The University of Iowa
IOWA INITIATIVE
FOR SUSTAINABLE COMMUNITIES

FINAL DELIVERABLE

Title	Waterloo Pedestrian Railroad Overpass
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Date Completed	May 2021
UI Department	Department of Civil & Environmental Engineering
Course Name	CEE:4850:0001 Project Design & Management
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Community Partners	City of Waterloo

This project was supported by the Iowa Initiative for Sustainable Communities (IISC), a community engagement program at the University of Iowa. IISC partners with rural and urban communities across the state to develop projects that university students and IISC pursues a dual mission of enhancing quality of life in Iowa while transforming teaching and learning at the University of Iowa.

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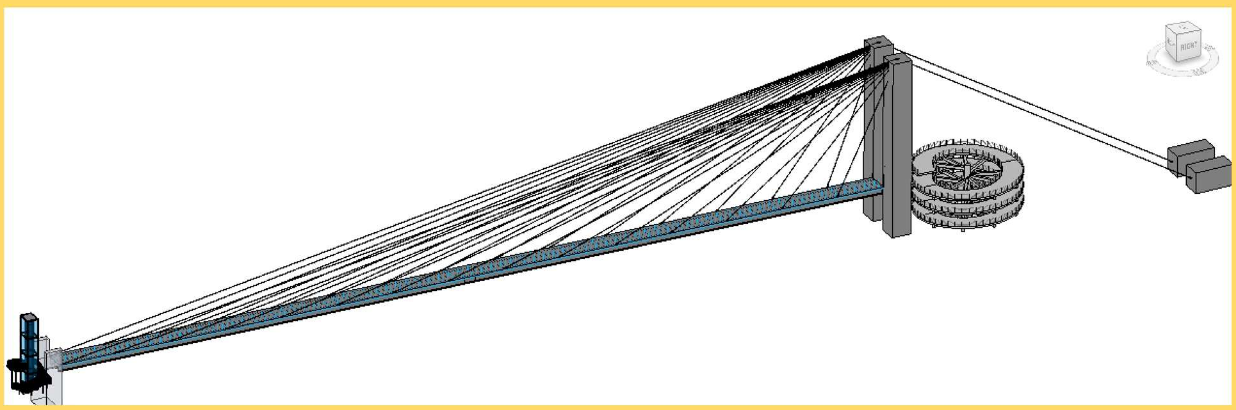
WATERLOO PEDESTRIAN RAIL BRIDGE



The top picture shown is the above-ground structure detailed in this report and the bottom picture is the University of Memphis pedestrian bridge that primarily inspired the design that follows.

Sydney Bortscheller
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Section I Executive Summary -

For this project we have designed a pedestrian cable-stayed bridge that crosses the Canadian National (CN) Railroad in Waterloo, IA. This overpass was designed by a team of senior University of Iowa students studying civil engineering in a capstone design class. During the Spring 2021 semester, the design group formulated a proposed structure for this area that they feel best fits the needs of the client and the people of Waterloo.

The structure designed is a cable-stayed pedestrian bridge over the north end of the CN railyard where it intersects East 4th Street. It will connect with the sidewalk running parallel to East 4th Street on the west side of the road. The south end of the structure is located near the intersection of East 4th Street and Dane Street, and the north end of the structure is located in the southeast corner of Five Sullivan Brothers Memorial Park.

The primary goal of this project was to provide the community with a safe method to cross the tracks, especially when train cars are parked in the intersection. While cars and trucks can easily take a detour when the tracks are blocked, there are no easy paths for foot traffic. Pedestrians are forced to either wait for up to an hour, or dodge between rail cars. Furthermore, this location sees a large amount of pedestrian foot traffic due to the nearby school, businesses, and local neighborhood. With the implementation of this overpass, the safety of pedestrians at this location will be reinstated.

The structure is in an urban setting with limited space, and the piers are placed outside of the railroad's right of way. Due to the limited space, an elevator and staircase were chosen as means of access on the south end of the bridge. On the north end, the bridge ties into a public park and is accessed by way of a spiral ramp. Having an elevator on one side and ramp on the other makes this bridge compliant with the Americans with Disabilities Act (ADA). In addition to this accessibility goal, it was important to design a structure that would enhance the uniqueness of the area. Elements like the spiral ramp access point in the park and the cable-stayed structure were chosen to prioritize the aesthetic of the bridge and its relation to its surroundings. To accomplish this, our team researched the local area and involved our client and the neighborhood services coordinator in the aesthetic decision-making process.

The overpass was designed according to Load and Resistance Factor Design (LRFD). It complies with the Iowa Department of Transportation (DOT) standards, as well as the regulations imposed by CN. The bridge was also designed to follow the Iowa Statewide Urban Design and Specifications (SUDAS) and be ADA compliant. Any other standards not found in these documents were from the Federal Railroad Administration (FRA). The American Association of State Highway and Transportation

Officials (AASHTO) LRFD Bridge Design Specifications serve as the foundation of the bridge superstructure and spiral ramp designs.

The design items included in the report that follows are superstructure, spiral ramp access point, elevator tower access point, substructure, site design, and complete construction cost estimate. The superstructure design includes that of the steel cables, concrete deck, and steel girders. The bridge deck is 27.5 feet above the ground to allow for the vertical clearance required by the railroad. The tower that supports the cables is located at the north end. This twin-tower style support extends 150 feet above the top of deck and is made of reinforced concrete. Each of the two identical towers has a 5.5 feet by 5.5 feet square cross section. Below deck, it expands to a 5.5 feet by 10 feet rectangular cross section, with the 10 foot segment in the east-west direction. The beams are supported on this end by a reinforced concrete beam that has cross section dimensions of 4 feet by 4.5 feet and connects into the tower. Cables are arranged in a radial pattern, extending from the top of the tower. 15 cables connect to the main span of the bridge on each tower, spaced every 35 feet along the bridge deck. Each cable is made up of 19 strands of prestressing wire, arranged in a hexagonal formation. Each strand shall be coated in a polyethylene coating and hot dip galvanized to prevent corrosion. Additionally, the entire cable shall be sheathed. The span is made up of a twin set of W36x853 shape I beams and an 8-inch reinforced concrete slab that will serve as the deck.

The spiral ramp serves as the north-end access point to the bridge deck and has an overall edge to edge diameter of approximately 63 feet. All elements are of reinforced concrete. The path has a width of 12 feet and thickness of 11 ¼ inches. It is supported by beams that are 18 inches wide and 20 inches tall. All beams frame into a circular center column with a diameter of 39 inches. At the south end, the bridge is accessed by way of elevator or stairs. The elevator tower has an overall height of 41 feet and takes up roughly a 9 foot by 7-foot area. The roof is a Vulcraft 3NL metal deck of 22-gauge steel. All beams are W8x10 and the four corner columns are W4x13. The voids in-between beams and columns will be filled with glass panels. The stairs that wrap around the elevator tower will be precast concrete long-span step treads. Each step will be 11 inches deep and 7 inches tall as required by the International Building Code (IBC). Both the ramp on the north and the stairs on the south end of the bridge are intended to have a cable railing installed to visually tie in with the suspension cables of the bridge deck.

The foundations on the project were designed as pile foundations. The foundation on the south side must support the load from the elevator tower, stairs, bridge, and wind loading. This foundation was designed to contain 12 HP14x117 steel piles with a length of 30 feet, spaced at distance of 4 feet on center. These piles where arranged symmetrically with 4 rows and 3 columns of piles running north to south. The pile cap was designed as a 12 by 15-foot-high strength concrete pile cap that has a thickness of

2 feet. The north foundation was designed to carry the deadload from the two main support towers, the spiral ramp, along with the reactionary forces from the bridge including horizontal and vertical loads plus an overturning moment. It was determined that this foundation will contain 64 HP18x204 steel piles with a length of 60 feet, spaced at a distance of 5 feet on center. These piles where arranged in a square formation with 8 rows and 8 columns of piles respectively. The pile cap was designed to be a 40 by 40 foot high strength concrete pile cap with a thickness of 2 feet. The anchorages where designed to be located at 178 feet due north of each tower. Each anchor will contain 1 back stay cable that is angled at 45 degrees with respect to the ground, a 25 foot long-6 inch diameter steel anchor pipe, and a 10 foot cube normal weight concrete anchor block. These blocks will be anchored at a depth of 15 feet and where designed to have enough resistance to prevent failure of the tower.

For this project site design was modeled around two key elements. The first of that being to ensure that our construction and project area did not impede CN right of way. This was important to us and the client due to not knowing the willingness or coordination between the City of Waterloo and the CR Railroad. The majority of our site is located in the southeast portion of Five Sullivan Brothers Park which is all owned by the City of Waterloo. In the park a quarter-acre staging area was designated for the contractor's use throughout the life of the project. This area has the ability to expand if need be further into the park. The other key element of site design is to ensure that the project site, once complete, will match its current state if not be in an improved condition. The measures done to achieve this include the prepping of the site, resurfacing and reseeding of vegetation, silt fencing to prevent erosion, and pre/post construction surveys of the site.

The cost estimate for the construction of the project was broken up into 3 major subsections: substructure, superstructure, and site design. The substructure, which consists of the abutments, tower, and pile foundations has a final cost of $340,000. The superstructure which includes the deck, girders lighting, railing, cables, access points, and tower has a cost of $1,500,000. Site design which consists of the cost of preparing the site for construction as well as restoration after project completion costs $98,000. All cost estimates prepared are for raw material and labor and do not include the contractors overhead/markups. A multiplier of 2.5 was used to calculate a final price that accounts for the contractor's overhead and risk in the project. This brings the total project cost to approximately $5,200,000.

Section II Organization Qualifications and Experience

The project has been designed by a team of four University of Iowa students in the capstone design class. Sydney Bortscheller is the project manager.

All members of the team are Spring 2021 graduates of the University of Iowa's Civil Engineering program. Sydney Bortscheller, Nick Moioffer, and April Vande Brake are specializing in Structures, Mechanics, and Materials. Alex Kettering's focus is Civil Practice. In this project Alex assisted with the superstructure calculations, but primarily focused on site design and construction estimating. April designed the access points of the bridge including the elevator and stairs on the south end and spiral ramp on the north end. Sydney designed the superstructure, including the girder and cable designs. Nick designed the foundations.

Section III Design Services

1. Project Scope

For this project we have designed a pedestrian suspension bridge that crosses the CN Railroad in Waterloo, IA. The design is positioned just northwest of the rail yard and crosses the tracks along the west side of East Fourth Street. The north landing of the bridge outlets into Five Sullivan Brothers Memorial Park. The south landing is south of the tracks, roughly one hundred feet north of Dane Street, west of East 4th Street. The primary goal of this project was to provide the community with a safe method to cross the tracks, especially when train cars are parked in the intersection. The deck that is atop the railroad right of way is enclosed on both sides. Included with the structure is lighting for both safety and aesthetic appeal. Along with safety, it was important to design a structure that would enhance the uniqueness of the area. Elements like the spiral ramp access point in the park and the suspension-type structure were chosen to prioritize the aesthetic of the bridge and its relation to its surroundings. All members of the community are able to access this overpass as all components are compliant with the ADA. The bridge access points include an elevator and stairs at the south entrance and a spiral ramp on the north end.

Our preliminary design of this pedestrian railroad crossing contains all relevant items to successfully completing this project. The items designed are as follows: superstructure, substructure, spiral ramp access point, elevator tower access point, aesthetics, site design, and a complete construction cost estimate. The superstructure design includes steel cable, concrete deck, and steel girder design. Substructure design contains the sizes and materials needed for the abutments, tower, and foundations. The elevator and spiral ramp designs contain the sizes and materials of all their structural members.

Aesthetics were modeled to fulfill the request of our client and duty as engineers to make a structure that is not only functional but will enhance the community in which it is in. Site design encompasses the preparation, use, and restoration of the site after construction in order to return to or improve the current site conditions. The final part of the design is the cost estimation of all materials, labor, and contingencies predicted as necessary from this point onward.

2. Work Plan

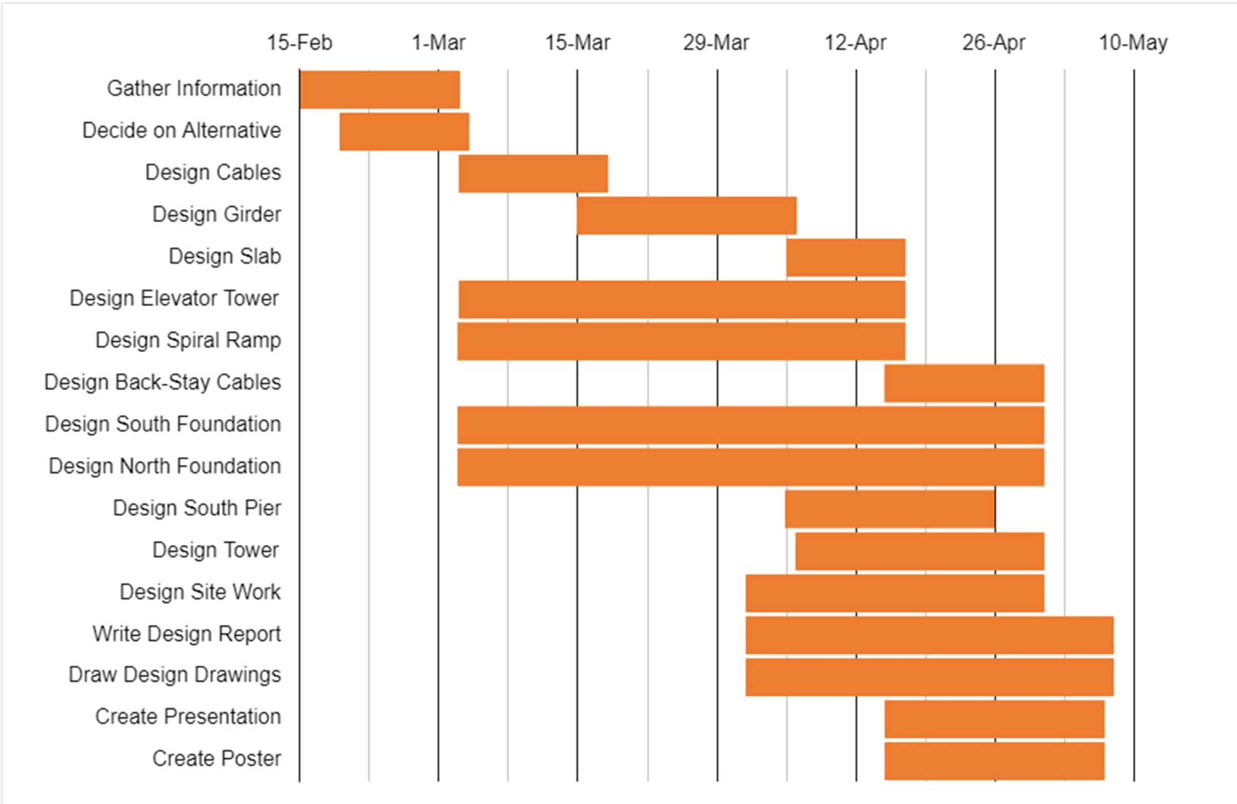
The Gantt chart in Table 2 below served as our schedule of the major tasks, now completed. It begins with gathering information. After a week to gather information, the design portion began, which included designing and/or ruling out possible alternatives. Once the initial design stage was completed, the report and drawings were drafted, along with the presentation and poster. After receiving feedback on those drafts, the design was edited and finalized. The presentation and poster was finalized for the presentation to the client. After feedback, the design report and drawings were also completed. The work was mainly divided by element of design. This means that the same team member that designed an element also prepared relevant report sections and drawings for that element.

The ownership of preliminary design elements is as follows:

Table 1: Table Showing Team Member and Tasks Completes

Name:	Aesthetics	Superstructure	Substructure	Ramp	Elevator	Site	Cost Est.
Sydney	**x**	**x**	**x**				
April	**x**			**x**	**x**		
Nick			**x**				
Alex		**x**				**x**	**x**

Table 2: Gantt Chart



Section IV Constraints, Challenges, and Impacts

1. Constraints

Due to the nature of this project, the design was subject to constraints. These constraints include the horizontal and vertical clearances that the overpass satisfies. Along with this, the profile and grade requirements (physical requirements) are satisfied as well in coordination with the IBC, CN rail standards, and Iowa DOT regulations. The overpass was designed in accordance with the AASHTO LRFD Bridge Design Specifications, such that there is a more than adequate resistance of loads, including but not limited to: dead, live, wind, and snow loads. Along with this, deflection considerations were assessed. The overpass contains one access point at each end which were designed in accordance with ADA specifications. The load applied to the overpass includes an allowance for the fencing specifications required in all railroad pedestrian overpasses.

The site layout presented our group with a few different challenges. The first one encountered was pier placement. Many different options were considered but ultimately the chosen location, west of and parallel to East 4th Street was chosen due to right of way restrictions and input from the city. The two biggest challenges with right of way were avoiding railroad property and avoiding the need to buy property from private landowners. The chosen configuration of the structure stays within the city's right of way.

2. Challenges

The final determination of the overpass location along with the direction of span in consideration with surrounding structures was a major challenge. Another challenge included the placement of the bridge abutments and access points. The final span length and width is greater than the minimum required, and the final dimensions were a challenge as they depended on the size, location, and direction of the abutments. Sizing and type of foundation were dependent on the surrounding soil, making it paramount that the final location of the overpass allowed enough surrounding space to account for this problem.

The task of designing an aesthetically pleasing structure that appeals to the entire community was heavily focused on by the team. The economic feasibility of an overpass was also a challenge that was considered when sizing and choosing materials for structural members. Although the design needed to be in accordance with the ADA, it was not allowed to take away from the functionality or look of the overpass.

3. Societal Impact within the Community and State of Iowa

The safety of Waterloo residents, those of the surrounding neighborhood in particular, is the most notable impact of this project. The design and construction of this overpass was and is significantly overdue as the railyard currently poses a fatal threat to pedestrians. It has been the site of over five dismemberments over the last thirty years (The Courier, 2019). The project site is only six blocks from the nearest high school. With the implementation of this overpass, the safety of students walking to and from school every day will be reinstated at long last. The layout of the bridge and its access points was chosen in part to optimize the safety of bridge patrons. With the spiral ramp access point in the park, there is considerable distance between it and the local bar across East 4th Street. This prevents any forced intermingling of those at the bar and those just passing by.

A priority of this project was to deliver an iconic structure for the neighborhood. Special care was taken to not only satiate the community's mobility and safety needs but also their aesthetic desires. The team met with neighborhood services coordinator, Felicia Smith-Nalls, to hear directly from someone whose priority is maintaining and increasing the local residents' quality of life. She provided critical insight on which existing neighborhood improvements to tie into and which to steer clear of. Her input was also essential in choosing the aesthetic intentions of the bridge. This design strives to bring people together in more ways than the obvious.

Section V Alternative Solutions That Were Considered

1. Underpass

In preparation of this project's proposal, it was determined that an overpass is the optimal solution for this railroad crossing. The most sensible alternative was an underpass structure, such as a tunnel. This would allow another easy mode of transportation for pedestrians to cross the tracks safely. Even though an underpass would present less winter maintenance and eliminate the risk of pedestrians jumping or throwing objects onto the tracks, it was ruled out due to restrictive clearance requirements beneath the track and the high flooding potential.

2. Access to Bridge

Within the bounds of ADA compliance, a spiral ramp and elevator tower were chosen for the bridge access points. Other options were continuous approaches and switchback ramps. These options were ruled out on the grounds of aesthetic appeal and available space. A spiral ramp requires a smaller area than a continuous approach and will add to the ambiance of the park to the north. An elevator tower requires an even smaller area, which is why it was chosen where space is especially limited on the south end.

3. Materials

Care was taken with each element in deciding whether it was best suited by steel or concrete. The team kept in mind that concrete is lower maintenance and less prone to damage by salt and ice melting chemicals. However, it makes for a heavier structure and has a higher potential of not being able to meet span requirements. Alternatively, using strictly steel would allow for the bridge to have a much longer span. It was noted that depending on the specific type of steel, it could be a cheaper alternative. Also to its advantage, steel can be covered with an epoxy-based clear sealer for preventative aid in graffiti removal. Steel as a material poses some possible issues though. One being that a steel walkway would not be as easy or as comfortable as walking on concrete. Also, steel is prone to rust, especially in climates in the Midwest.

4. Truss

A truss was briefly considered for this bridge. However, with the final span length being over 500 feet, it was determined that a cable-stay bridge would be more economical. In addition, the City of Waterloo is looking for a signature bridge. The cable-stay design was more aesthetically pleasing and gained the attention of the neighborhood services coordinator, Felicia Smith-Nalls, and the city engineer, Jamie Knutson. It was therefore decided that this team would move forward with the design of a suspension bridge.

5. Span

Multiple spans were considered for this bridge. There were considerations to bring the north side pier over to an empty lot. However, this lot had little room and probably would have required an additional elevator. Also, the City expressed their wish to open the bridge into the park, to make it a more welcoming environment.

The south side pier was considered to be placed in a larger open lot farther west of the decided on site. However, this would have been out of the way for pedestrians, and therefore would decrease the usability of the bridge. In addition, it would have expanded the span. The most viable possible span design is located in Appendix F.

Section VI Final Design Details

<u>Span Design:</u>

The span was designed to be 525 feet long. A depiction of the span can be found in Appendix F. On the North end, the bridge abutment will be located in the nearby park. The pedestrians can then use a spiral ramp to exit the bridge, leading them into the park. On the south side, a small plot of land currently owned by the city will be used. Because of the small size, an elevator and staircase was designed for this segment. These two pier placements were decided on for multiple reason. The first is that both slots of land are owned by the City. Both pier locations are also close to E. 4th street, which runs north-south, and would provide easy access for pedestrians.

Drawbacks to this location, is that it is not the shortest span possible. The shortest viable span is shown in Appendix F. It is also not on the same side of the road as the high school, so pedestrians will most likely have to cross the street if they wish to use the bridge.

Aesthetic Considerations:

The bridge requires a 10 foot tall safety fence, due to the fact that it is running over a railroad. We propose that this fence be a wire mesh fence, similar to that of the cable-stay pedestrian bridge on the University of Memphis campus. Other fencing could be considered, including an 8 foot tall fence curved at the top, as specified by the AASHTO LRFD Guide and Specifications for the Design of Pedestrian Bridges. A cheaper, chain link fence, is also a viable solution, but does not match the aesthetic intent maintained as a priority in other aspects of the design of this bridge.

Lighting is also a consideration for this bridge. Traditional light posts could be placed on the bridge deck. However, for a more aesthetically intriguing look, LED lighting could be considered. LED lighting would be programmable, so the colors could change throughout the day. These lights would be fastened on the edge of the bridge deck, and could also be placed on top of the tower. Additionally, flood lights could be placed near the access points. This lighting would both increase the safety for bridge patrons as well as add some visual appeal. While the lighting is not fully designed at this time, lighting was taken into consideration when assessing the strength of every component.

Cable Design:

The cables were designed using an iterative design method. It was assumed that there would be 15 cables on each side of the bridge, for 30 total cables. These cables would be spaced apart equally every 35' in the horizontal direction, and all cables would be anchored to the top of the tower, for a radial cable design pattern. This pattern is a common pattern in cable-stay bridge construction, and is effective. However, it requires extra detailing to handle the congestion that will be apparent when anchoring multiple cables to the top of the tower.

The area of each cable was determined based on the service load applied, which is the dead load and pedestrian live load acting on each set of cables, added with no additional factors. The tower will extend 150 feet above the top of deck. Dead load was approximated based on the weight of the girders, concrete deck, lighting, and safety cage. Pedestrian loading was based on the AASHTO LRFD Guide Specifications for the Design of Pedestrian Bridges requirements and was thus assumed that a 90 PSF live load would be applied to the deck. The dead load and live load of the entire bridge was then divided by two, as there will be two sets of cables. The computation and values for this loading can be found in Appendix C. This service loading was then used to find the total tension in each cable. The area in the cable was then determine using the strength based approach. Calculations for the cable design are shown in Appendix B. Through these calculations, it was determined that each cable shall consist of 19 strands of 0.6" diameter prestressing wired.

The strands are assumed to be arranged in a parallel design pattern, as it has the most strength efficiency. As such, area may be increased to accommodate for this design. Each strand should be hot-dipped galvanized and coated in a polyethylene polymer to prevent corrosion in the wires. Additionally, the each cable should be sheathed to minimize wind and rain effects. A cross sectional view of the cable can be found in Sheet 5: Superstructure Elevation of the drawing set.

Girder Design:

The entire superstructure, including the girders, cables, and tower, were modelled using a 2-D frame in Autodesk Robot. This model was used to determine the sizing of the girders, as well as the tower. This model can be seen in Appendix A.

The girders were designed in accordance with the AASHTO LRFD Bridge Design Manual. The models were first analyzed by applying the calculated dead load and live load to them. Then, these numbers were multiplied in accordance with the Strength I limit state, to approximate a maximum shear force, and maximum positive and negative moment. The axial compression model analyzed only the service loading to find maximum compression. Results of both of these models can be found in Appendix C.

The girder was analyzed for shear, and combined axial and flexural resistance. It was designed to act non-compositely with the deck. The shear resistance was designed in accordance to AASHTO LRFD Bridge Specifications Section 6.10.3.3 on shear resistance of steel members. The shear strength of the girders was much greater than the required shear strength, as determined by the Strength I load combination. Therefore, the section can resist shear without shear stiffeners.

The combined compressive resistance and flexural resistance was determined using AASHTO LRFD Handbook section 6.9.2.2. The compressive resistance was determined using section 6.9.2.1. It was also assumed that this section would have torsional bracing so that effective length for torsional buckling was equal to 5 feet. The torsional bracing itself has yet to be designed, but this will be one of the controlling parameters for said design. The flexural strength was determined in accordance with Section 6.10. It checked against lateral torsional buckling, flange local buckling, and flexural yielding of the material. The critical case was found to be the lateral torsional buckling case, for a moment resistance of approximately 13266 kip-ft. This was determined using a lateral bracing distance of 9.5 feet.

This process has resulted with the conclusion that there shall be 2 girders, which are both W36x853 standard steel section.

Slab Design:

The slab shall be an 8 inch concrete slab. It should be made out of pre-cast concrete, cast in 35' segments to match the spacing between the cables. This slab will require #4 rebar to run through it every foot for shrinkage and temperature requirements. This slab was designed as a one-way slab, using LRFD concrete design theory. It was designed to support the required live load, superimposed dead load, and the slab's self weight. This slab was checked in Strength I. The slab was designed to act completely non-compositely from the girder, and therefore does not contribute to the strength of the girder. The calculations for this can be found in Appendix G.

Tower Design:

The tower consists of two twin columns that extend 150 feet above the top of deck. The tower above the deck was analyzed using the same model of the superstructure used to analyze the girders. This model can be found in Appendix A. Each tower was analyzed using standard LRFD concrete design theory. The towers were assessed under Strength 1, Strength 3, and Service 1 load combinations in accordance with the AASHTO LRFD Bridge Design Manual. The loadings applied to the section can be found in and the results of the loads on the members can be found in Appendix C. The tower was analyzed for bending moment, compressive resistance, and buckling.

Above the deck, these columns are each 5.5' by 5.5' sections of normal weight concrete. They consist of 16 pieces of #18 bars, arranged symmetrically through the bar. The rebar is tied using #4 ties spaced every 2'. The Strength I loadings for this section were used to design this section, finding that buckling was critical. The sections were then checked in Strength 3 and Service 1 to account for strength. This included a bi-axial moment interaction check. The towers above the deck are doubly symmetric, and therefore have the same bending moment resistance in each direction.

The tower extends 27.5' down from the top of deck to the ground. The tower below the deck was modelled as a rigid frame, . A concrete beam, referred to as the pier cap, supports the girders and deck, and two concrete columns connected the frame to the ground. A model of this can be found in Appendix A. The girders rest on a Disktron fixed bearing plate, with 30% lateral load capacity. This Disktron will need to be specially designed to suit this bridge, and estimations of the dimensions for this bearing plate were made based on readily available models from RJ Watson Inc. The Disktron will distributed the load from the girders and deck properly to the pier cap.

The pier cap was analyzed for flexural strength, as well as shear strength. Tension-controlled design was used to analyze the flexural resistance. The pier cap is a 4' by 4.5' singly reinforced concrete beam. It consists of 8 pieces of #14 rebar, with #5 U-shaped stirrups placed every 1.5".

The columns below the deck were determined to be 5.5' by 10' sections. The 10' dimension will extend in the east-west direction. The concrete will taper to this dimension at a 4:1 ratio for aesthetic purposes, but the tapered section was not be counted towards strength. The columns require 54 pieces of #18 rebar, and the arrangement is detailed in Sheet 7: Tower of the drawing set. There will be #4 ties supporting the columns, spaced every 1.75 in. This section was checked similarly to the tower above the deck. Flexural resistance was checked in both the strong and weak axis of the towers. The towers were designed in Strength I, and checked in Strength 3 and Service 1. The combined axial compression and flexural resistance was critical in this section. The full calculations used to design this tower can be found in Appendix D, and the full drawing details can be found in Sheet 7: Tower of the drawing set.

Spiral Ramp Design:

The ADA specifications were used to set spiral outer-edge diameter to 62 feet and 11 ¼ inches and path width to 12 feet. This diameter was found by employing the 5% slope requirement set by the ADA and assuming a height between adjacent ramp levels of 8 feet. The Approximate Method of Analysis for Decks was used as specified in AASHTO LRFD Bridge Design Specifications Section 4.6.2 to estimate a maximum moment in the equivalent strip of slab of 114.2 kip-ft. This moment is generated by application of self-weight, lighting, railing, and pedestrian live loads. Using LRFD concrete design theory, that moment would require a mat of #4 bar spaced 9 $^7/_8$ inches on center (o.c.) in the transverse direction for shrinkage and temperature and 6 $^5/_8$ inches o.c. in the longitudinal direction for tensile strength. This shrinkage and temperature spacing was set by the ACI area of steel minimum of 0.18% of the concrete cross-sectional area. The tensile reinforcing steel spacing was found using simplified equations for tension-controlled design which assume the rebar will yield before the concrete.

Tributary area and static equilibrium were used to determine that the maximum moment and shear in each 18 inch by 20 inch cantilever beam were 2,907 kip-ft and 119.4 kip, respectively. Using LRFD concrete design theory, the moment would require 6 #7 bars centered over the top of each beam, spaced at 3 $^5/_8$ inches o.c. and the shear would require #3 stirrups spaced at 2 inches in the high shear zone which extends from the center column for 23 feet 8 inches. The spacing then widens to 4 $^3/_8$ inches until 29 feet 6 inches from the center column; at which point, stirrups are no longer required. The spacing (18 feet 8 inches o.c. along the outer edge of the path) of the beams was determined by the requirements of the deck analysis method. In order to analyze the deck as a single-spine beam of straight segments, those segments must each span a maximum central angle of 34°. Similar to the deck, the 6 #7 bars for tensile reinforcement assume tension-controlled design. Their spacing meets the requirement that they span a distance equal to one-tenth the deck clear span. The concrete of the beam's shear strength left a difference of 114.5 kip to be covered by reinforcing stirrups. The 2 inch spacing of the stirrups in the high shear zone was determined using that difference. Past the high shear zone, the depth of the beam section was the controlling factor that set the maximum spacing at 4 $^3/_8$ inches o.c.

Static equilibrium was used to estimate a total compressive force in the central column of 3,581 kip. This is the sum of the support reactions from all beams framing into the column. Using LRFD concrete design theory, that force would require a 39 inch diameter spiral column with 6 #14 bars spaced evenly along the inside of a ½ inch diameter, 2 $^3/_8$ inch pitch spiral cage. This size and amount of reinforcing steel is typical of spiral columns and fulfills the steel-to-concrete ratio set by the ACI. Detailed calculations for the spiral deck, beams, and central column can be found in Appendix I and their visual depictions in Sheet 9: Spiral Ramp and Sheet 10: Spiral Ramp Section Details.

Elevator Tower Design:

ASCE 7-16 was used to calculate a total load on the elevator tower roof of 46.8 PSF. Using the Nucor Vulcraft Inward Uniform Allowable Loads tables, the optimal roof decking was determined to be 3NL22. The total load includes a superimposed dead load of ¼ inch protective layer and liquid applied waterproofing as well as standard roof live and snow loads of 20 PSF and 25 PSF, respectively. The selected roof decking can support a 58 PSF loading over a 9 foot span, which is sufficient for the Evolution 100 elevator dimensions.

The concepts of tributary width and static equilibrium were used to determine the maximum moment due to gravity loads in each of the roof framing beams to be 19.6 kip-feet. Chapter 26 of ASCE 7-10 and flexible diaphragm analysis were used to calculate the distributed wind loading over each lateral load resisting beam in the four equally spaced diaphragms and determine the maximum moment in the roof beams to be 1.435 kip-feet. The AISC Steel Construction Manual Table 3-2 was used to select a roof beam size of W8x10. The load from the roof decking, self-weight of the beam, and a required safety beam capacity of 7.5 kip by the Evolution 100 elevator manufacturer compose the gravity load applied. As the W8x10 is a section with noncompact flanges, Table 3-2 factors that into its capacity estimate of 21.9 kip-feet after the application of the ASD flexural factor of safety of 1.67. As the sum of gravity and lateral loads on the roof beams is 21.1 kip-ft, the W8x10 was determined sufficient.

ASCE 7-16 was used to determine the maximum moment due to gravity load of the glass panels on the intermediate to be 727 pound-feet. This combined with the maximum moment due to wind loading of

7.893 kip-feet is far less than that of the roof beams; therefore, the same W8x10 beam size was selected as adequate. As is the conservative choice, the wind pressure was assumed to act on the wider pair of walls. After distributing the net wind load to the roof and two intermediate diaphragms using their tributary widths, the diaphragms were assumed to behave as simply-supported beams and the wind loads were separated to supporting beams accordingly. As stated above, the weight of glass was used to size the beams beneath each panel, but this could be reanalyzed with a different material such as acrylic.

Static equilibrium was used to calculate maximum compressive force and moment due to eccentric loading in each corner column of 19.3 kip and 2.4 kip-feet, respectively. Using AISC Steel Construction Manual Table 6-2 and Chapter H of the AISC Specification, a W4x13 shape was chosen and confirmed to be adequate for strength resistance. An unbraced length of 10 feet was used to select an initial section as the 41 foot tall columns are braced at the three levels of intermediate beams. The loads from the beams are applied at the edges of the columns and not through their centers; therefore, both axial and flexural strength of the section were verified. The interaction of this combined loading was also checked and found to be within the acceptable limit. All steel members are Grade 50. Detailed calculations for all elevator tower structural elements can be found in Appendix J and its elevation and plan drawings on Sheet 11: Elevator Tower With Stairs.

The stairs that wrap around the elevator tower will be precast concrete long span step treads on steel stringers. There are 5 foot wide landings at each corner that the staircase rounds. Step depth and height are set at the International Building Code (IBC) standard of 11 inches and 7 inches, respectively. A cable railing is suggested to aesthetically coordinate with the suspension bridge design. Design of the stair supports including the stringer girders was determined to be beyond the scope of preliminary design, but the step dimensions can be seen on Sheet 11: Elevator Tower With Stairs.

South Pier Design:

The south pier was designed using standard LRFD concrete design. It is composed of normal weight concrete, and supports the load from the girders. This pier is designed in a T-shape, as is standard in pier foundation design. A model of this pier can be found in Appendix A. The girders shall rest on a Disktron Unidirectional bearing pad with 30% lateral load capacity. This Disktron is a standard, available connection that can be purchased from RJ Watson Inc, and ensures proper distribution of forces from the girder system to the pier cap.

The pier cap is a singly reinforced concrete section that is 3' by 4'. There are 4 pieces of #14 rebar, and #5 U-shaped stirrups. These stirrups should be placed every 2". A detail of this section can be found in Sheet 8: South Pier of the drawing set. The pier cap was analyzed in Strength 1. The wind loading supplied by Strength 3 and Service 1 on the pier cap was found to be negligible, and therefore not assessed.

The pier column was assessed in Strength 1, Strength 3, and Service 1. The combined flexural strength and axial strength was assessed, as well as buckling. The pier column will be a 3' by 3' section, reinforced doubly symmetrically with 8 #14 bars. The column also includes #4 ties that shall be placed every 2 feet. The design calculations of the south pier can be found in Appendix K, and full drawing details can be found in Sheet 8: South Pier of the drawing set.

South Foundation Design:

The Standard Guidelines for the Installation and Design of Pile Foundations from ASCE was used to design this foundation. It was determined that a pile foundation would be suitable for this abutment, as space at this site on the project is limited and the loads could be high. Using an iterative design method, a pile size was first chosen to determine its capacity and resistance to the applied loads. Several piles were chosen to resist the total load acting on the foundation, and pile cap dimensions were selected to account for the number of piles in an economical way. The south foundation could experience three different loading types: Case 1 & 2 being a strength design and Case 3 being a serviceability design. When sizing this foundation all three loading cases had to be considered and can be seen in detail in Appendix L. Using these loading cases, it was found that an HP14x117 steel pile with a length of 30' would be suitable for this foundation. This pile and its corresponding length had enough capacity in pile point bearing, point load, and side friction capacity to prevent failure of an individual pile due to the loads acting on it. It was determined that a pile formation of 12 piles spaced at 4' O.C. would be able to resist each loading case with a corresponding pile cap dimension of 12'x15'x2' high strength concrete, see design sheet 12. This foundation had corresponding edge distances of 1'-6" in the N-S direction and 2' E-W direction as this pile cap is to be shaped as a rectangle. This formation of piles was shown to have enough capacity to resist failure of individual piles and the failure of the pile cap due to each loading case. The loads to be resisted in this analysis included moments and horizontal forces in only the N-S directions, along with the vertical reaction force from the bridge deck combined with the dead load from the superstructure above. The piles where oriented arbitrary with their weak axis of bending in the N-S direction. It was found in analysis that this orientation does not cause failure as the applied moment and horizontal force are relatively small.

Settlement of the piles where also analyzed in the design of this foundation. This included the settlement of an individual pile and the settlement of the pile group. Using the Bowles Method, the pile settlement for an individual pile for each loading case was found to be less than the required settlement of

3 times the pile diameter or 0.617", see Appendix L. The elastic settlement of the entire pile group was determined using the Strain Influence Factor Method with accordance to equivalent footings in pile foundation design and analysis. It was found that for each loading case, the settlement of the group did not reach or go past the maximum limit of 1".

Buckling of the piles was the final design criterion. According to ASCE, the maximum allowed stress in the piles before buckling occurs at 35% of the yield stress (50 ksi). In this case it was found that 17500 psi was the allowable limit, in which each loading case passed well within the range with a magnitude in the 10000-psi range.

Many assumptions about the surrounding soil were made during the design and analysis of this foundation. USGS geodata provided soil data for only about 6' into the ground. Since soil boring is not an option at this point in the design, it was assumed that the soil in this site is a purely granular soul. This assumption was made in accordance with the fact that the project site is excessively drained as seen in the geodata. Final design shall include a soil boring which will influence the design of this foundation as the piles are to be embedded at a depth of 30'.

It should be noted that this foundation will have to contain the necessary requirements for building an elevator. This includes but is not limited to, an elevator pit that is embedded into the foundation, along with a hydraulic ram that will penetrate the entire foundation in order to operate the elevator. These elements were not considered during the design and analysis of this foundation but due to the loads applied and the nature of how pile foundations react to given loads, these changes will not impact the strength greatly and the piles can be rearranged to account for this.

North Foundation Design:

The Standard Guidelines for the Installation and Design of Pile Foundations from ASCE was used to design this foundation. It was determined that the loading acting on this foundation would warrant the design of a pile formation as these loads have a very high magnitude. Using an iterative design method, a pile size was first chosen to determine its capacity and resistance to the applied loads. Several piles were chosen to resist the total load acting on the foundation, and pile cap dimensions were selected to account for the number of piles in an economical way. Through this design process, it was determined that the north foundation could experience three different loading types: Case 1 & 2 being a strength design and Case 3 being a serviceability design. When sizing this foundation all three loading cases had to be considered and can be seen in detail in Appendix L. Using these loading cases, it was found that an HP18x204 steel pile with a length of 60' would be suitable for this foundation. This pile and its corresponding length had enough capacity in pile point bearing, point load, and side friction capacity to prevent failure of an individual pile. It was determined that a pile formation of 64 piles spaced at 5’ O.C. would be able to resist each loading case with a corresponding pile cap dimension of 40’x40’x2’ high strength concrete. This foundation had corresponding edge distances of 1’-6” in both the N-S and E-W directions as this pile cap is shaped as a symmetric square. This formation of piles was shown to have enough capacity to resist failure of individual piles and the failure of the pile cap due to each loading case. The loads to be resisted in this analysis included moments and horizontal forces in both N-S and E-W directions, along with the vertical reaction force from the bridge deck combined with the dead load from the superstructure above. The piles are specifically oriented with their strong axis of bending in the N-S direction. The reason for doing this is due to the high horizontal force that will be acting on this foundation. It was found that orienting the piles in this fashion will be able to resist this load and orienting them along their weak axis would not, see design sheet 13.

Settlement of the piles where also analyzed in the design of this foundation. This included the settlement of an individual pile and the settlement of the pile group. Using the Bowles Method, the pile settlement for an individual pile for each loading case was found to be less than the required settlement of 3 times the pile diameter or 0.772”, see Appendix L. The elastic settlement of the entire pile group was determined using the Strain Influence Factor Method with accordance to equivalent footings in pile foundation design and analysis. It was found that for each loading case, the settlement of the group did not reach or go past the maximum limit of 1”.

Buckling of the piles was the final design criterion. According to ASCE, the maximum allowed stress in the piles before buckling occurs at 35% of the yield stress (60 ksi). In this case it was found that 21000 psi was the allowable limit, in which each loading case passed well within the range with a magnitude in the 10000-psi range.

It should be noted that many assumptions about the surrounding soil was made during the design and analysis of this foundation. USGS geodata provided soil data for only about 6’ into the ground. Since soil boring is not an option at this point in the design, it was assumed that the soil in this site is a purely granular soul. This assumption was made in accordance with the fact that the project site is excessively drained as seen in the geodata. Final design shall include a soil boring which will influence the design of this foundation as the piles are to be embedded at a depth of 60’.

<u>Backstay Cables Design:</u>

The sizing for the backstay cables was designed in the same way as the previous cables. This time, however, they were designed to take all the horizontal force that the cables would exert on the tower under the service load. It was also determined that the backstay cables would be placed at a 45 degree angle with the ground. Similarly to the main span cables, these backstay cables shall be anchored to the top of the tower. This anchors the cables 177.5' away from the north tower. The cables should be placed in line with the bridge to prevent unnecessary torsion on the tower. There shall be two backstay cables, each using 127 strands of 0.6" diameter prestressing wire, arranged hexagonally. They shall be treated and coated the same way as the other cables.

Using equations and references found in Foundation Analysis and Design, the anchorages were designed. With references to the USS Steel Sheet Piling Design Manual for information on anchors and tiebacks, a block anchor (deadman anchor) foundation was designed. This foundation was designed to counteract the load on the two support towers from the bridge deck and keep the towers upright and sturdy. These types of anchorages are common in suspension bridge design and was an ideal solution to use in this project due to the limitation of space on this project. The anchors were designed such that each tower will have one back-stay cable and one block anchor. Each block anchor will be a 10'x10'x10' block of normal weight concrete embedded at a depth of 15' with one 6" diameter anchor rod spanning 25', see design sheet 14. This anchor carries the necessary dimensions to develop the adequate passive resistance for the back stay cable. It was found that the anchorage resistance of this formation would be 3380 kips, which is suitable to counter act the load of each back-stay cable, see Appendix L. These anchors where only analyzed for the critical load case (strength based), which controls cable design. This anchor must be carefully backfilled both around the sides and on the top so that the assumed passive condition with friction can develop. Carefully looking through design manuals and text, an assumed water depth table should not influence the anchorage resistance but may be important to note during the construction process. The anchor block will be connected to the anchor rod through a typical anchor rod connection, see Appendix L. This Rod will be connected to the main back-stay cable through a concrete transition block that can be seen on design sheet 14. With all these elements in place these anchorages will be able to support the two towers and prevent the bridge from failing.

<u>Site Design:</u>

In order to properly prepare for the construction of our project we must take proper steps to prepare our site for construction. We began by identifying which area is best to use as a staging area for equipment, materials, and on-site fabrication. We decided that the best location would be North of the structure in Five Sullivan Brothers Park. This land is owned by the city and is currently open space with a few small trees. Clearing and preparing this site would be easy and convenient. The total area is approximately .3 acres, a drawing of the area on Google Maps is shown below in Appendix C.

The site is initially to be cleared and graded to provide a smooth and level surface for equipment and fabrication. Then a base/subbase of material will be brought in to preserve the ground from being damaged from constant movement of heavy equipment. Then a silt fence will be installed around the perimeter to prevent erosion of topsoil from the site. A survey crew will survey the land before and after construction to ensure that the site is restored to near its previous condition. After construction the base/subbase will be removed and the previously removed topsoil will be reapplied to the area. On the newly placed soil, sod matching the surrounding grass type (assuming Kentucky Bluegrass) will be planted. Six native Iowa trees will also be planted in lot after the grass is sewn.

Section VII Engineer's Cost Estimate

The estimated final cost of the complete project is $5,200,000. This was determined using materials quantity estimation from design calculations and industry sources for pricing. All quantities and sources are listed in the project estimation spreadsheet below. The document is split up into four major cost sections. The following sections are substructure, superstructure/pylon/access points, and site design.

Substructure is all the materials, labor, and equipment needed to drive pile and construct the pile caps and anchors. Superstructure/pylon/access points has all the costs related to the deck, cables, and end structures (elevator and ramp). Site design includes all necessary material and labor for preparing the site for construction as well as restoring it to its final condition once the bridge is complete. The costliest section of the project is the superstructure/pylon/access points section. This is due to the amount of material needed to be included in this section as well as the nature of it. With our superstructure being primarily steel it makes sense that it will be the most expensive portion of the project due to the material and labor required. The total time of construction for this project is estimated to be 12 full working weeks on site. This does not include fabrication times of objects that can be built off-site. The timeline of 12 weeks is ambitious for this project, but it is important to work quickly and efficiently around the CN property to minimize the delays construction may cause to their operations.

When calculating and estimating labor costs various online sources were used and are linked in our spreadsheet. Labor was estimated using our best judgement and experience from internships and construction estimating. Labor values were pulled directly from an Iowa DOT sheet showing union workers' wages based on trade. For materials, industrial companies and professional websites were used as guides for estimation. To value the true cost of the project as it would appear to the client, a 2.5 multiplier was applied to all costs for the contractor's overhead and liability on this project. This multiplier is used due to the fact that material and labor pricing was not adjusted for profit by the sources.

Overhead is also to include costs related to mobilization and other items and materials that will be used on the project outside of what is needed for the structure itself.

To see a full breakdown of pricing, reference Appendix N below. The price of the subsections with the 2.5 multiplier are: substructure ($850,000), superstructure ($3,600,000). site design ($250,000). The total of these combined sections is $5,200,000. This will display the line items in each section of the project cost analysis along with their unit pricing. A breakdown of each section's price is also given. In the cost estimation spreadsheet, the sources for where a price was obtained are linked for certain materials. Scratch work for material quantity estimation is also included below in the appendix.

Appendices

A. Robot Models

B. Cable Design

C. Main Span Applied Loadings

D. Tower Design

E. Girder Design

F. Span Design

G. Slab Design

H. Span Deflection

I. Spiral Ramp Design

J. Elevator Tower Design

K. South Pier Design

L. Foundation Design

M. Site Design

N. Project Cost Estimate

A. Robot Models

Superstructure model:

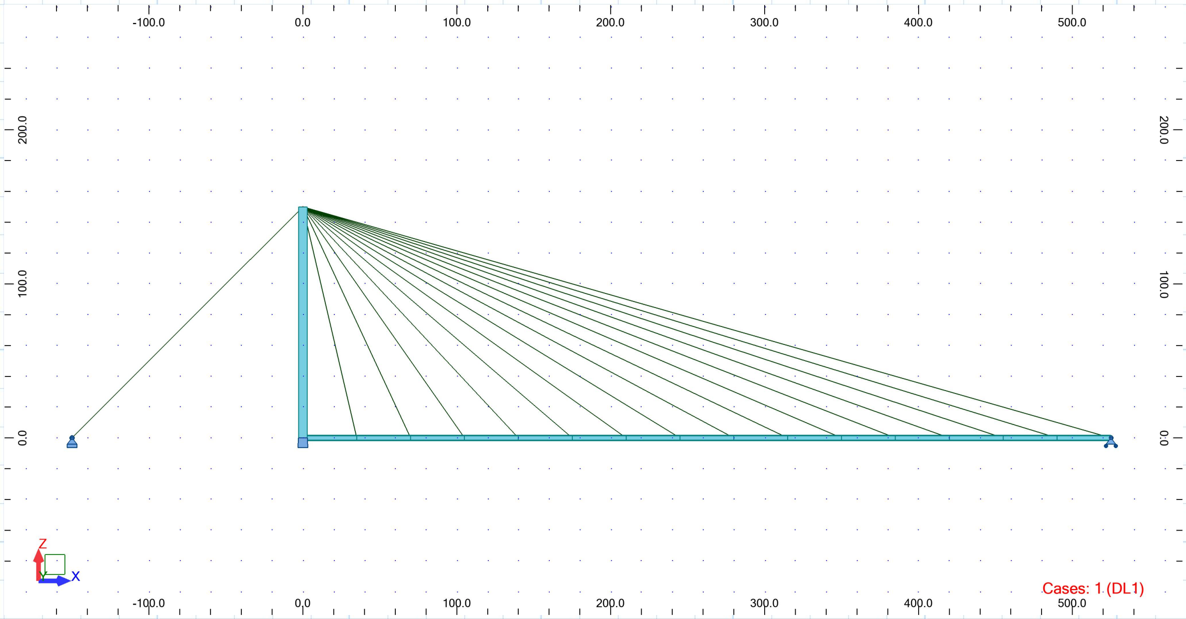


Table of Results: Unfactored Shear Force (kip)

		DC		DCT		LL-Ped	
x (ft)	Span	Pos	Neg	Pos	Neg	Pos	Neg
0	0	109.99	-	1.89	-	39.2	-
35	0.07	56.97	-	1.89	-	20.3	-
70	0.13	22.01	-8.37	0.22	-0.17	7.85	-2.98
105	0.20	18.58	-31.01	-	-0.13	6.62	-11.05
140	0.27	19.47	-34.45	-	-0.06	6.94	-12.28
175	0.33	22.09	-33.56	-	-0.02	7.87	-11.96
210	0.40	27.68	-30.94	0.01	-0.02	9.87	-11.03
245	0.47	36.95	-25.35	0.03	-	13.17	-9.03
280	0.53	48.71	-16.07	0.06	-	17.36	-5.73
315	0.60	59.71	-4.31	0.08	-	21.28	-1.54
350	0.67	65.09	-	0.09	-	23.2	-
385	0.73	59.3	-	0.09	-	21.14	-
420	0.80	37.16	-	0.07	-	13.25	-
455	0.87	-	-15.86	0.01	-0.09	-	-5.65
490	0.93	-	-67.9	-	-0.24	-	-24.2
525	1	-	-120.93	-	-0.24	-	-43.1

Note that the load case DCT is the dead load caused by the tower's self weight

Table of Results: Unfactored Moment (kip-ft)

		DC		DCT		LL-Ped	
x (ft)	Span	Pos	Neg	Pos	Neg	Pos	Neg
0	0	-	-3531.08	-	-61.6	-	-1258.6
35	0.07	-	-609.31	4.63	-	-	-217.18
70	0.13	25.524	-	12.39	-	9.1	-
105	0.20	-	-131.94	6.51	-	-	-47.03
140	0.27	-	-409.71	1.82	-	-	-146.04
175	0.33	-	-656.35	-	-0.32	-	-233.95
210	0.40	-	-811.19	-	-0.96	-	-289.14
245	0.47	-	-770.41	-	-0.66	-	-274.6
280	0.53	-	-405.07	0.5	-	-	-144.38
315	0.60	372	-	2.57	-	132.59	-
350	0.67	1533.92	-	5.42	-	546.74	-
385	0.73	2884.2	-	8.55	-	1028.03	-
420	0.80	4031.75	-	11.03	-	1437.06	-
455	0.87	4404.57	-	11.49	-	1569.94	-
490	0.93	3304.48	-	8.36	-	1177.84	-
525	1	0	-	0	-	0	-

Table of Results: Axial Compression (kip)

x(ft)	Span	DC	DCT	LL-Ped
0	0	846.07	-1.91	301.57
35	0.07	848.95	-1.91	302.59
70	0.13	848.95	-1.52	302.59
105	0.20	834.76	-1.36	297.54
140	0.27	800.05	-1.43	285.17
175	0.33	749.73	-1.48	267.23
210	0.40	684.81	-1.51	244.09
245	0.47	602.75	-1.55	214.84
280	0.53	501	-1.6	178.57
315	0.60	380.06	-1.65	135.47
350	0.67	245.61	-1.67	87.55
385	0.73	109.33	-1.67	38.97
420	0.80	-98.39	-1.62	-13.52
455	0.87	-131.58	-1.46	-46.9
490	0.93	-131.58	-1.15	-46.9
525	1	-99.05	-0.66	-35.31

Strength I

Using Load combination:
1.25*DC + 1.75*LL

Strength I Shear (kip)

		DC		DCT		LL-Ped	
x (ft)	Span	Pos	Neg	Pos	Neg	Pos	Neg
0	0	137.49	-	2.36	-	68.60	-
35	0.07	71.21	-	2.36	-	35.53	-
70	0.13	27.51	-10.46	0.28	-0.21	13.74	-5.22
105	0.20	23.23	-38.76	-	-0.16	11.59	-19.34
140	0.27	24.34	-43.06	-	-0.08	12.15	-21.49
175	0.33	27.61	-41.95	-	-0.03	13.77	-20.93
210	0.40	34.60	-38.68	0.01	-0.03	17.27	-19.30
245	0.47	46.19	-31.69	0.04	-	23.05	-16
280	0.53	60.89	-20	0.08	-	30.38	-10
315	0.60	74.64	-5	0.10	-	37.24	-3
350	0.67	81.36	-	0.11	-	40.60	-
385	0.73	74.13	-	0.11	-	37.00	-
420	0.80	46.45	-	0.088	-	23.19	-
455	0.87	-	-19.83	0.013	-0.11	-	-9.89
490	0.93	-	-84.88	-	-0.30	-	-42.35
525	1	-	-151.16	-	-0.30	-	-75.43

Strength I Bending Moment (kip-ft)

		DC		DCT		LL-Ped	
x(ft)	Span	Pos	Neg	Pos	Neg	Pos	Neg
0	0	-	-4413.85	-	-77	-	-2202.55
35	0.07	-	-761.64	5.788	-	-	-380.07
70	0.13	31.91	-	15.488	-	16	-
105	0.20	-	-164.93	8.138	-	-	-82.30
140	0.27	-	-512.14	2.275	-	-	-255.57
175	0.33	-	-820.44	-	-0.4	-	-409.41
210	0.40	-	-1013.99	-	-1.2	-	-506.00
245	0.47	-	-963	-	-0.825	-	-481
280	0.53	-	-506	0.63	-	-	-253
315	0.60	465.00	-	3.21	-	232.03	-
350	0.67	1917.40	-	6.78	-	956.80	-
385	0.73	3605.25	-	10.69	-	1799.05	-
420	0.80	5039.69	-	13.79	-	2514.86	-
455	0.87	5505.71	-	14.36	-	2747.40	-
490	0.93	4130.60	-	10.45	-	2061.22	-
525	1	0	-	0	-	0	-

Strength I Axial Compression (kip)

x (ft)	Span	DC	DCT	LL-Ped
0	0	1057.59	-2.39	527.75
35	0.07	1061.19	-2.39	529.53
70	0.13	1061.19	-1.90	529.53
105	0.20	1043.45	-1.70	520.70
140	0.27	1000.06	-1.79	499.05
175	0.33	937.16	-1.85	467.65
210	0.40	856.01	-1.89	427.16
245	0.47	753.44	-1.94	375.97
280	0.53	626.25	-2.00	312.50
315	0.60	475.08	-2.06	237.07
350	0.67	307.01	-2.09	153.21
385	0.73	136.66	-2.09	68.20
420	0.80	-122.99	-2.03	-23.66
455	0.87	-164.48	-1.83	-82.08
490	0.93	-164.48	-1.44	-82.08
525	1	-123.81	-0.83	-61.79

Strength I Design Shear (kip)

V_u	
Pos	Neg
208.45	-
109.10	-
41.53	-15.89
34.81	-58.26
36.48	-64.63
41.39	-62.91
51.89	-58.00
69.27	-47.49
91.34	-30.12
111.98	-8.08
122.08	-
111.23	-
69.73	-
-	-29.83
-	-127.53
-	-226.89

Strength I Design Moment (kip-ft)

M_u	
Pos	Neg
-	-6693.40
5.79	-1141.70
63.32	-
8.14	-247.23
2.28	-767.71
-	-1230.25
-	-1521.18
-	-1444.39
0.63	-759.00
700.25	-
2880.97	-
5414.99	-
7568.33	-
8267.47	-
6202.27	-
0.00	-

Strength I Axial Compression (kip)

P_u
1582.95
1588.33
1588.82
1562.45
1497.32
1402.97
1281.28
1127.47
936.75
710.09
458.14
202.77
-148.67
-248.38
-247.99
-186.43

Wind Loading

Applied to same model

Unfactored Shear Force in Weak direction (kip)

x (ft)	Span	Wind Pos	Wind Neg
0	0	26.81	-
35	0.07	23.32	-3.61
70	0.13	0.1	-7.11
105	0.20	1.83	-3.4
140	0.27	1.87	-1.67
175	0.33	1.72	-1.63
210	0.40	1.55	-1.78
245	0.47	1.49	-1.95
280	0.53	1.83	-2.01
315	0.60	2.83	-1.67
350	0.67	4.33	-0.67
385	0.73	5.51	-
420	0.80	4.88	-
455	0.87	1.38	-
490	0.93	-	-8.09
525	1	-	-11.59

Unfactored Bending Moment in Weak direction (kip-ft)

x (ft)	Span	Wind Pos	Wind Neg
0	0	651.93	-
35	0.07	-225.22	-
70	0.13	-37.46	-
105	0.20	20.42	-
140	0.27	16.67	-
175	0.33	12.47	-
210	0.40	13.69	-
245	0.47	19.94	-
280	0.53	27.16	-
315	0.60	29.77	-
350	0.67	15.82	-7.86
385	0.73	-	-74.44
420	0.80	-	-159.42
455	0.87	-	-212.57
490	0.93	-	-255.08
525	1	-	89.43

Note that torque was small and therefore neglected

Strength 3: Using Load Combination 1.24*DL+1.4*W

Strength 3 Factored Shear Force (kip)

V_u	
Pos	Neg
139.85	-
73.58	-
27.79	-11
23.23	-39
24.34	-43
27.61	-42
34.61	-39
46.23	-32
60.96	-20
74.74	-5
81.48	-
74.24	-
46.54	-
0.01	-20
-	-85
-	-151

Strength 3 Bending Moment (kip-ft)

M_u	
Pos	Neg
-	-4490.85
5.79	-761.64
47.39	-
8.14	-164.93
2.28	-512.14
-	-820.84
-	-1015.19
-	-963.84
-	-506.34
468.21	-
1924.18	-
3615.94	-
5053.48	-
5520.08	-
4141.05	-
0	-

Strength 3 Factored Shear Force in Weak Direction (kip)

Wind	
Pos	**Neg**
37.53	-
32.65	-5.05
0.14	-9.95
2.56	-4.76
2.62	-2.34
2.41	-2.28
2.17	-2.49
2.09	-2.73
2.56	-2.81
3.96	-2.34
6.06	-0.94
7.71	-
6.83	-
1.93	-
-	-11.33
-	-16.23

Strength 3 Factored Bending Moment in Weak Direction (kip-ft)

Wind	
Pos	**Neg**
912.70	-
-315.31	-
-52.44	-
28.59	-
23.34	-
17.46	-
19.17	-
27.92	-
38.02	-
41.68	-
22.15	-11.00
-	-104.22
-	-223.19
-	-297.60
-	-357.11
-	125.20

Service 1: Using Load Combination 1*DL+1*LL+0.3*W

Service 1 Factored Shear Force (kip)

V_u	
Pos	Neg
151.08	-
79.16	-
30.08	-11.52
25.2	-42.19
26.41	-46.79
29.96	-45.54
37.56	-41.99
50.15	-34.38
66.13	-21.8
81.07	-5.85
88.38	-
80.53	-
50.48	-
0.01	-21.6
-	-92.34
-	-164.27

Service 1 Bending Moment (kip-ft)

M_u	
Pos	Neg
-	-4851.28
4.63	-826.49
47.014	-
6.51	-178.97
1.82	-555.75
-	-890.62
-	-1101.29
-	-1045.67
0.5	-549.45
507.16	-
2086.08	-
3920.78	-
5479.84	-
5986	-
4490.68	-
0	-

Service 1 Factored Shear Force in Weak Direction (kip)

Wind	
Pos	Neg
8.04	-
7.00	-1.08
0.03	-2.13
0.55	-1.02
0.56	-0.50
0.52	-0.49
0.47	-0.53
0.45	-0.59
0.55	-0.60
0.85	-0.50
1.30	-0.20
1.65	-
1.46	-
0.41	-
-	-2.43
-	-3.48

Service 1 Factored Bending Moment in Weak Direction (kip-ft)

Wind	
Pos	Neg
195.58	-
-67.57	-
-11.24	-
6.13	-
5.00	-
3.74	-
4.11	-
5.98	-
8.15	-
8.93	-
4.75	-2.36
-	-22.33
-	-47.83
-	-63.77
-	-76.52
-	26.83

Reaction Forces

Unfactored Reactions on the Tower

	DC	DCT	LL-Ped
F_x	831.09	-1.98	296.23
F_z	1477.24	678.59	526.54
M_y	-5778.14	-73.35	-2059.53

Strength 1 Factored Reactions on Tower

	U
F_x	1554.79
F_z	3616.23
M_y	-10918.54

Unfactored Reactions on the South Pier

	DC	DCT	LL-Ped
F_z	149.23	0.43	53.19

Strength 1 Factored Reaction on South Pier

	U
F_z	280.16

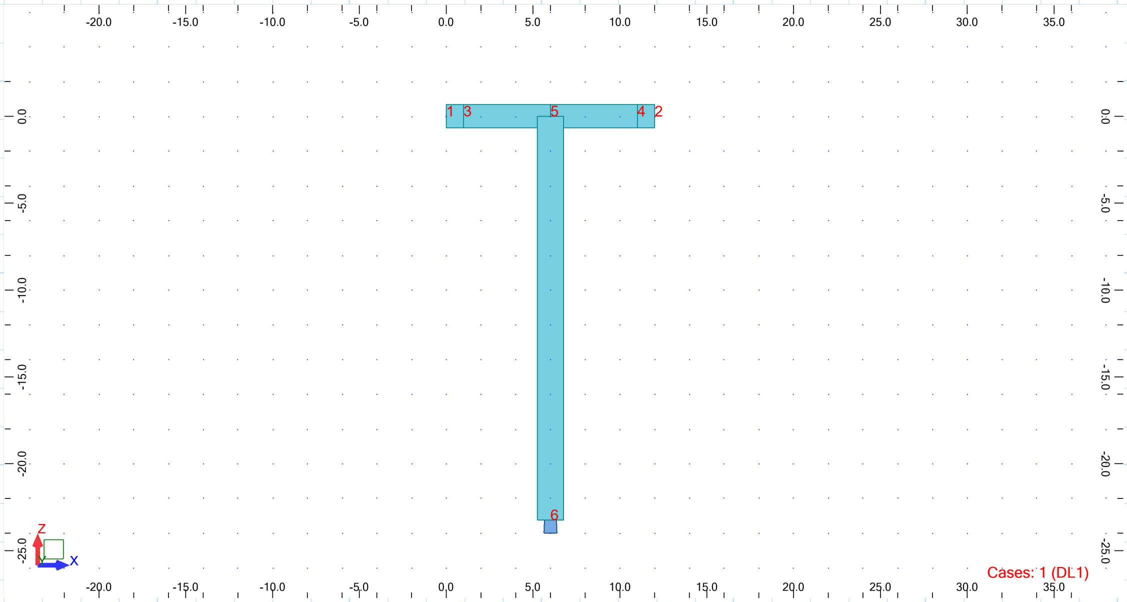
Unfactored Reactions on the Back Stay Anchorage

	DC	DCT	LL-Ped
F_x	-831.09	1.98	-296.23
F_z	-831.09	1.98	-296.23

Strength I Factored Reactions on the Back Stay Anchorage

	U
F_x	-1554.79
F_z	-1554.79

South Pier Model



Unfactored Shear Force on Pier Cap (kip)

DC	LL	SW
149.66	53.19	10.8

Unfactored Bending Moment on Pier Cap (kip-ft)

DC	LL	SW
-748.3	-265.95	-32.4

Strength I Shear Force on Pier Cap (kip)

V_u
293.66

Strength I Bending Moment on Pier Cap (kip-ft)

M_u
-1441.29

Wind loading was small on pier cap and therefore neglected

Unfactored axial compression on pier column (kip)

DC	LL	SW
299.32	106.38	52.97

Strength I factored axial compression on pier column (kip)

P_u
626.53

Bending moment caused by wind on pier column (kip-ft)

Wind
-25.64

Strength 3 factored loads on pier column

P_u	440.3625
M_u	-35.896

Service 1 factored loads on pier column

P_u	458.67
M_u	-7.692

Tower Above Deck

Using the same model as used for the girders

Unfactored Axial Compression (kip)

	DC	DCT	LL-Ped
Bottom	1367.25	676.7	487.34
Top	1367.25	-4.3	487.34

Unfactored Bending Moment (kip-ft)

	DC	DCT	LL-Ped
Bottom	-2247.05	-11.75	-800.93
Top	0	0	0

Strength 1 Factored Axial Compression (kip)

P_u
3407.78
2556.53

Strength 1 Factored Bending Moment (kip-ft)

M_u
-4225.13
0

Unfactored Wind Loading

	M_x	M_y	F_y
Bottom	335.63	2021.14	37.99
Top	335.63	-1653.04	10.99

Strength 3 Factored Force

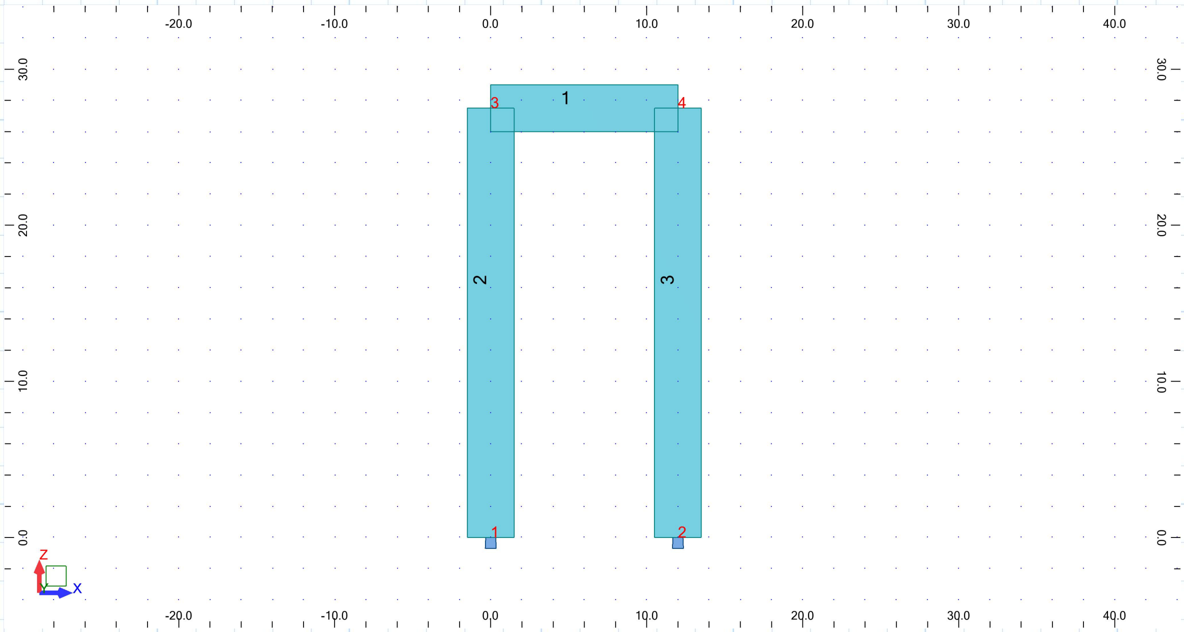
P_u	M_y	M_x
2554.938	-2823.5	2829.596

Service 1 Factored Force

P_u	M_y	M_x
2531.29	-3059.73	606.342

Tower Below Deck

Tower Model:



Unfactored Results on Beam supporting girders

	DC	LL	Wind	SW
F_z	0	0	-447.28	-16.2
F_y	0	0	0	0
M_y	0	0	2683.7	33.52
F_x	0	0	0.14	0.82

Strength I Factored Results on Beam supporting girders

F_z	-20.25
F_y	0
M_y	41.9
F_x	1.025

Strength 3 Factored Results on Beam supporting girders

F_z	-646.44
F_y	0
M_y	3799.08
F_x	0.20

Service 1 Factored Results on Beam supporting girders

F_z	-150.38
F_y	0
M_y	838.63
F_x	0.042

Unfactored Results on each Column

	DC	LL	SW	Wind
F_z	2155.83	526.54	0	-1434.83
F_y	829.11	296.23	234.27	0
M_y	0	0	10.06	-1177.5
F_x	0	0	0.82	-68.59
M_x	-28652	-10205.9	0	0

Strength 1 Factored Results on each Column

F_z	3616.23
F_y	1847.63
M_y	12.58
F_x	1.03
M_x	-53675.24

Strength 3 Factored Results on each Column

F_z	686.03
F_y	1329.23
M_y	-1635.93
F_x	-95.00
M_x	-35815.00

Service 1 Factored Results on each Column

F_z	2251.921
F_y	1359.61
M_y	-343.19
F_x	-19.757
M_x	-38857.9

B. Cable Design

Design Assumptions:

Number of cables	n	15	
Service Load (no combos)	w	2.482	KLF
Height of tower above deck	H	150	ft
Span Length	L	525	ft
Nominal tensile stress	f_{pu}	270	ksi
Allowable unit stress for cable steel	σ_a	121.5	ksi
Allowable stress under dead load effect	σ_d	108	ksi
Horizontal distance between cables	d	35	ft
Modulus of Elasticity of cables	E_s	28000	ksi
Area of strand	A_s	0.217	in^2
Area of cable	A	4.123	in^2
Number of strands per cable	n_s	19	
Unit weight of strand	γ	491.06	lbf/ft^3
Distance from top of deck to ground	h	27.5	ft

Using a 0.6" diameter, 7-wire pre-stressing strand

Each cable is spaced equally in the horizontal direction from the next cable. The vertical load is determined via tributary width of each cable.

Angle is determined based on height of tower, and distance from tower. Total tension is then determined via the angle and calculated vertical load
Sample Calculation below:

For cable 1: $w_t := 35\ \mathbf{ft}$ $\quad w := 2.482\ \mathbf{klf}$

$T_v := w_t \cdot w = 86.87\ \mathbf{kip}$ $\quad H := 75\ \mathbf{ft}$ $\quad n := 1$ $\quad d := 35\ \mathbf{ft}$

$$\theta_1 := \operatorname{atan}\left(\frac{H}{n \cdot d}\right) = 64.98\ \mathbf{deg}$$

$$T_1 := \frac{T_v}{\sin(\theta_1)} = 95.86\ \mathbf{kip} \qquad T_h := \frac{T_v}{\tan(\theta_1)} = 40.54\ \mathbf{kip}$$

$$L_{c1} := \frac{H}{\sin(\theta_1)} = 82.76\ \mathbf{ft}$$

Required area of the cable is: $A_{req} = \frac{\sigma_a}{T}$

The allowable unit stress was approximated to be the 45% nominal tensile stress of the steel wire.

$\sigma_a := 121.5 \text{ ksi}$

$A_1 := \frac{T_1}{\sigma_a} = 0.79 \text{ in}^2$

Table of Results: Cable Design

Cable	Tributary Width w_t (ft)	Angle from horizontal θ (deg)	θ (rad)	Total Tension T (kip)	Vertical Tension T_v (kip)	Horizontal Tension T_h (kip)	Length L_c (ft)	Area of Cable A (in^2)
1	35	76.87	1.34	89.20	86.87	20.27	154.0	0.734
2	35	64.98	1.13	95.86	86.87	40.54	165.5	0.789
3	35	55.01	0.96	106.04	86.87	60.81	183.1	0.873
4	35	46.97	0.82	118.83	86.87	81.08	205.2	0.978
5	35	40.60	0.71	133.48	86.87	101.35	230.5	1.099
6	35	35.54	0.62	149.46	86.87	121.62	258.1	1.230
7	35	31.48	0.55	166.37	86.87	141.89	287.3	1.369
8	35	28.18	0.49	183.96	86.87	162.16	317.6	1.514
9	35	25.46	0.44	202.05	86.87	182.43	348.9	1.663
10	35	23.20	0.40	220.53	86.87	202.70	380.8	1.815
11	35	21.29	0.37	239.29	86.87	222.97	413.2	1.969
12	35	19.65	0.34	258.28	86.87	243.24	446.0	2.126
13	35	18.25	0.32	277.46	86.87	263.51	479.1	2.284
14	35	17.02	0.30	296.77	86.87	283.78	512.4	2.443
15	18	15.95	0.28	158.11	43.435	152.02	546.0	1.301

maximum required area is 2.443 in^2
This required area determined the strand area, using 19 strands for a standard hexagonal configuration.

Backstay cable area was determined in a similar way. For strength, each backstay cable was assumed to take the total horizontal load that is acting on the tower due to the cables on the span.

Table of Results: Backstay Cable Design

Angle from horizontal		Total Tension	Vertical Tension	Horizontal Tension	Length	Required Area of Cable	Number of strands	Cable Area
θ (deg)	θ (rad)	T (kip)	T_v (kip)	T_h (kip)	L_c (ft)	A_{req} (in^2)	n	A (in^2)
45	0.79	3224.88	2280.34	2280.34	251.02	26.54	127	27.56

C. Main Span Applied Loadings

Girder Dead Load Calculations

$w_{path} := 10\ \mathbf{ft}$

$W_g := 853\ \mathbf{plf}$ Weight of girder W36x853

$\gamma_{conc} := 0.145\ \dfrac{\mathbf{kip}}{\mathbf{ft}^3}$ Specific weight of (NW) concrete

$\gamma_{rebar} := 0.005\ \dfrac{\mathbf{kip}}{\mathbf{ft}^3}$ Specific weight of rebar

$t_s := 8\ \mathbf{in}$

$L_{xs} := 12\ \mathbf{ft}$ Length of cross section

$q_{forms} := 7\ \mathbf{psf}$ weight of stay-in-place forms

$W_l := 20\ \mathbf{plf}$ weight of fence, and lighting throughout the bridge, on each side

$n := 2$ number of girders

Design Calculations

DC Components:
Deck Slab
Stay-in-place forms
Steel girders
Fence and Lighting

Deck Slab:

$$q_{deck} := t_s \cdot \left(\gamma_{conc} + \gamma_{rebar}\right) = 0.1\ \mathbf{ksf}$$

$$w_{deck} := q_{deck} \cdot \frac{L_{xs}}{n} = 0.6\ \mathbf{klf}$$

Stay in place forms:

$$w_{forms} := q_{forms} \cdot \frac{L_{xs}}{n} = 0.042\ \mathbf{klf}$$

Steel Girders:

$$w_{girder} := W_g = 0.853\ \mathbf{klf}$$

Fence and lighting

$$w_l := W_l \cdot \frac{2}{n} = 0.02\ \mathbf{klf}$$

$$w_{DC} := w_{girder} + w_{forms} + w_{deck} + w_l = 1.515\ \mathbf{klf}$$

Dead load applied per girder:

$$w_{DC} = 1.515\ \mathbf{klf}$$

Girder Live Load

$$q_{LL} := 90\ \mathbf{psf}$$

$$w_{LL} := q_{LL} \cdot \frac{L_{xs}}{n} = 0.54\ \mathbf{klf}$$

Live load applied per girder:

$$w_{LL} = 0.54\ \mathbf{klf}$$

Note: this same load is applied per set of cables

Slab Dead Load

$$q_{slab} := t_s \cdot \gamma_{conc} = 96.667\ \mathbf{psf}$$

$$q_{ds} := \frac{w_l}{12\ \mathbf{ft}} = 1.667\ \mathbf{psf}$$

$$q_{DL} := q_{slab} + q_{ds} = 98.333\ \mathbf{psf}$$

Dead load applied to slab

$$q_{DL} = 98.333\ \mathbf{psf}$$

Slab Live Load

$$q_{LL} = 90\ \mathbf{psf}$$

Dead Load of Tower Above Deck

$A_t := 5.5\ \mathbf{ft} \cdot 5.5\ \mathbf{ft} = 30.25\ \mathbf{ft}^2$

$w_{DCT} := A_t \cdot (\gamma_{conc} + \gamma_{rebar}) = 4.538\ \mathbf{klf}$

Dead load applied to tower above deck:

$w_{DCT} = 4.538\ \mathbf{klf}$

Dead Load of Tower Below Deck

Beam supporting girders:

$A_{tb} := 4\ \mathbf{ft} \cdot 4.5\ \mathbf{ft} = 18\ \mathbf{ft}^2$

$w_{tb} := A_{tb} \cdot (\gamma_{conc} + \gamma_{rebar}) = 2.7\ \mathbf{klf}$

Tower Columns:

$A_t := 5.5\ \mathbf{ft} \cdot 10\ \mathbf{ft} = 55\ \mathbf{ft}^2$

$w_{bar} := 13.6\ \mathbf{plf}$

$n_{bar} := 50$

$w_{DCT} := A_t \cdot (\gamma_{conc} + \gamma_{rebar}) = 8.25\ \mathbf{klf}$

Dead Load of South Pier

Pier Cap:

$A_p := 36\ \mathbf{in} \cdot 48\ \mathbf{in} = 12\ \mathbf{ft}^2$

$w_{DCP} := A_p \cdot (\gamma_{conc} + \gamma_{rebar}) = 1.8\ \mathbf{klf}$

Pier column:

$A_p := 3\ \mathbf{ft} \cdot 3\ \mathbf{ft} = 9\ \mathbf{ft}^2$

$w_{DCP} := A_p \cdot (\gamma_{conc} + \gamma_{rebar}) = 1.35\ \mathbf{klf}$

Wind Loadings

Design wind pressure for transverse wind loading:

$G := 0.85$

$K_{zt} := 1$

$H := 27.5\ \mathbf{ft}$

$h_g := \frac{H}{3} = 9.167\ \mathbf{ft}$

$z_{girder} := 25\ \mathbf{ft}$

$z_{towertop} := 27.5\ \mathbf{ft} + 150\ \mathbf{ft} = 177.5\ \mathbf{ft}$

Risk: II - T 1.5-1

$V := 108\ \mathbf{mph}$ - F 26.5-1D

$K_d := 0.85$

surfaceRoughness := "B"

exposure := "B"

$K_e := 1$

enclosure := "enclosed"

```
GCpi := if enclosure = "enclosed"
            ‖ 0.18
        else if enclosure = "partially enclosed"
            ‖ 0.55
        else if enclosure = "partially open"
            ‖ 0.18
        else
            ‖ 0
```

$GC_{pi} = 0.18$

$z_g := 1200\ \mathbf{ft}$

$\alpha := 7$

$$K_{z15} := 2.01 \cdot \left(\frac{15\ \mathbf{ft}}{z_g}\right)^{\frac{2}{\alpha}} = 0.575$$

$$K_{z20} := 2.01 \cdot \left(\frac{20\ \mathbf{ft}}{z_g}\right)^{\frac{2}{\alpha}} = 0.624$$

$$K_{zgirder} := 2.01 \cdot \left(\frac{z_{girder}}{z_g}\right)^{\frac{2}{\alpha}} = 0.665$$

$$K_{z25} := 2.01 \cdot \left(\frac{25\ \mathbf{ft}}{z_g}\right)^{\frac{2}{\alpha}} = 0.665$$

$K_{z30} := 0.7$ $K_{z40} := 0.76$ $K_{z50} := 0.81$ $K_{z60} := 0.85$ $K_{z70} := 0.89$

$K_{z80} := 0.93$ $K_{z90} := 0.96$ $K_{z100} := 0.99$ $K_{z120} := 1.04$ $K_{z140} := 1.09$

$K_{z160} := 1.13$ $K_{z180} := 1.17$

$$z_{towertop} := 27.5\ \mathbf{ft} + 150\ \mathbf{ft} = 177.5\ \mathbf{ft}$$

$$K_h := \frac{(177.5 - 180)}{160 - 180} \cdot 1.13 + \frac{(177.5 - 160)}{180 - 160} \cdot 1.17 = 1.165$$

$$q_{15} := 0.00256\ \frac{psf}{mph^2} \cdot K_{z15} \cdot K_{zt} \cdot K_d \cdot K_e \cdot V^2 = 14.587\ psf$$

$$q_{20} := 0.00256\ \frac{psf}{mph^2} \cdot K_{z20} \cdot K_{zt} \cdot K_d \cdot K_e \cdot V^2 = 15.837\ psf$$

$$q_{25} := 0.00256\ \frac{psf}{mph^2} \cdot K_{z25} \cdot K_{zt} \cdot K_d \cdot K_e \cdot V^2 = 16.879\ psf$$

$$q_{30} := 0.00256\ \frac{psf}{mph^2} \cdot K_{z30} \cdot K_{zt} \cdot K_d \cdot K_e \cdot V^2 = 17.767\ psf$$

$$q_{40} := 0.00256\ \frac{psf}{mph^2} \cdot K_{z40} \cdot K_{zt} \cdot K_d \cdot K_e \cdot V^2 = 19.289\ psf$$

$$q_{50} := 0.00256\ \frac{psf}{mph^2} \cdot K_{z50} \cdot K_{zt} \cdot K_d \cdot K_e \cdot V^2 = 20.558\ psf$$

$$q_{60} := 0.00256\ \frac{psf}{mph^2} \cdot K_{z60} \cdot K_{zt} \cdot K_d \cdot K_e \cdot V^2 = 21.574\ psf$$

$$q_{70} := 0.00256\ \frac{psf}{mph^2} \cdot K_{z70} \cdot K_{zt} \cdot K_d \cdot K_e \cdot V^2 = 22.589\ psf$$

$$q_{80} := 0.00256\ \frac{psf}{mph^2} \cdot K_{z80} \cdot K_{zt} \cdot K_d \cdot K_e \cdot V^2 = 23.604\ psf$$

$$q_{90} := 0.00256\ \frac{psf}{mph^2} \cdot K_{z90} \cdot K_{zt} \cdot K_d \cdot K_e \cdot V^2 = 24.366\ psf$$

$$q_{100} := 0.00256\ \frac{psf}{mph^2} \cdot K_{z100} \cdot K_{zt} \cdot K_d \cdot K_e \cdot V^2 = 25.127\ psf$$

$$q_{120} := 0.00256\ \frac{psf}{mph^2} \cdot K_{z120} \cdot K_{zt} \cdot K_d \cdot K_e \cdot V^2 = 26.396\ psf$$

$$q_{140} := 0.00256\ \frac{psf}{mph^2} \cdot K_{z140} \cdot K_{zt} \cdot K_d \cdot K_e \cdot V^2 = 27.665\ psf$$

$$q_{160} := 0.00256\ \frac{psf}{mph^2} \cdot K_{z160} \cdot K_{zt} \cdot K_d \cdot K_e \cdot V^2 = 28.68\ psf$$

$$q_{180} := 0.00256\ \frac{psf}{mph^2} \cdot K_{z180} \cdot K_{zt} \cdot K_d \cdot K_e \cdot V^2 = 29.696\ psf$$

$$q_{girder} := 0.00256\ \frac{psf}{mph^2} \cdot K_{zgirder} \cdot K_{zt} \cdot K_d \cdot K_e \cdot V^2 = 16.879\ psf$$

$$q_h := 0.00256\ \frac{psf}{mph^2} \cdot K_h \cdot K_{zt} \cdot K_d \cdot K_e \cdot V^2 = 29.569\ psf$$

$C_{p\_w} := 0.8$

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

$p_{15\_w\_p} := q_{15} \cdot G \cdot C_{p\_w} = 9.919$ **psf**

$p_{20\_w\_p} := q_{20} \cdot G \cdot C_{p\_w} = 10.769$ **psf**

$p_{25\_w\_p} := q_{25} \cdot G \cdot C_{p\_w} = 11.478$ **psf**

$p_{30} := q_{30} \cdot G \cdot C_{p\_w} = 12.081$ **psf**

$p_{40} := q_{40} \cdot G \cdot C_{p\_w} = 13.117$ **psf**

$p_{50} := q_{50} \cdot G \cdot C_{p\_w} = 13.98$ **psf**

$p_{60} := q_{60} \cdot G \cdot C_{p\_w} = 14.67$ **psf**

$p_{70} := q_{70} \cdot G \cdot C_{p\_w} = 15.36$ **psf**

$p_{80} := q_{80} \cdot G \cdot C_{p\_w} = 16.051$ **psf**

$p_{90} := q_{90} \cdot G \cdot C_{p\_w} = 16.569$ **psf**

$p_{100} := q_{100} \cdot G \cdot C_{p\_w} = 17.086$ **psf**

$p_{120} := q_{120} \cdot G \cdot C_{p\_w} = 17.949$ **psf**

$p_{140} := q_{140} \cdot G \cdot C_{p\_w} = 18.812$ **psf**

$p_{160} := q_{160} \cdot G \cdot C_{p\_w} = 19.503$ **psf**

$p_{180} := q_{180} \cdot G \cdot C_{p\_w} = 20.193$ **psf**

$p_{girder} := q_{girder} \cdot G \cdot C_{p\_w} = 11.478$ **psf**

$p_h := q_h \cdot G \cdot C_{p\_w} = 20.107$ **psf**

$p_{l\_p} := q_h \cdot G \cdot C_{p\_l} = -12.567$ **psf**

$p_{15\_w\_n} := q_{15} \cdot G \cdot C_{p\_w} = 9.919$ **psf**

$p_{20\_w\_n} := q_{20} \cdot G \cdot C_{p\_w} = 10.769$ **psf**

$p_{25\_w\_n} := q_{25} \cdot G \cdot C_{p\_w} = 11.478$ **psf**

$p_{30\_w\_n} := q_{30} \cdot G \cdot C_{p\_w} = 12.081$ **psf**

$p_{40\_w\_n} := q_{40} \cdot G \cdot C_{p\_w} = 13.117$ **psf**

$p_{50\_w\_n} := q_{50} \cdot G \cdot C_{p\_w} = 13.98$ **psf**

$p_{60\_w\_n} := q_{60} \cdot G \cdot C_{p\_w} = 14.67$ **psf**

$p_{70\_w\_n} := q_{70} \cdot G \cdot C_{p\_w} = 15.36$ **psf**

$p_{80\_w\_n} := q_{80} \cdot G \cdot C_{p\_w} = 16.051$ **psf**

$p_{90\_w\_n} := q_{90} \cdot G \cdot C_{p\_w} = 16.569$ **psf**

$p_{100\_w\_n} := q_{100} \cdot G \cdot C_{p\_w} = 17.086$ **psf**

$p_{120\_w\_n} := q_{120} \cdot G \cdot C_{p\_w} = 17.949$ **psf**

$p_{140\_w\_n} := q_{140} \cdot G \cdot C_{p\_w} = 18.812$ **psf**

$p_{160\_w\_n} := q_{160} \cdot G \cdot C_{p\_w} = 19.503$ **psf**

$p_{180\_w\_n} := q_{180} \cdot G \cdot C_{p\_w} = 20.193$ **psf**

$p_{h\_w\_n} := q_h \cdot G \cdot C_{p\_w} = 20.107$ **psf**

$p_{g\_w\_n} := q_{girder} \cdot G \cdot C_{p\_w} = 11.478$ **psf**

$p_{l\_n} := q_h \cdot G \cdot C_{p\_l} = -12.567$ **psf**

$p_{15\_n\_net} := p_{15\_w\_n} - p_{l\_n} = 22.486$ **psf**

$p_{20\_n\_net} := p_{20\_w\_n} - p_{l\_n} = 23.336$ **psf**

$p_{25\_n\_net} := p_{25\_w\_n} - p_{l\_n} = 24.044$ **psf**

$p_{30\_n\_net} := p_{30\_w\_n} - p_{l\_n} = 24.648$ **psf**

$p_{40\_n\_net} := p_{40\_w\_n} - p_{l\_n} = 25.684$ **psf**

$p_{50\_n\_net} := p_{50\_w\_n} - p_{l\_n} = 26.546$ **psf**

$p_{60\_n\_net} := p_{60\_w\_n} - p_{l\_n} = 27.237$ **psf**

$p_{70\_n\_net} := p_{70\_w\_n} - p_{l\_n} = 27.927$ **psf**

$p_{80\_n\_net} := p_{80\_w\_n} - p_{l\_n} = 28.618$ **psf**

$p_{90\_n\_net} := p_{90\_w\_n} - p_{l\_n} = 29.135$ **psf**

$p_{100\_n\_net} := p_{100\_w\_n} - p_{l\_n} = 29.653$ **psf**

$p_{120\_n\_net} := p_{120\_w\_n} - p_{l\_n} = 30.516$ **psf**

$p_{140\_n\_net} := p_{140\_w\_n} - p_{l\_n} = 31.379$ **psf**

$p_{160\_n\_net} := p_{160\_w\_n} - p_{l\_n} = 32.069$ **psf**

$p_{180\_n\_net} := p_{180\_w\_n} - p_{l\_n} = 32.76$ **psf**

$p_{h\_n\_net} := p_{h\_w\_n} - p_{l\_n} = 32.673$ **psf**

$p_{g\_n\_net} := p_{g\_w\_n} - p_{l\_n} = 24.044$ **psf**

Tower is 5.5' in width

$w_h := p_{h\_n\_net} \cdot 5.5\ \mathbf{ft} = 0.18\ \mathbf{klf}$

To be conservative, a consistent loading of 0.18 klf will be applied to the tower above deck

Wind loading applied to girder:

$w_g := p_{g\_n\_net} \cdot 51.1\ \mathbf{in} = 0.102\ \mathbf{klf}$

Wind loading applied to south pier:

Pier Cap:

$w_{pc} := p_{g\_n\_net} \cdot 48\ \mathbf{in} \cdot 36\ \mathbf{in} = 0.289\ \mathbf{kip}$

Pier column:

$w_{pcol} := p_{g\_n\_net} \cdot 36\ \mathbf{in} = 0.072\ \mathbf{klf}$

Wind loading applied to tower below deck:

$w_{ptower} := p_{g\_n\_net} \cdot 10\ \mathbf{ft} = 0.24\ \mathbf{klf}$

D. Tower Design

Concrete Properties:

$f'_c := 4\ \mathbf{ksi}$ $w_c := 145\ \mathbf{pcf}$ $\nu := 0.2$

$E_c := 1820 \cdot \sqrt{\frac{f'_c}{\mathbf{ksi}}} \cdot \mathbf{ksi} = 3640\ \mathbf{ksi}$

Cross-Section Dimensions: $b := 5.5\ \mathbf{ft}$ $h := 5.5\ \mathbf{ft}$

$A_g := b \cdot h = 30.25\ \mathbf{ft}^2$

Steel properties: $f_y := 60\ \mathbf{ksi}$ Using 16 #18 bars

$n_{bars} := 16$ $A_{bar} := 2.257\ \mathbf{in}^2$ $A_{st} := n_{bars} \cdot A_{bar}$

Required strength in Strength 1: $P_u := 3407.8\ \mathbf{kip}$ $M_u := 4225.13\ \mathbf{kip} \cdot \mathbf{ft}$

$\frac{P_u}{A_g} = 0.782\ \mathbf{ksi}$ $\rho_g := \frac{A_{st}}{A_g} = 0.008$

Axial load capacity for tied columns

$\phi P_n := 0.8 \cdot 0.65 \cdot \left(0.85 \cdot f'_c \cdot A_g + \left(f_y - 0.85 \cdot f'_c\right) \cdot A_{st}\right)$

$P_r := \phi P_n = 8764.256\ \mathbf{kip}$

$\frac{P_u}{P_r} \leq 1 = 1$

Combined Axial and Flexure: Create P-M interaction diagram

Point A

$P_{rA} := \phi P_n = 8764.256\ \mathbf{kip}$

$\phi M_{nA} := 0\ \mathbf{kip} \cdot \mathbf{ft}$ $M_{rA} := \phi M_{nA}$

Point B

$E_s := 29000\ \mathbf{ksi}$

$\varepsilon_{cu} := 0.003$

$d_{ties} := 0.5\ \mathbf{in}$ $\qquad d_{bar} := 1.693\ \mathbf{in}$

$c_c := 2\ \mathbf{in}$

$y_{bar} := \frac{h}{2} = 2.75\ \mathbf{ft}$

$A_{s1} := 5 \cdot A_{bar}$ $\qquad y_{s1} := c_c + d_{ties} + \frac{d_{bar}}{2} = 3.347\ \mathbf{in}$

$A_{s2} := 2 \cdot A_{bar}$

$A_{s3} := 2 \cdot A_{bar}$

$A_{s4} := 2 \cdot A_{bar}$

$A_{s5} := 5 \cdot A_{bar}$ $\qquad y_{s5} := h - \left(c_c + d_{ties} + \frac{d_{bar}}{2}\right) = 62.654\ \mathbf{in}$

$\frac{y_{s5} - y_{s1}}{4} = 1.236\ \mathbf{ft}$

$y_{s2} := y_{s1} + \frac{y_{s5} - y_{s1}}{4} = 18.173\ \mathbf{in}$

$y_{s3} := y_{s2} + \frac{y_{s5} - y_{s1}}{4} = 33\ \mathbf{in}$

$y_{s4} := y_{s3} + \frac{y_{s5} - y_{s1}}{4} = 47.827\ \mathbf{in}$

$\varepsilon_{ty} := \frac{f_y}{E_s} = 0.002$

$\varepsilon_{s1} := 0$ $\qquad \varepsilon_o := \frac{\varepsilon_{s1} \cdot h - \varepsilon_{cu} \cdot y_{s1}}{h - y_{s1}} = -1.602 \cdot 10^{-4}$

$c := \frac{\varepsilon_{cu} \cdot h}{\varepsilon_{cu} - \varepsilon_o} = 62.654\ \mathbf{in}$

$\beta_1 := 0.85$

$a := \beta_1 \cdot c = 4.438\ \mathbf{ft}$

$\varepsilon_{s2} := \frac{(h - y_{s2})}{h} \cdot \varepsilon_o + \frac{y_{s2}}{h} \cdot \varepsilon_{cu} = 0.00071$

$$\varepsilon_{s3} := \frac{(h - y_{s3})}{h} \cdot \varepsilon_o + \frac{y_{s3}}{h} \cdot \varepsilon_{cu} = 0.00142$$

$$\varepsilon_{s4} := \frac{(h - y_{s4})}{h} \cdot \varepsilon_o + \frac{y_{s4}}{h} \cdot \varepsilon_{cu} = 0.00213$$

$$\varepsilon_{s5} := \frac{(h - y_{s5})}{h} \cdot \varepsilon_o + \frac{y_{s5}}{h} \cdot \varepsilon_{cu} = 0.00284$$

stresses $\varepsilon_{ty} = 0.00207$

$f_{s1} := 0$ **ksi** strain was 0 $F_{s1} := A_{s1} \cdot f_{s1} = 0$ **kip**

$\varepsilon_{s2} < \varepsilon_{ty} = 1$ elastic in compression

$f_{s2} := E_s \cdot \varepsilon_{s2} - 0.85 \cdot f'_c = 17.188$ **ksi** $F_{s2} := A_{s2} \cdot f_{s2} = 77.588$ **kip**

$\varepsilon_{s3} < \varepsilon_{ty} = 1$ elastic in compression

$f_{s3} := E_s \cdot \varepsilon_{s3} - 0.85 \cdot f'_c = 37.777$ **ksi** $F_{s3} := A_{s3} \cdot f_{s3} = 170.523$ **kip**

$\varepsilon_{s4} > \varepsilon_{ty} = 1$ elastic in compression

$f_{s4} := f_y - 0.85 \cdot f'_c = 56.6$ **ksi** $F_{s4} := A_{s4} \cdot f_{s4} = 255.492$ **kip**

$\varepsilon_{s5} > \varepsilon_{ty} = 1$ yielding in compression

$f_{s5} := f_y - 0.85\ f'_c = 56.6$ **ksi** $F_{s5} := A_{s5} \cdot f_{s5} = 638.731$ **kip**

$A_c := b \cdot a = 24.409$ $\mathbf{ft}^2$

$F_c := 0.85 \cdot f'_c \cdot A_c = 11950.529$ **kip**

$y_c := h - \frac{a}{2} = 3.281$ **ft**

$P_n := F_c + F_{s1} + F_{s2} + F_{s3} + F_{s4} + F_{s5} = 13092.863$ **kip**

$$M_n := F_c \cdot (y_c - y_{bar}) + F_{s1} \cdot (y_{s1} - y_{bar}) + F_{s2} \cdot (y_{s2} - y_{bar}) + F_{s3} \cdot (y_{s3} - y_{bar}) + F_{s4} \cdot (y_{s4} - y_{bar}) + F_{s5} \cdot (y_{s5} - y_{bar})$$

$M_n = 8144.188$ **kip·ft**

$\varepsilon_t := \varepsilon_{s1} = 0$

$\phi := 0.65$

$P_{rB} := \phi \cdot P_n = 8510.361\ \textbf{kip}$

$M_{rB} := \phi \cdot M_n = 5293.722\ \textbf{kip} \cdot \textbf{ft}$

Point C

$\varepsilon_{s1} := \dfrac{-f_y}{E_s} = -0.002$

$\varepsilon_o := \dfrac{\varepsilon_{s1} \cdot h - \varepsilon_{cu} \cdot y_{s1}}{h - y_{s1}} = -0.002$

$c := \dfrac{\varepsilon_{cu} \cdot h}{\varepsilon_{cu} - \varepsilon_o} = 37.081\ \textbf{in}$

$a := \beta_1 \cdot c = 2.627\ \textbf{ft}$

$\varepsilon_{s2} := \dfrac{(h - y_{s2})}{h} \cdot \varepsilon_o + \dfrac{y_{s2}}{h} \cdot \varepsilon_{cu} = -0.00087$

$\varepsilon_{s3} := \dfrac{(h - y_{s3})}{h} \cdot \varepsilon_o + \dfrac{y_{s3}}{h} \cdot \varepsilon_{cu} = 0.00033$

$\varepsilon_{s4} := \dfrac{(h - y_{s4})}{h} \cdot \varepsilon_o + \dfrac{y_{s4}}{h} \cdot \varepsilon_{cu} = 0.00153$

$\varepsilon_{s5} := \dfrac{(h - y_{s5})}{h} \cdot \varepsilon_o + \dfrac{y_{s5}}{h} \cdot \varepsilon_{cu} = 0.00273$

stresses

$\varepsilon_{ty} = 0.00207$

$|\varepsilon_{s1}| \geq \varepsilon_{ty} = 1$ $\varepsilon_{s1} < 0 = 1$ yielded in tension

$f_{s1} := -f_y = -60\ \textbf{ksi}$ $F_{s1} := A_{s1} \cdot f_{s1} = -677.1\ \textbf{kip}$

$|\varepsilon_{s2}| < \varepsilon_{ty} = 1$ $\varepsilon_{s2} < 0 = 1$ elastic in tension

$f_{s2} := E_s \cdot \varepsilon_{s2} = -25.213\ \textbf{ksi}$ $F_{s2} := A_{s2} \cdot f_{s2} = -113.811\ \textbf{kip}$

$\varepsilon_{s3} < \varepsilon_{ty} = 1$ $\varepsilon_{s3} > 0 = 1$ elastic in compression

$f_{s3} := E_s \cdot \varepsilon_{s3} - 0.85 \cdot f'_c = 6.174\ \textbf{ksi}$ $F_{s3} := A_{s3} \cdot f_{s3} = 27870.149\ \textbf{lbf}$

$\varepsilon_{s4} < \varepsilon_{ty} = 1$ $\varepsilon_{s4} > 0 = 1$ elastic in compression

$f_{s4} := E_s \cdot \varepsilon_{s4} - 0.85 \cdot f'_c = 40.961 \ \mathbf{ksi}$ $F_{s4} := A_{s4} \cdot f_{s4} = 184899.023 \ \mathbf{lbf}$

$\varepsilon_{s5} > \varepsilon_{ty} = 1$ $\varepsilon_{s5} > 0 = 1$ yielding in compression

$f_{s5} := f_y - 0.85 \ f'_c = 56.6 \ \mathbf{ksi}$ $F_{s5} := A_{s5} \cdot f_{s5} = 638731 \ \mathbf{lbf}$

$A_c := b \cdot a = 14.446 \ \mathbf{ft}^2$

$F_c := 0.85 \cdot f'_c \cdot A_c = 7072761.819 \ \mathbf{lbf}$

$y_c := h - \frac{a}{2} = 4.187 \ \mathbf{ft}$

$P_n := F_c + F_{s1} + F_{s2} + F_{s3} + F_{s4} + F_{s5} = 7133.351 \ \mathbf{kip}$

$$M_n := F_c \cdot (y_c - y_{bar}) + F_{s1} \cdot (y_{s1} - y_{bar}) + F_{s2} \cdot (y_{s2} - y_{bar}) + F_{s3} \cdot (y_{s3} - y_{bar}) + F_{s4} \cdot (y_{s4} - y_{bar}) + F_{s5} \cdot (y_{s5} - y_{bar})$$

$M_n = 13782.287 \ \mathbf{kip \cdot ft}$

$\varepsilon_t := |\varepsilon_{s1}| = 0.002$

$\phi := 0.65$

$P_{rC} := \phi \cdot P_n = 4636.678 \ \mathbf{kip}$

$M_{rC} := \phi \cdot M_n = 8958.487 \ \mathbf{kip \cdot ft}$

Point D

$\varepsilon_{s1} := -0.005$ $\varepsilon_o := \frac{\varepsilon_{s1} \cdot h - \varepsilon_{cu} \cdot y_{s1}}{h - y_{s1}} = -0.005$

$c := \frac{\varepsilon_{cu} \cdot h}{\varepsilon_{cu} - \varepsilon_o} = 23.495 \ \mathbf{in}$

$a := \beta_1 \cdot c = 1.664 \ \mathbf{ft}$

$$\varepsilon_{s2} := \frac{(h - y_{s2})}{h} \cdot \varepsilon_o + \frac{y_{s2}}{h} \cdot \varepsilon_{cu} = -0.00311$$

$$\varepsilon_{s3} := \frac{(h - y_{s3})}{h} \cdot \varepsilon_o + \frac{y_{s3}}{h} \cdot \varepsilon_{cu} = -0.00121$$

$$\varepsilon_{s4} := \frac{(h - y_{s4})}{h} \cdot \varepsilon_o + \frac{y_{s4}}{h} \cdot \varepsilon_{cu} = 0.00068$$

$$\varepsilon_{s5} := \frac{(h - y_{s5})}{h} \cdot \varepsilon_o + \frac{y_{s5}}{h} \cdot \varepsilon_{cu} = 0.00257$$

stresses $\varepsilon_{ty} = 0.00207$

$|\varepsilon_{s1}| \geq \varepsilon_{ty} = 1$ $\varepsilon_{s1} < 0 = 1$ yielded in tension

$f_{s1} := -f_y = -60$ **ksi** $F_{s1} := A_{s1} \cdot f_{s1} = -677.1$ **kip**

$|\varepsilon_{s2}| \geq \varepsilon_{ty} = 1$ $\varepsilon_{s2} < 0 = 1$ yielded in tension

$f_{s2} := -f_y = -60$ **ksi** $F_{s2} := A_{s2} \cdot f_{s2} = -270.84$ **kip**

$|\varepsilon_{s3}| < \varepsilon_{ty} = 1$ $\varepsilon_{s3} < 0 = 1$ elastic in tension

$f_{s3} := E_s \cdot \varepsilon_{s3} = -35.196$ **ksi** $F_{s3} := A_{s3} \cdot f_{s3} = -158874.233$ **lbf**

$|\varepsilon_{s4}| < \varepsilon_{ty} = 1$ $\varepsilon_{s4} > 0 = 1$ elastic in compression

$f_{s4} := E_s \cdot \varepsilon_{s4} - 0.85 \cdot f'_c = 16.306$ **ksi** $F_{s4} := A_{s4} \cdot f_{s4} = 73606.05$ **lbf**

$\varepsilon_{s5} > \varepsilon_{ty} = 1$ $\varepsilon_{s5} > 0 = 1$ yielding in compression

$f_{s5} := f_y - 0.85\ f'_c = 56.6$ **ksi** $F_{s5} := A_{s5} \cdot f_{s5} = 638731$ **lbf**

$A_c := b \cdot a = 9.153$ $\mathbf{ft}^2$

$F_c := 0.85 \cdot f'_c \cdot A_c = 4481448.221$ **lbf**

$$y_c := h - \frac{a}{2} = 4.668 \ \mathbf{ft}$$

$P_n := F_c + F_{s1} + F_{s2} + F_{s3} + F_{s4} + F_{s5} = 4086.971$ **kip**

$$M_n := F_c \cdot (y_c - y_{bar}) + F_{s1} \cdot (y_{s1} - y_{bar}) + F_{s2} \cdot (y_{s2} - y_{bar}) + F_{s3} \cdot (y_{s3} - y_{bar}) + F_{s4} \cdot (y_{s4} - y_{bar}) + F_{s5} \cdot (y_{s5} - y_{bar})$$

$M_n = 12272.062\ \mathbf{kip \cdot ft}$

$\varepsilon_t := |\varepsilon_{s1}| = 0.005$

$\varepsilon_t > \varepsilon_{ty} = 1$

$\varepsilon_t < \varepsilon_{ty} + 0.003 = 1$

$$\phi := 0.65 + 0.25 \cdot \left(\frac{\varepsilon_t - \varepsilon_{ty}}{0.003} \right) = 0.894$$

$P_{rD} := \phi \cdot P_n = 3654.786\ \mathbf{kip}$

$M_{rD} := \phi \cdot M_n = 10974.327\ \mathbf{kip \cdot ft}$

Point E

$\varepsilon_{s1} := -0.02$

$$\varepsilon_o := \frac{\varepsilon_{s1} \cdot h - \varepsilon_{cu} \cdot y_{s1}}{h - y_{s1}} = -0.021$$

$$c := \frac{\varepsilon_{cu} \cdot h}{\varepsilon_{cu} - \varepsilon_o} = 8.172\ \mathbf{in}$$

$a := \beta_1 \cdot c = 0.579\ \mathbf{ft}$

$$\varepsilon_{s2} := \frac{(h - y_{s2})}{h} \cdot \varepsilon_o + \frac{y_{s2}}{h} \cdot \varepsilon_{cu} = -0.01456$$

$$\varepsilon_{s3} := \frac{(h - y_{s3})}{h} \cdot \varepsilon_o + \frac{y_{s3}}{h} \cdot \varepsilon_{cu} = -0.00911$$

$$\varepsilon_{s4} := \frac{(h - y_{s4})}{h} \cdot \varepsilon_o + \frac{y_{s4}}{h} \cdot \varepsilon_{cu} = -0.00367$$

$$\varepsilon_{s5} := \frac{(h - y_{s5})}{h} \cdot \varepsilon_o + \frac{y_{s5}}{h} \cdot \varepsilon_{cu} = 0.00177$$

stresses $\varepsilon_{ty} = 0.00207$

$|\varepsilon_{s1}| \geq \varepsilon_{ty} = 1$ $\varepsilon_{s1} < 0 = 1$ yielded in tension

$f_{s1} := -f_y = -60$ **ksi** $F_{s1} := A_{s1} \cdot f_{s1} = -677.1$ **kip**

$|\varepsilon_{s2}| \geq \varepsilon_{ty} = 1$ $\varepsilon_{s2} < 0 = 1$ yielded in tension

$f_{s2} := -f_y = -60$ **ksi** $F_{s2} := A_{s2} \cdot f_{s2} = -270.84$ **kip**

$|\varepsilon_{s3}| \geq \varepsilon_{ty} = 1$ $\varepsilon_{s3} < 0 = 1$ yielded in tension

$f_{s3} := -f_y = -60$ **ksi** $F_{s3} := A_{s3} \cdot f_{s3} = -270840$ **lbf**

$|\varepsilon_{s4}| \geq \varepsilon_{ty} = 1$ $\varepsilon_{s4} < 0 = 1$ yielded in tension

$f_{s4} := -f_y = -60$ **ksi** $F_{s4} := A_{s4} \cdot f_{s4} = -270840$ **lbf**

$|\varepsilon_{s5}| < \varepsilon_{ty} = 1$ $\varepsilon_{s5} > 0 = 1$ elastic in compression

$f_{s5} := E_s \cdot \varepsilon_{s5} - 0.85 \cdot f'_c = 47.974$ **ksi** $F_{s5} := A_{s5} \cdot f_{s5} = 541.383$ **kip**

$A_c := b \cdot a = 3.184$ **ft**^{2}

$F_c := 0.85 \cdot f'_c \cdot A_c = 1558.765$ **kip**

$y_c := h - \frac{a}{2} = 5.211$ **ft**

$P_n := F_c + F_{s1} + F_{s2} + F_{s3} + F_{s4} + F_{s5} = 610.527$ **kip**

$$M_n := F_c \cdot (y_c - y_{bar}) + F_{s1} \cdot (y_{s1} - y_{bar}) + F_{s2} \cdot (y_{s2} - y_{bar}) + F_{s3} \cdot (y_{s3} - y_{bar}) + F_{s4} \cdot (y_{s4} - y_{bar}) + F_{s5} \cdot (y_{s5} - y_{bar})$$

$M_n = 6846.469$ **kip · ft**

$\varepsilon_t := |\varepsilon_{s1}| = 0.02$

$\varepsilon_t := |\varepsilon_{s1}| = 0.02$

$\varepsilon_t > \varepsilon_{ty} = 1$

$\varepsilon_t > \varepsilon_{ty} + 0.003 = 1$

$\phi := 0.9$

$P_{rE} := \phi \cdot P_n = 549.475$ **kip**

$M_{rE} := \phi \cdot M_n = 6161.822$ **kip·ft**

Summary:

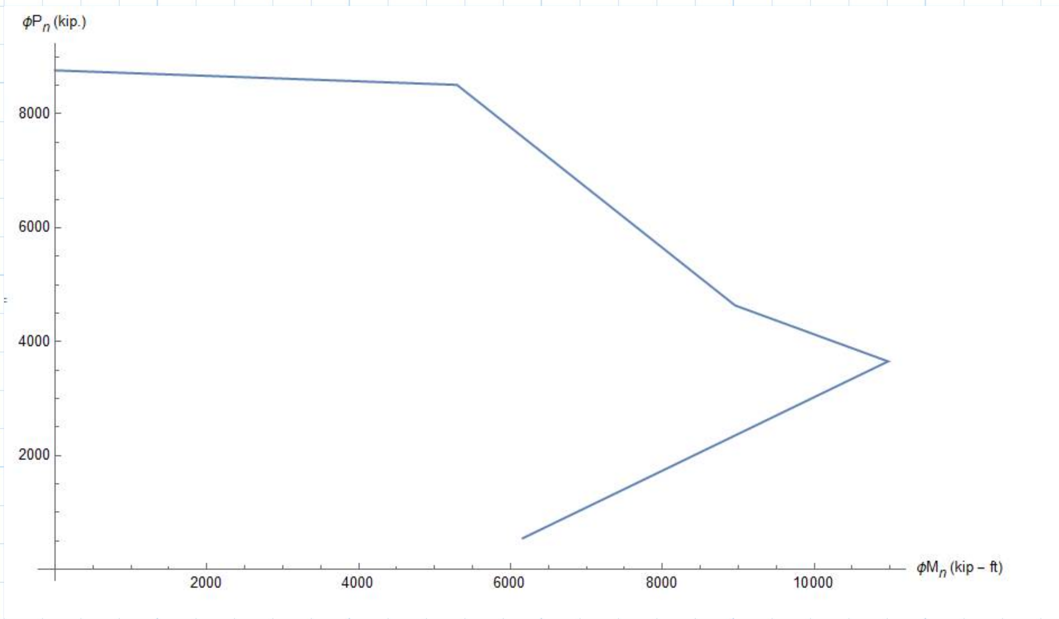
$P_{rA} = 8764.256$ **kip** $M_{rA} = 0$ **kip·ft**

$P_{rB} = 8510.361$ **kip** $M_{rB} = 5293.722$ **kip·ft**

$P_{rC} = 4636.678$ **kip** $M_{rC} = 8958.487$ **kip·ft**

$P_{rD} = 3654.786$ **kip** $M_{rD} = 10974.327$ **kip·ft**

$P_{rE} = 549.475$ **kip** $M_{rE} = 6161.822$ **kip·ft**



$P_u = 3407.8$ **kip** $M_u = 4225.13$ **kip·ft**

This falls under the curve, therefore OK

Buckling Strength

$h = 5.5\ \mathrm{ft}$ $b = 5.5\ \mathrm{ft}$

$A_{st} = 0.251\ \mathrm{ft}^2$ $A_g := h \cdot b = 30.25\ \mathrm{ft}^2$

$A_g = 30.25\ \mathrm{ft}^2$

$A_c := A_g - A_{st} = 29.999\ \mathrm{ft}^2$

$\phi P_n := 0.8 \cdot 0.65 \cdot \left(0.85 \cdot f'_c \cdot A_c + f_y \cdot A_{st}\right) = 8764.256\ \mathrm{kip}$

$r := 0.288 \cdot h = 1.584\ \mathrm{ft}$

$L := 150\ \mathrm{ft}$ $k := 2.1$

$$F_e := \frac{\pi^2 \cdot E_c}{\left(\frac{k \cdot L}{r}\right)^2} = 0.908\ \mathrm{ksi}$$

$P_{cr} := F_e \cdot A_g = 3957.111\ \mathrm{kip}$

Buckling controls. This force is greater than Pu

Reinforcement

minimal horizontal clear spacing

$s_{bc} := \max\left(1\ \mathrm{in}, d_{bar}\right) = 1.693\ \mathrm{in}$

minimal vertical clear spacing

$s_{hc} := 1\ \mathrm{in}$

$16 \cdot d_{bar} = 2.257\ \mathrm{ft}$

maximum vertical spacing
between ties

$b = 5.5\ \mathrm{ft}$

$48 \cdot d_{ties} = 2\ \mathrm{ft}$

$s := 2\ \mathrm{ft}$

Wind loading

Biaxial bending
The tower is symmetrical, therefore has the same flexural resistance in both the x and y directions.

Check Biaxial interaction

Strength 3
Tower must support:

$P_u := 2554.9\ \mathbf{kip}$

$M_{uy} := 2823.5\ \mathbf{kip \cdot ft}$

$M_{ux} := 2829.6\ \mathbf{kip \cdot ft}$

From the diagram.

$\phi\ P_{ny} := 8500\ \mathbf{kip}$ $\quad\quad \phi\ P_{nx} := 8500\ \mathbf{kip}$

$\phi\ P_{n0} := P_{rA} = 8764.256\ \mathbf{kip}$

$$\phi\ P_{neq} := \frac{1}{\left(\frac{1}{\phi\ P_{ny}} + \frac{1}{\phi\ P_{nx}} + \frac{1}{\phi\ P_{n0}}\right)} = 2862.099\ \mathbf{kip}$$

$P_u < \phi\ P_{neq} = 1$ Therefore OK in Strength 3

Service 1

$P_u := 2531.29\ \mathbf{kip}$

$M_{uy} := 606.34\ \mathbf{kip \cdot ft}$

$M_{ux} := 3059.73\ \mathbf{kip \cdot ft}$

From the diagram

$\phi\ P_{n0} := P_{rA} = 8764.256\ \mathbf{kip}$

$\phi\ P_{nx} := 8500\ \mathbf{kip}$ $\quad\quad \phi\ P_{ny} := 8850\ \mathbf{kip}$

$$\phi\ P_{neq} := \frac{1}{\left(\frac{1}{\phi\ P_{ny}} + \frac{1}{\phi\ P_{nx}} + \frac{1}{\phi\ P_{n0}}\right)} = 2900.727\ \mathbf{kip}$$

$P_u < \phi\, P_{neq} = 1$ Therefore OK in Service 1

Tower below girder

concrete beam to support girders

Pier Cap Design   Critical case is Strength 3

$c_c := 2\ \mathbf{in}$ $b := 4\ \mathbf{ft}$ $h := 4.5\ \mathbf{ft}$ $f_y := 60\ \mathbf{ksi}$

$d_{st} := 0.625\ \mathbf{in}$

$\beta_1 := 0.85$ $d_b := 1.693\ \mathbf{in}$ #14 rebar

$cover := c_c + d_{st} + \frac{d_b}{2}$ $A_b := 2.25\ \mathbf{in}^2$

$d := h - cover$

singly reinforced therefore $d_t := d = 50.529\ \mathbf{in}$

upper limit for tension for tension controlled design

$$A_{stension} := \frac{0.85 \cdot f'_c \cdot b \cdot \beta_1}{f_y} \cdot \left(\frac{3 \cdot d_t}{8}\right) = 43.808\ \mathbf{in}^2$$

$A_s := 8 \cdot A_b = 18\ \mathbf{in}^2$

$A_s < A_{stension} = 1$

$$A_{smin} := \max\left(\frac{200}{\left(\frac{f_y}{\mathbf{psi}}\right)}, \frac{3 \cdot \sqrt{\frac{f'_c}{\mathbf{psi}}}}{\frac{f_y}{\mathbf{psi}}}\right) \cdot b \cdot d = 8.085\ \mathbf{in}^2$$

$A_s > A_{smin} = 1$

$$a := \frac{A_s \cdot f_y}{0.85 \cdot f'_c \cdot b} = 6.618\ \mathbf{in}$$

$F_c := 0.85 \cdot f'_c \cdot b \cdot a = 1080\ \mathbf{kip}$

$F_s := A_s \cdot f_y = 1080\ \mathbf{kip}$

$M_n := F_s \cdot \left(d - \frac{a}{2}\right) = 4249.771\ \mathbf{kip \cdot ft}$

$M_r := 0.9 \cdot M_n = 3824.794\ \mathbf{kip \cdot ft}$

$M_u := 3799.08\ \mathbf{kip \cdot ft}$ in strength 3

$\frac{M_u}{M_r} = 0.993$ OK

$\phi_s := 0.75$

$\lambda := 1$

$$\lambda_s := \min\left(1, \sqrt{\frac{2}{1 + \frac{d}{\mathbf{in} \cdot 10}}}\right) = 0.575$$

$$\rho_w := \frac{A_s}{b \cdot d} = 0.007$$

$$V_c := \min\left(5 \cdot \lambda \cdot \sqrt{\frac{f'_c}{\mathbf{psi}}}\ \mathbf{psi}, 8 \cdot \lambda \cdot \lambda_s \cdot \rho_w^{\frac{1}{3}} \cdot \sqrt{\frac{f'_c}{\mathbf{psi}}}\ \mathbf{psi}\right) \cdot b \cdot d = 137.593\ \mathbf{kip}$$

$\phi V_c := \phi_s \cdot V_c = 103.195\ \mathbf{kip}$ $V_u := 646.44\ \mathbf{kip}$

Use #5 U stirrups

$f_{yt} := 40\ \mathbf{ksi}$ $d_{st} := 0.625\ \mathbf{in}$ $A_{st} := 0.31\ \mathbf{in}^2$

$A_s = 18\ \mathbf{in}^2$

$$V_{umax} := \phi_s \cdot \left(V_c + 8 \cdot \sqrt{\frac{f'_c}{\mathbf{psi}}}\ \mathbf{psi} \cdot b \cdot d\right) = 1023.557\ \mathbf{kip}$$ OK

$A_v := 2 \cdot A_{st} = 0.62\ \mathbf{in}^2$

$$s := \frac{A_v \cdot f_{yt} \cdot d}{\frac{V_u}{\phi_s} - V_c} = 1.73\ \mathbf{in}$$

space stirrups every 1.5 in

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Design axial load capacity for tied columns

Cross-Section Dimensions: $b := 5.5\ \mathbf{ft}$ $h := 10\ \mathbf{ft}$

$A_g := b \cdot h = 55\ \mathbf{ft}^2$

$P_u := 3480.32\ \mathbf{kip}$

$n_{bars} := 54$ $A_{bar} := 4\ \mathbf{in}^2$

$f_y := 60\ \mathbf{ksi}$ $A_{st} := n_{bars} \cdot A_{bar}$

$\frac{P_u}{A_g} = 0.439\ \mathbf{ksi}$ $\rho_g := \frac{A_{st}}{A_g} = 0.027$

$\phi P_n := 0.8 \cdot 0.65 \cdot \left(0.85 \cdot f'_c \cdot A_g + \left(f_y - 0.85 \cdot f'_c\right) \cdot A_{st}\right)$

$P_r := \phi P_n = 20359.872\ \mathbf{kip}$

$\frac{P_u}{P_r} \le 1 = 1$ $\frac{P_u}{P_r} = 0.171$

OK, now PM interaction

Point A

$P_{rA} := \phi P_n = 20359.872\ \mathbf{kip}$

$\phi M_{nA} := 0\ \mathbf{kip} \cdot \mathbf{ft}$ $M_{rA} := \phi M_{nA}$

Point B $E_s := 29000\ \mathbf{ksi}$ #18 rebar

$\varepsilon_{cu} := 0.003$ $d_{ties} := 0.5\ \mathbf{in}$ $d_{bar} := 2.257\ \mathbf{in}$

$c_c := 2\ \mathbf{in}$ $y_{bar} := \frac{h}{2} = 5\ \mathbf{ft}$

$A_{s1} := 14 \cdot A_{bar}$ $A_{s2} := 2 \cdot A_{bar}$ $A_{s3} := 2 \cdot A_{bar}$

$A_{s4} := 2 \cdot A_{bar}$ $A_{s5} := 2 \cdot A_{bar}$ $A_{s6} := 2 \cdot A_{bar}$

$A_{s7} := 2 \cdot A_{bar}$ $A_{s8} := 2 \cdot A_{bar}$ $A_{s9} := 2 \cdot A_{bar}$

$A_{s10} := 2 \cdot A_{bar}$ $A_{s11} := 2 \cdot A_{bar}$ $A_{s12} := 2 \cdot A_{bar}$

$A_{s13} := 2 \cdot A_{bar}$ $A_{s14} := 2 \cdot A_{bar}$ $A_{s15} := 14 \cdot A_{bar}$

$y_{s1} := c_c + d_{ties} + \frac{d_{bar}}{2} = 3.629 \text{ in}$ $y_{s15} := h - \left(c_c + d_{ties} + \frac{d_{bar}}{2}\right) = 9.698 \text{ ft}$

$s := \frac{y_{s15} - y_{s1}}{14} = 8.053 \text{ in}$

$y_{s2} := y_{s1} + s = 11.682 \text{ in}$ $y_{s3} := y_{s1} + 2 \cdot s = 19.735 \text{ in}$

$y_{s4} := y_{s1} + 3 \cdot s = 27.788 \text{ in}$ $y_{s5} := y_{s1} + 4 \cdot s = 2.987 \text{ ft}$

$y_{s6} := y_{s1} + 5 \cdot s = 3.658 \text{ ft}$ $y_{s7} := y_{s1} + 6 \cdot s = 4.329 \text{ ft}$

$y_{s8} := y_{s1} + 7 \cdot s = 5 \text{ ft}$ $y_{s9} := y_{s1} + 8 \cdot s = 5.671 \text{ ft}$

$y_{s10} := y_{s1} + 9 \cdot s = 6.342 \text{ ft}$ $y_{s11} := y_{s1} + 10 \cdot s = 7.013 \text{ ft}$

$y_{s12} := y_{s1} + 11 \cdot s = 7.684 \text{ ft}$ $y_{s13} := y_{s1} + 12 \cdot s = 8.355 \text{ ft}$

$y_{s14} := y_{s1} + 13 \cdot s = 9.027 \text{ ft}$

$\varepsilon_{ty} := \frac{f_y}{E_s} = 0.002$

$\varepsilon_{s1} := 0$ $\varepsilon_o := \frac{\varepsilon_{s1} \cdot h - \varepsilon_{cu} \cdot y_{s1}}{h - y_{s1}} = -9.354 \cdot 10^{-5}$

$$c := \frac{\varepsilon_{cu} \cdot h}{\varepsilon_{cu} - \varepsilon_o} = 116.372 \ \mathbf{in}$$

$$\beta_1 := 0.85$$

$$a := \beta_1 \cdot c = 8.243 \ \mathbf{ft}$$

$$\varepsilon_{s2} := \frac{(h - y_{s2})}{h} \cdot \varepsilon_o + \frac{y_{s2}}{h} \cdot \varepsilon_{cu} = 0.00021$$

$$\varepsilon_{s3} := \frac{(h - y_{s3})}{h} \cdot \varepsilon_o + \frac{y_{s3}}{h} \cdot \varepsilon_{cu} = 0.00042$$

$$\varepsilon_{s4} := \frac{(h - y_{s4})}{h} \cdot \varepsilon_o + \frac{y_{s4}}{h} \cdot \varepsilon_{cu} = 0.00062$$

$$\varepsilon_{s5} := \frac{(h - y_{s5})}{h} \cdot \varepsilon_o + \frac{y_{s5}}{h} \cdot \varepsilon_{cu} = 0.00083$$

$$\varepsilon_{s6} := \frac{(h - y_{s6})}{h} \cdot \varepsilon_o + \frac{y_{s6}}{h} \cdot \varepsilon_{cu} = 0.00104$$

$$\varepsilon_{s7} := \frac{(h - y_{s7})}{h} \cdot \varepsilon_o + \frac{y_{s7}}{h} \cdot \varepsilon_{cu} = 0.00125$$

$$\varepsilon_{s8} := \frac{(h - y_{s8})}{h} \cdot \varepsilon_o + \frac{y_{s8}}{h} \cdot \varepsilon_{cu} = 0.00145$$

$$\varepsilon_{s9} := \frac{(h - y_{s9})}{h} \cdot \varepsilon_o + \frac{y_{s9}}{h} \cdot \varepsilon_{cu} = 0.00166$$

$$\varepsilon_{s10} := \frac{(h - y_{s10})}{h} \cdot \varepsilon_o + \frac{y_{s10}}{h} \cdot \varepsilon_{cu} = 0.00187$$

$$\varepsilon_{s11} := \frac{(h - y_{s11})}{h} \cdot \varepsilon_o + \frac{y_{s11}}{h} \cdot \varepsilon_{cu} = 0.00208$$

$$\varepsilon_{s12} := \frac{(h - y_{s12})}{h} \cdot \varepsilon_o + \frac{y_{s12}}{h} \cdot \varepsilon_{cu} = 0.00228$$

$$\varepsilon_{s13} := \frac{(h - y_{s13})}{h} \cdot \varepsilon_o + \frac{y_{s13}}{h} \cdot \varepsilon_{cu} = 0.00249$$

$$\varepsilon_{s14} := \frac{(h - y_{s14})}{h} \cdot \varepsilon_o + \frac{y_{s14}}{h} \cdot \varepsilon_{cu} = 0.0027$$

$$\varepsilon_{s15} := \frac{(h - y_{s15})}{h} \cdot \varepsilon_o + \frac{y_{s15}}{h} \cdot \varepsilon_{cu} = 0.00291$$

stresses

$\varepsilon_{ty} = 0.00207$

$f_{s1} := 0$ **ksi** strain was 0

$F_{s1} := A_{s1} \cdot f_{s1} = 0$ **kip**

$\varepsilon_{s2} < \varepsilon_{ty} = 1$ elastic in compression

$f_{s2} := E_s \cdot \varepsilon_{s2} - 0.85 \cdot f'_c = 2.621$ **ksi**

$F_{s2} := A_{s2} \cdot f_{s2} = 20.964$ **kip**

$\varepsilon_{s3} < \varepsilon_{ty} = 1$ elastic in compression

$f_{s3} := E_s \cdot \varepsilon_{s3} - 0.85 \cdot f'_c = 8.641$ **ksi**

$F_{s3} := A_{s3} \cdot f_{s3} = 69.128$ **kip**

$\varepsilon_{s4} < \varepsilon_{ty} = 1$ elastic in compression

$f_{s4} := E_s \cdot \varepsilon_{s4} - 0.85 \cdot f'_c = 14.662$ **ksi** $F_{s4} := A_{s4} \cdot f_{s4} = 117.293$ **kip**

$\varepsilon_{s5} < \varepsilon_{ty} = 1$ elastic in compression

$f_{s5} := E_s \cdot \varepsilon_{s5} - 0.85\ f'_c = 20.682$ **ksi** $F_{s5} := A_{s5} \cdot f_{s5} = 165.457$ **kip**

$\varepsilon_{s6} < \varepsilon_{ty} = 1$ elastic in compression

$f_{s6} := E_s \cdot \varepsilon_{s6} - 0.85\ f'_c = 26.703$ **ksi** $F_{s6} := A_{s6} \cdot f_{s6} = 213.621$ **kip**

$\varepsilon_{s7} < \varepsilon_{ty} = 1$ elastic in compression

$f_{s7} := E_s \cdot \varepsilon_{s7} - 0.85\ f'_c = 32.723$ **ksi** $F_{s7} := A_{s7} \cdot f_{s7} = 261.785$ **kip**

$\varepsilon_{s8} < \varepsilon_{ty} = 1$ elastic in compression

$f_{s8} := E_s \cdot \varepsilon_{s8} - 0.85\ f'_c = 38.744$ **ksi** $F_{s8} := A_{s8} \cdot f_{s8} = 309.949$ **kip**

$\varepsilon_{s9} < \varepsilon_{ty} = 1$ elastic in compression

$f_{s9} := E_s \cdot \varepsilon_{s9} - 0.85\ f'_c = 44.764$ **ksi** $F_{s9} := A_{s9} \cdot f_{s9} = 358.113$ **kip**

$\varepsilon_{s10} < \varepsilon_{ty} = 1$ elastic in compression

$f_{s10} := E_s \cdot \varepsilon_{s10} - 0.85\ f'_c = 50.785$ **ksi** $F_{s10} := A_{s10} \cdot f_{s10} = 406.278$ **kip**

$\varepsilon_{s11} \geq \varepsilon_{ty} = 1$ yielded in compression

$f_{s11} := f_y - 0.85\ f'_c = 56.6$ **ksi** $F_{s11} := A_{s11} \cdot f_{s11} = 452.8$ **kip**

$\varepsilon_{s12} \geq \varepsilon_{ty} = 1$ yielded in compression

$f_{s12} := f_y - 0.85\ f'_c = 56.6$ **ksi** $F_{s12} := A_{s12} \cdot f_{s12} = 452.8$ **kip**

$\varepsilon_{s13} \geq \varepsilon_{ty} = 1$ yielded in compression

$f_{s13} := f_y - 0.85\ f'_c = 56.6$ **ksi** $F_{s13} := A_{s13} \cdot f_{s13} = 452.8$ **kip**

$\varepsilon_{s14} \geq \varepsilon_{ty} = 1$ yielded in compression

$f_{s14} := f_y - 0.85\ f'_c = 56.6$ **ksi** $F_{s14} := A_{s14} \cdot f_{s14} = 452.8$ **kip**

$\varepsilon_{s15} \geq \varepsilon_{ty} = 1$ yielded in compression

$f_{s15} := f_y - 0.85\ f'_c = 56.6\ \mathbf{ksi}$

$F_{s15} := A_{s15} \cdot f_{s15} = 3169.6\ \mathbf{kip}$

$A_c := b \cdot a = 45.336\ \mathbf{ft}^2$

$F_c := 0.85 \cdot f'_c \cdot A_c = 22196.7\ \mathbf{kip}$

$y_c := h - \frac{a}{2} = 5.879\ \mathbf{ft}$

$$P_n := F_c + F_{s1} + F_{s2} + F_{s3} + F_{s4} + F_{s5} + F_{s6} + F_{s7} + F_{s8} + F_{s9} + F_{s10} + F_{s11} + F_{s12} + F_{s13} + F_{s14} + F_{s15}$$

$P_n = 29100.088\ \mathbf{kip}$

$$M_n := F_c \cdot (y_c - y_{bar}) + F_{s1} \cdot (y_{s1} - y_{bar}) + F_{s2} \cdot (y_{s2} - y_{bar}) + F_{s3} \cdot (y_{s3} - y_{bar}) + F_{s4} \cdot (y_{s4} - y_{bar}) + F_{s5} \cdot (y_{s5} - y_{bar}) + F_{s6} \cdot (y_{s6} - y_{bar}) + F_{s7} \cdot (y_{s7} - y_{bar}) + F_{s8} \cdot (y_{s8} - y_{bar}) + F_{s9} \cdot (y_{s9} - y_{bar}) + F_{s10} \cdot (y_{s10} - y_{bar}) + F_{s11} \cdot (y_{s11} - y_{bar}) + F_{s12} \cdot (y_{s12} - y_{bar}) + F_{s13} \cdot (y_{s13} - y_{bar}) + F_{s14} \cdot (y_{s14} - y_{bar}) + F_{s15} \cdot (y_{s15} - y_{bar})$$

$M_n = 39218.139\ \mathbf{kip \cdot ft}$

$\varepsilon_t := \varepsilon_{s1} = 0$

$\phi := 0.65$

$P_{rB} := \phi \cdot P_n = 18915.057\ \mathbf{kip}$

$M_{rB} := \phi \cdot M_n = 25491.79\ \mathbf{kip \cdot ft}$

Point C

$\varepsilon_{s1} := \frac{-f_y}{E_s} = -0.002$

$\varepsilon_o := \frac{\varepsilon_{s1} \cdot h - \varepsilon_{cu} \cdot y_{s1}}{h - y_{s1}} = -0.002$

$c := \frac{\varepsilon_{cu} \cdot h}{\varepsilon_{cu} - \varepsilon_o} = 68.873\ \mathbf{in}$

$a := \beta_1 \cdot c = 4.878\ \mathbf{ft}$

$\varepsilon_{s2} := \frac{(h - y_{s2})}{h} \cdot \varepsilon_o + \frac{y_{s2}}{h} \cdot \varepsilon_{cu} = -0.00172$ $\varepsilon_{s9} := \frac{(h - y_{s9})}{h} \cdot \varepsilon_o + \frac{y_{s9}}{h} \cdot \varepsilon_{cu} = 0.00074$

$\varepsilon_{s3} := \frac{(h - y_{s3})}{h} \cdot \varepsilon_o + \frac{y_{s3}}{h} \cdot \varepsilon_{cu} = -0.00137$ $\varepsilon_{s10} := \frac{(h - y_{s10})}{h} \cdot \varepsilon_o + \frac{y_{s10}}{h} \cdot \varepsilon_{cu} = 0.00109$

$\varepsilon_{s4} := \frac{(h - y_{s4})}{h} \cdot \varepsilon_o + \frac{y_{s4}}{h} \cdot \varepsilon_{cu} = -0.00102$ $\varepsilon_{s11} := \frac{(h - y_{s11})}{h} \cdot \varepsilon_o + \frac{y_{s11}}{h} \cdot \varepsilon_{cu} = 0.00144$

$\varepsilon_{s5} := \frac{(h - y_{s5})}{h} \cdot \varepsilon_o + \frac{y_{s5}}{h} \cdot \varepsilon_{cu} = -0.00067$ $\varepsilon_{s12} := \frac{(h - y_{s12})}{h} \cdot \varepsilon_o + \frac{y_{s12}}{h} \cdot \varepsilon_{cu} = 0.00179$

$\varepsilon_{s6} := \frac{(h - y_{s6})}{h} \cdot \varepsilon_o + \frac{y_{s6}}{h} \cdot \varepsilon_{cu} = -0.00032$ $\varepsilon_{s13} := \frac{(h - y_{s13})}{h} \cdot \varepsilon_o + \frac{y_{s13}}{h} \cdot \varepsilon_{cu} = 0.00214$

$\varepsilon_{s7} := \frac{(h - y_{s7})}{h} \cdot \varepsilon_o + \frac{y_{s7}}{h} \cdot \varepsilon_{cu} = 0.00004$ $\varepsilon_{s14} := \frac{(h - y_{s14})}{h} \cdot \varepsilon_o + \frac{y_{s14}}{h} \cdot \varepsilon_{cu} = 0.00249$

$\varepsilon_{s8} := \frac{(h - y_{s8})}{h} \cdot \varepsilon_o + \frac{y_{s8}}{h} \cdot \varepsilon_{cu} = 0.00039$ $\varepsilon_{s15} := \frac{(h - y_{s15})}{h} \cdot \varepsilon_o + \frac{y_{s15}}{h} \cdot \varepsilon_{cu} = 0.00284$

stresses $\varepsilon_{ty} = 0.00207$

$|\varepsilon_{s1}| \geq \varepsilon_{ty} = 1$ $\varepsilon_{s1} < 0 = 1$ yielded in tension

$f_{s1} := -f_y = -60$ **ksi** $F_{s1} := A_{s1} \cdot f_{s1} = -3360$ **kip**

$|\varepsilon_{s2}| < \varepsilon_{ty} = 1$ $\varepsilon_{s2} < 0 = 1$ elastic in tension

$f_{s2} := E_s \cdot \varepsilon_{s2} = -49.827$ **ksi** $F_{s2} := A_{s2} \cdot f_{s2} = -398.619$ **kip**

$|\varepsilon_{s3}| < \varepsilon_{ty} = 1$ $\varepsilon_{s3} < 0 = 1$ elastic in tension

$f_{s3} := E_s \cdot \varepsilon_{s3} = -39.655$ **ksi** $F_{s3} := A_{s3} \cdot f_{s3} = -317.238$ **kip**

$|\varepsilon_{s4}| < \varepsilon_{ty} = 1$ $\varepsilon_{s4} < 0 = 1$ elastic in tension

$f_{s4} := E_s \cdot \varepsilon_{s4} = -29.482$ **ksi** $F_{s4} := A_{s4} \cdot f_{s4} = -235.857$ **kip**

$|\varepsilon_{s5}| < \varepsilon_{ty} = 1$ $\varepsilon_{s5} < 0 = 1$ elastic in tension

$f_{s5} := E_s \cdot \varepsilon_{s5} = -19.31$ **ksi** $F_{s5} := A_{s5} \cdot f_{s5} = -154.477$ **kip**

$|\varepsilon_{s6}| < \varepsilon_{ty} = 1$ $\varepsilon_{s6} < 0 = 1$ elastic in tension

$f_{s6} := E_s \cdot \varepsilon_{s6} = -9.137$ **ksi** $F_{s6} := A_{s6} \cdot f_{s6} = -73.096$ **kip**

$|\varepsilon_{s7}| < \varepsilon_{ty} = 1$ $\varepsilon_{s7} > 0 = 1$ elastic in compression

$f_{s7} := E_s \cdot \varepsilon_{s7} - 0.85 \cdot f'_c = -2.364$ **ksi** $F_{s7} := A_{s7} \cdot f_{s7} = -18.915$ **kip**

$|\varepsilon_{s8}| < \varepsilon_{ty} = 1$ $\varepsilon_{s8} > 0 = 1$ elastic in compression

$f_{s8} := E_s \cdot \varepsilon_{s8} - 0.85\ f'_c = 7.808$ **ksi** $F_{s8} := A_{s8} \cdot f_{s8} = 62.466$ **kip**

$|\varepsilon_{s9}| < \varepsilon_{ty} = 1$ $\varepsilon_{s9} > 0 = 1$ elastic in compression

$f_{s9} := E_s \cdot \varepsilon_{s9} - 0.85\ f'_c = 17.981$ **ksi** $F_{s9} := A_{s9} \cdot f_{s9} = 143.847$ **kip**

$|\varepsilon_{s10}| < \varepsilon_{ty} = 1$ $\varepsilon_{s10} > 0 = 1$ elastic in compression

$f_{s10} := E_s \cdot \varepsilon_{s10} - 0.85\ f'_c = 28.153$ **ksi** $F_{s10} := A_{s10} \cdot f_{s10} = 225.228$ **kip**

$|\varepsilon_{s11}| < \varepsilon_{ty} = 1$ $\varepsilon_{s11} > 0 = 1$ elastic in compression

$f_{s11} := E_s \cdot \varepsilon_{s11} - 0.85\ f'_c = 38.326$ **ksi** $F_{s11} := A_{s11} \cdot f_{s11} = 306.609$ **kip**

$|\varepsilon_{s12}| < \varepsilon_{ty} = 1$ $\varepsilon_{s12} > 0 = 1$ elastic in compression

$f_{s12} := E_s \cdot \varepsilon_{s12} - 0.85\ f'_c = 48.499$ **ksi** $F_{s12} := A_{s12} \cdot f_{s12} = 387.989$ **kip**

$|\varepsilon_{s13}| \ge \varepsilon_{ty} = 1$ $\varepsilon_{s13} > 0 = 1$ yielded in compression

$f_{s13} := f_y - 0.85\ f'_c = 56.6$ **ksi** $F_{s13} := A_{s13} \cdot f_{s13} = 452.8$ **kip**

$|\varepsilon_{s14}| \ge \varepsilon_{ty} = 1$ $\varepsilon_{s14} > 0 = 1$ yielded in compression

$f_{s14} := f_y - 0.85\ f'_c = 56.6$ **ksi** $F_{s14} := A_{s14} \cdot f_{s14} = 452.8$ **kip**

$|\varepsilon_{s15}| \ge \varepsilon_{ty} = 1$ $\varepsilon_{s15} > 0 = 1$ yielded in compression

$f_{s15} := f_y - 0.85\ f'_c = 56.6$ **ksi** $F_{s15} := A_{s15} \cdot f_{s15} = 3169.6$ **kip**

$A_c := b \cdot a = 26.832\ ft^2$

$F_c := 0.85 \cdot f'_c \cdot A_c = 13136822.396\ lbf$

$y_c := h - \frac{a}{2} = 7.561\ ft$

$P_n := F_c + F_{s1} + F_{s2} + F_{s3} + F_{s4} + F_{s5} + F_{s6} + F_{s7} + F_{s8} + F_{s9} + F_{s10} + F_{s11} + F_{s12} + F_{s13} + F_{s14} + F_{s15}$

$P_n = 13779.959\ kip$

$M_n := F_c \cdot (y_c - y_{bar}) + F_{s1} \cdot (y_{s1} - y_{bar}) + F_{s2} \cdot (y_{s2} - y_{bar}) + F_{s3} \cdot (y_{s3} - y_{bar}) + F_{s4} \cdot (y_{s4} - y_{bar}) + F_{s5} \cdot (y_{s5} - y_{bar}) + F_{s6} \cdot (y_{s6} - y_{bar}) + F_{s7} \cdot (y_{s7} - y_{bar}) + F_{s8} \cdot (y_{s8} - y_{bar}) + F_{s9} \cdot (y_{s9} - y_{bar}) + F_{s10} \cdot (y_{s10} - y_{bar}) + F_{s11} \cdot (y_{s11} - y_{bar}) + F_{s12} \cdot (y_{s12} - y_{bar}) + F_{s13} \cdot (y_{s13} - y_{bar}) + F_{s14} \cdot (y_{s14} - y_{bar}) + F_{s15} \cdot (y_{s15} - y_{bar})$

$M_n = 73438.374\ kip \cdot ft$

$\varepsilon_t := |\varepsilon_{s1}| = 0.002$

$\phi := 0.65$

$P_{rC} := \phi \cdot P_n = 8956.973\ kip$

$M_{rC} := \phi \cdot M_n = 47734.943\ kip \cdot ft$

Point D

$\varepsilon_{s1} := -0.005$

$\varepsilon_o := \frac{\varepsilon_{s1} \cdot h - \varepsilon_{cu} \cdot y_{s1}}{h - y_{s1}} = -0.005$

$c := \frac{\varepsilon_{cu} \cdot h}{\varepsilon_{cu} - \varepsilon_o} = 43.639\ in$

$a := \beta_1 \cdot c = 3.091\ ft$

$\varepsilon_{s2} := \frac{(h - y_{s2})}{h} \cdot \varepsilon_o + \frac{y_{s2}}{h} \cdot \varepsilon_{cu} = -0.00445$

$\varepsilon_{s3} := \frac{(h - y_{s3})}{h} \cdot \varepsilon_o + \frac{y_{s3}}{h} \cdot \varepsilon_{cu} = -0.00389$

$\varepsilon_{s9} := \frac{(h - y_{s9})}{h} \cdot \varepsilon_o + \frac{y_{s9}}{h} \cdot \varepsilon_{cu} = -0.00057$

$\varepsilon_{s10} := \frac{(h - y_{s10})}{h} \cdot \varepsilon_o + \frac{y_{s10}}{h} \cdot \varepsilon_{cu} = -0.00002$

$$\varepsilon_{s4} := \frac{(h - y_{s4})}{h} \cdot \varepsilon_o + \frac{y_{s4}}{h} \cdot \varepsilon_{cu} = -0.00334$$

$$\varepsilon_{s11} := \frac{(h - y_{s11})}{h} \cdot \varepsilon_o + \frac{y_{s11}}{h} \cdot \varepsilon_{cu} = 0.00054$$

$$\varepsilon_{s5} := \frac{(h - y_{s5})}{h} \cdot \varepsilon_o + \frac{y_{s5}}{h} \cdot \varepsilon_{cu} = -0.00279$$

$$\varepsilon_{s12} := \frac{(h - y_{s12})}{h} \cdot \varepsilon_o + \frac{y_{s12}}{h} \cdot \varepsilon_{cu} = 0.00109$$

$$\varepsilon_{s6} := \frac{(h - y_{s6})}{h} \cdot \varepsilon_o + \frac{y_{s6}}{h} \cdot \varepsilon_{cu} = -0.00223$$

$$\varepsilon_{s13} := \frac{(h - y_{s13})}{h} \cdot \varepsilon_o + \frac{y_{s13}}{h} \cdot \varepsilon_{cu} = 0.00164$$

$$\varepsilon_{s7} := \frac{(h - y_{s7})}{h} \cdot \varepsilon_o + \frac{y_{s7}}{h} \cdot \varepsilon_{cu} = -0.00168$$

$$\varepsilon_{s14} := \frac{(h - y_{s14})}{h} \cdot \varepsilon_o + \frac{y_{s14}}{h} \cdot \varepsilon_{cu} = 0.0022$$

$$\varepsilon_{s8} := \frac{(h - y_{s8})}{h} \cdot \varepsilon_o + \frac{y_{s8}}{h} \cdot \varepsilon_{cu} = -0.00112$$

$$\varepsilon_{s15} := \frac{(h - y_{s15})}{h} \cdot \varepsilon_o + \frac{y_{s15}}{h} \cdot \varepsilon_{cu} = 0.00275$$

stresses $\varepsilon_{ty} = 0.00207$

$|\varepsilon_{s1}| \geq \varepsilon_{ty} = 1$ $\varepsilon_{s1} < 0 = 1$ yielded in tension

$f_{s1} := -f_y = -60$ **ksi** $F_{s1} := A_{s1} \cdot f_{s1} = -3360$ **kip**

$|\varepsilon_{s2}| \geq \varepsilon_{ty} = 1$ $\varepsilon_{s2} < 0 = 1$ yielded in tension

$f_{s2} := -f_y = -60$ **ksi** $F_{s2} := A_{s2} \cdot f_{s2} = -480$ **kip**

$|\varepsilon_{s3}| \geq \varepsilon_{ty} = 1$ $\varepsilon_{s3} < 0 = 1$ yielded in tension

$f_{s3} := -f_y = -60$ **ksi** $F_{s3} := A_{s3} \cdot f_{s3} = -480$ **kip**

$|\varepsilon_{s4}| \geq \varepsilon_{ty} = 1$ $\varepsilon_{s4} < 0 = 1$ yielded in tension

$f_{s4} := -f_y = -60$ **ksi** $F_{s4} := A_{s4} \cdot f_{s4} = -480$ **kip**

$|\varepsilon_{s5}| \geq \varepsilon_{ty} = 1$ $\varepsilon_{s5} < 0 = 1$ yielded in tension

$f_{s5} := -f_y = -60$ **ksi** $F_{s5} := A_{s5} \cdot f_{s5} = -480$ **kip**

$|\varepsilon_{s6}| \geq \varepsilon_{ty} = 1$ $\varepsilon_{s6} < 0 = 1$ yielded in tension

$f_{s6} := -f_y = -60$ **ksi** $F_{s6} := A_{s6} \cdot f_{s6} = -480$ **kip**

$|\varepsilon_{s7}| < \varepsilon_{ty} = 1$ $\varepsilon_{s7} < 0 = 1$ elastic in tension

$f_{s7} := E_s \cdot \varepsilon_{s7} = -48.672$ **ksi** $F_{s7} := A_{s7} \cdot f_{s7} = -389.373$ **kip**

$|\varepsilon_{s8}| < \varepsilon_{ty} = 1$ $\varepsilon_{s8} < 0 = 1$ elastic in tension

$f_{s8} := E_s \cdot \varepsilon_{s8} = -32.617$ **ksi** $F_{s8} := A_{s8} \cdot f_{s8} = -260.935$ **kip**

$|\varepsilon_{s9}| < \varepsilon_{ty} = 1$ $\varepsilon_{s9} < 0 = 1$ elastic in tension

$f_{s9} := E_s \cdot \varepsilon_{s9} = -16.562$ **ksi** $F_{s9} := A_{s9} \cdot f_{s9} = -132.498$ **kip**

$|\varepsilon_{s10}| < \varepsilon_{ty} = 1$ $\varepsilon_{s10} < 0 = 1$ elastic in tension

$f_{s10} := E_s \cdot \varepsilon_{s10} = -0.507$ **ksi** $F_{s10} := A_{s10} \cdot f_{s10} = -4.06$ **kip**

$|\varepsilon_{s11}| < \varepsilon_{ty} = 1$ $\varepsilon_{s11} > 0 = 1$ elastic in compression

$f_{s11} := E_s \cdot \varepsilon_{s11} - 0.85\ f'_c = 12.147$ **ksi** $F_{s11} := A_{s11} \cdot f_{s11} = 97.178$ **kip**

$|\varepsilon_{s12}| < \varepsilon_{ty} = 1$ $\varepsilon_{s12} > 0 = 1$ elastic in compression

$f_{s12} := E_s \cdot \varepsilon_{s12} - 0.85\ f'_c = 28.202$ **ksi** $F_{s12} := A_{s12} \cdot f_{s12} = 225.616$ **kip**

$|\varepsilon_{s13}| < \varepsilon_{ty} = 1$ $\varepsilon_{s13} > 0 = 1$ elastic in compression

$f_{s13} := E_s \cdot \varepsilon_{s13} - 0.85\ f'_c = 44.257$ **ksi** $F_{s13} := A_{s13} \cdot f_{s13} = 354.054$ **kip**

$|\varepsilon_{s14}| \geq \varepsilon_{ty} = 1$ $\varepsilon_{s14} > 0 = 1$ yielded in compression

$f_{s14} := f_y - 0.85\ f'_c = 56.6$ **ksi** $F_{s14} := A_{s14} \cdot f_{s14} = 452.8$ **kip**

$|\varepsilon_{s15}| \geq \varepsilon_{ty} = 1$ $\varepsilon_{s15} > 0 = 1$ yielded in compression

$f_{s15} := f_y - 0.85\ f'_c = 56.6$ **ksi** $F_{s15} := A_{s15} \cdot f_{s15} = 3169.6$ **kip**

$A_c := b \cdot a = 17.001$ **ft**^{2}

$F_c := 0.85 \cdot f'_c \cdot A_c = 8323762.466$ **lbf**

$y_c := h - \frac{a}{2} = 8.454$ **ft**

$$P_n := F_c + F_{s1} + F_{s2} + F_{s3} + F_{s4} + F_{s5} + F_{s6} + F_{s7} + F_{s8} + F_{s9} + F_{s10} + F_{s11} + F_{s12} + F_{s13} + F_{s14} + F_{s15}$$

$$P_n = 6076.144\ \textbf{kip}$$

$$M_n := F_c \cdot (y_c - y_{bar}) + F_{s1} \cdot (y_{s1} - y_{bar}) + F_{s2} \cdot (y_{s2} - y_{bar}) + F_{s3} \cdot (y_{s3} - y_{bar}) + F_{s4} \cdot (y_{s4} - y_{bar}) + F_{s5} \cdot (y_{s5} - y_{bar}) + F_{s6} \cdot (y_{s6} - y_{bar}) + F_{s7} \cdot (y_{s7} - y_{bar}) + F_{s8} \cdot (y_{s8} - y_{bar}) + F_{s9} \cdot (y_{s9} - y_{bar}) + F_{s10} \cdot (y_{s10} - y_{bar}) + F_{s11} \cdot (y_{s11} - y_{bar}) + F_{s12} \cdot (y_{s12} - y_{bar}) + F_{s13} \cdot (y_{s13} - y_{bar}) + F_{s14} \cdot (y_{s14} - y_{bar}) + F_{s15} \cdot (y_{s15} - y_{bar})$$

$$M_n = 69849.456\ \textbf{kip} \cdot \textbf{ft}$$

$$\varepsilon_t := |\varepsilon_{s1}| = 0.005$$

$$\varepsilon_t > \varepsilon_{ty} = 1$$

$$\varepsilon_t < \varepsilon_{ty} + 0.003 = 1$$

$$\phi := 0.65 + 0.25 \cdot \left(\frac{\varepsilon_t - \varepsilon_{ty}}{0.003}\right) = 0.894$$

$$P_{rD} := \phi \cdot P_n = 5433.61\ \textbf{kip}$$

$$M_{rD} := \phi \cdot M_n = 62463.077\ \textbf{kip} \cdot \textbf{ft}$$

Point E

$$\varepsilon_{s1} := -0.02$$

$$\varepsilon_o := \frac{\varepsilon_{s1} \cdot h - \varepsilon_{cu} \cdot y_{s1}}{h - y_{s1}} = -0.021$$

$$c := \frac{\varepsilon_{cu} \cdot h}{\varepsilon_{cu} - \varepsilon_o} = 15.179\ \textbf{in}$$

$$a := \beta_1 \cdot c = 1.075\ \textbf{ft}$$

$$\varepsilon_{s2} := \frac{(h - y_{s2})}{h} \cdot \varepsilon_o + \frac{y_{s2}}{h} \cdot \varepsilon_{cu} = -0.01841$$

$$\varepsilon_{s9} := \frac{(h - y_{s9})}{h} \cdot \varepsilon_o + \frac{y_{s9}}{h} \cdot \varepsilon_{cu} = -0.00727$$

$$\varepsilon_{s3} := \frac{(h - y_{s3})}{h} \cdot \varepsilon_o + \frac{y_{s3}}{h} \cdot \varepsilon_{cu} = -0.01682$$

$$\varepsilon_{s10} := \frac{(h - y_{s10})}{h} \cdot \varepsilon_o + \frac{y_{s10}}{h} \cdot \varepsilon_{cu} = -0.00568$$

$$\varepsilon_{s4} := \frac{(h - y_{s4})}{h} \cdot \varepsilon_o + \frac{y_{s4}}{h} \cdot \varepsilon_{cu} = -0.01523$$

$$\varepsilon_{s11} := \frac{(h - y_{s11})}{h} \cdot \varepsilon_o + \frac{y_{s11}}{h} \cdot \varepsilon_{cu} = -0.00408$$

$$\varepsilon_{s5} := \frac{(h - y_{s5})}{h} \cdot \varepsilon_o + \frac{y_{s5}}{h} \cdot \varepsilon_{cu} = -0.01363 \qquad \varepsilon_{s12} := \frac{(h - y_{s12})}{h} \cdot \varepsilon_o + \frac{y_{s12}}{h} \cdot \varepsilon_{cu} = -0.00249$$

$$\varepsilon_{s6} := \frac{(h - y_{s6})}{h} \cdot \varepsilon_o + \frac{y_{s6}}{h} \cdot \varepsilon_{cu} = -0.01204 \qquad \varepsilon_{s13} := \frac{(h - y_{s13})}{h} \cdot \varepsilon_o + \frac{y_{s13}}{h} \cdot \varepsilon_{cu} = -0.0009$$

$$\varepsilon_{s7} := \frac{(h - y_{s7})}{h} \cdot \varepsilon_o + \frac{y_{s7}}{h} \cdot \varepsilon_{cu} = -0.01045 \qquad \varepsilon_{s14} := \frac{(h - y_{s14})}{h} \cdot \varepsilon_o + \frac{y_{s14}}{h} \cdot \varepsilon_{cu} = 0.00069$$

$$\varepsilon_{s8} := \frac{(h - y_{s8})}{h} \cdot \varepsilon_o + \frac{y_{s8}}{h} \cdot \varepsilon_{cu} = -0.00886 \qquad \varepsilon_{s15} := \frac{(h - y_{s15})}{h} \cdot \varepsilon_o + \frac{y_{s15}}{h} \cdot \varepsilon_{cu} = 0.00228$$

stresses $\qquad \varepsilon_{ty} = 0.00207$

$|\varepsilon_{s1}| \geq \varepsilon_{ty} = 1 \qquad \varepsilon_{s1} < 0 = 1 \qquad$ yielded in tension

$f_{s1} := -f_y = -60$ **ksi** $\qquad F_{s1} := A_{s1} \cdot f_{s1} = -3360$ **kip**

$|\varepsilon_{s2}| \geq \varepsilon_{ty} = 1 \qquad \varepsilon_{s2} < 0 = 1 \qquad$ yielded in tension

$f_{s2} := -f_y = -60$ **ksi** $\qquad F_{s2} := A_{s2} \cdot f_{s2} = -480$ **kip**

$|\varepsilon_{s3}| \geq \varepsilon_{ty} = 1 \qquad \varepsilon_{s3} < 0 = 1 \qquad$ yielded in tension

$f_{s3} := -f_y = -60$ **ksi** $\qquad F_{s3} := A_{s3} \cdot f_{s3} = -480$ **kip**

$|\varepsilon_{s4}| \geq \varepsilon_{ty} = 1 \qquad \varepsilon_{s4} < 0 = 1 \qquad$ yielded in tension

$f_{s4} := -f_y = -60$ **ksi** $\qquad F_{s4} := A_{s4} \cdot f_{s4} = -480$ **kip**

$|\varepsilon_{s5}| \geq \varepsilon_{ty} = 1 \qquad \varepsilon_{s5} < 0 = 1 \qquad$ yielded in tension

$f_{s5} := -f_y = -60$ **ksi** $\qquad F_{s5} := A_{s5} \cdot f_{s5} = -480$ **kip**

$|\varepsilon_{s6}| \geq \varepsilon_{ty} = 1 \qquad \varepsilon_{s6} < 0 = 1 \qquad$ yielded in tension

$f_{s6} := -f_y = -60$ **ksi** $\qquad F_{s6} := A_{s6} \cdot f_{s6} = -480$ **kip**

$|\varepsilon_{s7}| \geq \varepsilon_{ty} = 1 \qquad \varepsilon_{s7} < 0 = 1 \qquad$ yielded in tension

$f_{s7} := -f_y = -60$ **ksi** $\qquad F_{s7} := A_{s7} \cdot f_{s7} = -480$ **kip**

$|\varepsilon_{s8}| \geq \varepsilon_{ty} = 1 \quad \varepsilon_{s8} < 0 = 1$ yielded in tension

$f_{s8} := -f_y = -60\ \mathbf{ksi}$ $\quad F_{s8} := A_{s8} \cdot f_{s8} = -480\ \mathbf{kip}$

$|\varepsilon_{s9}| \geq \varepsilon_{ty} = 1 \quad \varepsilon_{s9} < 0 = 1$ yielded in tension

$f_{s9} := -f_y = -60\ \mathbf{ksi}$ $\quad F_{s9} := A_{s9} \cdot f_{s9} = -480\ \mathbf{kip}$

$|\varepsilon_{s10}| \geq \varepsilon_{ty} = 1 \quad \varepsilon_{s10} < 0 = 1$ yielded in tension

$f_{s10} := -f_y = -60\ \mathbf{ksi}$ $\quad F_{s10} := A_{s10} \cdot f_{s10} = -480\ \mathbf{kip}$

$|\varepsilon_{s11}| \geq \varepsilon_{ty} = 1 \quad \varepsilon_{s11} < 0 = 1$ yielded in tension

$f_{s11} := -f_y = -60\ \mathbf{ksi}$ $\quad F_{s11} := A_{s11} \cdot f_{s11} = -480\ \mathbf{kip}$

$|\varepsilon_{s12}| \geq \varepsilon_{ty} = 1 \quad \varepsilon_{s12} < 0 = 1$ yielded in tension

$f_{s12} := -f_y = -60\ \mathbf{ksi}$ $\quad F_{s12} := A_{s12} \cdot f_{s12} = -480\ \mathbf{kip}$

$|\varepsilon_{s13}| < \varepsilon_{ty} = 1 \quad \varepsilon_{s13} < 0 = 1$ elastic in tension

$f_{s13} := E_s \cdot \varepsilon_{s13} = -26.112\ \mathbf{ksi}$ $\quad F_{s13} := A_{s13} \cdot f_{s13} = -208.896\ \mathbf{kip}$

$|\varepsilon_{s14}| < \varepsilon_{ty} = 1 \quad \varepsilon_{s14} > 0 = 1$ elastic in compression

$f_{s14} := E_s \cdot \varepsilon_{s14} - 0.85\ f'_c = 16.645\ \mathbf{ksi}$ $\quad F_{s14} := A_{s14} \cdot f_{s14} = 133.163\ \mathbf{kip}$

$|\varepsilon_{s15}| \geq \varepsilon_{ty} = 1 \quad \varepsilon_{s15} > 0 = 1$ yielded in compression

$f_{s15} := f_y - 0.85\ f'_c = 56.6\ \mathbf{ksi}$ $\quad F_{s15} := A_{s15} \cdot f_{s15} = 3169.6\ \mathbf{kip}$

$A_c := b \cdot a = 5.913\ \mathbf{ft}^2$

$F_c := 0.85 \cdot f'_c \cdot A_c = 2895221.727\ \mathbf{lbf}$

$y_c := h - \dfrac{a}{2} = 9.462\ \mathbf{ft}$

$$P_n := F_c + F_{s1} + F_{s2} + F_{s3} + F_{s4} + F_{s5} + F_{s6} + F_{s7} + F_{s8} + F_{s9} + F_{s10} + F_{s11} + F_{s12} + F_{s13} + F_{s14} + F_{s15}$$

$P_n = -2650.911$ **kip**

$$M_n := F_c \cdot (y_c - y_{bar}) + F_{s1} \cdot (y_{s1} - y_{bar}) + F_{s2} \cdot (y_{s2} - y_{bar}) + F_{s3} \cdot (y_{s3} - y_{bar}) + F_{s4} \cdot (y_{s4} - y_{bar}) + F_{s5} \cdot (y_{s5} - y_{bar}) + F_{s6} \cdot (y_{s6} - y_{bar}) + F_{s7} \cdot (y_{s7} - y_{bar}) + F_{s8} \cdot (y_{s8} - y_{bar}) + F_{s9} \cdot (y_{s9} - y_{bar}) + F_{s10} \cdot (y_{s10} - y_{bar}) + F_{s11} \cdot (y_{s11} - y_{bar}) + F_{s12} \cdot (y_{s12} - y_{bar}) + F_{s13} \cdot (y_{s13} - y_{bar}) + F_{s14} \cdot (y_{s14} - y_{bar}) + F_{s15} \cdot (y_{s15} - y_{bar})$$

$M_n = 46971.891$ **kip·ft**

$\varepsilon_t := |\varepsilon_{s1}| = 0.02$

$\varepsilon_t := |\varepsilon_{s1}| = 0.02$

$\varepsilon_t > \varepsilon_{ty} = 1$

$\varepsilon_t > \varepsilon_{ty} + 0.003 = 1$

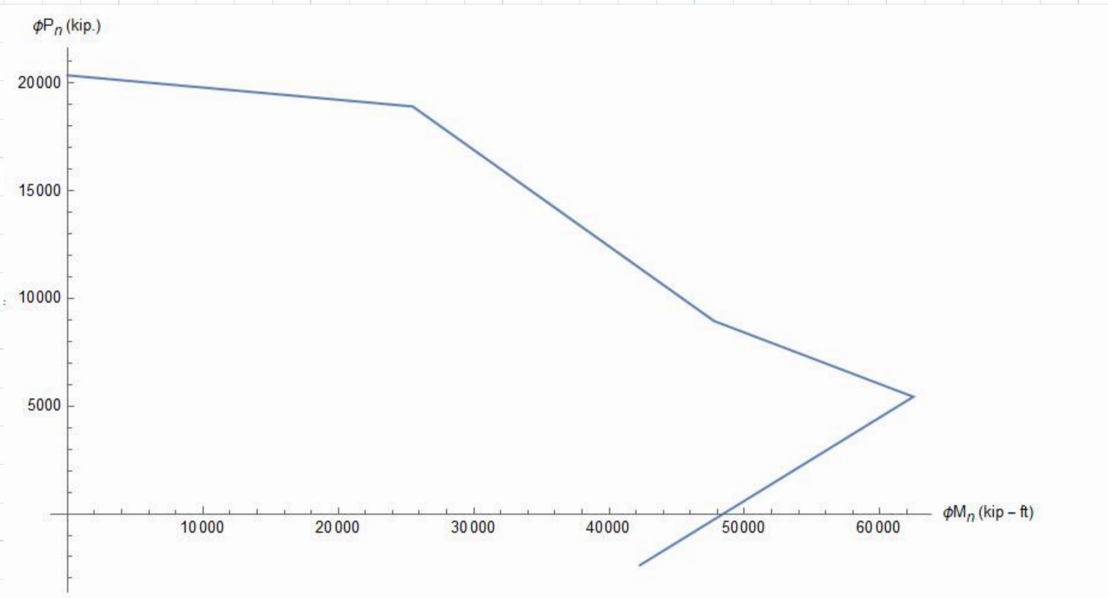
$\phi := 0.9$

$P_{rE} := \phi \cdot P_n = -2385.82$ **kip**

$M_{rE} := \phi \cdot M_n = 42274.702$ **kip·ft**

Summary:

$P_{rA} = 20359.872$ **kip**	$M_{rA} = 0$ **kip·ft**
$P_{rB} = 18915.057$ **kip**	$M_{rB} = 25491.79$ **kip·ft**
$P_{rC} = 8956.973$ **kip**	$M_{rC} = 47734.943$ **kip·ft**
$P_{rD} = 5433.61$ **kip**	$M_{rD} = 62463.077$ **kip·ft**
$P_{rE} = -2385.82$ **kip**	$M_{rE} = 42274.702$ **kip·ft**



In Strength I

$P_u := 3616.23\ \mathbf{kip}$

$M_u := 53675.2\ \mathbf{kip \cdot ft}$

This point falls within the curve, therefore OK

Shear strength

$d := h - y_{s1} = 9.698\ \mathbf{ft}$ $\lambda := 1$ NWC

$r := 0.288 \cdot d = 2.793\ \mathbf{ft}$

$M_u := 40176.5\ \mathbf{kip \cdot ft}$

$V_u := 1708.4\ \mathbf{kip}$

$$\phi\ V_c := 0.75 \cdot \min\left(3.3 \cdot \lambda \cdot \sqrt{\frac{f'_c}{\mathbf{psi}}}\ \mathbf{psi} \cdot h \cdot d, \left(0.6 \cdot \lambda \cdot \sqrt{\frac{f'_c}{\mathbf{psi}}}\ \mathbf{psi} + b \cdot \frac{\left(1.25 \cdot \lambda \cdot \sqrt{\frac{f'_c}{\mathbf{psi}}}\ \mathbf{psi}\right)}{\frac{M_u}{V_u} - \frac{b}{2}}\right) \cdot h \cdot d\right)$$

$\phi\ V_c = 616.728\ \mathbf{kip}$

$V_u > \phi\ V_c$

bars are required: ties

$A_{ties} := 0.2\ \mathbf{in}^2$

$A_t := 2 \cdot A_{ties} = 0.003\ \mathbf{ft}^2$

$\phi := 0.75$

$s_2 := \dfrac{A_t \cdot \phi \cdot f_y \cdot d}{V_u - \phi\ V_c} = 1.919\ \mathbf{in}$

$s_2 := 1.25\ \mathbf{in}$

$\rho_t := \dfrac{A_t}{h \cdot s_2} = 0.0027$ OK, minimum ratio of 0.0025

Buckling Strength

$A_{st} = 1.5\ \mathbf{ft}^2$

$A_g = 55\ \mathbf{ft}^2$

$A_c := A_g - A_{st} = 53.5\ \mathbf{ft}^2$

$\phi\ P_n := 0.8 \cdot 0.65 \cdot \left(0.85 \cdot f'_c \cdot A_c + f_y \cdot A_{st}\right) = 20359.872\ \mathbf{kip}$

$r := 0.288 \cdot h = 2.88\ \mathbf{ft}$

$L := 27.5\ \mathbf{ft}$

$k := 0.65$

$$F_e := \frac{\pi^2 \cdot E_c}{\left(\frac{k \cdot L}{r}\right)^2} = 932.597 \ \mathbf{ksi}$$

$$P_{cr} := F_e \cdot A_g = 7386168.062 \ \mathbf{kip}$$

Axial load capacity based on elastic buckling is much larger than that based on strength. Strength controls.

Reinforcement

minimal horizontal clear spacing

$$s_{bc} := \max\left(1 \ \mathbf{in}, d_{bar}\right) = 2.257 \ \mathbf{in}$$

minimal vertical clear spacing

$$s_{hc} := 1 \ \mathbf{in}$$

$16 \cdot d_{bar} = 3.009 \ \mathbf{ft}$

maximum vertical spacing between ties

$b = 5.5 \ \mathbf{ft}$

$48 \cdot d_{ties} = 2 \ \mathbf{ft}$

$s \leq \min\left(16 \cdot d_{bar}, 48 \cdot d_{ties}, b\right) = 1$ $\qquad s := 2 \ \mathbf{ft}$

<u>Wind Loading</u>

Biaxial interaction check

Create P-M diagram for the other direction of moment

Cross-Section Dimensions: $b := 10 \ \mathbf{ft}$ $\qquad h := 5.5 \ \mathbf{ft}$

$$A_g := b \cdot h = 55 \ \mathbf{ft}^2$$

$n_{bars} := 54$ $\qquad A_{bar} := 4 \ \mathbf{in}^2$

$f_y := 60 \ \mathbf{ksi}$ $\qquad A_{st} := n_{bars} \cdot A_{bar}$

$\frac{P_u}{A_g} = 0.457 \ \mathbf{ksi}$ $\qquad \rho_g := \frac{A_{st}}{A_g} = 0.027$

$\phi P_n := 0.8 \cdot 0.65 \cdot (0.85 \cdot f'_c \cdot A_g + (f_y - 0.85 \cdot f'_c) \cdot A_{st})$

$P_r := \phi P_n = 20359.872$ **kip**

Point A

$P_{rA} := \phi P_n = 20359.872$ **kip**

$\phi M_{nA} := 0$ **kip·ft** $\quad M_{rA} := \phi M_{nA}$

Point B $\quad E_s := 29000$ **ksi** $\quad$ #18 rebar

$\varepsilon_{cu} := 0.003 \quad d_{ties} := 0.5$ **in** $\quad d_{bar} := 2.257$ **in**

$c_c := 2$ **in** $\quad y_{bar} := \frac{h}{2} = 2.75$ **ft**

$A_{s1} := 15 \cdot A_{bar} \quad A_{s2} := 2 \cdot A_{bar} \quad A_{s3} := 2 \cdot A_{bar}$

$A_{s4} := 2 \cdot A_{bar} \quad A_{s5} := 2 \cdot A_{bar} \quad A_{s6} := 2 \cdot A_{bar}$

$A_{s7} := 2 \cdot A_{bar} \quad A_{s8} := 2 \cdot A_{bar} \quad A_{s9} := 2 \cdot A_{bar}$

$A_{s10} := 2 \cdot A_{bar} \quad A_{s11} := 2 \cdot A_{bar} \quad A_{s12} := 2 \cdot A_{bar}$

$A_{s13} := 2 \cdot A_{bar} \quad A_{s14} := 15 \cdot A_{bar}$

$y_{s1} := c_c + d_{ties} + \frac{d_{bar}}{2} = 3.629$ **in** $\quad y_{s14} := h - \left(c_c + d_{ties} + \frac{d_{bar}}{2}\right) = 5.198$ **ft**

$s := \frac{y_{s14} - y_{s1}}{13} = 4.519$ **in**

$y_{s2} := y_{s1} + s = 8.147$ **in** $\quad y_{s3} := y_{s1} + 2 \cdot s = 12.666$ **in**

$y_{s4} := y_{s1} + 3 \cdot s = 17.185\ \textbf{in}$ $y_{s5} := y_{s1} + 4 \cdot s = 1.809\ \textbf{ft}$

$y_{s6} := y_{s1} + 5 \cdot s = 2.185\ \textbf{ft}$ $y_{s7} := y_{s1} + 6 \cdot s = 2.562\ \textbf{ft}$

$y_{s8} := y_{s1} + 7 \cdot s = 2.938\ \textbf{ft}$ $y_{s9} := y_{s1} + 8 \cdot s = 3.315\ \textbf{ft}$

$y_{s10} := y_{s1} + 9 \cdot s = 3.691\ \textbf{ft}$ $y_{s11} := y_{s1} + 10 \cdot s = 4.068\ \textbf{ft}$

$y_{s12} := y_{s1} + 11 \cdot s = 4.445\ \textbf{ft}$ $y_{s13} := y_{s1} + 12 \cdot s = 4.821\ \textbf{ft}$

$$\varepsilon_{ty} := \frac{f_y}{E_s} = 0.002$$

$\varepsilon_{s1} := 0$ $$\varepsilon_o := \frac{\varepsilon_{s1} \cdot h - \varepsilon_{cu} \cdot y_{s1}}{h - y_{s1}} = -1.745 \cdot 10^{-4}$$

$$c := \frac{\varepsilon_{cu} \cdot h}{\varepsilon_{cu} - \varepsilon_o} = 62.372\ \textbf{in}$$

$\beta_1 := 0.85$

$a := \beta_1 \cdot c = 4.418\ \textbf{ft}$

$$\varepsilon_{s2} := \frac{(h - y_{s2})}{h} \cdot \varepsilon_o + \frac{y_{s2}}{h} \cdot \varepsilon_{cu} = 0.00022$$

$$\varepsilon_{s9} := \frac{(h - y_{s9})}{h} \cdot \varepsilon_o + \frac{y_{s9}}{h} \cdot \varepsilon_{cu} = 0.00174$$

$$\varepsilon_{s3} := \frac{(h - y_{s3})}{h} \cdot \varepsilon_o + \frac{y_{s3}}{h} \cdot \varepsilon_{cu} = 0.00043$$

$$\varepsilon_{s10} := \frac{(h - y_{s10})}{h} \cdot \varepsilon_o + \frac{y_{s10}}{h} \cdot \varepsilon_{cu} = 0.00196$$

$$\varepsilon_{s4} := \frac{(h - y_{s4})}{h} \cdot \varepsilon_o + \frac{y_{s4}}{h} \cdot \varepsilon_{cu} = 0.00065$$

$$\varepsilon_{s11} := \frac{(h - y_{s11})}{h} \cdot \varepsilon_o + \frac{y_{s11}}{h} \cdot \varepsilon_{cu} = 0.00217$$

$$\varepsilon_{s5} := \frac{(h - y_{s5})}{h} \cdot \varepsilon_o + \frac{y_{s5}}{h} \cdot \varepsilon_{cu} = 0.00087$$

$$\varepsilon_{s6} := \frac{(h - y_{s6})}{h} \cdot \varepsilon_o + \frac{y_{s6}}{h} \cdot \varepsilon_{cu} = 0.00109$$

$$\varepsilon_{s12} := \frac{(h - y_{s12})}{h} \cdot \varepsilon_o + \frac{y_{s12}}{h} \cdot \varepsilon_{cu} = 0.00239$$

$$\varepsilon_{s7} := \frac{(h - y_{s7})}{h} \cdot \varepsilon_o + \frac{y_{s7}}{h} \cdot \varepsilon_{cu} = 0.0013$$

$$\varepsilon_{s13} := \frac{(h - y_{s13})}{h} \cdot \varepsilon_o + \frac{y_{s13}}{h} \cdot \varepsilon_{cu} = 0.00261$$

$$\varepsilon_{s8} := \frac{(h - y_{s8})}{h} \cdot \varepsilon_o + \frac{y_{s8}}{h} \cdot \varepsilon_{cu} = 0.00152$$

$$\varepsilon_{s14} := \frac{(h - y_{s14})}{h} \cdot \varepsilon_o + \frac{y_{s14}}{h} \cdot \varepsilon_{cu} = 0.00283$$

stresses

$\varepsilon_{ty} = 0.00207$

$f_{s1} := 0$ **ksi** strain was 0

$F_{s1} := A_{s1} \cdot f_{s1} = 0$ **lbf**

$\varepsilon_{s2} < \varepsilon_{ty} = 1$ elastic in compression

$f_{s2} := E_s \cdot \varepsilon_{s2} - 0.85 \cdot f'_c = 2.903$ **ksi**

$F_{s2} := A_{s2} \cdot f_{s2} = 23.224$ **kip**

$\varepsilon_{s3} < \varepsilon_{ty} = 1$ elastic in compression

$f_{s3} := E_s \cdot \varepsilon_{s3} - 0.85 \cdot f'_c = 9.206$ **ksi**

$F_{s3} := A_{s3} \cdot f_{s3} = 73.648$ **kip**

$\varepsilon_{s4} < \varepsilon_{ty} = 1$ elastic in compression

$f_{s4} := E_s \cdot \varepsilon_{s4} - 0.85 \cdot f'_c = 15.509$ **ksi**

$F_{s4} := A_{s4} \cdot f_{s4} = 124.071$ **kip**

$\varepsilon_{s5} < \varepsilon_{ty} = 1$ elastic in compression

$f_{s5} := E_s \cdot \varepsilon_{s5} - 0.85\ f'_c = 21.812$ **ksi**

$F_{s5} := A_{s5} \cdot f_{s5} = 174.495$ **kip**

$\varepsilon_{s6} < \varepsilon_{ty} = 1$ elastic in compression

$f_{s6} := E_s \cdot \varepsilon_{s6} - 0.85\ f'_c = 28.115$ **ksi**

$F_{s6} := A_{s6} \cdot f_{s6} = 224.919$ **kip**

$\varepsilon_{s7} < \varepsilon_{ty} = 1$ elastic in compression

$f_{s7} := E_s \cdot \varepsilon_{s7} - 0.85\ f'_c = 34.418$ **ksi**

$F_{s7} := A_{s7} \cdot f_{s7} = 275.343$ **kip**

$\varepsilon_{s8} < \varepsilon_{ty} = 1$ elastic in compression

$f_{s8} := E_s \cdot \varepsilon_{s8} - 0.85\ f'_c = 40.721$ **ksi**

$F_{s8} := A_{s8} \cdot f_{s8} = 325.767$ **kip**

$\varepsilon_{s9} < \varepsilon_{ty} = 1$ elastic in compression

$f_{s9} := E_s \cdot \varepsilon_{s9} - 0.85\ f'_c = 47.024$ **ksi**

$F_{s9} := A_{s9} \cdot f_{s9} = 376.191$ **kip**

$\varepsilon_{s10} < \varepsilon_{ty} = 1$ elastic in compression

$f_{s10} := E_s \cdot \varepsilon_{s10} - 0.85\ f'_c = 53.327\ \mathbf{ksi}$ $F_{s10} := A_{s10} \cdot f_{s10} = 426.614\ \mathbf{kip}$

$\varepsilon_{s11} \geq \varepsilon_{ty} = 1$ yielded in compression

$f_{s11} := f_y - 0.85\ f'_c = 56.6\ \mathbf{ksi}$ $F_{s11} := A_{s11} \cdot f_{s11} = 452.8\ \mathbf{kip}$

$\varepsilon_{s12} \geq \varepsilon_{ty} = 1$ yielded in compression

$f_{s12} := f_y - 0.85\ f'_c = 56.6\ \mathbf{ksi}$ $F_{s12} := A_{s12} \cdot f_{s12} = 452.8\ \mathbf{kip}$

$\varepsilon_{s13} \geq \varepsilon_{ty} = 1$ yielded in compression

$f_{s13} := f_y - 0.85\ f'_c = 56.6\ \mathbf{ksi}$ $F_{s13} := A_{s13} \cdot f_{s13} = 452.8\ \mathbf{kip}$

$\varepsilon_{s14} \geq \varepsilon_{ty} = 1$ yielded in compression

$f_{s14} := f_y - 0.85\ f'_c = 56.6\ \mathbf{ksi}$ $F_{s14} := A_{s14} \cdot f_{s14} = 3396\ \mathbf{kip}$

$A_c := b \cdot a = 44.18\ \mathbf{ft}^2$

$F_c := 0.85 \cdot f'_c \cdot A_c = 21630.436\ \mathbf{kip}$

$y_c := h - \frac{a}{2} = 3.291\ \mathbf{ft}$

$$P_n := F_c + F_{s1} + F_{s2} + F_{s3} + F_{s4} + F_{s5} + F_{s6} + F_{s7} + F_{s8} + F_{s9} + F_{s10} + F_{s11} + F_{s12} + F_{s13} + F_{s14}$$

$P_n = 28409.109\ \mathbf{kip}$

$$M_n := F_c \cdot (y_c - y_{bar}) + F_{s1} \cdot (y_{s1} - y_{bar}) + F_{s2} \cdot (y_{s2} - y_{bar}) + F_{s3} \cdot (y_{s3} - y_{bar}) + F_{s4} \cdot (y_{s4} - y_{bar}) + F_{s5} \cdot (y_{s5} - y_{bar}) + F_{s6} \cdot (y_{s6} - y_{bar}) + F_{s7} \cdot (y_{s7} - y_{bar}) + F_{s8} \cdot (y_{s8} - y_{bar}) + F_{s9} \cdot (y_{s9} - y_{bar}) + F_{s10} \cdot (y_{s10} - y_{bar}) + F_{s11} \cdot (y_{s11} - y_{bar}) + F_{s12} \cdot (y_{s12} - y_{bar}) + F_{s13} \cdot (y_{s13} - y_{bar}) + F_{s14} \cdot (y_{s14} - y_{bar})$$

$M_n = 22312.091\ \mathbf{kip \cdot ft}$

$\varepsilon_t := \varepsilon_{s1} = 0$

$\phi := 0.65$

$P_{rB} := \phi \cdot P_n = 18465.921$ **kip**

$M_{rB} := \phi \cdot M_n = 14502.859$ **kip·ft**

Point C

$\varepsilon_{s1} := \frac{-f_y}{E_s} = -0.002$

$\varepsilon_o := \frac{\varepsilon_{s1} \cdot h - \varepsilon_{cu} \cdot y_{s1}}{h - y_{s1}} = -0.002$

$c := \frac{\varepsilon_{cu} \cdot h}{\varepsilon_{cu} - \varepsilon_o} = 36.914$ **in**

$a := \beta_1 \cdot c = 2.615$ **ft**

$\varepsilon_{s2} := \frac{(h - y_{s2})}{h} \cdot \varepsilon_o + \frac{y_{s2}}{h} \cdot \varepsilon_{cu} = -0.0017$

$\varepsilon_{s3} := \frac{(h - y_{s3})}{h} \cdot \varepsilon_o + \frac{y_{s3}}{h} \cdot \varepsilon_{cu} = -0.00133$

$\varepsilon_{s4} := \frac{(h - y_{s4})}{h} \cdot \varepsilon_o + \frac{y_{s4}}{h} \cdot \varepsilon_{cu} = -0.00097$

$\varepsilon_{s5} := \frac{(h - y_{s5})}{h} \cdot \varepsilon_o + \frac{y_{s5}}{h} \cdot \varepsilon_{cu} = -0.0006$

$\varepsilon_{s6} := \frac{(h - y_{s6})}{h} \cdot \varepsilon_o + \frac{y_{s6}}{h} \cdot \varepsilon_{cu} = -0.00023$

$\varepsilon_{s7} := \frac{(h - y_{s7})}{h} \cdot \varepsilon_o + \frac{y_{s7}}{h} \cdot \varepsilon_{cu} = 0.00013$

$\varepsilon_{s8} := \frac{(h - y_{s8})}{h} \cdot \varepsilon_o + \frac{y_{s8}}{h} \cdot \varepsilon_{cu} = 0.0005$

$\varepsilon_{s9} := \frac{(h - y_{s9})}{h} \cdot \varepsilon_o + \frac{y_{s9}}{h} \cdot \varepsilon_{cu} = 0.00087$

$\varepsilon_{s10} := \frac{(h - y_{s10})}{h} \cdot \varepsilon_o + \frac{y_{s10}}{h} \cdot \varepsilon_{cu} = 0.00124$

$\varepsilon_{s11} := \frac{(h - y_{s11})}{h} \cdot \varepsilon_o + \frac{y_{s11}}{h} \cdot \varepsilon_{cu} = 0.0016$

$\varepsilon_{s12} := \frac{(h - y_{s12})}{h} \cdot \varepsilon_o + \frac{y_{s12}}{h} \cdot \varepsilon_{cu} = 0.00197$

$\varepsilon_{s13} := \frac{(h - y_{s13})}{h} \cdot \varepsilon_o + \frac{y_{s13}}{h} \cdot \varepsilon_{cu} = 0.00234$

$\varepsilon_{s14} := \frac{(h - y_{s14})}{h} \cdot \varepsilon_o + \frac{y_{s14}}{h} \cdot \varepsilon_{cu} = 0.00271$

stresses

$\varepsilon_{ty} = 0.00207$

$|\varepsilon_{s1}| \geq \varepsilon_{ty} = 1$ $\quad \varepsilon_{s1} < 0 = 1$ $\quad$ yielded in tension

$f_{s1} := -f_y = -60$ **ksi**

$F_{s1} := A_{s1} \cdot f_{s1} = -3600$ **kip**

$|\varepsilon_{s2}| < \varepsilon_{ty} = 1$ $\varepsilon_{s2} < 0 = 1$ elastic in tension

$f_{s2} := E_s \cdot \varepsilon_{s2} = -49.35$ **ksi** $F_{s2} := A_{s2} \cdot f_{s2} = -394.801$ **kip**

$|\varepsilon_{s3}| < \varepsilon_{ty} = 1$ $\varepsilon_{s3} < 0 = 1$ elastic in tension

$f_{s3} := E_s \cdot \varepsilon_{s3} = -38.7$ **ksi** $F_{s3} := A_{s3} \cdot f_{s3} = -309.602$ **kip**

$|\varepsilon_{s4}| < \varepsilon_{ty} = 1$ $\varepsilon_{s4} < 0 = 1$ elastic in tension

$f_{s4} := E_s \cdot \varepsilon_{s4} = -28.05$ **ksi** $F_{s4} := A_{s4} \cdot f_{s4} = -224.403$ **kip**

$|\varepsilon_{s5}| < \varepsilon_{ty} = 1$ $\varepsilon_{s5} < 0 = 1$ elastic in tension

$f_{s5} := E_s \cdot \varepsilon_{s5} = -17.401$ **ksi** $F_{s5} := A_{s5} \cdot f_{s5} = -139.204$ **kip**

$|\varepsilon_{s6}| < \varepsilon_{ty} = 1$ $\varepsilon_{s6} < 0 = 1$ elastic in tension

$f_{s6} := E_s \cdot \varepsilon_{s6} = -6.751$ **ksi** $F_{s6} := A_{s6} \cdot f_{s6} = -54.006$ **kip**

$|\varepsilon_{s7}| < \varepsilon_{ty} = 1$ $\varepsilon_{s7} > 0 = 1$ elastic in compression

$f_{s7} := E_s \cdot \varepsilon_{s7} - 0.85 \cdot f'_c = 0.499$ **ksi** $F_{s7} := A_{s7} \cdot f_{s7} = 3.993$ **kip**

$|\varepsilon_{s8}| < \varepsilon_{ty} = 1$ $\varepsilon_{s8} > 0 = 1$ elastic in compression

$f_{s8} := E_s \cdot \varepsilon_{s8} - 0.85\ f'_c = 11.149$ **ksi** $F_{s8} := A_{s8} \cdot f_{s8} = 89.192$ **kip**

$|\varepsilon_{s9}| < \varepsilon_{ty} = 1$ $\varepsilon_{s9} > 0 = 1$ elastic in compression

$f_{s9} := E_s \cdot \varepsilon_{s9} - 0.85\ f'_c = 21.799$ **ksi** $F_{s9} := A_{s9} \cdot f_{s9} = 174.391$ **kip**

$|\varepsilon_{s10}| < \varepsilon_{ty} = 1$ $\varepsilon_{s10} > 0 = 1$ elastic in compression

$f_{s10} := E_s \cdot \varepsilon_{s10} - 0.85\ f'_c = 32.449$ **ksi** $F_{s10} := A_{s10} \cdot f_{s10} = 259.59$ **kip**

$|\varepsilon_{s11}| < \varepsilon_{ty} = 1$ $\varepsilon_{s11} > 0 = 1$ elastic in compression

$f_{s11} := E_s \cdot \varepsilon_{s11} - 0.85\ f'_c = 43.099$ **ksi** $F_{s11} := A_{s11} \cdot f_{s11} = 344.789$ **kip**

$|\varepsilon_{s12}| < \varepsilon_{ty} = 1$ $\varepsilon_{s12} > 0 = 1$ elastic in compression

$f_{s12} := E_s \cdot \varepsilon_{s12} - 0.85\ f'_c = 53.748$ **ksi** $F_{s12} := A_{s12} \cdot f_{s12} = 429.988$ **kip**

$\left|\varepsilon_{s13}\right| \geq \varepsilon_{ty} = 1 \qquad \varepsilon_{s13} > 0 = 1 \qquad$ yielded in compression

$f_{s13} := f_y - 0.85\ f'_c = 56.6\ \mathbf{ksi} \qquad F_{s13} := A_{s13} \cdot f_{s13} = 452.8\ \mathbf{kip}$

$\left|\varepsilon_{s14}\right| \geq \varepsilon_{ty} = 1 \qquad \varepsilon_{s14} > 0 = 1 \qquad$ yielded in compression

$f_{s14} := f_y - 0.85\ f'_c = 56.6\ \mathbf{ksi} \qquad F_{s14} := A_{s14} \cdot f_{s14} = 3396\ \mathbf{kip}$

$A_c := b \cdot a = 26.147\ \mathbf{ft}^2$

$F_c := 0.85 \cdot f'_c \cdot A_c = 12801686.731\ \mathbf{lbf}$

$y_c := h - \frac{a}{2} = 4.193\ \mathbf{ft}$

$P_n := F_c + F_{s1} + F_{s2} + F_{s3} + F_{s4} + F_{s5} + F_{s6} + F_{s7} + F_{s8} + F_{s9} + F_{s10} + F_{s11} + F_{s12} + F_{s13} + F_{s14}$

$P_n = 13230.413\ \mathbf{kip}$

$$M_n := F_c \cdot (y_c - y_{bar}) + F_{s1} \cdot (y_{s1} - y_{bar}) + F_{s2} \cdot (y_{s2} - y_{bar}) + F_{s3} \cdot (y_{s3} - y_{bar}) + F_{s4} \cdot (y_{s4} - y_{bar}) + F_{s5} \cdot (y_{s5} - y_{bar}) + F_{s6} \cdot (y_{s6} - y_{bar}) + F_{s7} \cdot (y_{s7} - y_{bar}) + F_{s8} \cdot (y_{s8} - y_{bar}) + F_{s9} \cdot (y_{s9} - y_{bar}) + F_{s10} \cdot (y_{s10} - y_{bar}) + F_{s11} \cdot (y_{s11} - y_{bar}) + F_{s12} \cdot (y_{s12} - y_{bar}) + F_{s13} \cdot (y_{s13} - y_{bar}) + F_{s14} \cdot (y_{s14} - y_{bar})$$

$M_n = 39871.107\ \mathbf{kip \cdot ft}$

$\varepsilon_t := \left|\varepsilon_{s1}\right| = 0.002$

$\phi := 0.65$

$P_{rC} := \phi \cdot P_n = 8599.768\ \mathbf{kip}$

$M_{rC} := \phi \cdot M_n = 25916.219\ \mathbf{kip \cdot ft}$

Point D

$\varepsilon_{s1} := -0.005 \qquad \varepsilon_o := \frac{\varepsilon_{s1} \cdot h - \varepsilon_{cu} \cdot y_{s1}}{h - y_{s1}} = -0.005$

$c := \frac{\varepsilon_{cu} \cdot h}{\varepsilon_{cu} - \varepsilon_o} = 23.389\ \mathbf{in}$

$a := \beta_1 \cdot c = 1.657$ **ft**

$\varepsilon_{s2} := \frac{(h - y_{s2})}{h} \cdot \varepsilon_o + \frac{y_{s2}}{h} \cdot \varepsilon_{cu} = -0.00442$

$\varepsilon_{s3} := \frac{(h - y_{s3})}{h} \cdot \varepsilon_o + \frac{y_{s3}}{h} \cdot \varepsilon_{cu} = -0.00384$

$\varepsilon_{s4} := \frac{(h - y_{s4})}{h} \cdot \varepsilon_o + \frac{y_{s4}}{h} \cdot \varepsilon_{cu} = -0.00326$

$\varepsilon_{s5} := \frac{(h - y_{s5})}{h} \cdot \varepsilon_o + \frac{y_{s5}}{h} \cdot \varepsilon_{cu} = -0.00268$

$\varepsilon_{s6} := \frac{(h - y_{s6})}{h} \cdot \varepsilon_o + \frac{y_{s6}}{h} \cdot \varepsilon_{cu} = -0.0021$

$\varepsilon_{s7} := \frac{(h - y_{s7})}{h} \cdot \varepsilon_o + \frac{y_{s7}}{h} \cdot \varepsilon_{cu} = -0.00152$

$\varepsilon_{s8} := \frac{(h - y_{s8})}{h} \cdot \varepsilon_o + \frac{y_{s8}}{h} \cdot \varepsilon_{cu} = -0.00094$

$\varepsilon_{s9} := \frac{(h - y_{s9})}{h} \cdot \varepsilon_o + \frac{y_{s9}}{h} \cdot \varepsilon_{cu} = -0.00036$

$\varepsilon_{s10} := \frac{(h - y_{s10})}{h} \cdot \varepsilon_o + \frac{y_{s10}}{h} \cdot \varepsilon_{cu} = 0.00022$

$\varepsilon_{s11} := \frac{(h - y_{s11})}{h} \cdot \varepsilon_o + \frac{y_{s11}}{h} \cdot \varepsilon_{cu} = 0.0008$

$\varepsilon_{s12} := \frac{(h - y_{s12})}{h} \cdot \varepsilon_o + \frac{y_{s12}}{h} \cdot \varepsilon_{cu} = 0.00138$

$\varepsilon_{s13} := \frac{(h - y_{s13})}{h} \cdot \varepsilon_o + \frac{y_{s13}}{h} \cdot \varepsilon_{cu} = 0.00196$

$\varepsilon_{s14} := \frac{(h - y_{s14})}{h} \cdot \varepsilon_o + \frac{y_{s14}}{h} \cdot \varepsilon_{cu} = 0.00253$

stresses

$\varepsilon_{ty} = 0.00207$

$|\varepsilon_{s1}| \geq \varepsilon_{ty} = 1$ $\quad \varepsilon_{s1} < 0 = 1$ yielded in tension

$f_{s1} := -f_y = -60$ **ksi** $\quad F_{s1} := A_{s1} \cdot f_{s1} = -3600$ **kip**

$|\varepsilon_{s2}| \geq \varepsilon_{ty} = 1$ $\quad \varepsilon_{s2} < 0 = 1$ yielded in tension

$f_{s2} := -f_y = -60$ **ksi** $\quad F_{s2} := A_{s2} \cdot f_{s2} = -480$ **kip**

$|\varepsilon_{s3}| \geq \varepsilon_{ty} = 1$ $\quad \varepsilon_{s3} < 0 = 1$ yielded in tension

$f_{s3} := -f_y = -60$ **ksi** $\quad F_{s3} := A_{s3} \cdot f_{s3} = -480$ **kip**

$|\varepsilon_{s4}| \geq \varepsilon_{ty} = 1$ $\quad \varepsilon_{s4} < 0 = 1$ yielded in tension

$f_{s4} := -f_y = -60$ **ksi** $\quad F_{s4} := A_{s4} \cdot f_{s4} = -480$ **kip**

$|\varepsilon_{s5}| \geq \varepsilon_{ty} = 1$ $\varepsilon_{s5} < 0 = 1$ yielded in tension

$f_{s5} := -f_y = -60$ **ksi** $F_{s5} := A_{s5} \cdot f_{s5} = -480$ **kip**

$|\varepsilon_{s6}| \geq \varepsilon_{ty} = 1$ $\varepsilon_{s6} < 0 = 1$ yielded in tension

$f_{s6} := -f_y = -60$ **ksi** $F_{s6} := A_{s6} \cdot f_{s6} = -480$ **kip**

$|\varepsilon_{s7}| < \varepsilon_{ty} = 1$ $\varepsilon_{s7} < 0 = 1$ elastic in tension

$f_{s7} := E_s \cdot \varepsilon_{s7} = -44.152$ **ksi** $F_{s7} := A_{s7} \cdot f_{s7} = -353.219$ **kip**

$|\varepsilon_{s8}| < \varepsilon_{ty} = 1$ $\varepsilon_{s8} < 0 = 1$ elastic in tension

$f_{s8} := E_s \cdot \varepsilon_{s8} = -27.344$ **ksi** $F_{s8} := A_{s8} \cdot f_{s8} = -218.755$ **kip**

$|\varepsilon_{s9}| < \varepsilon_{ty} = 1$ $\varepsilon_{s9} < 0 = 1$ elastic in tension

$f_{s9} := E_s \cdot \varepsilon_{s9} = -10.536$ **ksi** $F_{s9} := A_{s9} \cdot f_{s9} = -84.292$ **kip**

$|\varepsilon_{s10}| < \varepsilon_{ty} = 1$ $\varepsilon_{s10} < 0 = 0$ elastic in tension

$f_{s10} := E_s \cdot \varepsilon_{s10} = 6.271$ **ksi** $F_{s10} := A_{s10} \cdot f_{s10} = 50.172$ **kip**

$|\varepsilon_{s11}| < \varepsilon_{ty} = 1$ $\varepsilon_{s11} > 0 = 1$ elastic in compression

$f_{s11} := E_s \cdot \varepsilon_{s11} - 0.85\ f'_c = 19.679$ **ksi** $F_{s11} := A_{s11} \cdot f_{s11} = 157.435$ **kip**

$|\varepsilon_{s12}| < \varepsilon_{ty} = 1$ $\varepsilon_{s12} > 0 = 1$ elastic in compression

$f_{s12} := E_s \cdot \varepsilon_{s12} - 0.85\ f'_c = 36.487$ **ksi** $F_{s12} := A_{s12} \cdot f_{s12} = 291.899$ **kip**

$|\varepsilon_{s13}| < \varepsilon_{ty} = 1$ $\varepsilon_{s13} > 0 = 1$ elastic in compression

$f_{s13} := E_s \cdot \varepsilon_{s13} - 0.85\ f'_c = 53.295$ **ksi** $F_{s13} := A_{s13} \cdot f_{s13} = 426.363$ **kip**

$|\varepsilon_{s14}| \geq \varepsilon_{ty} = 1$ $\varepsilon_{s14} > 0 = 1$ yielded in compression

$f_{s14} := f_y - 0.85\ f'_c = 56.6$ **ksi** $F_{s14} := A_{s14} \cdot f_{s14} = 3396$ **kip**

$A_c := b \cdot a = 16.567\ \mathbf{ft}^2$

$F_c := 0.85 \cdot f'_c \cdot A_c = 8111413.575\ \mathbf{lbf}$

$y_c := h - \frac{a}{2} = 4.672\ \mathbf{ft}$

$$P_n := F_c + F_{s1} + F_{s2} + F_{s3} + F_{s4} + F_{s5} + F_{s6} + F_{s7} + F_{s8} + F_{s9} + F_{s10} + F_{s11} + F_{s12} + F_{s13} + F_{s14}$$

$P_n = 5777.017\ \mathbf{kip}$

$$M_n := F_c \cdot (y_c - y_{bar}) + F_{s1} \cdot (y_{s1} - y_{bar}) + F_{s2} \cdot (y_{s2} - y_{bar}) + F_{s3} \cdot (y_{s3} - y_{bar}) + F_{s4} \cdot (y_{s4} - y_{bar}) + F_{s5} \cdot (y_{s5} - y_{bar}) + F_{s6} \cdot (y_{s6} - y_{bar}) + F_{s7} \cdot (y_{s7} - y_{bar}) + F_{s8} \cdot (y_{s8} - y_{bar}) + F_{s9} \cdot (y_{s9} - y_{bar}) + F_{s10} \cdot (y_{s10} - y_{bar}) + F_{s11} \cdot (y_{s11} - y_{bar}) + F_{s12} \cdot (y_{s12} - y_{bar}) + F_{s13} \cdot (y_{s13} - y_{bar}) + F_{s14} \cdot (y_{s14} - y_{bar})$$

$M_n = 37483.873\ \mathbf{kip \cdot ft}$

$\varepsilon_t := |\varepsilon_{s1}| = 0.005$

$\varepsilon_t > \varepsilon_{ty} = 1$

$\varepsilon_t < \varepsilon_{ty} + 0.003 = 1$

$\phi := 0.65 + 0.25 \cdot \left(\frac{\varepsilon_t - \varepsilon_{ty}}{0.003} \right) = 0.894$

$P_{rD} := \phi \cdot P_n = 5166.114\ \mathbf{kip}$

$M_{rD} := \phi \cdot M_n = 33520.061\ \mathbf{kip \cdot ft}$

Point E

$\varepsilon_{s1} := -0.02$

$\varepsilon_o := \frac{\varepsilon_{s1} \cdot h - \varepsilon_{cu} \cdot y_{s1}}{h - y_{s1}} = -0.021$

$c := \frac{\varepsilon_{cu} \cdot h}{\varepsilon_{cu} - \varepsilon_o} = 8.135\ \mathbf{in}$

$a := \beta_1 \cdot c = 0.576\ \mathbf{ft}$

$\varepsilon_{s2} := \frac{(h - y_{s2})}{h} \cdot \varepsilon_o + \frac{y_{s2}}{h} \cdot \varepsilon_{cu} = -0.01833$

$\varepsilon_{s9} := \frac{(h - y_{s9})}{h} \cdot \varepsilon_o + \frac{y_{s9}}{h} \cdot \varepsilon_{cu} = -0.00667$

$$\varepsilon_{s3} := \frac{(h - y_{s3})}{h} \cdot \varepsilon_o + \frac{y_{s3}}{h} \cdot \varepsilon_{cu} = -0.01667$$

$$\varepsilon_{s10} := \frac{(h - y_{s10})}{h} \cdot \varepsilon_o + \frac{y_{s10}}{h} \cdot \varepsilon_{cu} = -0.005$$

$$\varepsilon_{s4} := \frac{(h - y_{s4})}{h} \cdot \varepsilon_o + \frac{y_{s4}}{h} \cdot \varepsilon_{cu} = -0.015$$

$$\varepsilon_{s11} := \frac{(h - y_{s11})}{h} \cdot \varepsilon_o + \frac{y_{s11}}{h} \cdot \varepsilon_{cu} = -0.00334$$

$$\varepsilon_{s5} := \frac{(h - y_{s5})}{h} \cdot \varepsilon_o + \frac{y_{s5}}{h} \cdot \varepsilon_{cu} = -0.01333$$

$$\varepsilon_{s12} := \frac{(h - y_{s12})}{h} \cdot \varepsilon_o + \frac{y_{s12}}{h} \cdot \varepsilon_{cu} = -0.00167$$

$$\varepsilon_{s6} := \frac{(h - y_{s6})}{h} \cdot \varepsilon_o + \frac{y_{s6}}{h} \cdot \varepsilon_{cu} = -0.01167$$

$$\varepsilon_{s13} := \frac{(h - y_{s13})}{h} \cdot \varepsilon_o + \frac{y_{s13}}{h} \cdot \varepsilon_{cu} = -4.3437 \cdot 10^{-6}$$

$$\varepsilon_{s7} := \frac{(h - y_{s7})}{h} \cdot \varepsilon_o + \frac{y_{s7}}{h} \cdot \varepsilon_{cu} = -0.01$$

$$\varepsilon_{s14} := \frac{(h - y_{s14})}{h} \cdot \varepsilon_o + \frac{y_{s14}}{h} \cdot \varepsilon_{cu} = 0.00166$$

$$\varepsilon_{s8} := \frac{(h - y_{s8})}{h} \cdot \varepsilon_o + \frac{y_{s8}}{h} \cdot \varepsilon_{cu} = -0.00834$$

stresses $\varepsilon_{ty} = 0.00207$

$|\varepsilon_{s1}| \geq \varepsilon_{ty} = 1$ $\varepsilon_{s1} < 0 = 1$ yielded in tension

$f_{s1} := -f_y = -60$ **ksi** $F_{s1} := A_{s1} \cdot f_{s1} = -3600$ **kip**

$|\varepsilon_{s2}| \geq \varepsilon_{ty} = 1$ $\varepsilon_{s2} < 0 = 1$ yielded in tension

$f_{s2} := -f_y = -60$ **ksi** $F_{s2} := A_{s2} \cdot f_{s2} = -480$ **kip**

$|\varepsilon_{s3}| \geq \varepsilon_{ty} = 1$ $\varepsilon_{s3} < 0 = 1$ yielded in tension

$f_{s3} := -f_y = -60$ **ksi** $F_{s3} := A_{s3} \cdot f_{s3} = -480$ **kip**

$|\varepsilon_{s4}| \geq \varepsilon_{ty} = 1$ $\varepsilon_{s4} < 0 = 1$ yielded in tension

$f_{s4} := -f_y = -60$ **ksi** $F_{s4} := A_{s4} \cdot f_{s4} = -480$ **kip**

$|\varepsilon_{s5}| \geq \varepsilon_{ty} = 1$ $\varepsilon_{s5} < 0 = 1$ yielded in tension

$f_{s5} := -f_y = -60$ **ksi** $F_{s5} := A_{s5} \cdot f_{s5} = -480$ **kip**

$|\varepsilon_{s6}| \geq \varepsilon_{ty} = 1$ $\varepsilon_{s6} < 0 = 1$ yielded in tension

$f_{s6} := -f_y = -60$ **ksi** $F_{s6} := A_{s6} \cdot f_{s6} = -480$ **kip**

$|\varepsilon_{s7}| \geq \varepsilon_{ty} = 1$ $\varepsilon_{s7} < 0 = 1$ yielded in tension

$f_{s7} := -f_y = -60$ **ksi** $F_{s7} := A_{s7} \cdot f_{s7} = -480$ **kip**

$|\varepsilon_{s8}| \geq \varepsilon_{ty} = 1$ $\varepsilon_{s8} < 0 = 1$ yielded in tension

$f_{s8} := -f_y = -60$ **ksi** $F_{s8} := A_{s8} \cdot f_{s8} = -480$ **kip**

$|\varepsilon_{s9}| \geq \varepsilon_{ty} = 1$ $\varepsilon_{s9} < 0 = 1$ yielded in tension

$f_{s9} := -f_y = -60$ **ksi** $F_{s9} := A_{s9} \cdot f_{s9} = -480$ **kip**

$|\varepsilon_{s10}| \geq \varepsilon_{ty} = 1$ $\varepsilon_{s10} < 0 = 1$ yielded in tension

$f_{s10} := -f_y = -60$ **ksi** $F_{s10} := A_{s10} \cdot f_{s10} = -480$ **kip**

$|\varepsilon_{s11}| \geq \varepsilon_{ty} = 1$ $\varepsilon_{s11} < 0 = 1$ yielded in tension

$f_{s11} := -f_y = -60$ **ksi** $F_{s11} := A_{s11} \cdot f_{s11} = -480$ **kip**

$|\varepsilon_{s12}| < \varepsilon_{ty} = 1$ $\varepsilon_{s12} < 0 = 1$ yielded in tension

$f_{s12} := E_s \cdot \varepsilon_{s12} = -48.449$ **ksi** $F_{s12} := A_{s12} \cdot f_{s12} = -387.59$ **kip**

$|\varepsilon_{s13}| < \varepsilon_{ty} = 1$ $\varepsilon_{s13} < 0 = 1$ elastic in tension

$f_{s13} := E_s \cdot \varepsilon_{s13} = -0.126$ **ksi** $F_{s13} := A_{s13} \cdot f_{s13} = -1.008$ **kip**

$|\varepsilon_{s14}| < \varepsilon_{ty} = 1$ $\varepsilon_{s14} > 0 = 1$ elastic in compression

$f_{s14} := E_s \cdot \varepsilon_{s14} - 0.85\ f'_c = 44.797$ **ksi** $F_{s14} := A_{s14} \cdot f_{s14} = 2687.812$ **kip**

$A_c := b \cdot a = 5.763$ **ft**^{2}

$F_c := 0.85 \cdot f'_c \cdot A_c = 2821361.243$ **lbf**

$y_c := h - \frac{a}{2} = 5.212$ **ft**

$$P_n := F_c + F_{s1} + F_{s2} + F_{s3} + F_{s4} + F_{s5} + F_{s6} + F_{s7} + F_{s8} + F_{s9} + F_{s10} + F_{s11} + F_{s12} + F_{s13} + F_{s14}$$

$$P_n = -3279.425\ \mathbf{kip}$$

$$\begin{aligned} M_n := {} & F_c \cdot (y_c - y_{bar}) + F_{s1} \cdot (y_{s1} - y_{bar}) + F_{s2} \cdot (y_{s2} - y_{bar}) + F_{s3} \cdot (y_{s3} - y_{bar}) \\ & + F_{s4} \cdot (y_{s4} - y_{bar}) + F_{s5} \cdot (y_{s5} - y_{bar}) + F_{s6} \cdot (y_{s6} - y_{bar}) + F_{s7} \cdot (y_{s7} - y_{bar}) \\ & + F_{s8} \cdot (y_{s8} - y_{bar}) + F_{s9} \cdot (y_{s9} - y_{bar}) + F_{s10} \cdot (y_{s10} - y_{bar}) + F_{s11} \cdot (y_{s11} - y_{bar}) \\ & + F_{s12} \cdot (y_{s12} - y_{bar}) + F_{s13} \cdot (y_{s13} - y_{bar}) + F_{s14} \cdot (y_{s14} - y_{bar}) \end{aligned}$$

$$M_n = 23484.647\ \mathbf{kip \cdot ft}$$

$$\varepsilon_t := |\varepsilon_{s1}| = 0.02$$

$$\varepsilon_t := |\varepsilon_{s1}| = 0.02$$

$$\varepsilon_t > \varepsilon_{ty} = 1$$

$$\varepsilon_t > \varepsilon_{ty} + 0.003 = 1$$

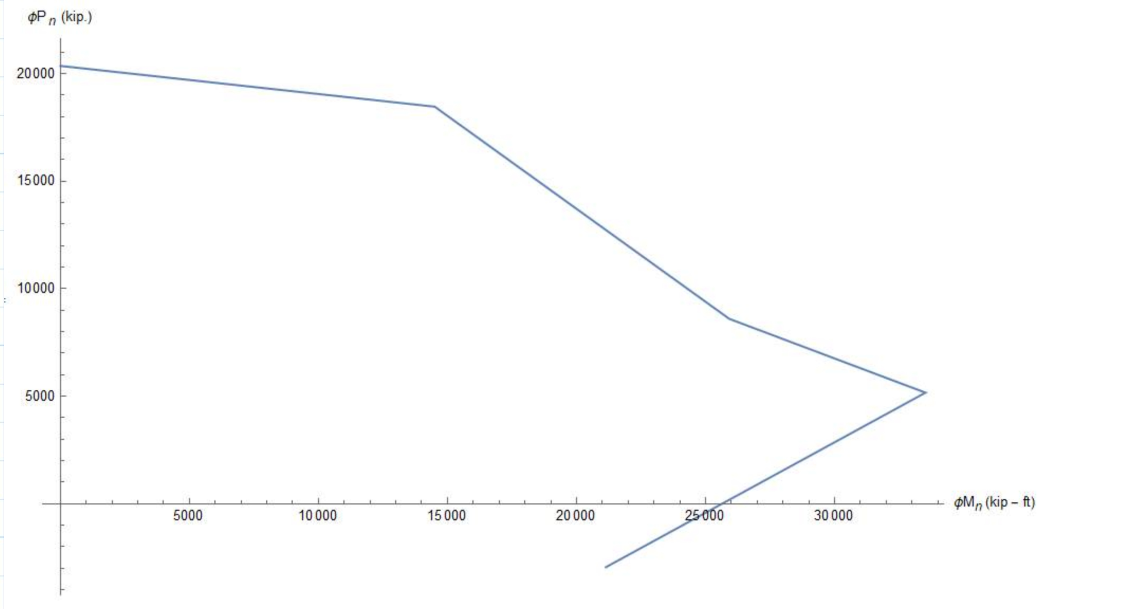
$$\phi := 0.9$$

$$P_{rE} := \phi \cdot P_n = -2951.482\ \mathbf{kip}$$

$$M_{rE} := \phi \cdot M_n = 21136.182\ \mathbf{kip \cdot ft}$$

Summary:

$P_{rA} = 20359.872\ \mathbf{kip}$	$M_{rA} = 0\ \mathbf{kip \cdot ft}$
$P_{rB} = 18465.921\ \mathbf{kip}$	$M_{rB} = 14502.859\ \mathbf{kip \cdot ft}$
$P_{rC} = 8599.768\ \mathbf{kip}$	$M_{rC} = 25916.219\ \mathbf{kip \cdot ft}$
$P_{rD} = 5166.114\ \mathbf{kip}$	$M_{rD} = 33520.061\ \mathbf{kip \cdot ft}$
$P_{rE} = -2951.482\ \mathbf{kip}$	$M_{rE} = 21136.182\ \mathbf{kip \cdot ft}$



Check Strength 3

$M_{ux} := 35815\ \textbf{kip} \cdot \textbf{ft}$

$M_{uy} := 1635.93\ \textbf{kip} \cdot \textbf{ft}$

$P_{ux} := 686.03\ \textbf{kip}$

From the diagram.

$\phi\ P_{ny} := 17000\ \textbf{kip}$ $\qquad \phi\ P_{nx} := 14000\ \textbf{kip}$

$\phi\ P_{n0} := P_{rA} = 20359.872\ \textbf{kip}$

$$\phi\ P_{neq} := \frac{1}{\left(\frac{1}{\phi\ P_{ny}} + \frac{1}{\phi\ P_{nx}} + \frac{1}{\phi\ P_{n0}}\right)} = 5575.12\ \textbf{kip}$$

$P_u < \phi\ P_{neq}$ Therefore OK in Strength 3

Service 1

$P_u := 2251.9\ \textbf{kip}$

$M_{uy} := 343.19\ \textbf{kip} \cdot \textbf{ft}$

$M_{ux} := 38857.9\ \textbf{kip} \cdot \textbf{ft}$

From the diagram

$\phi\, P_{n0} := P_{rA} = 20359.872$ **kip**

$\phi\, P_{nx} := 12500$ **kip** $\phi\, P_{ny} := 20000$ **kip**

$$\phi\, P_{neq} := \frac{1}{\left(\frac{1}{\phi\, P_{ny}} + \frac{1}{\phi\, P_{nx}} + \frac{1}{\phi\, P_{n0}}\right)} = 5582.967 \textbf{ kip}$$

$P_u < \phi\, P_{neq} = 1$ Therefore OK in Service 1

E. Girder Design

Define Variables:

Properties of the W36x853 girder

$I_x := 70000\ \mathbf{in}^4$ $A := 251\ \mathbf{in}^2$ $d := 43.1\ \mathbf{in}$ $I_y := 4600\ \mathbf{in}^4$

$t_f := 4.53\ \mathbf{in}$ $S_x := 3250\ \mathbf{in}^3$ $J := 1240\ \mathbf{in}^4$ $C_w := 1710000\ \mathbf{in}^6$

$b_f := 18.2\ \mathbf{in}$ $t_w := 2.52\ \mathbf{in}$ $\lambda_f := 2.01$ $\lambda_w := 12.9$

$E_s := 29000\ \mathbf{ksi}$ $G := 11200\ \mathbf{ksi}$

$F_{yt} := 50\ \mathbf{ksi}$ $F_{yc} := 50\ \mathbf{ksi}$ $F_{yw} := 50\ \mathbf{ksi}$

Concrete properties:

$E_c := 3600\ \mathbf{ksi}$ $t_s := 8\ \mathbf{in}$

Strength I factored values:

$V_u := 226.89\ \mathbf{kip}$

$M_{up} := 8267.47\ \mathbf{kip} \cdot \mathbf{ft}$

$M_{un} := 6693.4\ \mathbf{kip} \cdot \mathbf{ft}$

$P_u := 1582.95\ \mathbf{kip}$

Shear Resistance

Recall:

$\lambda_w = 12.9$ $t_w = 2.52$ **in** $F_{yw} = 50$ **ksi** $E_s = 29000$ **ksi**

$d = 43.1$ **in** $V_u = 226.89$ **kip** $\lambda_w = 12.9$

Following AASHTO LRFD Bridge Specifications 6.10.3.3

$D := d - 2 \cdot t_f$

$k := 5$ Assuming there are no stiffeners

$$\lambda_{pw} := 1.12 \cdot \sqrt{E_s \cdot \frac{k}{F_{yw}}} = 60.314$$

$$\lambda_{rw} := 1.40 \cdot \sqrt{E_s \cdot \frac{k}{F_{yw}}} = 75.392$$

$\lambda_w < \lambda_{pw}$ therefore $C := 1$

$$\frac{2 \cdot D \cdot t_w}{b_f \cdot t_f \cdot 2} \le 2.5$$

$V_p := 0.58 \cdot F_{yw} \cdot D \cdot t_w = 2487.64$ **kip**

$V_N := C \cdot V_p = 2487.64$ **kip**

$\phi_v := 1$

$V_u \le \phi_v \cdot V_N$ No stiffeners are necessary

$$OCR := \frac{V_u}{V_N} = 0.091$$

Combined Axial and Flexural Resistance

a) Compressive Resistance 6.9.2.1

Recall:

$C_w = 1710000\ \mathbf{in}^6$ $G = 11200\ \mathbf{ksi}$ $I_x = 70000\ \mathbf{in}^4$ $I_y = 4600\ \mathbf{in}^4$

$J = 1240\ \mathbf{in}^4$ $F_y := 50\ \mathbf{ksi}$

Using elastic torsional buckling and flexural torsional buckling resistance

$\phi_c := 0.9$ 6.5.4.2

$Q := 1$ 6.9.4.1.1

$P_o := Q \cdot F_y \cdot A = 12550\ \mathbf{kip}$

$K_z l_z := 15\ \mathbf{ft}$ Assuming a torsional bracing so that this is the effective length for torsional buckling

$$P_e := \left(\frac{\pi^2 \cdot E_s \cdot C_w}{\left(K_z l_z\right)^2} + G \cdot J\right) \cdot \frac{A}{I_x + I_y} = 97553.464\ \mathbf{kip}$$

$$\frac{P_e}{P_o} = 7.773$$

$$P_n := \left(0.658^{\left(\frac{P_o}{P_e}\right)}\right) P_o = 11892.111\ \mathbf{kip}$$ 6.9.4.1.1-1

$P_r := \phi_c \cdot P_n = 10702.9\ \mathbf{kip}$

b) Flexural Strength 6.10

Recall:

$\lambda_f = 2.01$ $E_s = 29000$ **ksi** $F_{yt} = 50$ **ksi** $F_{yc} = 50$ **ksi**

$M_{up} = 8267.47$ **kip·ft** $M_{un} = 6693.4$ **kip·ft**

Check positive flexure

$\lambda_{pf} := 0.38 \cdot \sqrt{\frac{E_s}{F_{yt}}} = 9.152$ $\phi_f := 1$

6.10.3.2
Neglecting flange lateral bending stress

$f_{bu} := \frac{M_{up}}{S_x} = 30.526$ **ksi**

$F_{nc} := F_{yc} = 50$ **ksi**

$f_{bu} \le \phi_f \cdot F_{nc}$

Negative flexure

$F_{yt} = 50$ **ksi** $\lambda_{pf} := 0.38 \cdot \sqrt{\frac{E_s}{F_{yc}}} = 9.152$

$f_{bu} := \frac{M_{un}}{S_x} = 24.714$ **ksi**

$f_{bu} \le \phi_f \cdot F_{nc}$

Lateral Torsional Buckling 6.10.8.2.3

Recall:

$b_f = 18.2$ **in** $t_w = 2.52$ **in** $t_s = 0.667$ **ft** $D = 34.04$ **in**

$$r_t := b_f \cdot \left(12 \cdot \left(1 + \frac{1}{3} \frac{D \cdot t_w}{b_f \cdot t_f}\right)\right)^{-0.5} = 4.527 \text{ in}$$

$$L_p := 1 \cdot r_t \cdot \sqrt{\frac{E_s}{F_{yc}}} = 9.086 \text{ ft}$$

$F_{yr} := 0.7 \cdot F_{yc} = 35\ \mathbf{ksi}$

$$L_r := \pi \cdot r_t \cdot \sqrt{\frac{E_s}{F_{yr}}} = 34.116\ \mathbf{ft}$$

$L_b := 20\ \mathbf{ft}$ Assuming a bracing so this is true

$L_p < L_b \le L_r = 1$ therefore section is noncompact

$C_b := 1.0$

$$a_{wc} := \frac{2 \cdot D \cdot t_w}{b_f \cdot t_f} = 2.081$$

$$\lambda_{rw} := 0.56 \cdot \sqrt{\frac{E_s}{F_{yw}}} = 13.487$$

$$R_b := 1 - \frac{a_{wc}}{1200 + 300 \cdot a_{wc}} \cdot \left(2 \cdot \frac{D}{t_w} - \lambda_{rw}\right) = 0.985$$

$R_h := 1$ assuming hybrid factor is 1

$$F_{ncLTB} := C_b \cdot \left(1 - \left(1 - \frac{F_{yr}}{R_h \cdot F_{yc}}\right) \cdot \frac{\left(L_b - L_p\right)}{L_r - L_p}\right) \cdot R_b \cdot R_h \cdot F_{yc} = 42.789\ \mathbf{ksi}$$

Flange Local Buckling 6.10.8.2.2

$\lambda_f \le \lambda_{pf}$

$F_{ncFLB} := R_b \cdot R_h \cdot F_{yc} = 49.228\ \mathbf{ksi}$

$$F_{nc} := \begin{array}{l} \text{if } F_{ncLTB} < F_{ncFLB} \\ \quad \| F_{ncLTB} \\ \text{else} \\ \quad \| F_{ncFLB} \end{array} = 42.789\ \mathbf{ksi}$$

Bottom Flange: Discretely Braced in Compression

$$f_{bu} := \frac{M_{up}}{S_x} = 30.526\ \mathbf{ksi}$$

$\phi_f := 1$

Discretely brace in compression

$$f_{bu} \leq \phi_f \cdot F_{nc} = 1$$

$$DCR := \frac{f_{bu}}{F_{nc}} = 0.713$$

Continuously braced in tension

$$f_{bu} \leq \phi_f \cdot F_{yc} = 1$$

$$DCR := \frac{f_{bu}}{F_{yc}} = 0.611$$

$$M_{rx} := \phi_f \cdot F_{nc} \cdot S_x = 11588.609\ \mathbf{kip \cdot ft}$$

$$M_{ux} := M_{up} = 8267.47\ \mathbf{kip \cdot ft}$$

Design equation from 6.9.2.2

$$\frac{P_u}{P_r} = 0.148 \qquad \frac{P_u}{P_r} < 0.2 \qquad \text{therefore:}$$

$$\frac{P_u}{2 \cdot P_r} + \left(\frac{M_{ux}}{M_{rx}}\right) = 0.787 \qquad \frac{P_u}{2 \cdot P_r} + \frac{M_{ux}}{M_{rx}} \leq 1$$

Vibration Check:

$$\Delta_{DL} := 0.57\ \mathbf{in}$$

$$g = 32.174\ \frac{\mathbf{ft}}{\mathbf{s}^2}$$

$$f := 0.18 \cdot \sqrt{\frac{g}{\Delta_{DL}}} = 4.685\ \frac{1}{\mathbf{s}} \qquad f > 3$$

Therefore, bridge is OK for vibrations

Wind Loading:

Wind creates moment in the y-direction

$\phi_f = 1$

$\lambda_f = 2.01$ $\lambda_{pf} := 0.38 \cdot \sqrt{\frac{E_s}{F_y}} = 9.152$ $S_y := 505\ \mathbf{in}^3$

$M_p := F_y \cdot S_y = 2104.167\ \mathbf{kip \cdot ft}$

$M_{ny} := M_p = 2104.167\ \mathbf{kip \cdot ft}$ $M_{ry} := \phi_f \cdot M_{ny} = 2104.167\ \mathbf{kip \cdot ft}$

In Strength 3

$M_{ux} := 5520\ \mathbf{kip \cdot ft}$

$M_{uy} := 37.5\ \mathbf{kip \cdot ft}$

$V_u := 151\ \mathbf{kip}$

$\frac{P_u}{P_r} = 0.148$ $\frac{P_u}{P_r} < 0.2$ therefore:

$\frac{P_u}{2 \cdot P_r} + \left(\frac{M_{ux}}{M_{rx}} + \frac{M_{uy}}{M_{ry}}\right) = 0.568$ $\frac{P_u}{2 \cdot P_r} + \frac{M_{ux}}{M_{rx}} \le 1$

In Service 1

$M_{ux} := 5986\ \mathbf{kip \cdot ft}$

$M_{uy} := 8.043\ \mathbf{kip \cdot ft}$

$V_u := 164.27\ \mathbf{kip}$

$\frac{P_u}{P_r} = 0.148$ $\frac{P_u}{P_r} < 0.2$ therefore:

$$\frac{P_u}{2 \cdot P_r} + \left(\frac{M_{ux}}{M_{rx}} + \frac{M_{uy}}{M_{ry}}\right) = 0.594 \qquad \frac{P_u}{2 \cdot P_r} + \frac{M_{ux}}{M_{rx}} \le 1$$

Design Summary:

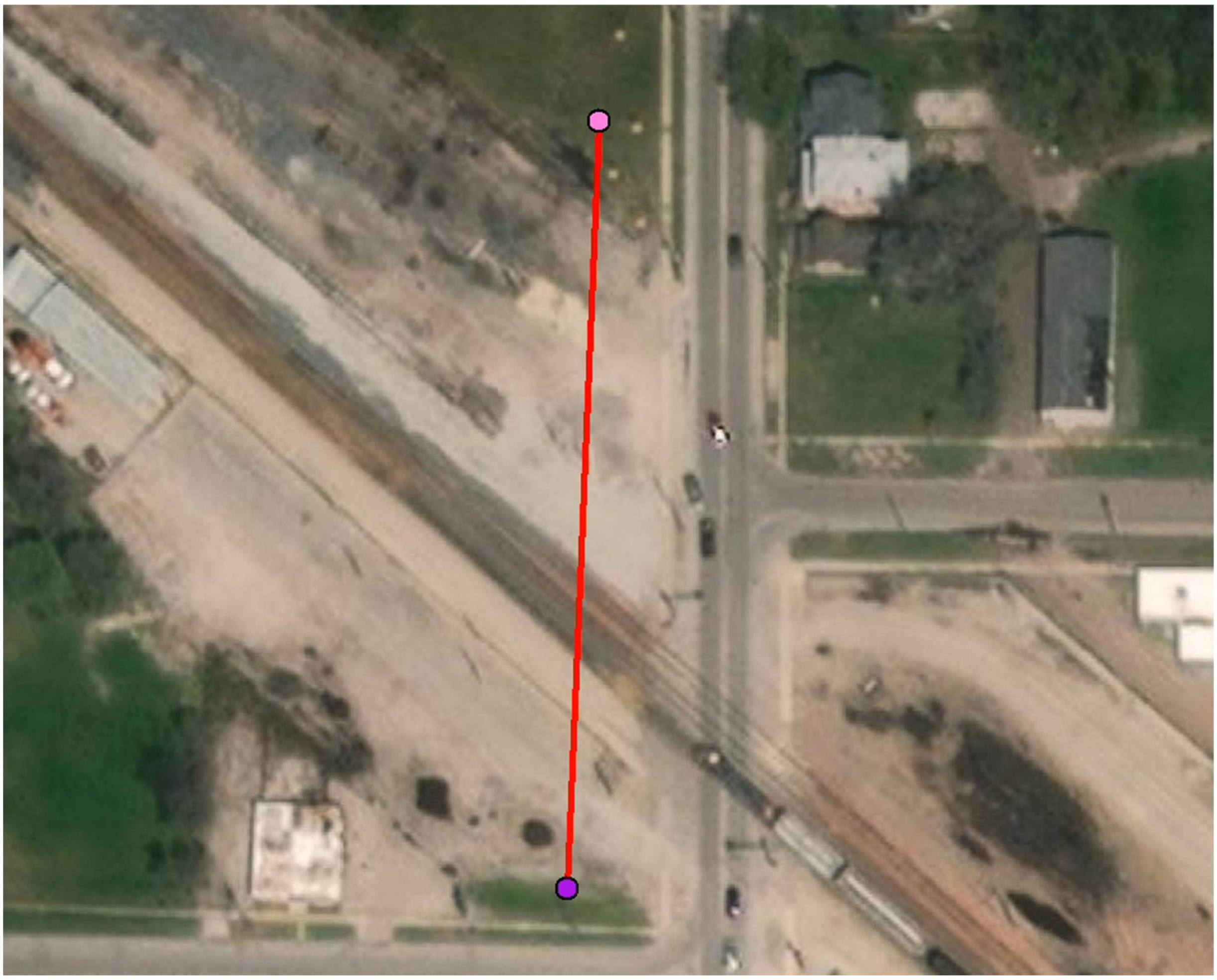
Use a W36x853 girder

$L_b = 20$ **ft** $\qquad K_z l_z = 15$ **ft**

No shear stiffeners are required

F. Span Design

Designed Span:



Span Alternative:

G. Slab Design

Concrete Properties:

$t_s := 8\ \mathbf{in}$ $w_c := 145\ \mathbf{pcf}$ $\beta_1 := 0.85$ $f'_c := 4000\ \mathbf{psi}$

Girder Properties:

$f_y := 60\ \mathbf{ksi}$ $b_f := 18.2\ \mathbf{in}$

considering the middle strip of concrete, in between the two W sections

$L_h := 10\ \mathbf{ft}$ $L_l := 35\ \mathbf{ft}$

clear span: $l_n := L_h - b_f = 8.483\ \mathbf{ft}$

$\frac{35}{12} = 2.917$ this is greater than 2, consider one-way slab design

The main tension reinforcement will be #4 bars, with 3/4 in clear cover. Use #4 for S&T

$c_c := 0.75\ \mathbf{in}$ $d_b := 0.5\ \mathbf{in}$

$d := t_s - c_c - \frac{d_b}{2} = 7\ \mathbf{in}$

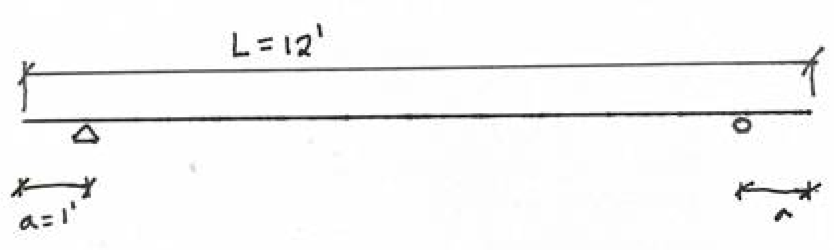
From Main Span Applied Loadings Appendix

$q_{LL} := 90\ \mathbf{psf}$ $q_{slab} := 96.7\ \mathbf{psf}$ $q_{ds} := 1.7\ \mathbf{psf}$

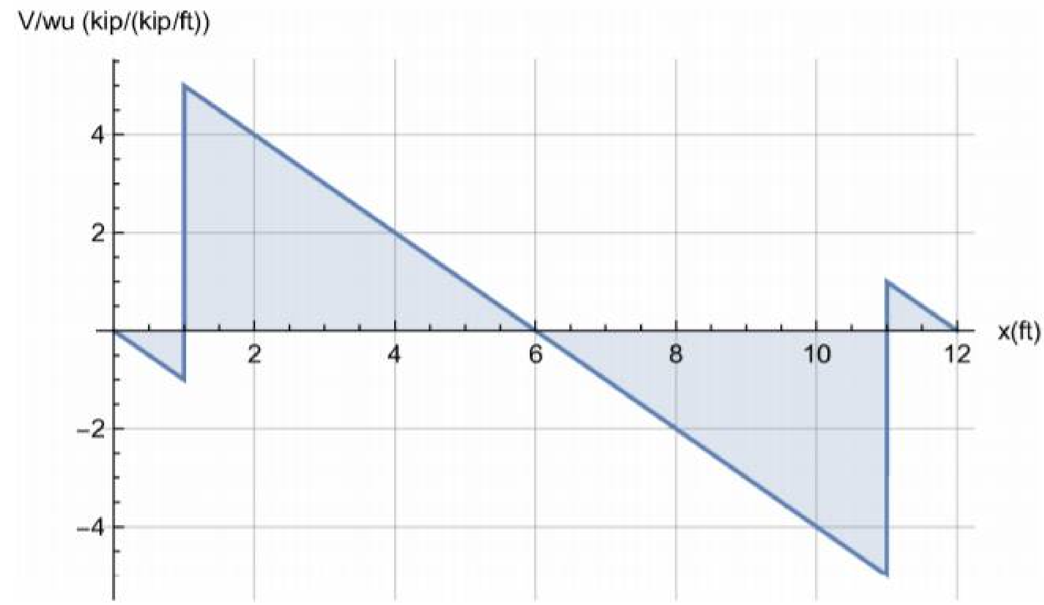
Consider a 12" strip in the direction of one-way loading in strength 1

$w_u := 1\ \mathbf{ft} \cdot \left(1.25 \cdot \left(q_{slab} + q_{ds}\right) + 1.75 \cdot q_{LL}\right) = 0.281\ \mathbf{klf}$

consider the slab to be a 12' long simply supported beam with 1' overhangs on either end

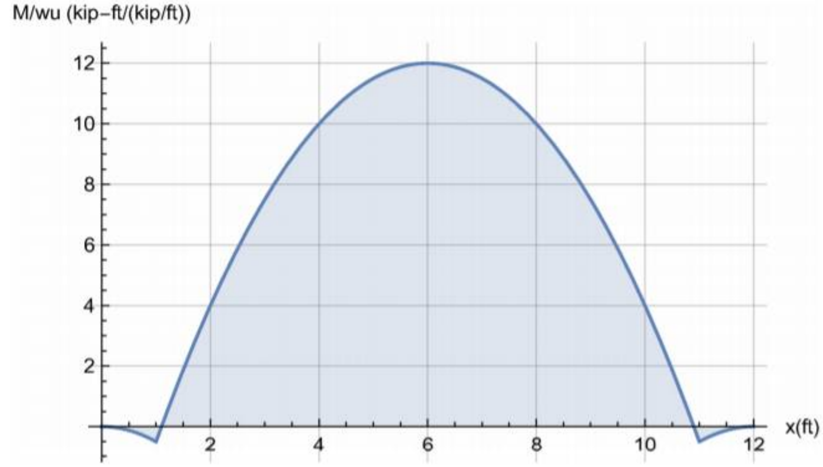


```
In[343]:= L = 12;
          a = 1;

In[345]:= Clear[wu, R1, R2];

In[346]:= ΣFy = R1 + R2 - wu * L;
          ΣMa = -wu * (L - a) ^2 / 2 + R2 * (L - 2 * a) + wu * a^2 / 2;
          s = Solve[{ΣFy == 0, ΣMa == 0}, {R1, R2}]

Out[348]= {{R1 → 6 wu, R2 → 6 wu}}

In[349]:= {R1} = s[[All, 1, 2]]

Out[349]= {6 wu}

In[350]:= {R2} = s[[All, 2, 2]]

Out[350]= {6 wu}

In[351]:= V1 = -wu * x;
          V2 = -wu * x + R1;
          V3 = -wu * x + R1 + R2;

In[354]:= wu = 1;

In[355]:= Plot[Piecewise[{{V1, x < a}, {V2, a < x < L - a}, {V3, L - a < x < L}}],
           {x, 0, L}, AxesLabel → {"x(ft)", "V/wu (kip/(kip/ft))"},
           Filling → Axis, Exclusions → None, GridLines → Automatic]
```




```
M1 = -wu * x^2 / 2;
M2 = -wu * x^2 / 2 + R1 * (x - a);
M3 = -wu * x^2 / 2 + R1 * (x - a) + R2 (x - (L - a));
```

```
Plot[Piecewise[{{M1, x < a}, {M2, a < x < L - a}, {M3, L - a < x < L}}],
 {x, 0, L}, AxesLabel → {"x(ft)", "M/wu (kip-ft/(kip/ft))"},
 Filling → Axis, Exclusions → None, GridLines → Automatic]
```



$M_{ubot} := 12\ \text{kip} \cdot \frac{\text{ft}}{\text{klf}} \cdot w_u = 3.366\ \text{kip} \cdot \text{ft}$

$M_{utop} := 0.5\ \text{kip} \cdot \frac{\text{ft}}{\text{klf}} \cdot w_u = 0.14\ \text{kip} \cdot \text{ft}$

$V_u := 5\ \frac{\text{kip}}{\text{klf}} \cdot w_u = 1.403\ \text{kip}$

Shear check

$\lambda := 1$ $\quad N_u := 0$

$b := 12\ \text{in}$

$\phi\ V_c := 0.75 \cdot 2 \cdot \lambda \cdot \sqrt{\frac{f'_c}{\text{psi}}}\ \text{psi} \cdot b \cdot d = 7.969\ \text{kip}$

shear resistance is greater than maximum shear, no need for any shear reinforcement

Shrinkage and Temperature steel

$A_b := 0.2\ \text{in}^2$ $\quad A_{stmin} := 0.0018 \cdot b \cdot t_s = 0.173\ \text{in}^2$

$s := 12\ \text{in} \cdot \frac{A_b}{A_{stmin}} = 13.889\ \text{in}$

$s_{max} := \min(5 \cdot t_s, 18\ in) = 18\ in$

Provide #4 bars at 12 in on center for S&T

Flexural reinforcement

$$s_{max} := \min\left(3 \cdot t_s, 18\ in, 15 \cdot \frac{(40000\ psi)}{\left(\frac{2}{3} f_y\right)}\ in - 2.5 \cdot c_c, 12\ in \cdot \frac{(40000\ psi)}{\left(\frac{2}{3} f_y\right)}\right) = 12\ in$$

$b = 12\ in$

$$A_{stensioncontrolled} := \frac{0.85 \cdot f'_c \cdot b \cdot \beta_1}{f_y} \cdot 3 \cdot \frac{d}{8} = 1.517\ in^2$$

$\phi := 0.9$

Top flexural reinforcement

$$\frac{\phi \cdot f_y^{\ 2}}{1.7 \cdot f'_c \cdot b} A_s^{\ 2} - \phi \cdot f_y \cdot d \cdot A_s + M_{utop} = 0$$

Guess Values

$A_s := 0.1\ in^2$

Constraints

$$f(A_s) := \frac{\phi \cdot f_y^{\ 2}}{1.7 \cdot f'_c \cdot b} A_s^{\ 2} - \phi \cdot f_y \cdot d \cdot A_s + M_{utop}$$

Solver

$\mathrm{root}(f(A_s), A_s) = 0.004\ in^2$

$A_s := 0.004\ in^2$

$s := 12\ \mathbf{in} \cdot \frac{A_b}{A_s} = 50\ \mathbf{ft}$ Provide #4 bars at 12" spacing

Bottom flexural reinforcement

Guess Values

$A_s := 0.1\ in^2$

Constraints

$f(A_s) := \frac{\phi \cdot f_y^2}{1.7 \cdot f'_c \cdot b} A_s^2 - \phi \cdot f_y \cdot d \cdot A_s + M_{ubot}$

Solver

$\mathrm{root}(f(A_s), A_s) = 0.108\ in^2$

$A_s := 0.108\ \mathbf{in}^2$

$s := 12\ \mathbf{in} \cdot \frac{A_b}{A_s} = 22.222\ \mathbf{in}$ Provide #4 bars at 12" spacing

Deflection check

$10\ \mathbf{in} \cdot \frac{12}{28} = 4.286\ \mathbf{in}$ slab thickness is greater than 4.3", deflection check is not needed.

Exterior slab check

$M_{utop} := 0.5\ \mathbf{kip} \cdot \frac{\mathbf{ft}}{\mathbf{klf}} \cdot w_u = 0.14\ \mathbf{kip} \cdot \mathbf{ft}$

$V_u := 1\ \frac{\mathbf{kip}}{\mathbf{klf}} \cdot w_u = 0.281\ \mathbf{kip}$

Shrinkage and temperature
would remain the same

Top flexural reinforcement

$$\frac{\phi \cdot f_y^2}{1.7 \cdot f'_c \cdot b} A_s^2 - \phi \cdot f_y \cdot d \cdot A_s + M_{utop} = 0$$

Guess Values

$$A_s := 0.1\ in^2$$

Constraints

$$f(A_s) := \frac{\phi \cdot f_y^2}{1.7 \cdot f'_c \cdot b} A_s^2 - \phi \cdot f_y \cdot d \cdot A_s + M_{utop}$$

Solver

$$\mathbf{root}(f(A_s), A_s) = 0.004\ in^2$$

$$A_s := 0.004\ \mathbf{in}^2$$

$$s := 12\ \mathbf{in} \cdot \frac{A_b}{A_s} = 50\ \mathbf{ft}$$

Provide #4 bars at 12" spacing

H. Span Deflection

Unfactored Deflection Due to Dead and Live Load (in)

Beam	x (ft) Beginning	End	Maximum deflection per span DC	DCT	LL-Ped
1	0	35	0.249	0.004	0.089
2	35	70	0.016	-0.001	0.006
3	70	105	-0.184	-0.001	-0.007
4	105	140	0.011	-0.001	0.004
5	140	175	0.044	0	0.016
6	175	210	0.070	0	0.025
7	210	245	0.078	0	0.028
8	245	280	0.052	0	0.018
9	280	315	-0.025	0	-0.009
10	315	350	-0.150	-0.001	-0.054
11	350	385	-0.313	-0.001	-0.112
12	385	420	-0.476	-0.001	-0.170
13	420	455	-0.575	-0.002	-0.205
14	455	490	-0.528	-0.001	-0.188
15	490	525	-0.245	-0.001	-0.087

Total deflection due to dead and live load

Beam	Total deflection (in)
1	0.341
2	0.021
3	-0.192
4	0.015
5	0.060
6	0.096
7	0.106
8	0.070
9	-0.034
10	-0.205
11	-0.426
12	-0.647
13	-0.782
14	-0.717
15	-0.333

Maximum deflection occurs in beam 13

$$\Delta := 0.782\ \mathbf{in}$$

$$\Delta_{max} := \frac{(525 \cdot 12\ \mathbf{in})}{360} = 17.5\ \mathbf{in}$$

$$\Delta < \Delta_{max}$$

Therefore beam meets deflection requirements.

I: Spiral Ramp Design

Legend:
requires up-to-date input
checks to verify
results

Define constants:

$\lambda := 1$

$f_c' := 5000 \ \frac{lb}{in^2}$

$f_y := 60000 \ \frac{lb}{in^2}$

$\beta_1 := 0.85$

$A_4 := 0.2 \ in^2$

$d_4 := 0.5 \ in$

Set overall dimensions:

$H := 27.5 \ ft$ top of deck from top of rail

$s := \frac{1}{20} = 0.05$ $17 \ ft \cdot 0.05 = 10.2 \ in$

$W := 12 \ ft$

$h := 8 \ ft$

$x := \frac{H}{s} = 550 \ ft$

$n_{spiral} := \frac{H}{h} = 3.438$

$D_{spiral} := \frac{x}{n_{spiral} \cdot \pi} + W = 62.93 \ ft$ uses centerline of path to fulfill slope requirement

$\theta := 34 \cdot \frac{\pi}{180} \ rad = 0.593$

Strength I

$$l_n := \theta \cdot \frac{D_{spiral}}{2} = 18.672\ ft$$

$$L := \text{if } l_n > 60\ ft \;\Big\|\; 60\ ft \;\; \text{else} \;\Big\|\; l_n = 18.672\ ft$$

Design slab:

$$\frac{L}{20} = 11.203\ in$$

$$t_s := 11.25\ in$$

$$d_t := t_s - 0.75\ in - \frac{d_4}{2} = 10.25\ in$$

$$E := 10\ in + 5 \cdot \sqrt[2]{L \cdot \frac{W}{ft^2}} \cdot in = 84.843\ in$$

$$w_s := 145\ \frac{lb}{ft^3} \cdot E \cdot t_s = 961.112\ \frac{lb}{ft}$$

$$w_{sdl} := (4 + 4 + 2.08 + 2.3) \cdot \frac{lb}{ft^2} \cdot W = 148.56\ \frac{lb}{ft}$$

$$w_u := 1.25 \cdot (w_s + w_{sdl}) + 1.75 \cdot 90 \cdot \frac{lb}{ft^2} \cdot W = (3.277 \cdot 10^3)\ \frac{lb}{ft}$$

$$M_{pos\_end} := \frac{w_u \cdot l_n^2}{14} = (8.161 \cdot 10^4)\ lb \cdot ft$$

$$M_{neg\_int1} := \frac{w_u \cdot l_n^2}{10} = (1.142 \cdot 10^5)\ lb \cdot ft$$

$$M_u := \text{if } M_{pos\_end} < M_{neg\_int1} \; \| \; M_{neg\_int1} \; \text{else} \; \| \; M_{pos\_end} = (1.142 \cdot 10^5)\ lb \cdot ft$$

$$V_u := 1.15 \cdot w_u \cdot \frac{l_n}{2} = (3.518 \cdot 10^4)\ lb$$

-moment and shear equations above from ACI 318 Table 6.5.2 and 6.5.4

$$\phi V_{c\_max} := 0.75 \cdot 2 \cdot \lambda \cdot \sqrt[2]{\frac{f_c'}{\frac{lb}{in^2}}} \cdot \frac{E}{in} \cdot \frac{d_t}{in}\ lb = (9.224 \cdot 10^4)\ lb$$

$$checkV := \text{if } V_u < \phi V_{c\_max} \;\Big\|\; \text{"ok"} \;\text{ else }\; \Big\| \;\text{"not ok"} \quad = \text{"ok"}$$

Vu < ϕV_c

$$A_g := E \cdot t_s = 954.484 \; in^2$$

$$s := \frac{A_4}{0.0018 \cdot t_s} = 9.877 \; in$$

$$s_{max} := \text{if } 5 \cdot t_s < 18 \; in \;\Big\|\; 5 \; t_s \;\text{ else }\; \Big\|\; 18 \; in \quad = 18 \; in$$

$$s_{ST} := \text{if } s < 18 \; in \;\Big\|\; s \;\text{ else }\; \Big\|\; 18 \; in \quad = 9.877 \; in$$

for S&T, provide #4 bars at 9 7/8 in o.c.

$$A_{s\_T} := 0.85 \cdot f_c{}' \cdot E \cdot \frac{\beta_1}{f_y} \cdot \left(\frac{3 \cdot d_t}{8}\right) = 19.635 \; in^2$$

$$Q_1 := 0.9 \cdot \frac{\left(\frac{f_y}{\frac{lb}{in^2}}\right)^2}{1.7 \cdot \left(\frac{f_c{}'}{\frac{lb}{in^2}}\right) \cdot \left(\frac{E}{in}\right)} = 4.493 \cdot 10^3$$

$A_s := 1$

$$Q_1 \cdot A_s^{\,2} - 0.9 \cdot \left(\frac{f_y}{\frac{lb}{in^2}}\right) \cdot \left(\frac{d_t}{in}\right) \cdot A_s + \left(\frac{M_u}{lb \cdot in}\right) = 0$$

$\text{find}\left(A_s\right) = 2.529$

$A_s := 2.529\ in^2$

$$checkA_s := \begin{array}{l} \text{if } A_s < A_{s\_T} \\ \quad \| \text{"ok"} \\ \text{else} \\ \quad \| \text{"not ok"} \end{array} = \text{"ok"}$$

As < A_{s_T}

$$s_{max} := \begin{array}{l} \text{if } 3 \cdot t_s < 18\ in \\ \quad \| 3 \cdot t_s \\ \text{else} \\ \quad \| 18\ in \end{array} = 18\ in$$

$$s_{max} := \text{if } s_{max} < \left(\frac{15 \cdot 40000 \cdot 3}{2 \cdot \left(\frac{f_y}{\frac{lb}{in^2}} \right)} - 2.5 \cdot 0.75 \right) \cdot in = 13.125\ in$$

$$\quad \| \ s_{max}$$

$$\quad \text{else}$$

$$\quad \| \left(\frac{15 \cdot 40000 \cdot 3}{2 \cdot \left(\frac{f_y}{\frac{lb}{in^2}} \right)} - 2.5 \cdot 0.75 \right) \cdot in$$

$$s_{max} := \text{if } s_{max} < \frac{12 \cdot 40000 \cdot 3}{2 \cdot \left(\frac{f_y}{\frac{lb}{in^2}} \right)} \cdot in = 12\ in$$

$$\quad \| \ s_{max}$$

$$\quad \text{else}$$

$$\quad \| \frac{12 \cdot 40000 \cdot 3}{2 \cdot \left(\frac{f_y}{\frac{lb}{in^2}} \right)} \cdot in$$

$$s := E \cdot \frac{A_4}{A_s} = 6.71\ in$$

$$s_s := \text{if } s < s_{max} = 6.71\ in$$

$$\quad \| \ s$$

$$\quad \text{else}$$

$$\quad \| \ s_{max}$$

for flexural, provide #4 at 6 5/8 in o.c.

Design beams:

$t_b := l_n$

$w_d := \frac{w_u}{W} = 273.091 \; \frac{lb}{ft^2}$

$b := 18 \; in$

$h_b := 20 \; in$

$w_b := 145 \; \frac{lb}{ft^3} \cdot b \cdot h_b = 362.5 \; \frac{lb}{ft}$

$D := w_b + w_d \cdot t_b = \left(5.462 \cdot 10^3\right) \; \frac{lb}{ft}$

$w_u := 1.2 \cdot D + 1.6 \cdot 90 \; \frac{lb}{ft^2} \cdot t_b = \left(9.243 \cdot 10^3\right) \; \frac{lb}{ft}$

$$M_u := w_u \cdot W \cdot \frac{\left(D_{spiral} - W\right)}{2} + 1.2 \cdot w_b \cdot \frac{\left(\frac{D_{spiral}}{2} - W\right)^2}{2} = \left(2.907 \cdot 10^6\right) \; lb \cdot ft$$

$$V_u := w_u \cdot W + 1.2 \; w_b \cdot \left(\frac{D_{spiral}}{2} - W\right) = \left(1.194 \cdot 10^5\right) \; lb$$

$d_6 := 0.75 \; in$

$d_7 := 0.875 \; in$

$d := h_b - 2 \; in - \frac{d_7}{2} = 17.563 \; in$

$$A_{s\_T} := 0.85 \cdot f_c' \cdot b \cdot \frac{\beta_1}{f_y} \cdot \left(\frac{3 \cdot d}{8}\right) = 7.138 \; in^2$$

$A_s := 1$

$$\frac{0.9 \cdot \left(\frac{f_y}{\frac{lb}{in^2}}\right)^2}{1.7 \cdot \left(\frac{f_c'}{\frac{lb}{in^2}}\right) \cdot \left(\frac{b}{in}\right)} \cdot A_s^{\,2} - 0.9 \cdot \left(\frac{f_y}{\frac{lb}{in^2}}\right) \cdot \left(\frac{d}{in}\right) \cdot A_s + \left(\frac{M_u}{lb \cdot ft}\right) = 0$$

$\text{find}\left(A_s\right) = 3.31$

$A_s := 3.31\ in^2$

$$A_{s\_min} := \text{if } \frac{200}{\frac{f_y}{\frac{lb}{in^2}}} > 3 \cdot \frac{\sqrt[2]{\frac{f_c'}{\frac{lb}{in^2}}}}{\left(\frac{f_y}{\frac{lb}{in^2}}\right)} = 1.273\ in^2$$

$$\quad \left\| \; 200 \cdot \frac{f_y}{\frac{lb}{in^2}} \cdot b \cdot h_b \right.$$

else

$$\quad \left\| \; 3 \cdot \frac{\sqrt[2]{\frac{f_c'}{\frac{lb}{in^2}}}}{\left(\frac{f_y}{\frac{lb}{in^2}}\right)} \cdot b \cdot h_b \right.$$

$checkA_s := \text{if } \left(A_s < A_{s\_T}\right) \wedge A_s > A_{s\_min} = \text{“ok”}$

‖ “ok”

else

‖ “not ok”

$A_6 := 0.44\ in^2$

$A_7 := 0.6\ in^2$

$n_{b\_bar} := \frac{A_s}{A_7} = 5.517$ $n_{b\_bar} := 6$ round up to be conservative

$s := \frac{l_n}{10 \cdot n_{b\_bar}} = 3.734\ in$ 6 #7 bar spaced 3.625 in centered over beam

$\phi := 0.75$ $\frac{\left(\frac{l_n}{10} - 18\ in\right)}{2} = 2.203\ in$

$V_c := 2 \cdot \sqrt{\frac{f_c'}{\frac{lb}{in^2}}} \cdot \frac{b}{in} \cdot \frac{d}{in}\ lb = \left(4.471 \cdot 10^4\right)\ lb$

$checkV_s :=$ if $0.5 \cdot \phi \cdot V_c > V_u$
  "do not need stirrups"
else
  "need stirrups"
= "need stirrups"

need stirrups - continue

$\phi V_{c\_max} := 0.75 \cdot \left(V_c + 8 \cdot \sqrt{\frac{f_c'}{\frac{lb}{in^2}}} \cdot \frac{b}{in} \cdot \frac{d}{in} \cdot lb\right) = \left(1.677 \cdot 10^5\right)\ lb$

$checkV_c :=$ if $\phi V_{c\_max} > V_u$
  "ok"
else
  "not ok"
= "ok"

not ok - revise section dimensions

$V_{s\_req} := \frac{V_u - \phi \cdot V_c}{\phi} = \left(1.145 \cdot 10^5\right)\ lb$ $\phi \cdot V_c = \left(3.353 \cdot 10^4\right)\ lb$

$A_3 := 0.11\ in^2$

$A_v := 2 \cdot A_3 = 0.22\ in^2$

$4 \cdot \sqrt{\frac{f_c'}{\frac{lb}{in^2}}} \cdot \frac{b}{in} \cdot \frac{d}{in} \cdot lb = \left(8.941 \cdot 10^4\right)\ lb$ $V_u = \left(1.194 \cdot 10^5\right)\ lb$

$$checkV := \text{if } 4 \cdot \sqrt[2]{\frac{f_c'}{\frac{lb}{in^2}}} \cdot \frac{b}{in} \cdot \frac{d}{in} \cdot lb > V_u \;\; \Big\| \text{"use d/2 and 24 in below"} \;\; \text{else} \;\; \Big\| \text{"use d/4 and 12 in below"} = \text{"use d/4 and 12 in below"}$$

Smax = min*:

$$\frac{A_v \cdot f_y}{0.75 \cdot \sqrt[2]{\frac{f_c'}{\frac{lb}{in^2}}} \cdot \frac{b}{in} \cdot lb} \cdot in = 13.828\ in \qquad \frac{A_v \cdot f_y}{50 \cdot \frac{b}{in} \cdot lb} \cdot in = 14.667\ in \qquad \frac{d}{4} = 4.391\ in \qquad 12\ \text{in}$$

*but not less than 3 in

$s_{max} := \frac{d}{4} = 4.391\ in$ $\qquad s_{max} := 4.375\ in$ round down to be conservative

$$V_{s\_max} := A_v \cdot f_y \cdot \frac{d}{s_{max}} = \left(5.299 \cdot 10^4\right)\ lb$$

$$checkV_{s\_max} := \text{if } V_{s\_max} > V_{s\_req} \;\; \Big\| \text{"ok"} \;\; \text{else} \;\; \Big\| \text{"add high shear zone"} = \text{"add high shear zone"}$$

if not ok, continue

$s := \frac{A_v \cdot f_y \cdot d}{\frac{V_u}{\phi} - V_c} = 2.025\ in$ $\qquad s := 2\ in$

$$V_h := \phi \cdot \left(V_c + V_{s\_max}\right) = \left(7.327 \cdot 10^4\right)\ lb$$

$$x_1 := \frac{D_{spiral}}{2} - W = 19.465\ ft$$

$$V_u - 1.2 \cdot w_b \cdot x_1 = \left(1.109 \cdot 10^5\right)\ lb$$

$$x_v := \frac{D_{spiral}}{2} = 31.465\ ft$$

$$V_u - \left(\frac{D_{spiral}}{2} - W\right) \cdot 1.2 \cdot w_b - w_u \cdot \left(x_v - \left(\frac{D_{spiral}}{2} - W\right)\right) = 0 \ lb$$

$$x_h := \frac{D_{spiral}}{2} - W + \frac{V_u - V_h - \left(\frac{D_{spiral}}{2} - W\right) \cdot 1.2 \cdot w_b}{w_u} = 23.537 \ ft$$

$$V_i := 0.5 \cdot \phi \cdot V_c = \left(1.677 \cdot 10^4\right) \ lb$$

$$x_i := \frac{V_u - V_i - \left(\frac{D_{spiral}}{2} - W\right) \cdot 1.2 \cdot w_b}{w_u} + \frac{D_{spiral}}{2} - W = 29.651 \ ft$$

$n_h := \frac{x_h}{s} = 141.223$ $n_h := 142$ round up to be conservative

$n_i := \frac{x_i - (n_h + 1) \cdot s}{s_{max}} = 15.957$ $n_i := 16$

$$n_t := 1 + n_h + n_i = 159$$

$n_h \cdot s = 23.667 \ ft$ space #3 stirrups at 2" o.c. until 23'-8" from centerline of column

$$n_h \cdot s = 23.667 \ ft$$

$n_h \cdot s + n_i \cdot s_{max} = 29.5 \ ft$ space #3 stirrups at 4 3/8" o.c. from end of high shear zone until 29'-6" from centerline of column

$$\frac{D_{spiral}}{2} - \left(n_h \cdot s + n_i \cdot s_{max}\right) = 23.577 \ in$$

Design column:

$A_f := x \cdot W = (6.6 \cdot 10^3)\ ft^2$

$n_P := \frac{x}{l_n} = 29.457$ $n_P := 30$

$P_u := V_u \cdot n_P = (3.581 \cdot 10^6)\ lb$

$A_{14} := 2.25\ in^2$

$A_{st} := A_{14} \cdot 6 = 13.5\ in^2$

$$A_g := \frac{\frac{P_u}{0.85 \cdot 0.75} - f_y \cdot A_{st}}{0.85 \cdot f_c'} + A_{st} = (1.145 \cdot 10^3)\ in^2$$

$d_c := \sqrt[2]{\frac{A_g \cdot 4}{\pi}} = 38.178\ in$ $d_c := 39\ in$

$P_u := P_u + (H - t_s) \cdot \frac{\pi}{4} \cdot d_c^{\ 2} \cdot 145\ \frac{lb}{ft^3} = (3.613 \cdot 10^6)\ lb$

$$A_g := \frac{\frac{P_u}{0.85 \cdot 0.75} - f_y \cdot A_{st}}{0.85 \cdot f_c'} + A_{st} = (1.157 \cdot 10^3)\ in^2$$

$d_c := \sqrt[2]{\frac{A_g \cdot 4}{\pi}} = 38.374\ in$ $d_c := 39\ in$

$A_g := \frac{\pi}{4} \cdot (d_c)^2 = (1.195 \cdot 10^3)\ in^2$

$\rho_g := \frac{A_{st}}{A_g} = 0.011$

$check\rho_g :=$ if $(\rho_g > 0.01) \land \rho_g < 0.08$ = "ok"
  "ok"
else
  "not ok"

6 #14 rebars meet ACI requirement for steel-to-concrete ratio

$$\phi P_n := 0.85 \cdot 0.75 \cdot \left(0.85 \cdot f_c{}' \cdot \left(A_g - A_{st}\right) + f_y \cdot A_{st}\right) = \left(3.716 \cdot 10^6\right)\ lb$$

$$c_c := 2\ in$$

$$h_c := d_c - 2 \cdot c_c = 35\ in \qquad \frac{\pi \cdot h_c}{6} = 18.326\ in$$

$$A_{ch} := \frac{\pi}{4} \cdot h_c{}^2 = 962.113\ in^2$$

$$\rho_{s\_min} := 0.45 \cdot \left(\frac{A_g}{A_{ch}} - 1\right) \frac{f_c{}'}{f_y} = 0.009$$

$$d_{sp} := \frac{1}{2}\ in$$

$$A_{sp} := \frac{\pi}{4} \cdot d_{sp}{}^2 = 0.196\ in^2$$

$$s := \frac{A_{sp} \cdot \pi \cdot \left(h_c - d_{sp}\right)}{\rho_{s\_min} \cdot A_{ch}} = 2.441\ in \qquad s := 2.375\ in$$

round down to be conservative

provide spiral cage with 1/2 inch smooth wire and a 2 3/8 inch pitch

Quantities:

$$L := n_{spiral} \cdot \pi \cdot D_{spiral} = 679.591\ ft$$

$$V := \frac{\left(W \cdot t_s \cdot L + b \cdot h_b \cdot \left(\frac{D_{spiral} - d_c}{2}\right) \cdot n_P + (H - t_s) \cdot \frac{\pi}{4} \cdot {d_c}^2\right)}{27\ ft^3} = 374.212$$

$$W_4 := 0.668\ \frac{lb}{ft} \cdot \pi \cdot D_{spiral} \cdot n_{spiral} \cdot W \cdot \left(\frac{1}{s_{ST}} + \frac{1}{s_s}\right) = \left(1.636 \cdot 10^4\right)\ lb$$

$$W_7 := 2.044\ \frac{lb}{ft} \cdot n_{b\_bar} \cdot n_P \cdot \left(\frac{D_{spiral} - d_c}{2}\right) = \left(1.098 \cdot 10^4\right)\ lb$$

$$W_3 := \left(0.376\ \frac{lb}{ft} \cdot (n_h + n_i) \cdot ((d - 2\ in) \cdot 2 + (b - 4\ in))\right) \cdot n_P = \left(6.702 \cdot 10^3\right)\ lb$$

$$W_{14} := 7.65\ \frac{lb}{ft} \cdot 6 \cdot (H - t_s) = \left(1.219 \cdot 10^3\right)\ lb$$

$$L_w := H - t_s = 26.563\ ft$$

$$\alpha := \operatorname{atan}\left(\frac{7}{11}\right) = 0.567$$

$$L_{rail} := n_{spiral} \cdot \pi \cdot (2 \cdot D_{spiral} - 2 \cdot W) + \frac{2 \cdot H}{\sin(\alpha)} + 4 \cdot \left(5\ ft + 8\ in + \frac{33}{64}\ in + 5\ ft + 1\ in + \frac{61}{64}\ in\right) = \left(1.246 \cdot 10^3\right)\ ft$$

J: Elevator Tower Design

Legend:

requires up-to-date input

checks to verify

results

Choose metal deck:

$E := 29000000 \frac{lb}{in^2}$ $F_y := 50000 \frac{lb}{in^2}$

$SDL := (0.8 + 1) \frac{lb}{ft^2} = 1.8 \frac{lb}{ft^2}$

$LL := 20 \frac{lb}{ft^2}$

$S := 25 \frac{lb}{ft^2}$

$w := SDL + LL + S = 46.8 \frac{lb}{ft^2}$

$w_d := 1.8 \frac{lb}{ft^2}$ self-weight of 3NL22 roof deck; chosen for capacity of single 8' span

$H := 27.5\ ft$

$OS := 12\ ft + 8\ in + 10\ in = 13.5\ ft$ required above upper level by evolution 100 elevator

$F_s := 7500\ lb = (7.5 \cdot 10^3)\ lb$ required safety bar capacity by evolution 100 elevator

$h := H + OS = 41\ ft$

$B := 7\ ft + 6\ in + 2 \cdot 5.5\ in = 8.417\ ft$

$W := 5\ ft + 9\ in + 2 \cdot 5.5\ in = 6.667\ ft$

Calculate wind load:

$h_g := \frac{h}{4} = 10.25\ ft$

$G := 0.85$

$K_{zt} := 1$

$z_{b1} := h_g = 10.25\ ft$

check all levels are included in Kz, q, Pp, Pp_net, Pn, Pn_net, W/R

$z_{b2} := 2 \cdot h_g = 20.5\ ft$

$z_{b3} := 3 \cdot h_g = 30.75\ ft$

$z_{roof} := h = 41\ ft$

$L := W = 6.667\ ft$

Risk: II - T 1.5-1

$V := 108\ mph$ - F 26.5-1D

$K_d := 0.85$

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

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

$K_e := 1$

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

$GC_{pi} :=$ if $enclosure = \text{"enclosed"}$
  ‖ 0.18
else if $enclosure = \text{"partially enclosed"}$
  ‖ 0.55
else if $enclosure = \text{"partially open"}$
  ‖ 0.18
else
  ‖ 0

$GC_{pi} = 0.18$

$z_g := 1200\ ft$

$\alpha := 7$

$K_{z15} := 2.01 \cdot \left(\frac{15\ ft}{z_g}\right)^{\frac{2}{\alpha}} = 0.575$

compare to levels to determine need

$K_{zb2} := 2.01 \cdot \left(\frac{z_{b2}}{z_g}\right)^{\frac{2}{\alpha}} = 0.628$

$K_{zb3} := 2.01 \cdot \left(\frac{z_{b3}}{z_g}\right)^{\frac{2}{\alpha}} = 0.706$

$K_{zroof} := 2.01 \cdot \left(\frac{z_{roof}}{z_g}\right)^{\frac{2}{\alpha}} = 0.766$

$q_{15} := 0.00256\ \frac{psf}{mph^2} \cdot K_{z15} \cdot K_{zt} \cdot K_d \cdot K_e \cdot V^2 = 14.587\ psf$

$q_{b2} := 0.00256\ \frac{psf}{mph^2} \cdot K_{zb2} \cdot K_{zt} \cdot K_d \cdot K_e \cdot V^2 = 15.949\ psf$

$q_{b3} := 0.00256\ \frac{psf}{mph^2} \cdot K_{zb3} \cdot K_{zt} \cdot K_d \cdot K_e \cdot V^2 = 17.908\ psf$

$q_{roof} := 0.00256\ \frac{psf}{mph^2} \cdot K_{zroof} \cdot K_{zt} \cdot K_d \cdot K_e \cdot V^2 = 19.442\ psf$

$\frac{L}{B} = 0.792$

$C_{p\_w} := 0.8$

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

$p_{15\_w\_p} := q_{15} \cdot G \cdot C_{p\_w} - q_{15} \cdot GC_{pi} = 7.293\ psf$

$p_{b2\_w\_p} := q_{b2} \cdot G \cdot C_{p\_w} - q_{b2} \cdot GC_{pi} = 7.974\ psf$

$p_{b3\_w\_p} := q_{b3} \cdot G \cdot C_{p\_w} - q_{b3} \cdot GC_{pi} = 8.954\ psf$

$p_{roof\_w\_p} := q_{roof} \cdot G \cdot C_{p\_w} - q_{roof} \cdot GC_{pi} = 9.721 \ psf$

$p_{l\_p} := q_{roof} \cdot G \cdot C_{p\_l} - q_{roof} \cdot GC_{pi} = -11.762 \ psf$

$p_{15\_p\_net} := p_{15\_w\_p} - p_{l\_p} = 19.056 \ psf$

$p_{b2\_p\_net} := p_{b2\_w\_p} - p_{l\_p} = 19.736 \ psf$

$p_{b3\_p\_net} := p_{b3\_w\_p} - p_{l\_p} = 20.716 \ psf$

$p_{roof\_p\_net} := p_{roof\_w\_p} - p_{l\_p} = 21.483 \ psf$

$p_{15\_w\_n} := q_{15} \cdot G \cdot C_{p\_w} - q_{15} \cdot -GC_{pi} = 12.545 \ psf$

$p_{b2\_w\_n} := q_{b2} \cdot G \cdot C_{p\_w} - q_{b2} \cdot -GC_{pi} = 13.716 \ psf$

$p_{b3\_w\_n} := q_{b3} \cdot G \cdot C_{p\_w} - q_{b3} \cdot -GC_{pi} = 15.4 \ psf$

$p_{roof\_w\_n} := q_{roof} \cdot G \cdot C_{p\_w} - q_{roof} \cdot -GC_{pi} = 16.72 \ psf$

$p_{l\_n} := q_{roof} \cdot G \cdot C_{p\_l} - q_{roof} \cdot -GC_{pi} = -4.763 \ psf$

$p_{15\_n\_net} := p_{15\_w\_n} - p_{l\_n} = 17.308 \ psf$

$p_{b2\_n\_net} := p_{b2\_w\_n} - p_{l\_n} = 18.479 \ psf$

$p_{b3\_n\_net} := p_{b3\_w\_n} - p_{l\_n} = 20.164 \ psf$

$p_{roof\_n\_net} := p_{roof\_w\_n} - p_{l\_n} = 21.483 \ psf$

Design roof+wind beam:

$$t_w := \frac{W}{2} = 3.333\ ft$$

$$w_b := (w + w_d) \cdot t_w = 162\ \frac{lb}{ft}$$

$$R_{roof\_g} := \frac{w_b \cdot B + F_s}{2} = (4.432 \cdot 10^3)\ lb$$

$$W_{roof} := p_{roof\_p\_net} \cdot \frac{h_g}{2} \cdot \frac{L}{2 \cdot B} \cdot \frac{s^2}{m} = 427.609\ \frac{lb}{ft}$$

$$R_{b4} := W_{roof} \cdot \frac{B}{2} = (1.8 \cdot 10^3)\ lb$$

$$M_u := (w_b + W_{roof}) \cdot \frac{B^2}{8} + F_s \cdot \frac{B}{4} = (2.1 \cdot 10^4)\ lb \cdot ft$$

W8x10 - chosen from AISC Table 3-2

$$w_{self} := 10\ \frac{lb}{ft}$$

$$M_u := (w_b + w_{self} + W_{roof}) \cdot \frac{B^2}{8} + F_s \cdot \frac{B}{4} = (2.109 \cdot 10^4)\ lb \cdot ft$$

$$M_n := 36573\ lb \cdot ft$$

$$\Omega_b := 1.67$$

$$\frac{M_n}{\Omega_b} = (2.19 \cdot 10^4)\ lb \cdot ft$$

$$checkM := \text{if } M_u < \left(\frac{M_n}{\Omega_b}\right) \ \| \ \text{"ok"} \ \text{else} \ \| \ \text{"not ok"} = \text{"ok"}$$

W8x10 section is sufficient to sustain applied loads

Design wind+glass beam:

$$w_g := 8\ \frac{lb}{ft^2} \cdot h_g = 82\ \frac{lb}{ft}$$

$$W_{roof} := p_{roof\_p\_net} \cdot \frac{h_g}{2} \cdot \frac{s^2}{m} = \left(1.08 \cdot 10^3\right)\ \frac{lb}{ft}$$

$$W_{b4} := W_{roof} \cdot \frac{B}{2 \cdot L} = 681.569\ \frac{lb}{ft}$$

$$R_{b4} := W_{b4} \cdot \frac{L}{2} = \left(2.272 \cdot 10^3\right)\ lb$$

$$W_{b3} := \left(p_{roof\_p\_net} \cdot \frac{h_g}{2} + p_{b3\_p\_net} \cdot \frac{h_g}{2}\right) \cdot \frac{B}{2 \cdot L} \cdot \frac{s^2}{m} = \left(1.339 \cdot 10^3\right)\ \frac{lb}{ft}$$

$$R_{b3} := W_{b3} \cdot \frac{L}{2} = \left(4.463 \cdot 10^3\right)\ lb$$

$$W_{b2} := \left(p_{b3\_p\_net} \cdot \frac{h_g}{2} + p_{b2\_p\_net} \cdot \frac{h_g}{2}\right) \cdot \frac{B}{2 \cdot L} \cdot \frac{s^2}{m} = \left(1.283 \cdot 10^3\right)\ \frac{lb}{ft}$$

$$R_{b2} := W_{b2} \cdot \frac{L}{2} = \left(4.278 \cdot 10^3\right)\ lb$$

$$W_{b1} := \left(p_{b2\_p\_net} \cdot \left(\frac{h_g}{2} - \left(15\ ft - h_g\right)\right) + p_{15\_p\_net} \cdot \left(15\ ft - h_g\right) + p_{15\_p\_net} \cdot \frac{h_g}{2}\right) \cdot \frac{B}{2 \cdot L} \cdot \frac{s^2}{m} = \left(1.211 \cdot 10^3\right)\ \frac{lb}{ft}$$

$$R_{b1} := W_{b1} \cdot \frac{L}{2} = \left(4.036 \cdot 10^3\right)\ lb$$

$$M_u := \left(W_{b3} + w_g\right) \cdot \frac{L^2}{8} = \left(7.893 \cdot 10^3\right)\ lb \cdot ft$$

choose largest W

W8x10 - chosen from AISC Table 3-2 and confirmed adequate as this moment is less than that applied to the W8x10 roof beams authorized above

Design glass beam:

$d_{W8x10} := 7.89\ in$

$w_{glass} := w_g = 82\ \frac{lb}{ft}$

$R_{glass} := w_{glass} \cdot \frac{B}{2} = 345.083\ lb$

$M_u := w_{glass} \cdot \frac{B^2}{8} = 726.113\ lb \cdot ft$

W8x10 - chosen from AISC Table 3-2 and confirmed adequate as this moment is less than that applied to the W8x10 roof beams authorized above

Design corner column:

$P_u := R_{roof\_g} + R_{b4} + R_{b3} + R_{b2} + R_{b1} + 2 \cdot R_{glass} + 10 \frac{lb}{ft} \cdot 4 \cdot (B + L) = \left(2.077 \cdot 10^4\right)\ lb$

$R_{roof\_g} = \left(4.432 \cdot 10^3\right)\ lb$

$R_{b4} = \left(2.272 \cdot 10^3\right)\ lb$

$R_{b3} = \left(4.463 \cdot 10^3\right)\ lb$

$R_{b2} = \left(4.278 \cdot 10^3\right)\ lb$

$R_{b1} = \left(4.036 \cdot 10^3\right)\ lb$

$R_{glass} = 345.083\ lb$

$L_b := h_g = 10.25\ ft$ $L_b := 11\ ft$ round up for conservative use of AISC Table 6-2

W4x13 - chosen from AISC Table 6-2

$W := 13 \frac{lb}{ft}$

$P_c := 40000\ lb$

$M_{cx} := 13100\ lb \cdot ft$

$d := 4.16\ in$

$b_f := 4.06\ in$

$t_f := 0.345\ in$

$Z_y := 2.92\ in^3$

$S_y := 1.9\ in^3$

$P_u := R_{roof\_g} + R_{b3} + R_{b2} + R_{b1} + 2 \cdot R_{glass} + 10 \frac{lb}{ft} \cdot 3 \cdot (B + L) + W \cdot H = \left(1.871 \cdot 10^4\right)\ lb$

$e_w := \frac{d}{2} = 2.08\ in$

$$e_g := \frac{b_f}{2} = 2.03\ in$$

$$M_{x\_w} := \left(R_{b3} + R_{b2} + R_{b1} + 3 \cdot R_{glass} + W \cdot 4 \cdot L\right) \cdot e_w = \left(2.454 \cdot 10^3\right)\ lb \cdot ft$$

$$M_{y\_g} := \left(R_{roof\_g} + 3 \cdot R_{glass} + W \cdot 4 \cdot B\right) \cdot e_g = 998.873\ lb \cdot ft$$

$$\lambda_f := \begin{array}{l} \text{if } \frac{b_f}{2 \cdot t_f} < 0.38 \cdot \sqrt[2]{\frac{E}{F_y}} \\ \quad \| \text{“compact”} \\ \text{else if } 0.38 \cdot \sqrt[2]{\frac{E}{F_y}} < \frac{b_f}{2 \cdot t_f} < \sqrt[2]{\frac{E}{F_y}} \\ \quad \| \text{“noncompact”} \\ \text{else} \\ \quad \| \text{“slender”} \end{array} = \text{“compact”}$$

FLB does not apply (F6.2)

$$M_{cy} := \begin{array}{l} \text{if } F_y \cdot Z_y < 1.6 \cdot F_y \cdot S_y \\ \quad \| \frac{F_y \cdot Z_y}{1.67} \\ \text{else} \\ \quad \| \frac{1.6 \cdot F_y \cdot S_y}{1.67} \end{array} = \left(7.285 \cdot 10^3\right)\ lb \cdot ft$$

$$\frac{P_u}{P_c} = 0.468$$

$$i := \begin{array}{l} \text{if } \frac{P_u}{P_c} < 0.2 \\ \quad \| \frac{P_u}{2 \cdot P_c} + \left(\frac{M_{x\_w}}{M_{cx}} + \frac{M_{y\_g}}{M_{cy}}\right) \\ \text{else} \\ \quad \| \frac{P_u}{P_c} + \frac{8}{9} \cdot \left(\frac{M_{x\_w}}{M_{cx}} + \frac{M_{y\_g}}{M_{cy}}\right) \end{array} = 0.756$$

$$checki := \text{if } i \le 1 \; \| \; \text{"ok"} \; \text{else} \; \| \; \text{"not ok"} = \text{"ok"}$$

W4x13 section has sufficient strength to be used as corner columns

Calculate quantities:

$$L_{W8x10} := \left(L - 2 \cdot d\right) \cdot 8 + \left(B - 2 \cdot b_f\right) \cdot 8 = 109.707\ ft$$

$$A_{glass} := \left(h_g - d_{W8x10}\right) \cdot \left(\left(B - 2 \cdot b_f\right) \cdot 7 + \left(L - 2 \cdot d\right) \cdot 8\right) = 978.115\ ft^2$$

$$L_{W4x13} := h \cdot 4 = 164\ ft$$

$$A_{plan} := B \cdot L = 56.111\ ft^2$$

$$n_{tread} := \frac{H}{7\ in} = 47.143$$

K. South Pier Design

Concrete Properties:

$f'_c := 4\ \mathbf{ksi}$ $w_c := 145\ \mathbf{pcf}$ $\nu := 0.2$

$E_c := 1820 \cdot \sqrt{\frac{f'_c}{\mathbf{ksi}}} \cdot \mathbf{ksi} = 3640\ \mathbf{ksi}$

Pier Cap Design

Designing it like a beam

#4 bars for stirrups $d_{stirrup} := 0.5\ \mathbf{in}$

$c_c := 2\ \mathbf{in}$ $b := 36\ \mathbf{in}$ $h := 48\ \mathbf{in}$ $f_y := 60\ \mathbf{ksi}$

$\beta_1 := 0.85$ $d_b := 1.693\ \mathbf{in}$ #14 rebar

$cover := c_c + d_{stirrup} + d_b$ $A_b := 2.25\ \mathbf{in}^2$

$d := h - cover$

singly reinforced therefore $d_t := d = 43.807\ \mathbf{in}$

upper limit for tension for tension controlled design

$$A_{stension} := \frac{0.85 \cdot f'_c \cdot b \cdot \beta_1}{f_y} \cdot \left(\frac{3 \cdot d_t}{8}\right) = 28.486\ \mathbf{in}^2$$

$A_s := 4 \cdot A_b = 9\ \mathbf{in}^2$

$A_s < A_{stension} = 1$

$$A_{smin} := \max\left(\frac{200}{\left(\frac{f_y}{\mathbf{psi}}\right)}, \frac{3 \cdot \sqrt{\frac{f'_c}{\mathbf{psi}}}}{\frac{f_y}{\mathbf{psi}}}\right) \cdot b \cdot d = 5.257\ \mathbf{in}^2$$

$A_s > A_{smin} = 1$

$$a := \frac{A_s \cdot f_y}{0.85 \cdot f'_c \cdot b} = 4.412\ \mathbf{in}$$

$F_c := 0.85 \cdot f'_c \cdot b \cdot a = 540 \ \mathbf{kip}$

$F_s := A_s \cdot f_y = 540 \ \mathbf{kip}$

$M_n := F_s \cdot \left(d - \frac{a}{2}\right) = 1872.05 \ \mathbf{kip \cdot ft}$

$M_r := 0.9 \cdot M_n = 1684.845 \ \mathbf{kip \cdot ft}$

$M_u := 1441.29 \ \mathbf{kip \cdot ft}$

$\frac{M_u}{M_r} = 0.855$ OK

Shear Resistance

$\phi_s := 0.75$

$\lambda := 1$

$\lambda_s := \min\left(1, \sqrt{\frac{2}{1 + \frac{d}{\mathbf{in} \cdot 10}}}\right) = 0.61$

$\rho_w := \frac{A_s}{b \cdot d} = 0.006$

$V_c := \min\left(5 \cdot \lambda \cdot \sqrt{\frac{f'_c}{\mathbf{psi}}} \ \mathbf{psi}, 8 \cdot \lambda \cdot \lambda_s \cdot \rho_w^{\frac{1}{3}} \cdot \sqrt{\frac{f'_c}{\mathbf{psi}}} \ \mathbf{psi}\right) \cdot b \cdot d = 86.935 \ \mathbf{kip}$

$\phi V_c := \phi_s \cdot V_c = 65.201 \ \mathbf{kip}$ $V_u := 293.66 \ \mathbf{kip}$

this is less than Vu therefore need shear reinforcement

Use #4 U stirrups

$f_{yt} := 40 \ \mathbf{ksi}$ $d_{st} := 0.5 \ \mathbf{in}$ $A_{st} := 0.2 \ \mathbf{in}^2$

$A_s = 9 \ \mathbf{in}^2$

$V_{umax} := \phi_s \cdot \left(V_c + 8 \cdot \sqrt{\frac{f'_c}{\mathbf{psi}}} \ \mathbf{psi} \cdot b \cdot d\right) = 663.65 \ \mathbf{kip}$ OK

$A_v := 2 \cdot A_{st} = 0.4\ \mathbf{in}^2$

$$s := \frac{A_v \cdot f_{yt} \cdot d}{\frac{V_u}{\phi_s} - V_c} = 2.301\ \mathbf{in}$$

space stirrups every 2 inches

Pier column

Design axial load capacity for tied columns

Cross-Section Dimensions: $b := 36\ \mathbf{in}$ $h := 36\ \mathbf{in}$

$A_g := b \cdot h = 9\ \mathbf{ft}^2$

$P_u := 554.4\ \mathbf{kip}$

$n_{bars} := 8$ $A_{bar} := 2.25\ \mathbf{in}^2$

$f_y := 60\ \mathbf{ksi}$ $A_{st} := n_{bars} \cdot A_{bar}$

$d_{bar} := 1.693\ \mathbf{in}$

$\frac{P_u}{A_g} = 0.428\ \mathbf{ksi}$ $\rho_g := \frac{A_{st}}{A_g} = 0.014$

$\phi\, P_n := 0.8 \cdot 0.65 \cdot \left(0.85 \cdot f'_c \cdot A_g + \left(f_y - 0.85 \cdot f'_c\right) \cdot A_{st}\right)$

$P_r := \phi\, P_n = 2821.104\ \mathbf{kip}$

$\frac{P_u}{P_r} = 0.197$

Buckling Strength

$A_{st} = 0.125\ \mathbf{ft}^2$

$A_g = 9\ \mathbf{ft}^2$

$A_c := A_g - A_{st} = 8.875\ \mathbf{ft}^2$

$\phi\, P_n := 0.8 \cdot 0.65 \cdot \left(0.85 \cdot f'_c \cdot A_c + f_y \cdot A_{st}\right) = 2821.104\ \mathbf{kip}$

$r := 0.288 \cdot h = 0.864\ \mathbf{ft}$

$L := 27.5\ \mathbf{ft}$ $k := 2.1$

$$F_e := \frac{\pi^2 \cdot E_c}{\left(\frac{k \cdot L}{r}\right)^2} = 8.041\ \mathbf{ksi}$$

$P_{cr} := F_e \cdot A_g = 10421.486\ \mathbf{kip}$

Axial load capacity based on elastic buckling is much larger than that based on strength. Strength controls.

Reinforcement

minimal horizontal clear spacing

$s_{bc} := \max\left(1\ \mathbf{in}, d_{bar}\right) = 1.693\ \mathbf{in}$

minimal vertical clear spacing

$d_{ties} := 0.5\ \mathbf{in}$

$s_{hc} := 1\ \mathbf{in}$

$16 \cdot d_{bar} = 2.257\ \mathbf{ft}$

maximum vertical spacing between ties

$b = 3\ \mathbf{ft}$

$48 \cdot d_{ties} = 2\ \mathbf{ft}$

$s \le \min\left(16 \cdot d_{bar}, 48 \cdot d_{ties}, b\right) = 1$ $s := 2\ \mathbf{ft}$

<u>Wind Loading</u>

wind creates a negligible amount of axial compression in the pier cap.
check axial moment caused in pier column

Combined Axial and Flexure Resistance

Point A

$P_{rA} := \phi P_n = 2821.104\ \mathbf{kip}$

$\phi M_{nA} := 0\ \mathbf{kip} \cdot \mathbf{ft}$ $M_{rA} := \phi M_{nA}$

Point B

$E_s := 29000\ \mathbf{ksi}$

$\varepsilon_{cu} := 0.003$

$d_{ties} = 0.042\ \mathbf{ft}$ $\quad d_{bar} = 1.693\ \mathbf{in}$

$c_c := 2\ \mathbf{in}$

$y_{bar} := \dfrac{h}{2} = 1.5\ \mathbf{ft}$

$A_{s1} := 3 \cdot A_{bar}$ $\quad y_{s1} := c_c + d_{ties} + \dfrac{d_{bar}}{2} = 3.347\ \mathbf{in}$

$A_{s2} := 2 \cdot A_{bar}$

$A_{s3} := 3 \cdot A_{bar}$ $\quad y_{s3} := h - \left(c_c + d_{ties} + \dfrac{d_{bar}}{2}\right) = 32.654\ \mathbf{in}$

$\dfrac{y_{s3} - y_{s1}}{2} = 1.221\ \mathbf{ft}$ $\quad y_{s2} := y_{s1} + \dfrac{y_{s3} - y_{s1}}{2} = 18\ \mathbf{in}$

$\varepsilon_{ty} := \dfrac{f_y}{E_s} = 0.002$

$\varepsilon_{s1} := 0$ $\quad \varepsilon_o := \dfrac{\varepsilon_{s1} \cdot h - \varepsilon_{cu} \cdot y_{s1}}{h - y_{s1}} = -3.075 \cdot 10^{-4}$

$c := \dfrac{\varepsilon_{cu} \cdot h}{\varepsilon_{cu} - \varepsilon_o} = 32.654\ \mathbf{in}$

$\beta_1 := 0.85$

$a := \beta_1 \cdot c = 2.313\ \mathbf{ft}$

$\varepsilon_{s2} := \dfrac{(h - y_{s2})}{h} \cdot \varepsilon_o + \dfrac{y_{s2}}{h} \cdot \varepsilon_{cu} = 0.00135$

$\varepsilon_{s3} := \dfrac{(h - y_{s3})}{h} \cdot \varepsilon_o + \dfrac{y_{s3}}{h} \cdot \varepsilon_{cu} = 0.00269$

stresses $\varepsilon_{ty} = 0.00207$

$f_{s1} := 0\ \mathbf{ksi}$ strain was 0 $F_{s1} := A_{s1} \cdot f_{s1} = 0\ \mathbf{lbf}$

$\varepsilon_{s2} < \varepsilon_{ty} = 1$ elastic in compression

$f_{s2} := E_s \cdot \varepsilon_{s2} - 0.85 \cdot f'_c = 35.642\ \mathbf{ksi}$ $F_{s2} := A_{s2} \cdot f_{s2} = 160.389\ \mathbf{kip}$

$\varepsilon_{s3} > \varepsilon_{ty} = 1$ yielded in compression

$f_{s3} := f_y - 0.85 \cdot f'_c = 56.6\ \mathbf{ksi}$ $F_{s3} := A_{s3} \cdot f_{s3} = 382050\ \mathbf{lbf}$

$A_c := b \cdot a = 6.939\ \mathbf{ft}^2$

$F_c := 0.85 \cdot f'_c \cdot A_c = 3397270.14\ \mathbf{lbf}$

$y_c := h - \dfrac{a}{2} = 1.844\ \mathbf{ft}$

$P_n := F_c + F_{s1} + F_{s2} + F_{s3} = 3939.709\ \mathbf{kip}$

$M_n := F_c \cdot (y_c - y_{bar}) + F_{s1} \cdot (y_{s1} - y_{bar}) + F_{s2} \cdot (y_{s2} - y_{bar}) + F_{s3} \cdot (y_{s3} - y_{bar})$

$M_n = 1633.567\ \mathbf{kip \cdot ft}$

$\varepsilon_t := \varepsilon_{s1} = 0$

$\phi := 0.65$

$P_{rB} := \phi \cdot P_n = 2560.811\ \mathbf{kip}$

$M_{rB} := \phi \cdot M_n = 1061.819\ \mathbf{kip \cdot ft}$

Point C

$\varepsilon_{s1} := \dfrac{-f_y}{E_s} = -0.002$ $\varepsilon_o := \dfrac{\varepsilon_{s1} \cdot h - \varepsilon_{cu} \cdot y_{s1}}{h - y_{s1}} = -0.003$

$c := \dfrac{\varepsilon_{cu} \cdot h}{\varepsilon_{cu} - \varepsilon_o} = 19.326\ \mathbf{in}$

$a := \beta_1 \cdot c = 1.369\ \mathbf{ft}$

$$\varepsilon_{s2} := \frac{(h - y_{s2})}{h} \cdot \varepsilon_o + \frac{y_{s2}}{h} \cdot \varepsilon_{cu} = 0.00021$$

$$\varepsilon_{s3} := \frac{(h - y_{s3})}{h} \cdot \varepsilon_o + \frac{y_{s3}}{h} \cdot \varepsilon_{cu} = 0.00248$$

stresses $\varepsilon_{ty} = 0.00207$

$|\varepsilon_{s1}| \geq \varepsilon_{ty} = 1$ $\varepsilon_{s1} < 0 = 1$ yielded in tension

$f_{s1} := -f_y = -60\ \mathbf{ksi}$ $F_{s1} := A_{s1} \cdot f_{s1} = -405\ \mathbf{kip}$

$|\varepsilon_{s2}| < \varepsilon_{ty} = 1$ $\varepsilon_{s2} > 0 = 1$ elastic in compression

$f_{s2} := E_s \cdot \varepsilon_{s2} - 0.85 \cdot f'_c = 2.567\ \mathbf{ksi}$ $F_{s2} := A_{s2} \cdot f_{s2} = 11.553\ \mathbf{kip}$

$|\varepsilon_{s3}| \geq \varepsilon_{ty} = 1$ $\varepsilon_{s3} > 0 = 1$ yielded in compression

$f_{s3} := f_y - 0.85 \cdot f'_c = 56.6\ \mathbf{ksi}$ $F_{s3} := A_{s3} \cdot f_{s3} = 382.05\ \mathbf{kip}$

$A_c := b \cdot a = 4.107\ \mathbf{ft}^2$

$F_c := 0.85 \cdot f'_c \cdot A_c = 2010.629\ \mathbf{kip}$

$y_c := h - \frac{a}{2} = 2.316\ \mathbf{ft}$

$P_n := F_c + F_{s1} + F_{s2} + F_{s3} = 1999.232\ \mathbf{kip}$

$$M_n := F_c \cdot (y_c - y_{bar}) + F_{s1} \cdot (y_{s1} - y_{bar}) + F_{s2} \cdot (y_{s2} - y_{bar}) + F_{s3} \cdot (y_{s3} - y_{bar})$$

$M_n = 2600.863\ \mathbf{kip \cdot ft}$

$\varepsilon_t := |\varepsilon_{s1}| = 0.002$

$\phi := 0.65$

$P_{rC} := \phi \cdot P_n = 1299.501\ \mathbf{kip}$ $M_{rC} := \phi \cdot M_n = 1690.561\ \mathbf{kip \cdot ft}$

Point D

$\varepsilon_{s1} := -0.005$

$\varepsilon_o := \dfrac{\varepsilon_{s1} \cdot h - \varepsilon_{cu} \cdot y_{s1}}{h - y_{s1}} = -0.006$

$c := \dfrac{\varepsilon_{cu} \cdot h}{\varepsilon_{cu} - \varepsilon_o} = 12.245\ \mathbf{in}$

$a := \beta_1 \cdot c = 0.867\ \mathbf{ft}$

$\varepsilon_{s2} := \dfrac{(h - y_{s2})}{h} \cdot \varepsilon_o + \dfrac{y_{s2}}{h} \cdot \varepsilon_{cu} = -0.00141$

$\varepsilon_{s3} := \dfrac{(h - y_{s3})}{h} \cdot \varepsilon_o + \dfrac{y_{s3}}{h} \cdot \varepsilon_{cu} = 0.00218$

stresses

$\varepsilon_{ty} = 0.00207$

$|\varepsilon_{s1}| \geq \varepsilon_{ty} = 1$ $\quad \varepsilon_{s1} < 0 = 1$ $\quad$ yielded in tension

$f_{s1} := -f_y = -60\ \mathbf{ksi}$ $\quad F_{s1} := A_{s1} \cdot f_{s1} = -405\ \mathbf{kip}$

$|\varepsilon_{s2}| < \varepsilon_{ty} = 1$ $\quad \varepsilon_{s2} < 0 = 1$ $\quad$ elastic in tension

$f_{s2} := E_s \cdot \varepsilon_{s2} = -40.888\ \mathbf{ksi}$ $\quad F_{s2} := A_{s2} \cdot f_{s2} = -183.997\ \mathbf{kip}$

$|\varepsilon_{s3}| \geq \varepsilon_{ty} = 1$ $\quad \varepsilon_{s3} > 0 = 1$ $\quad$ yielded in compression

$f_{s3} := f_y - 0.85 \cdot f'_c = 56.6\ \mathbf{ksi}$ $\quad F_{s3} := A_{s3} \cdot f_{s3} = 382.05\ \mathbf{kip}$

$A_c := b \cdot a = 2.602\ \mathbf{ft}^2$

$F_c := 0.85 \cdot f'_c \cdot A_c = 1273976.303\ \mathbf{lbf}$

$y_c := h - \dfrac{a}{2} = 2.566\ \mathbf{ft}$

$P_n := F_c + F_{s1} + F_{s2} + F_{s3} = 1067.029\ \mathbf{kip}$

$M_n := F_c \cdot (y_c - y_{bar}) + F_{s1} \cdot (y_{s1} - y_{bar}) + F_{s2} \cdot (y_{s2} - y_{bar}) + F_{s3} \cdot (y_{s3} - y_{bar})$

$M_n = 2319.554$ **kip · ft**

$\varepsilon_t := |\varepsilon_{s1}| = 0.005$

$\varepsilon_t > \varepsilon_{ty} = 1$

$\varepsilon_t < \varepsilon_{ty} + 0.003 = 1$

$\phi := 0.65 + 0.25 \cdot \left(\frac{\varepsilon_t - \varepsilon_{ty}}{0.003}\right) = 0.894$

$P_{rD} := \phi \cdot P_n = 954.194$ **kip**

$M_{rD} := \phi \cdot M_n = 2074.268$ **kip · ft**

Point E

$\varepsilon_{s1} := -0.02$

$\varepsilon_o := \frac{\varepsilon_{s1} \cdot h - \varepsilon_{cu} \cdot y_{s1}}{h - y_{s1}} = -0.022$

$c := \frac{\varepsilon_{cu} \cdot h}{\varepsilon_{cu} - \varepsilon_o} = 4.259$ **in**

$a := \beta_1 \cdot c = 0.302$ **ft**

$\varepsilon_{s2} := \frac{(h - y_{s2})}{h} \cdot \varepsilon_o + \frac{y_{s2}}{h} \cdot \varepsilon_{cu} = -0.00968$

$\varepsilon_{s3} := \frac{(h - y_{s3})}{h} \cdot \varepsilon_o + \frac{y_{s3}}{h} \cdot \varepsilon_{cu} = 0.00064$

stresses

$\varepsilon_{ty} = 0.00207$

$|\varepsilon_{s1}| \geq \varepsilon_{ty} = 1$ $\quad \varepsilon_{s1} < 0 = 1$ $\quad$ yielded in tension

$f_{s1} := -f_y = -60$ **ksi**

$F_{s1} := A_{s1} \cdot f_{s1} = -405$ **kip**

$|\varepsilon_{s2}| \geq \varepsilon_{ty} = 1$ $\varepsilon_{s2} < 0 = 1$ yielded in tension

$f_{s2} := -f_y = -60$ **ksi** $F_{s2} := A_{s2} \cdot f_{s2} = -270$ **kip**

$|\varepsilon_{s3}| < \varepsilon_{ty} = 1$ $\varepsilon_{s3} > 0 = 1$ elastic in compression

$f_{s3} := E_s \cdot \varepsilon_{s3} - 0.85 \cdot f'_c = 15.242$ **ksi** $F_{s3} := A_{s3} \cdot f_{s3} = 102886.074$ **lbf**

$A_c := b \cdot a = 0.905$ **ft**^{2}

$F_c := 0.85 \cdot f'_c \cdot A_c = 443.122$ **kip**

$y_c := h - \frac{a}{2} = 2.849$ **ft**

$P_n := F_c + F_{s1} + F_{s2} + F_{s3} = -128.992$ **kip**

$M_n := F_c \cdot (y_c - y_{bar}) + F_{s1} \cdot (y_{s1} - y_{bar}) + F_{s2} \cdot (y_{s2} - y_{bar}) + F_{s3} \cdot (y_{s3} - y_{bar})$

$M_n = 1218.033$ **kip·ft**

$\varepsilon_t := |\varepsilon_{s1}| = 0.02$

$\varepsilon_t := |\varepsilon_{s1}| = 0.02$

$\varepsilon_t > \varepsilon_{ty} = 1$

$\varepsilon_t > \varepsilon_{ty} + 0.003 = 1$

$\phi := 0.9$

$P_{rE} := \phi \cdot P_n = -116.093$ **kip**

$M_{rE} := \phi \cdot M_n = 1096.23$ **kip·ft**

Summary:

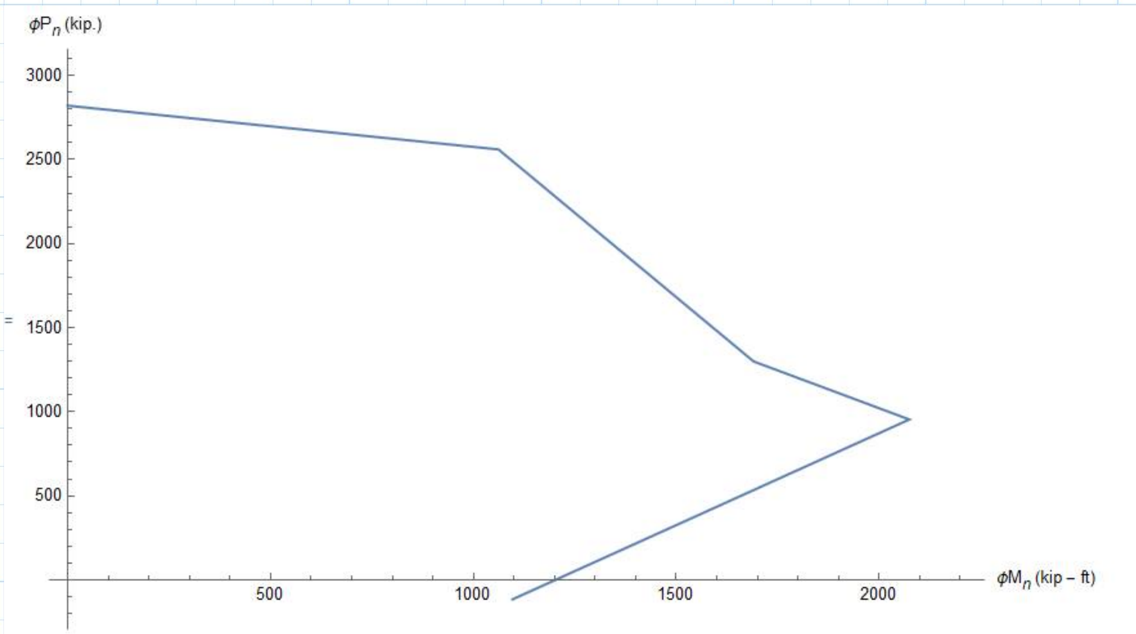
$P_{rA} = 2821.104$ **kip** $M_{rA} = 0$ **kip·ft**

$P_{rB} = 2560.811$ **kip** $M_{rB} = 1061.819$ **kip·ft**

$P_{rC} = 1299.501$ **kip** $M_{rC} = 1690.561$ **kip·ft**

$P_{rD} = 954.194$ **kip** $M_{rD} = 2074.268$ **kip·ft**

$P_{rE} = -116.093$ **kip** $M_{rE} = 1096.23$ **kip·ft**



In Strength 3

$M_u := 35.9$ **kip·ft** $P_u := 440.36$ **kip**

This falls within the envelop, therefore it is safe

In Service 1

$M_u := 7.7$ **kip·ft** $P_u := 458.7$ **kip** These also fall within the envelop.

Anchor Design
(Critical Case: Strength I)

Description:

The design of the anchors is for a proposed pedestrian overpass in Waterloo, IA. The anchors will be located on the north side of the bridge, and will be supporting the two main towers. Each tower will contain one back-stay cable with a transition block, anchorage cable, and concrete anchor block.

Project Goal:

Determine the most economical design for the anchors.

Variables:

$\gamma := 109 \frac{lb}{ft^3}$	Unit Weight of Soil
$\gamma_{sat} := 130 \frac{lb}{ft^3}$	Saturated Unit Weight of Soil
$\gamma_w := 62.4 \frac{lb}{ft^3}$	Unit Weight of Water
$\gamma' := \gamma_{sat} - \gamma_w = 67.6 \frac{lb}{ft^3}$	Submerged Unit Weight of Soil
$\gamma_c := 150 \frac{lb}{ft^3}$	Unit Weight of Concrete
$\phi := 38.5 °$	Angle of Friction
$D_w := 12 ft$	Depth of Water Table
$P_x := 1710000 lb$	Horizontal Load
$P_y := 1710000 lb$	Vertical Load
$P := \sqrt[2]{P_x{}^2 + P_y{}^2} = (2.418 \cdot 10^6) lb$	Load in Back-Stay Cable

Assumptions:
From the soil report from USGS geodata, it shows that from 0-18" the soil is a loamy sand, followed by sand per 18-30", gravelly course sand per 30-55", course sand per 55-70", and gravelly course sand per 70-80". Since the layer that is cohesive only goes down a short distance from soil grade, it may be reasonable to analyze this soil as a purely granular soil. It was also noted from the soils report that the soil surrounding the project site is excessively drained. The preliminary design with be with a 10'x10' concrete anchor block anchored at 15' below the ground surface.

Anchor Design
(Critical Case: Strength I)

$d_1 := 15\ ft$

$d_2 := 25\ ft$

$H := 10\ ft$

$\frac{d_1}{d_2} = 0.6$ Shallow Anchor, Greater than 0.5 but less than 0.7

Calculation of Friction Forces

F_{top} Neglected in this analysis

$\sigma'_o := d_1 \cdot \gamma + H \cdot \gamma_c = \left(3.135 \cdot 10^3\right) \frac{lb}{ft^2}$

$F_{bot} := H \cdot \sigma'_o \cdot \tan(\phi) = \left(2.494 \cdot 10^4\right) \frac{lb}{ft}$

Calculation Soil Pressures

$K_a := \tan\left(45° - \frac{\phi}{2}\right)^2 = 0.233$

$\sigma'_a := \left(d_1 \cdot \gamma + \left(d_2 - D_w\right) \cdot \gamma'\right) \cdot K_a = 584.847 \frac{lb}{ft^2}$

$P_a := \frac{1}{2} \cdot \left(\sigma'_a\right) \cdot \left(d_2\right) = \left(7.311 \cdot 10^3\right) \frac{lb}{ft}$

$K_p := \tan\left(45° + \frac{\phi}{2}\right)^2 = 4.298$

$\sigma'_p := \left(d_1 \cdot \gamma + \left(d_2 - D_w\right) \cdot \gamma'\right) \cdot K_p = \left(1.08 \cdot 10^4\right) \frac{lb}{ft^2}$

$P_p := \frac{1}{2} \cdot \left(\sigma'_p\right) \cdot \left(d_2\right) = \left(1.351 \cdot 10^5\right) \frac{lb}{ft}$

$FS_v := 1.5$

$P_{all} := \frac{\left(\frac{1}{2} \cdot P_p + F_{bot} - P_a\right)}{FS_v} = \left(5.677 \cdot 10^4\right) \frac{lb}{ft}$

Anchor Design
(Critical Case: Strength I)

Calculation of Anchorage Resistance

$T_{AR} := P$ $C := 0.65$ $SF := 1.5$ Block Anchor Factors

$L := 25\ ft$ Length of Anchor Rod

$$P_{ar} := \frac{C \cdot \gamma \cdot {d_2}^2 \cdot L \cdot K_p + \sigma'_o \cdot H^2}{SF} = \left(3.381 \cdot 10^6\right)\ lb \quad >/= \quad P = \left(2.418 \cdot 10^6\right)\ lb$$

Condition Satisfied

North Foundation Design
(Case 1: Strength I)

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North Foundation Design
(Case 1: Strength I)

Foundation Description:

The design of this foundation is for a proposed pedestrian overpass in Waterloo, IA. The foundation in need of attention is the foundations located on the north side of the bridge, and will be supporting two concrete towers. To guard against excessive settlement of the abutments that would be detrimental to bridge operations, the abutments shall be supported by a pile group.

Project Goal:

Determine the most economical design for the pile group.

Variables:

$\gamma := 109 \frac{lb}{ft^3}$	Unit Weight of Soil
$\gamma_{sat} := 130 \frac{lb}{ft^3}$	Saturated Unit Weight of Soil
$\gamma_w := 62.4 \frac{lb}{ft^3}$	Unit Weight of Water
$\gamma' := \gamma_{sat} - \gamma_w = 67.6 \frac{lb}{ft^3}$	Submerged Unit Weight of Soil
$D_w := 12\ ft$	Depth of Water Table
$L_p := 60\ ft$	Length of the Pile
$P_{rxn} := 7232466\ lb$	Reaction Force From Bridge Load
$P_{dead} := 1000000\ lb$	Dead Load of Both Tower
$P_g := P_{rxn} + P_{dead}$	Vertical Load
$V_y := 369526\ lb$	Horizontal Load N-S
$V_x := 2050\ lb$	Horizontal Load E-W
$M_y := 18825\ lb \cdot ft$	Moment N-S
$M_x := 107350600\ lb \cdot ft$	Moment E-W

Assumptions:
From the soil report from USGS geodata, it shows that from 0-18" the soil is a loamy sand, followed by sand per 18-30", gravelly course sand per 30-55", course sand per 55-70", and gravelly course sand per 70-80". Since the layer that is cohesive only goes down a short distance from soil grade, it may be reasonable to analyze this soil as a purely granular soil. It was also noted from the soils report that the soil surrounding the project site is excessively drained. The preliminary design with be with a HP18x204 steel pile cross section with an individual pile length of 60'.

North Foundation Design
(Case 1: Strength I)

Design Calculations for Individual Piles

For calculating pile point bearing capacity, the reduced rigidity index needs to be assessed. Therefore taking values for the static stress-strain modulus of elasticity from table 5-6 in the Bowles, *Foundation Analysis and Design* was found for the soil at the bottom of the pile.

$E_s := 50\ MPa$ Assuming course sand that is dense and wet.

$E_s := 1000000\ \frac{lb}{ft^2}$ Conversion to US units.

$\sigma_{z12}' := \gamma \cdot D_w = 1308\ \frac{lb}{ft^2}$ Vertical Effective Stress at Water Table Depth

$\sigma_{z40}' := \gamma \cdot D_w + \left(\gamma' \cdot \left(L_p - D_w\right)\right) = 4552.8\ \frac{lb}{ft^2}$ Vertical Effective Stress at Pile Depth

Using table 2-7 in the Bowles text, the Poisson's ratio for this soil was determined:

$\mu := 0.35$ Cohesionless, dense sand.

Taking the assumed internal angle of friction for poorly graded sand from Lindeburg, *Civil Engineering Reference for PE 8th Edition*, the critical depth can be found using Figure 3.10 from the Poulos and Davis text:

$\phi' := 38\ °$

$\phi := \frac{3}{4} \cdot \phi' + 10\ ° = 38.5\ °$

Zc / d ratio from Poulos & Davis, *Pile Foundation Analysis & Design*, Figure 3.10: = 10.5

$b_f := 18.1\ in$ $d := 18.3\ in$ HP18x204 steel pile.

$B_p := \sqrt[2]{\left(d^2\right) + \left(b_f^{\,2}\right)} = 25.739\ in$ Width of pile is diagonal

$A_p := b_f \cdot d = 331.23\ in^2$ Including Soil Plug

$z_c := 10.5 \cdot B_p = 22.522\ ft$ Critical Depth

$\sigma_p'' := D_w \cdot \gamma + \left(\left(z_c - D_w\right) \cdot \gamma'\right) = 2019.266\ \frac{lb}{ft^2}$ Vertical Effective Stress at Critical Depth

North Foundation Design
(Case 1: Strength I)

Calculation of bearing capacity factors ensues:

$G_s := \dfrac{E_s}{2 \cdot (1+\mu)} = 370370.37 \dfrac{lb}{ft^2}$ $\quad c' := 0$ Cohesionless sand.

$I_r := \dfrac{G_s}{c' + \sigma_{z40}' \cdot \tan(\phi')} = 104.123$ Rigidity Index

Bowles, *Foundation Analysis & Design*, Table on P.894 : Rigidity Index Within Range for Sandy Soil

$\varepsilon_v := \dfrac{(1+\mu) \cdot (1-2 \cdot \mu) \cdot (\sigma_{z40}')}{E_s \cdot (1-\mu)} = 0.003$ Volumetric Strain

$I_{rr} := \dfrac{I_r}{1 + \varepsilon_v \cdot I_r} = 80.381$ Reduced Rigidity Index

$K_o := 1 - \sin(\phi') = 0.384$ At rest earth pressure coefficient

Vesic's Bearing Capacity Factors:

$\eta := \dfrac{1 + 2 \cdot K_o}{3} = 0.59$

$N_q := \dfrac{3}{3 - \sin(\phi')} \cdot \exp\left(\left(\dfrac{\pi}{2} - \phi'\right) \tan(\phi')\right) \cdot \left(\tan\left(\dfrac{\pi}{4} + \dfrac{\phi'}{2}\right)\right)^2 \cdot I_{rr}^{\left(\frac{4 \cdot \sin(\phi')}{3 \cdot (1 + \sin(\phi'))}\right)} = 99.84$

$N_\gamma := 0.6 \cdot (N_q - 1) \tan(\phi') = 46.333$

$N_{c1} := (N_q - 1) \cot(\phi') = 126.509$

$N_{c2} := \dfrac{4}{3} \cdot (\ln(I_{rr}) + 1) + \dfrac{\pi}{2} + 1 = 9.753$

$N_c := \text{if}(\phi' = 0, N_{c2}, N_{c1}) = 126.509$

$q_p := \eta \cdot \sigma_{z40}' \cdot N_q + \dfrac{1}{2} \cdot \gamma' \cdot B_p \cdot N_\gamma = 271342.785 \dfrac{lb}{ft^2}$ Pile Point Bearing Capacity

$P_p := A_p \cdot q_p = 624144.936\ lb$ Pile Point Load Capacity

North Foundation Design
(Case 1: Strength I)

Calculation of Side Friction Capacity:

α - Method calculations will be used to calculate the pile side friction capacity.

From Tomlinson, *Pile Design and Construction Practice*, Table 4.10, Large displacement pile due to Steel HP shape. Range is based on soil density, therefore with our dense sand assumption:

$$\frac{K_s}{K_o} = 1.25$$

$K_s := K_o \cdot 1.25 = 0.48$ Lateral Earth-Pressure Coefficient for Side Friction

From Tomlinson, Table 4.11, Smooth Steel Pile at interface with sand:

$\delta_p := 0.6 \cdot \phi' = 22.8\ °$ Angle of Friction Between Pile and Soil

$f_s := K_s \cdot \sigma_p'' \cdot \tan\left(\delta_p\right) = 407.793\ \frac{lb}{ft^2}$ Side Friction Stress, Considering Critical Depth

Calculation of side friction stress profile:

$z_1 := D_w$

$\sigma_{z1} := D_w \cdot \gamma = 1308\ \frac{lb}{ft^2}$

$f_{s12} := K_s \cdot \sigma_{z1} \cdot \tan\left(\delta_p\right) = 264.152\ \frac{lb}{ft^2}$

$z_c = 22.522\ ft$

Layers	Avg. Side Friction Stress
0-12 ft	$f_{s1Bar} := \frac{1}{2} \cdot \left(0 + f_{s12}\right) = 132.076\ \frac{lb}{ft^2}$
12-22.5 ft	$f_{s2Bar} := \frac{1}{2} \cdot \left(f_{s12} + f_s\right) = 335.973\ \frac{lb}{ft^2}$
22.5-75 ft	$f_{s3Bar} := f_s = 407.793\ \frac{lb}{ft^2}$

$P_{p1} := b_f + b_f + d + d = 6.067\ ft$ Perimeter of Pile (Including Soil Plug)

$P_s := P_{p1} \cdot \left(12\ ft \cdot \left(f_{s1Bar}\right) + 10.5\ ft \cdot \left(f_{s2Bar}\right) + 52.5\ ft \cdot \left(f_{s3Bar}\right)\right) = 160898.831\ lb$ Side Friction Capacity

North Foundation Design
(Case 1: Strength I)

<u>Calculations for compression and tension load capacity:</u>

$L_p = 60\ ft$ — Length of Pile

$W_p := 204\ \frac{lb}{ft} \cdot L_p = 12240\ lb$ — Nominal Weight of pile type multiplied by pile length

From the *Standard Guidelines for the Design and Installation of Pile Foundations*, the factors of safety were determined from table A.1 and A.2. Assuming a pile group consisting of 64 piles, the design axial load will be distributed throughout these 64 piles.

$N := 64$ — Number of Piles

$P_{g1} := \frac{P_g}{N} = 64.316\ ton$

$F_1 := 2.0$ — Table A.1, Since this is a preliminary design, using driving formulas and static analysis to determine factors of safety

$F_2 := 1.1$ — Table A.2, HP Pile

$FS := F_1 \cdot F_2 = 2.2$

$P := P_{g1} + W_p = 140872.281\ lb$ </= $P_{all} := \frac{P_p + P_s}{FS} = 356838.076\ lb$

Condition Satisfied

For uplift, the factor of safety is approximately a 50% increase from the compression capacity factor of safety.

$FS_T := 1.5 \cdot FS = 3.3$ — Factor of Safety for Uplift

$T_{all} := \frac{P_s}{FS_T} + W_p = 60997.221\ lb$ — Tension Capacity

Each individual pile satisfies this condition for both combined axial load and bending moment. This design criteria is calculated when calculating to see if the piles will buckle in the analysis of pile groups. This value is also shown in the summary.

North Foundation Design
(Case 1: Strength I)

Pile settlement for an individual pile is as follows (Bowles Method):

$A_p := 60.2\ in^2$ AISC Table 1-4: HP 18x201

$E_p := 29000000\ \frac{lb}{in^2}$

$m := 1$ Shape Factor, m*Is = 1.0 (Relatively square pile)

$I_s := 1$

$\frac{L_p}{B_p} = 27.973$ Fox Embedment Factor Inequality Satisfaction

$I_F := 0.35$ Fox Embedment Factor

$F_1 := 0.25$ Reduction Factor, High side friction capacity compared to design load for an individual pile.

$q := \frac{P_{g1}}{A_p} = 2136.749\ \frac{lb}{in^2}$

$\delta_p := q \cdot B_p \cdot \left(\frac{1-\mu^2}{E_s}\right) \cdot m \cdot I_s \cdot I_F \cdot F_1 = 0.608\ in$ Point Bearing Settlement

For elastic settlement assume that the point load is equal to zero.

$P(z) := P_{g1} + \left(\frac{P_{g1}}{L_p}\right) \cdot z$

$\delta_E := \int_0^{L_p} \frac{P(z)}{E_p \cdot A_p}\, dz = 0.08\ in$ Elastic Shortening

$\delta_{pile} := \delta_p + \delta_E = 0.688\ in$ Total Pile Settlement

$B_p \cdot 0.03 = 0.772\ in$ Pile settlement should not be greater than 3% of pile diameter

Condition Satisfied

North Foundation Design
(Case 1: Strength I)

Design Calculations for Pile Group

Vertical load capacity of the entire pile group:

$P_s = 160898.831\ lb$ Single pile

$P_p = 624144.936\ lb$ Single pile

$P_{Group} := N \cdot (P_s + P_p) = 50242801.086\ lb$

$s := 2 \cdot B_p = 4.29\ ft$ Minimum Spacing Requirement

$s := 5\ ft$ Pile Spacing

$e := 1\ ft$ Edge Distance

$B_g := (7 \cdot s) + \left(\frac{(B_p)}{2} + e\right) + \left(\frac{(B_p)}{2} + e\right) = 39.145\ ft$ See Drawing for Dimensions

$L_g := (7 \cdot s) + \left(\frac{(B_p)}{2} + e\right) + \left(\frac{(B_p)}{2} + e\right) = 39.145\ ft$

$B_g := 40\ ft$ Simplified Dimension

$L_g := 40\ ft$ Simplified Dimension

$f_s := (f_{s1Bar}) + (f_{s2Bar}) + (f_{s3Bar}) = 875.842\ \frac{lb}{ft^2}$

$P_{gg} := (2 \cdot B_g) + (2 \cdot L_g) = 160\ ft$ Perimeter of Group

$A_g := L_g \cdot B_g = 1600\ ft^2$ Area of Group

$P_{NBlock} := P_{gg} \cdot L_p \cdot f_s + A_g \cdot q_p = 442556544.349\ lb$ Block Failure Capacity

$P_{Ngroup} := min\left(P_{Group}, P_{NBlock}\right) = 50242801.086\ lb$

$FS := 3$ Assumed FS for pile group

$P_{allGroup} := \frac{P_{Ngroup}}{FS} = 16747600.362\ lb$ >/= $P_g = 8232466\ lb$

Condition Satisfied

North Foundation Design
(Case 1: Strength I)

Predicted elastic settlement of the entire pile group:

$x := \frac{2}{3} \cdot L_p = 40\ ft$ Equivalent Footing Depth for Piles in Sand

$D_b := \frac{1}{3} \cdot L_p = 20\ ft$

$D_{bb} := \frac{2}{3} \cdot D_b = 13.333\ ft$ Starting of 4/1 Slope Stress Distribution

$B_{EQ} := B_g + 2 \cdot \left(\frac{D_{bb}}{4}\right) = 46.667\ ft$ $L_{EQ} := L_g + 2 \cdot \left(\frac{D_{bb}}{4}\right) = 46.667\ ft$

$q_{net} := \frac{P_g}{B_{EQ}^{\ 2}} = 3780.214\ \frac{lb}{ft^2}$

Strain Influence Factor Method:

$z_1 := B_{EQ} \cdot \left(0.5 + 0.555 \cdot \left(\frac{L_{EQ}}{B_{EQ}} - 1\right)\right) = 23.333\ ft$ </= $B_{EQ} = 46.667\ ft$ OK

$z_2 := B_{EQ} \cdot \left(2 + 0.222 \cdot \left(\frac{L_{EQ}}{B_{EQ}} - 1\right)\right) = 93.333\ ft$ </= $4 \cdot B_{EQ} = 186.667\ ft$ OK

$I_{z0} := 0.1 + 0.0111 \cdot \left(\frac{L_{EQ}}{B_{EQ}} - 1\right) = 0.1$ </= 0.2 OK

$\sigma_{zp}' := \gamma \cdot D_w + \gamma' \cdot \left(\left(x - D_w\right) + D_{bb} + z_1\right) = 5679.467\ \frac{lb}{ft^2}$ Vertical Effective Stress at z1 (Before Installation)

$I_{zMax} := 0.5 + 0.1 \cdot \sqrt[2]{\frac{q_{net}}{\sigma_{zp}'}} = 0.582$ All sand layer

$\Sigma = I_{zBar1} \cdot \frac{\Delta z_1}{E_s}$

Layer 1

$\Delta z_1 := z_1 = 23.333\ ft$ $E_s = 500\ \frac{ton}{ft^2}$ $I_{zBar1} := \frac{I_{z0} + I_{zMax}}{2} = 0.341$ $\Sigma_1 := 2.4258 \cdot 10^{-6}\ \frac{ft^3}{lb}$

Layer 2

$\Delta z_2 := z_2 - z_1 = 70\ ft$ $E_s = 500\ \frac{ton}{ft^2}$ $I_{zBar2} := \frac{I_{zMax}}{2} = 0.291$ $\Sigma_2 := 6.2068 \cdot 10^{-6}\ \frac{ft^3}{lb}$

$\Sigma := \Sigma_1 + \Sigma_2 = 0.0000086326\ \frac{ft^3}{lb}$

$\sigma_z' := \gamma \cdot D_w + \gamma' \cdot \left(\left(x - D_w\right) + D_{bb}\right) = 4102.133\ \frac{lb}{ft^2}$ Vertical Effective Stress at Equivalent Footing

$C_1 := 1 - 0.5 \cdot \left(\frac{\sigma_z'}{q_{net}}\right) = 0.457$

$t := 50$ Years

$C_2 := 1 + 0.2 \cdot \log\left(\frac{t}{0.1}\right) = 1.54$

$\delta_{pg} := C_1 \cdot C_2 \cdot q_{net} \cdot \Sigma = 0.276\ in$ Elastic Settlement of Pile Group

North Foundation Design
(Case 1: Strength I)

Elastic Shortening:

$P_{top} := P_{g1} = 128632.281\ lb$

$A_T := s^2 = 25\ ft^2$ Tributary Area of Each Pile

$P_{bot} := q_{net} \cdot A_T = 94505.349\ lb$

$L_{p1} := x + D_{bb} = 53.333\ ft$

$$P(z) := P_{top} + \left(\frac{P_{top} - P_{bot}}{L_{p1}} \right) \cdot z$$

$A_p = 60.2\ in^2$

$$\delta_E := \int_0^{L_{p1}} \frac{P(z)}{E_p \cdot A_p}\, dz = 0.053\ in$$ Elastic Shortening

$\delta_{PileGroup} := \delta_{pg} + \delta_E = 0.329\ in$

Satisfies requirement of a max settlement of 1 in.

North Foundation Design
(Case 1: Strength I)

Allowable Lateral Load Calculations (Brom's Method):

Assume Fixed-Head, Long Pile (Tomlinson)

$B_p = 25.739\ in$ $e := 27\ ft$ $S_x := 380\ in^3$ $S_y := 124\ in^3$ $\frac{e}{B_p} = 12.588$

$F_y := 60000\ \frac{lb}{in^2}$ Yield Stress of Pile

N-S Horizontal Load:

$M_Y := S_x \cdot F_y = 1900000\ lb \cdot ft$ Yield Moment N-S

$K_p := \tan\left(45° + \frac{\phi'}{2}\right)^2 = 4.204$

$\frac{M_Y}{K_p \cdot \gamma \cdot {B_p}^4} = 195.904$ Figure 7.12 from Poulous and Davis

$V_u := 150 \cdot K_p \cdot \gamma \cdot {B_p}^3 = 678249.205\ lb$ Ultimate Load

$FS := 1.67$ Factor of Safety for Steel

$V_{all} := \frac{V_u}{FS} = 406137.249\ lb$ >/= $V_y = 369526\ lb$

Condition Satisfied

E-W Horizontal Load:

$M_Y := S_y \cdot F_y = 620000\ lb \cdot ft$ Yield Moment E-W

$K_p := \tan\left(45° + \frac{\phi'}{2}\right)^2 = 4.204$

$\frac{M_Y}{K_p \cdot \gamma \cdot {B_p}^4} = 63.927$ Figure 7.12 from Poulous and Davis

$V_u := 75 \cdot K_p \cdot \gamma \cdot {B_p}^3 = 339124.603\ lb$ Ultimate Load

$FS := 1.67$ Factor of Safety for Steel

$V_{all} := \frac{V_u}{FS} = 203068.624\ lb$ >/= $V_x = 2050\ lb$

Condition Satisfied

North Foundation Design
(Case 1: Strength I)

Pile Buckling Calculations:

See AutoCAD drawing for pile arrangement

N-S Axis Bending (Overturning Moment)

$y_4 := \frac{s}{2} + s + s + s = 17.5\ ft$ $y_3 := \frac{s}{2} + s + s = 12.5\ ft$ $y_2 := \frac{s}{2} + s = 7.5\ ft$ $y_1 := \frac{s}{2} = 2.5\ ft$

$y_5 := y_1$ $y_6 := y_2$ $y_7 := y_3$ $y_8 := y_4$

$\Sigma y_{squared1} := 2 \cdot \left(8 \cdot \left(y_1^{\,2} + y_2^{\,2} + y_3^{\,2} + y_4^{\,2}\right)\right) = 8400\ ft^2$

$P_1 := \frac{P_g}{N} + \frac{M_y \cdot y_1}{\Sigma y_{squared1}} = 128637.884\ lb$ </= $P_{all} = 356838.076\ lb$

$P_2 := \frac{P_g}{N} + \frac{M_y \cdot y_2}{\Sigma y_{squared1}} = 128649.089\ lb$ </= $P_{all} = 356838.076\ lb$

$P_3 := \frac{P_g}{N} + \frac{M_y \cdot y_3}{\Sigma y_{squared1}} = 128660.295\ lb$ </= $P_{all} = 356838.076\ lb$

$P_4 := \frac{P_g}{N} + \frac{M_y \cdot y_4}{\Sigma y_{squared1}} = 128671.5\ lb$ </= $P_{all} = 356838.076\ lb$

$P_5 := \frac{P_g}{N} - \frac{M_y \cdot y_5}{\Sigma y_{squared1}} = 128626.679\ lb$ </= $P_{all} = 356838.076\ lb$

$P_6 := \frac{P_g}{N} - \frac{M_y \cdot y_6}{\Sigma y_{squared1}} = 128615.473\ lb$ </= $P_{all} = 356838.076\ lb$

$P_7 := \frac{P_g}{N} - \frac{M_y \cdot y_7}{\Sigma y_{squared1}} = 128604.268\ lb$ </= $P_{all} = 356838.076\ lb$

$P_8 := \frac{P_g}{N} - \frac{M_y \cdot y_8}{\Sigma y_{squared1}} = 128593.063\ lb$ </= $P_{all} = 356838.076\ lb$

All Piles are able to handle this loading.

E-W Axis Bending (Overturning Moment)

$x_4 := \frac{s}{2} + s + s + s = 17.5\ ft$ $x_3 := \frac{s}{2} + s + s = 12.5\ ft$ $x_2 := \frac{s}{2} + s = 7.5\ ft$ $x_1 := \frac{s}{2} = 2.5\ ft$

$x_5 := x_1$ $x_6 := x_2$ $x_7 := x_3$ $x_8 := x_4$

$\Sigma x_{squared1} := 2 \cdot \left(8 \cdot \left(x_1^{\,2} + x_2^{\,2} + x_3^{\,2} + x_4^{\,2}\right)\right) = 8400\ ft^2$

North Foundation Design
(Case 1: Strength I)

$P_1 := \frac{P_g}{N} + \frac{M_x \cdot y_1}{\Sigma x_{squared1}} = 160581.865\ lb$ </= $P_{all} = 356838.076\ lb$

$P_2 := \frac{P_g}{N} + \frac{M_x \cdot y_2}{\Sigma x_{squared1}} = 224481.031\ lb$ </= $P_{all} = 356838.076\ lb$

$P_3 := \frac{P_g}{N} + \frac{M_x \cdot y_3}{\Sigma x_{squared1}} = 288380.198\ lb$ </= $P_{all} = 356838.076\ lb$

$P_4 := \frac{P_g}{N} + \frac{M_x \cdot y_4}{\Sigma x_{squared1}} = 352279.365\ lb$ </= $P_{all} = 356838.076\ lb$

$P_5 := \frac{P_g}{N} - \frac{M_x \cdot y_5}{\Sigma x_{squared1}} = 96682.698\ lb$ </= $P_{all} = 356838.076\ lb$

$P_6 := \frac{P_g}{N} - \frac{M_x \cdot y_6}{\Sigma x_{squared1}} = 32783.531\ lb$ </= $T_{all} = 60997.221\ lb$

$P_7 := \frac{P_g}{N} - \frac{M_x \cdot y_7}{\Sigma x_{squared1}} = -31115.635\ lb$ </= $T_{all} = 60997.221\ lb$

$P_8 := \frac{P_g}{N} - \frac{M_x \cdot y_8}{\Sigma x_{squared1}} = -95014.802\ lb$ </= $T_{all} = 60997.221\ lb$

All Piles are able to handle this loading.

All piles satisfy axial load capacity

$A := 60.2\ in^2$

$P_d := P_4$ Maximum Axial Load Controls

$\sigma_d := \frac{P_d}{A} = 5851.817\ \frac{lb}{in^2}$

$\sigma_{all} := 0.35 \cdot F_y = 21000\ \frac{lb}{in^2}$ *Standard Guidelines for the Design and Installation of Pile Foundations*, (35% of Yield Stress)

σ_d </= σ_{all}

Criteria is satisfied. Piles will not buckle.

North Foundation Design
(Case 1: Strength I)

Design Summary

All requirements were satisfied for the design of the individual piles and the pile group. The piles are HP 18x204 and 60ft long. The pile cap contains 64 piles that are spaced at 5ft with a typical distribution of an isolated pile cap. Pile axial capacity is analyzed for each individual pile and passes the requirement for axial load. When combining axial load and bending moment the capacity is also satisfied and is shown in the analysis of pile groups portion of the calculations. The pile cap will be a 2ft thick concrete slab, which rests on top of the pile. This cap is a 40' x 40' square block.

North Foundation Design
(Case 2: Strength III)

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North Foundation Design
(Case 2: Strength III)

Foundation Description:

The design of this foundation is for a proposed pedestrian overpass in Waterloo, IA. The foundation in need of attention is the foundations located on the north side of the bridge, and will be supporting two concrete towers. To guard against excessive settlement of the abutments that would be detrimental to bridge operations, the abutments shall be supported by a pile group.

Project Goal:

Determine the most economical design for the pile group.

Variables:

$\gamma := 109 \frac{lb}{ft^3}$ Unit Weight of Soil

$\gamma_{sat} := 130 \frac{lb}{ft^3}$ Saturated Unit Weight of Soil

$\gamma_w := 62.4 \frac{lb}{ft^3}$ Unit Weight of Water

$\gamma' := \gamma_{sat} - \gamma_w = 67.6 \frac{lb}{ft^3}$ Submerged Unit Weight of Soil

$D_w := 12\ ft$ Depth of Water Table

$L_p := 60\ ft$ Length of the Pile

$P_{rxn} := 1372000\ lb$ Reaction Force From Bridge Load

$P_{dead} := 1000000\ lb$ Dead Load of Both Tower

$P_g := P_{rxn} + P_{dead}$ Vertical Load

$V_y := 265800\ lb$ Horizontal Load N-S

$V_x := 190000\ lb$ Horizontal Load E-W

$M_y := 3272000\ lb \cdot ft$ Moment N-S

$M_x := 71630000\ lb \cdot ft$ Moment E-W

Assumptions:

From the soil report from USGS geodata, it shows that from 0-18" the soil is a loamy sand, followed by sand per 18-30", gravelly course sand per 30-55", course sand per 55-70", and gravelly course sand per 70-80". Since the layer that is cohesive only goes down a short distance from soil grade, it may be reasonable to analyze this soil as a purely granular soil. It was also noted from the soils report that the soil surrounding the project site is excessively drained. The preliminary design with be with a HP18x204 steel pile cross section with an individual pile length of 60'.

North Foundation Design
(Case 2: Strength III)

Design Calculations for Individual Piles

For calculating pile point bearing capacity, the reduced rigidity index needs to be assessed. Therefore taking values for the static stress-strain modulus of elasticity from table 5-6 in the Bowles, *Foundation Analysis and Design* was found for the soil at the bottom of the pile.

$E_s := 50\ MPa$ — Assuming course sand that is dense and wet.

$E_s := 1000000\ \dfrac{lb}{ft^2}$ — Conversion to US units.

$\sigma_{z12}' := \gamma \cdot D_w = 1308\ \dfrac{lb}{ft^2}$ — Vertical Effective Stress at Water Table Depth

$\sigma_{z40}' := \gamma \cdot D_w + \left(\gamma' \cdot \left(L_p - D_w\right)\right) = 4552.8\ \dfrac{lb}{ft^2}$ — Vertical Effective Stress at Pile Depth

Using table 2-7 in the Bowles text, the Poisson's ratio for this soil was determined:

$\mu := 0.35$ — Cohesionless, dense sand.

Taking the assumed internal angle of friction for poorly graded sand from Lindeburg, *Civil Engineering Reference for PE 8th Edition*, the critical depth can be found using Figure 3.10 from the Poulos and Davis text:

$\phi' := 38\ °$

$\phi := \dfrac{3}{4} \cdot \phi' + 10\ ° = 38.5\ °$

Zc / d ratio from Poulos & Davis, *Pile Foundation Analysis & Design*, Figure 3.10: = 10.5

$b_f := 18.1\ in$ $\quad d := 18.3\ in$ — HP18x204 steel pile.

$B_p := \sqrt[2]{\left(d^2\right) + \left(b_f^{\,2}\right)} = 25.739\ in$ — Width of pile is diagonal

$A_p := b_f \cdot d = 331.23\ in^2$ — Including Soil Plug

$z_c := 10.5 \cdot B_p = 22.522\ ft$ — Critical Depth

$\sigma_p'' := D_w \cdot \gamma + \left(\left(z_c - D_w\right) \cdot \gamma'\right) = 2019.266\ \dfrac{lb}{ft^2}$ — Vertical Effective Stress at Critical Depth

North Foundation Design
(Case 2: Strength III)

Calculation of bearing capacity factors ensues:

$G_s := \frac{E_s}{2 \cdot (1+\mu)} = 370370.37 \ \frac{lb}{ft^2}$ $\quad c' := 0$ Cohesionless sand.

$I_r := \frac{G_s}{c' + \sigma_{z40}' \cdot \tan(\phi')} = 104.123$ Rigidity Index

Bowles, *Foundation Analysis & Design*, Table on P.894 : Rigidity Index Within Range for Sandy Soil

$\varepsilon_v := \frac{(1+\mu) \cdot (1-2 \cdot \mu) \cdot (\sigma_{z40}')}{E_s \cdot (1-\mu)} = 0.003$ Volumetric Strain

$I_{rr} := \frac{I_r}{1+\varepsilon_v \cdot I_r} = 80.381$ Reduced Rigidity Index

$K_o := 1 - \sin(\phi') = 0.384$ At rest earth pressure coefficient

Vesic's Bearing Capacity Factors:

$\eta := \frac{1+2 \cdot K_o}{3} = 0.59$

$N_q := \frac{3}{3-\sin(\phi')} \cdot \exp\left(\left(\frac{\pi}{2} - \phi'\right) \tan(\phi')\right) \cdot \left(\tan\left(\frac{\pi}{4} + \frac{\phi'}{2}\right)\right)^2 \cdot I_{rr}^{\left(\frac{4 \cdot \sin(\phi')}{3 \cdot (1+\sin(\phi'))}\right)} = 99.84$

$N_\gamma := 0.6 \cdot (N_q - 1) \tan(\phi') = 46.333$

$N_{c1} := (N_q - 1) \cot(\phi') = 126.509$

$N_{c2} := \frac{4}{3} \cdot (\ln(I_{rr}) + 1) + \frac{\pi}{2} + 1 = 9.753$

$N_c := \text{if}(\phi' = 0, N_{c2}, N_{c1}) = 126.509$

$q_p := \eta \cdot \sigma_{z40}' \cdot N_q + \frac{1}{2} \cdot \gamma' \cdot B_p \cdot N_\gamma = 271342.785 \ \frac{lb}{ft^2}$ Pile Point Bearing Capacity

$P_p := A_p \cdot q_p = 624144.936 \ lb$ Pile Point Load Capacity

North Foundation Design
(Case 2: Strength III)

Calculation of Side Friction Capacity:

α - Method calculations will be used to calculate the pile side friction capacity.

From Tomlinson, *Pile Design and Construction Practice*, Table 4.10, Large displacement pile due to Steel HP shape. Range is based on soil density, therefore with our dense sand assumption:

$$\frac{K_s}{K_o} = 1.25$$

$K_s := K_o \cdot 1.25 = 0.48$ Lateral Earth-Pressure Coefficient for Side Friction

From Tomlinson, Table 4.11, Smooth Steel Pile at interface with sand:

$\delta_p := 0.6 \cdot \phi' = 22.8\ °$ Angle of Friction Between Pile and Soil

$f_s := K_s \cdot \sigma_p'' \cdot \tan\left(\delta_p\right) = 407.793\ \frac{lb}{ft^2}$ Side Friction Stress, Considering Critical Depth

Calculation of side friction stress profile:

$z_1 := D_w$

$\sigma_{z1} := D_w \cdot \gamma = 1308\ \frac{lb}{ft^2}$

$f_{s12} := K_s \cdot \sigma_{z1} \cdot \tan\left(\delta_p\right) = 264.152\ \frac{lb}{ft^2}$

$z_c = 22.522\ ft$

Layers	Avg. Side Friction Stress
0-12 ft	$f_{s1Bar} := \frac{1}{2} \cdot \left(0 + f_{s12}\right) = 132.076\ \frac{lb}{ft^2}$
12-22.5 ft	$f_{s2Bar} := \frac{1}{2} \cdot \left(f_{s12} + f_s\right) = 335.973\ \frac{lb}{ft^2}$
22.5-75 ft	$f_{s3Bar} := f_s = 407.793\ \frac{lb}{ft^2}$

$P_{p1} := b_f + b_f + d + d = 6.067\ ft$ Perimeter of Pile (Including Soil Plug)

$P_s := P_{p1} \cdot \left(12\ ft \cdot \left(f_{s1Bar}\right) + 10.5\ ft \cdot \left(f_{s2Bar}\right) + 52.5\ ft \cdot \left(f_{s3Bar}\right)\right) = 160898.831\ lb$ Side Friction Capacity

North Foundation Design
(Case 2: Strength III)

Calculations for compression and tension load capacity:

$L_p = 60\ ft$ — Length of Pile

$W_p := 204\ \frac{lb}{ft} \cdot L_p = 12240\ lb$ — Nominal Weight of pile type multiplied by pile length

From the *Standard Guidelines for the Design and Installation of Pile Foundations*, the factors of safety were determined from table A.1 and A.2. Assuming a pile group consisting of 64 piles, the design axial load will be distributed throughout these 64 piles.

$N := 64$ — Number of Piles

$P_{g1} := \frac{P_g}{N} = 18.531\ ton$

$F_1 := 2.0$ — Table A.1, Since this is a preliminary design, using driving formulas and static analysis to determine factors of safety

$F_2 := 1.1$ — Table A.2, HP Pile

$FS := F_1 \cdot F_2 = 2.2$

$P := P_{g1} + W_p = 49302.5\ lb$ </= $P_{all} := \frac{P_p + P_s}{FS} = 356838.076\ lb$

Condition Satisfied

For uplift, the factor of safety is approximately a 50% increase from the compression capacity factor of safety.

$FS_T := 1.5 \cdot FS = 3.3$ — Factor of Safety for Uplift

$T_{all} := \frac{P_s}{FS_T} + W_p = 60997.221\ lb$ — Tension Capacity

Each individual pile satisfies this condition for both combined axial load and bending moment. This design criteria is calculated when calculating to see if the piles will buckle in the analysis of pile groups. This value is also shown in the summary.

North Foundation Design
(Case 2: Strength III)

Pile settlement for an individual pile is as follows (Bowles Method):

$A_p := 60.2\ in^2$ AISC Table 1-4: HP 18x201

$E_p := 29000000\ \frac{lb}{in^2}$

$m := 1$ Shape Factor, m*Is = 1.0 (Relatively square pile)

$I_s := 1$

$\frac{L_p}{B_p} = 27.973$ Fox Embedment Factor Inequality Satisfaction

$I_F := 0.35$ Fox Embedment Factor

$F_1 := 0.25$ Reduction Factor, High side friction capacity compared to design load for an individual pile.

$q := \frac{P_{g1}}{A_p} = 615.656\ \frac{lb}{in^2}$

$\delta_p := q \cdot B_p \cdot \left(\frac{1-\mu^2}{E_s}\right) \cdot m \cdot I_s \cdot I_F \cdot F_1 = 0.175\ in$ Point Bearing Settlement

For elastic settlement assume that the point load is equal to zero.

$P(z) := P_{g1} + \left(\frac{P_{g1}}{L_p}\right) \cdot z$

$\delta_E := \int_0^{L_p} \frac{P(z)}{E_p \cdot A_p}\, dz = 0.023\ in$ Elastic Shortening

$\delta_{pile} := \delta_p + \delta_E = 0.198\ in$ Total Pile Settlement

$B_p \cdot 0.03 = 0.772\ in$ Pile settlement should not be greater than 3% of pile diameter

Condition Satisfied

North Foundation Design
(Case 2: Strength III)

Design Calculations for Pile Group

Vertical load capacity of the entire pile group:

$P_s = 160898.831\ lb$ Single pile

$P_p = 624144.936\ lb$ Single pile

$P_{Group} := N \cdot (P_s + P_p) = 50242801.086\ lb$

$s := 2 \cdot B_p = 4.29\ ft$ Minimum Spacing Requirement

$s := 5\ ft$ Pile Spacing

$e := 1\ ft$ Edge Distance

$B_g := (7 \cdot s) + \left(\frac{(B_p)}{2} + e\right) + \left(\frac{(B_p)}{2} + e\right) = 39.145\ ft$ See Drawing for Dimensions

$L_g := (7 \cdot s) + \left(\frac{(B_p)}{2} + e\right) + \left(\frac{(B_p)}{2} + e\right) = 39.145\ ft$

$B_g := 40\ ft$ Simplified Dimension

$L_g := 40\ ft$ Simplified Dimension

$f_s := (f_{s1Bar}) + (f_{s2Bar}) + (f_{s3Bar}) = 875.842\ \frac{lb}{ft^2}$

$P_{gg} := (2 \cdot B_g) + (2 \cdot L_g) = 160\ ft$ Perimeter of Group

$A_g := L_g \cdot B_g = 1600\ ft^2$ Area of Group

$P_{NBlock} := P_{gg} \cdot L_p \cdot f_s + A_g \cdot q_p = 442556544.349\ lb$ Block Failure Capacity

$P_{Ngroup} := min\left(P_{Group}, P_{NBlock}\right) = 50242801.086\ lb$

$FS := 3$ Assumed FS for pile group

$P_{allGroup} := \frac{P_{Ngroup}}{FS} = 16747600.362\ lb$ >/= $P_g = 2372000\ lb$

Condition Satisfied

North Foundation Design
(Case 2: Strength III)

<u>Predicted elastic settlement of the entire pile group:</u>

$x := \frac{2}{3} \cdot L_p = 40\ ft$ Equivalent Footing Depth for Piles in Sand

$D_b := \frac{1}{3} \cdot L_p = 20\ ft$

$D_{bb} := \frac{2}{3} \cdot D_b = 13.333\ ft$ Starting of 4/1 Slope Stress Distribution

$B_{EQ} := B_g + 2 \cdot \left(\frac{D_{bb}}{4}\right) = 46.667\ ft$ $L_{EQ} := L_g + 2 \cdot \left(\frac{D_{bb}}{4}\right) = 46.667\ ft$

$q_{net} := \frac{P_g}{{B_{EQ}}^2} = 1089.184\ \frac{lb}{ft^2}$

Strain Influence Factor Method:

$z_1 := B_{EQ} \cdot \left(0.5 + 0.555 \cdot \left(\frac{L_{EQ}}{B_{EQ}} - 1\right)\right) = 23.333\ ft$ </= $B_{EQ} = 46.667\ ft$ OK

$z_2 := B_{EQ} \cdot \left(2 + 0.222 \cdot \left(\frac{L_{EQ}}{B_{EQ}} - 1\right)\right) = 93.333\ ft$ </= $4 \cdot B_{EQ} = 186.667\ ft$ OK

$I_{z0} := 0.1 + 0.0111 \cdot \left(\frac{L_{EQ}}{B_{EQ}} - 1\right) = 0.1$ </= 0.2 OK

$\sigma_{zp}' := \gamma \cdot D_w + \gamma' \cdot \left(\left(x - D_w\right) + D_{bb} + z_1\right) = 5679.467\ \frac{lb}{ft^2}$ Vertical Effective Stress at z1 (Before Installation)

$I_{zMax} := 0.5 + 0.1 \cdot \sqrt[2]{\frac{q_{net}}{\sigma_{zp}'}} = 0.544$ All sand layer

$\Sigma = I_{zBar1} \cdot \frac{\Delta z_1}{E_s}$

<u>Layer 1</u>

$\Delta z_1 := z_1 = 23.333\ ft$ $E_s = 500\ \frac{ton}{ft^2}$ $I_{zBar1} := \frac{I_{z0} + I_{zMax}}{2} = 0.322$ $\Sigma_1 := 2.4258 \cdot 10^{-6}\ \frac{ft^3}{lb}$

<u>Layer 2</u>

$\Delta z_2 := z_2 - z_1 = 70\ ft$ $E_s = 500\ \frac{ton}{ft^2}$ $I_{zBar2} := \frac{I_{zMax}}{2} = 0.272$ $\Sigma_2 := 6.2068 \cdot 10^{-6}\ \frac{ft^3}{lb}$

$\Sigma := \Sigma_1 + \Sigma_2 = 0.0000086326\ \frac{ft^3}{lb}$

$\sigma_z' := \gamma \cdot D_w + \gamma' \cdot \left(\left(x - D_w\right) + D_{bb}\right) = 4102.133\ \frac{lb}{ft^2}$ Vertical Effective Stress at Equivalent Footing

$C_1 := 1 - 0.5 \cdot \left(\frac{\sigma_z'}{q_{net}}\right) = -0.883$

$t := 50$ Years

$C_2 := 1 + 0.2 \cdot \log\left(\frac{t}{0.1}\right) = 1.54$

$\delta_{pg} := C_1 \cdot C_2 \cdot q_{net} \cdot \Sigma = -0.153\ in$ Elastic Settlement of Pile Group

North Foundation Design
(Case 2: Strength III)

Elastic Shortening:

$P_{top} := P_{g1} = 37062.5\ lb$

$A_T := s^2 = 25\ ft^2$ Tributary Area of Each Pile

$P_{bot} := q_{net} \cdot A_T = 27229.592\ lb$

$L_{p1} := x + D_{bb} = 53.333\ ft$

$P(z) := P_{top} + \left(\frac{P_{top} - P_{bot}}{L_{p1}}\right) \cdot z$

$A_p = 60.2\ in^2$

$\delta_E := \int_0^{L_{p1}} \frac{P(z)}{E_p \cdot A_p}\, dz = 0.015\ in$ Elastic Shortening

$\delta_{PileGroup} := \delta_{pg} + \delta_E = -0.138\ in$

Satisfies requirement of a max settlement of 1 in.

North Foundation Design
(Case 2: Strength III)

<u>Allowable Lateral Load Calculations (Brom's Method):</u>

Assume Fixed-Head, Long Pile (Tomlinson)

$B_p = 25.739\ in$ $e := 27\ ft$ $S_x := 380\ in^3$ $S_y := 124\ in^3$ $\frac{e}{B_p} = 12.588$

$F_y := 60000\ \frac{lb}{in^2}$ Yield Stress of Pile

<u>N-S Horizontal Load:</u>

$M_Y := S_x \cdot F_y = 1900000\ lb \cdot ft$ Yield Moment N-S

$K_p := \tan\left(45° + \frac{\phi'}{2}\right)^2 = 4.204$

$\frac{M_Y}{K_p \cdot \gamma \cdot {B_p}^4} = 195.904$ Figure 7.12 from Poulous and Davis

$V_u := 150 \cdot K_p \cdot \gamma \cdot {B_p}^3 = 678249.205\ lb$ Ultimate Load

$FS := 1.67$ Factor of Safety for Steel

$V_{all} := \frac{V_u}{FS} = 406137.249\ lb$ >/= $V_y = 265800\ lb$

Condition Satisfied

<u>E-W Horizontal Load:</u>

$M_Y := S_y \cdot F_y = 620000\ lb \cdot ft$ Yield Moment E-W

$K_p := \tan\left(45° + \frac{\phi'}{2}\right)^2 = 4.204$

$\frac{M_Y}{K_p \cdot \gamma \cdot {B_p}^4} = 63.927$ Figure 7.12 from Poulous and Davis

$V_u := 75 \cdot K_p \cdot \gamma \cdot {B_p}^3 = 339124.603\ lb$ Ultimate Load

$FS := 1.67$ Factor of Safety for Steel

$V_{all} := \frac{V_u}{FS} = 203068.624\ lb$ >/= $V_x = 190000\ lb$

Condition Satisfied

North Foundation Design
(Case 2: Strength III)

Pile Buckling Calculations:

See AutoCAD drawing for pile arrangement

N-S Axis Bending (Overturning Moment)

$y_4 := \frac{s}{2} + s + s + s = 17.5\ ft$ $y_3 := \frac{s}{2} + s + s = 12.5\ ft$ $y_2 := \frac{s}{2} + s = 7.5\ ft$ $y_1 := \frac{s}{2} = 2.5\ ft$

$y_5 := y_1$ $y_6 := y_2$ $y_7 := y_3$ $y_8 := y_4$

$\Sigma y_{squared1} := 2 \cdot \left(8 \cdot \left(y_1{}^2 + y_2{}^2 + y_3{}^2 + y_4{}^2\right)\right) = 8400\ ft^2$

$P_1 := \frac{P_g}{N} + \frac{M_y \cdot y_1}{\Sigma y_{squared1}} = 38036.31\ lb$ </= $P_{all} = 356838.076\ lb$

$P_2 := \frac{P_g}{N} + \frac{M_y \cdot y_2}{\Sigma y_{squared1}} = 39983.929\ lb$ </= $P_{all} = 356838.076\ lb$

$P_3 := \frac{P_g}{N} + \frac{M_y \cdot y_3}{\Sigma y_{squared1}} = 41931.548\ lb$ </= $P_{all} = 356838.076\ lb$

$P_4 := \frac{P_g}{N} + \frac{M_y \cdot y_4}{\Sigma y_{squared1}} = 43879.167\ lb$ </= $P_{all} = 356838.076\ lb$

$P_5 := \frac{P_g}{N} - \frac{M_y \cdot y_5}{\Sigma y_{squared1}} = 36088.69\ lb$ </= $P_{all} = 356838.076\ lb$

$P_6 := \frac{P_g}{N} - \frac{M_y \cdot y_6}{\Sigma y_{squared1}} = 34141.071\ lb$ </= $P_{all} = 356838.076\ lb$

$P_7 := \frac{P_g}{N} - \frac{M_y \cdot y_7}{\Sigma y_{squared1}} = 32193.452\ lb$ </= $P_{all} = 356838.076\ lb$

$P_8 := \frac{P_g}{N} - \frac{M_y \cdot y_8}{\Sigma y_{squared1}} = 30245.833\ lb$ </= $P_{all} = 356838.076\ lb$

All Piles are able to handle this loading.

E-W Axis Bending (Overturning Moment)

$x_4 := \frac{s}{2} + s + s + s = 17.5\ ft$ $x_3 := \frac{s}{2} + s + s = 12.5\ ft$ $x_2 := \frac{s}{2} + s = 7.5\ ft$ $x_1 := \frac{s}{2} = 2.5\ ft$

$x_5 := x_1$ $x_6 := x_2$ $x_7 := x_3$ $x_8 := x_4$

$\Sigma x_{squared1} := 2 \cdot \left(8 \cdot \left(x_1{}^2 + x_2{}^2 + x_3{}^2 + x_4{}^2\right)\right) = 8400\ ft^2$

North Foundation Design
(Case 2: Strength III)

$P_1 := \frac{P_g}{N} + \frac{M_x \cdot y_1}{\Sigma x_{squared1}} = 58380.952\ lb$ </= $P_{all} = 356838.076\ lb$

$P_2 := \frac{P_g}{N} + \frac{M_x \cdot y_2}{\Sigma x_{squared1}} = 101017.857\ lb$ </= $P_{all} = 356838.076\ lb$

$P_3 := \frac{P_g}{N} + \frac{M_x \cdot y_3}{\Sigma x_{squared1}} = 143654.762\ lb$ </= $P_{all} = 356838.076\ lb$

$P_4 := \frac{P_g}{N} + \frac{M_x \cdot y_4}{\Sigma x_{squared1}} = 186291.667\ lb$ </= $P_{all} = 356838.076\ lb$

$P_5 := \frac{P_g}{N} - \frac{M_x \cdot y_5}{\Sigma x_{squared1}} = 15744.048\ lb$ </= $P_{all} = 356838.076\ lb$

$P_6 := \frac{P_g}{N} - \frac{M_x \cdot y_6}{\Sigma x_{squared1}} = -26892.857\ lb$ </= $T_{all} = 60997.221\ lb$

$P_7 := \frac{P_g}{N} - \frac{M_x \cdot y_7}{\Sigma x_{squared1}} = -69529.762\ lb$ </= $T_{all} = 60997.221\ lb$

$P_8 := \frac{P_g}{N} - \frac{M_x \cdot y_8}{\Sigma x_{squared1}} = -112166.667\ lb$ </= $T_{all} = 60997.221\ lb$

All Piles are able to handle this loading.

All piles satisfy axial load capacity

$A := 60.2\ in^2$

$P_d := P_4$ Maximum Axial Load Controls

$\sigma_d := \frac{P_d}{A} = 3094.546\ \frac{lb}{in^2}$

$\sigma_{all} := 0.35 \cdot F_y = 21000\ \frac{lb}{in^2}$ *Standard Guidelines for the Design and Installation of Pile Foundations*, (35% of Yield Stress)

σ_d </= σ_{all}

Criteria is satisfied. Piles will not buckle.

North Foundation Design
(Case 2: Strength III)

Design Summary

All requirements were satisfied for the design of the individual piles and the pile group. The piles are HP 18x204 and 60ft long. The pile cap contains 64 piles that are spaced at 5ft with a typical distribution of an isolated pile cap. Pile axial capacity is analyzed for each individual pile and passes the requirement for axial load. When combining axial load and bending moment the capacity is also satisfied and is shown in the analysis of pile groups portion of the calculations. The pile cap will be a 2ft thick concrete slab, which rests on top of the pile. This cap is a 40' x 40' square block.

North Foundation Design
(Case 3: Service I)

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North Foundation Design
(Case 3: Service I)

Foundation Description:

The design of this foundation is for a proposed pedestrian overpass in Waterloo, IA. The foundation in need of attention is the foundations located on the north side of the bridge, and will be supporting two concrete towers. To guard against excessive settlement of the abutments that would be detrimental to bridge operations, the abutments shall be supported by a pile group.

Project Goal:

Determine the most economical design for the pile group.

Variables:

$\gamma := 109 \frac{lb}{ft^3}$	Unit Weight of Soil
$\gamma_{sat} := 130 \frac{lb}{ft^3}$	Saturated Unit Weight of Soil
$\gamma_w := 62.4 \frac{lb}{ft^3}$	Unit Weight of Water
$\gamma' := \gamma_{sat} - \gamma_w = 67.6 \frac{lb}{ft^3}$	Submerged Unit Weight of Soil
$D_w := 12\ ft$	Depth of Water Table
$L_p := 60\ ft$	Length of the Pile
$P_{rxn} := 4503842\ lb$	Reaction Force From Bridge Load
$P_{dead} := 1000000\ lb$	Dead Load of Both Tower
$P_g := P_{rxn} + P_{dead}$	Vertical Load
$V_y := 271922\ lb$	Horizontal Load N-S
$V_x := 39514\ lb$	Horizontal Load E-W
$M_y := 691440\ lb \cdot ft$	Moment N-S
$M_x := 77715800\ lb \cdot ft$	Moment E-W

Assumptions:
From the soil report from USGS geodata, it shows that from 0-18" the soil is a loamy sand, followed by sand per 18-30", gravelly course sand per 30-55", course sand per 55-70", and gravelly course sand per 70-80". Since the layer that is cohesive only goes down a short distance from soil grade, it may be reasonable to analyze this soil as a purely granular soil. It was also noted from the soils report that the soil surrounding the project site is excessively drained. The preliminary design with be with a HP18x204 steel pile cross section with an individual pile length of 60'.

North Foundation Design
(Case 3: Service I)

Design Calculations for Individual Piles

For calculating pile point bearing capacity, the reduced rigidity index needs to be assessed. Therefore taking values for the static stress-strain modulus of elasticity from table 5-6 in the Bowles, *Foundation Analysis and Design* was found for the soil at the bottom of the pile.

$E_s := 50\ MPa$ Assuming course sand that is dense and wet.

$E_s := 1000000\ \frac{lb}{ft^2}$ Conversion to US units.

$\sigma_{z12}' := \gamma \cdot D_w = 1308\ \frac{lb}{ft^2}$ Vertical Effective Stress at Water Table Depth

$\sigma_{z40}' := \gamma \cdot D_w + \left(\gamma' \cdot \left(L_p - D_w\right)\right) = 4552.8\ \frac{lb}{ft^2}$ Vertical Effective Stress at Pile Depth

Using table 2-7 in the Bowles text, the Poisson's ratio for this soil was determined:

$\mu := 0.35$ Cohesionless, dense sand.

Taking the assumed internal angle of friction for poorly graded sand from Lindeburg, *Civil Engineering Reference for PE 8th Edition*, the critical depth can be found using Figure 3.10 from the Poulos and Davis text:

$\phi' := 38\ °$

$\phi := \frac{3}{4} \cdot \phi' + 10\ ° = 38.5\ °$

Zc / d ratio from Poulos & Davis, *Pile Foundation Analysis & Design*, Figure 3.10: = 10.5

$b_f := 18.1\ in$ $d := 18.3\ in$ HP18x204 steel pile.

$B_p := \sqrt[2]{\left(d^2\right) + \left(b_f^{\ 2}\right)} = 25.739\ in$ Width of pile is diagonal

$A_p := b_f \cdot d = 331.23\ in^2$ Including Soil Plug

$z_c := 10.5 \cdot B_p = 22.522\ ft$ Critical Depth

$\sigma_p'' := D_w \cdot \gamma + \left(\left(z_c - D_w\right) \cdot \gamma'\right) = 2019.266\ \frac{lb}{ft^2}$ Vertical Effective Stress at Critical Depth

North Foundation Design
(Case 3: Service I)

Calculation of bearing capacity factors ensues:

$G_s := \frac{E_s}{2 \cdot (1+\mu)} = 370370.37 \frac{lb}{ft^2}$ $\quad c' := 0$ Cohesionless sand.

$I_r := \frac{G_s}{c' + \sigma_{z40}' \cdot \tan(\phi')} = 104.123$ Rigidity Index

Bowles, *Foundation Analysis & Design*, Table on P.894 : Rigidity Index Within Range for Sandy Soil

$\varepsilon_v := \frac{(1+\mu) \cdot (1 - 2 \cdot \mu) \cdot (\sigma_{z40}')}{E_s \cdot (1-\mu)} = 0.003$ Volumetric Strain

$I_{rr} := \frac{I_r}{1 + \varepsilon_v \cdot I_r} = 80.381$ Reduced Rigidity Index

$K_o := 1 - \sin(\phi') = 0.384$ At rest earth pressure coefficient

Vesic's Bearing Capacity Factors:

$\eta := \frac{1 + 2 \cdot K_o}{3} = 0.59$

$N_q := \frac{3}{3 - \sin(\phi')} \cdot \exp\left(\left(\frac{\pi}{2} - \phi'\right) \tan(\phi')\right) \cdot \left(\tan\left(\frac{\pi}{4} + \frac{\phi'}{2}\right)\right)^2 \cdot I_{rr}^{\left(\frac{4 \cdot \sin(\phi')}{3 \cdot (1 + \sin(\phi'))}\right)} = 99.84$

$N_\gamma := 0.6 \cdot (N_q - 1) \tan(\phi') = 46.333$

$N_{c1} := (N_q - 1) \cot(\phi') = 126.509$

$N_{c2} := \frac{4}{3} \cdot (\ln(I_{rr}) + 1) + \frac{\pi}{2} + 1 = 9.753$

$N_c := \text{if}(\phi' = 0, N_{c2}, N_{c1}) = 126.509$

$q_p := \eta \cdot \sigma_{z40}' \cdot N_q + \frac{1}{2} \cdot \gamma' \cdot B_p \cdot N_\gamma = 271342.785 \frac{lb}{ft^2}$ Pile Point Bearing Capacity

$P_p := A_p \cdot q_p = 624144.936\ lb$ Pile Point Load Capacity

North Foundation Design
(Case 3: Service I)

Calculation of Side Friction Capacity:

α - Method calculations will be used to calculate the pile side friction capacity.

From Tomlinson, *Pile Design and Construction Practice*, Table 4.10, Large displacement pile due to Steel HP shape. Range is based on soil density, therefore with our dense sand assumption:

$$\frac{K_s}{K_o} = 1.25$$

$K_s := K_o \cdot 1.25 = 0.48$ Lateral Earth-Pressure Coefficient for Side Friction

From Tomlinson, Table 4.11, Smooth Steel Pile at interface with sand:

$\delta_p := 0.6 \cdot \phi' = 22.8\ °$ Angle of Friction Between Pile and Soil

$f_s := K_s \cdot \sigma_p'' \cdot \tan\left(\delta_p\right) = 407.793\ \frac{lb}{ft^2}$ Side Friction Stress, Considering Critical Depth

Calculation of side friction stress profile:

$z_1 := D_w$

$\sigma_{z1} := D_w \cdot \gamma = 1308\ \frac{lb}{ft^2}$

$f_{s12} := K_s \cdot \sigma_{z1} \cdot \tan\left(\delta_p\right) = 264.152\ \frac{lb}{ft^2}$

$z_c = 22.522\ ft$

Layers	Avg. Side Friction Stress
0-12 ft	$f_{s1Bar} := \frac{1}{2} \cdot \left(0 + f_{s12}\right) = 132.076\ \frac{lb}{ft^2}$
12-22.5 ft	$f_{s2Bar} := \frac{1}{2} \cdot \left(f_{s12} + f_s\right) = 335.973\ \frac{lb}{ft^2}$
22.5-75 ft	$f_{s3Bar} := f_s = 407.793\ \frac{lb}{ft^2}$

$P_{p1} := b_f + b_f + d + d = 6.067\ ft$ Perimeter of Pile (Including Soil Plug)

$P_s := P_{p1} \cdot \left(12\ ft \cdot \left(f_{s1Bar}\right) + 10.5\ ft \cdot \left(f_{s2Bar}\right) + 52.5\ ft \cdot \left(f_{s3Bar}\right)\right) = 160898.831\ lb$ Side Friction Capacity

North Foundation Design
(Case 3: Service I)

Calculations for compression and tension load capacity:

$L_p = 60\ ft$ Length of Pile

$W_p := 204\ \frac{lb}{ft} \cdot L_p = 12240\ lb$ Nominal Weight of pile type multiplied by pile length

From the *Standard Guidelines for the Design and Installation of Pile Foundations*, the factors of safety were determined from table A.1 and A.2. Assuming a pile group consisting of 64 piles, the design axial load will be distributed throughout these 64 piles.

$N := 64$ Number of Piles

$P_{g1} := \frac{P_g}{N} = 42.999\ ton$

$F_1 := 2.0$ Table A.1, Since this is a preliminary design, using driving formulas and static analysis to determine factors of safety

$F_2 := 1.1$ Table A.2, HP Pile

$FS := F_1 \cdot F_2 = 2.2$

$P := P_{g1} + W_p = 98237.531\ lb$ </= $P_{all} := \frac{P_p + P_s}{FS} = 356838.076\ lb$

Condition Satisfied

For uplift, the factor of safety is approximately a 50% increase from the compression capacity factor of safety.

$FS_T := 1.5 \cdot FS = 3.3$ Factor of Safety for Uplift

$T_{all} := \frac{P_s}{FS_T} + W_p = 60997.221\ lb$ Tension Capacity

Each individual pile satisfies this condition for both combined axial load and bending moment. This design criteria is calculated when calculating to see if the piles will buckle in the analysis of pile groups. This value is also shown in the summary.

North Foundation Design
(Case 3: Service I)

Pile settlement for an individual pile is as follows (Bowles Method):

$A_p := 60.2\ in^2$ AISC Table 1-4: HP 18x201

$E_p := 29000000\ \frac{lb}{in^2}$

$m := 1$ Shape Factor, m*Is = 1.0 (Relatively square pile)

$I_s := 1$

$\frac{L_p}{B_p} = 27.973$ Fox Embedment Factor Inequality Satisfaction

$I_F := 0.35$ Fox Embedment Factor

$F_1 := 0.25$ Reduction Factor, High side friction capacity compared to design load for an individual pile.

$q := \frac{P_{g1}}{A_p} = 1428.53\ \frac{lb}{in^2}$

$\delta_p := q \cdot B_p \cdot \left(\frac{1-\mu^2}{E_s}\right) \cdot m \cdot I_s \cdot I_F \cdot F_1 = 0.407\ in$ Point Bearing Settlement

For elastic settlement assume that the point load is equal to zero.

$P(z) := P_{g1} + \left(\frac{P_{g1}}{L_p}\right) \cdot z$

$\delta_E := \int_0^{L_p} \frac{P(z)}{E_p \cdot A_p}\, dz = 0.053\ in$ Elastic Shortening

$\delta_{pile} := \delta_p + \delta_E = 0.46\ in$ Total Pile Settlement

$B_p \cdot 0.03 = 0.772\ in$ Pile settlement should not be greater than 3% of pile diameter

Condition Satisfied

North Foundation Design
(Case 3: Service I)

Design Calculations for Pile Group

<u>Vertical load capacity of the entire pile group:</u>

$P_s = 160898.831\ lb$ Single pile

$P_p = 624144.936\ lb$ Single pile

$P_{Group} := N \cdot (P_s + P_p) = 50242801.086\ lb$

$s := 2 \cdot B_p = 4.29\ ft$ Minimum Spacing Requirement

$s := 5\ ft$ Pile Spacing

$e := 1\ ft$ Edge Distance

$B_g := (7 \cdot s) + \left(\frac{(B_p)}{2} + e\right) + \left(\frac{(B_p)}{2} + e\right) = 39.145\ ft$ See Drawing for Dimensions

$L_g := (7 \cdot s) + \left(\frac{(B_p)}{2} + e\right) + \left(\frac{(B_p)}{2} + e\right) = 39.145\ ft$

$B_g := 40\ ft$ Simplified Dimension

$L_g := 40\ ft$ Simplified Dimension

$f_s := (f_{s1Bar}) + (f_{s2Bar}) + (f_{s3Bar}) = 875.842\ \frac{lb}{ft^2}$

$P_{gg} := (2 \cdot B_g) + (2 \cdot L_g) = 160\ ft$ Perimeter of Group

$A_g := L_g \cdot B_g = 1600\ ft^2$ Area of Group

$P_{NBlock} := P_{gg} \cdot L_p \cdot f_s + A_g \cdot q_p = 442556544.349\ lb$ Block Failure Capacity

$P_{Ngroup} := min\left(P_{Group}, P_{NBlock}\right) = 50242801.086\ lb$

$FS := 3$ Assumed FS for pile group

$P_{allGroup} := \frac{P_{Ngroup}}{FS} = 16747600.362\ lb$ >/= $P_g = 5503842\ lb$

Condition Satisfied

North Foundation Design
(Case 3: Service I)

<u>Predicted elastic settlement of the entire pile group:</u>

$x := \frac{2}{3} \cdot L_p = 40\ ft$ Equivalent Footing Depth for Piles in Sand

$D_b := \frac{1}{3} \cdot L_p = 20\ ft$

$D_{bb} := \frac{2}{3} \cdot D_b = 13.333\ ft$ Starting of 4/1 Slope Stress Distribution

$B_{EQ} := B_g + 2 \cdot \left(\frac{D_{bb}}{4}\right) = 46.667\ ft$ $L_{EQ} := L_g + 2 \cdot \left(\frac{D_{bb}}{4}\right) = 46.667\ ft$

$q_{net} := \frac{P_g}{{B_{EQ}}^2} = 2527.274\ \frac{lb}{ft^2}$

Strain Influence Factor Method:

$z_1 := B_{EQ} \cdot \left(0.5 + 0.555 \cdot \left(\frac{L_{EQ}}{B_{EQ}} - 1\right)\right) = 23.333\ ft$ </= $B_{EQ} = 46.667\ ft$ OK

$z_2 := B_{EQ} \cdot \left(2 + 0.222 \cdot \left(\frac{L_{EQ}}{B_{EQ}} - 1\right)\right) = 93.333\ ft$ </= $4 \cdot B_{EQ} = 186.667\ ft$ OK

$I_{z0} := 0.1 + 0.0111 \cdot \left(\frac{L_{EQ}}{B_{EQ}} - 1\right) = 0.1$ </= 0.2 OK

$\sigma_{zp}' := \gamma \cdot D_w + \gamma' \cdot \left(\left(x - D_w\right) + D_{bb} + z_1\right) = 5679.467\ \frac{lb}{ft^2}$ Vertical Effective Stress at z1 (Before Installation)

$I_{zMax} := 0.5 + 0.1 \cdot \sqrt[2]{\frac{q_{net}}{\sigma_{zp}'}} = 0.567$ All sand layer

$\Sigma = I_{zBar1} \cdot \frac{\Delta z_1}{E_s}$

<u>Layer 1</u>

$\Delta z_1 := z_1 = 23.333\ ft$ $E_s = 500\ \frac{ton}{ft^2}$ $I_{zBar1} := \frac{I_{z0} + I_{zMax}}{2} = 0.333$ $\Sigma_1 := 2.4258 \cdot 10^{-6}\ \frac{ft^3}{lb}$

<u>Layer 2</u>

$\Delta z_2 := z_2 - z_1 = 70\ ft$ $E_s = 500\ \frac{ton}{ft^2}$ $I_{zBar2} := \frac{I_{zMax}}{2} = 0.283$ $\Sigma_2 := 6.2068 \cdot 10^{-6}\ \frac{ft^3}{lb}$

$\Sigma := \Sigma_1 + \Sigma_2 = 0.0000086326\ \frac{ft^3}{lb}$

$\sigma_z' := \gamma \cdot D_w + \gamma' \cdot \left(\left(x - D_w\right) + D_{bb}\right) = 4102.133\ \frac{lb}{ft^2}$ Vertical Effective Stress at Equivalent Footing

$C_1 := 1 - 0.5 \cdot \left(\frac{\sigma_z'}{q_{net}}\right) = 0.188$

$t := 50$ Years

$C_2 := 1 + 0.2 \cdot \log\left(\frac{t}{0.1}\right) = 1.54$

$\delta_{pg} := C_1 \cdot C_2 \cdot q_{net} \cdot \Sigma = 0.076\ in$ Elastic Settlement of Pile Group

North Foundation Design
(Case 3: Service I)

Elastic Shortening:

$P_{top} := P_{g1} = 85997.531\ lb$

$A_T := s^2 = 25\ ft^2$ Tributary Area of Each Pile

$P_{bot} := q_{net} \cdot A_T = 63181.86\ lb$

$L_{p1} := x + D_{bb} = 53.333\ ft$

$$P(z) := P_{top} + \left(\frac{P_{top} - P_{bot}}{L_{p1}}\right) \cdot z$$

$A_p = 60.2\ in^2$

$$\delta_E := \int_0^{L_{p1}} \frac{P(z)}{E_p \cdot A_p}\, dz = 0.036\ in$$ Elastic Shortening

$\delta_{PileGroup} := \delta_{pg} + \delta_E = 0.112\ in$

Satisfies requirement of a max settlement of 1 in.

North Foundation Design
(Case 3: Service I)

Allowable Lateral Load Calculations (Brom's Method):

Assume Fixed-Head, Long Pile (Tomlinson)

$B_p = 25.739\ in$ $\quad e := 27\ ft$ $\quad S_x := 380\ in^3$ $\quad S_y := 124\ in^3$ $\quad \frac{e}{B_p} = 12.588$

$F_y := 60000\ \frac{lb}{in^2}$ Yield Stress of Pile

N-S Horizontal Load:

$M_Y := S_x \cdot F_y = 1900000\ lb \cdot ft$ Yield Moment N-S

$K_p := \tan\left(45° + \frac{\phi'}{2}\right)^2 = 4.204$

$\frac{M_Y}{K_p \cdot \gamma \cdot B_p^{\ 4}} = 195.904$ Figure 7.12 from Poulous and Davis

$V_u := 150 \cdot K_p \cdot \gamma \cdot B_p^{\ 3} = 678249.205\ lb$ Ultimate Load

$FS := 1.67$ Factor of Safety for Steel

$V_{all} := \frac{V_u}{FS} = 406137.249\ lb$ >/= $V_y = 271922\ lb$

Condition Satisfied

E-W Horizontal Load:

$M_Y := S_y \cdot F_y = 620000\ lb \cdot ft$ Yield Moment E-W

$K_p := \tan\left(45° + \frac{\phi'}{2}\right)^2 = 4.204$

$\frac{M_Y}{K_p \cdot \gamma \cdot B_p^{\ 4}} = 63.927$ Figure 7.12 from Poulous and Davis

$V_u := 75 \cdot K_p \cdot \gamma \cdot B_p^{\ 3} = 339124.603\ lb$ Ultimate Load

$FS := 1.67$ Factor of Safety for Steel

$V_{all} := \frac{V_u}{FS} = 203068.624\ lb$ >/= $V_x = 39514\ lb$

Condition Satisfied

North Foundation Design
(Case 3: Service I)

Pile Buckling Calculations:

See AutoCAD drawing for pile arrangement

N-S Axis Bending (Overturning Moment)

$y_4 := \frac{s}{2} + s + s + s = 17.5\ ft$ $y_3 := \frac{s}{2} + s + s = 12.5\ ft$ $y_2 := \frac{s}{2} + s = 7.5\ ft$ $y_1 := \frac{s}{2} = 2.5\ ft$

$y_5 := y_1$ $y_6 := y_2$ $y_7 := y_3$ $y_8 := y_4$

$\Sigma y_{squared1} := 2 \cdot \left(8 \cdot \left(y_1^2 + y_2^2 + y_3^2 + y_4^2\right)\right) = 8400\ ft^2$

$P_1 := \frac{P_g}{N} + \frac{M_y \cdot y_1}{\Sigma y_{squared1}} = 86203.317\ lb$ </= $P_{all} = 356838.076\ lb$

$P_2 := \frac{P_g}{N} + \frac{M_y \cdot y_2}{\Sigma y_{squared1}} = 86614.888\ lb$ </= $P_{all} = 356838.076\ lb$

$P_3 := \frac{P_g}{N} + \frac{M_y \cdot y_3}{\Sigma y_{squared1}} = 87026.46\ lb$ </= $P_{all} = 356838.076\ lb$

$P_4 := \frac{P_g}{N} + \frac{M_y \cdot y_4}{\Sigma y_{squared1}} = 87438.031\ lb$ </= $P_{all} = 356838.076\ lb$

$P_5 := \frac{P_g}{N} - \frac{M_y \cdot y_5}{\Sigma y_{squared1}} = 85791.746\ lb$ </= $P_{all} = 356838.076\ lb$

$P_6 := \frac{P_g}{N} - \frac{M_y \cdot y_6}{\Sigma y_{squared1}} = 85380.174\ lb$ </= $P_{all} = 356838.076\ lb$

$P_7 := \frac{P_g}{N} - \frac{M_y \cdot y_7}{\Sigma y_{squared1}} = 84968.603\ lb$ </= $P_{all} = 356838.076\ lb$

$P_8 := \frac{P_g}{N} - \frac{M_y \cdot y_8}{\Sigma y_{squared1}} = 84557.031\ lb$ </= $P_{all} = 356838.076\ lb$

All Piles are able to handle this loading.

E-W Axis Bending (Overturning Moment)

$x_4 := \frac{s}{2} + s + s + s = 17.5\ ft$ $x_3 := \frac{s}{2} + s + s = 12.5\ ft$ $x_2 := \frac{s}{2} + s = 7.5\ ft$ $x_1 := \frac{s}{2} = 2.5\ ft$

$x_5 := x_1$ $x_6 := x_2$ $x_7 := x_3$ $x_8 := x_4$

$\Sigma x_{squared1} := 2 \cdot \left(8 \cdot \left(x_1^2 + x_2^2 + x_3^2 + x_4^2\right)\right) = 8400\ ft^2$

North Foundation Design
(Case 3: Service I)

$P_1 := \frac{P_g}{N} + \frac{M_x \cdot y_1}{\Sigma x_{squared1}} = 109127.234\ lb$ </= $P_{all} = 356838.076\ lb$

$P_2 := \frac{P_g}{N} + \frac{M_x \cdot y_2}{\Sigma x_{squared1}} = 155386.638\ lb$ </= $P_{all} = 356838.076\ lb$

$P_3 := \frac{P_g}{N} + \frac{M_x \cdot y_3}{\Sigma x_{squared1}} = 201646.043\ lb$ </= $P_{all} = 356838.076\ lb$

$P_4 := \frac{P_g}{N} + \frac{M_x \cdot y_4}{\Sigma x_{squared1}} = 247905.448\ lb$ </= $P_{all} = 356838.076\ lb$

$P_5 := \frac{P_g}{N} - \frac{M_x \cdot y_5}{\Sigma x_{squared1}} = 62867.829\ lb$ </= $P_{all} = 356838.076\ lb$

$P_6 := \frac{P_g}{N} - \frac{M_x \cdot y_6}{\Sigma x_{squared1}} = 16608.424\ lb$ </= $T_{all} = 60997.221\ lb$

$P_7 := \frac{P_g}{N} - \frac{M_x \cdot y_7}{\Sigma x_{squared1}} = -29650.981\ lb$ </= $T_{all} = 60997.221\ lb$

$P_8 := \frac{P_g}{N} - \frac{M_x \cdot y_8}{\Sigma x_{squared1}} = -75910.385\ lb$ </= $T_{all} = 60997.221\ lb$

All Piles are able to handle this loading.

All piles satisfy axial load capacity

$A := 60.2\ in^2$

$P_d := P_4$ Maximum Axial Load Controls

$\sigma_d := \frac{P_d}{A} = 4118.031\ \frac{lb}{in^2}$

$\sigma_{all} := 0.35 \cdot F_y = 21000\ \frac{lb}{in^2}$ *Standard Guidelines for the Design and Installation of Pile Foundations*, (35% of Yield Stress)

σ_d </= σ_{all}

Criteria is satisfied. Piles will not buckle.

North Foundation Design
(Case 3: Service I)

Design Summary

All requirements were satisfied for the design of the individual piles and the pile group. The piles are HP 18x204 and 60ft long. The pile cap contains 64 piles that are spaced at 5ft with a typical distribution of an isolated pile cap. Pile axial capacity is analyzed for each individual pile and passes the requirement for axial load. When combining axial load and bending moment the capacity is also satisfied and is shown in the analysis of pile groups portion of the calculations. The pile cap will be a 2ft thick concrete slab, which rests on top of the pile. This cap is a 40' x 40' square block.

South Foundation Design
(Case 1: Strength I)

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South Foundation Design
(Case 1: Strength I)

Foundation Description:

The design of this foundation is for the elevator tower of a proposed pedestrian overpass in Waterloo, IA. The foundation in need of attention is the foundations located on the south side of the bridge, and will be supporting an elevator. To guard against excessive settlement of the abutments that would be detrimental to bridge operations, the abutments shall be supported by a pile group.

Project Goal:

Determine the most economical design for the pile group.

Variables:

$\gamma := 109 \frac{lb}{ft^3}$	Unit Weight of Soil
$\gamma_{sat} := 130 \frac{lb}{ft^3}$	Saturated Unit Weight of Soil
$\gamma_w := 62.4 \frac{lb}{ft^3}$	Unit Weight of Water
$\gamma' := \gamma_{sat} - \gamma_w = 67.6 \frac{lb}{ft^3}$	Submerged Unit Weight of Soil
$D_w := 12\ ft$	Depth of Water Table
$L_p := 30\ ft$	Length of the Pile
$P_{rxn} := 630000\ lb$	Reaction Force From Bridge Load
$P_{dead} := 200000\ lb$	Dead Load of Tower, Including Elevator Shaft
$P_g := P_{rxn} + P_{dead}$	Vertical Load
$M_g := 0\ lb \cdot ft$	Moment

Assumptions:

From the soil report from USGS geodata, it shows that from 0-18" the soil is a loamy sand, followed by sand per 18-30", gravelly course sand per 30-55", course sand per 55-70", and gravelly course sand per 70-80". Since the layer that is cohesive only goes down a short distance from soil grade, it may be reasonable to analyze this soil as a purely granular soil. It was also noted from the soils report that the soil surrounding the project site is excessively drained. The preliminary design with be with a HP14x117 steel pile cross section with an individual pile length of 30'.

South Foundation Design
(Case 1: Strength I)

Design Calculations for Individual Piles

For calculating pile point bearing capacity, the reduced rigidity index needs to be assessed. Therefore taking values for the static stress-strain modulus of elasticity from table 5-6 in the Bowles, *Foundation Analysis and Design* was found for the soil at the bottom of the pile.

$E_s := 50\ MPa$ Assuming course sand that is dense and wet.

$E_s := 1000000\ \frac{lb}{ft^2}$ Conversion to US units.

$\sigma_{z12}' := \gamma \cdot D_w = 1308\ \frac{lb}{ft^2}$ Vertical Effective Stress at Water Table Depth

$\sigma_{z40}' := \gamma \cdot D_w + \left(\gamma' \cdot \left(L_p - D_w\right)\right) = 2524.8\ \frac{lb}{ft^2}$ Vertical Effective Stress at Pile Depth

Using table 2-7 in the Bowles text, the Poisson's ratio for this soil was determined:

$\mu := 0.35$ Cohesionless, dense sand.

Taking the assumed internal angle of friction for poorly graded sand from Lindeburg, *Civil Engineering Reference for PE 8th Edition*, the critical depth can be found using Figure 3.10 from the Poulos and Davis text:

$\phi' := 38\ °$

$\phi := \frac{3}{4} \cdot \phi' + 10\ ° = 38.5\ °$

Zc / d ratio from Poulos & Davis, *Pile Foundation Analysis & Design*, Figure 3.10: = 10.5

$b_f := 14.9\ in$ $d := 14.2\ in$ HP14x117 steel pile.

$B_p := \sqrt[2]{\left(d^2\right) + \left(b_f^2\right)} = 20.583\ in$ Width of pile is diagonal

$A_p := b_f \cdot d = 211.58\ in^2$ Including Soil Plug

$z_c := 10.5 \cdot B_p = 18.01\ ft$ Critical Depth

$\sigma_p'' := D_w \cdot \gamma + \left(\left(z_c - D_w\right) \cdot \gamma'\right) = 1714.27\ \frac{lb}{ft^2}$ Vertical Effective Stress at Critical Depth

South Foundation Design
(Case 1: Strength I)

Calculation of bearing capacity factors ensues:

$G_s := \frac{E_s}{2 \cdot (1+\mu)} = 370370.37 \frac{lb}{ft^2}$ $\quad c' := 0$ Cohesionless sand.

$I_r := \frac{G_s}{c' + \sigma_{z40}' \cdot \tan(\phi')} = 187.758$ Rigidity Index

Bowles, *Foundation Analysis & Design*, Table on P.894 : Rigidity Index Within Range for Sandy Soil

$\varepsilon_v := \frac{(1+\mu) \cdot (1 - 2 \cdot \mu) \cdot (\sigma_{z40}')}{E_s \cdot (1-\mu)} = 0.002$ Volumetric Strain

$I_{rr} := \frac{I_r}{1 + \varepsilon_v \cdot I_r} = 144.946$ Reduced Rigidity Index

$K_o := 1 - \sin(\phi') = 0.384$ At rest earth pressure coefficient

Vesic's Bearing Capacity Factors:

$\eta := \frac{1 + 2 \cdot K_o}{3} = 0.59$

$N_q := \frac{3}{3 - \sin(\phi')} \cdot \exp\left(\left(\frac{\pi}{2} - \phi'\right) \tan(\phi')\right) \cdot \left(\tan\left(\frac{\pi}{4} + \frac{\phi'}{2}\right)\right)^2 \cdot I_{rr}^{\left(\frac{4 \cdot \sin(\phi')}{3 \cdot (1 + \sin(\phi'))}\right)} = 134.709$

$N_\gamma := 0.6 \cdot (N_q - 1) \tan(\phi') = 62.679$

$N_{c1} := (N_q - 1) \cot(\phi') = 171.14$

$N_{c2} := \frac{4}{3} \cdot (\ln(I_{rr}) + 1) + \frac{\pi}{2} + 1 = 10.539$

$N_c := \text{if}(\phi' = 0, N_{c2}, N_{c1}) = 171.14$

$q_p := \eta \cdot \sigma_p'' \cdot N_q + \frac{1}{2} \cdot \gamma' \cdot B_p \cdot N_\gamma = 139779.292 \frac{lb}{ft^2}$ Pile Point Bearing Capacity

$P_p := A_p \cdot q_p = 205378.491\ lb$ Pile Point Load Capacity

South Foundation Design
(Case 1: Strength I)

Calculation of Side Friction Capacity:

α - Method calculations will be used to calculate the pile side friction capacity.

From Tomlinson, *Pile Design and Construction Practice*, Table 4.10, Large displacement pile due to Steel HP shape. Range is based on soil density, therefore with our dense sand assumption:

$$\frac{K_s}{K_o} = 1.25$$

$K_s := K_o \cdot 1.25 = 0.48$ Lateral Earth-Pressure Coefficient for Side Friction

From Tomlinson, Table 4.11, Smooth Steel Pile at interface with sand:

$\delta_p := 0.6 \cdot \phi' = 22.8\ °$ Angle of Friction Between Pile and Soil

$f_s := K_s \cdot \sigma_p'' \cdot \tan\left(\delta_p\right) = 346.199\ \frac{lb}{ft^2}$ Side Friction Stress, Considering Critical Depth

Calculation of side friction stress profile:

$z_1 := D_w$

$\sigma_{z1} := D_w \cdot \gamma = 1308\ \frac{lb}{ft^2}$

$f_{s12} := K_s \cdot \sigma_{z1} \cdot \tan\left(\delta_p\right) = 264.152\ \frac{lb}{ft^2}$

$z_c = 18.01\ ft$

Layers	Avg. Side Friction Stress
0-12 ft	$f_{s1Bar} := \frac{1}{2} \cdot \left(0 + f_{s12}\right) = 132.076\ \frac{lb}{ft^2}$
12-18 ft	$f_{s2Bar} := \frac{1}{2} \cdot \left(f_{s12} + f_s\right) = 305.176\ \frac{lb}{ft^2}$
18-30 ft	$f_{s3Bar} := f_s = 346.199\ \frac{lb}{ft^2}$

$P_{p1} := b_f + b_f + d + d = 4.85\ ft$ Perimeter of Pile (Including Soil Plug)

$P_s := P_{p1} \cdot \left(12\ ft \cdot \left(f_{s1Bar}\right) + 6\ ft \cdot \left(f_{s2Bar}\right) + 12\ ft \cdot \left(f_{s3Bar}\right)\right) = 36716.233\ lb$ Side Friction Capacity

South Foundation Design
(Case 1: Strength I)

<u>Calculations for compression and tension load capacity:</u>

$L_p = 30\ ft$ Length of Pile

$W_p := 117 \frac{lb}{ft} \cdot L_p = 3510\ lb$ Nominal Weight of pile type multiplied by pile length

From the *Standard Guidelines for the Design and Installation of Pile Foundations*, the factors of safety were determined from table A.1 and A.2. Assuming a pile group consisting of 12 piles, the design axial load will be distributed throughout these 12 piles.

$N := 12$ Number of Piles

$P_{g1} := \frac{P_g}{N} = 34.583\ ton$

$F_1 := 2.0$ Table A.1, Since this is a preliminary design, using driving formulas and static analysis to determine factors of safety

$F_2 := 1.1$ Table A.2, HP Pile

$FS := F_1 \cdot F_2 = 2.2$

$P := P_{g1} + W_p = 72676.667\ lb$ </= $P_{all} := \frac{P_p + P_s}{FS} = 110043.056\ lb$

Condition Satisfied

For uplift, the factor of safety is approximately a 50% increase from the compression capacity factor of safety.

$FS_T := 1.5 \cdot FS = 3.3$ Factor of Safety for Uplift

$T_{all} := \frac{P_s}{FS_T} + W_p = 14636.131\ lb$ Tension Capacity

Each individual pile satisfies this condition for both combined axial load and bending moment. This design criteria is calculated when calculating to see if the piles will buckle in the analysis of pile groups. This value is also shown in the summary.

South Foundation Design
(Case 1: Strength I)

Pile settlement for an individual pile is as follows (Bowles Method):

$A_p := 34.4\ in^2$ AISC Table 1-4: HP 18x201

$E_p := 29000000\ \frac{lb}{in^2}$

$m := 1$ Shape Factor, m*Is = 1.0 (Relatively square pile)

$I_s := 1$

$\frac{L_p}{B_p} = 17.49$ Fox Embedment Factor Inequality Satisfaction

$I_F := 0.4$ Fox Embedment Factor

$F_1 := 0.25$ Reduction Factor, High side friction capacity compared to design load for an individual pile.

$q := \frac{P_{g1}}{A_p} = 2010.659\ \frac{lb}{in^2}$

$\delta_p := q \cdot B_p \cdot \left(\frac{1-\mu^2}{E_s}\right) \cdot m \cdot I_s \cdot I_F \cdot F_1 = 0.523\ in$ Point Bearing Settlement

For elastic settlement assume that the point load is equal to zero.

$P(z) := P_{g1} + \left(\frac{P_{g1}}{L_p}\right) \cdot z$

$\delta_E := \int_0^{L_p} \frac{P(z)}{E_p \cdot A_p}\, dz = 0.037\ in$ Elastic Shortening

$\delta_{pile} := \delta_p + \delta_E = 0.56\ in$ Total Pile Settlement

$B_p \cdot 0.03 = 0.617\ in$ Pile settlement should not be greater than 3% of pile diameter

Condition Satisfied

South Foundation Design
(Case 1: Strength I)

Design Calculations for Pile Group

Vertical load capacity of the entire pile group:

$P_s = 36716.233\ lb$ Single pile

$P_p = 205378.491\ lb$ Single pile

$P_{Group} := N \cdot (P_s + P_p) = 2905136.678\ lb$ Capacity of Pile Group based on piles failing individually

$s := 2 \cdot B_p = 3.43\ ft$ Minimum Spacing Requirement

$s := 4\ ft$ Pile Spacing

$e := 0.5\ ft$ Edge Distance

$B_g := (2 \cdot s) + \left(\frac{(B_p)}{2} + e\right) + \left(\frac{(B_p)}{2} + e\right) = 10.715\ ft$ See Drawing for Dimensions

$L_g := (3 \cdot s) + \left(\frac{(B_p)}{2} + e\right) + \left(\frac{(B_p)}{2} + e\right) = 14.715\ ft$

$B_g := 12\ ft$ Simplified Dimension

$L_g := 15\ ft$ Simplified Dimension

$f_s := (f_{s1Bar}) + (f_{s2Bar}) + (f_{s3Bar}) = 783.451\ \frac{lb}{ft^2}$

$P_{gg} := (2 \cdot B_g) + (2 \cdot L_g) = 54\ ft$ Perimeter of Group

$A_g := L_g \cdot B_g = 180\ ft^2$ Area of Group

$P_{NBlock} := P_{gg} \cdot L_p \cdot f_s + A_g \cdot q_p = 26429463.147\ lb$ Block Failure Capacity (failure of entire group)

$P_{Ngroup} := min\left(P_{Group}, P_{NBlock}\right) = 2905136.678\ lb$

$FS := 3$ Assumed FS for pile group

$P_{allGroup} := \frac{P_{Ngroup}}{FS} = 968378.893\ lb$ >/= $P_g = 830000\ lb$

Condition Satisfied

South Foundation Design
(Case 1: Strength I)

Predicted elastic settlement of the entire pile group:

$x := \frac{2}{3} \cdot L_p = 20\ ft$ Equivalent Footing Depth for Piles in Sand

$D_b := \frac{1}{3} \cdot L_p = 10\ ft$

$D_{bb} := \frac{2}{3} \cdot D_b = 6.667\ ft$ Starting of 4/1 Slope Stress Distribution

$B_{EQ} := B_g + 2 \cdot \left(\frac{D_{bb}}{4}\right) = 15.333\ ft$ $L_{EQ} := L_g + 2 \cdot \left(\frac{D_{bb}}{4}\right) = 18.333\ ft$

$q_{net} := \frac{P_g}{B_{EQ}^{2}} = 3530.246\ \frac{lb}{ft^2}$

Strain Influence Factor Method:

$z_1 := B_{EQ} \cdot \left(0.5 + 0.555 \cdot \left(\frac{L_{EQ}}{B_{EQ}} - 1\right)\right) = 9.332\ ft$ </= $B_{EQ} = 15.333\ ft$ OK

$z_2 := B_{EQ} \cdot \left(2 + 0.222 \cdot \left(\frac{L_{EQ}}{B_{EQ}} - 1\right)\right) = 31.333\ ft$ </= $4 \cdot B_{EQ} = 61.333\ ft$ OK

$I_{z0} := 0.1 + 0.0111 \cdot \left(\frac{L_{EQ}}{B_{EQ}} - 1\right) = 0.102$ </= 0.2 OK

$\sigma_{zp}' := \gamma \cdot D_w + \gamma' \cdot \left(\left(x - D_w\right) + D_{bb} + z_1\right) = 2930.287\ \frac{lb}{ft^2}$ Vertical Effective Stress at z1 (Before Installation)

$I_{zMax} := 0.5 + 0.1 \cdot \sqrt[2]{\frac{q_{net}}{\sigma_{zp}'}} = 0.61$ All sand layer

$\Sigma = I_{zBar1} \cdot \frac{\Delta z_1}{E_s}$

Layer 1

$\Delta z_1 := z_1 = 9.332\ ft$ $E_s = 500\ \frac{ton}{ft^2}$ $I_{zBar1} := \frac{I_{z0} + I_{zMax}}{2} = 0.356$ $\Sigma_1 := 2.4258 \cdot 10^{-6}\ \frac{ft^3}{lb}$

Layer 2

$\Delta z_2 := z_2 - z_1 = 22.001\ ft$ $E_s = 500\ \frac{ton}{ft^2}$ $I_{zBar2} := \frac{I_{zMax}}{2} = 0.305$ $\Sigma_2 := 6.2068 \cdot 10^{-6}\ \frac{ft^3}{lb}$

$\Sigma := \Sigma_1 + \Sigma_2 = 0.0000086326\ \frac{ft^3}{lb}$

$\sigma_z' := \gamma \cdot D_w + \gamma' \cdot \left(\left(x - D_w\right) + D_{bb}\right) = 2299.467\ \frac{lb}{ft^2}$ Vertical Effective Stress at Equivalent Footing

$C_1 := 1 - 0.5 \cdot \left(\frac{\sigma_z'}{q_{net}}\right) = 0.674$

$t := 50$ Years

$C_2 := 1 + 0.2 \cdot \log\left(\frac{t}{0.1}\right) = 1.54$

$\delta_{pg} := C_1 \cdot C_2 \cdot q_{net} \cdot \Sigma = 0.38\ in$ Elastic Settlement of Pile Group

South Foundation Design
(Case 1: Strength I)

Elastic Shortening:

$P_{top} := P_{g1} = 69166.667\ lb$

$A_T := s^2 = 16\ ft^2$ Tributary Area of Each Pile

$P_{bot} := q_{net} \cdot A_T = 56483.932\ lb$

$L_{p1} := x + D_{bb} = 26.667\ ft$

$P(z) := P_{top} + \left(\frac{P_{top} - P_{bot}}{L_{p1}}\right) \cdot z$

$A_p = 34.4\ in^2$

$\delta_E := \int_0^{L_{p1}} \frac{P(z)}{E_p \cdot A_p}\, dz = 0.024\ in$ Elastic Shortening

$\delta_{PileGroup} := \delta_{pg} + \delta_E = 0.404\ in$

Satisfies requirement of a max settlement of 1 in.

South Foundation Design
(Case 1: Strength I)

<u>Pile Buckling Calculations:</u>

See AutoCAD drawing for pile arrangement

<u>No Overturning Moment</u>

All piles satisfy axial load capacity

$A := 34.4\ in^2$ $\quad F_y := 50000\ \frac{lb}{in^2}$

$P_d := \frac{P_g}{N} = 69166.667\ lb$ Maximum Axial Load Controls

$\sigma_d := \frac{P_d}{A} = 2010.659\ \frac{lb}{in^2}$

$\sigma_{all} := 0.35 \cdot F_y = 17500\ \frac{lb}{in^2}$ *Standard Guidelines for the Design and Installation of Pile Foundations*, (35% of Yield Stress)

$\sigma_d \quad </= \quad \sigma_{all}$

Criteria is satisfied. Piles will not buckle.

South Foundation Design
(Case 1: Strength I)

Design Summary

All requirements were satisfied for the design of the individual piles and the pile group. The piles are HP 14x117 and 30ft long. The pile cap contains 12 piles that are spaced at 4ft with a typical distribution of an isolated pile cap. Pile axial capacity is analyzed for each individual pile and passes the requirement for axial load. When combining axial load and bending moment the capacity is also satisfied and is shown in the analysis of pile groups portion of the calculations. The pile cap will be a 2ft thick concrete slab, which rests on top of the pile. This cap is a 12' x 15' rectangular block.

South Foundation Design
(Case 2: Strength III)

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South Foundation Design
(Case 2: Strength III)

Foundation Description:

The design of this foundation is for the elevator tower of a proposed pedestrian overpass in Waterloo, IA. The foundation in need of attention is the foundations located on the south side of the bridge, and will be supporting an elevator. To guard against excessive settlement of the abutments that would be detrimental to bridge operations, the abutments shall be supported by a pile group.

Project Goal:

Determine the most economical design for the pile group.

Variables:

$\gamma := 109 \frac{lb}{ft^3}$	Unit Weight of Soil
$\gamma_{sat} := 130 \frac{lb}{ft^3}$	Saturated Unit Weight of Soil
$\gamma_w := 62.4 \frac{lb}{ft^3}$	Unit Weight of Water
$\gamma' := \gamma_{sat} - \gamma_w = 67.6 \frac{lb}{ft^3}$	Submerged Unit Weight of Soil
$D_w := 12\ ft$	Depth of Water Table
$L_p := 30\ ft$	Length of the Pile
$P_{rxn} := 441000\ lb$	Reaction Force From Bridge Load
$P_{dead} := 200000\ lb$	Dead Load of Tower, Including Elevator Shaft
$P_g := P_{rxn} + P_{dead}$	Vertical Load
$V_{gx} := 3000\ lb$	Horizontal Load
$M_{gy} := 36000\ lb \cdot ft$	Moment

Assumptions:

From the soil report from USGS geodata, it shows that from 0-18" the soil is a loamy sand, followed by sand per 18-30", gravelly course sand per 30-55", course sand per 55-70", and gravelly course sand per 70-80". Since the layer that is cohesive only goes down a short distance from soil grade, it may be reasonable to analyze this soil as a purely granular soil. It was also noted from the soils report that the soil surrounding the project site is excessively drained. The preliminary design with be with a HP14x117 steel pile cross section with an individual pile length of 30'.

South Foundation Design
(Case 2: Strength III)

Design Calculations for Individual Piles

For calculating pile point bearing capacity, the reduced rigidity index needs to be assessed. Therefore taking values for the static stress-strain modulus of elasticity from table 5-6 in the Bowles, *Foundation Analysis and Design* was found for the soil at the bottom of the pile.

$E_s := 50\ MPa$ Assuming course sand that is dense and wet.

$E_s := 1000000\ \frac{lb}{ft^2}$ Conversion to US units.

$\sigma_{z12}' := \gamma \cdot D_w = 1308\ \frac{lb}{ft^2}$ Vertical Effective Stress at Water Table Depth

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Using table 2-7 in the Bowles text, the Poisson's ratio for this soil was determined:

$\mu := 0.35$ Cohesionless, dense sand.

Taking the assumed internal angle of friction for poorly graded sand from Lindeburg, *Civil Engineering Reference for PE 8th Edition*, the critical depth can be found using Figure 3.10 from the Poulos and Davis text:

$\phi' := 38\ °$

$\phi := \frac{3}{4} \cdot \phi' + 10\ ° = 38.5\ °$

Zc / d ratio from Poulos & Davis, *Pile Foundation Analysis & Design*, Figure 3.10: = 10.5

$b_f := 14.9\ in$ $d := 14.2\ in$ HP14x117 steel pile.

$B_p := \sqrt[2]{\left(d^2\right) + \left(b_f^{\ 2}\right)} = 20.583\ in$ Width of pile is diagonal

$A_p := b_f \cdot d = 211.58\ in^2$ Including Soil Plug

$z_c := 10.5 \cdot B_p = 18.01\ ft$ Critical Depth

$\sigma_p'' := D_w \cdot \gamma + \left(\left(z_c - D_w\right) \cdot \gamma'\right) = 1714.27\ \frac{lb}{ft^2}$ Vertical Effective Stress at Critical Depth

South Foundation Design
(Case 2: Strength III)

Calculation of bearing capacity factors ensues:

$G_s := \frac{E_s}{2 \cdot (1+\mu)} = 370370.37 \frac{lb}{ft^2}$ $c' := 0$ Cohesionless sand.

$I_r := \frac{G_s}{c' + \sigma_{z40}' \cdot \tan(\phi')} = 187.758$ Rigidity Index

Bowles, *Foundation Analysis & Design*, Table on P.894 : Rigidity Index Within Range for Sandy Soil

$\varepsilon_v := \frac{(1+\mu) \cdot (1-2 \cdot \mu) \cdot (\sigma_{z40}')}{E_s \cdot (1-\mu)} = 0.002$ Volumetric Strain

$I_{rr} := \frac{I_r}{1 + \varepsilon_v \cdot I_r} = 144.946$ Reduced Rigidity Index

$K_o := 1 - \sin(\phi') = 0.384$ At rest earth pressure coefficient

Vesic's Bearing Capacity Factors:

$\eta := \frac{1 + 2 \cdot K_o}{3} = 0.59$

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$N_\gamma := 0.6 \cdot (N_q - 1) \tan(\phi') = 62.679$

$N_{c1} := (N_q - 1) \cot(\phi') = 171.14$

$N_{c2} := \frac{4}{3} \cdot (\ln(I_{rr}) + 1) + \frac{\pi}{2} + 1 = 10.539$

$N_c := \text{if}(\phi' = 0, N_{c2}, N_{c1}) = 171.14$

$q_p := \eta \cdot \sigma_p'' \cdot N_q + \frac{1}{2} \cdot \gamma' \cdot B_p \cdot N_\gamma = 139779.292 \frac{lb}{ft^2}$ Pile Point Bearing Capacity

$P_p := A_p \cdot q_p = 205378.491\ lb$ Pile Point Load Capacity

South Foundation Design
(Case 2: Strength III)

Calculation of Side Friction Capacity:

α - Method calculations will be used to calculate the pile side friction capacity.

From Tomlinson, *Pile Design and Construction Practice*, Table 4.10, Large displacement pile due to Steel HP shape. Range is based on soil density, therefore with our dense sand assumption:

$$\frac{K_s}{K_o} = 1.25$$

$K_s := K_o \cdot 1.25 = 0.48$ Lateral Earth-Pressure Coefficient for Side Friction

From Tomlinson, Table 4.11, Smooth Steel Pile at interface with sand:

$\delta_p := 0.6 \cdot \phi' = 22.8\ °$ Angle of Friction Between Pile and Soil

$f_s := K_s \cdot \sigma_p'' \cdot \tan(\delta_p) = 346.199\ \frac{lb}{ft^2}$ Side Friction Stress, Considering Critical Depth

Calculation of side friction stress profile:

$z_1 := D_w$

$\sigma_{z1} := D_w \cdot \gamma = 1308\ \frac{lb}{ft^2}$

$f_{s12} := K_s \cdot \sigma_{z1} \cdot \tan(\delta_p) = 264.152\ \frac{lb}{ft^2}$

$z_c = 18.01\ ft$

Layers	Avg. Side Friction Stress
0-12 ft	$f_{s1Bar} := \frac{1}{2} \cdot (0 + f_{s12}) = 132.076\ \frac{lb}{ft^2}$
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18-30 ft	$f_{s3Bar} := f_s = 346.199\ \frac{lb}{ft^2}$

$P_{p1} := b_f + b_f + d + d = 4.85\ ft$ Perimeter of Pile (Including Soil Plug)

$P_s := P_{p1} \cdot (12\ ft \cdot (f_{s1Bar}) + 6\ ft \cdot (f_{s2Bar}) + 12\ ft \cdot (f_{s3Bar})) = 36716.233\ lb$ Side Friction Capacity

South Foundation Design
(Case 2: Strength III)

Calculations for compression and tension load capacity:

$L_p = 30\ ft$ — Length of Pile

$W_p := 117 \frac{lb}{ft} \cdot L_p = 3510\ lb$ — Nominal Weight of pile type multiplied by pile length

From the *Standard Guidelines for the Design and Installation of Pile Foundations*, the factors of safety were determined from table A.1 and A.2. Assuming a pile group consisting of 12 piles, the design axial load will be distributed throughout these 12 piles.

$N := 12$ — Number of Piles

$P_{g1} := \frac{P_g}{N} = 26.708\ ton$

$F_1 := 2.0$ — Table A.1, Since this is a preliminary design, using driving formulas and static analysis to determine factors of safety

$F_2 := 1.1$ — Table A.2, HP Pile

$FS := F_1 \cdot F_2 = 2.2$

$P := P_{g1} + W_p = 56926.667\ lb$ </= $P_{all} := \frac{P_p + P_s}{FS} = 110043.056\ lb$

Condition Satisfied

For uplift, the factor of safety is approximately a 50% increase from the compression capacity factor of safety.

$FS_T := 1.5 \cdot FS = 3.3$ — Factor of Safety for Uplift

$T_{all} := \frac{P_s}{FS_T} + W_p = 14636.131\ lb$ — Tension Capacity

Each individual pile satisfies this condition for both combined axial load and bending moment. This design criteria is calculated when calculating to see if the piles will buckle in the analysis of pile groups. This value is also shown in the summary.

South Foundation Design
(Case 2: Strength III)

Pile settlement for an individual pile is as follows (Bowles Method):

$A_p := 34.4\ in^2$ AISC Table 1-4: HP 18x201

$E_p := 29000000\ \frac{lb}{in^2}$

$m := 1$ Shape Factor, m*Is = 1.0 (Relatively square pile)

$I_s := 1$

$\frac{L_p}{B_p} = 17.49$ Fox Embedment Factor Inequality Satisfaction

$I_F := 0.4$ Fox Embedment Factor

$F_1 := 0.25$ Reduction Factor, High side friction capacity compared to design load for an individual pile.

$q := \frac{P_{g1}}{A_p} = 1552.81\ \frac{lb}{in^2}$

$\delta_p := q \cdot B_p \cdot \left(\frac{1-\mu^2}{E_s}\right) \cdot m \cdot I_s \cdot I_F \cdot F_1 = 0.404\ in$ Point Bearing Settlement

For elastic settlement assume that the point load is equal to zero.

$P(z) := P_{g1} + \left(\frac{P_{g1}}{L_p}\right) \cdot z$

$\delta_E := \int_0^{L_p} \frac{P(z)}{E_p \cdot A_p}\, dz = 0.029\ in$ Elastic Shortening

$\delta_{pile} := \delta_p + \delta_E = 0.433\ in$ Total Pile Settlement

$B_p \cdot 0.03 = 0.617\ in$ Pile settlement should not be greater than 3% of pile diameter

Condition Satisfied

South Foundation Design
(Case 2: Strength III)

Design Calculations for Pile Group

Vertical load capacity of the entire pile group:

$P_s = 36716.233\ lb$ Single pile

$P_p = 205378.491\ lb$ Single pile

$P_{Group} := N \cdot (P_s + P_p) = 2905136.678\ lb$ Capacity of Pile Group based on piles failing individually

$s := 2 \cdot B_p = 3.43\ ft$ Minimum Spacing Requirement

$s := 4\ ft$ Pile Spacing

$e := 0.5\ ft$ Edge Distance

$B_g := (2 \cdot s) + \left(\frac{(B_p)}{2} + e\right) + \left(\frac{(B_p)}{2} + e\right) = 10.715\ ft$ See Drawing for Dimensions

$L_g := (3 \cdot s) + \left(\frac{(B_p)}{2} + e\right) + \left(\frac{(B_p)}{2} + e\right) = 14.715\ ft$

$B_g := 12\ ft$ Simplified Dimension

$L_g := 15\ ft$ Simplified Dimension

$f_s := (f_{s1Bar}) + (f_{s2Bar}) + (f_{s3Bar}) = 783.451\ \frac{lb}{ft^2}$

$P_{gg} := (2 \cdot B_g) + (2 \cdot L_g) = 54\ ft$ Perimeter of Group

$A_g := L_g \cdot B_g = 180\ ft^2$ Area of Group

$P_{NBlock} := P_{gg} \cdot L_p \cdot f_s + A_g \cdot q_p = 26429463.147\ lb$ Block Failure Capacity (failure of entire group)

$P_{Ngroup} := min\,(P_{Group}, P_{NBlock}) = 2905136.678\ lb$

$FS := 3$ Assumed FS for pile group

$P_{allGroup} := \frac{P_{Ngroup}}{FS} = 968378.893\ lb$ >/= $P_g = 641000\ lb$

Condition Satisfied

South Foundation Design
(Case 2: Strength III)

Predicted elastic settlement of the entire pile group:

$x := \frac{2}{3} \cdot L_p = 20\ ft$ Equivalent Footing Depth for Piles in Sand

$D_b := \frac{1}{3} \cdot L_p = 10\ ft$

$D_{bb} := \frac{2}{3} \cdot D_b = 6.667\ ft$ Starting of 4/1 Slope Stress Distribution

$B_{EQ} := B_g + 2 \cdot \left(\frac{D_{bb}}{4}\right) = 15.333\ ft$ $L_{EQ} := L_g + 2 \cdot \left(\frac{D_{bb}}{4}\right) = 18.333\ ft$

$q_{net} := \frac{P_g}{B_{EQ}^2} = 2726.371\ \frac{lb}{ft^2}$

Strain Influence Factor Method:

$z_1 := B_{EQ} \cdot \left(0.5 + 0.555 \cdot \left(\frac{L_{EQ}}{B_{EQ}} - 1\right)\right) = 9.332\ ft$ </= $B_{EQ} = 15.333\ ft$ OK

$z_2 := B_{EQ} \cdot \left(2 + 0.222 \cdot \left(\frac{L_{EQ}}{B_{EQ}} - 1\right)\right) = 31.333\ ft$ </= $4 \cdot B_{EQ} = 61.333\ ft$ OK

$I_{z0} := 0.1 + 0.0111 \cdot \left(\frac{L_{EQ}}{B_{EQ}} - 1\right) = 0.102$ </= 0.2 OK

$\sigma_{zp}' := \gamma \cdot D_w + \gamma' \cdot \left(\left(x - D_w\right) + D_{bb} + z_1\right) = 2930.287\ \frac{lb}{ft^2}$ Vertical Effective Stress at z1 (Before Installation)

$I_{zMax} := 0.5 + 0.1 \cdot \sqrt[2]{\frac{q_{net}}{\sigma_{zp}'}} = 0.596$ All sand layer

$\Sigma = I_{zBar1} \cdot \frac{\Delta z_1}{E_s}$

Layer 1

$\Delta z_1 := z_1 = 9.332\ ft$ $E_s = 500\ \frac{ton}{ft^2}$ $I_{zBar1} := \frac{I_{z0} + I_{zMax}}{2} = 0.349$ $\Sigma_1 := 2.4258 \cdot 10^{-6}\ \frac{ft^3}{lb}$

Layer 2

$\Delta z_2 := z_2 - z_1 = 22.001\ ft$ $E_s = 500\ \frac{ton}{ft^2}$ $I_{zBar2} := \frac{I_{zMax}}{2} = 0.298$ $\Sigma_2 := 6.2068 \cdot 10^{-6}\ \frac{ft^3}{lb}$

$\Sigma := \Sigma_1 + \Sigma_2 = 0.0000086326\ \frac{ft^3}{lb}$

$\sigma_z' := \gamma \cdot D_w + \gamma' \cdot \left(\left(x - D_w\right) + D_{bb}\right) = 2299.467\ \frac{lb}{ft^2}$ Vertical Effective Stress at Equivalent Footing

$C_1 := 1 - 0.5 \cdot \left(\frac{\sigma_z'}{q_{net}}\right) = 0.578$

$t := 50$ Years

$C_2 := 1 + 0.2 \cdot \log\left(\frac{t}{0.1}\right) = 1.54$

$\delta_{pg} := C_1 \cdot C_2 \cdot q_{net} \cdot \Sigma = 0.251\ in$ Elastic Settlement of Pile Group

South Foundation Design
(Case 2: Strength III)

Elastic Shortening:

$P_{top} := P_{g1} = 53416.667\ lb$

$A_T := s^2 = 16\ ft^2$ Tributary Area of Each Pile

$P_{bot} := q_{net} \cdot A_T = 43621.928\ lb$

$L_{p1} := x + D_{bb} = 26.667\ ft$

$P(z) := P_{top} + \left(\frac{P_{top} - P_{bot}}{L_{p1}}\right) \cdot z$

$A_p = 34.4\ in^2$

$\delta_E := \int_0^{L_{p1}} \frac{P(z)}{E_p \cdot A_p}\, dz = 0.019\ in$ Elastic Shortening

$\delta_{PileGroup} := \delta_{pg} + \delta_E = 0.27\ in$

Satisfies requirement of a max settlement of 1 in.

South Foundation Design
(Case 2: Strength III)

Allowable Lateral Load Calculations (Brom's Method):

Assume Fixed-Head, Long Pile (Tomlinson)

$B_p = 20.583\ in$

$e := 27\ ft$

$\frac{e}{B_p} = 15.741$

$I_x := 1220\ in^4$

$S_x := 172\ in^3$

$I_y := 443\ in^4$

$S_y := 59.5\ in^3$

$F_y := 50000\ \frac{lb}{in^2}$ Yield Stress of Pile

$M_y := S_y \cdot F_y = 247916.667\ lb \cdot ft$ Yield Moment

$K_p := \tan\left(45° + \frac{\phi'}{2}\right)^2 = 4.204$

$\frac{M_y}{K_p \cdot \gamma \cdot B_p^{\ 4}} = 62.511$ Figure 7.12 from Poulous and Davis

$V_u := 9 \cdot K_p \cdot \gamma \cdot B_p^{\ 3} = 20810.022\ lb$ Ultimate Load

$FS := 1.67$ Factor of Safety for Steel

$V_{all} := \frac{V_u}{FS} = 12461.091\ lb$ >/= $V_{gx} = 3000\ lb$

Condition Satisfied

South Foundation Design
(Case 2: Strength III)

Pile Buckling Calculations:

See AutoCAD drawing for pile arrangement

Overturning Moment (N-S) Only

$y_1 := s + \frac{s}{2} = 6\ ft$ $y_2 := \frac{s}{2} = 2\ ft$ $y_3 := y_2$ $y_4 := y_1$

$\Sigma y_{squared1} := 2 \cdot \left(3 \cdot \left(y_1^{\ 2} + y_2^{\ 2}\right)\right) = 240\ ft^2$

$P_1 := \frac{P_g}{N} + \frac{M_{gy} \cdot y_1}{\Sigma y_{squared1}} = 54316.667\ lb$ </= $P_{all} := \frac{P_p + P_s}{FS} = 144966.9\ lb$

$P_2 := \frac{P_g}{N} + \frac{M_{gy} \cdot y_2}{\Sigma y_{squared1}} = 53716.667\ lb$ </= $P_{all} := \frac{P_p + P_s}{FS} = 144966.9\ lb$

$P_3 := \frac{P_g}{N} - \frac{M_{gy} \cdot y_3}{\Sigma y_{squared1}} = 53116.667\ lb$ </= $P_{all} := \frac{P_p + P_s}{FS} = 144966.9\ lb$

$P_4 := \frac{P_g}{N} - \frac{M_{gy} \cdot y_4}{\Sigma y_{squared1}} = 52516.667\ lb$ </= $P_{all} := \frac{P_p + P_s}{FS} = 144966.9\ lb$

All piles satisfy axial load capacity

$A := 34.4\ in^2$ $F_y := 50000\ \frac{lb}{in^2}$

$P_d := P_1 = 54316.667\ lb$ Maximum Axial Load Controls

$\sigma_d := \frac{P_d}{A} = 1578.973\ \frac{lb}{in^2}$

$\sigma_{all} := 0.35 \cdot F_y = 17500\ \frac{lb}{in^2}$ *Standard Guidelines for the Design and Installation of Pile Foundations*, (35% of Yield Stress)

σ_d </= σ_{all}

Criteria is satisfied. Piles will not buckle.

South Foundation Design
(Case 2: Strength III)

Design Summary

All requirements were satisfied for the design of the individual piles and the pile group. The piles are HP 14x117 and 30ft long. The pile cap contains 12 piles that are spaced at 4ft with a typical distribution of an isolated pile cap. Pile axial capacity is analyzed for each individual pile and passes the requirement for axial load. When combining axial load and bending moment the capacity is also satisfied and is shown in the analysis of pile groups portion of the calculations. The pile cap will be a 2ft thick concrete slab, which rests on top of the pile. This cap is a 12' x 15' rectangular block.

South Foundation Design
(Case 3: Service I)

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South Foundation Design
(Case 3: Service I)

Foundation Description:

The design of this foundation is for the elevator tower of a proposed pedestrian overpass in Waterloo, IA. The foundation in need of attention is the foundations located on the south side of the bridge, and will be supporting an elevator. To guard against excessive settlement of the abutments that would be detrimental to bridge operations, the abutments shall be supported by a pile group.

Project Goal:

Determine the most economical design for the pile group.

Variables:

$\gamma := 109 \frac{lb}{ft^3}$	Unit Weight of Soil
$\gamma_{sat} := 130 \frac{lb}{ft^3}$	Saturated Unit Weight of Soil
$\gamma_w := 62.4 \frac{lb}{ft^3}$	Unit Weight of Water
$\gamma' := \gamma_{sat} - \gamma_w = 67.6 \frac{lb}{ft^3}$	Submerged Unit Weight of Soil
$D_w := 12\ ft$	Depth of Water Table
$L_p := 30\ ft$	Length of the Pile
$P_{rxn} := 460000\ lb$	Reaction Force From Bridge Load
$P_{dead} := 200000\ lb$	Dead Load of Tower, Including Elevator Shaft
$P_g := P_{rxn} + P_{dead}$	Vertical Load
$V_{gx} := 1000\ lb$	Horizontal Load
$M_{gy} := 8000\ lb \cdot ft$	Moment

Assumptions:

From the soil report from USGS geodata, it shows that from 0-18" the soil is a loamy sand, followed by sand per 18-30", gravelly course sand per 30-55", course sand per 55-70", and gravelly course sand per 70-80". Since the layer that is cohesive only goes down a short distance from soil grade, it may be reasonable to analyze this soil as a purely granular soil. It was also noted from the soils report that the soil surrounding the project site is excessively drained. The preliminary design with be with a HP14x117 steel pile cross section with an individual pile length of 30'.

South Foundation Design
(Case 3: Service I)

Design Calculations for Individual Piles

For calculating pile point bearing capacity, the reduced rigidity index needs to be assessed. Therefore taking values for the static stress-strain modulus of elasticity from table 5-6 in the Bowles, *Foundation Analysis and Design* was found for the soil at the bottom of the pile.

$E_s := 50\ MPa$ Assuming course sand that is dense and wet.

$E_s := 1000000\ \frac{lb}{ft^2}$ Conversion to US units.

$\sigma_{z12}' := \gamma \cdot D_w = 1308\ \frac{lb}{ft^2}$ Vertical Effective Stress at Water Table Depth

$\sigma_{z40}' := \gamma \cdot D_w + (\gamma' \cdot (L_p - D_w)) = 2524.8\ \frac{lb}{ft^2}$ Vertical Effective Stress at Pile Depth

Using table 2-7 in the Bowles text, the Poisson's ratio for this soil was determined:

$\mu := 0.35$ Cohesionless, dense sand.

Taking the assumed internal angle of friction for poorly graded sand from Lindeburg, *Civil Engineering Reference for PE 8th Edition*, the critical depth can be found using Figure 3.10 from the Poulos and Davis text:

$\phi' := 38\ °$

$\phi := \frac{3}{4} \cdot \phi' + 10\ ° = 38.5\ °$

Zc / d ratio from Poulos & Davis, *Pile Foundation Analysis & Design*, Figure 3.10: = 10.5

$b_f := 14.9\ in$ $d := 14.2\ in$ HP14x117 steel pile.

$B_p := \sqrt[2]{(d^2) + (b_f^2)} = 20.583\ in$ Width of pile is diagonal

$A_p := b_f \cdot d = 211.58\ in^2$ Including Soil Plug

$z_c := 10.5 \cdot B_p = 18.01\ ft$ Critical Depth

$\sigma_p'' := D_w \cdot \gamma + ((z_c - D_w) \cdot \gamma') = 1714.27\ \frac{lb}{ft^2}$ Vertical Effective Stress at Critical Depth

South Foundation Design
(Case 3: Service I)

Calculation of bearing capacity factors ensues:

$G_s := \frac{E_s}{2 \cdot (1+\mu)} = 370370.37 \ \frac{lb}{ft^2}$

$c' := 0$ Cohesionless sand.

$I_r := \frac{G_s}{c' + \sigma_{z40}' \cdot \tan(\phi')} = 187.758$ Rigidity Index

Bowles, *Foundation Analysis & Design*, Table on P.894 : Rigidity Index Within Range for Sandy Soil

$\varepsilon_v := \frac{(1+\mu) \cdot (1-2 \cdot \mu) \cdot (\sigma_{z40}')}{E_s \cdot (1-\mu)} = 0.002$ Volumetric Strain

$I_{rr} := \frac{I_r}{1 + \varepsilon_v \cdot I_r} = 144.946$ Reduced Rigidity Index

$K_o := 1 - \sin(\phi') = 0.384$ At rest earth pressure coefficient

Vesic's Bearing Capacity Factors:

$\eta := \frac{1 + 2 \cdot K_o}{3} = 0.59$

$N_q := \frac{3}{3 - \sin(\phi')} \cdot \exp\left(\left(\frac{\pi}{2} - \phi'\right) \tan(\phi')\right) \cdot \left(\tan\left(\frac{\pi}{4} + \frac{\phi'}{2}\right)\right)^2 \cdot I_{rr}^{\left(\frac{4 \cdot \sin(\phi')}{3 \cdot (1 + \sin(\phi'))}\right)} = 134.709$

$N_\gamma := 0.6 \cdot (N_q - 1) \tan(\phi') = 62.679$

$N_{c1} := (N_q - 1) \cot(\phi') = 171.14$

$N_{c2} := \frac{4}{3} \cdot (\ln(I_{rr}) + 1) + \frac{\pi}{2} + 1 = 10.539$

$N_c := \text{if}(\phi' = 0, N_{c2}, N_{c1}) = 171.14$

$q_p := \eta \cdot \sigma_p'' \cdot N_q + \frac{1}{2} \cdot \gamma' \cdot B_p \cdot N_\gamma = 139779.292 \ \frac{lb}{ft^2}$ Pile Point Bearing Capacity

$P_p := A_p \cdot q_p = 205378.491 \ lb$ Pile Point Load Capacity

South Foundation Design
(Case 3: Service I)

Calculation of Side Friction Capacity:

α - Method calculations will be used to calculate the pile side friction capacity.

From Tomlinson, *Pile Design and Construction Practice*, Table 4.10, Large displacement pile due to Steel HP shape. Range is based on soil density, therefore with our dense sand assumption:

$$\frac{K_s}{K_o} = 1.25$$

$K_s := K_o \cdot 1.25 = 0.48$ Lateral Earth-Pressure Coefficient for Side Friction

From Tomlinson, Table 4.11, Smooth Steel Pile at interface with sand:

$\delta_p := 0.6 \cdot \phi' = 22.8\ °$ Angle of Friction Between Pile and Soil

$f_s := K_s \cdot \sigma_p'' \cdot \tan\left(\delta_p\right) = 346.199\ \frac{lb}{ft^2}$ Side Friction Stress, Considering Critical Depth

Calculation of side friction stress profile:

$z_1 := D_w$

$\sigma_{z1} := D_w \cdot \gamma = 1308\ \frac{lb}{ft^2}$

$f_{s12} := K_s \cdot \sigma_{z1} \cdot \tan\left(\delta_p\right) = 264.152\ \frac{lb}{ft^2}$

$z_c = 18.01\ ft$

Layers	Avg. Side Friction Stress
0-12 ft	$f_{s1Bar} := \frac{1}{2} \cdot \left(0 + f_{s12}\right) = 132.076\ \frac{lb}{ft^2}$
12-18 ft	$f_{s2Bar} := \frac{1}{2} \cdot \left(f_{s12} + f_s\right) = 305.176\ \frac{lb}{ft^2}$
18-30 ft	$f_{s3Bar} := f_s = 346.199\ \frac{lb}{ft^2}$

$P_{p1} := b_f + b_f + d + d = 4.85\ ft$ Perimeter of Pile (Including Soil Plug)

$P_s := P_{p1} \cdot \left(12\ ft \cdot \left(f_{s1Bar}\right) + 6\ ft \cdot \left(f_{s2Bar}\right) + 12\ ft \cdot \left(f_{s3Bar}\right)\right) = 36716.233\ lb$ Side Friction Capacity

South Foundation Design
(Case 3: Service I)

Calculations for compression and tension load capacity:

$L_p = 30\ ft$ Length of Pile

$W_p := 117\ \frac{lb}{ft} \cdot L_p = 3510\ lb$ Nominal Weight of pile type multiplied by pile length

From the *Standard Guidelines for the Design and Installation of Pile Foundations*, the factors of safety were determined from table A.1 and A.2. Assuming a pile group consisting of 12 piles, the design axial load will be distributed throughout these 12 piles.

$N := 12$ Number of Piles

$P_{g1} := \frac{P_g}{N} = 27.5\ ton$

$F_1 := 2.0$ Table A.1, Since this is a preliminary design, using driving formulas and static analysis to determine factors of safety

$F_2 := 1.1$ Table A.2, HP Pile

$FS := F_1 \cdot F_2 = 2.2$

$P := P_{g1} + W_p = 58510\ lb$ </= $P_{all} := \frac{P_p + P_s}{FS} = 110043.056\ lb$

Condition Satisfied

For uplift, the factor of safety is approximately a 50% increase from the compression capacity factor of safety.

$FS_T := 1.5 \cdot FS = 3.3$ Factor of Safety for Uplift

$T_{all} := \frac{P_s}{FS_T} + W_p = 14636.131\ lb$ Tension Capacity

Each individual pile satisfies this condition for both combined axial load and bending moment. This design criteria is calculated when calculating to see if the piles will buckle in the analysis of pile groups. This value is also shown in the summary.

South Foundation Design
(Case 3: Service I)

Pile settlement for an individual pile is as follows (Bowles Method):

$A_p := 34.4\ in^2$ AISC Table 1-4: HP 18x201

$E_p := 29000000\ \frac{lb}{in^2}$

$m := 1$ Shape Factor, m*Is = 1.0 (Relatively square pile)

$I_s := 1$

$\frac{L_p}{B_p} = 17.49$ Fox Embedment Factor Inequality Satisfaction

$I_F := 0.4$ Fox Embedment Factor

$F_1 := 0.25$ Reduction Factor, High side friction capacity compared to design load for an individual pile.

$q := \frac{P_{g1}}{A_p} = 1598.837\ \frac{lb}{in^2}$

$\delta_p := q \cdot B_p \cdot \left(\frac{1-\mu^2}{E_s}\right) \cdot m \cdot I_s \cdot I_F \cdot F_1 = 0.416\ in$ Point Bearing Settlement

For elastic settlement assume that the point load is equal to zero.

$P(z) := P_{g1} + \left(\frac{P_{g1}}{L_p}\right) \cdot z$

$\delta_E := \int_0^{L_p} \frac{P(z)}{E_p \cdot A_p}\, dz = 0.03\ in$ Elastic Shortening

$\delta_{pile} := \delta_p + \delta_E = 0.446\ in$ Total Pile Settlement

$B_p \cdot 0.03 = 0.617\ in$ Pile settlement should not be greater than 3% of pile diameter

Condition Satisfied

South Foundation Design
(Case 3: Service I)

Design Calculations for Pile Group

Vertical load capacity of the entire pile group:

$P_s = 36716.233\ lb$ Single pile

$P_p = 205378.491\ lb$ Single pile

$P_{Group} := N \cdot (P_s + P_p) = 2905136.678\ lb$ Capacity of Pile Group based on piles failing individually

$s := 2 \cdot B_p = 3.43\ ft$ Minimum Spacing Requirement

$s := 4\ ft$ Pile Spacing

$e := 0.5\ ft$ Edge Distance

$B_g := (2 \cdot s) + \left(\frac{(B_p)}{2} + e\right) + \left(\frac{(B_p)}{2} + e\right) = 10.715\ ft$ See Drawing for Dimensions

$L_g := (3 \cdot s) + \left(\frac{(B_p)}{2} + e\right) + \left(\frac{(B_p)}{2} + e\right) = 14.715\ ft$

$B_g := 12\ ft$ Simplified Dimension

$L_g := 15\ ft$ Simplified Dimension

$f_s := (f_{s1Bar}) + (f_{s2Bar}) + (f_{s3Bar}) = 783.451\ \frac{lb}{ft^2}$

$P_{gg} := (2 \cdot B_g) + (2 \cdot L_g) = 54\ ft$ Perimeter of Group

$A_g := L_g \cdot B_g = 180\ ft^2$ Area of Group

$P_{NBlock} := P_{gg} \cdot L_p \cdot f_s + A_g \cdot q_p = 26429463.147\ lb$ Block Failure Capacity (failure of entire group)

$P_{Ngroup} := min\left(P_{Group}, P_{NBlock}\right) = 2905136.678\ lb$

$FS := 3$ Assumed FS for pile group

$P_{allGroup} := \frac{P_{Ngroup}}{FS} = 968378.893\ lb$ >/= $P_g = 660000\ lb$

Condition Satisfied

South Foundation Design
(Case 3: Service I)

Predicted elastic settlement of the entire pile group:

$x := \frac{2}{3} \cdot L_p = 20\ ft$ Equivalent Footing Depth for Piles in Sand

$D_b := \frac{1}{3} \cdot L_p = 10\ ft$

$D_{bb} := \frac{2}{3} \cdot D_b = 6.667\ ft$ Starting of 4/1 Slope Stress Distribution

$B_{EQ} := B_g + 2 \cdot \left(\frac{D_{bb}}{4}\right) = 15.333\ ft$ $L_{EQ} := L_g + 2 \cdot \left(\frac{D_{bb}}{4}\right) = 18.333\ ft$

$q_{net} := \frac{P_g}{{B_{EQ}}^2} = 2807.183\ \frac{lb}{ft^2}$

Strain Influence Factor Method:

$z_1 := B_{EQ} \cdot \left(0.5 + 0.555 \cdot \left(\frac{L_{EQ}}{B_{EQ}} - 1\right)\right) = 9.332\ ft$ </= $B_{EQ} = 15.333\ ft$ OK

$z_2 := B_{EQ} \cdot \left(2 + 0.222 \cdot \left(\frac{L_{EQ}}{B_{EQ}} - 1\right)\right) = 31.333\ ft$ </= $4 \cdot B_{EQ} = 61.333\ ft$ OK

$I_{z0} := 0.1 + 0.0111 \cdot \left(\frac{L_{EQ}}{B_{EQ}} - 1\right) = 0.102$ </= 0.2 OK

$\sigma_{zp}' := \gamma \cdot D_w + \gamma' \cdot \left(\left(x - D_w\right) + D_{bb} + z_1\right) = 2930.287\ \frac{lb}{ft^2}$ Vertical Effective Stress at z1 (Before Installation)

$I_{zMax} := 0.5 + 0.1 \cdot \sqrt[2]{\frac{q_{net}}{\sigma_{zp}'}} = 0.598$ All sand layer

$\Sigma = I_{zBar1} \cdot \frac{\Delta z_1}{E_s}$

Layer 1

$\Delta z_1 := z_1 = 9.332\ ft$ $E_s = 500\ \frac{ton}{ft^2}$ $I_{zBar1} := \frac{I_{z0} + I_{zMax}}{2} = 0.35$ $\Sigma_1 := 2.4258 \cdot 10^{-6}\ \frac{ft^3}{lb}$

Layer 2

$\Delta z_2 := z_2 - z_1 = 22.001\ ft$ $E_s = 500\ \frac{ton}{ft^2}$ $I_{zBar2} := \frac{I_{zMax}}{2} = 0.299$ $\Sigma_2 := 6.2068 \cdot 10^{-6}\ \frac{ft^3}{lb}$

$\Sigma := \Sigma_1 + \Sigma_2 = 0.0000086326\ \frac{ft^3}{lb}$

$\sigma_z' := \gamma \cdot D_w + \gamma' \cdot \left(\left(x - D_w\right) + D_{bb}\right) = 2299.467\ \frac{lb}{ft^2}$ Vertical Effective Stress at Equivalent Footing

$C_1 := 1 - 0.5 \cdot \left(\frac{\sigma_z'}{q_{net}}\right) = 0.59$

$t := 50$ Years

$C_2 := 1 + 0.2 \cdot \log\left(\frac{t}{0.1}\right) = 1.54$

$\delta_{pg} := C_1 \cdot C_2 \cdot q_{net} \cdot \Sigma = 0.264\ in$ Elastic Settlement of Pile Group

South Foundation Design
(Case 3: Service I)

Elastic Shortening:

$P_{top} := P_{g1} = 55000\ lb$

$A_T := s^2 = 16\ ft^2$ Tributary Area of Each Pile

$P_{bot} := q_{net} \cdot A_T = 44914.934\ lb$

$L_{p1} := x + D_{bb} = 26.667\ ft$

$P(z) := P_{top} + \left(\frac{P_{top} - P_{bot}}{L_{p1}}\right) \cdot z$

$A_p = 34.4\ in^2$

$\delta_E := \int_0^{L_{p1}} \frac{P(z)}{E_p \cdot A_p}\, dz = 0.019\ in$ Elastic Shortening

$\delta_{PileGroup} := \delta_{pg} + \delta_E = 0.284\ in$

Satisfies requirement of a max settlement of 1 in.

South Foundation Design
(Case 3: Service I)

Allowable Lateral Load Calculations (Brom's Method):

Assume Fixed-Head, Long Pile (Tomlinson)

$B_p = 20.583\ in$

$e := 27\ ft$

$\frac{e}{B_p} = 15.741$

$I_x := 1220\ in^4$

$S_x := 172\ in^3$

$I_y := 443\ in^4$

$S_y := 59.5\ in^3$

$F_y := 50000\ \frac{lb}{in^2}$ Yield Stress of Pile

$M_y := S_y \cdot F_y = 247916.667\ lb \cdot ft$ Yield Moment

$K_p := \tan\left(45° + \frac{\phi'}{2}\right)^2 = 4.204$

$\frac{M_y}{K_p \cdot \gamma \cdot B_p^{\ 4}} = 62.511$ Figure 7.12 from Poulous and Davis

$V_u := 9 \cdot K_p \cdot \gamma \cdot B_p^{\ 3} = 20810.022\ lb$ Ultimate Load

$FS := 1.67$ Factor of Safety for Steel

$V_{all} := \frac{V_u}{FS} = 12461.091\ lb$ >/= $V_{gx} = 1000\ lb$

Condition Satisfied

South Foundation Design
(Case 3: Service I)

Pile Buckling Calculations:

See AutoCAD drawing for pile arrangement

Overturning Moment (N-S) Only

$y_1 := s + \frac{s}{2} = 6\ ft$ $y_2 := \frac{s}{2} = 2\ ft$ $y_3 := y_2$ $y_4 := y_1$

$\Sigma y_{squared1} := 2 \cdot \left(3 \cdot \left(y_1^2 + y_2^2\right)\right) = 240\ ft^2$

$P_1 := \frac{P_g}{N} + \frac{M_{gy} \cdot y_1}{\Sigma y_{squared1}} = 55200\ lb$ </= $P_{all} := \frac{P_p + P_s}{FS} = 144966.9\ lb$

$P_2 := \frac{P_g}{N} + \frac{M_{gy} \cdot y_2}{\Sigma y_{squared1}} = 55066.667\ lb$ </= $P_{all} := \frac{P_p + P_s}{FS} = 144966.9\ lb$

$P_3 := \frac{P_g}{N} - \frac{M_{gy} \cdot y_3}{\Sigma y_{squared1}} = 54933.333\ lb$ </= $P_{all} := \frac{P_p + P_s}{FS} = 144966.9\ lb$

$P_4 := \frac{P_g}{N} - \frac{M_{gy} \cdot y_4}{\Sigma y_{squared1}} = 54800\ lb$ </= $P_{all} := \frac{P_p + P_s}{FS} = 144966.9\ lb$

All piles satisfy axial load capacity

$A := 34.4\ in^2$ $F_y := 50000\ \frac{lb}{in^2}$

$P_d := P_1 = 55200\ lb$ Maximum Axial Load Controls

$\sigma_d := \frac{P_d}{A} = 1604.651\ \frac{lb}{in^2}$

$\sigma_{all} := 0.35 \cdot F_y = 17500\ \frac{lb}{in^2}$ *Standard Guidelines for the Design and Installation of Pile Foundations*, (35% of Yield Stress)

σ_d </= σ_{all}

Criteria is satisfied. Piles will not buckle.

South Foundation Design
(Case 3: Service I)

Design Summary

All requirements were satisfied for the design of the individual piles and the pile group. The piles are HP 14x117 and 30ft long. The pile cap contains 12 piles that are spaced at 4ft with a typical distribution of an isolated pile cap. Pile axial capacity is analyzed for each individual pile and passes the requirement for axial load. When combining axial load and bending moment the capacity is also satisfied and is shown in the analysis of pile groups portion of the calculations. The pile cap will be a 2ft thick concrete slab, which rests on top of the pile. This cap is a 12' x 15' rectangular block.

Engineering Costs:

Item	Hours	Salary/hr	Overhead Mult	Cost
Planning and Data Collection	90	$ 30.00	2.5	$ 6,750.00
Bridge Design	250	$ 30.00	2.5	$ 18,750.00
Design Alternatives	150	$ 30.00	2.5	$ 11,250.00
Report and Presentations Production	50	$ 30.00	2.5	$ 3,750.00
Meetings	15	$ 30.00	2.5	$ 1,125.00
Travel, Material, Supplies				$ 100.00
			Total	**$ 41,725.00**

Substructure	Unit	Quantity	Price/Unit	Total
HP18X204 Steel Piles (ft)	60	60	60	$ 216,000.00
HP14X117 Steel Piles (ft)	30	12	40	$ 14,400.00
CC Pile Caps/Anchors (yd^3)	130	2	200	$ 52,000.00
Excavation (yd^3)	210	1	60	$ 12,600.00
Crane+Operator(hr)	40	2	350	$ 28,000.00
Deisel Hammer(week)	1	2	1380	$ 2,760.00
Laborers(hr)	40	4	19.4	$ 3,104.00
Piledrivermen (hr)	40	2	24.15	$ 1,932.00
Concrete Finisher (hr)	16	4	24.8	$ 1,587.20
Construction Inspector/Engineer(hr)	40	1	60	$ 2,400.00
Carpenter (hr)	16	2	24.15	$ 772.80
Rebar (ft^2)	2400	1	0.2	$ 480.00
			Total	**$ 336,036.00**

Superstructure/Pylon/App	Unit	Quantity	Price/Unit	Total
W36X853 Steel Beams (ft)	525	2	383.85	$ 403,042.50
CC Deck 8" depth x 10' wide(yd^3)	132	1	200	$ 26,400.00
Deck Rebar (ft^2)	5250	1	0.2	$ 1,050.00
Stay Cables (ft)	8850	2	3	$ 53,100.00
Pylon/Pier CC (yd^3)	2	120	200	$ 48,000.00
Elevator Tower Steel Framing W4x13(ft)	630	1	17	$ 10,710.00
Pylon/Pier Rebar (ft^2)	3500	2	0.6	$ 4,200.00
Elevator	1	1	108965.75	$ 108,965.75
Ramp Approach CC(yd^3)	375	1	200	$ 75,000.00
Ramp Approach Rebar (lb)	18000	1	1	$ 18,000.00
Steel Paint (LS)	1	1	35000	$ 35,000.00
Ramp Approach Stirrups	1	160	2	$ 320.00
Laborers(hr)	400	8	19.4	$ 62,080.00
Carpenter (hr)	400	2	24.15	$ 19,320.00
Construction Inspector/Engineer(hr)	400	2	60	$ 48,000.00
Roller Support	1	2	1000	$ 2,000.00
Tower Support	1	2	15000	$ 30,000.00
Superintendent(hr)	400	2	50	$ 40,000.00
Welder(hr)	350	3	25.25	$ 26,512.50
Electrician(hr)	40	2	22.25	$ 1,780.00
Equipment Operator	400	2	29.05	$ 23,240.00
Crane Oiler/Spotter (hr)	400	1	24.15	$ 9,660.00
Concrete Finisher (hr)	80	4	24.8	$ 7,936.00
Crane+Operator(hr)	400	1	350	$ 140,000.00
Saftey Fencing (/7ft)	186	1	270	$ 50,220.00
Deck Lighting (fixture)	1	12	250	$ 3,000.00
Structure Lighting (fixture)	1	6	600	$ 3,600.00
Elevator Tower Windows	1	8	4000	$ 32,000.00
Bridge Steel Handrail (ft)	650	2	80	$ 104,000.00
Precast CC Steps (step)	30	1	650	$ 19,500.00
Elevator Tower Roof (ft^2)	634	1	12	$ 7,608.00
CC Stair Landings	1	3	150	$ 450.00
Elevator Tower Steel Framing W4x13(ft)	164	1	17	$ 2,788.00
Elevator Tower Steel Framing W8x10	110	1	12	$ 1,320.00
			Total	**$ 1,418,802.75**

Site Design	Unit	Quantity	Price/Unit	Total
Land Grading (ft^2)	20000	1	1.5	$ 30,000.00
Subbase (yd^3)	484	1	15	$ 7,260.00
Surveying Crew(hr)	60	1	96	$ 5,760.00
4 Equipment Operator(hr)	160	2	29.05	$ 9,296.00
Laborers(hr)	160	6	19.4	$ 18,624.00
Concrete Finisher (hr)	8	4	24.8	$ 793.60
Construction Inspector/Engineer(hr)	160	1	60	$ 9,600.00
Superintendent(hr)	160	1	50	$ 8,000.00
Lamposts	1	3	1000	$ 3,000.00
Silt Fence (ft)	600	1	1	$ 600.00
Native Grass (acre)	0.3	1	2000	$ 600.00
Trees	1	6	150	$ 900.00
CC Sidwalk (yd^3)	30	1	100	$ 3,000.00
Rebar (ft^2)	700	1	0.2	$ 140.00
			Total	**$ 97,573.60**

Cost before profit	**$ 1,894,137.35**
Cost Multiplier	**2.5**
Total Project Cost	**$ 4,735,343.38**

Notes:

1. Concrete Price includes forms
2. Cost of Cables is unkown and estimated from a much smaller product. Will contact manufacturer for specific product cost.
3. Elevator Cost includes installation
4. Survey Crew Includes 2 Liscensed Surveyors $30/hr and 2 untrained assistants @$18/hr.

Project Cost Breakdown	
Category	**Cost**
Substructure	**$ 840,090.00**
Superstructre+Approaches	**$ 3,547,006.88**
Site Design	**$ 243,934.00**
Contigency (10%)	**$ 463,103.09**
Total Cost	**$ 5,094,133.96**

Project Cost Breakdown

Category	Cost
Substructure	$ 850,000.00
Superstructure+Approaches	$ 3,600,000.00
Site Design	$ 250,000.00
Contigency (10%)	$ 470,000.00
Total Cost	$ 5,170,000.00

Cost Est

Substructure

Excavation: 2.89 yd³ + 80 piles

Piles → using augur:
(60.2 in²) →

$$\frac{60.2}{144} \cdot 25' = \frac{10.45}{27} = .4\,yd^3$$

$$.4 \cdot 80 = 32\,yd^3$$

Total Excavation = 178 + 32 = **210 yd³**

(178 ← Excavator; 32 ← augur)

Labor: Zone 4

Super Structure / Pylon

W36X853 → 3

price/1ft = .45 · 853 = $383.85

CC Deck: $\frac{8}{12} \cdot 525 = \frac{351.75}{27} = 13\,yd^3 \cdot 10$
10' wide   132 yd³

Stay cable: Total Length From Excel

Pylon Tower: $(235' \cdot 5' \cdot 10' \cdot 4.150)\ \frac{}{27} = 240\,yd^3$

↓ Area = (27.5·(10+5)·2) + (3·4·150) = 3000²

Super Structure

Elevator

Stairs: 27' up ≈ 30 steps

Stair Landing pads (10'×10'×.5')·2

= 4yd³ @$130/yd³ + $20 rebar

windows - 12'×12'

Project Site

Silt Fence: 140' × 175' × 225' = 540'

head wid H.span

Sidewalk: 140' × 5' × .5'

Length width Depth

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