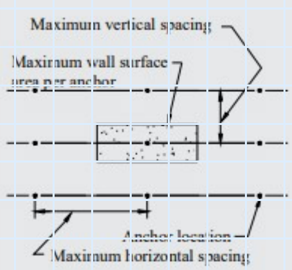
CANADA

PRINT FORM
RESET FORM SAVE FORM
SAVE FORM LOCK & HIDE BUTTONS

16" by 16" OC Spacing is recommended

Anchor Veneer (ref. 2)

Table 1—Anchor Spacing Requirements (ref. 2)



Backing	Type of anchor	Max. wall surface area, ft² (m²)[a]	Anchor spacing Max. vertical spacing, in. (mm)	Max. horizontal spacing, in. (mm)
Masonry	wire, adjustable, or joint reinforcement	2.67 (0.25)	18 (457)	32 (813)
Concrete	adjustable	2.67 (0.25)	18 (457)	32 (813)
Wood stud	adjustable two-piece, anchors of wire size W1.7 (MW 11), or 22 gauge (0.8 mm) corrugated sheet metal	2.67 (0.25)	18 (457)	32 (813)
	all other anchors	3.5 (0.33)	18 (457)	32 (813)
Steel stud	adjustable	2.67 (0.25)	18 (457)	32 (813)

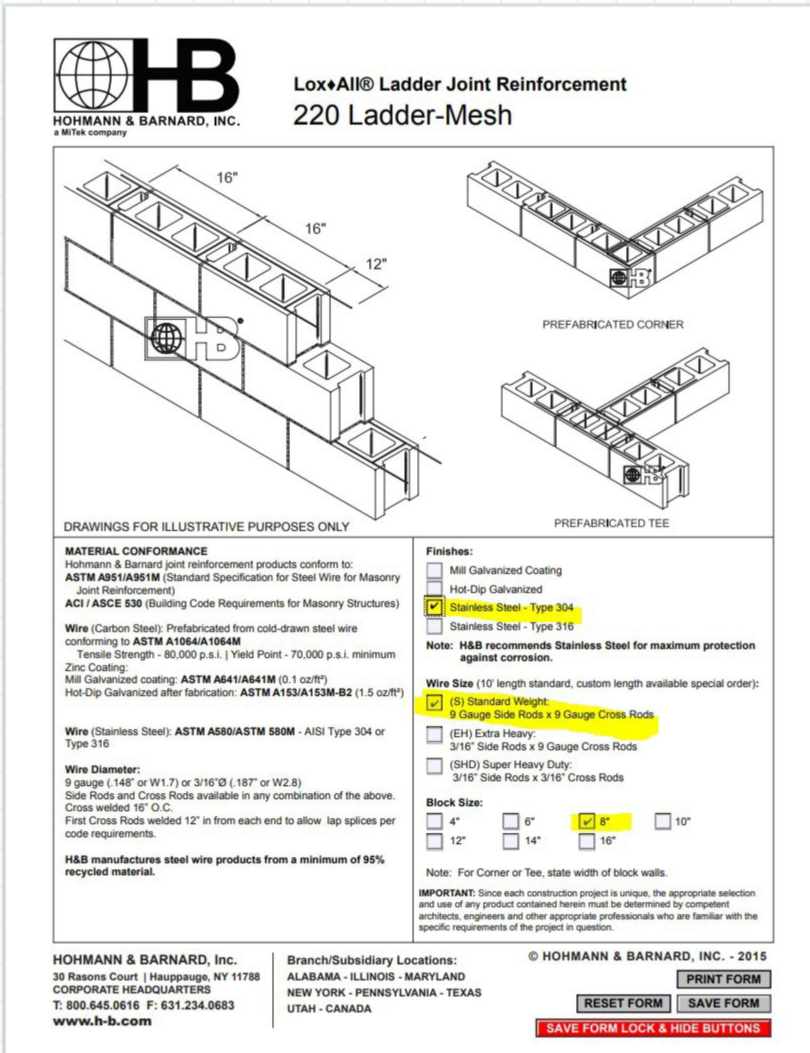
Additional requirements:

- When anchored veneer is laid in other than running bond, the veneer shall have joint reinforcement of at least one W1.7 (MW 11) wire, spaced at a maximum of 18 in. (457 mm) on center vertically to increase the flexural strength of the veneer in the horizontal span.
- Around openings larger than 16 in. (406 mm) in either dimension, space anchors around perimeter of opening at a maximum of 3 ft (0.91 m) on center, and place anchors within 12 in. (305 mm) of opening.

[a] For Seismic Design Categories D, E and F, reduce maximum wall area supported by each anchor to 75% of values shown

Selecting Horizontal Reinforcement

Link: https://www.h-b.com/index.php?main_page=product_info&products_id=75
Spec Sheet: https://www.h-b.com/images/submittal/08220LAPRITEL.pdf


HOHMANN & BARNARD, INC.
a MiTek company

Lox♦All® Ladder Joint Reinforcement
220 Ladder-Mesh

DRAWINGS FOR ILLUSTRATIVE PURPOSES ONLY

MATERIAL CONFORMANCE
Hohmann & Barnard joint reinforcement products conform to:
ASTM A951/A951M (Standard Specification for Steel Wire for Masonry Joint Reinforcement)
ACI / ASCE 530 (Building Code Requirements for Masonry Structures)

Wire (Carbon Steel): Prefabricated from cold-drawn steel wire conforming to **ASTM A1064/A1064M**
Tensile Strength - 80,000 p.s.i. | Yield Point - 70,000 p.s.i. minimum
Zinc Coating:
Mill Galvanized coating: **ASTM A641/A641M** (0.1 oz/ft²)
Hot-Dip Galvanized after fabrication: **ASTM A153/A153M-B2** (1.5 oz/ft²)

Wire (Stainless Steel): **ASTM A580/ASTM 580M** - AISI Type 304 or Type 316

Wire Diameter:
9 gauge (.148" or W1.7) or 3/16"Ø (.187" or W2.8)
Side Rods and Cross Rods available in any combination of the above.
Cross welded 16" O.C.
First Cross Rods welded 12" in from each end to allow lap splices per code requirements.

H&B manufactures steel wire products from a minimum of 95% recycled material.

Finishes:
- [ ] Mill Galvanized Coating
- [ ] Hot-Dip Galvanized
- [x] Stainless Steel - Type 304
- [ ] Stainless Steel - Type 316

Note: H&B recommends Stainless Steel for maximum protection against corrosion.

Wire Size (10' length standard, custom length available special order):
- [x] (S) Standard Weight: 9 Gauge Side Rods x 9 Gauge Cross Rods
- [ ] (EH) Extra Heavy: 3/16" Side Rods x 9 Gauge Cross Rods
- [ ] (SHD) Super Heavy Duty: 3/16" Side Rods x 3/16" Cross Rods

Block Size:
- [ ] 4"
- [ ] 6"
- [x] 8"
- [ ] 10"
- [ ] 12"
- [ ] 14"
- [ ] 16"

Note: For Corner or Tee, state width of block walls.

IMPORTANT: Since each construction project is unique, the appropriate selection and use of any product contained herein must be determined by competent architects, engineers and other appropriate professionals who are familiar with the specific requirements of the project in question.

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UTAH - CANADA

PRINT FORM
RESET FORM SAVE FORM
SAVE FORM LOCK & HIDE BUTTONS

For CMU horizontal reinforcing, 220 Ladder-Mesh to prevent cracking, placed every layer

SPECIFYING BRICK VENEER CONTROL JOINTS

Link: https://ncma.org/resource/crack-control-for-concrete-brick-and-other-concrete-masonry-veneers/

Crack Control Recommendations for Concrete Masonry Veneers[a]

- Control joints: maximum panel length to height ratio of 1 1/2, and maximum spacing of 20 ft (6.1 m) and where stress concentrations occur
- Joint reinforcement: at 16 in. (406 mm) o.c.
- Mortar: Type N

[a] Adjust as needed to suit local conditions and experience.

Height - 10 feet

Every 15 feet there will be an control joint and also at stress concentrations

Determining Lintel Size

Link: https://ncma.org/resource/precast-concrete-lintels-for-concrete-masonry-construction/
Link: https://ncma.org/resource/asd-of-concrete-masonry-lintels-2012-ibc-2011-msjc/

Design for the longest span...

A modular lintel length should be specified, with a minimum length of the clear span plus 8 in. (203 mm), to provide at least 4 in. (102 mm) bearing at each end (ref. 1). Additionally, if lintels are subjected to tensile stresses during storage, transportation, handling, or placement, it is recommended that steel reinforcement be provided in both the top and bottom to prevent cracking. Minimum concrete cover over the steel should be 1 ½ in. (13 mm). The lintel width, or width of the combination of side-by-side lintels, should equal the width of the supported masonry wythe.

Longest clear span is 8 feet

$Length := 5.5\ ft + 4\ in + 4\ in = 6.167\ ft$

$Width := 8\ in$ Width of wythe

Reinforcement will be needed

Need to design for flexure and shear

equal or exceed the factored loads. Precast concrete strength reduction factors are 0.9 and 0.85 for flexure and shear, respectively (ref. 2).

Flexure strength

$$\phi M_n = \phi [A_s f_y (d - a/2)],\ \phi = 0.9$$

Shear strength

$$\phi V_n = \phi (2) (f'_c)^{1/2} bd,\ \phi = 0.85$$

Table 1 - Shear and Moment Capacity for 4 x 8 in. (102 x 203 mm) Reinforced Concrete Lintels

Reinforcing bar size (No.)	No. of bars	f'_c, psi (MPa): 3000 (20.7) ϕV_n lb (kN)	3000 (20.7) ϕM_n in.-lb (kN m)	3500 (24.1) ϕV_n lb (kN)	3500 (24.1) ϕM_n in.-lb (kN m)	4000 (27.6) ϕV_n lb (kN)	4000 (27.6) ϕM_n in.-lb (kN m)
3 (10M)	1	2,000 (8.9)	33,140 (3.75)	2,160 (9.6)	33,450 (3.78)	2,310 (10.3)	33,670 (3.81)
4 (13M)	1	1,980 (8.8)	56,440 (6.38)	2,140 (9.5)	57,440 (6.49)	2,290 (10.2)	58,190 (6.57)
5 (16M)	1	1,960 (8.7)	80,450 (9.09)	2,110 (9.4)	82,860 (9.36)	2,260 (10.1)	84,670 (9.57)

Table 2 - Shear and Moment Capacity for 6 x 8 in. (152 x 203 mm) Reinforced Concrete Lintels

Reinforcing bar size (No.)	No. of bars	f'_c, psi (MPa): 3000 (20.7) ϕV_n lb (kN)	3000 (20.7) ϕM_n in.-lb (kN m)	3500 (24.1) ϕV_n lb (kN)	3500 (24.1) ϕM_n in.-lb (kN m)	4000 (27.6) ϕV_n lb (kN)	4000 (27.6) ϕM_n in.-lb (kN m)
4 (13M)	1	3,070 (13.7)	58,930 (6.66)	3,320 (14.8)	59,570 (6.73)	3,550 (15.8)	60,060 (6.79)
5 (16M)	1	3,040 (13.5)	86,440 (9.77)	3,280 (14.6)	87,990 (9.94)	3,510 (15.6)	89,160 (10.1)
3 (10M)	2	3,110 (13.8)	65,070 (7.35)	3,350 (14.9)	65,850 (7.44)	3,590 (16.0)	66,430 (7.51)
4 (13M)	2	3,070 (13.7)	108,820 (12.3)	3,320 (14.8)	111,410 (12.6)	3,550 (15.8)	113,340 (12.8)
5 (16M)	2	[2]	[2]	[2]	[2]	3,510 (15.6)	162,040 (18.3)

Table 3 - Shear and Moment Capacity for 8 x 8 in. (203 x 203 mm) Reinforced Concrete Lintels

Reinforcing bar size (No.)	No. of bars	f'_c, psi (MPa): 3000 (20.7) ϕV_n lb (kN)	3000 (20.7) ϕM_n in.-lb (kN m)	3500 (24.1) ϕV_n lb (kN)	3500 (24.1) ϕM_n in.-lb (kN m)	4000 (27.6) ϕV_n lb (kN)	4000 (27.6) ϕM_n in.-lb (kN m)
4 (13M)	1	4,170 (18.6)	60,110 (6.79)	4,500 (20.0)	60,590 (6.85)	4,810 (21.4)	60,950 (6.89)
5 (16M)	1	4,120 (18.4)	89,290 (10.1)	4,450 (19.8)	90,430 (10.2)	4,760 (21.2)	91,290 (10.3)
6 (19M)	1	4,080 (18.2)	120,490 (13.6)	4,410 (19.6)	122,790 (13.9)	4,710 (21.0)	124,520 (14.1)
4 (13M)	2	4,170 (18.6)	113,560 (12.8)	4,500 (20.0)	115,470 (13.0)	4,810 (21.4)	116,900 (13.2)
5 (16M)	2	4,120 (18.4)	162,570 (18.4)	4,450 (19.8)	167,150 (18.9)	4,760 (21.2)	170,580 (19.3)
6 (19M)	2	[2]	[2]	[2]	[2]	4,710 (21.0)	224,840 (25.4)

Table 4 - Shear and Moment Capacity for 8 x 16 in. (203 x 406 mm) Reinforced Concrete Lintels[3]

Reinforcing bar size (No.)	No. of bars	f'_c, psi (MPa): 3000 (20.7) ϕV_n lb (kN)	3000 (20.7) ϕM_n in.-lb (kN m)	3500 (24.1) ϕV_n lb (kN)	3500 (24.1) ϕM_n in.-lb (kN m)	4000 (27.6) ϕV_n lb (kN)	4000 (27.6) ϕM_n in.-lb (kN m)
6 (19M)	1	9,760 (43.4)	310,570 (35.1)	10,540 (46.9)	312,870 (35.4)	11,270 (50.1)	314,600 (35.5)
4 (13M)	2	9,850 (43.8)	286,360 (32.4)	10,640 (47.3)	288,270 (32.6)	11,370 (50.6)	289,700 (32.7)
5 (16M)	2	9,800 (43.6)	430,410 (48.6)	10,590 (47.1)	434,990 (49.1)	11,320 (50.4)	438,420 (49.5)
6 (19M)	2	9,760 (43.4)	588,870 (66.5)	10,540 (46.9)	598,090 (67.6)	11,270 (50.1)	605,000 (68.4)

1. Tables based on strength design method as described in ref. 2, assuming 1.5 in. (38 mm) concrete cover and Grade 60 reinforcement, f_y = 60,000 psi (413 MPa).
2. Reinforcement at listed effective depth exceeds the maximum reinforcing ratio of 0.75 ρ_b.
3. When determining minimum end bearing, the bearing stress of the masonry supporting the lintel should be checked to ensure it does not exceed $0.25f'_m$ (ref. 1).

Use precast lintel chart

Loading

LC	LRFD	ASD
1	$1.4D$	D
2	$1.2D + 1.6L + 0.5(L_r \text{ or } S \text{ or } R)$	$D+L$
3	$1.2D + 1.6(L_r \text{ or } S \text{ or } R) + (L \text{ or } 0.5W)$	$D + (L_r \text{ or } S \text{ or } R)$
4	$1.2D + W + L + 0.5(L_r \text{ or } S \text{ or } R)$	$D + 0.75L + 0.75(L_r \text{ or } S \text{ or } R)$
5	$0.9D + W$	$D+0.6W$
6	$1.2D + E_v + E_h + L + 0.2S$	$D + 0.75L + 0.75(0.6W) + 0.75(L_r \text{ or } S \text{ or } R)$
7	$0.9D - E_v + E_h$	$0.6D+0.6W$
8		$D+0.7E_v+0.7E_h$
9		$D+0.525E_v+0.525E_h + 0.75L + 0.75S$
10		$0.6D-0.7E_v+0.7E_h$

Where:
D= Dead Load
L= Live Load
Lr= Roof Live Load
S= Snow Load
W= Wind Load
R= Rain Load

Loads:

$Dead_{Membrane} := 1\ psf$

$Dead_{MEP} := 5\ psf$

$Dead_{RoofInsulation} := 0.365\ psf$

$Dead_{CompositeDeck} := 63.9\ psf$

$D := Dead_{Membrane} + (1\ in \cdot 145\ pcf) + Dead_{RoofInsulation} + Dead_{MEP} + Dead_{CompositeDeck} = 82.348\ psf$

Additional 1 inch of concrete for roof sloping, specified to installer

$L := 100\ psf$

$S := 35.7\ psf$

$W := 0.391\ psf$

$W_{uplift} := -13.671\ psf$

LRFD Load Combos:

$1.4\ D = 115.288\ psf$ **(1)**

$1.2\ D + 1.6\ L + 0.5\ S = 276.668\ psf$ **(2)**

$1.2\ D + 1.6\ S + L = 255.938\ psf$ **(3)** Neglect wind load in combo 3 because it is smaller than L

$1.2\ D + W + L + 0.5\ S = 217.059\ psf$ **(4)**

$0.9\ \left(\left(D + Dead_{CompositeDeck}\right)\right) + W_{uplift} = 117.953\ psf$ **(5)** Uplift is not an issue

$MaxLoad := 276.668\ psf$

Tributary Area Over the Largest Opening....

$Trib := \dfrac{16.6666\ ft}{2} + 4\ ft = 12.333\ ft$ At Overhang

$Dead_{Wall} := \left(125\ pcf \cdot 16\ in \cdot 8\ in\right) + \left(120\ pcf \cdot 4\ in \cdot 16\ in\right) = 164.444\ plf$

$w := \left(MaxLoad \cdot Trib\right) + Dead_{Wall} \cdot 1.2 = 3.61\ klf$ Add in factored dead load of wall

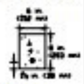
$$M_{Max} := \frac{w \cdot Length^2}{8} = \left(2.059 \cdot 10^5\right)\ in \cdot lbf$$

$$V := w \cdot \frac{Length}{2} = \left(1.113 \cdot 10^4\right)\ lbf$$

Table 1 - Shear and Moment Capacity for 4 x 8 in. (102 x 203 mm) Reinforced Concrete Lintels

Reinforcing bar size (No.)	No. of bars	f'_c, psi (MPa) 3000 (20.7) ϕV_n lb (kN)	3000 (20.7) ϕM_n in.-lb (kN·m)	3500 (24.1) ϕV_n lb (kN)	3500 (24.1) ϕM_n in.-lb (kN·m)	4000 (27.6) ϕV_n lb (kN)	4000 (27.6) ϕM_n in.-lb (kN·m)
3 (10M)	1	2,000 (8.9)	33,140 (3.75)	2,160 (9.6)	33,450 (3.78)	2,310 (10.3)	33,670 (3.81)
4 (13M)	1	1,980 (8.8)	56,440 (6.38)	2,140 (9.5)	57,440 (6.49)	2,290 (10.2)	58,190 (6.57)
5 (16M)	1	1,960 (8.7)	80,450 (9.09)	2,110 (9.4)	82,860 (9.36)	2,260 (10.1)	84,670 (9.57)

Table 2 - Shear and Moment Capacity for 6 x 8 in. (152 x 203 mm) Reinforced Concrete Lintels



Reinforcing bar size (No.)	No. of bars	f'_c, psi (MPa) 3000 (20.7) ϕV_n lb (kN)	3000 (20.7) ϕM_n in.-lb (kN·m)	3500 (24.1) ϕV_n lb (kN)	3500 (24.1) ϕM_n in.-lb (kN·m)	4000 (27.6) ϕV_n lb (kN)	4000 (27.6) ϕM_n in.-lb (kN·m)
4 (13M)	1	3,070 (13.7)	58,930 (6.66)	3,320 (14.8)	59,570 (6.73)	3,550 (15.8)	60,060 (6.79)
5 (16M)	1	3,040 (13.5)	86,440 (9.77)	3,280 (14.6)	87,990 (9.94)	3,510 (15.6)	89,160 (10.1)
3 (10M)	2	3,110 (13.8)	65,070 (7.35)	3,350 (14.9)	65,850 (7.44)	3,590 (16.0)	66,430 (7.51)
4 (13M)	2	3,070 (13.7)	108,820 (12.3)	3,320 (14.8)	111,410 (12.6)	3,550 (15.8)	113,340 (12.8)
5 (16M)	2	[2]	[2]	[2]	[2]	3,510 (15.6)	162,040 (18.3)

Table 3 - Shear and Moment Capacity for 8 x 8 in. (203 x 203 mm) Reinforced Concrete Lintels

Reinforcing bar size (No.)	No. of bars	f'_c, psi (MPa) 3000 (20.7) ϕV_n lb (kN)	3000 (20.7) ϕM_n in.-lb (kN·m)	3500 (24.1) ϕV_n lb (kN)	3500 (24.1) ϕM_n in.-lb (kN·m)	4000 (27.6) ϕV_n lb (kN)	4000 (27.6) ϕM_n in.-lb (kN·m)
4 (13M)	1	4,170 (18.6)	60,110 (6.79)	4,500 (20.0)	60,590 (6.85)	4,810 (21.4)	60,950 (6.89)
5 (16M)	1	4,120 (18.4)	89,290 (10.1)	4,450 (19.8)	90,430 (10.2)	4,760 (21.2)	91,290 (10.3)
6 (19M)	1	4,080 (18.2)	120,490 (13.6)	4,410 (19.6)	122,790 (13.9)	4,710 (21.0)	124,520 (14.1)
4 (13M)	2	4,170 (18.6)	113,560 (12.8)	4,500 (20.0)	115,470 (13.0)	4,810 (21.4)	116,900 (13.2)
5 (16M)	2	4,120 (18.4)	162,570 (18.4)	4,450 (19.8)	167,150 (18.9)	4,760 (21.2)	170,580 (19.3)
6 (19M)	2	[2]	[2]	[2]	[2]	4,710 (21.0)	224,840 (25.4)

Table 4 - Shear and Moment Capacity for 8 x 16 in. (203 x 406 mm) Reinforced Concrete Lintels[3]

Reinforcing bar size (No.)	No. of bars	f'_c, psi (MPa) 3000 (20.7) ϕV_n lb (kN)	3000 (20.7) ϕM_n in.-lb (kN·m)	3500 (24.1) ϕV_n lb (kN)	3500 (24.1) ϕM_n in.-lb (kN·m)	4000 (27.6) ϕV_n lb (kN)	4000 (27.6) ϕM_n in.-lb (kN·m)
6 (19M)	1	9,760 (43.4)	310,570 (35.1)	10,540 (46.9)	312,870 (35.4)	11,270 (50.1)	314,600 (35.5)
4 (13M)	2	9,850 (43.8)	286,360 (32.4)	10,640 (47.3)	288,270 (32.6)	11,370 (50.6)	289,700 (32.7)
5 (16M)	2	9,800 (43.6)	430,410 (48.6)	10,590 (47.1)	434,990 (49.1)	11,320 (50.4)	438,420 (49.5)
6 (19M)	2	9,760 (43.4)	588,870 (66.5)	10,540 (46.9)	598,090 (67.6)	11,270 (50.1)	605,000 (68.4)

1. Tables based on strength design method as described in ref. 2, assuming 1.5 in. (38 mm) concrete cover and Grade 60 reinforcement, f_y = 60,000 psi (413 MPa).
2. Reinforcement at listed effective depth exceeds the maximum reinforcing ratio of 0.75 ρ_b.
3. When determining minimum end bearing, the bearing stress of the masonry supporting the lintel should be checked to ensure it does not exceed $0.25f'_m$ (ref. 1).

Specify a 8x16 lintel with 1 #6 bar for reinforcement and 1.5 inches of concrete cover with 4000 psi compressive strength concrete

Phase One Restroom Insulation Calcs

Link to International Energy Conservation Code:
https://codes.iccsafe.org/content/IECC2021P2/chapter-4-ce-commercial-energy-efficiency

Considering this building a class C - commercial building

CHAPTER 4 [CE] COMMERCIAL ENERGY EFFICIENCY

C401.2.1 International Energy Conservation Code.

Commercial buildings shall comply with one of the following:

1. Prescriptive Compliance. The Prescriptive Compliance option requires compliance with Sections C402 through C406 and Section C408. Dwelling units and sleeping units in Group R-2 buildings without systems serving multiple units shall be deemed to be in compliance with this chapter, provided that they comply with Section R406.
2. Total Building Performance. The Total Building Performance option requires compliance with Section C407.

Exception: Additions, alterations, repairs and changes of occupancy to existing buildings complying with Chapter 5.

This building is going to comply with the prescriptive compliance... for insulation on building we will start off in section C4.02 Building Envelope Requirements

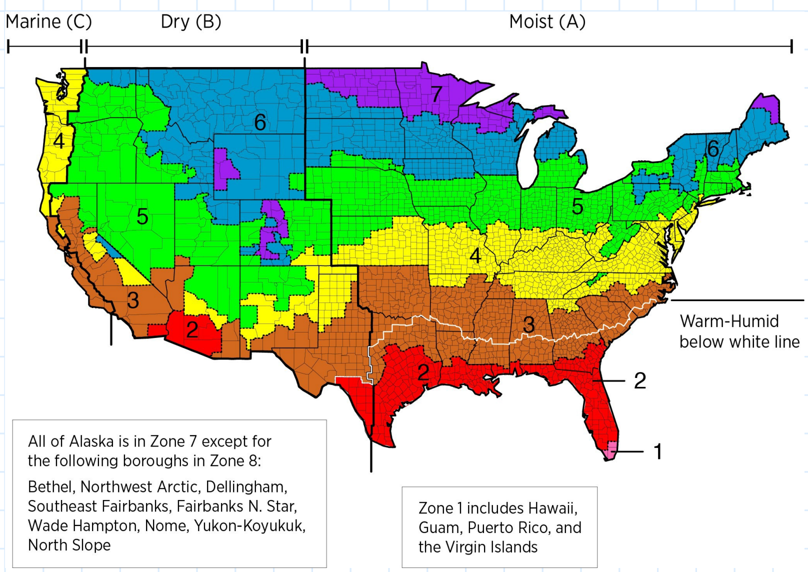
C402.1 General.

Building thermal envelope assemblies for buildings that are intended to comply with the code on a prescriptive basis in accordance with the compliance path described in Item 1 of Section C401.2.1 shall comply with the following:

1. The opaque portions of the *building thermal envelope* shall comply with the specific insulation requirements of Section C402.2 and the thermal requirements of either the *R*-value-based method of Section C402.1.3; the *U*-, *C*- and *F*-factor-based method of Section C402.1.4; or the component performance alternative of Section C402.1.5.
2. Roof solar reflectance and thermal emittance shall comply with Section C402.3.
3. Fenestration in building envelope assemblies shall comply with Section C402.4.
4. Air leakage of building envelope assemblies shall comply with Section C402.5.

Alternatively, where buildings have a vertical fenestration area or skylight area exceeding that allowed in Section C402.4, the building and *building thermal envelope* shall comply with Item 2 of Section C401.2.1 or Section C401.2.2.

Walk-in coolers, walk-in freezers, refrigerated warehouse coolers and refrigerated warehouse freezers shall comply with Section C403.11.



Maquoketa is in Climate Zone 5

For Roof Insulation

C402.2.3 Floors.

The thermal properties (component *R*-values or assembly *U*-, *C*- or *F*-factors) of floor assemblies over outdoor air or unconditioned space shall be as specified in Table C402.1.3 or C402.1.4 based on the construction materials used in the floor assembly. Floor framing *cavity insulation* or structural slab insulation shall be installed to maintain permanent contact with the underside of the subfloor decking or structural slabs.

"Mass floors" where used as a component of the thermal envelope of a building shall provide one of the following weights:

1. 35 pounds per square foot (171 kg/m^2) of floor surface area.
2. 25 pounds per square foot (122 kg/m^2) of floor surface area where the material weight is not more than 120 pounds per cubic foot (1923 kg/m^3).

	Floors										
Mass[e]	NR	NR	R-6.3ci	R-8.3ci	R-10ci	R-10ci	R-14.6ci	R-16.7ci	R-14.6ci	R-16.7ci	R-16.
Joist/framing	R-13	R-13	R-30	R-30	R-30	R-30	R-30	R-30	R-30	R-30	R-3

Considering it a mass floor because floor composite deck is over 35 psf... R value needed is R-14.6 ci (continuous insulation)

We are going with a closed cell foam insulation product

Benefits of Closed Cell Foam

Closed cell foam is the best choice for robust insulating where space is an issue, as it can achieve 2x the R-Value of open cell inside a standard wall. Its rigid nature also adds to the structural integrity of the building and E84 fire rated versions are available. The closed cell also acts as a vapor barrier, so water and moisture will be less likely to get inside the home, and the foam itself is unharmed by water damage.

Product: Handi-Foam™ Quick Cure E-84 (Class 1)

https://www.energyefficientsolutions.com/Fire_Rated.asp?item=FOAM605E84&gclid=EAIaIQobChMI3LfOk-j1_QIVyv_jBx19aQAPEAQYAyABEgK6K_D_BwE

R Value: R-6.2... required thickness of 2.5 inches

$750 for 605 board feet = 605 ft^2 at 1 in depth

Amount of spray foam needed

$$76\,ft - 16\,in \cdot 16\,ft + 8\,in - 16\,in \cdot 2.5 = \left(\left(2.862 \cdot 10^3\right)\right) ft^2$$

$$\frac{2.862 \cdot 10^3}{605} = 4.731$$

Units

Cost: $5 \cdot 750 = 3.75 \cdot 10^3$

$3750 for closed cell spray foam insulation for roof

Product: Handi-Foam™ Quick Cure E-84 (Class

Meets R value requirement

$$Density := 1.75 \frac{lbf}{ft^3}$$

$$DeadLoad := Density \cdot 2.5\ in = 0.365\ psf$$

C402.5.1.3 Materials. P

Materials with an air permeability not greater than 0.004 cfm/ft^2 (0.02 L/s × m^2) under a pressure differential of 0.3 inch water gauge (75 Pa) when tested in accordance with ASTM E2178 shall comply with this section. Materials in Items 1 through 16 shall be deemed to comply with this section, provided that joints are sealed and materials are installed as air barriers in accordance with the manufacturer's instructions.

1. Plywood with a thickness of not less than $^3/_8$ inch (10 mm).
2. Oriented strand board having a thickness of not less than $^3/_8$ inch (10 mm).
3. Extruded polystyrene insulation board having a thickness of not less than $^1/_2$ inch (12.7 mm).
4. Foil-back polyisocyanurate insulation board having a thickness of not less than $^1/_2$ inch (12.7 mm).
5. Closed-cell spray foam having a minimum density of 1.5 pcf (2.4 kg/m^3) and having a thickness of not less than $1^1/_2$ inches (38 mm).
6. Open-cell spray foam with a density between 0.4 and 1.5 pcf (0.6 and 2.4 kg/m^3) and having a thickness of not less than 4.5 inches (113 mm).
7. Exterior or interior gypsum board having a thickness of not less than $^1/_2$ inch (12.7 mm).
8. Cement board having a thickness of not less than $^1/_2$ inch (12.7 mm).
9. Built-up roofing membrane.
10. Modified bituminous roof membrane.
11. Single-ply roof membrane.
12. A Portland cement/sand parge, or gypsum plaster having a thickness of not less than $^5/_8$ inch (15.9 mm).
13. Cast-in-place and precast concrete.
14. Fully grouted concrete block masonry.
15. Sheet steel or aluminum.
16. Solid or hollow masonry constructed of clay or shale masonry units.

Meets air barrier compliance

For Floor Slab

	Slab-on-grade floors											
	Unheated slabs	NR	NR	NR	NR	NR	R-10 for 24" below	R-15 for 24" below	R-15 for 24" below	R-15 for 24" below	R-20 for 24" below	R-20 f 24" bel

No insulation requirement needed because slab does not sit 24" below grade (unheated slab)

For Wall Insulation

Can be considered a mass wall because CMU blocks have a high density which results in a weight not less than 35 psf

C402.2.2 Above-grade walls.

The minimum thermal resistance (*R*-value) of materials installed in the wall cavity between framing members and continuously on the walls shall be as specified in Table C402.1.3, based on framing type and construction materials used in the wall assembly. The *R*-value of integral insulation installed in concrete masonry units shall not be used in determining compliance with Table C402.1.3 except as otherwise noted in the table. In determining compliance with Table C402.1.4, the use of the *U*-factor of concrete masonry units with integral insulation shall be permitted.

"Mass walls" where used as a component in the thermal envelope of a building shall comply with one of the following:

1. Weigh not less than 35 pounds per square foot (171 kg/m^2) of wall surface area.
2. Weigh not less than 25 pounds per square foot (122 kg/m^2) of wall surface area where the material weight is not more than 120 pcf (1900 kg/m^3).
3. Have a heat capacity exceeding 7 $Btu/ft^2 \times °F$ (144 $kJ/m^2 \times K$).
4. Have a heat capacity exceeding 5 $Btu/ft^2 \times °F$ (103 $kJ/m^2 \times K$), where the material weight is not more than 120 pcf (1900 kg/m^3).

						Walls, above grade					
Mass[f]	R-5.7ci[c]	R-5.7ci[c]	R-5.7ci[c]	R-7.6ci	R-7.6ci	R-9.5ci	R-9.5ci	R-11.4ci	R-11.4ci	R-13.3ci	R-13.:
Metal building	R-13 + R-6.5ci	R-13 + R-6.5ci	R13 + R-6.5ci	R-13 + R-13ci	R-13 + R-6.5ci	R-13 + R-13ci	R-13 + R-13ci	R-13 + R-14ci	R-13 + R-14ci	R-13 + R-14ci	R-13 R-14
Metal framed	R-13 + R-5ci	R-13 + R-5ci	R-13 + R-5ci	R-13 + R-7.5ci	R-13 + R-7.5ci	R-13 + R-7.5ci	R-13 + R-7.5ci	R-13 + R-7.5ci	R-13 + R-10ci	R-13 + R-10ci	R-13 R-12.!
Wood framed and other	R-13 + R-3.8ci or R-20	R-13 + R-3.8ci or R-20	R-13 + R-3.8ci or R-20	R-13 + R-3.8ci or R-20	R-13 + R-3.8ci or R-20	R-13 + R-3.8ci or R-20	R-13 + R-3.8ci or R-20	R-13 + R-3.8ci or R-20	R-13 + R-7.5ci or R20 + R-3.8ci	R-13 + R-7.5ci or R-20 + R-3.8ci	R-13 R-7.5c R-20 R-3.8

Considering it a mass wall above grade, the R value is R-11.4ci (continuous insulation)

We are going with rigid foam board insulation commonly used with masonry walls

Product: R-Tech 1 1/2 in x 48 in. x 8 ft. R-5.78 EPS Rigid Foam Board Insulation

https://www.homedepot.com/p/R-Tech-1-1-2-in-x-48-in-x-8-ft-R-5-78-EPS-Rigid-Foam-Board-Insulation-320817/202532855

R-value: 5.78

$5.78 \cdot 2 = 11.56$

Need two layers of this insulation

How much will you need?

Please note: calculations are estimates and can only be made using whole numbers.

Calculate by Square Footage

Area 1

Width:	Height:
152 ft	10 ft

Area 2

Width:	Height:
34 ft	10 ft

+ Add Area

Calculate

64 units will cover 2048.00 sq. ft.

$1,341.44 Est. Total

Include an extra 10% to cover potential waste and breaks

Double this...

Cost: $2682.88 for 128 units of R-Tech 1 1/2 in x 48 in. x 8 ft. R-5.78 EPS Rigid Foam Board Insulation

Weight in psf

$$Density := 4 \frac{lbf}{8\ ft \cdot 1.5\ in \cdot 48\ in} = 1\ pcf$$

$$WallInsulation := 3\ in \cdot Density = 0.25\ psf$$

C402.5.1.3 Materials.

Materials with an air permeability not greater than 0.004 cfm/ft^2 (0.02 L/s × m^2) under a pressure differential of 0.3 inch water gauge (75 Pa) when tested in accordance with ASTM E2178 shall comply with this section. Materials in Items 1 through 16 shall be deemed to comply with this section, provided that joints are sealed and materials are installed as air barriers in accordance with the manufacturer's instructions.

1. Plywood with a thickness of not less than $^3/_8$ inch (10 mm).
2. Oriented strand board having a thickness of not less than $^3/_8$ inch (10 mm).
3. Extruded polystyrene insulation board having a thickness of not less than $^1/_2$ inch (12.7 mm).
4. Foil-back polyisocyanurate insulation board having a thickness of not less than $^1/_2$ inch (12.7 mm).
5. Closed-cell spray foam having a minimum density of 1.5 pcf (2.4 kg/m^3) and having a thickness of not less than 1$^1/_2$ inches (38 mm).
6. Open-cell spray foam with a density between 0.4 and 1.5 pcf (0.6 and 2.4 kg/m^3) and having a thickness of not less than 4.5 inches (113 mm).
7. Exterior or interior gypsum board having a thickness of not less than $^1/_2$ inch (12.7 mm).
8. ~~Cement board having a thickness of not less than $^1/_2$ inch (12.7 mm).~~
9. Built-up roofing membrane.
10. Modified bituminous roof membrane.
11. Single-ply roof membrane.
12. A Portland cement/sand parge, or gypsum plaster having a thickness of not less than $^5/_8$ inch (15.9 mm).
13. Cast-in-place and precast concrete.
14. Fully grouted concrete block masonry.
15. Sheet steel or aluminum.
16. Solid or hollow masonry constructed of clay or shale masonry units.

Meets air barrier compliance

ous Footing Design for the Restroom:

riteria:

ckness: $t_{cmu} := FIF$ "8" $t_{brick} := FIF$ "4" $t_{air} := FIF$ "1" $t_{insulation} := FIF$ "3"

$t_{wall} := t_{cmu} + t_{brick} + t_{air} + t_{insulation} = 16\ in$

USE SOLID GROUT

ight: $\gamma_{fill} := 120\ pcf$ $\gamma_{brick} := 120\ pcf$ $\gamma_{cmu} := 130\ pcf$ $\gamma_{conc} := 150\ pcf$

f safety: $FS_{overturning} := 1.5$ $FS_{sliding} := 1.5$ $FS_{bearing} := 3$

of wall: $H_{total} := FIF$ "12' 6" $H_{wall} := FIF$ "9' 6" Need an extra 3 ft of height due to frost line

ss of footing/ $t_f := FIF$ "1' 8""

f footing: $B := FIF$ "3'"

f Footing: $D_f := 3\ ft + t_f$ DUE TO FROST LINE AT 4 ft.

arth Pressure:

$$K_a := \tan\left(\frac{\pi}{4} - \frac{\phi'}{2}\right)^2 = 0.3333$$

$$\cdot \gamma_{fill} \cdot \left(\left(H_{total} - H_{wall} + t_f\right)\right)^2 \cdot K_a = 0.4356\ \frac{kip}{ft}$$

$20\ psf$

$$_{wind} \cdot H_{wall} = 0.19\ \frac{kip}{ft}$$

ing Moment:

$$_{rning} := P_a \cdot \frac{\left(\left(H_{total} - H_{wall} + t_f\right)\right)}{3} + P_w \cdot \left(\frac{H_{wall}}{2} + 3\ ft\right) = 2.15\ \frac{kip \cdot ft}{ft}$$

g Moment:

ent weights: moment arms:

$$W_m := \left(\left(t_{cmu}\right)\right) \cdot \left(\left(H_{wall}\right)\right) \cdot \gamma_{cmu} + \left(\left(t_{brick}\right) \cdot \left(\left(H_{wall}\right)\right) \cdot \gamma_{brick}\right) = 1.2033\ \frac{kip}{ft} \qquad r_m := \frac{B}{2} = 1.5\ ft$$

$$W_e := 2\left(\frac{B}{2} - \left(0.5 \cdot \left(t_{wall}\right)\right)\right) \cdot 3\ ft \cdot \gamma_{fill} = 0.6\ \frac{kip}{ft} \qquad r_e := \frac{B}{2} = 1.5\ ft$$

on wall: $W_{...} := \left(\left(t_{...}\right)\right) \cdot \left(\left(H_{total} - H_{wall}\right)\right) \cdot \gamma_{...} = 0.6\ kip$ $r_{...} := \frac{B}{...} = 1.5\ ft$

$W_m + W_e + W_f + W_{wall} + W_{roof} = 13.1533\ \dfrac{kip}{ft}$

$:= \dfrac{\Sigma W \cdot \tan\left(\phi'\right)}{P_a} = 17.4354$ $\qquad FS_{sliding} = 1.5$

Technically would have a passive pressure but is a conservative answer

$> FS_{sliding}$ OK

Pressure:

$\dfrac{\Sigma W}{B} = 4384.4444\ psf$

capacity:

's continuous foundation equation:

psf $\qquad B = 3\ ft$ $\qquad \gamma' := \gamma_{fill} = 120\ pcf$

7.2 $\qquad N_q := 22.5$ $\qquad N_\gamma := 20.1$

$D_f \cdot \gamma_{fill} = 560\ psf$

$\cdot N_c + \sigma'_{zD} \cdot N_q + 0.5 \cdot \gamma' \cdot B \cdot N_\gamma = 16218\ psf$

$\dfrac{q_{ult}}{q_{gross}} = 3.699$ $\qquad FS_{bearing} = 3$

$> FS_{bearing}$ OK

or Settlement Failure: Boussinesq's Simple Elastic Settlement method

FIF "14' 0" $\qquad B = 3\ ft$ $\qquad H := 5 \cdot B$ $\qquad \mu_s := 0.3$ $\qquad \alpha := 4$ (Footing Center) $\qquad E_s := 750\ \dfrac{tonf}{ft^2}$ $\qquad \delta_{all} := 0.5\ in$

$\dfrac{L}{2} = 7\ ft$ $\qquad B' := \dfrac{B}{2} = 1.5\ ft$

$= \dfrac{L'}{B'} = 4.6667$ $\qquad N := \dfrac{H}{B'}$

ence Factors:

$$\ln\left(\frac{r_4+B}{L}\right)+B\cdot\ln\left(\frac{r_4+L}{B}\right)-\frac{r_4^3-L^3-B^3}{3\cdot L\cdot B}=8.4093\ ft$$

$$\ln\left(\frac{r_3+B}{r_1}\right)+B\cdot\ln\left(\frac{r_3+L}{r_2}\right)-\frac{r_3^3-r_2^3-r_1^3+r^3}{3\cdot L\cdot B}=5.154\ ft$$

$$\cdot\ln\left(\frac{\left(B+r_2\right)\cdot r_1}{\left(B+r_3\right)\cdot r}\right)+\frac{r^2}{B}\cdot\ln\left(\frac{\left(L+r_1\right)\cdot r_2}{\left(L+r_3\right)\cdot r}\right)=2.042\ ft$$

$$\frac{\cdot\left(r_1+r_2-r_3-r\right)}{L\cdot B}=0.4251\ ft$$

$$\operatorname{atan}\left(\frac{L\cdot B}{r\cdot r_3}\right)=2.4029\ ft$$

$$\frac{\cdot Y_1+\beta_2\cdot Y_2+\beta_3\cdot Y_3+\beta_4\cdot Y_4+\beta_5\cdot Y_5}{\left(\beta_1+\beta_2\right)\cdot Y_1}=0.7076$$

Pressure:

$4384.4444\ psf$

$D_f)\cdot\gamma_{fill}=560\ psf$

$'_{gross}-\sigma'_{zo}=3824.444\ psf$

ion Settlement:

$$=\alpha\cdot I_s\cdot I_f\cdot\left(\frac{q_{net}\cdot\left(1-\mu_s^{2}\right)}{E_s}\right)\cdot B'$$

$=0.094\ in$

$0.93\cdot\delta_{flexible}=0.087\ in$ OK

$5\ in$

sign:

of and steel: $f'_c:=4000\ psi$ $f_y:=60\ ksi$

APPENDIX B—STEEL REINFORCEMENT INFORMATION

aid to users of the ACI Building Code, information on sizes, areas, and weights of various steel reinforcement is pressure:

$$\frac{W}{B} = 4384.4444\ psf$$

ft (Long dimension. Use 1 ft analysis strip)

(short dimension)

(width of wall)

$$W_m + W_{wall} + 10\ \frac{kip}{ft} = 11.8033\ klf$$

$${}_{ay} := P_u \cdot \left(\frac{B - c - 2 \cdot d}{B}\right) = -4.3443\ klf$$

$${}_{Way} := \frac{0.75 \cdot \left(2 \cdot \lambda \cdot \sqrt{f'_c \cdot psi} \cdot L_2 \cdot d\right)}{1\ ft} = 56.7787\ klf$$

= if $V_{uOneWay} < \phi V_{cOneWay}$
‖ "The footing has adequate shear strength"
else
‖ "The footing has inadequate shear strength"

= "The footing has adequate shear strength"

lexural Strength:

$$\frac{c}{2} = 1.1667\ ft$$

$$\frac{{}_u \cdot l^2}{2 \cdot B} \quad \frac{kip \cdot ft}{ft}$$ (required flexural resistance)

0.0018

$$\rho_{min} \cdot d \cdot \frac{12\ in}{1 \cdot ft} = 0.3591\ \frac{in^2}{ft}$$

Rebars: $A_{\#6} := 0.44\ in^2$

A ... 0.44 in^2 ... 1 #6 bar in each ...

35

$${}_{nControlled} := \frac{\beta_1 \cdot 0.85 \cdot f'_c \cdot b}{f_y} \cdot \frac{3 \cdot d_t}{8} = 10.8104\ in^2$$

The design is tension controlled $A_s \le A_{s\_TC}$

Concrete Compression block:

$$\frac{A_s \cdot f_y}{5 \cdot f'_c \cdot b} = 0.2157\ in$$

$$6776\ \frac{kip \cdot ft}{ft}$$

$$_s \cdot f_y \cdot \left(d - \frac{a}{2}\right) = 36.3377\ kip \cdot ft$$

$$\frac{0.9 \cdot M_n}{1\ ft} = 32.704\ \frac{kip \cdot ft}{ft}$$ (flexural strength)

= if $M_u < \phi M_n$
‖ "The footing has adequate flexural strength"
else
‖ "The footing has inadequate flexural strength "

= "The footing has adequate flexural strength"

Development Length of Flexural 180 degree Hooked Rebars:

ervative assumptions:

$0\quad \psi_r := 1.0 \quad \psi_o := 1.0 \quad \psi_c := 0.6 + \frac{f'_c}{15000\ psi} = 0.8667 \quad d_{bar} := D_{\#6}$

$$\max\left(6\ in, 8 \cdot d_{bar}, \left(\frac{\psi_e \cdot \psi_r \cdot \psi_o \cdot \psi_c \cdot f_y}{55 \cdot \lambda \cdot min\left(100\ psi, \sqrt{f'_c \cdot psi}\right)}\right) \cdot 1\ in \cdot \left(\frac{d_{bar}}{in}\right)\right) = 9.7096\ in$$

f bars from the critical section: $\frac{(B - t_{wall})}{2} - c_c = 7\ in$

= if $l_{dhook} < \frac{(B - t_{wall})}{2}$
‖ "There is adequate room to develop the hooked bars"
else
‖

make sure footing is thick enough to accommodate development length:

$$\cdot 6 = 2.25\ in$$ (radius of dowel bar bend)

$$_{xuralHooks} := \left(\left(\left(D_{\#6} \cdot 6\right)\right) + \left(\left(2 \cdot D_{\#6}\right)\right)\right) + D_{\#6} = 6.75\ in$$

$$_{ar} + r = 3\ in$$ (distance required for hook)

$$depth_{FlexuralHooks} = 6.75\ in$$

= if $h_{min} < t_f$
"The footing thickness is adequate to accomodate the hooked bar"
else
"The footing thickness is not adequate to accomodate the hooked bar"

= "The footing thickness is adequate to accomodate the hooked bar"

earing Capacity of Column at Base:

$$1\ ft \qquad h := t_f$$

$$in \cdot L_{wall} = 96\ in^2$$

$$\left(L_{wall}, \left(2 \cdot h + t_{wall} + 2 \cdot h\right)\right) = 1\ ft$$

$$= 144\ in^2$$

$$55 \cdot \left(0.85 \cdot f'_c \cdot A_1\right) = 212.16\ kip$$

$$55 \cdot min\left(\left(\left(0.85 \cdot f'_c \cdot A_1\right) \cdot \sqrt{\frac{A_2}{A_1}}\right), \left(2 \cdot 0.85 \cdot f'_c \cdot A_1\right)\right) = 259.8419\ kip$$

$$_{earing} := \frac{min\left(N_1, N_2\right)}{1\ ft} = 212.16\ klf$$

= 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"

= "The footing has adequate bearing strength at the base"

$_n := 12 \cdot d_{dowel} = 7.5\ in$

make sure footing is thick enough to accommodate development length:

$\frac{_{vel} \cdot 6}{2} = 1.875\ in$ (radius of dowel bar bend)

$_{owel} + r = 2.5\ in$ (distance required for hook)

$_{dc} + 2 \cdot H + c_c = 20\ in$

= if $h_{min} \le h$
‖ "The footing thickness is adequate"
else
‖ "The footing thickness is inadequate"

="The footing thickness is adequate"

ebars in Wall:

$.5\ in$ $d_{bar} := D_{\#4}$ column bars are #4

ment Length:

$$\max\left(8\ in, \frac{0.02 \cdot f_y \cdot d_{bar}}{\lambda \cdot min\left(100\ psi, \sqrt{f'_c \cdot psi}\right)}, \frac{0.0003}{psi} \cdot f_y \cdot d_{bar}\right) = 9.4868\ in$$

ngth for rebars in compression:

$$\max\left(12\ in, l_{dcCol}, \frac{0.0005}{psi} \cdot f_y \cdot d_{bar} \cdot \alpha_s\right) = 15\ in$$

(round up to a 24in (2ft) splice)

ous Footing Design for the Ramp:

riteria:

kness: $t_{cmu} := FIF$ "8" $t_{brick} := FIF$ "4" $t_{air} := FIF$ "1"

$t_{wall} := t_{cmu} + 2\ t_{brick} + 2\ t_{air} = 18\ in$

USE SOLID GROUT

ght: $\gamma_{fill} := 120\ pcf$ $\gamma_{brick} := 120\ pcf$ $\gamma_{cmu} := 130\ pcf$ $\gamma_{conc} := 150\ pcf$

f safety: $FS_{overturning} := 1.5$ $FS_{sliding} := 1.5$ $FS_{bearing} := 3$

f wall: $H_{total} := FIF$ "18'" $H_{wall} := FIF$ "15'"   Need an extra 3 ft of height due to frost line

s of footing/ $t_f := FIF$ "1' 8""

f footing: $B := FIF$ "3' 8"

f Footing: $D_f := 3\ ft + t_f$ DUE TO FROST LINE AT 4 ft.

arth Pressure:

$$K_a := \tan\left(\frac{\pi}{4} - \frac{\phi'}{2}\right)^2 = 0.3333$$

$$\cdot \gamma_{fill} \cdot \left(H_{total} - H_{wall} + t_f\right)^2 \cdot K_a = 0.4356\ \frac{kip}{ft}$$

20 psf

$$_{wind} \cdot H_{wall} = 0.3\ \frac{kip}{ft}$$

ing Moment:

$$_{ning} := P_a \cdot \frac{\left(\left(H_{total} - H_{wall} + t_f\right)\right)}{3} + P_w \cdot \left(\frac{H_{wall}}{2} + 3\ ft\right) = 3.8275\ \frac{kip \cdot ft}{ft}$$

g Moment:

ent weights:

$$W_m := \left(\left(t_{cmu}\right) \cdot \left(H_{wall}\right)\right) \cdot \gamma_{cmu} + \left(\left(t_{brick} \cdot \left(H_{wall}\right)\right) \cdot \gamma_{brick}\right) = 1.9\ \frac{kip}{ft}$$

$$W_e := 2\left(\frac{B}{2} - \left(0.5 \cdot \left(t_{wall}\right)\right)\right) \cdot 3\ ft \cdot \gamma_{fill} = 0.78\ \frac{kip}{ft}$$

moment arms:

$$r_m := \frac{B}{2} = 1.8333\ ft$$

$$r_e := \frac{B}{2} = 1.8333\ ft$$

n wall: $W_{...} := \left(\left(t ... \right)\right) \cdot \left(H ... H ...\right) \cdot \gamma ... = 0.675\ kip$ $r_{wall} := \frac{B}{2} = 1.8333\ ft$

$:= \frac{\Sigma W \cdot \tan \phi'}{P_a} = 5.6623$ $FS_{sliding} = 1.5$

Technically would have a passive pressure but is a conservative answer

$> FS_{sliding}$ OK

Pressure:

$\frac{\Sigma W}{B} = 1165\ psf$

capacity:

's continuous foundation equation:

psf $B = 3.6667\ ft$ $\gamma' := \gamma_{fill} = 120\ pcf$

7.2 $N_q := 22.5$ $N_\gamma := 20.1$

$D_f \cdot \gamma_{fill} = 560\ psf$

$' \cdot N_c + \sigma'_{zD} \cdot N_q + 0.5 \cdot \gamma' \cdot B \cdot N_\gamma = 17022\ psf$

$\frac{q_{ult}}{_{gross}} = 14.6112$ $FS_{bearing} = 3$

$> FS_{bearing}$ OK

or Settlement Failure: Boussinesq's Simple Elastic Settlement method

FIF “14' 0” $B = 3.6667\ ft$ $H := 5 \cdot B$ $\mu_s := 0.3$ $\alpha := 4$ (Footing Center) $E_s := 750\ \frac{tonf}{ft^2}$ $\delta_{all} := 0.5\ in$

$\frac{L}{2} = 7\ ft$ $B' := \frac{B}{2} = 1.8333\ ft$

$= \frac{L'}{B'} = 3.8182$ $N := \frac{H}{B'}$

ence Factors:

$\frac{1}{\pi} \cdot \left(M \cdot \ln\left(\frac{\left(1 + \sqrt{M^2 + 1}\right) \cdot \sqrt{M^2 + N^2}}{M \cdot \left(1 + \sqrt{M^2 + N^2 + 1}\right)} \right) + \ln\left(\frac{\left(M + \sqrt{M^2 + 1}\right) \cdot \sqrt{1 + N^2}}{M + \sqrt{M^2 + N^2 + 1}} \right) \right) = 0.7356$

$$\cdot \ln\left(\frac{\left(B+r_2\right) \cdot r_1}{\left(B+r_3\right) \cdot r}\right) + \frac{r}{B} \cdot \ln\left(\frac{\left(L+r_1\right) \cdot r_2}{\left(L+r_3\right) \cdot r}\right) = 2.4427\ ft$$

$$\frac{\cdot \left(r_1+r_2-r_3-r\right)}{L \cdot B} = 0.5083\ ft$$

$$\text{atan}\left(\frac{L \cdot B}{r \cdot r_3}\right) = 2.8853\ ft$$

$$\frac{\cdot Y_1+\beta_2 \cdot Y_2+\beta_3 \cdot Y_3+\beta_4 \cdot Y_4+\beta_5 \cdot Y_5}{\left(\beta_1+\beta_2\right) \cdot Y_1} = 0.723$$

Pressure:

$1165\ psf$

$D_f) \cdot \gamma_{fill} = 560\ psf$

$_{gross} - \sigma'_{zo} = 605\ psf$

ion Settlement:

$$= \alpha \cdot I_s \cdot I_f \cdot \left(\frac{q_{net} \cdot \left(1-\mu_s^2\right)}{E_s}\right) \cdot B'$$

$= 0.018\ in$

$0.93 \cdot \delta_{flexible} = 0.017\ in$ OK

$5\ in$

sign:

of and steel: $f'_c := 4000\ psi$ $f_y := 60\ ksi$

APPENDIX B—STEEL REINFORCEMENT INFORMATION

id to users of the ACI Building Code, information on sizes, areas, and weights of various steel reinforcement is .

TANDARD REINFORCING BARS

Bar size, no.	Nominal diameter, in.	Nominal area, in.^{2}	Nominal weight, lb/ft
3	0.375	0.11	0.376
4	0.500	0.20	0.668
5	0.625	0.31	1.043

pressure:

W
$= 1165\ psf$
B

ft (Long dimension. Use 1 ft analysis strip)

(short dimension)

(width of wall)

$W_m + W_{wall} = 2.575\ klf$

$\boxed{ay} := P_u \cdot \left(\frac{B - c - 2 \cdot d}{B}\right) = -0.4243\ klf$

$\boxed{Way} := \frac{0.75 \cdot \left(2 \cdot \lambda \cdot \sqrt{f'_c \cdot psi} \cdot L_2 \cdot d\right)}{1\ ft} = 69.3962\ klf$

= if $V_{uOneWay} < \phi V_{cOneWay}$
‖ "The footing has adequate shear strength"
else
‖ "The footing has inadequate shear strength "

= "The footing has adequate shear strength"

lexural Strength:

c
2
$= 1.4583\ ft$

u
$= 0.747\ \frac{kip \cdot ft}{ft}$ (required flexural resistance)
$2 \cdot B$

0.0018

$\rho_{min} \cdot d \cdot \frac{12\ in}{1 \cdot ft} = 0.3591\ \frac{in^2}{ft}$

Rebars: $\boxed{A_{\#6}} := 0.44\ in^2$

$f'_c \leq 4000\ psi$

0.85

se

$\max\left(0.65, 0.85 - 0.05 \cdot \frac{f'_c - 4000\ psi}{1000\ psi}\right)$

35

$_{nControlled} := \frac{\beta_1 \cdot 0.85 \cdot f'_c \cdot b}{f_y} \cdot \frac{3 \cdot d_t}{8} = 13.2127\ in^2$ The design is tension controlled $A_s \leq A_{s\_TC}$

Concrete Compression block:

$\frac{A_s \cdot f_y}{5 \cdot f'_c \cdot b} = 0.1765\ in$

$7468\ \frac{kip \cdot ft}{ft}$

$_s \cdot f_y \cdot \left(d - \frac{a}{2}\right) = 36.3809\ kip \cdot ft$

$\frac{0.9 \cdot M_n}{1\ ft} = 32.7428\ \frac{kip \cdot ft}{ft}$ (flexural strength)

$=$ if $M_u < \phi M_n$
"The footing has adequate flexural strength"
else
"The footing has inadequate flexural strength"

= "The footing has adequate flexural strength"

Development Length of Flexural <u>180 degree Hooked</u> Rebars:

ervative assumptions:

$\psi_r := 1.0$ $\psi_o := 1.0$ $\psi_c := 0.6 + \frac{f'_c}{15000\ psi} = 0.8667$ $d_{bar} := D_{\#6}$

$\max\left(6\ in, 8 \cdot d_{bar}, \left(\frac{\psi_e \cdot \psi_r \cdot \psi_o \cdot \psi_c \cdot f_y}{55 \cdot \lambda \cdot min\left(100\ psi, \sqrt{f'_c \cdot psi}\right)}\right) \cdot 1\ in \cdot \left(\frac{d_{bar}}{in}\right)^{1.5}\right) = 9.7096\ in$

$ce_{max} := min(3 \cdot t_f, 18\ in) = 18\ in$

$ce := \frac{B - 2\ c_c}{2} = 19\ in$ use 12 in spacing to be conservative

make sure footing is thick enough to accommodate development length:

$\frac{\cdot\ 6}{2} = 2.25\ in$ (radius of dowel bar bend)

$xuralHooks := ((D_{\#6} \cdot 6) + (2 \cdot D_{\#6})) + D_{\#6} = 6.75\ in$

$_{ar} + r = 3\ in$ (distance required for hook)

$depth_{FlexuralHooks} = 6.75\ in$

= if $h_{min} < t_f$
‖"The footing thickness is adequate to accomodate the hooked bar"
else
‖"The footing thickness is not adequate to accomodate the hooked bar "

= "The footing thickness is adequate to accomodate the hooked bar"

earing Capacity of Column at Base:

$1\ ft$ $h := t_f$

$in \cdot L_{wall} = 96\ in^2$

$(L_{wall}, (2 \cdot h + t_{wall} + 2 \cdot h)) = 1\ ft$

$= 144\ in^2$

$5 \cdot (0.85 \cdot f'_c \cdot A_1) = 212.16\ kip$

$5 \cdot min\left(\left((0.85 \cdot f'_c \cdot A_1) \cdot \sqrt{\frac{A_2}{A_1}}\right), (2 \cdot 0.85 \cdot f'_c \cdot A_1)\right) = 259.8419\ kip$

$earing := \frac{min(N_1, N_2)}{1\ ft} = 212.16\ klf$

= if $P_u < \phi P_n$

ment Length:

.625 in

$D_{\#5}$

$ax\left(8\ in,\ \lambda \cdot min\left(100\ psi,\ \dfrac{0.02 \cdot f_y \cdot d_{dowel}}{\sqrt{f'_c \cdot psi}}\right),\ \dfrac{0.0003}{psi} \cdot f_y \cdot d_{dowel}\right) = 11.8585\ in$

in (round up to get an appropriate constructible dimension)

$:= 12 \cdot d_{dowel} = 7.5\ in$

make sure footing is thick enough to accommodate development length:

$\dfrac{{}_{vel} \cdot 6}{2} = 1.875\ in$ (radius of dowel bar bend)

${}_{wel} + r = 2.5\ in$ (distance required for hook)

${}_{dc} + 2 \cdot H + c_c = 20\ in$

= if $h_{min} \le h$
"The footing thickness is adequate"
else
"The footing thickness is inadequate"

="The footing thickness is adequate"

ebars in Wall:

.5 in $d_{bar} := D_{\#4}$ column bars are #4

ment Length:

$max\left(8\ in,\ \lambda \cdot min\left(100\ psi,\ \dfrac{0.02 \cdot f_y \cdot d_{bar}}{\sqrt{f'_c \cdot psi}}\right),\ \dfrac{0.0003}{psi} \cdot f_y \cdot d_{bar}\right) = 9.4868\ in$

ngth for rebars in compression:

Isolated Footing Design for Bathroom columns from roof

esign Parameters:

F "4' 4" $H := 3\ ft$ $B_{col} := FIF$ "1' 4" $t_f := FIF$ "1' 8" $P_{des} := 12\ kip$

$ft + t_f$

$H + t_f = 4.6667\ ft$

or Bearing Failure:

Equation for Bearing Capacity:

$\cdot N_c \cdot s_c \cdot d_c \cdot i_c \cdot b_c \cdot g_c + \sigma'_z \cdot N_q \cdot s_q \cdot d_q \cdot i_q \cdot b_q \cdot g_q + 0.5 \cdot \gamma_{backfill} \cdot B' \cdot N_\gamma \cdot s_\gamma \cdot d_\gamma \cdot i_\gamma \cdot b_\gamma \cdot g_\gamma$

Capacity Factors:

le of bearing capacity factors for ϕ' = 30 degrees and using Vesic's Equation:

.4 $N_\gamma := 22.4$

Inclination Factors:

level ground, therefore, ground inclination factors are equal to 1:

$g_\gamma := 1$

clination Factors:

$i_\gamma := 1$

lination Factors:

is not inclined, therefore the base inclination factors are equal to 1:

$b_\gamma := 1$

For continuous footings, $B/L \to 0$, so s_c, s_q, and s_γ become equal to 1. This means factors may be ignored when analyzing continuous footings.

In line with continuous footing

actors:

$s_\gamma := 1$

actors:

$\cdot 2 \cdot \frac{D_f}{B} \cdot \tan\phi' \cdot (1 - \sin\phi')^2 = 1.3109$ $d_\gamma := 1$

Unit Weight:

or Settlement Failure:

$B = 4.3333\ ft$ $\quad H := 5 \cdot B \quad \mu_s := 0.3 \quad \alpha := 4$ (Footing Center) $\quad \delta_{all} := 0.5\ in$

$= 2.1667\ ft \quad B' := \dfrac{B}{2} = 2.1667\ ft \quad M := \dfrac{L'}{B'} = 1 \quad N := \dfrac{H}{B'}$

e Factors:

$$\cdot \left(M \cdot \ln\left(\frac{\left(1 + \sqrt{M^2 + 1}\right) \cdot \sqrt{M^2 + N^2}}{M \cdot \left(1 + \sqrt{M^2 + N^2 + 1}\right)} \right) + \ln\left(\frac{\left(M + \sqrt{M^2 + 1}\right) \cdot \sqrt{1 + N^2}}{M + \sqrt{M^2 + N^2 + 1}} \right) \right) = 0.4979$$

$$\frac{N}{\pi} \cdot \operatorname{atan}\left(\frac{M}{N \cdot \sqrt{M^2 + N^2 + 1}} \right) = 0.0158$$

orrection Factor:

$$\left(\frac{1 - 2 \cdot \mu_s}{1 - \mu_s} \right) \cdot I_2 = 0.5069$$

th Correction Factor, If:

$4 \cdot \mu_s \quad \beta_2 := 5 - 12 \cdot \mu_s + 8 \cdot \mu_s^2 \quad \beta_3 := -4 \cdot \mu_s \cdot \left(\left(1 - 2 \cdot \mu_s\right)\right) \quad \beta_4 := -1 + 4 \cdot \mu_s - 8 \cdot \mu_s^2 \quad \beta_5 := -4 \cdot \left(\left(1 - 2 \cdot \mu_s\right)\right)^2$

$D_f = 9.3333\ ft$

$L^2 + r^2 = 10.2902\ ft$

$B^2 + r^2 = 10.2902\ ft$

$L^2 + B^2 + r^2 = 11.1654\ ft$

$L^2 + B^2 = 6.1283\ ft$

$$\ln\left(\frac{r_4 + B}{L} \right) + B \cdot \ln\left(\frac{r_4 + L}{B} \right) - \frac{r_4^3 - L^3 - B^3}{3 \cdot L \cdot B} = 6.442\ ft$$

$$\ln\left(\frac{r_3 + B}{r_1} \right) + B \cdot \ln\left(\frac{r_3 + L}{r_2} \right) - \frac{r_3^3 - r_2^3 - r_1^3 + r^3}{3 \cdot L \cdot B} = 3.0925\ ft$$

$$\cdot \ln\left(\frac{\left(\left(B + r_2\right)\right) \cdot r_1}{\left(\left(B + r_3\right)\right) \cdot r} \right) + \frac{r^2}{B} \cdot \ln\left(\frac{\left(\left(L + r_1\right)\right) \cdot r_2}{\left(\left(L + r_3\right)\right) \cdot r} \right) = 1.5872\ ft$$

of Rebar in Isolated Footing:

One-Way Shear Strength:

F "4' 4"　　$H := FIF$ "3'"　　$B_{col1} := FIF$ "2' 8"　　$B_{col2} := FIF$ "2'"　　$t_f := FIF$ "1' 10"　　$h := t_f$

2 kip　　$\gamma_{conc} := 150\ pcf$　　$\gamma_{backfill} := 120\ pcf$　　$f'_c := 4000\ psi$　　$f_y := 60\ ksi$

e depth of footing:

a 3in clear cover, #6 rebars and bars in both directions:

n　　$D_{\#6} := 0.750\ in$

$= c_c + \dfrac{D_{\#6}}{2} = 3.375\ in$　　$cover_2 := c_c + D_{\#6} + \dfrac{D_{\#6}}{2} = 4.125\ in$

$_g := \dfrac{cover_1 + cover_2}{2} = 3.75\ in$

$cover_{Avg} = 18.25\ in$

Pressure:

$\dfrac{P_{des}}{\cdot B} + (\gamma_{conc} \cdot t_f) + 2 \cdot (\gamma_{backfill} \cdot H) + \dfrac{2\ \gamma_{conc} \cdot FIF \text{ "1' 10"} \cdot 1\ ft \cdot 3\ ft}{B \cdot B} = 1721.9231\ psf$

$L_2 := B$　　$c_1 := B_{col1}$　　$c_2 := B_{col2}$　　$\lambda := 1$

$_{ay} := q_u \cdot L_2 \cdot \left(\dfrac{L_1 - c_1}{2} - d\right) = -5.1299\ kip$

$_{Vay} := 0.75 \cdot \left(2 \cdot \lambda \cdot \sqrt{f'_c \cdot psi} \cdot L_2 \cdot d\right) = 90.03\ kip$

$=$ if $V_{uOneWay} < \phi V_{cOneWay}$
　"The footing has adequate shear strength"
else
　"The footing has inadequate shear strength"

= "The footing has adequate shear strength"

Punching Shear Strength:

$:= q_u \cdot (L_1 \cdot L_2 - (c_1 + d) \cdot (c_2 + d)) = 6.9467\ kip$

$\rho_{min} \cdot B \cdot h = 2.0592\ in^2$

6 Rebars: $A_{\#6} := 0.440\ in^2$

$A_{\#6} = 2.2\ in^2$ 5 #6 bars is adequate

cing:

$ace_{max} := min\left(3 \cdot h, 18\ in\right) = 18\ in$

$ce := \dfrac{B - 2 \cdot c_c}{4} = 11.5\ in$

e flexural strength of a singly reinforced rectangular section:

$= h$ $y_{s1} := cover_1$ $A_s = 2.2\ in^2$ $b := B$

$pth - y_{s1} = 18.625\ in$

$f'_c \leq 4000\ psi$

0.85

se

$max\left(0.65, 0.85 - 0.05 \cdot \dfrac{f'_c - 4000\ psi}{1000\ psi}\right)$

5

$_{nControlled} := \dfrac{\beta_1 \cdot 0.85 \cdot f'_c \cdot b}{f_y} \cdot \dfrac{3 \cdot d_t}{8} = 17.4935\ in^2$

ign is tension controlled

f Concrete Compression block:

$\dfrac{A_s \cdot f_y}{5 \cdot f'_c \cdot b} = 0.7466\ in$ $M_u = 2.5909\ kip \cdot ft$

$_s \cdot f_y \cdot \left(d - \dfrac{a}{2}\right) = 196.6437\ kip \cdot ft$

$0.9 \cdot M_n = 176.9793\ kip \cdot ft$ (flexural strength)

$\left.\left.{}_{oll}\right)\right) = 10\ in$

$= \text{if } l_{dhook} \le \frac{\left(B - B_{coll}\right)}{2}$

"There is adequate room to develop the hooked bars"

else

"There is inadequate room to develop the hooked bars"

$heck =$ "There is adequate room to develop the hooked bars"

o make sure footing is thick enough to accommodate development length:

$\cdot 6 \quad = 2.25\ in$ (radius of dowel bar bend)

$_{uralHooks} := \left(\left(\left(d_{bar} \cdot 6\right)\right) + \left(2 \cdot d_{bar}\right)\right) = 6\ in$

$_n := \max\left(4 \cdot d_{bar}, 2.5\ in\right) = 3\ in$

$_r + r = 3\ in$ (distance required for hook)

$depth_{FlexuralHooks} = 6\ in$

Bearing Capacity of Column at Base:

$_{oll} \cdot B_{col2} = 768\ in^2$ $L := B = 4.3333\ ft$ $l := min\left(L, \left(2 \cdot h + B_{coll} + 2 \cdot h\right)\right) = 4.3333\ ft$

$= 2704\ in^2$

$55 \cdot \left(0.85 \cdot f'_c \cdot A_1\right) = 1697.28\ kip$ $N_2 := 0.65 \cdot min\left(\left(\left(0.85 \cdot f'_c \cdot A_1\right) \cdot \sqrt{\frac{A_2}{A_1}}\right), \left(2 \cdot 0.85 \cdot f'_c \cdot A_1\right)\right) = 3184.7565\ kip$

$_{earing} := min\left(N_1, N_2\right) = 1697.28\ kip$

$= \text{if } P_{des} < \phi P_{BaseBearing}$

"The footing has adequate bearing strength at the base"

else

"The footing has inadequate bearing strength at the base"

$=$ "The footing has adequate bearing strength at the base"

90 Degree Hooked Dowel Bars in Column:

$_n := 12 \cdot d_{dowel} = 9\ in$

$_{owel} + r = 3\ in$ (distance required for hook)

$_{dc} + 2 \cdot H + c_c + D_{\#6} = 21.75\ in$

$=$ if $h_{min} \le h$
"There footing thickness is adequate"
else
"There footing thickness is inadequate"

$=$"There footing thickness is adequate"

Rebars in Column:

$D_{\#4}$ column bars are #4

ment Length:

$\max\left(8\ in, \lambda \cdot min\left(\dfrac{0.02 \cdot f_y \cdot d_{bar}}{100\ psi, \sqrt{f'_c} \cdot psi}\right), \dfrac{0.0003}{psi} \cdot f_y \cdot d_{bar}\right) = 9.4868\ in$

ength for rebars in compression:

$\max\left(12\ in, l_{dcCol}, \dfrac{0.0005}{psi} \cdot f_y \cdot d_{bar} \cdot \alpha_s\right) = 15\ in$ (round up to a 24in (2ft) splice)

MWFRS DESIGN WIND PRESSURES FOR WALLS AND ROOF - PHASE 1

Determining Wind Load Parameters:

Exposure Category:	B	(ASCE 7-22: Section 26.7.3)
Risk Category:	**II**	(ASCE 7-22: Table 1.5-1)
Importance Factor:	$I := 1$	(ASCE 7-22: Table 1.5-2)
Basic Wind Speed:	$V := 108\ mph$	(ASCE 7-22: Figure 26.5-1B)
Wind Directionality Factor:	$K_d := 0.85$	(ASCE 7-22: Figure 26.6-1)
Topographic Factor:	$K_{zt} := 1$	(ASCE 7-22: Section 26.8.1)
Ground Elevation Factor:	$K_e := 1$	(ASCE 7-22: Table 26.9-1)
Gust Effect Factor:	$G := 0.85$	(ASCE 7-22: Section 26.11.1)
Enclosure Classification:	Building Enclosed	(ASCE 7-22: Section 26.2)
Internal Pressure Coefficient:	$GC_{pi} := 0.18$	(ASCE 7-22: Table 26.13-1)

Velocity Pressure Exposure Coefficient: $K_z := 0.57$ (ASCE 7-22: Table 26.10-1)

Velocity Pressure:

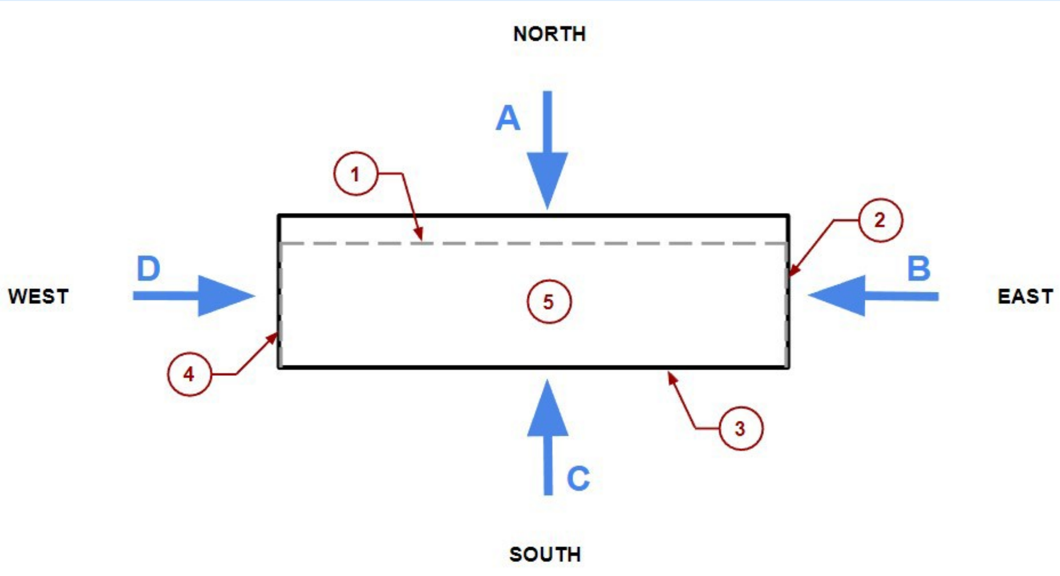
$$q_z := 0.00256 \frac{psf}{mph^2} \cdot K_z \cdot K_{zt} \cdot K_d \cdot V^2 \cdot I = 14.467\ psf \qquad q_i := q_z$$

(ASCE 7-22: Equation 26.10-1)

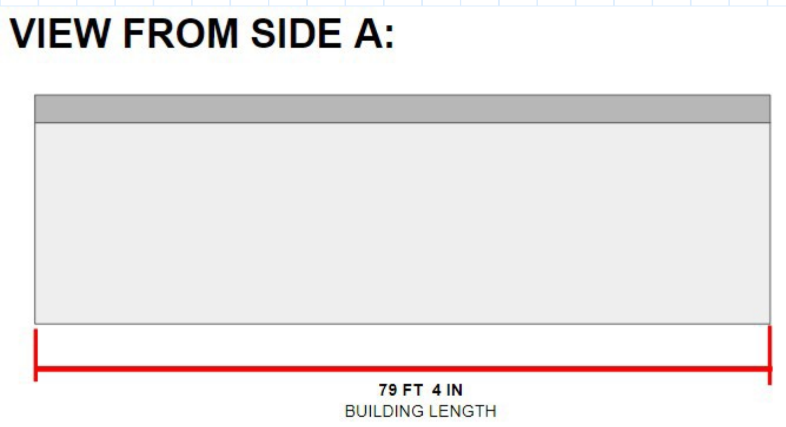
Mean Roof Height: $h := \dfrac{10\ ft + 9.458\ ft}{2} = 9.729\ ft$ (Height Modeled on Revit)

DIAGRAMS FOR WIND PRESSURE CALCULATIONS:

<u>**TOP VIEW:** OF PHASE 1 RESTROOM STRUCTURE</u>



<u>**PROFILE VIEWS:** OF PHASE 1 RESTROOM STRUCTURE</u>



Building Dimensions:

$L_A := 79\ ft + 4\ in$

$B_A := 16\ ft + 8\ in$

Overhang Dimensions:

$l_{EW.OH} := 0\ ft$

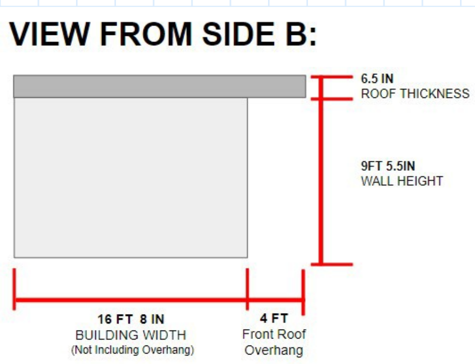
$l_{NS.OH} := 4\ ft$

Equivalent Dimensions:

$L_C := L_A \qquad B_C := B_A$

$L_B := B_A \qquad B_B := L_A$

$L_D := L_B \qquad B_D := B_B$



NORTH WIND CALCULATIONS (WIND DIRECTION A)

Wall Pressures Coefficients:

$L := L_A = 79.333\ ft$ $B := B_A = 16.667\ ft$

Variables Needed for Table: $\frac{L}{B} = 4.76$

External Pressure Coefficients: $C_{pWW} := 0.8$ $C_{pLW} := -0.2$ $C_{pS} := -0.7$ (ASCE 7-22: Figure 27.3-1)

Wall Pressures:

$p := p_{iext}$ +/- p_{iint}

Surface 1: (windward)

$p_{N1ext} := q_z \cdot G \cdot C_{pWW} = 9.84\ psf$

$p_{N1int} := q_i \cdot (\backslash(GC_{pi}\backslash)) = 2.6\ psf$

$p_{N1plus} := p_{N1ext} + p_{N1int} = 12.442\ psf$

$p_{N1minus} := p_{N1ext} - p_{N1int} = 7.234\ psf$

Surface 2: (side)

$p_{N2ext} := q_z \cdot G \cdot C_{pS} = -8.61\ psf$

$p_{N2int} := q_i \cdot (\backslash(GC_{pi}\backslash)) = 2.6\ psf$

$p_{N2plus} := p_{N2ext} + p_{N2int} = -6.004\ psf$

$p_{N2minus} := p_{N2ext} - p_{N2int} = -11.212\ psf$

Surface 3: (leeward)

$p_{N3ext} := q_z \cdot G \cdot C_{pLW} = -2.46\ psf$

$p_{N3int} := q_i \cdot (\backslash(GC_{pi}\backslash)) = 2.6\ psf$

$p_{N3plus} := p_{N3ext} + p_{N3int} = 0.145\ psf$

$p_{N3minus} := p_{N3ext} - p_{N3int} = -5.063\ psf$

Surface 4: (side)

$p_{N4ext} := q_z \cdot G \cdot C_{pS} = -8.61\ psf$

$p_{N4int} := q_i \cdot (\backslash(GC_{pi}\backslash)) = 2.6\ psf$

$p_{N4plus} := p_{N4ext} + p_{N4int} = -6.004\ psf$

$p_{N4minus} := p_{N4ext} - p_{N4int} = -11.212\ psf$

Roof Pressures Coefficients:

$L := L_A + l_{EW.OH} = 79.333\ ft$ $B := B_A + l_{NS.OH} = 20.667\ ft$

Variables Needed for Table: $\frac{h}{L} = 0.1226$ $\frac{h}{2} = 4.9\ ft$ $h = 9.7\ ft$ $2\ h = 19.5\ ft$ $\theta := 0$

External Pressure Coefficient: $C_{p1} := -0.9$ $C_{p2} := -0.18$ (ASCE 7-22: Figure 27.3-1)

Roof Pressures:

$p := p_{iext}$ +/- p_{iint}

Surface 5:

$p_{N5ext1} := q_z \cdot G \cdot C_{p1} = -11.07\ psf$ $p_{N5ext2} := q_z \cdot G \cdot C_{p2} = -2.21\ psf$

$p_{N5int1} := q_i \cdot (\backslash(GC_{pi})\backslash) = 2.6\ psf$ $p_{N5int2} := q_i \cdot (\backslash(GC_{pi})\backslash) = 2.6\ psf$

$p_{N5plus1} := p_{N5ext1} + p_{N5int1} = -8.463\ psf$ $p_{N5plus2} := p_{N5ext2} + p_{N5int2} = 0.391\ psf$

$p_{N5minus1} := p_{N5ext1} - p_{N5int1} = -13.671\ psf$ $p_{N5minus2} := p_{N5ext2} - p_{N5int2} = -4.818\ psf$

Overhang Pressures:

$p_{Noh} := q_z \cdot G \cdot C_{p1} = -11.067\ psf$

IMPORTANT NOTE: p_{oh} Must be added to the positive external pressure on windward faces with overhang (surface 1))

EAST WIND CALCULATIONS (WIND DIRECTION B)

Wall Pressures Coefficients:

$L := L_B = 16.667\ ft$ $B := B_B = 79.333\ ft$

Variables Needed for Table: $\frac{L}{B} = 0.21$

External Pressure Coefficients: $C_{pWW} := 0.8$ $C_{pLW} := -0.5$ $C_{pS} := -0.7$ (ASCE 7-22: Figure 27.3-1)

Wall Pressures:

$p := p_{iext}$ +/- p_{iint}

Surface 1: (side)

$p_{E1ext} := q_z \cdot G \cdot C_{pS} = -8.61\ psf$

$p_{E1int} := q_i \cdot (\backslash(GC_{pi}\backslash)) = 2.6\ psf$

$p_{E1plus} := p_{E1ext} + p_{E1int} = -6.004\ psf$

$p_{E1minus} := p_{E1ext} - p_{E1int} = -11.212\ psf$

Surface 2: (windward)

$p_{E2ext} := q_z \cdot G \cdot C_{pWW} = 9.84\ psf$

$p_{E2int} := q_i \cdot (\backslash(GC_{pi}\backslash)) = 2.6\ psf$

$p_{E2plus} := p_{E2ext} + p_{E2int} = 12.442\ psf$

$p_{E2minus} := p_{E2ext} - p_{E2int} = 7.234\ psf$

Surface 3: (side)

$p_{E3ext} := q_z \cdot G \cdot C_{pS} = -8.61\ psf$

$p_{E3int} := q_i \cdot (\backslash(GC_{pi}\backslash)) = 2.6\ psf$

$p_{E3plus} := p_{E3ext} + p_{E3int} = -6.004\ psf$

$p_{E3minus} := p_{E3ext} - p_{E3int} = -11.212\ psf$

Surface 4: (leeward)

$p_{E4ext} := q_z \cdot G \cdot C_{pLW} = -6.15\ psf$

$p_{E4int} := q_i \cdot (\backslash(GC_{pi}\backslash)) = 2.6\ psf$

$p_{E4plus} := p_{E4ext} + p_{E4int} = -3.544\ psf$

$p_{E4minus} := p_{E4ext} - p_{E4int} = -8.753\ psf$

Roof Pressures Coefficients:

$L := L_B + l_{NS.OH} = 20.667\ ft$ $B := B_B + l_{EW.OH} = 79.333\ ft$

Variables Needed for Table: $\frac{h}{L} = 0.4708$ $\frac{h}{2} = 4.9\ ft$ $h = 9.7\ ft$ $2\ h = 19.5\ ft$ $\theta := 0$

External Pressure Coefficient: $C_{p1} := -0.9$ $C_{p2} := -0.18$ (ASCE 7-22: Figure 27.3-1)

Roof Pressures:

$p := p_{iext}$ +/- p_{iint}

Surface 5:

$p_{E5ext1} := q_z \cdot G \cdot C_{p1} = -11.07\ psf$

$p_{E5int1} := q_i \cdot (\backslash(GC_{pi}\backslash)) = 2.6\ psf$

$p_{E5plus1} := p_{E5ext1} + p_{E5int1} = -8.463\ psf$

$p_{E5minus1} := p_{E5ext1} - p_{E5int1} = -13.671\ psf$

$p_{E5ext2} := q_z \cdot G \cdot C_{p2} = -2.21\ psf$

$p_{E5int2} := q_i \cdot (\backslash(GC_{pi}\backslash)) = 2.6\ psf$

$p_{E5plus2} := p_{E5ext2} + p_{E5int2} = 0.391\ psf$

$p_{E5minus2} := p_{E5ext2} - p_{E5int2} = -4.818\ psf$

SOUTH WIND CALCULATIONS (WIND DIRECTION C)

Wall Pressures Coefficients:

$L := L_C = 79.333\ ft$ $B := B_C = 16.667\ ft$

Variables Needed for Table: $\frac{L}{B} = 4.76$

External Pressure Coefficients: $C_{pWW} := 0.8$ $C_{pLW} := -0.2$ $C_{pS} := -0.7$ (ASCE 7-22: Figure 27.3-1)

Wall Pressures:

$p := p_{iext}$ +/- p_{iint}

Surface 1: (leeward)

$p_{S1ext} := q_z \cdot G \cdot C_{pLW} = -2.46\ psf$

$p_{S1int} := q_i \cdot \left(\left(GC_{pi}\right)\right) = 2.6\ psf$

$p_{S1plus} := p_{S1ext} + p_{S1int} = 0.145\ psf$

$p_{S1minus} := p_{S1ext} - p_{S1int} = -5.063\ psf$

Surface 2: (side)

$p_{S2ext} := q_z \cdot G \cdot C_{pS} = -8.61\ psf$

$p_{S2int} := q_i \cdot \left(\left(GC_{pi}\right)\right) = 2.6\ psf$

$p_{S2plus} := p_{S2ext} + p_{S2int} = -6.004\ psf$

$p_{S2minus} := p_{S2ext} - p_{S2int} = -11.212\ psf$

Surface 3: (windward)

$p_{S3ext} := q_z \cdot G \cdot C_{pWW} = 9.84\ psf$

$p_{S3int} := q_i \cdot \left(\left(GC_{pi}\right)\right) = 2.6\ psf$

$p_{S3plus} := p_{S3ext} + p_{S3int} = 12.442\ psf$

$p_{S3minus} := p_{S3ext} - p_{S3int} = 7.234\ psf$

Surface 4: (side)

$p_{S4ext} := q_z \cdot G \cdot C_{pS} = -8.61\ psf$

$p_{S4int} := q_i \cdot \left(\left(GC_{pi}\right)\right) = 2.6\ psf$

$p_{S4plus} := p_{S4ext} + p_{S4int} = -6.004\ psf$

$p_{S4minus} := p_{S4ext} - p_{S4int} = -11.212\ psf$

Roof Pressures Coefficients:

$L := L_C + l_{EW.OH} = 79.333\ ft$ $B := B_C + l_{NS.OH} = 20.667\ ft$

Variables Needed for Table: $\frac{h}{L} = 0.1226$ $\frac{h}{2} = 4.9\ ft$ $h = 9.7\ ft$ $2\ h = 19.5\ ft$ $\theta := 0$

External Pressure Coefficient: $C_{p1} := -0.9$ $C_{p2} := -0.18$ (ASCE 7-22: Figure 27.3-1)

Roof Pressures:

$p := p_{iext}$ +/- p_{iint}

Surface 5:

$p_{S5ext1} := q_z \cdot G \cdot C_{p1} = -11.07\ psf$

$p_{S5int1} := q_i \cdot \left(\left(GC_{pi}\right)\right) = 2.6\ psf$

$p_{S5plus1} := p_{S5ext1} + p_{S5int1} = -8.463\ psf$

$p_{S5minus1} := p_{S5ext1} - p_{S5int1} = -13.671\ psf$

$p_{S5ext2} := q_z \cdot G \cdot C_{p2} = -2.21\ psf$

$p_{S5int2} := q_i \cdot \left(\left(GC_{pi}\right)\right) = 2.6\ psf$

$p_{S5plus2} := p_{S5ext2} + p_{S5int2} = 0.391\ psf$

$p_{S5minus2} := p_{S5ext2} - p_{S5int2} = -4.818\ psf$

WEST WIND CALCULATIONS (WIND DIRECTION D)

Wall Pressures Coefficients:

$L := L_D = 16.667\ ft$ $B := B_D = 79.333\ ft$

Variables Needed for Table: $\frac{L}{B} = 0.21$

External Pressure Coefficients: $C_{pWW} := 0.8$ $C_{pLW} := -0.5$ $C_{pS} := -0.7$ (ASCE 7-22: Figure 27.3-1)

Wall Pressures:

$p := p_{iext}$ +/- p_{iint}

Surface 1: (side)

$p_{W1ext} := q_z \cdot G \cdot C_{pS} = -8.61\ psf$

$p_{W1int} := q_i \cdot (\backslash(GC_{pi}\backslash)) = 2.6\ psf$

$p_{W1plus} := p_{W1ext} + p_{W1int} = -6.004\ psf$

$p_{W1minus} := p_{W1ext} - p_{W1int} = -11.212\ psf$

Surface 2: (leeward)

$p_{W2ext} := q_z \cdot G \cdot C_{pLW} = -6.15\ psf$

$p_{W2int} := q_i \cdot (\backslash(GC_{pi}\backslash)) = 2.6\ psf$

$p_{W2plus} := p_{W2ext} + p_{W2int} = -3.544\ psf$

$p_{W2minus} := p_{W2ext} - p_{W2int} = -8.753\ psf$

Surface 3: (side)

$p_{W3ext} := q_z \cdot G \cdot C_{pS} = -8.61\ psf$

$p_{W3int} := q_i \cdot (\backslash(GC_{pi}\backslash)) = 2.6\ psf$

$p_{W3plus} := p_{W3ext} + p_{W3int} = -6.004\ psf$

$p_{W3minus} := p_{W3ext} - p_{W3int} = -11.212\ psf$

Surface 4: (windward)

$p_{W4ext} := q_z \cdot G \cdot C_{pWW} = 9.84\ psf$

$p_{W4int} := q_i \cdot (\backslash(GC_{pi}\backslash)) = 2.6\ psf$

$p_{W4plus} := p_{W4ext} + p_{W4int} = 12.442\ psf$

$p_{W4minus} := p_{W4ext} - p_{W4int} = 7.234\ psf$

Roof Pressures Coefficients:

$L := L_D + l_{NS.OH} = 20.667\ ft$ $B := B_D + l_{EW.OH} = 79.333\ ft$

Variables Needed for Table: $\frac{h}{L} = 0.4708$ $\frac{h}{2} = 4.9\ ft$ $h = 9.7\ ft$ $2\ h = 19.5\ ft$ $\theta := 0$

External Pressure Coefficient: $C_{p1} := -0.9$ $C_{p2} := -0.18$ (ASCE 7-22: Figure 27.3-1)

Roof Pressures:

$p := p_{iext}$ +/- p_{iint}

Surface 5:

$p_{W5ext1} := q_z \cdot G \cdot C_{p1} = -11.07\ psf$

$p_{W5int1} := q_i \cdot (\backslash(GC_{pi}\backslash)) = 2.6\ psf$

$p_{W5plus1} := p_{W5ext1} + p_{W5int1} = -8.463\ psf$

$p_{W5minus1} := p_{W5ext1} - p_{W5int1} = -13.671\ psf$

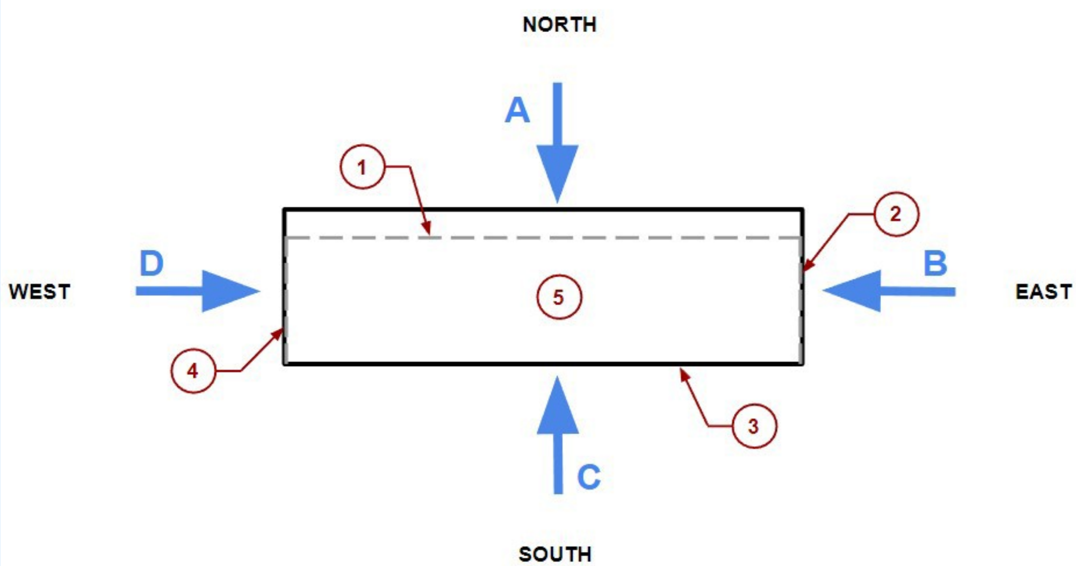
$p_{W5ext2} := q_z \cdot G \cdot C_{p2} = -2.21\ psf$

$p_{W5int2} := q_i \cdot (\backslash(GC_{pi}\backslash)) = 2.6\ psf$

$p_{W5plus2} := p_{W5ext2} + p_{W5int2} = 0.391\ psf$

$p_{W5minus2} := p_{W5ext2} - p_{W5int2} = -4.818\ psf$

SUMMARY CALCULATIONS



Maximum Positive Pressure on each wall and roof surface:

Surface 1: $\max\left(\left(p_{N1plus}, p_{E1plus}, p_{S1plus}, p_{W1plus}, p_{N1minus}, p_{E1minus}, p_{S1minus}, p_{W1minus}\right)\right) = 12.442\ psf$

Surface 2: $\max\left(\left(p_{N2plus}, p_{E2plus}, p_{S2plus}, p_{W2plus}, p_{N2minus}, p_{E2minus}, p_{S2minus}, p_{W2minus}\right)\right) = 12.442\ psf$

Surface 3: $\max\left(\left(p_{N3plus}, p_{E3plus}, p_{S3plus}, p_{W3plus}, p_{N3minus}, p_{E3minus}, p_{S3minus}, p_{W3minus}\right)\right) = 12.442\ psf$

Surface 4: $\max\left(\left(p_{N4plus}, p_{E4plus}, p_{S4plus}, p_{W4plus}, p_{N4minus}, p_{E4minus}, p_{S4minus}, p_{W4minus}\right)\right) = 12.442\ psf$

Surface 5: $\max\left(\left(p_{N5plus1}, p_{N5plus2}, p_{E5plus1}, p_{E5plus2}, p_{S5plus1}, p_{S5plus2}, p_{W5plus1}, p_{W5plus2}\right)\right) = 0.391\ psf$

Maximum Negative Pressure on each wall and roof surface:

Surface 1: $min\left(\left(p_{N1plus}, p_{E1plus}, p_{S1plus}, p_{W1plus}, p_{N1minus}, p_{E1minus}, p_{S1minus}, p_{W1minus}\right)\right) = -11.212\ psf$

Surface 2: $min\left(\left(p_{N2plus}, p_{E2plus}, p_{S2plus}, p_{W2plus}, p_{N2minus}, p_{E2minus}, p_{S2minus}, p_{W2minus}\right)\right) = -11.212\ psf$

Surface 3: $min\left(\left(p_{N3plus}, p_{E3plus}, p_{S3plus}, p_{W3plus}, p_{N3minus}, p_{E3minus}, p_{S3minus}, p_{W3minus}\right)\right) = -11.212\ psf$

Surface 4: $min\left(\left(p_{N4plus}, p_{E4plus}, p_{S4plus}, p_{W4plus}, p_{N4minus}, p_{E4minus}, p_{S4minus}, p_{W4minus}\right)\right) = -11.212\ psf$

Surface 5: $min\left(\left(p_{N5minus1}, p_{N5minus2}, p_{E5minus1}, p_{E5minus2}, p_{S5minus1}, p_{S5minus2}, p_{W5minus1}, p_{W5minus2}\right)\right) = -13.671\ psf$

Overhang Pressures: $p_{oh} := q_z \cdot G \cdot C_{p1} = -11.067\ psf$

Appendix E: Restroom Phase Two Design Calculations

MWFRS DESIGN WIND PRESSURES FOR WALLS AND ROOF - PHASE 2

Determining Wind Load Parameters:

Exposure Category:	B		(ASCE 7-22: Section 26.7.3)
Risk Category:	II		(ASCE 7-22: Table 1.5-1)
Importance Factor:	$I := 1.0$		(ASCE 7-22: Table 1.5-2)
Basic Wind Speed:	$V := 108\ mph$		(ASCE 7-22: Figure 26.5-1B)
Wind Directionality Factor:	$K_d := 0.85$	MWFRS system	(ASCE 7-22: Figure 26.6-1)
Topographic Factor:	$K_{zt} := 1$	No ridges or hills near this site	(ASCE 7-22: Section 26.8.1)
Ground Elevation Factor:	$K_e := 1$	Conservative approximation	(ASCE 7-22: Table 26.9-1)
Gust Effect Factor:	$G := 0.85$	Rigid Building	(ASCE 7-22: Section 26.11.1)
Enclosure Classification:	Partially Open		(ASCE 7-22: Section 26.2)
Internal Pressure Coefficient:	$GC_{pi} := 0.18$		(ASCE 7-22: Table 26.13-1)

Velocity Pressure Exposure Coefficient: $K_z := 0.57$ (ASCE 7-22: Table 26.10-1)

Using directional procedure so not 0.7

Velocity Pressure:

$$q_z := 0.00256\ \frac{psf}{mph^2} \cdot K_z \cdot K_{zt} \cdot K_d \cdot V^2 \cdot I = 14.467\ psf \qquad q_i := q_z$$

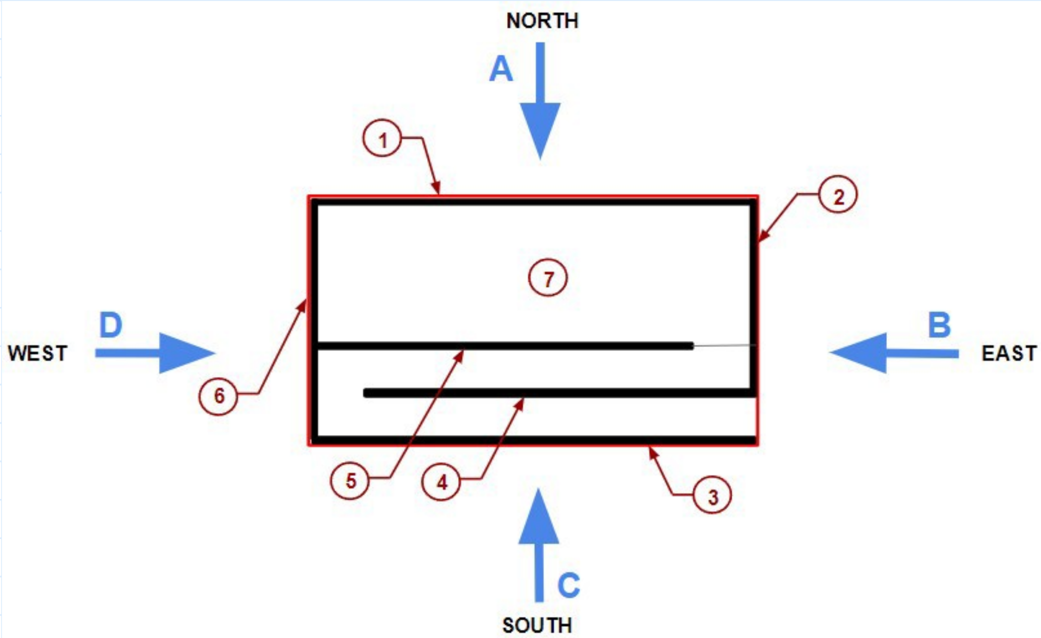
(ASCE 7-22: Equation 26.10-1)

Mean Roof Height: $h := \dfrac{10\ ft + 9.458\ ft}{2} = 9.729\ ft$ (Height Modeled on Revit)

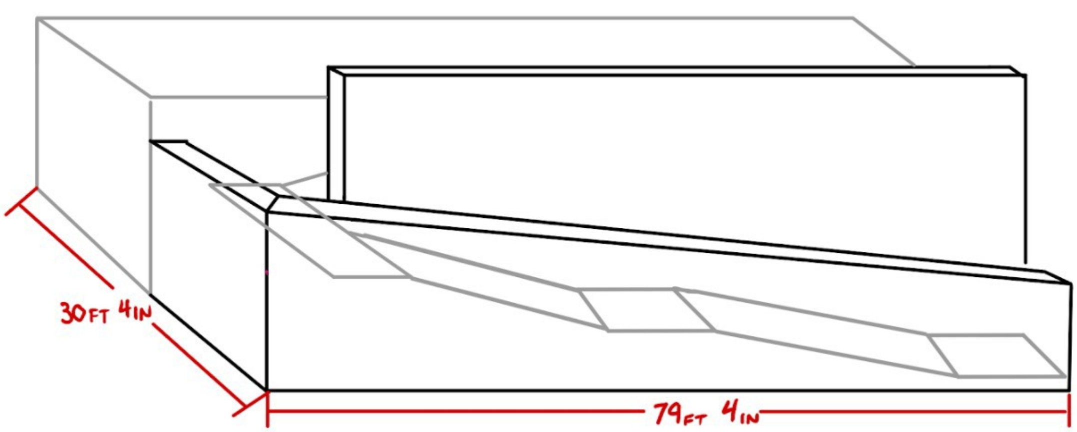
Based on 5" roof deck .3.5" concrete, 1.5" metal decking

DIAGRAMS FOR WIND PRESSURE CALCULATIONS:

<u>**TOP VIEW:** OF PHASE 2 RESTROOM STRUCTURE</u>



<u>**3D VIEW:** OF PHASE 2 RESTROOM STRUCTURE FROM SOUTHWEST</u>



Building Dimensions:

$L_A := 79\ ft + 4\ in$

$B_A := 30\ ft + 4\ in$

Equivalent Dimensions: $L_C := L_A$ $B_C := B_A$

$L_B := B_A$ $B_B := L_A$

$L_D := L_B$ $B_D := B_B$

NORTH WIND CALCULATIONS (WIND DIRECTION A)

Wall Pressures Coefficients:

$L := L_A = 79.333\ ft$ $B := B_A = 30.333\ ft$

Variables Needed for Table: $\frac{L}{B} = 2.615$

External Pressure Coefficients: $C_{pWW} := 0.8$ $C_{pLW} := -0.267$ $C_{pS} := -0.7$ (ASCE 7-22: Figure 27.3-1)

Wall Pressures: $p := \boxed{p_{iext}}$ +/- p_{iint}

Surface 1: (windward)

$p_{N1ext} := q_z \cdot G \cdot C_{pWW} = 9.84\ psf$

$p_{N1int} := q_i \cdot ((GC_{pi})) = 2.6\ psf$

$p_{N1plus} := p_{N1ext} + p_{N1int} = 12.442\ psf$

$p_{N1minus} := p_{N1ext} - p_{N1int} = 7.234\ psf$

Surface 2: (side)

$p_{N2ext} := q_z \cdot G \cdot C_{pS} = -8.61\ psf$

$p_{N2int} := q_i \cdot ((GC_{pi})) = 2.6\ psf$

$p_{N2plus} := p_{N2ext} + p_{N2int} = -6.004\ psf$

$p_{N2minus} := p_{N2ext} - p_{N2int} = -11.212\ psf$

Surface 3: (leeward)

$p_{N3ext} := q_z \cdot G \cdot C_{pLW} = -3.28\ psf$

$p_{N3int} := q_i \cdot ((GC_{pi})) = 2.6\ psf$

$p_{N3plus} := p_{N3ext} + p_{N3int} = -0.679\ psf$

$p_{N3minus} := p_{N3ext} - p_{N3int} = -5.887\ psf$

Surface 4: (leeward)

$p_{N4ext} := q_z \cdot G \cdot C_{pLW} = -3.28\ psf$

$p_{N4int} := q_i \cdot ((GC_{pi})) = 2.6\ psf$

$p_{N4plus} := p_{N4ext} + p_{N4int} = -0.679\ psf$

$p_{N4minus} := p_{N4ext} - p_{N4int} = -5.887\ psf$

Surface 5: (leeward)

$p_{N5ext} := q_z \cdot G \cdot C_{pLW} = -3.28\ psf$

$p_{N5int} := q_i \cdot ((GC_{pi})) = 2.6\ psf$

$p_{N5plus} := p_{N5ext} + p_{N5int} = -0.679\ psf$

$p_{N5minus} := p_{N5ext} - p_{N5int} = -5.887\ psf$

Surface 6: (side)

$p_{N6ext} := q_z \cdot G \cdot C_{pS} = -8.61\ psf$

$p_{N6int} := q_i \cdot ((GC_{pi})) = 2.6\ psf$

$p_{N6plus} := p_{N6ext} + p_{N6int} = -6.004\ psf$

$p_{N6minus} := p_{N6ext} - p_{N6int} = -11.212\ psf$

EAST WIND CALCULATIONS (WIND DIRECTION B)

Wall Pressures Coefficients:

$L := L_B = 30.333\ ft$ $B := B_B = 79.333\ ft$

Variables Needed for Table: $\frac{L}{B} = 0.382$

External Pressure Coefficients: $C_{pWW} := 0.8$ $C_{pLW} := -0.5$ $C_{pS} := -0.7$ (ASCE 7-22: Figure 27.3-1)

Wall Pressures: $p := p_{iext}$ +/- p_{iint}

Surface 1: (side)

$p_{E1ext} := q_z \cdot G \cdot C_{pS} = -8.61\ psf$

$p_{E1int} := q_i \cdot ((GC_{pi})) = 2.6\ psf$

$p_{E1plus} := p_{E1ext} + p_{E1int} = -6.004\ psf$

$p_{E1minus} := p_{E1ext} - p_{E1int} = -11.212\ psf$

Surface 2: (windward)

$p_{E2ext} := q_z \cdot G \cdot C_{pWW} = 9.84\ psf$

$p_{E2int} := q_i \cdot ((GC_{pi})) = 2.6\ psf$

$p_{E2plus} := p_{E2ext} + p_{E2int} = 12.442\ psf$

$p_{E2minus} := p_{E2ext} - p_{E2int} = 7.234\ psf$

Surface 3: (side)

$p_{E3ext} := q_z \cdot G \cdot C_{pS} = -8.61\ psf$

$p_{E3int} := q_i \cdot ((GC_{pi})) = 2.6\ psf$

$p_{E3plus} := p_{E3ext} + p_{E3int} = -6.004\ psf$

$p_{E3minus} := p_{E3ext} - p_{E3int} = -11.212\ psf$

Surface 4: (side)

$p_{E4ext} := q_z \cdot G \cdot C_{pS} = -8.61\ psf$

$p_{E4int} := q_i \cdot ((GC_{pi})) = 2.6\ psf$

$p_{E4plus} := p_{E4ext} + p_{E4int} = -6.004\ psf$

$p_{E4minus} := p_{E4ext} - p_{E4int} = -11.212\ psf$

Surface 5: (side)

$p_{E5ext} := q_z \cdot G \cdot C_{pS} = -8.61\ psf$

$p_{E5int} := q_i \cdot ((GC_{pi})) = 2.6\ psf$

$p_{E5plus} := p_{E5ext} + p_{E5int} = -6.004\ psf$

$p_{E5minus} := p_{E5ext} - p_{E5int} = -11.212\ psf$

Surface 6: (leeward)

$p_{E6ext} := q_z \cdot G \cdot C_{pLW} = -6.15\ psf$

$p_{E6int} := q_i \cdot ((GC_{pi})) = 2.6\ psf$

$p_{E6plus} := p_{E6ext} + p_{E6int} = -3.544\ psf$

$p_{E6minus} := p_{E6ext} - p_{E6int} = -8.753\ psf$

SOUTH WIND CALCULATIONS (WIND DIRECTION C)

Wall Pressures Coefficients:

$L := L_C = 79.333\ ft$ $B := B_C = 30.333\ ft$

Variables Needed for Table: $\frac{L}{B} = 2.615$

External Pressure Coefficients: $C_{pWW} := 0.8$ $C_{pLW} := -0.267$ $C_{pS} := -0.7$ (ASCE 7-22: Figure 27.3-1)

Wall Pressures: $p := p_{iext}$ +/- p_{iint}

Surface 1: (leeward)

$p_{S1ext} := q_z \cdot G \cdot C_{pLW} = -3.28\ psf$

$p_{S1int} := q_i \cdot \left(\left(GC_{pi}\right)\right) = 2.6\ psf$

$p_{S1plus} := p_{S1ext} + p_{S1int} = -0.679\ psf$

$p_{S1minus} := p_{S1ext} - p_{S1int} = -5.887\ psf$

Surface 2: (side)

$p_{S2ext} := q_z \cdot G \cdot C_{pS} = -8.61\ psf$

$p_{S2int} := q_i \cdot \left(\left(GC_{pi}\right)\right) = 2.6\ psf$

$p_{S2plus} := p_{S2ext} + p_{S2int} = -6.004\ psf$

$p_{S2minus} := p_{S2ext} - p_{S2int} = -11.212\ psf$

Surface 3: (windward)

$p_{S3ext} := q_z \cdot G \cdot C_{pWW} = 9.84\ psf$

$p_{S3int} := q_i \cdot \left(\left(GC_{pi}\right)\right) = 2.6\ psf$

$p_{S3plus} := p_{S3ext} + p_{S3int} = 12.442\ psf$

$p_{S3minus} := p_{S3ext} - p_{S3int} = 7.234\ psf$

Surface 4: (windward)

$p_{S4ext} := q_z \cdot G \cdot C_{pWW} = 9.84\ psf$

$p_{S4int} := q_i \cdot \left(\left(GC_{pi}\right)\right) = 2.6\ psf$

$p_{S4plus} := p_{S4ext} + p_{S4int} = 12.442\ psf$

$p_{S4minus} := p_{S4ext} - p_{S4int} = 7.234\ psf$

Surface 5: (windward)

$p_{S5ext} := q_z \cdot G \cdot C_{pWW} = 9.84\ psf$

$p_{S5int} := q_i \cdot \left(\left(GC_{pi}\right)\right) = 2.6\ psf$

$p_{S5plus} := p_{S5ext} + p_{S5int} = 12.442\ psf$

$p_{S5minus} := p_{S5ext} - p_{S5int} = 7.234\ psf$

Surface 6: (side)

$p_{S6ext} := q_z \cdot G \cdot C_{pS} = -8.61\ psf$

$p_{S6int} := q_i \cdot \left(\left(GC_{pi}\right)\right) = 2.6\ psf$

$p_{S6plus} := p_{S6ext} + p_{S6int} = -6.004\ psf$

$p_{S6minus} := p_{S6ext} - p_{S6int} = -11.212\ psf$

WEST WIND CALCULATIONS (WIND DIRECTION D)

Wall Pressures Coefficients:

$L := L_D = 30.333\ ft$ $B := B_D = 79.333\ ft$

Variables Needed for Table: $\frac{L}{B} = 0.382$

External Pressure Coefficients: $C_{pWW} := 0.8$ $C_{pLW} := -0.5$ $C_{pS} := -0.7$ (ASCE 7-22: Figure 27.3-1)

Wall Pressures: $p := p_{iext}$ +/- p_{iint}

Surface 1: (side)

$p_{W1ext} := q_z \cdot G \cdot C_{pS} = -8.61\ psf$

$p_{W1int} := q_i \cdot \left(\left(GC_{pi}\right)\right) = 2.6\ psf$

$p_{W1plus} := p_{W1ext} + p_{W1int} = -6.004\ psf$

$p_{W1minus} := p_{W1ext} - p_{W1int} = -11.212\ psf$

Surface 2: (leeward)

$p_{W2ext} := q_z \cdot G \cdot C_{pLW} = -6.15\ psf$

$p_{W2int} := q_i \cdot \left(\left(GC_{pi}\right)\right) = 2.6\ psf$

$p_{W2plus} := p_{W2ext} + p_{W2int} = -3.544\ psf$

$p_{W2minus} := p_{W2ext} - p_{W2int} = -8.753\ psf$

Surface 3: (side)

$p_{W3ext} := q_z \cdot G \cdot C_{pS} = -8.61\ psf$

$p_{W3int} := q_i \cdot \left(\left(GC_{pi}\right)\right) = 2.6\ psf$

$p_{W3plus} := p_{W3ext} + p_{W3int} = -6.004\ psf$

$p_{W3minus} := p_{W3ext} - p_{W3int} = -11.212\ psf$

Surface 4: (side)

$p_{W4ext} := q_z \cdot G \cdot C_{pS} = -8.61\ psf$

$p_{W4int} := q_i \cdot \left(\left(GC_{pi}\right)\right) = 2.6\ psf$

$p_{W4plus} := p_{W4ext} + p_{W4int} = -6.004\ psf$

$p_{W4minus} := p_{W4ext} - p_{W4int} = -11.212\ psf$

Surface 5: (side)

$p_{W5ext} := q_z \cdot G \cdot C_{pS} = -8.61\ psf$

$p_{W5int} := q_i \cdot \left(\left(GC_{pi}\right)\right) = 2.6\ psf$

$p_{W5plus} := p_{W5ext} + p_{W5int} = -6.004\ psf$

$p_{W5minus} := p_{W5ext} - p_{W5int} = -11.212\ psf$

Surface 6: (windward)

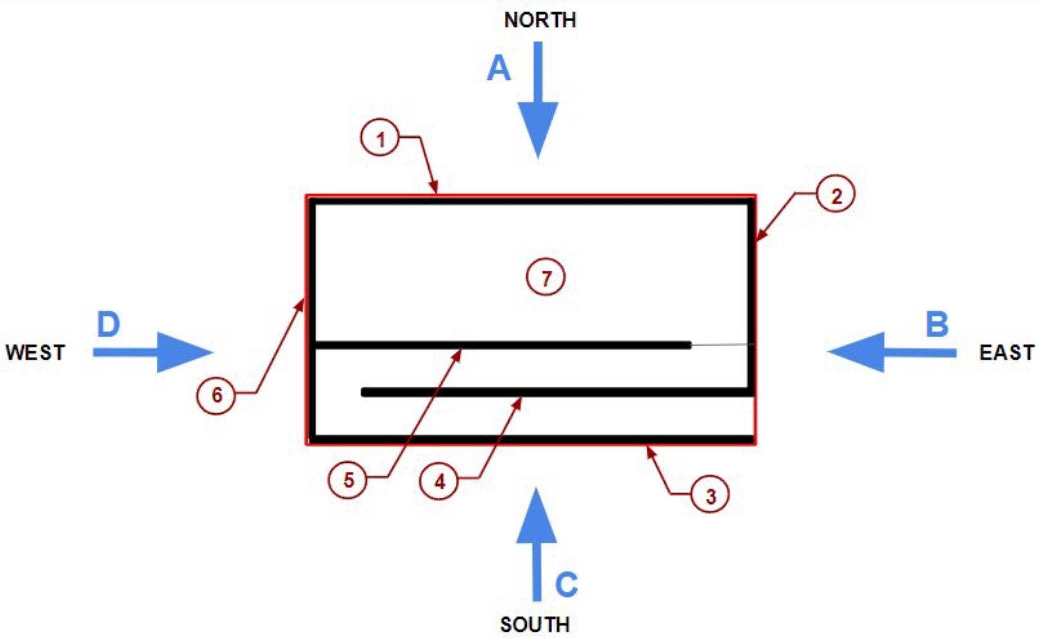
$p_{W6ext} := q_z \cdot G \cdot C_{pWW} = 9.84\ psf$

$p_{W6int} := q_i \cdot \left(\left(GC_{pi}\right)\right) = 2.6\ psf$

$p_{W6plus} := p_{W6ext} + p_{W6int} = 12.442\ psf$

$p_{W6minus} := p_{W6ext} - p_{W6int} = 7.234\ psf$

SUMMARY CALCULATIONS



Maximum Positive Pressure on each wall and roof surface:

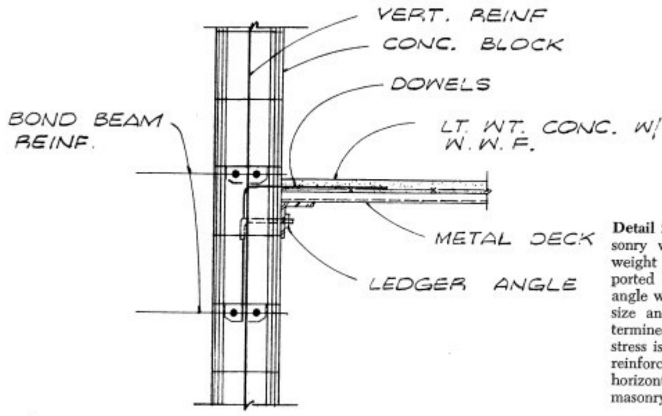
Surface 1: $\max\left(\left(p_{N1plus}, p_{E1plus}, p_{S1plus}, p_{W1plus}, p_{N1minus}, p_{E1minus}, p_{S1minus}, p_{W1minus}\right)\right) = 12.442\ psf$
Surface 2: $\max\left(\left(p_{N2plus}, p_{E2plus}, p_{S2plus}, p_{W2plus}, p_{N2minus}, p_{E2minus}, p_{S2minus}, p_{W2minus}\right)\right) = 12.442\ psf$
Surface 3: $\max\left(\left(p_{N3plus}, p_{E3plus}, p_{S3plus}, p_{W3plus}, p_{N3minus}, p_{E3minus}, p_{S3minus}, p_{W3minus}\right)\right) = 12.442\ psf$
Surface 4: $\max\left(\left(p_{N4plus}, p_{E4plus}, p_{S4plus}, p_{W4plus}, p_{N4minus}, p_{E4minus}, p_{S4minus}, p_{W4minus}\right)\right) = 12.442\ psf$
Surface 5: $\max\left(\left(p_{N5plus}, p_{E5plus}, p_{S5plus}, p_{W5plus}, p_{N5minus}, p_{E5minus}, p_{S5minus}, p_{W5minus}\right)\right) = 12.442\ psf$
Surface 6: $\max\left(\left(p_{N6plus}, p_{E6plus}, p_{S6plus}, p_{W6plus}, p_{N6minus}, p_{E6minus}, p_{S6minus}, p_{W6minus}\right)\right) = 12.442\ psf$

Maximum Negative Pressure on each wall and roof surface:

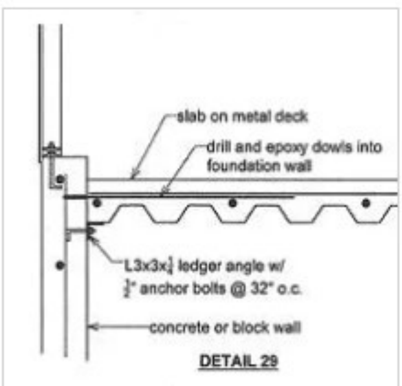
Surface 1: $min\left(\left(p_{N1plus}, p_{E1plus}, p_{S1plus}, p_{W1plus}, p_{N1minus}, p_{E1minus}, p_{S1minus}, p_{W1minus}\right)\right) = -11.212\ psf$
Surface 2: $min\left(\left(p_{N2plus}, p_{E2plus}, p_{S2plus}, p_{W2plus}, p_{N2minus}, p_{E2minus}, p_{S2minus}, p_{W2minus}\right)\right) = -11.212\ psf$
Surface 3: $min\left(\left(p_{N3plus}, p_{E3plus}, p_{S3plus}, p_{W3plus}, p_{N3minus}, p_{E3minus}, p_{S3minus}, p_{W3minus}\right)\right) = -11.212\ psf$
Surface 4: $min\left(\left(p_{N4plus}, p_{E4plus}, p_{S4plus}, p_{W4plus}, p_{N4minus}, p_{E4minus}, p_{S4minus}, p_{W4minus}\right)\right) = -11.212\ psf$
Surface 5: $min\left(\left(p_{N5plus}, p_{E5plus}, p_{S5plus}, p_{W5plus}, p_{N5minus}, p_{E5minus}, p_{S5minus}, p_{W5minus}\right)\right) = -11.212\ psf$
Surface 6: $min\left(\left(p_{N6plus}, p_{E6plus}, p_{S6plus}, p_{W6plus}, p_{N6minus}, p_{E6minus}, p_{S6minus}, p_{W6minus}\right)\right) = -11.212\ psf$

Use Roof Pressures from phase 1 Wind Calcs for more accurate surface 7 pressures

Ramp Dead Loads



Detail 21. A section of a concrete block masonry wall and a metal decking with lightweight concrete. The metal decking is supported by and welded to a continuous ledger angle which is bolted to the masonry wall. The size and spacing of the angle bolts are determined by calculation. The floor diaphragm stress is transferred to the masonry wall by the reinforcing dowels and the ledger angle. The horizontal bars act as a bond beam in the masonry wall.



From top down:

Liquid Applied Waterproofing Membrane

Clear Waterproofing Membrane

Link to Website: https://waterstop.com.au/product/clear-waterproofing-membrane-10l/

Link to Datasheet: https://waterstop.com.au/wp-content/uploads/2021/06/Data-Sheet-Clear-Waterproofing-Membrane-RV2-2.pdf

Waterproofing membranes:	
Bituminous, gravel-covered	5.5
Bituminous, smooth surface	1.5
Liquid applied	1
Single-ply, sheet	0.7
Wood sheathing (per in. thickness)	3
Wood shakes and shingles	3

ASCE 7-22 Table C3.1-1a Minimum Design Dead Loads (psf)

$Dead_{Membrane} := 1\ psf$

Normal Weight Concrete Composite Deck with Welded-Wire Reinforcements

1.5VL-36 COMPOSITE DECK-SLAB Normal Weight Concrete - 22 gage - 5" total depth - 3.5" topping

Link to Website: https://vulcraft.com/Products/Deck#composite-deck

Link to Datasheet:https://vulcraft.com/catalogs/Deck/CompositeDeck/LRFD-1.5VL-36-1.5VLI-36-1.5PLVLI-36_Composite_Deck-Slab.pdf

$Dead_{CompositeDeck} := 50.3\ psf$

Slab Depth			Maximum Unshored Spans			Composite Deck-Slab Properties			
		Deck	Maximum Unshored Construction Clear Span			Concrete + Deck	Deflection $I_d = (I_{cr}+I_u)/2$	Moment ØM_{no}	Shear ØV_{no}
Total	Topping	Gage	1	2	3	(psf)	(in⁴/ft)	(kip-ft/ft)	(kip/ft)
3½"	2"	22	6'-6"	7'-7"	7'-8"	32.2	2.64	2.73	3.02
		20	7'-10"	9'-0"	9'-3"	32.6	2.85	3.22	3.02
		19	8'-4"	9'-11"	10'-3"	32.9	3.03	3.67	3.02
		18	8'-9"	10'-7"	10'-11"	33.2	3.19	4.07	3.02
		16	9'-6"	11'-10"	11'-8"	33.9	3.52	4.91	3.02
5"	3½"	22	5'-8"	6'-7"	6'-8"	50.3	7.62	4.79	4.93
		20	6'-9"	7'-9"	7'-11"	50.7	8.18	5.69	4.93
		19	7'-3"	8'-7"	8'-10"	51.0	8.68	6.54	4.93
		18	7'-8"	9'-2"	9'-5"	51.3	9.12	7.30	4.93
		16	8'-4"	10'-3"	10'-4"	52.0	10.02	8.92	4.93
6"	4½"	22	5'-3"	6'-1"	6'-2"	62.4	13.11	6.30	6.41
		20	6'-3"	7'-2"	7'-4"	62.8	14.02	7.51	6.41
		19	6'-10"	7'-11"	8'-2"	63.1	14.85	8.64	6.41
		18	7'-2"	8'-5"	8'-9"	63.4	15.57	9.67	6.41
		16	7'-10"	9'-6"	9'-8"	64.1	17.06	11.87	6.41

Total Ramp Dead Load

$$Dead_{Ramp} := Dead_{Membrane} + Dead_{CompositeDeck} = 51.3\ psf$$

The dead load for the ramp is approximately 52 psf

Snow Loads for Ramp

Used ASCE 7-16

Balanced Snow Load

Exposure Coefficient

Table 7.3-1. Exposure Factor, C_e.

	Exposure of Roof[a]		
Surface Roughness Category	Fully Exposed[b]	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	N/A
In Alaska, in areas where trees do not exist within a 2 mi (3 km) radius of the site	0.7	0.8	N/A

The terrain category and roof exposure condition chosen shall be representative of the anticipated conditions during the life of the structure. An exposure factor shall be determined for each roof of a structure.

[a] Partially Exposed: All roofs not Fully Exposed or Sheltered. Fully Exposed: Roofs exposed on all sides with no shelter[b] afforded by terrain, higher structures, or trees. Roofs that contain several large pieces of mechanical equipment, parapets that extend above the height of the balanced snow load (h_b), or other obstructions are not in this category. Sheltered: Roofs located tight in among conifers that qualify as obstructions.

[b] Obstructions within a distance of $10h_o$ provide "shelter," where h_o is the height of the obstruction above the roof level. If the only obstructions are a few deciduous trees that are leafless in winter, the "fully exposed" category shall be used. Note that these are heights above the roof. Heights used to establish the Exposure Category in Section 26.7 are heights above the ground.

$C_e := 1.0$

Surface Roughness B/Partially Exposed...
because we're in downtown Maquoketa

ASCE 7-16 Table 7.3-1 Exposure Coefficient, Ce

Thermal Factor

$C_t := 1.2$

Unheated Open Air Structure

Heated closed structure
without a ventilated roof

ASCE 7-16 Table 7.3-2 Thermal Factor, Ct

Didn't use ASCE 7-22
because C_t is requires
known material properties

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 $25°F \times h \times ft^2/Btu$ ($4.4\ K \times m^2/W$)	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 $2.0°F \times h \times ft^2/Btu$ ($0.4\ K \times m^2/W$)	0.85

[a] These conditions shall be representative of the anticipated conditions during winters for the life of the structure.

[b] Greenhouses with a constantly maintained interior temperature of 50°F (10°C) or more at any point 3 ft (0.9 m) above the floor level during winters and having either a maintenance attendant on duty at all times or a temperature alarm system to provide warning in the event of a heating failure.

Importance Factor

Table 1.5-2 Importance Factors by Risk Category of Buildings and Other Structures for Snow, Ice, and Earthquake Loads

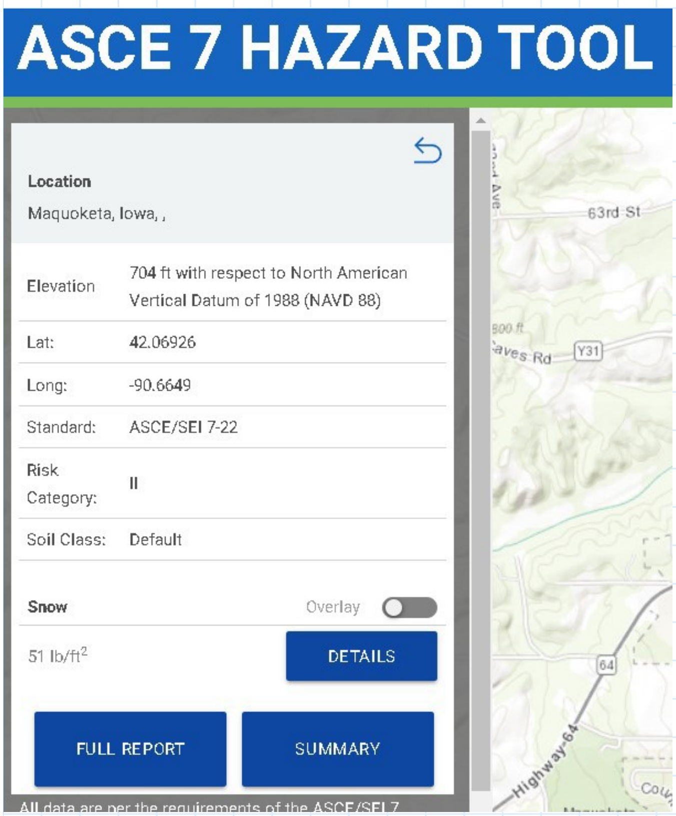
Risk Category from Table 1.5-1	Snow Importance Factor, I_s	Ice Importance Factor—Thickness, I_i	Ice Importance Factor—Wind, I_w	Seismic Importance Factor, I_e
I	0.80	0.80	1.00	1.00
II	1.00	1.00	1.00	1.00
III	1.10	1.15	1.00	1.25
IV	1.20	1.25	1.00	1.50

Note: The component importance factor, I_p, applicable to earthquake loads, is not included in this table because it depends on the importance of the individual component rather than that of the building as a whole, or its occupancy. Refer to Section 13.1.3.

$I_s := 1.0$

Risk Category 2
ASCE 7-16 Table 1.5-2

Ground Snow Load



ASCE Hazard Tool

$p_g := 51\ psf$

No other snow load indicated in the Maquoketa Code of Ordinances

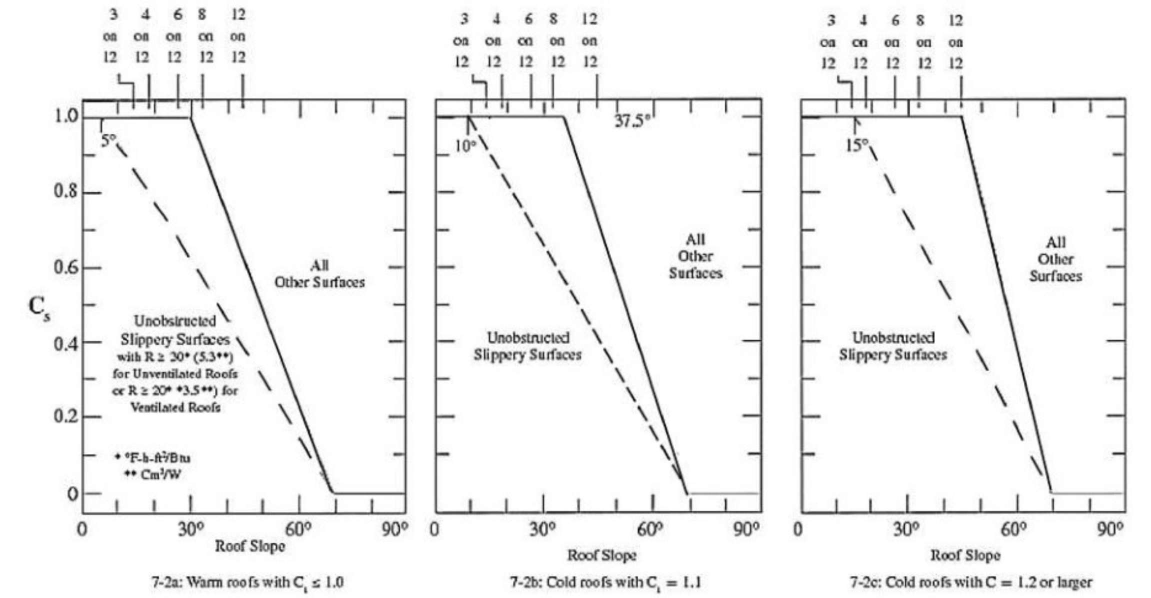


FIGURE 7.4-1 Graphs for Determining Roof Slope Factor, C_s, for Warm and Cold Roofs (See Table 7.3-2 for C_t Definitions)

Sloped Roof Factor $C_s := 1.0$ Using ramp slope as the roof slope

Sloped Roof Snow Load $p_s := 0.7 \cdot C_s \cdot C_e \cdot C_t \cdot I_s \cdot p_g = 42.84\ psf$

$BalancedSnow_{Load} := p_s = 42.84\ psf$

The snow load is approximately 43 psf

Live Loads

Used ASCE 7-22

Table 4.3-1. *(Continued)* Minimum Uniformly Distributed Live Loads, L_o, and Minimum Concentrated Live Loads

Occupancy or Use	Uniform, L_o psf (kN/m²)	Live Load Reduction Permitted? (Sec. No.)	Multiple-Story Live Load Reduction Permitted? (Sec. No.)	Concentrated lb (kN)	Also See Section
Penal institutions					
Cell blocks	40 (1.92)	Yes (4.7.2)	Yes (4.7.2)		
Corridors	100 (4.79)	Yes (4.7.2)	Yes (4.7.2)		
Recreational uses					
Bowling alleys, poolrooms, and similar uses	75 (3.59)	No (4.7.5)	No (4.7.5)		
Dance halls and ballrooms	100 (4.79)	No (4.7.5)	No (4.7.5)		
Gymnasiums	100 (4.79)	No (4.7.5)	No (4.7.5)		
Residential					
One- and two-family dwellings					
Uninhabitable attics without storage	10 (0.48)	Yes (4.7.2)	Yes (4.7.2)		4.12.1
Uninhabitable attics with storage	20 (0.96)	Yes (4.7.2)	Yes (4.7.2)		4.12.2
Habitable attics and sleeping areas	30 (1.44)	Yes (4.7.2)	Yes (4.7.2)		
All other areas except stairs	40 (1.92)	Yes (4.7.2)	Yes (4.7.2)		
All other residential occupancies					
Private rooms and corridors serving them	40 (1.92)	Yes (4.7.2)	Yes (4.7.2)		
Public rooms	100 (4.79)	No (4.7.5)	No (4.7.5)		
Corridors serving public rooms	100 (4.79)	Yes (4.7.2)	Yes (4.7.2)		
Roofs					
Ordinary flat, pitched, and curved roofs	20 (0.96)	Yes (4.8.2)	—		4.8.1
Roof areas used for occupants	Same as occupancy served	Yes (4.8.3)	—		
Roof areas used for assembly purposes	100 (4.70)	Yes (4.8.3)			
Vegetative and landscaped roofs					
Roof areas not intended for occupancy	20 (0.96)	Yes (4.8.2)	—		
Roof areas used for assembly purposes	100 (4.70)	Yes (4.8.3)	—		
Roof areas used for other occupancies	Same as occupancy served	Yes (4.8.3)	—		
Awnings and canopies					
Fabric construction supported by a skeleton structure	5 (0.24)	No (4.8.2)	—		
Screen enclosure support frame	5 (0.24) based on the tributary area of the roof supported by the frame member	No (4.8.2)	—	200 (0.89)	

$Live_{Load} := 100\ psf$

For roof area used for assembly purposes

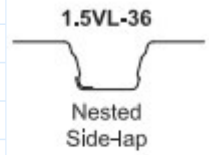
Since the roof won't be occupied at the same time as snowstorm, use the larger number load (snow or live load).

$Live_{Load} = 100\ psf$ >> $BalancedSnow_{Load} = 42.84\ psf$

Regardless, we will test each load combination for sufficient strength of members

1.5VL-36 COMPOSITE DECK GRADE 50 STEEL

Link to Website: https://vulcraft.com/Products/Deck#composite-deck



Link to Datasheet: https://vulcraft.com/catalogs/Deck/CompositeDeck/LRFD-1.5VL-36-1.5VLI-36-1.5PLVLI-36_Composite_Deck-Slab.pdf

Slab Makeup:
Total - 5"
Topping - 3.5" of Normal weight concrete (145 pcf)
Deck Gauge - 22

Superimposed Load: Based on LRFD Load Combinations

LC	LRFD	ASD
1	$1.4D$	D
2	$1.2D + 1.6L + 0.5(L_r \text{ or } S \text{ or } R)$	$D+L$
3	$1.2D + 1.6(L_r \text{ or } S \text{ or } R) + (L \text{ or } 0.5W)$	$D + (L_r \text{ or } S \text{ or } R)$
4	$1.2D + W + L + 0.5(L_r \text{ or } S \text{ or } R)$	$D + 0.75L + 0.75(L_r \text{ or } S \text{ or } R)$
5	$0.9D + W$	$D+0.6W$
6	$1.2D + E_v + E_h + L + 0.2S$	$D + 0.75L + 0.75(0.6W) + 0.75(L_r \text{ or } S \text{ or } R)$
7	$0.9D - E_v + E_h$	$0.6D + 0.6W$
8		$D + 0.7E_v + 0.7E_h$
9		$D + 0.525E_v + 0.525E_h + 0.75L + 0.75S$
10		$0.6D - 0.7E_v + 0.7E_h$

Where:
D= Dead Load
L= Live Load
Lr= Roof Live Load
S= Snow Load
W= Wind Load
R= Rain Load

Loads:

$Dead_{Membrane} := 1 \ psf$

$Dead_{CompositeDeck} := 50.3 \ psf$

$D := Dead_{Membrane} = 1 \ psf$ **Superimposed**

$L := 100 \ psf$

$S := 43 \ psf$

$W := 12.442 \ psf$

$W_{uplift} := -11.212 \ psf$

LRFD Load Combos:

$1.4\ D = 1.4\ psf$ **(1)**

$1.2\ D + 1.6\ L + 0.5\ S = 182.7\ psf$ **(2)**

$1.2\ D + 1.6\ S + L = 170\ psf$ **(3)** Neglect wind load in combo 3 because it is smaller than L

$1.2\ D + W + L + 0.5\ S = 135.142\ psf$ **(4)**

$0.9\ \left(\left(D + Dead_{CompositeDeck}\right)\right) + W_{uplift} = 34.958\ psf$ **(5)** Uplift is not an issue

$MaxLoad := 182.7\ psf$

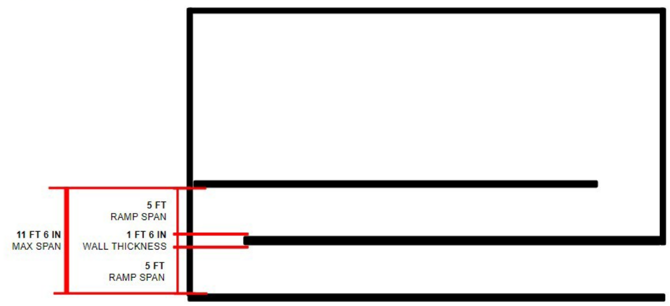
Allowable Superimposed

Span:

Designing for max span of 11'-6"

$Allowable := 192\ psf$

AERIAL VIEW OF PHASE 2:



$MaxLoad = 182.7\ psf$ < $Allowable = 192\ psf$

Sufficient for Allowable Deflection... which is the limiting factor!

Superimposed Design Load, $øW_n$, / Deflection at L/360 (psf) NWC (145 pcf), f'_c = 3000 psi

Total Slab Depth	Deck Gage	Span (ft-in.) 4'-0"	5'-0"	6'-0"	7'-0"	8'-0"	9'-0"	10'-0"	12'-0"
3½"	22	1327/1804	835/923	568/534	407/336	302/225	231/158	179/115	113/66
	20	1471/1944	990/995	676/576	486/362	363/243	278/170	218/124	139/72
	19	1471/2071	1135/1060	776/613	560/386	419/258	323/181	254/132	164/76
	18	1471/2179	1168/1115	864/645	624/406	469/272	362/191	285/139	186/80
	16	1470/2401	1168/1229	966/711	761/448	573/300	444/210	352/153	232/88
5"	22	2336/5206	1473/2665	1004/1542	722/971	538/650	412/457	323/333	205/192
	20	2405/5585	1761/2859	1204/1654	868/1042	650/698	501/490	394/357	255/206
	19	2404/5929	1911/3035	1391/1756	1006/1106	756/741	584/520	461/379	302/219
	18	2404/6228	1911/3188	1559/1845	1129/1162	850/778	659/546	522/398	343/230
	16	2403/6842	1910/3503	1581/2027	1346/1276	1052/855	818/600	651/437	433/253
6"	22	3075/8955	1941/4585	1325/2653	953/1670	712/1119	547/786	429/573	275/331
	20	3130/9574	2327/4902	1593/2836	1150/1786	863/1196	666/840	525/612	341/354
	19	3129/10137	2488/5190	1845/3003	1335/1891	1004/1267	777/889	615/648	404/375
	18	3129/10630	2488/5443	2060/3149	1502/1983	1132/1328	878/933	697/680	461/393
	16	3128/11651	2487/5965	2060/3452	1754/2174	1407/1456	1095/1022	873/745	582/431

Allowable Reaction at Supports (Based on LRFD Load Combinations)

Treat as a one way slab since the length is much much longer than the width

16 Deck Gauge
One Flange Loading
End bearing is 4"+

Design Reactions at Supports Based on Web Crippling, ϕR_n (lb/ft)

	Bearing Length of Webs											
	One-Flange Loading						Two-Flange Loading					
Deck	End Bearing				Interior Bearing		End Bearing				Interior Bearing	
Gage	1½"	2"	3"	4"	3"	4"	1½"	2"	3"	4"	3"	4"
22	1235	1357	1563	1706	2204	2383	1289	1389	1556	1672	2728	2966
20	1763	1932	2215	2408	3164	3406	1949	2093	2333	2497	3960	4286
19	2344	2562	2927	3169	4222	4527	2702	2893	3213	3426	5324	5740
18	2954	3221	3669	3959	5334	5699	3515	3754	4156	4417	6762	7265
16	4525	4915	5568	5967	8206	8709	5681	6043	6651	7023	10487	11191

$$Allowable := 1563\ plf$$

$$b_{tributary} := 5\ ft + 0.5\ \ 1.5\ ft = 5.75\ ft$$

$$Actual := MaxLoad \cdot b_{tributary} = 1050.525\ plf$$

$$Actual = 1050.525\ plf \qquad < \qquad Allowable = 1563\ plf$$

Sufficient for Web Crippling, will not be an issue

Shear Stud Spacing

https://vulcraft.com/DesignTools/HighPerformanceDeckSlabDiaphragmStrength

Required Shear

$$WindLoad_{Max} := 12.442\ psf$$

$$Shear_{Required} := \left(WindLoad_{Max}\right) \cdot 7\ ft = 87.094\ plf$$

(Half of largest wall height)

Round up to 90 plf

Input

High Performance Deck-Slab Diaphragm Strength

Input Design Criteria

NUCOR VULCRAFT

Print

Design Basis

Unit System	Imperial
Design Method	LRFD
Required Factored Diaphragm Shear, $\bar{v}$ (plf)	90

Deck Selection

Deck Option	Composite Deck
Deck Type	1.5VL-36
Deck Gage	22
Deck Grade	Grade 50

Concrete Slab

Total Slab Thickness (in.)	3.5 ≤ 5.00 ≤ 7.5
Structural Concrete Unit Weight (pcf)	90 ≤ 145 ≤ 160
Structural Concrete Strength (psi)	2500 ≤ 3000 ≤ 5000

Structural Concrete Reinforcement

Concrete Reinforcement Type	Dramix® Steel Fibers
Fiber Type	4D 65/60BG
Fiber Dosage (pcy)	15 ≤ 35.00 ≤ 66

Support Connections

Deck to Support Connection Type	1/2" Steel Headed Stud Anchor
Support Member Thickness (in.)	0.2 ≤ 0.300
Support Member	A572 GR50

Deck-Slab Diaphragm Shear Strength

22 ga 1.5VL-36 Grade 50 Composite Deck

5 in. total slab depth, f'c = 3000 psi, 145 pcf NWC

35 pcy of Bekaert® Dramix® 4D 65/60BG Fiber Reinforcing [4]

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1.5VL-36/1.5VLI-36/1.5PLVLI-36 Composite Deck-Slab Information $f'_c = 3000$ psi

Total Slab Depth (in.)	Cover Depth (in.)	Theoretical Concrete Volume ($yd^3/100\ ft^2$)	Min. A_s for T&S ($in.^2$)	Recommended Reinforcing for Temperature and Shrinkage: WWR	(OR) Bekaert Dramix® Steel Fiber Alternate to WWR (lb/yd^3): 4D 65/60BG
Normal Weight Concrete (145 pcf)					
3½	2	0.78	0.028	6x6-W1.4xW1.4	23
4	2½	0.94	0.028	6x6-W1.4xW1.4	18
4½	3	1.09	0.028	6x6-W1.4xW1.4	15
5	3½	1.24	0.032	6x6-W2.1xW2.1	15
5½	4	1.40	0.036	6x6-W2.1xW2.1	15
6	4½	1.55	0.041	6x6-W2.1xW2.1	15
Light Weight Concrete (110 pcf)					
3½	2	0.78	0.028	6x6-W1.4xW1.4	33
4	2½	0.94	0.028	6x6-W1.4xW1.4	25
4½	3	1.09	0.028	6x6-W1.4xW1.4	20
4¾	3¼	1.17	0.029	6x6-W2.1xW2.1	20
5	3½	1.24	0.032	6x6-W2.1xW2.1	20
5¾	4¼	1.48	0.038	6x6-W2.1xW2.1	20

Notes:
1. FRC reinforcement is based on IAPMO UES ER-465.
2. Dramix® fibers may be used in UL or ULC fire rated assemblies in lieu of WWR. See UL file R19307 for additional information.

Output

Deck-Slab Diaphragm Shear Strength

22 ga 1.5VL-36 Grade 50 Composite Deck

5 in. total slab depth, f'c = 3000 psi, 145 pcf NWC

35 pcy of Bekaert® Dramix® 4D 65/60BG Fiber Reinforcing [4]

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1/2" Steel Headed Stud Anchor at Chords & Collectors for Shear Transfer

Perpendicular Connection Pattern[1]			1 every 6th rib	
Parallel Connection Attachment (maximum)	1	row at	16	in. o.c.

Minimum Connections to Supporting Members[2]

Minimum connections to all supports may be any of the following: arc spot welds, fillet welds, PAF's, screws, Shearflex® anchors, welded studs or other mechanical connections[3].

Perpendicular Connection Pattern[1]	1 every other rib		
Parallel Perimeter Connections for deck spans greater than 5 ft		36	in. o.c.

Governing Deck-Slab Strength and Stiffness

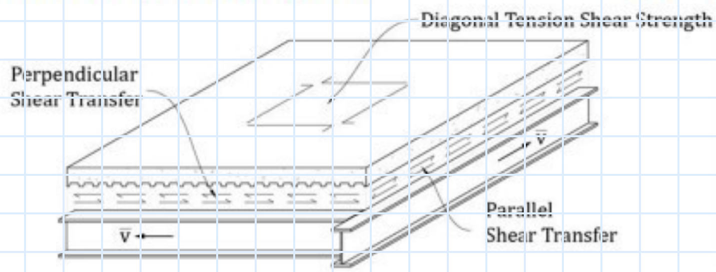
Available Diaphragm Design Strength	**Va = ΦQN =**	**1401**	**plf**
Controlled by Connections to Chords and Collectors			
Deck-Slab Diaphragm Design Shear Stiffness	G' =	1170	kip/in

Deck-Slab Shear Strength

Deck-Slab Diagonal Tension Design Shear Strength	ΦSn =	11593	plf

Chords & Collector Shear Transfer Strength

Chord & Collectors Design Shear Transfer Strength	ΦQN =	1401	plf

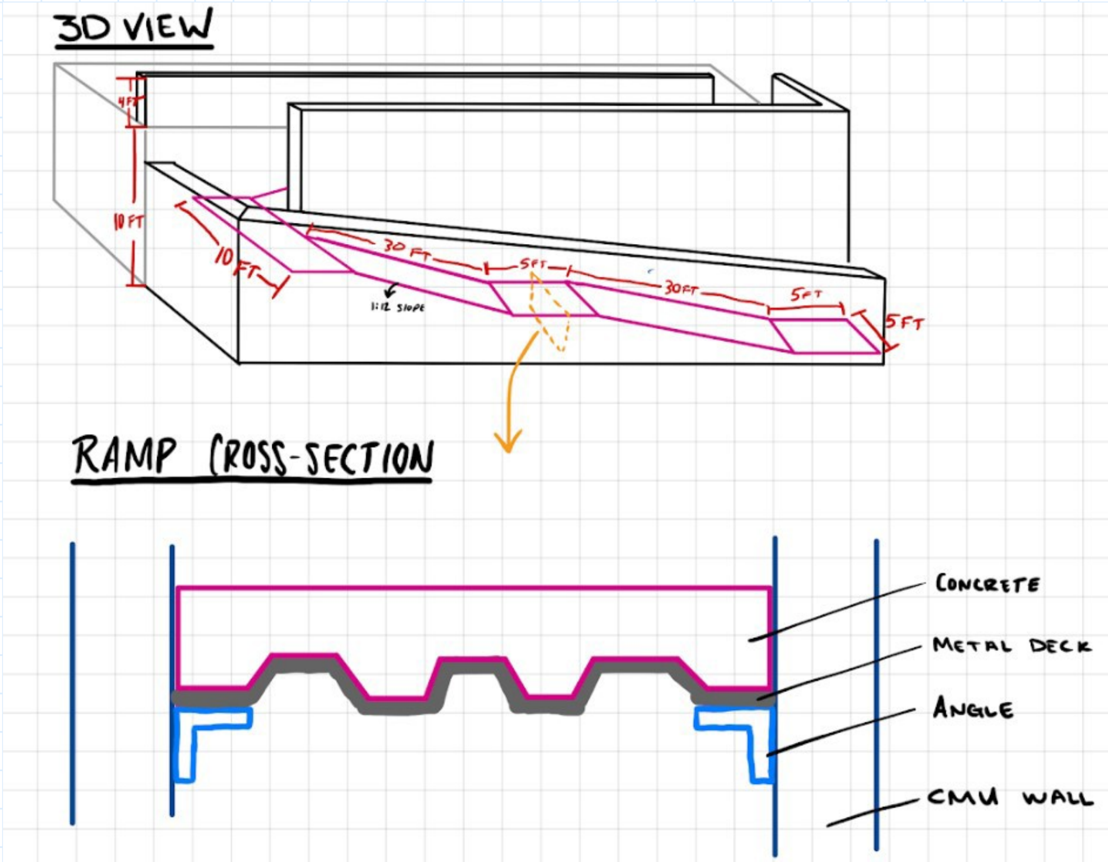


Notes:

1. For UL Fire rated assemblies, refer to UL Design Number for support and sidelap connection requirements.
2. Minimum connections to supporting members do not contribute to the diaphragm shear strength.
3. Support welds at interlocking sidelaps may be 3/8" x 1 1/4" arc seam welds in lieu of arc spot welds.
4. Dramix Steel Fibers up to 66 lb/cy are approved in lieu of welded wire fabric in all UL D700, D800 , and 900 Series Designs, and G229.
5. Sidelap connections between steel deck panels may be VSC2, button punch, screw, 1-1/2 in. arc seam weld or 1-1/2 in. top arc seam weld. The maximum sidelap connection spacing shall not exceed 36 in. o.c.

Calculations generated Per 2018 IBC & IAPMO ER-0652 using calculator V1.1 Date: 3/30/2023

STEEL ANGLE CONNECTION:



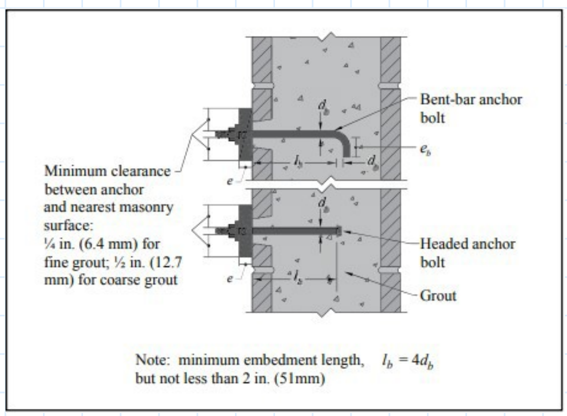
Live Load: $L := 100\ psf$

Dead Load: $D := 52\ psf$

Snow Load: $S := 43\ psf$

Wind Load: $W := 0.391\ psf$

Uplift Load: $W_{uplift} := -13.671\ psf$



Width of Ramp: $l := 5\ ft$

Slab Strip Width: $l_s := 1\ ft$

LRFD load combos:

$w_1 := 1.4\ D = 72.8\ psf$

$w_2 := 1.2\ D + 1.6\ L + 0.5\ S = 243.9\ psf$

$w_3 := 1.2\ D + 1.6\ S + L = 231.2\ psf$

$w_4 := 1.2\ D + W + L + 0.5\ S = 184.291\ psf$

$w_5 := 0.9\ D + W_{uplift} = 33.129\ psf$

Finding Required Strength

Factored Distributed Load: $w_u := \max\left(\left(w_1, w_2, w_3, w_4, w_5\right)\right) \cdot l_s = 0.244\ kip \div ft$

Load Bearing on one Angle: $P_u := w_u \cdot l \div 2 = 0.61\ kip$ (Load distributed to angles on each side of ramp)

Max Moment: $M_{max} := w_u \cdot l^2 \div 12 = 0.508\ kip \cdot ft$

Max Shear: $V_{max} := w_u \cdot (l \div 2) = 0.61\ kip$

(AISC TABLES - CASE 15 - FIXED AT BOTH ENDS WITH UNIFORMLY DISTRIBUTED LOAD)

Angle Selection

Angle: **L3x3x1/2**

Length of Legs: $b := 3\ in$

Leg Thickness: $t := 0.5\ in$

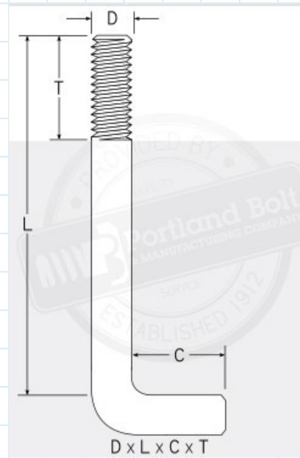
Plastic Section: $Z_x := 1.91\ in^3$

Grade of Steel: **A36**

Steel Yield Strength: $F_y := 36\ ksi$

Ultimate Strength: $F_u := 58\ ksi$

Modulus of Elasticity: $E := 29000\ ksi$



Anchor Bolt Size (D x L x C x T)			
Diam.	Length	Hook	Thread
½	8	2	4

6

Bolt Selection

Bolt Type: **Bent Bar Anchor Bolt**

Bolt Grade: **F1554 Grade 36**

Yield Strength: $F_{yb} := 36\ ksi$

Ultimate Strength: $F_{ub} := 58\ ksi$

Bolt Diameter: $d_b := 0.5\ in$

Bolt Hole: $d_h := d_b + (1 \div 16)\ in$ (Standard holes)

Threads are excluded (X bolts) or included (N bolts) in the shear plane: $Bolt :=$ "N"

Check Flexural Strength based on Flange Local Buckling

Width to Thickness Ratio: $\lambda_f := b \div t = 6$

Limiting flange slenderness parameter: $\lambda_{pf} := 0.54\ \sqrt{E \div F_{yb}} = 15.326$

Limiting flange slenderness parameter: $\lambda_{rf} := 0.91\ \sqrt{E \div F_{yb}} = 25.828$

For Compact Flange ($\lambda_f \le \lambda_{pf}$):

Plastic Moment about x-axis: $M_p := F_y \cdot Z_x = 5.73\ kip \cdot ft$

Flexural Strength based on FLB: $\phi M_n := 0.9 \cdot M_p = 5.157\ kip \cdot ft$

Required Flexural Strength: $M_{max} = 0.508\ kip \cdot ft$

The section IS adequate for flexure since $M_{max} < \phi M_n$

Check for Shear Strength of Bolt

Available shear strength of bolt:

$$F_{nv} := \text{if } Bolt = \text{“X”} \;\; \| \; 0.563 \cdot F_{ub} \;\; \text{else if } Bolt = \text{“N”} \;\; \| \; 0.45 \cdot F_{ub} \quad = 26.1 \; ksi$$

Area of Bolt: $A_b := \pi \cdot \left(\left(d_b^2 \div 4\right)\right) = 0.196 \; in^2$

Bolt Shear Strength: $\phi R_{nShear} := 0.75 \cdot \left(\left(F_{nv}\right)\right) \cdot A_b = 3.844 \; kip$

Required Shear Strength: $V_{max} = 0.61 \; kip$

The section IS adequate for shear since $V_{max} < \phi R_{nShear}$

Check Bearing Strength

https://ncma.org/resource/design-of-anchor-bolts-embedded-in-concrete-masonry/

Input into "Masonry Anchor Bolt Design Calculator":

Date: 4/13/2023
(MM/DD/YYYY)

2013 MSJC Anchor Bolt Design
Allowable Stress Design

**** user input indicated by blue cells*

DATA INPUT AND SUMMARY OF DESIGN

Propeties and Geometry

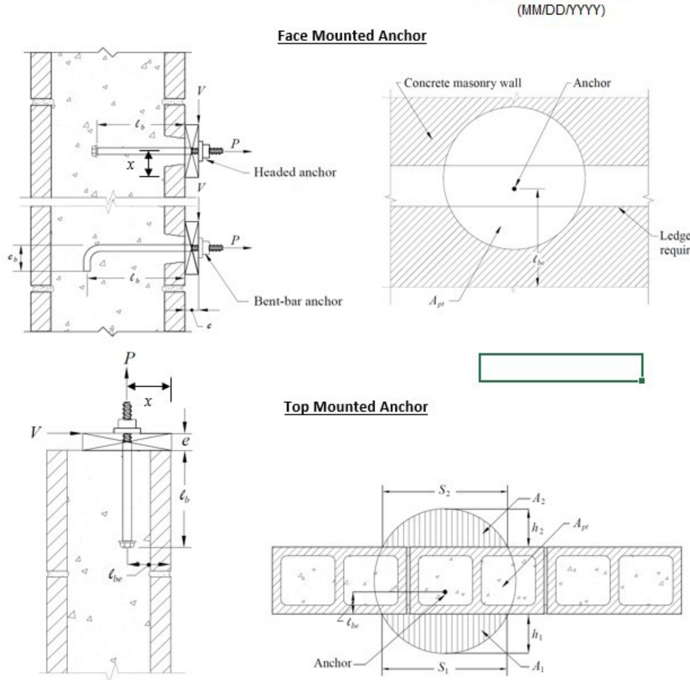
Weather or Soil Exposure	NO	
Top or Face Mount	Face	

***Assumed adequate distance from top/bottom of wall and spacing from adjacent anchors to develop breakout cone*

Anchor Type =	bent-bar	
Anchor Yield Strength =	36000	psi
Anchor Diameter, d_b =	1/2	in
Anchor Hook Length, e_b =	2.00	in
f'_m =	2000	psi
Wall thickness, t =	18.000	in
Edge Distance, l_{be} =	3.50	in
Net Anchor Area, A_b =	0.142	in^2
Effective Embed. Length, l_b =	7.00	in

Loading

Shear Force, V_{total} =	610	lbs
Offset distance, e =	0.50	in
Dist. From C.L. of Bolt To Edge of Ledger, x =	1.50	in
Direct Tension Force, P_{total} =	610	lbs



<<Anchor design is satisfactory. See detailed analysis>>

Output of "Masonry Anchor Bolt Design Calculator":

Detailed Analysis

Check minimum embed.

[TMS 402-13 §1.17.3] l_b=	7.00	in	>	l_{b_min} = min(4d_b,2 in) =	2.00 in	**<Satisfactory>**

Check minimum cover

[TMS 402-13 §1.16.4.1] $cover_{min}$ = 1.50 in

$cover_{actual}$= (for top mounted) $t - l_{ab}$ = / (for face mounted) $t - l_b$ = 11.00 in **<Satisfactory>**

Total Tension Force Considering Ecc., b_{af}

$b_{af} = P_u + \frac{V_u e}{(\frac{5}{6})d} =$ 354 lbs ****assuming that moment arm is (5/6) of 'd'*

Determine Tensile Capacity

h_1=	0.00	in	s_1=	0.00	in	A_1=	0.00	in^2
h_2=	0.00	in	s_2=	0.00	in	A_2=	0.00	in^2

$A_{pt} = \pi l_b{}^2 =$ 153.94 in^2

[TMS 402-13 Eqn. 8-1] Masonry Tensile Breakout	$B_{ab} = 1.25A_{pt}\sqrt{f'_m} =$	8605 lbs			
[TMS 402-13 Eqn. 8-2] Steel Tensile Yield	$B_{as} = 0.6A_b f_y =$	3067 lbs			
[TMS 402-13 Eqn. 8-4] Anchor Pullout	$B_{ap} = (0.6f'_m e_b d_b + 120\pi(l_b + e_b + d_b)d_b) =$	2991 lbs			
Design Axial Strength	$B_a =$	2991 lbs	>	b_{af}	**<Satisfactory>**

Governing Failure Mode: ***Pullout***

Determine Shear Capacity

$A_{pv} = \frac{\pi l_{be}{}^2}{2} =$ 19.24 in^2

[TMS 402-13 Eqn. 8-6] Masonry Shear Breakout	$B_{vb} = 1.25A_{pv}\sqrt{f'_m} =$	1076 lbs			
[TMS 402-13 Eqn. 8-7] Masonry Crushing	$B_{vc} = 350\sqrt[4]{f'_m A_b} =$	1437 lbs			
[TMS 402-13 Eqn. 8-8] Anchor Shear Pryout	$B_{vpry} = 2.0B_{ab} = 2.5A_{pt}\sqrt{f'_m} =$	17211 lbs			
[TMS 402-13 Eqn. 8-9] Steel Shear Yielding	$B_{vs} = 0.36A_b f_y =$	1840 lbs			
Design Shear Strength	$B_v =$	1076 lbs	>	b_{vf}	**<Satisfactory>**

Governing Failure Mode: ***Breakout***

Check Combined Tension and Shear Interaction

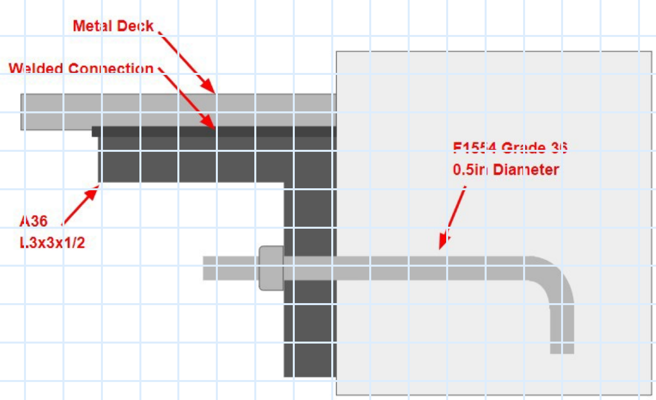
[TMS 402-13 Eqn. 8-10] $\frac{b_a}{B_a} + \frac{b_v}{B_v} =$ 0.353 < 1.000 **<Satisfactory>**

NATIONAL CONCRETE MASONRY ASSOCIATION

NCMA TEK TECHNICAL SOLUTIONS

Revision Date: 03/19/2019

Overview:



Appendix F: Restroom Phase Three Design Calculations

HASE 3 ROOF CALCULATIONS

ASCE 7-16 Chapters 26 and 27

Open Structure Wind Calculations with a Monoslope Roof

$V := 108 \frac{mi}{hr}$

Chapter 26

$Kd := 0.85 \qquad Ke := 1 \qquad Kz := 0.605$

$q := 0.00256 \frac{psf}{mph^2} \cdot Kz \cdot Kd \cdot Ke \cdot V^2 = 15.355\ psf$

South Wind Loads

$CnW := 1.2 \qquad G := 0.85$

$p := q \cdot G \cdot CnW = 15.663\ psf$

North Wind Loads

Figure 27.3-4

$CnW := -1.1 \qquad G := 0.85$

$p := q \cdot G \cdot CnW = -14.357\ psf$

East Wind Load

$Cn := 0.3 \qquad G := 0.85$

$p := q \cdot G \cdot Cn = 3.916\ psf$

West Wind Load

Figure 27.3-7

$Cn := 0.3 \qquad G := 0.85$

$p := q \cdot G \cdot Cn = 3.916\ psf$

Balanced Snow Load

Exposure Coefficient

Table 7.3-1. Exposure Factor, C_e.

Surface Roughness Category	Exposure of Roof[a]		
	Fully Exposed[b]	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	N/A
In Alaska, in areas where trees do not exist within a 2 mi (3 km) radius of the site	0.7	0.8	N/A

The terrain category and roof exposure condition chosen shall be representa-

$C_e := 0.9$

Surface Roughness B/Partially Exposed...
because we're in downtown Maquoketa

ASCE 7-16 Table 7.3-1 Exposure Coefficient

Importance Factor

Table 1.5-2 Importance Factors by Risk Category of Buildings and Other Structures for Snow, Ice, and Earthquake Loads

Risk Category from Table 1.5-1	Snow Importance Factor, I_s	Ice Importance Factor—Thickness, I_i	Ice Importance Factor—Wind, I_w	Seismic Importance Factor, I_e
I	0.80	0.80	1.00	1.00
II	1.00	1.00	1.00	1.00
III	1.10	1.15	1.00	1.25
IV	1.20	1.25	1.00	1.50

Note: The component importance factor, I_p, applicable to earthquake loads, is not included in this table because it depends on the importance of the individual component rather than that of the building as a whole, or its occupancy. Refer to Section 13.1.3.

$I_s := 1.0$

Risk Category 2
ASCE 7-16 Table 1.5-2

Ground Snow Load

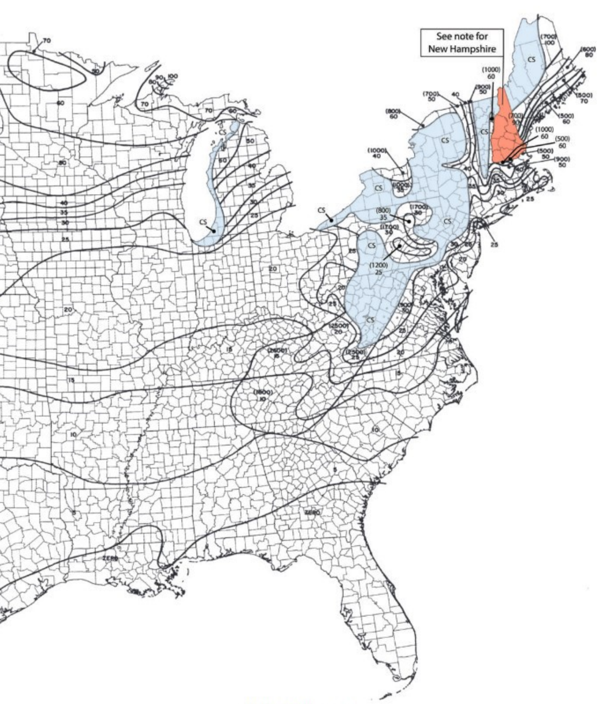


FIGURE 7.2-1 (*Continued*)

hip, and gable roofs with slopes less than 15° and to curved roofs where the vertical angle from the eaves to the crown is less than 10°. The minimum roof snow load for low-slope roofs shall be obtained using the following formula:

Where p_g is 20 lb/ft^2 (0.96 kN/m^2) or less:

$$p_m = I_s p_g \quad \text{(Importance Factor times } p_g\text{)}$$

Where p_g exceeds 20 lb/ft^2 (0.96 kN/m^2):

$$p_m = 20(I_s) \quad \text{(20 lb/ft}^2\text{ times Importance Factor)}$$

$$p_m = 0.96(I_s) \,\text{(0.96 kN/m}^2\text{ times Importance Factor)}$$

This minimum roof snow load is a separate uniform load case. It need not be used in determining or in combination with drift, sliding, unbalanced, or partial loads.

Minimum Design Loads and Associated Criteria for Buildings and Other Structures 53

$p_g := 25\ psf$

ASCE 7-16 Figure 7.2-1

No other snow load indicated in the Maquoketa Code of Ordinances

Sloped Roof Factor $C_s := 1$ Essentially a flat roof

Sloped Roof Snow Load $p_s := 0.7 \cdot C_s \cdot C_e \cdot C_t \cdot I_s \cdot p_g = 18.9\ psf$

ase 3 Roof Calculations

Metal Deck

60 solar panels at 40 lbs each or 2.8psf

5.5 ft x 3.25 ft

$A := 5.5\ ft \cdot 3.25\ ft = 17.875\ ft^2$

Uplift Calculations(Ignoring Weight of solar panels)

Accounting for metal deck and steel slates(keep water off decking)

$DeckDead := 2.5\ psf + 1\ psf$

$WL := -14.375\ psf$

$wUplift := 0.6 \cdot DeckDead + 0.75 \cdot 0.6 \cdot WL = -4.369\ psf$

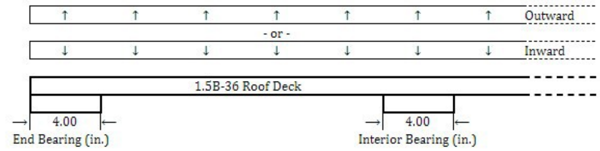
Deck can support uplift, design OK

18 Gage 1.5B-36 Grade 50
Uniform Design Load Table, LRFD (psf)
For Butted End Deck

NUCOR VULCRAFT GROUP

36 / 4 Connection Pattern to Supports with #12 Screw

Support Member A992 GR50
$0.25 \le t_2$ (in.) ≤ 0.5



Inward Uniform Design Load Table, LRFD (psf)

Span	(ft-in)	8'-0"	8'-6"	9'-0"	9'-6"	10'-0"	10'-6"	11'-0"	11'-6"
1	ΦWn	143	127	113	102	92	83	76	69
	L/240	35	30	25	21	18	16	14	12
2	ΦWn	148	131	117	105	95	86	79	72
	L/240	89	75	63	53	46	40	34	30
3	ΦWn	185	164	146	131	119	108	98	90
	L/240	70	58	49	42	36	31	27	24

Uplift (Outward) Uniform Design Load Table, LRFD (psf)

Span	(ft-in)	8'-0"	8'-6"	9'-0"	9'-6"	10'-0"	10'-6"	11'-0"	11'-6"
1	ΦWn	149	132	118	106	95	87	79	72
	L/240	37	31	26	22	19	16	14	13
2	ΦWn	92	87	82	77	74	70	67	64
	L/240	85	71	60	51	44	38	33	29
3	ΦWn	105	98	93	88	84	80	76	73
	L/240	67	56	47	40	34	30	26	23

Steel Deck Properties

t	Fy	wdd	Id+	Id-	Se+	Se-	ΦMn+	ΦMn-	ΦVn
in	ksi	psf	in.^{4}/ft	in.^{4}/ft	in.^{3}/ft	in.^{3}/ft	lbs-ft/ft	lbs-ft/ft	lbs/ft
0.0474	50	2.6	0.277	0.29	0.306	0.318	1148	1193	6398

Where: W ≤ ΦWn

W = Required strength of the governing LRFD load combintaion
ΦWn = Design Strength

Joists Supporting Deck

Using W10x12

$L := 20\ ft = 20\ ft \qquad a := 4.5\ ft$

6 psf acting from roof down: 3 psf from deck, 1 psf from panels, 2 psf from panels

$wDead := 6\ psf \qquad wLive := 0\ psf \qquad wSnow := 19\ psf \qquad wWind := 15.7\ psf$

$wu := 1.2 \cdot wDead + 1.6 \cdot wSnow + 0.5 \cdot wWind = 45.45\ psf$

$tb := 8\ ft$

$w := wu \cdot tb = 363.6\ plf \qquad Rroof := w \cdot L = 7.272\ kip$

$$M1 := \frac{w}{8 \cdot L^2} \cdot (L+a)^2 \cdot (L-a)^2 = 16.386\ ft \cdot kip$$

$$M2 := \frac{w \cdot a^2}{2} = 3.681\ ft \cdot kip$$

$Fy := 50\ ksi \qquad E := 29000\ ksi$

$$Zx := \frac{M1}{Fy} = 3.933\ in^3$$

Zx of W10x12 is 12.6in^3, so design OK

Using a coped end

$$\frac{c}{ho}=0.572 \qquad \frac{c}{d}=0.456$$

$$k:=\frac{2.2}{\left(\frac{c}{ho}\right)^{1.65}}=5.533 \qquad f:=2\cdot\left(\frac{c}{d}\right)=0.912$$

$$k1:=\max\left(f\cdot k, 1.61\right)=5.046$$

$$\lambda p:=0.475\cdot\sqrt{\frac{k1\cdot E}{Fy}}=25.696$$

$$h1:=\frac{bf\cdot tf\cdot\left(ho-0.5\cdot tf\right)+tw\cdot\left(ho-tf\right)\cdot\frac{\left(ho-tf\right)}{2}}{bf\cdot tf+tw\cdot\left(ho-tf\right)}=5.261\ in$$

$$Inet:=\frac{1}{12}\cdot bf\cdot tf^{3}+bf\cdot tf\cdot\left(ho-h1-0.5\cdot tf\right)^{2}+\frac{1}{12}\cdot tw\cdot\left(ho-tf\right)^{3}+tw\cdot\left(ho-tf\right)\cdot\left(0.5\cdot\left(ho-tf\right)-h1\right)^{2}=15.314\ in$$

$$Snet:=\frac{Inet}{h1}=2.911\ in^{3}$$

$$My:=Fy\cdot Snet=12.129\ ft\cdot kip$$

$$\phi Mn:=0.9\cdot\left(Mp-\left(Mp-My\right)\cdot\left(\frac{\lambda}{\lambda p}-1\right)\right)=14.173\ ft\cdot kip$$

Since flexural strength phi Mn is greater than Mu, a W10X12 member with a cope length of 4.5 in and depth of 2 in is suitable

Girders Supporting Joists

Only load it is supporting is from the joists, which acts at the mid point

$$Mmax:=\frac{RJoist\cdot L}{4}=18.68\ ft\cdot kip$$

$$Zx:=\frac{Mmax}{Fy}=4.483\ in^{3}$$

Use W12X14 to allow for single coped beam.

Columns Supporting Girders

Using W12x14

$$Pcolumn:=2.5\cdot RJoist=9.34\ kip$$

$$L:=9.5\ ft \qquad A:=3.54\ in^{2}$$

$$Lcx:=L=9.5\ ft$$

$$Lcy:=0.8\cdot L=7.6\ ft$$

Connections

Panels to deck

Any sort of clamp will suffice

Clamp at each end of the panel(4 corners) to the metal deck, or where best seen fit

Deck to Joist connection was previously shown, but will be shown again

18 Gage 1.5B-36 Grade 50

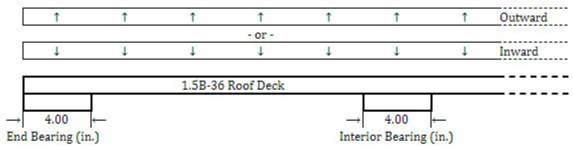
Uniform Design Load Table, LRFD (psf)

For Butted End Deck

NUCOR VULCRAFT GROUP

36 / 4 Connection Pattern to Supports with #12 Screw

Support Member A992 GR50
$0.25 \le t_2$ (in.) ≤ 0.5



Inward Uniform Design Load Table, LRFD (psf)

Span	(ft-in)	8'-0"	8'-6"	9'-0"	9'-6"	10'-0"	10'-6"	11'-0"	11'-6"
1	ΦWn	143	127	113	102	92	83	76	69
	L/240	35	30	25	21	18	16	14	12
2	ΦWn	148	131	117	105	95	86	79	72
	L/240	89	75	63	53	46	40	34	30
3	ΦWn	185	164	146	131	119	108	98	90
	L/240	70	58	49	42	36	31	27	24

Uplift (Outward) Uniform Design Load Table, LRFD (psf)

Span	(ft-in)	8'-0"	8'-6"	9'-0"	9'-6"	10'-0"	10'-6"	11'-0"	11'-6"
1	ΦWn	149	132	118	106	95	87	79	72
	L/240	37	31	26	22	19	16	14	13
2	ΦWn	92	87	82	77	74	70	67	64
	L/240	85	71	60	51	44	38	33	29
3	ΦWn	105	98	93	88	84	80	76	73
	L/240	67	56	47	40	34	30	26	23

Steel Deck Properties

t	Fy	wdd	Id+	Id-	Se+	Se-	ΦMn+	ΦMn-	ΦVn
in	ksi	psf	$in.^4/ft$	$in.^4/ft$	$in.^3/ft$	$in.^3/ft$	lbs-ft/ft	lbs-ft/ft	lbs/ft
0.0474	50	2.6	0.277	0.29	0.306	0.318	1148	1193	6398

Where: $W \le \Phi W_n$

W = Required strength of the governing LRFD load combintaion
ΦWn = Design Strength

36/4 connection pattern with #12 Screw

Meets required strength for what is needed for on top of deck

Joist to Girder

Start with half inch plate, 7 in height, 7 in width
3 5/8 in standard bolts, A325N

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Shear Strength of Bolt

$$db := \frac{5}{8}\ in \qquad Ab := \frac{\pi}{4} \cdot db^2 = 0.307\ in^2$$

$$FuBolt := 120\ ksi$$

$$Fnv := 0.45 \cdot FuBolt = 54\ ksi$$

Joist to Column

Start with half inch plate, 7 in height, 7 in width
3 5/8 in standard bolts, A325N

Shear Strength of Bolt

$db := \frac{5}{8}\ in$ $\qquad Ab := \frac{\pi}{4} \cdot db^2 = 0.307\ in^2$

$FuBolt := 120\ ksi$

$Fnv := 0.45 \cdot FuBolt = 54\ ksi$

$\phi RnShear := 0.75 \cdot Fnv \cdot Ab = 12.425\ kip$

Strength of Connection

$Le := 1.5\ in$

$dh := db + \frac{1}{16}\ in = 0.688\ in$

$Lce := Le - 0.5 \cdot dh = 1.156\ in$

$t := 0.225\ in$

$Fu := 65\ ksi$

$\phi RnEndBolt := 0.75 \cdot \min\ 1.2 \cdot Lce \cdot t \cdot Fu , 2.4 \cdot db \cdot t \cdot Fu\ = 15.219\ kip$

$s := 2\ in$ $\qquad Lc := s - dh = 1.313\ in$

$\phi RnIntBolt := 0.75 \cdot \min\ 1.2 \cdot Lc \cdot t \cdot Fu , 2.4 \cdot db \cdot t \cdot Fu\ = 16.453\ kip$

$\phi RnBearing := \phi RnEndBolt \cdot 3 = 45.657\ kip$

$ConnectionStrength := \min\ \phi RnShear , \phi RnBearing\ = 12.425\ kip$

This value is larger than the force experienced, so OK

Girder to Column

Welding the girder to 0.5 in plate, then weld plate to column flange
11 in transverse weld, one on each side of girder web

$L := 11\ in$ $\qquad Fy := 50\ ksi$ $\qquad Fu := 65\ ksi$ $\qquad t := 0.5\ in$

$\phi RnBM := \min\ 1 \cdot 0.6 \cdot Fy \cdot t \cdot L , 0.75 \cdot 0.6 \cdot Fu \cdot t \cdot L = 160.875\ kip$

$w := t - \frac{1}{16}\ in = 0.438\ in$ $\qquad \frac{L}{w} = 25.143$ $\qquad Fexx := 70\ ksi$

$$l := \max\left(\frac{N-0.95 \cdot d}{2}, \frac{B-0.8 \cdot bf}{2}, \frac{1}{4} \cdot \sqrt{d \cdot bf}\right) = 4.416\ in$$

$$tp := l \cdot \sqrt{\frac{2 \cdot Pu}{0.9 \cdot B \cdot N \cdot Fy}} = 0.242\ in$$

Anything 2/8 inches or more will suffice

Anchor Bolts

Start with 3/4 in

4 anchors

$$P := Pcolumn = 9.34\ kip$$

$$Fwind := wWind \cdot 16\ ft \cdot 9.5\ ft = 2.386\ kip$$

$$Mo := Fwind \cdot 9.5\ ft = 22.671\ ft \cdot kip$$

$$Pw := \frac{Mo}{16\ ft} = 1.417\ kip$$

$$Pu := P = 9.34\ kip$$

$$Uplift := 0.9 \cdot Pu - Pw = 6.989\ kip$$

$$Tu := Uplift = 6.989\ kip$$

$$FuRod := 58\ ksi$$

$$db := \frac{3}{4}\ in$$

$$\phi Rn := 0.75 \cdot 0.75 \cdot FuRod \cdot \left(\frac{\pi}{4} \cdot db^2\right) = 14.413\ kip$$

$$\frac{Tu}{4} = 1.747\ kip$$

$:= L$

Anchor rods have enough strength, design OK