Spring 2023

Pedestrian Bridge for Discovery & Innovation

Matthew Hale

Dylan Stegman

Jake Porter

SANTA CLARA UNIVERSITY

Department of Civil, Environmental, and
Sustainable Engineering

I hereby recommend that the
SENIOR DESIGN PROJECT REPORT
prepared under my supervision by

MATTHEW HALE
DYLAN STEGMAN
&
JAKE PORTER

entitled

PEDESTRIAN BRIDGE FOR DISCOVERY & INNOVATION

be accepted in partial fulfillment of the requirements for
the degree of

BACHELOR OF SCIENCE IN CIVIL ENGINEERING

Advisor — 6/15/2023
Date

Department Chair — 6/14/2023
Date

PEDESTRIAN BRIDGE FOR DISCOVERY & INNOVATION

by

MATTHEW HALE
DYLAN STEGMAN
&
JAKE PORTER

SENIOR DESIGN PROJECT REPORT

submitted to
the Department of Civil, Environmental,
and Sustainable Engineering

of

SANTA CLARA UNIVERSITY

in partial fulfillment of the requirements
for the degree of
Bachelor of Science in Civil Engineering

Santa Clara, California

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MATTHEW HALE, DYLAN STEGMAN, & JAKE PORTER

Department of Civil, Environmental, and
Sustainable Engineering
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ACKNOWLEDGMENTS

Throughout this project we were fortunate enough to receive assistance from not only our University's professors, but also local engineering and design professionals. We would not have achieved the level of success we achieved on this project without our structural and geotechnical design professors; Tracy Abbott, and Sukhmander Singh, as well as engineering and design professionals; Ray Fassett, Nick Ramirez, and Alec Nicholas.

In addition,

We would also like to thank the City of San Jose as well as the Children's Discovery Museum of San Jose for lending us their time to meet with our project's design group and discuss their interest regarding the project. In particular, a special thanks to the Children's Discovery Museum Staff; Richard Turner and Marilee Jennings.

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ABSTRACT

A new pedestrian bridge was designed adjacent to the Children's Discovery Museum of San Jose spanning across the Guadalupe River in the Downtown San Jose area. This design includes structural, geotechnical, and construction components. The architectural aspects of the bridge include two intertwined mass timber arches, galvanized steel cables, and a deck consisting of wide flange I-beams and a proprietary structural glass deck system. These features allow the bridge to seamlessly integrate into the surrounding environment as well as expose structural components of a pedestrian bridge that would otherwise be covered by a traditional bridge deck. All disciplinary components of the bridge were constructed for all relevant California building and design codes. Some assumptions for non-typical structural design components were made in order to ensure a conservative and safe design, and as such design strength values may exceed the minimum requirements outlined by the relevant design provisions. This bridge design provides both; a symbolic connection between the youth of San Jose and its larger community, as well as a corridor between private development and the Children's Discovery Museum of San Jose. The bridge design promotes engagement in civil, environmental, and sustainable engineering amongst middle and high school aged students therefore increasing participation in engineering in upper-level education. The Pedestrian Bridge for Discovery & Innovation is a landmark structure that will improve the existing site, educate the local community, and add to the character of Downtown San Jose.

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INTRODUCTION

There is perhaps no other structure that resonates better with the field of civil engineering than a bridge. Bridges have been designed for many reasons throughout history including but not limited to: connecting communities, providing quicker methods of transportation, avoiding obstacles, and creating economic corridors. In addition to the practicalities of bridges, the best bridges are also pleasing to the eye. This is especially prevalent in pedestrian bridges due to the relatively small weight they must support. Pedestrian bridges only have to hold the weight of pedestrians so they require less materials. This means that more of the costs can be allocated to design and fabrication of unique and often intricate parts. Many bridges are designed by famous architects and designers and act as outlets for creativity and artistry. They can often be symbols of communication, union, and technological progress. Some examples of this are the Gateshead Millennium Bridge in London, the Peace Bridge in Calgary, and the Sundial Bridge in Redding, California. These all are attributes that not only align with the traits of successful civil engineers, but also the communities that are connected by these bridges. Then, it should not come as a surprise that in devising a project that could best showcase this team's skills as civil engineering students and the usefulness to the immediate community that the team chose to design a pedestrian bridge.

While it is important for any design team to be passionate about their project, the bottom-line is that there must be a significant issue or opportunity that the project is addressing, especially in the field of civil engineering where projects are costly and the hours spent working by civil engineers is in ever-increasing need to be allocated more precisely. There are less students studying civil engineering in comparison to not only years prior, but also in regards to other engineering disciplines. The apparent lack of interest is a plight that is exacerbated by public marketing and funding in other engineering industries that tend to overshadow that of civil engineering. While other fields might showcase advances in extraterrestrial travel or the latest achievement in the design of computers and tablet technology, the typical civil engineering project is much more sedate in comparison. This explains why children are driven towards these more flashy engineering disciplines from a young age, however, this is not to say that there are no innovative and inspiring examples of civil engineering out there.

Although there are incredible achievements in civil engineering across the world, many are inaccessible or unapproachable for the typical middle school or highschool aged student. Fortunately, there are institutions like the Children's Discovery Museum of San Jose that provide an opportunity for students to examine hands on many different science, technology, engineering, and mathematics (STEM) exhibits. The museum has been around since 1990 and has allowed for the type of foundationary learning experience that inspires and excites children to pursue careers in engineering fields. Ironically, the grounds of the museum currently lack any type of civil engineering exhibit, while there are several that represent electrical, mechanical, and aerospace engineering fields. The team wanted to address this issue while also taking advantage of an opportunity presented by the museum leadership team.

The team's proposal was to build a pedestrian bridge across the Guadalupe River that will connect the *Discovery Lawn* of the Children's Discovery Museum of San Jose with an existing parking lot and future development. The bridge will span over 120 feet across the river and will feature a curved steel and glass deck that is suspended by galvanized steel cables connected to twin mass timber arches. The resulting structure will be an awe-inspiring design that will grab the attention of not only students, but the community of San Jose as well. Along the railings on either side of the deck there will be infographic plaques and screens. These plaques and screens will allow visitors to learn more about the different structural aspects of the bridge as well as some additional facts about civil and structural engineering as a whole.

Figure 1: Site overview and proposed location of pedestrian bridge

In terms of an opportunity, such a pedestrian bridge will aid in educating the next generation of civil engineers, but it will also become a landmark structure for the museum and the City of San Jose. When meeting with museum directors, the plan of relocating the entrance of the museum was explained to the team. In the past, families have had trouble finding the entrance of the museum. To fix this issue, there is a proposal to move the entrance to the Northeast side of the building, the side facing the Guadalupe River. With this, the museum staff would like to see a new pedestrian bridge built over the river connecting to a path which will lead directly to the proposed entrance. It will provide much needed direction to the museum grounds as it becomes the newest and most direct entrance to the museum, not to mention that it will link future outdoor exhibits that the museum plans to expand upon with their existing interior attractions. In addition, the bridge will give the museum more exposure by attracting people from the new mixed use development to come check out the museum or at least learn more about it. The bridge will also exhibit many of the architectural traits that make the museum special as well as those of the proposed development across the Guadalupe River. This is a nod to the works of the renowned architects who design these structures as well as careful planning to tie into existing infrastructure. The bridge will stand out, but it will not take away from the existing pathways and surrounding environment.

By the end of this project, the team delivered the following design considerations, including: a set of architectural and structural plans for the bridge, structural analysis and calculations for all structural members and foundation, and an additional analysis of the constructability and cost of the bridge. In addition to these items, the team also performed and included an analysis and calculation of some of the connections used in constructing the bridge. All software calculations included hand calculations to ensure the validity and reliability of the computer results. The team also built a complete digital model of the bridge that portrays the serviceability and deflection of the bridge under different loading scenarios that can be used for later loading iterations as well. Along with these deliverables, the team also completed the generic requirements required for a Senior Capstone Project, including: a thesis, presentation, and project report for the designated advisors.

COMPARATIVE ALTERNATIVE ANALYSIS

There are currently few educational and easily accessible exhibits of both structural engineering and minimally invasive architecture. As a forward-thinking community, we require a structure which embodies these characteristics in order to inspire younger generations, and to aid students pursuing higher education in understanding engineering concepts through real-world applications. First, the team had to find a location/client in need of a new bridge or repairs to an existing bridge. The Children's Discovery Museum has an existing pedestrian bridge that crosses the Guadalupe River, but the team believed that they might benefit from a new design or a new bridge all together. The list of alternatives, shown below, were evaluated based on cost, environmental impact, longevity, aesthetic value, accessibility, design and construction time, maintenance, client input/feedback, and technical feasibility. Although these values intersect with the values of many potential clients regarding similar projects, there is no way of knowing the client's particular values without meeting and communicating with them directly.

In October of 2022, the team was fortunate enough to meet with the Executive Director and Director of Exhibits of the Children's Museum of San Jose. Although it may have occurred under the false pretense that they were a professional design team, the museum presented this team with a detailed list of requirements and recommendations regarding the project. The team discovered that the Board of Directors responsible for the funding and upkeep of the museum had been considering a design similar to this team's pedestrian bridge design for the last few years. It was stated that there was not only a need to change the flow of traffic, but also a need to introduce a new interactive exhibit to liven the area surrounding the backside of the museum. As such, the team was presented with a unique opportunity in regards to most other student design projects. Before the team had begun the alternative analysis required for the prerequisite senior design class, they had conducted a thorough and detailed alternative analysis with the client. The outcome of the team's previous work was that their academically-required alternatives analysis became easily streamlined.

The below listed options describe the alternatives the design team considered.

<u>Option (1): Status Quo:</u>

The status quo alternative implies that there will be nothing done to the existing bridge and there will not be a design of any proposed bridge. This option is very economical and sustainable since it does not require any new materials. Also, this option is very technically feasible and would not take any time. It scores low on longevity compared to the other alternatives since the existing bridge already shows signs of cosmetic wear and tear. Social impact and aesthetic value are low compared to the others because the existing bridge is not aesthetically pleasing. While it may not look pretty, this design has withstood the test of time and currently has not shown any signs of not being able to handle the local traffic or museum demands. It could be argued that the need for an option that is alternative to this one, reflects either the museum's or the team's own desire to improve the site or meet the demand of future development and expansion.

As there is a $200 million development slated for 2024 lying directly across the Guadalupe River, it would be unlikely that the existing bridge would be able to satisfy the increased traffic, not to mention the fact that tickets to the museum could increase in demand as tenants moved into the nearby mixed-use development. This adds substance to the claims of the client for the need of increased and directed infrastructure towards the museum. Leaving the site as is may not be an inadequate solution for the time being, however, there is a high chance that increased infrastructure will be required in the near future.

Option (2): Retrofit Existing Bridge:

Alternative two (2) involves a retrofitting of the existing bridge near the Children's Discovery Museum of San Jose. This option would be very economical and sustainable because it requires less materials than building a new bridge, while being very practical. The redesign would also improve the longevity of the existing bridge, however, the social impact and aesthetic value would be limited due to restrictions on the retrofit design. This option does not greatly change the accessibility or maintenance required and it will also take less time than designing a new bridge. It does not fully satisfy the client's input/feedback, but still remains a relatively easy and technically feasible alternative. If the client wished for the design team to go along with the cheapest option besides leaving the site as it currently exists, then retrofitting the existing bridge may be an adequate option.

There are some drawbacks of this option, including the fact that retrofitting the bridge may be putting a bandaid on an issue that is not bandage-able. One of the main concerns of the museum is that there will be increased traffic. They never mentioned that the existing bridge was in need of a structural retrofit. As there is nothing wrong structurally with the existing bridge, a non-structural retrofit is what this option is considering. Any retrofit having to do with handling an increased traffic demand could include: adding another deck to the bridge, or even widening the existing deck. Both of these options would cost nearly as much as constructing a new bridge, while requiring advanced engineering to see if this design is even feasible. The worst knock on this option is the fact that throughout the construction of a retrofit, there will be no pedestrian crossing for quite some distance across the Guadalupe River.

<u>Option (3): Remove Existing Bridge/ Design New Bridge:</u>

Alternative three (3) involves removing the existing bridge and replacing it with a new, innovative, and aesthetically pleasing design. This option has the lowest economical and sustainable ranking because it requires completely removing a functional bridge that could be used elsewhere, however, a new design would be optimal for longevity and aesthetic value. It would also have a beneficial social impact for the community by educating children and families on structural engineering and the environment. This is an option that may potentially attract more people to the Children's Discovery Museum of San Jose and add to the value of Downtown San

Jose. The accessibility and maintenance would relatively remain the same and it would not take as long to complete compared to other alternatives that require relocating the existing bridge. This alternative would also satisfy the client's input/feedback that the team received in previous meetings.

An addendum to this alternative, and also what the team ultimately decided to go along with is to allow the existing bridge to stay as well as design a new bridge. This way they are able to maintain the existing traffic demand as well as create an exhibit that will satisfy an important social demand that exists in the local community. An important aspect involved in leaving the existing bridge is to allow the required personnel to access both sides of the bridge during construction. This will reduce the need for excess construction costs including having to construct a scaffolding system or temporary bridge to access both sides of the river. This is not a requirement that the team was able to discuss with the client, but it ended up being an important consideration later on in their project, especially when considering the constructability of a new bridge in this location and the requirements of the site’s topography.

Option (4): Relocate Existing Bridge/ Design New Bridge:

Alternative four (4) requires relocating the existing bridge to a location in need of a pedestrian crossing and replacing it with a new design. This is a very economical and sustainable option because it makes good use of the existing bridge by relocating it for the community to use. It would also increase longevity and have a large social impact for the community by educating children and families on structural engineering and the environment, and providing a new location for pedestrians to cross. This would be optimal for aesthetic value and would fully satisfy the client's input/feedback, without changing the accessibility, however, this alternative is less technically feasible, would take the longest to complete, and would require the highest construction cost due to the relocation of the existing bridge.

The one real drawback of this option is the logistics of transporting a bridge of the size of the one that currently exists. There will need to be special precautions taken when transporting the bridge and it will have to be transported in a way that preserves its structural integrity. This is especially difficult to do when considering that the existing bridge is of reinforced concrete construction. This type of construction has a very high level of rigidity and would not handle excess vibration well, such as the type that arises from transporting it on a highway. This means that the bridge would have to be specially inspected both before and after being transported with no guarantee that the bridge will still be serviceable when it arrives at its proposed location.

Option (5): Find New Location in Need of a Bridge:

Alternative five (5) involves finding a new location in need of a bridge that would benefit the local community. This option has an average ranking of three (3) for economical and sustainability because it is unclear how these criteria would be affected by changing locations. It would also have little impact on longevity considering the team retrofits an existing bridge or designs a new one. This alternative has one of the lowest rankings for social impact and aesthetic value because it is difficult to find an alternative location where the community would gain the same benefits that would be achieved through embodying the education and innovation of the Children's Discovery Museum. The accessibility would not change provided the team finds another public location. Although this alternative would require more planning, design, and construction time, it is still technically feasible. It is unclear how this option would affect maintenance, and it completely neglects the client's input/feedback.

The 10 different criteria used were ranked one through five (1-5) based on their importance to the project. The economical, time, maintenance, and technical feasibility were the lowest criteria ranked at two (2) out of five (5). Sustainability and client input/feedback were the next most important criteria ranked three (3) of five (5). Longevity, aesthetic value, and accessibility received a ranking of four (4), and the most important criteria was social impact. The final score for each alternative was found after ranking each alternative in every category then taking into account the weighted value of each criteria.

After evaluating the four alternatives against the criteria, alternative four (4) received the highest ranking of 111 due to the high scores it received from economical, longevity, social impact, aesthetic value, accessibility, and client input/feedback. Alternative five (5), however, received low scores for time, maintenance, and technical feasibility. The next best option was alternative three (3) with a ranking of 103 points. This alternative had an average score of three (3) in the majority of the categories, but received the lowest score for sustainable. The worst option was alternative five (5) with a ranking of 77 points. It was difficult to evaluate alternative five (5) because it requires finding a new location for the bridge which can drastically change the criteria rankings. It is clear from the analysis of alternatives that alternative four (4), relocating the existing bridge and designing a new bridge, is the best choice and will have a large social impact on the community.

Table 1: Criteria and Alternatives.

Option:	Category, (value):		
Alt.	Economical Value: (2)	Sustainability: (3)	Design and Construction Cost: (2)
(1)	5	5	5
(2)	4	4	4
(3)	3	1	2
(4)	4	3	1
(5)	3	3	4

Table 1: Criteria and Alternatives (Cont.).

Option:	Category, (value):		
Alt.	Structural Longevity: (4)	Social Impact: (5)	Aesthetic Value: (4)
(1)	3	0	1
(2)	4	2	3
(3)	4	3	5
(4)	4	4	5
(5)	4	1	2

Table 1: Criteria and Alternatives (Cont.).

Option:	Category, (value):		
Alt.	Accessibility: (4)	Design and Construction Time: (2)	Maintenance Cost: (2)
(1)	4	5	3
(2)	4	3	3
(3)	4	3	3
(4)	4	2	1
(5)	4	2	2

Table 1: Criteria and Alternatives (Cont.).

Option:	Category, (value):	
Alternative	Client Design Input: (3)	Technical Design Feasibility: (2)
(1)	1	5
(2)	1	4
(3)	3	3
(4)	4	2
(5)	1	3

Table 2: Criteria and Alternatives (Totals).

Totals:	
Option (1)	91
Option (2)	102
Option (3)	105
Option (4)	112
Option (5)	81

DESIGN CRITERIA AND PERFORMANCE REQUIREMENTS

The project, which is primarily the design of the bridge, has two main components which require adherence to applicable codes and standards; the structural design and the construction plan. The structural design can be broken down into different pieces which require different codes and regulations, but the main code that had to be utilized was the 2020 Caltrans Highway Design Manual (HDM). The pedestrian bridge section of this code works in conjunction with and is supplementary to the American Association of State Highway and Transportation Officials 2009 Load and Resistance Factor Design (AASHTO LRFD) Guide Specification for the Design of Pedestrian Bridges. These in addition to the general AASHTO LRFD Bridge Design Guide are the main codes used for general design requirements, such as required loading and dimensions. In addition to these codes, the bridge will consist of different materials, such as steel framing members. The design of these structural members in addition to any steel connectors were done in accordance with specifications in the 2016 American Institute of Steel Construction (AISC) Manual for Steel Construction. The bridge will consist of two large timber arches and the design of these follow the 2018 National Design Specification for Wood Construction (NDS). In addition to these general codes and specifications, more specific and niche design guides were utilized for the designing of curved steel members. Aside from these specific structural design codes, the bridge must comply with accessibility codes and local ordinances such as the Americans with Disabilities Act (ADA). Since the bridge spans over the Guadalupe River, it must also comply with the 2023 California Code of Regulations (CCR), specifically, about flood protection. Since the project was designed above the floodplain (near top of bank), the bridge will not impede the flow of the river and hence will not be subject to some guidelines within the CCR.

The construction plan of the bridge must also comply with various codes and regulations related to transportation and accessibility. The proposed bridge design adheres to both; the 2020 Caltrans Highway Design Manual, and AASHTO, which each have regulations that apply to any kind of construction related to transportation. This is essential as the bridge design will act as an essential corridor across the Guadalupe River that will be subject to daily use of pedestrian and cyclist traffic. Some future precautions would have to be taken including: conducting a traffic study on the nearby existing pedestrian bridge, as well as creating an emergency egress plan for the event of a partial structural failure. If time allowed, this would become part of a package presented to the City of San Jose Building Department and local Fire Marshall to be approved and permitted for pedestrian use. In addition to these items, there are various accessibility standards regarding the width of the bridge's walkways/arteries, slope of the bridge deck, grip of the decking materials, railing heights, fencing types, and height of obstructions that could pose a danger to moving traffic. These sizing regulations have all been accounted for in the design, they are reflected in the bridge's ultimate dimensions, and have been shown on all relevant planset drawings.

There are codes that must be adhered to regarding zoning and local ordinances, such as the 2023 City of San Jose Local Ordinances & Zoning Code and the 2023 San Jose Municipal Code. The construction of the bridge would have to comply with the Occupational Safety and Health Administration of the United (OSHA) regulations for public and worker safety, but since the scope of the project did not include a detailed construction plan, many of these local and national regulations were not accounted for. That scope would involve a site specific logistics plan and construction phasing plan that would have to be approved by the City of San Jose. The Santa Clara Valley Water District would also need to be notified of the construction as the Guadalupe River, which the bridge spans across, is under the jurisdiction of the aforementioned authority.

When projects of this nature are proposed, the project would certainly be subject to environmental and sustainability regulations. The most significant one would be the California Environmental Quality Act (CEQA). CEQA is a statute that requires the state and local agencies to address and mitigate the significant environmental impacts of proposed projects. A large project that proposes significant alterations to riparian and adjacent environments (such as a pedestrian bridge, river crossing) would certainly invoke CEQA. One of the results of CEQA being invoked is that an external agency would have to prepare a study on the existing environment as well as the environmental impact of the pedestrian bridge. This will be produced as an Environmental Impact Report (EIR) for projects under the state or local jurisdiction (CEQA) or as an Environmental Impact Statement (EIS) for projects that are under national jurisdiction (NEPA). These studies on the existing environment are important because they ensure that one of the main goals of CEQA is withheld; ensuring that existing environmental conditions are protected.

A project of this size would also require that a notice to the public be released regarding the proposed project and its environmental impact. This is not only presented by allowing the public access to the Environmental Impact Report (EIR), but also in the form of public hearings where concerned individuals or organizations will get a chance to voice their concerns and opinions regarding the project as well as any proposed environmental mitigation measures. This is significant because if the public feels that the project and the design team have not taken adequate measures to reduce its impact on the environment, they have the right to challenge the responsible agency's decision in court. While the ultimate decision would fall with the lead agency responsible for the project, the public does have much more say over the project then they would have if these environmental regulations did not exist.

In addition to adhering to all of these codes and regulations, the design was informed by the team's core values. These main values were; (1) safety, (2) longevity, and (3) the social impact on children visiting the museum. It goes without saying that the structure needs to be safe for anyone to use, especially, during a variety of different loading sources and patterns. The main loads that the team considered for this project were gravity and earthquake loading, however, there are some more unusual loading cases the team also had to acknowledge. For example, an unequal loading of large groups of people must be considered, such as a school field-trip group all looking over one side of the bridge or all running towards one end. These specific, but plausible loading scenarios must be considered due to the fact that the bridge's target audience is children. Children do unusual things, and as a result of this universal truth: the team designed this bridge to withstand unusual loads that may occur. The design of the bridge can even withstand the loss of over half of its cables and still remain structurally serviceable. Due to time constraints, the in-depth design was for gravity loading. In regards to lateral loading, earthquake loading was considered and analyzed in the model, however the results did not inform the design. The team chose to do an earthquake analysis rather than wind loading because of the bridge location and bridge geometry. San Jose has high seismic activity and the lateral face of the bridge has a very small area, meaning the lateral force generated by wind loading would be very small and likely insignificant relating to the structural design of the bridge.

Even if the bridge is safe, the team must also consider its longevity. This is important because the team wants the structure to be an exhibit for the museum and a symbol for the City that lasts a long time. The bridge is designed to become a landmark structure: something that over time should age well rather than become a nuisance for those that have to maintain it. As such, it should not only be around for a long time, but also never cease in having its original and attractive appearance. To accomplish this goal, the proposed design considers factors like corrosion, cleanliness of the glass decking, and normal wear and tear of all the bridge components. Many of these elements, historically, do not age well over time. As a result, the team was forced to come up with innovative ways to ensure that the bridge breaks this norm. For example, the main arches of the bridge are constructed of mass timber: a material that does not normally perform well over time in close proximity to riparian environments. To remedy this fact, the team found a third-party manufacturer that specializes in treating mass-timber products in a way that allows them to resist the typical effects of weathering while also not taking away the natural aesthetic of the timber material.

The design was highly informed by the value of having an educational impact on the surrounding community. It was designed to be very visually appealing and showcase qualities of structural engineering. The team had to come up with a reliable way to present different aspects of the bridge, while also maintaining its structural integrity. One unorthodox way that the team discovered and utilized was to design the deck to be made out of structural glass panels. This not only allows the bridge to seamlessly blend in with the surrounding environment, but it also exposes various beam connections and other structural components that are normally hidden on other pedestrian bridges. Another interesting aspect that arose from choosing a glass deck was that the below riparian environment was now exposed to pedestrians in a new light. It would be possible to watch migratory salmon or steelhead from the deck of the bridge by looking down straight through the bridge. Now, the bridge would be able to inform children not only about structural engineering, but also about sustainability and channel design: different aspects under the umbrella of civil engineering. This is something that would perfectly integrate the structure as an exhibit that not only connects the communities of San Jose as a practical pedestrian bridge but also invites families and inspires children by igniting their imaginations of what is possible in the field of civil and structural engineering.

SUMMARY DESCRIPTION OF SUPERSTRUCTURE DESIGN

The design of the bridge has many components but a general flow of key steps helps to create a design timeline. After initial steps like identifying the problem and selecting a solution that was informed by specific values which were set by the design team, the structural design process began with gravity loading analysis. This was started by reviewing applicable codes and determining gravity loading on the structure.

According to the 2009 AASHTO LRFD Guide Specification for the Design of Pedestrian Bridges, the bridge had to be designed to withstand 90 pounds per square foot (psf) of live load. The dead load was determined by adding up material weights that are on the deck of the bridge. This primarily includes the weight of the glass panels. Once the loading was factored according to specification in the design guide, a preliminary design of the beams that span the width of the deck was conducted. Once this was done, a preliminary design of the girders was completed. These were both done in conjunction with Simple SAP models because although they can be modeled as simply supported members, the cables connecting them to the arches are not directly vertical. The angle of the cables to the members will create compression and tension in different parts of the members, especially the girders. This requires the girders and beams to be designed for combined loading, either flexure and compression, or flexure and tension.

Located at the midpoint of the bridge deck, there are curved members that cantilever off of the main girders. These required second order analysis calculations that were prepared differently than other members. In order to ensure that these complex calculations were performed correctly, the curves on the bridge had to be analyzed in a structural analysis program. The aptly named, Structural Analysis Program 2000 (SAP 2000), is the software application the team chose to analyze the bridge model. This digital model of the full bridge had to be designed in order to confirm that the hand-calculations were reasonable.

The design of a complete three-dimensional model was a particular challenge for the team. To start, the bridge was designed with joints and members in Civil 3D which is an AutoCAD software used to draft in 3D. Using this software the team was able to then draft and design all of the intricate curves of the bridge design, however, the structural analysis program utilized was not able to import curved 3D elements. The team was met with many error notifications until it was discovered that the program can only accept straight, linear members from Civil 3D. As such, the team had to use a separate tool in AutoCAD that was able to linearize these curved and problematic components. After many iterations and adjustments, the team was able to place nodes and members into the AutoCAD model that was able to be successfully imported into the SAP 2000 modeling software and then successfully analyzed. This process was repeated for several arch sizes and beam layouts that were considered in the design. Once the most efficient geometry of the arch and deck was found, the team used that particular model as the ultimate model for the remainder of the team's structural calculations in the SAP software. The hand calculations were also adjusted, accordingly.

With the full SAP 2000 model complete, the design process shifted to include not only gravity loading, but also lateral forces such as those caused by earthquakes. A modal analysis was performed that analyzes the different types of vibrations that may occur in the structure. This was a difficult process to conduct, as the cables in the SAP 2000 analysis program do not always behave as expected with different loading scenarios, especially lateral ones. The team needed the cables to be loaded equally for our bridge to behave correctly. This was not straightforward as this behavior is in contrast to the unequal geometry of the cables and deck layout of the structural system. Therefore, the brides cables had to be both composed of different thicknesses as well as pretensioned to force the desired behavior. The team spent a few days fiddling with the different settings and preferences in the SAP 2000 software until the team had a result that they felt comfortable with as well as one that reflected the anticipated behavior of the greater system.

Moving to connections between members; they were designed based on how members had interacted in the structural analysis program. Developing connections was a process; they had to be constructed in a way that both allowed loads to move through our theoretical load path as well as maintain the serviceability of the structure. While in some areas leniency in the connection (non-moment resisting) was desired, in others rigidity (moment resisting) was desired. Choosing the areas where these connections were appropriate was important to maintaining some level of efficiency in the design. While the structure might work by utilizing the strongest connection possible for each member, it would certainly be very costly and wasteful in terms of design.

The team ended up placing rigid, welded connections in the locations where there would be curved members, and flexible, bolted connections for square members. This allowed the bridge to ‘breathe’ in locations that required play in the design, an aspect that was important to ensuring that the glass deck system experienced the least amount of vibration. Not only would this reduce the amount of vibration damping material needed in the deck and foundation, but also allow for easier constructability with lower tolerances in building. The bolted connections are simple enough to bolt together on site. But due to the height of the bridge above the Guadalupe River, the welded connections would most likely have to be constructed off site. And, there are also many connections involved in the proposed bridge design requiring specialized third-party professionals to construct.

In regards to material procurement, there are many components of the structure that will use materials which are inherently unsustainable and costly. These include the glass and steel components as well as other stainless, galvanized, or weather-protected materials that will be used on this project. In order to remedy this issue, the team encourages the use of recycled materials that meet the same structural and design requirements as their non-recycled counterparts. Much of the main structural system of the bridge will be constructed using mass timber which is a sustainable material when the proper forestry practices are conducted. Even the steel substructure of the deck can be procured in part from recycled or reclaimed steel. Although steel products have a large environmental impact during the forging and hot-rolling process, the material itself (A995 steel) is mostly made of recycled scrap steel products like those left behind from retired automobiles. Although any proposed construction will involve the use of new materials, these sustainable design practices can be used to offset this design parameter.

The other issue that arises has to do with the recurring maintenance and longevity of the bridge. Since it will be located in close proximity to a body of water, it will be more prone to deterioration over time. These aspects will be addressed by implementing treated wood products where wood will be used, weather-resistant surfacing for steel components, and protective paint where applicable. The use of these coverings and shields are not permanent and will have to be reapplied throughout time on a certain interval. This is particularly true for surfaces which are highly-trafficked, such as the steel and glass deck. Subsequently, a company that will provide these maintenance services will have to be hired or the design must be done in a way that limits the use of the materials that will need to be serviced.

Regarding the environmental impact of the structure, there are sustainability concerns as well as environmental impacts that had to be considered. For sustainability, the two most prominent issues that arose had to do with the longevity and recurring maintenance of the structure as well as material implementation and procurement. There are several environmental issues that were taken into consideration throughout the design of the bridge structure. These include but were not limited to the following: byproducts and pollution from construction, disturbance of the existing terrain, wildlife, and wetlands, as well as hydraulic effects, such as the flow of the river. Although it will not be included as a formal document in the team's final submission package, an environmental impact statement could be prepared to determine what kind of permits would be required for bridge construction at this location. This is an important consideration not only for this design, but also for the communities that live downstream and alongside the Guadalupe River.

One way to address these environmental issues and the concerns of the public was to design using materials that are not known to leech or decay into the environment. One byproduct requiring taking care of the bridge structure is making sure that small pieces of the bridge are not being deposited into the environment over time. This is especially important in a location that is in close proximity to the Guadalupe River, which is a prominent waterway in the State of California. Over the last few years, several native species of migrational fish, including steelhead and salmon, have been sighted in this waterway. This is an indication of the work conducted by the Santa Clara Valley Water District (SCVWD) and California Department of Fish and Wildlife to maintain and restore the habitat and water quality required for these species to survive. On the other hand, it means that there has to be a high degree of consideration taken in regards to the impact the proposed bridge would have on the existing riparian environment. Especially, when it comes to pollution caused by general construction of the project as well as ongoing pollution caused by the aforementioned degradation of the structure.

Another aspect of the bridge design that can be altered to reduce the effect on the nearby environment is the bridge foundation design. Some bridge designs that require columns or footings to be placed in the path of the flow of water will have an unknown or unforeseen effect on the waterway. It is not uncommon for designs that impede or alter the flow of a waterway to have lastings effects on both upstream and downstream communities. As such, a design that will not place columns and supports in the path of the flow of water was utilized for this design. This will eliminate the chances of this bridge changing the behavior of the channel during a flood event as well as interfering with the overall behavior of the river. This design even goes a step further by not placing any footings or underground utilities below the top of the bank of the Guadalupe River. This not only reduces the amount of regulations that the design will undergo in the jurisdiction, but also vastly decreases the chances of altering the existing hydraulic or riparian behavior of this portion and any other portion of this waterway.

GEOTECHNICAL ANALYSIS AND FOUNDATION DESIGN

The design of the foundation not only took into account the vertical and lateral loads, but also underwent an extensive site investigation. Although earthquakes and liquefaction were not explicitly considered in this project, the geotechnical investigation required assessing the soil properties, understanding groundwater conditions, and analyzing any subsurface factors that might influence the design.

To determine the most suitable foundation type, the unique loading requirements and specific characteristics of the site were assessed. This involved considering multiple factors, such as estimating the nominal bearing resistance, assessing the potential for sliding, and predicting any settlement. The calculations provide a comprehensive analysis to ensure a stable and long lasting bridge foundation. In addition to these critical considerations, the design also complied with both the 2020 Caltrans Highway Design Manual and the 2015 American Association of State Highway and Transportation Officials (AASHTO) Load and Resistance Factor Design (LRFD) requirements. These regulations ensured the foundation would meet the standards and requirements for safety and serviceability.

The geology surrounding San Jose and San Francisco Bay was the result of forces along the Pacific and North American plates. Over time these tectonic plates shaped the local land and geological features, resulting in the San Andreas fault. When examining the geology of California, there are 11 distinct geomorphic provinces that have distinct geologic characteristics. San Jose and the Children's Discovery Museum predominantly reside within the Coastal Ranges Province, which have an abundance of hills and valleys. A defining characteristic of this region is the presence of multiple fault lines that span across the state and greatly influence its geological composition. The four (4) main active strike-slip faults are the San Andreas, San Gregorio, Hayward, and Calaveras faults. Although earthquakes were not considered in the design, these fault systems have the potential for seismic events and offer insights into the geological history of the area.

The mapped 2023 United States Geological Survey (USGS) provides detailed geological maps of the Bay Area. These maps depict the surface geology, fault lines, rock formations, and other geological features in the area that give insights into the geological composition of the site. The region encompasses a wide range of rock types including sedimentary deposits, igneous, and metamorphic rocks. The local geology of the site in San Jose, CA, as described by the USGS soil descriptions, is characterized by a combination of natural and anthropogenic deposits. This area primarily sits on alluvial deposits, which are a result of the historical floodplain of the Guadalupe River. These alluvial deposits consist of a mixture of sand, silt, clay, and gravel that were carried and deposited by the river over time. The nature of these deposits can vary, ranging from coarse-grained sands to fine-grained silts and clays. Additionally, the site also has areas of artificial fill deposits that are man-made and consist of various materials, such as sand, gravel, and debris that are used to raise the ground level for construction purposes. It is also important to note that the local geology has been influenced by factors such as, proximity to the Guadalupe River, previous land use, and historical development.

There were numerous considerations encountered in the site exploration that influenced the foundation design. The soil layers have many different properties, such as cohesion, permeability, and bearing capacity. Understanding the bearing capacity of the soil was crucial for selecting an appropriate type of foundation. Boring logs were analyzed from a flood control project geotechnical report from 2002 by Santa Clara Valley Water District. These boring logs from the report indicated that the first 10 to 15 feet was mostly lean clay, fat clay, and silty sand which had a low bearing capacity, low permeability, and weak shear strength. Although this indicated that the upper soils were weak for structural loads, driven piles were still not necessary. Another condition discovered in the report was the high groundwater level, which caused soil erosion and affected the design of the foundation. After a detailed analysis of the geotechnical report and numerous calculations, a shallow foundation was determined to be appropriate for the bridge.

There were a few geotechnical hazards and concerns at the site location. Much of the San Francisco Bay Area has a soil composition at a very high risk of liquefaction. This is because a majority of the soil in the area was composed of young alluvial deposits which are subject to strength loss during liquefaction. These massive liquefaction zones extend from San Francisco through most of Santa Clara and San Jose. The site location has a liquefaction probability of 5 to 10% for a M7.8 San Andreas Fault earthquake scenario. Another significant geotechnical hazard was flooding. In the past, the Guadalupe River has experienced major floods in the years 1936, 1952, 1972, 1973, 1978, 1987, 1991, and 1997. These dangerous floods could impact the structure and its foundation if not properly accounted for. Given the location of the bridge foundation, riprap would have to be used along the embankment to prevent erosion. Additionally, the abutment was designed to account for lateral stability and flood-resisting performance.

There were many types of abutment foundations that would be appropriate for this project. After careful consideration, a typical stub abutment was chosen that has several advantages over other types of abutments. Stub abutments provided excellent stability and support for bridges and created a secure connection between the bridge deck and foundation, minimizing the risk of movement or displacement. They ensured long-term stability and resistance to corrosion which enhanced the overall functionality. Additionally, the balanced load distribution helped to prevent excessive stress on individual parts of the foundation, which reduced the risk of failure.

The final dimensions of the foundation were 10.5 feet deep by 8 feet wide. The rectangular foundation also spanned across the entire width of the bridge to allow support directly under both of the arches, and is constructed with reinforced concrete. This design resisted various factors, such the vertical and lateral loads that came from the arches. The proposed design helped to distribute the load evenly and prevented excessive settlement, sliding, and provided ample bearing capacity. Another concern with the foundation was shear failure, due to its close proximity to the embankment. To prevent this, the stem of the abutment was placed closer to the embankment, creating eccentric loading that counteracted the moments from the lateral loads. The foundation design also included weep holes along the wall to relieve hydrostatic water pressure and ensure proper drainage. This was a concern due to the high groundwater level and the potential for flooding that allows water to accumulate behind the abutment. A structural backfill was used to replace the soft layer of soil behind the wall. This has many advantages, such as increased load distribution, stability, settlement control, and drainage. The structural backfill also helped to distribute the loads from the bridge superstructure and reduced concentrated forces, minimizing the risk of settlement and sliding. Structural backfill also included materials with good drainage properties that allowed water to flow through and away from the foundation, which if left unchecked could lead to soil instability or erosion.

COST ESTIMATE

The Children's Discovery Museum is a public institution that is funded by the City of San Jose. In most cases, the museum would need to fund the majority of the cost for a new pedestrian bridge. Since the museum is a public entity of the City of San Jose, they may be entitled to some improvement fees from nearby projects. An improvement fee of one percent (1%) for a nearby $200,000,000 dollar development may be enough to completely cover the costs of improving the site (i.e. a pedestrian bridge construction). A fee of this amount is reasonable considering that the average improvement fee ranges from two to five percent (2-5%) in Santa Clara County for mixed-use constructions. This is dependent upon the fact of the private developer deciding to dedicate the improvement costs to the bridge rather than another site improvement measure such as sidewalk, road, or utility improvement. In the case the developer does choose this option, it would expedite the bridge construction, help improve the site, and add to the attractiveness of this new development. There is no stated limit on the available funds for the new pedestrian bridge by the museum, however, the team had to give the project a cost that was comparable to nearby pedestrian bridges.

The cost estimate was broken up into four (4) high level categories: materials, construction cost and equipment, labor, and maintenance. Materials make up more than half the estimate of the total cost. The materials needed for this design include: steel girders, timber arches, glass deck, steel framing members, concrete, steel H-piles, railings, steel cables, steel connections, general fill material, utilities, lighting, and large stone boulders. Due to the location of the bridge at the Guadalupe River, there are additional costs for riprap to prevent erosion on the side of the bank.

Machine hire and labor accounts for a large portion of the total cost. You can expect approximately four (4) weeks for site work and to install the abutments. The final installation and site clean-up can take up to a week. Rental prices and labor costs can vary significantly depending on your location and the type of construction site you are working with. San Jose has a significantly higher cost for construction labor compared to other parts of the country and will increase the estimate for labor. Labor rates were estimated based on general pay information gathered from similar projects in the area. The type of rental equipment needed includes excavators, trailers and pickups, dump trucks, concrete mixers, cranes, backhoes, generators, welders, skid loaders, and pile drivers.

As shown in the tables below, the cost estimate analysis found the total cost of materials and labor to be approximately $858,000, with the majority of the cost coming from the glass deck and timber arches. Additionally, the total estimated cost for construction is approximately $570,000. This includes factors, such as a construction survey, excavation, clearing and grubbing, mobilization, and permitting. The total estimated cost for this bridge is approximately $1,400,000 to $1,900,000 after using a contingency of 35%.

Table 3: Material and Labor Estimates.

Item:	Unit:	Quantity:	Rate:	Total:
STRUCTURAL CONCRETE	CY	17.8	1000	$17,800
REINFORCED STEEL	LB	13,200	10	$132,000
ENGINEERED TIMBER	LF	257.4	1000	$257,400
STRUCTURAL STEEL	LF	12,000	10	$120,000
GLASS PANELS	SF	1,315.00	250	$328,750
			Total:	$858,100

Table 4: Construction Cost Estimates.

Item:	Unit:	Total:
CONSTRUCTION SURVEY	LS	$50,000
EXCAVATE	LS	$50,000
CLEARING & GRUBBING	LS	$20,000
MOBILIZATION	LS	$250,000
PERMITTING	LS	$200,000
	Total:	$570,000

Table 5: Total Cost Estimate.

Total Cost	$1,428,100
Total Cost + Fee/Contingency (35%)	$1,927,935

ETHICAL, ENVIRONMENTAL, & SOCIAL IMPACTS

All civil engineering projects have ethical, environmental, and social impacts that ought to be considered. These impacts and concerns were thought about even before the design of the project began. Defining an ethical justification for the team's project was a large part of the planning process and it was integral to creating a design that was valuable to the immediate community. The team had to consider the social implications of the project including the overarching goal of the project; which was to improve the site in order to further educate and inspire children about the discipline of engineering.

They first had to discuss the potential ethical impacts the project would have on future generations of engineers. (1) What can the team teach them with our project and (2) what does it mean to be a successful engineer? The team considered this by reminding themselves of the ASCE, Code of Ethics and embodying these guidelines throughout the project. There were a few rules in particular that they decided to focus on. Firstly, the team knew that the bridge had to be constructed as safely and as reasonably as possible.. From the ASCE, Code of Ethics; “First and foremost, protect the health, safety, and welfare of the public.” Although the team wanted to be creative and innovative with the design, public safety was our number one concern. Secondly, they wanted to embody the value of, “enhancing the quality of life for humanity.” The project accomplished this by not only attracting more sightseers, but also by improving the experience of the existing pedestrian bridge crossing. Lastly, they had to, “acknowledge the diverse historical, social, and cultural needs of the community.” The team satisfied this by working directly with the museum to see exactly how they could maximize the utility of the bridge to best suit their needs.

The project inevitably had ethical issues that the team had to consider deeply. Although there are no blanket solutions for all of these challenges, the team worked to develop some potential mitigation measures throughout the project. While walking around the proposed location for the bridge and talking to the museum representatives, they noticed that a large number of homeless people lived in the area. This was a very important consideration for the team, especially since our project attracted families with children. They thought about how the bridge could affect interactions with the homeless as well as museum visitors. Considering this in the design of the bridge, they had to take measures to reduce any chances of harmful interactions between visitors and the homeless population.. Although there were already safety regulations in place for pedestrian bridge design, the team went beyond these rules to ensure that there were no inherent safety issues that could arise over time. The team designed the bridge in a way that reduced blind turns and areas that could potentially be used to sleep or to hide. Although these types of safety precautions would not be seen in a typical structural engineering code, they were important to consider in terms of common sense planning. Since the bridge design was innovative in construction, the team had to also consider the balance of cost and creativity. The museum is owned by The City of San Jose which means that the budget for the project was coming from a public entity. The team wanted to be as creative as possible, but they also had to consider how funding for public structures would be procured and the types of costs that would be both; feasible, and valuable.

The team also had to take into account many societal and social impacts in an effort to help better understand and benefit the community. The local community around the Children's Discovery Museum of San Jose is diverse in the range of ages, ethnicities, and cultures. As they considered the impact of their work in this specific community, they addressed the needs of everyone no matter their background in the community. That being said, the team anticipated that children and families would benefit most from the bridge design. The purpose of the Children's Discovery Museum of San Jose is to let kids explore science, humanities, art, and nature concepts in an approachable environment. The team incorporated similar concepts that add social value through educational and creative hands-on learning experiences. That way children could be introduced to civil engineering concepts without being overwhelmed by the breadth of the discipline. The bridge featured several educational plaques placed at different locations along the walkway of the bridge that children could interact with and read about specific aspects of the bridge. Not only would these plaques be filled with civil engineering material, but also information about the surrounding area and local ecosystem of the Guadalupe River. This was an effort by the team to blend together community education with sustainability and environmental awareness: hopefully, increasing a common awareness about the importance of the project site itself.

CONCLUSION

The detailed design of the bridge meets project needs because it is not only structurally sound and safe for public use, but also embodies integral values of inspiring youth and symbolizing the museum. The location of the pedestrian bridge will connect the Children's Discovery Museum of San Jose with the future commercial development across the Guadalupe River. The design will feature a curved steel and glass deck that is suspended by two mass timber beams and will embody the characteristics of the Children's Discovery Museum. The Discovery Bridge senior design project takes into account many social impacts in an effort to benefit the community. The local community around the Children's Discovery Museum is very diverse with a broad range of ages, ethnicities, and cultures. As the team considers the impact of the project in society, the team must address the needs and wants of everyone in the community. The team anticipates young kids and families will benefit most from the bridge design. The purpose of the Children's Discovery Museum is to allow kids to explore science, humanities, art, and nature concepts. This project incorporates similar concepts and adds social value through educational and fun hands-on learning experiences in the design, so kids can be introduced to civil engineering concepts. With an innovative design and educational exhibits, the design can help inspire creativity and curiosity for children of all ages.

The analysis of alternatives included (5) different options that were evaluated, and a final approach was decided based on the scores each option received. Alternative 4, to relocate the existing bridge and design a new bridge, received the highest score. This option is fairly economical, has a large social impact, and allows for a great deal of creativity in an aesthetic design. It also meets the criteria for accessibility and supports the needs of the Children's Discovery Museum. Cost was another criteria that was considered in the analysis of alternatives. Although the Children's Discovery Museum would be funding the project, there is no proposed limit for the final cost of the design. Considering similar projects in the area, along with materials, construction, equipment, and labor, the final cost estimate for the Discovery Bridge is expected to be approximately $1,500,000 to $2,000,000.

Concluding the complete design of the pedestrian bridge, a set of architectural and structural plans for the bridge, structural analysis and calculations for all members, and an additional analysis of the constructability and cost of the bridge was provided. The structural analysis provided software and hand calculations ensured the structural integrity of the design. The final design was not only reliable, but also could be implemented to help connect the community, and further educate and inspire the next generation of civil engineers. The bridge provided an alternative entrance and an exhibit for the museum that became a beacon for museum visitors and local sightseers. The bridge would become a landmark structure for the museum, the City of San Jose, and its surrounding community.

APPENDIX A - CITY CODE REQUIREMENTS

The team followed city codes and regulations throughout the entire design process. All of the codes and provisions utilized are listed below:

I. CalTrans Highway Design Manual (2020)
II. AASHTO LRFD Bridge Design Guide (2012)
III. AASHTO LRFD Guide Specifications for the Design of Pedestrian Bridges (2009)
IV. AISC Manual for Steel Construction (2016)
V. NDS for Wood Construction (2018)
VI. California Code of Regulations (2023)
VII. Americans with Disabilities Act (2010)

Deck Loading:

*All values and information pulled from AASHTO Pedestrian Bridge Design Guide, AASHTO LRFD 2012 Bridge Design Specifications (6th Ed.) and AASHTO Standard Specifications for Structural Supports for Highway Signs, Luminaires, and Traffic Signals 2009 (5th Ed.)

$PL := 90\ psf$ — Pedestrian Loading (AASHTO Pedestrian Bridge 3.1)

$LL := 0\ psf$ — Vehicle Load (AASHTO Pedestrian Bridge 3.2) - Not applicable but bridge must have permanent physical vehicle blockage

$WS = P_z$ — Wind Loads (AASHTO Pedestrian Bridge 3.4) - Refer to AASHTO Signs, Articles 3.8 and 3.9

$$P_z = 0.00256 \cdot K_z \cdot G \cdot V^2 \cdot I_r \cdot C_d$$

$K_z := 1.34$ — Height and Exposure Factor (assuming a height of 40ft) (AASHTO Signs section 3.8.4)

$G := 1.14$ — Gust Effect Factor (AASHTO Signs section 3.8.5)

$V := 38\ mph$ — Basic Wind Speed (AASTO Signs figure 3.2)

$I_r := 1.15$ — Wind Importance Factor

$C_d := 2$ — Drag Coefficient (AASHTO Signs Table 3-6)

$$P_z := 0.00256 \cdot K_z \cdot G \cdot V^2 \cdot I_r \cdot C_d \cdot \frac{kg}{m^3} = 0.054\ psf$$

$WS := P_z = 0.054\ psf$

$LL = P_{NW}$ — Fatigue Load (3.5) - Refer to AASHTO Signs section 11

$P_{NW} := 5.2 \cdot C_d \cdot I_r \cdot psf = 11.96\ psf$ — Natural Wind Gust (AASHTO Sign section 11.7.3)

$\boxed{LL} := P_{NW} = 11.96\ psf$

$DC := 20\ psf$ — Dead Load of Structural Components and Nonstructural Attachments (AASHTO LRFD Section 3.3.2)

Load Factoring:

$Q = \sum \eta_i \cdot \gamma_i \cdot Q_i$ Total Factored Force Effect (AASHTO LRFD eq. 3.4.1-1)

η_i Load Modifier (AASHTO LRFD Article 1.3.2)

$\eta_D := 1$ Factor relating to ductility (AASHTO LRFD Article 1.3.3)

$\eta_R := 1$ Factor relating to redundancy (AASHTO LRFD Article 1.3.4)

$\eta_I := 1$ Factor relating to operational classification (AASHTO LRFD Article 1.3.5)

$\eta_i := \eta_D \cdot \eta_R \cdot \eta_I = 1$ For loads for which a maximum value of γ_i is appropriate

$\eta_i := \dfrac{1}{\eta_D \cdot \eta_R \cdot \eta_I} = 1$ For loads for which a minimum value of γ_i is appropriate

Load Combinations Applicable to Pedestrian Bridges (AASHTO LRFD Table 3.4.1-1):

$\gamma_p := 1.25$ Maximum Load Factor for DC (Dead Load) (Table 3.4.1-2)

$Strength\_I := (\eta_i \cdot \gamma_p \cdot DC) + (\eta_i \cdot 1.75 \cdot LL) + (\eta_i \cdot 1.75 \cdot PL) = 203.43\ \mathbf{psf}$

$Strength\_III := (\eta_i \cdot \gamma_p \cdot DC) + (\eta_i \cdot 1 \cdot WS) = 25.054\ \mathbf{psf}$

$ExtremeEvent\_II := (\eta_i \cdot 1 \cdot DC) + (\eta_i \cdot 0.5 \cdot LL) + (\eta_i \cdot 0.5 \cdot PL) = 70.98\ \mathbf{psf}$

$Service\_I := 1 \cdot (\eta_i \cdot 1 \cdot DC) + (\eta_i \cdot 1 \cdot LL) + (\eta_i \cdot 1 \cdot PL) + (\eta_i \cdot 1 \cdot WS) = 122.014\ \mathbf{psf}$

$Service\_II := (\eta_i \cdot 1 \cdot DC) + (\eta_i \cdot 1.3 \cdot LL) + (\eta_i \cdot 1.3 \cdot PL) = 152.548\ \mathbf{psf}$

$$Service\_III := (\eta_i \cdot 1 \cdot DC) + (\eta_i \cdot 0.8 \cdot LL) + (\eta_i \cdot 0.8 \cdot PL) = 101.568\ psf$$

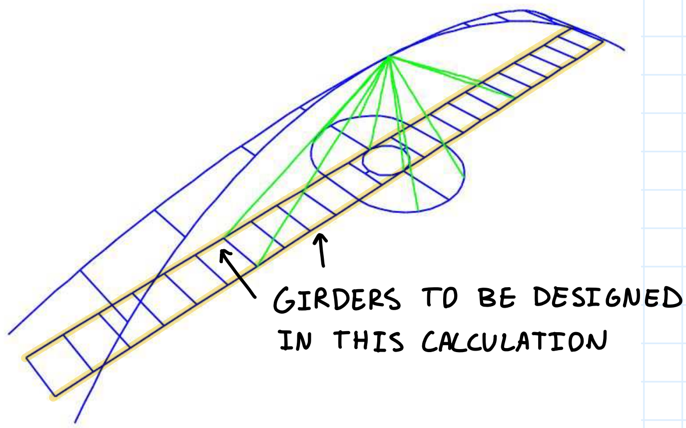
$$Service\_IV := (\eta_i \cdot 1 \cdot DC) + (\eta_i \cdot 1 \cdot WS) = 20.054\ psf$$

$$Fatigue\_I := (\eta_i \cdot 1.75 \cdot LL) + (\eta_i \cdot 1.75 \cdot PL) = 178.43\ psf$$

GOVERNING LOAD COMBINATION FOR GRAVITY LOADING:

$$Strength\_I := (\eta_i \cdot \gamma_p \cdot DC) + (\eta_i \cdot 1.75 \cdot LL) + (\eta_i \cdot 1.75 \cdot PL) = 203.43\ psf$$

Steel Girder Gravity Load Design



Assumptions:

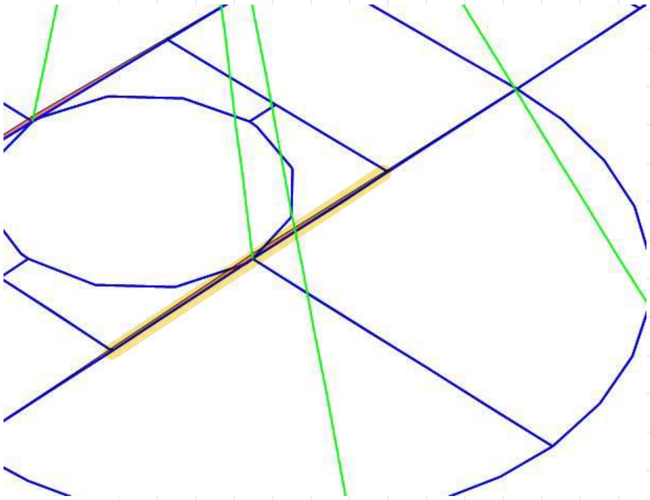
- Bridge deck will consist of glass panels supported by steel beam framing
- Panels will be placed in two rows end to end across full bridge span
- Panels will be 48"X60" and 1 1/8" thickness
- Connections modelled as simply supported
- This design will follow the specifications provided by the AISC Manual, CalTrans, and AASHTO LRFD provisions.
- The girders are laterally braced every 5 feet by cross beams
- Beam to girder connections are moment resisting.

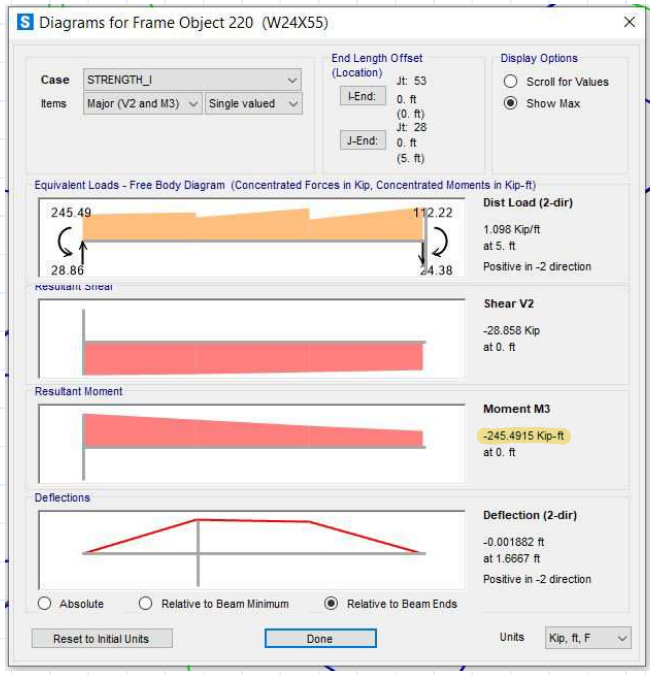
$L_{bY} := 5\ ft$	Girder Weak Axis Unraced Length (beam spacing)
$L_{bX} := 30\ ft$	Girder Strong Axis Unraced Length (cable spacing)
$L_{bZ} := 5\ ft$	Girder Torsional Unraced Length (beam spacing)

Moment and Axial Load in Girder:

$M_{max} := 246\ \boldsymbol{kip} \cdot \boldsymbol{ft}$

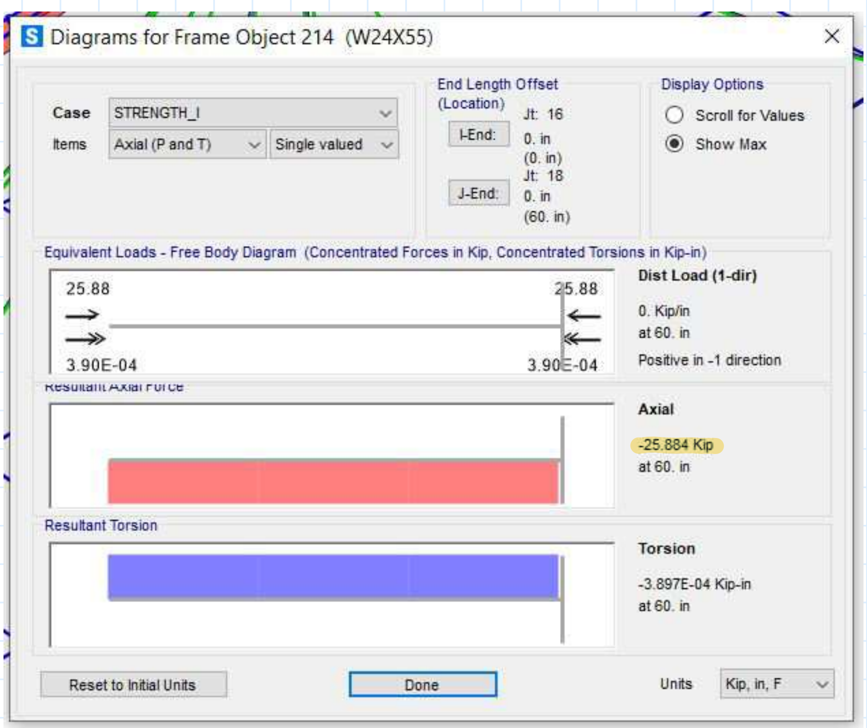
Maximum Moment in Girder Generated by Loading in SAP2000 (required moment)





$P_r := 29\ \boldsymbol{kip}$

Required Axial Compression in Girders Generated by Loading in SAP2000 (compression from cables)



DESIGN AS STEEL MEMBER SUBJECT TO FLEXURE AND COMPRESSION:

MATERIAL PROPERTIES (ASTM A992):

$F_y := 50 \cdot ksi$ $\quad F_u := 65 \cdot ksi$ $\quad E := 29000 \cdot ksi$ $\quad G := 11200 \cdot ksi$ $\quad \nu := 0.3$

INPUT

SECTION:

$Section := \text{"W24X55"}$

SECTION PROPERTIES EXTRACTED FROM AISC SHAPES DATABASE:

SECTION PROPERTIES

$w := \|\text{vlookup}(Section, M, 1)\| \cdot plf = 55\ plf$

$A := \|\text{vlookup}(Section, M, 2)\| \cdot in^2 = 16.2\ in^2$

$d := \|\text{vlookup}(Section, M, 3)\| \cdot in = 23.6\ in$

$b_f := \|\text{vlookup}(Section, M, 4)\| \cdot in = 7.01\ in$

$t_w := \|\text{vlookup}(Section, M, 5)\| \cdot in = 0.395\ in$

$t_f := \|\text{vlookup}(Section, M, 6)\| \cdot in = 0.505\ in$

$b_f\_2t_f := \|\text{vlookup}(Section, M, 7)\| = 6.94$

$h\_t_w := \|\text{vlookup}(Section, M, 8)\| = 54.6$

$I_x := \|\text{vlookup}(Section, M, 9)\| \cdot in^4 = (1.35 \cdot 10^3)\ in^4$

$Z_x := \|\text{vlookup}(Section, M, 10)\| \cdot in^3 = 134\ in^3$

$S_x := \|\text{vlookup}(Section, M, 11)\| \cdot in^3 = 114\ in^3$

$r_x := \|\text{vlookup}(Section, M, 12)\| \cdot in = 9.11\ in$

$J := \|\text{vlookup}(Section, M, 13)\| \cdot in^4 = 1.18\ in^4$

$C_w := \|\text{vlookup}(Section, M, 21)\| \cdot in^6 = (3.87 \cdot 10^3)\ in^6$

$r_{ts} := \|\text{vlookup}(Section, M, 14)\| \cdot in = 1.72\ in$

$$I_y := \left\| \text{vlookup}\left(Section, M, 15\right) \right\| \cdot in^4 = 29.1\ in^4$$

$$Z_y := \left\| \text{vlookup}\left(Section, M, 16\right) \right\| \cdot in^3 = 13.3\ in^3$$

$$S_y := \left\| \text{vlookup}\left(Section, M, 17\right) \right\| \cdot in^3 = 8.3\ in^3$$

$$r_y := \left\| \text{vlookup}\left(Section, M, 18\right) \right\| \cdot in = 1.34\ in$$

$$h_o := \left\| \text{vlookup}\left(Section, M, 19\right) \right\| \cdot in = 23.1\ in$$

$$T := \left\| \text{vlookup}\left(Section, M, 20\right) \right\| \cdot in = 20.75\ in$$

SECTION CLASSIFICATION

SLENDERNESS CHECK (AISCM, CHP. B, TABLE B4.1B):

	COMPACT LIMIT λ_p	NONCOMPACT LIMIT λ_r
FLANGE:	$\lambda_{p\_flange} := 1.12 \cdot \sqrt{\frac{E}{F_y}} = 26.973$	$\lambda_{r\_flange} := 1.40 \sqrt{\frac{E}{F_y}} = 33.716$
WEB:	$\lambda_{p\_web} := 2.42 \cdot \sqrt{\frac{E}{F_y}} = 58.281$	$\lambda_{r\_web} := 5.70 \cdot \sqrt{\frac{E}{F_y}} = 137.274$

ACTUAL FLANGE AND WEB PROPERTIES FROM DATABASE:

$$\lambda_{flange} := b_{f}\_2t_f = 6.94$$

$$\lambda_{web} := h\_t_w = 54.6$$

$$Flange_{slenderness} := \left\| \begin{array}{l} \text{if } \lambda_{flange} \le \lambda_{p\_flange} \\ \quad \| \text{“C”} \\ \text{else if } \lambda_{flange} \ge \lambda_{r\_flange} \\ \quad \| \text{“S”} \\ \text{else} \\ \quad \| \text{“NC”} \end{array} \right| = \text{“C”}$$

C = COMPACT
NC = NON-COMPACT
S = SLENDER

$$Web_{slenderness} := \left\| \begin{array}{l} \text{if } \lambda_{web} \le \lambda_{p\_web} \\ \quad \| \text{“C”} \\ \text{else if } \lambda_{web} \ge \lambda_{r\_web} \\ \quad \| \text{“S”} \\ \text{else} \\ \quad \| \text{“NC”} \end{array} \right| = \text{“C”}$$

$\therefore$ W24X55 IS COMPACT

FLEXURAL STRENGTH - AISCM, CHP. F, SECTION F7

LIMIT STATES TO BE CHECKED: YIELDING, LATERAL TORSIONAL BUCKLING :

$\phi_b := 0.9$

$M_{px} := F_y \cdot Z_x = 558.333\ kip \cdot ft$ EQ. (F2-1)

YIELDING LIMIT STATE:

AISCM SECTION F2.1 ... RECALL: C FLANGE, C WEB

$\phi_b M_n$ DUE TO YIELDING:

$\phi_b M_{n\_y} := \phi_b \cdot M_{px} = 502.5\ kip \cdot ft$ EQ. (F2-1)

LTB LIMIT STATE

AISCM SECTION F2.2

$\boxed{L_{bY}} := 5\ ft$

$L_p := 1.76 \cdot r_y \cdot \sqrt{\frac{E}{F_y}} = 4.733\ ft$ EQ. (F2-5)

$$L_r := 1.95 \cdot r_{ts} \cdot \frac{E}{0.7 \cdot F_y} \cdot \sqrt{\frac{J}{S_x \cdot h_o} + \sqrt{\left(\frac{J}{S_x \cdot h_o}\right)^2 + 6.76 \cdot \left(\frac{0.7 \cdot F_y}{E}\right)^2}} = 13.93\ ft$$ EQ. (F2-6)

SINCE $L_p < L_b \le L_r$ LTB APPLIES WITH EQUATION F2-2:

$C_b := 1$

$$M_{n\_ltb} := C_b \cdot \left(M_{px} - \left(M_{px} - 0.7 \cdot F_y \cdot S_x\right) \cdot \left(\frac{L_{bY} - L_p}{L_r - L_p}\right)\right) = 551.78\ kip \cdot ft$$ EQ. (F2-2)

$\phi_b M_{n\_ltb} := \phi_b \cdot M_{n\_ltb} = 496.602\ kip \cdot ft$

FINAL MOMENT CAPACITY VALUE:

$\phi_b M_{nx} := min\left(\phi_b M_{n\_y}, \phi_b M_{n\_ltb}\right) = 496.602\ kip \cdot ft$ USE SMALLEST $\phi_b M_n$ VALUE

COMPESSIVE STRENGTH - AISCM, CHP. 16.1, SECTION E

SECTION CLASSIFICATION

SLENDERNESS CHECK (AISCM, CHP. B, TABLE B4.1A):

SLENDERNESS LIMIT λ_p

FLANGE: $\lambda_{p\_flange} := 0.56 \cdot \sqrt{\frac{E}{F_y}} = 13.487$

WEB: $\lambda_{p\_web} := 1.49 \cdot \sqrt{\frac{E}{F_y}} = 35.884$

ACTUAL FLANGE AND WEB PROPERTIES FROM DATABASE:

$\lambda_{flange} := b_{f}\_2t_f = 6.94$

$\lambda_{web} := h\_t_w = 54.6$

$$Flange_{slenderness} := \begin{array}{l} \text{if } \lambda_{flange} \le \lambda_{p\_flange} \\ \quad \text{"NS"} \\ \text{else} \\ \quad \text{"S"} \end{array} = \text{"NS"}$$

NS = NONSLENDER
S = SLENDER

$$Web_{slenderness} := \begin{array}{l} \text{if } \lambda_{web} \le \lambda_{p\_web} \\ \quad \text{"NS"} \\ \text{else} \\ \quad \text{"S"} \end{array} = \text{"S"}$$

∴ W24X55 CONTAINS SLENDER ELEMENTS

*NEED TO CHECK FOR FLEXURAL BUCKLING, TORSIONAL BUCKLING, AND LOCAL BUCKLING (TABLE USER NOTE E1.1)

FOLLOWING SECTION E7:

FLEXURAL BUCKLING:

$$K_x := 1$$

$$L_{cx} := K_x \cdot L_{bX} = 30\ ft$$

$$F_e := \frac{\pi^2 \cdot E}{\left(\frac{L_{cx}}{r_x}\right)^2} = 183.286\ ksi$$ EQ. (E3-4)

$$F_{cr} := \begin{Vmatrix} \text{if } \frac{L_{cx}}{r_x} \le 4.71 \cdot \sqrt{\frac{E}{F_y}} \\ \quad \left\| \left(0.658^{\frac{F_y}{F_e}}\right) \cdot F_y \right. \\ \text{else} \\ \quad \left\| 0.877 \cdot F_e \right. \end{Vmatrix} = 44.605\ ksi$$ EQ. (E3-2)

FINDING EFFECTIVE AREA:

FLANGE:

$$\lambda := b_{f\_}2t_f$$ (FROM SECTION PROPERTIES LISTED ABOVE)

$$\lambda_r := 0.56 \cdot \sqrt{\frac{E}{F_y}} = 13.487$$

$$b_f = 7.01\ in$$ (FROM SECTION PROPERTIES LISTED ABOVE)

$$c_1 := 0.18$$ (TABLE E7.1)

$$c_2 := 1.31$$ (TABLE E7.1)

$$F_{el} := \left(c_2 \cdot \frac{\lambda_r}{\lambda}\right)^2 \cdot F_y = 324.039\ ksi$$ EQ. (E7-5)

$$b_e := \begin{Vmatrix} \text{if } \lambda \le \lambda_r \cdot \sqrt{\dfrac{F_y}{F_{cr}}} \\ \quad \Vert\, b_f \\ \text{else} \\ \quad \Vert\, b_f \cdot \left(1 - c_1 \cdot \sqrt{\dfrac{F_{el}}{F_{cr}}}\right) \cdot \sqrt{\dfrac{F_{el}}{F_{cr}}} \end{Vmatrix} = 7.01 \; in$$

EQ. (E7-2/3)

WEB:

$\boxed{\lambda} := h\_t_w$ (FROM SECTION PROPERTIES LISTED ABOVE)

$\boxed{\lambda_r} := 1.49 \cdot \sqrt{\dfrac{E}{F_y}} = 35.884$

$h := T = 20.75 \; in$ (FROM SECTION PROPERTIES LISTED ABOVE)

$\boxed{c_1} := 0.22$ (TABLE E7.1)

$\boxed{c_2} := 1.49$ (TABLE E7.1)

$\boxed{F_{el}} := \left(c_2 \cdot \dfrac{\lambda_r}{\lambda}\right)^2 \cdot F_y = 47.947 \; ksi$ EQ. (E7-5)

$$h_e := \begin{Vmatrix} \text{if } \lambda \le \lambda_r \cdot \sqrt{\dfrac{F_y}{F_{cr}}} \\ \quad \Vert\, h \\ \text{else} \\ \quad \Vert\, h \cdot \left(1 - c_1 \cdot \sqrt{\dfrac{F_{el}}{F_{cr}}}\right) \cdot \sqrt{\dfrac{F_{el}}{F_{cr}}} \end{Vmatrix} = 16.606 \; in$$

EQ. (E7-2/3)

$A_{e\_fb} := 2 \cdot (b_e \cdot t_f) + (h_e \cdot t_w) = 13.64 \; in^2$

$P_{n\_fb} := F_{cr} \cdot A_{e\_fb} = 608.391 \; kip$ EQ. (E7-1)

$\phi_c P_{n\_fb} := 0.9 \cdot P_{n\_fb} = 547.552 \; kip$

Torsional Buckling:

$K_z := 1$

$L_{bZ} = 5\ ft$

$L_{cz} := K_z \cdot L_{bZ} = 5\ ft$

$$F_e := \left(\frac{\pi^2 \cdot E \cdot C_w}{L_{cz}^{\ 2}} + G \cdot J \right) \cdot \frac{1}{I_x + I_y} = 232.689\ ksi$$ Eq. (E3-4)

$$F_{cr} := \begin{cases} \text{if } \dfrac{L_{cx}}{r_x} \le 4.71 \cdot \sqrt{\dfrac{E}{F_y}} \\ \quad \left(0.658^{\frac{F_y}{F_e}} \right) \cdot F_y \\ \text{else} \\ \quad 0.877 \cdot F_e \end{cases} = 45.699\ ksi$$ Eq. (E3-2)

Finding Effective Area:

FLANGE:

$\lambda := b_{f\_}2t_f$ (from section properties listed above)

$\lambda_r := 0.56 \cdot \sqrt{\dfrac{E}{F_y}} = 13.487$

$b_f = 7.01\ in$ (from section properties listed above)

$c_1 := 0.18$ (Table E7.1)

$c_2 := 1.31$ (Table E7.1)

$$F_{el} := \left(c_2 \cdot \frac{\lambda_r}{\lambda} \right)^2 \cdot F_y = 324.039\ ksi$$ Eq. (E7-5)

$$b_e := \begin{cases} b_f & \text{if } \lambda \le \lambda_r \cdot \sqrt{\dfrac{F_y}{F_{cr}}} \\ b_f \cdot \left(1 - c_1 \cdot \sqrt{\dfrac{F_{el}}{F_{cr}}}\right) \cdot \sqrt{\dfrac{F_{el}}{F_{cr}}} & \text{else} \end{cases} = 7.01\ in$$

Eq. (E7-2/3)

WEB:

$$\lambda := h\_t_w$$

$$\lambda_r := 1.49 \cdot \sqrt{\frac{E}{F_y}} = 35.884$$

(FROM SECTION PROPERTIES LISTED ABOVE)

$$h := T = 20.75\ in$$

(FROM SECTION PROPERTIES LISTED ABOVE)

$$c_1 := 0.22$$

(TABLE E7.1)

$$c_2 := 1.49$$

(TABLE E7.1)

$$F_{el} := \left(c_2 \cdot \frac{\lambda_r}{\lambda}\right)^2 \cdot F_y = 47.947\ ksi$$

Eq. (E7-5)

$$h_e := \begin{cases} h & \text{if } \lambda \le \lambda_r \cdot \sqrt{\dfrac{F_y}{F_{cr}}} \\ h \cdot \left(1 - c_1 \cdot \sqrt{\dfrac{F_{el}}{F_{cr}}}\right) \cdot \sqrt{\dfrac{F_{el}}{F_{cr}}} & \text{else} \end{cases} = 16.465\ in$$

Eq. (E7-2/3)

$$A_{e\_tb} := 2 \cdot \left(b_e \cdot t_f\right) + \left(h_e \cdot t_w\right) = 13.584\ in^2$$

$$P_{n\_tb} := F_{cr} \cdot A_{e\_tb} = 620.763\ kip$$

Eq. (E7-1)

$$\phi_c P_{n\_tb} := 0.9 \cdot P_{n\_tb} = 558.686\ kip$$

$\phi_c P_n := min(\phi_c P_{n\_tb}, \phi_c P_{n\_fb}) = 547.552\ kip$

CHOOSING MINIMUM $\phi_c P_n$ TO USE IN INTERACTION EQUATION

BEAM-COLUMN INTERACTION CALCULATIONS (FLEXURE/ COMPRESSION):

AISCM Ch. H, Sec. H1

Summarizing:

Capacity:

$P_c := \phi_c P_n = 547.552\ kip$

$M_{cx} := \phi_b M_{nx} = 496.602\ kip \cdot ft$

Demand:

$P_r = 29\ kip$

$M_{rx} := M_{max} = 246\ kip \cdot ft$

Interaction Eqn. Check:

$$\frac{P_r}{P_c} = 0.053$$

$$PM_{ratio} := \begin{cases} \text{if } \dfrac{P_r}{P_c} \ge 0.2 \\ \quad \dfrac{P_r}{P_c} + \dfrac{8}{9} \cdot \left(\dfrac{M_{rx}}{M_{cx}}\right) \\ \text{else} \\ \quad \dfrac{P_r}{2 \cdot P_c} + \left(\dfrac{M_{rx}}{M_{cx}}\right) \end{cases} = 0.522$$

Eq. (H1-1a, H1-1b)

TENSILE STRENGTH - AISCM, CHP. 16.1, SECTION D

SLENDERNESS LIMITATIONS:

$$\frac{L}{r_x} \le 300$$ SLENDERNESS RATIO LIMIT (USER NOTE D1)

$$L := 30\ ft$$ LENGTH OF TENSION MEMBER

$$\frac{L}{r_x} = 39.517$$

TENSILE STRENGTH (YIELDING IN GROSS SECTION):

$$P_n := F_y \cdot A = 810\ kip$$ EQ. (D2-1)

$$\phi_t P_n := 0.9 \cdot P_n = 729\ kip$$

BEAM-COLUMN INTERACTION CALCULATIONS (FLEXURE/ TENSION):

AISCM CH. H, SEC. H1

SUMMARIZING:

CAPACITY:

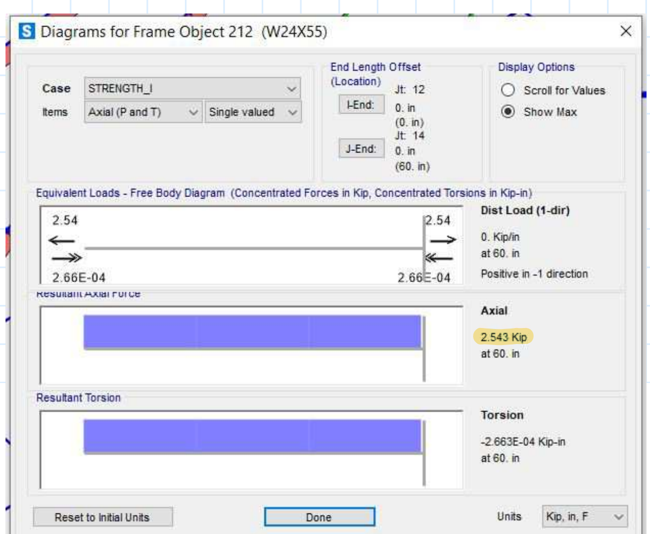
$$P_t := \phi_t P_n = 729\ kip$$

$$\boxed{M_{cx}} := \phi_b M_{nx} = 496.602\ kip \cdot ft$$

DEMAND:

$$\boxed{P_r} := 2.6\ kip$$ GENERATED USING SAP2000

$$\boxed{M_{rx}} := M_{max} = 246\ kip \cdot ft$$



INTERACTION EQN. CHECK:

$$\frac{P_r}{P_t} = 0.004$$

$$PM_{ratio} := \left\| \begin{array}{l} \text{if } \dfrac{P_r}{P_c} \geq 0.2 \\ \quad \left\| \dfrac{P_r}{P_c} + \dfrac{8}{9} \cdot \left(\dfrac{M_{rx}}{M_{cx}} \right) \right. \\ \text{else} \\ \quad \left\| \dfrac{P_r}{2 \cdot P_c} + \left(\dfrac{M_{rx}}{M_{cx}} \right) \right. \end{array} \right\| = 0.498$$

EQ. (HI-IA, HI-IB)

LIVE LOAD DEFLECTION CHECK:

Deflection Along Deck Length:

$L := 120\ ft$ — TOTAL BRIDGE SPAN

$\delta_{max} := 2.9\ in$ — MAXIMUM DEFLECTION IN BRIDGE SPAN DUE TO LIVE LOADING (GENERATED USING SAP2000)

$\delta_{limit} := \frac{L}{360} = 4\ in$ — BRIDGE SPAN DEFLECTION LIMIT

$$DCR_{\delta\_L} := \frac{\delta_{max}}{\delta_{limit}} = 0.725$$

Deflection in Cantilevered Section:

$L := 10\ ft$ — CANTILEVERED LENGTH

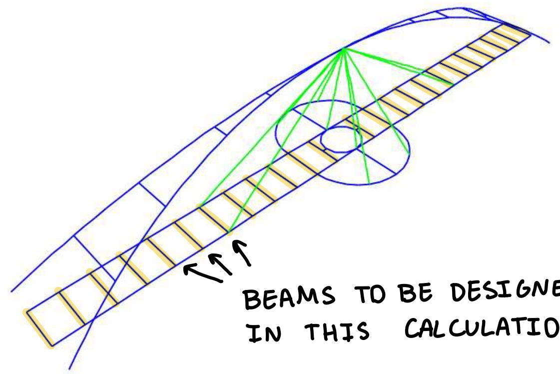
$\delta_{max} := 0.11\ in$ — MAXIMUM DEFLECTION IN CANTILEVERED SECTION DUE TO LIVE LOADING (GENERATED USING SAP2000)

$\delta_{limit} := \frac{L}{220} = 0.545\ in$ — CANTILEVERED SECTION DEFLECTION LIMIT

$$DCR_{\delta\_C} := \frac{\delta_{max}}{\delta_{limit}} = 0.202$$

FINAL GIRDER DESIGN: W24X55

Steel Deck Beams Gravity Load Design:



Assumptions:

- Bridge deck will consist of glass panels supported by steel framing
- Panels will be placed end to end across full bridge span
- Panels will be 48"X60" and 1 1/8" thickness
- Connections modelled as simply supported
- This design will follow the specifications provided by the AISC Manual, Cal Trans, and AASHTO LRFD provisions
- The steel beams will be W sections
- Beam to girder connection are moment resisting

$L_b := 96\ in$ — Beam Unbraced Length (distance between girders)

Loading:

$W_L := 90\ psf$ — Pedestrian Live Load (CalTrans)

$W_D := 20\ psf$ — Dead Load (weight of glass plus 3 psf buffer)

$W := 203.43\ psf$ — Factored Distributed Load (STRENGTH I)

$t_w := 60\ in$ — Tributary Width (beam spacing)

$W_{beam} := W \cdot t_w = 1.017\ klf$ — Distributed Load along Beam

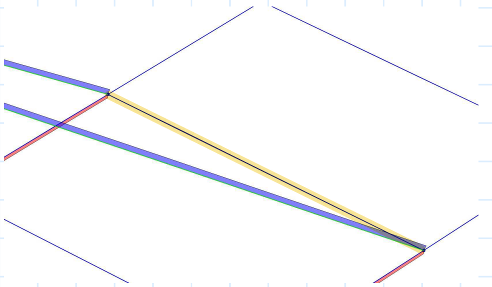
Maximum Moment in Beam:

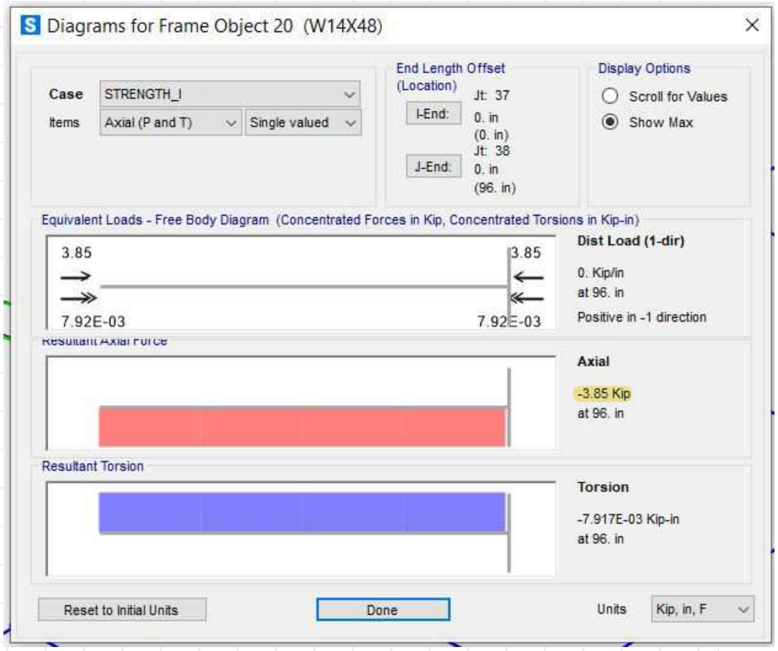
$$M_{max} := \frac{W_{beam} \cdot L_b^{\ 2}}{8} = 8.137\ kip \cdot ft$$

Maximum Moment in Beam Generated by Distributed Load on Simply Supported Beam (Required Moment)

$M_{rx} := M_{max} = 8.137\ kip \cdot ft$ MAXIMUM MOMENT IS THE REQUIRED MOMENT

$P_r := 4\ kip$ REQUIRED AXIAL LOAD (GENERATED USING SAP2000)





DESIGN AS STEEL MEMBER SUBJECT TO FLEXURE AND COMPRESSION:

MATERIAL PROPERTIES (ASTM A992):

$F_y := 50 \cdot ksi$ $F_u := 62 \cdot ksi$ $E := 29000 \cdot ksi$ $G := 11200 \cdot ksi$ $\nu := 0.3$

INPUT

SECTION:

$Section :=$ "W14X48"

SECTION PROPERTIES EXTRACTED FROM AISC SHAPES DATABASE:

SECTION PROPERTIES

$w := \| \text{vlookup}(Section, M, 1) \| \cdot plf = 48\ plf$

$A := \| \text{vlookup}(Section, M, 2) \| \cdot in^2 = 14.1\ in^2$

$d := \| \text{vlookup}(Section, M, 3) \| \cdot in = 13.8\ in$

$b_f := \|\text{vlookup}(Section, M, 4)\| \cdot in = 8.03\ in$

$t_w := \|\text{vlookup}(Section, M, 5)\| \cdot in = 0.34\ in$

$t_f := \|\text{vlookup}(Section, M, 6)\| \cdot in = 0.595\ in$

$b_f2t_f := \|\text{vlookup}(Section, M, 7)\| = 6.75$

$ht_w := \|\text{vlookup}(Section, M, 8)\| = 33.6$

$I_x := \|\text{vlookup}(Section, M, 9)\| \cdot in^4 = 484\ in^4$

$Z_x := \|\text{vlookup}(Section, M, 10)\| \cdot in^3 = 78.4\ in^3$

$S_x := \|\text{vlookup}(Section, M, 11)\| \cdot in^3 = 70\ in^3$

$r_x := \|\text{vlookup}(Section, M, 12)\| \cdot in = 5.85\ in$

$J := \|\text{vlookup}(Section, M, 13)\| \cdot in^4 = 1.45\ in^4$

$C_w := \|\text{vlookup}(Section, M, 21)\| \cdot in^6 = \left(2.24 \cdot 10^3\right)\ in^6$

$r_{ts} := \|\text{vlookup}(Section, M, 14)\| \cdot in = 2.2\ in$

$I_y := \|\text{vlookup}(Section, M, 15)\| \cdot in^4 = 51.4\ in^4$

$Z_y := \|\text{vlookup}(Section, M, 16)\| \cdot in^3 = 19.6\ in^3$

$S_y := \|\text{vlookup}(Section, M, 17)\| \cdot in^3 = 12.8\ in^3$

$r_y := \|\text{vlookup}(Section, M, 18)\| \cdot in = 1.91\ in$

$h_0 := \|\text{vlookup}(Section, M, 19)\| \cdot in = 13.2\ in$

SECTION CLASSIFICATION

SLENDERNESS CHECK (AISCM, CHP. B, TABLE B4.1B):

COMPACT LIMIT λ_p — NONCOMPACT LIMIT λ_r

FLANGE: $\lambda_{p\_flange} := 1.12 \cdot \sqrt{\frac{E}{F_y}} = 26.973$ $\qquad$ $\lambda_{r\_flange} := 1.40 \sqrt{\frac{E}{F_y}} = 33.716$

WEB: $\lambda_{p\_web} := 2.42 \cdot \sqrt{\frac{E}{F_y}} = 58.281$ $\qquad$ $\lambda_{r\_web} := 5.70 \cdot \sqrt{\frac{E}{F_y}} = 137.274$

ACTUAL FLANGE AND WEB PROPERTIES FROM DATABASE:

$\lambda_{flange} := b_f 2t_f = 6.75$

$\lambda_{web} := ht_w = 33.6$

$$Flange_{slenderness} := \left\| \begin{array}{l} \text{if } \lambda_{flange} \le \lambda_{p\_flange} \\ \quad \| \text{“C”} \\ \text{else if } \lambda_{flange} \ge \lambda_{r\_flange} \\ \quad \| \text{“S”} \\ \text{else} \\ \quad \| \text{“NC”} \end{array} \right\| = \text{“C”}$$

C = COMPACT
NC = NON-COMPACT
S = SLENDER

$$Web_{slenderness} := \left\| \begin{array}{l} \text{if } \lambda_{web} \le \lambda_{p\_web} \\ \quad \| \text{“C”} \\ \text{else if } \lambda_{web} \ge \lambda_{r\_web} \\ \quad \| \text{“S”} \\ \text{else} \\ \quad \| \text{“NC”} \end{array} \right\| = \text{“C”}$$

$\therefore$ W4X13 IS COMPACT

FLEXURAL STRENGTH - AISCM 16.1, CHP. F, SECTION F2

LIMIT STATES: YIELDING (Y), LATERAL TORSIONAL BUCKLING (LTB)

YIELDING LIMIT STATE (SECTION F2):

$\phi_b := 0.9$

$M_{px} := F_y \cdot Z_x = 326.667 \; \boldsymbol{kip \cdot ft}$ EQ. (F2-1)

$\phi_b M_{n\_y} := \phi_b \cdot M_{px} = 294 \; \boldsymbol{kip \cdot ft}$

LTB LIMIT STATE:

$L_b = 8 \; \boldsymbol{ft}$ BRACED LENGTH

$L_p := 1.76 \cdot r_y \cdot \sqrt{\frac{E}{F_y}} = 6.747 \; \boldsymbol{ft}$ EQ. (F2-5)

$L_r := 1.95 \cdot r_{ts} \cdot \left(\frac{E}{0.7 \cdot F_y}\right) \cdot \sqrt{\frac{J}{S_x \cdot h_0} + \sqrt{\left(\frac{J}{S_x \cdot h_0}\right)^2 + 6.76 \cdot \left(\frac{0.7 \cdot F_y}{E}\right)^2}} = 21.094 \; \boldsymbol{ft}$ EQ. (F2-6)

SINCE $L_p < L_b < L_r$ INELASTIC LTB APPLIES:

$C_b := 1$

$M_{n\_ltb} := C_b \cdot \left(M_{px} - \left(M_{px} - 0.7 \cdot F_y \cdot S_x\right) \cdot \left(\frac{L_b - L_p}{L_r - L_p}\right)\right) = 316.015 \; \boldsymbol{kip \cdot ft}$ EQ. (F2-2)

$\phi_b M_{n\_ltb} := \phi_b \cdot M_{n\_ltb} = 284.414 \; \boldsymbol{kip \cdot ft}$

FINAL MOMENT CAPACITY VALUE:

$\phi_b M_n := min\left(\phi_b M_{n\_y}, \phi_b M_{n\_ltb}\right) = 284.414 \; \boldsymbol{kip \cdot ft}$ USE SMALLEST $\phi_b M_n$ VALUE

COMPESSIVE STRENGTH - AISCM, CHP. 16.1, SECTION E

SECTION CLASSIFICATION

SLENDERNESS CHECK (AISCM, CHP. B, TABLE B4.1A):

SLENDERNESS LIMIT λ_p

FLANGE: $\boxed{\lambda_{p\_flange}} := 0.56 \cdot \sqrt{\frac{E}{F_y}} = 13.487$

WEB: $\boxed{\lambda_{p\_web}} := 1.49 \cdot \sqrt{\frac{E}{F_y}} = 35.884$

ACTUAL FLANGE AND WEB PROPERTIES FROM DATABASE:

$\boxed{\lambda_{flange}} := b_f 2 t_f = 6.75$

$\boxed{\lambda_{web}} := h t_w = 33.6$

$$\boxed{Flange_{slenderness}} := \begin{Vmatrix} \text{if } \lambda_{flange} \le \lambda_{p\_flange} \\ \quad \Vert \text{"NS"} \\ \text{else} \\ \quad \Vert \text{"S"} \end{Vmatrix} = \text{"NS"}$$

NS = NONSLENDER
S = SLENDER

$$\boxed{Web_{slenderness}} := \begin{Vmatrix} \text{if } \lambda_{web} \le \lambda_{p\_web} \\ \quad \Vert \text{"NS"} \\ \text{else} \\ \quad \Vert \text{"S"} \end{Vmatrix} = \text{"NS"}$$

$\therefore$ W14X48 DOES NOT CONTAINS SLENDER ELEMENTS

*Need to check for flexural buckling, torsional buckling, (table user note E1.1)

Flexural Buckling (E3):

$K_x := 0.5$ Table C-A-7.1

$L_{bX} := 8\ ft$

$L_{cx} := K_x \cdot L_{bX} = 4\ ft$

$$F_e := \frac{\pi^2 \cdot E}{\left(\frac{L_{cx}}{r_x}\right)^2} = \left(4.251 \cdot 10^3\right)\ ksi$$ Eq. (E3-4)

$$F_{cr} := \begin{cases} \text{if } \frac{L_{cx}}{r_x} \le 4.71 \cdot \sqrt{\frac{E}{F_y}} \\ \quad \left(0.658^{\frac{F_y}{F_e}}\right) \cdot F_y \\ \text{else} \\ \quad 0.877 \cdot F_e \end{cases} = 49.754\ ksi$$ Eq. (E3-2)

$P_{n\_f} := F_{cr} \cdot A = 701.538\ kip$

$\phi_c P_{n\_f} := 0.9 \cdot P_{n\_f} = 631.384\ kip$

Torsional Buckling (E4):

$K_z := 0.5$

$L_{bZ} := 8\ ft$

$L_{cz} := K_z \cdot L_{bZ} = 4\ ft$

$$\boxed{F_e} := \left(\frac{\pi^2 \cdot E \cdot C_w}{L_{cz}^{\ 2}} + G \cdot J\right) \cdot \frac{1}{I_x + I_y} = 550.071\ ksi$$ Eq. (E3-4)

$$F_{cr} := \left\| \begin{array}{l} \text{if } \dfrac{L_{cx}}{r_x} \le 4.71 \cdot \sqrt{\dfrac{E}{F_y}} \\ \quad \left\| \left(0.658^{\frac{F_y}{F_e}}\right) \cdot F_y \right. \\ \text{else} \\ \quad \left\| 0.877 \cdot F_e \right. \end{array} \right\| = 48.133 \; \boldsymbol{ksi}$$

Eq. (E3-2)

$$P_{n\_tb} := F_{cr} \cdot A = 678.682 \; \boldsymbol{kip}$$

$$\phi_c P_{n\_tb} := 0.9 \cdot P_{n\_tb} = 610.814 \; \boldsymbol{kip}$$

$$\phi_c P_n := min\left(\phi_c P_{n\_tb}, \phi_c P_{n\_f}\right) = 610.814 \; \boldsymbol{kip}$$

BEAM-COLUMN INTERACTION CALCULATIONS

X-AXIS:

$K := 1$ — Effective Length Factor (beam modeled as simply supported)

$L_{cx} := K\ L_b = 8\ ft$ — Beam Effective Length

$\phi_c P_n = 610.814\ kip$ — Maximum compressive axial load for chosen section at effective length

$\phi_b M_{nx} := \phi_b M_n = 284.414\ kip \cdot ft$ — $\phi_b M_n$ for strong axis flexure as calculated above

Summarizing:

Capacity:

$P_c := \phi_c P_n = 610.814\ kip$

$M_{cx} := \phi_b M_{nx} = 284.414\ kip \cdot ft$

Demand:

$P_r = 4\ kip$

$M_{rx} = 8.137\ kip \cdot ft$

Interaction Eqn. Check:

$$\frac{P_r}{P_c} = 0.007$$

$$PM_{ratio} := \begin{cases} \text{if } \dfrac{P_r}{P_c} \geq 0.2 \\ \quad \dfrac{P_r}{P_c} + \dfrac{8}{9} \cdot \left(\dfrac{M_{rx}}{M_{cx}}\right) \\ \text{else} \\ \quad \dfrac{P_r}{2 \cdot P_c} + \left(\dfrac{M_{rx}}{M_{cx}}\right) \end{cases} = 0.032$$

Eq. (H1-1a, H1-1b)

BEAM STIFFNESS CHECK (FOLLOWING AISCM CH. 16.1 APPENDIX 6 SEC. 6.3):

GIRDER BRACING REQUIRED STRENGTH:

$C_d := 1$

$M_r := 246\ kip \cdot ft$ — REQUIRED FLEXURAL STRENGTH OF GIRDERS (GENERATED USING SAP2000)

$h_0 = 13.2\ in$ — DISTANCE BETWEEN FLANGE CENTROIDS

$$P_{br1} := 0.02 \cdot \left(\frac{M_r \cdot C_d}{h_0}\right) = 4.473\ kip$$ EQ. (A-6-7)

$P_{br2} := 2 \cdot P_{br1} = 8.945\ kip$ — MULTIPLY BY TWO SINCE BEAMS MUST BRACE TWO GIRDERS

< CAPACITY OF SECTION IN COMPRESSION (OKAY)

GIRDER BRACING REQUIRED STIFFNESS:

$\phi := 0.75$

$L_{br} := 5\ ft$ — UNBRACED LENGTH OF GIRDER

$$\beta_{br1} := \frac{1}{\phi} \cdot \left(\frac{10 \cdot M_r \cdot C_d}{L_{br} \cdot h_0}\right) = 596.364\ \frac{kip}{ft}$$ EQ. (A-6-8A)

$$\beta_{br2} := \beta_{br1} \cdot 2 = \left(1.193 \cdot 10^3\right)\ \frac{kip}{ft}$$ MULTIPLY BY TWO SINCE BEAMS MUST BRACE TWO GIRDERS

AVAILABLE SECTION STIFFNESS:

$$E=\left(2.9\cdot 10^{4}\right)\ ksi$$

$$I_x=484\ in^{4}$$

$L:=8\ ft$ LENGTH OF BEAMS

$$w:=1.017\ klf$$

$\delta_{max}:=\dfrac{5\cdot w\cdot L^{4}}{384\cdot E\cdot I_x}$ MAX DEFLECTION OF UNIFORMLY LOADED SIMPLY SUPPORTED BEAM (TABLE 3-23 CASE 1)

$\beta:=\dfrac{w\cdot L}{\delta_{max}}=\left(1.462\cdot 10^{4}\right)\dfrac{kip}{ft}$ AVAILABLE STIFFNESS OF BEAMS (FORCE/DISPLACEMENT)

$$DCR_{\beta}:=\frac{\beta_{br2}}{\beta}=0.082$$

FINAL BEAM DESIGN: W14X48

1) - Span Dimensions & Loading Conditions

1.1 - Span Length:

$L := 120\ ft$

Note: This span is the horizontal span length, not the arc length of the curved member.

1.2 - Arc Geometry:

$L_a := 140.73\ ft$ $\qquad R := 73.13\ ft$

1.3 - Girder Loading:

$L_g := 120\ ft$ $\qquad w_g := 150\ plf$ $\qquad n := 2$

$P_g := L_g \cdot w_g \cdot n = 36\ kip$

1.4 - Arch Loading:

$$P := \frac{P_g}{2}$$

Note: Point load is located at center of arc member and the member is modeled as simply supported.

2) - Member Stresses & Rough Estimate for Member Size

2.1 - Member Material Selection (Glulam) (26F-V1) (DF/DF):

$F_{bx.S} := 2600\ psi$ $\qquad F_{bx.W} := 1950\ psi$

$F_{perp} := 650\ psi$ $\qquad F_{shear} := 265\ psi$

$E_{x.true} := 2.1 \cdot 10^6\ psi$ $\qquad E_{aprox} := 2.0 \cdot 10^6\ psi$

$E_{x.min} := 1.06 \cdot 10^6\ psi$

2.2 - Approximation for Member Section Geometry:

$$M_{max} := P \cdot \frac{L}{2} = \left(1.296 \cdot 10^4\right)\ kip \cdot in$$

$$\sigma_b := F_{bx.S}$$

$$S_{x.req} := \frac{M_{max}}{\sigma_b} = \left(4.985 \cdot 10^3\right)\ in^3$$

2.21 - SAP 2000 Data:

$M_{SAP2000} := 1000\ kip \cdot ft$

$V_{SAP2000} := 60\ kip$

Note: *These values come from combined loading combination in structural analysis model.*

2.3 - Member Section Size Selection:

$b := 16\ in$ $\qquad$ $d := 48\ in$

$$S_x := b \cdot \frac{d^2}{6} = \left(6.144 \cdot 10^3\right)\ in^3$$

$$I_x := b \cdot \frac{d^3}{12} = \left(1.475 \cdot 10^5\right)\ in^4$$

$$\begin{array}{l|l} \text{if } S_x > S_{x.req} & = \text{“Good”} \\ \quad \| \text{“Good”} & \\ \text{else} & \\ \quad \| \text{“Bad”} & \end{array}$$

3) - ASD Adjustment Factors for Size & Curvature

3.1 - Volume Factor: AWC-NDS 2018, 5.3.6

$x := 10$ 10, Used for Douglas Fir Species

$$C_v := \left(\frac{21}{L}\right)^{\frac{1}{x}} \left(\frac{12}{d}\right)^{\frac{1}{x}} \left(\frac{5.125}{b}\right)^{\frac{1}{x}} \cdot ft^{\frac{3}{10}} = 1.073$$

$$\begin{array}{l|l} C_v := \text{if } C_v \geq 1.0 & = 1 \\ \qquad \| 1.0 & \\ \qquad \text{else} & \\ \qquad \| C_v & \end{array}$$

3.2 - Curvature Factor: AWC-NDS 2018, 5.3-3

$t := 1.5\ in$ 1.5" for Douglas Fir Laminations

$$C_c := 1 - (2000)\left(\frac{t \cdot in}{R^2}\right) = 0.996$$

4) - Preliminary Stress Calculations

4.1 - Additional Stress Factors:

4.12 - Duration Factor: AWC-NDS 2018, 5.3.2

$C_D := 0.9$ *Live loading duration factor

4.13 - Temperature Factor: AWC-NDS 2018, 5.3.4

$C_T := 1.0$ *Temperatures do not exceed 150 degrees

4.14 - Flat Use Factor AWC-NDS 2018, 5.3.7

$C_{FU} := 1.0$ *Not used in flat use

4.15 - Stress Interaction Factor AWC-NDS 2018, 5.3.9

$C_I := 1.0$ *No tapering on member

4.16 - Wet Service Factor AWC-NDS 2018, 5.3.3

$C_M := 1.0$ *Normal moisture content

4.17 - *Beam Stability Factor: AWC-NDS 2018, 3.3.3

$$L_e := \text{if } \frac{L}{d} \geq 7 \; \| \; 1.37 \cdot L + 3 \cdot d \;\; \text{else} \; \| \; 1.80 \cdot L \quad = 176.4 \; ft$$

AWC-NDS 2018, Table 3.3.3

$$R_b := \sqrt{\frac{(L_e \cdot d)}{b^2}} = 19.922$$

$$F_{bE} := \frac{(1.20 \cdot E_{x.min})}{{R_b}^2} = (3.205 \cdot 10^3)\ psi$$

$$F''_b := F_{bx.S} \cdot C_D \cdot C_M \cdot C_T \cdot C_{FU} \cdot C_c \cdot C_I = (2.331 \cdot 10^3)\ psi$$

$$C_L := \frac{\left(1 + \left(\frac{F_{bE}}{F''_b}\right)\right)}{1.9} - \sqrt{\left(\frac{\left(1 + \left(\frac{F_{bE}}{F''_b}\right)\right)}{1.9}\right)^2 - \left(\frac{\left(\frac{F_{bE}}{F''_b}\right)}{0.95}\right)} = 0.911$$

***Note:** The lesser of Cv and Cl shall be used for structural glue laminated timber members.

$$C_{LorV} := min(C_L, C_v) = 0.911$$

4.18 - Shear Reduction Factor:

$C_{VR} := 1.0$ *Member is curved, but prismatic across span length

4.2 - Available Bending Stress:

$$F'_{bx} := F''_b \cdot C_{LorV} = 2.123\ ksi$$

4.3 - Available Shear Stress:

$$F'_{vx} := F_{shear} \cdot C_D \cdot C_M \cdot C_T \cdot C_{VR} = 238.5\ psi$$

4.4 - Adjusted Modulus of Elasticity:

$$E'_x := E_{x.true} \cdot C_M \cdot C_T = (2.1 \cdot 10^6)\ psi$$

4.5 - Available Radial Tension Stress: ***Not Applicable to Use**

$F_{rt} := 15\ psi$ Value for Douglas Fir species under dead or live loading

$$F'_{rt} := F_{rt} \cdot C_D \cdot C_M{}^2 \cdot C_T{}^2 = 13.5 \; psi$$

5) - Special Design Consideration for Curved Bending Members

5.1 - Radial Stress Induced by Bending Moment: AWC NDS 2018, 5.4.1

$$R_m := R - \frac{d}{2} = 71.13 \; ft$$

$$f_{rt} := \frac{(3 \cdot M_{max})}{2 \cdot R_m \cdot b \cdot d} = 29.655 \; psi$$

6) - Demand Capacity Ratio's

6.1 - Bending Stress:

$$F_{b.stress} := \frac{\left(M_{SAP2000} \cdot \frac{d}{2}\right)}{I_x} = 1.953 \; ksi$$

$$DCR := \frac{F_{b.stress}}{F'_{bx}} = 0.92$$

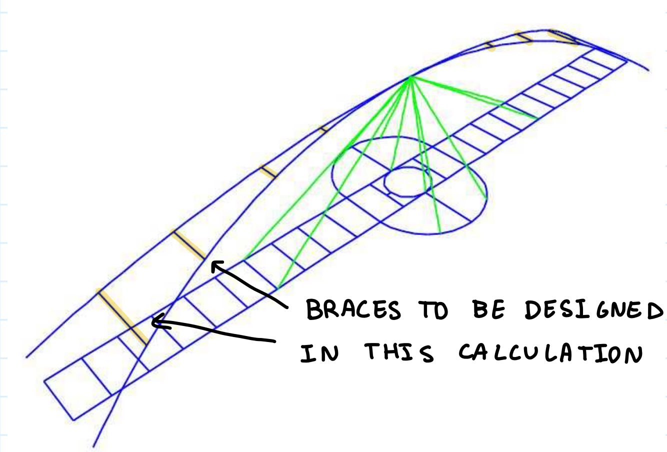
6.2 - Shear Stress:

$$V_{stress} := \frac{\left(V_{SAP2000} \cdot \left(\frac{d}{4} \cdot b \cdot d\right)\right)}{I_x \cdot b} = 234.375 \; psi$$

$$DCR := \frac{V_{stress}}{F'_{vx}} = 0.983$$

6.3 - Deflection:

Arch Brace Gravity Load Design:



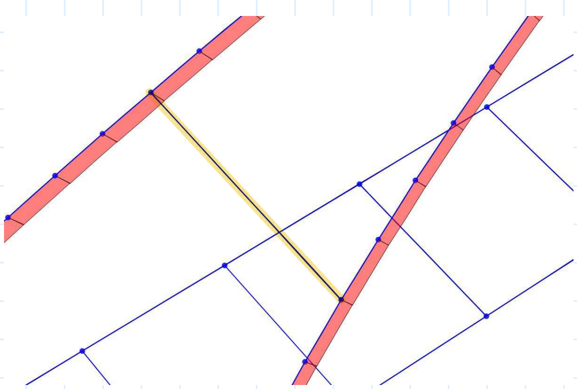
For the arch braces, buckling can occur through the strong axis, weak axis, or through torsion. Calculations to see which one of these governs are conducted. However, for round HSS and tube sections without slender elements, the only limit state that must be checked is flexural buckling.

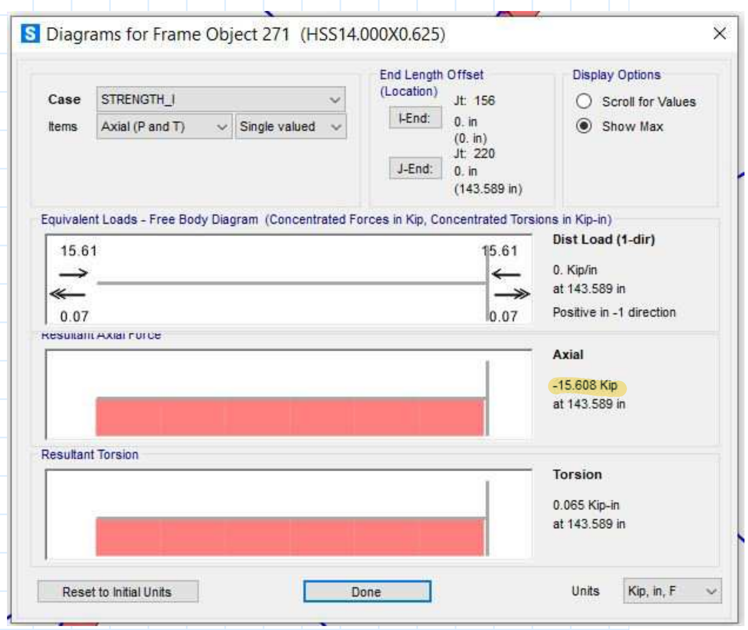
Unsupported Lengths:

$L := 12\ ft$

The brace must resist 13 kips of horizontal force. With a 16 kip horizontal reaction, the required axial resistance through the column must be greater than 16 kips.

$P := 16\ kip$ maximum compression in brace generated using SAP2000





EFFECTIVE LENGTH FACTORS:

THESE FACTORS DEPEND ON THE SUPPORT TYPES OF THE COLUMN. SINCE ALL OF THE CONNECTIONS ARE PINNED, THE EFFECTIVE LENGTH FACTOR IS ONE.
(AISCM CH. 16.1 TABLE C-A-7.1)

$K := 0.5$ ... FLEXURAL BUCKLING (STRONG AND WEAK AXIS)

MATERIAL PROPERTIES:

$F_y := 50 \cdot ksi$ $F_u := 65 \cdot ksi$ $E := 29000 \cdot ksi$

SECTION:

$\boxed{Section} :=$ "HSS14X.625"

SECTION PROPERTIES EXTRACTED FROM AISC SHAPES DATABASE:

SECTION PROPERTIES

$w := \|\text{vlookup}(Section, M, 1)\| \cdot plf = 89.36\ plf$

$A := \|\text{vlookup}(Section, M, 2)\| \cdot in^2 = 24.5\ in^2$

$OD := \|\text{vlookup}(Section, M, 3)\| \cdot in = 14\ in$

$t_{nom} := \|\text{vlookup}(Section, M, 5)\| \cdot in = 0.625\ in$

$t_{des} := \|\text{vlookup}(Section, M, 6)\| \cdot in = 0.581\ in$

$D\_t := \|\text{vlookup}(Section, M, 7)\| = 24.1$

$I_x := \|\text{vlookup}(Section, M, 8)\| \cdot in^4 = 552\ in^4$

$Z_x := \|\text{vlookup}(Section, M, 9)\| \cdot in^3 = 105\ in^3$

$S_x := \|\text{vlookup}(Section, M, 10)\| \cdot in^3 = 78.9\ in^3$

$r_x := \|\text{vlookup}(Section, M, 11)\| \cdot in = 4.75\ in$

$I_y := \|\text{vlookup}(Section, M, 12)\| \cdot in^4 = 552\ in^4$

$Z_y := \|\text{vlookup}(Section, M, 13)\| \cdot in^3 = 105\ in^3$

$S_y := \|\text{vlookup}(Section, M, 14)\| \cdot in^3 = 78.9\ in^3$

$r_y := \|\text{vlookup}(Section, M, 15)\| \cdot in = 4.75\ in$

$J := \|\text{vlookup}(Section, M, 16)\| \cdot in^4 = (1.1 \cdot 10^3)\ in^4$

$C := \|\text{vlookup}(Section, M, 17)\| \cdot in^3 = 158\ in^3$

CALCULATIONS

SLENDERNESS:

SLENDERNESS CALCULATIONS ARE DONE TO DETERMINE THE LIMIT STATES OF THE COLUMN.

OVERALL BUCKLING (EFFECTIVE LENGTH):

$L_c := K \cdot L = 6\ ft$ ... FLEXURAL BUCKLING (STRONG AND WEAK AXIS)

LOCAL BUCKLING (ELEMENT SLENDERNESS)
ELEMENT (AISC 360, CHAPTER B, TABLE B4.1A):

WALLS:

$$D\_t_{limit} := 0.11 \cdot \sqrt{\frac{E}{F_y}} = 2.649$$

$$Wall_{class} := \left\| \begin{array}{l} \text{if } D\_t > D\_t_{limit} \\ \quad \| \text{“S”} \\ \text{else} \\ \quad \| \text{“Not Slender”} \end{array} \right| = \text{“S”}$$

THIS SECTION IS A ROUND HSS COLUMN WITHOUT SLENDER ELEMENTS SO THE ONLY LIMIT STATE THAT MUST BE CHECKED IS FLEXURAL BUCKLING.

ELASTIC BUCKLING STRESSES:

FLEXURAL BUCKLING (STRONG AND WEAK AXIS):

$$F_e := \frac{\pi^2 \cdot E}{\left(\frac{L_c}{r_x}\right)^2} = (1.246 \cdot 10^3)\ ksi$$ (AISCM CH. 16.1, SEC. E4)

LRFD Design Strength:

Design Stress:

$$F_{cr} := \begin{cases} \text{if } \dfrac{F_y}{F_e} \le 2.25 \\ \quad 0.658^{\left(\frac{F_y}{F_e}\right)} \cdot F_y \\ \text{else} \\ \quad 0.877 \cdot F_e \end{cases} = 49.167\ \mathbf{ksi}$$

(AISCM Ch. 16.1, Sec. E3)

$$A_e := \left(\frac{0.038 \cdot E}{F_y \cdot D\_t} + \frac{2}{3}\right) \cdot A = 38.739\ \mathbf{in}^2$$

Design Strength:

$$\phi_c := 0.9$$

$$\phi_c P_n := \phi_c \cdot F_{cr} \cdot A = \left(1.084 \cdot 10^3\right)\ \mathbf{kip}$$

$$DCR_P := \frac{P}{\phi_c P_n} = 0.015$$

BEAM STIFFNESS CHECK (FOLLOWING AISCM CH. 16.1 APPENDIX 6 SEC. 6.4):

SINCE ARCHES SEE HIGH COMPRESSION AND FLEXURE STIFFNESS FOR COMPRESSION (SECTION 6.2) AND FLEXURE (SECTION 6.3) MUST BE CHECKED SEPARATELY.

FIRST CHECKING COMPRESSION:

ARCH BRACING REQUIRED STRENGTH (REQUIRED STRENGTH IS THE SUM OF EQ. A-6-3 AND A-6-7):

$P_r := 216\ kip$ — REQUIRED AXIAL STRENGTH OF ARCHES (GENERATED USING SAP2000)

$P_{br6.3} := 0.01 \cdot P_r = 2.16\ kip$ — EQ. (A-6-3)

USING A-6-7:

$C_d := 1$

$M_{max} := 223\ kip \cdot ft$

$h_0 := OD = 14\ in$ — OUTER DIAMETER OF SECTION

$$P_{br6.7} := 0.02 \cdot \left(\frac{M_{max} \cdot C_d}{h_0}\right) = 3.823\ kip$$ EQ. (A-6-7)

$P_{br1} := P_{br6.3} + P_{br6.7} = 5.983\ kip$

$P_{br2} := 2 \cdot P_{br1} = 11.966\ kip$ — MULTIPLY BY TWO SINCE BEAMS MUST BRACE TWO ARCHES

< CAPACITY OF SECTION IN COMPRESSION (OKAY)

ARCH BRACING REQUIRED STIFFNESS (REQUIRED STRENGTH IS THE SUM OF EQ. A-6-4A AND A-6-8A):

$$L_{br} := 12\ ft$$

$$\phi := 0.75$$

$$\beta_{br6.4} := \frac{1}{\phi} \cdot \left(\frac{8 \cdot P_r}{L_{br}} \right) = 192\ \frac{kip}{ft}$$ EQ. (A-6-4A)

USING A-6-8A:

$$\phi := 0.75$$

$$L_{br} := 5\ ft$$ UNBRACED LENGTH OF GIRDER

$$\beta_{br6.8} := \frac{1}{\phi} \cdot \left(\frac{10 \cdot M_{max} \cdot C_d}{L_{br} \cdot h_0} \right) = 509.714\ \frac{kip}{ft}$$ EQ. (A-6-8A)

$$\beta_{br1} := \beta_{br6.4} + \beta_{br6.8} = 701.714\ \frac{kip}{ft}$$

$$\beta_{br2} := 2 \cdot \beta_{br1} = \left(1.403 \cdot 10^3\right)\ \frac{kip}{ft}$$ MULTIPLY BY TWO SINCE BRACES MUST BRACE TWO ARCHES

AVAILABLE SECTION STIFFNESS:

$$E = \left(2.9 \cdot 10^4\right)\ ksi$$

$$I_x = 552\ in^4$$

$$L := 12\ ft$$

$$\delta_{max} := \frac{w \cdot L^4}{384 \cdot E \cdot I_x}$$ MAX DEFLECTION OF UNIFORMLY LOADED FIXED END BEAM (TABLE 3-23 CASE 15)

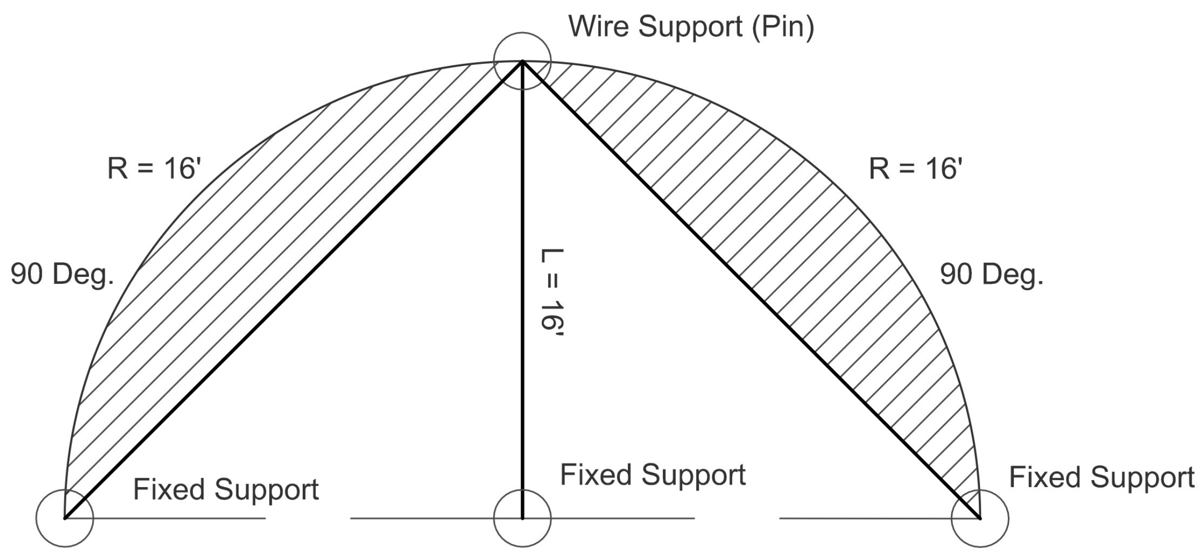
$$\beta := \frac{w \cdot L}{\delta_{max}} = \left(2.47 \cdot 10^4\right)\ \frac{kip}{ft}$$ AVAILABLE STIFFNESS OF BEAMS (FORCE/DISPLACEMENT)

$$DCR_\beta := \frac{\beta_{br2}}{\beta} = 0.057$$

FINAL BRACE DESIGN: HSS14X0.625

Calculation for Exterior and Interior Steel Deck Members Curved in Plan

(1) - Geometry of Member:



(2) - Material Properties:

ASTM A992 Steel $Fy := 50\ ksi$ $Fu := 36\ ksi$

ASTM A36 Steel $Fy2 := 36\ ksi$ $Fu2 := 58\ ksi$

(3) - Span Information:

Radius:

$R := 10\ ft = 120\ in$

Span Angle:

$\theta := 90\ deg \left(\frac{\pi}{180\ deg}\right) = 1.571\ rad$

Angle between torsional restraints:

$\phi := 90\ deg = 1.571\ rad$

Developed Span Length:

$Lds := 8\ ft \cdot (\theta) = 150.796\ in$

Brace Length:

$Ldb := 8\ ft \cdot (\phi) = 150.796\ in$

(4) - Loading Condition (Uniformly Loaded):

Tributary Area for Curved Members:

$$A1 := \frac{\left(\frac{1}{2}\pi \cdot (R)^2 - \frac{1}{2}(32\ ft) \cdot (16\ ft)\right)}{2} = -49.46\ ft^2$$

Tributary Area for Bracing Member:

$$A2 := \frac{1}{2} \cdot 32\ ft \cdot 16\ ft = 256\ ft^2$$

(5) - Acting Gravity Loads:

Live Load:

$$L := 90\ psf$$

Dead Load:

$$D := 50\ psf$$

Factored Loading Condition:

$$Wo := 1.6 \cdot L + 1.2 \cdot D = 204\ psf$$

Load per Linear Foot of Curved Member:

Point Load from Tributary Area:

$$P := Wo \cdot A1 = -1.009 \cdot 10^4\ lbf$$

Circumference of circle divided by 4 for 90 degrees of the arc:

$$C := \frac{2 \cdot \pi \cdot (R)}{4} = 15.708\ ft$$

Assume: Load evenly distributed throughout arc length:

$$W := \frac{P}{C} = -642.342\ plf$$

(6) - Beam End Reactions (AISC Design Guide 33, Curved Member Design):

Reaction at Pinned End:

$$Rup := 3 \cdot \frac{W \cdot Lds}{8} = -3.027\ kip$$

Reaction at Continuous Support:

$$Ruc := 5 \cdot W \cdot \frac{Lds}{8} = -5.045\ kip$$

(7) - Maximum Positive and Negative Moment:

Maximum Positive Moment:

$$Mup := 9 \cdot W \cdot \frac{Lds^2}{128} = -7.132\ kip \cdot ft$$

Maximum Negative Moment Located @ Continuous Support:

$$Mun := -W \cdot \frac{Lds^2}{8} = 12.679\ kip \cdot ft$$

(7) - Torsional Loads:

Pursuant to AISC Design Guide 33, Curved Member Design, Section 7.3.2 utilizing the M/R Method to determine torsional loads.

For a horizontally curved member the maximum torsional moment is at the supports and is zero at midspan. Torsion must be accounted for in the design of member connections.

$$M_{ux} := \frac{\left(W \cdot Lds^3\right)}{24 \cdot R} = -5.311\ kip \cdot ft$$

Eq. (7-13), AISC Design Guide 33

(8) - Corrected Moments for Flexure & Warping

Correction Factor (C): Eq. 7-11, AISC Design Guide 33

$$Cf := 1 - \frac{\theta}{30} + \frac{\theta^2}{6.2} = 1.346$$

Corrected Flexural & Warping Moments:

$$Munc := Mun \cdot Cf = 17.061\ kip \cdot ft$$

$$Mtc := M_{ux} \cdot Cf = -7.147\ kip \cdot ft$$

(9) - Available Shear Strength

AISC, *Specification* Section G2.1

Check for Slenderness: **TRY W12X30**

$h := 12\ in$ $\quad$ $E := 29000\ ksi$

$t_w := 0.440\ in$ $\quad$ $F_y := 50\ ksi$

$$r_1 := \frac{h}{t_w} = 27.273 \qquad r_2 := 2.24 \cdot \sqrt{\frac{E}{F_y}} = 53.946$$

$$h/t_w = 37.5$$

$$2.24\sqrt{\frac{E}{F_y}} = 2.24\sqrt{\frac{29{,}000\text{ ksi}}{50\text{ ksi}}} = 53.9$$

$$\text{if } r_1 < r_2 \;\|\; \text{"Good"} \;\text{else}\; \|\; \text{"Bad"} = \text{"Good"}$$

Therefore, use $C_{v1} := 1.0$

Spec. Eq. G2-2

Nominal Shear Strength:

$$A_w := t_w \cdot h = 5.28 \; in^2$$

$$\phi := 0.9$$

$$\phi V_n := \phi \cdot 0.6 \cdot A_w \cdot C_{v1} \cdot F_y = 142.56 \; kip$$

Demand Capacity Ratio:

$$DCR := \frac{-Ruc}{\phi V_n} = 0.035$$

More than adequate!

(10) - Flexural Strength

AISC *Specification* Section F2

Local Buckling:

$b_f := 7.56 \; in$ $\quad t_f := 0.695 \; in$

$$\lambda_f := \frac{b_f}{2 \cdot t_f} = 5.439$$

$$\lambda_w := \frac{h}{t_w} = 27.273$$

Limiting Ratios (AISC, *Spec.,* Table B4.1b):

$$\lambda_{pf} := 0.38 \cdot \sqrt{\frac{E}{F_y}} = 9.152$$

$$\lambda_{pw} := 3.76 \cdot \sqrt{\frac{E}{F_y}} = 90.553$$

$$\left.\begin{array}{l}\text{if } \lambda_f < \lambda_{pf} \\ \quad \|\ \text{"Good"} \\ \text{else} \\ \quad \|\ \text{"Bad"}\end{array}\right| = \text{"Good"}$$

$$\left.\begin{array}{l}\text{if } \lambda_w < \lambda_{pw} \\ \quad \|\ \text{"Good"} \\ \text{else} \\ \quad \|\ \text{"Bad"}\end{array}\right| = \text{"Good"}$$

Therefore, Section is compact.

Lateral Torsional Buckling:

Plastic Bending Moment for W18x60 is

$Z_x := 123\ in^3$

$M_p := F_y \cdot Z_x = 512.5\ kip \cdot ft$ AISC, Spec. Eq. F2-1

Curvature Factor:

$C_{bs} := 1.0$

$C_{bo} := C_{bs} \cdot \left(1 - \left(\frac{\theta}{\pi}\right)^2\right)^2 = 0.563$ AISC Design Man. 33, Eq. 7-24

Use AISC *Specification* Section F2 w/ Length Braced = Ldb and Cb = Cbo

AISC Manual Table 3-6 for W18x60

$L_b := Ldb = 12.566\ ft$ $C_b := C_{bo}$

$L_p := 5.93\ ft$ $L_r := 18.2\ ft$

$$\left.\begin{array}{l}\text{if } L_p < L_b < L_r \\ \quad \|\ \text{"Good"} \\ \text{else} \\ \quad \|\ \text{"Bad"}\end{array}\right| = \text{"Good"}$$

Nominal Flexural Strength:

$S_x := 108\ in^3$

$$\phi M_n := \phi \cdot C_b \cdot \left(M_p - \left(M_p - 0.7 \cdot F_y \cdot S_x\right) \cdot \frac{\left(L_b - L_p\right)}{\left(L_r - L_p\right)}\right) = 205.375\ kip \cdot ft$$

$$\text{if } M_p > \phi M_n \;\|\; \text{"Good"} \;\text{ else }\; \|\; \text{"Bad"} \quad = \text{"Good"}$$

(11) - Second Order Effects

Elastic Critical Lateral Torsional Buckling Moment:

$r_{ts} := 2.02\ in$ Effective Radius of Gyration from ASIC Manual

$J := 2.17\ in^4$

$$F_{cr} := \frac{\left(C_{bo} \cdot \pi^2 \cdot E\right)}{\left(\frac{L_b}{r_{ts}}\right)^2} \cdot \sqrt{1 + 0.078 \cdot \left(\frac{J}{S_x \cdot h}\right) \cdot \left(\frac{L_b}{r_{ts}}\right)^2} = 37.974\ ksi$$

$$M_{eo} := F_{cr} \cdot S_x = 341.77\ kip \cdot ft$$

Second Order Amplification Factor:

$$B_o := \frac{0.85}{1 - \alpha \cdot \frac{Munc}{M_{eo}}} = 0.85$$

$$B_o := \text{if } B_o < 1.0 \;\|\; 1.0 \;\text{ else }\; \|\; B_o \quad = 1$$

Second Order Flange Warping Moment:

$$M_{uw} := Munc \cdot B_o = 17.061\ kip \cdot ft$$

Warping Strength:

$$Z_f := \frac{\left(t_f \cdot {b_f}^2\right)}{4} = 9.93\ in^3$$

Nominal Flexural Strength of Isolated Flange:

$$\phi M_{nw} := \phi \cdot F_y \cdot Z_f = 37.239\ kip \cdot ft$$ AISC Design Man. 33, Eq. 7-33

Combined Loading Scenario (Out of plane flexural moment & Flange warping moment:

$$DCR_1 := \frac{Mtc}{\phi M_n} + \frac{8}{9} \cdot \frac{M_{uw}}{\phi M_{nw}} = 0.372$$ More Than Adequate!

(12) - Serviceability Requirements -- ASCI Design Manual 33, Section 7.7

Using criteria for first yield:

$$S_f := \frac{\left(t_f \cdot {b_f}^2\right)}{6} = 6.62\ in^3$$

Service Load Warping Stress:

$$\sigma_{rw} := \frac{M_{ux}}{S_f} = -9.627\ ksi$$

Service Level Out of Plane Stress:

$$\sigma_{ro} := \frac{M_{ux}}{S_x} = -0.59\ ksi$$

Combined Stress for First Yield:

$$Y := \sigma_{rw} + \sigma_{ro} = -10.217\ ksi$$

```
if Y < F_y          = "Good"
  ‖ "Good"
else
  ‖ "Bad"
```

(13) - Deflection using Conservative Methods:

1. St. Venant torsion neglected
2. Torsion exclusively handled by warping
3. Isolate flanges and treat them as independent rectangular beams

AISC Design Manual 33, Section 7.3.2, M/R Method

$$f_{fc} := \frac{Munc}{h} = 17.061\ kip$$

$$I_f := \frac{\left(t_f \cdot b_f^{\ 3}\right)}{12} = 25.025\ in^4$$

$$\Delta_{max} := \frac{\left(f_{fc} \cdot Lds^4\right)}{185 \cdot E \cdot I_f \cdot 1\ ft} = 5.476\ in$$

NOT ADEQUATE!

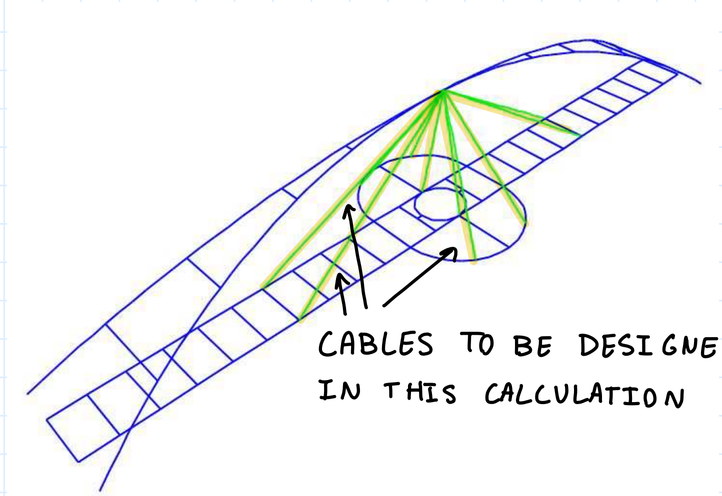
Torsional Rotation:

$$\theta_1 := \operatorname{atan}\left(2 \cdot \frac{\Delta_{max}}{h}\right) = 42.385\ °$$

NOT ADEQUATE!

***Note:** These equations do not align with our displacement results from the SAP2000 structural analysis model. Please refer to structural analysis model for correct displacement results.

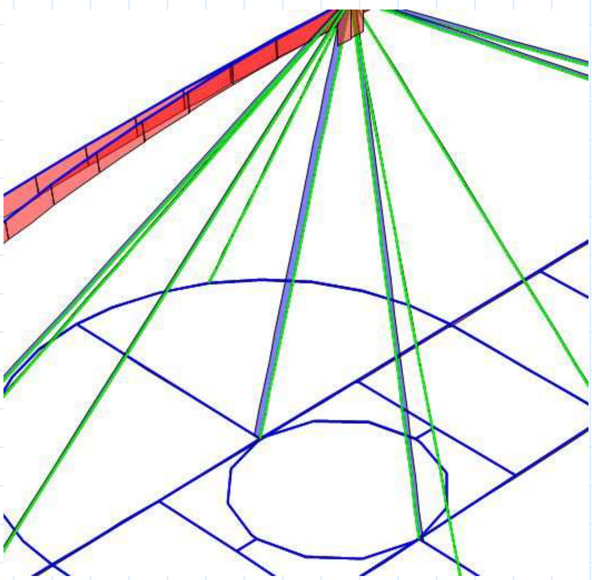
Steel Cable Gravity Load Design:

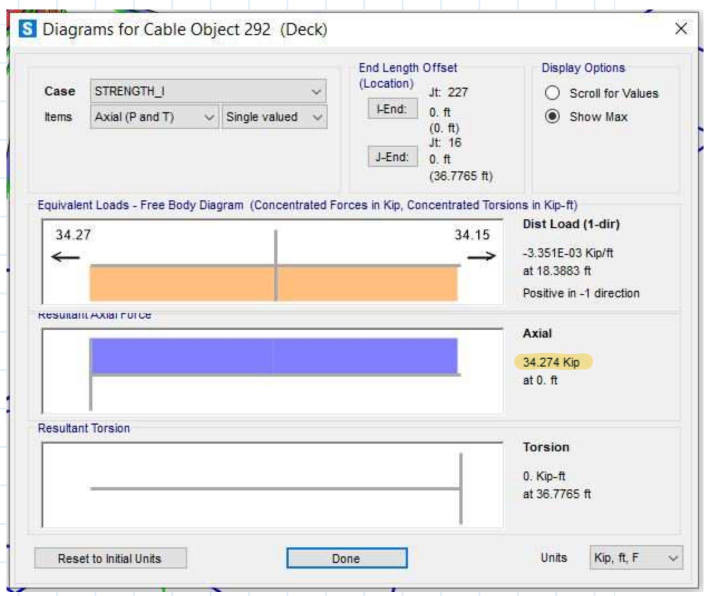


The capacity values provided by cable manufacturer represent the breaking strength of the cables so a factor of safety of 4 (or DCR of at most 0.25) is used.

Outer Deck Cables:

$P_{req} := 35\ kip$ Required axial tension on cable (generated using SAP2000)



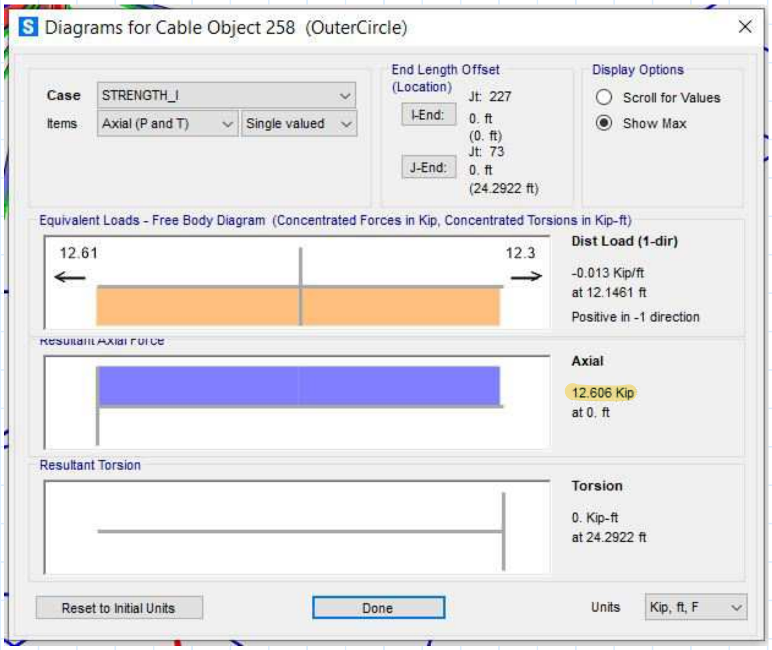


$P_n := 87.8\ tonf$ Capacity in axial tension of cable

$$DCR_{P\_1} := \frac{P_{req}}{P_n} = 0.199$$

OUTER CIRCLE CABLES:

$P_{req} := 13\ kip$ — REQUIRED AXIAL TENSION ON CABLE (GENERATED USING SAP2000)

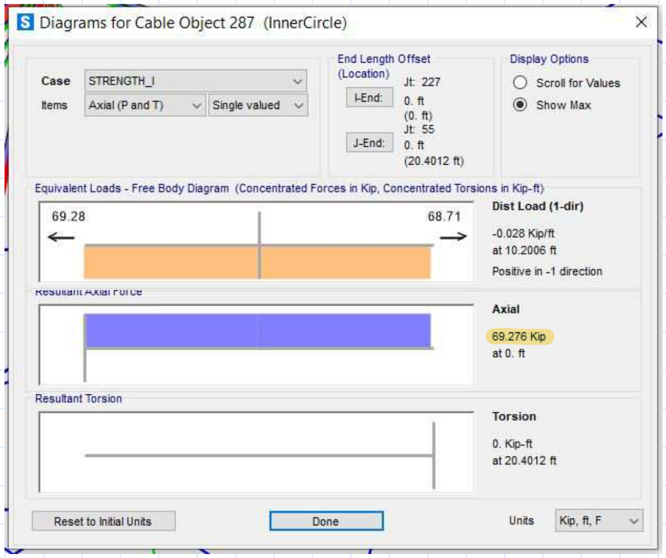


$P_n := 35\ tonf$ — CAPACITY IN AXIAL TENSION OF CABLE

$$DCR_{P\_1} := \frac{P_{req}}{P_n} = 0.186$$

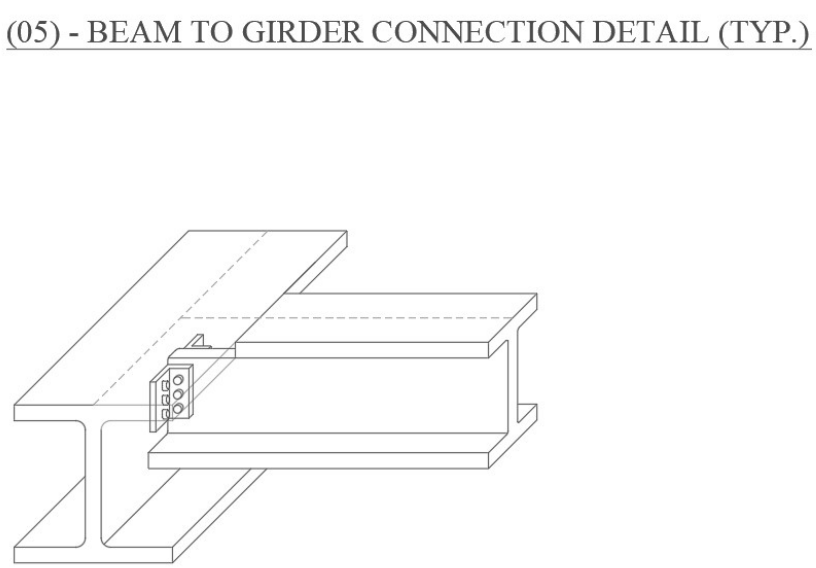
Inner Circle Cables:

$P_{req} := 70\ kip$ REQUIRED AXIAL TENSION ON CABLE (GENERATED USING SAP2000)



$P_n := 164\ tonf$ CAPACITY IN AXIAL TENSION OF CABLE

$$DCR_{P\_1} := \frac{P_{req}}{P_n} = 0.213$$



1 - Member Geometry & Sizing

Girder --> W24x55 Beam --> W10x26

1.1 - Material Properties:

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

Angle Plate (ASTM A36 Steel): $F_{y.p} := 36\ ksi$ $F_{u.p} := 58\ ksi$

Use: 5/8" ASTM A325-N Bolts, (Group A Bolt)

1.2 - Member Thickness:

$T_{girder.w} := 0.395\ in$ $T_{beam.w} := 0.260\ in$

$T_{plate} := \frac{T_{girder.w}}{2} = 0.198\ in$ **Use:** 0.25"

2 - Bolt Spacing & Loading Conditions:

$D_{bolt} := \frac{5}{8}\ in$ **Thread Condition: N, Group: A**
AISC Steel Manual, Table 7-1

$V_n := 12.4\ kip$ AISC Steel Manual, Table 7-1

2.1 - Check for Shear Strength:

$V_{SAP2000} := 25\ kip$

$N_{bolts.min} := \frac{V_{SAP2000}}{V_n} = 2.016$ **Use:** 4 Bolts

2.2 - Bolt Minimum Edge Distance:

$Coping := 2\ in$

$WorkableHeight := 10.3\ in - Coping = 8.3\ in$

Minimum Distance per **AISC Spec. Table J3.4, (5/8" Bolt --> 7/8" Distance).**

$LeftoverSpace := WorkableHeight - 2 \cdot \left(\frac{7}{8}\right) in = 6.55\ in$

2.3 - Minimum & Maximum Spacing:

$S_{min} := D_{bolt} \cdot 2.67 = 1.669\ in$ $S_{max} := 12 \cdot 0.25\ in = 3\ in$

Per AISC Spec. J3.0 & J5.0

Use: 2" Spacing

3 - Bolt Tearout & Bearing Strength:

3.1 - Bolt Bearing Strength:

$dt := D_{bolt} \cdot T_{plate} = 0.123\ in^2$ $F_{u.p} := 58\ ksi$

$R_n := 2.4 \cdot dt \cdot F_{u.p} = 17.183\ kip$ **AISC Spec. J10.1(i)**

Using deformation at the bolt hole as a design consideration.

3.2 - Bolt Tearout Strength:

$L_c := 3\ in$ *Longest distance from bolt hole center to edge of material.*

$R_{n.2} := 1.2 \cdot L_c \cdot T_{plate} \cdot F_{u.p} = 41.238\ kip$ **AISC Spec. J10.1(i)**

Using deformation at the bolt hole as a design consideration.

3.3 - DCR

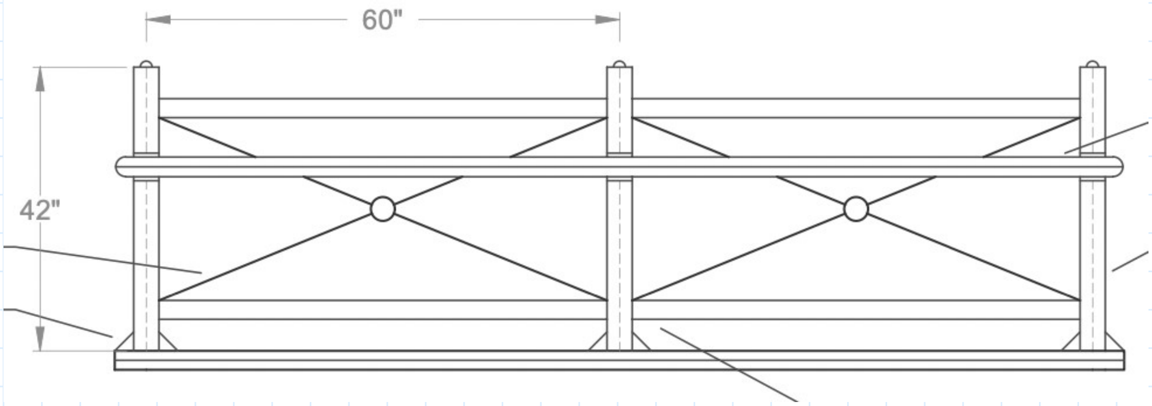
$$DCR_{Shear} := \frac{V_{SAP2000}}{4 \cdot V_n} = 0.504$$

$$DCR_{bearing} := \frac{V_{SAP2000}}{4 \cdot R_n} = 0.364$$

$$DCR_{tearout} := \frac{V_{SAP2000}}{4 \cdot R_{n.2}} = 0.152$$

Calculation of Safety Railings Along Exterior of Bridge

(S-09) - SAFETY RAILING DETAIL (TYP.)



$Spacing := 60$

$h := 42\ in$

Minimum required height for compliance. Must be measured from walkway surface.

$P_{LL} := 0.20 + 0.050 \cdot Spacing = 3.2$

$P'_{LL} := 3.2\ kip$

$P_L := 5\ kip$ *Conservative Increase*

Assumption: Design member for combined 5kip compression and 5kip lateral load @ top of post.

$$P_{LL} = 0.20 + 0.050L \qquad (13.8.2\text{-}1)$$

where:

L = post spacing (ft)

The design load for chain link or metal fabric fence shall be 0.015 ksf acting normal to the entire surface.

The application of loads shall be as indicated in Figure 13.8.2-1, in which the shapes of rail members are illustrative only. Any material or combination of materials specified in Article 13.5 may be used.

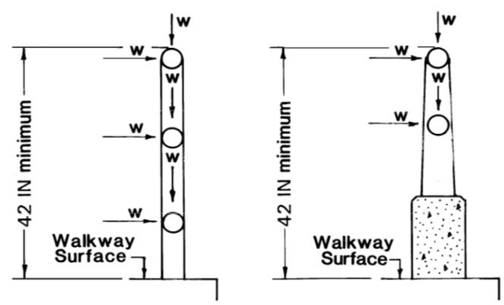


Figure 13.8.2-1—Pedestrian Railing Loads—To be used on the outer edge of a sidewalk when highway traffic is separated from pedestrian traffic by a traffic railing. Railing shape illustrative only.

Excerpt from AASHTO, LRFD 2012, SECTION 13, "Railing Design."

1 - Member Selection & Geometry:

TRY: Round HSS4x0.125 **Required Limit States:** Y, LB

1.1 - Member Material Properties:

$F_y := 46\ ksi \qquad E := 29000\ ksi$

1.2 - Member Geometry:

$W := 5.18\ \frac{lb}{ft} \qquad A := 1.42\ in^2 \qquad D.t := 34.5 \qquad I := 2.67\ in^4 \qquad S := 1.34\ in^3$

$r := 1.37\ in \qquad Z := 1.75\ in^3 \qquad J := 5.34\ in^4 \qquad C := 2.67\ in^3$

1.3 - Compactness / Slenderness Test:

$Ratio := 0.07 \cdot \frac{E}{F_y} = 44.13$ **AISC Steel Manual, Spec. Section B4, Table B4.1b.-20**

$Ratio_2 := 0.31 \cdot \frac{E}{F_y} = 195.435$ *Not applicable, member is not slender.

$$Test := \text{if } Ratio < D.t \;\; \text{"Not Good"} \;\; \text{else} \;\; \text{"Good"} = \text{"Good"}$$

Pursuant to **Spec Section F8-2**, "(a) for compact sections, limit state for buckling does not apply."

2 - Limit State: Yielding

$M_n := 0.9 \cdot F_y \cdot Z = 6.038\ kip \cdot ft$ **Section (F8-1)**

$M_{Req.} := P_L \cdot h = 17.5\ kip \cdot ft$

2.1 - Required Z Value:

$Z_{req.} := \frac{M_{Req.}}{F_y} = 4.565\ in^3$

2.2 - New Member Selection:

USE: Round HSS5x0.375

$W := 18.54\ \frac{lb}{ft}$ $A := 5.10\ in^2$ $D.t := 14.3$ $I := 13.9\ in^4$ $S := 5.55\ in^3$

$r := 1.65\ in$ $Z := 7.56\ in^3$ $J := 27.7\ in^4$ $C := 11.1\ in^3$

2.3 - Limit State: Yielding CHECK

$M_n := 0.9 \cdot F_y \cdot Z = 26.082\ kip \cdot ft$

$M_{Req.} := P_L \cdot h = 17.5\ kip \cdot ft$

2.4 - Demand Capacity Ratio (Yielding):

$$DCR := \frac{M_{Req.}}{M_n} = 0.671$$

3 - Compression / Column Behavior:

3.1 - Effective Length:

$L_c := 2.0 \cdot 42\ in = 7\ ft$ *K=2.0, Fixed @ Bottom Only*

3.2 - Critical Stress:

$$F_e := \frac{\left(\pi^2 \cdot E\right)}{\left(\frac{L_c}{r}\right)^2} = \left(1.104 \cdot 10^5\right)\ psi$$ **Section (E3-2)**

$$F_{cr} := \text{if } \frac{L_c}{r} \le 4.71 \cdot \sqrt{\frac{E}{F_y}} \;\; \left\| \left(0.658^{\frac{F_y}{F_e}}\right) \cdot F_y \right. \;\; \text{else} \;\; \left\| 0.877 \cdot F_e \right. = \left(3.864 \cdot 10^4\right)\ psi$$ **Section (E3-3)**

3.2 - Compressive Strength of Member:

$P_n := F_{cr} \cdot A = 197.067\ kip$ **Section (E3-1)**

$P_{req.} := P_L = 5\ kip$

**Member is more than adequate.*

4 - Combined Loading Scenario:

$$Limit := \frac{P_{req.}}{P_n} = 0.025$$

4.1 - When Pr/Pc < 0.2 Use... **Section (H1-1b)**

$$DCR_{Comb.} := \frac{P_{req.}}{2 \cdot P_n} + \left(\frac{M_{Req.}}{M_n}\right) = 0.684$$

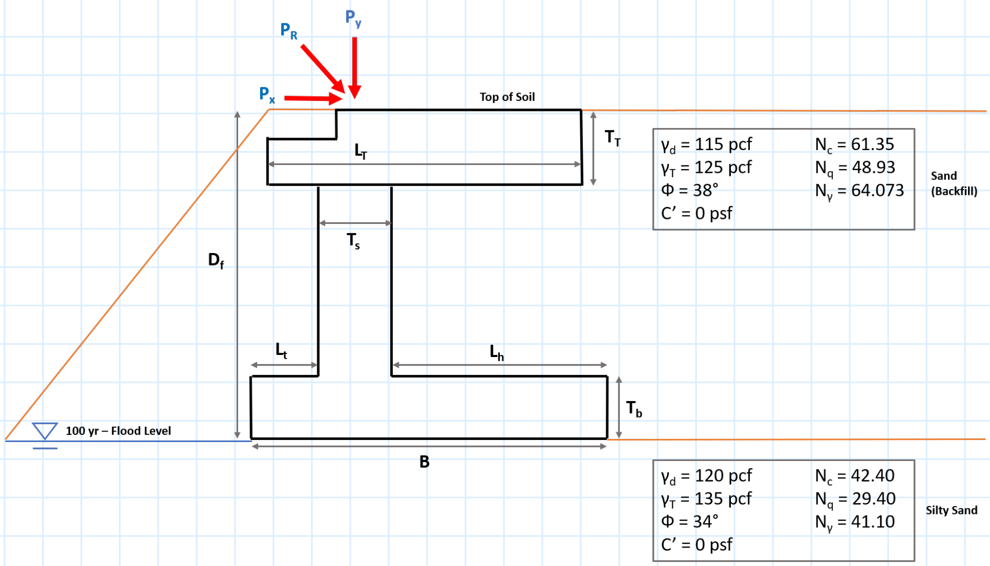
$kip \equiv 1000\ lb$

$psf \equiv 1\ \frac{lb}{ft^2}$

$pcf \equiv 1\ \frac{lb}{ft^3}$

Bridge Foundation Conditions:

(1) - Geometry of Foundation:



Depth of Footing:	$D_f := 10.5\ ft$	Length of Toe:	$L_t := 1.5\ ft$
Width of Footing:	$B := 8.5\ ft$	Length of Heel:	$L_h := 4\ ft$
Length of Footing:	$L := 25\ ft$	Stem Height:	$H_{stem} := 5\ ft$
Thickness of Stem:	$T_s := 3\ ft$	Thickness of Top:	$T_t := 3.42\ ft$
Thickness of Base:	$T_b := 2\ ft$	Length of Top:	$L_T := 8.5\ ft$
		Cross-Sectional Area:	$A := 104.25\ ft^2$

(2) - Foundation Properties:

Unit Weight of Concrete: $\gamma_c := 150\ pcf$

Volume of Concrete: $V_c := A \cdot L = \left(2.606 \cdot 10^3\right)\ ft^3$

Weight of Foundation: $W_f := \gamma_c \cdot V_c = 390.938\ kip$

Unit Weight of Water: $\gamma_w := 62.4\ \frac{lb}{ft^3}$

(2) - Soil Properties: | Silty Sand: | Sand (Backfill):

	Silty Sand:	Sand (Backfill):
Dry Density:	$\gamma_{D1} := 120\ pcf$	$\gamma_{D2} := 115\ pcf$
Total Unit Weight:	$\gamma_{T1} := 135\ pcf$	$\gamma_{T2} := 125\ pcf$
Angle of Internal Friction:	$\phi_1 := 34°$	$\phi_2 := 38°$
Effective Cohesion:	$C_1' := 0\ psf$	$C_2' := 0\ psf$
Undrained Shear Strength:	$S_{u1} := 2000\ psf$	$S_{u2} := 0\ psf$
Unit Weight of Water:	$\gamma_{w1} := 62.40\ pcf$	$\gamma_{w2} := 62.40\ pcf$
Bearing Capacity Factors:	$N_{c1} := 42.40$	$N_{c2} := 61.35$
	$N_{q1} := 29.40$	$N_{q2} := 48.93$
	$N_{\gamma 1} := 41.10$	$N_{\gamma 2} := 64.073$

(3) - Loading Conditions:

Vertical Load: $P_y := 225.64\ kip$

Lateral Load: $P_x := 373.46\ kip$

Resultant Force: $P_R := 436.33\ kip$

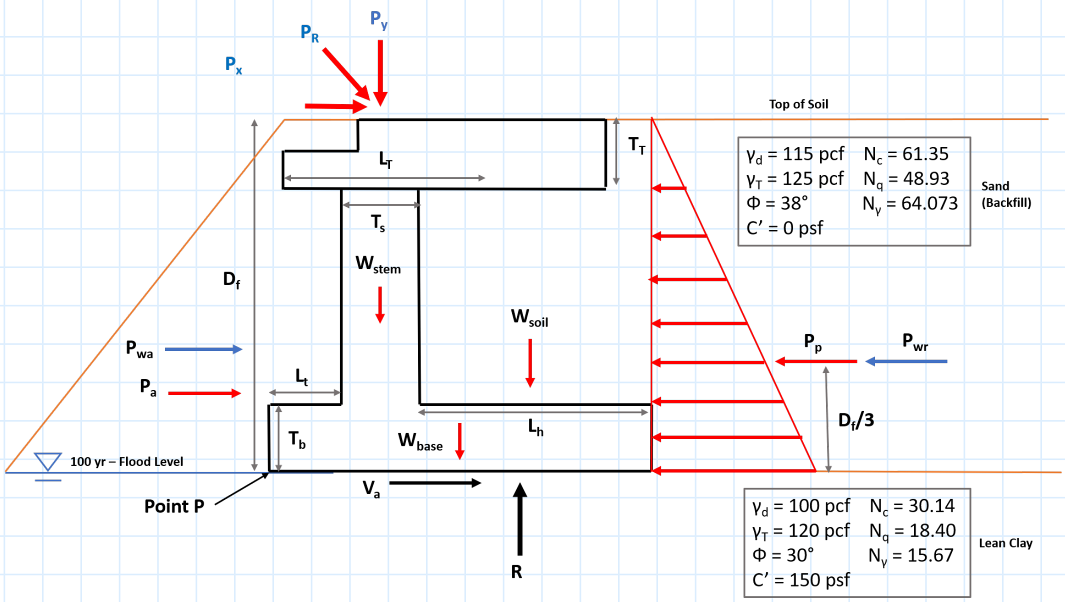
(4) - Bearing Pressure:

Pore Water Pressure: $u_D := D_f \cdot \gamma_w = 655.2\ \frac{lb}{ft^2}$

Bearing Pressure: $q := \frac{P_y + W_f}{A} - u_D = \left(5.259 \cdot 10^3\right) \frac{lb}{ft^2}$

Bridge Foundation - Lateral Load Analysis

(1) - Lateral Load Diagram:



(2) - Horizontal & Vertical Forces:

Rankine's Active Earth-Pressure Coefficients:

Slope of Soil: $\beta := -38$

Active Earth-Pressure Coefficient:

$$K_a := \left(\frac{\cos(\phi_2)}{1 + \sqrt[2]{\sin(\phi_2) \cdot (\sin(\phi_2) - \cos(\phi_2) \cdot \tan(\beta))}} \right)^2 = 0.208$$

Passive Earth-Pressure Coefficient:

$$K_p := \tan\left(45\ deg + \frac{\phi_2}{2}\right)^2 = 4.204$$

Active & Passive Lateral Earth Pressure: AASHTO LRFD 3.11.5.3

$$P_p := \frac{1}{2} \cdot K_p \cdot \gamma_{T2} \cdot {D_f}^2 \cdot L = 724.161\ kip$$

$$P_w := \frac{\gamma_w \cdot {D_f}^2}{2} \cdot L = 85.995\ kip$$

$$P_a := \frac{1}{2} \cdot K_a \cdot \gamma_{T1} \cdot {D_f}^2 \cdot L = 38.702\ kip$$

Forces Acting on Foundation:

Weight of Stem: $W_{stem} := \gamma_c \cdot \left(H_{stem} \cdot T_s\right) \cdot L = 56.25\ kip$

Weight of Base: $W_{base} := \gamma_c \cdot \left(T_b \cdot B\right) \cdot L = 63.75\ kip$

Weight of Soil: $W_{soil1} := \gamma_{T2} \cdot \left(H_{stem} \cdot L_h\right) \cdot L = 62.5\ kip$

$W_{soil2} := \gamma_{T2} \cdot \left(L_t \cdot H_{stem}\right) \cdot L = 23.438\ kip$

Weight of Top: $W_{top} := \gamma_c \cdot \left(T_t \cdot L_T\right) \cdot L = 109.013\ kip$

Uplift on Base: $U_p := \left(\dfrac{\gamma_w \cdot D_f \cdot B}{2}\right) \cdot L = 69.615\ kip$

(3) - Balance Forces:

Bearing Resultant: $F_y := W_f + W_{soil1} + W_{soil2} + P_y - U_p = 632.9\ kip$

(4) - Sliding:

<u>Allowable Shear Capacity</u>:

Interface Coefficient of Friction: $\mu := \tan\left(\phi_1\right) = 0.675$

Sliding Shear Resistance Factor: $\phi_{1R} := 0.8$

ASSHTO Resistance Factors (Table 3.4.1-2):

Sliding Shear Resistance Factor: $\phi_{2R} := 0.5$

Factored Nominal Shear Resistance: $A_{base} := B \cdot L = 212.5\ ft^2$

$$\phi V_n := \phi_{1R} \cdot \left(P_y + W_f - u_D \cdot A_{base}\right) \cdot \mu + \phi_{2R} \cdot \left(0.5 \cdot P_p\right) = 438.62\ kip$$

Resultant Passive Pressure: $P_{Pactual} := P_a + P_w + P_x = 498.157\ kip$

FS Sliding: $FS_{slide} := \dfrac{P_p + \phi V_n}{P_{Pactual}} = 2.334$ $FS_{slide} > 2.1$

(5) - Sum of Moments (around point P):

Righting Moment (due to the weight of foundation and soil):

$$RM := W_{stem} \cdot \left(L_t + \frac{T_s}{2}\right) + \left(W_{base}\right) \cdot \left(\frac{B}{2}\right) + W_{soil1} \cdot \left(L_t + T_s + \frac{L_h}{2}\right) + W_{soil2} \cdot \left(\frac{L_t}{2}\right) = 863.516\ ft \cdot (kip)$$

Overturning Moment (due to passive earth pressure):

$$OM_{Pp} := P_{Pactual} \cdot \left(\frac{D_f}{3}\right) = \left(1.744 \cdot 10^3\right)\ ft \cdot (kip)$$

Overturning Moment (due to active earth pressure):

$$OM_{Pa} := P_a \cdot \left(\frac{D_f}{6}\right) = 67.729\ ft \cdot (kip)$$

Overturning Moment (due to Px):

$$OM_{Px} := P_x \cdot D_f = \left(3.921 \cdot 10^3\right)\ ft \cdot (kip)$$

Overturning Moment (due to Py):

$$OM_{Py} := P_y \cdot \left(L_t + \frac{T_s}{2}\right) = 676.92\ ft \cdot (kip)$$

Overturning Moment (due to Pw):

$$OM_{Pw} := P_w \cdot \left(\frac{D_f}{3}\right) = 300.983\ ft \cdot (kip)$$

Overturning Moment (due to Uplift):

$$OM_{Up} := U_p \cdot \left(\frac{B}{3}\right) = 197.243\ ft \cdot (kip)$$

Total Overturning Moment:

$$OM_{total} := RM - OM_{Pp} - OM_{Up} + OM_{Pw} + OM_{Px} + OM_{Py} + OM_{Pa} = \left(3.89 \cdot 10^3\right)\ ft \cdot (kip)$$

Resulting Bearing Pressure (distance from point P):

$$x := \frac{OM_{total}}{F_y} = 6.146\ ft \qquad e := x - \left(L_t + \frac{T_s}{2}\right) = 3.146\ ft$$

$\frac{B}{6} = 1.417\ ft$ e < B/6 (e not within 1/3 of center)

Effective Foundation Width: $B' := B - 2 \cdot e = 2.208\ ft$

(6) - Bearing Capacity:

Max & Min Bearing Pressure:

$$q_{min} := \left(\frac{P_y + W_f}{A} - u_D\right) \cdot \left(1 - \frac{6 \cdot e}{B'}\right) = -3.969 \cdot 10^4\ \frac{lb}{ft^2}$$

$$q_{max} := \left(\frac{P_y + W_f}{A} - u_D\right) \cdot \left(1 + \frac{6 \cdot e}{B'}\right) = \left(5.021 \cdot 10^4\right)\ \frac{lb}{ft^2}$$

Vesic's Bearing Capacity Equation:

Shape Factors:

$$s_c := 1 + \left(\frac{B'}{L}\right) \cdot \left(\frac{N_{q1}}{N_{c1}}\right) = 1.061 \qquad s_q := 1 + \left(\frac{B'}{L}\right) \cdot \tan\left(\phi_1\right) = 1.06$$

$$s_\gamma := 1 - 0.4\left(\frac{B'}{L}\right) = 0.965$$

Depth Factors: $$k := \operatorname{atan}\left(\frac{D_f}{B'}\right) = 1.363$$

$$d_c := 1 + 0.4 \cdot k = 1.545$$ $$d_q := 1 + 2 \cdot k \cdot \tan\left(\phi_1\right) \cdot \left(1 - \sin\left(\phi_1\right)\right)^2 = 1.357$$

$$d_\gamma := 1$$

Load Inclination Factors: $$m := \frac{2 + \frac{B'}{L}}{1 + \frac{B'}{L}} = 1.919$$

$$i_c := 1 - \frac{m \cdot P_x}{A_{base} \cdot S_{u1} \cdot N_{c1}} = 0.96$$

$$i_q := 1 \qquad i_\gamma := 1$$

Base Inclination Factors (level footing): $b_c := 1$ $b_q := 1$ $b_\gamma := 1$

Ground Inclination Factors:

$$g_c := 1 - \frac{\beta}{147} = 1.259 \qquad g_q := \left(1 - \tan\left(\beta\right)\right)^2 = 1.717 \qquad g_\gamma := \left(1 - \tan\left(\beta\right)\right)^2 = 1.717$$

Buoyant Unit Weight of Soil: $$\gamma' := \gamma_{T2} - \gamma_{w1} = 62.6\ pcf$$

<u>Vesic's Bearing Capacity:</u> $$\sigma'_{zD} := D_f \cdot \gamma_{T1} - u_D = 762.3\ \frac{lb}{ft^2}$$

$$q_n := C_1' \cdot N_{c1} \cdot s_c \cdot d_c \cdot i_c \cdot b_c \cdot g_c + \sigma'_{zD} \cdot N_{q1} \cdot s_q \cdot d_q \cdot i_q \cdot b_q \cdot g_q + (0.5) \cdot \gamma' \cdot B' \cdot N_{\gamma 1} \cdot s_\gamma \cdot d_\gamma \cdot i_\gamma \cdot b_\gamma \cdot g_\gamma = \left(6.005 \cdot 10^4\right) \frac{lb}{ft^2}$$

$$q_{allow} := \frac{q_n}{3} = \left(2.002 \cdot 10^4\right) \frac{lb}{ft^2}$$

$$FS_{bearing} := \frac{q_{allow}}{q} = 3.806$$

$q_{allow} > q_{max}$

(7) - Settlement:

$$E := 200000 \frac{lb}{ft^2}$$

$$\frac{D_f}{B} = 1.235 \qquad I_0 := 0.98$$

$$\frac{L}{B} = 2.941 \qquad I_1 := 1.0$$

Total Settlement: $\delta := I_0 \cdot I_1 \cdot \left(\frac{q \cdot B'}{E}\right) = 0.683\ in$

$\delta < 1\ in$

AASHTO LRFD 10.6.2.4

APPENDIX C - MANUFACTURERS CATALOG INFORMATION

Image 1: Structural Glass Deck, Technical Information

CIRCLE REDMONT®
LIGHT PENETRATING SYSTEMS

SECTION ________

91R™ FACTORY PREFABRICATED *"PLANK"* GLASS AND METAL PANELS

PART 1 - GENERAL

1.1 DESCRIPTION OF WORK

A. Extent of the Glass and Metal Panels as indicated on the drawings.

B. Principal Work in this section includes:

1. 91R™ Metal Framed Composite Glass Panels
2. Reinforcement, anchors, fasteners and similar items
3. Glazing materials and sealant for panels
4. Fasteners at perimeter of panels

1.2 DESCRIPTION OF PANELS

A. Panels shall be factory prefabricated and glazed 91R™ Panels manufactured by Light Penetrating Systems, LLC dba Circle Redmont®, 1213 Medina Rd, Medina OH 44256 (800-358-3888 / 321-259-7374). Panels shall be of overall sizes as detailed on the drawings. Metal framework shall be designed and produced to meet or exceed the specified structural performance as well as the composite glass panel construction tolerance requirements of plus, minus .050 maximum. Glass units shall be Circle Redmont® standard, traffic bearing, composite glass units. Glass units shall be initially factory installed in respective cell openings using Circle Redmont® standard waterproof sealant and bedding material.

1.3 QUALITY ASSURANCE

A. Structural: Design, engineer and fabricate the panels to support live load capacity of one hundred (100) lbs per square foot (*amend as required*) on a 4'-0" span supported on four sides and to the stringent tolerance of (+) (-) .050 required for structural glass performance.

B. Thermal Movement: Design the framing system to provide for such expansion and contraction of component materials as will be caused by the surface temperature range of -10 degrees F to 120 degrees F, without causing buckling stresses on glass, failure of joint seals, undue stress on structural elements, damaging loads on fasteners, reduction in performance, or other detrimental effects.

C. Water Penetration: Water penetration is defined as the appearance of or damage from water, other than condensation, on the underside of the panels. The panels shall be totally watertight.

D. Manufacturer of Panels: The manufacturer of the shop fabricated panels shall be a firm with not less than ten (10) years of successful experience in supplying the same type of panels as required for this project and solely employs and is responsible for all of the personnel required for all facets of production required in manufacturing the units at its production facility.

E. Design Criteria: The drawings and specifications are based on a specific type of panel by a single manufacturer. Equivalent type panels by another manufacturer may be acceptable only if deviations in dimension, profile, manufacturing tolerance, appearance and performance history are minor and do not materially detract from the design concept of intended performances, as judged solely by the Architect.

If a substitute is being offered as an "Or Equal" to the specified products, in order to be considered, the Contractor will notify the Architect within five (5) business days of such intent of substitution and upon notification by the Architect the Contractor will have three business days to provide adequate proof of

Image 2: Structural Glass Deck, Technical Information (Cont.)

equivalency, i.e.; documentation that the "Or Equal" meets the minimum characteristics of the ordering description or specification. Submission of proof of equivalency and samples shall be at the bidder's expense and no compensation shall be offered by Architect or Owner.

Failure to provide this documentation which must include Substitute's Manufacturer History, Technical Specifications, Specifications, Brochures, Samples, Catalogs, Etc., will render the substitution non responsive and ineligible for award.

F. Warranty: Submit written warranties signed by the manufacturer, installer and contractor, agreeing to repair or replace defective materials and workmanship during the warranty period. Manufacturer must be able to document its warranty based upon at least fifty (50) projects with at least five (5) years of successful field performance.

Defective materials and workmanship in manufacture include abnormal deterioration, aging or weathering of work, leakage of water (except at perimeter), structural failure, deterioration of finishes in excess of normal weathering and aging.

The manufacturer shall warrant the panels to be free from manufacturing defects in either materials or workmanship for a period of one (1) year from the date of shipment. The manufacturer's warranty does not include glass breakage, delaminating, damage caused by improper handling or use, improper installation, vandalism, abuse or natural conditions exceeding standard performance requirements.

Where the manufacturer is contacted and upon inspection finds that performance failure is due to defective materials of fabrication of the factory prefabricated ***91R™ Plank Glass™ & Metal Frame Panels***, all materials necessary to repair will be provided by the manufacturer at no extra cost to the customer. Freight and related charges not included.

The manufacturer's liability is limited to materials in the factory prefabricated panels only and the manufacturer shall not be liable for handling by contractor and installer, installation procedures, and consequential damages or expenses.

The contractor and installer shall furnish a written warranty covering defects in handling, installation procedures, materials and workmanship and field applied perimeter sealant work for a period of one (1) year from the date of installation. The contractor and installer's liability shall be limited to the prompt repair or replacement of panels, materials and/or perimeter sealant work.

1.4 SUBMITTALS:

A. Shop Drawings: Submit shop and erection drawings at large scale clearly showing sections of panels with all fasteners, joinery techniques, provisions for expansion/contraction, metal thickness and profiles. Identify all materials including metal alloys and fasteners. Locate and identify shop and field sealants on drawings. Show adjacent structural elements. G.C. shall provide accurate field dimensions to Light Penetrating Systems, LLC dba Circle Redmont® and coordinate all efforts with Light Penetrating Systems, LLC dba Circle Redmont® to insure Architects design intent.

B. Samples: Submit samples for color and finish of the Plank Glass™ that are to be factory installed in the metal grid.

C. Structural Calculations: Submit structural calculations for live and dead loads on framing members confirming the panel's capacity to withstand the specified load requirements.

PART 2 - PRODUCTS

2.1 MATERIALS

A. Panels shall be factory fabricated Circle Redmont® ***91R™ Plank Glass™ & Metal Frame Panels*** as manufactured by Light Penetrating Systems, LLC dba Circle Redmont, Inc.®, 1213 Medina Rd, Medina OH 44256 (800-358-3888 / 321-259-7374). Panels by other manufacturers will be considered only if the manufacturer and the panels comply fully with these specifications and the drawings.

Image 3: Structural Glass Deck, Technical Information (Cont.)

B. The factory prefabricated panels shall consist of the following:

1. Panels shall be factory fabricated and glazed, of overall sizes as detailed on the shop drawings. Metal framework and Circle Redmont® standard traffic bearing glass properly spaced according to Circle Redmont standard dimensions.

   a. Finish of exposed structural metal grid with Circle Redmont® standard factory applied epoxy prime and 2 part urethane topcoat painted finish.

   b. Standard traffic bearing glass with factory sandblasting of the top surfaces of glass is recommended for walking surfaces (Fritted Glass available). Plank Glass units shall be factory installed in respective cell openings using Circle Redmont® standard waterproof sealant and bedding material.

2. Exposed top glass joints weather sealed with sealant of a type recommended and warranted by the panel manufacturer.

2.2 FABRICATION AND WORKMANSHIP

A. Maintain the visual design concept including member sizes, profiles and alignment components. Coordinate this work with that of other trades.

B. Factory assemble each panel in the manufacturing facility employing qualified personnel solely engaged in prefabricated panel production.

C. Fit joints accurately in exposed metal work, and secure rigidly with hairline contacts.

D. Fabricate and fasten metal work so that the work will not be distorted nor the fasteners overstressed from expansion and contraction.

PART 3 - EXECUTION

3.1 STORAGE AND HANDLING

A. G.C. shall coordinate and cooperate with Light Penetrating Systems, LLC dba Circle Redmont® to ensure handling of panels in a manner that will prevent undue stress on component parts, sealants, and structural members. Do not rack, torque, or cause load forces in an inappropriate manner. Lift panels from top only unless specifically instructed by the manufacturer. Prevent damage to finished surfaces. Do not install components that have been damaged or stained.

B. Store panels in a dry place, off the ground. Bear fully along all supported edges on level and true structural supports.

C. Handle materials to prevent damage to finished surfaces. Do not install components which have been damaged.

D. Upon completion of installation, G.C. shall protect panels from damage caused by ensuing work of other trades.

3.2 PANEL INSTALLATION

A. Contractor shall coordinate and set panels into prepared openings and shall bear fully along all supported edges on structural framing supports. Top of panels shall finish flush with adjoining surfaces unless shown otherwise on drawings. Where necessary, build up support ledges and beams as required with materials similar to support framing members, prior to placement of panels. Panels shall be set to proper pitch (minimum ¼" pf) and cross falls to ensure proper drainage of surface water and avoid ponding. Fasten panels as indicated on shop drawings.

Image 4: Structural Glass Deck, Technical Information (Cont.)

B. Allow 1/2" spacing between panels and adjacent surfaces and between adjoining panels to permit installation of closed cell foam backer rods and sealants. All joints which are exposed to the weather and to surface traffic shall be sealed with sealant manufacturer's recommended sealant.

C. Protect installed panels from damage during ensuing construction operations. Prior to the date of substantial completion replace any cracked, broken or otherwise damaged glazing units.

3.3 FIELD TESTING

A. After panel installations are completed they shall be field tested for leakage. Tests shall be conducted by flooding the surface of panels with a sprinkler hose for a period of 15 minutes while observations are made of the undersides. Correct any deficiencies that are found in a manner to make panels completely watertight. Conduct testing in the presence of the Architect or the Architect's designated representative.

3.4 PAINTING

A. All interior surfaces of the prefabricated metal grid that are exposed to view shall be finished after installation in accordance with the requirements of SPECIAL COATING SECTION. Care shall be taken so that the coating will not extend onto the glazing units. If spray application is used, mask all glazing.

3.5 CLEANING

A. Maintain installed panels, including glazing, in reasonably clean condition during construction operations. Remove any stains or materials that may have an adverse effect on panel materials and finishes. Remove any excess glazing compound and sealants.

B. Immediately prior to the date of substantial completion clean glazing units to remove any accumulations of dirt, paint stains, etc. Glazing shall be cleaned on both inside and outside surfaces.

3.6 REPLACEMENT INSTRUCTION

A. G.C. shall furnish the Owner with a copy of the panel manufacturer's complete printed instructions for replacement of any damaged glazing units.

END OF SECTION

Image 5: Structural Cable, Technical Information

Lexco Cable
7320 W. Agatite Ave.
Norridge, IL 60706
Toll Free: 1-800-626-6556
Phone: 1-773-588-8890

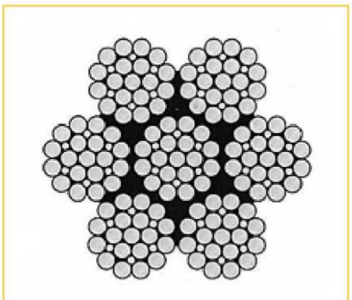
Phone: 1-708-697-7310
Fax: 1-773-478-4584
Email: Sales@LexcoCable.com
Website: www.lexcocable.com

Galvanized Structural Bridge Rope ASTM A603

Minimum breaking strength in tons of 2000 lbs Actual strand construction varies by diameter.
For example, 3/8-3/4"" is typically 7x7, 1"" is typically 6x25.

Warning: Wire rope & aircraft cable should never be used at breaking strength. 10:1 safety factor minimum is recommended for critical/overhead application. Do not use for overhead lifting without consulting a wire rope rigging professional. End fittings may not hold to 100% of strength efficiency so a safety factor such as 5:1 should be

+ more



1 2

Item #	Dia (in)	Class A Coating Throughout	Class A Coating Inner Wires/ Class B Coating Outer Wires	Class A Coating Inner Wires/ Class C Coating Outer Wires	Class C Coating Throughout
ASTM603-3/8	3/8	6.5	6.3	6.1	5.9
ASTM603-7/16	7/16	8.8	8.5	8.2	8
ASTM603-1/2	1/2	11.5	11.1	10.7	10.5
ASTM603-9/16	9/16	14.5	14	13.5	13.2
ASTM603-5/8	5/8	18	17.4	16.8	16.4
ASTM603-11/16	11/16	21.5	20.8	20	19.5

Image 6: Structural Cable, Technical Information (Cont.)

Item #	Dia (in)	Class A Coating Throughout	Class A Coating Inner Wires/ Class B Coating Outer Wires	Class A Coating Inner Wires/ Class C Coating Outer Wires	Class C Coating Throughout
ASTM603-3/4	3/4	26	25.1	24.2	23.6
ASTM603-13/16	13/16	30	29	28	27.3
ASTM603-7/8	7/8	35	33.8	32.6	31.8
ASTM603-15/16	15/16	40	38.6	37.3	36.4
ASTM603-1	1	45.7	44.1	42.6	41.5
ASTM603-1 1/8	1 1/8	57.8	55.8	53.9	52.5
ASTM603-1 1/4	1 1/4	72.2	69.7	67.3	65.6
ASTM603-1 3/8	1 3/8	87.8	84.8	81.8	79.8
ASTM603-1 1/2	1 1/2	104	100	96.9	94.5
ASTM603-1 5/8	1 5/8	123	120	117	112
ASTM603-1 3/4	1 3/4	143	140	136	130
ASTM603-1 7/8	1 7/8	164	160	156	149
ASTM603-2	2	186	182	177	169
ASTM603-2 1/8	2 1/8	210	205	200	191
ASTM603-2 1/4	2 1/4	235	230	224	214
ASTM603-2 3/8	2 3/8	261	255	249	237
ASTM603-2 1/2	2 1/2	288	281	275	262

Image 7: Structural Cable, Technical Information (Cont.)

Item #	Dia (in)	Class A Coating Throughout	Class A Coating Inner Wires/ Class B Coating Outer Wires	Class A Coating Inner Wires/ Class C Coating Outer Wires	Class C Coating Throughout
ASTM603-2 5/8	2 5/8	317	310	302	288
ASTM603-2 3/4	2 3/4	347	339	331	315

1 2

APPENDIX D - EXISTING TOPOGRAPHY DATA

Table 1: Summary of Data Collector Points

Pt. No.	Northing	Easting	Elev.	Name
1	1000.002	1000.002	100	Bm1
2	1096.136	1000.002	99.413	Bm2
3	939.498	1008.552	101.934	tp1
4	937.235	1026.884	100.594	tp2
5	935.384	1046.06	99.259	tp3
6	934.158	1065.651	98.209	tp4
7	931.333	1086.698	97.767	tp5
8	925.892	1108.201	97.269	tp6
9	913.549	1125.576	97.583	walkway
10	959.117	1147.892	97.484	walkway2
11	983.585	1155.943	97.421	walkway3
12	1008.829	1160.067	97.478	walkway4
13	1030.889	1161.862	97.525	walkway5
14	1051.802	1162.903	97.365	walkway6
15	1081.077	1163.288	97.404	walkway7
16	1105.762	1162.993	97.499	walkway8
17	1116.706	1160.221	97.451	walkway9
18	1125.353	1153.148	97.237	walkway10
19	1130.559	1142.416	97.008	walkway11
20	1129.998	1093.73	97.609	walkway12
21	1129.711	1050.633	98.522	walkway13
22	1129.843	1026.455	99.289	walkway14
23	1129.253	995.867	100.471	walkway15
24	1145.308	1004.181	100.242	walkway16
25	1145.419	1019.753	99.613	walkway17
26	1146.378	1067.871	97.811	walkway18
27	1146.974	1131.679	97.173	walkway19
28	1152.68	1153.351	97.393	walkway20
29	1170.976	1161.443	97.515	walkway21

30	1166.669	1174.27	97.993	walkway22
31	1157.908	1175.043	97.822	walkway23
32	1156.538	1174.109	97.792	walkway24
33	1140.017	1198.685	98.461	walkway25
34	1121.785	1174.357	97.825	walkway26
35	1113.261	1175.719	97.823	walkway27
36	1093.727	1176.25	97.546	walkway28
37	1044.14	1175.72	97.658	walkway29
38	991.78	1170.827	97.434	walkway30
39	986.797	1134.151	97.283	tp50
40	980.272	1084.858	98.214	tp51
41	988.561	1043.915	99.24	tp52
42	994.613	1003.141	99.853	tp53
43	1036.499	1000.88	99.708	tp54
44	1035.158	1045.305	98.943	tp55
45	1041.391	1093.767	97.958	tp56
46	1029.524	1186.988	96.14	dt1
47	967.808	1185.003	95.601	tree1
48	974.3	1185.528	94.771	tree2
49	986.89	1206.776	88.849	tree3
50	986.691	1206.694	88.849	tree4
51	974.47	1197.725	91.123	bank11
52	956.349	1194.851	91.207	bank12
53	936.84	1191.297	89.807	bank13
54	1003.136	1199.923	89.535	bank14
55	1015.673	1198.168	90.303	bank15
56	1044.385	1191.841	93.195	bank16
57	1094.059	1185.427	96.865	topbank1
58	1070.506	1180.965	97.562	topbank2
59	1045.163	1183.48	97.065	topbank3
60	1011.635	1182.833	96.788	topbank4
61	959.896	1179.287	97.279	topbank5
62	931.218	1169.883	97.29	topbank6

63	912.252	1140.107	97.891	tree10
64	929.907	1130.298	99.157	utility1
65	929.614	1147.635	97.905	lp2
66	1002.852	1173.224	97.361	lp3
67	1085.794	1177.172	97.8	lp4
68	1109.155	1181.98	97.987	topbank50
69	1171.88	1179.255	97.433	topbank51
70	1206.31	1177.546	97.16	topbank52
71	1260.54	1165.006	96.81	topbank53
72	1323.71	1142.666	97.016	topbank54
73	1357.182	1116.753	95.582	exbridge1
74	1362.889	1125.297	96.17	exbridge2
75	1369.947	1113.421	95.712	exbridge3
76	1373.041	1121.245	96.185	exbridge4
77	1388.461	1161.234	97.218	exbridge5
78	1393.278	1197.413	97.348	exbridge6
79	1413.391	1244.815	95.976	exbridge7
80	1402.59	1249.448	95.543	brgwalkway1
81	1377.954	1259.794	94.449	brgwalkway2
82	1342.491	1265.277	94.632	brgwalkway3
83	1349.2	1276.584	94.435	brgwalkway4
84	1271.566	1277.994	95.819	brgwalkway5
85	1221.752	1289.845	96.301	brgwalkway6
86	1152.437	1302.552	95.599	brgwalkway7
87	1125.862	1305.804	95.034	brgwalkway8
88	1095.554	1299.622	95.731	topbank100
89	1017.991	1292.603	97.755	topbank101
90	1219.561	1281.699	96.096	topbank102
91	1266.218	1269.313	95.396	topbank103
92	1319.628	1242.001	94.847	topbank104
93	1363.45	1227.235	94.447	topbank105
94	1190.732	1156.179	97.21	utility5
95	1128.278	1160.379	97.59	mh1

96	928.302	1148.402	97.965	bench1
97	965.369	1005.454	101.948	conpad1
98	945.473	1005.521	102.152	conpad2
99	911.254	997.339	102.221	conpad3
100	913.232	1030.838	98.758	conpad4
101	910.126	1086.411	97.953	conpad5
102	949.559	1063.973	98.096	lp15
103	1055.524	990.096	99.619	bm3

End of Table 1: Summary of Data Collector Points

APPENDIX E - DRAWINGS

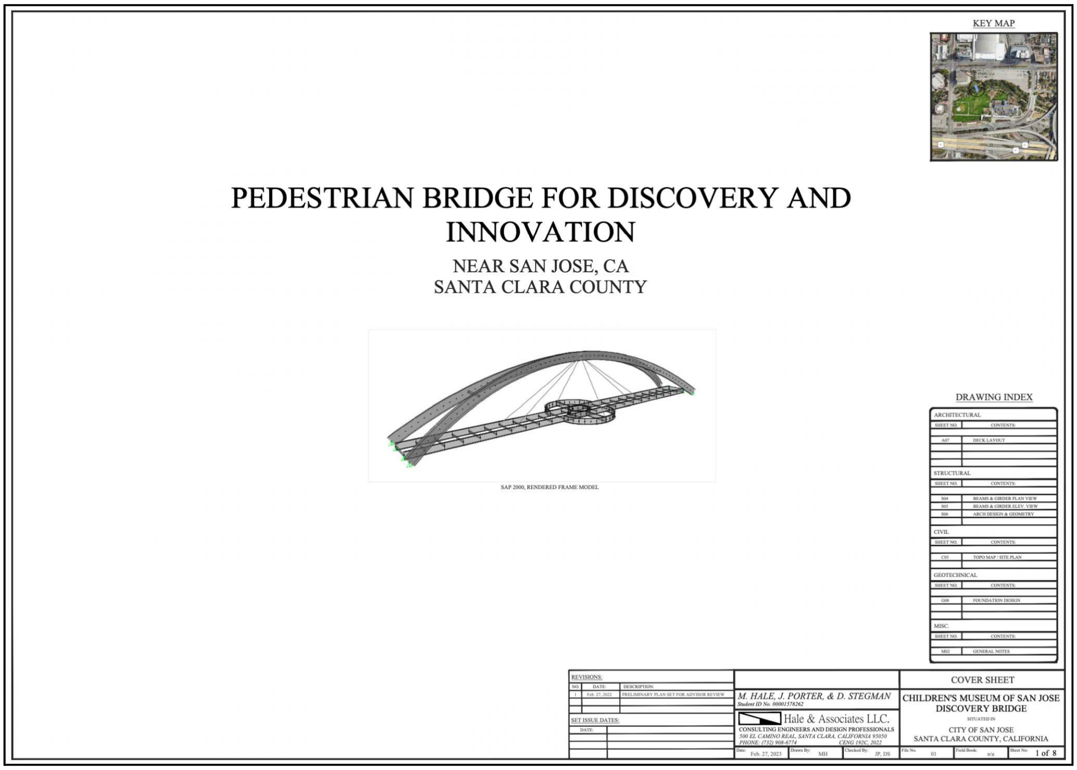


Image 1: Screenshot of Cover Sheet

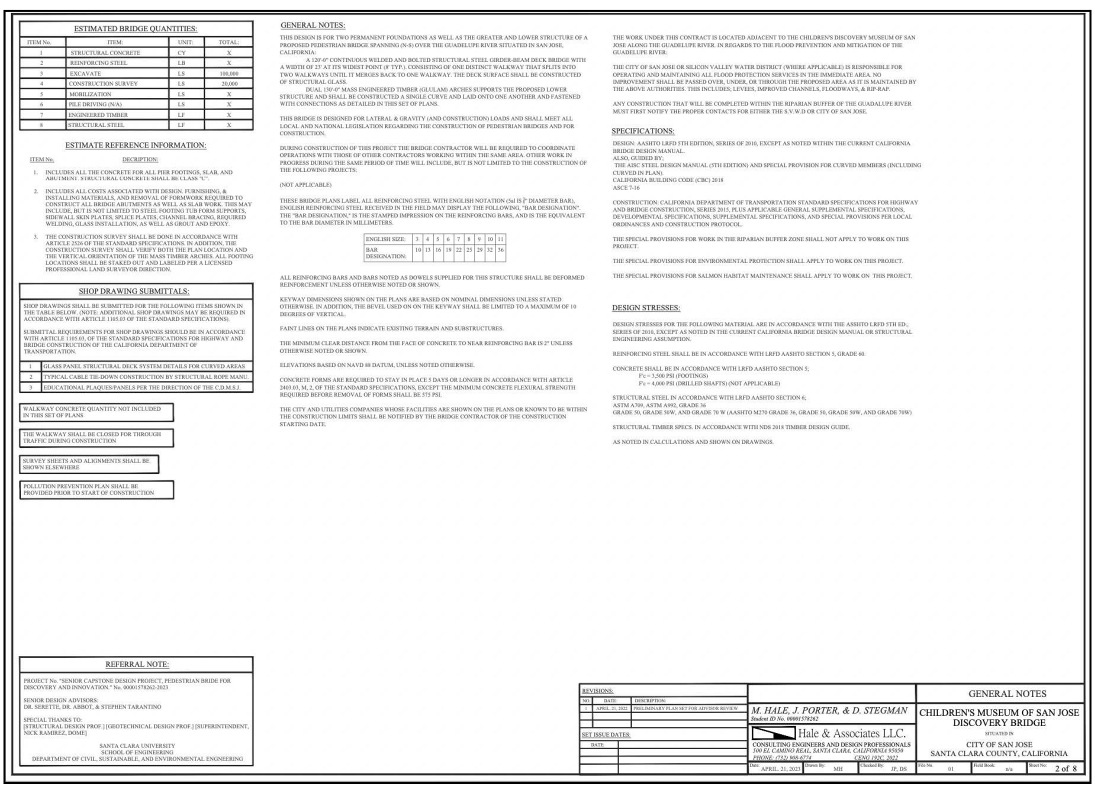


Image 2: Screenshot of General Notes

APPENDIX E - DRAWINGS

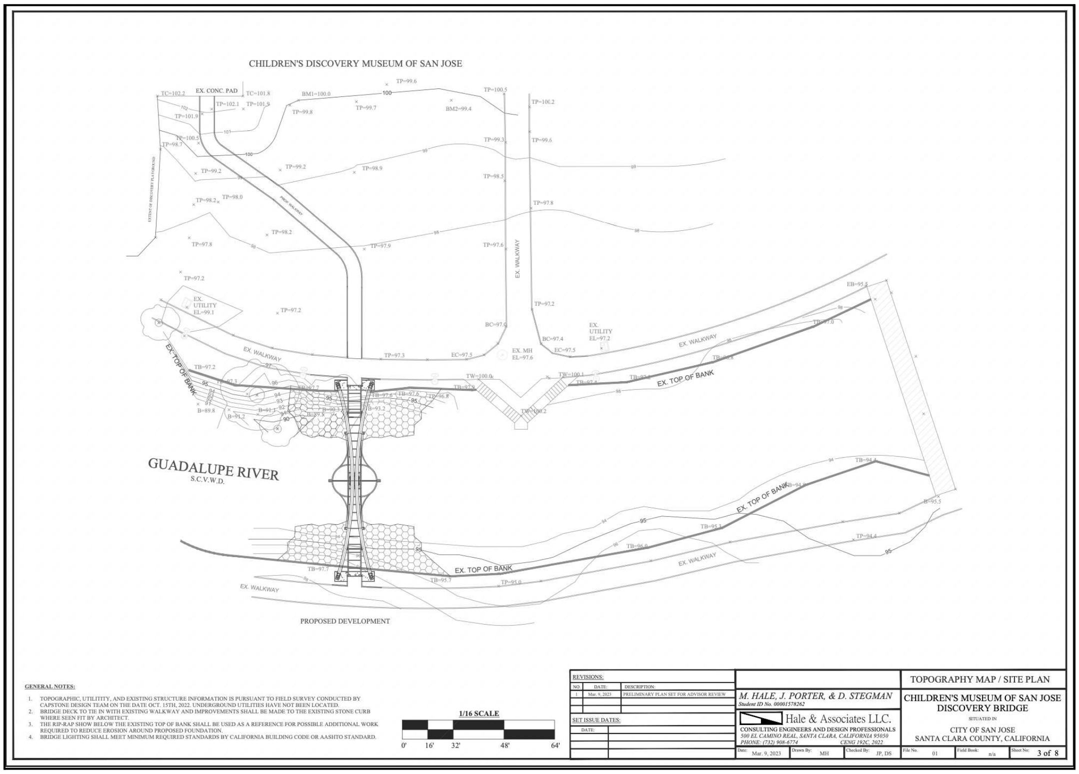


Image 3: Screenshot of Site Plan

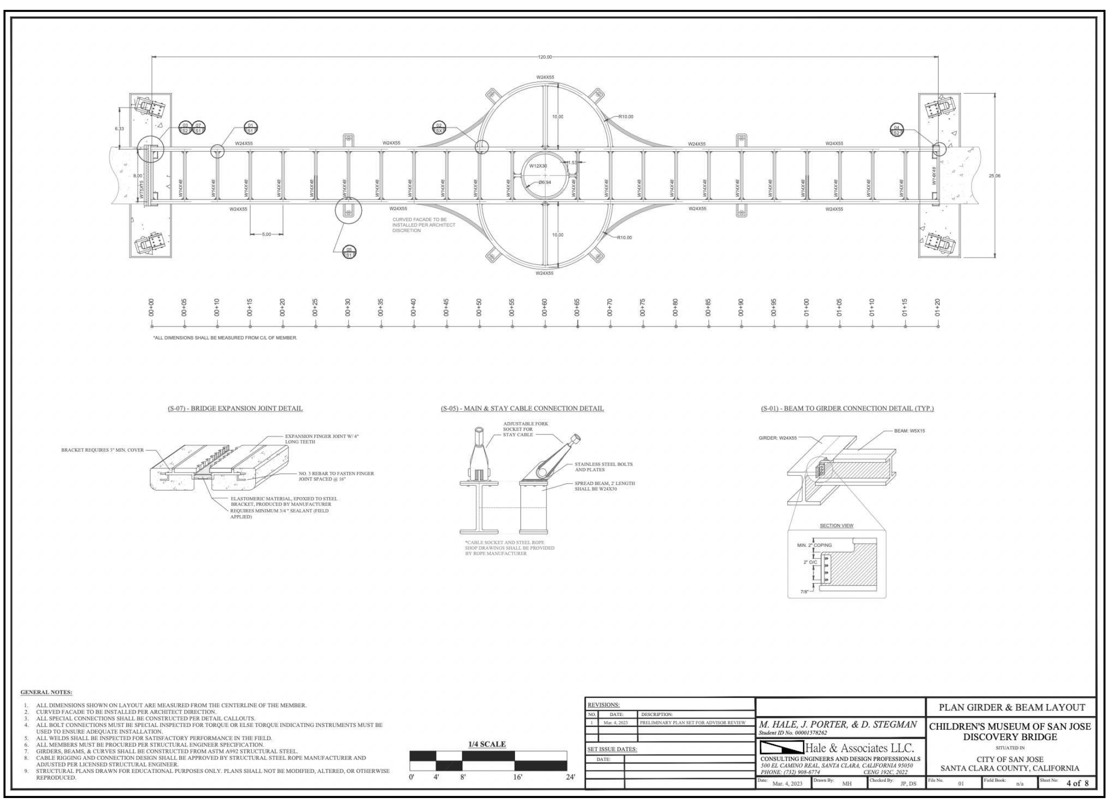


Image 4: Screenshot of Girder & Beam Layout

APPENDIX E - DRAWINGS

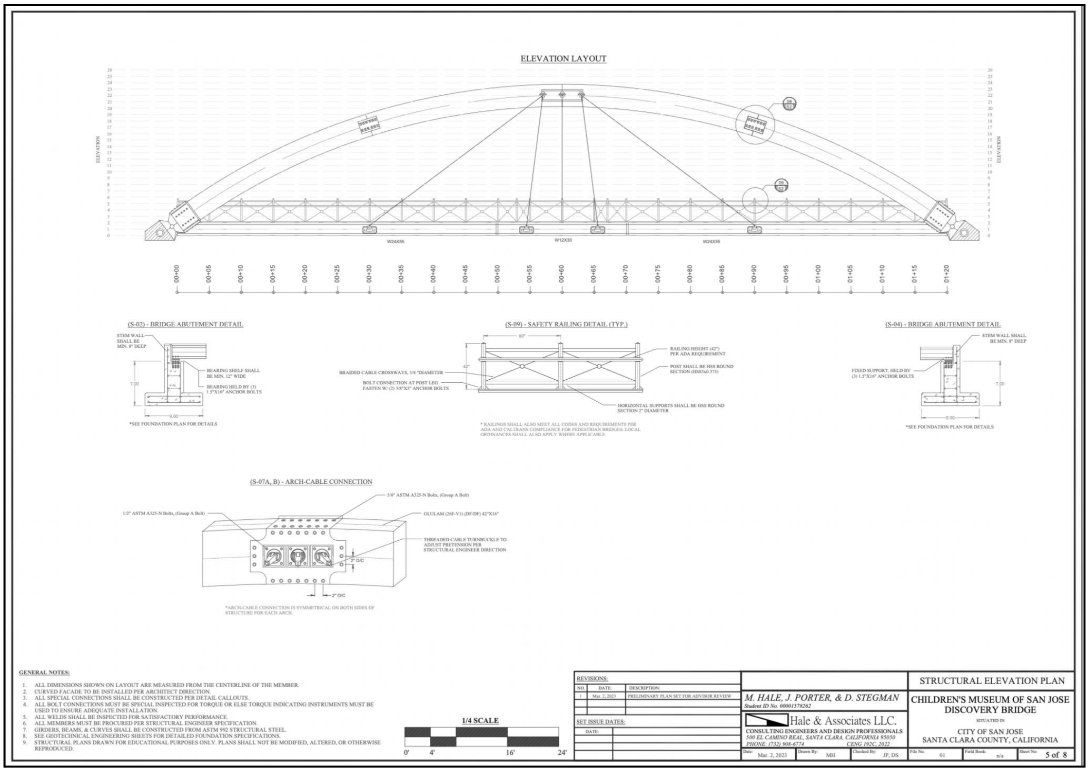


Image 5: Screenshot of Elevation Plan

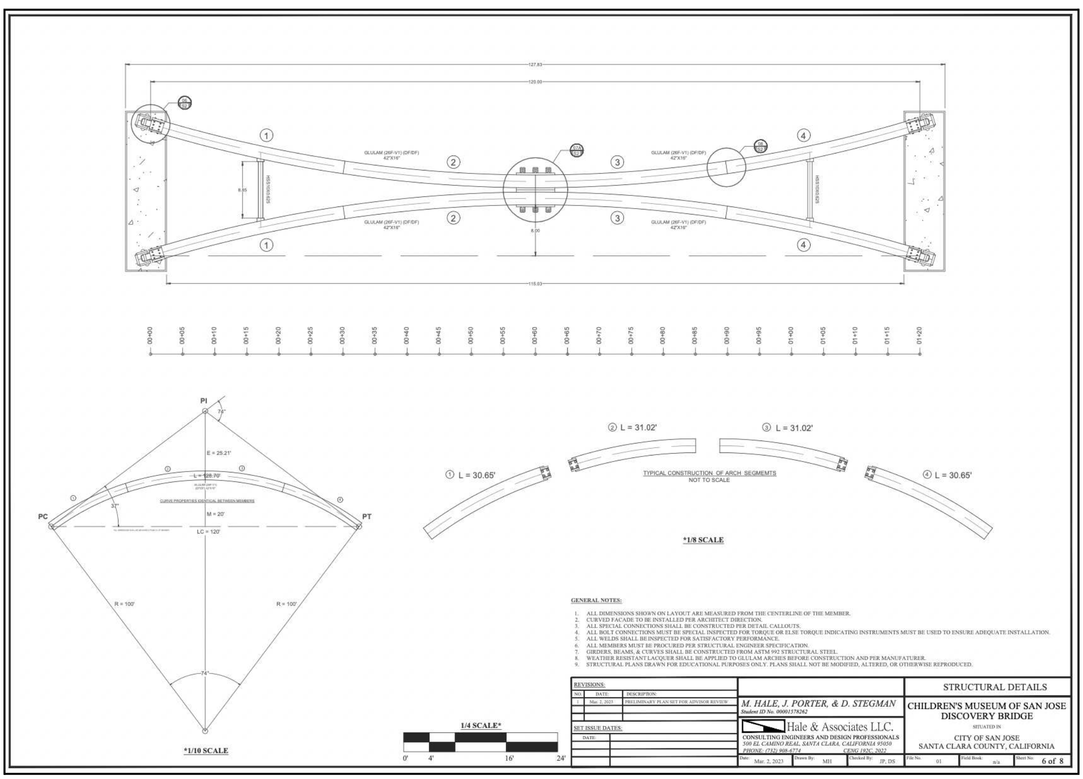


Image 6: Screenshot of Structural Details

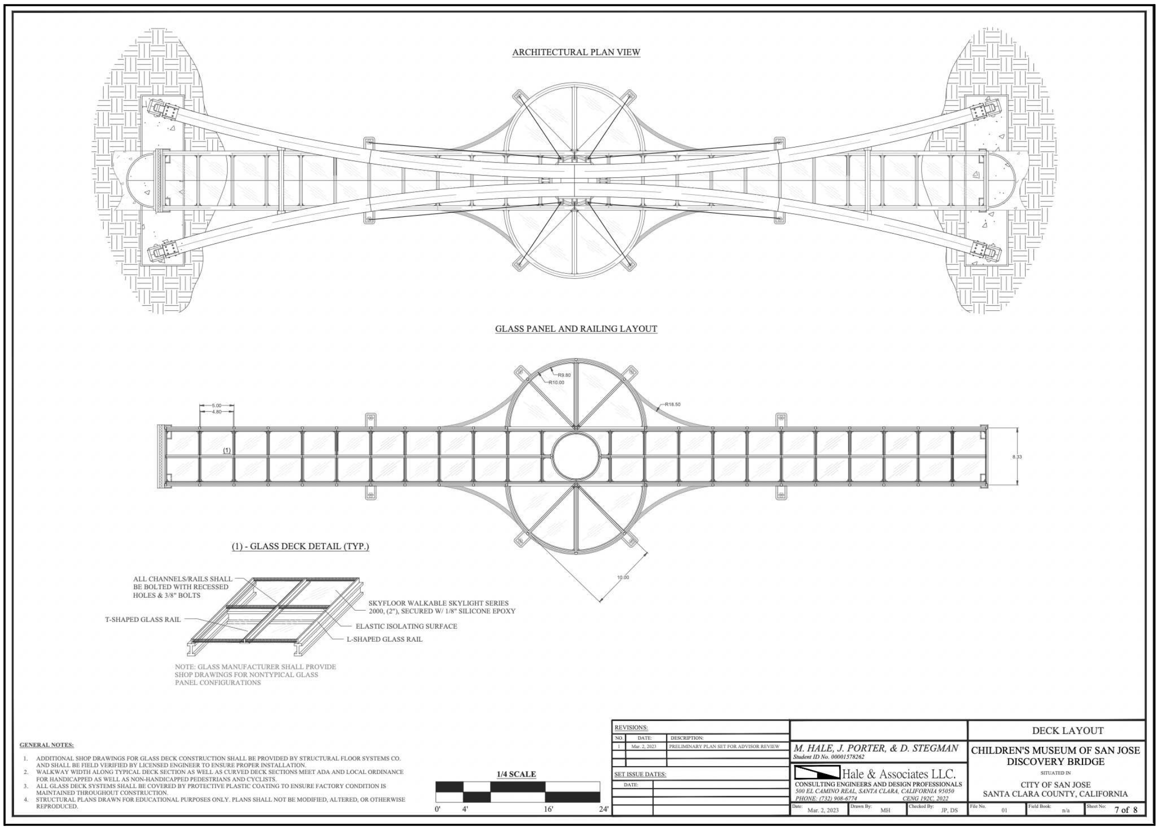


Image 7: Screenshot of Deck Layout

APPENDIX F - SAP ANALYSIS MODELS

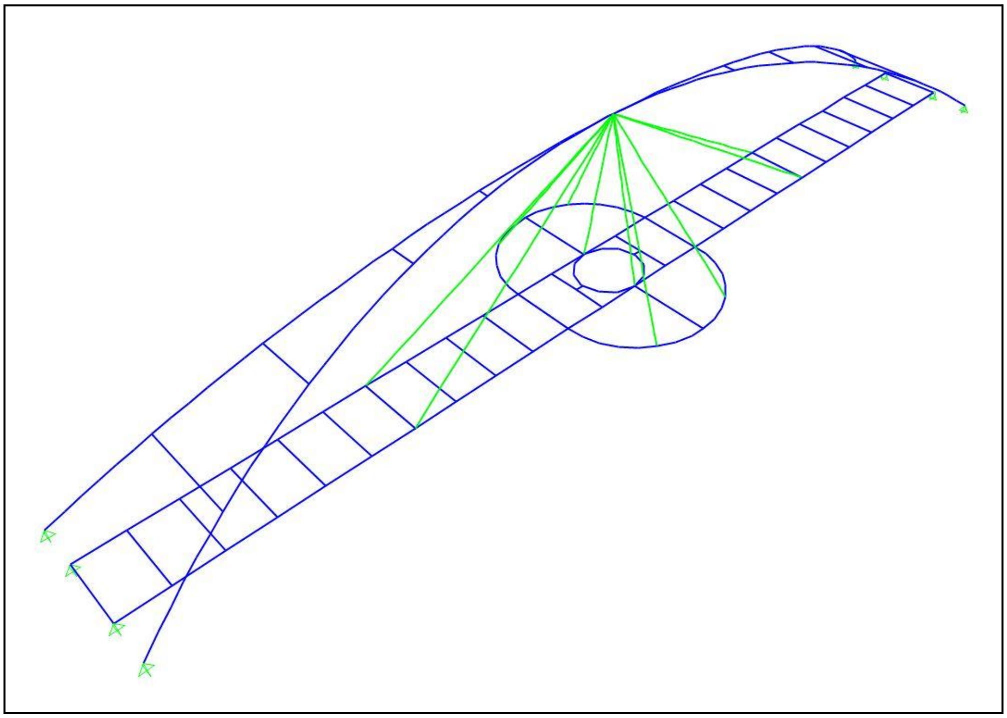


Figure F-1: SAP2000 Model before analysis.

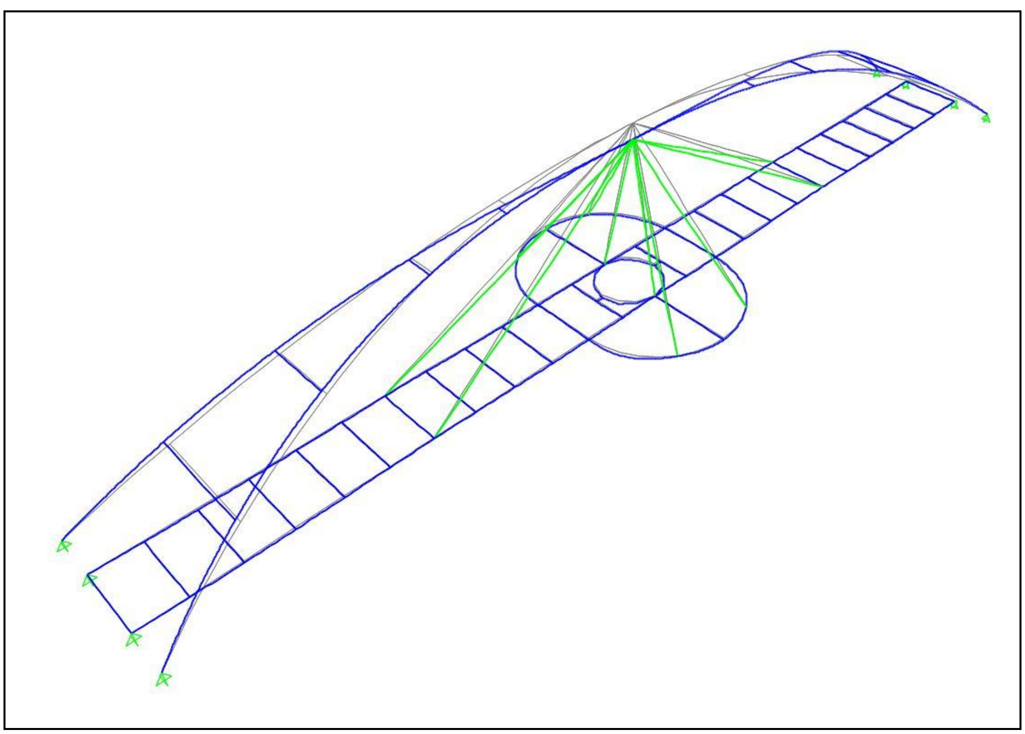


Figure F-2: Deflected shape from dead loading.

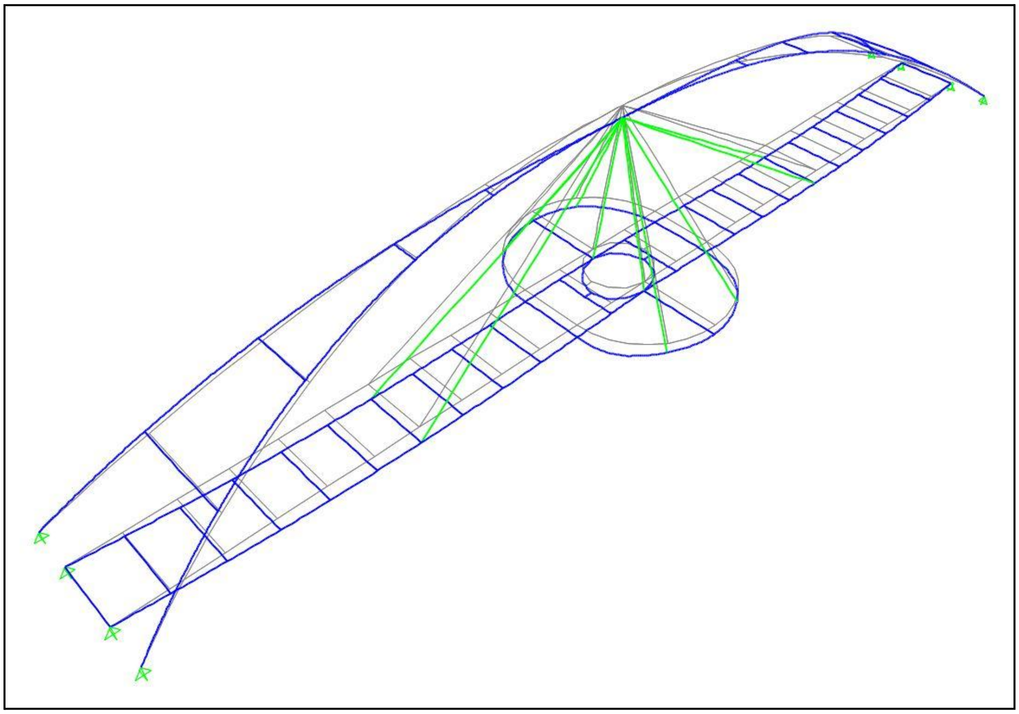


Figure F-3: Deflected shape from STRENGTH I load combination.

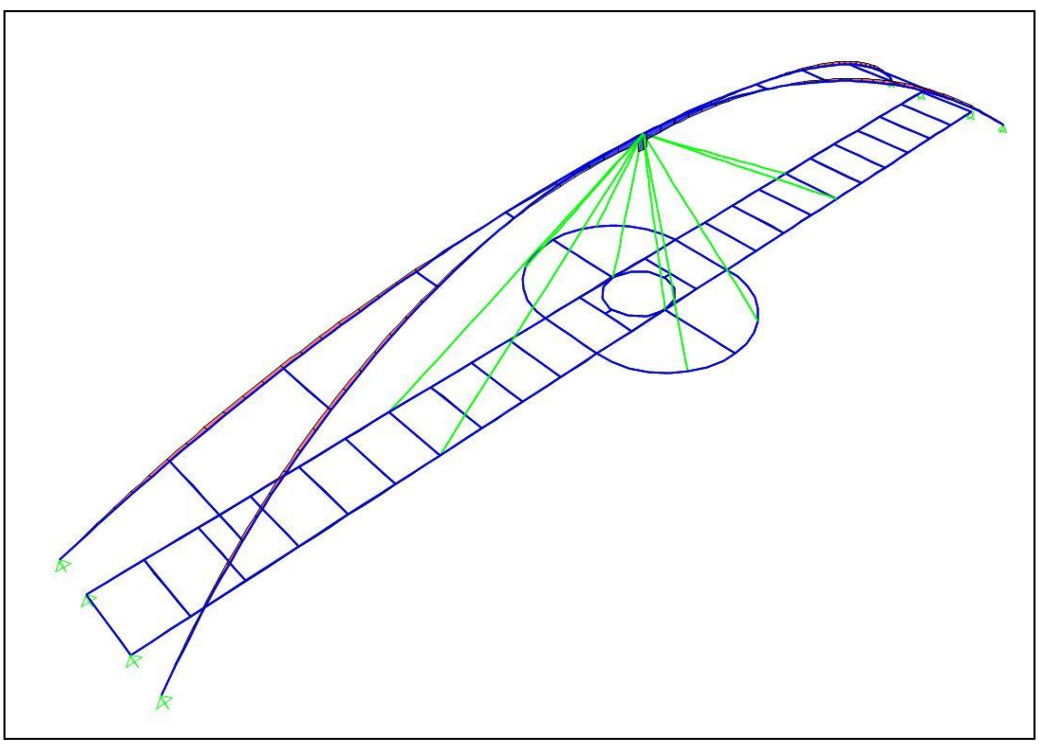


Figure F-4: Moment generated from gravity loading.

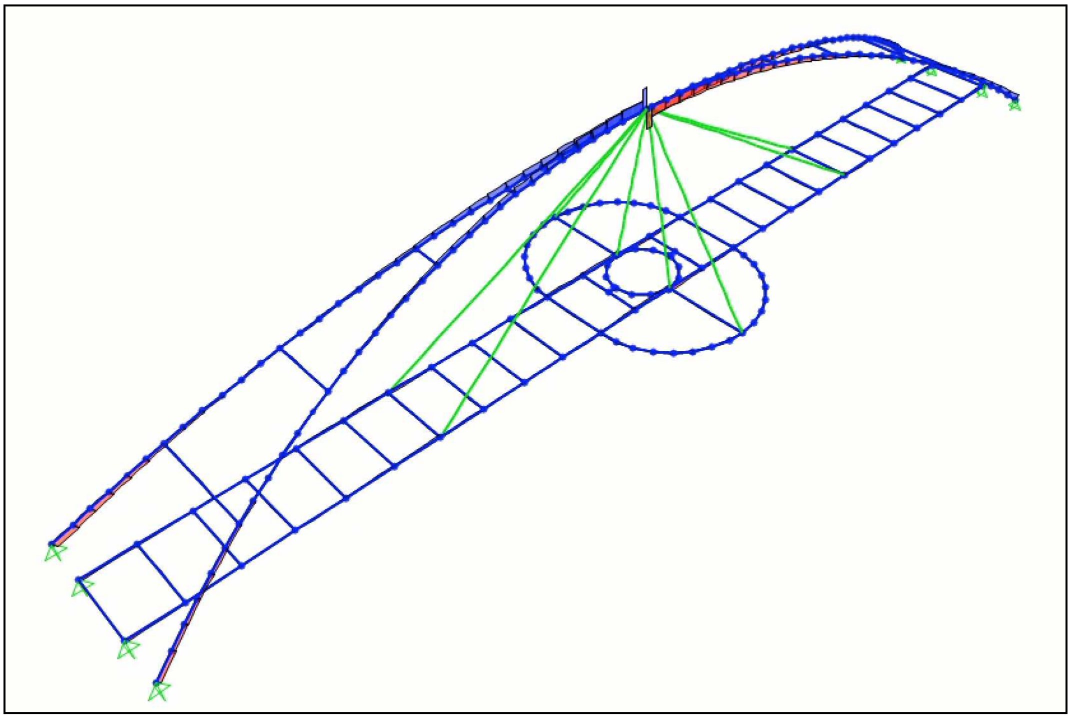


Figure F-5: Shear generated from gravity loading.

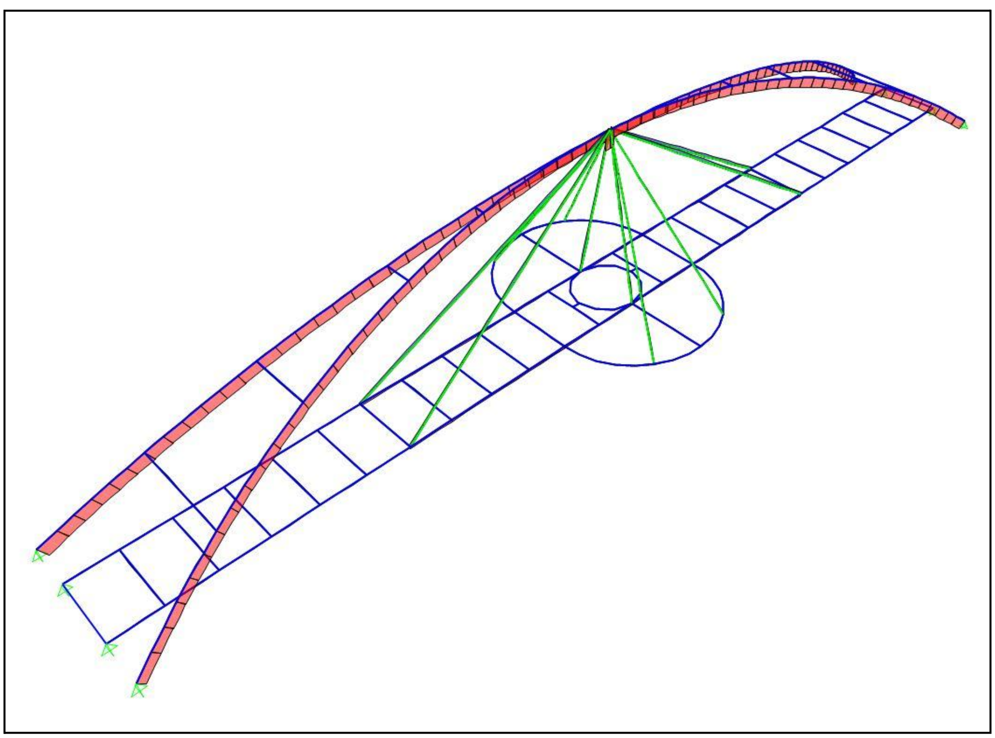


Figure F-6: Axial load generated from gravity loading.

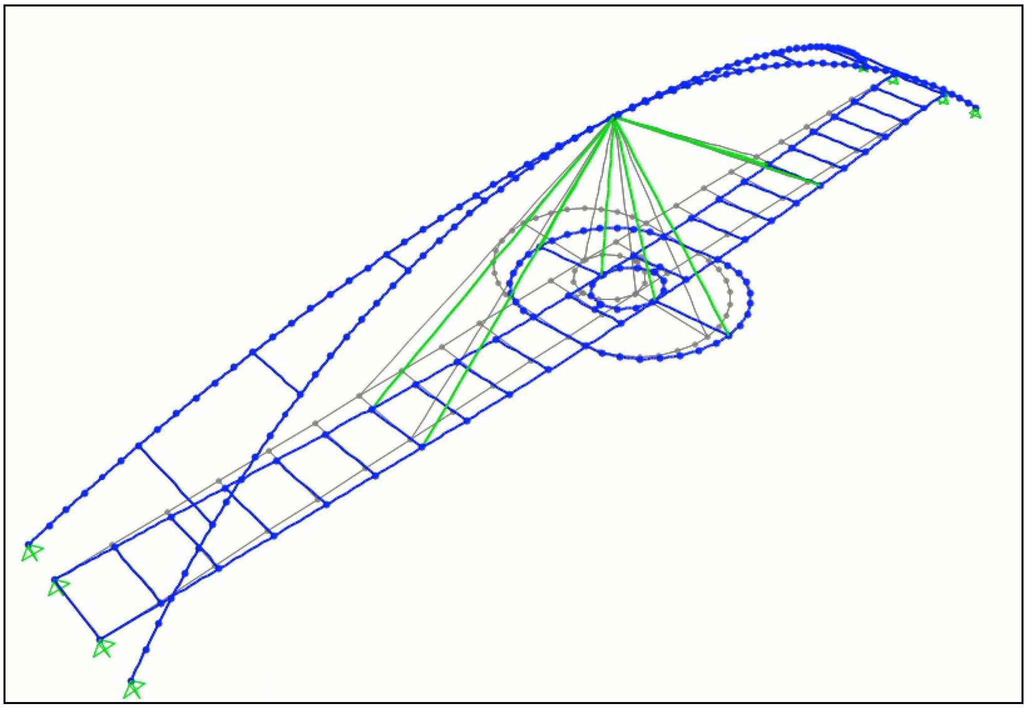


Figure F-7: Deflected shape from mode 1 of modal analysis.