THE UNIVERSITY OF IOWA
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FINAL DELIVERABLE

Title	Manchester Fire Station Expansion
Completed By	Soe Htet, Francis Hart, Abbie Dirks, Benjamin Rowley
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UI Department	Civil & Environmental Engineering
Course Name	Project Design and Management - CEE: 4850:0001
Instructor	Paul Hanley
Community Partners	City of Manchester, Manchester Fire Department

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COLD STORAGE & PARKING LOT EXPANSION

400 E Main St, Manchester, IA 52057
Manchester, IA Fire Department

HHDR Consultants
University of Iowa
Department of Civil & Environmental Engineering

HHDR
CONSULTANTS

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<u>Section 1:</u> Executive Summary

The city of Manchester, Iowa requested the services of HHDR Consultants to design a new cold storage facility and parking lot for their fire department. HHDR Consultants is made up of four civil engineering students led by project manager Soe M. Htet. He has worked on structural design of the new storage building with team member Abbie Dirks. The site design was completed by team members Benjamin Rowley and Francis Hart. The initial team meeting for this project began on August 24, 2020 and a finished report, presentation, and poster was delivered on December 11, 2020.

Infrequently used equipment currently occupies space in the existing fire station that could be better utilized for everyday needs of the department. The new cold storage facility will serve as the primary location to store this equipment, including but not limited to a boat, a trailer, kayaks, and ice water and rescue equipment. The department initially proposed a 30'x30' storage facility in the northwest corner of the lot; however, this configuration would not have been adequate to meet their storage requirements. Two building size alternatives were considered and presented to the Manchester Fire Department as well as multiple locations corresponding with maximized parking designs. After discussion, the final building design of 36'x40' was chosen, and a location just north of the generator was selected in order to maximize parking.

The Manchester Fire Department initially considered adding a wash bay but ultimately decided against pursuing this option at this time due to economics and parking capacity constraints. However, design considerations were made to allow for the addition of a wash bay on the north end of the building should the department decide to add one in the future. The storage facility will need to be expanded to 36'x49' to fit a wash bay, and it should be noted that this larger design will reduce the amount of parking available by 2 stalls. Additionally, if the wash bay is added, a portion of the sidewalk and verge will need to be removed to accommodate an additional paved entrance into the parking lot. This is required so that firetrucks may enter and exit the wash bay without reversing or turning around.

The final design with no wash bay consists of a 36'x40' cold storage facility located just north of the existing generator and setback several feet further to the east than the existing fire station. This location maximized parking and allows for a total of 28 parking spaces including 12 spaces on the existing concrete, 14 new regular parking stalls, and 2 new ADA compliant parking spaces on the new concrete. Existing drains, located on the north end of the existing concrete and in the alleyway south of the parking lot, will collect runoff from concrete. The concrete slope has been designed to facilitate this.

The storage building utilizes double fan trusses readily available from local retailers. Said trusses were designed to withstand gravity and lateral loads appropriate for a storage facility of its nature in accordance with applicable building codes and design standards. 14' high and 8" thick reinforced concrete masonry unit (CMU) walls are responsible for supporting the roof trusses, providing lateral support, and transferring applied loads to the building's foundations. The floor of the facility is a sloped 8" thick slab-on-grade foundation with gutter openings for easy drainage. Cast-in-place continuous footings run the length of the building beneath the CMU walls to support the structure. Two 12' tall garage doors will be located on the east wall. These doors are 12' wide and 10' wide to allow easy access for the trailer and boat. Additionally, one standard entrance door will be placed on the south wall and three windows will be located on the west wall of the storage building. A 6'x6' utility closet is partitioned on the south-east corner for the electricity panel, water heater, and any other utility storage equipment. A sink will be located on the south wall near the standard entrance door and shelving for storage will be added to the south and west walls.

The storage building roof and walls will be insulated with blow-in cellulose insulation to achieve required R values for Climate Zone 6 in which Manchester, Iowa is located. Electricity and heating layouts have both been completed for the new storage building. The walls of the new building will be covered with brick façade to match the existing fire station. The interior walls of the storage building will be covered with fiber reinforced plastic which can be easily washed down by hose.

The main challenge faced during this project was striking a balance between maximum storage space and maximum parking spaces on site. An additional challenge was keeping the overall project costs to a minimum in order to make implementation of the designs feasible. Constraints for this project included restrictions on the building's placement location due to city codes and setbacks requirements. Finally, an oak tree in the northwest corner of the site required protection. No construction or changes to the site are to be implemented within a 15' radius of this tree in to protect the roots. Additionally, during construction, the tree and its protected radius should be roped off to avoid any heavy vehicles driving above and damaging the roots.

The site design was completed using Civil 3D and includes the locations of existing and future utilities, stormwater drainage, access roads and sidewalks. Similarly, the parking lot has been designed using Civil 3D as well to account for size, location, numbers of stalls, vehicle tracking, pavement type and thickness. The structural drawings of the cold storage facility were completed using Revit and includes the framing plans of the roof, floor, and foundations.

The site design totals $26,700 with the highest costs resulting from the pavement and utility connections. The cost of structural design for the cold storage facility is estimated at $55,800 with the walls and overhead doors accounting for highest cost items. Interior structural costs, such as interior finishes and electrical and heating components, are estimated to cost $13,400. The total construction cost was found to be $140,500 which includes overhead and profit as well as a multiplier for inflation from 2011 to 2020.

Contingency is estimated to be 10% of construction costs. Engineering and administration are estimated to be 20% of construction costs. This results in an added $14,100 for contingency and $28,100 for engineering and administration bringing the total project cost to $183,000.

<u>Section 2:</u> Organization and Qualifications

1. <u>Name of Organization</u>

   HHDR Consultants

2. <u>Organization Location and Contact Information</u>

   Soe M. Htet – *Project Manager*

   (347) 604 – 4099

   shtet@uiowa.edu

3. <u>Organization and Design Team Description</u>

   Soe M. Htet - *Project manager (Structural Engineering)*

   Francis Hart - *Civil designer (Management)*

   Abbie Dirks - *Structural designer (Structural Engineering)*

   Benjamin Rowley - *Civil designer (Transportation Engineering)*

<u>Section 3:</u> Proposed Services

1. <u>Project Scope</u>

The Manchester Fire Station requested that our team design a cold storage facility to add storage space for a boat, a trailer, snow and ice rescue equipment, kayaks, grills, tents, and other infrequently used equipment. A 36’x40’ one story structure was designed to accommodate these storage requirements. Special considerations have been made when sizing the overhead doors to ensure that the boat and trailer are able to access the facility. The design includes one walk-in standard size door located on the south side of the facility and two 12’ tall overhang garage doors on the east side of the facility for full accessibility. The garage doors are 10’ and 12’ wide. In addition, the client expressed an interest in installing a wash bay within the new facility. The wash bay component was ultimately removed from the design due to cost considerations. However, the site has reserved a sufficient area to allow for the possibility of a future wash bay addition.

<u>Site Design</u>

The site has been designed to connect utilities (water, electric, and sewer) from their existing locations to the new cold storage facility. A new parking lot has been designed that connects with the old parking lot with the principal goal of maximizing on-site parking. The final parking design without a wash bay has 28 parking spaces and the parking design for the site with a wash bay added has 27 parking spaces. For both designs, the total spaces include two ADA stalls. Parking lot and site design aspects follow specifications from the SUDAS Design and Specifications Manual. Parking standards and ADA compliance will be found in the Iowa DOT Design Manual.

<u>Storage Building Design</u>

Structural design for the 36’x40’ cold storage building was separated into three main phases: roof design, wall design, and foundation design. The roof framing consists of 36’ 4/12 double fan trusses. 8” CMU walls support the roof framing and provide lateral load resistance. The foundation consists of continuous footings with slab-on-grade flooring. Two 12’ tall garage doors will be located on the east wall of the storage building. These doors are 10’ and 12’ wide to allow easy access for the trailer and boat. Additionally, one standard entrance door is placed on the south wall, one window is on the south wall, and three windows will be located on the west wall of the storage building. A 6’x 6’ utility closet is partitioned in the south-east corner of the building to store electricity panels, a water heater, and other utility storage equipment. A sink will be located on the south wall just west of the entry door while shelving for storage will be added to the south and west walls.

2. Work Plan

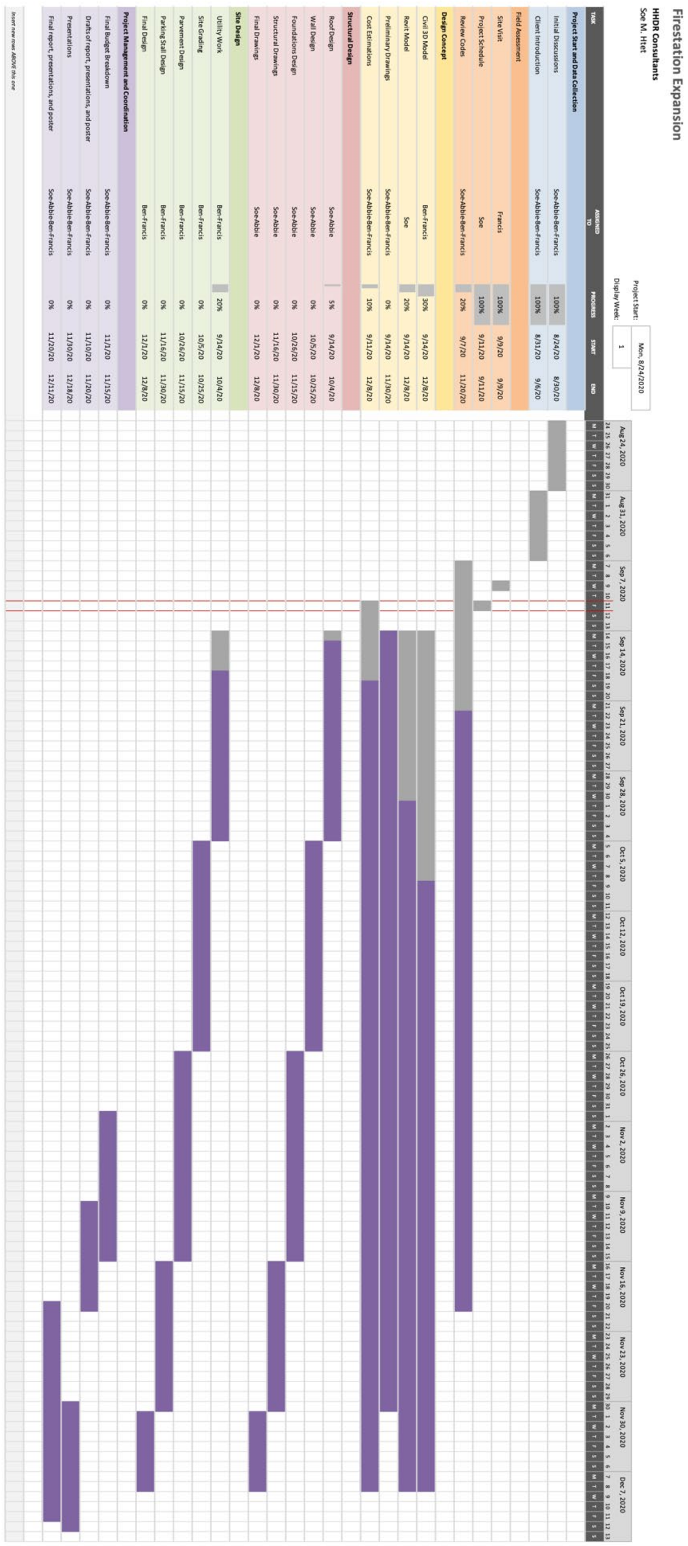

Figure 1: Fire Station Expansion Work Plan

1. Project Start and Data Collection
    i. Initial meetings with Professor Hanley and team to discuss progress and objectives
    ii. Introductory client contact meeting over Zoom

2. Concept Development and Field Assessment
    i. Site visit with client
    ii. Determine and report a project schedule
    iii. Review details of the current site and applicable codes

3. Design Concept Development
    i. Civil 3D model of the current site and parking lot
    ii. Revit 3D model of storage building and alternative options
    iii. Preliminary drawings
    iv. Preliminary cost estimate

4. Structural Design
    i. <u>Roof Design</u>: Design of roof framing members using structural analysis software to comply with applicable design codes.
    ii. <u>Wall Framing Design</u>: Design of wall framing members using structural analysis software to comply with applicable design codes.
    iii. <u>Foundation Design</u>: Design of foundations and flooring using structural analysis software to comply with applicable design codes.
    iv. <u>Preliminary MEP Design</u>
    v. <u>Structural Drawings:</u> Drafting of appropriate structural drawings using Revit

5. Site Design
    i. <u>Drainage</u>
    ii. <u>Grading and Parking Pavement Elevations:</u> Design new concrete to line up with existing parking lot and alleyway elevations
    iii. <u>Parking Layout:</u> Maximize parking spaces on lot
    iv. <u>Utilities:</u> Connecting water, sanitary sewer, and electricity to the new storage building

6. Project Management and Coordination
    i. Determine cost of project and budget break down
    ii. Compose drafts of final report, presentations, and poster
    iii. Presentation of design
    iv. Compose final report, presentations, and poster

Project manager Soe Htet and team member Abbie Dirks split up work on structural design. Team members Benjamin Rowley and Francis Hart worked on site design. All team members took part in data collection, design concept development, and project management and coordination.

<u>Section 4:</u> Constraints, Challenges, and Impacts

1. <u>Constraints</u>

   This project was primarily constrained by the space available on site. The current fire station, the generator on the east side of the empty lot and a tree in the northwest corner all are fixed elements that could not be changed, restricting where building was acceptable on site. The current parking lot pavement was required to remain as is, therefore the new parking lot will be expanded off the existing lot. In addition, setback limits enforced by the city have been considered and limited expansion near edges of the lot.

2. <u>Challenges</u>

   The fire department requested storage space for several large objects including a boat, trailer, and wash bay. In order to accommodate all these elements, the storage building constructed needed to be relatively large. The department also requested that the parking capacity on site be maximized, however, space is limited on site. One challenged was balancing the amount of space allotted to parking and to the storage building in order to maximize both. A specific budget was not discussed, but since this project is supported by the community keeping project costs to a minimum was an important consideration at every step of the process. The size of the cold storage facility was limited by removing the wash bay in order to make implementation of the project realistic. A large portion of the existing sidewalk is greatly damaged and will need to be repaved. Finally, our team was asked to create the parking lot so that it was as accessible as possible for snowplows. Our design allows for easy access to the majority of the lot, although it may still be difficult to clear the site between the new storage building, generator, and existing fire building. This was a comprise made to allow for more parking and storage.

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

   The addition of a storage building will give the fire department easy access to equipment and improve overall organization, which in turn will have a positive impact on the community by allowing the department to respond to calls as quickly as possible. The parking on site is sometimes used for nearby community events, therefore, expanding the parking will positively impact community members who choose to park here. The church next door was specifically mentioned as a community who uses the lot for overflow parking, and this expansion will benefit them. Another advantage of an expanded parking lot is that extra pavement provides additional space for outdoor storage and room for outdoor work such as cleaning of equipment when cars are not using the space.

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<u>Section 5:</u> Alternative Designs That Were Considered

Two alternatives for storage buildings were discussed with Manchester at our second meeting. Several locations for the storage facility with corresponding parking lot options were explored as well. One such location considered was adjacent to and north of the generator and current fire station, slightly set back from the west property line. This location allowed for more parking spaces but made snow removal more difficult between the existing building, generator, and new storage building. Another alternative location considered was along the west property line setback to be parallel with the current Manchester Fire Department building. This option resulted in fewer parking spaces and made the possibility a future wash bay more difficult to implement but would have made snow removal simpler near the generator.

The first building size presented to Manchester was a 36'x36' building. This was the original footprint proposed during the project kickoff and would have provided enough space for a boat, a trailer as well as limited shelving. This option was determined to be unsatisfactory and failed to meet all the storage requirements requested.

The second building size presented had a larger dimension of 36'x49'. This option provided enough space for a wash bay in addition to the storage space for the equipment mentioned previously. This option was the best fit for the fire departments storage needs. However, it would have cost more and reduced the area available for parking.

After a second meeting with the Manchester Fire Department and discussing the available options, it was concluded that the 36'x40' storage facility was the most ideal option. The extra 4' length would allow for additional storage shelves without sacrificing parking capacity. Ultimately, it was decided that a wash bay was too expensive to include at this time but may be an addition the department could reconsider at a future date. The first location discussed, closer to the generator and slightly set back from the west property setback line, was therefore chosen. This location allowed for maximizing the parking capacity and made the possibility of a future wash bay easier to implement.

<u>Section 6:</u> Final Design Details

Site Design:

<u>Paving and elevations</u>

A new parking lot has been designed to line up with the existing lot in order to increase parking capacity. This can be accomplished by clearing the existing ground then preparing and compacting a 6" subgrade using native materials. A 6" granular subbase will then sit on top of the compacted subgrade. Finally, the subbase will be topped with a 6" layer of Portland Cement Concrete (PCC) paving. The 6" PCC was determined by using a website called Pavement Designer to see the thickness that would be needed to support the loads. The thickness from Pavement Designer was compared to the minimum thickness required by SUDAS. Since the thickness determined by Pavement Designer was 3.75", it is under the SUDAS minimum so the minimum SUDAS thickness of 6" was used for the parking lot thickness. A parking surface drawing is available in the Project Drawing Set (P-1) and includes elevations taken at the top of pavement. The subbase and subgrade should be elevated accordingly. The east side and south side of the parking lot will match the existing elevation of the existing parking lot (east) and the existing alley (south).

<u>Drainage</u>

The parking lot will funnel all water that falls onto the pavement centrally and then to one of the existing storm water intakes on both the south and east sides of the pavement. The parking lot is designed into two sections, a top half, and a bottom half. The top half is designed so water slopes from the north and south to meet in the middle and then flows to the east towards the existing intake. This top half is designed to have a slope of 1.5%. Runoff calculations were performed using the Rational Method and using chapter 3 of the Iowa Storm Water Management Manual. The 5-Yr and 25-Yr peak runoff flows were 0.31 ft^3/s and 0.48 ft^3/s, respectively. The bottom half is designed so the water slopes from the east and west to meet in the middle and then flows south towards an existing storm intake which is in the alley. This bottom half is designed to have a slope of 3.5%. The bottom half of the parking lot was calculated using the same method as above. The 5-Yr and 25-Yr peak runoff flows were 0.16 ft^3/s and 0.25 ft^3/s, respectively. The slope of the top half and bottom half don't meet the same slope because the existing alley and parking lot are at different elevations so in order to have the proposed lot meet existing, the slopes of the proposed lot will have to vary as shown in the Project Drawing Set (P-1).

<u>Parking layout</u>

A parking space striping layout is also included in the Project Drawing Set (P-2). The new parking lot design without a wash bay will allow for 12 parking spaces on the original pavement, 14 additional regular spaces on the new pavement, and 2 ADA accessible spaces also on the new pavement. According to SUDAS, with less than 25 parking stalls, there should be 1 ADA accessible parking stall. One additional stall than was necessary was added to create more ADA accessibility. If a wash bay is added there will be room for the same 12 original parking spaces on the old pavement as well as 13 new regular spaces and 2 new ADA spaces on the new concrete. This results in a total of 28 parking spaces without a wash bay and 27 if a wash bay is added.

<u>Utilities</u>

Both the existing sanitary sewer and water service utility lines have been designed to connect to the new storage facility. There will be a new 6” PVC sanitary sewer that connects from the storage building to the existing 8” sanitary sewer in the alley. This pipe will run north to south as the new storage facility is located north of the existing sanitary line. On the north end of the storage building there will be a clean out added for a potential drain if a wash bay is added in the future. In the middle of the storage building a floor drain will be included and pavement slab will be laid at a 0.5% slope to direct water towards this central drain. The water service will run from the north side of the property, as shown on the Proposed Utilities Drawing in the Project Drawing Set (P-5 and P-6), down to the south side of the storage building. The service will connect to the existing pipe at a gate valve and will be a ¾” PVC pipe that ends at the sink located on the south wall of the building. There is no new proposed storm sewer since the pavement should allow the water to flow to the existing intakes in the parking lot and alley. More information in Appendix A.

Storage Building Design:

Structural components were calculated using structural analysis programs and are designed to comply with local building ordinances, IBC, ASCE, ACI, AISC and all other applicable standards and codes. 3D renderings of the proposed structure are provided for reference and structural drawing have been drafted as part of the final deliverable.

Roof

A hip and gable roof with an east-west orientation spanning 36’ and sloped at a pitch of 4:12 was chosen in order to match the aesthetics of existing fire station and to maintain a low profile within the neighborhood. Double fan wooden trusses were selected as the primary roof framing member over steel alternatives due to their relative ease of availability and cost effectiveness for a facility of this size and loading. Due to wide variability in species, grading and sizing of truss components amongst manufacturers, all truss calculations presented assumed the use of 2”x8” No.1 Douglas Fir Larch for bottom chord, top chord, and web members. Said truss members were modeled in RISA 2D as pinned simply supported beams and subjected to the appropriate gravity and lateral load criteria. The results of the analysis were used to ensure the truss members met the required bending, shear and bearing resistance in accordance with NDS 2018. Design calculations for the truss can be reviewed on pages 44 through 47 of Appendix C. The trusses themselves are spaced 4’-0” OC across the 40’ north-south span of the facility. CFR insulated steel metal roof panels will be implemented for weathertight roof performance. An insulation R value of 49 was calculated as necessary for roof insulation in Manchester, Iowa which is located in Climate Zone 6 according to the US Department of Energy (see Appendix B Figure 5 and Figure 6). 42” wide panels with 6” deep insulation will provide an R value of 49.75 which is greater than the requirement of 49. These panels use vertical side seaming and a lock and groove system to reduce joints that need to be sealed and make installation simple. The panels are predrilled for fasteners so that they can be easily screwed onto the wooden truss roof system below.

Walls

8”x8”x16” grouted CMU blocks made up the building’s exterior walls. A brick façade is applied to the exterior to match the aesthetic of the existing fire station. The exterior walls extend 14’ above grade, 1’-6” below grade and are designed using NWC (w = 150 PCF) with a compressive strength of 4000 PSI. In accordance with Ch. 11 ACI 318-19, 60 grade #4 vertical reinforcement bars placed 1’-6” OC and 60 grade #4 horizontal reinforcement bars placed 1’-0” OC are provided for shear and flexural resistance. The reinforced CMU was determined to meet the required shear, bending, axial and slenderness ratio requirements as recommended by ACI 318-19. These calculations can be reviewed on pages 55 to 57 of Appendix D.

Prefabricated steel lintels were selected to support the load bearing on the windows and doors. Three separate lintel designs have been calculated for the 12’ wide garage door, the 10’ wide garage door, and the standard size entrance door and windows. Dead load calculations for bearing on these lintels can be viewed on pages 57-62 of Appendices D. Drawings of these lintels can be reviewed on S-09. Lintels will sit on steel plates above doors and windows and be anchored to the CMU walls. Dead loads bearing on the lintels were checked to be sufficiently smaller than the acceptable total ultimate dead load allowed for each prefabricated lintel. Lengths of lintels were calculated to be greater than the length of the clear span plus two times the height of the lintel according to the AISC 15th Edition Steel Design Manual. The 12’ wide opening uses a lintel with length of 13.8 ft. The 10’ garage opening will be fitted with a lintel that is 13.3’ long. Finally, the standard sized entry door and 4 standard sized window gaps will be fitted with lintels that are 6.9’ long.

Fiber reinforced plastic (FRP) is used for the interior finish for its water resistance and ability to be hosed down for cleaning purposes. According to the US Department of Energy walls in Climate Zone 6 require insulation R values of 13, see Appendix B Figure 5 and Figure 6. It is most cost effective to install a 3.5” thick layer of blown in cellulose insulation alongside a 0.9” thick sheet of FRP rather than exclusively using FRP. The insulation from the blown in cellulose will result in an insulation R value of 13.3, greater than the requirement of 13.

Foundation and Footings

Continuous footings were chosen to act as the foundations for the structure. The footings are designed to be cast-in-place using NWC (wc = 150 PCF) with a compressive strength f’c of 4000 PSI. To accommodate for a soil bearing capacity of 100 kN/m^2, the footings are 1’-6” thick, 4’ wide and run the entire length of the structure beneath the walls. In accordance with ASCE 7-16, the footings were designed to meet bearing and uplift requirements. 60 grade #4 rebars spaced 10’ along the transverse direction and 22 60 grade #8 rebars along the span of the footings are provided as recommended by ACI 318-19. A sloped 8” thick concrete slab is designed to rest directly above the grade to serve as flooring. This is in accordance with ACI 318 bearing strength requirements which can be viewed in Table 8 of Appendix B. A channel located at the center of the slab running along the north-south span accommodates the gutter.

Interior Building Design:

Electrical Design and Layout

The storage building was designed to maximize the productivity of those working inside it. Twenty-two 15-amp duplex receptacle outlets were chosen to be positioned across all 3 walls, inside the utility closet, in between the overhead doors and in the ceiling centered with the overhead doors. The 6 outlets positioned on the north wall reside on one 15-amp circuit breaker, the 6 outlets positioned on the west wall reside on one 15-amp circuit breaker, and 6 outlets positioned on the south wall and in the utility closet reside on another 15-amp circuit breaker. The final 4 outlets, two between the overhead doors and one for each overhead door, remain on one 15-amp circuit breaker. US National Electrical Code, Section 210.52, calls for outlets to be no more than 6 feet apart and the outlets designed in the storage building are spaced every 6 feet, except in the corners of the building. Two 100-amp electrical panels will be required to power the storage building. This takes into account everything in the current plan, while also leaving a buffer for future growth. A visual representation of the aforementioned electrical design and layout is included in the Project Drawing Set, drawing E-1.

Lighting Fixture Layout

Three columns of 3 LED tube lights were chosen to illuminate the space. Visual 3D (see Appendix G - Bibliography) was used to select the specific model of light fixture, EMS L48 6000LM IMACD MD 80CRI 40K. Each column of light fixtures is controlled by their own switch. There are two locations for these switches, one set is located on the south wall and the second set is located between the overhead doors. All exterior walls will be illuminated by 12.5" LED dusk-to-dawn soffit lights, 4 on each side spaced 10' apart on the east and west walls, and 9' apart on the north and south walls. The two sets of switches to control these exterior lights will be located in the northeast corner of the storage building and just inside the door on the south wall. A visual representation of the aforementioned lighting fixture layout is included in the Project Drawing Set, drawing E-2.

Heating, Ventilation

A forced air furnace is to be installed in the utility closet. This furnace will exhaust to the exterior through a short PVC pipe. Duct work will connect to the furnace to transfer warm air throughout the building and for general circulation. A tankless water heater will also be installed in the same utility closet, then being connected to the sink. The tankless heater is designed to handle 1-2 bathrooms, so will prove sufficient for the storage building. The main focus of the heating design is to keep everything from freezing. Those working inside the storage building will not do so at extended periods of time so it was deemed not necessary to heat the interior to that of a residential building. A visual representation of the aforementioned heating and ventilation layout is included in the Project Drawing Set, drawing HV-1.

Section 7: Engineer's Cost Estimate

Both a basic construction estimate and an estimate with this construction cost plus overhead and profit increased for inflation from 2011 to 2020 were calculated. Costs for the project can be separated into three main categories site design, structural design, and interior building costs such as heating and electricity. Site design costs include grading and utility work and lines. The largest costs from structural design are due to CMU walls, wood trusses, and roofing materials. These three categories together total a cost of $98,000. When overhead and profit are added in this results in a total construction cost of $121,000 in 2011 and jumps to $140,500 in 2020 after inflation is factored in. A breakdown of individual costs per item can be viewed in Figure 2 below. All individual costs were found and calculated using RS Means. RS Means rounding standards were applied to our costs and total cost estimations and can be viewed in Table 3.

After this total construction cost was calculated allowances for contingency as well as engineering and administration were included. 10% of the 2020 construction cost with overhead and profit was added as contingency and 20% of this cost was added for engineering and administration. A total of $14,100 was added for contingency and $28,100 was added for engineering and administration. This brings the total project cost to $183,000.

Qty	Line Number	Description	Unit	Extended Total	Extended Total O&P
1.00	220576100180	Cleanout, floor type, round or square, scoriated nickel bı	Ea.	$424.00	$517.00
11.00	054413600270	Roof truss, using galv LB metal studs, fink (W) or King F	Ea.	$2,508.00	$3,179.00
127.00	312323142020	Backfill, structural, common earth, 80 H.P. dozer, 50' hau	L.C.Y.	$118.11	$157.48
80.00	077123300400	Aluminum gutters, stock units, enameled, 5" box, .032"	L.F.	$538.40	$705.60
12.00	265113508610	Accent lights, interior, 0.5 W low voltage incandescent, c	L.F.	$371.88	$416.52
98.00	331113253960	Water supply distribution piping, polyvinyl chloride press	L.F.	$69.58	$98.00
70.00	333113252040	Public Sanitary Utility Sewerage Piping, piping polyvinyl	L.F.	$441.70	$566.30
28.00	077123100700	Aluminum downspouts, round, corrugated, 4" diameter,	L.F.	$151.76	$201.04
2128.00	097733100030	Fiberglass Reinforced Plastic Panels, on walls, adhesive ı	S.F.	$5,511.52	$7,043.68
1440.00	033053404840	Structural concrete, in place, slab on grade (3500 psi), c	S.F.	$5,011.20	$5,990.40
6600.00	033053404820	Structural concrete, in place, slab on grade (3500 psi), c	S.F.	$18,084.00	$21,978.00
2128.00	042210420300	Concrete masonry unit (CMU), screen block, 2000 psi, 8	S.F.	$18,917.92	$25,833.92
1440.00	061210100570	Structural insulated panels, 7/16" OSB both sides, straw	S.F.	$15,681.60	$17,668.80
644.44	312216100011	Fine grading, finish grading granular subbase for highwa	S.Y.	$302.89	$393.11
12.00	265113100360	Fixture hangers, flexible, 1/2" diameter, 6" long	Ea.	$543.60	$750.00
1.00	331113254170	Water supply distribution piping, fitting, 90 degree elbov	Ea.	$6.32	$8.79
1.00	331113254340	Water supply distribution piping, fitting, tee, class 200 p	Ea.	$9.88	$13.66
2.00	083613102300	Doors, overhead, commercial, stock, fiberglass and alum	Ea.	$6,170.00	$7,160.00
1.00	019313151883	Clean single sink	Ea.	$10.11	$15.39
1.00	333113253080	Public Sanitary Utility Sewerage Piping, piping polyvinyl	Ea.	$66.50	$93.50
12.00	052119106400	Individual steel bearing plate, 6" x 6" x 1/4", with J-hool	Ea.	$106.92	$128.28
1.00	333113253160	Public Sanitary Utility Sewerage Piping, tees, polyvinyl cl	Ea.	$125.00	$168.50
1.00	221319140400	Drain, floor receptor, 12-1/2" square top, 25 square inch	Ea.	$176.00	$193.00
1.00	331213154100	Water Service Connection, ductile iron, cement lined, 6"	Ea.	$257.00	$385.00
644.44	312216101050	Fine grading, fine grade for small irregular areas, to 15,(	S.Y.	$1,205.10	$1,585.32
240.00	031113550020	C.I.P. concrete forms, mat foundation, plywood, 1 use, i	SFCA	$2,253.60	$3,201.60
11.00	260519550150	Non-metallic sheathed cable, copper with ground wire, 6	C.L.F.	$1,919.50	$2,739.00
1.00	331216103810	Water Utility distribution Valves, gate valves, cast iron, n	Ea.	$658.50	$786.00
22.00	260590104015	Receptacle devices, resi, duplex outlet, ivory, type NM ca	Ea.	$805.20	$1,122.00
4.00	080153810140	Windows, solid vinyl replacement, casement, insulated g	Ea.	$1,192.00	$1,460.00
7.00	034843401250	Precast lintel, 5' to 12' long, 8" wide, 8" high, 12' long, s	Ea.	$1,564.50	$1,918.00
180.00	032105750100	Splice rebar, standard, self-aligning type, taper threaded	Ea.	$1,962.00	$2,754.00
120.00	032105750120	Splice rebar, standard, self-aligning type, taper threaded	Ea.	$2,010.00	$2,790.00
10.00	262413400160	Circuit breakers, 1 pole, 240 V, 15 to 60 amp, FA frame,	Ea.	$1,655.00	$2,005.00
1520.00	053123503350	Metal roof decking, steel, open type N wide rib, galvaniz	S.F.	$3,465.60	$4,240.80
1.00	235413101040	Furnace, hot air heating, blowers, electric, 17.1 MBH, U.	Ea.	$692.00	$841.00
1	000000000001	Water heater, instantaneous, electric, point of use (6.6 C	Ea.	$580.00	$116.00
12.00	000000000002	Motion Sensor Dusk to Dusk LED Flushmount Button Lig	Ea.	$1,080.00	$359.88
1.00	081116100030	Doors & Frames, aluminum, entrance, narrow stile, clea	Ea.	$1,130.00	$1,500.00
				$97,776.89	$121,083.57

Figure 2: RS Means Cost Estimation (2011 Prices)

Table 1: Construction Costs

Category	Items	Cost
Structural Design	Walls	$36,700
	Windows and Doors	$8,500
	Foundation, Floor, and Drain	$6,425
	Roof	$4,200
	Total Structural Costs	**$55,800**
Site Design	6" Pavement	$23,000
	Utilities	$2,075
	Grading	$1625
	Total Site Costs	**$26,700**
Interior Structural Costs	Electrical	$6,525
	Interior Finishes	$5,525
	Heating	$1,325
	Total Interior Costs	**$13,400**
Total Construction Cost		**$98,000**
Construction Cost Including Overhead and Profit (2020)		**$140,500**

Table 2: Total Project Cost

Category	Cost
Construction Cost Including Overhead and Profit (2020)	$140,500
Contingency	$14,100
Engineering and Administration	$28,100
Total Project Cost	**$183,000**

Table 3: RS Means Rounding Standards

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

<u>Section 8:</u> Proposal Attachments

Appendix A: Site Design Calculations and References

Pavement Standards:

Figure 3: Pavementdesigner.org Screenshot of Parking Lot Requirement Results

Pavement Designer website did not reach the minimum requirements that were specified in SUDAS so the pavement thickness was altered according to SUDAS Design Manual. Pavement was designed with a 6" subbase and 6" subgrade for the equivalence of the 12" subgrade. That is why the desirable value of a 6" pavement was chosen.

Table 3: SUDAS Table 8B-1.03: Pavement Thickness for Light Loads

Table 8B-1.03: Pavement Thickness for Light Loads
(Parking lots with 200 or less cars/day and/or 2 or less trucks/day or equivalent axle loads)

Subgrade CBR	Surface Material	On 12" of Prepared Subgrade		On 12" of Prepared Subgrade with 4" Granular Subbase	
		Minimum	*Desirable*	*Minimum*	*Desirable*
9	Rigid	5"	6"	4"	5"
	Flexible	5"	6"	4"	5"
6	Rigid	5"	6"	4"	5"
	Flexible	5"	6"	4"	5"
3	Rigid	5"	6"	4"	5"
	Flexible	6"	6"	5"	5"

Parking Standards:

Most of the design criteria for the parking lot and parking stalls was found in SUDAS Design Manual, Chapter 8. For the parking stalls, according to SUDAS Design Manual Chapter 8B-part C.1, the Facility Type is for "Low turnover" which requires a stall width of 8'-6".

Table 4: SUDAS Table 8B-1.01 Recommended Minimum Widths for Parking Stalls

Table 8B-1.01: Recommended Minimum Widths for Parking Stalls

Facility Type	Width
Low turnover (employees, students, etc)	8'-6"
Moderate to high turnover (retail, medical facilities, etc.)	9'-0"

Source: Urban Land Institute, National Parking Association

The parking dimensions for all the spots were done using Vehicle Tracking from Civil 3D which were up to SUDAS standards for the different parking angles.

Table 5: SUDAS Table 8B-1.02: Minimum Parking Dimensions

Table 8B-1.02: Minimum Parking Dimensions

Parking Lot Dimension				Parking Angle (θ)				
				Two-way Aisle			One-way Aisle	
				90°	60°	45°	60°	45°
Stall Projection			SP	18'-0"	15'-7"	12'-9"	15'-7"	12'-9"
Aisle Width			A	24'-0"	25'-10"	29'-8"	20'-4"	21'-6"
Base Module			M_1	60'-0"	57'-0"	55'-2"	51'-6"	47'-0"
Single Loaded Module			M_2	42'-0"	39'-0"	37'-7"	32'-6"	29'-5"
Wall to Interlock			M_3	60'-0"	55'-10"	52'-2"	49'-4"	44'-0"
Interlock to Interlock			M_4	60'-0"	53'-8"	49'-2"	47'-2"	41'-0"
Overhang			o	2'-6"	2'-2"	1'-9"	2'-2"	1'-9"
Stall Width	8'-6"	Width Projection	WP	8'-6"	9'-10"	12'-0"	9'-10"	12'-0"
		Interlock	i	0'-0"	2'-2"	3'-0"	2'-2"	3'-0"
	9'-0"	Width Projection	WP	9'-0"	10'-5"	12'-9"	10'-5"	12'-9"
		Interlock	i	0'-0"	2'-3"	3'-2"	2'-3"	3'-2"

The number of parking spaces in the new lot are determined based off of the design that is chosen by the client. All options are under 25 parking spaces. The number of accessible spaces is based off of SUDAS Design Manual Chapter 8C-part B.2.

Table 6: SUDAS Table 8C-1.02: Minimum Accessible Parking Ratios

Table 8C-1.02: Minimum Accessible Parking Ratios

Total Number of Spaces Provided	Minimum Number of Accessible Spaces
1 to 25	1
26 to 50	2
51 to 75	3
76 to 100	4
101 to 150	5
151 to 200	6
201 to 300	7
301 to 400	8
401 to 500	9
501 to 1,000	2% of total
1,001 and over	20, plus 1 for each 100, or fraction thereof, over 1,000

For spaces between 1 to 25, the minimum number of accessible spaces is 1 and that space needs to be van accessible. 1 van accessible spot and 1 car accessible spot was deemed sufficient.

Water Service Placement:

Water mains and services need to be placed at a certain depth below the surface in order to be under the freeze line. According to SUDAS Design Manual Chapter 4C part B.6, the minimum depth of cover in Delaware County should be 5'.

Figure 4C-1.01: Minimum Depth of Cover for Water Main Installation

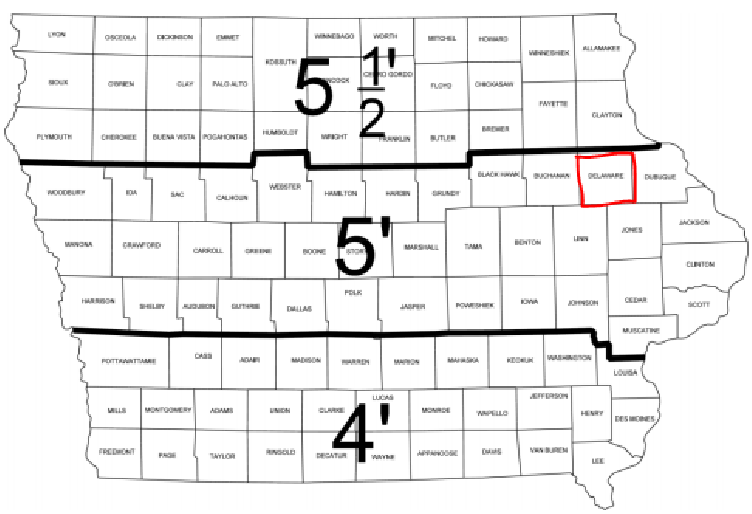

Figure 4: SUDAS Figure 4C-1.01 Minimum Depth of Cover for Water Main Installation

Sanitary Sewer Placement:

The separation of the water service and the sanitary sewer should be at least 18" from the top of the sanitary and the bottom of the water service as stated in SUDAS Design Manual Chapter 8C part G.1.

Appendix B: Structural Design References

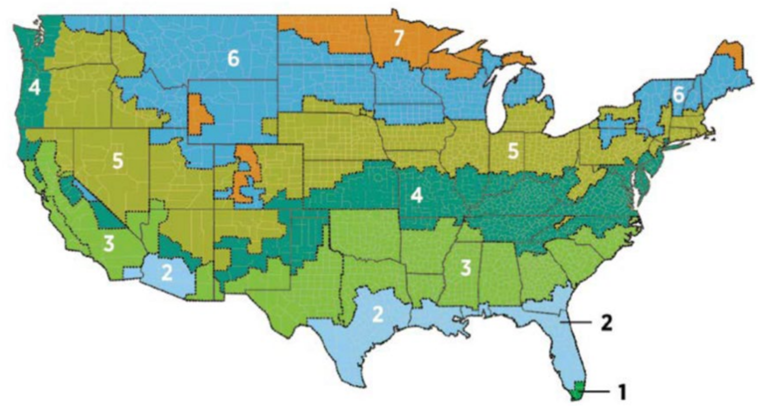

Figure 5: Insulation Climate Zones from Energystar.gov

Insulation R-Values for Location, Heat Type & Area*						
Location	**Heat Type**	**Attic**	**Wall**	**Floor**	**Crawl Space Wall****	**Basement Wall**
Zone 1	Natural Gas	38-49	13	13	13	11
	Oil Furnace	38-49	13	13	13	11
	Electric Furnace	38-49	13	13	13	11
	Electric Baseboard	38-49	13	13	13	11
	Heat Pump	38-49	13	13	13	11
	LPG Furnace	38-49	13	13	13	11
Zone 2	Natural Gas	38	13	13-19	13	11
	Oil Furnace	38	13	13-19	13-25	11
	Electric Furnace	38-49	13	19-25	25	11
	Electric Baseboard	38-49	13	13-25	13-25	11
	Heat Pump	38	13	13-19	13	11
	LPG Furnace	38-49	13	19-30	25	11
Zone 3	Natural Gas	30-38	13	13-19	13-25	11
	Oil Furnace	38	13	13-19	13	11
	Electric Furnace	38	13	13-19	13-25	11
	Electric Baseboard	38	13	13-19	13	11
	Heat Pump	30-38	13	13	13	11
	LPG Furnace	38-49	13	13-30	13-25	11
Zone 4	Natural Gas	38-49	13	25-30	25	11
	Oil Furnace	49	13	30	25	11
	Electric Furnace	38-49	13	25-30	25	25
	Electric Baseboard	49	13	30	25	11
	Heat Pump	38-49	13	13-25	13-25	11
	LPG Furnace	49	13	30	25	11-25
Zone 5	Natural Gas	38	13	25	25	11
	Oil Furnace	49	13	30	25	11-15
	Electric Furnace	49	13	30	25	25
	Electric Baseboard	49	13	30	25	11
	Heat Pump	38	13	30	25	11
	LPG Furnace	49	13	30	25	25
Zone 6-8	Natural Gas	49	13	30	25	25
	Oil Furnace	49	13	30	25	25
	Electric Furnace	49	13	30	25	25
	Electric Baseboard	49	13	30	25	25
	Heat Pump	49	13	30	25	25
	LPG Furnace	49	13	30	25	25

Figure 6: Insulation Climate Zone R-Value Requirements from Energystar.gov

Figure 7: Soil analysis for our site by the United States Department of Agriculture (USDA)

Table 7: Food and Agricultural Organization (FAO) of the United Nations Soil Bearing Capacities

Table 5.6 ***Soil Bearing Capacities***

Soil Type	**kN/m²**
Soft, wet, pasty or muddy soil	27 - 35
Alluvial soil, loam, sandy loam (clay +40 to 70% sand)	80 - 160
Sandy clay loam (clay +30% sand), moist clay	215 - 270
Compact clay, nearly dry	215 - 270
Solid clay with very fine sand	- 430
Dry compact clay (thick layer)	320 - 540
Loose sand	160 - 270
Compact sand	215 - 320
Red earth	- 320
Murram	- 430
Compact gravel	750 - 970
Rock	- 1700

Table 8: ACI Table A1.2: Allowable Distribution Loads

Table A1.2—Allowable distribution loads, unjointed aisles, uniform loading, and variable layout; PCA method

Slab thickness, in.	Working stress, psi	Critical aisle width, in.*	Allowable load, lb/ft² At critical aisle width	At other aisle widths 6 ft aisle	8 ft aisle	10 ft aisle	12 ft aisle	14 ft aisle
Subgrade $k = 50$ lb/in.³†								
5	300	5.6	610	615	670	815	1050	1215
	350	5.6	710	715	785	950	1225	1420
	400	5.6	815	820	895	1085	1400	1620
6	300	6.4	670	675	695	780	945	1175
	350	6.4	785	785	810	910	1100	1370
	400	6.4	895	895	925	1040	1260	1570
8	300	8.0	770	800	770	800	880	1010
	350	8.0	900	935	900	935	1025	1180
	400	8.0	1025	1070	1025	1065	1175	1350
10	300	9.4	845	930	855	850	885	960
	350	9.4	985	1085	1000	990	1035	1120
	400	9.4	1130	1240	1145	1135	1185	1285
12	300	10.8	915	1065	955	915	925	965
	350	10.8	1065	1240	1115	1070	1080	1125
	400	10.8	1220	1420	1270	1220	1230	1290
14	300	12.1	980	1225	1070	1000	980	995
	350	12.1	1145	1430	1245	1170	1145	1160
	400	12.1	1310	1630	1425	1335	1310	1330
Subgrade $k = 100$ lb/in.³†								
5	300	4.7	865	900	1090	1470	1745	1810
	350	4.7	1010	1050	1270	1715	2035	2115
	400	4.7	1155	1200	1455	1955	2325	2415
6	300	5.4	950	955	1065	1320	1700	1925
	350	5.4	1105	1115	1245	1540	1985	2245
	400	5.4	1265	1275	1420	1760	2270	2565
8	300	6.7	1095	1105	1120	1240	1465	1815
	350	6.7	1280	1285	1305	1445	1705	2120
	400	6.7	1460	1470	1495	1650	1950	2420
10	300	7.9	1215	1265	1215	1270	1395	1610
	350	7.9	1420	1475	1420	1480	1630	1880
	400	7.9	1625	1645	1625	1690	1860	2150
12	300	9.1	1320	1425	1325	1330	1400	1535
	350	9.1	1540	1665	1545	1550	1635	1795
Subgrade $k = 100$ lb/in.³†								
5	300	4.7	865	900	1090	1470	1745	1810
	350	4.7	1010	1050	1270	1715	2035	2115
	400	4.7	1155	1200	1455	1955	2325	2415
6	300	5.4	950	955	1065	1320	1700	1925
	350	5.4	1105	1115	1245	1540	1985	2245
	400	5.4	1265	1275	1420	1760	2270	2565
8	300	6.7	1095	1105	1120	1240	1465	1815
	350	6.7	1280	1285	1305	1445	1705	2120
	400	6.7	1460	1470	1495	1650	1950	2420
10	300	7.9	1215	1265	1215	1270	1395	1610
	350	7.9	1420	1475	1420	1480	1630	1880
	400	7.9	1625	1645	1625	1690	1860	2150
12	300	9.1	1320	1425	1325	1330	1400	1535
	350	9.1	1540	1665	1545	1550	1635	1795
	400	9.1	1755	1900	1770	1770	1865	2050
14	300	10.2	1405	1590	1445	1405	1435	1525
	350	10.2	1640	1855	1685	1640	1675	1775
	400	10.2	1875	2120	1925	1875	1915	2030
Subgrade $k = 200$ lb/in.³†								
5	300	4.0	1225	1400	1930	2450	2565	2520
	350	4.0	1425	1630	2255	2860	2990	2940
	400	4.0	1630	1865	2575	3270	3420	3360
6	300	4.5	1340	1415	1755	2395	2740	2810
	350	4.5	1565	1650	2050	2800	3200	3275
	400	4.5	1785	1890	2345	3190	3655	3745
8	300	5.6	1550	1550	1695	2045	2635	3070
	350	5.6	1810	1810	1980	2385	3075	3580
	400	5.6	2065	2070	2615	2730	3515	4095
10	300	6.6	1730	1745	1775	1965	2330	2895
	350	6.6	2020	2035	2070	2290	2715	3300
	400	6.6	2310	2325	2365	2620	3105	3860
12	300	7.6	1890	1945	1895	1995	2230	2610
	350	7.6	2205	2270	2210	2330	2600	3045
	400	7.6	2520	2595	2525	2660	2972	3480
14	300	8.6	2025	2150	2030	2065	2210	2480
	350	8.6	2360	2510	2365	2405	2580	2890
	400	8.6	2700	2870	2705	2750	2950	3305

*Critical aisle width equals 2.209 times the radius of relative stiffness.
†k of subgrade; disregard increase in k due to subbase.
Notes: Assumed load width = 300 in.; allowable load varies only slightly for other load widths. Allowable stress = 1/2 flexural strength.

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Appendix C: Structural Design Roof Calculations

1. Loading Criteria (Live & Dead)

Live Load

$L_r := 20\ psf$ — Upper Chord (Typical Roof Live Load)

$L := 0\ psf$

Dead Load

Upper Chord

$D_1 := 13\ psf$ — Light-frame wood roof + wood sheathing + 1/2" + asphalt shingle (3 psf)

Lower Chord

$W := 18\ ft$

$D_2 := 0.14\ \frac{psf}{in} \cdot 16\ in = 2.24\ psf$ — Insulation - Blown in cellulose

$D_3 := \frac{243\ lbf}{W \cdot 2 \cdot 4\ ft} = 1.688\ psf$ — Self weight (243lbs from Menards) @ 4' OC

$D_4 := 2\ psf$ — Gypsum board Ceiling

$D_5 := 1\ psf$ — Lighting and Electrical

Total Design Load

$S := 26.6\ psf$ — Reference Snow Loading

$W_{wind} := 19.5\ psf$ — Reference Wind Loading

$$D_{top} := \text{ceil}\left(\frac{D_1}{psf}\right) psf = 13\ psf$$

$$D_{bottom} := \text{ceil}\left(\frac{D_2 + D_3 + D_4 + D_5}{psf}\right) psf = 7\ psf$$

Top Chord

$D_{1top} := D_{top} = 13\ psf$

ASCE load combos

$D_{2top} := D_{top} + L = 13\ psf$

$D_{3top} := D_{top} + \max(L_r, S) = 39.6\ psf$

$D_{4top} := D_{top} + 0.75 \cdot L + 0.75\ \max(L_r, S) = 32.95\ psf$

$D_{5top} := D_{top} + 0.6\ W_{wind} = 24.7\ psf$

$D_{6top} := D_{top} + 0.75 \cdot L + 0.75\ (0.6\ W_{wind}) + 0.75\ \max(L_r, S) = 41.725\ psf$

$D_{7top} := 0.6\ D_{top} + 0.6\ W_{wind} = 19.5\ psf$

Bottom Chord

$D_{1bottom} := D_{bottom} = 7\ psf$

No Live, Roof Live or Snow Load

$\theta := 18.44\ deg$

4/12 Pitch

RISA Loading Criteria

Post Frame Truss placed @ 4' OC

2' - 9' OC allowed, what's optimal?

$w_{upper} := \max(D_{1top}, D_{2top}, D_{3top}, D_{4top}, D_{5top}, D_{6top}, D_{7top}) \cdot \cos(\theta) \cdot 4\ ft = 158.331\ plf$

$w_{upper} := \text{ceil}\left(\frac{w_{upper}}{plf}\right) plf = 159\ plf$

$w_{bottom} := D_{bottom} \cdot 4\ ft = 28\ plf$

$w_{bottom} := \text{ceil}\left(\frac{w_{bottom}}{plf}\right) plf = 29\ plf$

$w_{purlin} := (D_1 + S) \cdot \cos(\theta) \cdot 4\ ft = 150.267\ plf$

IOWA CIVIL & ENVIRONMENTAL ENGINEERING	Project				Job Ref.	
	Section				Sheet no./rev. 1	
	Calc. by S	Date 11/12/2020	Chk'd by	Date	App'd by	Date

SNOW LOADING

In accordance with ASCE7-16

Tedds calculation version 1.0.09

Building details

Roof type	Hip and gable
Width of roof (left on elevation)	b_1 = **18.00** ft
Width of roof (right on elevation)	b_2 = **18.00** ft
Slope of roof (left on elevation)	α_1 = **18.44** deg
Slope of roof (right on elevation)	α_2 = **18.44** deg

Ground snow load

Ground snow load (Figure 7.2-1)	p_g = **30.00** lb/ft^2
Density of snow	γ = min(0.13 * p_g / 1ft + 14lb/ft^3, 30lb/ft^3) = **17.90** lb/ft^3
Terrain typeSect. 26.7	B
Exposure condition (Table 7.3-1)	Partially exposed
Exposure factor (Table 7.3-1)	C_e = **1.00**
Thermal condition (Table 7.3-2)	All
Thermal factor (Table 7.3-2)	C_t = **1.00**
Importance category (Table 1.5-1)	I
Importance factor (Table 1.5-2)	I_s = **0.80**
Flat roof snow load (Sect 7.3)	p_f = 0.7 * C_e * C_t * I_s * p_g = **16.80** lb/ft^2

Warm roof slope factor (C_t <= 1.0)

Roof surface type	Slippery
Ventilation	Ventilated
Thermal resistance (R-value)	R = **30.00** °F h ft^2 / Btu
Roof slope factor - left Fig 7.4-1a (dashed line)	C_{s_l} = **0.79**
Roof slope factor - right Fig 7.4-1a (dashed line)	C_{s_r} = **0.79**

Hip and gable roof loads

Balanced sloped snow load - left (Cl.7.4)	p_{s_l} = C_{s_l} * p_f = **13.33** lb/ft^2
Balanced sloped snow load - right (Cl.7.4)	p_{s_r} = C_{s_r} * p_f = **13.33** lb/ft^2
Slope of left roof	S_l = 1 / tan(α_1) = **3.00**
Slope of right roof	S_r = 1 / tan(α_2) = **3.00**
Unbalanced load - left roof windward	p_{s_lw} = 0.3 * p_{s_l} = **4** lb/ft^2
Unbalanced load - right roof leeward	p_{s_rl} = p_{s_r} = **13.33** lb/ft^2
Length eaves to ridge for drift height	$l_{u\_ww\_l}$ = b_1 = **18.00** ft
Drift height	h_{dr_r} = min($\sqrt{(I_s)}$ * (0.43 * (max($l_{u\_ww\_l}$, 20 ft) * 1ft^2)$^{1/3}$ * (p_g / 1 lb/ft^2 + 10)$^{1/4}$ - 1.5 * 1ft), $\sqrt{(I_s * p_g * l_{u\_ww\_l} / (4 * \gamma))}$) = **1.28** ft
Rectangular surcharge to part leeward side	$p_{s\_rl\_sur}$ = h_{dr_r} * γ / $\sqrt{(S_r)}$ = **13.27** lb/ft^2
Length of rectangular surcharge	$l_{u\_rl\_sur}$ = min(8 / 3 * h_{dr_r} * $\sqrt{(S_r)}$, b_2) = **5.93** ft
Unbalanced load - left roof leeward	p_{s_ll} = p_{s_l} = **13.33** lb/ft^2
Unbalanced load - right roof windward	p_{s_rw} = 0.3 * p_{s_r} = **4** lb/ft^2
Length eaves to ridge for drift height	$l_{u\_ww\_r}$ = b_2 = **18.00** ft
Drift height	h_{dr_l} = min($\sqrt{(I_s)}$ * (0.43 * (max($l_{u\_ww\_r}$, 20 ft) * 1ft^2)$^{1/3}$ * (p_g / 1 lb/ft^2 + 10)$^{1/4}$ - 1.5 * 1ft), $\sqrt{(I_s * p_g * l_{u\_ww\_r} / (4 * \gamma))}$) = **1.28** ft

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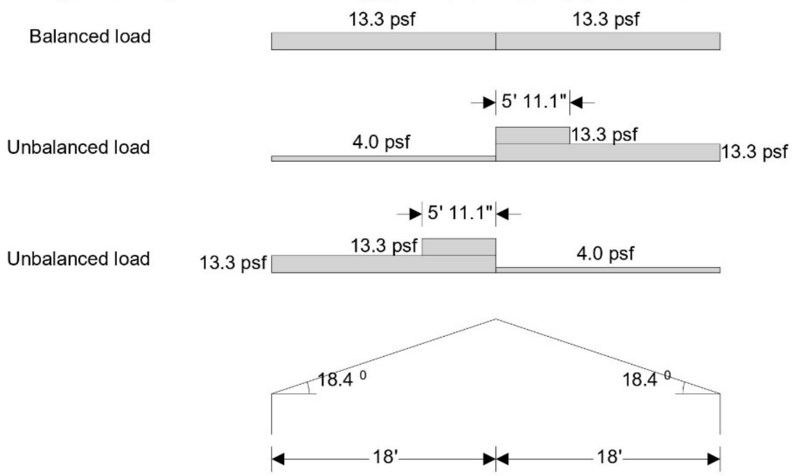
Rectangular surcharge to part leeward side $p_{s\_ll\_sur} = h_{dr\_l} * \gamma / \sqrt{(S_l)} = \mathbf{13.27}$ lb/ft^2

Length of rectangular surcharge $l_{u\_ll\_sur} = \min(8 / 3 * h_{dr\_l} * \sqrt{(S_l)}, b_1) = \mathbf{5.93}$ ft



Roof elevation

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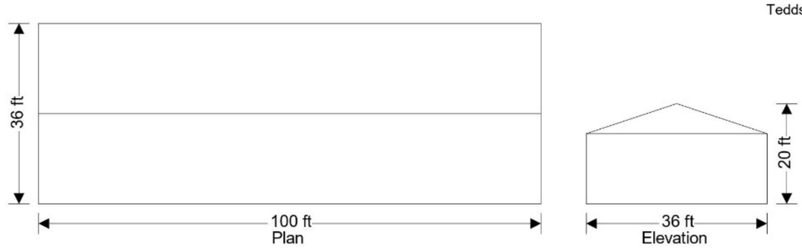
WIND LOADING

In accordance with ASCE7-16

Using the directional design method

Tedds calculation version 2.1.07



Building data

Type of roof	Gable
Length of building	b = **100.00** ft
Width of building	d = **36.00** ft
Height to eaves	H = **14.00** ft
Pitch of roof	α_0 = **18.4** deg
Mean height	h = **17.00** ft

General wind load requirements

Basic wind speed	V = **102.0** mph
Risk category	I
Velocity pressure exponent coef (Table 26.6-1)	K_d = **0.85**
Ground elevation above sea level	z_{gl} = **0** ft
Ground elevation factor	$K_e = \exp(-0.0000362 * z_{gl}/1ft)$ = **1.00**
Exposure category (cl 26.7.3)	B
Enclosure classification (cl.26.12)	Enclosed buildings
Internal pressure coef +ve (Table 26.13-1)	GC_{pi_p} = **0.18**
Internal pressure coef –ve (Table 26.13-1)	GC_{pi_n} = **-0.18**

Gust effect factor for rigid structures

Terrain exposure constants (Table 26.11-1)	
Integral length scale factor	l = **320.0** ft
Turbulence intensity factor	c = **0.30**
Minimum equivalent height	z_{min} = **30.0** ft
Peak factor for background response	g_Q = **3.400**
Peak factor for wind response	g_v = **3.400**
Integral length scale power law exponent	$\bar{\varepsilon}$ = **0.333**
Equivalent height of the structure	$\bar{z} = \max(0.6 * h, z_{min})$ = **30.00** ft
Intensity of turbulence (Eqn. 26.11-7)	$I_{\bar{z}} = c * (33\text{ ft} / \bar{z})^{1/6}$ = **0.30**
Integral length scale of turbulence (Eqn. 26.11-9)	$L_{\bar{z}} = l * (\bar{z} / 33\text{ ft})^{\bar{\varepsilon}}$ = **310.00** ft
Background response (Eqn. 26.11-8)	$Q = \sqrt{(1 / (1 + 0.63 * ((\min(B, L) + h) / L_{\bar{z}})^{0.63}))}$ = **0.910**
Gust effect factor (Eqn. 26.11-10)	$G = G_f = 0.925 * (1 + 1.7 * g_Q * I_{\bar{z}} * Q) / (1 + 1.7 * g_v * I_{\bar{z}})$ = **0.87**

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Minimum design wind loading (cl.27.4.7) p_{min_r} = **8** lb/ft^2

Topography

Topography factor not significant K_{zt} = 1.0

Velocity pressure equation $q = 0.00256 * K_z * K_{zt} * K_d * V^2 * 1psf/mph^2$

Velocity pressures table

z (ft)	K_z (Table 26.10-1)	q_z (psf)
14.00	0.57	12.90
15.00	0.57	12.90
15.00	0.57	12.90
17.00	0.59	13.36
20.00	0.62	14.04

Peak velocity pressure for internal pressure

Peak velocity pressure – internal (as roof press.) q_i = **13.36** psf

Pressures and forces

Net pressure $p = q * G_f * C_{pe} - q_i * GC_{pi}$

Net force $F_w = p * A_{ref}$

Roof load case 1 - Wind 0, GC_{pi} 0.18, $-c_{pe}$

Zone	Ref. height (ft)	Ext pressure coefficient c_{pe}	Peak velocity pressure q_p (psf)	Net pressure p (psf)	Area A_{ref} (ft^2)	Net force F_w (kips)
A (-ve)	17.00	-0.48	13.36	-7.98	1897.42	-15.15
B (-ve)	17.00	-0.57	13.36	-9.03	1897.42	-17.13

Total vertical net force $F_{w,v}$ = **-30.62** kips

Total horizontal net force $F_{w,h}$ = **0.63** kips

Walls load case 1 - Wind 0, GC_{pi} 0.18, $-c_{pe}$

Zone	Ref. height (ft)	Ext pressure coefficient c_{pe}	Peak velocity pressure q_p (psf)	Net pressure p (psf)	Area A_{ref} (ft^2)	Net force F_w (kips)
A	14.00	0.80	12.90	6.60	1400.00	9.24
B	17.00	-0.50	13.36	-8.23	1400.00	-11.52
C	17.00	-0.70	13.36	-10.56	612.03	-6.46
D	17.00	-0.70	13.36	-10.56	612.03	-6.46

Overall loading

Projected vertical plan area of wall $A_{vert\_w\_0} = b * H =$ **1400.00** ft^2

Projected vertical area of roof $A_{vert\_r\_0} = b * d/2 * \tan(\alpha_0) =$ **600.18** ft^2

Minimum overall horizontal loading $F_{w,total\_min} = p_{min\_w} * A_{vert\_w\_0} + p_{min\_r} * A_{vert\_r\_0} =$ **27.20** kips

Leeward net force $F_l = F_{w,wB} =$ **-11.5** kips

Windward net force $F_w = F_{w,wA} =$ **9.2** kips

Overall horizontal loading $F_{w,total} = \max(F_w - F_l + F_{w,h}, F_{w,total\_min}) =$ **27.2** kips

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Roof load case 2 - Wind 0, GC_{pi} -0.18, $-0c_{pe}$

Zone	Ref. height (ft)	Ext pressure coefficient c_{pe}	Peak velocity pressure q_p (psf)	Net pressure p (psf)	Area A_{ref} (ft^2)	Net force F_w (kips)
A (+ve)	17.00	-0.03	13.36	2.00	1897.42	3.80
B (+ve)	17.00	-0.57	13.36	-4.22	1897.42	-8.01

Total vertical net force $F_{w,v}$ = **-4.00** kips

Total horizontal net force $F_{w,h}$ = **3.74** kips

Walls load case 2 - Wind 0, GC_{pi} -0.18, $-0c_{pe}$

Zone	Ref. height (ft)	Ext pressure coefficient c_{pe}	Peak velocity pressure q_p (psf)	Net pressure p (psf)	Area A_{ref} (ft^2)	Net force F_w (kips)
A	14.00	0.80	12.90	11.41	1400.00	15.97
B	17.00	-0.50	13.36	-3.42	1400.00	-4.79
C	17.00	-0.70	13.36	-5.75	612.03	-3.52
D	17.00	-0.70	13.36	-5.75	612.03	-3.52

Overall loading

Projected vertical plan area of wall $A_{vert\_w\_0} = b * H =$ **1400.00** ft^2

Projected vertical area of roof $A_{vert\_r\_0} = b * d/2 * \tan(\alpha_0) =$ **600.18** ft^2

Minimum overall horizontal loading $F_{w,total\_min} = p_{min\_w} * A_{vert\_w\_0} + p_{min\_r} * A_{vert\_r\_0} =$ **27.20** kips

Leeward net force $F_l = F_{w,wB} =$ **-4.8** kips

Windward net force $F_w = F_{w,wA} =$ **16.0** kips

Overall horizontal loading $F_{w,total} = \max(F_w - F_l + F_{w,h}, F_{w,total\_min}) =$ **27.2** kips

Roof load case 3 - Wind 90, GC_{pi} 0.18, $-c_{pe}$

Zone	Ref. height (ft)	Ext pressure coefficient c_{pe}	Peak velocity pressure q_p (psf)	Net pressure p (psf)	Area A_{ref} (ft^2)	Net force F_w (kips)
A (-ve)	17.00	-0.90	13.36	-12.89	322.58	-4.16
B (-ve)	17.00	-0.90	13.36	-12.89	322.58	-4.16
C (-ve)	17.00	-0.50	13.36	-8.23	645.16	-5.31
D (-ve)	17.00	-0.30	13.36	-5.90	2504.53	-14.77

Total vertical net force $F_{w,v}$ = **-26.94** kips

Total horizontal net force $F_{w,h}$ = **0.00** kips

Walls load case 3 - Wind 90, GC_{pi} 0.18, $-c_{pe}$

Zone	Ref. height (ft)	Ext pressure coefficient c_{pe}	Peak velocity pressure q_p (psf)	Net pressure p (psf)	Area A_{ref} (ft^2)	Net force F_w (kips)
A_1	15.00	0.80	12.90	6.60	537.00	3.54
A_2	15.00	0.80	12.90	6.60	0.00	0.00
A_3	20.00	0.80	14.04	7.39	75.03	0.55
B	17.00	-0.26	13.36	-5.45	612.03	-3.33

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Zone	Ref. height (ft)	Ext pressure coefficient c_{pe}	Peak velocity pressure q_p (psf)	Net pressure p (psf)	Area A_{ref} (ft^2)	Net force F_w (kips)
C	17.00	-0.70	13.36	-10.56	1400.00	-14.78
D	17.00	-0.70	13.36	-10.56	1400.00	-14.78

Overall loading

Projected vertical plan area of wall $A_{vert\_w\_90} = d * H + d^2 * \tan(\alpha_0) / 4 = \mathbf{612.03}\ ft^2$

Projected vertical area of roof $A_{vert\_r\_90} = \mathbf{0.00}\ ft^2$

Minimum overall horizontal loading $F_{w,total\_min} = p_{min\_w} * A_{vert\_w\_90} + p_{min\_r} * A_{vert\_r\_90} = \mathbf{9.79}$ kips

Leeward net force $F_l = F_{w,wB} = \mathbf{-3.3}$ kips

Windward net force $F_w = F_{w,wA\_1} + F_{w,wA\_2} + F_{w,wA\_3} = \mathbf{4.1}$ kips

Overall horizontal loading $F_{w,total} = \max(F_w - F_l + F_{w,h}, F_{w,total\_min}) = \mathbf{9.8}$ kips

Roof load case 4 - Wind 90, GC_{pi} -0.18, +c_{pe}

Zone	Ref. height (ft)	Ext pressure coefficient c_{pe}	Peak velocity pressure q_p (psf)	Net pressure p (psf)	Area A_{ref} (ft^2)	Net force F_w (kips)
A (+ve)	17.00	-0.18	13.36	0.31	322.58	0.10
B (+ve)	17.00	-0.18	13.36	0.31	322.58	0.10
C (+ve)	17.00	-0.18	13.36	0.31	645.16	0.20
D (+ve)	17.00	-0.18	13.36	0.31	2504.53	0.77

Total vertical net force $F_{w,v} = \mathbf{1.11}$ kips

Total horizontal net force $F_{w,h} = \mathbf{0.00}$ kips

Walls load case 4 - Wind 90, GC_{pi} -0.18, +c_{pe}

Zone	Ref. height (ft)	Ext pressure coefficient c_{pe}	Peak velocity pressure q_p (psf)	Net pressure p (psf)	Area A_{ref} (ft^2)	Net force F_w (kips)
A_1	15.00	0.80	12.90	11.41	537.00	6.13
A_2	15.00	0.80	12.90	11.41	0.00	0.00
A_3	20.00	0.80	14.04	12.20	75.03	0.92
B	17.00	-0.26	13.36	-0.64	612.03	-0.39
C	17.00	-0.70	13.36	-5.75	1400.00	-8.05
D	17.00	-0.70	13.36	-5.75	1400.00	-8.05

Overall loading

Projected vertical plan area of wall $A_{vert\_w\_90} = d * H + d^2 * \tan(\alpha_0) / 4 = \mathbf{612.03}\ ft^2$

Projected vertical area of roof $A_{vert\_r\_90} = \mathbf{0.00}\ ft^2$

Minimum overall horizontal loading $F_{w,total\_min} = p_{min\_w} * A_{vert\_w\_90} + p_{min\_r} * A_{vert\_r\_90} = \mathbf{9.79}$ kips

Leeward net force $F_l = F_{w,wB} = \mathbf{-0.4}$ kips

Windward net force $F_w = F_{w,wA\_1} + F_{w,wA\_2} + F_{w,wA\_3} = \mathbf{7.0}$ kips

Overall horizontal loading $F_{w,total} = \max(F_w - F_l + F_{w,h}, F_{w,total\_min}) = \mathbf{9.8}$ kips

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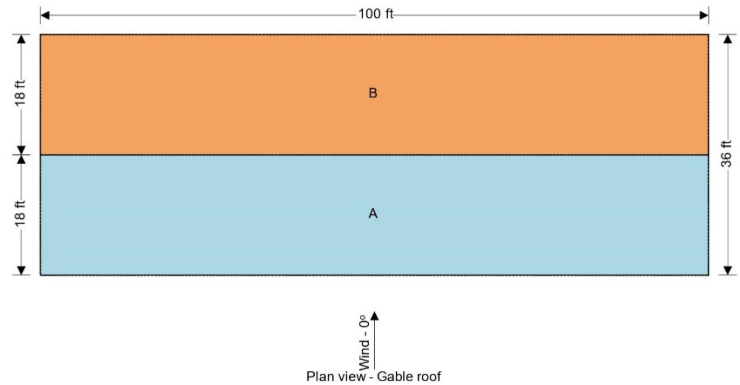
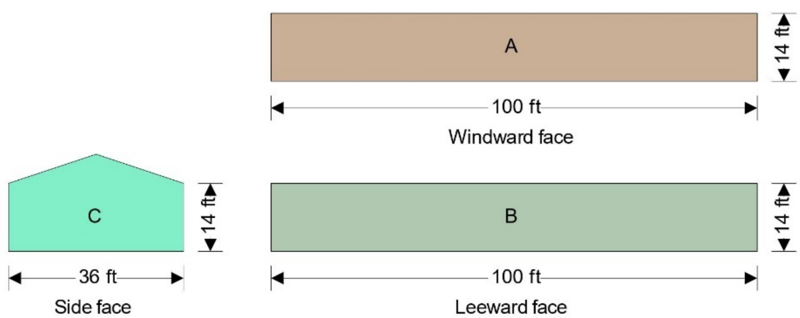
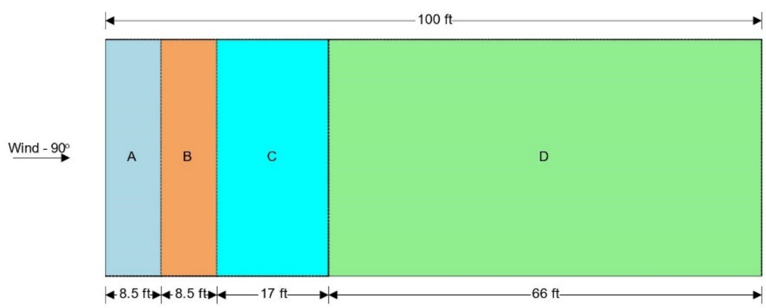


Plan view - Gable roof





Plan view - Gable roof

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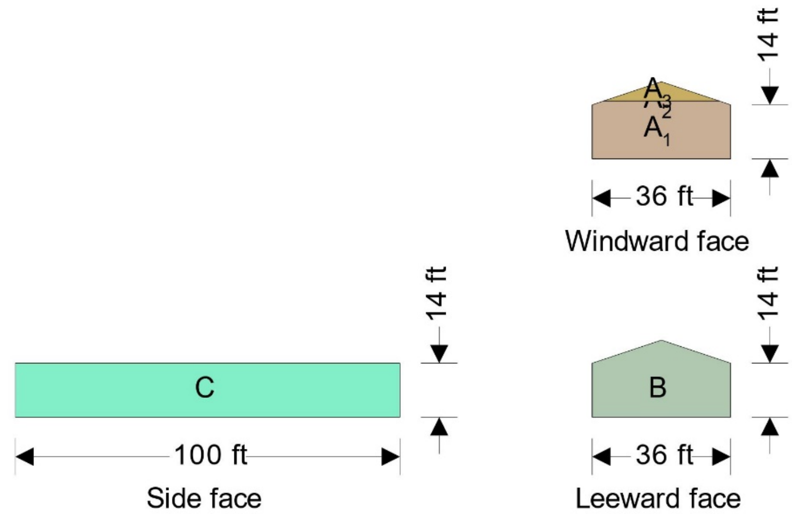

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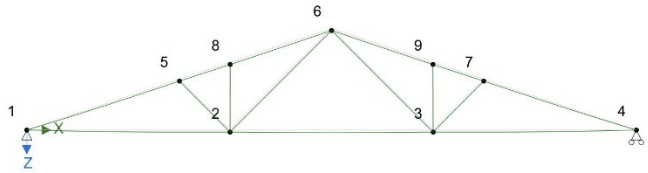
ANALYSIS

Tedds calculation version 1.0.35

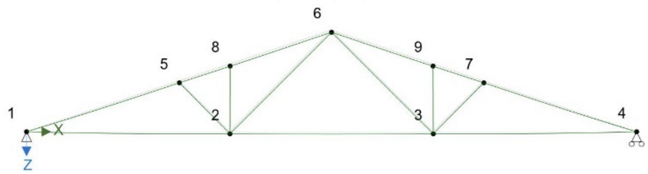
Results

Total deflection

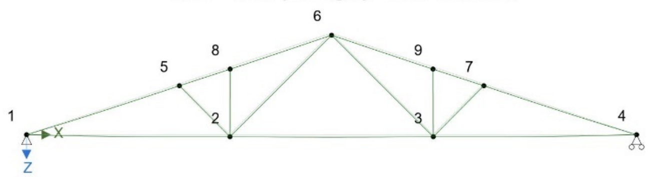
1.0D (Strength) - Total deflection



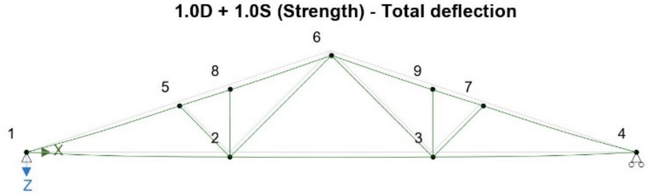
1.0D + 1.0L (Strength) - Total deflection



1.0D + 1.0Lr (Strength) - Total deflection



1.0D + 1.0S (Strength) - Total deflection



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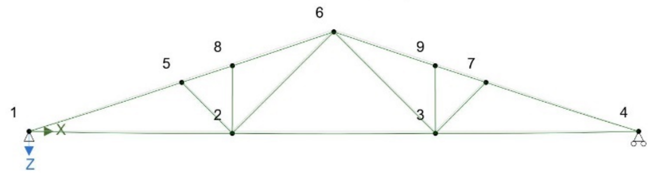
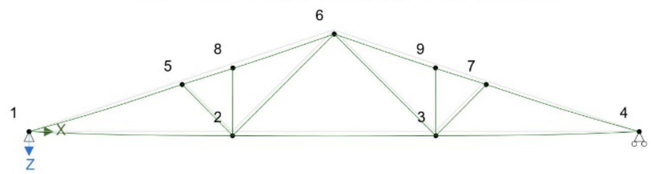
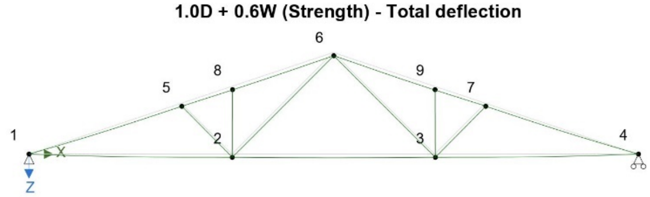
1.0D + 0.75L + 0.75Lr (Strength) - Total deflection



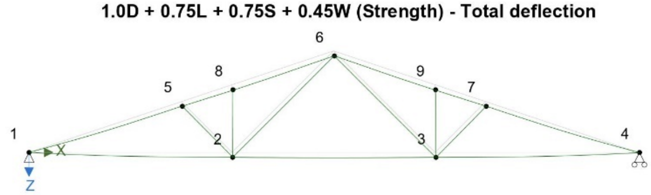
1.0D + 0.75L + 0.75S (Strength) - Total deflection



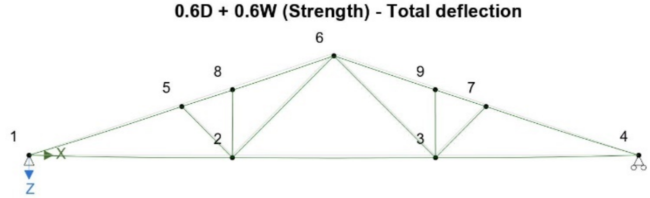
1.0D + 0.6W (Strength) - Total deflection



1.0D + 0.75L + 0.75S + 0.45W (Strength) - Total deflection



0.6D + 0.6W (Strength) - Total deflection



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Node deflections

Load combination: 1.0D (Strength)

Node	Deflection X (in)	Deflection Z (in)	Rotation (°)	Co-ordinate system
1	0	0	0.1338	
2	0.016	0.143	0.01356	
3	0.027	0.143	-0.01356	
4	0.043	0	-0.1338	
5	0.03	0.131	0.02221	
6	0.021	0.144	0	
7	0.013	0.131	-0.02221	
8	0.027	0.143	0.01356	
9	0.015	0.143	-0.01356	

Load combination: 1.0D + 1.0L (Strength)

Node	Deflection X (in)	Deflection Z (in)	Rotation (°)	Co-ordinate system
1	0	0	0.1338	
2	0.016	0.143	0.01356	
3	0.027	0.143	-0.01356	
4	0.043	0	-0.1338	
5	0.03	0.131	0.02221	
6	0.021	0.144	0	
7	0.013	0.131	-0.02221	
8	0.027	0.143	0.01356	
9	0.015	0.143	-0.01356	

Load combination: 1.0D + 1.0Lr (Strength)

Node	Deflection X (in)	Deflection Z (in)	Rotation (°)	Co-ordinate system
1	0	0	0.1338	
2	0.016	0.143	0.01356	
3	0.027	0.143	-0.01356	
4	0.043	0	-0.1338	
5	0.03	0.131	0.02221	
6	0.021	0.144	0	
7	0.013	0.131	-0.02221	
8	0.027	0.143	0.01356	
9	0.015	0.143	-0.01356	

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Load combination: 1.0D + 1.0S (Strength)

Node	Deflection		Rotation	Co-ordinate system
	X (in)	Z (in)	(°)	
1	0	0	0.30002	
2	0.038	0.336	0.03245	
3	0.064	0.336	-0.03245	
4	0.102	0	-0.30002	
5	0.069	0.308	0.05318	
6	0.051	0.339	0	
7	0.032	0.308	-0.05318	
8	0.065	0.336	0.03245	
9	0.037	0.336	-0.03245	

Load combination: 1.0D + 0.75L + 0.75Lr (Strength)

Node	Deflection		Rotation	Co-ordinate system
	X (in)	Z (in)	(°)	
1	0	0	0.1338	
2	0.016	0.143	0.01356	
3	0.027	0.143	-0.01356	
4	0.043	0	-0.1338	
5	0.03	0.131	0.02221	
6	0.021	0.144	0	
7	0.013	0.131	-0.02221	
8	0.027	0.143	0.01356	
9	0.015	0.143	-0.01356	

Load combination: 1.0D + 0.75L + 0.75S (Strength)

Node	Deflection		Rotation	Co-ordinate system
	X (in)	Z (in)	(°)	
1	0	0	0.25846	
2	0.033	0.288	0.02773	
3	0.055	0.288	-0.02773	
4	0.087	0	-0.25846	
5	0.059	0.264	0.04544	
6	0.044	0.29	0	
7	0.028	0.264	-0.04544	
8	0.056	0.288	0.02773	
9	0.031	0.288	-0.02773	

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Load combination: 1.0D + 0.6W (Strength)

Node	Deflection		Rotation	Co-ordinate system
	X (in)	Z (in)	(°)	
1	0	0	0.20718	
2	0.026	0.228	0.0219	
3	0.043	0.228	-0.0219	
4	0.069	0	-0.20718	
5	0.047	0.209	0.03588	
6	0.034	0.23	0	
7	0.022	0.209	-0.03588	
8	0.044	0.228	0.0219	
9	0.025	0.228	-0.0219	

Load combination: 1.0D + 0.75L + 0.75S + 0.45W (Strength)

Node	Deflection		Rotation	Co-ordinate system
	X (in)	Z (in)	(°)	
1	0	0	0.3135	
2	0.04	0.351	0.03398	
3	0.067	0.351	-0.03398	
4	0.107	0	-0.3135	
5	0.073	0.323	0.05569	
6	0.053	0.354	0	
7	0.034	0.323	-0.05569	
8	0.068	0.351	0.03398	
9	0.038	0.351	-0.03398	

Load combination: 0.6D + 0.6W (Strength)

Node	Deflection		Rotation	Co-ordinate system
	X (in)	Z (in)	(°)	
1	0	0	0.15367	
2	0.019	0.171	0.01648	
3	0.032	0.171	-0.01648	
4	0.052	0	-0.15367	
5	0.035	0.157	0.027	
6	0.026	0.172	0	
7	0.016	0.157	-0.027	
8	0.033	0.171	0.01648	
9	0.019	0.171	-0.01648	

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Total base reactions

Load case/combination	Force FX (kips)	Force FZ (kips)
1.0D (Strength)	0	2.981
1.0D + 1.0L (Strength)	0	2.981
1.0D + 1.0Lr (Strength)	0	2.981
1.0D + 1.0S (Strength)	0	7.004
1.0D + 0.75L + 0.75Lr (Strength)	0	2.981
1.0D + 0.75L + 0.75S (Strength)	0	5.998
1.0D + 0.6W (Strength)	0	4.757
1.0D + 0.75L + 0.75S + 0.45W (Strength)	0	7.33
0.6D + 0.6W (Strength)	0	3.565

Element end forces

Load combination: 1.0D (Strength)

Element	Length (ft)	Nodes Start/End	Axial force (kips)	Shear force (kips)	Moment (kip_ft)
1	12	1	3.387	-0.142	-0.016
		2	-3.387	-0.194	-0.294
2	12	2	2.27	-0.168	0.292
		3	-2.27	-0.168	-0.292
3	12	3	3.387	-0.194	0.294
		4	-3.387	-0.142	0.016
4	9.49	1	-3.64	-0.208	0.016
		5	3.484	-0.26	-0.261
5	9.49	5	-3.219	-0.219	0.252
		6	3.063	-0.249	-0.396
6	9.49	6	-3.063	-0.249	0.396
		7	3.219	-0.219	-0.252
7	9.49	7	-3.484	-0.26	0.261
		4	3.64	-0.208	-0.016
8	4.24	5	-0.547	-0.023	0.009
		2	0.547	0.023	0.089
9	8.49	2	1.023	0.013	-0.086
		6	-1.023	-0.013	-0.024
10	8.49	6	1.023	-0.013	0.024
		3	-1.023	0.013	0.086
11	4.24	3	-0.547	0.023	-0.089
		7	0.547	-0.023	-0.009
12	4	2	0	0	0
		8	0	0	0

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Element	Length (ft)	Nodes Start/End	Axial force (kips)	Shear force (kips)	Moment (kip_ft)
13	4	3	0	0	0
		9	0	0	0

Load combination: 1.0D + 1.0L (Strength)

Element	Length (ft)	Nodes Start/End	Axial force (kips)	Shear force (kips)	Moment (kip_ft)
1	12	1	3.387	-0.142	-0.016
		2	-3.387	-0.194	-0.294
2	12	2	2.27	-0.168	0.292
		3	-2.27	-0.168	-0.292
3	12	3	3.387	-0.194	0.294
		4	-3.387	-0.142	0.016
4	9.49	1	-3.64	-0.208	0.016
		5	3.484	-0.26	-0.261
5	9.49	5	-3.219	-0.219	0.252
		6	3.063	-0.249	-0.396
6	9.49	6	-3.063	-0.249	0.396
		7	3.219	-0.219	-0.252
7	9.49	7	-3.484	-0.26	0.261
		4	3.64	-0.208	-0.016
8	4.24	5	-0.547	-0.023	0.009
		2	0.547	0.023	0.089
9	8.49	2	1.023	0.013	-0.086
		6	-1.023	-0.013	-0.024
10	8.49	6	1.023	-0.013	0.024
		3	-1.023	0.013	0.086
11	4.24	3	-0.547	0.023	-0.089
		7	0.547	-0.023	-0.009
12	4	2	0	0	0
		8	0	0	0
13	4	3	0	0	0
		9	0	0	0

Load combination: 1.0D + 1.0Lr (Strength)

Element	Length (ft)	Nodes Start/End	Axial force (kips)	Shear force (kips)	Moment (kip_ft)
1	12	1	3.387	-0.142	-0.016
		2	-3.387	-0.194	-0.294
2	12	2	2.27	-0.168	0.292
		3	-2.27	-0.168	-0.292
3	12	3	3.387	-0.194	0.294
		4	-3.387	-0.142	0.016

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Element	Length (ft)	Nodes Start/End	Axial force (kips)	Shear force (kips)	Moment (kip_ft)
4	9.49	1	-3.64	-0.208	0.016
		5	3.484	-0.26	-0.261
5	9.49	5	-3.219	-0.219	0.252
		6	3.063	-0.249	-0.396
6	9.49	6	-3.063	-0.249	0.396
		7	3.219	-0.219	-0.252
7	9.49	7	-3.484	-0.26	0.261
		4	3.64	-0.208	-0.016
8	4.24	5	-0.547	-0.023	0.009
		2	0.547	0.023	0.089
9	8.49	2	1.023	0.013	-0.086
		6	-1.023	-0.013	-0.024
10	8.49	6	1.023	-0.013	0.024
		3	-1.023	0.013	0.086
11	4.24	3	-0.547	0.023	-0.089
		7	0.547	-0.023	-0.009
12	4	2	0	0	0
		8	0	0	0
13	4	3	0	0	0
		9	0	0	0

Load combination: 1.0D + 1.0S (Strength)

Element	Length (ft)	Nodes Start/End	Axial force (kips)	Shear force (kips)	Moment (kip_ft)
1	12	1	8.043	-0.118	-0.405
		2	-8.043	-0.218	-0.2
2	12	2	5.413	-0.168	0.23
		3	-5.413	-0.168	-0.23
3	12	3	8.043	-0.218	0.2
		4	-8.043	-0.118	0.405
4	9.49	1	-8.701	-0.667	0.405
		5	8.227	-0.755	-0.821
5	9.49	5	-7.473	-0.675	0.843
		6	6.999	-0.747	-1.185
6	9.49	6	-6.999	-0.747	1.185
		7	7.473	-0.675	-0.843
7	9.49	7	-8.227	-0.755	0.821
		4	8.701	-0.667	-0.405
8	4.24	5	-1.616	-0.035	-0.022
		2	1.616	0.035	0.17
9	8.49	2	2.099	0.029	-0.199

IOWA CIVIL & ENVIRONMENTAL ENGINEERING	Project			Job Ref.	
	Section			Sheet no./rev. 9	
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Element	Length (ft)	Nodes Start/End	Axial force (kips)	Shear force (kips)	Moment (kip_ft)
		6	-2.099	-0.029	-0.049
10	8.49	6	2.099	-0.029	0.049
		3	-2.099	0.029	0.199
11	4.24	3	-1.616	0.035	-0.17
		7	1.616	-0.035	0.022
12	4	2	0	0	0
		8	0	0	0
13	4	3	0	0	0
		9	0	0	0

Load combination: 1.0D + 0.75L + 0.75Lr (Strength)

Element	Length (ft)	Nodes Start/End	Axial force (kips)	Shear force (kips)	Moment (kip_ft)
1	12	1	3.387	-0.142	-0.016
		2	-3.387	-0.194	-0.294
2	12	2	2.27	-0.168	0.292
		3	-2.27	-0.168	-0.292
3	12	3	3.387	-0.194	0.294
		4	-3.387	-0.142	0.016
4	9.49	1	-3.64	-0.208	0.016
		5	3.484	-0.26	-0.261
5	9.49	5	-3.219	-0.219	0.252
		6	3.063	-0.249	-0.396
6	9.49	6	-3.063	-0.249	0.396
		7	3.219	-0.219	-0.252
7	9.49	7	-3.484	-0.26	0.261
		4	3.64	-0.208	-0.016
8	4.24	5	-0.547	-0.023	0.009
		2	0.547	0.023	0.089
9	8.49	2	1.023	0.013	-0.086
		6	-1.023	-0.013	-0.024
10	8.49	6	1.023	-0.013	0.024
		3	-1.023	0.013	0.086
11	4.24	3	-0.547	0.023	-0.089
		7	0.547	-0.023	-0.009
12	4	2	0	0	0
		8	0	0	0
13	4	3	0	0	0
		9	0	0	0

Load combination: 1.0D + 0.75L + 0.75S (Strength)

IOWA CIVIL & ENVIRONMENTAL ENGINEERING	Project			Job Ref.	
	Section			Sheet no./rev. 10	
	Calc. by S	Date 11/12/2020	Chk'd by	Date	App'd by	Date

Element	Length (ft)	Nodes Start/End	Axial force (kips)	Shear force (kips)	Moment (kip_ft)
1	12	1	6.879	-0.124	-0.308
		2	-6.879	-0.212	-0.224
2	12	2	4.627	-0.168	0.245
		3	-4.627	-0.168	-0.245
3	12	3	6.879	-0.212	0.224
		4	-6.879	-0.124	0.308
4	9.49	1	-7.435	-0.552	0.308
		5	7.041	-0.631	-0.681
5	9.49	5	-6.409	-0.561	0.696
		6	6.015	-0.623	-0.988
6	9.49	6	-6.015	-0.623	0.988
		7	6.409	-0.561	-0.696
7	9.49	7	-7.041	-0.631	0.681
		4	7.435	-0.552	-0.308
8	4.24	5	-1.349	-0.032	-0.014
		2	1.349	0.032	0.15
9	8.49	2	1.83	0.025	-0.171
		6	-1.83	-0.025	-0.043
10	8.49	6	1.83	-0.025	0.043
		3	-1.83	0.025	0.171
11	4.24	3	-1.349	0.032	-0.15
		7	1.349	-0.032	0.014
12	4	2	0	0	0
		8	0	0	0
13	4	3	0	0	0
		9	0	0	0

Load combination: 1.0D + 0.6W (Strength)

Element	Length (ft)	Nodes Start/End	Axial force (kips)	Shear force (kips)	Moment (kip_ft)
1	12	1	5.443	-0.131	-0.188
		2	-5.443	-0.205	-0.253
2	12	2	3.658	-0.168	0.264
		3	-3.658	-0.168	-0.264
3	12	3	5.443	-0.205	0.253
		4	-5.443	-0.131	0.188
4	9.49	1	-5.874	-0.411	0.188
		5	5.578	-0.478	-0.509
5	9.49	5	-5.097	-0.42	0.513
		6	4.801	-0.469	-0.744
6	9.49	6	-4.801	-0.469	0.744

HHDR CONSULTANTS

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Element	Length (ft)	Nodes Start/End	Axial force (kips)	Shear force (kips)	Moment (kip_ft)
		7	5.097	-0.42	-0.513
7	9.49	7	-5.578	-0.478	0.509
		4	5.874	-0.411	-0.188
8	4.24	5	-1.019	-0.028	-0.005
		2	1.019	0.028	0.125
9	8.49	2	1.498	0.02	-0.136
		6	-1.498	-0.02	-0.035
10	8.49	6	1.498	-0.02	0.035
		3	-1.498	0.02	0.136
11	4.24	3	-1.019	0.028	-0.125
		7	1.019	-0.028	0.005
12	4	2	0	0	0
		8	0	0	0
13	4	3	0	0	0
		9	0	0	0

Load combination: 1.0D + 0.75L + 0.75S + 0.45W (Strength)

Element	Length (ft)	Nodes Start/End	Axial force (kips)	Shear force (kips)	Moment (kip_ft)
1	12	1	8.421	-0.116	-0.436
		2	-8.421	-0.22	-0.193
2	12	2	5.668	-0.168	0.225
		3	-5.668	-0.168	-0.225
3	12	3	8.421	-0.22	0.193
		4	-8.421	-0.116	0.436
4	9.49	1	-9.111	-0.704	0.436
		5	8.611	-0.795	-0.866
5	9.49	5	-7.818	-0.712	0.891
		6	7.318	-0.787	-1.249
6	9.49	6	-7.318	-0.787	1.249
		7	7.818	-0.712	-0.891
7	9.49	7	-8.611	-0.795	0.866
		4	9.111	-0.704	-0.436
8	4.24	5	-1.703	-0.036	-0.025
		2	1.703	0.036	0.176
9	8.49	2	2.186	0.031	-0.208
		6	-2.186	-0.031	-0.051
10	8.49	6	2.186	-0.031	0.051
		3	-2.186	0.031	0.208
11	4.24	3	-1.703	0.036	-0.176
		7	1.703	-0.036	0.025

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	Section				Sheet no./rev. 12	
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Element	Length (ft)	Nodes Start/End	Axial force (kips)	Shear force (kips)	Moment (kip_ft)
12	4	2	0	0	0
		8	0	0	0
13	4	3	0	0	0
		9	0	0	0

Load combination: 0.6D + 0.6W (Strength)

Element	Length (ft)	Nodes Start/End	Axial force (kips)	Shear force (kips)	Moment (kip_ft)
1	12	1	4.088	-0.074	-0.181
		2	-4.088	-0.127	-0.135
2	12	2	2.75	-0.101	0.148
		3	-2.75	-0.101	-0.148
3	12	3	4.088	-0.127	0.135
		4	-4.088	-0.074	0.181
4	9.49	1	-4.418	-0.328	0.181
		5	4.184	-0.374	-0.404
5	9.49	5	-3.809	-0.333	0.412
		6	3.575	-0.369	-0.586
6	9.49	6	-3.575	-0.369	0.586
		7	3.809	-0.333	-0.412
7	9.49	7	-4.184	-0.374	0.404
		4	4.418	-0.328	-0.181
8	4.24	5	-0.8	-0.019	-0.008
		2	0.8	0.019	0.089
9	8.49	2	1.089	0.015	-0.102
		6	-1.089	-0.015	-0.025
10	8.49	6	1.089	-0.015	0.025
		3	-1.089	0.015	0.102
11	4.24	3	-0.8	0.019	-0.089
		7	0.8	-0.019	0.008
12	4	2	0	0	0
		8	0	0	0
13	4	3	0	0	0
		9	0	0	0

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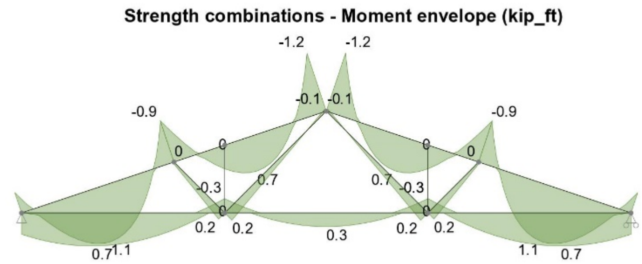
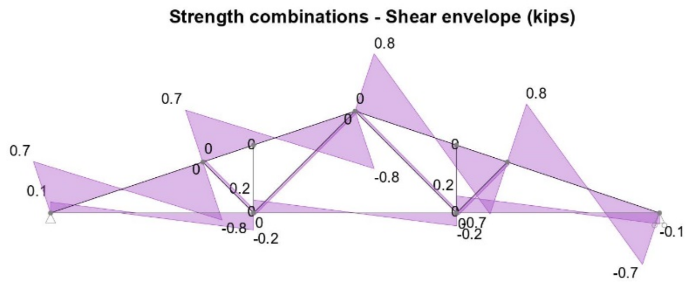
Forces





Member results

Envelope - Strength combinations

Member	Shear force		Moment			
	Pos (ft)	Max abs (kips)	Pos (ft)	Max (kip_ft)	Pos (ft)	Min (kip_ft)
Bottom Chord	24	0.22	4.13	0.675	24	-0.294
Left Rafter	9.49	-0.795	4.46	1.133 (max)	18.97	-1.249 (min)
Right Rafter	9.49	0.795 (max abs)	14.52	1.133 (max)	0	-1.249 (min)

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WOOD MEMBER DESIGN (NDS 2018)

In accordance with the ANSI/AF&PA NDS 2018 using the ASD method

Tedds calculation version 2.2.07

Design section 1

User note: Check beam at support

Member details

Service condition — Dry

Load duration - Table 2.3.2 — Ten years

Sawn lumber section details

Number of sections in member — N = **1**

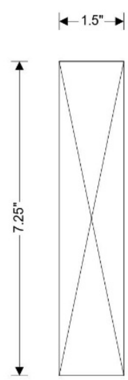
Nominal breadth of sections — b_{nom} = **2** in

Breadth of sections — b = **1.5** in

Nominal depth of sections — d_{nom} = **8** in

Depth of sections — d = **7.25** in

Material — **Douglas Fir-Larch, 2" && wider, Select Structural grade**



2"x8" sawn lumber section
Cross-sectional area, A, 10.875 in^2
Section modulus, S_x, 13.1 in^3
Section modulus, S_y, 2.7 in^3
Second moment of area, I_x, 47.6 in^4
Second moment of area, I_y, 2 in^4
Radius of gyration, r_x, 2.093 in
Radius of gyration, r_y, 0.433 in
Douglas Fir-Larch, 2" & wider, Select Structural grade
Bending, F_b, 1500 psi
Shear parallel to grain, F_v, 180 psi
Compression parallel to grain, F_c, 1700 psi
Compression perpendicular to grain, F_{c_perp}, 625 psi
Tension parallel to grain, F_t, 1000 psi
Modulus of elasticity, E, 1900000 psi
Minimum modulus of elasticity, E_{min}, 690000 psi
Density, ρ, 34.204 lbm/ft^3
Specific gravity, G, 0.5

Span details

Unbraced length - Major axis — L_x = **12** ft

Effective bending length - Major axis — $L_{e,x}$ = 1.63 * L_x + 3 * d = **21.373** ft

Column buckling length - Major axis — $L_{b,x}$ = L_x = **12** ft

Unbraced length - Minor axis — L_y = **0** ft

Bearing length — L_b = **4** in

Analysis results

Design bending moment - Major axis — M_x = **1249** lb_ft

Design shear force - Major axis — V_x = **795** lb

Design perpendicular compression - Major axis — R_x = **795** lb

Section s1 results summary	Unit	Capacity	Maximum	Utilization	Result
Bending stress	lb/in^2	1800	1141	0.634	PASS

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Shear stress	lb/in^2	180	110	0.609	PASS
Bearing stress	lb/in^2	625	133	0.212	PASS

Adjustment factors - Table 4.3.1

Load duration factor - Table 2.3.2 — C_D = **1**

Size factor for bending - Table 4A — C_{Fb} = **1.2**

Bending members - Flexure - cl.3.3

Design bending moment — M_x = **1249** lb_ft

Design bending stress - Table 4.3.1 — $F_{b,x}' = F_b * C_D * C_{Fb}$ = **1800** lb/in^2

Actual bending stress - eq.3.3-2 — $f_{b,x} = M_x / S_x$ = **1141** lb/in^2

$f_{b,x} / F_{b,x}'$ = **0.634**

PASS - Design bending stress exceeds actual bending stress

Bending members - Shear - cl.3.4

Design shear force — V_x = **795** lb

Design shear stress - Table 4.3.1 — $F_{v,x}' = F_v * C_D$ = **180** lb/in^2

Actual shear stress - eq.3.4-2 — $f_{v,x} = 3 * V_x / (2 * b * d)$ = **110** lb/in^2

$f_{v,x} / F_{v,x}'$ = **0.609**

PASS - Design shear stress exceeds actual shear stress

Design for bearing - cl.3.10

Design perpendicular compression — R_x = **795** lb

Design bearing stress - Table 4.3.1 — $F_{c\_perp,x}' = F_{c\_perp}$ = **625** lb/in^2

Actual bearing stress — $f_{c\_perp,x} = R_x / (b * L_b)$ = **133** lb/in^2

$f_{c\_perp,x} / F_{c\_perp,x}'$ = **0.212**

PASS - Design bearing stress exceeds actual bearing stress perpendicular to grain

<u>Design section 2</u>

User note: Check beam at mid-span

Member details

Service condition — Dry

Load duration - Table 2.3.2 — Ten years

Sawn lumber section details

Number of sections in member — N = **1**

Nominal breadth of sections — b_{nom} = **2** in

Breadth of sections — b = **1.5** in

Nominal depth of sections — d_{nom} = **8** in

Depth of sections — d = **7.25** in

Material — **Hem-Fir, 2" && wider, Select Structural grade**

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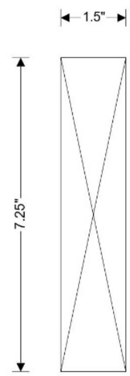

2"x8" sawn lumber section
Cross-sectional area, A, 10.875 in^2
Section modulus, S_x, 13.1 in^3
Section modulus, S_y, 2.7 in^3
Second moment of area, I_x, 47.6 in^4
Second moment of area, I_y, 2 in^4
Radius of gyration, r_x, 2.093 in
Radius of gyration, r_y, 0.433 in
Hem-Fir, 2" & wider, Select Structural grade
Bending, F_b, 1400 psi
Shear parallel to grain, F_v, 150 psi
Compression parallel to grain, F_c, 1500 psi
Compression perpendicular to grain, F_{c_perp}, 405 psi
Tension parallel to grain, F_t, 925 psi
Modulus of elasticity, E, 1600000 psi
Minimum modulus of elasticity, E_{min}, 580000 psi
Density, ρ, 29.743 lbm/ft^3
Specific gravity, G, 0.43

Span details

Unbraced length - Major axis L_x = **12** ft
Effective bending length - Major axis $L_{e,x} = 1.63 * L_x + 3 * d$ = **21.373** ft
Column buckling length - Major axis $L_{b,x} = L_x$ = **12** ft
Unbraced length - Minor axis L_y = **0** ft
Bearing length L_b = **4** in

Analysis results

Design bending moment - Major axis M_x = **675** lb_ft
Design shear force - Major axis V_x = **220** lb
Design perpendicular compression - Major axis R_x = **220** lb

Section s2 results summary	Unit	Capacity	Maximum	Utilization	Result
Bending stress	lb/in^2	1680	616	0.367	PASS
Shear stress	lb/in^2	150	30	0.202	PASS
Bearing stress	lb/in^2	405	37	0.091	PASS

Adjustment factors - Table 4.3.1

Load duration factor - Table 2.3.2 C_D = **1**
Size factor for bending - Table 4A C_{Fb} = **1.2**

Bending members - Flexure - cl.3.3

Design bending moment M_x = **675** lb_ft
Design bending stress - Table 4.3.1 $F_{b,x}' = F_b * C_D * C_{Fb}$ = **1680** lb/in^2
Actual bending stress - eq.3.3-2 $f_{b,x} = M_x / S_x$ = **616** lb/in^2
$f_{b,x} / F_{b,x}'$ = **0.367**

PASS - Design bending stress exceeds actual bending stress

Bending members - Shear - cl.3.4

Design shear force V_x = **220** lb
Design shear stress - Table 4.3.1 $F_{v,x}' = F_v * C_D$ = **150** lb/in^2
Actual shear stress - eq.3.4-2 $f_{v,x} = 3 * V_x / (2 * b * d)$ = **30** lb/in^2

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$f_{v,x} / F_{v,x}' =$ **0.202**

PASS - Design shear stress exceeds actual shear stress

Design for bearing - cl.3.10

Design perpendicular compression $R_x =$ **220** lb

Design bearing stress - Table 4.3.1 $F_{c\_perp,x}' = F_{c\_perp} =$ **405** lb/in^2

Actual bearing stress $f_{c\_perp,x} = R_x / (b * L_b) =$ **37** lb/in^2

$f_{c\_perp,x} / F_{c\_perp,x}' =$ **0.091**

PASS - Design bearing stress exceeds actual bearing stress perpendicular to grain

Wind Pressure Calculations

$he := 14\ ft$ $\quad h_{peak} := 19\ ft$ $\quad T := 288\ K$ $\quad g := 32.174\ \frac{ft}{s^2}$ $\quad R := 287\ \frac{(N \cdot m)}{kg \cdot K}$

$h := \frac{he + h_{peak}}{2} = 16.5\ ft$ Average roof height

$G := 0.85$ Gust Effect Factor for rigid buildings

$K_d := 0.85$ Wind Directionality Factor

$zg := 942\ ft$ Elevation of Manchester, IA

$K_e := e^{\frac{-g \cdot zg}{R \cdot T}} = 0.967$ Ground Elevation Factor

$K_{zt} := 1.0$ Topographic Factor

$GCp_i := 0.18$ Internal Pressure Coefficient for enclosed buildings

$V := 103\ mph$ Basic Wind Speed of Manchester, IA

Z locations: $z1 := 10\ ft$ $\quad z2 := he = 14\ ft$ $\quad z3 := h_{peak} = 19\ ft$

Exposure B (<30ft): $\alpha := 7$ $\quad zg := 1200\ ft$

$K_{10} := 2.01 \cdot \left(\frac{15 \cdot ft}{zg}\right)^{\frac{2}{\alpha}} = 0.575$ $\quad K_h := 2.01 \cdot \left(\frac{h}{zg}\right)^{\frac{2}{\alpha}} = 0.591$

$K_{14} := 2.01 \cdot \left(\frac{15 \cdot ft}{zg}\right)^{\frac{2}{\alpha}} = 0.575$ $\quad K_{19} := 2.01 \cdot \left(\frac{z3}{zg}\right)^{\frac{2}{\alpha}} = 0.615$

$$q_{10} := 0.00256 \cdot \frac{psf}{mph^2} \cdot K_{10} \cdot K_{zt} \cdot K_d \cdot K_e \cdot V^2 = 12.823\ psf$$

$$q_{14} := 0.00256 \cdot \frac{psf}{mph^2} \cdot K_{14} \cdot K_{zt} \cdot K_d \cdot K_e \cdot V^2 = 12.823\ psf$$

$$q_h := 0.00256 \cdot \frac{psf}{mph^2} \cdot K_h \cdot K_{zt} \cdot K_d \cdot K_e \cdot V^2 = 13.177\ psf$$

$$q_{19} := 0.00256 \cdot \frac{psf}{mph^2} \cdot K_{19} \cdot K_{zt} \cdot K_d \cdot K_e \cdot V^2 = 13.719 \ psf$$

Transverse Wind: $L_T := 36 \ ft$ $B_T := 40 \ ft$

Walls: $\frac{L_T}{B_T} = 0.9$ $Cp_{TWwind} := 0.8$ $Cp_{TWlee} := -0.5$

Roof: $\operatorname{atan}\left(\frac{4}{12}\right) = 0.322$ $0.322 \ rad = 18.449 \ °$

Wind is normal to ridge with θ = 18.449 degrees > 10 degrees

$\frac{h}{L_T} = 0.458$ $Cp_{TR1wind} := -0.493$ $Cp_{TR1lee} := -0.569$

$Cp_{TR2wind} := -0.0558$ $Cp_{TR2lee} := 0$

Longitudinal Wind: $B_L := 40 \ ft$ $L_L := 90 \ ft$

Walls: $\frac{L_L}{B_L} = 2.25$ $Cp_{LWwind} := 0.8$ $Cp_{LWlee} := -0.287$

Roof: Wind is parallel to ridge $\frac{h}{L_L} = 0.183$

from 0 - 20 ft: $Cp_{LR1} := -1.3$ $Cp_{LR2} := -0.18$

greater than 20 ft: $Cp_{LR3} := -0.7$ $Cp_{LR2} = -0.18$

a. Longitudinal design wind pressure for walls

Walls with positive pressure:

$p_{10WP} := q_{10} \cdot G \cdot Cp_{LWwind} - q_h \cdot GCp_i = 6.348 \text{ psf}$

$p_{14WP} := q_{14} \cdot G \cdot Cp_{LWwind} - q_h \cdot GCp_i = 6.348 \text{ psf}$

$p_{19WP} := q_{19} \cdot G \cdot Cp_{LWwind} - q_h \cdot GCp_i = 6.957 \text{ psf}$

$p_{hLP} := q_h \cdot G \cdot Cp_{LWlee} - q_h \cdot GCp_i = -5.586 \text{ psf}$

Walls with negative pressure:

$p_{10WN} := q_{10} \cdot G \cdot Cp_{LWwind} - q_h \cdot -GCp_i = 11.092 \text{ psf}$

$p_{14WN} := q_{14} \cdot G \cdot Cp_{LWwind} - q_h \cdot -GCp_i = 11.092 \text{ psf}$

$p_{19WP} := q_{19} \cdot G \cdot Cp_{LWwind} - q_h \cdot -GCp_i = 11.701 \text{ psf}$

$p_{hLN} := q_h \cdot G \cdot Cp_{LWlee} - q_h \cdot -GCp_i = -0.843 \text{ psf}$

b. Transverse design wind pressure for walls

Walls with positive internal pressure:

$p_{10WP} := q_{10} \cdot G \cdot Cp_{TWwind} - q_h \cdot GCp_i = 6.348 \text{ psf}$

$p_{14WP} := q_{14} \cdot G \cdot Cp_{TWwind} - q_h \cdot GCp_i = 6.348 \text{ psf}$

$p_{hLP} := q_h \cdot G \cdot Cp_{TWlee} - q_h \cdot GCp_i = -7.972 \text{ psf}$

Walls with negative internal pressure:

$p_{10WN} := q_{10} \cdot G \cdot Cp_{TWwind} - q_h \cdot -GCp_i = 11.092 \text{ psf}$

$p_{14WN} := q_{14} \cdot G \cdot Cp_{TWwind} - q_h \cdot -GCp_i = 11.092 \text{ psf}$

$p_{hLN} := q_h \cdot G \cdot Cp_{TWlee} - q_h \cdot -GCp_i = -3.228 \text{ psf}$

c. Longitudinal design wind pressure for roof

Roofs with positive pressure:

from 0 - 10 ft:

$p_{LRP1} := q_h \cdot G \cdot Cp_{LR1} - q_h \cdot GCp_i = -16.933\ \mathbf{psf}$

$p_{LRP2} := q_h \cdot G \cdot Cp_{LR2} - q_h \cdot GCp_i = -4.388\ \mathbf{psf}$

from 10 ft - 20 ft:

$p_{LRP3} := q_h \cdot G \cdot Cp_{LR1} - q_h \cdot GCp_i = -16.933\ \mathbf{psf}$

$p_{LRP4} := q_h \cdot G \cdot Cp_{LR2} - q_h \cdot GCp_i = -4.388\ \mathbf{psf}$

from 20 ft - 40 ft:

$p_{LRP5} := q_h \cdot G \cdot Cp_{LR3} - q_h \cdot GCp_i = -10.212\ \mathbf{psf}$

$p_{LRP6} := q_h \cdot G \cdot Cp_{LR2} - q_h \cdot GCp_i = -4.388\ \mathbf{psf}$

Roofs with negative pressure:

from 0 - 10 ft:

$p_{LRP1} := q_h \cdot G \cdot Cp_{LR1} - q_h \cdot -GCp_i = -12.189\ \mathbf{psf}$

$p_{LRP2} := q_h \cdot G \cdot Cp_{LR2} - q_h \cdot -GCp_i = 0.356\ \mathbf{psf}$

from 10 ft - 20 ft:

$p_{LRP3} := q_h \cdot G \cdot Cp_{LR1} - q_h \cdot -GCp_i = -12.189\ \mathbf{psf}$

$p_{LRP4} := q_h \cdot G \cdot Cp_{LR2} - q_h \cdot -GCp_i = 0.356\ \mathbf{psf}$

from 20 ft - 40 ft:

$p_{LRP5} := q_h \cdot G \cdot Cp_{LR3} - q_h \cdot -GCp_i = -5.469\ \mathbf{psf}$

$p_{LRP6} := q_h \cdot G \cdot Cp_{LR2} - q_h \cdot -GCp_i = 0.356\ \mathbf{psf}$

d. Transverse design wind pressure for roof

Roofs with positive internal pressure:

$p_{WP1} := q_h \cdot G \cdot Cp_{TR1wind} - q_h \cdot GCp_i = -7.894\ \mathbf{psf}$

$p_{WP2} := q_h \cdot G \cdot Cp_{TR2wind} - q_h \cdot GCp_i = -2.997\ \mathbf{psf}$

$p_{LP1} := q_h \cdot G \cdot Cp_{TR1lee} - q_h \cdot GCp_i = -8.745\ \mathbf{psf}$

$p_{LP1} := q_h \cdot G \cdot Cp_{TR2lee} - q_h \cdot GCp_i = -2.372\ \mathbf{psf}$

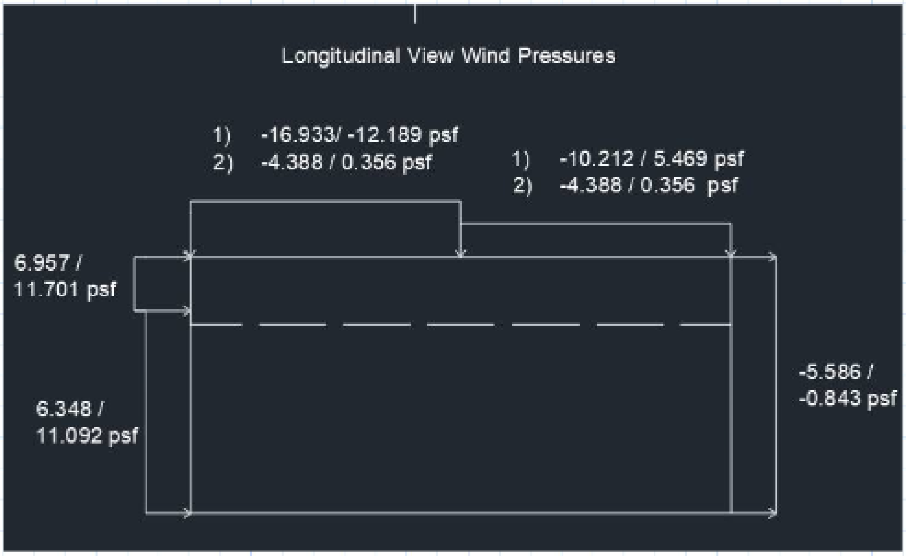
Roofs with negative internal pressure:

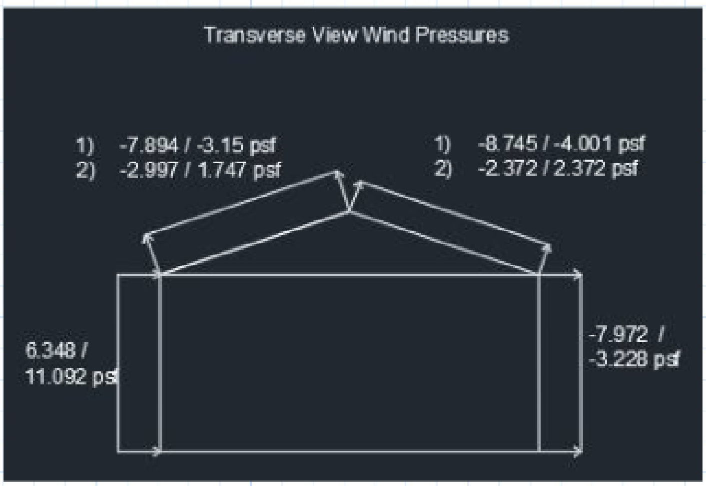
$p_{WN1} := q_h \cdot G \cdot Cp_{TR1wind} - q_h \cdot -GCp_i = -3.15\ \mathbf{psf}$

$p_{WN2} := q_h \cdot G \cdot Cp_{TR2wind} - q_h \cdot -GCp_i = 1.747\ \mathbf{psf}$

$p_{LN1} := q_h \cdot G \cdot Cp_{TR1lee} - q_h \cdot -GCp_i = -4.001\ \mathbf{psf}$

$p_{LN1} := q_h \cdot G \cdot Cp_{TR2lee} - q_h \cdot -GCp_i = 2.372\ \mathbf{psf}$


Longitudinal View Wind Pressures
1) -16.933/ -12.189 psf
2) -4.388 / 0.356 psf
1) -10.212 / 5.469 psf
2) -4.388 / 0.356 psf
6.957 / 11.701 psf
6.348 / 11.092 psf
-5.586 / -0.843 psf


Transverse View Wind Pressures
1) -7.894 / -3.15 psf
2) -2.997 / 1.747 psf
1) -8.745 / -4.001 psf
2) -2.372 / 2.372 psf
6.348 / 11.092 psf
-7.972 / -3.228 psf

Appendix D: Structural Design Wall Calculations

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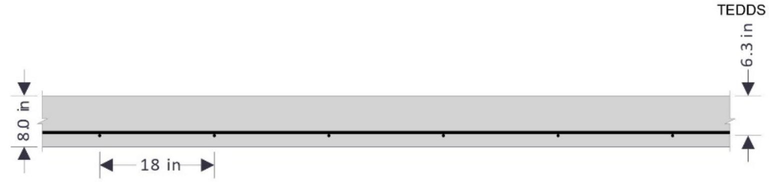
RC WALL DESIGN (ACI318-19)

TEDDS calculation version 1.2.03



Geometry of wall

Depth of wall h = **8.00** in

Clear cover to reinforcement (both sides) c_c = **1.50** in

Unsupported height of wall l_u = **168.0** in

Effective height factor k = **1.00**

Reinforcement of wall

Numbers of reinforcement layers N_l = **1**

Vertical steel bar diameter number D_{ver_num} = **4**

Spacing of vertical steel s_v = **18.00** in

Diameter of vertical steel bar D_{ver} = **0.500** in

Horizontal steel bar diameter number D_{hor_num} = **4**

Spacing of horizontal steel s_h = **12.00** in

Diameter of horizontal bar D_{hor} = **0.500** in

Specified yield strength of reinforcement f_y = **60000** psi

Specified compressive strength of concrete f'_c = **4000** psi

Modulus of elasticity of bar reinforcement E_s = **29** × **10⁶** psi

Modulus of elasticity of concrete $E_c = 57000 \times f'_c{}^{1/2} \times (1\text{psi})^{1/2}$ = **3604997** psi

Ultimate design strain ε_c = **0.003** in/in

Compression-controlled strain limit ε_{ty} = **0.002**

Check for minimum area of vertical steel of single layer reinforcement wall to cl. 11.6.1

Gross area of wall per running foot length $A_g = h \times 12\text{in}$ = **96.000** in^2

Numbers of vertical bars per running foot length $N_v = 12\text{in}/s_v$ = **0.667**

Area of vertical steel per running foot length $A_{st\_v} = N_v \times (\pi \times D_{ver}^2) / 4$ = **0.131** in^2

Minimum area of vertical steel required $A_{st\_v\_min}$= **0.115** in^2

PASS- Minimum vertical steel check

Check for minimum area of horizontal steel of single layer reinforcement wall to cl. 11.6.1

Gross area of wall per running foot height $A_g = h \times 12\text{in}$ = **96.000** in^2

Numbers of horizontal bar per running foot height $N_h = 12\text{in} / s_h$ = **1.000**

Area of horizontal steel per running foot height $A_{st\_h} = N_h \times (\pi \times D_{hor}^2) / 4$ = **0.196** in^2

Minimum area of horizontal steel required $A_{st\_h\_min}$= **0.192** in^2

PASS- Minimum horizontal steel check

Braced wall slenderness check to 6.2.5

Maximum slenderness ratio limit s_{r_max} = 100

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Permissible slenderness ratio s_{r_perm} = min(34 + 12 × (M_{1_act} / M_{2_act}), 40) = **34.0**

Slenderness check for braced wall

Radius of gyration r_{min} = 0.3 × h = **2.40** in

Actual slenderness ratio s_{r_act} = k × l_u / r_{min} = **70.00**

Wall slenderness limit OK, wall is braced slender wall

Design loads for wall subjected to shear and axial load

Ultimate axial force per running foot P_{u_act} = **1.00** kips/ft

Ultimate shear force per running foot V_{u_act} = **1.00** kips/ft

Magnified moment for braced slender wall to 6.6.4

Moment of inertia of section I_g = (12in×h³)/12 = **512.000** in⁴

Euler's buckling load P_c = (π^2 × 0.4 × E_c × I_g)/((1+β_d)×(k × l_u)^2 ×1ft) = **156.470** kips/ft

Correction factor for actual to equiv. mmt. diagram C_m = 0.6 - (0.4 * M_{1_act} / M_{2_act}) = **0.600**

Moment magnifier δ_{ns} = max(1.0, C_m / (1-(P_{u_act}/(0.75 × P_c)))) = **1.000**

Minimum uniaxial moment for slender section M_{2_min} = P_{u_act} * (0.6 in + 0.03 * h) = **0.070** kip_ft/ft

Magnified uniaxial moment M_c= δ_{ns} × max(M_{2_min}, M_{2_act}) = **0.070** kip_ft/ft

Axial load capacity of single layer reinforcement wall subjected to bending

c/d_t ratio r = **1.172**

Effective cover to reinforcement d' = c_c + (D_{ver}/2) = **1.750** in

Depth of tension steel d_t = h - d' = **6.250** in

Depth of NA from extreme compression face c = r × d_t = **7.326** in

Factor of depth of compressive stress block β_1= **0.850**

Depth of equivalent rectangular stress block a = min((β_1× c), h)= **6.227** in

Strain in 'tension' reinforcement ε_s = ε_c × (1 - d_t / c) = **0.000441**

f_s Stress in 'tension' reinforcement f_s = min(E_s * ε_s, f_y) = **12780.6** psi

Compression force in concrete C_c = 0.85 × f'c × a × 12in/1ft= **254.074** kips/ft

Area of vertical tension steel per running foot A_s = A_{st_v} = **0.131** in²

Force in 'tension' steel T_s = A_s × f_s/1ft = **1.673** kips/ft

Nominal axial load strength P_n = C_c + T_s = **255.747** kips/ft

Strength reduction factor ϕ = 0.65 = **0.65**

Ultimate axial load carrying capacity of wall P_u = ϕ × P_n = **166.236** kips/ft

Check for axial load capacity of wall

Wall is safe in axial loading

Bending capacity of single layer reinforcement wall

Centroid of wall y = h × 0.5 = **4.000** in

Nominal moment strength M_n = C_c × (y - 0.5 × a) - T_s × (d_t - y)= **18.453** kip_ft/ft

Ultimate moment strength capacity of wall M_u = ϕ × M_n = **11.994**kip_ft/ft

Check for uniaxial bending capacity of wall

Wall is safe for bending

Check for shear capacity of wall cl. 22.5

Required shear strength V_{u_act} = **1.000** kips/ft

Strength reduction factor ϕ_v = **0.75**

Effective cover to reinforcement d' = c_c + (D_{ver}/2) = **1.750** in

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Depth of tension steel — $d_t = h - d' =$ **6.250** in

Size effect factor – cl. 22.5.5.1.3 — $\lambda_s = \min(\sqrt{(2 / (1 + (d_t / 1\text{ in}) / 10))}, 1.0) =$ **1.00**

Ratio of longitudinal reinforcement — $\rho_w = A_s / (d_t \times 12\text{ in}) =$ **0.002**

Maximum shear force resisting capacity of wall — $V_{max} = 5 \times \lambda \times \sqrt{(f'_c \times 1\text{ psi})} \times d_t =$ **23.717** kips/ft

Shear strength provided by concrete — $V_{c1} = (8 \times \lambda_s \times \lambda \times (\rho_w)^{1/3} \times \sqrt{(f'_c \times 1\text{ psi})} + P_{u\_act} / (6 \times A_g) \times 12\text{ in}) \times d_t =$ **4.699** kips/ft

Design shear capacity of section — $\phi V_c = \phi_v \times \min(V_{c1}, V_{max}) =$ **3.524** kips/ft

PASS- Wall is safe in shear force

Design summary

Wall is 8" thick with 4000 psi concrete and 60000 psi steel

Vertical reinforcement is No.4 bars at 18" spacing

Horizontal reinforcement is No.4 bars at 12" spacing

Design status

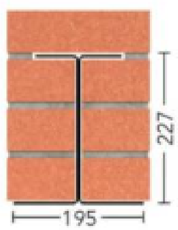
PASS-Wall is safe

$wc := 150 \frac{lbf}{ft^3}$ $ws := 493.18 \frac{lbf}{ft^3}$ $LG1 := 12\ ft$ $LG2 := 10\ ft$ $L3 := 4\ ft$

$wm := 84 \frac{lbf}{ft^2}$

Browse for Image...

12 ft Wide x 12 ft Tall Garage Door: IBeam 3C (for 215 mm solid walls)



Length (mm)	Height (mm)	Thickness (mm)	Total UDL (Kn)
600 1800	227	2.5	45
1950 2100	227	2.5	40
2250 3000	227	2.9	40
3150 4050	227	3.2	40
4200 4800	227	3.2	35

$bf := 195\ mm$ $hw := 227\ mm$ $tw := 3.2\ mm$ $tf := 3.2\ mm$

$h := hw + tf \cdot 2 = 9.189\ in$

$L_{IB3C1} := LG1 + h \cdot 2 = 13.531\ ft$ must be less than or equal to $4200\ mm = 13.78\ ft$

$A_{IB3C} := bf \cdot tf \cdot 2 + hw \cdot tw = 3.06\ in^2$

$xc := \frac{bf}{2} = 3.839\ in$ $yc := \frac{hw + 2 \cdot tf}{2} = 4.594\ in$

$$Ix := \frac{hw^3 \cdot tw + 8 \cdot bf \cdot tf^3 + 12 \cdot bf \cdot hw \cdot tf^2 + 6 \cdot bf \cdot hw^2 \cdot tf}{12} = 47.218\ \mathbf{in}^4$$

$$Iy := \frac{1}{12} \cdot hw \cdot tw^3 + \frac{2}{12} \cdot bf^3 \cdot tf = 9.502\ \mathbf{in}^4$$

Roof Load:

$$Roof := 0.12\ \frac{\mathbf{kip}}{\mathbf{ft}} + 0.24\ \frac{\mathbf{kip}}{\mathbf{ft}} = 0.36\ \frac{\mathbf{kip}}{\mathbf{ft}}$$

Self Weight:

$$SW := A_{IB3C} \cdot ws = 10.481\ \frac{\mathbf{lbf}}{\mathbf{ft}}$$

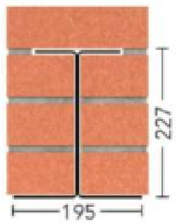
Wall Load (no arching action therefore consider full rectangular load):

$$DL := ((14\ \mathbf{ft} - (12\ \mathbf{ft}))) \cdot wm = 168\ \frac{\mathbf{lbf}}{\mathbf{ft}}$$

$$w := Roof + SW + DL = 538.481\ \frac{\mathbf{lbf}}{\mathbf{ft}}$$

$W_{G1} := w \cdot L_{IB3C1} = 32.412\ \mathbf{kN}$ must be less than or equal to $UDL_{3c1} := 35\ \mathbf{kN}$

10 ft Wide x 12 ft Tall Garage Door: IBeam 3C (for 215 mm solid walls)



Length (mm)	Height (mm)	Thickness (mm)	Total UDL (Kn)
600 1800	227	2.5	45
1950 2100	227	2.5	40
2250 3000	227	2.9	40
3150 4050	227	3.2	40
4200 4800	227	3.2	35

$bf := 195\ mm$ $\quad hw := 227\ mm$ $\quad tw := 3.2\ mm$ $\quad tf := 3.2\ mm$

$L_{IB3C2} := LG2 + h \cdot 2 = 11.531\ ft$ must be less than or equal to $4050\ mm = 13.287\ ft$

$A_{IB3C} = 3.06\ in^2$

$xc = 3.839\ in$ $\quad yc = 4.594\ in$ $\quad Ix = 47.218\ in^4$ $\quad Iy = 9.502\ in^4$

Roof Load:

$$Roof := 0.12\ \frac{kip}{ft} + 0.24\ \frac{kip}{ft} = 0.36\ \frac{kip}{ft}$$

Self Weight:

$$SW := A_{IB3C} \cdot ws = 10.481\ \frac{lbf}{ft}$$

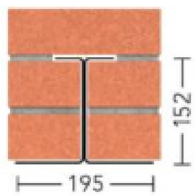
Wall Load (no arching action therefore consider full rectangular load):

$$DL := ((14\ ft - (12\ ft))) \cdot 84\ \frac{lbf}{ft^2} = 168\ \frac{lbf}{ft}$$

$$w := Roof + SW + DL = 538.481\ \frac{lbf}{ft}$$

$W_{G2} := L_{IB3C2} \cdot w = 27.621\ kN$ must be less than or equal to $UDL_{3c2} := 40\ kN$

Entrance Door 4ft x 7ft: IBeam 2C (for 215 mm solid walls)



Overall Length (mm)	Height (mm)	Thickness (mm)	Total UDL (Kn)
600 2100	152	2.5	30
2250 3000	152	2.9	30

$bf := 195\ mm \qquad hw := 152\ mm \qquad tw := 2.5\ mm \qquad tf := 2.5\ mm$

$h := hw + 2 \cdot tf = 6.181\ in$

$L_{IB2C} := L3 + h \cdot 2 = 5.03\ ft$ must be less than or equal to $2100\ mm = 6.89\ ft$

$A_{IB2C} := bf \cdot tf \cdot 2 + hw \cdot tw = 2.1\ in^2$

$xc := \frac{bf}{2} = 3.839\ in \qquad yc := \frac{hw + 2 \cdot tf}{2} = 3.091\ in$

$$Ix := \frac{hw^3 \cdot tw + 8 \cdot bf \cdot tf^3 + 12 \cdot bf \cdot hw \cdot tf^2 + 6 \cdot bf \cdot hw^2 \cdot tf}{12} = 15.738\ in^4$$

$$Iy := \frac{1}{12} \cdot hw \cdot tw^3 + \frac{2}{12} \cdot bf^3 \cdot tf = 7.423\ in^4$$

Roof Load:

$$Roof := 0.12\ \frac{kip}{ft} + 0.24\ \frac{kip}{ft} = 0.36\ \frac{kip}{ft}$$

Self Weight:

$$SW := A_{IB2C} \cdot ws = 7.193\ \frac{lbf}{ft}$$

Wall Load (no arching action therefore consider full rectangular load):

$$DL := ((14\ ft - (7 \cdot ft))) \cdot 84\ \frac{lbf}{ft^2} = 588\ \frac{lbf}{ft}$$

$$w := Roof + SW + DL = 955.193\ \frac{lbf}{ft}$$

$W_{G2} := L_{IB2C} \cdot w = 21.373\ kN$ must be less than or equal to $UDL_{3c2} := 30\ kN$

Appendix E: Structural Design Foundation and Footing Calculations

IOWA CIVIL & ENVIRONMENTAL ENGINEERING

Project: Storage Building Structural Design
Section: Slab on Grade Calculations

11/10/2020

Inputs:

Slab thickness:	$t := 14\ in$
Concrete strength, fc':	$f'c := 4000\ psi$
Normal weight concrete, wc:	$wc := 150\ \frac{lbf}{ft^3}$
Normal CMU weight, wm:	$wm := 125\ \frac{lbf}{ft^3}$
Subgrade modulus, k:	$k := 100\ \frac{lbf}{in^3}$
Poisson's Ratio, μ:	$\mu := 0.15$
Distributed roof load: wr	$wr := \frac{2.4\ kip \cdot 11}{40\ ft} = 0.66\ \frac{kip}{ft}$
due to 11 wood trusses	
Distributed load due to CMU walls, ww:	$ww := wm \cdot (8\ in \cdot 14\ ft) = 1.167\ \frac{kip}{ft}$
due to 8 inch thick walls and 14 ft tall walls	
Total distributed load, w:	$w := wr = 0.66\ \frac{kip}{ft}$
Round to...	$w := 0.7\ \frac{kip}{ft}$

ACI Bearing Strength

Table A1.2—Allowable distribution loads, unjointed aisles, uniform loading, and variable layout; PCA method

Slab thickness, in.	Working stress, psi	Critical aisle width, in.*	Allowable load, lb/ft²: At critical aisle width	At other aisle widths: 6 ft aisle	8 ft aisle	10 ft aisle	12 ft aisle	14 ft aisle
Subgrade $k = 50$ lb/in.³†								
5	300	5.6	610	615	670	815	1050	1215
	350	5.6	710	715	785	950	1225	1420
	400	5.6	815	820	895	1085	1400	1620
6	300	6.4	670	675	695	780	945	1175
	350	6.4	785	785	810	910	1100	1370
	400	6.4	895	895	925	1040	1260	1570
8	300	8.0	770	800	770	800	880	1010
	350	8.0	900	935	900	935	1025	1180
	400	8.0	1025	1070	1025	1065	1175	1350
10	300	9.4	845	930	855	850	885	960
	350	9.4	985	1085	1000	990	1035	1120
	400	9.4	1130	1240	1145	1135	1185	1285
12	300	10.8	915	1065	955	915	925	965
	350	10.8	1065	1240	1115	1070	1080	1125
	400	10.8	1220	1420	1270	1220	1230	1290
14	300	12.1	980	1225	1070	1000	980	995
	350	12.1	1145	1430	1245	1170	1145	1160
	400	12.1	1310	1630	1425	1335	1310	1330
Subgrade $k = 100$ lb/in.³†								
5	300	4.7	865	900	1090	1470	1745	1810
	350	4.7	1010	1050	1270	1715	2035	2115
	400	4.7	1155	1200	1455	1955	2325	2415
6	300	5.4	950	955	1065	1320	1700	1925
	350	5.4	1105	1115	1245	1540	1985	2245
	400	5.4	1265	1275	1420	1760	2270	2565
8	300	6.7	1095	1105	1120	1240	1465	1815
	350	6.7	1280	1285	1305	1445	1705	2120
	400	6.7	1460	1470	1495	1650	1950	2420
10	300	7.9	1215	1265	1215	1270	1395	1610
	350	7.9	1420	1475	1420	1480	1630	1880
	400	7.9	1625	1645	1625	1690	1860	2150
12	300	9.1	1320	1425	1325	1330	1400	1535
	350	9.1	1540	1665	1545	1550	1635	1795

Subgrade $k = 100$ lb/in.³†								
5	300	4.7	865	900	1090	1470	1745	1810
	350	4.7	1010	1050	1270	1715	2035	2115
	400	4.7	1155	1200	1455	1955	2325	2415
6	300	5.4	950	955	1065	1320	1700	1925
	350	5.4	1105	1115	1245	1540	1985	2245
	400	5.4	1265	1275	1420	1760	2270	2565
8	300	6.7	1095	1105	1120	1240	1465	1815
	350	6.7	1280	1285	1305	1445	1705	2120
	400	6.7	1460	1470	1495	1650	1950	2420
10	300	7.9	1215	1265	1215	1270	1395	1610
	350	7.9	1420	1475	1420	1480	1630	1880
	400	7.9	1625	1645	1625	1690	1860	2150
12	300	9.1	1320	1425	1325	1330	1400	1535
	350	9.1	1540	1665	1545	1550	1635	1795
	400	9.1	1755	1900	1770	1770	1865	2050
14	300	10.2	1405	1590	1445	1405	1435	1525
	350	10.2	1640	1855	1685	1640	1675	1775
	400	10.2	1875	2120	1925	1875	1915	2030
Subgrade $k = 200$ lb/in.³†								
5	300	4.0	1225	1400	1930	2450	2565	2520
	350	4.0	1425	1630	2255	2860	2990	2940
	400	4.0	1630	1865	2575	3270	3420	3360
6	300	4.5	1340	1415	1755	2395	2740	2810
	350	4.5	1565	1650	2050	2800	3200	3275
	400	4.5	1785	1890	2345	3190	3655	3745
8	300	5.6	1550	1550	1695	2045	2635	3070
	350	5.6	1810	1810	1980	2385	3075	3580
	400	5.6	2065	2070	2615	2730	3515	4095
10	300	6.6	1730	1745	1775	1965	2330	2895
	350	6.6	2020	2035	2070	2290	2715	3300
	400	6.6	2310	2325	2365	2620	3105	3860
12	300	7.6	1890	1945	1895	1995	2230	2610
	350	7.6	2205	2270	2210	2330	2600	3045
	400	7.6	2520	2595	2525	2660	2972	3480
14	300	8.6	2025	2150	2030	2065	2210	2480
	350	8.6	2360	2510	2365	2405	2580	2890
	400	8.6	2700	2870	2705	2750	2950	3305

*Critical aisle width equals 2.209 times the radius of relative stiffness.

†k of subgrade; disregard increase in k due to subbase.

Notes: Assumed load width = 300 in.; allowable load varies only slightly for other load widths. Allowable stress = 1/2 flexural strength.

Modulus of elasticity, Ec: $Ec := 57000 \cdot \sqrt{f'c \cdot psi} = \left(3.605 \cdot 10^{6}\right)\ psi$

Radius of Stiffness, Lr $Lr := \left(\dfrac{Ec \cdot t^3}{12 \cdot \left(1 - \mu^2\right) \cdot k}\right)^{0.25} = 53.889\ in$

k of subgrade (lbf/in^2) $k := 100\ \dfrac{lbf}{in^3}$

$t := 14\ in$ with 400 psi working stress and critical aisle widths is acceptable

Check Calculations:

Modulus of rupture, MR:	$MR := 9 \cdot \sqrt{f'c \cdot psi} = 569.21\ psi$
Allowable bending stress, Fb:	$Fb := 1.6 \cdot \sqrt{f'c \cdot psi} = 101.193\ psi$
Factor of safety, FS:	$FS := \frac{MR}{Fb} = 5.625$
Width, b:	$b := 12\ in$
Moment of inertia, I:	$I := \frac{1}{12} \cdot b \cdot t^3 = (2.744 \cdot 10^3)\ in^4$
Section Modulus, Sx:	$Sx := \frac{I}{0.5 \cdot t \cdot b} = 392\ \frac{in^3}{ft}$
Stiffness factor, λ:	$\lambda := \left(\frac{k \cdot b}{4 \cdot Ec \cdot I}\right)^{0.25} = 0.013\ \frac{1}{in}$
Coefficient, $B\lambda x$	$B\lambda x := 0.3224$
Allowable Wall Load:	$Pc_{allow} := 4 \cdot Fb \cdot Sx \cdot \lambda = (2.094 \cdot 10^3)\ \frac{lbf}{ft}$
	$Pe_{allow} := \frac{Fb \cdot Sx \cdot \lambda}{B\lambda x} = (1.624 \cdot 10^3)\ \frac{lbf}{ft}$

Using iterative excel calculation:

$tc_{min} := 13.68\ in$	center slab or key/dowled joints
$te_{min} := 16.77\ in$	near free edge of slab

Excel calculation pictures:

CALCULATIONS:

Design Parameters:

MR =	492.95	psi	MR = 9*SQRT(f 'c)
Fb =	87.64	psi	Fb = 1.6*SQRT(f 'c)
FS =	5.625		FS = MR/Fb
S =	128.00	in.^3	S = b*t^2/6
Ec =	3122019	psi	Ec = 57000*SQRT(f 'c)
b =	12.00	in.	b = 12" (assumed)
I =	512.00	in.^4	I = b*t^3/12
λ =	0.0208		λ = (k*b/(4*Ec*I))^(0.25)
Bλx =	0.3224		Bλx = coefficient from "Beams on Elastic Foundations" by M. Hetenyi

Near Center of Slab or Keyed/Doweled Joints:

Pc =	933.91	lb./ft.	Pc = 4*Fb*S*λ
	933.91		= 12.8*SQRT(f 'c)*t^2*(k/(19000*SQRT(f 'c)*t^3))^(0.25)

Near Free Edge of Slab:

Pe =	724.18	lb./ft.	Pe = Fb*S*λ/Bλx
	724.18		= 9.9256*SQRT(f 'c)*t^2*(k/(19000*SQRT(f 'c)*t^3))^(0.25)

Determine Minimum Slab Thickness for Given Wall Loading: (iterative solutions for t(min))

Iteration #	Eqn. for Pc	Eqn. for Pe	ti
1	-1757.59	-1773.17	1.00
2	-1735.26	-1755.86	1.25
3	-1711.77	-1737.65	1.50
4	-1687.29	-1718.66	1.75
5	-1661.91	-1698.98	2.00
6	-1635.72	-1678.67	2.25
7	-1608.79	-1657.80	2.50
8	-1581.19	-1636.39	2.75
9	-1552.94	-1614.49	3.00
10	-1524.10	-1592.12	3.25
11	-1494.70	-1569.33	3.50
12	-1464.77	-1546.12	3.75
13	-1434.34	-1522.52	4.00
14	-1403.43	-1498.55	4.25
15	-1372.06	-1474.22	4.50
16	-1340.25	-1449.55	4.75
17	-1308.02	-1424.56	5.00
18	-1275.38	-1399.25	5.25
19	-1242.35	-1373.64	5.50
20	-1208.95	-1347.74	5.75
21	-1175.17	-1321.55	6.00
22	-1141.05	-1295.09	6.25
23	-1106.58	-1268.36	6.50
24	-1071.78	-1241.38	6.75
25	-1036.66	-1214.14	7.00
26	-1001.22	-1186.66	7.25
27	-965.47	-1158.94	7.50
28	-929.43	-1130.99	7.75
29	-893.09	-1102.82	8.00
30	-856.47	-1074.42	8.25
31	-819.57	-1045.80	8.50

32	-782.39	-1016.98	8.75
33	-744.96	-987.95	9.00
34	-707.25	-958.71	9.25
35	-669.30	-929.28	9.50
36	-631.09	-899.65	9.75
37	-592.64	-869.83	10.00
38	-553.95	-839.83	10.25
39	-515.02	-809.64	10.50
40	-475.85	-779.27	10.75
41	-436.46	-748.73	11.00
42	-396.85	-718.01	11.25
43	-357.01	-687.12	11.50
44	-316.96	-656.06	11.75
45	-276.69	-624.84	12.00
46	-236.21	-593.45	12.25
47	-195.53	-561.90	12.50
48	-154.64	-530.19	12.75
49	-113.55	-498.33	13.00
50	-72.26	-466.32	13.25
51	-30.78	-434.15	13.50
52	10.89	-401.83	13.75
53	52.76	-369.37	14.00
54	94.81	-336.76	14.25
55	137.05	-304.01	14.50
56	179.47	-271.12	14.75
57	222.07	-238.08	15.00
58	264.84	-204.91	15.25
59	307.80	-171.61	15.50
60	350.92	-138.16	15.75
61	394.22	-104.59	16.00
62	437.69	-70.88	16.25
63	481.33	-37.05	16.50
64	525.13	-3.08	16.75
65	569.09	31.01	17.00
66	613.22	65.23	17.25
67	657.50	99.57	17.50
68	701.95	134.03	17.75
69	746.55	168.62	18.00
t(min) =	13.68	16.77	
	13.75	17.00	

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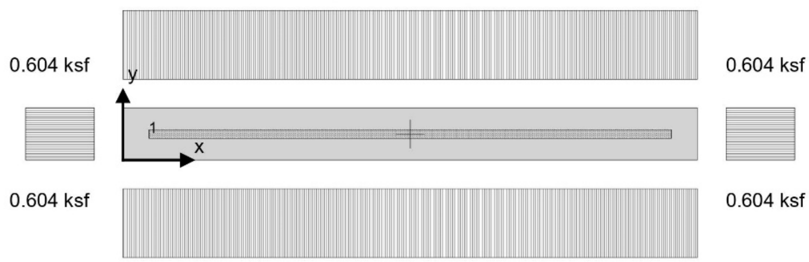
Foundation analysis & design (ACI318) in accordance with ACI318-14

Tedds calculation version 3.2.10

<u>FOOTING ANALYSIS</u>

Length of foundation	L_x = **44** ft
Width of foundation	L_y = **4** ft
Foundation area	$A = L_x \times L_y$ = **176** ft^2
Depth of foundation	h = **18** in
Depth of soil over foundation	h_{soil} = **18** in
Density of concrete	γ_{conc} = **150.0** lb/ft^3



Column no.1 details

Length of column	l_{x1} = **480.00** in
Width of column	l_{y1} = **8.00** in
position in x-axis	x_1 = **264.00** in
position in y-axis	y_1 = **24.00** in

Soil properties

Gross allowable bearing pressure	q_{allow_Gross} = **3** ksf
Density of soil	γ_{soil} = **120.0** lb/ft^3
Angle of internal friction	ϕ_b = **30.0** deg
Design base friction angle	δ_{bb} = **30.0** deg
Coefficient of base friction	$\tan(\delta_{bb})$ = **0.577**
Self weight	$F_{swt} = h * \gamma_{conc}$ = **225** psf
Soil weight	$F_{soil} = h_{soil} * \gamma_{soil}$ = **180** psf

Column no.1 loads

Dead load in z	F_{Dz1} = **15.0** kips
Live roof load in z	F_{Lrz1} = **15.0** kips
Snow load in z	F_{Sz1} = **20.0** kips
Wind load moment in x	M_{Wx1} = **28.0** kip_ft

Footing analysis for soil and stability

Load combinations per ASCE 7-16

1.0D (0.163)

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1.0D + 1.0L (0.163)
1.0D + 1.0Lr (0.192)
1.0D + 1.0S (0.201)
1.0D + 0.75L + 0.75Lr (0.185)
1.0D + 0.75L + 0.75S (0.192)
1.0D + 0.6W (0.168)
1.0D + 0.75L + 0.75Lr + 0.45W (0.188)
1.0D + 0.75L + 0.75S + 0.45W (0.195)
0.6D + 0.6W (0.102)

Combination 4 results: 1.0D + 1.0S

Forces on foundation

Force in z-axis $F_{dz} = \gamma_D * A * (F_{swt} + F_{soil}) + \gamma_D * F_{Dz1} + \gamma_S * F_{Sz1}$ = **106.3** kips

Moments on foundation

Moment in x-axis, about x is 0 $M_{dx} = \gamma_D * (A * (F_{swt} + F_{soil}) * L_x / 2) + \gamma_D * (F_{Dz1} * x_1) + \gamma_S * (F_{Sz1} * x_1)$ = **2338.2** kip_ft

Moment in y-axis, about y is 0 $M_{dy} = \gamma_D * (A * (F_{swt} + F_{soil}) * L_y / 2) + \gamma_D * (F_{Dz1} * y_1) + \gamma_S * (F_{Sz1} * y_1)$ = **212.6** kip_ft

Uplift verification

Vertical force F_{dz} = **106.28** kips

PASS - Foundation is not subject to uplift

Bearing resistance

Eccentricity of base reaction

Eccentricity of base reaction in x-axis $e_{dx} = M_{dx} / F_{dz} - L_x / 2$ = **0** in

Eccentricity of base reaction in y-axis $e_{dy} = M_{dy} / F_{dz} - L_y / 2$ = **0** in

Pad base pressures

$q_1 = F_{dz} * (1 - 6 * e_{dx} / L_x - 6 * e_{dy} / L_y) / (L_x * L_y)$ = **0.604** ksf

$q_2 = F_{dz} * (1 - 6 * e_{dx} / L_x + 6 * e_{dy} / L_y) / (L_x * L_y)$ = **0.604** ksf

$q_3 = F_{dz} * (1 + 6 * e_{dx} / L_x - 6 * e_{dy} / L_y) / (L_x * L_y)$ = **0.604** ksf

$q_4 = F_{dz} * (1 + 6 * e_{dx} / L_x + 6 * e_{dy} / L_y) / (L_x * L_y)$ = **0.604** ksf

Minimum base pressure $q_{min} = \min(q_1,q_2,q_3,q_4)$ = **0.604** ksf

Maximum base pressure $q_{max} = \max(q_1,q_2,q_3,q_4)$ = **0.604** ksf

Allowable bearing capacity

Allowable bearing capacity $q_{allow} = q_{allow\_Gross}$ = **3** ksf

q_{max} / q_{allow} = **0.201**

PASS - Allowable bearing capacity exceeds design base pressure

FOOTING DESIGN (ACI318)

In accordance with ACI318-14

Material details

Compressive strength of concrete f'_c = **4000** psi

Yield strength of reinforcement f_y = **60000** psi

Compression-controlled strain limit (21.2.2) ε_{ty} = **0.00200**

Cover to reinforcement c_{nom} = **3** in

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Concrete type	Normal weight
Concrete modification factor	λ = **1.00**
Column type	Concrete

Analysis and design of concrete footing

Load combinations per ASCE 7-16

1.4D (0.007)

1.2D + 1.6L + 0.5Lr (0.008)

Combination 2 results: 1.2D + 1.6L + 0.5Lr

Forces on foundation

Ultimate force in z-axis: $F_{uz} = \gamma_D * A * (F_{swt} + F_{soil}) + \gamma_D * F_{Dz1} + \gamma_{Lr} * F_{Lrz1}$ = **111.0** kips

Moments on foundation

Ultimate moment in x-axis, about x is 0: $M_{ux} = \gamma_D * (A * (F_{swt} + F_{soil}) * L_x / 2) + \gamma_D * (F_{Dz1} * x_1) + \gamma_{Lr} * (F_{Lrz1} * x_1)$ = **2442.8** kip_ft

Ultimate moment in y-axis, about y is 0: $M_{uy} = \gamma_D * (A * (F_{swt} + F_{soil}) * L_y / 2) + \gamma_D * (F_{Dz1} * y_1) + \gamma_{Lr} * (F_{Lrz1} * y_1)$ = **222.1** kip_ft

Eccentricity of base reaction

Eccentricity of base reaction in x-axis: $e_{ux} = M_{ux} / F_{uz} - L_x / 2$ = **0** in

Eccentricity of base reaction in y-axis: $e_{uy} = M_{uy} / F_{uz} - L_y / 2$ = **0** in

Pad base pressures

$q_{u1} = F_{uz} * (1 - 6 * e_{ux} / L_x - 6 * e_{uy} / L_y) / (L_x * L_y)$ = **0.631** ksf

$q_{u2} = F_{uz} * (1 - 6 * e_{ux} / L_x + 6 * e_{uy} / L_y) / (L_x * L_y)$ = **0.631** ksf

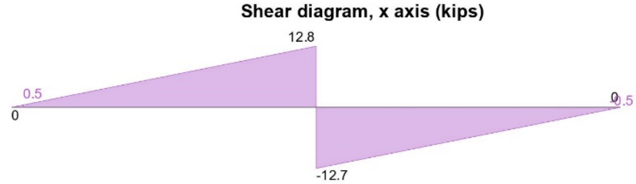
$q_{u3} = F_{uz} * (1 + 6 * e_{ux} / L_x - 6 * e_{uy} / L_y) / (L_x * L_y)$ = **0.631** ksf

$q_{u4} = F_{uz} * (1 + 6 * e_{ux} / L_x + 6 * e_{uy} / L_y) / (L_x * L_y)$ = **0.631** ksf

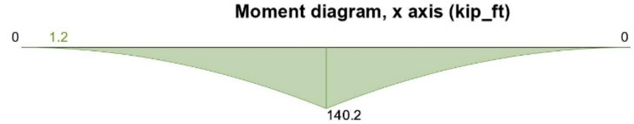
Minimum ultimate base pressure: $q_{umin} = \min(q_{u1},q_{u2},q_{u3},q_{u4})$ = **0.631** ksf

Maximum ultimate base pressure: $q_{umax} = \max(q_{u1},q_{u2},q_{u3},q_{u4})$ = **0.631** ksf

Shear diagram, x axis (kips)



Moment diagram, x axis (kip_ft)



Moment design, x direction, positive moment

Ultimate bending moment: $M_{u.x.max}$ = **1.159** kip_ft

Tension reinforcement provided: 2 No.4 bottom bars (41.5 in c/c)

Area of tension reinforcement provided: $A_{sx.bot.prov}$ = **0.4** in^2

Minimum area of reinforcement (8.6.1.1) $A_{s.min} = 0.0018 * L_y * h =$ **1.555** in^2

FAIL - Minimum area of reinforcement required exceeds area of reinforcement provided

Maximum spacing of reinforcement (8.7.2.2) $s_{max} =$ min(2 * h, 18 in) = **18** in

FAIL - Actual reinforcement spacing exceeds maximum permitted

Depth to tension reinforcement $d = h - c_{nom} - \phi_{x.bot} / 2 =$ **14.750** in

Depth of compression block $a = A_{sx.bot.prov} * f_y / (0.85 * f'_c * L_y) =$ **0.147** in

Neutral axis factor $\beta_1 =$ **0.85**

Depth to neutral axis $c = a / \beta_1 =$ **0.173** in

Strain in tensile reinforcement $\varepsilon_t = 0.003 * d / c - 0.003 =$ **0.25277**

Minimum tensile strain(8.3.3.1) $\varepsilon_{min} = 0.004 =$ **0.00400**

PASS - Tensile strain exceeds minimum required

Nominal moment capacity $M_n = A_{sx.bot.prov} * f_y * (d - a / 2) =$ **29.353** kip_ft

Flexural strength reduction factor $\phi_f = \min(\max(0.65 + 0.25 * (\varepsilon_t - \varepsilon_{ty}) / (0.005 - \varepsilon_{ty}), 0.65), 0.9) =$ **0.900**

Design moment capacity $\phi M_n = \phi_f * M_n =$ **26.418** kip_ft

$M_{u.x.max} / \phi M_n =$ **0.044**

PASS - Design moment capacity exceeds ultimate moment load

One-way shear design, x direction

Ultimate shear force $V_{u.x} =$ **0.489** kips

Depth to reinforcement $d_v = h - c_{nom} - \phi_{x.bot} / 2 =$ **14.75** in

Shear strength reduction factor $\phi_v =$ **0.75**

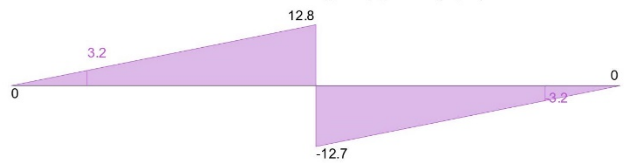
Nominal shear capacity (Eq. 22.5.5.1) $V_n = 2 * \lambda * \sqrt{(f'_c * 1\ \text{psi})} * L_y * d_v =$ **89.556** kips

Design shear capacity $\phi V_n = \phi_v * V_n =$ **67.167** kips

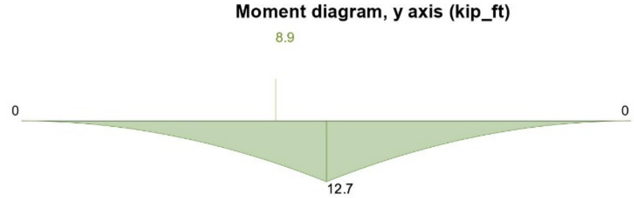
$V_{u.x} / \phi V_n =$ **0.007**

PASS - Design shear capacity exceeds ultimate shear load

Shear diagram, y axis (kips)



Moment diagram, y axis (kip_ft)



Moment design, y direction, positive moment

Ultimate bending moment $M_{u.y.max} =$ **8.858** kip_ft

Tension reinforcement provided 30 No.7 bottom bars (17.9 in c/c)

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Area of tension reinforcement provided	$A_{sy.bot.prov}$ = **18** in^2
Minimum area of reinforcement (8.6.1.1)	$A_{s.min}$ = 0.0018 * L_x * h = **17.107** in^2
	PASS - Area of reinforcement provided exceeds minimum
Maximum spacing of reinforcement (8.7.2.2)	s_{max} = min(2 * h, 18 in) = **18** in
	PASS - Maximum permissible reinforcement spacing exceeds actual spacing
Depth to tension reinforcement	d = h - c_{nom} - $\phi_{x.bot}$ - $\phi_{y.bot}$ / 2 = **14.063** in
Depth of compression block	a = $A_{sy.bot.prov}$ * f_y / (0.85 * f'_c * L_x) = **0.602** in
Neutral axis factor	β_1 = **0.85**
Depth to neutral axis	c = a / β_1 = **0.708** in
Strain in tensile reinforcement	ε_t = 0.003 * d / c - 0.003 = **0.05661**
Minimum tensile strain(8.3.3.1)	ε_{min} = 0.004 = **0.00400**
	PASS - Tensile strain exceeds minimum required
Nominal moment capacity	M_n = $A_{sy.bot.prov}$ * f_y * (d - a / 2) = **1238.553** kip_ft
Flexural strength reduction factor	ϕ_f = min(max(0.65 + 0.25 * (ε_t - ε_{ty}) / (0.005 - ε_{ty}), 0.65), 0.9) = **0.900**
Design moment capacity	ϕM_n = ϕ_f * M_n = **1114.698** kip_ft
	$M_{u.y.max}$ / ϕM_n = **0.008**
	PASS - Design moment capacity exceeds ultimate moment load
Footing geometry factor (13.3.3.3)	β_f = L_x / L_y = **11.000**
Area of reinf. req. for uniform distribution (CRSI)	A_{sreq} = ($M_{u.y.max}$ / (ϕ_f * f_y * (d - a / 2))) * 2 * β_f / (β_f + 1) = **0.262** in^2
	PASS - Reinforcement can be distributed uniformly
One-way shear design, y direction	
Ultimate shear force	$V_{u.y}$ = **3.156** kips
Depth to reinforcement	d_v = h - c_{nom} - $\phi_{x.bot}$ - $\phi_{y.bot}$ / 2 = **14.063** in
Shear strength reduction factor	ϕ_v = **0.75**
Nominal shear capacity (Eq. 22.5.5.1)	V_n = 2 * λ * $\sqrt{f'_c * 1\ psi}$ * L_x * d_v = **939.196** kips
Design shear capacity	ϕV_n = ϕ_v * V_n = **704.397** kips
	$V_{u.y}$ / ϕV_n = **0.004**
	PASS - Design shear capacity exceeds ultimate shear load
Two-way shear design at column 1	
Depth to reinforcement	d_{v2} = **14.406** in
Shear perimeter length (22.6.4)	l_{xp} = **494.406** in
Shear perimeter width (22.6.4)	l_{yp} = **22.406** in
Shear perimeter (22.6.4)	b_o = 2 * (l_{x1} + d_{v2}) + 2 * (l_{y1} + d_{v2}) = **1033.625** in
Shear area	A_p = $l_{x,perim}$ * $l_{y,perim}$ = **11077.790** in^2
Surcharge loaded area	A_{sur} = A_p - l_{x1} * l_{y1} = **7237.790** in^2
Ultimate bearing pressure at center of shear area	$q_{up.avg}$ = **0.631** ksf
Ultimate shear load	F_{up} = γ_D * F_{Dz1} + γ_{Lr} * F_{Lrz1} + γ_D * A_p * F_{swt} + γ_D * A_{sur} * F_{soil} - $q_{up.avg}$ * A_p = **8.594** kips
Ultimate shear stress from vertical load	v_{ug} = max(F_{up} / (b_o * d_{v2}),0 psi) = **0.577** psi
Column geometry factor (Table 22.6.5.2)	β = l_{x1} / l_{y1} = **60.00**
Column location factor (22.6.5.3)	α_s =**40**
Concrete shear strength (22.6.5.2)	v_{cpa} = (2 + 4 / β) * λ * $\sqrt{f'_c * 1\ psi}$ = **130.707** psi
	v_{cpb} = (α_s * d_{v2} / b_o + 2) * λ * $\sqrt{f'_c * 1\ psi}$ = **161.751** psi

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$v_{cpc} = 4 * \lambda * \sqrt{(f'_c * 1 \text{ psi})}$ = **252.982** psi

$v_{cp} = \min(v_{cpa}, v_{cpb}, v_{cpc})$ = **130.707** psi

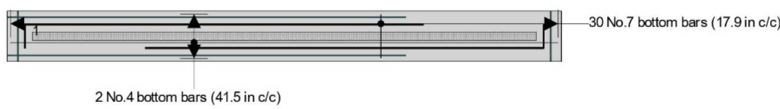
Shear strength reduction factor ϕ_v = **0.75**

Nominal shear stress capacity (Eq. 22.6.1.2) $v_n = v_{cp}$ = **130.707** psi

Design shear stress capacity (8.5.1.1(d)) $\phi v_n = \phi_v * v_n$ = **98.031** psi

$v_{ug} / \phi v_n$ = **0.006**

PASS - Design shear stress capacity exceeds ultimate shear stress load



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SLAB ON GROUND (ACI 360R)

In accordance with Guide to Design of Slab-on-Ground per ACI 360R-10

Tedds calculation version 1.0.02

Slab foundation-refer to Section 4.1 of ACI 360R

Design method

Design method publisher	**Portland Cement Association**

Materials and site properties

Slab thickness	h = **12** in
Specified compressive strength of concrete	f'_c = **4000** psi
Subgrade modulus	k = **100** lb/in^3

Wheel specifications

Axle type	**Single-axle**
Number of wheels at end of each axle	**Single wheel**
Wheel center to center spacing	S = **40** in

Loading details

Load location	**Interior**
Axle load	P_a = **30** kips
Safety factor	FS = **2**
Load contact area per wheel	A_c = **50** in^2
By iteration assumend trial thickness	h_{trial} = **10** in

The following output is based on the use of this trial thickness in the design charts in Appendix A of ACI 360R

Effective contact area per wheel (Fig. A1.2)	A_{c_eff} = **61.5** in^2

Slab thickness design

Modulus of rupture of concrete	$f_r = 9 * \sqrt{(f'_c * 1 \text{ psi})}$ = **569.2** psi
Concrete working stress	$f_{t\_allow} = f_r / FS$ = **284.6** psi
Slab stress / 1000 lb axle load	$f_t = f_{t\_allow} / (P_a / 1 \text{ kips})$ = **9.5** psi
Required slab thickness (Fig. A1.1)	h_{min} = **9.92** in
	h_{min} / h = **0.827**

PASS - Slab thickness is adequate to avoid live load-induced cracks

Crack control options

Reinforcement type	**Unreinforced**
Concrete type	**Typical concrete**
Actual joint spacing	S_{joint} = **15.0** ft
Recommended joint spacing (Fig.6.6)	S_{joint_max} = **22.3** ft
	$S_{joint} / S_{joint\_max}$ = **0.674**

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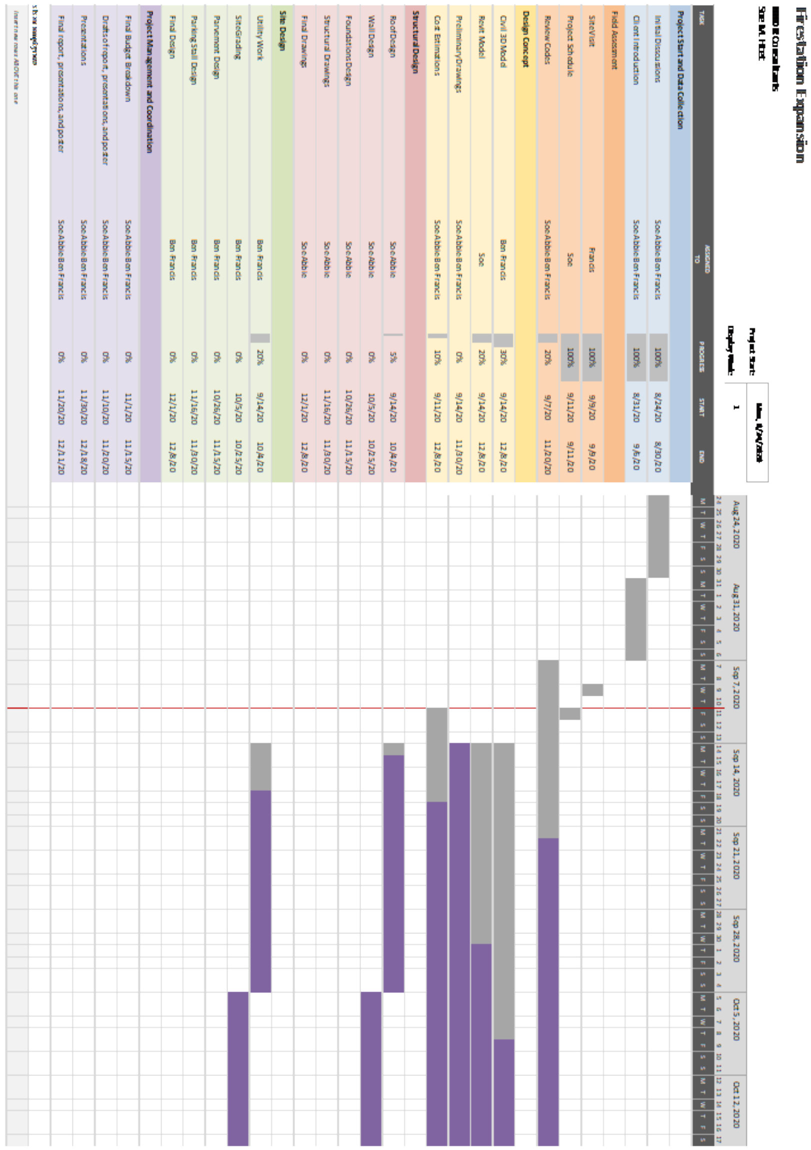
PASS - Joint spacing is adequate to avoid live load-induced cracks

Appendix F: Gantt Chart



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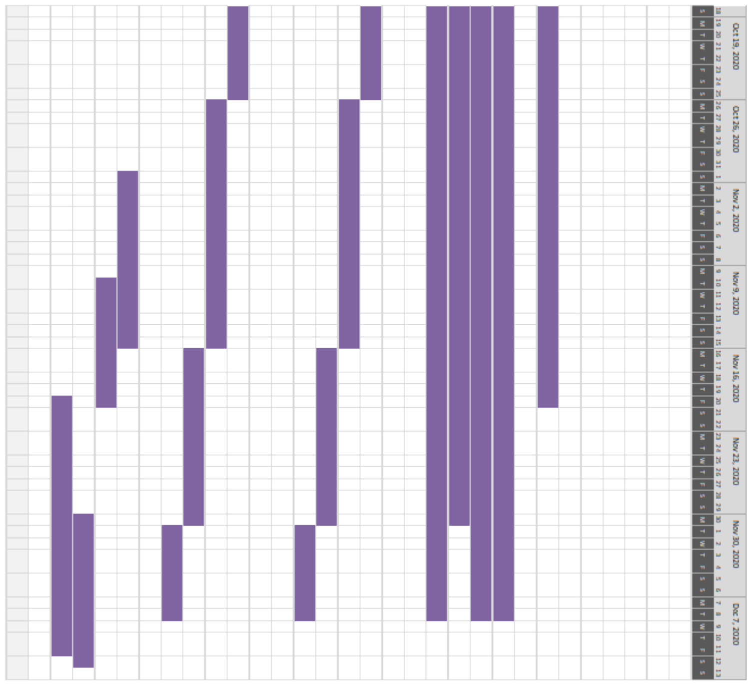
Oct 19, 2020
Oct 26, 2020
Nov 2, 2020
Nov 9, 2020
Nov 16, 2020
Nov 23, 2020
Nov 30, 2020
Dec 7, 2020

Appendix G: Bibliography

AASHTO LRFD bridge design specifications. (2017). Washington, DC: American Association of State Highway and Transportation Officials.

AISC 15th Edition Interactive Reference List. (n.d.). Retrieved November 20, 2020, from https://www.aisc.org/publications/steel-construction-manual-resources/15-interactive-reference-list/

“ASCE 7-16 Minimum Design Loads and Associated Criteria for Building and Other Structures.” (n.d.). *American Society of Civil Engineers (ASCE)*, <https://www.asce.org/asce-7/> (Nov. 20, 2020).

Building code requirements for reinforced concrete (ACI 318-89) and commentary--ACI 318R-89. (1992). Detroit: The Institute.

“Footings and Foundations.” (n.d.). *Food and Agriculture Organization of the United Nations*, FAO Fiat Panis, <http://www.fao.org/3/s1250e/S1250E0h.htm> (Nov. 19, 2020).

“IBC® 2015 Code and Commentary Combo, Vol. 1 & 2.” (2015). International Code Council, <https://iccsafe.org/2015-ibcr-code-and-commentary-combo-vol-1-2-42682.html> (Nov. 20, 2020).

Iowa DOT Design Manual. (n.d.). Retrieved November 20, 2020, from https://iowadot.gov/design/design-manual

Iowa Statewide Urban Design and Specifications. (n.d.). Retrieved November 20, 2020, from https://iowasudas.org/

NDS®, National Design Specification® for wood construction with commentary. (2015). Leesburg, VA: American Wood Council.

NFPA 70®. (n.d.). Retrieved November 20, 2020, from https://www.nfpa.org/codes-and-standards/all-codes-and-standards/list-of-codes-and-standards/detail?code=70

Pavement Designer. (2018). Retrieved November 20, 2020, from https://www.pavementdesigner.org/

“Recommended Home Insulation R–Values.” (2020). <https://www.energystar.gov/campaign/seal_insulate/identify_problems_you_want_fix/diy_checks_inspections/insulation_r_values> (Nov. 20, 2020).

Visual Interior Tool. (2020). Retrieved November 20, 2020, from https://www.visual-3d.com/tools/interior/

SDPWS Special design provisions for wind & seismic with commentary, 2015 edition. (2015). Leesburg, VA: American Wood Council.

United States Inflation Calculator. (2009, April 15). Retrieved December 03, 2020, from https://www.usinflationcalculator.com/page/18/