Final Design Report

Bondurant Public Works Facility

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Section I Executive Summary

Bondurant is a growing city in central Iowa, northeast of Iowa's capital Des Moines. From 2010 to 2020 Bondurant experienced a 90% increase in population, and this trend is expected to continue. With the population growing rapidly, it became clear that to properly serve their community, the City of Bondurant would need to expand their Civic Campus. Bondurant's new Civic Campus will include buildings for law enforcement, emergency services, and public works.

Our team of four civil engineering students from the University of Iowa provided architectural and structural design services for the new Public Works Facility that is included in Bondurant's Civic Campus Master Plan. The Public Works Facility served as our capstone project and was directed under the supervision of University of Iowa faculty member Christopher Stoakes, Ph.D.

The City of Bondurant, IA requested an architectural and structural design for a new public works facility which will be part of their new Civic Campus on the city's east side. The city currently uses a facility of about 15,000 SF and a secondary building for excess vehicle storage. Their current facility does not have enough space to store all their vehicles inside or allow easy access. This issue will only get worse as the city acquires more vehicles to address its growing needs. The new facility's primary use will be storage for the Public Works Department's numerous vehicles and equipment and anticipating and meeting the department's capacity needs in the future. Much of the new facility's storage area will be designed as a space to park vehicles with driving lanes so the vehicles can enter and exit the building easily.

It was important to consider the constraints, challenges and impacts of the new Public Works Facility. We identified three major constraints for the design. First the design must fit on the designated 7.91-acre site within the Bondurant Civic Campus. The facility also had to be designed to house 20-25 employees. Lastly the design had to match the aesthetic of the Bondurant Emergency Services building. The biggest design challenge was arranging vehicle storage so that all vehicles can freely move around the facility and be easily accessed while parked in their designated spots. Having a distinct flow of traffic while also fitting spots for a minimum of 30 vehicles was a large challenge for the design. Another challenge is that the largest vehicle in the Public Work's fleet is 30ft long, so the storage area needed to be designed to handle vehicles of that size.

The team developed three design alternatives for the facility. All alternatives featured a warehouse with vehicle storage space, a wash bay, and a mechanics bay in addition to an office space with a conference room, locker rooms, a training room, a reception area, open office space, and closed offices. The first design alternative featured a 62,250 sq-ft building with approximately 30% of this area dedicated to office space and employee amenities. This alternative had areas sticking out of the north and south sides which worked to completely hide back-of-house operations from the front view on Pleasant Street. Both alternative one and alternative two had a one-way north to south traffic pattern in the warehouse. These alternatives also had the office and mechanic bay on the west (front) side of the facility.

Alternative two was a completely rectangular building made of 60,000 ft^2. This alternative offered a condensed office area with only 10% of the building's area dedicated to office space. Alternative 2 provided a basic layout that will be easy to construct and maneuver through. It was also the alternative with the smallest square footage, which provides the lowest construction cost.

At 70,000 sq-ft, alternative three was the largest of the alternatives and differed from the others by having a two-way, east-to-west, traffic pattern. Alternative three placed the office area on the north wall of the building and the mechanic's bay in the northeast corner of the facility. After obtaining feedback from the client on these three alternatives, the team worked to combine all preferred features into one design.

The final facility layout was a 68,418 sq-ft facility within the designated 7.91-acre site within Civic Campus. The facility had one vehicle entrance on the east side and two vehicle exits on the south side creating an L shaped one-way traffic pattern. The vehicle storage area contains three rows of parking spots and two driving lanes. The warehouse also includes restrooms, a mechanics bay with three separate entrances/exits, a chemical storage room and an individual wash bay.

The office space is positioned in the northwest corner of the facility and is made up of 8,860 sq-ft dedicated to employee amenities and workspace. The office area contains an enclosed reception area where the public will enter the facility. Nine enclosed offices and 10 cubicles make up the workspace for the employees. The office area was designed with extra space for more enclosed offices to be added in the future as needed. The space has a large break room with space for trainings, as well as a large conferences room for group meetings. Both men's and women's locker rooms were included and a private first aid room. The utility room for the facility is also included in the office space.

Our team compiled a construction cost estimate for the entire Public Works Facility project. The cost estimate was prepared in two sections. First the cost of materials and labor for the structural elements was calculated using unit costs from Heavy Construction Cost with RS Mean data and quantities taken from Revit. Next, the cost of materials and labor for the architectural, mechanical, electrical, and plumbing systems was calculated using Square Foot Costs with RS Mean Data. A square foot cost method was used in both the office and the warehouse areas. The material and construction cost were added together then a 10% contingency was added along with a 20% administrative cost. The final total project cost was estimated to be $20,417,631.

The new Public Works Facility is tailored to the community's needs. The new design improved the work environment by allowing an easier flow of vehicle traffic throughout the storage, a separate office area, and outdoor material storage. These changes will help the department operate at a higher level of service for their community.

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Section II Organization Qualifications and Experience

1. Organization and Design Team Description

The team is comprised of four senior Civil Engineering students from the University of Iowa. As part of our capstone design class, we were tasked to complete the architectural and structural design for the Bondurant Public Works Facility.

Luke Maharry was the project manager and led the team throughout the project. Luke was responsible for facilitating work to the team members and led the organization throughout the complete design process.

Olivia Marchiori was the design specialist for the team and led the structural, architectural, and visual designs for the project. Olivia modeled all the architectural & structural features and completed the project's construction cost analysis.

Avery Liss was a team design specialist and helped with the facility's structural design as well as report production.

Braeden Smith was the technology specialist for the team and created files and documents for the project.

Section III Design Services

1. Project Scope

The city of Bondurant, IA requested an architectural and structural design for a new public works facility which will be part of their new Civic Campus on the city's east side. The city currently uses a facility of about 15,000 SF and a secondary building for excess vehicle storage. Their current facility does not have enough space to store all their vehicles inside or allow them to be easily accessed. This issue will only get worse asmore vehicles are acquired to address the city's growth. Additionally, the location of their current facility is located in a residential area which makes it inconvenient. To correct these issues, they requested a larger facility with an area in the estimated range of 58,000 to 64,000 SF as stated in the Request for Proposal to be placed in the 7.91-acre site dedicated to them according to the Civic Campus Master Plan. The building's location on the site and its shape should be strategic in the way they are used to hide the back-of-house operations from the highway and residential street as requested by the clients. Its aesthetic should match the future Emergency Services Building currently in design by OPN Architects.

The new facility's primary use will be storing the Public Works Department's numerous vehicles and equipment. Much of the new facility's storage area should be designed as a space to park vehicles with driving lanes so vehicles can enter and exit the building easily via garage doors. Vehicles should enter on the east side and exit on the south to avoid winds from the north and stay out of public view. The department currently has about 20 vehicles and expects to have up to 30 vehicles within the next five to ten years. The design should have enough parking stalls to house all the expected vehicles in an organized manner. The current facility does not have a mechanic's bay or indoor wash bay. For the new facility, an indoor wash bay and outdoor wash bay should be included in the designs. Additionally, a space inside the storage area should be set aside for a mechanic's bay that can be accessed via garage doors. Another section of the storage area should be used for excess floor storage including storage racks along the walls and a chemical storage area located near the mechanic's bay. The client also requested a salt storage facility for about 4,000 tons of salt and a covered storage area for other materials. These should be towards the east side of the site.

A smaller portion of the building will be used for employees' office space. There are nine employees operating the current facility, but that number is expected to grow to 25 employees at full buildout. The office space needs to be large enough to house at least nine enclosed offices. The office area also needs to include a reception room, a conference room, men's and women's locker rooms, a break room, a mechanical room, storage rooms, a personal care room, and two single stall bathrooms.

The project required the programming and layout of the building's interior spaces, the selection of building materials, the structural design of the building, meeting the client's aesthetics goals, creation of a 3D model, and renderings of the final design. A construction cost estimate including contingency and construction administration services.

2. Work plan

Our team created a Gantt Chart as a project management tool to keep the team on track to complete the final design by our deadline. The column on the left listed all the major tasks that needed to be completed during the duration of the Public Works Facility project. Under the middle column is our team member that led the completion of each task. On the right was the schedule for all the tasks. The schedule showed the order in which each task was completed and how long each task was predicted to take to be completed. The Gantt Chart helped our team stick to a specified timeline for each task and prevented our team from falling behind schedule.

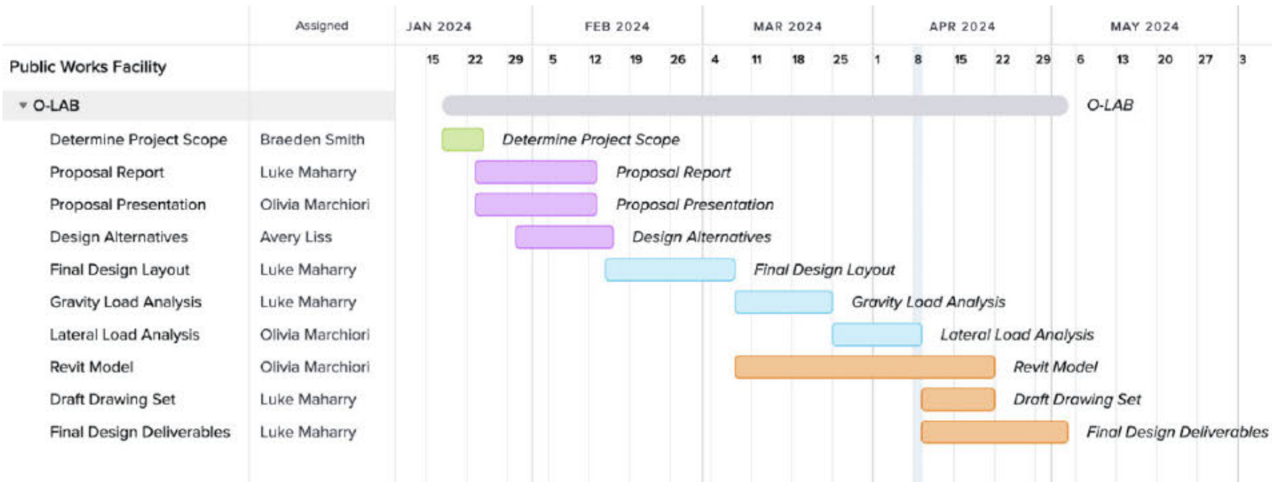


Figure 1. Public Works Facility Design Phase Gantt Chart

3. Methods and Design Guides

The Public Works Facility design complies with the City of Bondurant Code of Ordinances. Bondurant's Code of Ordinances is an adoption of the International Building Code (IBC), 2021 edition. The Allowable Strength Design (ASD) method, in accordance with ASCE/SEI 7-22, was used to complete the facility's structural design. The steel columns and girders were designed using the American Institute of Steel Construction's Manual (AISC) *Steel Construction Manual,* 16th Ed. Cast-in-place structural concrete members were designed using the *Building Code Requirements for Structural Concrete* (ACI 318-19) from the American Concrete Institute. Precast concrete members were designed using the PCI Design Handbook, *Precast and Prestressed Concrete,* 4th Ed. The roof framing system of the equipment area was developed in accordance with Nucor Vulcraft's design guides and specifications. Similarly, the roof framing system of the office area was developed using the prefabricated design specifications from ClarkDietrich. The cast-in-place foundations and footings were sized in accordance with chapter 18 of the IBC, with a presumptive allowable soil pressure of 15,000 pounds per square inch.

Section IV Constraints, Challenges, and Impacts

1. Constraints

A. Site & Building

a. The building must be designed in a way that would cover the back-of-house works.
b. Site and building limited to 7.91-acre lot.
c. The building must match the exterior architectural aesthetics of the Civic Campus, specifically the Emergency Services Building.

B. Office Space
    a. The office area must have capacity for 20-25 people at full buildout.
    b. Employee entrances must be separate from the public entrance.
    c. Must have a conference room that can accommodate 10-15 people for public meetings.
    d. Needs a men's and women's locker room that can be easily accessed from the warehouse and office.
    e. Must include a medical care room.

C. Warehouse
    a. The warehouse must have capacity for 30 large vehicles (ranging from pickup trucks to large bulldozers) including room for their additional equipment.
    b. Mechanics bay must accommodate vehicles as large as firetrucks and school buses along with the additional equipment that is needed.
    c. There must be a one-way traffic lane for vehicles to enter and exit the building.
    d. There cannot be garage doors on the north face of the building.

2. Challenges

A. Office area must have excess space to accommodate future growth.
B. There will be vehicles up to 40 ft in length that can move through (and be stored in) the warehouse and mechanic's bay.
C. Office space and warehouse must be designed to adequately accommodate any safety concerns such as fire or tornado emergencies.
D. The whole building must be within the city's budget to build while containing all requests from the client.
E. The building must block the back-of-house works while maintaining the architectural requirements of the civic campus.

3. Societal Impact within the Community/ State of Iowa

A. The new building will increase the Public Works Department's efficiency which will allow them to better serve their community. For example, it will allow snowplows to be mobilized easily, which will result in faster snow removal and enhanced public safety.
B. There are houses up the road from the planned building site. The residents will have to deal with the site's development and construction. They will also have to adapt to the influx of traffic on the roads and intersections near the site. The Public Works Facility and the rest of the campus will change the scenery around these homes. What is now an open field will eventually be a fully developed commercial site. To address the change in scenery, the buildings are designed to be aesthetically pleasing. We also designed the Public Works Facility to hide the back-of-house operations from the adjacent roads, so it is not as visible to the public.

C. The construction of this building will create more jobs for the community to fill (which can lead to economic growth).
D. Local businesses and services sourced locally, leading to increased business for suppliers, contractors, subcontractors, and vendors in the community can see an increase in business. This can boost the local economy and support small businesses.
E. Construction projects can have environmental consequences such as habitat disruption, air and water pollution, and increased energy consumption. Implementing sustainable construction practices and mitigating environmental impacts are essential to minimize negative effects on the community and ecosystem.
F. The construction process can bring together diverse stakeholders, including government agencies, community organizations, and residents, to collaborate on a shared goal. This can foster social cohesion, networking, and community pride.

Section V Alternative Solutions That Were Considered

In developing alternatives for a new Public Works Wuilding, three distinct designs were proposed, each tailored to address the current and future needs of the public works department. Design Alternative 1 prioritized ample storage space, offering 62,550 square feet with vehicular access from all sides, while also concealing back-of-house operations from residential view. Design Alternative 2, however, focused on a more compact layout of 60,000 square feet, providing a balance between office and storage space and emphasizing cost-effectiveness and ease of construction. In contrast, Design Alternative 3 emerged as the largest option at 70,000 square feet, emphasizing expansive office facilities and a warehouse layout conducive to efficient workflow. Each alternative presents unique advantages, ranging from scalability and cost efficiency to enhanced office amenities and operational convenience, reflecting a comprehensive exploration of possibilities to meet the client's needs.

1. Design Alternative I

Design Alternative I provided adequate space for the current demand of the employees and vehicles of the public works department while providing space to grow into. The design occupied a usable space of 62,550 square feet, with 70% designated for vehicle/ equipment storage, as shown in Figure 3. This allotted space can store more than twice the number of vehicles currently used by the public works department while also providing a large maintenance bay to operate equipment. The design provided vehicular access to the storage space from all sides of the building and allowed for safe travel throughout the facility. The internal parking configuration prioritized one-way traffic through the facility, entering from the north access and exiting through the south access The west end of the facility housed all the assembly spaces such as a locker room, break room, reception area, and various office spaces This alternative hid the back-of-house operations from the residential street view from Pleasant St. SE. Alternative I also provided a general site layout plan shown in Figure 2. The site plan is based on the Civic Campus Master Plan for the City of Bondurant and displayed locations for outdoor storage areas and a wash bay for equipment. This was done to provide the client with an alternative similar to what they already imagined.

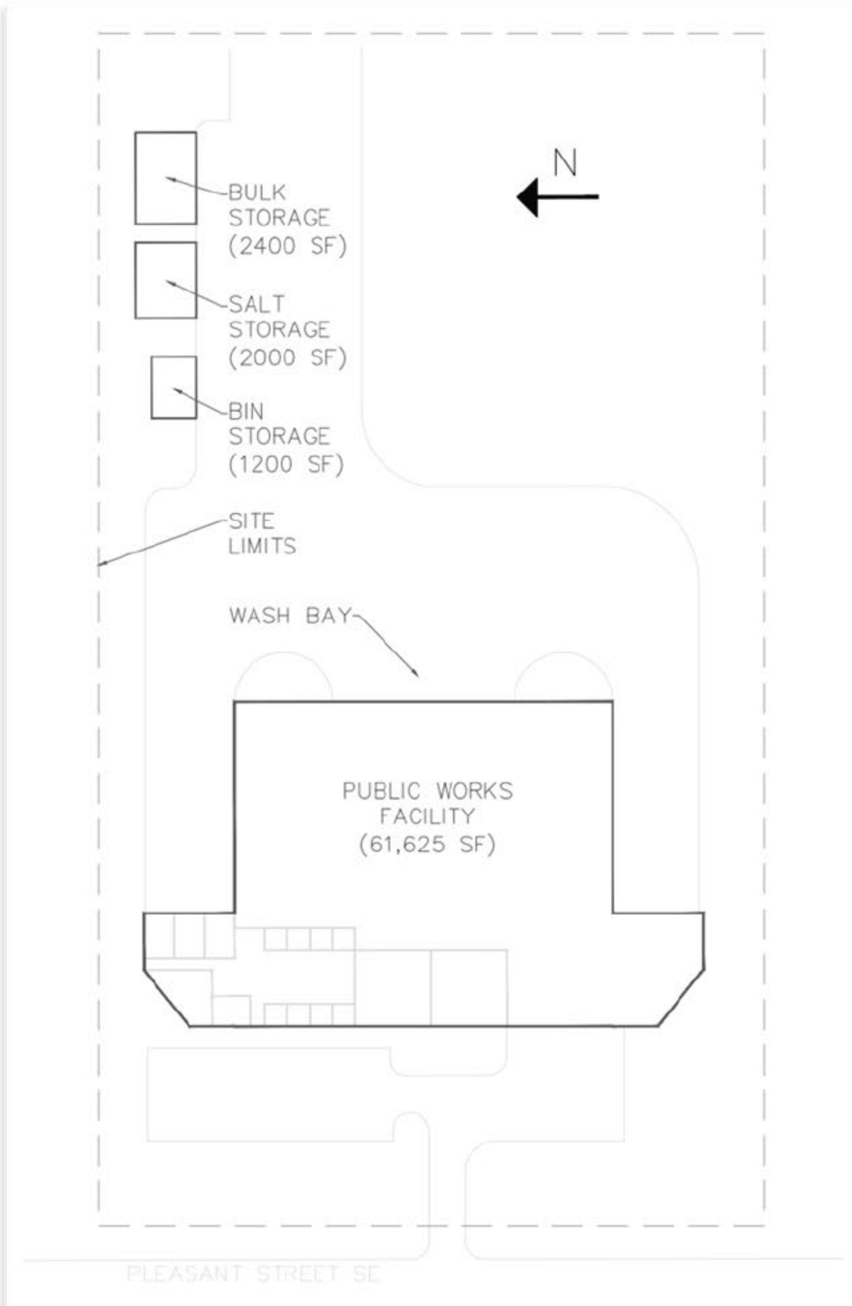


Figure 2. Design Alternative I – Proposed Site Plan

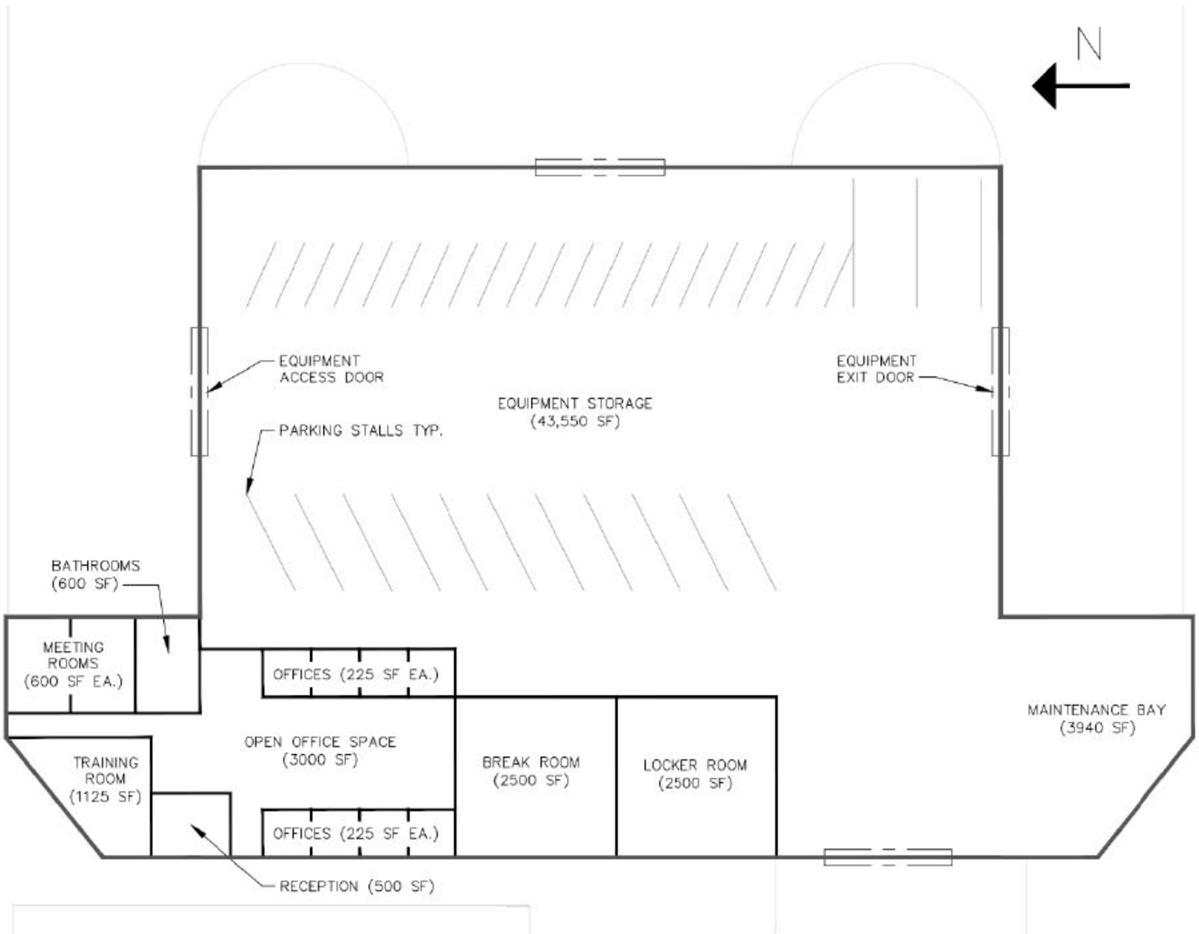


Figure 3. Design Alternative I – Proposed Floor Plan

2. Design Alternative II

Design alternative II is the smallest of the three alternatives. It proposed 60,000 square feet to meet Bondurant's staff and equipment capacity concerns in a simple compact layout. The facility is located on the west side of the site as seen in Figure 4. 6,600 square feet were dedicated as office and community space for the employees, while the other 53,400 square feet were dedicated for vehicle and equipment storage. The design allowed vehicles to enter from the north and exit to the south side of the facility using a one-way driving lane. The front of the facility faced towards west and provided cover to block the back-of-house operations including the outdoor storage facilities and the wash basin. The 6,600 square feet set aside for the employees was in the northwest corner of the facility and was divided into multiple rooms, shown in Figure 5. The area included private offices, space for additional cubicles, two conference rooms, a break room, two unisex bathrooms, a reception area, and locker rooms for both men and women. Alternative II provided a very basic layout that will be easy to construct and maneuver through. Due to its smaler square footage, it also has the lowest construction cost.

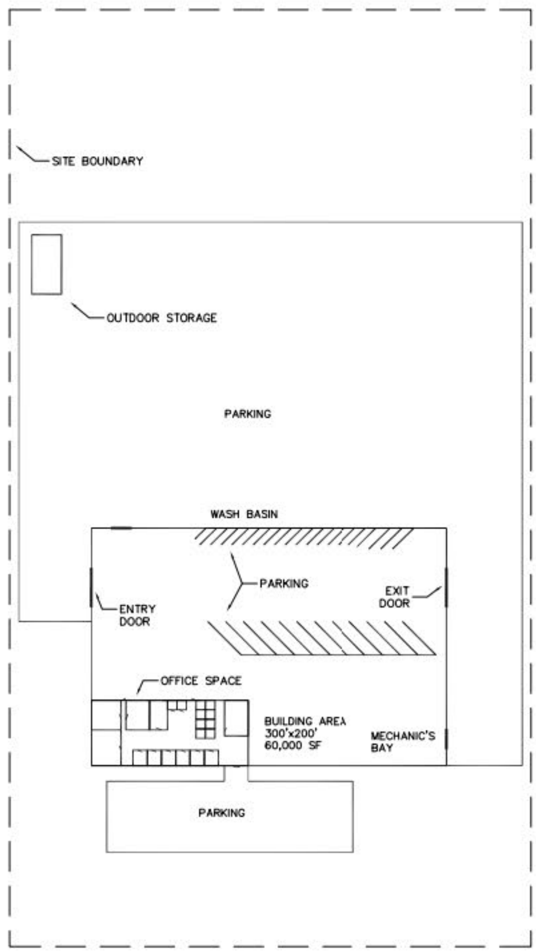


Figure 4. Design Alternative II – Proposed Site Plan

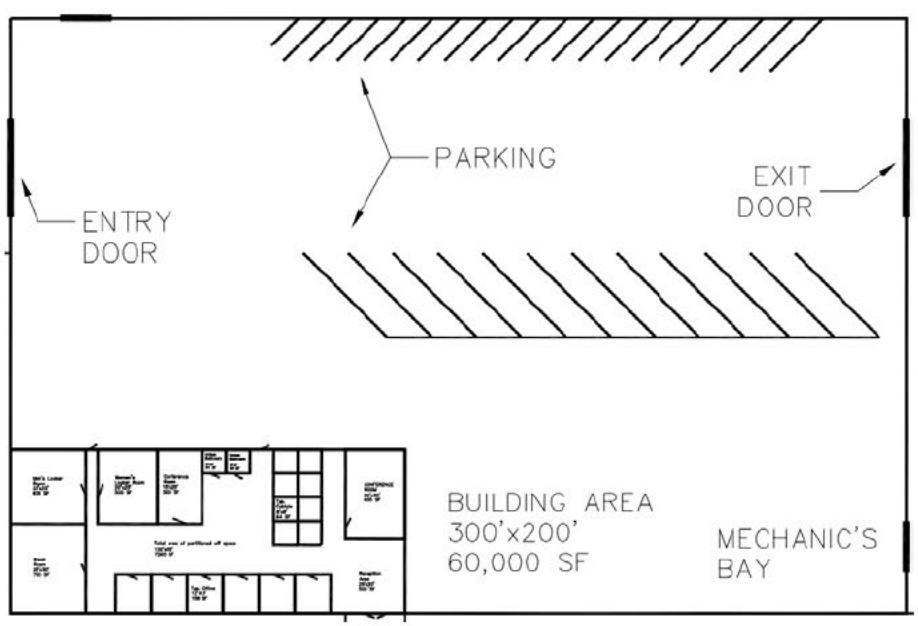


Figure 5. Design Alternative II – Proposed Floor Plan

3. Design Alternative III

Design alternative III proposed a larger and more unique layout compared to alternative I and II. At 70,00 square feet, this facility was positioned on the west side of the site as seen in Figure 6. The emphasis on this alternative was having a large office space of 14,00 square feet that has adequate space for the Public Works Department to grow into. As seen in Figure 7, it featured ten enclosed offices, 2,600 square feet for additional office space to grow into, multiple conference and meeting rooms, two private bathrooms in addition to the bathrooms located in the locker rooms, a training room, break room and locker room. The locker rooms' layout provided convenience for maintenance workers as they would have direct access from the office and warehouse. The office section of the building was completely separated from the equipment storage to provide unimpeded space from the equipment area. The 56,000-square-foot warehouse featured two large overhead doors on the east and west ends, which provided two-way traffic in and out of the facility. This would allow workers to access the road and facility quicker than having to drive through the entire campus. The maintenance bay's location was intentional as all vehicles and equipment could directly access it through the east side bay doors.

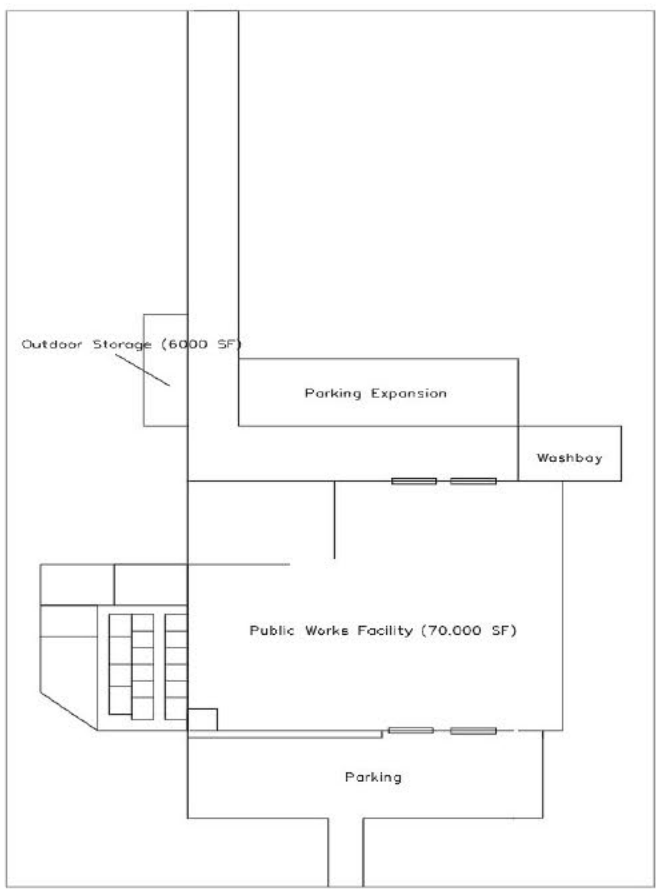


Figure 6. Design Alternative III – Proposed Site Plan

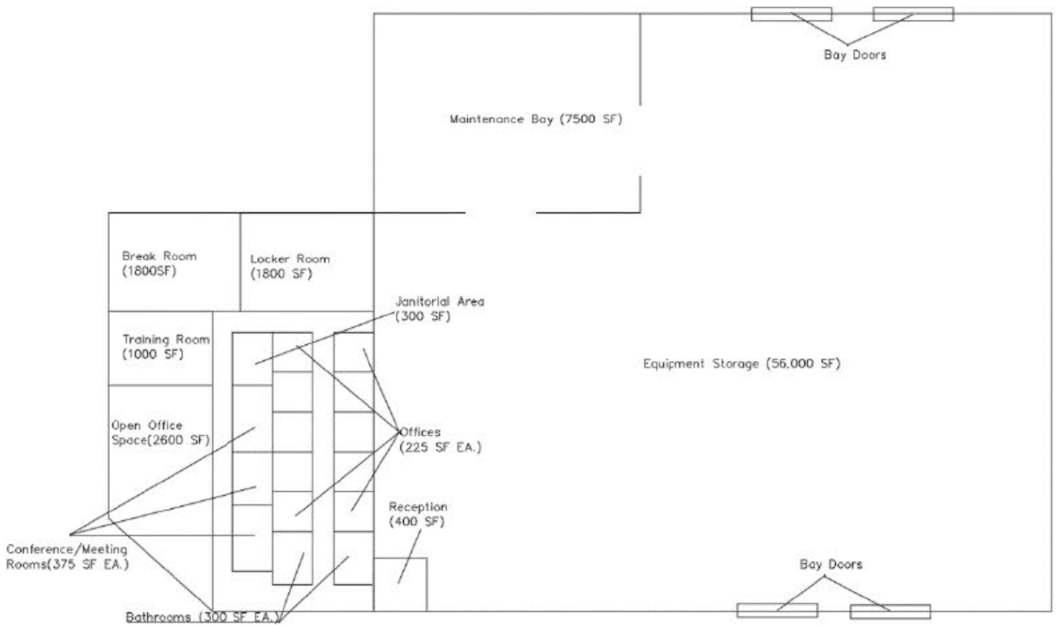


Figure 7. Design Alternative III – Proposed Floor Plan

Section VI Final Design Details

1. Architectural Design

The final architectural design we recommend incorporates features from Design Alternatives I and II identified during a meeting with the project stakeholders. The client chose Alternative I as the basis for the footprint of the building. Stating that the frontal extensions would provide the best exterior aesthetic and block the most back-of-house work and equipment. The client also expressed concerns about having vehicular openings on the north side of the building due to strong northern winds in the winter season. They stated the preference for one-way traffic lanes in the equipment area with no access/exit along the west side of the facility. Therefore, the access point was moved to the east side of the building, and two exits were designed along the south wall to provide an efficient exit from the building. The client also expressed the need for separate access to the mechanic's bay instead of being accessed only from within the facility. To accommodate this, the mechanics bay was extended from its location in Alternative I, and separate doors were provided. Additionally, the client clarified their need for both an interior and exterior wash bay adjacent to one another. The wash bays were kept in their proposed location along the east side of the facility with a portion of the area enclosed and attached to the facility.

The client decided they preferred the office layout of Alternative II. They liked the compact layout of the office space and stated they would not need as much space as is provided in alternatives one and three.

The final office layout was consolidated into the northwest corner, while keeping all previously mentioned rooms and amenities. An additional room was added for the personal care of the employees. Another addition to the office was the inclusion of an employee entrance toward the office space's north side. Using Autodesk's Revit, we were able to model our final design and get rendering of the facility. A rendering of the facility without a roof can be seen in Figure 8. This rendering helps showcase the building and the individual spaces within it. Architectural floors plan of the final layout can be seen in Figure 9 and 10. The final layout was designed to accommodate the full anticipated capacity of Bondurant's Public Works Department. The layout incorporates all the client's wants and needs into one building. Due to the resiliency of the structural materials is expected to last for more than 50 years.

Figure 8. Architectural Design – Final Design Plan View Rendering

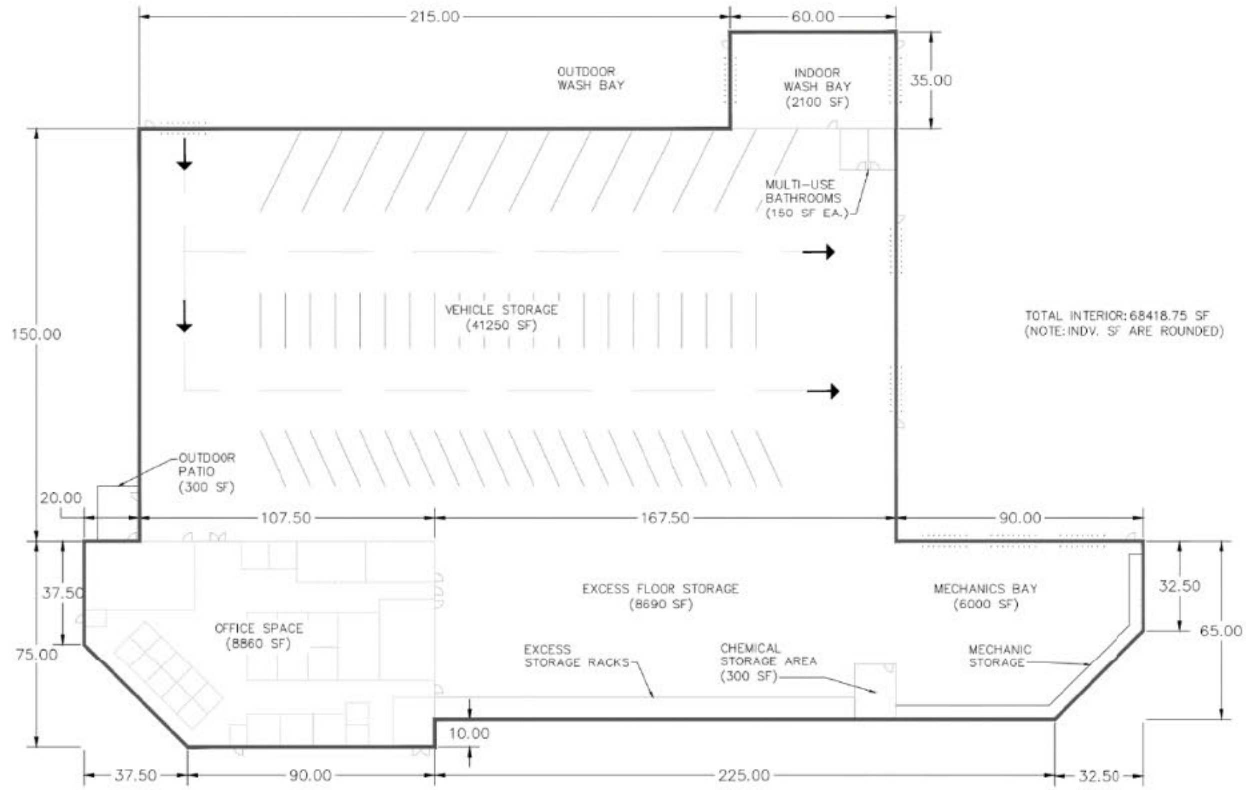


Figure 9. Final Design of the Public Works Facility

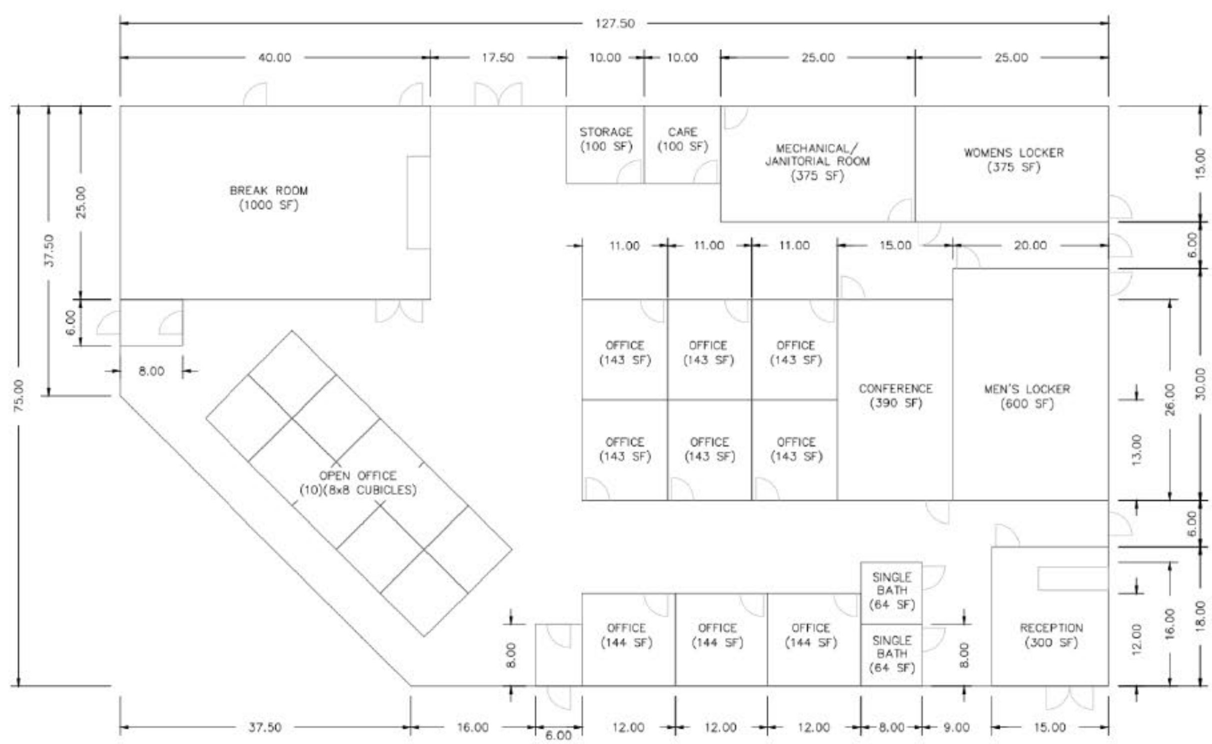


Figure 10. Final Design of the Public Works Facility – Office space

The architectural finishes were determined for both the storage area and the office area. The facility's storage area was left relatively unfinished, with the floor being concrete slab and the roof joists left exposed. The office area was fully finished. The floors are mostly carpet, except for tile and wood in a few places. Tile is used in the bathrooms, the locker rooms, the janitors' closet, and mechanical room. Wood is used for the break room and reception area's floors. The office was modeled with furniture to help better visualize the space. Large windows, exterior wood paneling, and steel overhangs were added to match the aesthetic of the Emergency Services Building, which can be seen in Figure 11 below.

Figure 11. Architectural Design – Final Design Northwest Exterior Rendering

2. Structural Design

Steel and concrete were selected as the primary materials for the facility's structural design. These materials are the most used in building construction and are readily available in large quantities. Steel was selected as the material for the roof members because the roof is a large system and required a magnitude of strength that steel could provide. The roof members in the storage area were placed strategically to provide the largest amount of uninterrupted space possible. The office area roof members were placed to match the room walls' layout. Precast concrete was chosen for the exterior walls to match the aesthetic of the Emergency Services Building. Once the materials were chosen, the elements were sized by completing both a gravity and lateral load analysis. They were initially sized using the gravity loads and then checked to verify that they would withstand the lateral load.

A. Gravity Load Analysis:

The gravity load analysis included the sizing of joists, girders, columns, footings, and bearing walls. The various members are labeled in Figures 12 and 13. The roof members were analyzed using ASD load combinations, the Steel Construction Manual, the Vulcraft Steel Joists & Joist Girder Manual, and the Clark-Dietrich TradeReady Steel Joist Tables.

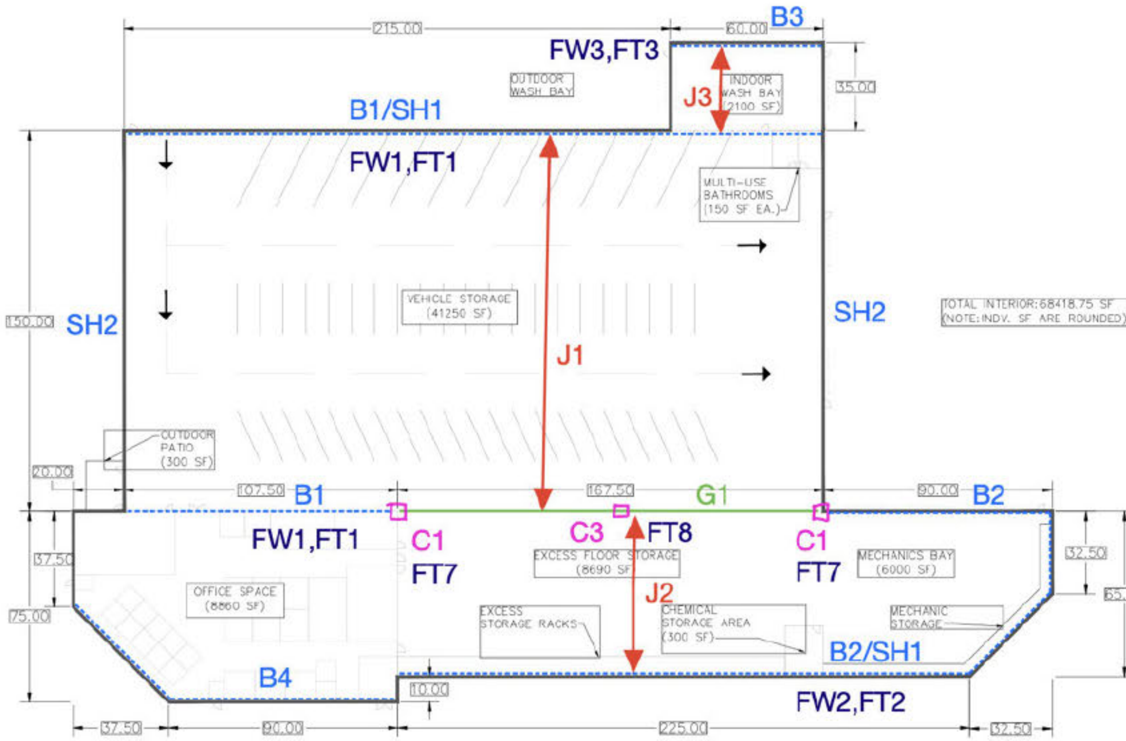


Figure 12. Labeled Member Diagram

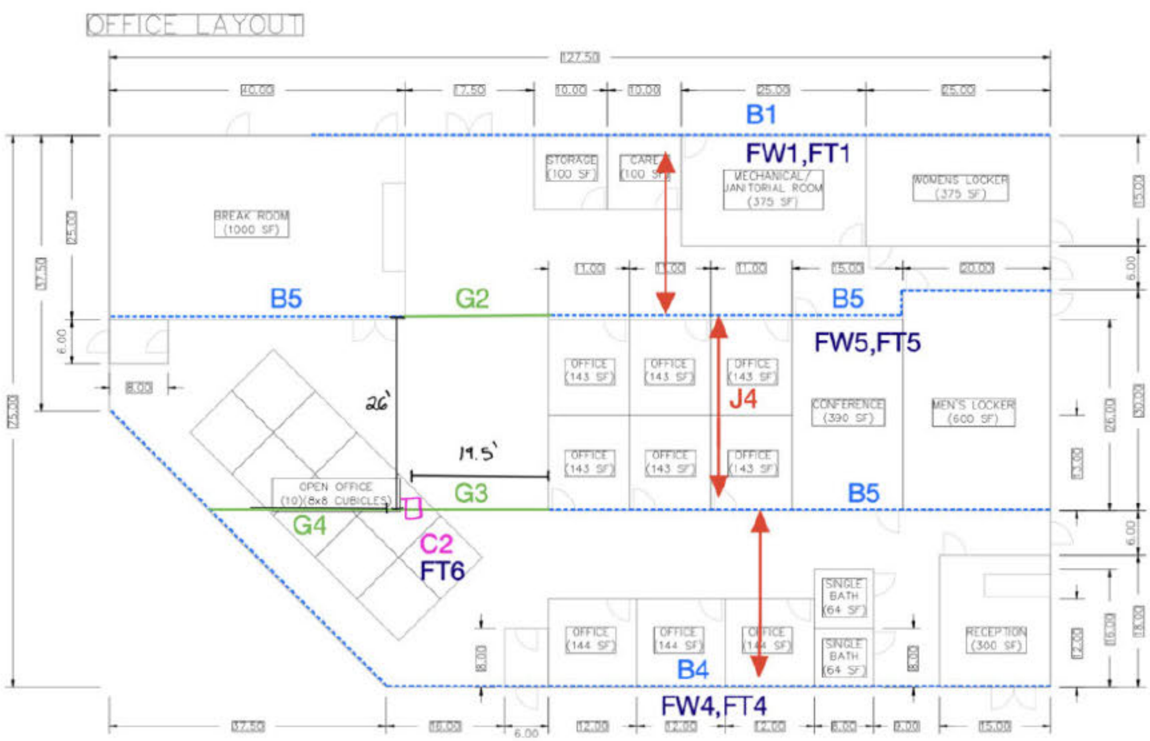


Figure 13. Labeled Member Diagram – Office

The ASD Load combinations were comprised of area loads for dead loads, live loads, and snow loads. All area loads were obtained from ASCE-7 and were applied to the roof decking. From the combination of area loads and spacing of joists, uniform loads were calculated to design the roof joists. The limiting design factor of the joists was the maximum deflection limit of L/240. which is the span length, L, divided by 240. The uniform loads of the joists were then used to calculate the point load reaction forces applied to their respective bearing member. The magnitude and spacing of the applied point loads were converted into uniform loads to design the supporting girders. The girders were checked for shear moment and deflection limits in accordance with AISC. The governing factor was the deflection limit of L/240. All steel members were designed using A992 steel with a yield strength of 50 ksi. The resulting roof member sizes for the joists and girder are shown below in Table 1.

Table 1. Roof Member Sizes

Equipment Area		
Element	**Abbreviation**	**Design**
Joist	J1	96DLH19
Joist	J2	36LH13
Joist	J3	22K11
Joist Girder	G1	83.75G15N41K
Office Area		
Element	**Abbreviation**	**Design**
Joist	J4	925TDJ24-175-97 Joist
Girder	G2	W14x22
Girder	G3	W14x22
Girder	G4	W14x43

The uniform loads applied to the girders were used to calculate the point load reaction forces applied to the facility's steel columns. The steel columns were designed for compression and flexure in accordance with AISC. The precast concrete bearing walls were designed using the point load reaction forces from the respective joists they support. The PCI Design Handbook was used to size the precast concrete wall panels, which have a design strength of 4000 psi. The sizes of the respective bearing walls and columns are represented below in Table 2.

Table 2. Wall and Column Member Sizes

Equipment Area		
Element Type	**Abbreviation**	**Design**
Bearing/Shear	B1/SH1	1’-3” x 33.5’
Bearing/Shear	B2/SH1	1’-3” x 33.5’
Bearing	B3	1’-3” x 33.5’
Column	C1	W12x65
Column	C3	W14x120
Office Area		
Element Type	**Abbreviation**	**Design**
Bearing	B4	6"
Bearing	B5	3 5/8"
Column	C2	W8x15

The design of the footings and foundations were based on the assumed soil properties of the site and the applied loads from the columns and bearing walls. Due to the soil type on the site an allowable vertical bearing pressure of 1500 psf was determined from the 2021 IBC. This was then used to size the continuous and spread footings. The footings of the columns are all square spread footings bearing just below the floor slab. The foundations of the bearing walls are designed as continuous footings and foundation walls. The continuous footings are designed to be 3’-6” below grade to withstand the effects of the frost depth in Iowa. The foundation walls act merely as an extension of the bearing wall to transfer the loads to the continuous footings. The sizes of the foundations are represented below in Table 3.

Table 3. Foundation and Footing Member Sizes

Equipment Area		
Element Type	**Abbreviation**	**Design (B x H)**
Foundation Wall	FW1	1'-3" x 2'
Footing	FT1	10'-6" x 1'-6"
Foundation Wall	FW2	1'-3" x 2'
Footing	FT2	8' x 1'-6"
Foundation Wall	FW3	1'-3" x 2'
Footing	FT3	2'-3" x 1'-6"
Column footing	FT7	18' x 1'-6"
Column footing	FT8	25' x 1'-6"
Office Area		
Element Type	**Abbreviation**	**Design (B x H)**
Foundation Wall	FW4	1'-3" x 2'
Footing	FT4	2'-3" x 1'-6"
Foundation Wall	FW5	1'-3" x 2'
Footing	FT5	2'-3" x 1'-6"
Column Footing	FT6	5'-3" x 1'-6"

B. Lateral Load Analysis

The lateral load analysis checked the lateral resistance of the building due to applied winds from both the east/west and north/south directions. Using ASCE-7 and a wind speed of 122 mph, the wind loads were determined to be 20.9 psf in the windward direction and 13.4 psf in the leeward direction for the warehouse area. For the office space, the wind load was determined to be 18.3 psf in the windward direction and 11.4 psf in the leeward direction. The roof metal decking was designed as the primary lateral force-resisting element. Using Vulcraft's design aids, the metal decking of the equipment area and office area roofs were designed as 18 gage grade 40 3.5D dovetail and 26 gage 1.0C-32 grade 80 non-composite decking respectively.

3. Ancillary Structures

Our team is recommending a prefabricated ClearSpan Fabric Structure for the outdoor material storage area. By using a prefabricated structure, the Public Works Department will be able to better customize the material storage structure to fit their needs. The department will be able to select the exact width, length, and height that fits their needs. As the department experiences growth, they can purchase additional prefabricated structures. After researching our team determined that a ClearSpan Round Extra-Tall HD Building by FarmTek would best fit the needs of the Public Works Department. It is recommended that the department purchases a configuration of 56'x60'.

Section VII Engineer's Construction Cost Estimate

After completing the facility design, we prepared a construction cost estimate that included the costs of materials, labor, overhead, profit, contingency, final design, and construction administration cost for construction of the Public Works Facility. RSMean 2018 Heavy Construction Cost with Gordian's data was used to calculate the material cost for the project. From the book we were able to find the unit cost for each structural element in our design. In Table 4, the total quantity of each element and unit cost were multiplied to get a cost per element. All the elements were then summed to the structural cost subtotal. In Table 4, the materials were split into categories by type, walls, columns, and framing members. Gordian's 2019 Square Foot Costs with RSMeans data was used to estimate the architectural, plumbing, mechanical, and electrical labor, and construction costs for the project. The labor and material subtotals calculated were then summed to get a total construction cost. In Table 5, an additional 10% of the construction cost was added for contingency, and an additional 20% was added for administrative costs. With these extra costs added to the construction cost, our team estimated the total project cost. The total construction cost is estimated to be $20,417,631.

Table 4. Total Construction Cost

Type	Quantity	Unit	Unit Cost		Extended Cost	
Cast-In-Place Pedestal	2,622	SF	$	9.30	$	24,400
Precast Walls	63,410	SF	$	19.45	$	1,233,500
Stud Walls	14,644	SF	$	7.85	$	115,000
Cast-In-Place Foundation	91,343	SF	$	9.30	$	849,500
W12x65	350	LF	$	115.50	$	40,500
W8x15	28	LF	$	35.50	$	995
W14x120	209	LF	$	202.00	$	42,200
W14x22	39	LF	$	49.50	$	1,925
W14x43	26.5	LF	$	78.50	$	20,100
925TDJ24-175-97	9450	LF	$	5.45	$	51,500
96DLH19	103.7	Ton	$	3,125.00	$	324,000
36LH13	42	Ton	$	2,950.00	$	124,000
22K11	1.1	Ton	$	2,725.00	$	2,950
83.75G15N41K	2.2	Ton	$	7,350.00	$	15,800
Office MEPA	8860	SF	$	275.00	$	2,436,500
Warehouse MEPA	59,559	SF	$	175.00	$	10,423,000
				Subtotal	$	15,706,000
				10% Contingency	$	1,570,000
				20% Administration	$	3,140,000
				Construction Cost	**$**	**20,416,000**

Appendix A: Design Calculations

J2

$L := 65$ ft
$s := 6$ ft
$w_d := 26$ psf
$w_l := 20$ psf $w_w := 30$ plf
$w_s := 45.54$ psf

$w_D := w_d \cdot s + w_w = 186$ plf
$w_L := w_l \cdot s = 120$ plf
$w_S := w_s \cdot s = 273.24$ plf

ASD Load Combos:

$P_1 := w_D = 186$ plf

$P_2 := w_D + w_L = 306$ plf

$P_3 := w_D + \max(w_L, w_S) = 459.24$ plf

$P_4 := w_D + 0.75\ w_L + 0.75\ \max(w_L, w_S) = 480.93$ plf

$P_5 := w_D = 186$ plf

$P_6 := w_D + 0.75\ w_L + 0.75\ \max(w_L, w_S) = 480.93$ plf

$P_7 := 0.6\ w_D = 111.6$ plf

$w_{uJ2} := \max(P_1, P_2, P_3, P_4, P_5, P_6, P_7) = 480.93$ plf

Vulcraft Joist:

36LH13 pg. 205

$d := 36$ in
$\boxed{w_w} := 30$ plf

Total Capacity=562 plf > $w_{uJ2} = 480.93$ plf OK

$\frac{L}{\frac{d}{12}} = 21.667$ good (b/w 20-24)

J3

$L := 35 \quad ft$
$s := 6 \quad ft$
$w_d := 26 \quad psf$
$w_l := 20 \quad psf$
$w_s := 45.54 \quad psf$ $\qquad w_w := 11.9 \quad plf$

$w_D := w_d \cdot s + w_w = 167.9 \quad plf$
$w_L := w_l \cdot s = 120 \quad plf$
$w_S := w_s \cdot s = 273.24 \quad plf$

ASD Load Combos:

$P_1 := w_D = 167.9 \quad plf$

$P_2 := w_D + w_L = 287.9 \quad plf$

$P_3 := w_D + \max\left(w_L, w_S\right) = 441.14 \quad plf$

$P_4 := w_D + 0.75\ w_L + 0.75\ \max\left(w_L, w_S\right) = 462.83 \quad plf$

$P_5 := w_D = 167.9 \quad plf$

$P_6 := w_D + 0.75\ w_L + 0.75\ \max\left(w_L, w_S\right) = 462.83 \quad plf$

$P_7 := 0.6\ w_D = 100.74 \quad plf$

$w_{uJ3} := \max\left(P_1, P_2, P_3, P_4, P_5, P_6, P_7\right) = 462.83 \quad plf$

Vulcraft Joist:

22K11 pg. 155

$d := 22 \quad in$
$w_w := 11.9 \quad plf$

Total capacity=494 plf > $w_{uJ3} = 462.83 \quad plf$ OK

$\dfrac{L}{\dfrac{d}{12}} = 19.091$ okay (not b/w 20-24)

Bearing Walls (B4, B5) $q_{DB} := 5\ psf$ $q_{wall\_arch} := 5\ psf$

Joist Load (J4) $q_{Do} := 17\ psf$

$s_{J4} := ft$ $L_{J4} := 26\ ft$

$w_{D\_J4} := q_{Do} \cdot s_{J4} = 0.017\ klf$ $R_{D\_J4} := \dfrac{w_{D\_J4} \cdot L_{J4}}{2} = 0.221\ kip$

$b_{B4} := in$ $h_{B4} := 12\ ft$ $L_{B4} := 90\ ft + 53\ ft = 143\ ft$

$b_{B5} := in$ $h_{B5} := 12\ ft$ $L_{B5} := 40\ ft$

$w_{weight\_B4} := h_{B4} \cdot q_{DB} = 0.06\ klf$ $w_{weight\_arch\_B4} := q_{wall\_arch} \cdot h_{B4} = 0.06\ klf$

$w_{weight\_B5} := h_{B5} \cdot q_{DB} = 0.06\ klf$ $w_{weight\_arch\_B5} := q_{wall\_arch} \cdot h_{B5} = 0.06\ klf$

$$w_{D\_B4} := w_{weight\_B4} + w_{weight\_arch\_B4} + \frac{R_{D\_J4}}{s_{J4}} = 0.341\ klf$$

$$w_{D\_B5} := w_{weight\_B5} + w_{weight\_arch\_B5} + \frac{2 \cdot R_{D\_J4}}{s_{J4}} = 0.562\ klf$$

Wall Studs (STB4-STB5)

$s_{STB4} := 24\ in$ $s_{STB5} := 24\ in$

$L_{STB4} := 12\ ft$ $L_{STB5} := 12\ ft$

$P_{D\_STB4} := w_{D\_B4} \cdot s_{STB4} = 0.682\ kip$

$P_{D\_STB5} := w_{D\_B5} \cdot s_{STB5} = 1.124\ kip$

Bearing wall 5 experiences ~49kip total axial compression

Bearing wall 4 experiences ~92kip total axial

ALLOWABLE COMBINED AXIAL & LATERAL LOADS (Kips/Stud)

Wind = 20psf			S162 (1-5/8" Flange)						
	Stud length (ft)	Spacing (in) o.c.	-33 (20ga) 33ksi	-43 (18ga) 33ksi	-54 (16ga) 50ksi	-68 (14ga) 50ksi	-97 (12ga) 50ksi	-33 (20ga) 33ksi	-43 (18ga) 33ksi
3-5/8" Stud	8	12	1.22 a	1.98 a	3.53 a	4.75 a	7.17 a	1.55 a	2.56 a
		16	0.96 a	1.71 a	3.27 a	4.49 a	6.88 a	1.27 a	2.27 a
		24	0.48 b	1.21 a	2.78 a	3.99 a	6.32 a	0.74 b	1.71 a
	9	12	0.95 a	1.67 a	3.09 a	4.21 a	6.41 a	1.25 a	2.19 a
		16	0.65 b	1.36 a	2.79 a	3.90 a	6.06 a	0.92 a	1.85 a
		24	0.11 d	0.78 c	2.23 b	3.32 a	5.41 a	0.32 c	1.21 b
	10	12	0.69 b	1.35 a	2.64 a	3.65 a	5.61 a	0.95 a	1.82 a
		16	0.36 d	1.00 c	2.31 a	3.30 a	5.22 a	0.59 c	1.43 b
		24	—	0.37 d	1.70 c	2.66 b	4.49 a	—	0.74 c
	12	12	0.22 e	0.76 d	1.78 c	2.56 b	4.05 a	0.41 d	1.11 c
		16	—	0.37 e	1.42 d	2.18 c	3.62 b	0.01 e	0.69 c
		24	—	—	0.79 e	1.52 e	2.86 d	—	—
	14	12	—	0.28 e	1.11 e	1.71 d	2.82 c	—	0.54 e
		16	—	—	0.76 e	1.34 e	2.40 d	—	0.12 e
		24	—	—	0.16 f	0.71 f	1.67 e	—	—
	16	12	—	—	0.63 f	1.10 e	1.94 d	—	0.12 f
		16	—	—	0.31 f	0.75 f	1.54 e	—	—
		24	—	—	—	0.16 f	0.87 f	—	—

INTERIOR WALL HEIGHTS

With structural framing

	Member	Spacing (in) o.c.	5psf L/120	5psf L/240	5psf L/360
3-5/8" Structural Framing	362S137-33	12	23' 3"	18' 5"	16' 1"
		16	21' 1"	16' 9"	14' 8"
		24	17' 6"	14' 8"	12' 10"
	362S137-43	12	25' 3"	20' 1"	17' 6"
		16	23' 0"	18' 3"	15' 11"
		24	20' 1"	15' 11"	13' 11"
	362S137-54	12	27' 1"	21' 6"	18' 9"
		16	24' 7"	19' 6"	17' 1"
		24	21' 6"	17' 1"	14' 11"
	362S137-68	12	28' 11"	22' 11"	20' 1"
		16	26' 3"	20' 10"	18' 3"
		24	22' 11"	18' 3"	15' 11"
	362S137-97	12	31' 10"	25' 3"	22' 1"
		16	28' 11"	22' 11"	20' 1"
		24	25' 3"	20' 1"	17' 6"
	362S162-33	12	24' 4"	19' 4"	16' 11"
		16	22' 2"	17' 7"	15' 4"
		24	18' 9"	15' 4"	13' 5"
	362S162-43	12	26' 6"	21' 0"	18' 5"
		16	24' 1"	19' 1"	16' 8"
		24	21' 0"	16' 8"	14' 7"
	362S162-54	12	28' 5"	22' 6"	19' 8"
		16	25' 10"	20' 6"	17' 11"
		24	22' 6"	17' 11"	15' 7"
	362S162-68	12	30' 5"	24' 1"	21' 1"
		16	27' 7"	21' 11"	19' 2"
		24	24' 1"	19' 2"	16' 9"
	362S162-97	12	33' 6"	26' 7"	23' 3"
		16	30' 5"	24' 2"	21' 1"
		24	26' 7"	21' 1"	18' 5"

Choose 3-5/8" 362S137-33 studs for load bearing wall 5.
Spaced at 24" OC
Member height is 12'

INTERIOR WALL HEIGHTS

With structural framing

6" Structural Framing

Member	Spacing (in) o.c.	5psf L/120	5psf L/240	5psf L/360
600S137-33	12	33' 1"	27' 3"	23' 10"
	16	28' 7"	24' 9"	21' 8"
	24	23' 4"	21' 8"	18' 11"
600S137-43	12	37' 8"	29' 11"	26' 2"
	16	34' 3"	27' 2"	23' 9"
	24	28' 1"	23' 9"	20' 9"
600S137-54	12	40' 5"	32' 1"	28' 0"
	16	36' 9"	29' 2"	25' 6"
	24	32' 1"	25' 6"	22' 3"
600S137-68	12	43' 4"	34' 4"	30' 0"
	16	39' 4"	31' 3"	27' 3"
	24	34' 4"	27' 3"	23' 10"
600S137-97	12	47' 11"	38' 0"	33' 2"
	16	43' 6"	34' 6"	30' 2"
	24	38' 0"	30' 2"	26' 4"
600S162-33	12	35' 6"	28' 8"	25' 0"
	16	30' 9"	26' 0"	22' 9"
	24	25' 2"	22' 9"	19' 10"
600S162-43	12	39' 4"	31' 2"	27' 3"
	16	35' 9"	28' 4"	24' 9"
	24	31' 1"	24' 9"	21' 8"
600S162-54	12	42' 2"	33' 6"	29' 3"
	16	38' 4"	30' 5"	26' 7"
	24	33' 6"	26' 7"	23' 3"
600S162-68	12	45' 3"	35' 11"	31' 4"
	16	41' 1"	32' 7"	28' 6"
	24	35' 11"	28' 6"	24' 11"
600S162-97	12	50' 1"	39' 9"	34' 9"
	16	45' 6"	36' 2"	31' 7"
	24	39' 9"	31' 7"	27' 7"

6" Stud

Wind = 20psf		S162 (1-5/8" Flange) -33 (20ga) 33ksi	-43 (18ga) 33ksi	-54 (16ga) 50ksi
Stud length (ft)	Spacing (in) o.c.			
8	12	1.98 a	2.96 a	5.20 a
	16	1.80 a	2.78 a	5.02 a
	24	1.43 a	2.42 a	4.68 a
9	12	1.82 a	2.80 a	5.04 a
	16	1.59 a	2.57 a	4.82 a
	24	1.13 a	2.12 a	4.38 a
10	12	1.64 a	2.62 a	4.85 a
	16	1.36 a	2.34 a	4.57 a
	24	0.81 a	1.79 a	4.03 a
12	12	1.21 a	2.16 a	4.35 a
	16	0.83 a	1.77 a	3.95 a
	24	0.12 c	1.03 a	3.20 a
14	12	0.74 a	1.63 a	3.62 a
	16	0.28 c	1.15 b	3.13 a
	24	—	0.27 d	2.22 c
16	12	0.29 c	1.09 b	2.85 a
	16	—	0.55 d	2.29 c
	24	—	—	1.30 d

Choose 6" 600S162-54 (16ga) studs for exterior load bearing wall 4.
Spaced at 24" OC
Member height is 16'

EXPOSURE CATEGORY: C

Ground Elevation: 940 ft

$V := 122$ $K_d := 0.85$ $K_e := 0.97$ $G := 0.85$

$K_{zt} := 1.0$

$W := 0\ klf$ $z_g := 2460\ ft$ $\alpha := 9.8$

Z Heights:

$GC_{pi} := 0.18$

Warehouse Walls: 15', 30',33.5'
Office Walls: 12', 15.5'

$$K_{12} := 2.41\left(\frac{12\ ft}{z_g}\right)^{\frac{2}{\alpha}} = 0.813 \qquad K_{15} := 2.41\left(\frac{15\ ft}{z_g}\right)^{\frac{2}{\alpha}} = 0.851$$

$$K_{15.5} := 2.41\left(\frac{15.5\ ft}{z_g}\right)^{\frac{2}{\alpha}} = 0.857 \qquad K_{30} := 2.41\left(\frac{30\ ft}{z_g}\right)^{\frac{2}{\alpha}} = 0.98$$

$$K_{33.5} := 2.41\left(\frac{33.5\ ft}{z_g}\right)^{\frac{2}{\alpha}} = 1.003$$

VELOCITY PRESSURES

$q_{12} := 0.00256\ K_{12} \cdot K_{zt} \cdot K_d \cdot K_e \cdot V^2 = 25.549$ psf

$q_{15} := 0.00256\ K_{15} \cdot K_{zt} \cdot K_d \cdot K_e \cdot V^2 = 26.74$ psf

$q_{15.5} := 0.00256\ K_{15.5} \cdot K_{zt} \cdot K_d \cdot K_e \cdot V^2 = 26.919$ psf

$q_{30} := 0.00256\ K_{30} \cdot K_{zt} \cdot K_d \cdot K_e \cdot V^2 = 30.803$ psf

$q_{33.5} := 0.00256\ K_{33.5} \cdot K_{zt} \cdot K_d \cdot K_e \cdot V^2 = 31.505$ psf

EXTERNAL PRESSURES

$GC_{pnW} := 1.5$ $GC_{pnL} := -1.0$

Parapet Pressures:

Windward

$p_{pw15.5} := q_{15.5} \cdot K_d \cdot GC_{pnW} = 34.322$ psf

$p_{pw33.5} := q_{33.5} \cdot K_d \cdot GC_{pnW} = 40.168$ psf

Leeward

$p_{pl15.5} := q_{15.5} \cdot K_d \cdot GC_{pnL} = -22.881$ psf

$p_{pl33.5} := q_{33.5} \cdot K_d \cdot GC_{pnL} = -26.779$ psf

Overall L/B Ratio: 260/385 = 0.675 < 1

$C_{pW} := 0.8 \qquad C_{pL} := -0.5$

Warehouse Walls:

Windward

$p_{15} := q_{15} \cdot G \cdot C_{pW} = 18.183$ psf

$p_{30} := q_{30} \cdot G \cdot C_{pW} = 20.946$ psf

Leeward

$p_{33.5} := q_{33.5} \cdot G \cdot C_{pL} = -13.389$ psf

Office Walls:

Windward

$p_{12} := q_{12} \cdot G \cdot C_{pW} = 17.374$ psf

$p_{15.5} := q_{15.5} \cdot G \cdot C_{pW} = 18.305$ psf

Leeward

$p_{lw15.5} := q_{15.5} \cdot G \cdot C_{pL} = -11.441$ psf

Wind Load Distribution (PC1 3-34)

$h_{eq} := 30\ ft \qquad h_{para} := 3.5\ ft \qquad h_{off} := 12\ ft$

South - North (Equipment Area)

$q_{eq\_ww} := p_{30} \cdot psf = 20.946\ psf \qquad q_{eq\_lw} := p_{33.5} \cdot psf = -13.389\ psf$

$q_{para\_eq\_ww} := p_{pw33.5} \cdot psf = 40.168\ psf \qquad q_{para\_eq\_lw} := p_{pl33.5} \cdot psf = -26.779\ psf$

$q_{off\_ww} := p_{12} \cdot psf = 17.374\ psf \qquad q_{off\_lw} := p_{lw15.5} \cdot psf = -11.441\ psf$

$q_{para\_off\_ww} := p_{pl15.5} \cdot psf = -22.881\ psf \qquad q_{para\_off\_lw} := p_{pl15.5} \cdot psf = -22.881\ psf$

Wind force to roof from Windward side $\qquad L_{ww\_SN\_eq} := 215\ ft \qquad L_{eff\_eq} := 150\ ft$

$$w_{ww\_SN\_eq} := \left(q_{eq\_ww} \cdot \frac{h_{eq}}{2}\right) + \left(q_{para\_eq\_ww} \cdot h_{para}\right) = 0.455\ klf \qquad W_{ww\_SN\_eq} := w_{ww\_SN\_eq} \cdot L_{ww\_SN\_eq} = 97.778\ kip$$

$$V_{ww\_SN\_eq} := \frac{W_{ww\_SN\_eq}}{2} = 48.889\ kip \qquad v_{ww\_SN\_eq} := \frac{V_{ww\_SN\_eq}}{275\ ft} = 177.778\ \frac{lbf}{ft}$$

$$M_{dia\_ww\_SN\_eq} := \frac{W_{ww\_SN\_eq} \cdot L_{eff\_eq}}{8} = \left(1.833 \cdot 10^{3}\right)\ kip \cdot ft$$

Wind force to roof from Leeward side

$$w_{lw\_SN\_eq} := \left(q_{eq\_lw} \cdot \frac{h_{eq}}{2}\right) + \left(q_{para\_eq\_lw} \cdot h_{para}\right) = -0.295\ klf \qquad W_{lw\_SN\_eq} := w_{lw\_SN\_eq} \cdot L_{ww\_SN\_eq} = -63.332\ kip$$

$$V_{lw\_SN\_eq} := \frac{W_{lw\_SN\_eq}}{2} = -31.666\ kip \qquad v_{lw\_SN\_eq} := \frac{V_{lw\_SN\_eq}}{275\ ft} = -115.149\ \frac{lbf}{ft}$$

$$M_{dia\_lw\_SN\_eq} := \frac{W_{lw\_SN\_eq} \cdot L_{eff\_eq}}{8} = -1.187 \cdot 10^{3}\ kip \cdot ft$$

$$v_{SN\_eq} := v_{ww\_SN\_eq} + \left|v_{lw\_SN\_eq}\right| = 292.927\ \frac{lbf}{ft}$$

East - West (Equipment Area)

Wind force to roof from Windward side $L_{ww\_EW\_eq} := 385\ ft$ $L_{eff\_eq} := 275\ ft$

$$w_{ww\_EW\_eq} := \left(q_{eq\_ww} \cdot \frac{h_{eq}}{2}\right) + \left(q_{para\_eq\_ww} \cdot h_{para}\right) = 0.455\ klf \quad W_{ww\_EW\_eq} := w_{ww\_EW\_eq} \cdot L_{ww\_EW\_eq} = 175.09\ kip$$

$$V_{ww\_EW\_eq} := \frac{W_{ww\_EW\_eq}}{2} = 87.545\ kip \qquad v_{ww\_EW\_eq} := \frac{V_{ww\_EW\_eq}}{150\ ft} = 583.634\ \frac{lbf}{ft}$$

$$M_{dia\_ww\_EW\_eq} := \frac{W_{ww\_EW\_eq} \cdot L_{eff\_eq}}{8} = \left(6.019 \cdot 10^{3}\right)\ kip \cdot ft$$

Wind force to roof from Leeward side

$$w_{lw\_EW\_eq} := \left(q_{eq\_lw} \cdot \frac{h_{eq}}{2}\right) + \left(q_{para\_eq\_lw} \cdot h_{para}\right) = -0.295\ klf \quad W_{lw\_EW\_eq} := w_{lw\_EW\_eq} \cdot L_{eff\_eq} = -81.006\ kip$$

$$V_{lw\_EW\_eq} := \frac{W_{lw\_EW\_eq}}{2} = -40.503\ kip \qquad v_{lw\_EW\_eq} := \frac{V_{lw\_EW\_eq}}{275\ ft} = -147.284\ \frac{lbf}{ft}$$

$$M_{dia\_lw\_EW\_eq} := \frac{W_{lw\_EW\_eq} \cdot L_{eff\_eq}}{8} = -2.785 \cdot 10^{3}\ kip \cdot ft$$

$$v_{EW\_eq} := v_{ww\_EW\_eq} + \left|v_{lw\_EW\_eq}\right| = 730.918\ \frac{lbf}{ft}$$

East - West (Office Area)

Wind force to roof from Windward side $L_{ww\_EW\_off} := 127.5\ ft$ $L_{eff\_off} := 127.5\ ft$

$$w_{ww\_EW\_off} := \left(q_{off\_ww} \cdot \frac{h_{eq}}{2}\right) + \left(q_{para\_off\_ww} \cdot h_{para}\right) = 0.181\ klf$$

$$W_{ww\_EW\_off} := w_{ww\_EW\_off} \cdot L_{ww\_EW\_off} = 23.016\ kip$$

$$V_{ww\_EW\_off} := \frac{W_{ww\_EW\_off}}{2} = 11.508\ kip \qquad v_{ww\_EW\_off} := \frac{V_{ww\_EW\_off}}{127.5\ ft} = 90.259\ \frac{lbf}{ft}$$

$$M_{dia\_ww\_EW\_off} := \frac{W_{ww\_EW\_off} \cdot L_{eff\_off}}{8} = 366.82\ kip \cdot ft$$

$$v_{ww\_EW\_off} = 90.259\ \frac{lbf}{ft}$$

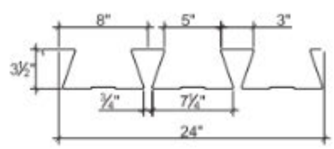
Equipment Area Deck

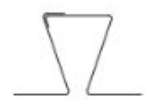
3.5D DOVETAIL ROOF DECK
GRADE 40 STEEL

ASD

3.5D DOVETAIL ROOF DECK
- **Enhanced 2-Coat Polyester Paint**
- **White Factory Primer Paint**
- **Galvanized Finish**
- **FM Listed**

Nominal Dimensions





Nested
Side-lap

Section Properties

Deck Gage	Deck Weight w_{dd} (psf)	Base Metal Thickness t (in.)	Yield Strength F_y (ksi)	Effective Moment of Inertia at Service Load $I_d = (2I_e+I_g)/3$ I_d+ (in^4/ft)	I_d- (in^4/ft)	Effective Section Modulus at F_y = 40 ksi S_e+ (in^3/ft)	S_e- (in^3/ft)	Allowable Moment M_n+/Ω (lb-ft/ft)	M_n-/Ω (lb-ft/ft)	Vertical Web Shear V_n/Ω (lb/ft)
20	3.3	0.0358	40	1.762	1.646	0.676	0.781	1349	1559	3435
18	4.3	0.0474	40	2.415	2.272	0.980	1.070	1956	2136	6012
16	5.4	0.0598	40	3.133	2.968	1.317	1.377	2629	2749	8313

Allowable Reactions at Supports Based on Web Crippling, R_n/Ω (lb/ft)

	Bearing Length of Webs											
	One-Flange Loading						Two-Flange Loading					
	End Bearing				Interior Bearing		End Bearing				Interior Bearing	
Deck Gage	2"	3"	4"	5"	4"	6"	2"	3"	4"	5"	4"	6"
20	693	794	880	955	1459	1670	714	796	865	926	1724	1991
18	1168	1330	1467	1588	2422	2753	1310	1450	1568	1672	2927	3360
16	1793	2032	2233	2410	3681	4162	2137	2352	2533	2693	4515	5157

Standard Features
- ASTM A653 SS GR 40 Min. with G90
- Standard lengths – 6'-0" to 42'-0"
- Tables conform to ANSI/SDI RD-2017
- IAPMO UES ER-423, FM and UL Listed

Optional Features
- Inquire regarding cost and lead times for:
  - -19 gage
  - -Short cuts < 6'-0"
  - -Alternative metallic and painted finishes
- Acoustical Version

18 ga 3.5D-24 Grade 40 Roof Deck

Wind Diaphragm Shear

For Both Ends Butted Deck

NUCOR VULCRAFT GROUP

#12 Screw Connections to Supports
24 / 6 Perpendicular Connection Pattern to Supports
#12 Screw Sidelap Connections

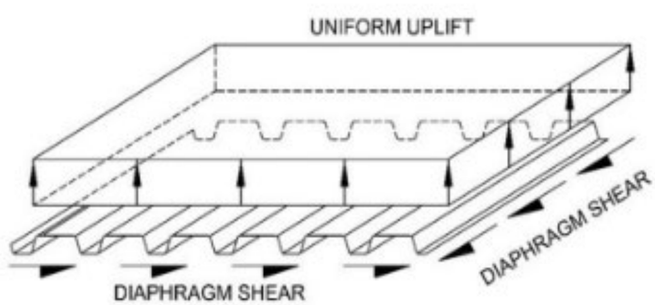
A572 GR50 Support Member or Equivalent
0.5 ≤ Support Thickness (in.) ≤ 0.5
4 in. Minimum Deck End Bearing Length

ASD Allowable Wind Diaphragm Shear Strength Sn/Ω (plf) — Generic 3 Span Condition

Sidelap Connections per Span	Span 3'-0"	3'-6"	4'-0"	4'-6"	5'-0"	5'-6"	6'-0"	6'-6"	7'-0"
3	818	713	631	566	512	468	430	398	370
4	849	844	751	675	613	561	516	479	446
5	849	849	849	780	710	651	601	557	520
6	849	849	849	849	803	738	682	634	592
7	849	849	849	849	849	821	761	708	662
8	849	849	849	849	849	849	837	780	730
9	849	849	849	849	849	849	849	849	797

Average Connection Spacing to Supports at Parallel Chords & Collectors (in.)

Sidelap Connections per Span	Span 3'-0"	3'-6"	4'-0"	4'-6"	5'-0"	5'-6"	6'-0"	6'-6"	7'-0"
3	12	14	16	18	20	22	24	20	21
4	12	11	12	14	15	17	18	20	21
5	12	11	12	14	15	13	14	16	17
6	12	11	12	11	12	13	14	16	17
7	10	11	12	11	10	11	12	13	14
8	9	11	12	11	10	11	10	11	12
9	8	10	11	11	10	11	10	11	12



18 ga 3.5D-24 Grade 40 Roof Deck

Seismic Diaphragm Shear

For Both Ends Butted Deck

NUCOR
VULCRAFT GROUP

#12 Screw Connections to Supports
24 / 6 Perpendicular Connection Pattern to Supports
#12 Screw Sidelap Connections

A572 GR50 Support Member or Equivalent
0.5 ≤ Support Thickness (in.) ≤ 0.5
4 in. Minimum Deck End Bearing Length

Seismic or Wind Diaphragm Shear Stiffness, G' (kip/in.) — Generic 3 Span Condition

Sidelap Connections per Span	Span 3'-0"	3'-6"	4'-0"	4'-6"	5'-0"	5'-6"	6'-0"	6'-6"	7'-0"
3	13	15	16	17	17	18	18	18	18
4	14	15	16	17	18	19	19	19	20
5	14	15	17	18	19	19	20	20	21
6	14	16	17	18	19	20	20	21	21
7	14	16	17	18	19	20	21	22	22
8	14	16	17	19	20	21	21	22	23
9	14	16	18	19	20	21	22	23	23

ASD Allowable Seismic Diaphragm Shear Strength Sn/Ω (plf) — Generic 3 Span Condition

Sidelap Connections per Span	Span 3'-0"	3'-6"	4'-0"	4'-6"	5'-0"	5'-6"	6'-0"	6'-6"	7'-0"
3	711	620	549	492	445	407	374	346	322
4	836	734	653	587	533	488	449	416	388
5	849	841	751	678	617	566	522	485	452
6	849	849	844	765	698	642	593	551	515
7	849	849	849	846	775	714	662	616	576
8	849	849	849	849	848	784	728	678	635
9	849	849	849	849	849	849	791	739	693

Average Connection Spacing to Supports at Parallel Chords & Collectors (in.)

Sidelap Connections per Span	Span 3'-0"	3'-6"	4'-0"	4'-6"	5'-0"	5'-6"	6'-0"	6'-6"	7'-0"
3	12	14	16	18	20	22	24	20	21
4	12	14	16	14	15	17	18	20	21
5	12	11	12	14	15	17	18	20	21
6	12	11	10	11	12	13	14	16	17
7	10	11	10	11	12	13	12	13	14
8	9	11	10	11	10	11	12	13	14
9	8	10	10	11	10	9	10	11	12

Bare Deck Diaphragm V5.3 in accordance with:
AISI S100-16 (2020) w/ S2-20
IAPMO UES ER-0423
IAPMO UES ER-0652
CAN/CSA-S136 (R2021) for Canadian references

Date: 4/4/2024

3.5D DOVETAIL ROOF DECK
GRADE 40 STEEL

Inward Uniform Allowable Loads, ASD (psf)

Deck Gage	Spans	Criteria	Span (ft-in.) 11'-0"	12'-0"	13'-0"	14'-0"	15'-0"	16'-0"	17'-0"	18'-0"	19'-0"	20'-0"	21'-0"
20	Single	W_n / Ω	89	75	64	55	48	42	37	33	30	27	24
		L/240	87	67	53	42	34	28	24	20	17	14	12
	Double	W_n / Ω	101	85	73	63	55	48	43	38	34	31	28
		L/240	---	---	---	---	---	---	---	---	---	---	28
	Triple	W_n / Ω	125	106	90	78							
		L/240	---	---	---	74							
18	Single	W_n / Ω	129	109	93	80	70	61	54	48	43	39	35
		L/240	119	92	72	58	47	39	32	27	23	20	17
	Double	W_n / Ω	139	117	100	86	75	66	59	52	47	43	39
		L/240	---	---	---	---	---	---	---	---	---	---	---
	Triple	W_n / Ω	173	146	125	108							
		L/240	---	---	---	102							
16	Single	W_n / Ω	174	146	124	107	93	82	73	65	58	53	48
		L/240	154	119	93	75	61	50	42	35	30	26	22
	Double	W_n / Ω	180	151	129	111	97	85	76	68	61	55	50
		L/240	---	---	---	---	---	---	---	---	---	---	---
	Triple	W_n / Ω	224	188	161	139							
		L/240	---	---	---	134							

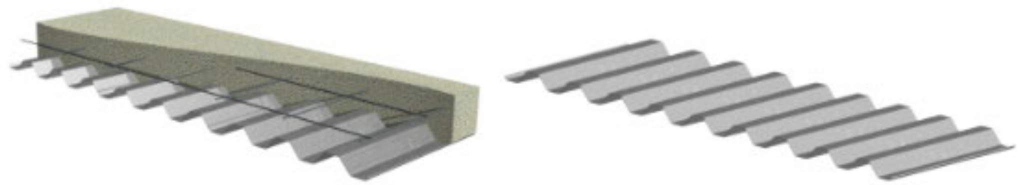
Notes:

1. Table does not account for web crippling. Required bearing should be determined based on specific span conditions.
2. The symbol "---" indicates that the uniform allowable load based on deflection exceeds the allowable load based on stress.

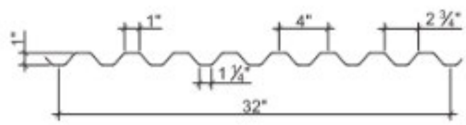
Office Area Deck

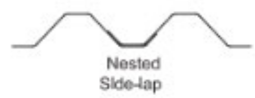
1.0C-32 NON-COMPOSITE & ROOF DECK
GRADE 80 STEEL

ASD



Nominal Dimensions





Section Properties

Deck Gage	Deck Weight w_{dd} (psf)	Base Metal Thickness t (in.)	Yield Strength F_y (ksi)	Effective Moment of Inertia at Service Load $I_d = (2I_e+I_g)/3$: I_d+ (in^4/ft)	I_d- (in^4/ft)	Effective Section Modulus at F_y = 60 ksi: S_e+ (in^3/ft)	S_e- (in^3/ft)	Allowable Moment: M_n+/Ω (lb-ft/ft)	M_n-/Ω (lb-ft/ft)	Vertical Web Shear V_n/Ω (lb/ft)
26	0.9	0.0179	60	0.041	0.043	0.067	0.071	201	213	1673
24	1.2	0.0239	60	0.057	0.058	0.098	0.103	293	308	2922
22	1.5	0.0295	60	0.071	0.071	0.130	0.134	389	401	3598
20	1.9	0.0358	60	0.090	0.090	0.168	0.166	503	497	4353

Allowable Reactions at Supports Based on Web Crippling, R_n/Ω (lb/ft)

Bearing Length of Webs

One-Flange Loading

Deck Gage	End Bearing 1½"	End Bearing 2"	End Bearing 3"	Interior Bearing 1½"	Interior Bearing 2"	Interior Bearing 3"
26	479	530	617	724	792	906
24	815	899	1039	1250	1361	1547
22	1198	1317	1516	1856	2014	2278
20	1707	1870	2144	2668	2884	3247

Standard Features
- ASTM A653 SS GR80 with G60
- Standard lengths – 6'-0" to 42'-0"
- IAPMO UES ER-0652 and UL Listed
- Tables conform to ANSI/SDI NC-2017 and RD-2017

Optional Features
- Inquire regarding cost and lead times for:
  - -Short cuts < 6'-0"
  - -Sheet Lengths > 42'-0"
  - -Alternative metallic and painted finishes
- Side-lap or bottom flange slot venting

26 ga 1.0C-32 Grade 80 Non-Composite Deck - No Fill

Wind Diaphragm Shear

For Both Ends Butted Deck

NUCOR VULCRAFT GROUP

#10 Screw Connections to Supports
32 / 3 Perpendicular Connection Pattern to Supports
#8 Screw Sidelap Connections

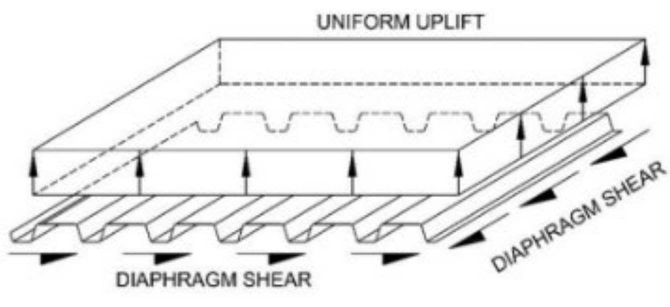
A572 GR50 Support Member or Equivalent
0.1 ≤ Support Thickness (in.) ≤ 0.175
4 in. Minimum Deck End Bearing Length

ASD Allowable Wind Diaphragm Shear Strength Sn/Ω (plf) — Generic 3 Span Condition

Sidelap Connections per Span	Span								
	0'-0"	0'-6"	1'-0"	1'-6"	2'-0"	2'-6"	3'-0"	3'-6"	4'-0"
3	-	211	205	197	186	175	163	152	142
4	-	212	207	201	192	183	173	163	154
5	-	212	209	203	197	189	180	172	163
6	-	212	210	205	200	193	186	178	171
7	-	213	210	207	202	197	190	184	177
8	-	213	211	208	204	199	194	188	182
9	-	213	211	209	205	201	197	191	186

Average Connection Spacing to Supports at Parallel Chords & Collectors (in.)

Sidelap Connections per Span	Span								
	0'-0"	0'-6"	1'-0"	1'-6"	2'-0"	2'-6"	3'-0"	3'-6"	4'-0"
3	-	3	7	10	14	15	18	21	24
4	-	3	6	8	11	14	17	19	22
5	-	2	5	7	9	12	14	16	16
6	-	2	4	6	8	10	12	14	16
7	-	2	3	5	7	9	10	12	14
8	-	2	3	5	6	8	9	11	12
9	-	1	3	4	6	7	8	10	11



26 ga 1.0C-32 Grade 80 Non-Composite Deck - No Fill

Seismic Diaphragm Shear

For Both Ends Butted Deck

NUCOR VULCRAFT GROUP

#10 Screw Connections to Supports
32 / 3 Perpendicular Connection Pattern to Supports
#8 Screw Sidelap Connections

A572 GR50 Support Member or Equivalent
$0.1 \le$ Support Thickness (in.) ≤ 0.175
4 in. Minimum Deck End Bearing Length

Seismic or Wind Diaphragm Shear Stiffness, G' (kip/in.) Generic 3 Span Condition

Sidelap Connections per Span	Span 0'-0"	0'-6"	1'-0"	1'-6"	2'-0"	2'-6"	3'-0"	3'-6"	4'-0"
3	-	1	2	3	4	4	5	6	7
4	-	1	2	3	4	4	5	6	7
5	-	1	2	3	4	4	5	6	7
6	-	1	2	3	4	4	5	6	7
7	-	1	2	3	4	4	5	6	7
8	-	1	2	3	4	4	5	6	7
9	-	1	2	3	4	4	5	6	7

ASD Allowable Seismic Diaphragm Shear Strength Sn/Ω (plf) Generic 3 Span Condition

Sidelap Connections per Span	Span 0'-0"	0'-6"	1'-0"	1'-6"	2'-0"	2'-6"	3'-0"	3'-6"	4'-0"
3	-	184	179	171	162	152	142	132	123
4	-	184	180	174	167	159	150	142	134
5	-	185	182	177	171	164	157	149	142
6	-	185	182	179	174	168	162	155	148
7	-	185	183	180	176	171	166	160	154
8	-	185	183	181	177	173	169	163	158
9	-	185	184	182	179	175	171	166	162

Average Connection Spacing to Supports at Parallel Chords & Collectors (in.)

Sidelap Connections per Span	Span 0'-0"	0'-6"	1'-0"	1'-6"	2'-0"	2'-6"	3'-0"	3'-6"	4'-0"
3	-	3	7	10	14	15	18	21	24
4	-	3	6	8	11	14	17	19	22
5	-	2	5	7	9	12	14	16	18
6	-	2	4	6	8	10	12	14	16
7	-	2	3	5	7	9	10	12	14
8	-	2	3	5	6	8	9	11	12
9	-	1	3	4	6	7	8	10	11

Bare Deck Diaphragm V5.3 in accordance with:
AISI S100-16 (2020) w/ S2-20
IAPMO UES ER-0423
IAPMO UES ER-0652
CAN/CSA-S136 (R2021) for Canadian references

Date: 4/4/2024

1.0C-32 NON-COMPOSITE & ROOF DECK
GRADE 80 STEEL

ASD

Inward Uniform Allowable Loads, ASD (psf)

Deck Gage	Spans	Criteria	Span (ft-in.)										
			2'-0"	2'-6"	3'-0"	3'-6"	4'-0"	4'-6"	5'-0"	5'-6"	6'-0"	6'-6"	7'-0"
26	Single	W_n / Ω	401	257	178	131	100	79	64	53	45	38	33
		L/240	336	172	100	63	42	29	22	16	12	10	8
	Double	W_n / Ω	405	264	185	137	105	83	67	56	47	40	35
		L/240	---	---	---	---	---	75	54	41	31	25	20
	Triple	W_n / Ω	497	325	229	170	131	103	84	70	59	50	43
		L/240	---	---	197	124	83	58	43	32	25	19	16
24	Single	W_n / Ω	587	376	261	192	147	116	94	78	65	56	48
		L/240	467	239	138	87	58	41	30	22	17	14	11
	Double	W_n / Ω	596	386	270	199	153	121	98	81	68	58	50
		L/240	---	---	---	---	143	101	73	55	42	33	27
	Triple	W_n / Ω	735	478	335	248	190	151	122	101	85	73	63
		L/240	---	459	266	167	112	79	57	43	33	26	21
22	Single	W_n / Ω	778	498	346	254	195	154	125	103	86	74	64
		L/240	582	298	172	109	73	51	37	28	22	17	14
	Double	W_n / Ω	773	501	351	259	199	157	128	106	89	76	65
		L/240	---	---	---	---	175	123	90	67	52	41	33
	Triple	W_n / Ω	951	620	435	322	247	196	159	132	111	94	82
		L/240	---	562	325	205	137	96	70	53	41	32	26
20	Single	W_n / Ω	1006	644	447	328	251	199	161	133	112	95	82
		L/240	738	378	219	138	92	65	47	35	27	21	17
	Double	W_n / Ω	956	620	434	320	246	195	158	131	110	94	81
		L/240	---	---	---	---	222	156	114	85	66	52	41
	Triple	W_n / Ω	1175	767	538	398	306	243	197	163	137	117	101
		L/240	---	713	413	260	174	122	89	67	52	41	32

Notes:

1. Table does not account for web crippling. Required bearing should be determined based on specific span conditions.
2. The symbol "---" indicates that the uniform allowable load based on deflection exceeds the allowable load based on stress.

J1 Analysis

$L_{j1} := 150\ ft$ $\quad s_{j1} := 5.8\ ft$

$w_d := 26\ psf$ $\quad w_l := 20\ psf$ $\quad w_s := 45.5\ psf$

$w_w := 80\ plf$ $\quad w_{lj1} := w_l \cdot s_{j1} = 116\ plf$ $\quad w_{sj1} := w_s \cdot s_{j1} = 263.9\ plf$

$w_{dj1} := w_d \cdot s_{j1} + w_w = 230.8\ plf$

ASD Load Combinations

$P_{ASD1} := w_{dj1} = 230.8\ plf$

$P_{ASD2} := w_{dj1} + w_{lj1} = 346.8\ plf$

$P_{ASD3} := w_{dj1} + \max(w_{lj1}, w_{sj1}) = 494.7\ plf$

$P_{ASD4} := w_{dj1} + 0.75 \cdot w_{lj1} + 0.75 \cdot \max(w_{lj1}, w_{sj1}) = 515.725\ plf$

$P_{ASD5} := w_{dj1} = 230.8\ plf$

$P_{ASD6} := w_{dj1} + 0.75 \cdot w_{lj1} + 0.75 \cdot \max(w_{lj1}, w_{sj1}) = 515.725\ plf$

$P_{ASD7} := 0.6 \cdot w_{dj1} = 138.48\ plf$

$w_{uj1} := \max(P_{ASD1}, P_{ASD2}, P_{ASD3}, P_{ASD4}, P_{ASD5}, P_{ASD6}, P_{ASD7}) = 515.725\ plf$

$w_{uj1} := 515.725\ plf$

Required Strength: $M_{uj1} := \dfrac{w_{uj1} \cdot L_{j1}^{\ 2}}{8} = (1.45 \cdot 10^3)\ kip \cdot ft$

Vulcraft Designation

96DLH19 $\quad d := 96\ in$ $\quad w_w := 66\ plf$

Total safe uniform capacity:

$$w_u := 561\ plf + (150\ ft - 148\ ft) \cdot \left(\frac{539\ plf - 561\ plf}{151\ ft - 148\ ft}\right)$$

$w_u = 546.333\ plf$ $\quad > \quad w_{uj1} = 515.725\ plf$ $\quad$ Adequate

G2 Analysis

$L_{g2} := 19.5\ ft$ $s_{g2} := 25\ ft \cdot 0.5 + 26\ ft \cdot 0.5 = 25.5\ ft$ $E := 29000\ ksi$

$w_d := 17\ psf$ $w_l := 20\ psf$ $w_s := 45.5\ psf$

$w_w := 25\ plf$ $w_{lg2} := w_l \cdot s_{g2} = 510\ plf$ $w_{sg2} := w_s \cdot s_{g2} = \left(1.16 \cdot 10^3\right)\ plf$

$w_{dg2} := w_d \cdot s_{g2} + w_w = 458.5\ plf$

ASD Load Combinations

$P_{ASD1} := w_{dg2} = 458.5\ plf$ $P_{ASD2} := w_{dg2} + w_{lg2} = 968.5\ plf$

$P_{ASD3} := w_{dg2} + \max\left(w_{lg2}, w_{sg2}\right) = \left(1.619 \cdot 10^3\right)\ plf$

$P_{ASD4} := w_{dg2} + 0.75 \cdot w_{lg2} + 0.75 \cdot \max\left(w_{lg2}, w_{sg2}\right) = \left(1.711 \cdot 10^3\right)\ plf$

$P_{ASD5} := w_{dg2} = 458.5\ plf$ $P_{ASD7} := 0.6 \cdot w_{dg2} = 275.1\ plf$

$P_{ASD6} := w_{dg2} + 0.75 \cdot w_{lg2} + 0.75 \cdot \max\left(w_{lg2}, w_{sg2}\right) = \left(1.711 \cdot 10^3\right)\ plf$

$w_{ug2} := \max\left(P_{ASD1}, P_{ASD2}, P_{ASD3}, P_{ASD4}, P_{ASD5}, P_{ASD6}, P_{ASD7}\right) = \left(1.711 \cdot 10^3\right)\ plf$

Required Strength: $M_{ug2} := \dfrac{w_{ug2} \cdot L_{g2}^{\ 2}}{8} = 81.335\ kip \cdot ft$

Minimum Zx

$\Omega_b := 1.67$ $F_y := 50\ ksi$ $Z_{xmin} := \dfrac{M_{ug2} \cdot \Omega_b}{F_y} = 32.599\ in^3$

Use: W14x22

$Z_x := 33.2\ in^3$ $I_x := 199\ in^4$

Nominal Strength: $M_p := F_y \cdot Z_x = 138.333\ kip \cdot ft$

Allowable Flexural Strength: $\dfrac{M_p}{\Omega_b} = 82.834\ kip \cdot ft$ $> M_{ug2} = 81.335\ kip \cdot ft$ Adequate

Deflection check

$\Delta_{max} := \dfrac{5 \cdot w_{ug2} \cdot L_{g2}^{\ 4}}{384 \cdot E \cdot I_x} = 0.965\ in$ $<$ $\dfrac{L_{g2}}{240} = 0.975\ in$

Bearing Wall 1

$L_{sh1} := 1\ ft$ $L_{j1} := 150\ ft$ $h_{sh1} := 30\ ft$ $b_{sh1} := 1.25\ ft$

$w_{uj1} := 515.725\ plf$ $f'_c := 4000\ psi$ $A_g := h_{sh1} \cdot b_{sh1} = 37.5\ ft^2$

Table 14.3.1.1

$\frac{h_{sh1}}{24} = 1.25\ ft$

$$P_{j1} := \frac{w_{uj1} \cdot L_{j1}}{2} \cdot 30 \cdot \frac{1}{184.33} = \left(6.295 \cdot 10^3\right)\ lbf$$

SH1, FW1, FT1
loads are lbf/unit length

$$P_{ush1} := P_{j1} + \left(150\ pcf \cdot b_{sh1} \cdot h_{sh1} \cdot L_{sh1}\right) = \left(1.192 \cdot 10^4\right)\ lbf$$

$$P_{nsh1} := 0.6 \cdot f'_c \cdot A_g \cdot \left(1 - \left(\frac{h_{sh1}}{32 \cdot b_{sh1}}\right)^2\right) = \left(5.67 \cdot 10^6\right)\ lbf$$

$P_{ush1} < P_{nsh1} = 1$

Foundation Wall 1

$h_{sh1} := 33.5\ ft$ $h_{fw1} := 2\ ft$ $b_{fw1} := 1.25\ ft$ $A_g := b_{fw1} \cdot h_{fw1}$

$$P_{ufw1} := P_{j1} + \left(150\ pcf \cdot b_{sh1} \cdot h_{sh1} \cdot L_{sh1}\right) + \left(150\ pcf \cdot b_{fw1} \cdot h_{fw1} \cdot L_{sh1}\right) = \left(1.295 \cdot 10^4\right)\ lbf$$

$$P_{nfw1} := 0.6 \cdot f'_c \cdot A_g \cdot \left(1 - \left(\frac{h_{fw1}}{32 \cdot b_{fw1}}\right)^2\right) = \left(8.618 \cdot 10^5\right)\ lbf$$

$P_{ufw1} < P_{nfw1} = 1$

Wall Footing 1

From IBC Section 1806 Table 1806.2: $\sigma_v := 1500\ psf$ $\sigma_h := 100\ psf$

$h_{ft1} := 1.5\ ft$ $b_{ft1} := 10.5\ ft$ $L_{ft1} := 184\ ft + 3\ in$

$$P_{uft1} := P_{ufw1} \cdot 184.33 + \left(150\ pcf \cdot b_{ft1} \cdot h_{ft1} \cdot L_{ft1}\right) = \left(2.823 \cdot 10^6\right)\ lbf$$

$$P_{nft1} := b_{ft1} \cdot L_{ft1} \cdot \sigma_v = \left(2.902 \cdot 10^6\right)\ lbf$$

$P_{uft1} < P_{nft1} = 1$

SH1 - 215 x 1.25 x 33.5 FW1 - 215 x 1.25 x 2 FT1 - 215 x 10 x 1.5

Bearing Wall 2

Table 14.3.1.1

$$\frac{h_{sh2}}{24} = 1.25\ ft$$

$L_{sh2} := 1\ ft$ $\quad L_{j2} := 65\ ft$ $\quad h_{sh2} := 30\ ft$ $\quad b_{sh2} := 1.25\ ft$

$w_{uj2} := 480.93\ plf$ $\quad f'_c := 4000\ psi$ $\quad A_g := h_{sh2} \cdot b_{sh2} = 37.5\ ft^2$

$$P_{j2} := \frac{w_{uj2} \cdot L_{j2}}{2} \cdot 38 \cdot \frac{1}{225} = \left(2.64 \cdot 10^3\right)\ lbf$$

$$P_{ush2} := P_{j2} + \left(150\ pcf \cdot b_{sh2} \cdot h_{sh2} \cdot L_{sh2}\right) = \left(8.265 \cdot 10^3\right)\ lbf$$

$$P_{nsh2} := 0.6 \cdot f'_c \cdot A_g \cdot \left(1 - \left(\frac{h_{sh2}}{32 \cdot b_{sh2}}\right)^2\right) = \left(5.67 \cdot 10^6\right)\ lbf$$

$P_{ush2} < P_{nsh2} = 1$

Foundation Wall 2

$h_{sh2} := 33.5\ ft$ $\quad h_{fw2} := 2\ ft$ $\quad b_{fw2} := 1.25\ ft$ $\quad A_g := b_{fw2} \cdot h_{fw2}$

$$P_{ufw2} := P_{j2} + \left(150\ pcf \cdot b_{sh2} \cdot h_{sh2} \cdot L_{sh2}\right) + \left(150\ pcf \cdot b_{fw2} \cdot h_{fw2} \cdot L_{sh2}\right) = \left(9.296 \cdot 10^3\right)\ lbf$$

$$P_{nfw2} := 0.6 \cdot f'_c \cdot A_g \cdot \left(1 - \left(\frac{h_{fw2}}{32 \cdot b_{fw2}}\right)^2\right) = \left(8.618 \cdot 10^5\right)\ lbf$$

$P_{ufw2} < P_{nfw2} = 1$

Wall Footing 2

From IBC Section 1806 Table 1806.2: $\quad \sigma_v := 1500\ psf$ $\quad \sigma_h := 100\ psf$

$h_{ft2} := 1.5\ ft$ $\quad b_{ft2} := 8\ ft$ $\quad L_{sh2} := 225\ ft$

$$P_{uft2} := P_{ufw2} \cdot 225 + \left(150\ pcf \cdot h_{ft2} \cdot b_{ft2} \cdot L_{sh2}\right) = \left(2.497 \cdot 10^6\right)\ lbf$$

$$P_{nft2} := b_{ft2} \cdot L_{sh2} \cdot \sigma_v = \left(2.7 \cdot 10^6\right)\ lbf$$

$P_{nft2} > P_{uft2} = 1$

SH2 - 225 x 1.25 x 33.5 $\quad$ FW2 - 225 x 1.25 x 2 $\quad$ FT2 - 225 x 8 x 1.5

Bearing Wall 3

Table 14.3.1.1

$\frac{h_{sh3}}{24} = 1.25\ ft$

$L_{sh3} := 1\ ft$ $L_{j3} := 35\ ft$ $h_{sh3} := 30\ ft$ $b_{sh3} := 1.25\ ft$

$w_{uj3} := 462.83\ plf$ $f'_c := 4000\ psi$ $A_g := h_{sh3} \cdot b_{sh3} = 37.5\ ft^2$

$$P_{j3} := \frac{w_{uj3} \cdot L_{j3}}{2} \cdot 10 \cdot \frac{1}{60} = \left(1.35 \cdot 10^3\right)\ lbf$$

$$P_{ush3} := P_{j3} + \left(150\ pcf \cdot b_{sh3} \cdot h_{sh3} \cdot L_{sh3}\right) = \left(6.975 \cdot 10^3\right)\ lbf$$

$$P_{nsh3} := 0.6 \cdot f'_c \cdot A_g \cdot \left(1 - \left(\frac{h_{sh3}}{32 \cdot b_{sh3}}\right)^2\right) = \left(5.67 \cdot 10^6\right)\ lbf$$

$P_{ush1} < P_{nsh1} = 1$

Foundation Wall 3

$h_{sh3} := 33.5\ ft$ $h_{fw3} := 2\ ft$ $b_{fw3} := 1.25\ ft$ $A_g := b_{fw3} \cdot h_{fw3}$

$$P_{ufw3} := P_{j3} + \left(150\ pcf \cdot b_{sh3} \cdot h_{sh3} \cdot L_{sh3}\right) + \left(150\ pcf \cdot b_{fw3} \cdot h_{fw3} \cdot L_{sh3}\right) = \left(8.006 \cdot 10^3\right)\ lbf$$

$$P_{nfw3} := 0.6 \cdot f'_c \cdot A_g \cdot \left(1 - \left(\frac{h_{fw3}}{32 \cdot b_{fw3}}\right)^2\right) = \left(8.618 \cdot 10^5\right)\ lbf$$

$P_{ufw1} < P_{nfw1} = 1$

Wall Footing 3

From IBC Section 1806 Table 1806.2: $\sigma_v := 1500\ psf$ $\sigma_h := 100\ psf$

$h_{ft3} := 1.5\ ft$ $b_{ft3} := 2\ ft + 3\ in$ $L_{sh3} := 60\ ft$

$$P_{uft3} := P_{ufw3} \cdot 60 + \left(150\ pcf \cdot h_{ft3} \cdot b_{ft3} \cdot L_{sh3}\right) = \left(5.107 \cdot 10^5\right)\ lbf$$

$$P_{nft3} := b_{ft3} \cdot L_{sh3} \cdot \sigma_v = \left(2.025 \cdot 10^5\right)\ lbf$$

$P_{nft1} > P_{uft1} = 1$

SH3 - 60 x 1.25 x 33.5 FW3 - 60 x 1.25 x 2 FT3 - 60 x 2.25 x 1.5

West Wind Load

$w_{ww} := 20.946\ \textbf{psf}$ $\quad w_{wp} := 40.168\ \textbf{psf}$ $\quad w_{lp} := -26.779\ \textbf{psf}$

$h_w := 30\ \textbf{ft}$ $\quad h_p := 3.5\ \textbf{ft}$ $\quad l_{sh} := 42\ \textbf{ft}$ $\quad l := 385\ \textbf{ft}$

$$W := \left(w_{ww} \cdot \frac{h_w}{2} \cdot l\right) + \left(w_{wp} \cdot h_p \cdot l\right) - \left(w_{lp} \cdot h_p \cdot l\right) = 211.174\ \textbf{kip}$$

$$V_l := \frac{W}{2} = 105.587\ \textbf{kip} \qquad V_r := \frac{W}{2} = 105.587\ \textbf{kip}$$

$$M_{diaphram} := \frac{W \cdot l}{8} = \left(1.016 \cdot 10^4\right)\ \textbf{kip} \cdot \textbf{ft}$$

Check sliding resistance

Example 3.7.2

$$W_{footing} := \left(150\ \textbf{pcf} \cdot 1.5\ \textbf{ft} \cdot 2.25\ \textbf{ft} \cdot l_{sh}\right) = 21.263\ \textbf{kip}$$

$$W_{wall} := \left(150\ \textbf{pcf} \cdot 35.5\ \textbf{ft} \cdot 1.25\ \textbf{ft} \cdot l_{sh}\right) = 279.563\ \textbf{kip}$$

$$W_{backfill} := \left(100\ \textbf{pcf} \cdot 3.5\ \textbf{ft} \cdot l_{sh} \cdot 1.25\ \textbf{ft}\right) = 18.375\ \textbf{kip}$$

$$W_{total} := W_{footing} + W_{wall} + W_{backfill} = 319.2\ \textbf{kip}$$

$\mu_s := 0.5$ $\quad V_{sliding} := \mu_s \cdot W_{total} = 159.6\ \textbf{kip}$ $\quad FS := \frac{V_{sliding}}{V_l} = 1.512$ > 1.5, good

Check overturning resistance

$$M_{app} := V_l \cdot (33.5\ \textbf{ft}) = \left(3.537 \cdot 10^3\right)\ \textbf{kip} \cdot \textbf{ft}$$

$M_{resis} := W_{total} \cdot \left(0.5 \cdot l_{sh}\right) = \left(6.703 \cdot 10^3\right)\ \textbf{kip} \cdot \textbf{ft}$ $\quad FS := \frac{M_{resis}}{M_{app}} = 1.895$ > 1.5, good

North Wind Load

$w_{ww} := 20.946\ psf$ $\quad w_{wp} := 40.168\ psf$ $\quad w_{lp} := -26.779\ psf$

$h_w := 30\ ft$ $\quad h_p := 3.5\ ft$ $\quad l_{sh} := 30\ ft$ $\quad l := 225\ ft$

$$W := \left(w_{ww} \cdot \frac{h_w}{2} \cdot l\right) + \left(w_{wp} \cdot h_p \cdot l\right) - \left(w_{lp} \cdot h_p \cdot l\right) = 123.414\ kip$$

$V_l := \frac{W}{2} = 61.707\ kip$ $\qquad V_r := \frac{W}{2} = 61.707\ kip$

$$M_{diaphram} := \frac{W \cdot l}{8} = \left(3.471 \cdot 10^3\right)\ kip \cdot ft$$

Check sliding resistance

Example 3.7.2

$W_{footing} := \left(150\ pcf \cdot 1.5\ ft \cdot 2.25\ ft \cdot l_{sh}\right) = 15.188\ kip$

$W_{wall} := \left(150\ pcf \cdot 35.5\ ft \cdot 1.25\ ft \cdot l_{sh}\right) = 199.688\ kip$

$W_{backfill} := \left(100\ pcf \cdot 3.5\ ft \cdot l_{sh} \cdot 1.25\ ft\right) = 13.125\ kip$

$W_{total} := W_{footing} + W_{wall} + W_{backfill} = 228\ kip$

$\mu_s := 0.5$ $\quad V_{sliding} := \mu_s \cdot W_{total} = 114\ kip$ $\quad FS := \frac{V_{sliding}}{V_l} = 1.847$

Check overturning resistance

$M_{app} := V_l \cdot (33.5\ ft) = \left(2.067 \cdot 10^3\right)\ kip \cdot ft$

$M_{resis} := W_{total} \cdot \left(0.5 \cdot l_{sh}\right) = \left(3.42 \cdot 10^3\right)\ kip \cdot ft$ $\quad FS := \frac{M_{resis}}{M_{app}} = 1.654$

Using 15' panels with 5' tie spacing
NS - 2 panels
EW - 3 panels

Shear ties in roof

$$V_{app} := \left(\frac{\left(\frac{l}{2} - 12\ ft \right)}{0.5 \cdot l} \right) \cdot V_l = 55.125\ kip$$

$$V_u := \frac{V_{app} \cdot 1.3 \cdot 1.2}{l_{sh}} = 2.866\ \frac{kip}{ft}$$ using #4 ties $$s_{req} := \frac{15.3\ kip}{V_u} = 5.338\ ft$$

Shear ties at shear wall

$$V_u := \frac{V_l \cdot 1.3 \cdot 1.2}{l_{sh}} = 3.209\ \frac{kip}{ft}$$ $$s_{req} := \frac{15.3\ kip}{V_u} = 4.768\ ft$$

Reinforcement

Shear

$f_y := 60\ ksi$ $b_w := 1.25\ ft$ $s := 12\ in$ $d_b := 1.27\ in$ using #10

$$A_v := 50\ psi \cdot b_w \cdot \frac{s}{f_y} = 0.15\ in^2$$

#10 spaces 12" on center
#3 ties

Ties
- Clear spacing = (4/3)dagg
- Center-to-center < 16db (longitudinal), 48db (tie)
- Either
  - #3 enclosing #10 or smaller long bars - 1.5in cover
  - #4 enclosing #11 or smaller long bars
- Bottom tie located not more than 1/2 tie spacing from top of footing or slab

G1 (Vulcraft Joist Girder Specs)

$D := 26\ psf$ $L := 20\ psf$ $S := 45.54\ psf$ $w_{weight} := 310\ plf$ $s := \frac{67}{12}\ ft$

$$P_{D\_G1} := D \cdot s \cdot \left(\frac{150\ ft}{2} + \frac{65\ ft}{2}\right) + w_{weight} \cdot s = 17.336\ kip$$

$$P_{L\_G1} := L \cdot s \cdot \left(\frac{150}{2}\ ft + \frac{65}{2}\ ft\right) = 12.004\ kip$$

$$P_{S\_G1} := S \cdot s \cdot \left(\frac{150}{2}\ ft + \frac{65}{2}\ ft\right) = 27.333\ kip$$

ASD Load Combinations

$$P_{D\_G1} + P_{L\_G1} + P_{S\_G1} = 56.674\ kip$$

1a. D
2a. D + L
3a. D + (Lr or 0.7S or R)
4a. D + 0.75L + 0.75(Lr or 0.7S or R)
5a. D + 0.6(W or WT)
6a. D + 0.75L + 0.75(0.6(W or WT) + 0.75(Lr or 0.7S or R)
7a. 0.6D + 0.6(W or WT)

- **ASD**

1a.)

$$P_{ASD\_G\_1a} := \left[P_{D\_G1}\right] = [17.336]\ kip$$

2a.)

$$P_{ASD\_G\_2a} := \left[P_{D\_G1} + P_{L\_G1}\right] = [29.34]\ kip$$

3a.)

$$P_{ASD\_G\_3a} := \left[P_{D\_G1} + \max\left(P_{L\_G1}, 0.7 \cdot P_{S\_G1}\right)\right] = [36.47]\ kip$$

4a.)

$$P_{ASD\_G\_4a} := \left[P_{D\_G1} + 0.75 \cdot P_{L\_G1} + 0.75 \cdot \max\left(P_{L\_G1}, 0.7 \cdot P_{S\_G1}\right)\right] = [40.689]\ kip$$

5a.)

$$P_{ASD\_G\_5a} := \left[P_{D\_G1}\right] = [17.336]\ kip$$

6a.)

$$P_{ASD\_G\_6a} := \left[P_{D\_G1} + 0.75 \cdot P_{L\_G1} + 0.75 \cdot \max\left(P_{L\_G1}, 0.7 \cdot P_{S\_G1}\right)\right] = [40.689]\ kip$$

7a.)

$$P_{ASD\_G\_7a} := \left[0.6 \cdot P_{D\_G1}\right] = [10.402]\ kip$$

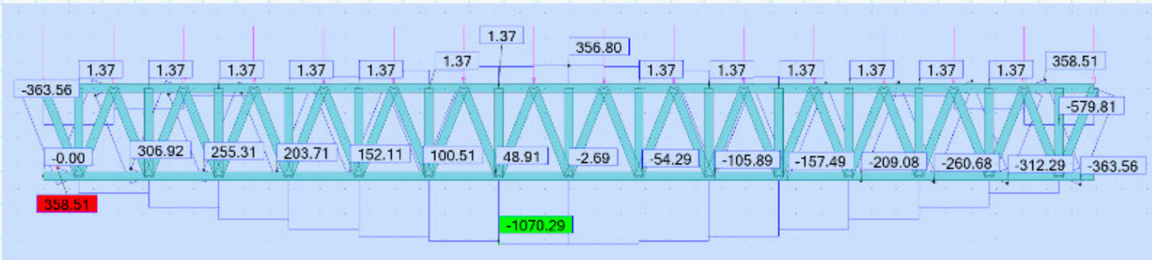
GIRDER SPAN (ft)	JOIST SPACES (ft)	GIRDER DEPTH (in)	JOIST GIRDER WEIGHT -- POUNDS PER LINEAR FOOT																			
			LOAD ON EACH PANEL POINT -- KIPS																			
			ASD																			
			6	8	10	12	14	16	18	20	22	24	26	28	30	35	40	45	50	55	60	65
			LRFD																			
			9	12	15	18	21	24	27	30	33	36	39	42	45	52.5	60	67.5	75	82.5	90	97.5
90	9N@ 10.00	72	46	55	64	81	92	98	117	119	141	143	159	160	182	202	233	238	287			
		84	48	50	60	75	84	88	102	121	124	135	148	154	165	180	218	228	254	286	298	
		90	56	57	62	72	85	88	99	105	125	128	138	152	159	174	211	218	251	272	294	
		96	57	58	64	69	80	91	98	107	110	128	131	142	155	170	199	208	246	263	290	302
		102	57	59	62	69	75	87	95	105	112	130	133	134	146	167	191	196	241	257	281	296
	10N@ 9.00	72	48	61	72	85	99	118	130	142	155	160	170	182	186	236	248	255				
		84	49	58	69	81	97	115	117	137	148	153	165	173	177	210	240	252	292	307		
		90	50	56	66	79	89	100	107	126	129	141	157	167	176	186	232	248	283	292		
		96	48	56	66	74	87	95	108	113	129	133	153	159	168	173	217	223	259	268		
		102	48	57	65	76	84	97	105	115	124	131	137	155	162	170	200	222	245	257	288	
	11N@ 8.18	72	51	65	78	99	119	120	143	150	172	182	188	208	219	240	273					
		84	50	62	74	87	100	113	126	138	150	166	177	192	194	215	258	288				
		90	51	59	72	85	93	107	128	129	142	158	170	182	188	204	250	272	291			
		96	53	60	71	81	95	105	113	132	134	148	167	174	183	194	242	266	296	310		
		102	57	61	70	82	94	101	116	124	138	150	163	167	177	191	235	254	278	292		
	12N@ 7.50	78	53	68	79	102	111	124	149	162	172	183	193	210	225	258	276					
		84	52	65	79	91	105	125	137	149	166	176	188	195	213	245	270	277				
		90	52	68	79	89	106	126	128	151	152	169	182	190	205	219	262	269				
		96	52	63	76	90	103	110	129	132	153	156	175	185	198	210	252	265	293			
		108	55	64	76	85	97	107	115	135	137	160	168	179	183	198	237	261	290	295		
	15N@ 6.00	78	66	82	99	121	145	148	179	188	201	216	237	252	270							
		84	62	76	97	122	125	149	169	183	192	204	220	247	252	296						
		90	60	78	90	106	127	140	153	178	183	200	213	221	249	284	308					
		96	58	72	93	108	129	131	154	173	176	196	198	218	226	272	300					
		108	59	72	87	101	115	136	139	168	172	183	186	206	218	263	275	299				
	18N@ 5.00	78	74	99	120	145	159	177	198	215	238	253	270	292								
		84	73	89	113	137	151	169	192	207	216	244	260	272	298							
		90	70	90	106	129	153	166	185	198	212	228	250	267	280							
		96	68	87	108	131	144	158	179	192	204	220	235	260	275	291						
		108	64	85	103	120	139	151	172	189	192	216	231	243	263	276						

GIRDER SPAN (ft)	JOIST SPACES (ft)	GIRDER DEPTH (in)																				
		90	52	69	81	95	118	133	145	146	177	179	182	202	212	246	280	294				
80	8N@ 10.00	60	37	45	56	64	75	88	97	103	112	127	137	156	162	189	208	234	261	277		
		66	35	45	52	62	70	77	90	103	105	113	129	131	155	176	197	226	243	258	287	
		72	33	41	48	59	68	76	87	92	106	108	116	126	141	170	187	203	235	250	268	288
		78	33	41	47	56	64	73	81	88	94	109	111	118	136	156	178	195	216	238	255	269
		84	35	39	48	56	63	71	79	83	96	98	112	114	129	146	173	184	210	216	247	256
		90	56	57	58	63	70	79	79	90	95	103	105	118	127	134	169	179	205	210	243	252
	10N@ 8.00	60	41	53	68	76	97	103	112	129	139	159	180	191	195	234	255	267				
		66	39	52	62	75	90	100	107	115	132	154	167	178	187	210	245	257				
		72	43	55	63	74	87	97	106	120	127	151	161	171	182	195	238	252				
		78	42	51	63	71	86	90	100	112	122	130	155	166	176	187	229	245	281	290		
		84	42	51	61	70	78	91	100	109	115	125	131	157	166	178	222	230	256	277		
		90	40	49	60	68	77	87	92	102	111	118	132	136	160	169	197	221	239	261	293	
	12N@ 6.67	66	50	65	73	90	103	115	130	161	172	180	195	207	220	254	290					
		72	47	59	72	86	101	107	125	133	165	174	183	196	210	243	273					
		78	46	60	69	80	94	108	114	129	136	167	176	189	197	220	263	276				
		84	47	56	70	79	92	99	111	121	138	140	170	175	193	207	250	265	301			
		90	44	56	66	74	86	101	113	116	125	143	149	170	177	193	242	259	281	301		
		96	43	54	68	75	85	98	104	117	120	130	147	156	180	195	233	251	277	295	308	
	14N@ 5.71	66	57	73	89	103	113	129	160	182	186	207	221	231	262	288						
		72	54	67	79	101	106	125	143	165	184	198	211	224	243	277						
		78	50	66	78	95	109	118	136	149	173	191	197	216	226	266	297					
		84	50	64	74	92	99	112	124	143	169	177	191	203	218	250	276					
		90	48	61	74	86	100	115	121	136	146	172	181	195	208	227	264	275				
		96	47	61	74	84	100	108	118	127	145	152	177	181	201	215	258	266				
	16N@ 5.00	66	62	78	101	113	130	161	184	197	212	233	253	268	288	308						
		72	57	76	93	109	118	145	167	187	203	218	246	258	270	291						
		78	58	73	91	104	120	137	149	181	191	208	219	242	252	282						
		84	54	69	84	100	115	126	143	174	185	195	211	223	250	271	315					
		90	54	70	80	101	114	119	144	155	180	191	207	220	233	267	306					
		96	55	68	81	94	110	121	133	155	164	186	201	211	226	264	298	302				

83.75G15N40.69K

83.75G15N41K

Axial Loads of truss chords Top:(2L6x6x3/4) Bottom:(2L8x8x3/4)



Tension

$F_y := 50\ ksi$ $\sigma_{all.T} := 0.6\ F_y = 30\ ksi$ $P_{max.top} := 356.8\ kip$ $E := 29000\ ksi$

$A_{top} := 16.9\ in^2$ $A_{bot} := 30.2\ in^2$ $\sigma := \dfrac{P_{max.top}}{A_{top}} = 21.112\ ksi$ good

Compression $P_{max.bot} := 1070.3\ kip$

$k := 1$ $l := 83.75\ ft$ $I := 56.2\ in^4$ $r := 1.82\ in$ $Q := 1$

$\dfrac{k \cdot l}{r} = 552.198$ $4.71 \cdot \sqrt{\dfrac{E}{Q \cdot F_y}} = 113.432$

$F_e := \dfrac{\pi^2 \cdot E}{\left(\dfrac{k \cdot l}{r}\right)^2} = 0.939\ ksi$ $F_{cr} := 0.877 \cdot F_e = 0.823\ ksi$

$\sigma_{all.C} := 0.6 \cdot F_{cr} = 0.494\ ksi$ good

G3 (AISC-specs Section F7)

$D := 17\ psf$ $L := 20\ psf$ $S := 45.54\ psf$ $w_{weight} := 25\ plf$

$$w_{D\_G3} := D \cdot \left(\frac{26}{2}\ ft + \frac{24}{2}\ ft\right) + w_{weight} = 0.45\ klf$$

$$w_{L\_G3} := L \cdot \left(\frac{26}{2}\ ft + \frac{24}{2}\ ft\right) = 0.5\ klf$$

$$w_{S\_G3} := S \cdot \left(\frac{26}{2}\ ft + \frac{24}{2}\ ft\right) = 1.139\ klf$$

- **ASD**

1a.)

$$w_{ASD\_G\_1a} := \left[w_{D\_G3}\right] = \left[0.45\right]\ klf$$

2a.)

$$w_{ASD\_G\_2a} := \left[w_{D\_G3} + w_{L\_G3}\right] = \left[0.95\right]\ klf$$

3a.)

$$w_{ASD\_G\_3a} := \left[w_{D\_G3} + \max\left(w_{L\_G3}, 0.7 \cdot w_{S\_G3}\right)\right] = \left[1.247\right]\ klf$$

4a.)

$$w_{ASD\_G\_4a} := \left[w_{D\_G3} + 0.75 \cdot w_{L\_G3} + 0.75 \cdot \max\left(w_{L\_G3}, 0.7 \cdot w_{S\_G3}\right)\right] = \left[1.423\right]\ klf$$

5a.)

$$w_{ASD\_G\_5a} := \left[w_{D\_G3}\right] = \left[0.45\right]\ klf$$

6a.)

$$w_{ASD\_G\_6a} := \left[w_{D\_G3} + 0.75 \cdot w_{L\_G3} + 0.75 \cdot \max\left(w_{L\_G3}, 0.7 \cdot w_{S\_G3}\right)\right] = \left[1.423\right]\ klf$$

7a.)

$$w_{ASD\_G\_7a} := \left[0.6 \cdot w_{D\_G3}\right] = \left[0.27\right]\ klf$$

G3(AISC-specs Section F7) $\Omega_b := 1.67$

$w_{G3} := w_{ASD\_G\_4a} = [1.423]\ klf$ $L_{G3} := 19.5\ ft = 19.5\ ft$

Flexure

$M_{max\_G3} := \dfrac{w_{G3} \cdot L_{G3}^{\ 2}}{8} = [67.623]\ kip \cdot ft$ $M_u := M_{max\_G3} = [67.623]\ kip \cdot ft$

Needed Strength

$M_u \cdot \Omega_b = [112.931]\ kip \cdot ft$

$Z_{x\_needed} := \dfrac{M_u \cdot \Omega_b}{50\ ksi} = [27.103]\ in^3$

W14x22

$F_y := 50\ ksi$ $Z_x := 33.2\ in^3$ $E := 29000\ ksi$

Yielding

$M_p := F_y \cdot Z_x = 138.333\ kip \cdot ft$

$M_n := M_p = 138.333\ kip \cdot ft$ $\left(\dfrac{M_n}{\Omega_b}\right) = 82.834\ kip \cdot ft$

$\dfrac{\left(\dfrac{M_n}{\Omega_b}\right)}{M_u} = [1.225]$ good, Mn/Ob > Mu

Shear

$V_{max\_G3} := \dfrac{w_{G3} \cdot L_{G3}}{2} = [13.871]\ kip$ $V_u := V_{max\_G3}$

$d := 13.75\ in$ $t_w := 0.23\ in$ $htw := 53.3$ $2.24 \cdot \sqrt{\dfrac{E}{F_y}} = 53.946$

$A_w := d \cdot t_w = 3.163\ in^2$ $C_{v1} := 1$

$V_n := 0.6 \cdot F_y \cdot A_w \cdot C_{v1} = 94.875\ kip$

$\dfrac{\left(\dfrac{V_n}{\Omega_b}\right)}{V_u} = [4.096]$ good, Vn/Ob > Vu

Deflection $I_{G3} := 199\ in^4$

$\delta_{max} := \dfrac{5 \cdot w_{G3} \cdot L_{G3}^{\ 4}}{384 \cdot E \cdot I_{G3}} = [0.802]\ in$ $\dfrac{L_{G3}}{260} = 0.9\ in$ good

G4

$D := 17\ psf$ $L := 20\ psf$ $S := 45.54\ psf$ $w_{weight} := 45\ plf$

$$w_{D\_G4} := D \cdot \left(\frac{26}{2}\ ft + \frac{24}{2}\ ft\right) + w_{weight} = 0.47\ klf$$

$$w_{L\_G4} := L \cdot \left(\frac{26}{2}\ ft + \frac{24}{2}\ ft\right) = 0.5\ klf$$

$$w_{S\_G4} := S \cdot \left(\frac{26}{2}\ ft + \frac{24}{2}\ ft\right) = 1.139\ klf$$

- **ASD**

1a.)

$$w_{ASD\_G\_1a} := \left[w_{D\_G4}\right] = [0.47]\ klf$$

2a.)

$$w_{ASD\_G\_2a} := \left[w_{D\_G4} + w_{L\_G4}\right] = [0.97]\ klf$$

3a.)

$$w_{ASD\_G\_3a} := \left[w_{D\_G4} + \max\left(w_{L\_G4}, 0.7 \cdot w_{S\_G4}\right)\right] = [1.267]\ klf$$

4a.)

$$w_{ASD\_G\_4a} := \left[w_{D\_G4} + 0.75 \cdot w_{L\_G4} + 0.75 \cdot \max\left(w_{L\_G4}, 0.7 \cdot w_{S\_G4}\right)\right] = [1.443]\ klf$$

5a.)

$$w_{ASD\_G\_5a} := \left[w_{D\_G4}\right] = [0.47]\ klf$$

6a.)

$$w_{ASD\_G\_6a} := \left[w_{D\_G4} + 0.75 \cdot w_{L\_G4} + 0.75 \cdot \max\left(w_{L\_G4}, 0.7 \cdot w_{S\_G4}\right)\right] = [1.443]\ klf$$

7a.)

$$w_{ASD\_G\_7a} := \left[0.6 \cdot w_{D\_G4}\right] = [0.282]\ klf$$

$$\Omega_b := 1.67$$

$w_{G4} := w_{ASD\_G\_4a} = [1.443]\ klf$ $L_{G4} := 26.5\ ft = 26.5\ ft$

Flexure

$M_{max\_G4} := \dfrac{w_{G4} \cdot L_{G4}^{\ 2}}{8} = [126.643]\ \boldsymbol{kip \cdot ft}$ $\qquad M_u := M_{max\_G4} = [126.643]\ \boldsymbol{kip \cdot ft}$

Needed Strength

$M_u \cdot \Omega_b = [211.494]\ \boldsymbol{kip \cdot ft}$

$Z_{x\_needed} := \dfrac{M_u \cdot \Omega_b}{50\ \boldsymbol{ksi}} = [50.759]\ \boldsymbol{in}^3$

Try W14x43

$F_y := 50\ \boldsymbol{ksi}$ $\qquad Z_x := 69.6\ \boldsymbol{in}^3$ $\qquad E := 29000\ \boldsymbol{ksi}$

Yielding

$M_p := F_y \cdot Z_x = 290\ \boldsymbol{kip \cdot ft}$

$M_n := M_p = 290\ \boldsymbol{kip \cdot ft}$ $\qquad \left(\dfrac{M_n}{\Omega_b}\right) = 173.653\ \boldsymbol{kip \cdot ft}$

$\dfrac{\left(\dfrac{M_n}{\Omega_b}\right)}{M_u} = [1.371]$ $\qquad$ good, Mn/Ob > Mu

Shear

$V_{max\_G4} := \dfrac{w_{G4} \cdot L_{G4}}{2} = [19.116]\ \boldsymbol{kip}$ $\qquad V_u := V_{max\_G4}$

$d := 13.625\ \boldsymbol{in}$ $\qquad t_w := 0.305\ \boldsymbol{in}$ $\qquad htw := 37.4$ $\qquad 2.24 \cdot \sqrt{\dfrac{E}{F_y}} = 53.946$

$A_w := d \cdot t_w = 4.156\ \boldsymbol{in}^2$ $\qquad C_{v1} := 1$

$V_n := 0.6 \cdot F_y \cdot A_w \cdot C_{v1} = 124.669\ \boldsymbol{kip}$

$\dfrac{\left(\dfrac{V_n}{\Omega_b}\right)}{V_u} = [3.905]$ $\qquad$ good, Vn/Ob > Vu

Deflection $\qquad I_{G4} := 428\ \boldsymbol{in}^4$

$\delta_{max} := \dfrac{5 \cdot w_{G4} \cdot L_{G4}^{\ 4}}{384 \cdot E \cdot I_{G4}} = [1.29]\ \boldsymbol{in}$ $\qquad \dfrac{L_{G4}}{240} = 1.325\ \boldsymbol{in}$ $\qquad$ good

COLUMNS (C1, C2, C3)

$L_{C1} := 30\ ft$ $\qquad P_{C1} := 380\ kip + 65\ plf \cdot L_{C1} = 381.95\ kip$

$L_{C2} := 12\ ft$ $\qquad P_{C2} := V_{max\_G4} + V_{max\_G3} + 30\ plf \cdot L_{C2} = [33.347]\ kip$

$L_{C3} := 30\ ft$ $\qquad P_{C3} := 2 \cdot P_{C1} + 120\ plf \cdot L_{C3} = 767.5\ kip$

C1

Required Axial Strength

$F_y = 50\ ksi$ $\qquad L_{c\_C1} := L_{C1} = 30\ ft$ $\qquad P_r := P_{C1} = 381.95\ kip$

Try W12x65 $\qquad r_{C1} := 5.18\ in$ $\qquad A_g := 19.1\ in^2$ $\qquad Z_x := 96.8\ in^3$

Allowable Axial Strength

$$4.71 \cdot \sqrt{\frac{E}{F_y}} = 113.432 \qquad \frac{L_{c\_C1}}{r_{C1}} = 69.498 \qquad F_e := \frac{\pi^2 \cdot E}{\left(\frac{L_{c\_C1}}{r_{C1}}\right)^2} = 59.259\ ksi$$

$$F_{cr} := \left(0.658^{\frac{F_y}{F_e}}\right) \cdot F_y = 35.123\ ksi$$

$$P_n := F_{cr} \cdot A_g = 670.857\ kip \qquad P_c := \frac{P_n}{\Omega_b} = 401.711\ kip$$

$$\frac{P_c}{P_r} = 1.052$$ good

C2

Required Axial Strength

$F_y = 50\ ksi$ $\quad L_{c\_C2} := L_{C2} = 12\ ft$ $\quad P_r := P_{C2} = [33.347]\ kip$

Try W8x15 $\quad r_{C2} := 3.29\ in$ $\quad A_g := 4.44\ in^2$ $\quad Z_x := 27.2\ in^3$

Allowable Axial Strength

$$4.71 \cdot \sqrt{\frac{E}{F_y}} = 113.432 \qquad \frac{L_{c\_C2}}{r_{C2}} = 43.769 \qquad F_e := \frac{\pi^2 \cdot E}{\left(\frac{L_{c\_C2}}{r_{C2}}\right)^2} = 149.405\ ksi$$

$$F_{cr} := \left(0.658^{\frac{F_y}{F_e}}\right) \cdot F_y = 43.465\ ksi$$

$$P_n := F_{cr} \cdot A_g = 192.984\ kip \qquad P_c := \frac{P_n}{\Omega_b} = 115.559\ kip$$

$$\frac{P_c}{P_r} = [3.465]$$ good

C3

Required Axial Strength

$F_y = 50\ ksi$ $\quad L_{c\_C3} := L_{C3} = 30\ ft$ $\quad P_r := P_{C3} = 767.5\ kip$

Try W14x120 $\quad r_{C3} := 6.24\ in$ $\quad A_g := 35.3\ in^2$ $\quad Z_x := 175\ in^3$

Allowable Axial Strength

$$4.71 \cdot \sqrt{\frac{E}{F_y}} = 113.432 \qquad \frac{L_{c\_C3}}{r_{C3}} = 57.692 \qquad F_e := \frac{\pi^2 \cdot E}{\left(\frac{L_{c\_C3}}{r_{C3}}\right)^2} = 85.993\ ksi$$

$$F_{cr} := \left(0.658^{\frac{F_y}{F_e}}\right) \cdot F_y = 39.199\ ksi$$

$$P_n := F_{cr} \cdot A_g = \left(1.384 \cdot 10^3\right)\ kip \qquad P_c := \frac{P_n}{\Omega_b} = 828.584\ kip$$

$$\frac{P_c}{P_r} = 1.08$$ good

Foundations

FT6

$q_{max} := 1500\ psf$ $\gamma_{conc} := 150\ pcf$ $b_{FT6} := 5.25\ ft$ $h_{FT6} := 5.25\ ft$ $d_{FT6} := 1\ ft$

$A_{FT6} := b_{FT6} \cdot h_{FT6} = 27.563\ ft^2$

Applied Stress

$$\frac{P_{C2}}{q_{max}} = [22.232]\ ft^2$$

$$q_{FT6} := \frac{P_{C2}}{A_{FT6}} + d_{FT6} \cdot \gamma_{conc} = [1.36 \cdot 10^3]\ psf$$

$q_{FT6} < q_{max}$ good

FT7

$q_{max} := 1500\ psf$ $\gamma_{conc} := 150\ pcf$ $b_{FT7} := 18\ ft$ $h_{FT7} := 18\ ft$ $d_{FT7} := 1\ ft$

$A_{FT7} := b_{FT7} \cdot h_{FT7} = 324\ ft^2$

$$A_{min} := \frac{P_{C1}}{q_{max}} = 254.633\ ft^2$$

Applied Stress

$$q_{FT7} := \frac{P_{C1}}{A_{FT7}} + d_{FT7} \cdot \gamma_{conc} = (1.329 \cdot 10^3)\ psf$$

$q_{FT7} < q_{max}$ good

FT8

$q_{max} := 1500\ psf$ $\gamma_{conc} := 150\ pcf$ $b_{FT8} := 25\ ft$ $h_{FT8} := 25\ ft$ $d_{FT8} := 1\ ft$

$A_{FT8} := b_{FT8} \cdot h_{FT8} = 625\ ft^2$

$$A_{min} := \frac{P_{C3}}{q_{max}} = 511.667\ ft^2$$

Applied Stress

$$q_{FT8} := \frac{P_{C3}}{A_{FT8}} + d_{FT8} \cdot \gamma_{conc} = (1.378 \cdot 10^3)\ psf$$

$q_{FT8} < q_{max}$ good

Slabs

Equipment Area

Axle Load = 32 kips
Wheel Spacing = 72 in
# of wheels per axle = 2
Tire pressure = 110 psi

Tire contact area = (32000/2)/110 = 146 sq in

Subgrade modulus, k = 200 pci
Concrete flexural strength, MR = 640 psi
Safety Factor = 2

Working Stress = MR/SF = 320 psi

Slab stress per kip of axle load = 320pci/32 kips = 10 psi

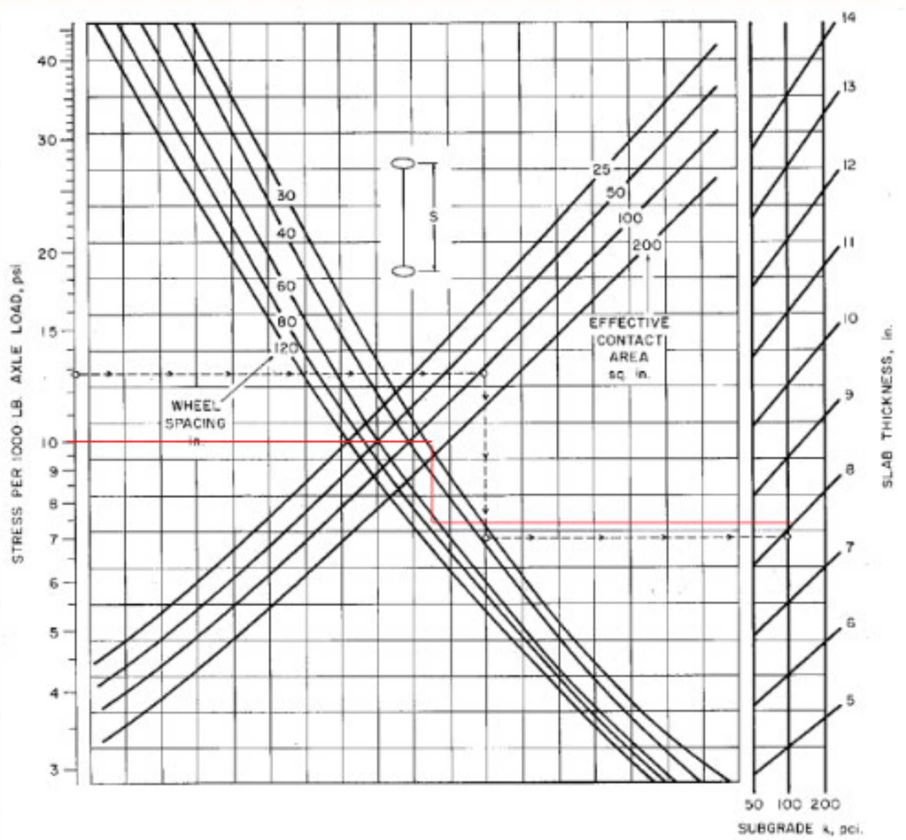


Fig. 3. Design chart for axles with single wheels.

8 in slab

Office Area

5 in slab

Occupancy **	Min. Slab Thickness	Reinforcement ‡
Sub-slabs under other slabs	2″	None
Domestic or light commercial (loaded less than 100 psf)	4″	One layer 6 x 6 10/10 welded wire fabric, minimum for ideal conditions; 6 x 6 8/8 for average conditions.
Commercial—institutional—barns (loaded 100-200 psf)	5″	One layer 6 x 6 8/8 welded wire fabric or one layer 6 x 6 6/6.
Industrial (loaded not over 400-500 psf) and pavements for industrial plants, gas stations, and garages	6″	One layer 6 x 6 6/6 welded wire fabric or one layer 6 x 6 4/4.
Industrial (loaded 600–800 psf) and heavy pavements for industrial plants, gas stations, and garages	6″	Two layers 6 x 6 6/6 welded wire fabric or two layers 6 x 6 4/4
Industrial (loaded 1500 psf) †	7″	Two mats of bars (one top, one bottom), each of #4 bars @ 12″ c/c, each way
Industrial (loaded 2500 psf) †	8″	Two mats of bars (one top, one bottom), each of #5 bars @ 12″ c/c, each way
Industrial (loaded 3000–3500 psf) †	9″	Two mats of bars (one top, one bottom), each of #5 bars @ 8″ to 12″ c/c, each way

Storage Racks

$A_{rack} := 48\ in \cdot 96\ in = \left(4.608 \cdot 10^{3}\right)\ in^{2}$ 3 levels

$q_{rack} := 150\ psf$

$P_{post} := 3 \cdot A_{rack} \cdot q_{rack} = 14.4\ kip$

$A_{contact} := 64\ in^{2}$

$k := 200\ pci$ $MR := 640\ psi$ $SF := 3$

$WS := \dfrac{MR}{SF} = 213.333\ psi$ $\sigma_{slab} := \dfrac{214}{14.4} = 14.861$ psi per kip of post load

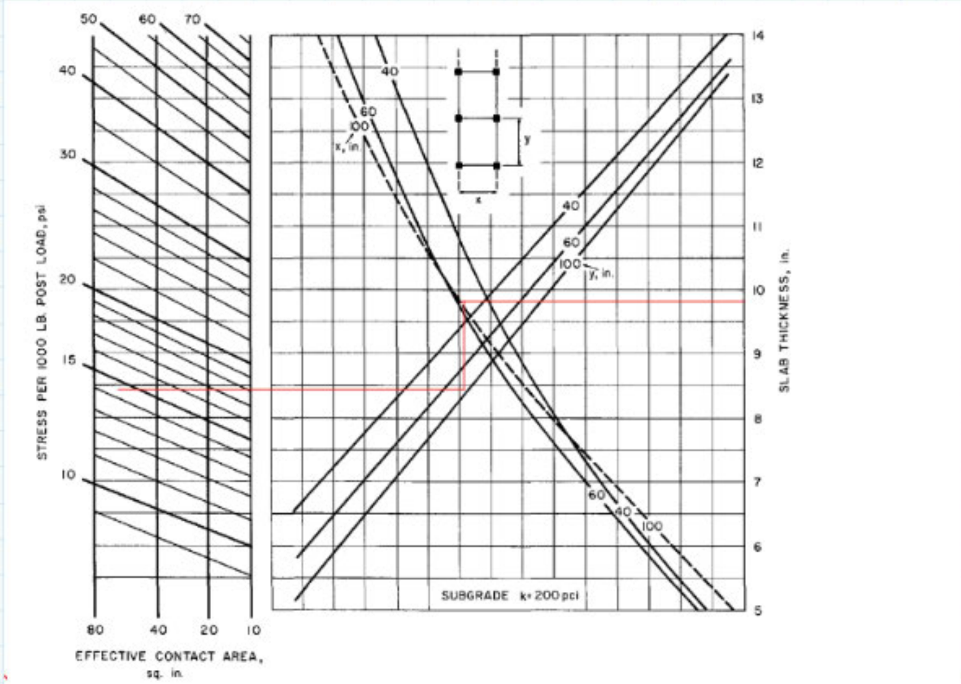


Fig. 7c. Design chart for post loads, subgrade k = 200 pci.

10 in slab

Appendix B: Outdoor Storage Specifications

FarmTek

ClearSpan™ Round Extra-Tall HD Buildings

Select configuration

Width *	Length *	Height *	Cover Color *
56'	60'	26'11"	Gray

ClearSpan Round Extra-Tall HD Building - 56'W x 60'L Gray

$26,655.00/EA

Quantity*
1

Add to Cart

Item Number: TT5606020FG

Availability: Usually available in 10 days (Manufactured Product)

ClearSpan™ HD Buildings are designed, manufactured and constructed with the highest structural integrity.

• High clearance and wide-open space of these structures make them ideal for virtually any application.

• 12.5 oz., 24 mil rip-stop polyethylene covers are UV resistant and available in your choice of four colors.

• Durable frames are manufactured from our American-made, triple-galvanized structural steel, which is resistant to corrosive environments and long lasting.

• 56'W buildings are 26'9-7/8"H.

• Truss spacing is 20' on center.

• Available in freestanding round style.

• Industry-leading 20 year warranty on cover and 50 year warranty on frame.

• Custom covers, end panels and accessories are available, all sold separately.

Appendix C: Additional Renderings

Figure A1. Reception Area

Figure A2. Hallway to ADA Restrooms

Figure A3. Single ADA Restrooms

Figure A4. West Hallway

Figure A5. Private Office Along West Wall

Figure A6. Private Center Office

Figure A7. Cubicle/ Open Office Area

Figure A8. Break and Training Room from Southwest Corner

Figure A9. Break and Training Room from South Wall

Figure A10. Break and Training Room from East Wall

Figure A11. Outdoor Employee Patio

Figure A12. East Hallway

Figure A13. Janitor's Closet

Figure A14. First Aid Room

Figure A15. Conference Room

Figure A16. Women's Locker Room

Figure A17. Women's Locker Room Showers

Figure A18. Mechanical Room

Figure A19. Men's Locker Room Restroom

Figure A20. Men's Locker Room

Figure A21. Men's Locker Room Showers

Figure A22. Men's Locker Room Restrooms

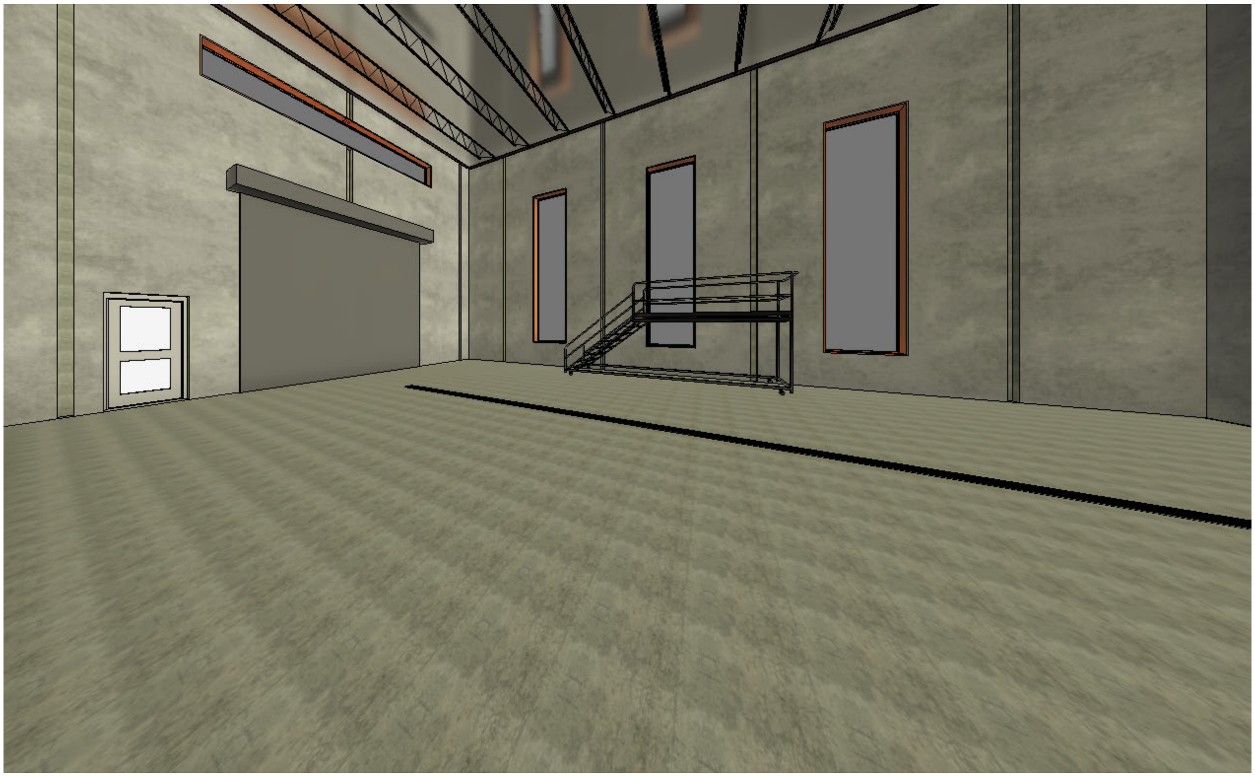


Figure A23. Indoor Wash Bay

Figure A24. West Office Exterior

Figure A25. Northwest Office Exterior

Figure A26. Facility from Above Without Roof

Figure A27. Facility Exterior from Southeast Corner

Figure A28. Warehouse Interior from Northeast Corner

Figure A29. Mechanic's Bay Interior

Figure A30. Office Interior Plan View