Hurstville Lime Kilns Improvement Final Design Report

CEE 4850 – Project Design and Management

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The University of Iowa
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Table of Contents

Executive Summary……………………………………………………………………………pg 3

Organization, Qualifications, and Experience…………………………………………………pg 4

Proposed Services ………………………………………………………………………….…pg 5

Constraints, Challenges, and Impacts ……………………………………………….......… pg 9

Alternative Solutions………………………………………………….....………....………pg 11

Final Design Details………………………………………………………………………....pg 11

Engineer's Cost Estimate…………………………………………………………………..pg 20

Appendices……………………………………………………………………………….pg 22

Section I Executive Summary

The Hurstville Lime Kilns are located one mile north of Maquoketa, IA along U.S. Highway 61. The site contains four lime kilns, the rock crusher, and the bridge that spans the North Fork of the Maquoketa River. The kilns and rock crusher lie on land owned by the Jackson County Historic Society while the bridge is on the land of a private owner. Our team had been requested to provide structural analysis of the three items and provide engineering recommendations on how to rehabilitate them.

The site is an important landmark to Maquoketa for its historical value and the education it provides to the public. The kilns have been there since the late 19th century and is a source of knowledge to the community in highlighting the growth of the area as the land slowly industrialized as the kilns provided a key component for mortar which in turn gave back to the land with the urbanization of Iowa and surrounding states. The preservation of the site is of the upmost importance to continue this spread of knowledge to the community for years to come, which is why our team will provide the best options for keeping the three items standing for as long as possible.

The first item, the kilns and the retaining walls connected, have undergone renovation in the 1980's so both items are rather structurally sound, but analysis was still conducted due to visible deterioration of the surfaces and the future addition of a trail system residing behind the structures that would add additional stresses. Soil nails were designed to be implemented into the retaining wall to counteract the increased soil pressures resulting from the trail and are placed in a diagonal formation and covered with decorative star caps. There are openings on the sides of the kilns where there are existing grates that need to be replaced. A platform will also be constructed at the top of the kilns to provide a better view of the inside as well as provide protection against weather from getting inside the kilns. Protection is also needed to prevent visitors from climbing into the kilns, which is a current problem as stated by the client.

The second item is the rock crusher structure. The building is in dire need of renovation or else a loss of structure will happen in the future. The supports need to be replaced as well as other surfaces such as walls and roofs since there is considerable damage to them with holes and rotting. Any work done on the structure needs to be as safe as possible by having a secure site during work and proper outside support for the structure itself.

The third item is the bridge where part of the span has collapsed. Onsite observation showed that the bracing for the bridge is damaged and needs replacing. The supports at the pier are also in need of work. As the same for the rock crusher, any work done on the bridge needs to be supported with outside reinforcement and the site closed off to prevent any trespassers which is evident with graffiti in the rock crusher structure. It was viewed to see if any material could be salvaged from the collapsed span on the opposite side of the river, but it was deemed that new components are needed to support the standing bridge section. Clearing of vegetation is also needed for the bridge and rock crusher since there are many fallen trees around each item and overgrown plants.

The project came with constraints regarding proposed improvements. A goal for the site was to maintain as much originality as possible to not tarnish its historical significance. Our group was sure to base all our decisions by keeping this in mind when designing our alternatives, such as providing additional support to the rock crusher building rather than rebuilding it. Flooding was also seen as an issue for the site but was discovered that most of the water came from ground water and our designed creations were unaffected. If construction on the site is initiated, proper designation of property lines needs to be established since the site shares a close border of ownership between the Historic Society and the private landowner. Budgeting for the project is provided by donations to the Historic Society so total expenses for each design creation were considered. The final constraint that was looked into was that the site is on the National Register of Historic Places which means that any digging on the site was to be done as an archaeological excavation to look out for artifacts.

There are no existing environmental hazards in the area, however for being near the river, precautionary actions need to be taken during the construction process. Fuel for construction equipment would need to be stored over a tarp covered area that will have no chance of spilling and leaking into the ground water table.

The Hurstville Lime Kilns are of great importance to the area which is why preserving the originality of the site is the top goal for this project. With the introduction of a more public friendly area, safety is a priority, which is why the existing structures need to have additional supports to withstand the coming of time and public interaction. There were many challenges crossed during this endeavor, but the conclusion of this project reflects the hard work that was put into it and the care needed to keep providing the educational value that the site has.

Section II Organization Qualifications and Experience

1. Organization and Design Team Description

The Project Team is three engineering students in the senior design capstone class at the University of Iowa. The project lead was Carson Schuler. Carson was in charge of coordinating project tasks, preparing meeting agendas and organizing presentations. He was also the main contact person for this project. The report editor was Samantha Olson. Samantha's responsibilities include coordinating the writing of all reports, preparing graphics, and editing. Technology support was provided by Caden Fedeler. Caden was in charge of creating a shared electronic drive for all documents produced by the team and helping with all technology needs in relation to the project.

4. Description of Experience with Similar Projects

All members of this team have a focus in structures, mechanics, and materials. Carson has experience with design of structures with classes that provide many areas of design such as concrete, wood and steel structures as well as foundation design. Carson's internship experience in the past has been working with a municipality and working closely with construction inspection with the repair and resurfacing of streets. Samantha has been working for Hubbard Merrell Engineering since May 2021. She has worked on a wide variety of projects, including retaining walls, wood buildings, and steel structures. During these projects, Samantha was responsible for designing loads, members, foundations, retaining walls, and connections, as well as reviewing steel submittals and compiling calculation packets. Samantha also has class experience with design of wood, concrete, and steel structures, as well as foundation design. Caden has experience working in an engineering team from working at Snyder & Associates for two summers. In these internships he learned how to use engineering software programs and developed his problem-solving skills. He has completed many structural classes which have prepared him for this project.

Section III Proposed Services

1. Project Scope

The goal of this project was to rehabilitate the Hurstville Lime Kilns. Structural analysis was performed on the existing structures to determine effects of loads and the distribution of internal stresses. Structures that were considered in evaluation include: four historic lime kilns, three spans of retaining wall between the kilns, and a rock crusher building. There were a few areas of focus for this project. First, the overall safety of the site needed to be improved. There were multiple locations that were potential hazards to visitors including the grates at the sides of the kilns, near the rock crusher, on the bridge, and at the top of the kilns themselves. Another focus of the project was to raise awareness about the kilns. Currently, the historic site is often driven by and can easily be seen from the road, but most people don't know about how the kilns originated. Finally, the kilns and other structures needed to be protected from frequent flooding events.

A viewing platform was designed to be placed on top of one of the kilns. It will allow visitors to safely view the inside of the kilns. The platform incorporates watertight decking so that rainwater is diverted away from the kiln and its smokestack. Also, at the base of the kiln, the existing grates are planned to be removed and replaced with a safer wire mesh grate. Finally, the designed platform will add stresses to the kiln, so we ensured the kiln wouldn't fail under the additional loads.

Figure 3.01 – Existing Method of Viewing at the Top of Kilns

The stability of the retaining wall was analyzed, considering the new loads from the trail and platform. There is a noticeable bulge in the north span and lots of noticeable cracking. We determined that this is likely due to the two large trees located between the northernmost kilns. The trees are planned to be removed and a soil nailing plan will be implemented to repair the adjacent retaining wall.

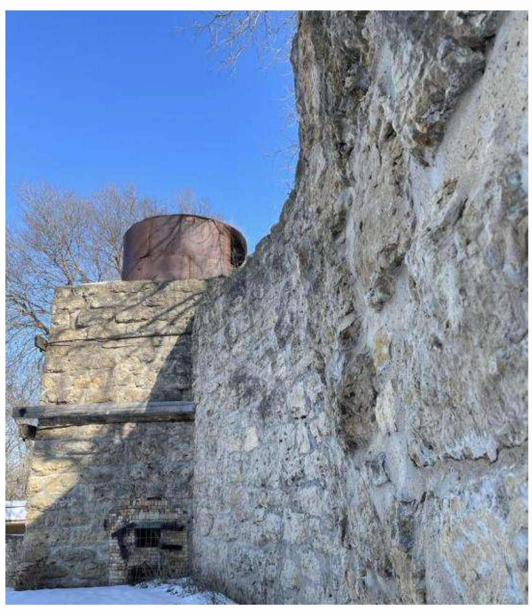

Figure 3.02 – Span of The Retaining Wall with Bulge

It is clear that the rock crusher building is in poor condition. The rotted supports and missing boards make it unsafe. To prevent this building from collapsing, we identified critical members of the building and created a plan to stabilize them. The stabilization plan consists of both temporary and permanent supports.

Figure 3.03 – Rotted Members on The Rock Crusher Building

Finally, a nearby bridge needs repair. Currently, only half of a two-span bridge remains standing on site. The bridge's original purpose was to transport limestone from the quarry across the North Fork Maquoketa River. It should be noted that the rehabilitation of the kilns, retaining wall, and rock crusher take precedence over this bridge, so we only created a plan to temporarily stabilize the bridge rather than restore it completely.

The final deliverables for this project include a presentation, construction drawings, a poster, and this design report.

2. Work Plan

Our group completed the design for this project over the last three months. To track the progress of the project, the Gantt chart shown in *Figure 3.04* was used. The project was divided into seven primary components. The work plan was used as a timeline to ensure that the design phase was completed on schedule.

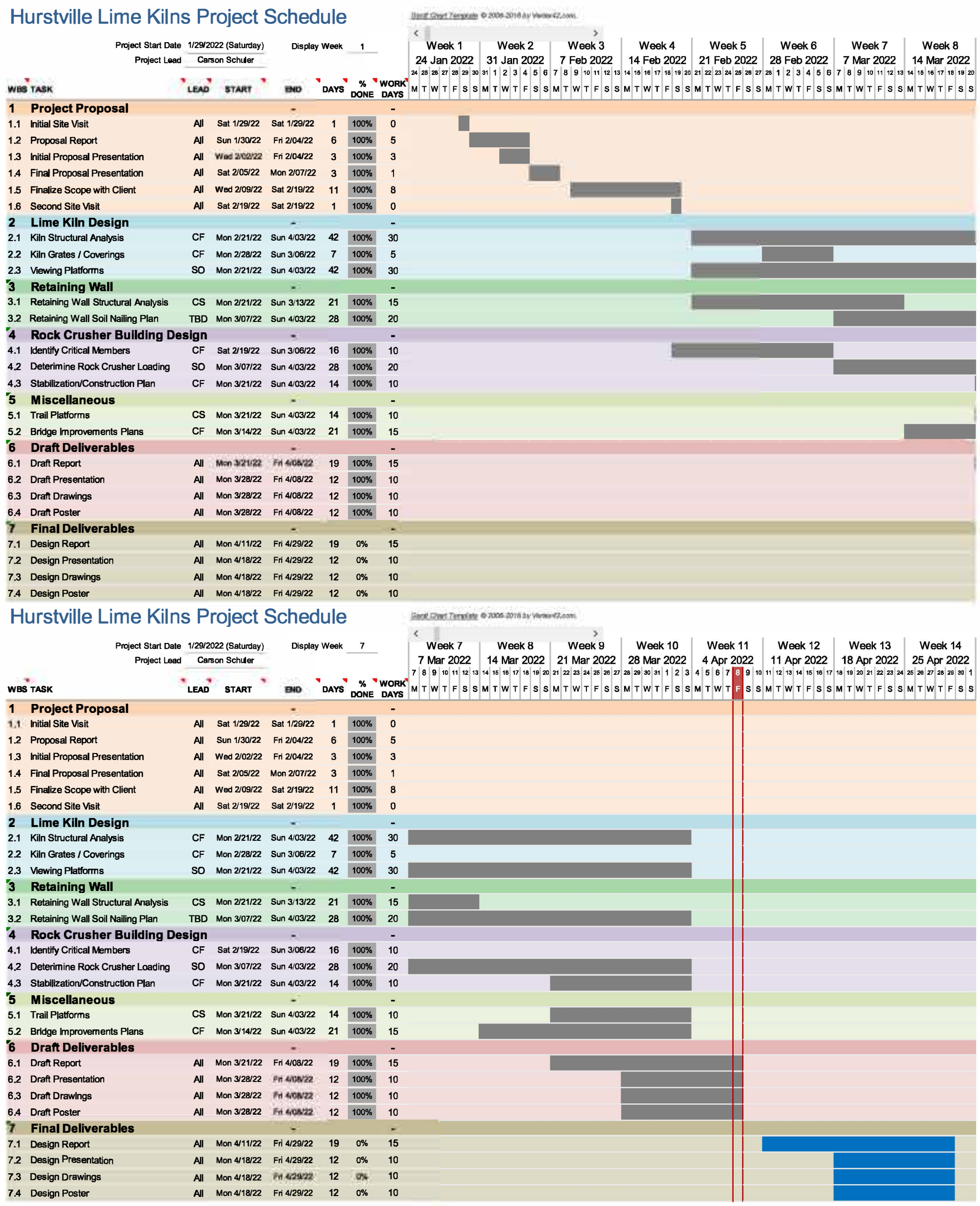

Figure 3.04 - Gantt Chart

3. Methods and Design Guides

This project will be completed in accordance with the Iowa Statewide Urban Design and Specifications (SUDAS) along with the International Building Code 2015 as per the state of Iowa's adopted codes. The Load and Resistance Factor Design (LRFD) was used to evaluate the design of the kilns and retaining wall, as well as to design the anchor bolts. Allowable Stress Design (ASD) was used to design the kiln platforms and wood supports for the bridge and rock crusher. These different designs have different load factors, which suggest different degrees of uncertainty for different loads. Additionally, all improvements made to the bridge follow the Iowa DOT's Bridge Design Manual.

Section IV Constraints, Challenges, and Impacts

1. Constraints

Our team was not given a set budget for this project, but we understand that the past efforts to restore the site relied significantly on public donations. For this reason, we discussed multiple solutions with the client at various price points to allow the clients to select options that best fit their needs and budgetary constraints.

Another constraint on our design was the connection of the kiln and viewing platforms to the trail system. Two engineering groups worked to develop designs for the site simultaneously. Our team focused on the structural elements for the site, while the other group worked on the site development aspects of this project, specifically including the trail design. The design and placement of both the platform that is designed for the top of the kilns as well as the viewing platforms that shall hold the glass etchings relied on the site development group's trail placement and design. Thus, the platforms were designed to fit around the plans made by the other group.

However, perhaps the most important constraint was the safety of those who visit the site. The site features a large retaining wall, on top of which our plans place a trail and a large viewing platform that will place visitors at heights of over 30 feet. Additionally, visitors are currently entering the rock crusher building, which is in poor condition, as well as climbing into the kilns themselves. Our designs meet or exceed the standards presented in the International Building Code so that visitors can safely access the site. In our designs, we included measures, such as grates and additional members, to prevent the lime kilns and rock crusher building from being accessed by the public.

2. Challenges

The Hurstville Lime Kilns are part of the Hurstville Historic District, and the site is listed on the National Register of Historic Places. This site is one of only two in Iowa on which historic lime kilns remain standing. Our team exercised great care to ensure that our designs, once constructed, will not damage the site, as doing so could result in a significant historical loss. Since this is a historic site, our designs strive to strike a balance between historical accuracy and authenticity of the site, while also considering the ability of visitors to enjoy and learn about the lime kilns now and in the future.

Additionally, the site is located in a non-coastal A-Zone and thus experiences a significant level of flooding that occurs on at least a yearly basis. We considered the regular flooding in both our design and selection of materials. Our team worked with the Site Development Team to best address the flooding concerns and mitigation for this challenge.

Another challenge that our team faced was the lack of engineering drawings for the existing structures on the site. Due to this, our team was required to make certain assumptions as we worked on our design. Some of these assumptions included the type of retaining wall and its reinforcement, the type of wood used in the rock crusher building, and the steel grade of the bridge. We strived to make conservative assumptions based on the information we were able to obtain from online resources and the site visits, as well as the experience of both ourselves and our mentors. In order to be conservative, we assumed that the retaining wall was a gravity wall that is only as deep as the bottom of the kilns that we observed on our site visit. We obtained the soil data from boring logs in the nearby area. A miscellaneous load was added to the rock crusher dead loads to account for unknowns, such as the framing in the unviewable second level of Building 1.

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

Community Resources - As mentioned, the Hurstville Lime Kiln site is one of only two sites in Iowa where historic lime kilns remain standing. The other site is the Birdsall Lime Kiln in Winneshiek County. This means that the site has great significance not only to the local area, but it is of great importance to Iowa's history. It is thus important to the community that our design maintains the historical aspects of this site to allow future generations to view and understand pieces of Iowa's history.

Additionally, our team has been informed by a local landowner that there may be a Native American burial ground by the nearby water tower. Any significant excavation of the site runs the risk of uncovering historical artifacts related to these grounds. While this could mean a historical discovery, it also has the potential to delay work on the site and incur additional costs.

Personal and Property Rights - A portion of the project that we worked on included the bridge on Bob Garien's property. Both Bob and the Jackson County Historical Society expressed interest in repairing the span of the bridge that is still standing and connecting it to the Hurstville Lime Kiln site so it can be accessible to the public in the future. However, the current path to the bridge is located on both Bob Garien and Jerry Schwenker's properties. To obtain bridge access, the site would require an easement or purchase of property.

Our team did need to scale back the original scope of the project, so there was little design work done for the bridge. However, we did make recommendations to support the bridge for the time being so that a future project could address the goal of connecting the bridge to the site. Legal agreement would be required for any construction work on Bob's property.

Public Safety - The client informed both groups that visitors to the site are climbing into the rock crusher building and the kilns, despite current efforts to prevent visitors from using the site in this way. Specifically, the rock crusher seemed to be a gathering place for visitors who are participating in illegal activities. We evaluated the stability of the rock crusher and developed a plan to increase its current stability and also plan to block current entrances to the building to keep visitors out. Additionally, our design adds grates to the openings in the sides of the kilns as well as a kiln platform that prevents visitors from falling or climbing inside of the kilns. All of these efforts will improve site safety, and thus public safety, as well as minimize risk.

Population Characteristics - The improvements our design makes to the site also have the potential to increase the number of visitors to the site and surrounding area. This can lead to a beneficial economic impact as visitors would likely patronize local businesses.

Section V Alternative Solutions That Were Considered

Many design alternatives were taken into account for this project with three goals in mind of originality, safety, and cost effectiveness. Certain alternatives were omitted due to challenges associated with the project, two of which being time and the goal of originality.

A sheet pile wall was considered to be installed to prevent flooding over a large area of the site, with flood waters reaching six to eight feet multiple times a year. It was sought out that a majority of the flooding was from the creek and river next to the site, but it was discovered that the flooding stemmed from ground water. While on a site visit, sediment deposits on the snow were noticed at the low points of the site which confirmed where the flooding originated from. With this discovery, implementing a sheet pile wall would not prevent excessive flooding of the area and the solution was proper drainage through the site with the installation of a culvert leading into the creek.

On the basis of safety, an alternative of tearing down the rock crusher and replacing it with new lumber was considered but this went against the goal of originality and cost effectiveness. While rebuilding the structure would provide more safety to the area and allow people to see it up close and having detailed photographs of the structure in the past, it was decided that reinforcing critical sections of the structure, such as the deteriorating supports.

Geogrid reinforcement and tiebacks were considered for the reinforcement of the retaining wall. These would maximize strength against the added pressures from the trail, but both shared the same issue. To do any of the two reinforcement ideas would involve excavation in the area which would fall under the archaeological excavation route. This would add a delay to the project's time and therefore increase the costs of the project. Utilizing soil nails would avoid the extra excavation time and by using the existing retaining wall as support for the nails, shotcrete is not needed for the process.

Section VI Final Design Details

Kiln Platforms

The kiln platforms were designed with the goal of allowing visitors to view the inside of the kilns, prevent water from entering the top of the kilns, and preventing visitors from climbing into the kilns. Multiple preliminary layout options were discussed with the client. The clients decided that they preferred the large glass viewport that encompassed the kiln's smokestack and a larger platform that would allow stairs to be attached to the side of the platform and better connect with the trail. This preliminary layout that was selected by the clients for design is pictured below in

Figure 6.01. Additionally, the clients chose from multiple decking options, but ultimately opted for the DuxxBak decking option because of its low maintenance and that it provides better water protection to the kilns. The clients also decided on a metal handrail that would be durable and yet still fit with the aesthetics of the site. It was also decided that the framing members for the kiln platforms would be designed using Douglas Fir – Larch #1 due to the material's strength and history of performance as an outdoor building material.

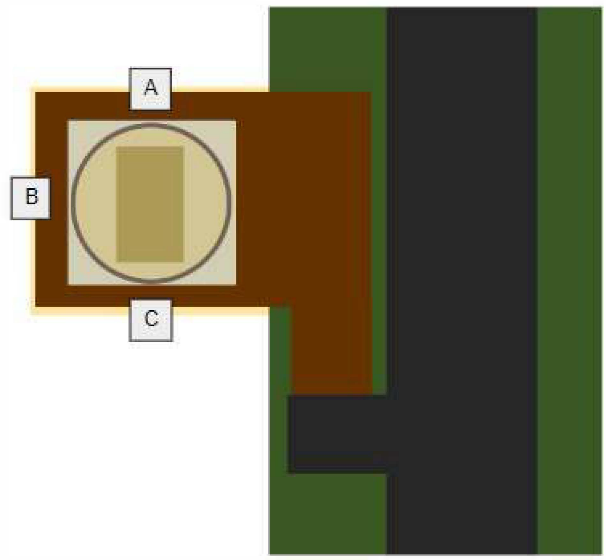

Figure 6.01: Preliminary Platform Layout Selected by Clients

The design loads were determined per the ASCE 7-16 and by using information from specific product sites. The risk category was determined from ASCE 7-16, Chapter C1, Figure C1.5-1 Approximate Relationship between Number of Lives Placed at Risk by a Failure and Occupancy Category. Based on this figure, the risk category was determined as risk category II as there could be multiple people on the platform at the same time, but there would not be greater than 100 persons at risk. The dead load for the joists of 8 psf was determined from ASCE 7-16, Chapter C3, Table C3.1-1a Minimum Design Dead Loads. The decking dead load was determined to be 13 psf per information provided by DuxxBak Composite Decking, the makers of the DuxxBak composite decking specified. The glass for the viewport was selected per the guidance of Glass Flooring Systems Inc. and the design loads and drawings they provided (see appendix 8.4), and was selected to be 1.5" thick and have a dead load of 18 psf. The platform live load was determined to be 60 psf. This was determined from ASCE 7-16, Chapter 4, Table 4.3-1 Minimum Uniformly Distributed Live Loads, L_0, and Minimum Concentrated Live Loads and the Occupancy or Use category of walkways and elevated platforms. The stair live load was determined to be 60 psf as well based on this value. Additionally, the handrail loads were determined per ASCE 7-16, Chapter 4, 4.5 Loads on Handrail, Guardrail, Grab Bar, and Vehicle Barrier Systems, and on Fixed Ladders. This section specifies a distributed load of 50 plf and a concentrated load of 200 lbs for handrails, both applied in any direction, but not simultaneously.

The ground snow load was determined by using the ATC Design Hazards site (See Appendix 8.1) and was determined to be 25 psf. The flat roof snow load, equation (7.3-1) from ASCE 7-16, Chapter 7, 7.3 Flat Roof Snow Loads, p_f, was used to calculate the snow load on the platform. The flat roof snow load was determined to be 21 psf. The wind loads were also designed according to the ASCE 7-16. First, the wind speed was determined using the ATC Hazards site and was determined to be 115 mph. Then steps 1 through 5 given in ASCE 7-16, Chapter 27, 27.2 General Requirements, Table 27.2-1 Steps to Determine MWFRS Wind Loads for Enclosed, Partially Enclosed, and Open Building of All Heights and equations from ASCE 7-16, Chapter 26 were referenced. However, only Components and Cladding (C&C) wind loads were calculated and used for member design because of the platform's size and layout. These loads were calculated using ACSE 7-16, Chapter 30, Part 5. The calculated worst-case C&C load of 86 psf was used as the wind load for the entire platform. It was determined that, due to the location of the site, seismic design would not be considered for this project. All kiln platform design loads and supporting calculations can be found in Appendix 8.1.

All of the wood platform members were designed in accordance with the National Design Specification for Wood Construction, 2015 Edition and the National Design Specification Design Values for Wood Construction, 2015 Edition. All wood members were designed using Allowable Stress Design (ASD) loads. However, reactions at the ends of the beams were calculated both for ASD loads as well as for the individual load types. The ASD loads were used in member design, and the individual loads were used to calculate the Load and Resistance Factor Design (LRFD) load combinations. LRFD design was utilized for the design of the anchor bolts and to evaluate the kiln on top of which the platform columns were placed. The applied load combinations were determined by selecting the maximum load as determined from the ASD load combinations. First, the dimensions of the platform were laid out. These dimensions did change throughout the design process and an additional four columns were added. The final member layout and dimensions are shown below in *Figures 6.02 and 6.03*. The FTool program was used to determine the reaction, shear, and moment forces on the joists and beams. It was also used to calculate the member deflection in the cases where the distributed and point loads varied across the members. Checks for bending, shear, bearing, and deflection were performed on each joist and beam. The joists were determined to be 2x14 DF#1 spaced at 16" on center. This beam depth was used as the governing depth for the other beam members which were all determined to be 2x14, (2) 2x14, or (3) 2x14 DF#1. While not in the design calculations, joist blocking at 6'-0" on center minimum is required for the DuxxBak decking system as per specification from DuxxBak Composite Decking. All of the columns were designed to resist the worst-case individual loads and a maximum point load of 9527.57 lbs, rounded to 9.53 kips, was determined to act on the column. The column was designed as cantilevered. A 6x6 DF#1 column was determined to be the required column size. The stairs were designed similarly to the beams with the same checks. The stair dimensions are based on the IBC 2018 stair standards. (See supporting documentation in Appendix 8.2 for complete calculations.)

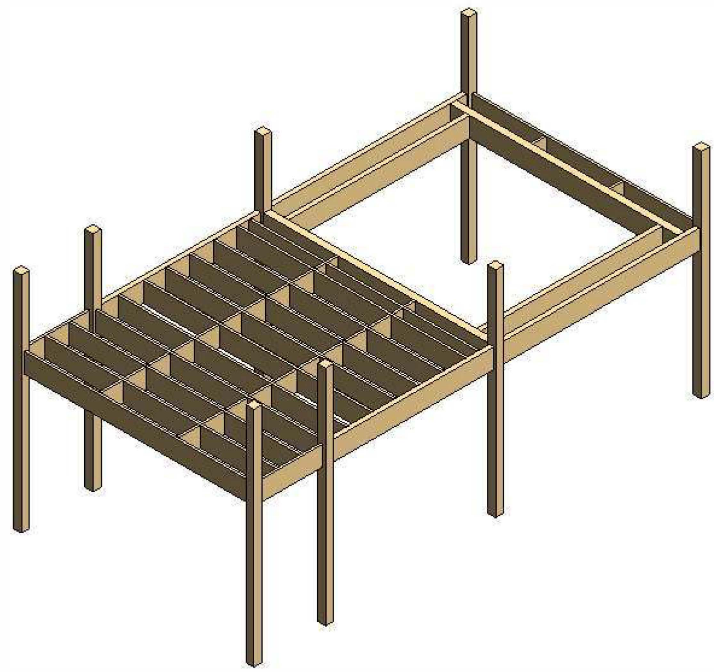

Figure 6.02: Member Layout

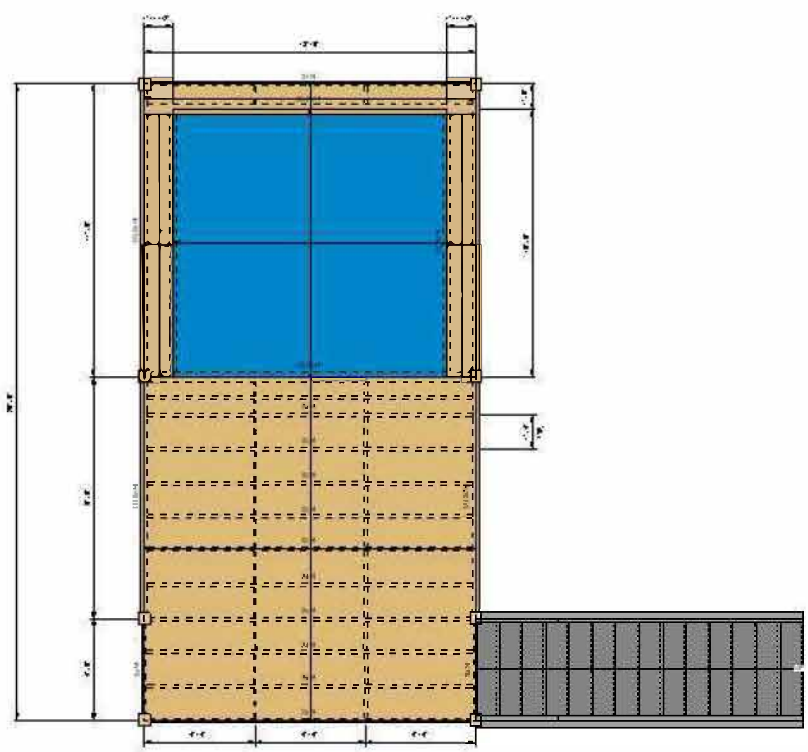

Figure 6.03: Platform and Member Dimensions

The handrails, as stated earlier, must conform to their own set of design loads. The handrails shall be built per the design provided by Thompson Fabricating, LLC for IBC Public Access Handrails (see Appendix 8.3). The OSHA requirements for the toeboard and self-closing gate need not apply.

Connections were selected for each member based on the Simpson Strong-Tie Wood Construction Connectors document and using the maximum reaction forces at the ends of the members. Members of the same size will require the same connector for ease of construction, regardless of the variation in reaction loads. 2x14 members will use LUS210 hangers, (2) 2x14 members will use HUCQ210-2-SDS hangers, and (3) 2x14 members will use HUCQ210-3-SDS hangers. The exception to this rule will be the stair stringers, as they must be connected at an angle. Stair stringers will be connected with an LSSR210-2Z hanger.

The foundation design was based off of the Allowable Soil Bearing Pressure. The IBC 2018, Chapter 18, Table 1806.2 Presumptive Load-Bearing Values was used to determine an allowable bearing pressure value of 1500 psf. Then the maximum uplift and download forces for the columns on the ground, as opposed to the kilns, were determined. The maximum uplift capacity was determined based on the weight of the footing itself and neglected the soil above. The download capacity was determined using the area of the footing and the allowable bearing pressure. The footing shall be placed below the frostline, with the top of the footing at 4'-0" below the soil surface.

It was important to get the correct anchor bolt design so the platform could be safely attached to the kiln. Simpson Strong-Tie Anchor Designer was used to design the anchor bolts. The tensile strength of the anchor, the breakout strength of the base material, and the adhesive strength are what governed the design. The recommended anchor is SET-3G with #4 A706 Gr. 60 Rebar. The base plate is to be 12-in x 12-in with a thickness of ¼". The bore holes shall be spaced 8-in apart with 2-in clear cover on each side of the plate. The anchors must be spaced a minimum of 9-in from the edge of the kiln with an effective embedment depth of 7.5-in. This anchor design should sufficiently resist the shear and moment loads transferred from the columns.

Kiln Structural Analysis

The designed platform creates additional loads on the kiln it rests on. This kiln was analyzed to ensure that the added stresses don't cause it to collapse. Four of the platform columns are supported at the top of the kiln: one at each corner. The column loads were factored using LRFD load combinations (opposed to ASD like the platforms), because there was a higher degree of uncertainty for the kilns. Also, the addition of a platform on the kiln upgrades it to Risk Category II since it increases the occupancy of the structure. Because of this, we had to add another factor of safety to the kiln loads. The strength of the kiln's masonry walls was calculated based on methods presented in *Masonry Structures Behavior and Design* by Robert F. Drysdale and Ahmad A. Hamid.

The columns add mostly vertical loads to the kiln. Five different LRFD load combinations were checked to determine the worst-case scenario. The maximum downward load was about 970 kips total, with the columns accounting for less than 10 kips. This minor increase in weight is not enough to significantly impact the bearing strength of the soil underneath the kiln. Also, the compressive strength of limestone is much greater than that of concrete, and it will be able to handle the additional weight of the platform. When checking for uplift of the platform, we found that there is a possibility that the columns could experience tension. It is important to check how this case affects the kilns because the mortar that binds the limestone together has poor tensile strength. In this case, the possible upwards force from the columns isn't enough to counteract the weight of the smokestack and slab, so the mortar won't experience any tensile stresses

In addition to vertical load, the columns also transfer lateral loads onto the kiln. Each column also acts as a railing for the platform. According to section 4.5 of ASCE 7-16, handrail systems must be able to resist 50 pounds per foot in any direction. Using this rule and the dimensions of the platform, we found the design lateral load and design moment of the columns to be 525 pounds and 6,563 pound*ft, respectively. The walls' shear strength was dependent on the shear strength of the bed joint mortar and the normal stresses on each layer. Since the wall was so heavy, the shear strength of each bed joint was increased. But even if the weight of the wall was neglected, the shear strength of the mortar would be sufficient in resisting the lateral loads from the columns (see Appendix 8.6 for all analysis of the kiln loading).

Finally, the last thing we did with the kilns was specify a plan to repair the openings at the base. Currently, the openings are in poor condition and their grates need to be replaced. The current grates are too weak and have holes that are too large. This makes it too easy for people to pull and mangle the existing grates. We recommend replacing the bent grates with ¾" hole carbon steel mesh (or any similar steel grid). This type of cover will inhibit visitors from reaching into the kiln, but still allows for any rainwater to drain.

Retaining Wall and Soil Nailing Plan

The retaining wall of the limestone kilns has been standing for over a hundred years, but due to the anticipated increase in visitors to the site with the new improvements, along with the additional loading the new additions will bring, analysis was done on the walls to design the best specifications of soil nails for the site.

The first step of the process was to determine the additional soil stresses the new trail and pedestrian live load would impose onto the wall. To do so, the loadings were designed as a strip loading, one positioned 10 feet away from the wall and one against the wall simulating the trail extension to the viewing platform being built on top of a kiln. This was done using the Boussinesq method with steps labelled in *Foundation Design – Principles and Practices* by Donald P. Coduto. The combined stresses were taken and compared to the capacity of the wall, and it was discovered that the system had a factor of safety under 1.5, less than a safe structure needs to be. This supplies the evidence needed for the justification of installing soil nails into the wall.

With cost in mind, soil nail sizes were chosen from the soil nail manufacturer Williams Form Engineering Corp. To find the total length of the soil nail, the nail was split into two sections, one length outside the Rankine failure zone and one length for the boring length that would be filled with cement grout. These equations were pulled from *Principles of Foundation Engineering* by Braja M. Das. A majority of the forces were concentrated near the surface of the existing ground behind the retaining wall so this was labeled as the critical section, where further lengths calculated for the latter soil nails would be based on. It was found that a total required length of soil nail was 26 ft, and since the retaining wall is about a foot in thickness, total length was increased to 27 ft. Certain specifications such as sizing for the subsequent soil nails and spacing was found using the *Soil Nail Walls Reference Manual* from the U.S. Department of Transportation Federal Highway Administration. Such specifications were a minimum of 3.5 ft from the surface is needed for installation with the design being 4 ft from the surface. A range of vertical and horizontal spacing of the nails ranged from 4 ft to 6 ft with a 5 ft typical spacing being chosen. The length of soil nails was also specified with the range being 20 ft to 40 ft with the design length falling at 27 ft, within the threshold. Soil nail diameter will be 1.25 inches and components for the nail will follow the same diameter. A diameter of 7 in is also required for drilling into the wall face to install the soil nails to accommodate the size of the nails and the required cement grouting. Finally, the nails should be installed at a 15-degree angle in relevance to the wall.

Once an appropriate length was chosen for the soil nail and the diameter it needs to be, the components needed for the full assembly were chosen. Corrosion protection was also chosen to help increase the lifespan of the soil nails for years to come. For normal soil nail installations, shotcrete is used on the surface to provide added support for the soil nails as it keeps the soil together, but since work is being done on an existing wall, the soil nails will be supported by the wall itself. With this, a 9 in square hole a half foot in depth should be drilled out on center with each 7 in diameter hole where a 9 in square bearing plate will be placed for the support of the nail. To cover up the openings, decorative star caps are chosen to continue to follow along with the originality of the site while mimicking the star anchors used to stabilize older brick structures (*Figure 6.04)*.

Figure: 6.04: Retaining Wall Star Placement

Rock Crusher Stabilization

The rock crusher building, and the old bridge are in poor condition. To stabilize these structures, both temporary and permanent supports were designed.

The rock crusher building has many critical elements that need to be repaired. Firstly, one of the concrete foundation walls has significant cracking throughout. This wall also retains five feet of soil. A new retaining wall design was designed to go directly in front of the cracked wall. Using ACI codes and standards, we designed the new retaining wall to support all the loads from the retained soil assuming that the existing wall was failing. Our design would be sufficient to resist overturning moment and shear in this section. However, before the retaining wall can be installed, there are two patches of concrete inside the rock crusher that must be removed.

Another issue with the rock crusher is that some of the studs in the wall are bending. All the studs are either 2-in x 10-in or 2-in x 12-in. These need to be straightened out by providing blocking at mid-height. The studs are spaced every 2 feet so a series of 2-in x 10-in x 2-ft members will be enough to straighten the studs and prevent any further buckling.

Since the rock crusher building is so old, there is a lot of rotting in some of the members. Specifically, there are several beams that have lost connection to the columns because of rot. We recommend sistering another beam of the same size to them. Doing this strengthens the member and re-establishes the connection between the rotted beam and the column.

The rock crusher also has many deteriorating columns. These supports carry the roof loads and floor loads of the building, so it is essential that these are repaired. Since the blueprint of the building is uncertain, higher factors were needed when estimating the loads. LRFD load combinations were used to determine the maximum possible load experienced by any given column. The National Design Specification for Wood Construction (NDS 2015) was used to design temporary lumber supports for these columns. The temporary supports consist of two beams and two smaller columns. The beams were designed to be 2-in x 10-in x 3-ft Douglas Fir Strength I members on either side of the building's column. The beams were designed to have adequate shear strength and flexural strength for the maximum possible column load from the rock crusher. The columns for the temporary support are meant to carry the load from the beams to the ground. Each of these column members needed to be designed to have enough compressive strength to carry half of the load from the beams. We found that 4-in x 4-in Douglas Fir Strength I columns have adequate compressive strength to support these loads. Even without lateral bracing, the columns had enough compressive strength to carry the loads from the rock crusher.

Bridge Stabilization

There are many things that need to be done to the bridge before it can be fully restored. Due to the time constraints of this project our group did not address all these issues. We dealt with the repair of the horizontal bracing of the bridge. We have created a general plan for removing and replacing the damaged cross bracing and crooked out-of-plane bracing. These bracings are intended to resist horizontal loads and limit the bridge's lateral movement/turning. They do not support any of the bridge's vertical loads. Therefore, the specified temporary supports for the bridge were designed to resist any lateral loads while the bracing was removed and replaced.

Chapter 29 of ASCE 7-16 was used to find the lateral wind force that could act on the bridge. Using formulas for open frame structures we found a design force of 1.8 kips acting on the bridge. The geometry of the temporary lumber cross bracing shown in *Figure 6.05* is simple, but it is all that is needed to resist the design load.

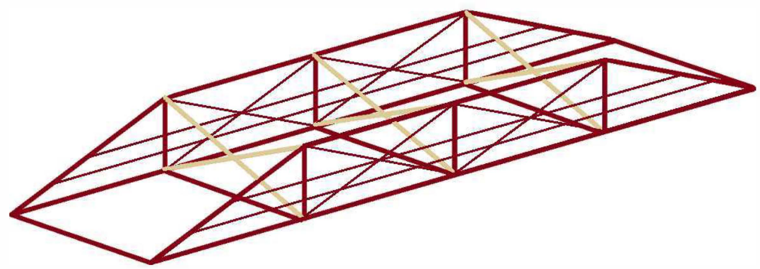

Figure 6.05: Temporary Cross Bracing Plan

Viewing Platforms

Viewing platforms showing an artistic depiction of the original Hurstville Lime Kilns shall be placed along the trail in the direction of the kilns itself to provide an interactive history aspect to the site. General construction of the platforms will consist of two 4x4s with a half inch piece of plexiglass inserted between the two with the rendition lining up with a section of the site showing what it would have looked like while the site was active (*Figure 6.06)*.

The length of the 4x4s are 11 ft with 4 ft below the surface in compliance with general fence construction for having supporting posts being one third to one half the above length underground. The tops of the posts and plexiglass are covered with a 2x4 piece of lumber that is 4 ft in length and connected by 2 16d nails on either side. The plexiglass will be supported by 3 2-inch bolts on either side. The holes should be no less than twice the thickness of the posts where it will be filled in with gravel for support. Loose limestone will be chosen from the site to create a step up to view through the plexiglass with the dimensions being 4 ft in line with the plexiglass and 3 ft in depth with having no more than a foot in height. Installation of the limestone will be level with the rest of the assembly and the existing ground.

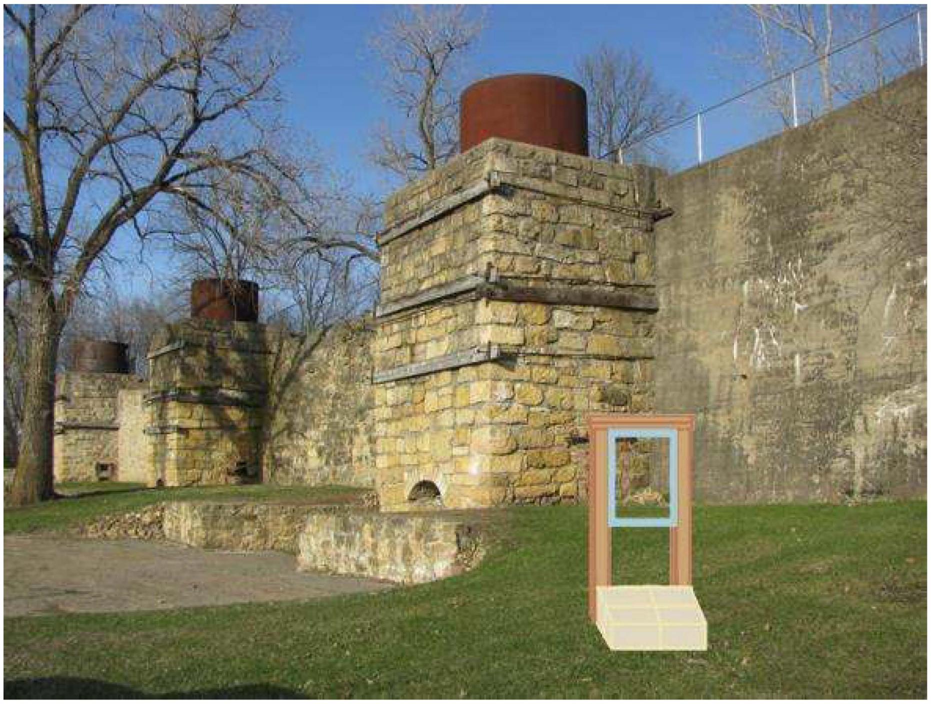

Figure 6.06: Viewing Platform Positioning

Section VII Engineer's Cost Estimate

The estimated final cost for the structural part of this project is $119,500. This includes the cost of construction plus 20% and 10% for contingencies and engineering fees respectively. The costs of the project are split up into different sections of the project: rock crusher and bridge stabilization, grates, kiln platforms, soil nailing, and glass etching platforms.

We determined the overall material cost of the project by determining the unit costs of materials and the costs of labor. We used RSMeans and local hardware store prices to determine the unit costs of materials.

	Material Cost Estimate				
	Item	Unit	Unit Price	Quantity	Cost
Rock Crusher and Bridge Stabilizatoin	Concrete	CY	$150.00	3.5	$525.00
	Concrete removal	SY	$40.00	4	$160.00
	# 4 Rebar	LF	$2.55	200	$510.00
	8x8 DF#1	BF	$12.00	110	$1,320.00
	2x10 DF#1	BF	$12.00	60	$720.00
	4x4 DF#1	BF	$12.00	60	$720.00
	2x12 Pine	BF	$8.00	12	$96.00
	2x4 Pine	BF	$8.00	80	$640.00
	Stuctural Steel	CWT	$360.00	30	$10,800.00
Grates / Side Openings	Clay Brick Masonry	SF	$25.00	50	$1,250.00
	Welded Wire 2" Mesh (bundle)	EA	$150.00	1	$150.00
Kiln Platforms	6x6 DF#1	BF	$12.00	384	$4,608.00
	2x14 DF#1	BF	$12.00	1100	$13,200.00
	Decking	SF	$5,347.00	303.75	$5,320.47
	Glass Panel (10-6"x10-6")	CT	$4,540.00	1	$4,540.00
	3.5'x3.5'x24"	CY	$125.00	0.908	$113.50
	#5 Rebar	LF	$22.48	210	$236.04
	Handrails	LF	$80.00	104.5	$8,360.00
	2x14 hangers (LUS210)	CT	$1.84	64	$117.76
	(2) 2x14 hangers (HUCQ210-2-SDS)	CT	$42.21	12	$506.52
	(3) 2x14 hangers (HUCQ210-3-SDS)	CT	$46.60	4	$186.40
	Stair hangers (LSSR210-2Z)	CT	$27.39	3	$82.17
	Anchor rebar (4 at each)	LF	$0.63	25	$15.85
	Set 3G epoxy (sold in 8.5 OZ)	OZ	$24.62	33.28	$98.48
	A36 Base Plates (1/2x12x12)	CT	$85.00	10	$850.00
	A36 Knife Plate (1/2x8x5.5)	CT	$44.57	10	$445.70
	A307 1/2"x7" Hex Bolt	CT	$2.12	60	$127.20
	A3071/2" Hex Nut and Washer	CT	$0.67	60	$40.20
	Welds	LF	$70.00	10	$700.00
	Miscellaneous Construction	-	-	-	$9,280.50
Soil Nailing	Grade 75 #10 Nail @ 27 ft	EA	$400.00	56	$22,400.00
	Star Anchor Plates	EA	$15.00	56	$840.00
Glass Etching Platforms	4x4 Douglas Fir posts @ 12ft	CT	$31.00	8	$248.00
	2x4 Framing Lumber @ 4ft	CT	$5.09	4	$20.36
	3-1/2" 16D Box Nail - 5 lb. Box	CT	$19.38	1	$19.38
	3/4 x 4 x 8 Plywood Sheathing	CT	$56.38	1	$56.38
	1/2" Clear Acrylic Plexiglass Sheet	CT	$600.00	4	$2,400.00
	5/16" -18 x 2" Zince Grade 2 Hex Bolt	CT	$1.09	24	$26.16
	5/16" -18 Blue Zinc Grade 2 Hex Nut	CT	$2.99	24	$71.76
				TOTAL:	**$91,900.00**

Figure 7.01 – Material Cost Estimate

Construction Subtotal	$91,900.00
10% Contingencies	$9,200.00
20% Engineering and Administration	$18,400.00
Total Project Cost	**$119,500.00**

Figure 7.02 – Total Project Cost

Section VIII Appendices

Please see the attached documents for further information.

8.01 Kiln Platform Design Loads

8.02 Kiln Platform Design

8.03 Kiln Platform Handrails

8.04 Kiln Platform Glass Viewport

8.05 Kiln Platform Anchor Bolt Design

8.06 Kiln Analysis Loading

8.07 Kiln Analysis Shear Checks

8.08 Kiln Analysis Settlement Checks

8.09 Kiln Opening Repair

8.10 Retaining Wall Analysis

8.11 Retaining Wall Soil Nails

8.12 Rock Crusher Design Loads

8.13 Rock Crusher Supports

8.14 Bridge Stabilization Plan

Section IX Bibliography

American Society of Civil Engineers., (2017). *Minimum design loads and associated criteria for buildings and other structures: ASCE/SEI 7-16*.

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Das, Braja M. Principles of Foundation Engineering. Pacific Grove, CA: Thomson/Brooks/Cole, 2004

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Simpson Strong-Tie., (2019). *Wood Construction Connectors Catalog 2019.* Pleasanton, CA.

Hazards by Location

Search Information

Address: Hurstville Historic District, IA 52060, USA

Coordinates: 42.09751869999999, -90.6831911

Elevation: 668 ft

Timestamp: 2022-02-25T16:19:44.032Z

Hazard Type: Snow

ASCE 7-16

Ground Snow Load 25 lb/sqft

ASCE 7-10

Ground Snow Load 25 lb/sqft

ASCE 7-05

Ground Snow Load 25 lb/sqft

The results indicated here DO NOT reflect any state or local amendments to the values or any delineation lines made during the building code adoption process. Users should confirm any output obtained from this tool with the local Authority Having Jurisdiction before proceeding with design.

Disclaimer

Hazard loads are interpolated from data provided in ASCE 7 and rounded up to the nearest whole integer.

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ATC Hazards by Location

Search Information

Address: Hurstville Historic District, IA 52060, USA

Coordinates: 42.09751869999999, -90.6831911

Elevation: 668 ft

Timestamp: 2022-02-25T16:18:54.073Z

Hazard Type: Wind

ASCE 7-16

MRI 10-Year	74 mph
MRI 25-Year	81 mph
MRI 50-Year	86 mph
MRI 100-Year	93 mph
Risk Category I	101 mph
Risk Category II	108 mph
Risk Category III	115 mph
Risk Category IV	120 mph

ASCE 7-10

MRI 10-Year	76 mph
MRI 25-Year	84 mph
MRI 50-Year	90 mph
MRI 100-Year	96 mph
Risk Category I	105 mph
Risk Category II	115 mph
Risk Category III-IV	120 mph

ASCE 7-05

ASCE 7-05 Wind Speed	90 mph

The results indicated here DO NOT reflect any state or local amendments to the values or any delineation lines made during the building code adoption process. Users should confirm any output obtained from this tool with the local Authority Having Jurisdiction before proceeding with design.

Disclaimer

Hazard loads are interpolated from data provided in ASCE 7 and rounded up to the nearest whole integer. Per ASCE 7, islands and coastal areas outside the last contour should use the last wind speed contour of the coastal area – in some cases, this website will extrapolate past the last wind speed contour and therefore, provide a wind speed that is slightly higher. NOTE: For queries near wind-borne debris region boundaries, the resulting determination is sensitive to rounding which may affect whether or not it is considered to be within a wind-borne debris region.

Mountainous terrain, gorges, ocean promontories, and special wind regions shall be examined for unusual wind conditions.

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	Hurstville Lime Kilns	Designer	Date
		SRO	02/22

Kiln Platform: Design Loads

Miscellaneous Info

Design Guide	=	ASCE 7-16		
Risk Category	=	II		

Dead Loads

Decking DL	=	13	psf	*assume DuxxBak Decking
Viewport DL	=	18	psf	*assume 1 1/2" thick
Joists	=	8	psf	*assume 2x12 at 16" o.c.

Live Loads

Platform LL	=	60	psf	*elevated platforms
Stair LL	=	60	psf	*stairs and exit ways
Handrails/Guardrails LL**	=	50	plf	
Handrails/Guardrails LL**	=	200	lb	*single concentrated load

**Handrail and guardrail systems shall be designed to resist a single concentrated load of 200 pounds applied in any direction. Handrail and giardrail systems shall also be designed to resist 50 lb/ft applied in any direction along the handrail. These loads need not be assumed to be concurrent.

Snow Loads

SL, ground	=	25	psf
Surface Roughness	=	C	
Exposure	=	Partially Exposed	
Ce	=	1	
Thermal Condition	=	Open air	
Ct	=	1.2	
Is	=	1	
SL, flat roof	=	0.7*Ce*Ct*Is*SL, ground	
SL, flat roof	=	21	psf

Kiln Platform: Design Loads cont.

Wind Loads - MWFRS

*ASCE 7-16 Ch. 27.3-4, wind speed from ASCE 7-10

Risk Category	=	II	
Basic Wind Speed, V	=	115	mph
Directionality factor, K_d	=	0.85	
Exposure category	=	C	
Topographic factor, K_{zt}	=	1	
Elevation above sea level	=	668	ft
Ground elevation factor, K_e	=	1	
Gust effect factor, G	=	0.85	
Enclosure classification	=	Open	
building height, h	=	41	
K_z	=	1.05	
Gcpi	=	0	
Velocity pressure, q_z	=	31	psf
Wind flow	=	Clear	
Roof angle	=	0	degrees

Table 1: Net Pressure Coefficients, C_N

Load Case	C_{NW}	C_{NL}
A	1.2	0.3
B	-1.1	-0.1

Table 2: Wind Pressure, p (psf)

Load Case	C_{NW}	C_{NL}
A	32	8
B	-28	-2

Kiln Platform: Design Loads cont.

Wind Loads - C&C
*ASCE 7-16 Ch. 30, part 5

Velocity pressure, q_z = 31 psf
Gust effect factor, G = 0.85
Wind flow = Clear
Roof angle = 0 degrees

Table 1: Net Pressure Coefficients, C_N

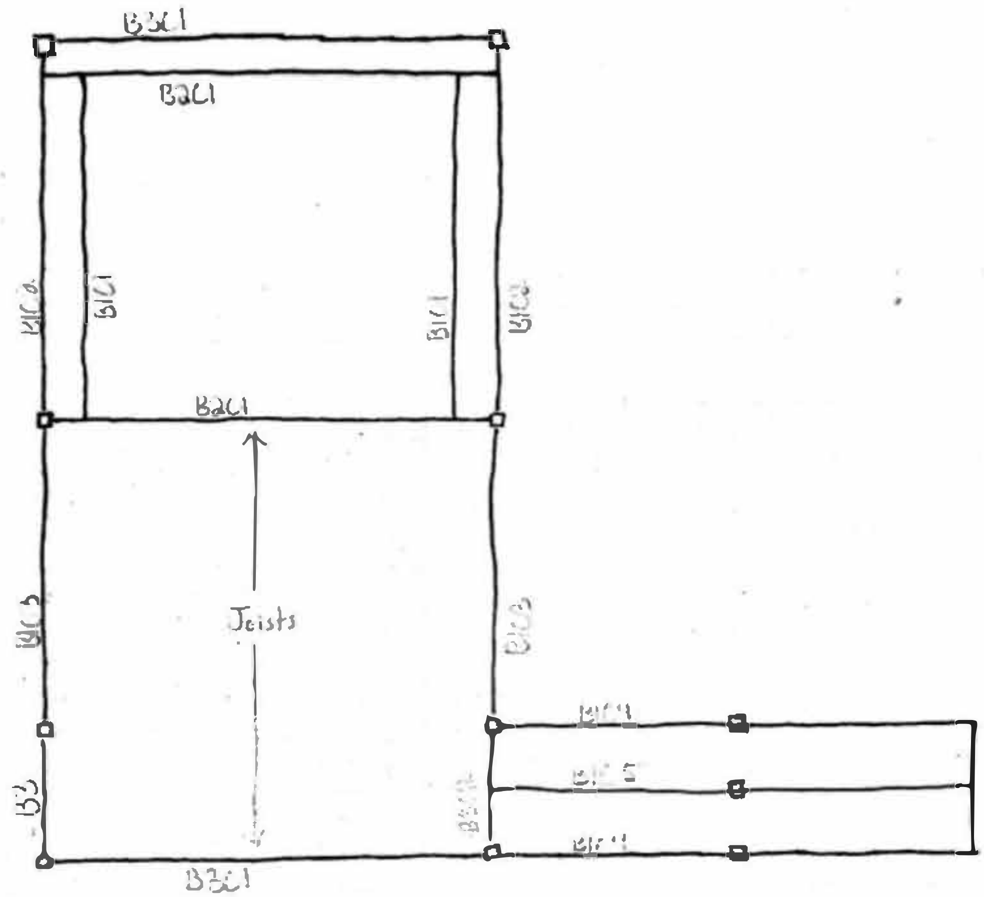
Effective Wind Area	Zone 3 +	Zone 3 -	Zone 2 +	Zone 2 -	Zone 1 +	Zone 1 -
$\leq a^2$	2.4	-3.3	1.8	-1.7	1.2	-1.1
$>a^2, \leq 4.0a^2$	1.8	-1.7	1.8	-1.7	1.2	-1.1
$>4.0a^2$	1.2	-1.1	1.2	-1.1	1.2	-1.1

Table 2: Wind Pressure, p (psf)

Effective Wind Area	Zone 3 +	Zone 3 -	Zone 2 +	Zone 2 -	Zone 1 +	Zone 1 -
$\leq a^2$	64	-86	48	-44	32	-28
$>a^2, \leq 4.0a^2$	48	-44	48	-44	32	-28
$>4.0a^2$	32	-28	32	-28	32	-28

		Designer	Date
	Hurstville Lime Kilns	SRO	03/22

Beam Key Plan



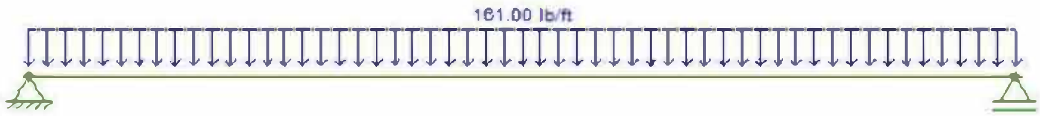
Kiln Platform: Joists

Spacing	=	1.34	ft o.c.
Max Length	=	13	ft
DL	=	21	psf
LL	=	60	psf
SL	=	21	psf
WL	=	86	psf
Distributed Load			
DL	=	28.14	plf
LL	=	80.4	plf
SL	=	28.14	plf
WL	=	115.24	plf
Applied Distributed Load	=	161	plf

	Hurstville Lime Kilns	Designer	Date
		SRO	03/22

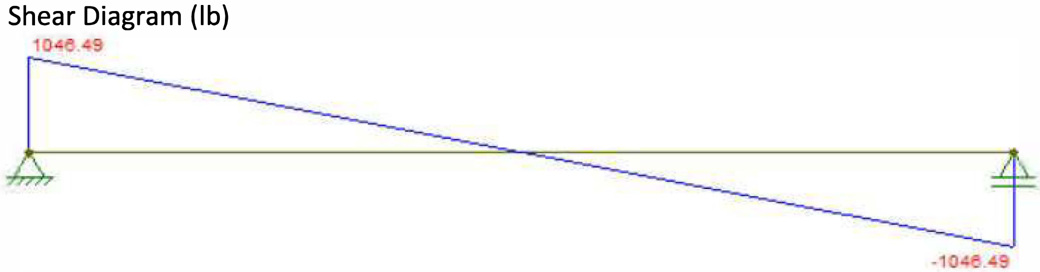
Kiln Platform: Joists

Loading Diagram

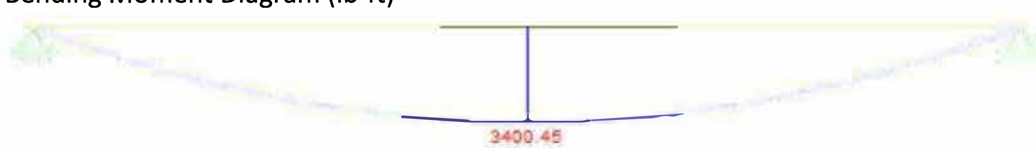


Reaction Diagram (lb)

Shear Diagram (lb)



Bending Moment Diagram (lb-ft)



Kiln Platform: Joists

M	=	3400.45	lb-ft
	=	40806	lb-in
Fb	=	1000	psi
CD	=	1.60	
CM	=	1.00	
Ct	=	1.00	
CL	=	1.00	
Cf	=	1.00	
Cfu	=	1.00	
Ci	=	1.00	
Cr	=	1.15	
Fb'	=	1840	psi
Sx	>	M/Fb	
43.89	>	22.18	
Beam selection	=	2x14 DF #1	
M/Sx fb	=	929.73	< 1840 psi

	Hurstville Lime Kilns	Designer	Date
		SRO	03/22

Kiln Platform: Joists

Shear Check

V	=	1046.49	lbs	
d	=	13.25	in	
d	=	1.5	in	
fv	=	78.98	psi	< 288
Fv	=	180	psi	
Fv'	=	288	psi	

Bearing Check

Fc(perp)	=	625	psi	
Cb	=	1.25		
Fc(perp)'	=	781.25	psi	
fc (perp)	=	465	psi	< 781.25
lb	=	1.5	inches	

Deflection Check

DL Deflection limit	=	l/360	per IBC 2018	
DL Deflection limit	=	0.433	inches	
DL +LL Deflection Limit	=	l/240	per IBC 2018	
DL +LL Deflection Limit	=	0.650	inches	
E	=	620000	psi	
I	=	290.8	in^4	
dst	=	0.101	in	< 0.433
dlt	=	0.287	in	
Δ DL + LL	=	0.54	in	< 0.650
Kcr	=	1.5		

Use = 2x14 DF #1 with Simpson LUS210 hanger

		Designer	Date
	Hurstville Lime Kilns	SRO	02/22

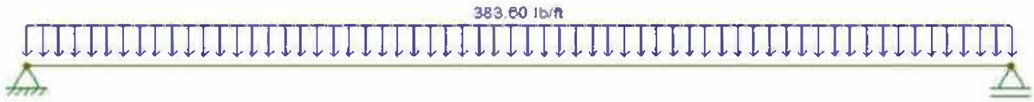
Kiln Platform: B1C1

Max Length	=	10.5	ft
Glass Viewport Loading			
DL	=	18	psf
LL	=	60	psf
SL	=	21	psf
WL	=	86	psf
Trib	=	2.625	ft
Decking Loading			
DL	=	21	psf
LL	=	60	psf
SL	=	21	psf
WL	=	86	psf
Trib	=	0.625	ft
Combined Glass and Decking Load			
DL	=	60.38	plf
LL	=	195	plf
SL	=	68.25	plf
WL	=	279.5	plf
Applied Distributed Load	=	383.6	plf
Reactions at Ends			
DL	=	316.97	lbs
LL	=	1023.75	lbs
SL	=	358.32	lbs
WL	=	1467.38	lbs

	Hurstville Lime Kilns	Designer	Date
		SRO	03/22

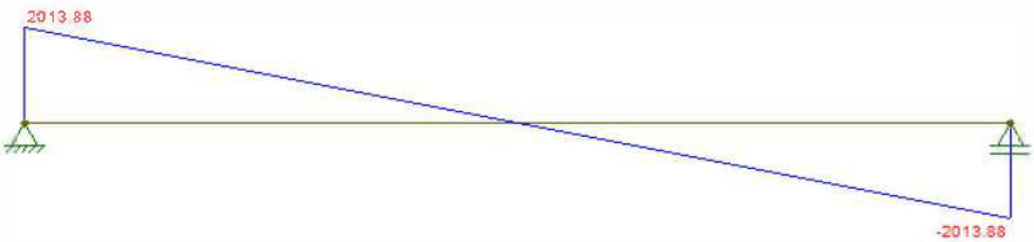
Kiln Platform: B1C1

Loading Diagram



Reaction Diagram (lb)

Shear Diagram (lb)



Bending Moment Diagram (lb-ft)

		Designer	Date
	Hurstville Lime Kilns	SRO	03/22

Kiln Platform: B1C1

	M	=	5285.45	lb-ft
		=	63426	lb-in
	Fb	=	1000	psi
	CD	=	1.60	
	CM	=	1.00	
	Ct	=	1.00	
	CL	=	1.00	
	Cf	=	1.00	
	Cfu	=	1.00	
	Ci	=	1.00	
	Cr	=	1.00	
	Fb'	=	1600	psi
	Sx	>	M/Fb	
	87.78	>	39.64	
	Beam selection	=	(2) 2x14 DF #1	
M/Sx	fb	=	722.56	< 1600 psi

	Hurstville Lime Kilns	Designer	Date
		SRO	03/22

Kiln Platform: B1C1

Shear Check

V	=	2013.88	lbs	
d	=	13.25	in	
d	=	3	in	
fv	=	76.00	psi	< 288
Fv	=	180	psi	
Fv'	=	288	psi	

Bearing Check

Fc(perp)	=	625	psi	
Cb	=	1.25		
Fc(perp)'	=	781.25	psi	
fc (perp)	=	448	psi	< 781.25
lb	=	1.5	inches	

Deflection Check

DL Deflection limit	=	l/360	per IBC 2018	
DL Deflection limit	=	0.350	inches	
DL+LL Deflection Limit	=	l/240	per IBC 2018	
DL+LL Deflection Limit	=	0.525	inches	
E	=	620000	psi	
I	=	581.6	in^4	
dst	=	0.0458	in	< 0.350
dlt	=	0.1479	in	
Δ DL+LL	=	0.268	in	< 0.525
Kcr	=	1.5		

Use = (2) 2x14 DF #1 with Simpson LUS214-2 hanger

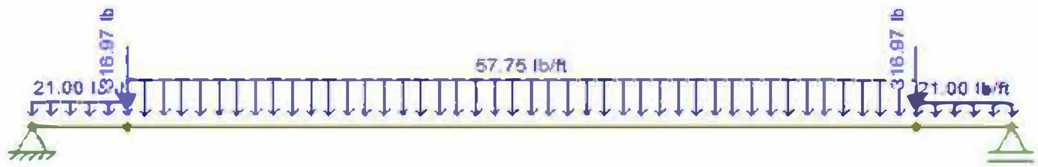
Kiln Platform: B2C1

Max Length	=	13	ft			
Glass Viewport Loading	from 1.25' to 12.75'					
DL	=	18	psf	=	47.25	plf
LL	=	60	psf	=	157.5	plf
SL	=	21	psf	=	55.13	plf
WL	=	86	psf	=	225.75	plf
Glass Trib	=	2.625	ft			
Decking Loading	from 2.25' to 12.75'					
DL	=	21	psf	=	10.5	plf
LL	=	60	psf	=	30	plf
SL	=	21	psf	=	10.5	plf
WL	=	86	psf	=	43	plf
Glass Trib	=	0.5	ft			
Decking Loading	from 0' to 2.25' and 12.75' to 13'					
DL	=	21	psf	=	21	plf
LL	=	60	psf	=	60	plf
SL	=	21	psf	=	21	plf
WL	=	86	psf	=	86	plf
Glass Trib	=	1	ft			
B2C2 Point load	at 2.25' and 12.75'					
DL	=	316.97	lbs			
LL	=	1023.75	lbs			
SL	=	358.32	lbs			
WL	=	1467.38	lbs			

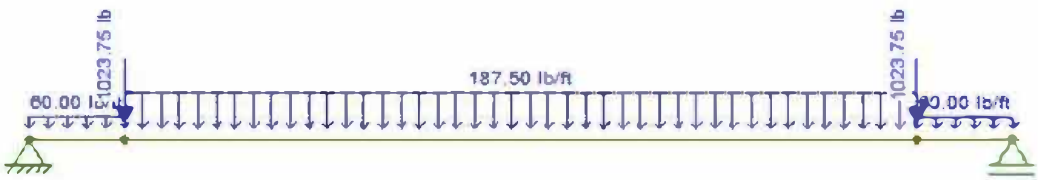
		Designer	Date
	Hurstville Lime Kilns	SRO	02/22

Kiln Platform: B2C1

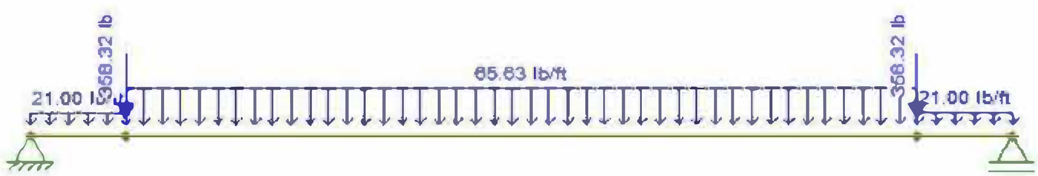
Dead Load



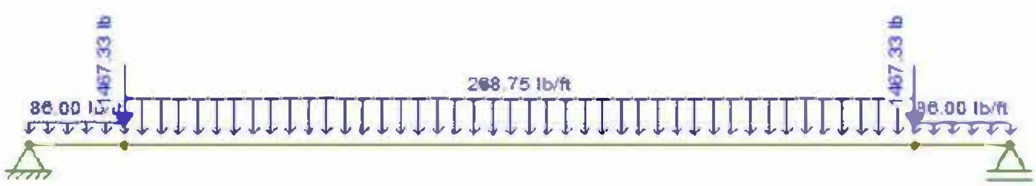
Live Load



Snow Load



Wind Load



Reactions at ends

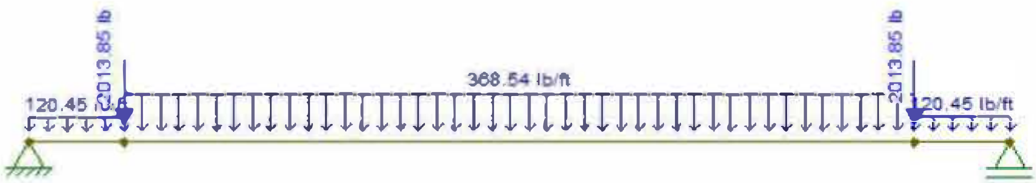
DL	=	646.4	lbs
LL	=	2083.11	lbs
SL	=	729.12	lbs
WL	=	2985.75	lbs

		Designer	Date
	Hurstville Lime Kilns	SRO	03/22

Kiln Platform: B2C1

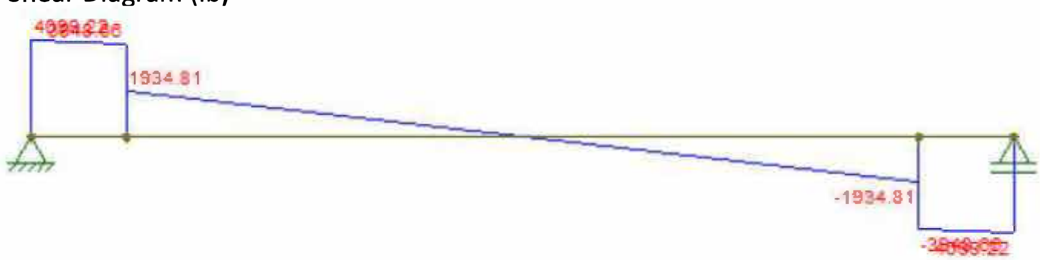
Total Applied Loads			
from 1.25' to 11.75'	=	368.54	plf
from 0' to 1.25' and 12.75' to 13'	=	120.45	plf
at 1.25' and 11.75'	=	2013.85	lbs

Loading Diagram

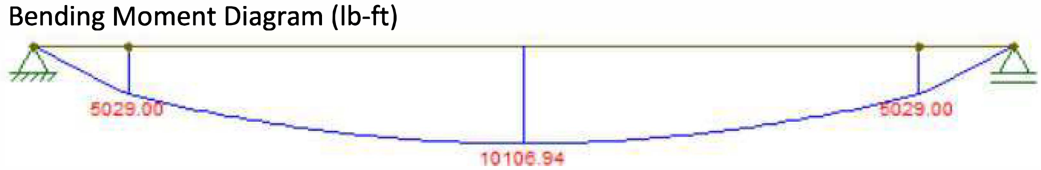


Reaction Diagram (lb)

Shear Diagram (lb)



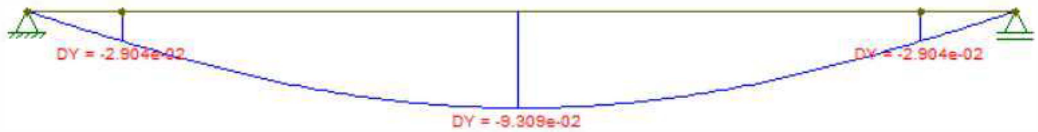
Bending Moment Diagram (lb-ft)



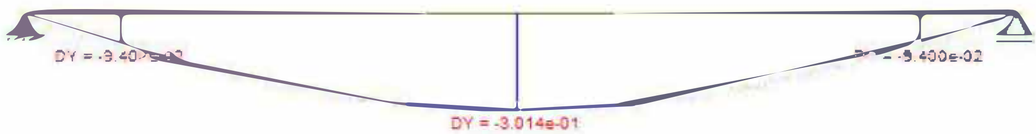
		Designer	Date
	Hurstville Lime Kilns	SRO	03/22

Kiln Platform: B2C1

DL Deflection (in)



LL Deflection (in)



Kiln Platform: B2C1

M	=	10106.94	lb-ft
	=	121284	lb-in
Fb	=	1000	psi
CD	=	1.60	
CM	=	1.00	
Ct	=	1.00	
CL	=	1.00	
Cf	=	1.00	
Cfu	=	1.00	
Ci	=	1.00	
Cr	=	1.00	
Fb'	=	1600	psi
Sx	>	M/Fb	
131.67	>	75.80	
Beam selection	=	(3) 2x14 DF #1	
M/Sx fb	=	921.12	< 1600 psi

	Hurstville Lime Kilns	Designer	Date
		SRO	03/22

Kiln Platform: B2C1

Shear Check

V	=	4099.22	lbs	
d	=	13.25	in	
b	=	4.5	in	
fv	=	103.13	psi	< 288
Fv	=	180	psi	
Fv'	=	288	psi	

Bearing Check

Fc(perp)	=	625	psi	
Cb	=	1.1875		
Fc(perp)'	=	742.19	psi	
fc (perp)	=	455	psi	< 742.19
lb	=	2	inches	

Deflection Check

DL Deflection limit	=	l/360	per IBC 2018	
DL Deflection limit	=	0.433	inches	
DL +LL Deflection Limit	=	l/240	per IBC 2018	
DL +LL Deflection Limit	=	0.650	inches	
E	=	620000	psi	
I	=	872.4	in^4	
dst	=	0.09309	in	< 0.433
dlt	=	0.3014	in	
Δ DL + LL	=	0.55	in	< 0.650
Kcr	=	1.5		

Use = (3) 2x14 DF #1 with Simpson HUCQ210-3-SDS Hanger

		Designer	Date
	Hurstville Lime Kilns	SRO	02/22

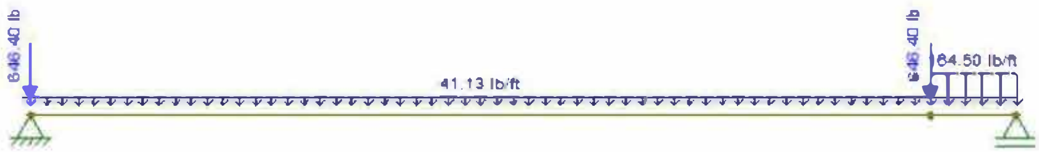
Kiln Platform: B1C2

Max Length	=	11.5	ft			
Decking Loads from 0' to 10.5'						
DL	=	21	psf	=	13.13	plf
LL	=	60	psf	=	37.5	plf
SL	=	21	psf	=	13.13	plf
WL	=	86	psf	=	53.75	plf
Trib	=	0.625	ft			
Decking Loads from 10.5' to 11.5'						
DL	=	21	psf	=	136.5	plf
LL	=	60	psf	=	390	plf
SL	=	21	psf	=	136.5	plf
WL	=	86	psf	=	559	plf
Trib	=	6.5	ft			
Handrail						
DL	=	28	plf			
LL	=	50	plf			
SL	=	0	plf			
WL	=	0	plf			
B2C1 Point Loads at 0' and 10.5'						
DL	=	646.4	lbs			
LL	=	2083.11	lbs			
SL	=	729.12	lbs			
WL	=	2985.75	lbs			

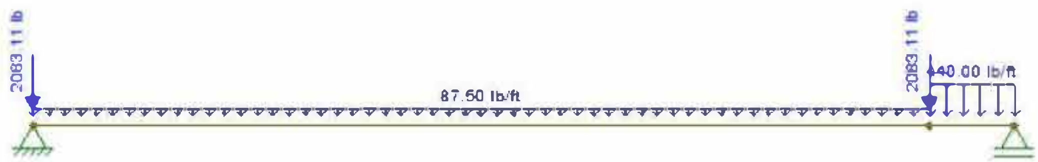
	Hurstville Lime Kilns	Designer	Date
		SRO	02/22

Kiln Platform: B1C2

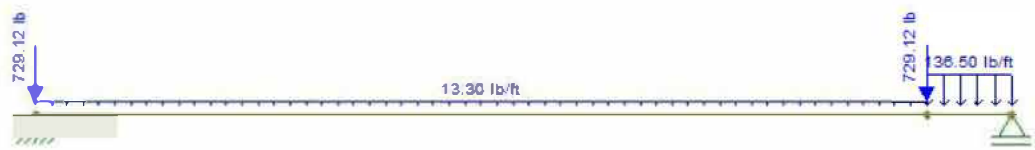
Dead Load



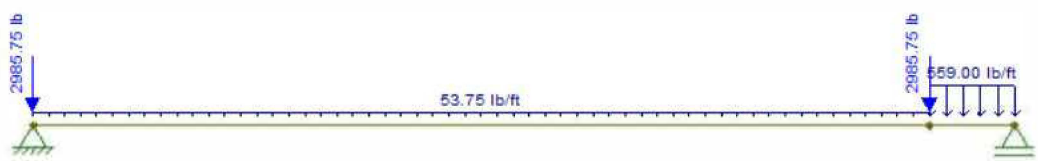
Live Load



Snow Load



Wind Load



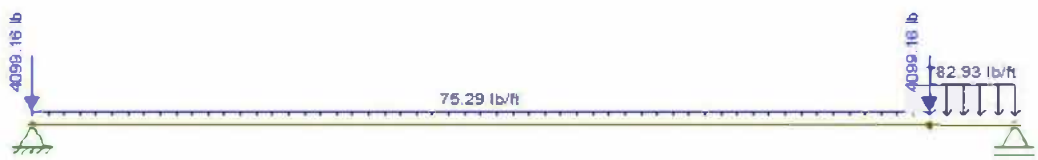
Reactions at C3 and C6			Reactions at C4 and C5		
DL =	944.47	lbs	DL =	944.69	lbs
LL =	2782.69	lbs	LL =	2742.26	lbs
SL =	874.35	lbs	SL =	860.03	lbs
WL =	3578.41	lbs	WL =	3518.46	lbs

		Designer	Date
	Hurstville Lime Kilns	SRO	03/22

Kiln Platform: B1C2

Total Applied Loads			
from 0' to 10.5'	=	75.29	plf
from 10.5' to 11.5'	=	782.93	plf
at 0' and 10.5'	=	4099.16	lbs

Loading Diagram



Reaction Diagram (lb)

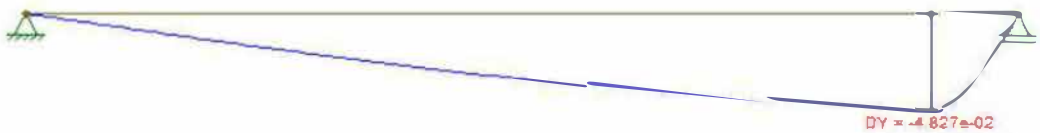
Shear Diagram (lb)

Bending Moment Diagram (lb-ft)

	Hurstville Lime Kilns	Designer	Date
		SRO	03/22

Kiln Platform: B1C2

DL Deflection (in)



LL Deflection (in)

		Designer	Date
	Hurstville Lime Kilns	SRO	03/22

Kiln Platform: B1C2

M	=	4460.21	lb-ft
	=	53523	lb-in
Fb	=	1000	psi
CD	=	1.60	
CM	=	1.00	
Ct	=	1.00	
CL	=	1.00	
Cf	=	1.00	
Cfu	=	1.00	
Ci	=	1.00	
Cr	=	1.00	
Fb'	=	1600	psi
Sx	>	M/Fb	
87.78	>	33.45	
Beam selection	=	(2) 2x14 DF #1	
M/Sx fb	=	609.74	< 1600 psi

		Designer	Date
	Hurstville Lime Kilns	SRO	03/22

Kiln Platform: B1C2

Shear Check

V	=	4852.49	lbs	
d	=	13.25	in	
b	=	3	in	
fv	=	183.11	psi	< 288
Fv	=	180	psi	
Fv'	=	288	psi	

Bearing Check

Fc(perp)	=	625	psi	
Cb	=	1.125		
Fc(perp)'	=	703.13	psi	
fc (perp)	=	539	psi	< 703.13
lb	=	3	inches	

Deflection Check

DL Deflection limit	=	l/360	per IBC 2018	
DL Deflection limit	=	0.383	inches	
DL+LL Deflection limit	=	l/240	per IBC 2018	
DL+LL Deflection limit	=	0.575	inches	
E	=	620000	psi	
I	=	581.6	in^4	
dst	=	0.01664	in	< 0.383
dlt	=	0.04827	in	
Δ total	=	0.09	in	< 0.575
Kcr	=	1.5		

Use = (2) 2x14 DF #1 with Simpson HUCQ210-2-SDS

	Hurstville Lime Kilns	Designer	Date
		SRO	02/22

Kiln Platform: B1C3

Max Length	=	9.5	ft			
Decking Loads from 0' to 9.5'						
DL	=	21	psf	=	136.5	plf
LL	=	60	psf	=	390	plf
SL	=	21	psf	=	136.5	plf
WL	=	86	psf	=	559	plf
Trib	=	6.5	ft			
Handrail						
DL	=	28	plf			
LL	=	50	plf			
SL	=	0	plf			
WL	=	0	plf			

		Designer	Date
	Hurstville Lime Kilns	SRO	02/22

Kiln Platform: B1C3

Dead Load

Live Load

Snow Load

Wind Load

Reactions

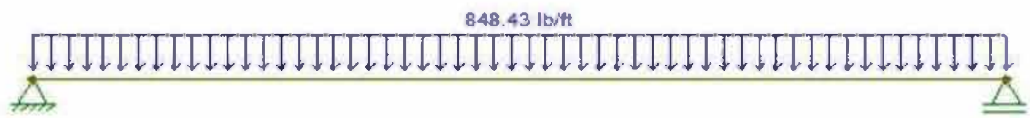
DL	=	781.375	lbs
LL	=	2090	lbs
SL	=	648.375	lbs
WL	=	2655.25	lbs

		Designer	Date
	Hurstville Lime Kilns	SRO	03/22

Kiln Platform: B1C3

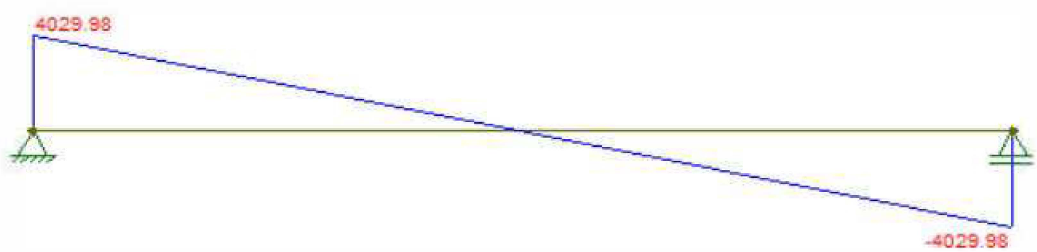
Total Applied Loads
from 0' to 9.5' = 848.43 plf

Loading Diagram

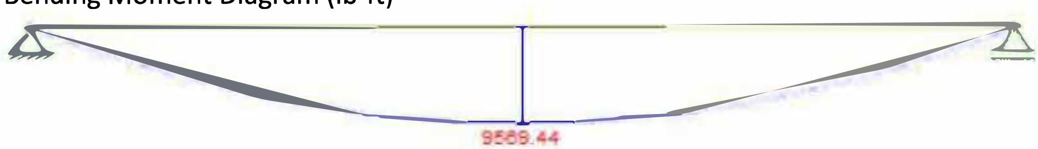


Reaction Diagram (lb)

Shear Diagram (lb)



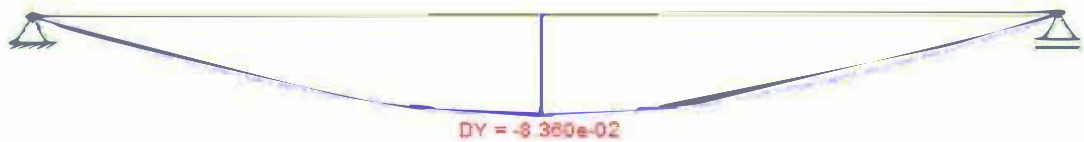
Bending Moment Diagram (lb-ft)



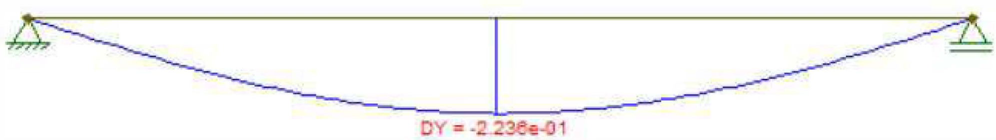
	Hurstville Lime Kilns	Designer	Date
		SRO	03/22

Kiln Platform: B1C3

DL Deflection (in)



LL Deflection (in)



		Designer	Date
	Hurstville Lime Kilns	SRO	03/22

Kiln Platform: B1C3

M	=	9569.44	lb-ft
	=	114834	lb-in
Fb	=	1000	psi
CD	=	1.60	
CM	=	1.00	
Ct	=	1.00	
CL	=	1.00	
Cf	=	1.00	
Cfu	=	1.00	
Ci	=	1.00	
Cr	=	1.00	
Fb'	=	1600	psi
Sx	>	M/Fb	
87.78	>	71.77	
Beam selection	=	(2) 2x14 DF #1	
M/Sx fb	=	1308.20	< 1600 psi

Kiln Platform: B1C3

Shear Check

V	=	7104.21	lbs	
d	=	13.25	in	
b	=	3	in	
fv	=	268.08	psi	< 288
Fv	=	180	psi	
Fv'	=	288	psi	

Bearing Check

Fc(perp)	=	625	psi	
Cb	=	1.09375		
Fc(perp)'	=	683.6	psi	
fc (perp)	=	592	psi	< 683.6
lb	=	4	inches	

Deflection Check

DL Deflection limit	=	l/360	per IBC 2018	
DL Deflection limit	=	0.317	inches	
DL+LL Deflection limit	=	l/240	per IBC 2018	
DL+LL Deflection limit	=	0.475	inches	
E	=	620000	psi	
I	=	581.6	in^4	
dst	=	0.00836	in	< 0.317
dlt	=	0.02236	in	
Δ total	=	0.05	in	< 0.475
Kcr	=	1.5		

Use = (2) 2x14 DF #1 with Simpson HUCQ210-2-SDS

Kiln Platform: B3C2

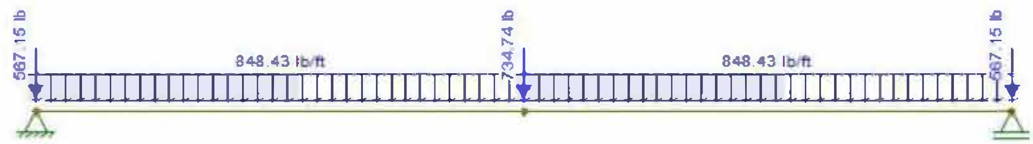
Max Length	=	4	ft				
Decking Loads	from 0' to 4'			Decking + Handrail			
DL	=	21	psf		=	164.5	plf
LL	=	60	psf		=	440	plf
SL	=	21	psf		=	136.5	plf
WL	=	86	psf		=	559	plf
Trib	=	6.5	ft				
Handrail							
DL	=	28	plf				
LL	=	50	plf				
SL	=	0	plf				
WL	=	0	plf				
Outer Stair Stringer Loads	at 0'-0" and 4'-0"						
DL	=	149.45	lbs				
LL	=	335.5	lbs				
SL	=	64.05	lbs				
WL	=	262.3	lbs				
Total	=	567.15	lbs				
Center Stair Stringer Load	at 2'-0"						
DL	=	128.1	lbs				
LL	=	365.99	lbs				
SL	=	128.1	lbs				
WL	=	524.59	lbs				
Total	=	734.74	lbs				
Reactions							
DL	=	393.05	lbs				
LL	=	1063	lbs				
SL	=	337.05	lbs				
WL	=	1380.3	lbs				

	Hurstville Lime Kilns	Designer	Date
		SRO	03/22

Kiln Platform: B3C2

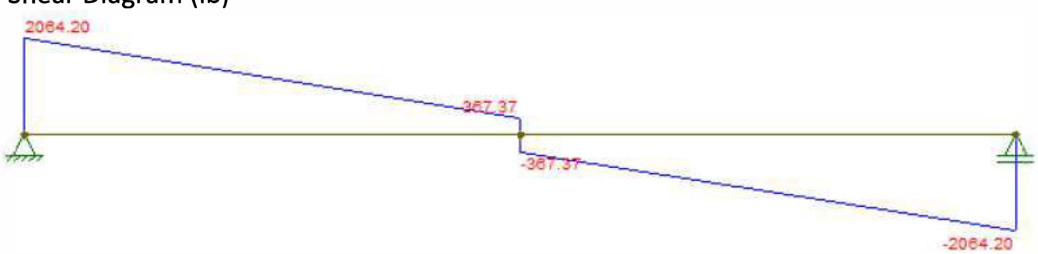
Total Applied Loads			
from 0' to 4'	=	848.43	plf
pt loads at 0' and 4'	=	567.15	lbs
pt load at 2'	=	734.74	lbs

Loading Diagram

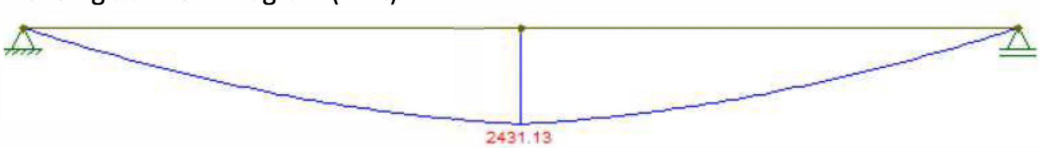


Reaction Diagram (lb)

Shear Diagram (lb)



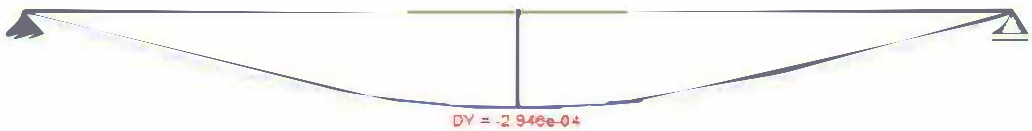
Bending Moment Diagram (lb-ft)



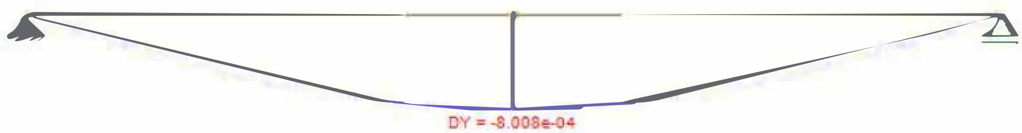
	Hurstville Lime Kilns	Designer	Date
		SRO	03/22

Kiln Platform: B3C2

DL Deflection (in)



LL Deflection (in)



	Hurstville Lime Kilns	Designer	Date
		SRO	03/22

Kiln Platform: B3C2

	M	=	2431.13	lb-ft	
		=	29174	lb-in	
	Fb	=	1000	psi	
	CD	=	1.60		
	CM	=	1.00		
	Ct	=	1.00		
	CL	=	1.00		
	Cf	=	1.00		
	Cfu	=	1.00		
	Ci	=	1.00		
	Cr	=	1.00		
	Fb'	=	1600	psi	
	Sx	>	M/Fb		does not work with (4) 2x14
	43.89	>	18.23		
	Beam selection	=	(1) 2x14 DF #1		
M/Sx	fb	=	664.71	< 1600 psi	

	Hurstville Lime Kilns	Designer	Date
		SRO	03/22

Kiln Platform: B3C2

Shear Check

V	=	2064.2	lbs	
d	=	13.25	in	
b	=	1.5	in	
fv	=	155.79	psi	< 288
Fv	=	180	psi	
Fv'	=	288	psi	

Bearing Check

Fc(perp)	=	625	psi	
Cb	=	1.09375		
Fc(perp)'	=	683.6	psi	
fc (perp)	=	344	psi	< 683.6
lb	=	4	inches	

Deflection Check

DL Deflection limit	=	l/360	per IBC 2018	
DL Deflection limit	=	0.133	inches	
DL +LL Deflection Limit	=	l/240	per IBC 2018	
DL +LL Deflection Limit	=	0.200	inches	
E	=	620000	psi	
I	=	290.8	in^4	
dst	=	0.000295	in	< 0.133
dlt	=	0.000801	in	
Δ total	=	0.01	in	< 0.200
Kcr	=	1.5		

Use = 2x14 DF #1 with Simpson HU214

		Designer	
	Hurstville Lime Kilns	SRO	02/22

Kiln Platform: B3C1

Length	=	13	ft			
Decking Loads				Decking + Handrail Loads		
DL	=	21	psf	=	42.07	plf
LL	=	60	psf	=	90.2	plf
SL	=	21	psf	=	14.07	plf
WL	=	86	psf	=	57.62	plf
Trib	=	0.67	ft			
Reactions at End						
DL	=	273.46	lbs			
LL	=	586.3	lbs			
SL	=	91.46	lbs			
WL	=	374.53	lbs			
Total	=	950.32	lbs			

Use = 2x12 DF #1 with Simpson LUC210Z hanger
(ok by inspection)

Kiln Platform: Platform Columns

Max Height	=	12	ft
Max Axial Load	at 8.5'		
DL	=	1725.85	lbs
LL	=	4872.69	lbs
SL	=	1522.73	lbs
WL	=	6233.66	lbs
Total	=	9327.57	lbs
Axial Load 2	at 12'		
DL	=	0	lbs
LL	=	200	lbs
SL	=	0	lbs
WL	=	0	lbs
Total	=	200	lbs
Total Axial Load	=	9527.57	lbs
	=	9.53	kips

	Hurstville Lime Kilns	Designer	Date
		SRO	02/22

Kiln Platform: Platform Columns

Use: 6x6 DF #1 Column

Height	=	12	ft
d	=	5.5	inches
b	=	5.5	inches

Reference Design Values

Fc	=	1500	psi
E,min	=	620000	psi

Adjustment Factors

C,D	=	1.6	
C,M	=	1	
C,t	=	1	
C,F	=	1	for Fc
C,i	=	1	
C,P	=	see below	

Find F,cE

le/d	=	52.37	
Kx	=	2.5	
F,cE	=	465	psi

Calc Pc

Fc*	=	2400	psi	
F,cE/Fc*	=	0.194		
c	=	0.8		
C,P	=	0.190		
F,c'	=	456.0	psi	
Pc	=	13.794	kips	> 9.53 kips

Use = 6x6 DF#1 with Simpson ABU88Z

		Designer	Date
	Hurstville Lime Kilns	SRO	02/22

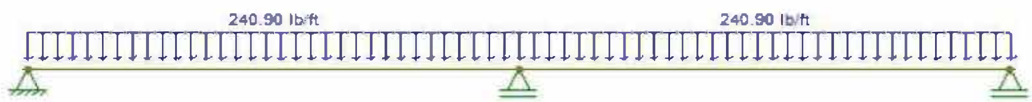
Kiln Platform: Center Stair Stringer (B1C5)

Total Length	=	16.25	ft
Span 1	=	8.125	ft
Span 2	=	8.125	ft
Stair Loads			
DL	=	21	psf
LL	=	60	psf
SL	=	21	psf
WL	=	86	psf
Trib	=	2	ft
Distributed Loads			
DL	=	42	plf
LL	=	120	plf
SL	=	42	plf
WL	=	172	plf
Total Distributed Load	=	240.9	plf

	Hurstville Lime Kilns	Designer	Date
		SRO	03/22

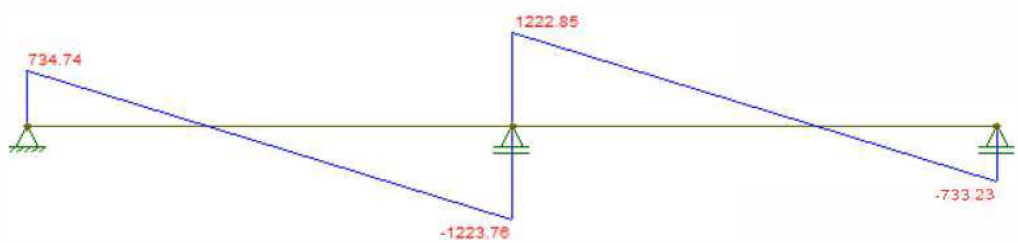
Kiln Platform: Center Stair Stringer (B1C5)

Loading Diagram

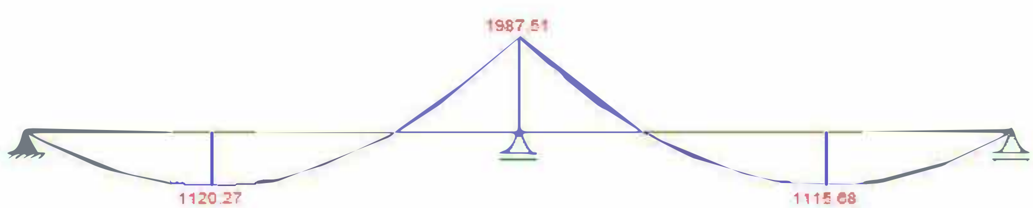


Reaction Diagram (lb)

Shear Diagram (lb)



Bending Moment Diagram (lb-ft)



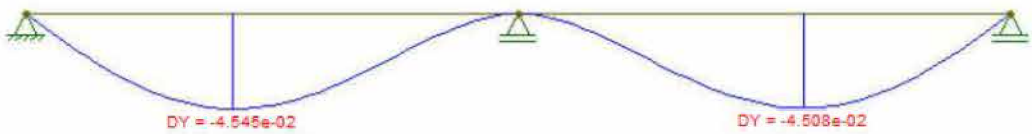
Reaction at ends

DL	=	128.1	lbs
LL	=	365.99	lbs
SL	=	128.1	lbs
WL	=	524.59	lbs

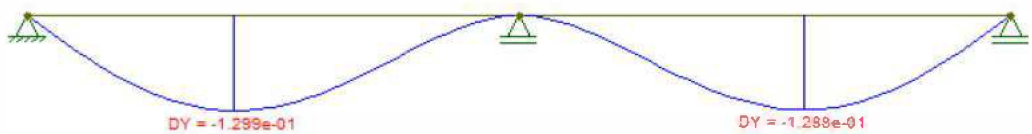
	Hurstville Lime Kilns	Designer	Date
		SRO	03/22

Kiln Platform: Center Stair Stringer (B1C5)

DL Deflection (in)



LL Deflection (in)



	Hurstville Lime Kilns	Designer	Date
		SRO	03/22

Kiln Platform: Center Stair Stringer (B1C5)

M	=	1987.51	lb-ft
	=	23851	lb-in
Fb	=	1000	psi
CD	=	1.60	
CM	=	1.00	
Ct	=	1.00	
CL	=	1.00	
Cf	=	1.00	
Cfu	=	1.00	
Ci	=	1.00	
Cr	=	1.15	
Fb'	=	1840	psi
Sx	=	19.53	
Sx	>	M/Fb	
39.07	>	12.96	

Beam selection = (2) 2x14 DF #1 notched for stairs

M/Sx fb = 610.47 < 1840 psi

	Hurstville Lime Kilns	Designer	Date
		SRO	03/22

Kiln Platform: Center Stair Stringer (B1C5)

Shear Check

V	=	611.88	lbs	
d	=	6.25	in	(2x14 notched 0'-7")
b	=	3	in	
fv	=	48.95	psi	< 288
Fv	=	180	psi	
Fv'	=	288	psi	

Bearing Check

Fc(perp)	=	625	psi	
Cb	=	1.25		
Fc(perp)'	=	781.25	psi	
fc (perp)	=	272	psi	< 781.25
lb	=	1.5	inches	

Deflection Check

DL Deflection limit	=	l/360	per IBC 2018	
DL Deflection limit	=	0.542	inches	
DL +LL Deflection Limit	=	l/240	per IBC 2018	
DL +LL Deflection Limit	=	0.813	inches	
E	=	620000	psi	
I	=	122.08	in^4	
dst	=	0.04508	in	< 0.542
dlt	=	0.1288	in	
Δ DL + LL	=	0.24	in	< 0.813
Kcr	=	1.5		

Use = (2) 2x14 DF #1 notched for stairs
with Simpson LSSR210-2Z

		Designer	Date
	Hurstville Lime Kilns	SRO	02/22

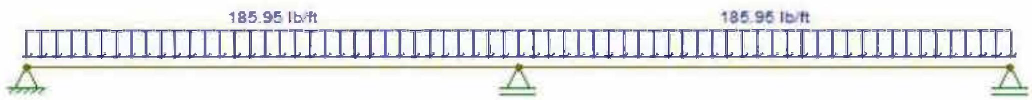
Kiln Platform: Outer Stair Stringers (B1C4)

Total Length	=	16.25	ft
Span 1	=	8.125	ft
Span 2	=	8.125	ft
Stair Loads			
DL	=	21	psf
LL	=	60	psf
SL	=	21	psf
WL	=	86	psf
Trib	=	1	ft
Handrail Loads			
DL	=	28	plf
LL	=	50	plf
SL	=	0	plf
WL	=	0	plf
Distributed Loads			
DL	=	49	plf
LL	=	110	plf
SL	=	21	plf
WL	=	86	plf
Total Distributed Load	=	185.95	plf

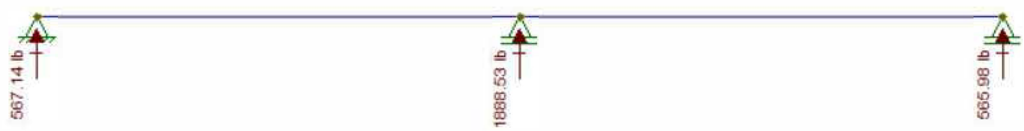
	Hurstville Lime Kilns	Designer	Date
		SRO	03/22

Kiln Platform: Outer Stair Stringers (B1C4)

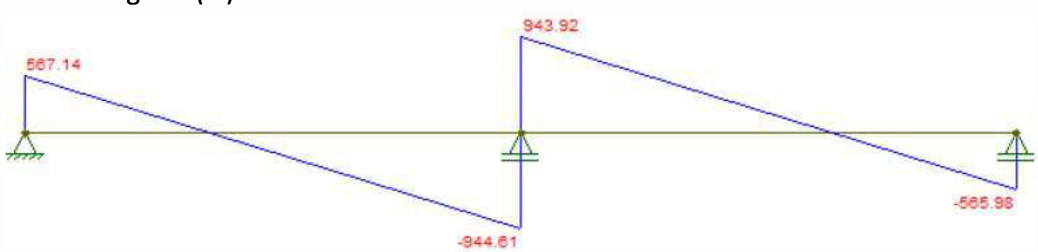
Loading Diagram



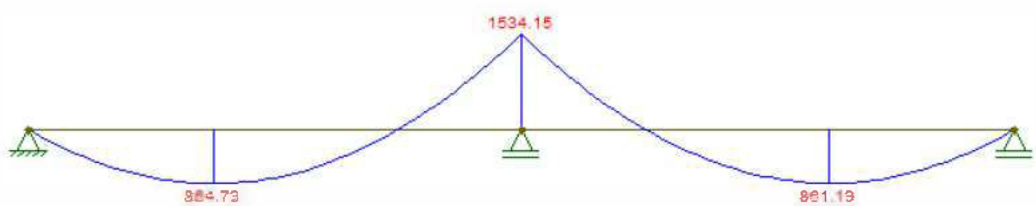
Reaction Diagram (lb)



Shear Diagram (lb)



Bending Moment Diagram (lb-ft)



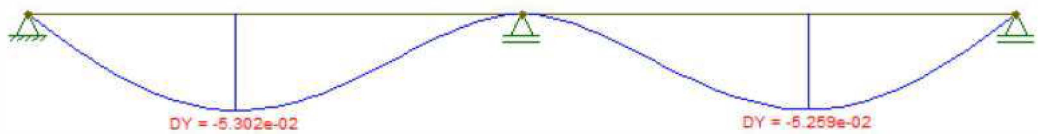
Reaction at ends

DL	=	149.45	lbs
LL	=	335.5	lbs
SL	=	64.05	lbs
WL	=	262.3	lbs

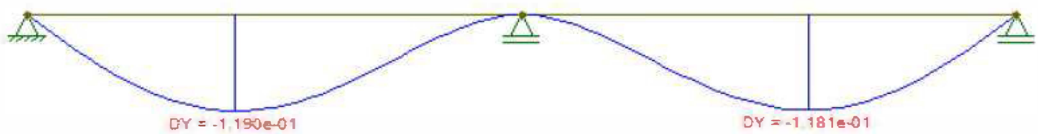
	Hurstville Lime Kilns	Designer	Date
		SRO	03/22

Kiln Platform: Outer Stair Stringers (B1C4)

DL Deflection (in)



LL Deflection (in)



		Designer	Date
	Hurstville Lime Kilns	SRO	03/22

Kiln Platform: Outer Stair Stringers (B1C4)

M	=	1534.15	lb-ft
	=	18410	lb-in
Fb	=	1000	psi
CD	=	1.60	
CM	=	1.00	
Ct	=	1.00	
CL	=	1.00	
Cf	=	1.00	
Cfu	=	1.00	
Ci	=	1.00	
Cr	=	1.15	
Fb'	=	1840	psi
Sx	=	19.53	
Sx	>	M/Fb	
39.07	>	10.01	

Beam selection = (2) 2x14 DF #1 notched for stairs

M/Sx fb = 471.21 < 1840 psi

	Hurstville Lime Kilns	Designer	Date
		SRO	03/22

Kiln Platform: Outer Stair Stringers (B1C4)

Shear Check

V	=	944.61	lbs	
d	=	6.25	in	(2x14 notched 0'-7")
b	=	3	in	
fv	=	75.57	psi	< 288
Fv	=	180	psi	
Fv'	=	288	psi	

Bearing Check

Fc(perp)	=	625	psi	
Cb	=	1.25		
Fc(perp)'	=	781.25	psi	
fc (perp)	=	420	psi	< 781.25
lb	=	1.5	inches	

Deflection Check

DL Deflection limit	=	l/360	per IBC 2018	
DL Deflection limit	=	0.542	inches	
DL +LL Deflection Limit	=	l/240	per IBC 2018	
DL +LL Deflection Limit	=	0.813	inches	
E	=	620000	psi	
I	=	122.08	in^4	
dst	=	0.05302	in	< 0.542
dlt	=	0.119	in	
Δ DL + LL	=	0.24	in	< 0.813
Kcr	=	1.5		

Use = (2) 2x14 DF #1 notched for stairs
with Simpson LSSR210-2Z

	Hurstville Lime Kilns	Designer	Date
		SRO	03/22

Kiln Platform: Off-Kiln Footing

ASBP	=	1.5	ksf	
Max Loads				
DL	=	1323.88	lbs	
LL	=	3488.5	lbs	
SL	=	1049.48	lbs	
WL	=	4297.85	lbs	
Max Uplift	=	-1.7843	k	
Max Download	=	6.6614	lbs	
Length	=	3.5	ft	
Width	=	3.5	ft	
Depth	=	2	ft	
Max Uplift Allowed	=	2.131	k	> 1.7843 k
Max Download Allowed	=	18.375	k	> 6.6614 k
Total Rebar Required	=	6	#5 Rebar	
Rebar at top and bottom	=	3	#5 Rebar	

Use = 3'-6" x 3'-6" x 2'-0" footing with (3) #5 rebar, top and bottom both ways

Thompson Fabricating LLC
An Industry Leader
Thompson Fabricating, LLC
1411 Commerce Place
PO Box 170160
Tarrant, AL 35217
205-841-0441

Design Specifications for Public-Access Guardrail

(International Building Code)

1. Guardrails and Handrails shall be the product of a company normally engaged in the manufacture of pipe railing. Railings shall be shop assembled in lengths not to exceed 24 feet for field erection.

2. The handrail shall be made of pipes joined together with component fittings. Samples of all components, bases, toeboard and pipe must be submitted for approval at the request of the engineer. Components that are pop-riveted or glued at the joints will not be acceptable. All components must be mechanically fastened with stainless steel hardware. **Handrail and components shall be TUFRAIL, as manufactured by Thompson Fabricating, LLC (Birmingham, Alabama) or an approved equal.**

3. Railings shall be 1 1/2" Schedule 40 aluminum pipe alloy 6105-T5, ASTM-B-429 or ASTM-B-221. Post shall be 1 1/2" Schedule 80 aluminum pipe of the same alloy. Post spacing shall be a maximum of 6'-0”.

4. Guardrails and Handrails shall be designed to withstand a 200lb concentrated load applied in any direction and at any point on the top rail. Guardrails and Handrails shall also be designed to withstand a uniform load of 50 lb/ft applied horizontally to the top rail. Uniform loads are not to be applied simultaneously with the concentrated loads.

5. Pickets and intermediate railings shall be provided such that a 4-inch diameter sphere cannot pass through any opening up to a height of 34 inches. From a height of 34 inches to 42 inches above the adjacent walking surface, a sphere 8 inches in diameter shall not pass. The triangular openings formed by the riser, tread and bottom rail at the open side of a stairway shall be of a maximum size such that a sphere of 6 inches in diameter cannot pass through the opening.

6. Pickets and intermediate railings shall be designed to withstand a horizontally applied normal load of 50lb on an area not to exceed one square foot including openings and spaces between rails.

7. The manufacturer shall submit calculations for approval at the request of the Engineer. Testing of base castings or base extrusions by an independent lab or manufacturer's lab (if manufacturer's lab meets the requirements of the Aluminum Association) will be an acceptable substitute for calculations. Calculations will be required for approval of all other design aspects.

8. Posts shall not interrupt the continuation of the top rail at any point along the railing, including corners and end terminations (OSHA 1910.23). The top surface of the top railing shall be smooth and shall not be interrupted by projected fittings.

Thompson Fabricating, LLC
1411 Commerce Place
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Design Specifications for Public-Access Guardrail (page2)

(International Building Code)

9. The mid-rail at a corner return shall be able to withstand a 200lb load without loosening. The manufacturer is to determine this dimension for their system and provide physical tests from a laboratory to confirm compliance.

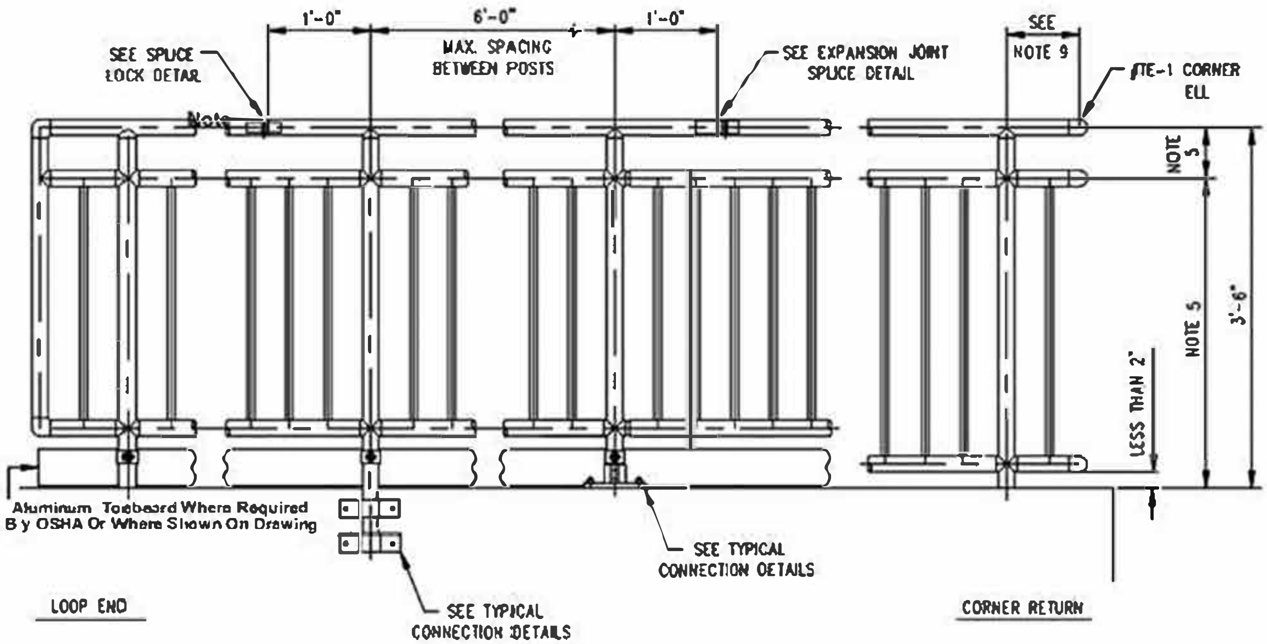
10. Concrete anchors shall be stainless steel type 303 or 304 wedge anchors and shall be furnished by the handrail manufacturer. The anchor design shall include the appropriate reduction factors for spacing and edge distances in accordance with the manufacturers published data.

11. Toeboard shall conform to OSHA standards. Toeboard shall be a minimum of 4" high and shall be an extrusion that attaches to the posts with clamps that will allow for expansion and contraction between posts. Toeboards shall be set 1/4" above the walking surface. Toeboards shall be provided on handrails as required by OSHA and/or as shown on drawings. Toeboards shall be shipped in stock lengths for field installation.

12. A self-closing gate shall guard openings in the railing (OSHA 1910.23). Safety chains shall not be used unless specifically shown on the drawings.

13. Finish shall be Aluminum Association M10-C22-A41 (215-R1). The pipe shall be plastic-wrapped. The plastic wrap is to be removed after erection.

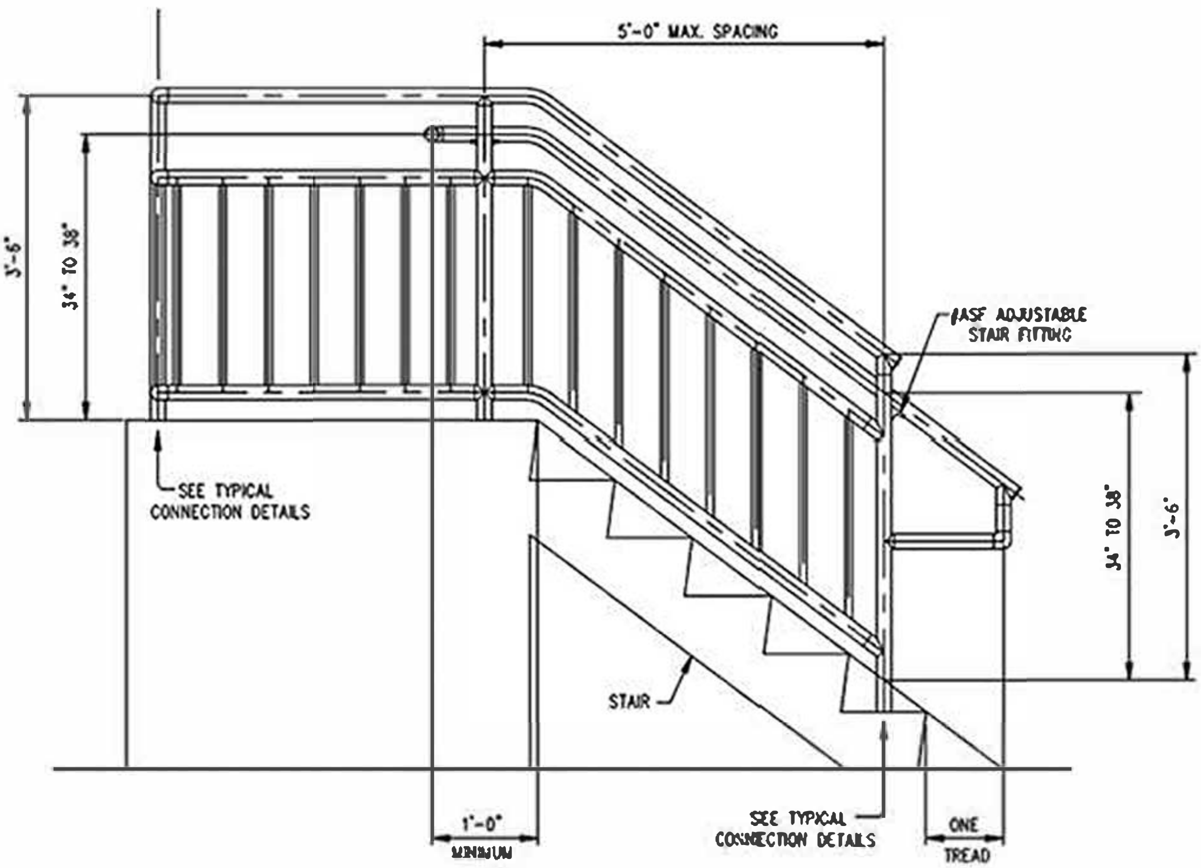
14. Aluminum surfaces in contact with concrete, grout or dissimilar metals will be protected with a coat of bituminous paint, Mylar isolators or other approved material.



TYPICAL TYPE I GUARDRAIL

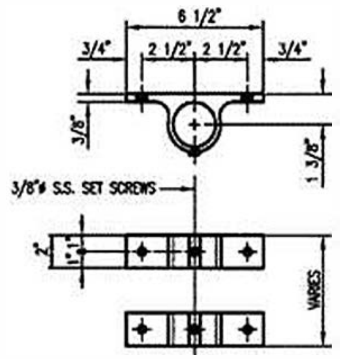
GUARDRAILS SHALL BE TOP-MOUNTED OR SIDE-MOUNTED, AS SHOWN ON PLANS

Note 5. Pickets and intermediate railings shall be provided such that a 4-inch diameter sphere cannot pass through any opening up to a height of 34 inches. From a height of 34 inches to 42 inches above the adjacent walking surface, a sphere 8 inches in diameter shall not pass. The triangular openings formed by the riser, tread and bottom rail at the open side of a stairway shall be of a size such that a sphere of 6 inches in diameter cannot pass through the opening.


5'-0" MAX. SPACING
3'-6"
34" TO 38"
BASE ADJUSTABLE
STAIR FITTING
SEE TYPICAL
CONNECTION DETAILS
34" TO 38"
3'-6"
STAIR
1'-0"
MINIMUM
SEE TYPICAL
CONNECTION DETAILS
ONE
TREAD

#TSM-1.50 Side-Mt. Bracket (2 Piece)

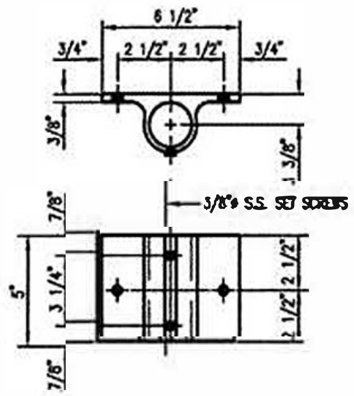
– SIDE MOUNT TO CONCRETE –



#TSM-1.50 Side-Mt. Bracket (1 Piece)

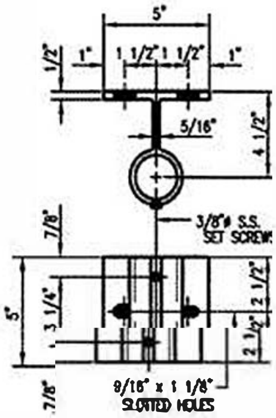
–SIDE MOUNT TO CONCRETE SLAB–

–SIDE MOUNT TO CHANNEL–



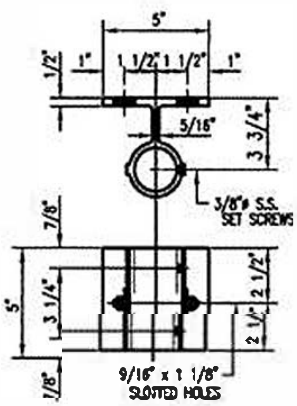
#SMB-2 Side-Mt. Bracket

–SIDE MOUNT TO BEAM OR CHANNEL–



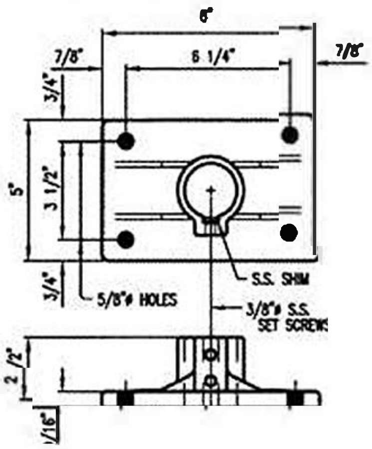
#SMB-3 Side-Mt. Bracket

–SIDE MOUNT TO BEAM OR CHANNEL–



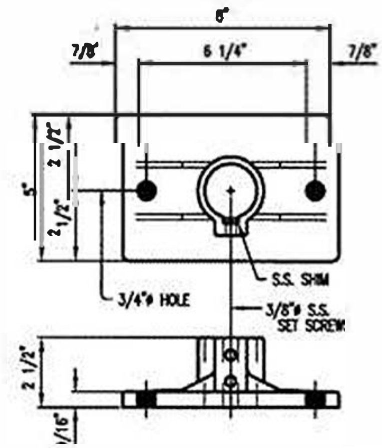
#TBF-3.4 Base Flange

– TOP MOUNT TO CONCRETE –



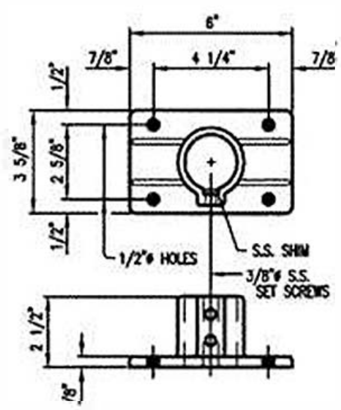
#TBF-3.2 Base Flange

– TOP MOUNT TO CONCRETE CURB –



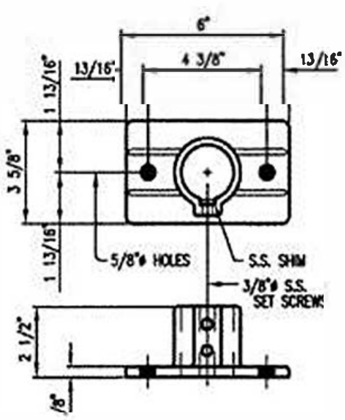
#TBF-1.4 Base Flange

–TOP MOUNT TO CONCRETE–



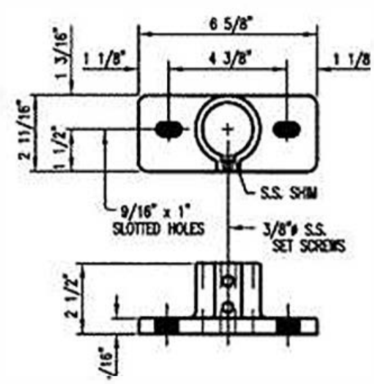
#TBF-1.2 Base Flange

– TOP MOUNT TO CONCRETE CURB –



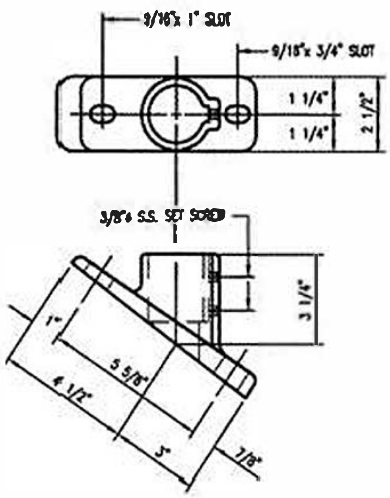
#TBF-2 Base Flange

– TOP MOUNT TO BEAM OR CHANNEL –

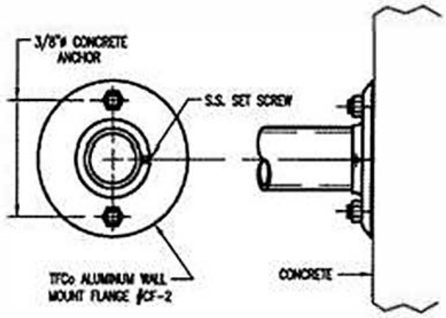


#ABF Base Flange

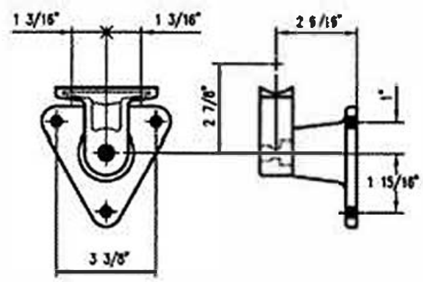
- RANGES FROM 30° TO 44° -
(TOP MOUNT TO STRINGER)



#CF-2 Wall Mount Flange

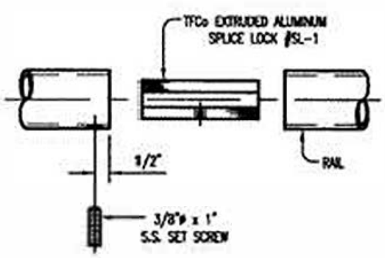


#AWF Adjustable Wall Rail Bracket

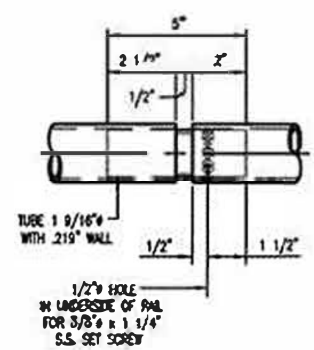


Field Splice Locks

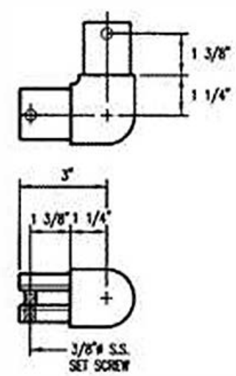
#SL-1 Splice Lock



#ES-1 Expansion Splice



#TE-1 90° Corner Elbow



FDR ENGINEERS

Glass Flooring System Inc.:

Walkable Aluminum Skylight Rafter System

General Notes and Assumptions:

1. Calculations based on ASCE7/10 and 2010 ADM
2. Deflection limitations are based on L/500 for the total load (TL)
3. Mullions and structural members and components shall be 6063-T6 unwelded aluminum (min)
4. "I" and "T" Rafters shall have section properties as shown on the following sheet
5. Glass lights were assumed to be 30" maximum width
6. For multiple light conditions, the girder mullions are assumed to be continuous, spanning in the short direction of overall opening with intermediate rafters notched to fit at the point of intersection. No notching of the girder mullion is allowed
7. Aluminum perimeter zee shapes are assumed to be in constant bearing with structural steel or concrete and anchored at 24" o.c. min.
8. Mullions assumed to be connected to perimeter zee with (2) 1/4" thick aluminum angle with 5/8" fasteners each end
9. Span tables are for reference only, project specific engineering shall be required for unique project conditions and loading prior to ordering of materials.

	Exterior - Roof Applications (20 psf Live Load)					
	Single Light		Double Light		Triple Light	
	Max Length	Max End Reaction	Max Length	Max End Reaction	Max Length	Max End Reaction
I Rafter	13'-1"	772 (lbf)	12'-6"	818 (lbf)	11'-9"	853 (lbf)
T Rafter	8'-3"	493 (lbf)	7'-7"	535 (lbf)	7'-5"	603 (lbf)

- Max span based on 20psf dead load, 20 psf live load, 25psf wind uplift and 30psf snow load
- Max end reaction indicates the max bearing or uplift each end of rafter

	Residential Applications (40 psf Live Load)					
	Single Light		Double Light		Triple Light	
	Max Length	Max Reaction	Max Length	Max End Reaction	Max Length	Max End Reaction
I Rafter	13'-8"	949 (lbf)	13'-0"	986 (lbf)	12'-8"	1047 (lbf)
T Rafter	8'-9"	595 (lbf)	8'-2"	639 (lbf)	8'-0"	711 (lbf)

- Max span based on 20psf dead load, 40 psf live load, and 5psf wind uplift
- Max end reaction indicates the max bearing each end of rafter

	Residential Balcony Applications (60 psf Live Load)					
	Single Light		Double Light		Triple Light	
	Max Length	Max End Reaction	Max Length	Max End Reaction	Max Length	Max End Reaction
I Rafter	12'-10"	1142 (lbf)	12'-0"	1171 (lbf)	11'-8"	1246 (lbf)
T Rafter	8'-2"	715 (lbf)	7'-8"	775 (lbf)	7'-6"	864 (lbf)

- Max span based on 20psf dead load, 60 psf live load, and 25psf wind uplift
- Max end reaction indicates the max bearing each end of rafter

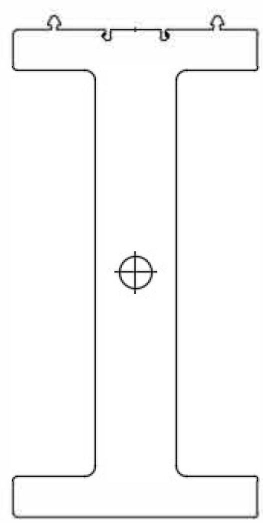
Glass Flooring System Inc.:

Walkable Aluminum Skylight Rafter System

	Commercial Load Applications (100 psf Live Load)					
	Single Light		Double Light		Triple Light	
	Max Length	Max End Reaction	Max Length	Max End Reaction	Max Length	Max End Reaction
I Rafter	11'-3"	1435 (lbf)	10'-6"	1498 (lbf)	10'-0"	1593 (lbf)
T Rafter	7'-6"	946 (lbf)	6'-11"	1032 (lbf)	6'-9"	1169 (lbf)

- Max span based on 20psf dead load, 100 psf live load, and 5psf wind uplift
- Max end reaction indicates the max bearing each end of rafter

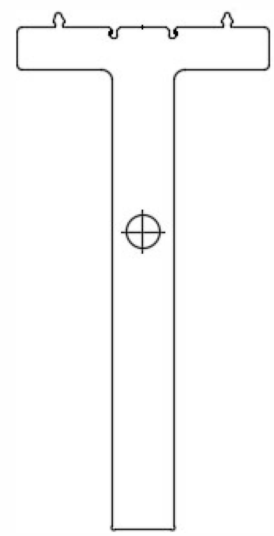
"I" Rafter



REGIONS

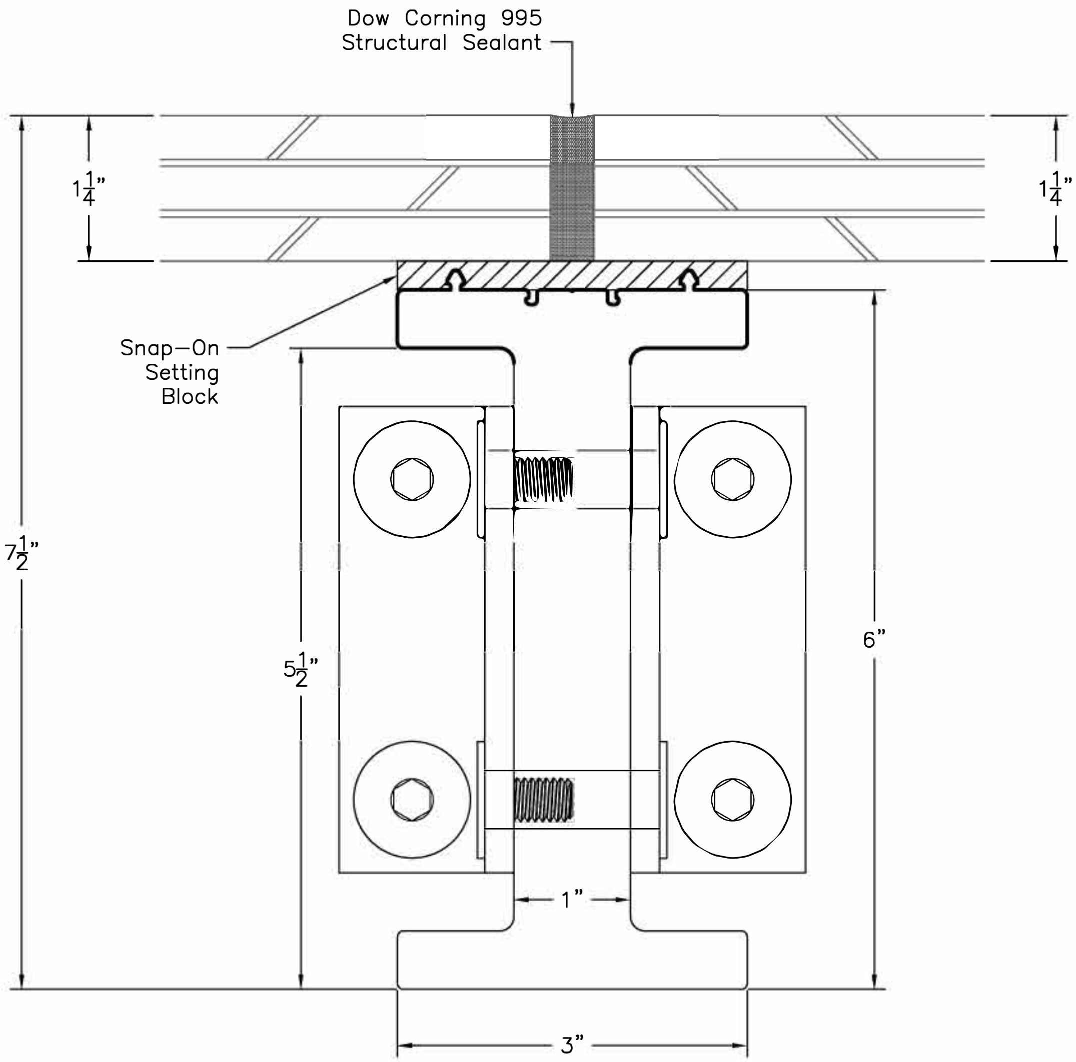
Area: 8.0221
Perimeter: 22.7803
Bounding box: X: -1.5000 -- 1.5000
Y: -3.0053 -- 3.1597
Centroid: X: 0.0000
Y: 0.0000
Moments of inertia: X: 33.3564
Y: 2.6920
Product of inertia: XY: 0.0000
Radii of gyration: X: 2.0391
Y: 0.5793

"T" Rafter



REGIONS

Area: 5.6415
Perimeter: 18.9306
Bounding box: X: -1.5000 -- 1.5000
Y: -3.5585 -- 2.6065
Centroid: X: 0.0000
Y: 0.0000
Moments of inertia: X: 20.4275
Y: 1.3457
Product of inertia: XY: 0.0000
Radii of gyration: X: 1.9029
Y: 0.4884



Glass Flooring Systems, Inc.			
TITLE: Skyfloor™ Non–Thermal I–Rafter			
REFERENCE FIXTURE:			SHEET 1 of 1
SCALE NTS	PART NUMBER GFS–AD3	DATE 2–2–16	REVISION

GLASS FLOORING SYSTEMS INC

MULTIPLE PATENTS PENDING

		Designer	Date
	Hurstville Lime Kilns	SRO	04/22

Kiln Platform: Column Connection

D	=	0.5	in	
l,m	=	0.25	in	
l,s	=	2.62	in	
F,em	=	87000	psi	
F,es	=	3150	psi	
F,yb	=	45000	psi	
θ	=	0	degrees	
K,θ	=	1		
R,e	=	27.62		
k,3	=	0.57		
Yield Mode Z values				
I,m	=	2718.8	lb	
I,s	=	2063.3	lb	
III,s	=	1370.8	lb	
IV	=	1492.2	lb	
Z	=	1370.8	lb	
C,D	=	1.6		
C,M	=	1		
C,t	=	1		
C,Δ	=	1		
C,g	=	0.893	*calculation below	
C,eg	=	1		
C,di	=	1		
C,tn	=	1		
Z'	=	1958.6	lb	for 1 bolt
Minimum End Distance	=	2	in	
Minimum Bolt Spacing	=	2	in	
l/D	=	0.5		
Minimum Edge Distance	=	0.75	in	
number of bolts in a row, n	=	3		
number of rows	=	2		
Total Z'	=	11751.6	lbs	> 9530 lbs

	Hurstville Lime Kilns	Designer	Date
		SRO	04/22

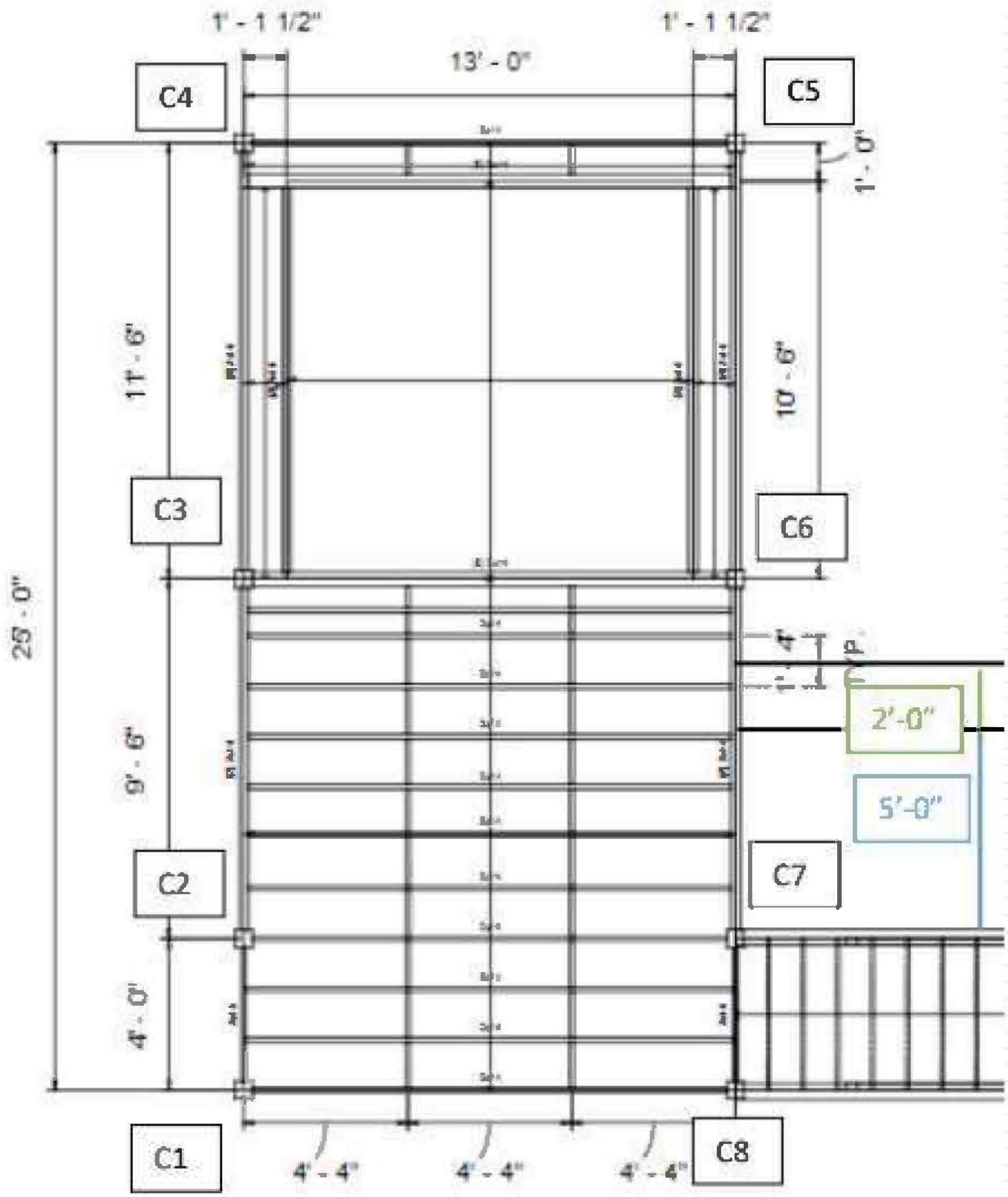
Kiln Platform: Column Connection

E,s	=	620000	psi
A,s	=	28.82	in^2
E,m	=	29,000,000	psi
A,m	=	1.375	in^2
R,EA	=	0.449	
y	=	95460	lbs/in
s	=	2	in
u	=	1.1	
m	=	0.65	
n	=	3	
C,g	=	0.893	

Weld Design

D	=	2	/16 in	
l	=	5.5	in	
F,EXX	=	70	ksi	
Ω	=	2		
R,n/Ω	=	10210	lbs	> 9530 lbs

Use = 0'- 0 1/2"x0'-5 1/2"x8" ASTM A36 Steel Knife plate with 1/2" dia. hex bolts, spaced per detail



$S_1 := 11.5\ ft$

$S_2 := 9.5\ ft$

vertical load:

Column Loads Summary:

	C1	C2	C3	C4	C5	C6	C7	C8
DL	666.51	1174.425	1725.845	1218.15	1218.15	1725.845	1323.875	815.96
LL	1649.3	3153	4872.69	3328.56	3328.56	4872.69	3488.5	1984.8
SL	428.51	985.425	1522.725	951.49	951.49	1522.725	1049.475	492.56
WL	1754.83	4035.55	6233.66	3892.99	3892.99	6233.66	4297.85	2017.13

horizontal load:

*Handrail and guardrail systems shall be designed to resist a single concentrated load of 200 pounds applied in any direction. Handrail and guardrail systems shall also be designed to resist 50 lb/ft applied in any direction along the handrail. These loads need not be assumed to be concurrent.

Vertical loads:

$D := 1725.845\ lbf$

$L := 4872.69\ lbf$

$S := 1522.725\ lbf$

$W := 6233.66\ lbf$

$F_a := 0\ lbf$ *no load from flooding on the platforms

horizontal loads:

$$V_{max} := \max\left(200\ lbf, \frac{S_1 + S_2}{2} \cdot 50\ \frac{lbf}{ft}\right) = 525\ lbf$$

$$M_{max} := V_{max} \cdot 12.5\ ft = 6.563\ kip \cdot ft$$

Load combo 1 - 1.4D

$1.4 \cdot D = 2.416\ kip$

Load combo 2 - 1.2D + 1.6L + 0.3S

$1.2 \cdot D + 1.6\ L + 0.3\ S = 10.324\ kip$ highest compressive load

Load combo 3 - 1.2D + 1.0S + 1.0L

$1.2 \cdot D + 1.0\ S + 1.0\ L = 8.466\ kip$

Load combo 4 - 1.2D +/- 0.5W + 1.0Fa + 0.3S

$1.2 \cdot D + 0.5\ W + 1.0\ F_a + 0.3\ S = 5.645\ kip$

$1.2 \cdot D - 0.5\ W + 1.0\ F_a + 0.3\ S = -0.589\ kip$

Load combo 5 - 0.9D +/- 0.5W + 1.0Fa

$0.9 \cdot D + 0.5\ W + 1.0\ F_a = 4.67\ kip$

$0.9 \cdot D - 0.5\ W + 1.0\ F_a = -1.564\ kip$ worst case for uplift

1.Project information

Customer company:
Customer contact name:
Customer e-mail:
Comment:

Project description:
Location:
Fastening description:

2. Input Data & Anchor Parameters

General
Design method:ACI 318-14
Units: Imperial units

Anchor Information:
Anchor type: Bonded anchor
Material: A706 Grade 60 Rebar
Diameter (inch): 0.500
Effective Embedment depth, h_{ef} (inch): 6.000
Code report: ICC-ES ESR-4057
Anchor category: -
Anchor ductility: Yes
h_{min} (inch): 7.25
c_{ac} (inch): 13.04
C_{min} (inch): 1.75
S_{min} (inch): 2.50

Base Material
Concrete: Normal-weight
Concrete thickness, h (inch): 12.00
State: Cracked
Compressive strength, f'_c (psi): 4000
$\Psi_{c,V}$: 1.0
Reinforcement condition: B tension, B shear
Supplemental reinforcement: Not applicable
Reinforcement provided at corners: No
Ignore concrete breakout in tension: No
Ignore concrete breakout in shear: No
Hole condition: Dry concrete
Inspection: Continuous
Temperature range, Short/Long: 150/110°F
Ignore 6do requirement: Not applicable
Build-up grout pad: No

Base Plate
Length x Width x Thickness (inch): 12.00 x 12.00 x 0.25

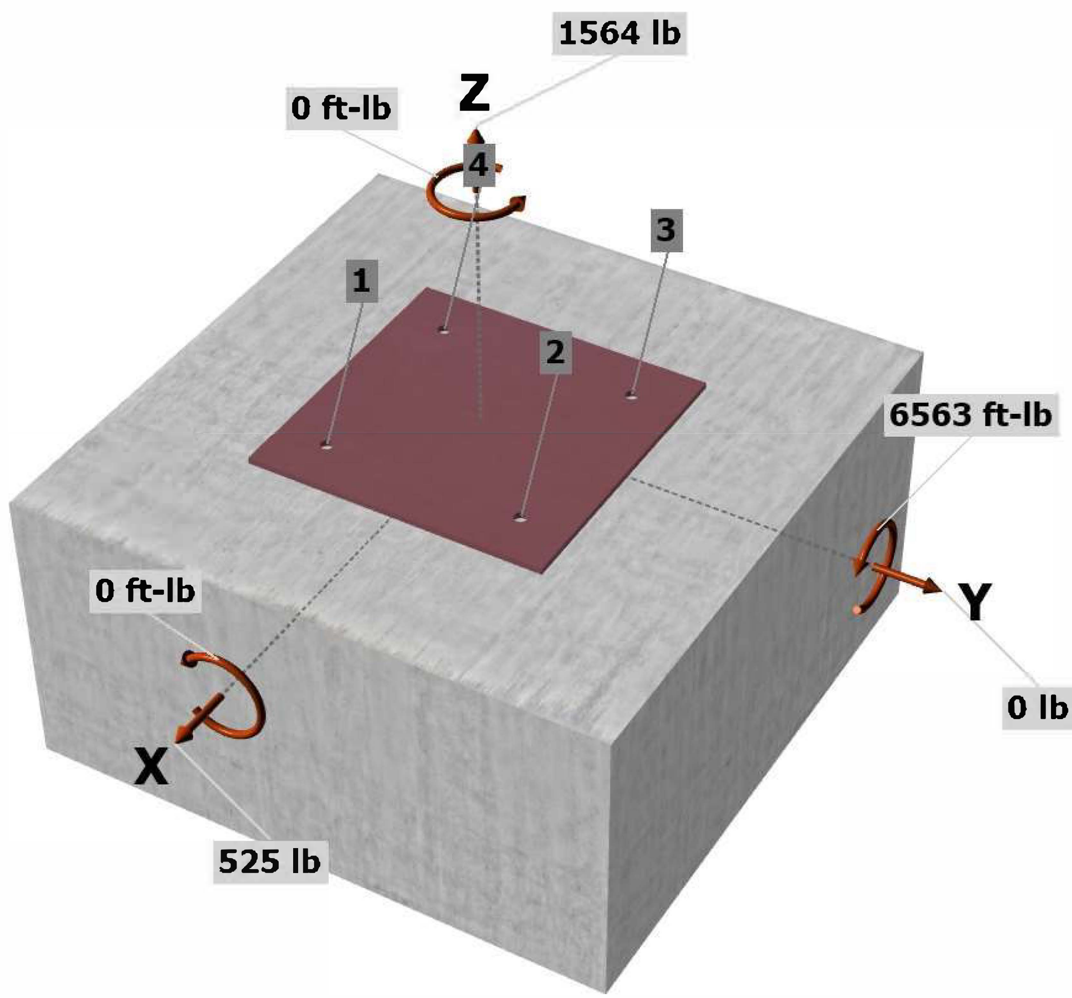
Recommended Anchor
Anchor Name: SET-3G - SET-3G w/ #4 A706 Gr. 60 Rebar
Code Report: ICC-ES ESR-4057

Load and Geometry
Load factor source: ACI 318 Section 5.3
Load combination: not set
Seismic design: No
Anchors subjected to sustained tension: No
Apply entire shear load at front row: No
Anchors only resisting wind and/or seismic loads: No

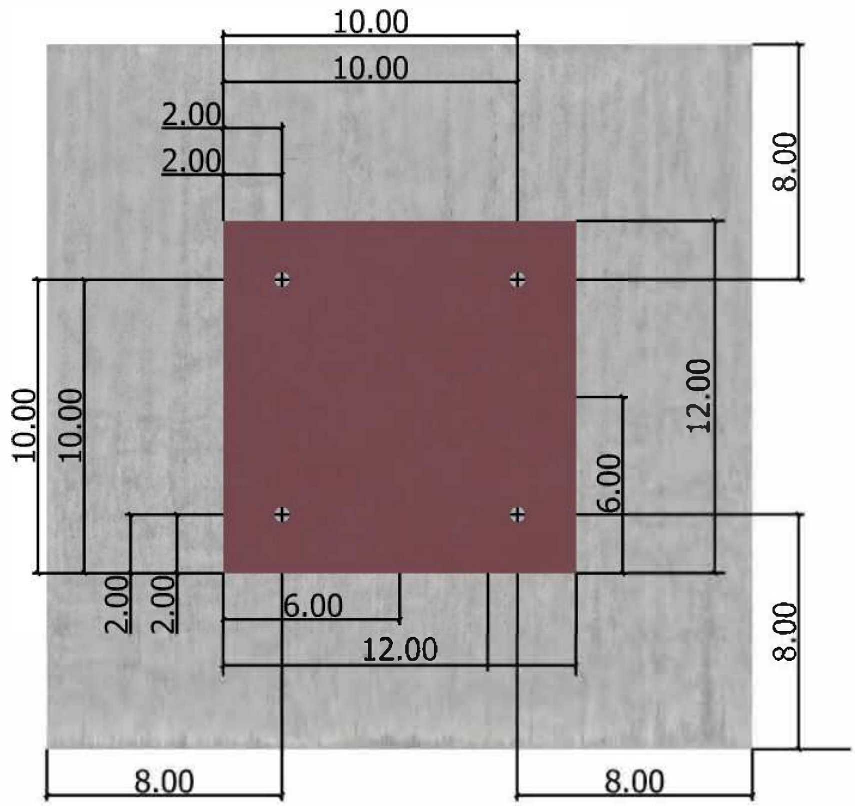
Strength level loads:

N_{ua} [lb]: 1564
V_{uax} [lb]: 525
V_{uay} [lb]: 0
M_{ux} [ft-lb]: 0
M_{uy} [ft-lb]: 6563
M_{uz} [ft-lb]: 0

<Figure 1>



<Figure 2>

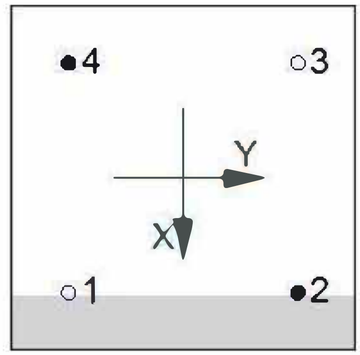

10.00
10.00
2.00
2.00
8.00
10.00
10.00
12.00
6.00
2.00
2.00
6.00
12.00
8.00
8.00
8.00

3. Resulting Anchor Forces

Anchor	Tension load, N_{ua} (lb)	Shear load x, V_{uax} (lb)	Shear load y, V_{uay} (lb)	Shear load combined, $\sqrt{(V_{uax})^2+(V_{uay})^2}$ (lb)
1	50.7	131.3	0.0	131.3
2	50.7	131.3	0.0	131.3
3	4646.0	131.3	0.0	131.3
4	4646.0	131.3	0.0	131.3
Sum	9393.4	525.0	0.0	525.0

Maximum concrete compression strain (‰): 0.16
Maximum concrete compression stress (psi): 683
Resultant tension force (lb): 9393
Resultant compression force (lb): 7830
Eccentricity of resultant tension forces in x-axis, e'_{Nx} (inch): 0.00
Eccentricity of resultant tension forces in y-axis, e'_{Ny} (inch): 3.91
Eccentricity of resultant shear forces in x-axis, e'_{Vx} (inch): 0.00
Eccentricity of resultant shear forces in y-axis, e'_{Vy} (inch): 0.00

<Figure 3>



4. Steel Strength of Anchor in Tension (Sec. 17.4.1)

N_{sa} (lb)	ϕ	ϕN_{sa} (lb)
16000	0.75	12000

5. Concrete Breakout Strength of Anchor in Tension (Sec. 17.4.2)

$N_b = k_c \lambda_a \sqrt{f'_c} h_{ef}^{1.5}$ (Eq. 17.4.2.2a)

k_c	λ_a	f'_c (psi)	h_{ef} (in)	N_b (lb)
17.0	1.00	4000	5.333	13243

$\phi N_{cbg} = \phi\ (A_{Nc} / A_{Nco})\, \Psi_{ec,N} \Psi_{ed,N} \Psi_{c,N} \Psi_{cp,N} N_b$ (Sec. 17.3.1 & Eq. 17.4.2.1b)

A_{Nc} (in^2)	A_{Nco} (in^2)	$c_{a,min}$ (in)	$\Psi_{ec,N}$	$\Psi_{ed,N}$	$\Psi_{c,N}$	$\Psi_{cp,N}$	N_b (lb)	ϕ	ϕN_{cbg} (lb)
576.00	256.00	8.00	0.671	1.000	1.00	1.000	13243	0.65	13005

6. Adhesive Strength of Anchor in Tension (Sec. 17.4.5)

$\tau_{k,cr} = \tau_{k,cr} f_{short\text{-}term} K_{sat} (f'_c / 2{,}500)^n$

$\tau_{k,cr}$ (psi)	$f_{short\text{-}term}$	K_{sat}	f'_c (psi)	n	$\tau_{k,cr}$ (psi)
1402	1.00	1.00	4000	0.25	1577

$N_{ba} = \lambda_a \tau_{cr} \pi d_a h_{ef}$ (Eq. 17.4.5.2)

λ_a	τ_{cr} (psi)	d_a (in)	h_{ef} (in)	N_{ba} (lb)
1.00	1577	0.50	6.000	14861

$\phi N_{ag} = \phi\ (A_{Na} / A_{Na0})\, \Psi_{ec,Na} \Psi_{ed,Na} \Psi_{cp,Na} N_{ba}$ (Sec. 17.3.1 & Eq. 17.4.5.1b)

A_{Na} (in^2)	A_{Na0} (in^2)	c_{Na} (in)	$c_{a,min}$ (in)	$\Psi_{ec,Na}$	$\Psi_{ed,Na}$	$\Psi_{cp,Na}$	N_{ba}(lb)	ϕ	ϕN_{ag} (lb)
482.43	195.00	6.98	8.00	0.641	1.000	1.000	14861	0.65	15314

8. Steel Strength of Anchor in Shear (Sec. 17.5.1)

V_{sa} (lb)	ϕ_{grout}	ϕ	$\phi_{grout}\phi V_{sa}$ (lb)
9600	1.0	0.65	6240

9. Concrete Breakout Strength of Anchor in Shear (Sec. 17.5.2)

Shear perpendicular to edge in x-direction:

$V_{bx} = \min|7(l_e / d_a)^{0.2}\sqrt{d_a}\lambda_a\sqrt{f'_c}c_{a1}^{1.5};\ 9\lambda_a\sqrt{f'_c}c_{a1}^{1.5}|$ (Eq. 17.5.2.2a & Eq. 17.5.2.2b)

l_e (in)	d_a (in)	λ_a	f'_c (psi)	c_{a1} (in)	V_{bx} (lb)
4.00	0.500	1.00	4000	8.00	10737

$\phi V_{cbgx} = \phi\ (A_{Vc} / A_{Vco})\Psi_{ec,V}\Psi_{ed,V}\Psi_{c,V}\Psi_{h,V}V_{bx}$ (Sec. 17.3.1 & Eq. 17.5.2.1b)

A_{Vc} (in^2)	A_{Vco} (in^2)	$\Psi_{ec,V}$	$\Psi_{ed,V}$	$\Psi_{c,V}$	$\Psi_{h,V}$	V_{bx} (lb)	ϕ	ϕV_{cbgx} (lb)
288.00	288.00	1.000	0.900	1.000	1.000	10737	0.70	6764

Shear parallel to edge in y-direction:

$V_{bx} = \min|7(l_e / d_a)^{0.2}\sqrt{d_a}\lambda_a\sqrt{f'_c}c_{a1}^{1.5};\ 9\lambda_a\sqrt{f'_c}c_{a1}^{1.5}|$ (Eq. 17.5.2.2a & Eq. 17.5.2.2b)

l_e (in)	d_a (in)	λ_a	f'_c (psi)	c_{a1} (in)	V_{bx} (lb)
4.00	0.500	1.00	4000	8.00	10737

$\phi V_{cbgy} = \phi\ (2)(A_{Vc} / A_{Vco})\Psi_{ec,V}\Psi_{ed,V}\Psi_{c,V}\Psi_{h,V}V_{bx}$ (Sec. 17.3.1, 17.5.2.1(c) & Eq. 17.5.2.1b)

A_{Vc} (in^2)	A_{Vco} (in^2)	$\Psi_{ec,V}$	$\Psi_{ed,V}$	$\Psi_{c,V}$	$\Psi_{h,V}$	V_{bx} (lb)	ϕ	ϕV_{cbgy} (lb)
288.00	288.00	1.000	1.000	1.000	1.000	10737	0.70	15031

10. Concrete Pryout Strength of Anchor in Shear (Sec. 17.5.3)

$\phi V_{cpg} = \phi \min|k_{cp}N_{ag}\ ;\ k_{cp}N_{cbg}| = \phi \min|k_{cp}(A_{Na} / A_{Na0})\Psi_{ec,Na}\Psi_{ed,Na}\Psi_{cp,Na}N_{ba}\ ;\ k_{cp}(A_{Nc} / A_{Nco})\Psi_{ec,N}\Psi_{ed,N}\Psi_{c,N}\Psi_{cp,N}N_b|$ (Sec. 17.3.1 & Eq. 17.5.3.1b)

k_{cp}	A_{Na} (in^2)	A_{Na0} (in^2)	$\Psi_{ed,Na}$	$\Psi_{ec,Na}$	$\Psi_{cp,Na}$	N_{ba} (lb)	N_a (lb)
2.0	482.43	195.00	1.000	1.000	1.000	14861	36766

A_{Nc} (in^2)	A_{Nco} (in^2)	$\Psi_{ec,N}$	$\Psi_{ed,N}$	$\Psi_{c,N}$	$\Psi_{cp,N}$	N_b (lb)	N_{cb} (lb)	ϕ
576.00	256.00	1.000	1.000	1.000	1.000	13243	29796	0.70

ϕV_{cpg} (lb)
41715

11. Results

Interaction of Tensile and Shear Forces (Sec. 17.6.)

Tension	Factored Load, N_{ua} (lb)	Design Strength, $øN_n$ (lb)	Ratio	Status
Steel	4646	12000	0.39	Pass
Concrete breakout	**9393**	**13005**	**0.72**	**Pass (Governs)**
Adhesive	9393	15314	0.61	Pass

Shear	Factored Load, V_{ua} (lb)	Design Strength, $øV_n$ (lb)	Ratio	Status
Steel	131	6240	0.02	Pass
T Concrete breakout x+	**525**	**6764**	**0.08**	**Pass (Governs)**

|| Concrete breakout y+	**263**	**15031**	**0.02**	**Pass (Governs)**
Pryout	525	41715	0.01	Pass

Interaction check	$N_{ua}/\phi N_n$	$V_{ua}/\phi V_n$	Combined Ratio	Permissible	Status
Sec. 17.6..1	0.72	0.00	72.2%	1.0	Pass

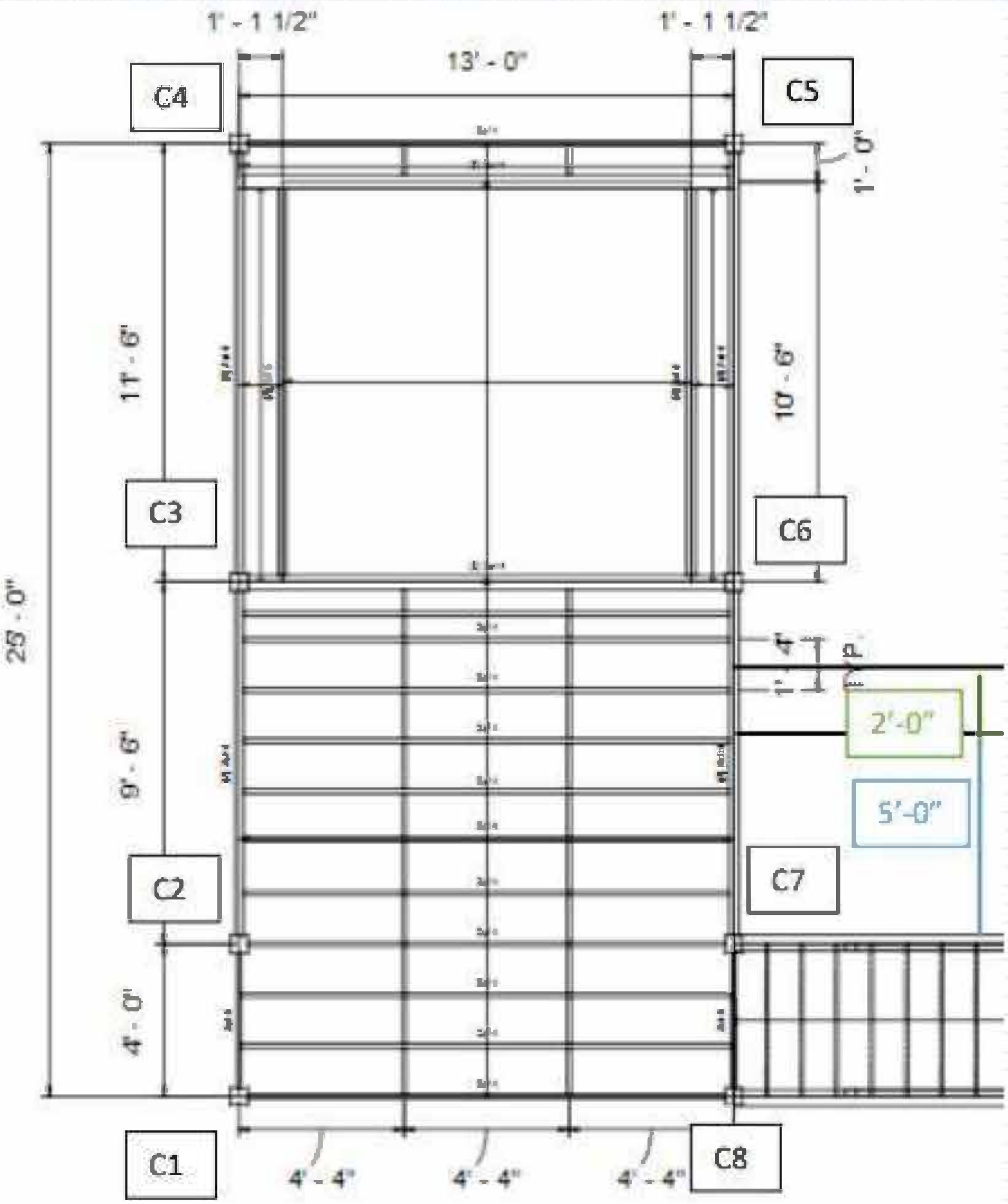
SET-3G w/ #4 A706 Gr. 60 Rebar with hef = 6.000 inch meets the selected design criteria.

12. Warnings

- Designer must exercise own judgement to determine if this design is suitable.

- Refer to manufacturer's product literature for hole cleaning and installation instructions.

Loads from platforms



$S_1 := 11.5\ ft$

$S_2 := 9.5\ ft$

Column Loads Summary:

	C1	C2	C3	C4	C5	C6	C7	C8
DL	666.51	1174.425	1725.845	1218.15	1218.15	1725.845	1323.875	815.96
LL	1649.3	3153	4872.69	3328.56	3328.56	4872.69	3488.5	1984.8
SL	428.51	985.425	1522.725	951.49	951.49	1522.725	1049.475	492.56
WL	1754.83	4035.55	6233.66	3892.99	3892.99	6233.66	4297.85	2017.13

$C_{3D} := 2054.85\ lbf$ $\quad C_{4D} := 1218.15\ lbf$ $\quad C_{5D} := C_{3D}$ $\quad C_{6D} := 2155.94\ lbf$

$C_{3L} := 5752.69\ lbf$ $\quad C_{4L} := 3328.56\ lbf$ $\quad C_{5L} := C_{3L}$ $\quad C_{6L} := 6041.54\ lbf$

$C_{3S} := 1795.73\ lbf$ $\quad C_{4S} := 951.49\ lbf$ $\quad C_{5S} := C_{3S}$ $\quad C_{6S} := 1896.83\ lbf$

$C_{3W} := 7351.66\ lbf$ $\quad C_{4W} := 3893.99\ lbf$ $\quad C_{5W} := C_{3W}$ $\quad C_{6W} := 7765.69\ lbf$

$$C_{25h} := \frac{S_1 + S_2}{2} \cdot 50\ \frac{lbf}{ft} = 525\ lbf \qquad C_{34h} := \frac{13\ ft}{2} \cdot 50\ \frac{lbf}{ft} = 325\ lbf$$

Dead Loads

Smokestacks

Kilns 1 and 4

$W_{ssa} := 2400.9\ lbf$

$A_a := 139.63\ ft^2$

$c_a := 41.89\ ft$

Kilns 2 and 3

$W_{ssb} := 1637.7\ lbf$

$A_b := 81.2\ ft^2$

$c_b := 31.9\ ft$

Concrete Slab

$t := 5\ in$ $\qquad \gamma_{conc} := 145\ pcf$

Kilns 1 and 4

$W_{slaba} := 116.4\ ft^2 \cdot t \cdot \gamma_{conc} = 7.033\ kip$

$A_{slaba} := 116.4\ ft^2$

Kilns 2 and 3

$W_{slabb} := 174.8\ ft^2 \cdot t \cdot \gamma_{conc} = 10.561\ kip$

$A_{slabb} := 174.8\ ft^2$

Limestone Block Walls

$\gamma_{limestone} := 170\ pcf$ $\qquad \gamma_{brick} := 110\ pcf$ $\qquad H := 32\ ft$

$t_{limestone} := 1.5\ ft$ $\qquad t_{brick} := 8\ in$

$$w_{wall} := H \cdot \left(\gamma_{limestone} \cdot t_{limestone} + \gamma_{brick} \cdot t_{brick}\right) = 10.507\ \frac{kip}{ft}$$

Soil

$\gamma_{soil} := 123\ pcf$ $\qquad \phi' := 32\ deg$ $\qquad K_o := 1 - \sin\left(\phi'\right) = 0.47$

$d_e := 4.5\ ft$ (depth of embedment)

Snow Loads

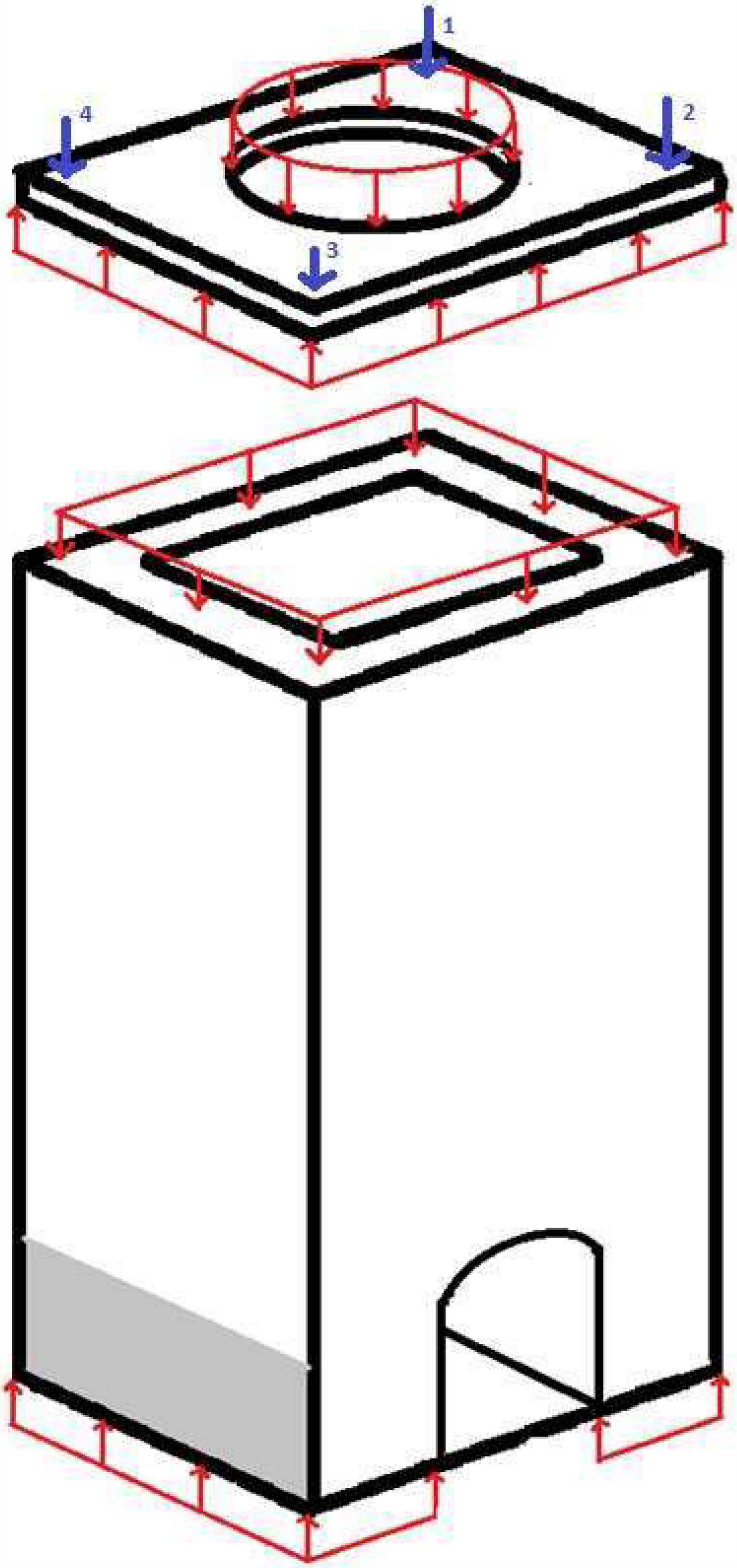
$p_{fI} := 25\ psf$ minimum for risk category I

$p_{fII} := 30\ psf$ minimum for risk category II

Wind Loads

windward wall	leeward wall	
$p_{wwI} := 11.046\ psf$	$p_{lwI} := 13.366\ psf$	risk category I
$p_{wwII} := 12.631\ psf$	$p_{lwII} := 15.283\ psf$	risk category II

LOAD COMBINATION 1 - 1.4D



$$w_{1a} := \frac{1.4\ W_{ssa}}{c_a} = 0.08\ \frac{kip}{ft}$$

$$w_{1b} := \frac{1.4\ W_{ssb}}{c_b} = 0.072\ \frac{kip}{ft}$$

$$P_1 := 1.4 \cdot C_{3D} = 2.877\ kip$$

$$P_2 := 1.4 \cdot C_{4D} = 1.705\ kip$$

$$P_3 := 1.4\ C_{5D} = 2.877\ kip$$

$$P_4 := 1.4 \cdot C_{6D} = 3.018\ kip$$

$$w_{2a} := w_{1a} + \frac{1.4\ W_{slaba}}{64\ ft} = 0.234\ \frac{kip}{ft}$$

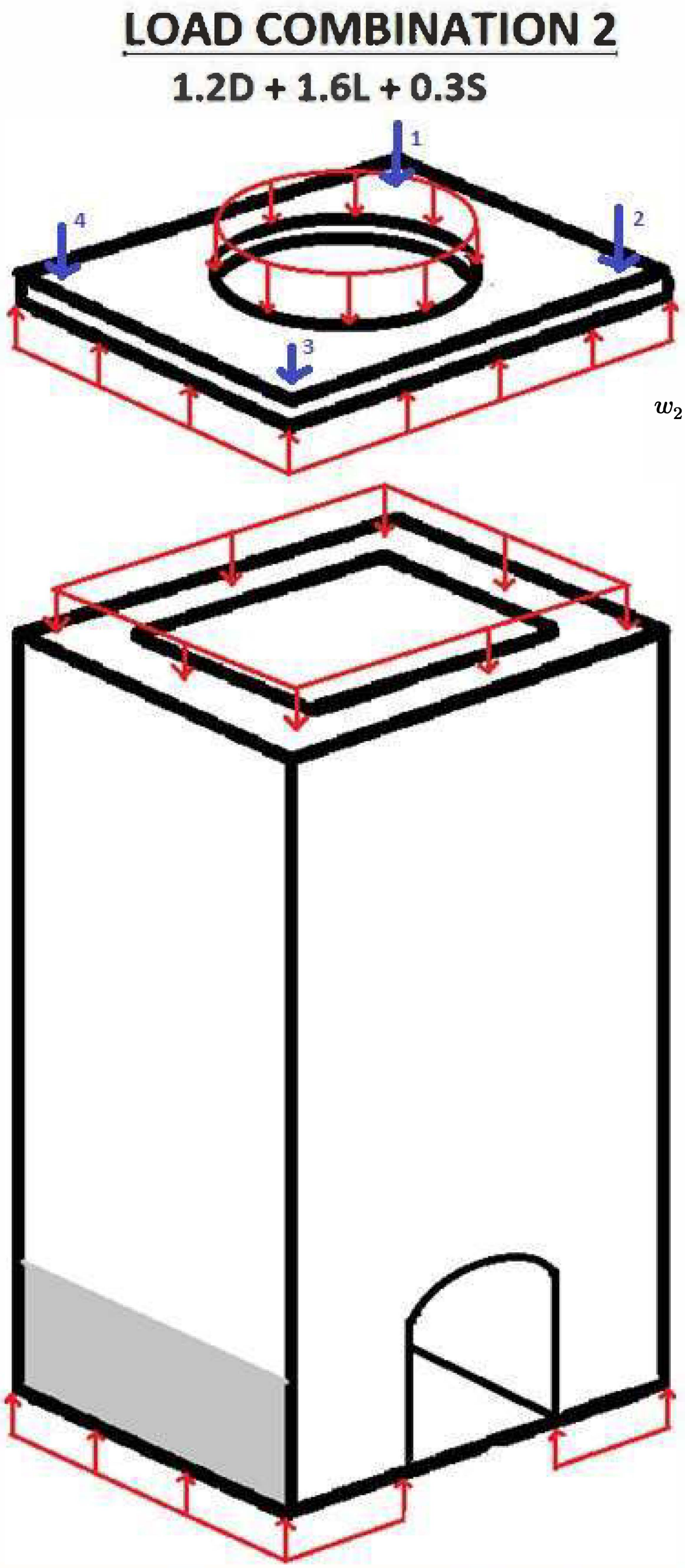
$$w_{2b} := w_{1b} + \frac{1.4\ W_{slabb}}{64\ ft} = 0.303\ \frac{kip}{ft}$$

$$w_{3a} := w_{2a} + 1.4\ w_{wall} = 14.943\ \frac{kip}{ft}$$

$$w_{3b} := w_{2b} + 1.4\ w_{wall} = 15.012\ \frac{kip}{ft}$$

$$W := w_{3b} \cdot 64\ ft + P_1 + P_2 + P_3 + P_4 = 971.26\ kip$$

analyzing kiln 2 for live load



$$w_1 := \frac{1.2\ W_{ssb}}{c_b} = 0.062\ \frac{kip}{ft}$$

$$P_1 := 1.2 \cdot C_{3D} + 1.6 \cdot C_{3L} + 0.3 \cdot C_{3S} = 12.209\ kip$$
$$P_2 := 1.2 \cdot C_{4D} + 1.6 \cdot C_{4L} + 0.3 \cdot C_{4S} = 7.073\ kip$$
$$P_3 := 1.2 \cdot C_{5D} + 1.6 \cdot C_{5L} + 0.3 \cdot C_{5S} = 12.209\ kip$$
$$P_4 := 1.2 \cdot C_{6D} + 1.6 \cdot C_{6L} + 0.3 \cdot C_{6S} = 12.823\ kip$$

$$w_2 := w_1 + \frac{1.2\ W_{slabb} + 0.3 \cdot p_{fII} \cdot A_{slabb}}{64\ ft} = 0.284\ \frac{kip}{ft}$$

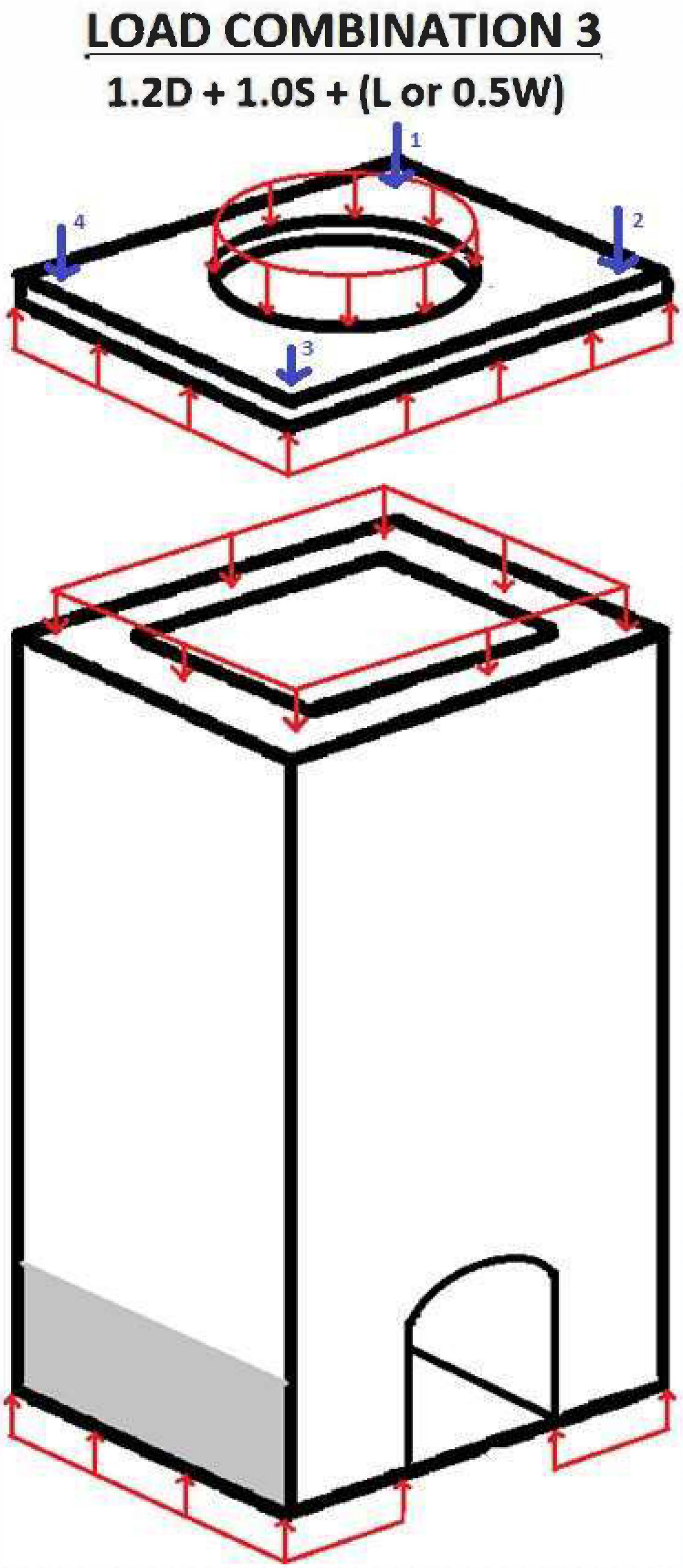
$$P_1 + P_2 + P_3 + P_4 = 44.313\ kip$$

$$w_3 := w_2 + 1.2\ w_{wall} = ?\ \frac{kip}{ft}$$

$$w_3 \cdot 64\ ft = 825.101\ kip$$

$$W := w_3 \cdot 64\ ft + P_1 + P_2 + P_3 + P_4 = 869.414\ kip$$

analyzing kiln 2 for live load



$$w_1 := \frac{1.2\ W_{ssb}}{c_b} = 0.062\ \frac{kip}{ft}$$

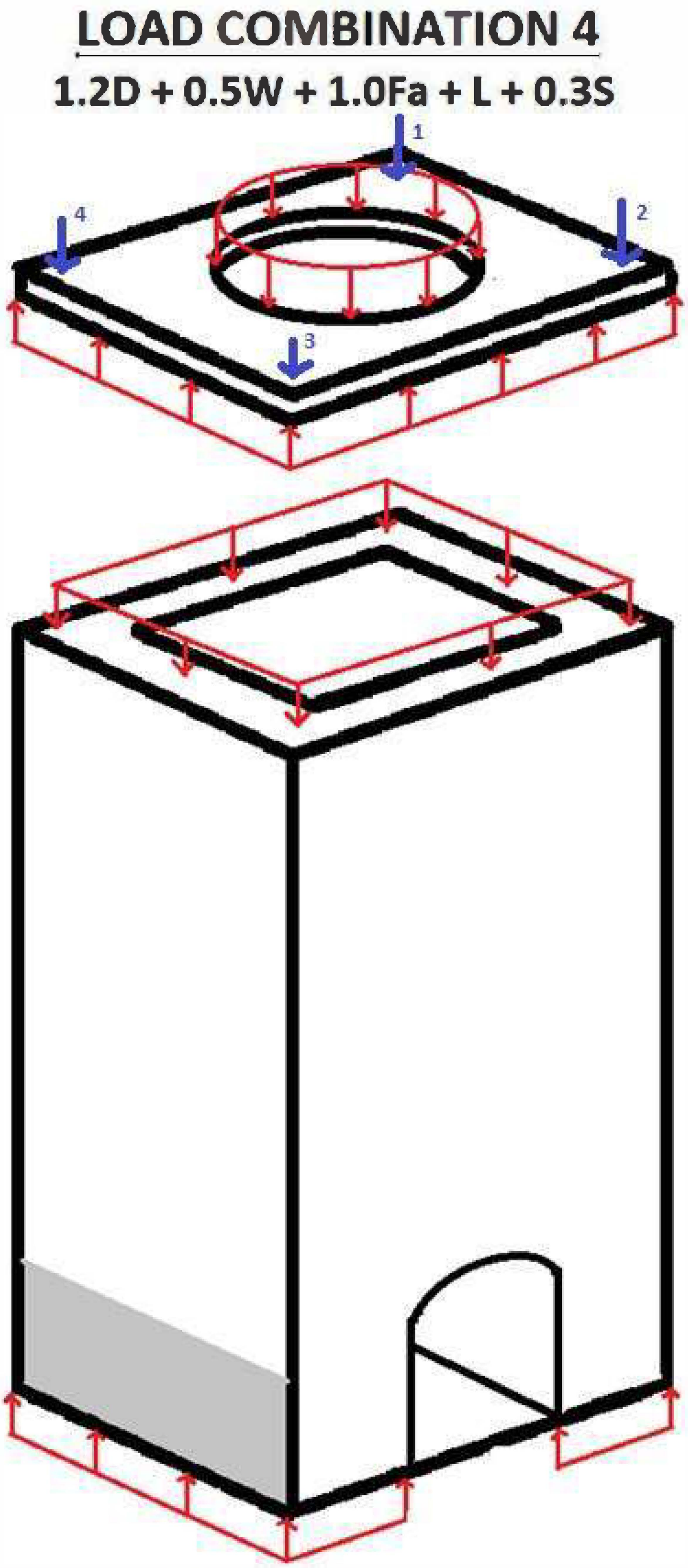
$$P_1 := 1.2 \cdot C_{3D} + 1.0 \cdot C_{3L} + 1.0 \cdot C_{3S} = 10.014\ kip$$
$$P_2 := 1.2 \cdot C_{4D} + 1.0 \cdot C_{4L} + 1.0 \cdot C_{4S} = 5.742\ kip$$
$$P_3 := 1.2 \cdot C_{5D} + 1.0 \cdot C_{5L} + 1.0 \cdot C_{5S} = 10.014\ kip$$
$$P_4 := 1.2 \cdot C_{6D} + 1.0 \cdot C_{6L} + 1.0 \cdot C_{6S} = 10.525\ kip$$

$$w_2 := w_1 + \frac{1.2\ W_{slabb} + 1.0 \cdot p_{fII} \cdot A_{slabb}}{64\ ft} = 0.342\ \frac{kip}{ft}$$

$$w_3 := w_2 + 1.2\ w_{wall} = 12.95\ \frac{kip}{ft}$$

$$W := w_3 \cdot 64\ ft + P_1 + P_2 + P_3 + P_4 = 865.068\ kip$$

analyzing kiln 2 for live load



$$w_1 := \frac{1.2\ W_{ssb}}{c_b} = 0.062\ \frac{kip}{ft}$$

$$P_1 := 1.2 \cdot C_{3D} + 1 \cdot C_{3L} + 0.3 \cdot C_{3S} + 0.5\ C_{3W} = 12.433\ kip$$
$$P_2 := 1.2 \cdot C_{4D} + 1 \cdot C_{4L} + 0.3 \cdot C_{4S} + 0.5\ C_{4W} = 7.023\ kip$$
$$P_3 := 1.2 \cdot C_{5D} + 1 \cdot C_{5L} + 0.3 \cdot C_{5S} + 0.5\ C_{5W} = 12.433\ kip$$
$$P_4 := 1.2 \cdot C_{6D} + 1 \cdot C_{6L} + 0.3 \cdot C_{6S} + 0.5\ C_{6W} = 13.081\ kip$$

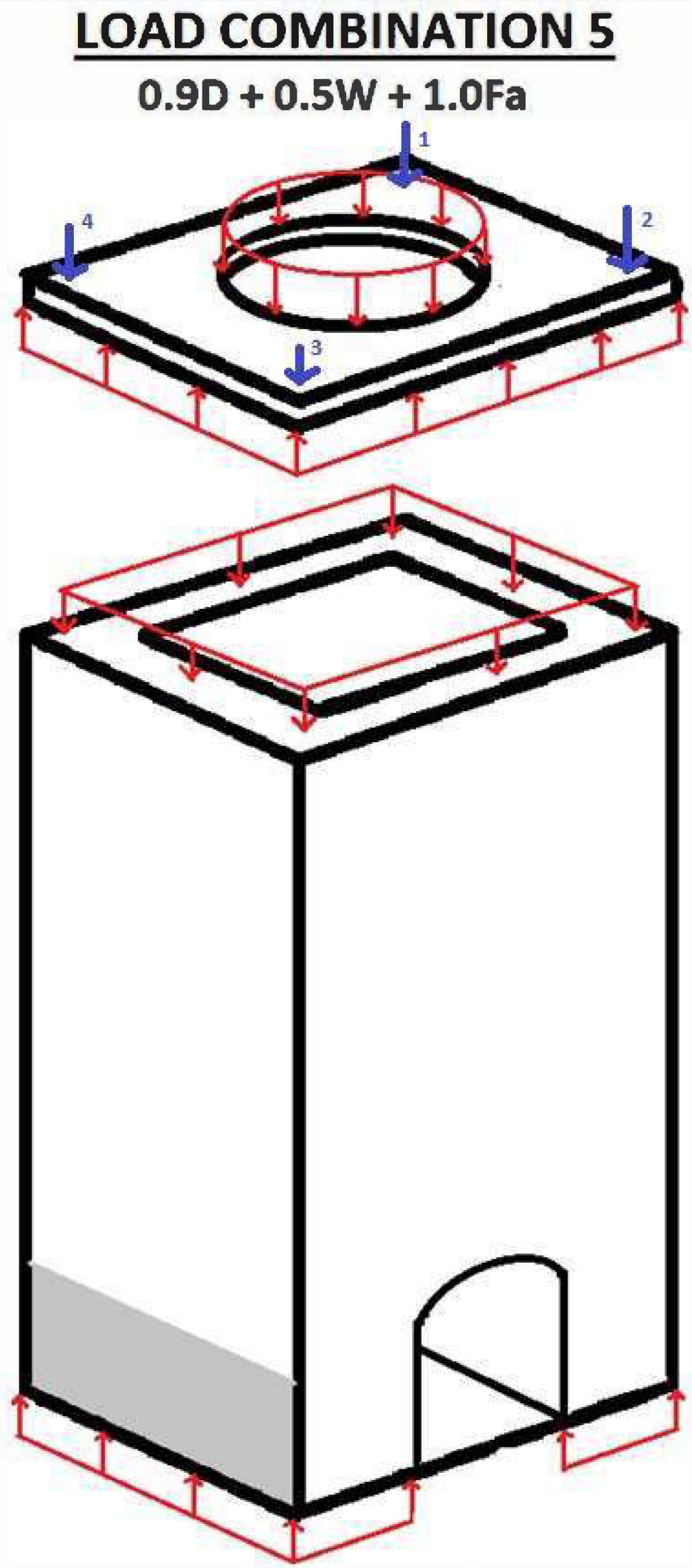
$$P_1 + P_2 + P_3 + P_4 = 44.969\ kip$$

$$w_2 := w_1 + \frac{1.2\ W_{slabb} + 0.3 \cdot p_{fII} \cdot A_{slabb}}{64\ ft} = 0.284\ \frac{kip}{ft}$$

$$w_3 := w_2 + 1.2\ w_{wall} = 12.892\ \frac{kip}{ft}$$

$$W := w_3 \cdot 64\ ft + P_1 + P_2 + P_3 + P_4 = 870.07\ kip$$

checking for uplift (using negative wind)



$$w_1 := \frac{0.9\ W_{ssb}}{c_b} = 0.046\ \frac{kip}{ft}$$

$$P_1 := 0.9 \cdot C_{3D} - 0.5\ C_{3W} = -1.826\ kip$$

$$P_2 := 0.9 \cdot C_{4D} - 0.5\ C_{4W} = -0.851\ kip$$

$$P_3 := 0.9\ C_{5D} - 0.5\ C_{5W} = -1.826\ kip$$

$$P_4 := 0.9 \cdot C_{6D} - 0.5\ C_{6W} = -1.942\ kip$$

$$w_2 := w_1 + \frac{0.9\ W_{slabb}}{64\ ft} = 0.195\ \frac{kip}{ft}$$

$$W_1 := w_2 \cdot 64\ ft + P_1 + P_2 + P_3 + P_4 = 6.016\ kip$$

$$w_3 := w_2 + 0.9\ w_{wall} = 9.651\ \frac{kip}{ft}$$

$$W := w_3 \cdot 64\ ft + P_1 + P_2 + P_3 + P_4 = 611.2\ kip$$

LOAD COMBINATION 1 - 1.4D

$w_{soil} := 1.4\ K_o \cdot d_e \cdot \gamma_{soil} = 364.266\ psf$

32 ft

LOAD COMBINATION 2

1.2D + 1.6L + 0.3S

$w_{soil} := 1.2 \cdot K_o \cdot d_e \cdot \gamma_{soil} = 312.228\ psf$

32 ft

LOAD COMBINATION 3

1.2D + 1.0S + (L or 0.5W)

$w_{soil} := 1.2 \cdot K_o \cdot d_e \cdot \gamma_{soil} = 312.228\ psf$

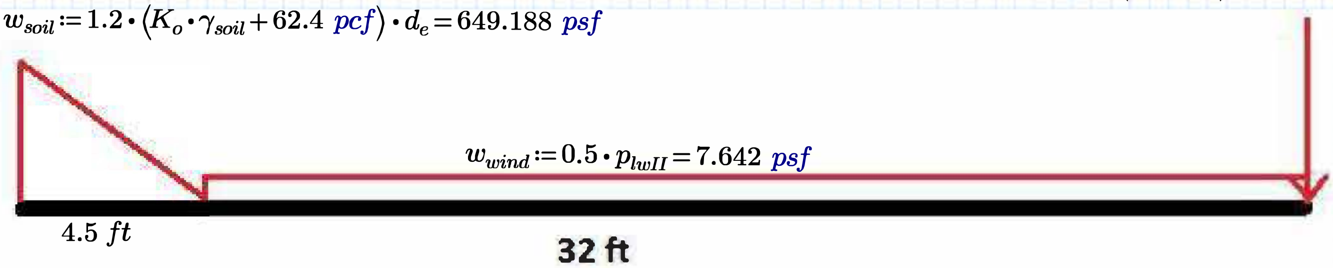
32 ft

LOAD COMBINATION 4

1.2D + 0.5W + 1.0Fa + L + 0.3S

$$P := 50\ \frac{lbf}{ft} \cdot \left(\frac{S_1 + S_2}{2}\right) = 0.525\ kip$$

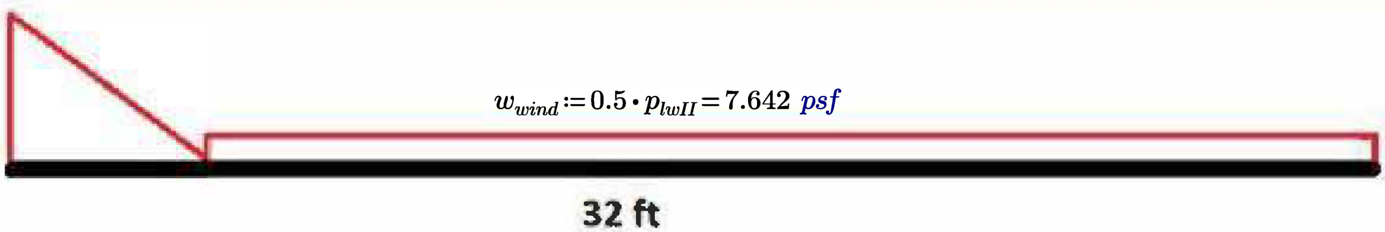
$$w_{soil} := 1.2 \cdot \left(K_o \cdot \gamma_{soil} + 62.4\ pcf\right) \cdot d_e = 649.188\ psf$$



LOAD COMBINATION 5

0.9D + 0.5W + 1.0Fa

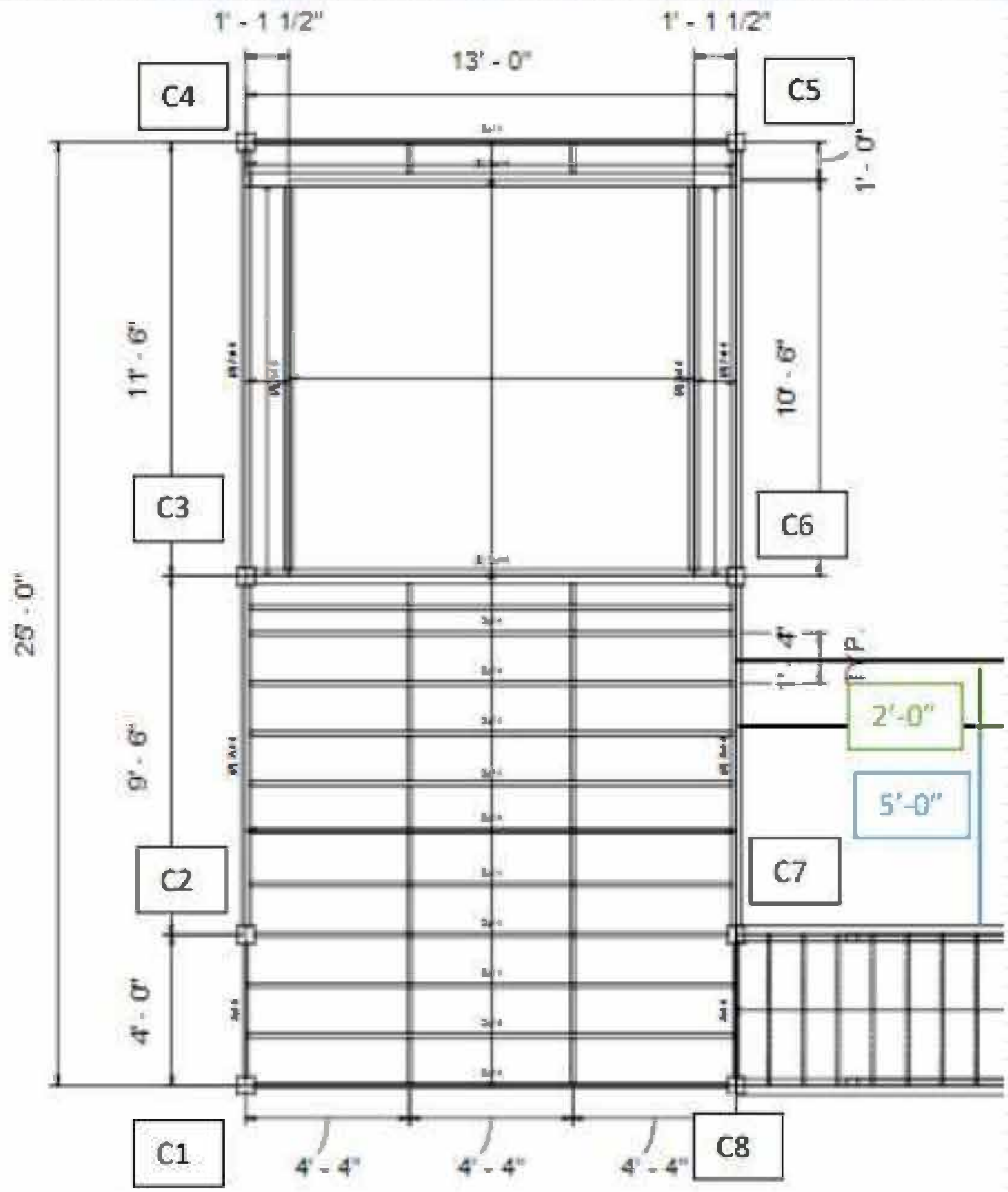
$$w_{soil} := 0.9\ \left(K_o \cdot \gamma_{soil} + 62.4\ pcf\right) \cdot d_e = 486.891\ psf$$



LOAD COMBO 2 is the worst case for vertical loads

LOAD COMBO 4 is the worst case for horizontal loads

Loads from platforms



$S_1 := 11.5\ ft$

$S_2 := 9.5\ ft$

Column Loads Summary:

	C1	C2	C3	C4	C5	C6	C7	C8
DL	666.51	1174.425	1725.845	1218.15	1218.15	1725.845	1323.875	815.96
LL	1649.3	3153	4872.69	3328.56	3328.56	4872.69	3488.5	1984.8
SL	428.51	985.425	1522.725	951.49	951.49	1522.725	1049.475	492.56
WL	1754.83	4035.55	6233.66	3892.99	3892.99	6233.66	4297.85	2017.13

$C_{3D} := 2054.85\ lbf$ $\quad C_{4D} := 1218.15\ lbf$ $\quad C_{5D} := C_{3D}$ $\quad C_{6D} := 2155.94\ lbf$

$C_{3L} := 5752.69\ lbf$ $\quad C_{4L} := 3328.56\ lbf$ $\quad C_{5L} := C_{3L}$ $\quad C_{6L} := 6041.54\ lbf$

$C_{3S} := 1795.73\ lbf$ $\quad C_{4S} := 951.49\ lbf$ $\quad C_{5S} := C_{3S}$ $\quad C_{6S} := 1896.83\ lbf$

$C_{3W} := 7351.66\ lbf$ $\quad C_{4W} := 3893.99\ lbf$ $\quad C_{5W} := C_{3W}$ $\quad C_{6W} := 7765.69\ lbf$

$$C_{25h} := \frac{S_1 + S_2}{2} \cdot 50\ \frac{lbf}{ft} = 525\ lbf \qquad C_{34h} := \frac{13\ ft}{2} \cdot 50\ \frac{lbf}{ft} = 325\ lbf$$

$t_{limestone} := 1.5\ ft$ thickness of limestone block

$t_{brick} := 8\ in$ thickness of brick

$A_{wall} := 16\ ft \cdot (t_{limestone} + t_{brick}) = 34.667\ ft^2$ cross sectional area of wall

$A_{unit} := 1\ ft \cdot (t_{limestone} + t_{brick}) = 2.167\ ft^2$ area of 1 ft strip of wall

$V := C_{25h} = 0.525\ kip$ shear force on wall

$\tau_{wall} := \dfrac{2 \cdot V}{A_{wall}} = 0.21\ psi$ shear stress applied on wall

$\tau_{unit} := \dfrac{V}{A_{unit}} = 1.683\ psi$ shear stress applied to 1 ft strip (one unit)

Joint shear strength

τ_o ranges from 35 psi to 100 psi

assumptions: $\tau_o := 35\ psi$

$\mu := 0.1$

$\sigma_N := 0\ psi$

$\tau := \tau_o + \mu \cdot \sigma_N = 35\ psi$

$\tau > \tau_{unit}$ $\tau > \tau_{wall}$

GIVEN VALUES

$P_{SL} := 45\ kip$ $D_f := 4.5\ ft$ $\gamma := 123\ pcf$ $H := 15\ ft$

$t_f := 10\ in$ $\gamma_c := 150\ pcf$

$\gamma_{bf} := 120\ pcf$ $H_{bf} := 4.5\ ft - 10\ in$

$\alpha := 4$ $B := 16\ ft$ $L := B$ $A_f := B \cdot L = 256\ ft^2$ $B' := \frac{B}{2}$ $L' := \frac{L}{2}$

$H := 5\ B$ $E_{s1} := 160\ \frac{tonf}{ft^2}$ $E_{s2} := 250\ \frac{tonf}{ft^2}$ $\mu_{s1} := 0.3$ $\mu_{s2} := 0.3$

$$E_s := \frac{E_{s1} \cdot 4.5\ ft + E_{s2} \cdot (H - 4.5\ ft)}{H} = 244.938\ \frac{tonf}{ft^2} \qquad \mu_s := \frac{\mu_{s1} \cdot 4.5\ ft + \mu_{s2} \cdot (H - 4.5\ ft)}{H}$$

$$q_{net} := \frac{P_{SL}}{A_f} = 175.8\ psf$$

DEPTH FACTOR CALCS

$\beta_1 := 3 - 4 \cdot \mu_s$ $r := 2\ D_f = 9\ ft$

$\beta_2 := 5 - 12 \cdot \mu_s + 8 \cdot \mu_s^2$ $r_1 := \sqrt{L^2 + r^2} = 18.358\ ft$

$\beta_3 := -4 \cdot \mu_s \cdot (1 - 2 \cdot \mu_s)$ $r_2 := \sqrt{B^2 + r^2} = 18.358\ ft$

$\beta_4 := -1 + 4 \cdot \mu_s - 8 \cdot \mu_s^2$ $r_3 := \sqrt{L^2 + B^2 + r^2} = 24.352\ ft$

$\beta_5 := -4 \cdot (1 - 2 \cdot \mu_s)^2$ $r_4 := \sqrt{B^2 + L^2} = 22.627\ ft$

$$Y_1 := L \cdot \ln\left(\frac{r_4 + B}{L}\right) + B \cdot \ln\left(\frac{r_4 + L}{B}\right) - \frac{r_4^3 - L^3 - B^3}{3 \cdot L \cdot B} = 23.786\ ft$$

$$Y_2 := L \cdot \ln\left(\frac{r_3 + B}{r_1}\right) + B \cdot \ln\left(\frac{r_3 + L}{r_2}\right) - \frac{r_3^3 - r_2^3 - r_1^3 + r^3}{3 \cdot L \cdot B} = 21.562\ ft$$

$$Y_3 := \frac{r^2}{L} \cdot \ln\left(\frac{(r_2 + B) \cdot r_1}{(B + r_3) \cdot r}\right) + \frac{r^2}{B} \cdot \ln\left(\frac{(r_1 + L) \cdot r_2}{(L + r_3) \cdot r}\right) = 5.589\ ft$$

$$Y_4 := \frac{r^2 \cdot (r_1 + r_2 - r_3 - r)}{L \cdot B} = 1.064\ ft$$

$$Y_5 := r \cdot \operatorname{atan}\left(\frac{L \cdot B}{r \cdot r_3}\right) = 7.765\ ft$$

$$I_F := \frac{\beta_1 \cdot Y_1 + \beta_2 \cdot Y_2 + \beta_3 \cdot Y_3 + \beta_4 \cdot Y_4 + \beta_5 \cdot Y_5}{(\beta_1 + \beta_2) \cdot Y_1} = 0.861$$

SHAPE FACTOR CALCS

$$M := \frac{L'}{B'} \qquad N := \frac{H}{B'} \qquad I_1 := \frac{1}{\pi} \cdot \left(M \cdot \ln\left(\frac{\left(1 + \sqrt{M^2 + 1}\right) \cdot \sqrt{M^2 + N^2}}{M \cdot \left(1 + \sqrt{M^2 + N^2 + 1}\right)}\right) + \ln\left(\frac{\left(M + \sqrt{M^2 + 1}\right) \cdot \sqrt{1 + N^2}}{M + \sqrt{M^2 + N^2 + 1}}\right)\right) = 0.498$$

$$I_2 := \frac{N}{2\ \pi} \cdot \operatorname{atan}\left(\frac{M}{N \cdot \sqrt{M^2 + N^2 + 1}}\right) = 0.016 \qquad I_s := I_1 + \left(\frac{1 - 2\ \mu_s}{1 - \mu_s}\right) \cdot I_2 = 0.507$$

Footing Rigidity Factor, Ir $I_r := 0.93$

$$\delta_{im} := \alpha \cdot q_{net} \cdot B' \cdot \frac{\left(1 - \mu_s^2\right)}{E_s} \cdot I_s \cdot I_F \cdot I_r = 0.051\ in$$

Kiln 1

Brick is in good condition - only replace grates

Kiln 2

Brick is in good condition - only replace grates
remove protruding steel

Kiln 3

Brick is in good condition - only replace grates
remove protruding steel

Kiln 4

Brick is in poor condition - replace grate and section of bricks
remove protruding steel

	Hurstville Lime Kilns	Designer	Date
		SRO	02/22

Rock Crusher: Design Loads

Miscellaneous Info

Design Guide	=	ASCE 7-16	
Risk Category	=	II	

Dead Loads

Metal Roof	=	3	psf
Wood boards	=	8	psf
Roof Joists	=	5	psf
Total Roof DL	=	16	psf
Floor Joists	=	7	psf
Floor Boards	=	8	psf
Total 2nd Floor DL	=	15	psf
Wood boards	=	8	psf
Wood studs	=	2.5	psf
Miscellaneous	=	5	psf
Total Wall DL Building 1	=	15.5	psf
Metal Siding	=	3	psf
Wood boards	=	8	psf
Wood Studs	=	2	psf
Total Wall DL Building 2	=	13	psf

Live Loads

Roof LL	=	20	psf
2nd Floor LL	=	20	psf

Rock Crusher: Design Loads

Snow Loads

SL, ground	=	25	psf
Surface Roughness	=	C	
Exposure	=	Partially Exposed	
Ce	=	1	
Thermal Condition	=	Open air	
Ct	=	1.2	
Is	=	1	
SL, flat roof	=	0.7*Ce*Ct*Is*SL, ground	
SL, flat roof	=	21	psf

Building 1

Roof Slope	=	45	degrees
C,s	=	1	
SL, sloped roof	=	C,s*SL, flat roof	
SL, sloped roof	=	21	psf

Building 2

Roof Slope	=	42.51	degrees
C,s	=	1	
SL, sloped roof	=	C,s*SL, flat roof	
SL, sloped roof	=	21	psf

Walkway Roof

Roof Slope	=	45	degrees
C,s	=	1	
SL, sloped roof	=	C,s*SL, flat roof	
SL, sloped roof	=	21	psf

		Designer	Date
	Hurstville Lime Kilns	SRO	02/22

Rock Crusher: Design Loads cont.

To Old Rock Crusher Building

Roof Slope	=	75	degrees
C,s	=	0	
SL, sloped roof	=	C,s*SL, flat roof	
SL, sloped roof	=	0	psf

Building 1 Opening Roof

Roof Slope	=	22.26	degrees
C,s	=	1	
SL, sloped roof	=	C,s*SL, flat roof	
SL, sloped roof	=	21	psf

*No Unbalanced Snow Loads Applicable

Drift Snow Loads

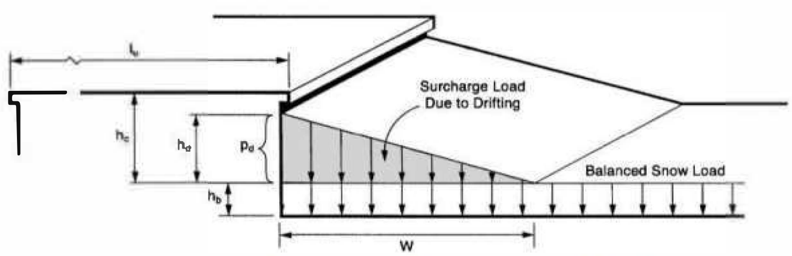


FIGURE 7.7-2 Configuration of Snowdrifts on Lower Roofs

l,u	=	30	ft
h,d/sqrt(I,s)	=	1.75	ft
I,s	=	1	ft
h,d	=	1.75	ft
y	=	17.25	pcf
p,d	=	31	psf
h,c	=	5	ft
w	=	2.45	ft

$$\frac{h_d}{\sqrt{I_s}} = \left(0.43\sqrt[3]{l_u}\sqrt[4]{p_g+10}\right) - 1.5$$

Sliding Snow Loads

W	=	3	ft
Max sliding snow load	=	26	psf

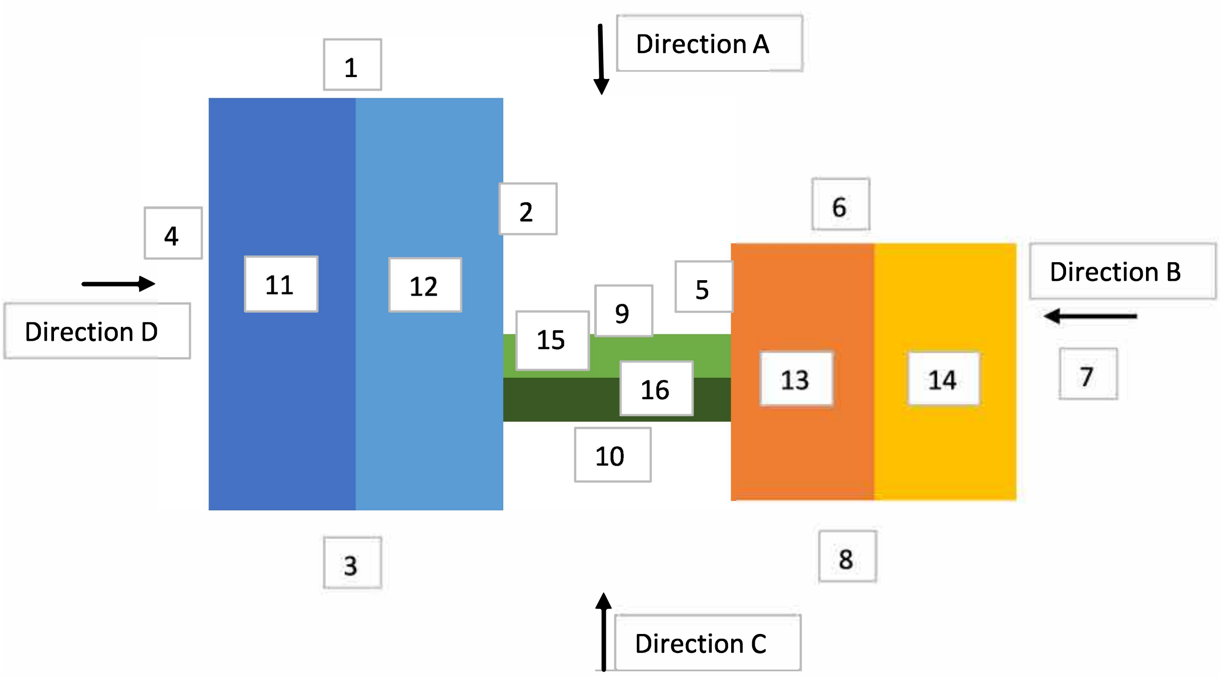
	Hurstville Lime Kilns	Designer	Date
		SRO	02/22

Rock Crusher: Design Loads cont.

Wind Loads

*Equations from ASCE 7-16, wind speed from ASCE 7-10

Risk Category	=	I	
Basic Wind Speed, V	=	105	mph
Directionality factor, K_d	=	0.85	
Exposure category	=	C	
Topographic factor, K_{zt}	=	1	
Elevation above sea level	=	668	ft
Ground elevation factor, K_e	=	1	
Gust effect factor, G	=	0.85	
Enclosure classification	=	Partially Enclosed	
building height, h	=	30	
K_z	=	0.98	
GC,pi	=	0.55	+/-
Velocity pressure, q_z	=	24	psf



Building 1 is in light blue and dark blue, Building 2 is in orange and yellow, and Connection Path is in greens

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		SRO	02/22

Rock Crusher: Design Loads cont.

Building 1 Wind Loads - MWFRS Direction A

*ASCE 7-16 27.3-1

p = q*GC,p-q,i*(GC,pi)

Roof C_p Interpolation

h/L, Directions A, C for Building 1 = 0.80 >0.5, <1.0

For roof angle of 45 degrees normal to ridge

	Windward	Leeward	
h/L	$-C_p$	$+C_p$	C_p
0.5	0.0	0.4	-0.6
≥ 1.0	0.0	0.3	-0.6
0.8	0.0	0.34	-0.6

Roof Pressure Coefficients, C_p, for use with q_h

		Windward								Leeward		
		Angle, θ (degrees)								Angle, θ (degrees)		
Wind Direction	h/L	10	15	20	25	30	35	45	≥60^c	10	15	≥20
Normal	≤0.25	−0.7	−0.5	−0.3	−0.2	−0.2	0.0^a					
to Ridge		−0.18	0.0^a	0.2	0.3	0.3	0.4	0.4	0.01 θ	−0.3	−0.5	−0.6
for	0.5	−0.9	−0.7	−0.4	−0.3	−0.2	−0.2	0.0^a				
θ ≥ 10°		−0.18	−0.18	0.0^a	0.2	0.2	0.3	0.4	0.01 θ	−0.5	−0.5	−0.6
	≥1.0	−1.3^b	−1.0	−0.7	−0.5	−0.3	−0.2	0.0^a				
		0.18	0.18	0.18	0.0^a	0.2	0.2	0.3	0.01 θ	0.7	0.6	0.6

Wind Direction	h/L	Horizontal Distance from Windward Edge	C_p
Normal to Ridge for θ < 10° and Parallel to Ridge for All θ	≤0.5	0 to $h/2$	−0.9, −0.18
		$h/2$ to h	−0.9, −0.18
		h to $2h$	−0.5, −0.18
		$>2h$	−0.3, −0.18
	≥1.0	0 to $h/2$	−1.3^b, −0.18
		$>h/2$	−0.7, −0.18

Wall Pressure Coefficients, C_p

Surface	L/B	C_p	Use With
Windward wall	All values	0.8	q_z
	0–1	−0.5	q_h
Leeward wall	2	−0.3	q_h
	≥4	−0.2	q_h
Sidewall	All values	−0.7	q_h

Rock Crusher: Design Loads cont.

Building 1 Wind Loads - MWFRS Direction A

*ASCE 7-16 27.3-1

h/L	=	0.80		
All loads	+/-	13.2	psf	
p, Walls				
L/B	=	0.43		
1	=	16.32	psf	WW
2	=	-14.28	psf	SW
3	=	-10.20	psf	LW
4	=	-14.28	psf	SW
p, Roof				
11	=	-23.26	psf	Parallel
12	=	-23.26	psf	Parallel

Building 1 Wind Loads - MWFRS Direction B

*ASCE 7-16 27.3-1

h/L	=	1.88		
All loads	+/-	13.2	psf	
p, Walls				
L/B	=	2.34		
1	=	-14.28	psf	SW
2	=	16.32	psf	WW
3	=	-14.28	psf	SW
4	=	-5.77	psf	LW
p, Roof				
11	=	-12.24	psf	LW
12	=	6.94	psf	WW

Rock Crusher: Design Loads cont.

Building 1 Wind Loads - MWFRS Direction C

*ASCE 7-16 27.3-1

h/L	=	0.80		
All loads	+/-	13.2	psf	
p, Walls				
L/B	=	0.43		
1	=	-10.20	psf	LW
2	=	-14.28	psf	SW
3	=	16.32	psf	WW
4	=	-14.28	psf	SW
p, Roof				
11	=	-23.26	psf	Parallel
12	=	-23.26	psf	Parallel

Building 1 Wind Loads - MWFRS Direction D

*ASCE 7-16 27.3-1

h/L	=	1.88		
All loads	+/-	13.2	psf	
p, Walls				
L/B	=	2.34		
1	=	-14.28	psf	SW
2	=	-5.77	psf	LW
3	=	-14.28	psf	SW
4	=	16.32	psf	WW
p, Roof				
11	=	6.94	psf	WW
12	=	-12.24	psf	LW

	Hurstville Lime Kilns	Designer	Date
		SRO	02/22

Rock Crusher: Design Loads cont.

Building 2 Wind Loads - MWFRS Direction A

*ASCE 7-16 27.3-1

h/L	=	1.94		
All loads	+/-	13.2	psf	
p, Walls				
L/B	=	1.16		
6	=	16.32	psf	WW
7	=	-14.28	psf	SW
8	=	-9.54	psf	LW
5	=	-14.28	psf	SW
p, Roof				
13	=	-26.52	psf	Parallel
14	=	-26.52	psf	Parallel

Building 2 Wind Loads - MWFRS Direction B

*ASCE 7-16 27.3-1

h/L	=	1.67		
All loads	+/-	13.2	psf	
p, Walls				
L/B	=	0.86		
6	=	-14.28	psf	SW
7	=	16.32	psf	WW
8	=	-14.28	psf	SW
5	=	-10.20	psf	LW
p, Roof				
13	=	-12.24	psf	LW
14	=	6.12	psf	WW

Rock Crusher: Design Loads cont.

Building 2 Wind Loads - MWFRS Direction C

*ASCE 7-16 27.3-1

h/L	=	1.94		
All loads	+/-	13.2	psf	
p, Walls				
L/B	=	1.16		
6	=	-9.54	psf	LW
7	=	-14.28	psf	SW
8	=	16.32	psf	WW
5	=	-14.28	psf	SW
p, Roof				
13	=	-26.52	psf	Parallel
14	=	-26.52	psf	Parallel

Building 2 Wind Loads - MWFRS Direction D

*ASCE 7-16 27.3-1

h/L	=	1.67		
All loads	+/-	13.2	psf	
p, Walls				
L/B	=	0.86		
6	=	-14.28	psf	SW
7	=	-10.20	psf	LW
8	=	-14.28	psf	SW
5	=	16.32	psf	WW
p, Roof				
13	=	6.12	psf	WW
14	=	-12.24	psf	LW

Rock Crusher: Design Loads cont.

Connection Path Wind Loads - MWFRS Direction A

*ASCE 7-16 27.3-1

h/L	=	1.50		
All loads	+/-	13.2	psf	
p, Walls				
L/B	=	0.53		
9	=	16.32	psf	WW
10	=	-10.20	psf	LW
p, Roof				
15	=	6.12	psf	WW
16	=	-12.24	psf	LW

Connection Path Wind Loads - MWFRS Direction B

*ASCE 7-16 27.3-1

h/L	=	0.80		
All loads	+/-	13.2	psf	
p, Roof				
15	=	-23.26	psf	Parallel
16	=	-23.26	psf	Parallel

Rock Crusher: Design Loads cont.

Connection Path Wind Loads - MWFRS Direction C
*ASCE 7-16 27.3-1

h/L	=	1.50		
All loads	+/-	13.2	psf	
p, Walls				
L/B	=	0.53		
9	=	-10.20	psf	LW
10	=	16.32	psf	WW
p, Roof				
15	=	-12.24	psf	LW
16	=	6.12	psf	WW

Connection Path Wind Loads - MWFRS Direction D
*ASCE 7-16 27.3-1

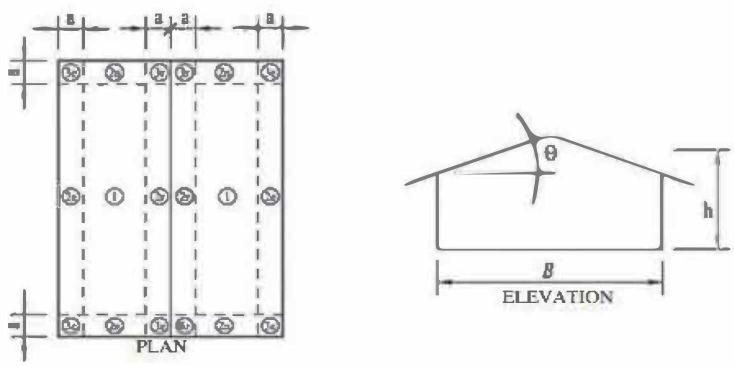
h/L	=	0.80		
All loads	+/-	13.2	psf	
p, Roof				
15	=	-23.26	psf	Parallel
16	=	-23.26	psf	Parallel

		Designer	Date
	Hurstville Lime Kilns	SRO	02/22

Kiln Platform: Design Loads cont.

Wind Loads - C&C Roof
*ASCE 7-16 Ch. 30, part 1

Velocity pressure, q_z = 24 psf
Gust effect factor, G = 0.85



GCp Roof (angle of 27 to 45 degrees)

Effective Wind Area	1	2e	2n	2r	3e	3r
1	-1.8	-1.8	-2	-1.8	-3.2	-2
10	-1.8	-1.8	-2	-1.8	-2.5	-2
20	-1.5	-1.5	-1.8	-1.5	-2.2	-1.8
50	-1.1	-1.1	-1.5	-1.1	-1.8	-1.5
100	-0.8	-0.8	-1.2	-0.8	-1.5	-1.2

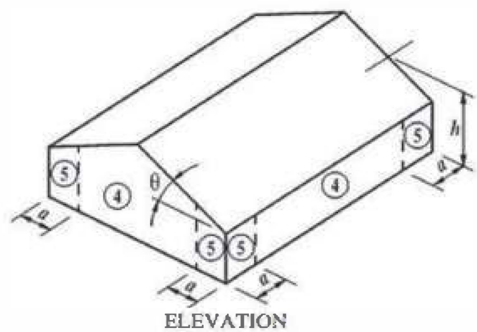
p (psf) Roof

Effective Wind Area	1	2e	2n	2r	3e	3r	
1	-43.20	-43.20	-48.00	-43.20	-76.80	-48.00	+/- 13.20
10	-43.20	-43.20	-48.00	-43.20	-60.00	-48.00	+/- 13.20
20	-36.00	-36.00	-43.20	-36.00	-52.80	-43.20	+/- 13.20
50	-26.40	-26.40	-36.00	-26.40	-43.20	-36.00	+/- 13.20
100	-19.20	-19.20	-28.80	-19.20	-36.00	-28.80	+/- 13.20

	Hurstville Lime Kilns	Designer	Date
		SRO	02/22

Kiln Platform: Design Loads cont.

Wind Loads - C&C Walls
*ASCE 7-16 Ch. 30, part 1

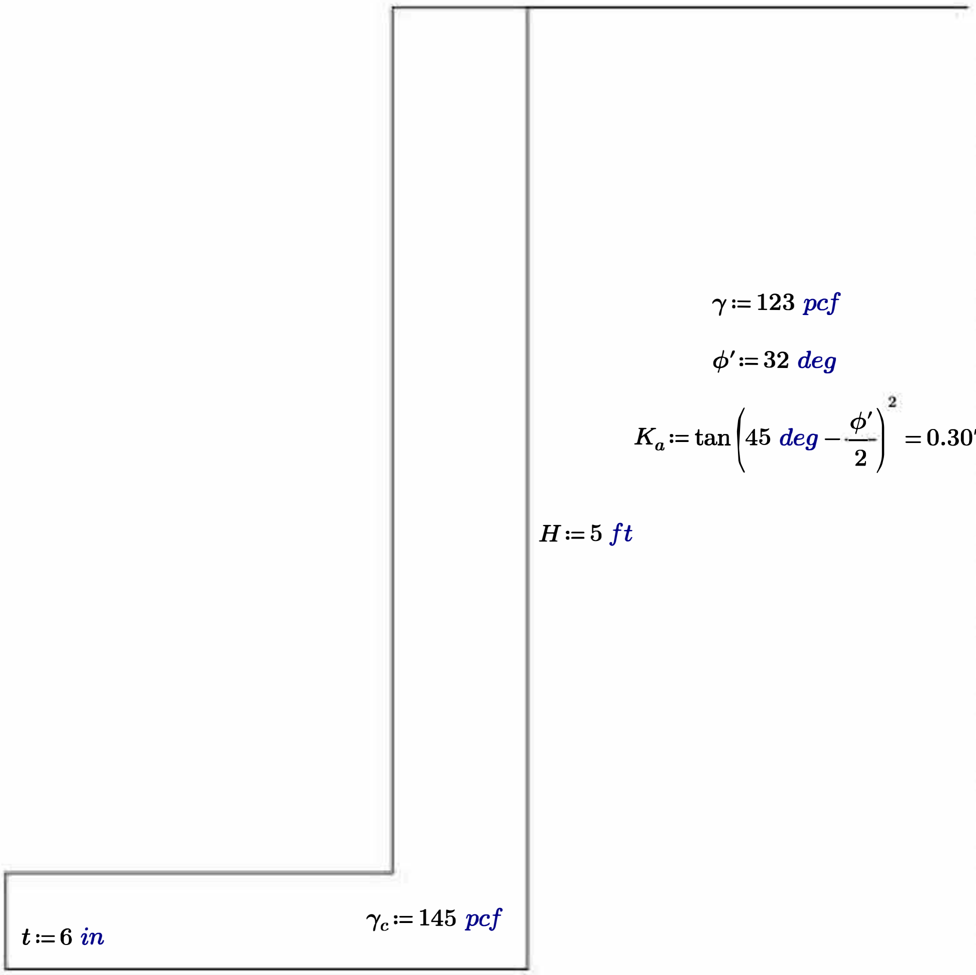


GCp Walls

Effective Wind Area	-4	-5	+ 4&5
1	-1.1	-1.4	1
10	-1.1	-1.4	1
20	-1.05	-1.3	0.9
50	-1	-1.15	0.85
100	-0.95	-1.5	0.8
200	-0.9	-0.95	0.8
500	-0.8	-0.8	0.7

p, Walls

Effective Wind Area	-4	-5	+ 4&5			
1	-26.40	-33.60	24.00	psf	+/-	13.20
10	-26.40	-33.60	24.00	psf	+/-	13.20
20	-25.20	-31.20	21.60	psf	+/-	13.20
50	-24.00	-27.60	20.40	psf	+/-	13.20
100	-22.80	-36.00	19.20	psf	+/-	13.20
200	-21.60	-22.80	19.20	psf	+/-	13.20
500	-19.20	-19.20	16.80	psf	+/-	13.20



$t_w := 8\ in$

$\gamma := 123\ pcf$

$\phi' := 32\ deg$

$K_a := \tan\left(45\ deg - \frac{\phi'}{2}\right)^2 = 0.307$

$H := 5\ ft$

$t := 6\ in$ $\qquad \gamma_c := 145\ pcf$

$b := 2\ ft + 8\ in$

$P_a := \frac{1}{2}\gamma \cdot K_a \cdot H^2 \cdot \cos(\phi')\ ft = 0.401\ kip$ per linear foot

overturning moment

$M_o := P_a \cdot \frac{H}{3} = 0.668\ kip \cdot ft$

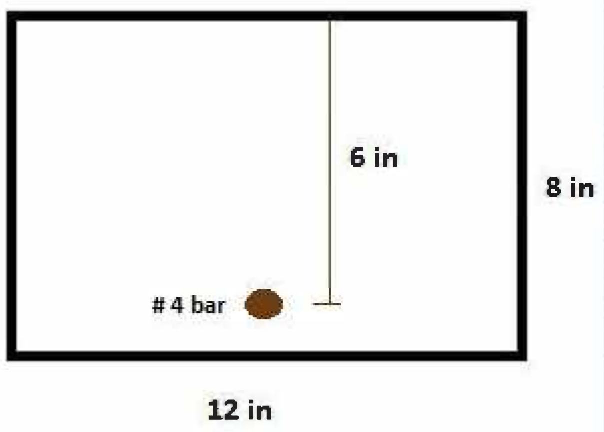
resisting moment

$W_{wall} := 1\ ft \cdot 8\ in \cdot 5\ ft \cdot \gamma_c = 0.483\ kip$
$W_{toe} := 1\ ft \cdot 1\ ft \cdot 6\ in \cdot \gamma_c = 0.073\ kip$ per linear foot

$M_r := W_{wall} \cdot (2\ ft + 4\ in) + W_{toe} \cdot 1\ ft = 1.2\ kip \cdot ft$

$FS_o := \frac{M_r}{M_o} = 1.798$ $\qquad FS_o > 1.5$

Check against shear



rebar spacing @ 1 ft

$\lambda := 1$

$f'_c := 4\ ksi$

$b := 12\ in$

$d := 6\ in$

$V := P_a = 0.401\ kip$

$$\tau_a := min\left(5 \cdot \lambda \cdot \sqrt{\frac{f'_c}{psi}} \cdot psi, 2 \cdot \lambda \cdot \sqrt{\frac{f'_c}{psi}} \cdot psi\right) = 126.491\ psi$$

$$V_c := \tau_a \cdot b \cdot d = 9.107\ kip$$

$$\phi V_c := 0.75 \cdot V_c = 6.831\ kip$$

$\phi V_c > V$ so this section is adequate for resisting shear

Check for flexural strength

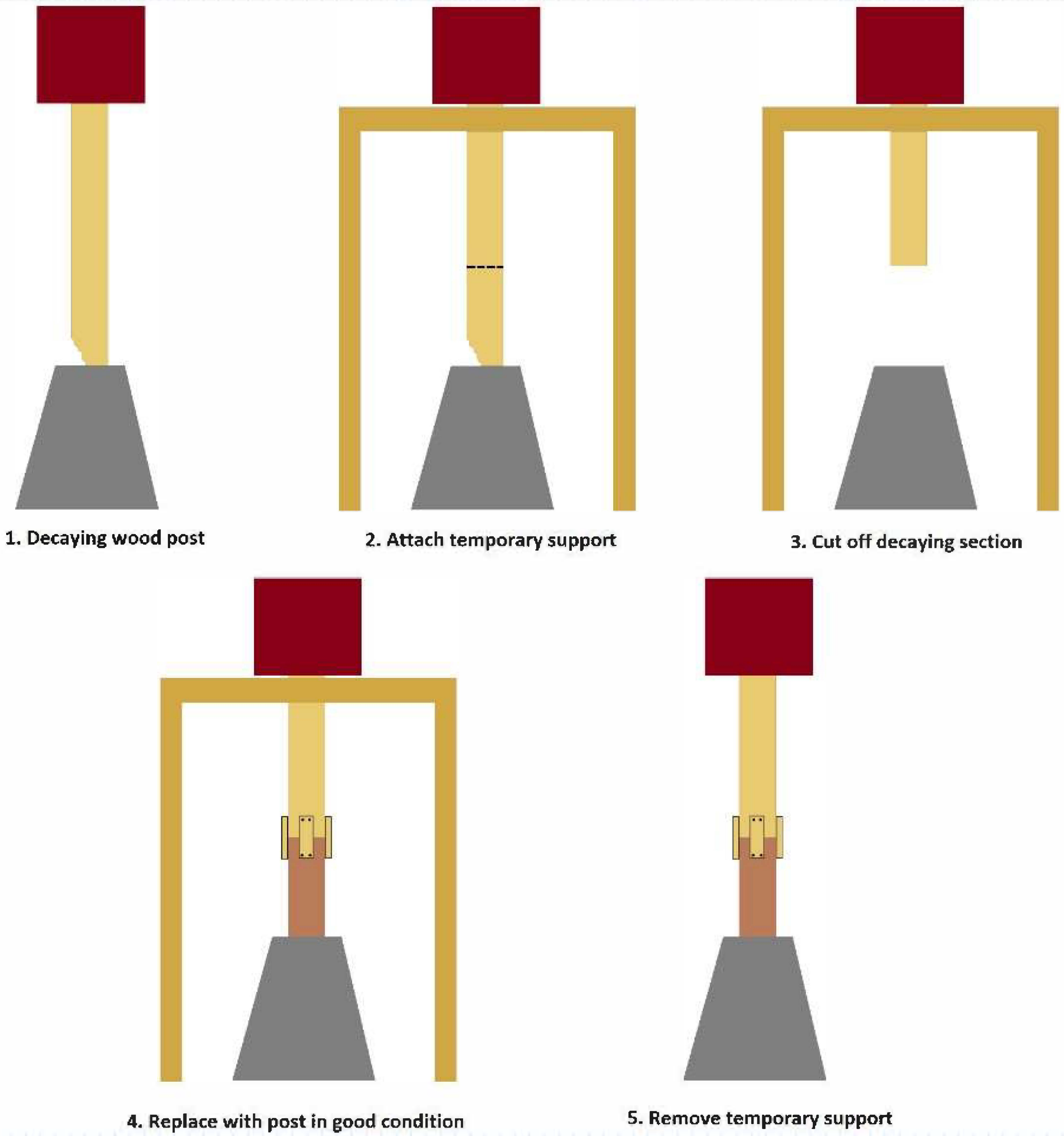
$$M_{max} := P_a \cdot \left(\frac{H}{3} - 6\ in\right) = 0.467\ kip \cdot ft \qquad A_s := 0.20\ in^2 \qquad f_y := 60\ ksi$$

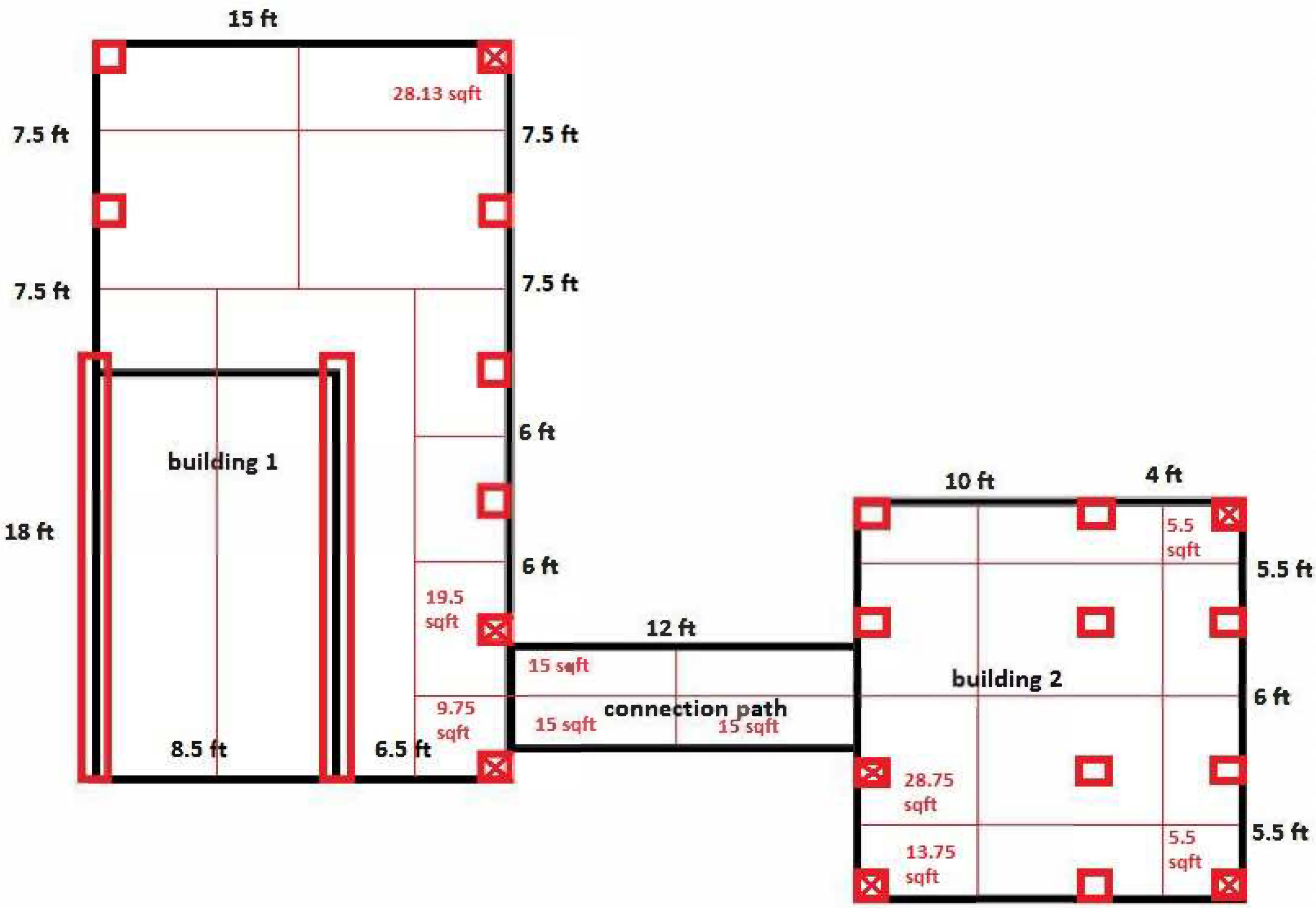
$$a := \frac{A_s \cdot f_y}{0.85 \cdot f'_c \cdot b} = 0.294\ in$$

$$\phi M_n := 0.9 \cdot A_s \cdot f_y \cdot \left(d - \frac{a}{2}\right) = 5.268\ kip \cdot ft$$

$\phi M_n > M_{max}$ so this section can resist the max moment

Temporary supports for columns





*From design loads spreadsheet

$DL_1 := 16\ psf + 15\ psf + 15.5\ psf = 46.5\ psf$

$DL_2 := 16\ psf + 15\ psf + 13\ psf = 44\ psf$

$LL := 40\ psf$

$SL := 21\ psf$

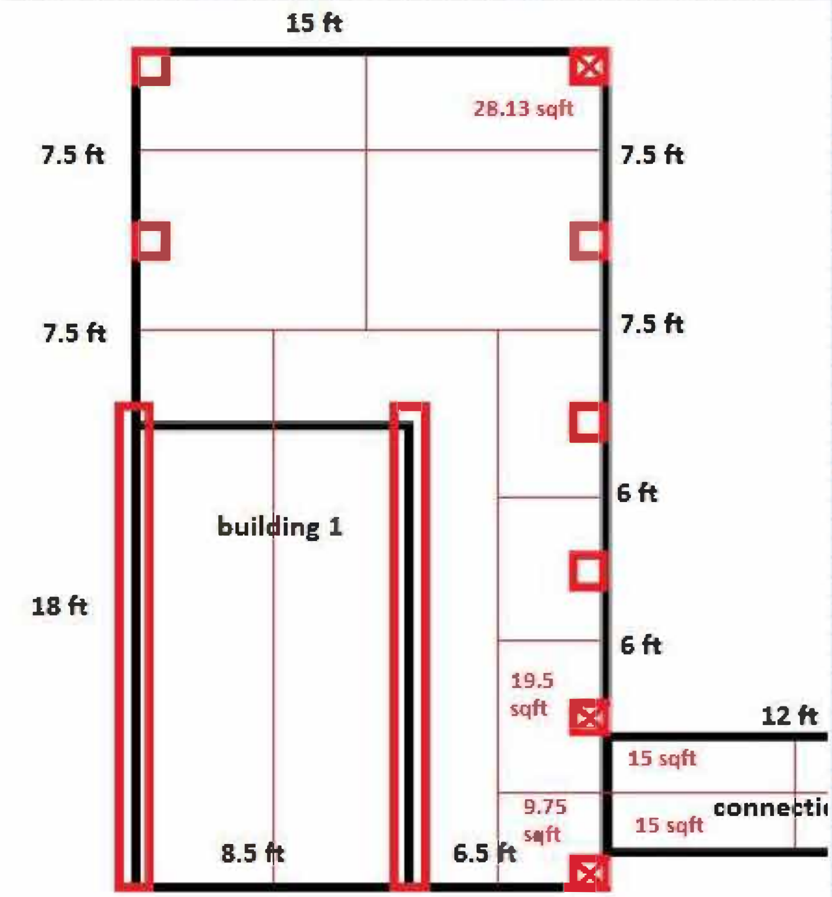
$WL_{pos} := 6.94\ psf$

$WL_{neg} := -76.8\ psf$

$Fa := 0\ psf$

$DL_{path} := 16\ psf$

Building 1



$C1 := 1.4 \cdot DL_1 = 65.1\ psf$

$C2 := 1.2\ DL_1 + 1.6\ LL + 0.3\ SL = 126.1\ psf$

$C3 := 1.2\ DL_1 + SL + LL = 116.8\ psf$

$C4a := 1.2\ DL_1 + 0.5\ WL_{pos} + Fa + LL + 0.3\ SL$

$C4a = 105.57\ psf$

$C4b := 1.2\ DL_1 + 0.5\ WL_{neg} + Fa + LL + 0.3\ SL$

$C4b = 63.7\ psf$

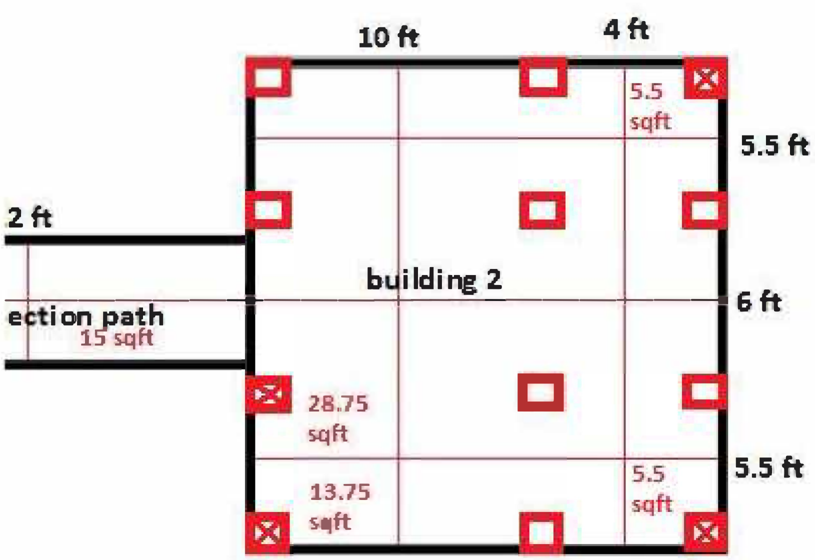
$C5 := 0.9\ DL_1 + 0.5\ WL_{neg} = 3.45\ psf$

$q := \max(C1, C2, C3, C4a, C4b, C5) = 126.1\ psf$

$q \cdot 28.13\ ft^2 = 3.547\ kip$

$q \cdot 19.5\ ft^2 + 1.4 \cdot (15\ ft^2 \cdot DL_{path}) = 2.795\ kip$

Building 2



$C1 := 1.4 \cdot DL_2 = 61.6\ psf$

$C2 := 1.2\ DL_2 + 1.6\ LL + 0.3\ SL = 123.1\ psf$

$C3 := 1.2\ DL_2 + SL + LL = 113.8\ psf$

$C4a := 1.2\ DL_2 + 0.5\ WL_{pos} + Fa + LL + 0.3\ SL$

$C4a = 102.57\ psf$

$C4b := 1.2\ DL_2 + 0.5\ WL_{neg} + Fa + LL + 0.3\ SL$

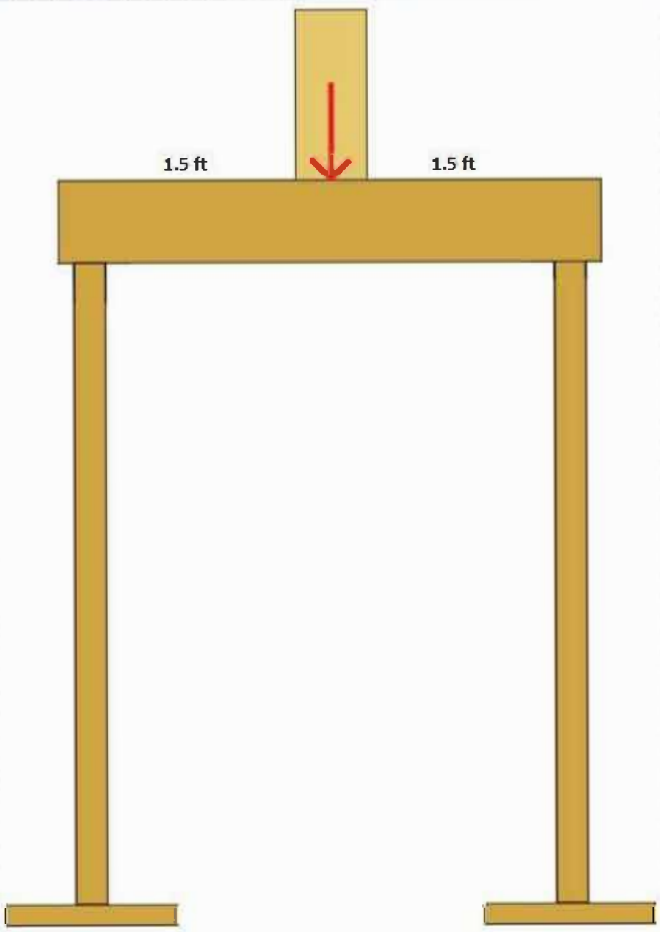
$C4b = 60.7\ psf$

$C5 := 0.9\ DL_2 + 0.5\ WL_{neg} = 1.2\ psf$

$q := \max(C1, C2, C3, C4a, C4b, C5) = 123.1\ psf$

$q \cdot 28.75\ ft^2 + 1.4 \cdot (15\ ft^2 \cdot DL_{path}) = 3.875\ kip$

$F := 4\ kip$



$V_{max} := \frac{F}{2} = 2\ kip$

$l_u := 3\ ft$

$l_e := 1.8\ l_u = 5.4\ ft$

$M_{max} := \frac{F \cdot l_u}{4} = 3\ kip \cdot ft$

For Douglas Fir-Larch No. 1:

$F_b := 1200\ psi$

$F_v := 180\ psi$

$E_{min} := 660\ ksi$

$F_c := 1550\ psi$

For a rectangular bending member of breadth, b, and depth, d, this becomes:

$$f_b = \frac{M}{S} = \frac{6M}{bd^2} \quad (3.3\text{-}2)$$

Flexural Design Equations - (NDS for Wood Construction)

1 2x8 board $\quad b := 1.5\ in \quad d := 7.25\ in \quad f_{b18} := \frac{6 \cdot M_{max}}{b \cdot d^2} = \left(2.74 \cdot 10^3\right)\ psi$

$R_{B18} := \sqrt{\frac{l_e \cdot d}{b^2}} = 14.45 \qquad F_{bE} := \frac{1.2 \cdot E_{min}}{{R_{B18}}^2} = \left(3.793 \cdot 10^3\right)\ psi$

1 2x10 board $\quad b := 1.5\ in \quad d := 9.25\ in \quad f_{b110} := \frac{6 \cdot M_{max}}{b \cdot d^2} = \left(1.683 \cdot 10^3\right)\ psi$

$R_{B110} := \sqrt{\frac{l_e \cdot d}{b^2}} = 16.322 \qquad F_{bE} := \frac{1.2 \cdot E_{min}}{{R_{B110}}^2} = \left(2.973 \cdot 10^3\right)\ psi$

2 2x8 boards $\quad b := 3\ in \quad d := 7.25\ in \quad f_{b28} := \frac{6 \cdot M_{max}}{b \cdot d^2} = \left(1.37 \cdot 10^3\right)\ psi$

$R_{B28} := \sqrt{\frac{l_e \cdot d}{b^2}} = 7.225 \qquad F_{bE} := \frac{1.2 \cdot E_{min}}{{R_{B28}}^2} = \left(1.517 \cdot 10^4\right)\ psi$

2 2x10 boards $\quad b := 3\ in \quad d := 9.25\ in \quad \boxed{f_{b210} := \frac{6 \cdot M_{max}}{b \cdot d^2} = 841.49\ psi} \quad < F_b = \left(1.2 \cdot 10^3\right)\ psi$ -> OK

$R_{B210} := \sqrt{\frac{l_e \cdot d}{b^2}} = 8.161 \qquad F_{bE} := \frac{1.2 \cdot E_{min}}{{R_{B210}}^2} = \left(1.189 \cdot 10^4\right)\ psi$

Shear Design Equations - (NDS for Wood Construction)

For a rectangular bending member of breadth, b, and depth, d, this becomes:

$$f_v = \frac{3V}{2bd} \quad (3.4\text{-}2)$$

1 2x8 board $b := 1.5\ in$ $d := 7.25\ in$ $f_v := \dfrac{3 \cdot V_{max}}{2 \cdot b \cdot d} = 275.862\ psi$

1 2x10 board $b := 1.5\ in$ $d := 9.25\ in$ $f_v := \dfrac{3 \cdot V_{max}}{2 \cdot b \cdot d} = 216.216\ psi$

2 2x8 boards $b := 3\ in$ $d := 7.25\ in$ $f_v := \dfrac{3 \cdot V_{max}}{2 \cdot b \cdot d} = 137.931\ psi$

2 2x10 boards $b := 3\ in$ $d := 9.25\ in$ $f_v := \dfrac{3 \cdot V_{max}}{2 \cdot b \cdot d} = 108.108\ psi$ < $F_v = 180\ psi$ -> OK

Axial load on columns $P := \dfrac{F}{2} = 2\ kip$ $L := 4\ ft$ $K := 2$ $l_e := K \cdot L = 8\ ft$ $F_c = 1.55\ ksi$

$$C_p = \frac{1 + (F_{cE}/F_c^*)}{2c} - \sqrt{\left[\frac{1 + (F_{cE}/F_c^*)}{2c}\right]^2 - \frac{F_{cE}/F_c^*}{c}} \quad (3.7\text{-}1)$$

$c := 0.9$

$d := 3.5\ in$

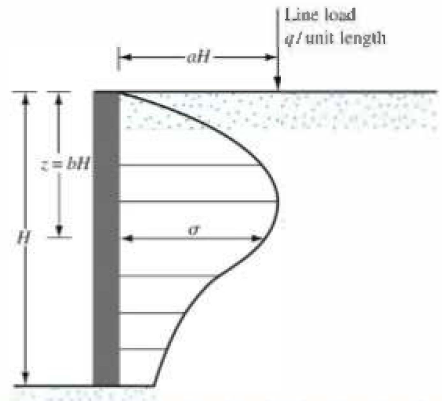
$\dfrac{l_e}{d} = 27.429$ slenderness doesn't exceed 50 -> OK

$$F_{cE} := \frac{0.822 \cdot E_{min}}{\left(\frac{l_e}{d}\right)^2} = 0.721\ ksi$$

$$C_p := \frac{1 + \left(\frac{F_{cE}}{F_c}\right)}{2\ c} - \sqrt{\left(\frac{1 + \left(\frac{F_{cE}}{F_c}\right)}{2\ c}\right)^2 - \frac{\frac{F_{cE}}{F_c}}{c}} = 0.432$$ column stability factor

$\dfrac{P}{d^2} = 163.265\ psi$ < $C_p \cdot F_c = 670.092\ psi$

Soil Stresses 234 Coduto



LATERAL EARTH PRESSURE
DUE TO SURCHARGE LOADS
Using Boussinesq

Lateral Earth Pressure due to a strip load

$w_c := 150\ pcf$ $w_{sb} := 135\ pcf$

$LL := 100\ psf$

$a' := 10\ ft$

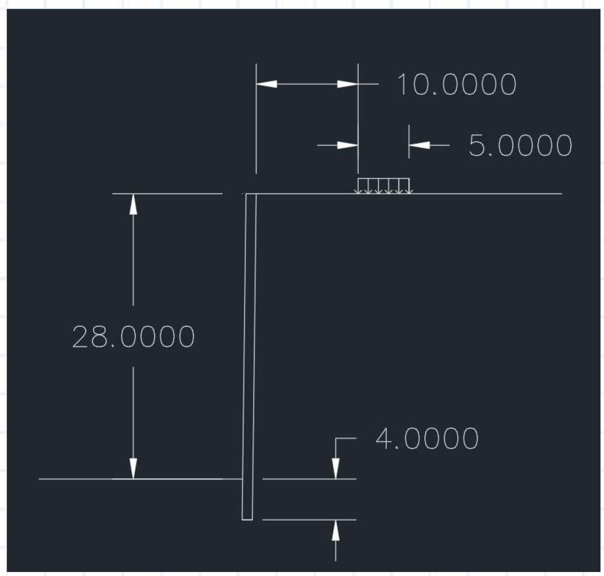
Lateral Earth Pressure due to a Strip Load

- Elastic Solution

$b' := 4\ ft$

$$\sigma_h(z) = \frac{q_s}{\pi}(\beta - \sin\beta\cos 2\alpha)$$

$t_c := 4\ in$ $t_{sb} := 6\ in$



$DL_c := t_c \cdot w_c + t_{sb} \cdot w_{sb} = 117.5\ psf$

$LL_c := LL = 100\ psf$

$q_s := 1.2 \cdot DL_c + 1.6 \cdot LL_c = 0.301\ ksf$

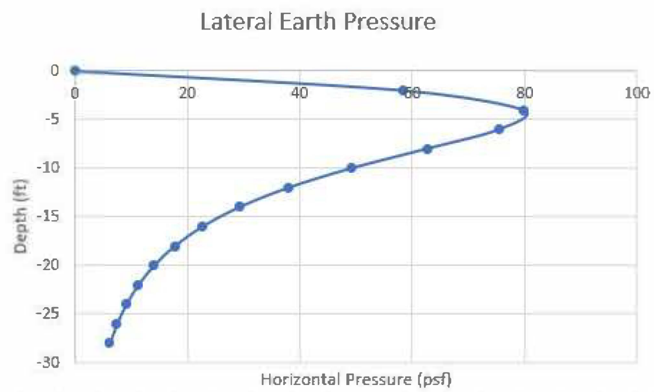
Using the modified form

$z := 28\ ft$

$\alpha := \operatorname{atan}\left(\dfrac{b' + 0.5 \cdot a'}{z}\right) = 0.311$ $\beta := \operatorname{atan}\left(\dfrac{a' + b'}{z}\right) - \operatorname{atan}\left(\dfrac{b'}{z}\right) = 0.322$

$$\sigma_h(z) := \frac{q_s}{\pi} \cdot (\beta - \sin(\beta) \cdot \cos(2 \cdot \alpha)) = 6.203\ psf$$

z (ft)	σh (psf)
0	0
-2	58.274
-4	79.766
-6	75.5
-8	62.568
-10	49.189
-12	37.976
-14	29.268
-16	22.697
-18	17.775
-20	14.079
-22	11.283
-24	9.145
-26	7.494
-28	6.203



z	n	Pi		zi	ziPi	
0	1	58.274		1.333333	77.69867	
2	2	138.04		3.051898	421.284	
4	3	155.266		4.990842	774.908	
6	4	138.068		6.968779	962.1653	
8	5	111.757		8.960095	1001.353	
10	6	87.165		10.95712	955.0773	
12	7	67.244		12.95683	871.2693	
14	8	51.965		14.95785	777.2847	
16	9	40.472		16.95946	686.3833	
18	10	31.854		18.96132	603.994	
20	11	25.362		20.96325	531.67	
22	12	20.428		22.96511	469.1313	
24	13	16.639		24.96693	415.4247	
26	14	13.697		26.96858	369.3887	
28						
sum		956.231	plf		8917.033	lb*ft/ft
zbar	9.325187	ft				

Wall

$$A_w := 32\ ft \cdot 1\ ft = 32\ ft^2$$

$$w_w := \gamma_{ls} \cdot A_w = 5.44\ klf$$

$$x_w := 12\ in \cdot \frac{1}{2} = 0.5\ ft$$

$$M_{rw} := w_w \cdot x_w = 2.72\ kip \cdot \frac{ft}{ft}$$

$$M_{ow} := \frac{8917.033\ kip \cdot \frac{ft}{ft}}{1000}$$

$$FS_{oc} := \frac{M_{rw}}{M_{ow}} = 0.305 \qquad FS_{oc} \geq FS_o = 0$$

Lateral Earth Pressure due to a Strip Load

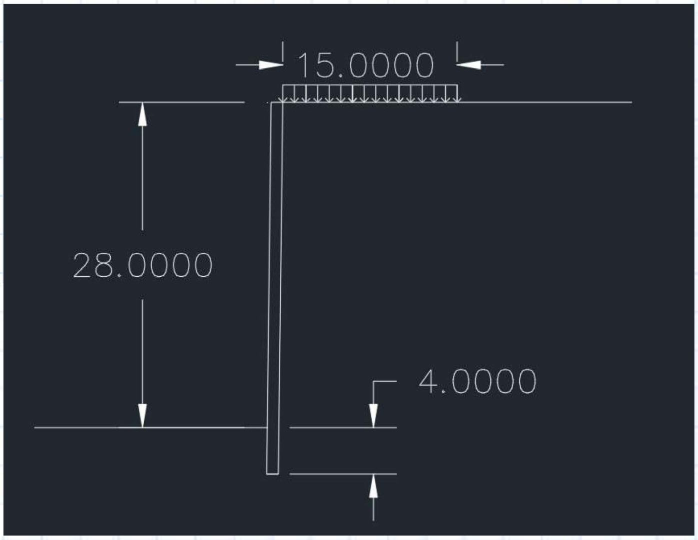
- Elastic Solution

$$\sigma_h(z) = \frac{q_s}{\pi}(\beta - \sin\beta\cos 2\alpha)$$

$DL_c := t_c \cdot w_c + t_{sb} \cdot w_{sb} = 0.118\ ksf$

$LL_c := LL = 0.1\ ksf$

$q_s := 1.2 \cdot DL_c + 1.6 \cdot LL_c = 0.301\ ksf$



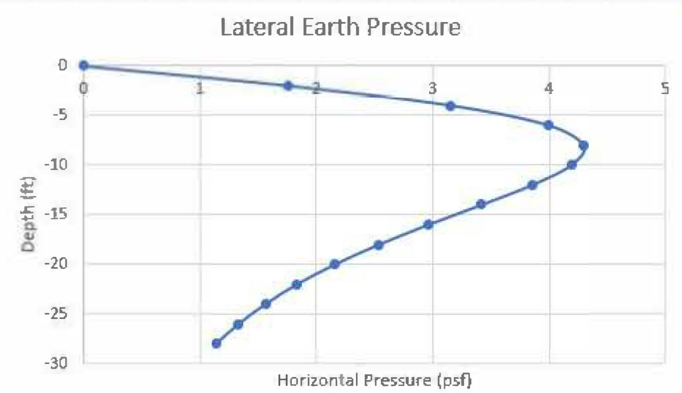
$z := 28\ ft$ $H := 28\ ft$

$a' := 1\ ft$ $b' := 14\ ft$

$$\alpha := \operatorname{atan}\left(\frac{b' + 0.5 \cdot a'}{z}\right) = 0.478 \qquad \beta := \operatorname{atan}\left(\frac{a' + b'}{z}\right) - \operatorname{atan}\left(\frac{b'}{z}\right) = 0.028$$

$$\sigma_h(z) := \frac{q_s}{\pi} \cdot (\beta - \sin(\beta) \cdot \cos(2 \cdot \alpha)) = 1.141\ psf$$

z (ft)	σh (psf)
0	0
-2	1.757
-4	3.151
-6	3.99
-8	4.288
-10	4.187
-12	3.854
-14	3.418
-16	2.966
-18	2.541
-20	2.164
-22	1.839
-24	1.564
-26	1.334
-28	1.141



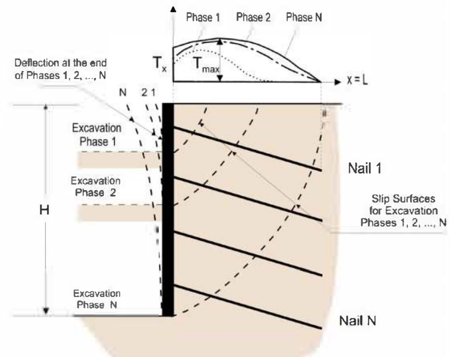
z	n	Pi		zi	ziPi	
0	1	1.757		1.333333	2.342667	
2	2	4.908		3.994675	15.18867	
4	3	7.141		5.039164	35.98467	
6	4	8.278		7.012	58.04533	
8	5	8.475		8.996028	76.24133	
10	6	8.041		10.9862	88.34	
12	7	7.272		12.98001	94.39067	
14	8	6.384		14.9764	95.60933	
16	9	5.507		16.97428	93.47733	
18	10	4.705		18.97329	89.26933	
20	11	4.003		20.97294	83.95467	
22	12	3.403		22.97306	78.17733	
24	13	2.898		24.97354	72.37333	
26	14	2.475		26.97401	66.76067	
28						
sum		75.247	plf		950.1553	lb*ft/ft

$$M_{ow} := \frac{950.1553\ kip \cdot \frac{ft}{ft}}{1000}$$

$$FS_{oc} := \frac{M_{rw}}{M_{ow}} = 2.863$$

$$FS_{oc} \geq FS_o = 1$$

Soil Nail

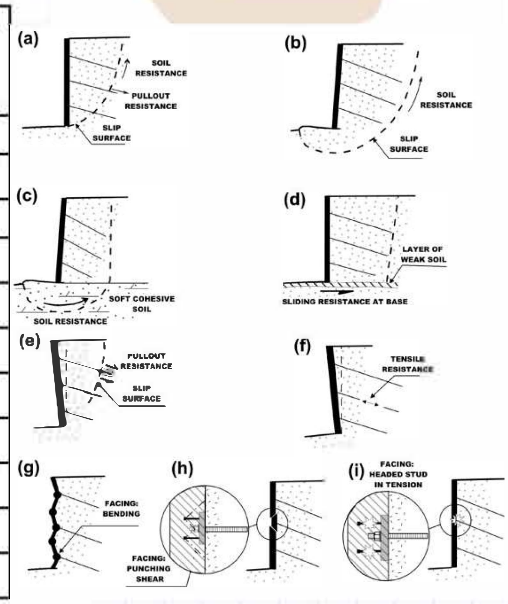


Borings drilled behind the wall should be spaced up to 150 ft along the alignment, be located within H to 1.5H behind the wall, and be advanced at least to a depth 2H below final grades. If the ground behind the wall slopes up, borings should be drilled within a horizontal distance of 1.5H to 2H from the wall. These borings should be deep enough to allow assessing potentially larger sliding masses occurring up the slope.

Borings drilled in front of the wall should be spaced up to 200 ft along the alignment, be located between 0.75H to H in front of the wall, and be advanced at least to a depth H below the planned bottom of the excavation.

Table 5.1: Minimum Recommended Factors of Safety for the Design of Soil Nail Walls Using the ASD Method (1)

Limit State	Condition	Symbol	Minimum Recomm. Factors of Safety, Static Loads	Minimum Recomm. Factors of Safety, Seismic Loads
Overall	Overall Stability	FS_{OS}	1.5(2)	1.1(6)
Overall	Short Term Condition, Excavation	FS_{OS}	1.25-1.33(3)	NA
Overall	Basal Heave	FS_{BH}	2.0(4), 2.5(5)	2.3(5)
Strength – Geotechnical	Pullout Resistance	FS_{PO}	2.0	1.5
Strength – Geotechnical	Lateral Sliding	FS_{LS}	1.5	1.1
Strength – Structural	Tendon Tensile Strength (Grades 60 and 75)	FS_T	1.8	1.35
Strength – Structural	Tendon Tensile Strength (Grades 95 and 150)	FS_T	2.0	1.50
Strength – Structural	Facing Flexural	FS_{FF}	1.5	1.1
Strength – Structural	Facing Punching Shear	FS_{FP}	1.5	1.1
Strength – Structural	Headed Stud Tensile (A307 Bolt)	FS_{FH}	2.0	1.5
Strength – Structural	Headed Stud Tensile (A325 Bolt)	FS_{FH}	1.7	1.3



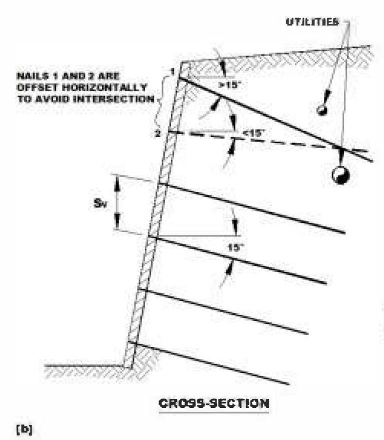
- Upper half of the wall: Soil nails whose heads are in this zone should have a uniform length, L.

- Lower half of the wall: Soil nails whose heads are in this zone should be increasingly shorter toward the bottom. The lengths of these nails must be determined by linear interpolation from value L at the wall mid height, to R × L at the base of the wall. R is < 1.0 and is selected depending on subsurface and geometric conditions and other factors, as indicated below.

  - For very dense, coarse-grained granular soils: $0.15 \le R \le 0.30$
  - For silty sand, sand, to gravelly sand: $0.25 \le R \le 0.40$
  - For fined-grained soils: $0.30 \le R \le 0.45$

  R has been estimated for the following conditions: safety factor for pullout $FS_{PO} = 2.0$, drill hole diameter (D_{DH}) between 4 to 8 in., horizontal and vertical nail spacing (S_H and S_V) between 4 and 6 ft, and typical ranges of bond strengths (q_U) for the soil types listed above.

  In addition, the following ranges of soil properties were considered to be consistent with the listed soil types: soil unit weight of retained soils (γ_s) between 110 and 130 pcf, and ratio of maximum soil nail length to wall height (L/H) between 0.75 and 1.0. In general, larger values of D_{DH} and q_U, in conjunction with lower values of S_H, S_V, and γ_s, would produce lower values of R.



6.3.4 Step 4c Select Soil Nail Pattern on Wall Face

Soil nails are installed on the excavation face in "square" or, more commonly, "staggered" (also referred to as triangular or offset) patterns (Figure 6.1). The pattern of nails on the excavation face can become irregular at locations with space restrictions.

In the square pattern, nails are vertically aligned in rows. This pattern allows the easy construction of vertical joints in shotcrete and an easier installation of precast concrete panels (if used). Drain strips are equidistant from nails in this pattern. A staggered pattern results in more uniform earth-pressure distributions, better soil arching effects, and provides a slightly larger resistance compared to those from a square pattern.

6.3.5 Step 4d Evaluate Soil Nail Horizontal Splaying

Nails may need to be splayed on plan view to: (i) avoid manholes and other obstructions, (ii) avoid external corners due to interference with adjacent nails (Figure 6.2c); or (iii) to possibly improve stability at internal corners. The engineer must consider nail splaying before using a design computer program because these programs do not account for the splay angle.

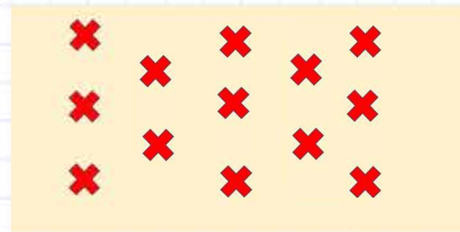
6.3.6 Step 4e Detail Corrosion Protection

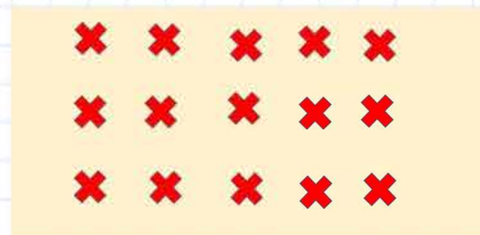
The designer must select the corrosion protection technique or techniques that meet the level of corrosion protection established during the Initial Design Considerations phase. This selection involves specifying a material or process that is suitable for the nail type and installation procedures. Guidelines for selection of corrosion protection materials are provided in Chapters 7 and 10.

6.3.7 Step 4f Select Soil Nail Type and Material Properties

The engineer must select a grade of steel for the soil nail bar and other metallic parts. Information on steel grades and sizes is presented in Chapter 3 and Appendix A.

In traditional Design/Bid/Build contracts, the engineer may estimate a practical minimum drill hole diameter to provide the bond resistance required for stability. However, the drill hole diameter is ultimately selected by the Contractor to obtain the specified, nominal pullout resistance, and to possibly allow cleaning the drill hole, or accommodating a tremie pipe, tendon couplers, and centralizers.





Install at 15 degrees

Soil nails are installed at 10 to 20 degrees from the horizontal, and most commonly at 15 degrees. The grout can flow at these inclinations from the bottom of the drill hole to the head. Grout generally can fill the hole without leaving air pockets for typical drill-hole dimensions and grout mixes.

Pullout Resistance P 161 (201)

6.6.2 Step 7a Verify Pullout Resistance

Pullout resistance is mobilized behind the slip surface, along the length, L_p, and contributes to overall stability. The length, L_P, can be estimated from the graphical output of soil nail design programs, where critical slip surfaces and soil nails are shown to an appropriate scal
The nominal (i.e., ultimate) pullout resistance per unit length, r_{PO}, is expressed as:

$$r_{PO} = \pi \, q_u \, D_{DH}$$

Equation 6.1: Nominal unit pullout resistance.

Where:

q_u = bond strength of the nail-grout-soil interface (force/unit area)
D_{DH} = diameter of the drill hole

Distributions of bond stresses along the grout-soil interface can be complex and exhibit variations along L_p (Figure 6.4).

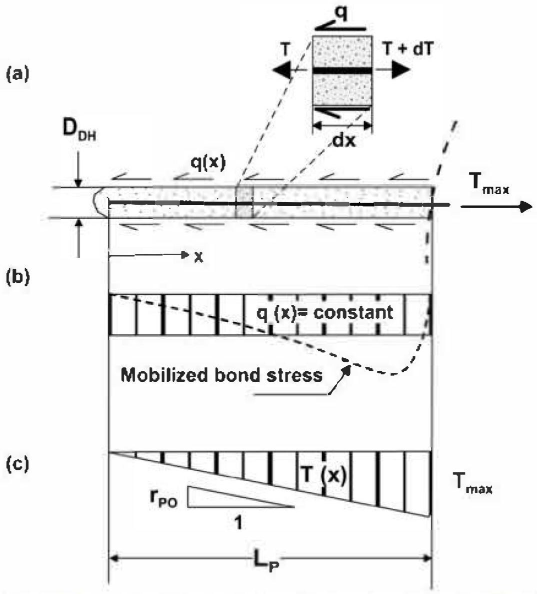


Figure 6.4: Illustration. Single nail stress-transfer mode: (a) soil nail layout, (b) distribution of mobilized bond stresses, and (c) hypothetical distribution of loads along the nail.

The distribution is assumed to be constant along the nail pullout length for simplicity, and the bond stress is considered to have an apparent, average value. When the bond stress increases to its maximum value, the bond strength, q_u, is mobilized.

The nominal pullout resistance, R_{PO}, is calculated as follows:

$$R_{PO} = r_{PO}\, L_P$$

Equation 6.2: Nominal pullout resistance.

Pullout resistance is evaluated as follows:

$$CDR = \frac{\phi_{PO}\ R_{PO}}{\gamma\ T_{max}} \geq 1.0$$

Equation 6.3: Capacity-to-demand ratio (CDR).

Chosen Soil Nail

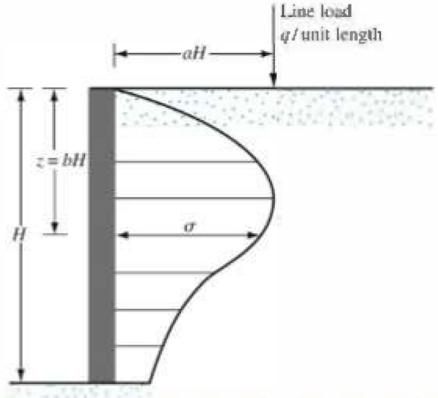
$f_y := 75\ ksi$ $\quad A_b := 0.44\ in^2$ $\quad \sigma' := 66.251\ psf$ @ 4ft

BAR DESIGNATION NOMINAL DIAMETER & PITCH	MINIMUM NET AREA THRU THREADS	MINIMUM ULTIMATE STRENGTH	GRADE 75 MINIMUM YIELD STRENGTH	GRADE 80 MINIMUM YIELD STRENGTH	NOMINAL WEIGHT	APPROXIMATE THREAD MAJOR DIAMETER	PART NUMBER
#6 – 3/4" – 5 (19 mm)	0.44 in² (284 mm²)	44 kips (196 kN)	33 kips (147 kN)	35 kips (156 kN)	1.5 lbs/ft (2.4 kg/m)	7/8" (22 mm)	R61-06

$S_v := 10\ ft$ $\quad S_H := 15\ ft$ $\quad z := 2\ ft$

Soil Stresses 234 Coduto

LATERAL EARTH PRESSURE
DUE TO SURCHARGE LOADS
Using Boussinesq



Lateral Earth Pressure due to a strip load

$w_c := 150\ pcf$ $w_{sb} := 135\ pcf$

$LL := 100\ psf$ $FS_o := 1.5$

$a' := 10\ ft$

$\gamma_{ls} := 170\ pcf$

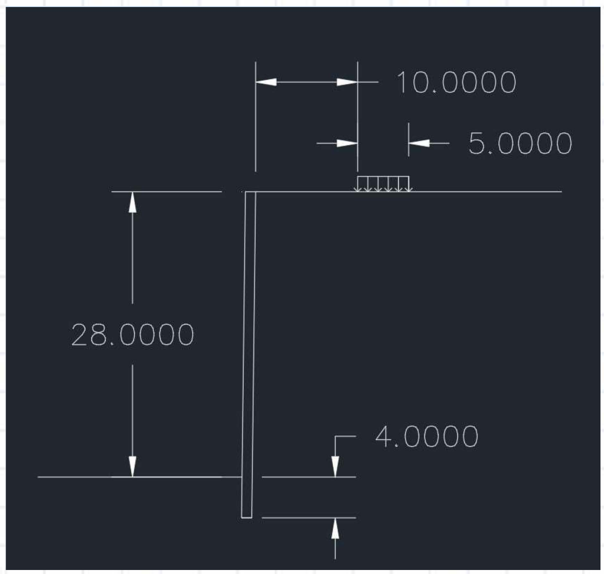
$b' := 4\ ft$

$t_c := 4\ in$ $t_{sb} := 6\ in$

Lateral Earth Pressure due to a Strip Load

- Elastic Solution

$$\sigma_h(z) = \frac{q_s}{\pi}(\beta - \sin\beta\cos 2\alpha)$$



$DL_c := t_c \cdot w_c + t_{sb} \cdot w_{sb} = 117.5\ psf$

$LL_c := LL = 100\ psf$

$q_s := 1.2 \cdot DL_c + 1.6 \cdot LL_c = 0.301\ ksf$

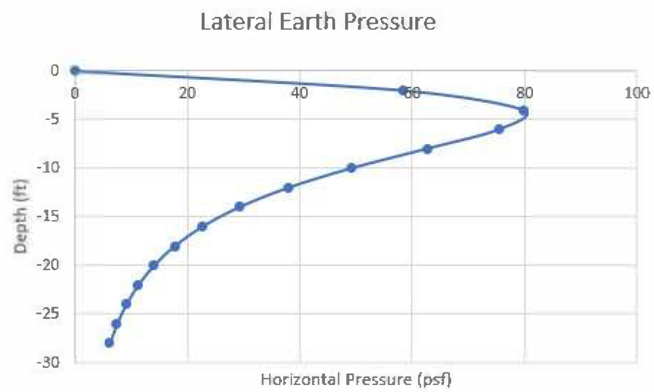
Using the modified form

$z := 28\ ft$

$$\alpha := \operatorname{atan}\left(\frac{b' + 0.5 \cdot a'}{z}\right) = 0.311 \qquad \beta := \operatorname{atan}\left(\frac{a' + b'}{z}\right) - \operatorname{atan}\left(\frac{b'}{z}\right) = 0.322$$

$$\sigma_h(z) := \frac{q_s}{\pi} \cdot (\beta - \sin(\beta) \cdot \cos(2 \cdot \alpha)) = 6.203\ psf$$

z (ft)	σh (psf)
0	0
-2	58.274
-4	79.766
-6	75.5
-8	62.568
-10	49.189
-12	37.976
-14	29.268
-16	22.697
-18	17.775
-20	14.079
-22	11.283
-24	9.145
-26	7.494
-28	6.203



z	n	Pi		zi	ziPi	
0	1	58.274		1.333333	77.69867	
2	2	138.04		3.051898	421.284	
4	3	155.266		4.990842	774.908	
6	4	138.068		6.968779	962.1653	
8	5	111.757		8.960095	1001.353	
10	6	87.165		10.95712	955.0773	
12	7	67.244		12.95683	871.2693	
14	8	51.965		14.95785	777.2847	
16	9	40.472		16.95946	686.3833	
18	10	31.854		18.96132	603.994	
20	11	25.362		20.96325	531.67	
22	12	20.428		22.96511	469.1313	
24	13	16.639		24.96693	415.4247	
26	14	13.697		26.96858	369.3887	
28						
sum		956.231	plf		8917.033	lb*ft/ft
zbar	9.325187	ft				

Wall

$$A_w := 32\ ft \cdot 1\ ft = 2.97289728\ m^2$$

$$w_w := \gamma_{ls} \cdot A_w = 5.44\ klf$$

$$x_w := 12\ in \cdot \frac{1}{2} = 0.152\ m$$

$$M_{rw} := w_w \cdot x_w = 2.72\ kip \cdot \frac{ft}{ft}$$

$$M_{ow} := \frac{8917.033\ kip \cdot \frac{ft}{ft}}{1000}$$

$$FS_{oc} := \frac{M_{rw}}{M_{ow}} = 0.305 \qquad FS_{oc} \geq FS_o = 0$$

Lateral Earth Pressure due to a Strip Load

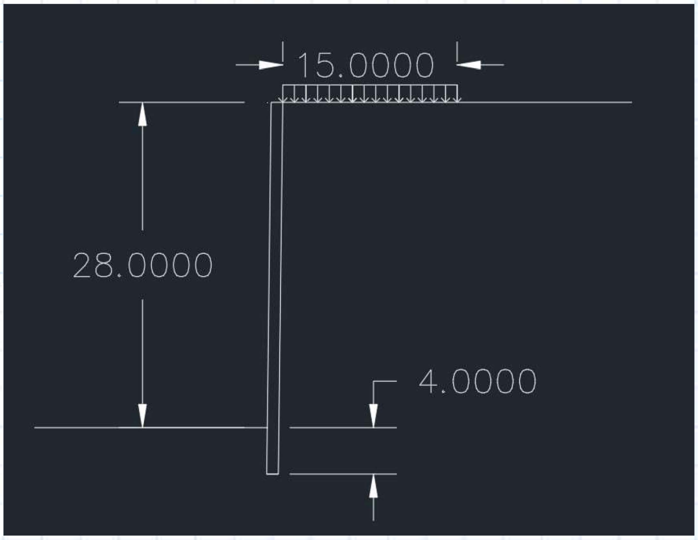
- Elastic Solution

$$\sigma_h(z) = \frac{q_s}{\pi}(\beta - \sin\beta\cos 2\alpha)$$

$DL_c := t_c \cdot w_c + t_{sb} \cdot w_{sb} = 0.118\ ksf$

$LL_c := LL = 0.1\ ksf$

$q_s := 1.2 \cdot DL_c + 1.6 \cdot LL_c = 0.301\ ksf$



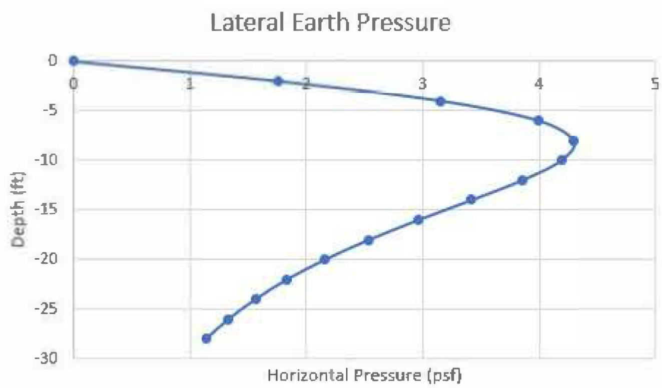
$z := 28\ ft$ $\quad H := 28\ ft$

$a' := 1\ ft$ $\quad b' := 14\ ft$

$\alpha := \operatorname{atan}\left(\frac{b' + 0.5 \cdot a'}{z}\right) = 0.478$ $\quad \beta := \operatorname{atan}\left(\frac{a' + b'}{z}\right) - \operatorname{atan}\left(\frac{b'}{z}\right) = 0.028$

$\sigma_h(z) := \frac{q_s}{\pi} \cdot (\beta - \sin(\beta) \cdot \cos(2 \cdot \alpha)) = 1.141\ psf$

z (ft)	σh (psf)
0	0
-2	1.757
-4	3.151
-6	3.99
-8	4.288
-10	4.187
-12	3.854
-14	3.418
-16	2.966
-18	2.541
-20	2.164
-22	1.839
-24	1.564
-26	1.334
-28	1.141



z	n	Pi		zi	ziPi	
0	1	1.757		1.333333	2.342667	
2	2	4.908		3.094675	15.18867	
4	3	7.141		5.039164	35.98467	
6	4	8.278		7.012	58.04533	
8	5	8.475		8.996028	76.24133	
10	6	8.041		10.9862	88.34	
12	7	7.272		12.98001	94.39067	
14	8	6.384		14.9764	95.60933	
16	9	5.507		16.97428	93.47733	
18	10	4.705		18.97329	89.26933	
20	11	4.003		20.97294	83.95467	
22	12	3.403		22.97306	78.17733	
24	13	2.898		24.97354	72.37333	
26	14	2.475		26.97401	66.76067	
28						
sum		75.247	plf		950.1553	lb*ft/ft

$$M_{ow} := \frac{950.1553\ kip \cdot \frac{ft}{ft}}{1000}$$

$$FS_{oc} := \frac{M_{rw}}{M_{ow}} = 2.863$$

$$FS_{oc} \geq FS_o = 1$$

$FS_P := 1.5 \quad \gamma_1 := 123\ pcf \quad z := 4\ ft \quad S_v := 5\ ft \quad w := 1.25\ in$

$c_a := 130\ psf \quad d := 7\ in \quad \phi' := 32\ deg \quad S_H := 5\ ft \quad \phi'_u := 20\ deg$

$\sigma' := 79.766\ psf \quad H := 28\ ft$

$$l_e := \frac{FS_P \cdot \sigma' \cdot S_v \cdot S_H}{\pi \cdot d \cdot c_a} = 3.827\ m$$

$$l_r := \frac{H - z}{\tan\left(45\ deg + \frac{\phi'}{2}\right)} = 4.055\ m$$

$$FS_P \cdot \sigma' \cdot S_v \cdot S_H = \left(1.331 \cdot 10^4\right)\ N$$

$$L := l_e + l_r = 7.882\ m$$

$$2 \cdot w \cdot \left(\gamma_1 \cdot z\right) \cdot \tan\left(\phi'_u\right) = 544.454\ \frac{kg}{s^2}$$

Because the wall is about 12 in thick, an additional foot should be added to the rods to sustain the correct length for embankment

Soil Nail Specifics

CORROSION PROTECTION TYPE	ABRASION RESISTANCE (4=BEST)	TYPICAL THICKNESS	RELATIVE COST (4=HIGHEST)	PRODUCTION LEAD TIME	CAN BE APPLIED TO ACCESSORIES?	APPLIED IN THE FIELD?
Hot Dip Galvanizing	4	3-4 mils	2	2-4 weeks	yes	no

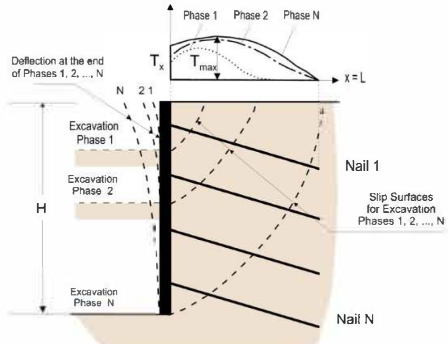
#10 – 1-1/4" – 3 (32 mm)	1.27 in^2 (819 mm^2)	127 kips (565 kN)	95 kips (424 kN)	102 kips (454 kN)	4.3 lbs/ft (5.5 kg/m)	1-3/8" (35 mm)	R61-10

#10 - 1-1/4" (32 mm)	2" (51 mm)	2.31" (59 mm)	2" (51 mm)	R63-10

#10 - 1-1/4" (32 mm)	2-1/2" (64 mm)	1-3/8" (35 mm)	5/32" (4 mm)	R9F-10-436

Depth (ft)	Pressure from trail (psf)	le (ft)	lr (ft)	L (ft)	FSp	L actual (ft)	Rod #	Diameter (in)	Rod Unit	Washer Unit	Hex Nut Unit
4	79.766	12.556	13.303	25.859	1.5	27	10	1.25	R61-10	R9F-10-436	R63-10
9	55.8785	8.796	10.532	19.327	1.5	27	10	1.25	R61-10	R9F-10-436	R63-10
14	29.268	4.607	7.76	12.367	1.5	27	10	1.25	R61-10	R9F-10-436	R63-10

Soil Nail



Borings drilled behind the wall should be spaced up to 150 ft along the alignment, be located within H to 1.5H behind the wall, and be advanced at least to a depth 2H below final grades. If the ground behind the wall slopes up, borings should be drilled within a horizontal distance of 1.5H to 2H from the wall. These borings should be deep enough to allow assessing potentially larger sliding masses occurring up the slope.

Borings drilled in front of the wall should be spaced up to 200 ft along the alignment, be located between 0.75H to H in front of the wall, and be advanced at least to a depth H below the planned bottom of the excavation.

Table 5.1: Minimum Recommended Factors of Safety for the Design of Soil Nail Walls Using the ASD Method (1)

Limit State	Condition	Symbol	Minimum Recomm. Factors of Safety, Static Loads	Minimum Recomm. Factors of Safety, Seismic Loads
Overall	Overall Stability	FS_{OS}	$1.5^{(2)}$	$1.1^{(6)}$
Overall	Short Term Condition, Excavation	FS_{OS}	$1.25\text{-}1.33^{(3)}$	NA
Overall	Basal Heave	FS_{BH}	$2.0^{(4)}$, $2.5^{(5)}$	$2.3^{(5)}$
Strength – Geotechnical	Pullout Resistance	FS_{PO}	2.0	1.5
Strength – Geotechnical	Lateral Sliding	FS_{LS}	1.5	1.1
Strength – Structural	Tendon Tensile Strength (Grades 60 and 75)	FS_T	1.8	1.35
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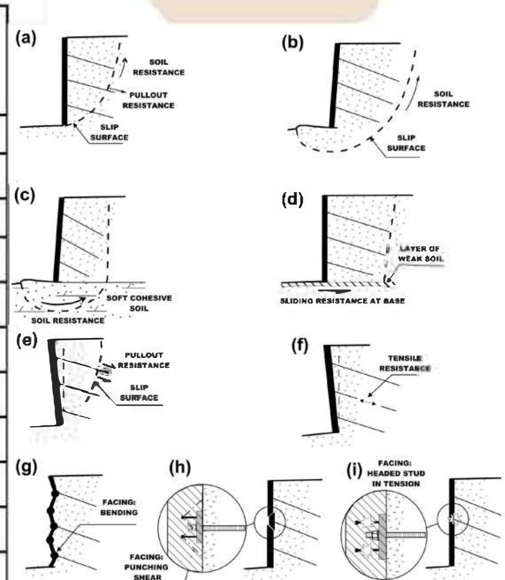


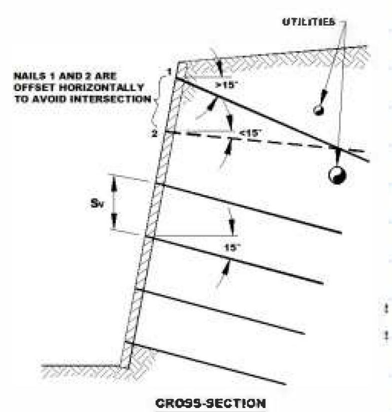
- Upper half of the wall: Soil nails whose heads are in this zone should have a uniform length, L.

- Lower half of the wall: Soil nails whose heads are in this zone should be increasingly shorter toward the bottom. The lengths of these nails must be determined by linear interpolation from value L at the wall mid height, to R × L at the base of the wall. R is < 1.0 and is selected depending on subsurface and geometric conditions and other factors, as indicated below.

  - For very dense, coarse-grained granular soils: $0.15 \leq R \leq 0.30$
  - For silty sand, sand, to gravelly sand: $0.25 \leq R \leq 0.40$
  - For fined-grained soils: $0.30 \leq R \leq 0.45$

  R has been estimated for the following conditions: safety factor for pullout $FS_{PO} = 2.0$, drill hole diameter (D_{DH}) between 4 to 8 in., horizontal and vertical nail spacing (S_H and S_V) between 4 and 6 ft, and typical ranges of bond strengths (q_U) for the soil types listed above.

  In addition, the following ranges of soil properties were considered to be consistent with the listed soil types: soil unit weight of retained soils (γ_s) between 110 and 130 pcf, and ratio of maximum soil nail length to wall height (L/H) between 0.75 and 1.0. In general, larger values of D_{DH} and q_U, in conjunction with lower values of S_H, S_V, and γ_s, would produce lower values of R.

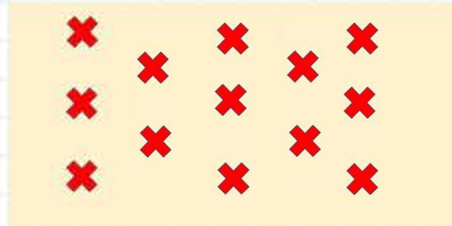


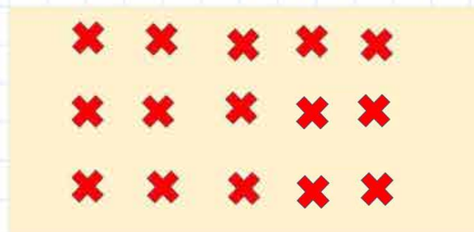
[b]

6.3.4 Step 4c Select Soil Nail Pattern on Wall Face

Soil nails are installed on the excavation face in "square" or, more commonly, "staggered" (also referred to as triangular or offset) patterns (Figure 6.1). The pattern of nails on the excavation face can become irregular at locations with space restrictions.

In the square pattern, nails are vertically aligned in rows. This pattern allows the easy construction of vertical joints in shotcrete and an easier installation of precast concrete panels (if used). Drain strips are equidistant from nails in this pattern. A staggered pattern results in more uniform earth-pressure distributions, better soil arching effects, and provides a slightly larger resistance compared to those from a square pattern.





6.3.5 Step 4d Evaluate Soil Nail Horizontal Splaying

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6.3.6 Step 4e Detail Corrosion Protection

The designer must select the corrosion protection technique or techniques that meet the level of corrosion protection established during the Initial Design Considerations phase. This selection involves specifying a material or process that is suitable for the nail type and installation procedures. Guidelines for selection of corrosion protection materials are provided in Chapters 7 and 10.

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The engineer must select a grade of steel for the soil nail bar and other metallic parts. Information on steel grades and sizes is presented in Chapter 3 and Appendix A.

In traditional Design/Bid/Build contracts, the engineer may estimate a practical minimum drill hole diameter to provide the bond resistance required for stability. However, the drill hole diameter is ultimately selected by the Contractor to obtain the specified, nominal pullout resistance, and to possibly allow cleaning the drill hole, or accommodating a tremie pipe, tendon couplers, and centralizers.

Install at 15 degrees

Soil nails are installed at 10 to 20 degrees from the horizontal, and most commonly at 15 degrees. The grout can flow at these inclinations from the bottom of the drill hole to the head. Grout generally can fill the hole without leaving air pockets for typical drill-hole dimensions and grout mixes.

Pullout Resistance P 161 (201)

6.6.2 Step 7a Verify Pullout Resistance

Pullout resistance is mobilized behind the slip surface, along the length, L_p, and contributes to overall stability. The length, L_P, can be estimated from the graphical output of soil nail design programs, where critical slip surfaces and soil nails are shown to an appropriate scal

The nominal (i.e., ultimate) pullout resistance per unit length, r_{PO}, is expressed as:

$$r_{PO} = \pi \, q_u \, D_{DH}$$

Equation 6.1: Nominal unit pullout resistance.

Where:

q_u = bond strength of the nail-grout-soil interface (force/unit area)

D_{DH} = diameter of the drill hole

Distributions of bond stresses along the grout-soil interface can be complex and exhibit variations along L_p (Figure 6.4).

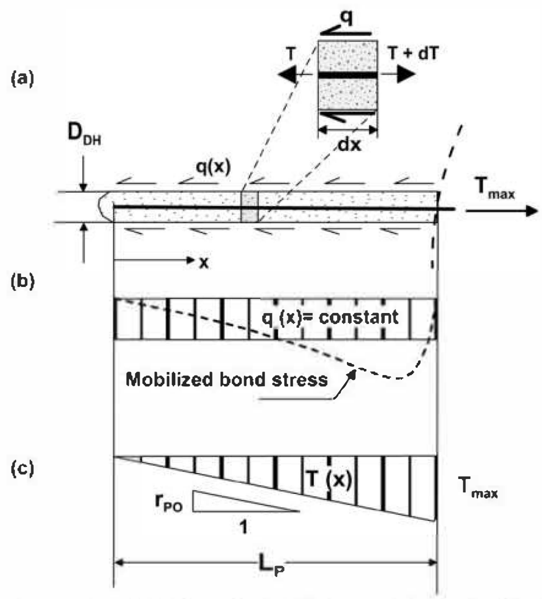


Figure 6.4: Illustration. Single nail stress-transfer mode: (a) soil nail layout, (b) distribution of mobilized bond stresses, and (c) hypothetical distribution of loads along the nail.

The distribution is assumed to be constant along the nail pullout length for simplicity, and the bond stress is considered to have an apparent, average value. When the bond stress increases to its maximum value, the bond strength, q_u, is mobilized.

The nominal pullout resistance, R_{PO}, is calculated as follows:

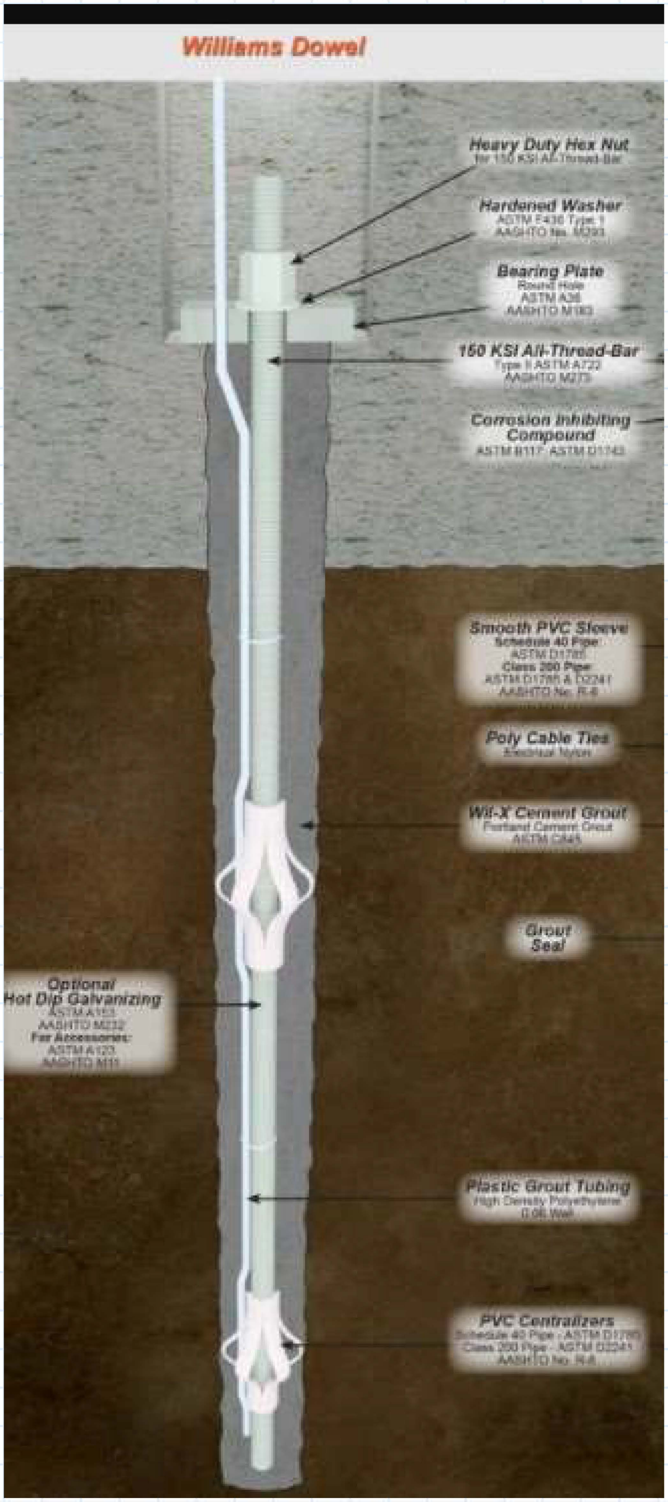
$$R_{PO} = r_{PO}\, L_P$$

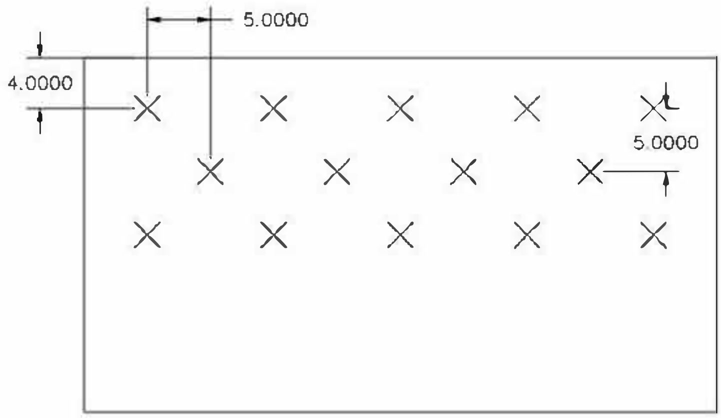
Equation 6.2: Nominal pullout resistance.

Pullout resistance is evaluated as follows:

$$CDR = \frac{\phi_{PO}\; R_{PO}}{\gamma\, T_{max}} \geq 1.0$$

Equation 6.3: Capacity-to-demand ratio (CDR).


Williams Dowel
Heavy Duty Hex Nut
for 150 KSI All-Thread-Bar
Hardened Washer
ASTM F436 Type 1
AASHTO No. M293
Bearing Plate
Round Hole
ASTM A36
AASHTO M183
150 KSI All-Thread-Bar
Type II ASTM A722
AASHTO M275
Corrosion Inhibiting Compound
ASTM B117 ASTM D1743
Smooth PVC Sleeve
Schedule 40 Pipe
ASTM D1785
Class 200 Pipe
ASTM D1785 & D2241
AASHTO No. R-6
Poly Cable Ties
Electrical Nylon
Wil-X Cement Grout
Portland Cement Grout
ASTM C845
Grout Seal
Optional Hot Dip Galvanizing
ASTM A153
AASHTO M232
For Accessories:
ASTM A123
AASHTO M111
Plastic Grout Tubing
High Density Polyethylene
0.06 Wall
PVC Centralizers
Schedule 40 Pipe - ASTM D1785
Class 200 Pipe - ASTM D2241
AASHTO No. R-6



6.3.3b Soil Nail Spacing

Soil nails are installed in a grid pattern. The horizontal nail spacing, S_H, is often the same as the vertical nail spacing, S_V (Figure 6.1). Nail spacing in both directions generally ranges from 4 to 6 ft and occasionally up to 6.5 ft, and is routinely selected at 5 ft. The spacing can be checked such that $S_H \times S_V$ is less than approximately 36 to 42 ft^2.

The first row of nails should not be installed deeper than approximately 2 to 3.5 ft from the top edge of the wall to reduce the potential for instability of the upper excavation lift and to reduce cantilever effects on the temporary facing. The lowermost row of nails should be installed about 2 to 3 ft above the base of the excavation. These requirements are the result of the limited ability of the facing to work as a cantilever at the top and bottom of the wall. However, these limits may be adjusted for project-specific conditions, and when based on suitable analysis.

$FS_P := 1.5$ $\gamma_1 := 123\ pcf$ $z := 4\ ft$ $S_v := 5\ ft$ $w := 1.25\ in$

$c_a := 130\ psf$ $d := 7\ in$ $\phi' := 32\ deg$ $S_H := 5\ ft$ $\phi'_u := 20\ deg$

$\sigma' := 79.766\ psf$ $H := 28\ ft$

$$l_e := \frac{FS_P \cdot \sigma' \cdot S_v \cdot S_H}{\pi \cdot d \cdot c_a} = 12.556\ ft$$

$$l_r := \frac{H - z}{\tan\left(45\ deg + \frac{\phi'}{2}\right)} = 13.303\ ft$$

$$FS_P \cdot \sigma' \cdot S_v \cdot S_H = \left(2.991 \cdot 10^3\right)\ lbf$$

$$L := l_e + l_r = 25.859\ ft$$

$$2 \cdot w \cdot \left(\gamma_1 \cdot z\right) \cdot \tan\left(\phi'_u\right) = \left(1.2 \cdot 10^3\right)\ \frac{lb}{s^2}$$

Because the wall is about 12 in thick, an additional foot should be added to the rods to sustain the correct length for embankment

Soil Nail Specifics

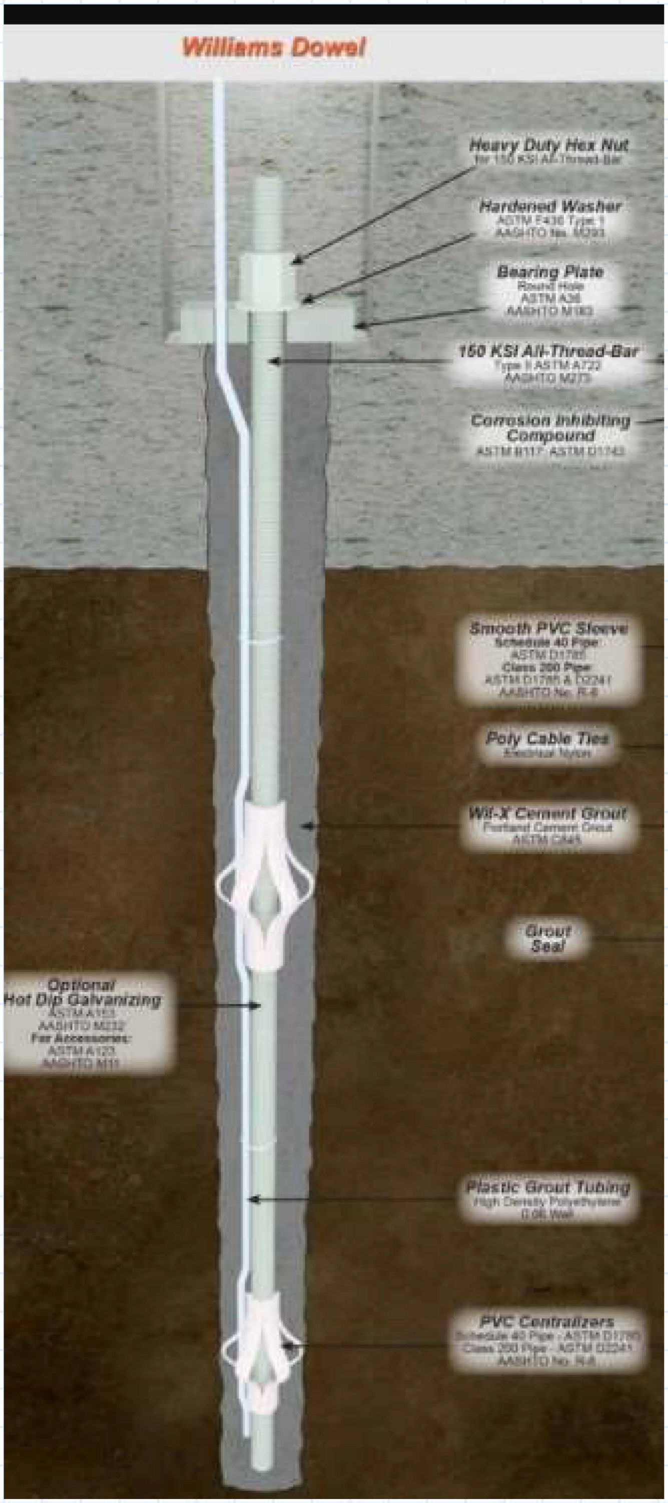
CORROSION PROTECTION TYPE	ABRASION RESISTANCE (4=BEST)	TYPICAL THICKNESS	RELATIVE COST (4=HIGHEST)	PRODUCTION LEAD TIME	CAN BE APPLIED TO ACCESSORIES?	APPLIED IN THE FIELD?
Hot Dip Galvanizing	4	3-4 mils	2	2-4 weeks	yes	no

#10 – 1-1/4" – 3 (32 mm)	1.27 in² (819 mm²)	127 kips (565 kN)	95 kips (424 kN)	102 kips (454 kN)	4.3 lbs/ft (5.5 kg/m)	1-3/8" (35 mm)	R61-10

#10 - 1-1/4" (32 mm)	2" (51 mm)	2.31" (59 mm)	2" (51 mm)	R63-10

#10 – 1-1/4" (32 mm)	2-1/2" (64 mm)	1-3/8" (35 mm)	5/32" (4 mm)	R9F-10-436

Depth (ft)	Pressure from trail (psf)	le (ft)	lr (ft)	L (ft)	FSp	L actual (ft)	Rod #	Diameter (in)	Rod Unit	Washer Unit	Hex Nut Unit
4	79.766	12.556	13.303	25.859	1.5	27	10	1.25	R61-10	R9F-10-436	R63-10
9	55.8785	8.796	10.532	19.327	1.5	27	10	1.25	R61-10	R9F-10-436	R63-10
14	29.268	4.607	7.76	12.367	1.5	27	10	1.25	R61-10	R9F-10-436	R63-10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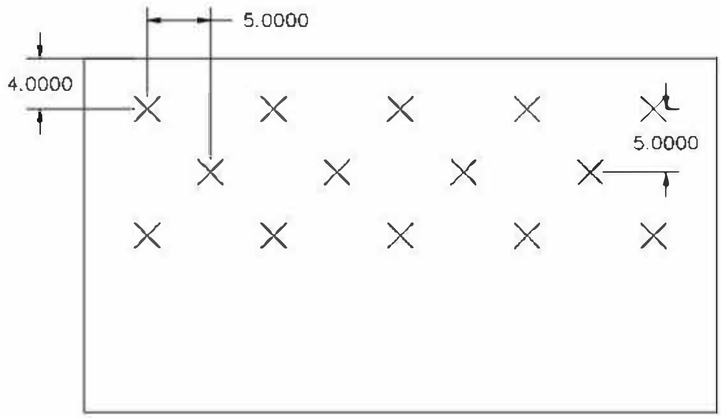
Grout Seal
Optional Hot Dip Galvanizing
Plastic Grout Tubing
PVC Centralizers



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