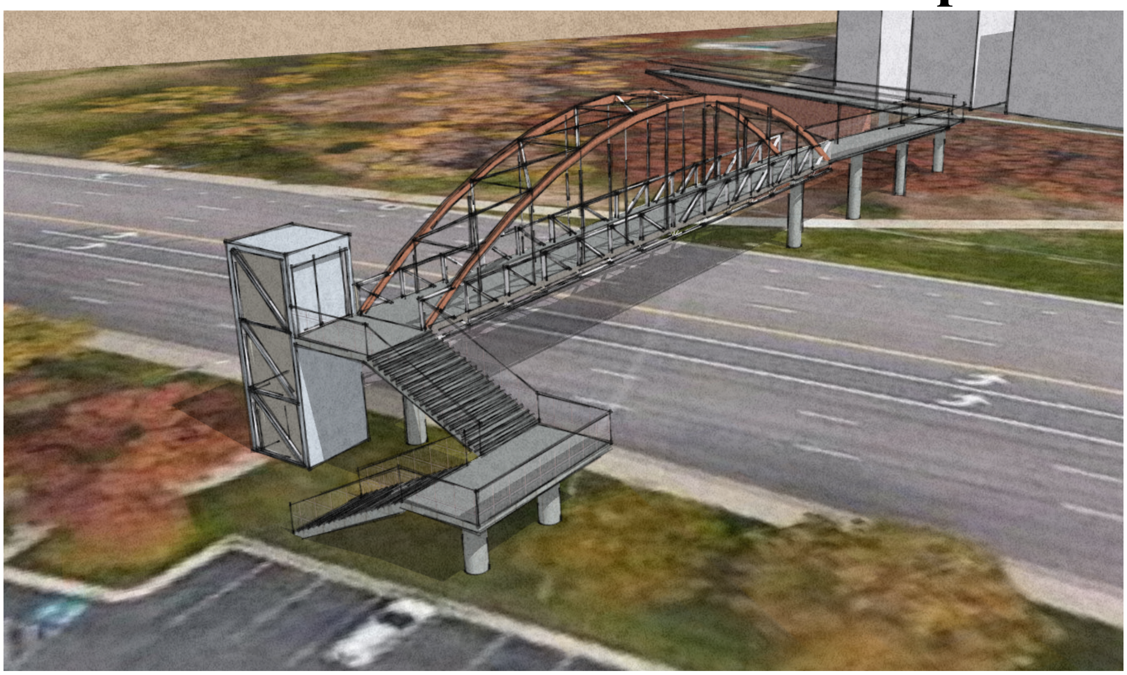
Red Butte Creek Pedestrian Overpass



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Disclaimer:

The study described herein is associated with the courses CvEEN 4900 and 4910 Professional Practice & Design conducted at the University of Utah, 2022-23. As such, it is intended for academic use only. Engineering opinions and descriptions provided are likewise only for academic use.

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OVERALL SUMMARY

The work encompassed in this report represents the research and design for a pedestrian bridge spanning Foothill Drive near Wakara Way. This proposed crossing is designed to facilitate and effectively allow pedestrians and cyclists to cross Foothill without the need for an at grade crosswalk. Furthermore, this trail will advance Salt Lake City's master plan by connecting the Red Butte Creek trailhead to the future Bonneville Creek trail.

The design within this report consists of a prefabricated truss bridge with non load bearing arches for the appearance of a tied arch bridge. The west end connects to the existing sidewalk via a staircase/elevator combination with bike rails to allow parties of all abilities to access the crossing. The east end will connect to the future Bonneville Creek Trail via a ramp supported by a system of columns and MSE wall support.

Site planning included analyses of soil bearing capacities, riparian corridor concerns, and identification of existing trees and utilities. These analyses have been factored into the current design and are the reasoning behind the current placement of the crossing, ramps, stairs, and columns.

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1. Project Summary

1.1 Project Needs Statement

Vehicular traffic on Foothill Drive poses a danger to pedestrians crossing the road due to long crosswalks and high traffic speeds. There is one documented pedestrian death on Foothill Drive within the project area from a vehicle collision. As such, there is a need for a grade-separated crossing over Foothill Drive between Mario Capecchi and Wakara Way to safely connect the future Red Butte Creek Trail.

This project is planned within the scope of long term master plans for both Salt Lake City and UDOT. This project aims to design an overpass crossing; this crossing will meet the needs of the stakeholders and will serve as a starting point for future conversations involving the implementation of pedestrian-friendly infrastructure and complete streets in Salt Lake City.

1.2 Project Goals and Vision

Along with increasing the safety for crossing, the crossing will allow Sunnyside residents a direct connection to the University of Utah, as well as connection to the trails system above Foothill. With a well-used pedestrian connection, Foothill may also see a decrease in the high traffic volume which currently experiences a level of service of F. The crossing hopes to incentivise drivers to switch to pedestrian transportation and reduce the pedestrian crossing times of Foothill, leading to a more efficient roadway. The project site is directly adjacent to Red Butte Creek, and the crossing is expected to tie into a future multi-use trail, currently named the Red Butte Creek Trail; this will provide a connection to the existing trails along the Wasatch Front including the Bonneville Shoreline Trail.

1.3 Project Participants and Organization

Salt Lake City and students in the University of Utah Civil and Environmental engineering capstone class have collaborated to design a grade-separated, multi-use crossing over Foothill Drive. Salt Lake City has served as the main client, with city staff reviewing the work produced from students. In addition to the instructional team, mentors from Parametrix have provided guidance and feedback in frequent meetings on the project’s progress.

1.4 Stakeholders

Relevant project stakeholders have been evaluated using a three-dimensional stakeholder cube, as shown in Figure 1.4.1. The cube is more complex than a traditional 2x2 matrix, however it provides the opportunity to include the high power, low interest who oppose the project and don’t fit on the matrix. There are eight different types of stakeholders with the influential active blocker having the highest priority and the insignificant passive backer having the lowest priority. The prioritization of stakeholders specific to this project is provided in a later section.

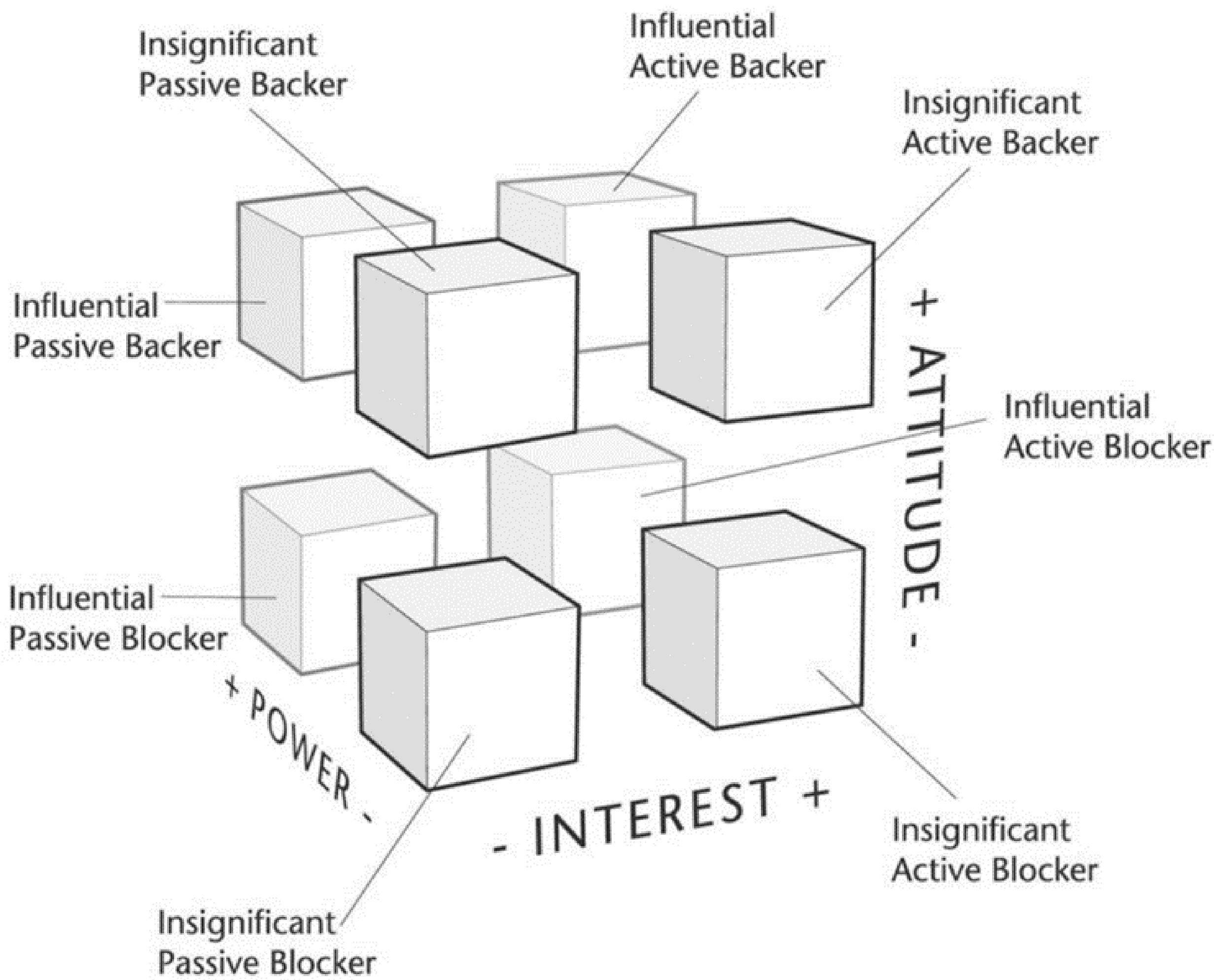


Figure 1.4.1 - Graphic of Stakeholder Analysis Cube [1]

An in depth analysis of stakeholders is located in section 3.

2. SITE DESCRIPTION AND ANALYSIS

2.1 Location Details and Observations

Figure 2.1.1 - Google Earth view of project site [2]

The project site is located east of Salt Lake City along Foothill Drive near Red Butte Creek. The boundary for which a crossing location is being explored spans from the Mario Capecchi Drive intersection to just southeast from the Wakara Way intersection, as shown in Figure 2.1.1. At this location, Foothill Drive is approximately 100 feet in width and has multiple lanes for each direction, which include turn pockets at the intersections. There are sidewalks along both sides of the roadway that are frequently used by college students and workers from Research Park. A bus stop is also in the project location, situated on the northeast corner of Foothill Drive and Wakara Way. Many utilities are present in the area including sewer, water, storm drains, buried and overhead communications and power. The easternmost edge of the street is empty of any significant structures or obstacles, with the exception of a single power pole. The terrain here is most likely non native fill which has been landscaped to create a natural aesthetic. This area is property of the University of Utah. On the adjacent side, there exists the VA Hospital and parking lot. There is the potential for a portion of the parking lot to be set aside for the structure of the crossing, though more information on easements and the right of way is needed for a conclusive design.

Figure 2.1.2 is a map that contains the locations of sewer laterals in Salt Lake City. As seen in the figure the ideal project area crosses a sewer line that is 12 feet deep. The node within the circle will be relocated.

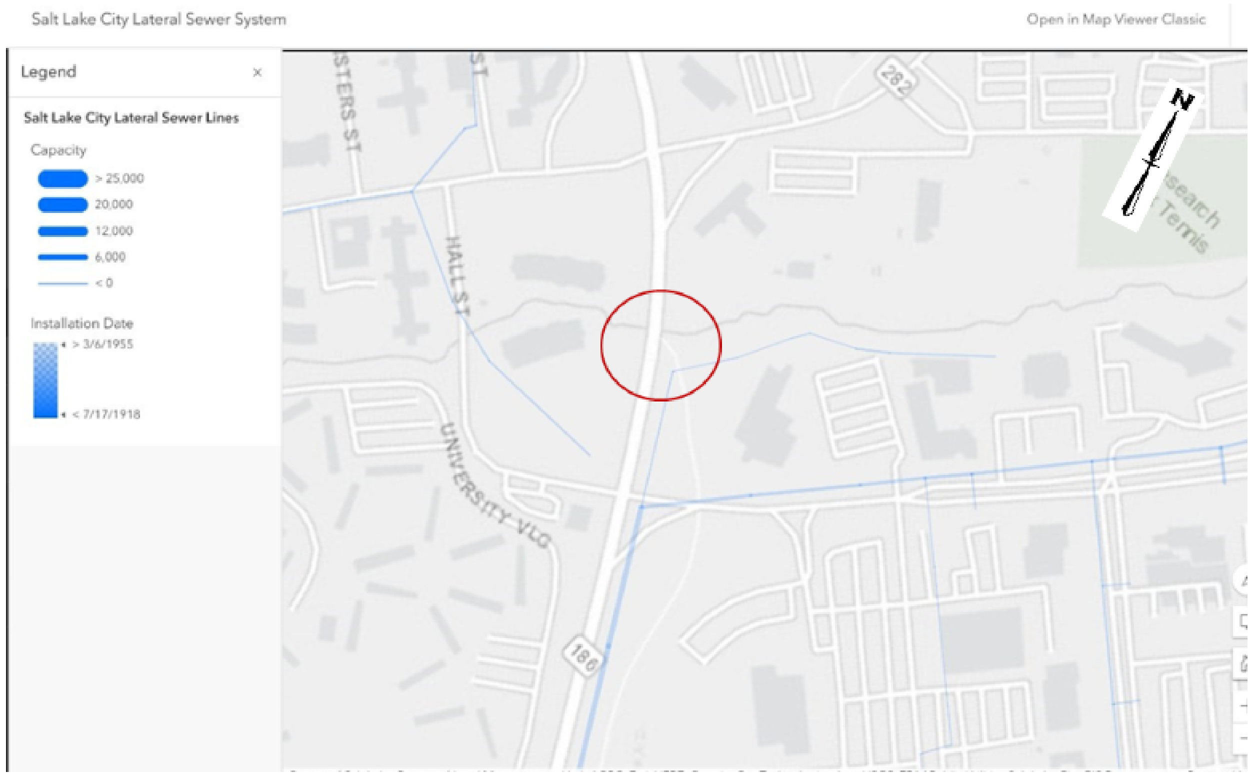


Figure 2.1.2 - Salt Lake City Lateral Sewer System Provided by USGS [3]

2.2 General Site Usage

The current at-grade crossing at the intersection of Foothill Drive and Wakara Way is used primarily by students and employees associated with the University of Utah, in addition to pedestrians and bicyclists. The location of current and future University housing complexes provides a steady pedestrian traffic flow of students and employees biking and walking to work/school. Furthermore, this intersection sees traffic from pedestrians and cyclists traveling to the foothills for recreational purposes. In addition to University and recreation related travel, the placement of the VA Hospital means the intersection sees a volume of patients and workers from the hospital.

Figure 2.2.1 - Foothill Drive looking toward Mario Capecchi intersection [4]

2.3 Geologic and Geotechnical

Figure 2.3.1 - Map of soils present in project area [5]

Figure 2.3.1 is a map of the soil present at the site location. The crossing will be situated along the stony terrace escarpments which is denoted by SP, though it is important to identify the surrounding soils and how they might be impacted by construction. Stony terrace escarpments consist of long, narrow, rocky areas that rise abruptly from the mean tide line to the coastal plain terraces or plateaus. This land type typically consists of steep faces that separate the terraces from the lower lying land. The faces are composed of soft coastal sandstone, hard shale, or hard, weather-resistant, fine-grained sandstone. Vegetation is sparse and is made up of dwarfed shrubs, patches of grass, lichens, and moss. In seepage areas, water grasses, a few cypress and oaks, and various weathered conifers also grow. Areas of terrace escarpments are used mainly for watersheds and as wildlife habitat.

2.4 Hydrologic and Hydraulic

Red Butte Creek is a collection of drainages from Red Butte Creek Canyon that combine into a single waterway as it flows toward the east bench of Salt Lake City. This creek flows underneath Foothill Drive through a 72-inch by 60-inch box culvert that transitions to a 72-inch steel corrugated pipe and flows toward the Sunnyside Community. Evidence of animal tracks and trails are present along each side of the creek as well as some human used paths.

Figure 2.4.1 - Red Butte Creek inlet [6]

Figure 2.4.2 - Red Butte Creek outlet [7]

2.5 Topographic

The topography of the surrounding area consists of a big slope near Red Butte Creek as shown in Figure 2.5.1 below. The west side of the pedestrian bridge topography consists of a small slope, while the east side of the pedestrian bridge has steep topography towards the east. The slope across Foothill Boulevard is gradual, but it does impact structure height to maintain vertical clearance for tall vehicles. The topography that will most affect the structure is the ramp on the east side on the project. The east side ramp column heights needed to be adjusted in size due to the topography of the terrain. Shown in Figure 2.5.1 below is the elevation change from the west side to the east side of the project drawn on a chart. The min elevation in the project area is 4768 feet while the max which is on the east side is 4776 feet.

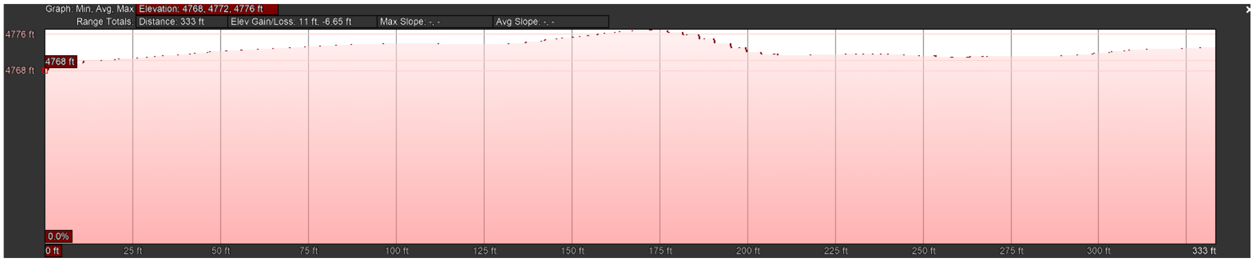


Figure 2.5.1- Existing elevation [8]

Figures 2.5.2 and 2.5.3 show the existing topography for the area surrounding the project site.

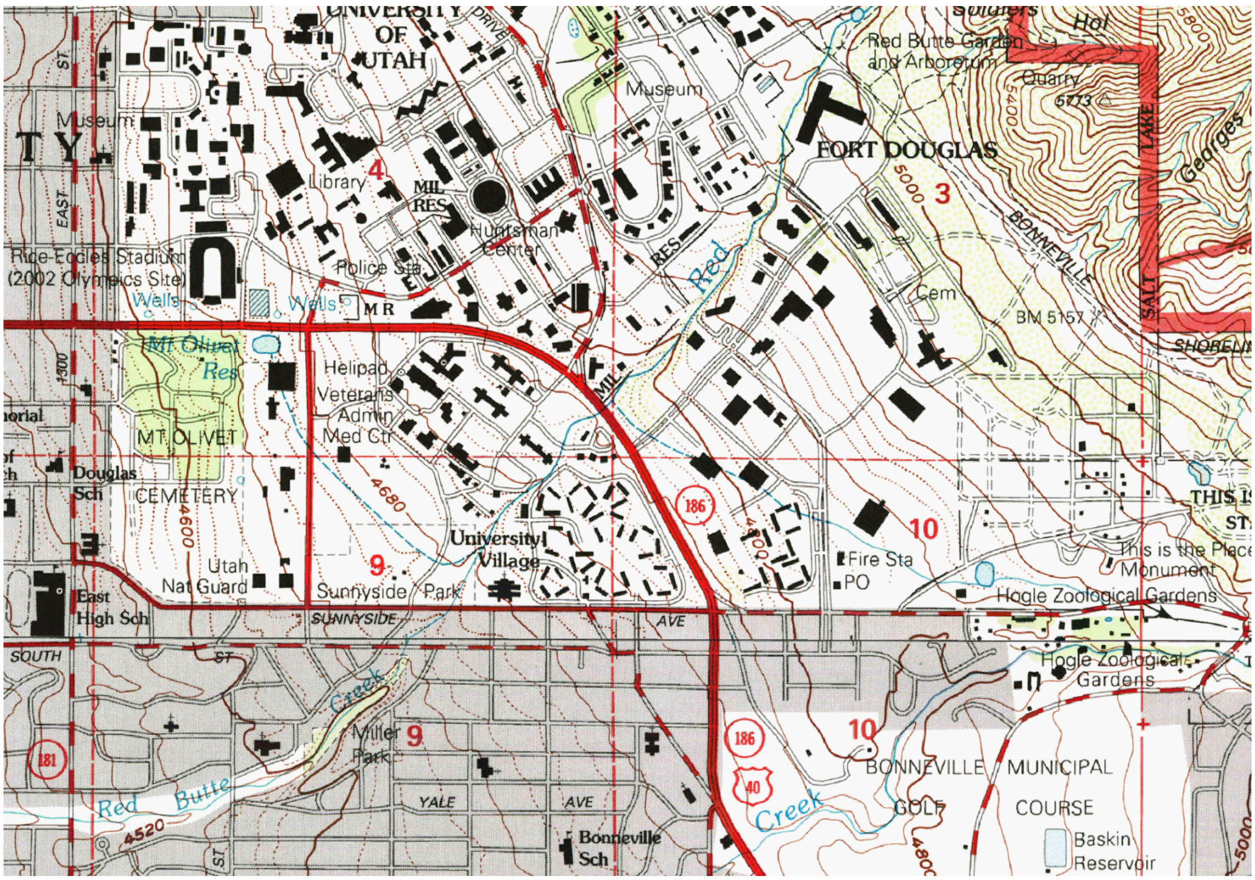


Figure 2.5.2 - Topographic Map of area surrounding project site [9]

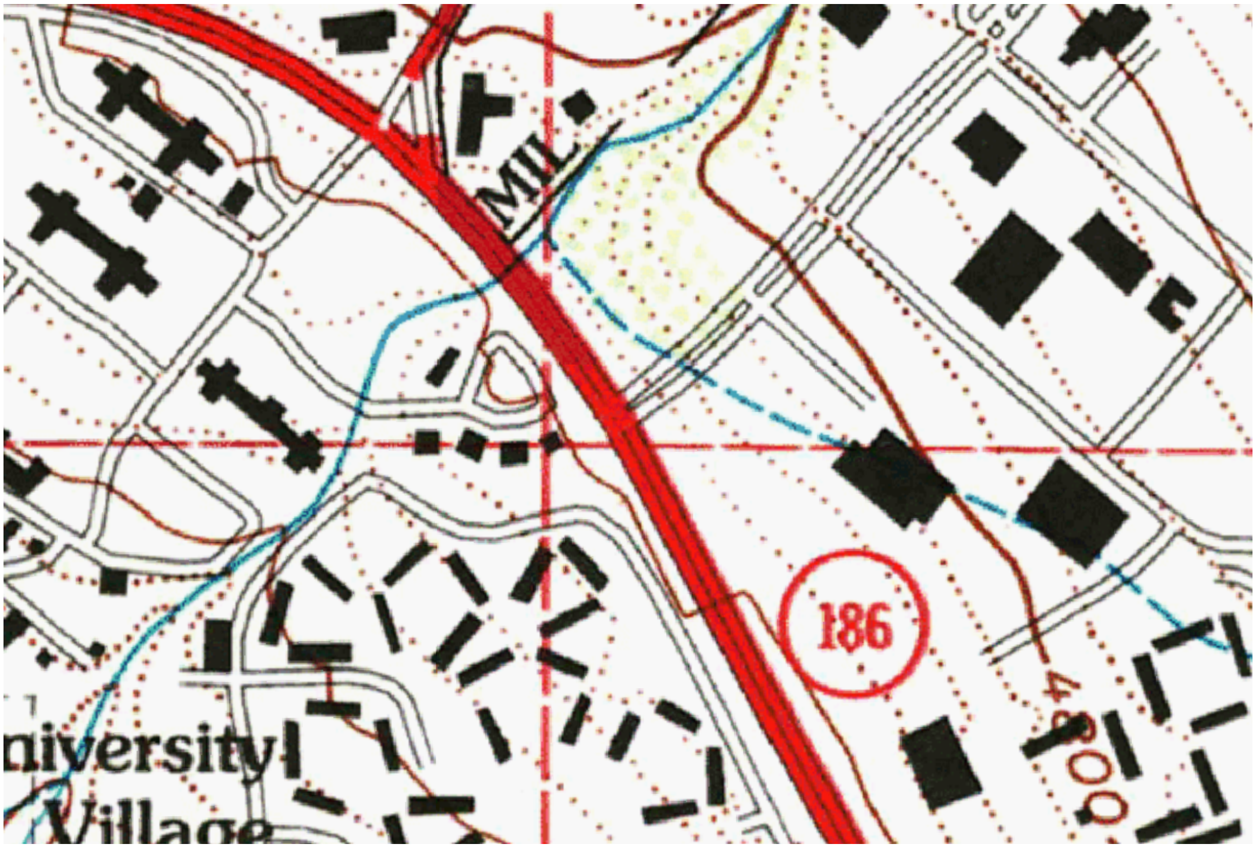


Figure 2.5.3 - Topographic Map of area near project site [9]

3. SUMMARY OF CRITERIA

3.1 Project Criteria

Criteria for the project was included for each main aspect of the project. The criteria was mainly guided by the purpose of enabling pedestrians of all types, ages, and abilities to comfortably use the overpass.

Accessibility was therefore a main criteria. Modes of access to the overpass considered pedestrians who walked, cycled, used wheelchairs, walkers, strollers, and those who cannot easily ascend or descend stairs. Therefore, ramps were used on the east side of Foothill to provide a smooth transition from the trail to the overpass. The slope met ADA requirements, with five foot wide landings at 30 foot intervals. Space constraints on the west of Foothill required stairs; these were supplemented with an elevator. The elevator interior is a 12-foot by 12-foot box, with the total structure measuring 14-feet by 14-feet; the large size is intended to accommodate bicycles pulling strollers.

The elevator not only enables disabled persons to use the bridge, but cyclists, strollers, and fatigued persons can also use the elevator in lieu of the stairs. This accessibility provides for all people to use the overpass, regardless of age or ability.

The scenic nature of the overpass was also a significant criteria; the client wanted the overpass to enhance the view of the area, and perhaps tie to the local community heritage. The George S. Eccles 2002 Legacy Bridge served as a guide for aesthetic enhancement of the community. Therefore, a tied arch look was favored over the conventional girder bridge. The specific process of alternative development and selection is outlined in section 4.

The overpass is also meant to serve as a connection between the sunnyside area and the wasatch trail system. UDOT plans to design a trail along Red Butte Creek to provide residents of the area access to the Bonneville Shoreline Trail and the overpass will help serve this goal. With the Red Butte Creek in mind, impacts to the riparian corridor need to be minimized.

Criteria of space constraints played a role as an obstacle. The project crosses over a UDOT-owned highway; furthermore, the crossing lands on VA and University of Utah property. Fort Douglas of the United States Army was located just north of the project area. Sensitivity to each entity’s concerns and requirements served as a guide for the layout and type of structures to be implemented.

The University of Utah indicated their approval of construction on their property; the VA, however, was not comfortable in removing parking spaces in their lot. The original design was to use a ramp on the west-side (on va prop.), but switched to stairs etc.

3.2 Basis of Design

The basis of design for this project is dependent on multiple elements, discussed below. These elements include project needs/desires and the ISI Envision analysis.

The basis of design includes must-have and desirable elements. Must-have elements are those that are required and absolutely necessary for the project to function appropriately. Desirable elements are those that are secondary to the project, but are convenient if included.

A must-have element includes an overpass over Foothill Drive that allows pedestrians to safely cross without disrupting traffic or risking their lives. The main purpose of the project is to provide a safer way to cross Foothill, and therefore, it is imperative that this crossing be safe for all pedestrians while allowing traffic to continue underneath.

Another must-have element includes minimal encroachment on surrounding properties. The project is unique in that there are multiple property owners nearby with differing opinions and interests in the project. These include UDOT, the University of Utah, and the United States Military. It is essential that the crossing minimize its invasion on these properties and only use space necessary to accomplish the main goal of moving pedestrians across Foothill.

However, the workable space for the crossing presents unique challenges due to site constraints from the creek, powerlines, and more.

A must-have element includes ease of construction due to the busy nature of Foothill Drive. The construction should not disturb traffic for long periods of time, especially because of the low level of service Foothill is currently at. The construction must be relatively quiet due to close proximity to businesses and neighborhoods, and should not disrupt the UDOT right-of-way; this is due to the UDOT bond that will be needed for construction.

Furthermore, minimal impact to Red Butte Creek and the Riparian Corridors is a must-have element. The health of the creek and Riparian Corridors is overseen by outside authorities, including the Salt Lake Riparian Corridor Overlay District (RCO) and United States Army Corps of Engineers (USACE). These organizations protect the stream and are likely to grant or deny permission for construction near these areas. For the project to be feasible and completable, the project must avoid these areas; otherwise, building permits may not be granted and the project will stagnate.

It is also critical that the right of way is preserved; this includes maintaining the current roadway width and volume. Foothill Drive is at level of service F, which is unfavorable; any encroachment upon the right of way may further reduce traffic volume and cause the level of service to worsen.

Desirable elements of the project include low maintenance. For example, it is desirable to prevent corrosion of the steel elements if a steel truss is used; however, it may be necessary for frequent applications of anti-corrosion coatings. Other maintenance includes lighting along the overpass, draining and runoff, and more. Another trait is the beauty and aesthetics of the finished bridge product. The aesthetics of the bridge can provide a connection to the historic heritage of the local community, and may serve as a landmark for the area.

Furthermore, it is desirable, but not critical, for the project to directly connect the trails system from the Sunnyside Community up to the Bonneville Trailhead. This direct connection would provide convenience for trail users and may encourage recreation among community members. This would also fulfill Salt Lake City's vision of a joint trail network in the area, according to Lynn Jacobs. Minimal cost is also a consideration in the project, but is not a hindrance. It may be advantageous to add extra elements beyond the bare minimum as an investment. For example, designing the crossing to connect to future TRAX stations may be a worthwhile cost. The probability of the olympic games returning to Salt Lake City may encourage the crossing to include extra elements, and thus added costs.

The ISI Envision Analysis is a tool for infrastructure projects to review its overall sustainability. This tool consists of a series of questions related to specific sustainability categories, each of which require a score. The scoring ranges from 0 to 2; 0 meaning not applicable, 1 representing a basic opportunity and 2 meaning an ability to go above and beyond for little cost. The categories are: Quality of Life, Leadership, Resource Allocation, Natural World and Climate, and Resilience.

The ISI Envision Analysis was performed for the Red Butte Creek Pedestrian Crossing; the details of the analysis are included in the appendix. The specific sustainability of each category was investigated, and the resulting score for each category was determined. As discussed above, the investigation considered not only environmental sustainability, but also social sustainability and equity. With these in mind, the scores were calculated and compared. For areas of uncertainty, the scores were estimated.

It is important to note that the potential future TRAX station in Research Park is an important driver for sustainability in the “leadership” category. The overpass will connect the Sunnyside community to Research Park, and can stimulate economic growth if companies rent space in Research Park in which their employees can access via public transportation.

The highest ranking categories included Quality of Life and Natural world. This result is intuitive based on the nature of the project; it is focused on providing a new crossing that connects to the existing trail network and aims to preserve the surrounding environment. This project has the ability to provide infrastructure that will allow for a safer and more comfortable crossing. The crossing can also be integrated into the natural landscape to be used by recreational, student and commuting users. Additionally, if Sunnyside residents utilize the crossing, they will use their vehicles less, leading to cleaner air.

Using the ISI Envision worksheet in Appendix A2, the total ISI points divided by the available points totals at 62.5%. This is considered as platinum grade, and means the project would be very effective at enhancing the overall sustainability in the community.

3.3 Stakeholder Prioritization

The stakeholders are ranked by priority: first by their level of influence, followed by urgency and then by attitude. It’s important to acknowledge the actively-opposing people of power first to address their concerns for the completion of the project and then work down the priority list. The list of stakeholders is shown in Table 3.3.1 below.

Table 3.3.1 - Prioritization of Stakeholders

Stakeholder	Assessment	Key Concerns	Priority
VA Salt Lake City Regional Office	Influential Active Blocker	Doesn’t want it cut through their parking lot Private Property	1
US Army Corp of Engineering	Influential Passive Blocker	Minimize impact to stream banks and channel	2
Utah State Engineer's Office	Influential Passive Blocker	Minimize impact to stream banks and channel	2
Utah Department of Public Safety	Influential Passive Blocker	Doesn’t conflict with highway safety and management	2

Salt Lake City Ordinance Riparian Corridor Overlay District (RCO)	Insignificant Active Blocker	Preserve and Protect Riparian Corridors	3
Salt Lake County Engineering and Flood Control	Insignificant Active Blocker	No impact within 20 feet of top of channel bank or flood control facility	3
Salt Lake County Watershed Planning and Restoration	Insignificant Active Blocker	Zero net impact to downstream systems and facilities	3
UDOT	Influential Active Backer	No at-grade pedestrian crossing due to proximity of adjacent intersections (Mario Capecchi and Wakara Way) No change in street width and volume Minimal disruption to traffic during construction Preserve R.o.W.	4
Sunnyside Neighborhood	Influential Active Backer	Direct connection to future trail Direct passage to other side of Foothill Drive No unnecessary construction noise or interruptions No impact on street parking and passage No construction debris	6
Salt Lake City Engineering	Insignificant Active Backer	Integrated usage and purpose Improved equity, access, and inclusion Low maintenance Low construction Minimal invasion to existing property Safety No impact to off-site city infrastructure Minimize impacts to storm drainage and conveyance systems Minimize impacts to utilities Preserving and protecting Riparian Corridors Connect to historic heritage Promote sustainable design, construction, and usage	7

Salt Lake City Trails	Insignificant Active Backer	Pedestrians and cyclists of all ability levels Minimize unnecessary elevation changes Connect to future trail points Width necessary to accommodate two-way passage Cross-slopes and centerline profiles that accommodate all mobility levels (compliance with ADA) Minimal cost Possible natural walking trail path on one side of Red Butte Creek and northeast side of Foothill Drive	7

A portion of the stakeholders can also be presented graphically which is shown in Figure 3.3.1. The parcel information of each segment shows the location and owner of that area which impacts the process of the project. As seen in Figure xxxx, the east side of Foothill Drive between Mario Capecchi and Wakara Way is split into two owners, the United States of America and the University of Utah. Developing the crossing on the University of Utah property will be easiest for permitting and construction.

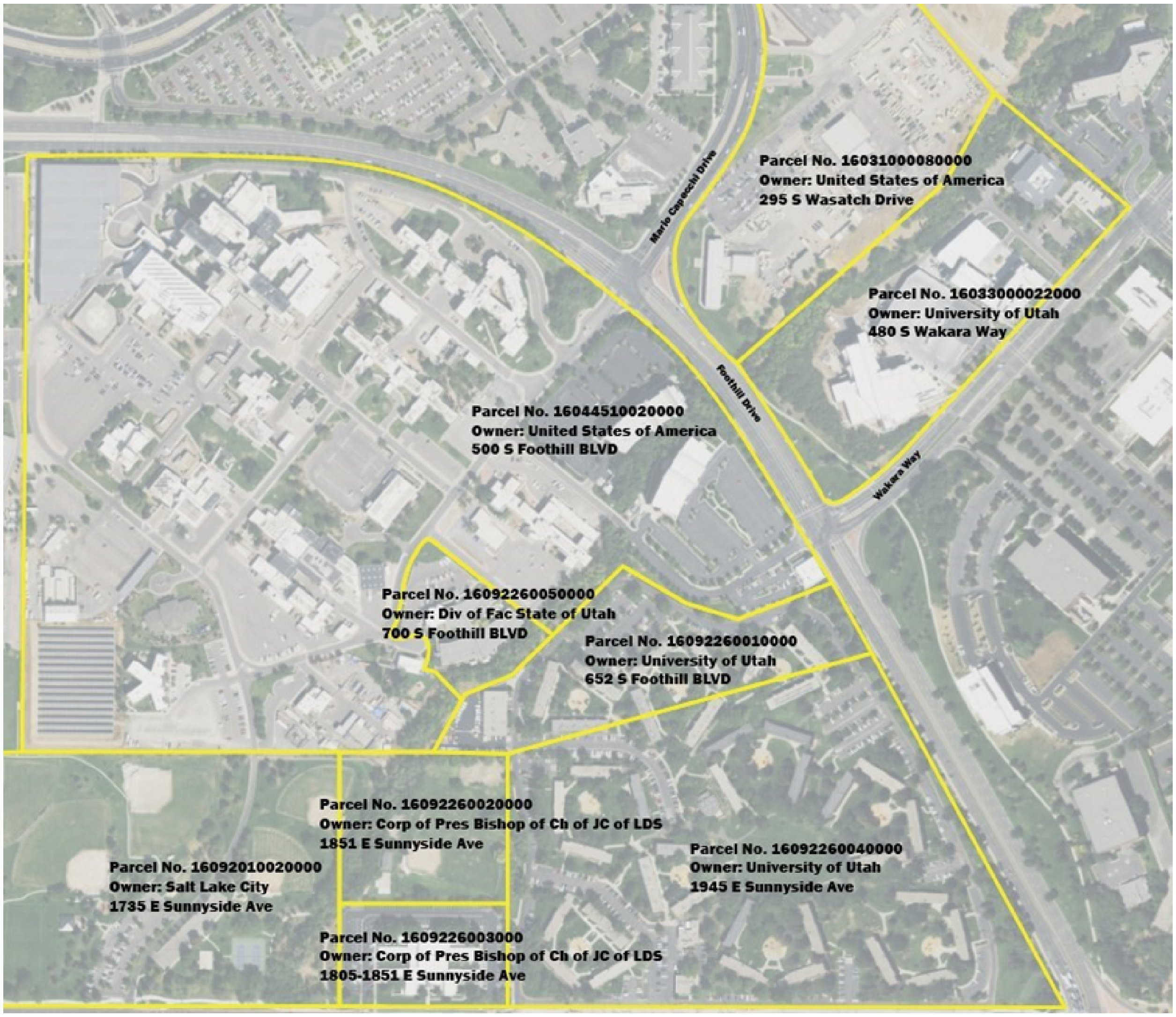


Figure 3.3.1 - Parcel information surrounding the site [10].

3.4 Decision Criteria

Decisions for the design were based on a number of factors that can be categorized into 3 categories: site constraints, accessibility and Salt Lake City feedback. Each of these categories were explored during the initial development, as well as throughout the design process.

Numerous site constraints are present within the project area which include: Federally Owned Land, Red Butte Creek Riparian Corridor and existing overhead and underground utilities. In order to develop a design that balanced impacts with each of these constraints, the design team opted to minimize the footprint of the structure. This required an elevator and staircase transition from the structure to the future trail/existing sidewalk on the West side in lieu of a large ADA compliant ramp system that was initially proposed.

The proximity to Red Butte Creek, the riparian corridor and existing utilities was a major factor in placement of the structure and connections. The project team made the decision early in the design to minimize impacts to the existing environment near the creek. This resulted in the decision to push the structure to the edge of the riparian corridor. However, this placement did result in conflicts with overhead and underground utilities. To resolve those conflicts, the project team reviewed the site for potential utility relocation and came to the conclusion that there is ample room for their relocation and that it is worth the cost to create this separation between the utilities and the structure and connections.

Accessibility was another large decision criteria that aided in the design. This stems from Salt Lake City's desire to ensure the crossing and future trail is accessible by all users. The result of prioritizing accessibility in the design was multiple ramp configurations, all of which would be utilized by different types of users. It also resulted in a wide trail/structure cross section to allow for comfortable two direction usability that accommodates many user types.

Feedback from Salt Lake City was also crucial for design decisions. Their feedback ultimately shaped the structure design from the initial proposal of a spiral ramp configuration, to a switchback ramp configuration to the final curved east side connection and stair/elevator combination on the west side. Other crucial features such as bike rails for the stairs, as well as prioritized connections to the future trail instead of existing facilities, were a direct result from design reviews by Salt Lake City.

3.5 Design Criteria

Because the bridge is located over a highway, AASHTO LRFD load combinations were used for structural analysis of the overpass. Analysis for non-overpass structures (ramps, columns, etc) utilized ASCE LRFD load combinations, found in ASCE 7-16.

Additional guidance related to specific materials was found in ACI 318 *Building Code Requirements for Structural Concrete* and the *AISC Steel Construction Manual* 15th edition for concrete and steel, respectively.

Slope requirements for the ramps used were governed by standards outlined in the 2010 *ADA Standards for Accessible Design*.

The overpass is designed to appear as a tied arch structure with two ribs spanning the length of the bridge; these ribs are purely architectural in purpose and the entire system is supported with a prefabricated truss bridge. The design criteria for the structure is thus dependent on truss bridge criteria only. However, basic tied arch criteria was used in the aesthetic design of the arches.

The structure is created to be at least 25 feet away from the annual high water line of Red Butte Creek to limit impacts to the riparian corridor and avoid the flood plain.

4. ALTERNATIVE DEVELOPMENT, ANALYSIS, AND SELECTION

The development, analysis, and selection of alternatives was detailed in P10 Universe of Alternatives for Group 4. That work is summarized below [11]. Development of alternatives juggled between aesthetics, practicality, and budget. Several alternatives were brought forth.

Within each alternative, different layouts were discussed; these included a direct tie-in to the VA buildings, as shown in Figure 4.1, a two-span, forked bridge shown in Figure 4.2, and the traditional single-span bridge layout as shown in Figure 4.3.

Figure 4.1 - Plan View of Overpass Connection to VA Structure: A ramp and stairs are shown on the east side while the west side depicts a direct connection to the VA building, with indoor elevator/stairs for 24/7 access [11].

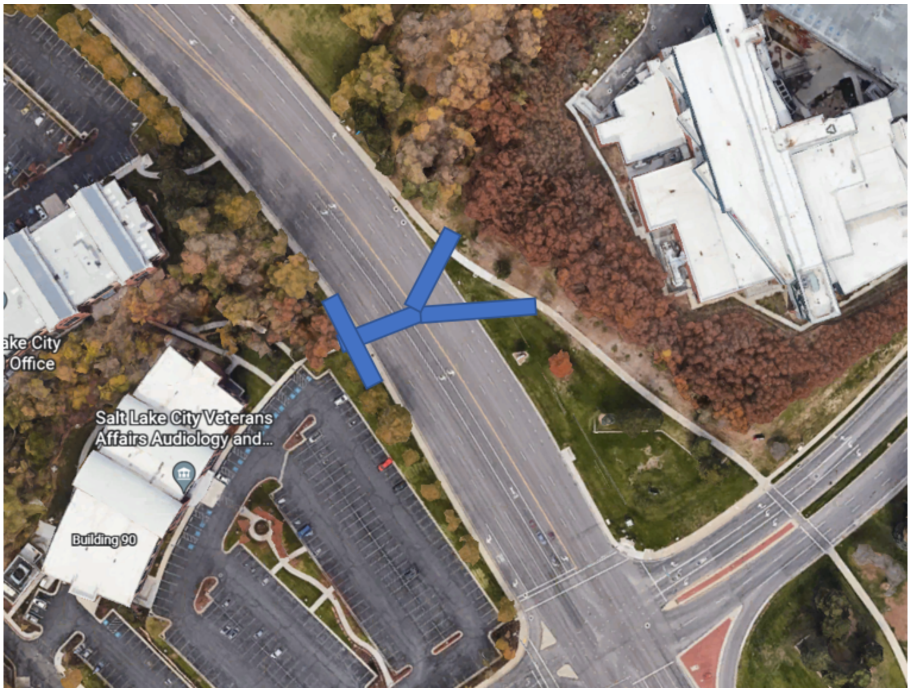


Figure 4.2 - Plan View of Forked Bridge: the west side includes a ramp with north and south approaches. The truss bridge forks out on the east side to meet the trail connection and the sidewalk for transit access. A bridge pier will land on the inner left turn lane in the road [11].

Figure 4.3 - Plan View of Single Span Bridge with Elevator and Stairs: Stairs are shown in blue while the elevators are in yellow [11].

For the specific bridge type, the alternatives considered included a prefabricated truss bridge, girder-style bridge, cable-stayed bridge, suspension bridge, and tied-arch bridge. Other concepts briefly investigated included a buried bridge overpass, a roll-out bridge, and even an aqueduct style bridge. Each alternative was viewed through the lens of several factors, as discussed below.

Factors in the analysis process for alternative selection included cost, given that the overall cost would likely determine the feasibility of the project. While cost was a significant driver, the uniqueness and “legacy” of the project was also considered. The Eccles 2002 bridge on Mario Cappechi served as an example of making an overpass into a community icon. Thus, aesthetics and the local heritage were investigated to find an overpass that would add to the beauty of the site, while simultaneously fitting into the landscape.

Another factor in the analysis included constructability. Closure of Foothill was negatively received by the client, and rapid construction was a priority for the project. This included the concept of accelerated bridge construction (ABC). The prefabricated truss and girder bridges were viewed as most compatible with the ABC erecting method.

Available space was also examined. The VA does not want to give up parking spaces in their lot, and thus the parking strip on the west side of Foothill in Figure 4.1 is the only place for the landing of the ramp or stairs. This space constraint was a large factor in determining which bridge type is appropriate.

Each of the above factors were discussed in choosing the final bridge type. However, the bridge type was primarily controlled by the aesthetic ability of the overpass to become a symbol for the local community. Secondary (but important) considerations were cost and construction. Potential alternatives were narrowed down, beginning at the layouts, and subsequently selecting a bridge type.

Several bridge layouts were eliminated early due to construction and feasibility. It was decided the VA would likely not approve of a direct public connection to their building; the forked bridge was also removed because the middle pier would occupy a much-needed turning lane for Foothill traffic. Thus, a traditional straight, single-span bridge layout was chosen.

For the specific bridge types, the selection also considered feedback from the client. Due to lengthy closures for construction, the buried bridge was eliminated; the roll-out bridge was also eliminated due to its cost and negative feedback from the client.

The cable-stayed and suspension bridges were dropped due to their cost, lengthy construction period, and large space the pylons and ramps appeared to occupy. While the girder bridge was a cheap option, the client desired a more visually-pleasing style; a tied arch bridge was therefore selected.

In discussing how to connect the crossing to the current grade, multiple preliminary concepts were investigated, including spiral ramps, elevator/stair combinations, and MSE supported walls. Ultimately, cost and space restraints resulted in a switchback staircase with an elevator for the western end and a ramp supported by a system of columns and MSE walls for the west end.

5. DESIGN DEVELOPMENT SUMMARY

5.1 Process

This subsection will describe the process in how the design was selected, formed, and created for the final design submission. The design outlined in this report was created and modified by Team 4 with mentorship from practicing engineers as well as the mentor team. The prefabricated pedestrian bridge with a tied arch feature was Salt Lake City's preferred design alternative of the three alternatives that were proposed to Salt Lake City Engineers. Once the tied arch prefabricated bridge was selected, the project team contacted Contech for a preliminary design of a prefabricated bridge and started designing the ramps, elevator structure, tied arch feature, and many other additives to the design of the overall project. A complicated portion of the design was ensuring that the ramp has proper ADA compliance for proper design and construction of the pedestrian crossing for all users. Structural calculations according to the ASC code in steel and concrete bridge design, environmental concerns and permits were researched for proper environmental designs near and on the project area. Geotechnical data was taken and carefully looked at for proper geotechnical footing design for the structure. Aesthetic features were also added to the design according to the requests from Salt Lake City Engineering. Safety features were also added for all users of the pedestrian crossing.

5.2 Design Data and Specification Summary

Design data and specifications for the project encompasses multiple disciplines, including structural, hydrological, site, and transportation. The basis of modeling, including the assumptions, limitations, and inputs are outlined for each discipline below.

The project structure was broken into two parts - those controlled by AASHTO LRFD and ASCE 7-16 load combinations. The bridge structure itself is controlled by AASHTO load combinations, and therefore the arch rib and bridge abutments utilized those combinations; the truss bridge is designed by ContechES and was not covered in this modeling. The truss bridge is designed to carry all loading; the arch ribs are purely aesthetical, and were only designed to withstand their self-weight.

The components controlled by ASCE 7-16 load combinations, namely the ramps and columns, used LRFD load factors and combinations.

A major constraint was the limited knowledge of seismic behavior of structures and foundations. The dynamic behavior of the foundations and structure under seismic and wind loading was not considered; simplified equivalent static loads were used in place of sophisticated analysis methods. A computer model of the system was not created; instead, hand calculations provided a satisfactory model for this level of design. Before construction documents are created, it is recommended to create an accurate computer model to ensure the designed components are satisfactory for the project; this includes the verification of component constructability and economy.

Each component was analyzed with each respective loading. As an assumption, live loading was taken as 100 psf everywhere. The analyzed components are labeled in Figure 5.2.1 for quick reference. The governing load values using the appropriate combinations is given in Table 5.2.1 below. Downwards is taken as positive for forces.

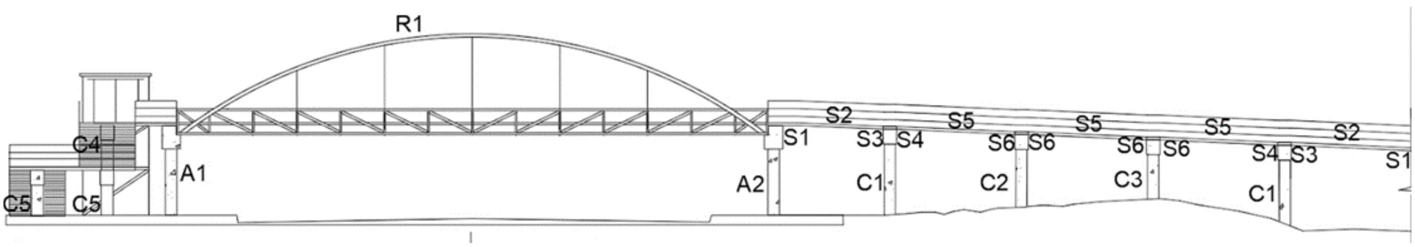


Figure 5.2.1 - Reference for Structural Load Schedules

Table 5.2.1 - Load Schedule for Bridge Abutments, Ribs, and Columns

LOAD SCHEDULE												
	Force (k)						Moment (k-ft)					
Criteria	AASHTO LRFD											
Type	Dead	Live	Snow	Wind	Seismic	Governing	Dead	Live	Snow	Wind	Seismic	Governing
A1	74.0	93.0	24.7	0.0	-15.0	284.0	73.5	138.0	36.7	2.5	77.5	475.0
A2	83.0	91.0	26.3	0.0	-17.0	307.0	54.6	126.0	33.5	2.5	77.5	425.0
R2	7.1	0.0	0.0	0.0	-1.4	8.6	0.0	0.0	0.0	0.0	0.0	0.0
Criteria	ASCE 7-16 LRFD											
Type	Dead	Live	Snow	Wind	Seismic	Governing	Dead	Live	Snow	Wind	Seismic	Governing
C1	70.8	36.0	9.6	0.0	-14.5	147.4	0.0	0.0	0.0	2.5	77.5	77.5
C2	67.2	36.0	9.6	0.0	-13.7	143.0	0.0	0.0	0.0	1.4	43.6	43.6
C3	63.7	36.0	9.6	0.0	-13.0	138.8	0.0	0.0	0.0	0.6	19.4	19.4
C4	51.8	22.4	6.0	0.0	-10.6	101.0	80.9	44.6	11.9	2.5	94.0	193.5
C5	52.0	26.6	7.1	0.0	-10.6	108.5	80.9	44.6	11.9	0.8	39.9	174.4

As discussed previously, ACI 318 section 6.5 was used to approximate the loads over the continuous slab. It is assumed to be a one-way slab; slab deflections are not needed according to section 6.5 because the slab meets thickness requirements. Table 5.2.2 shows the loads for each mark on the slab. For moments, the American sign convention is used (negative moments depict a “hogging” curvature of the slab).

Table 5.2.2 - ACI 318 Section 6.5 Slab Loads

Mark	Moment (k-ft)	Shear (k)
S1	-15.9	4.7
S2	18.1	0.7
S3	-25.4	5.4
S4	-23.1	4.7
S5	15.9	0.5
S6	-23.1	4.7

The load results were used to design the different structural components, including sizing of members, proportioning of reinforcement, and so forth.

Hydrological and stormwater data for the Red Butte Creek was analyzed in StreamStats. The creek's interception with Foothill drive was chosen as the point of analysis. Then a two hour storm with different occurrence frequencies was delineated in order to determine the maximum creek flows in cubic feet per second. The determined values are given in Table 5.2.3 below.

Table 5.2.3 - Hydrological and Stormwater Analysis

Average Return Period (Years)	Occurrence Frequency In Given Year	Intensity (ft^3/s)	Storm Duration
2	50%	41.5	2 Hours
10	10%	94.8	2 Hours
100	1%	172	2 Hours
500	0.2%	239	2 Hours

*Source StreamStats [12]

Table 5.2.4 lists the measured traffic volume for the project site, as sourced from a 2019 study conducted by UDOT. Due to the continued growth of population in the area, the current numbers may be slightly higher, though these values describe the approximate volume with which to be considered.

Table 5.2.4 - Traffic Information

Roadway	AADT*	Collection Year
Foothill Drive	51,000	2019
Mario Capecchi	24,000	2019
Wakara Way	17,000	2019

*Source: UDOT AADT Google Earth KMZ [13]

The total ramp length for the east side ramp was calculated using the height above existing grade, design slope and number of 5 foot flat landings as follows:

$$Ramp\ Total\ Length\ (ft) = \frac{Height\ above\ grade\ (ft)}{ramp\ slope} + (number\ of\ landings\ x\ 5ft)$$

Table 5.2.5 - Ramp Length for East Ramp

Height above grade:	20 ft
Design Slope:	8%
Ramp Length = Height above grade / ramp slope	250 ft
Number of 5 ft landings:	9
Ramp total length:	295 ft

5.3 Operations and Maintenance Summary

The current bridge design does not need much maintenance throughout the lifetime of the bridge; the only items that will possibly need maintenance are the elevators, snow removal, and paint due to rusting. Elevators require annual inspections and also a series of maintenance checks to maintain optimal performance. A structural inspection will be required every 2 years during the lifetime of the bridge to ensure optimal performance and safety. During the winter months the pedestrian bridge and stairs will need snow removal for optimum safety using the pedestrian crossing. The bridge will require paint between 15-25 years of the lifetime of the project.

5.4 Construction Needs and Phasing Summary

This subsection will describe what will need to be constructed, though the phasing plan for how the construction will take place will be up to the general contractor that wins the bid for the project. Some notes that will be added will be about the different traffic conditions and potential traffic control ideas for the project for more workable installation. The construction of the pedestrian bridge will first require the construction of the columns and ramps on the east and west side of the project. Once the columns are constructed, the bridge is able to be shipped to the project and installed in one evening over Foothill Blvd. One of the challenges with the pedestrian crossing over Foothill Blvd is traffic control. Foothill Blvd is a heavily trafficked roadway, exceeding an average of 50,000 vehicles per day.

One of the proposed Detour routes is shown in Figure 5.4.1; the blue line shows the detour route which requires northbound motorists to take a left on Wakara Way, going around the VA, and getting back on Foothill Boulevard at Mario Capecchi Drive. This proposed detour will require the permission from the VA. This route could also be used exclusively for emergency services if UDOT, Salt Lake City and the emergency responders were concerned with other proposed detour routes.

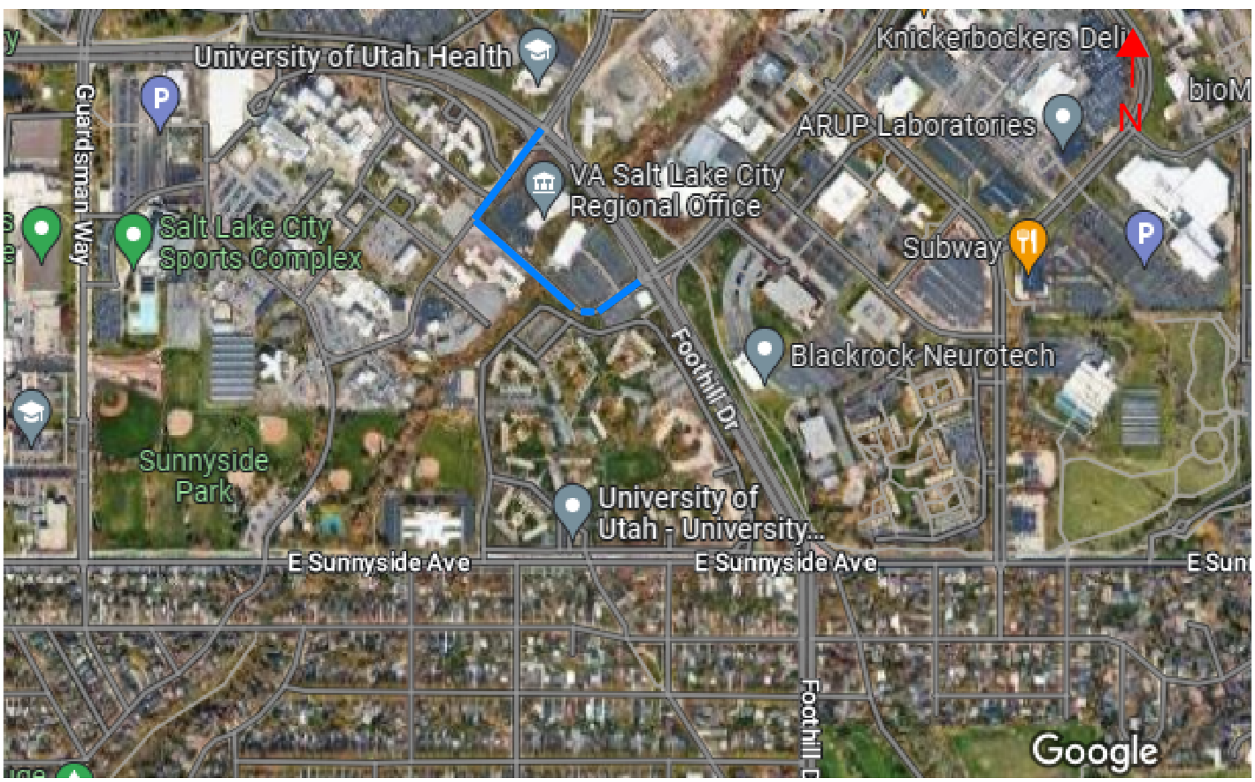


Figure 5.4.1 - Proposed detour route through VA area [14]

The other proposed detour route shown in Figure 5.4.2; the red line shows the detour route which requires northbound motorists to make a left turn onto Sunnyside Avenue, then a right turn on Guardsman Way, and then a right onto Foothill Drive. This detour route would need to be coordinated with UDOT and Salt Lake City Engineering.

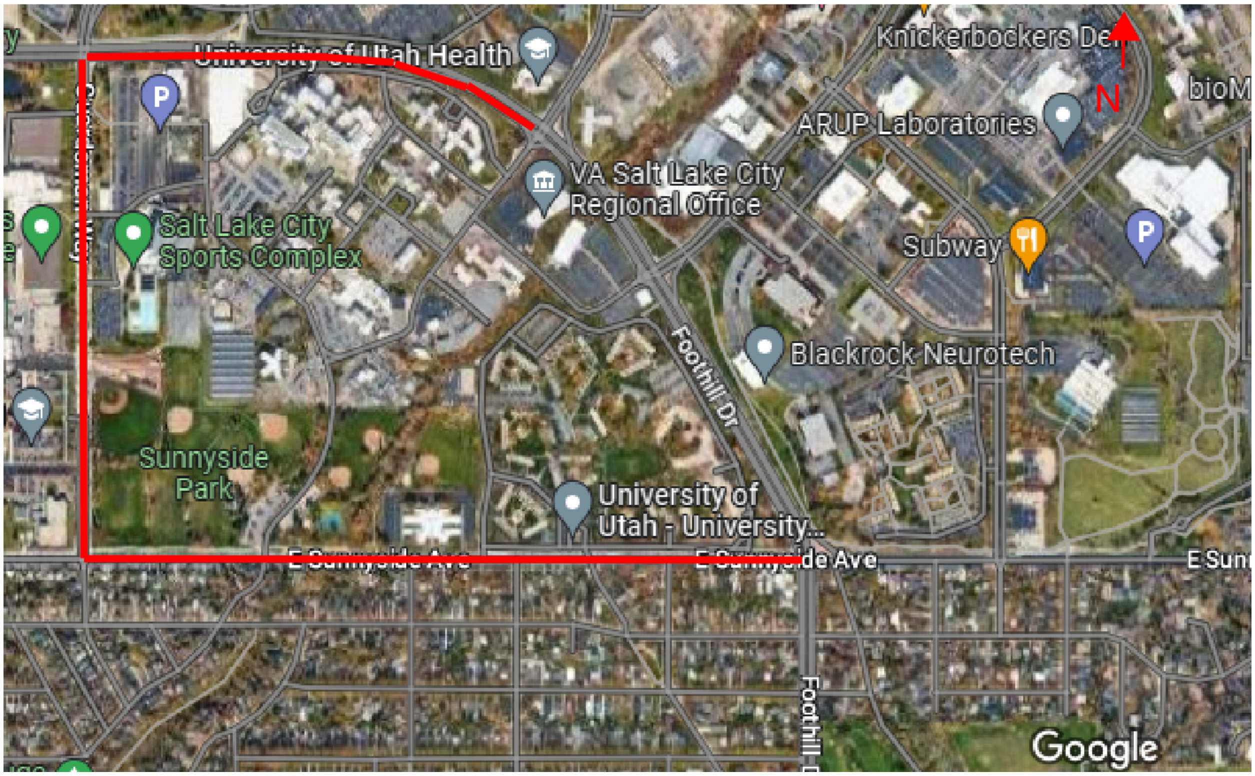


Figure 5.4.2 - Proposed detour route onto Sunnyside Ave and Guardsman Way [14]

6. DESIGN SUMMARY EFFECTIVENESS

The goal for the Red Butte Creek Pedestrian Crossing was to design a grade separated crossing over Foothill Drive which was accessible by all users. This design needed to account for many user types including bicyclists, bicyclists with trailers, wheelchairs as well as pedestrians. Impacts to Red Butte Creek, adjacent landowners and Foothill Drive were all considered and balanced during the design process. In order for an effective design to be achieved, the project team focused on three factors: location, access and safety.

6.1 Location

Placement of the crossing, as well as connections with the future trail and other existing facilities, played a vital role in the design. The crossing was situated close to the creek while still ensuring impacts to the riparian corridor were minimized. This location provides for the least amount of out-of-direction travel for users who are wanting to cross Foothill Drive and continue on the Red Butte Creek trail down to the Sunnyside Community. Also, the inclusion of close connections to the existing sidewalks allows pedestrians in the area to use the crossing as a safe alternative to crossing Foothill at the signalized intersections. This has the potential to reduce congestion due to long pedestrian crossing times at the already busy signalized intersections.

6.2 Access

The Red Butte Creek Pedestrian crossing was designed to be accessible by all users. Multiple elements were incorporated in order for this to be achieved. On the east side of the crossing, an ADA compliant ramp with an 8% slope and level landings at 30 foot intervals ensures all users have direct access to the trail. Also, an elevator and stair design was provided on the east side as optional features to increase connectivity with the existing pedestrian facilities; an included bike rail provides more access for bicyclists. The reason it wasn't proposed as part of the base design was to minimize cost while still meeting the primary project goal of a trail crossing.

The west side of the crossing features a large traction elevator and switchback stair design which incorporates bike rails. This allows users to choose which method best suits their needs in order to transition to and from the crossing. The switchback design allows for a direct connection to the existing sidewalk that runs along Foothill Drive. It is the project team's assumption that the future trail will incorporate these existing facilities to access the trail on the west side of Foothill Drive.

6.3 Safety

The crossing design enhances safety by removing the conflict between trail users and motorists on Foothill Drive, which exceeds 50,000 vehicles on average each day. Additional features that follow Crime Prevention Through Environmental Design (CPTED) principles such as fencing, lighting and security cameras were also included to ensure safety and comfort for the users. The overpass was also designed with a 12 foot trail width to provide safe access for pedestrians and bicyclists going both directions.

7. COST ESTIMATE

The accompanying cost estimate for the preliminary design of the proposed overpass located on Foothill Drive near Wakara Way has been assembled for the purpose of estimating the feasibility of the project. This cost estimate is based on a cost per unit system and encompasses each aspect of construction with respect to uncertainties and contingencies.

The current cost estimate utilizes prior construction knowledge from Group 4 members in addition to an existing UVU pedestrian crossing over I-15. An engineering cost estimate for this case study was retrieved from the Utah Department of Transportation website. The comparative nature of both the existing I-15 crossing and the proposed Foothill crossing make this analysis suitable to be included in the cost estimate. Furthermore, the pedestrian bridge structure item in the current cost estimate was given by Contech Engineered Solutions for a pre-fabricated truss style bridge.

The current quantity values in the cost estimate are approximated due to uncertainties in the exact project details. The current estimate is a preliminary design cost estimate for the project and may need refining once more details for the project emerge. However, this estimate adequately encompasses the design features outlined in the drawings. These uncertainties in exact project details are covered via multiple design options with respective cost estimates.

As detailed in the Construction Estimate, the addition of a staircase and elevator combination for a sidewalk connection on the east side would have an estimated cost of approximately $850,000; this increase in the total project cost is presented for the purpose of informing Salt Lake City of the additional connection options along with the pedestrian ramp.

The current cost estimate combined total for all bid items with a 30% contingency is $3,435,321.67. This is the preliminary estimate for the current design and approximates the final completion of the project. It is important to note some costs may vary within the project based on environmental factors, inflation, and projected start date of the project. Furthermore, any changes within the scope of work for the project will change the cost of the project. With the location of this project being near the VA building, University of Utah, Fort Douglas, and over a UDOT right of way, discussions will need to take place for possible land acquisition which could further add to the estimated cost and timeline of the project.

Appendices

A.1 Calculations

STRUCTURAL CALCULATIONS

Calculations reviewed by Braden Miles

Loading:

Live: $L = 100\,psf$

Dead: $D = 75\,psf$ (This is the initial guess of dead load)

Snow: $p_g = 38\,psf \quad C_e = 1 \quad C_t = 1 \quad I_p = 1$

$$S = 0.7 \cdot C_e \cdot C_t \cdot I_p \cdot p_g \rightarrow 27\,psf$$

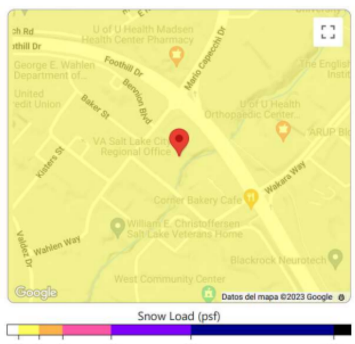
2/10/23, 7:33 PM Utah Snow Load | USU

Utah Ground Snow Load Map

Latitude: 40.756
Longitude: -111.836
Elevation: 4,753 ft

Ground Snow Load:
38 psf / 1.80 kPa

*This document is not legally binding. The user is urged to verify ground snow load values with the local authority having jurisdiction.



Wind: $p_{wind} = 0.00256 \cdot \left(109^2\right) \cdot psf \rightarrow 30.4\,psf$

ATC Hazards by Location

Search Information

Coordinates:	40.75623861411172, -111.83468985564105
Elevation:	4773 ft
Timestamp:	2023-03-25T15:24:09.855Z
Hazard Type:	Wind

ASCE 7-16

MRI 10-Year	74 mph
MRI 25-Year	79 mph
MRI 50-Year	84 mph
MRI 100-Year	89 mph
Risk Category I	97 mph
Risk Category II	103 mph
Risk Category III	109 mph
Risk Category IV	113 mph

ASCE 7-10

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

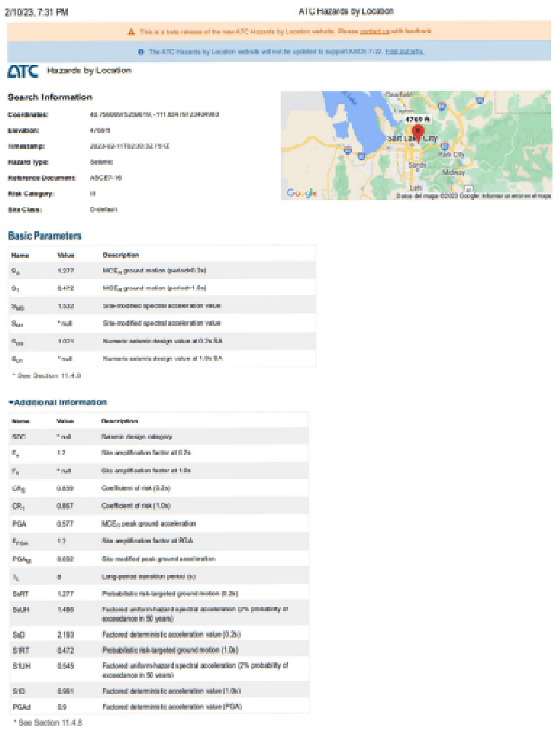
ASCE 7-05

ASCE 7-05 Wind Speed	90 mph

Category III is utilized.

Seismic: $S_{DS} = 1.021$

$I = 1.0$


2/10/23, 7:31 PM ATC Hazards by Location

This is a beta release of the new ATC Hazards by Location website. Please contact us with feedback.

The ATC Hazards by Location website will not be updated to support ASCE 7-22. Find out why.

ATC Hazards by Location

Search Information

Coordinates:	[illegible]
Elevation:	[illegible]
Timestamp:	[illegible]
Hazard Type:	Seismic
Reference Document:	ASCE7-16
Risk Category:	III
Site Class:	D-default

Basic Parameters

Name	Value	Description
S_S	1.277	MCE_R ground motion (period=0.2s)
S_1	0.472	MCE_R ground motion (period=1.0s)
S_{MS}	1.532	Site-modified spectral acceleration value
S_{M1}	* null	Site-modified spectral acceleration value
S_{DS}	1.021	Numeric seismic design value at 0.2s SA
S_{D1}	* null	Numeric seismic design value at 1.0s SA

* See Section 11.4.8

Additional Information

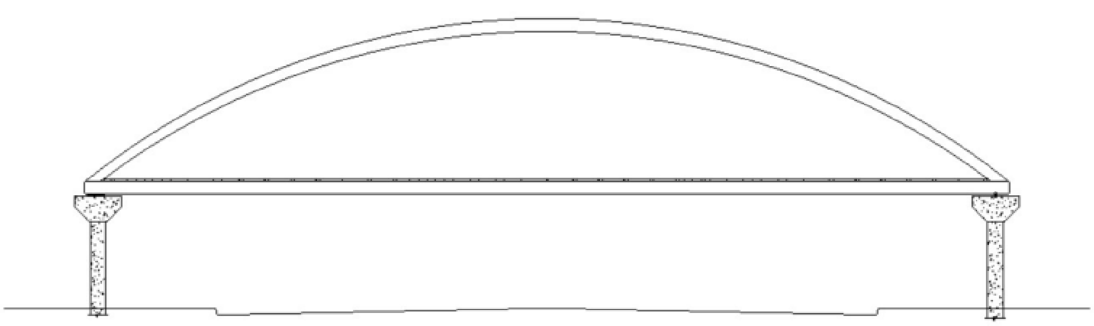
Name	Value	Description
SDC	* null	Seismic design category
F_a	1.2	Site amplification factor at 0.2s
F_v	* null	Site amplification factor at 1.0s
CR_S	0.899	Coefficient of risk (0.2s)
CR_1	0.887	Coefficient of risk (1.0s)
PGA	0.577	MCE_G peak ground acceleration
F_{PGA}	1.2	Site amplification factor at PGA
PGA_M	0.692	Site modified peak ground acceleration
T_L	8	Long-period transition period (s)
SsRT	1.277	Probabilistic risk-targeted ground motion (0.2s)
SsUH	1.486	Factored uniform-hazard spectral acceleration (2% probability of exceedance in 50 years)
SsD	2.183	Factored deterministic acceleration value (0.2s)
S1RT	0.472	Probabilistic risk-targeted ground motion (1.0s)
S1UH	0.545	Factored uniform-hazard spectral acceleration (2% probability of exceedance in 50 years)
S1D	0.981	Factored deterministic acceleration value (1.0s)
PGAd	0.9	Factored deterministic acceleration value (PGA)

* See Section 11.4.8

Category III is utilized.

ARCH RIB DESIGN.

Shown is a general schematic of the arch (not including the truss component or risers).



The arch is designed using a parabolic function; parabolic is selected over catenary function forms because of the designer's familiarity with parabolic functions. No load will be placed on the ribs; the ribs must be designed to support only self-weight.

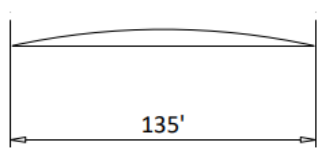
Per guidance from the instructional team and mentors, a 1:5 ratio of height to span is used for the bridge; therefore, with a span of 135 feet, the total height of the structure is 27 ft.

$$span = 135\,ft \qquad ratio = \frac{1}{5} \qquad height = ratio \cdot span \rightarrow 27\,ft$$

An allowance of 3.5 feet on top of the mandatory 18.5 feet of clearance to the bridge deck is used. With Contech's design, this proved beneficial. The 3.5 feet is the approximate length from the bottom of truss to the deck; on top of this will be the arch. The height of the arch is then found:

$$allowance = 3.5\,ft$$

$$archHeight = height - allowance \rightarrow 23.5\,ft$$



$$a = \frac{-archHeight}{\left(\frac{span}{2}\right)^2} \rightarrow -0.0052\,ft^{-1}$$

$$ArchEqn = ax^2 + archHeight$$

$$ArchEqnDeriv = 2 \cdot a \cdot x$$

To find the required length of HSS arch ribs, the length of the arch is calculated using integral calculus.

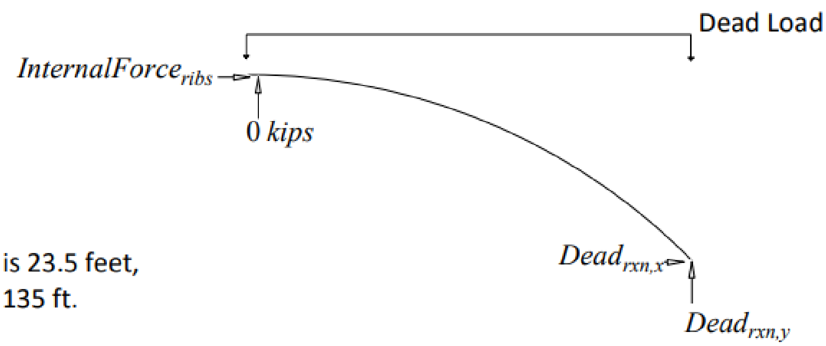
Limits of integration range from -67.5' to 67.5'

The integral yields a length of 145.4 ft.

$$\int_{-67.5'}^{67.5'} \sqrt{(ArchEqnDeriv)^2 + 1}\, dx$$

$$Length = 145.4\,ft$$

Approximated as a 3 hinge arch according to Hansen Structural Analysis text



The height of the arch is 23.5 feet, while the total span is 135 ft.

The width is 12 ft.

Using an initial estimated dead load of 20 plf

$$\omega_{ribsD} = 20\ plf$$

$$Dead_{rxn,y} = \frac{\omega_{ribsD} \cdot span}{2} \rightarrow 1.35\ kips$$

$$InternalForce_{ribs} = \frac{\frac{\omega_{ribsD} \cdot span^2}{2}}{archHeight} \rightarrow 7.76\ kips$$

This is the force at the peak of the arch, using statics

$$Dead_{rxn,x} = InternalForce_{ribs} \rightarrow 7.76\ kips$$

$$Resultant_{rxn} = \sqrt{\left(Dead_{rxn,x}\right)^2 + \left(Dead_{rxn,y}\right)^2} \rightarrow 7.87\ kips$$

This is the resultant force at the base of the arch

Being a two-force member, the HSS section is designed (ideally) for pure axial compression. AISC Table 4-4, the HSS section is selected based on the effective length.

A cable spacing of 20' is used to eliminate stringers. The unbraced length of the rib is then approximately 23'

Choose Square HSS 6x6x3⁄16 with:

$$\phi P_{n,rib} = 66.3\ kips \qquad \omega_{rib} = 14.53 \frac{lbs}{ft}$$

A 6x6 HSS shape was chosen to match the size of the diagonals and verticals in the truss. This is presumed to simplify the rib connection to the truss.

Using AASHTO LRFD load combinations, the reaction the rib makes with the truss is found:

$$\omega_{Du,ribs} = n_1 \cdot \gamma_D \cdot \omega_{rib} \rightarrow 18.2\,plf$$

$$\omega_{u,rib} = \omega_{Du,ribs} \rightarrow 18.2\,plf$$

$$Rxn_y = \frac{\omega_{u,rib} \cdot span}{2} \rightarrow 1.23\,kips$$

Using statics, the force at the peak of the arch is found: $$InternalForce_{ribs,u} = \frac{\frac{\omega_{u,rib} \cdot span^2}{2}}{archHeight} \rightarrow 7.0\,kips$$

$$Rxn_x = InternalForce_{ribs,u} \rightarrow 7.0\,kips$$

$$Resultant_{rxn,u} = \sqrt{\left(Rxn_x\right)^2 + \left(Rxn_y\right)^2} \rightarrow 7.1\,kips$$

Using AASHTO LRFD load combinations, the vertical seismic component is added:

$$P_{ribaashto} = Resultant_{rxn,u} + n_1 \cdot 0.2 \cdot S_{DS} \cdot Resultant_{rxn,u} \rightarrow 8.61\,kips$$

Using AISC manual tables, this still falls far below ϕP_n of 66.3 kips, and thus is sufficient.

Connection to truss bridge:

$$F_{nv,A325N} = 54\,ksi$$

$$m_{shear} = 2$$

A plate on each side of the rib will be used, putting the bolt in double shear.

$$d_{bolt} = \left(\frac{1}{2}\right)\,in$$

$$A_{bolt} = \frac{\pi \cdot \left(d_{bolt}^{\;2}\right)}{4} \rightarrow 0.2\,in^2$$

$$\phi R_{nbolt} = 0.75 \cdot m_{shear} \cdot F_{nv,A325N} \cdot A_{bolt} \rightarrow 15.9\,kips$$

$$NumberBolts_x = \frac{Rxn_x}{\phi R_{nbolt}} \rightarrow 0.44 \qquad NumberBolts_y = \frac{Rxn_y}{\phi R_{nbolt}} \rightarrow 0.077$$

To obtain the necessary resistance, 1 bolt needs to be placed. For redundancy, 3 total bolts will be placed.

Tear-out and bearing are verified: An A572 Grade 50 ½" plate is proposed:

Bolt hole dimension, from AISC Table J3.3: $BoltHole = \left(\frac{9}{16}\right)\ in$

Minimum edge distance from center of hole to edge, from AISC Table J3.4: $EdgeDist_{min} = \left(\frac{3}{4}\right)\ in$

A standard edge distance from center of bolt to edge of plate is 3"

$$L_c = 3\ in - \left(\frac{BoltHole}{2}\right) \rightarrow 2.72\,in \quad > \quad 2 \cdot d_{bolt} \rightarrow 1\ in$$

Therefore, bearing controls over tear-out

$$\phi R_{n,brg,x} = 0.75 \cdot 2.4 \cdot d_{bolt} \cdot \left(\frac{1}{2}\right)\ in \cdot 65\ ksi \rightarrow 29.3\,kips \quad > \quad Rxn_x \rightarrow 7.0\,kips$$

$$\phi R_{n,brg,y} = 0.75 \cdot 2.4 \cdot d_{bolt} \cdot \left(\frac{1}{2}\right)\ in \cdot 65\ ksi \rightarrow 29.3\,kips \quad > \quad Rxn_y \rightarrow 1.23\,kips$$

There is sufficient strength to use a ½" A572 Gr 50 plate on each side of the arch rib to chord connection on each rib.

Arch Rib splices:

For conservatism, the resultant force at the base of the arch is used as a basis for all of the splices along the arch.

$$NumberBolts_{splice} = \frac{Resultant_{rxn,u}}{\phi R_{nbolt}} \rightarrow 0.45$$

Therefore, use 1 bolt total for each splice in the rib. However, to develop a moment connection, 2 bolts on each face of the HSS section will be used.

Cable Sizing:

The cables do not support any load. Therefore, they are not sized based on stress calculations.

To prevent the cables from being cut, a sufficient diameter must be chosen to resist bolt cutters. Thus, a **1.0 inch diameter is chosen**.

Stainless steel wire rope is chosen to resist corrosion. According to https://www.wwewirerope.com/galvanized-wire-rope.html#stainless-steel-wire-rope, the approximate weight per foot of stainless steel wire rope is:

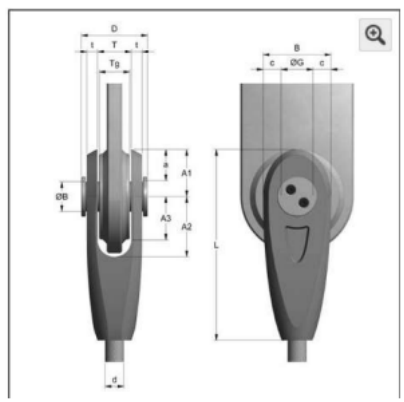
$$\omega_{rope} = 1.85 \frac{lbs}{ft}$$

The weight of the longest cable is found:

$$W_{rope} = \omega_{rope} \cdot archHeight \rightarrow 43.5\, lbs$$

According to https://www.steelwirerope.com/WireRopes/Structural/open-spelter-socket.html, an open-spelter socket is used to connect the cable strand to the arch and chords.

The weld to the heaviest cable is designed, and this weld is applied to all subsequent cables in the arch.



The weld of the socket plate to the rib is designed. www.steelrope.com offers the smallest space of 1.8 inches for T_g; therefore initial plate thickness of 1" is used.

The required length of weld is found:

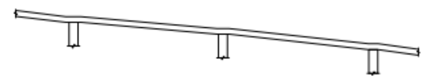
$$l_{weld} = \frac{W_{rope}}{0.75 \cdot 0.6 \cdot F_{E70} \cdot \left(1 + sin^{1.5}\left(\theta_{weld}\right)\right) \cdot 0.707 \cdot \left(\frac{3}{16}\right)\, in} \rightarrow 0.0052\, in$$

Therefore, use a 1" total weld.

The minimum B according to the supplier is 4 inches. Therefore, use a 4" wide plate with 1" thickness

RAMP DESIGN.

A sketch of the ramp (not to scale) is shown to the right. Although the ramp is sloped, it is believed a reasonable approximation of the loading will be obtained by treating the ramp as flat.



Alternate live loading and skipping of live loads must be accounted for, along with the indeterminate nature of the ramps. For simplification, t**he ACI coefficient method is used to obtain the shear and moment envelope at critical slab locations**. This is found in ACI 318 section 6.5.

$$Span_{ramp} = 30\,ft$$

$$h_{slab} = \frac{Span_{ramp}}{28} \rightarrow 12.9\,in$$ For "both ends continuous" Solid One-way slabs, according to ACI

$$h_{slab,use} = 13\,in$$

$$D_{ramp} = \gamma_{concrete} \cdot h_{slab,use} \rightarrow 157\,psf \qquad L \rightarrow 100\,psf$$

$$\omega_{D,ramp} = D_{ramp} \cdot 1\,ft \rightarrow 157\,plf \qquad \omega_L = L \cdot 1\,ft \rightarrow 100\,plf$$

$$\omega_{u,ramp} = \left(1.2 \cdot \omega_{D,ramp}\right) + \left(1.6 \cdot \omega_L\right) \rightarrow 0.35\,klf$$

$$\omega_{Lu} = 1.6 \cdot \omega_L \rightarrow 0.16\,klf$$

The ACI coefficient method is used to obtain the moment and shear values at critical locations of the slab. The outer spans represent the outer edges of the ramp at the bridge landing and the MSE wall. For conservatism, any maximum moments or shears that may occur here are used to design the entire ramp.

$$M_{u,ramp,positive} = 18.1\,kip \cdot ft$$

$$M_{u,ramp,negative} = -25.4\,kip \cdot ft$$

$$V_{u,ramp} = 5.4\,kips$$

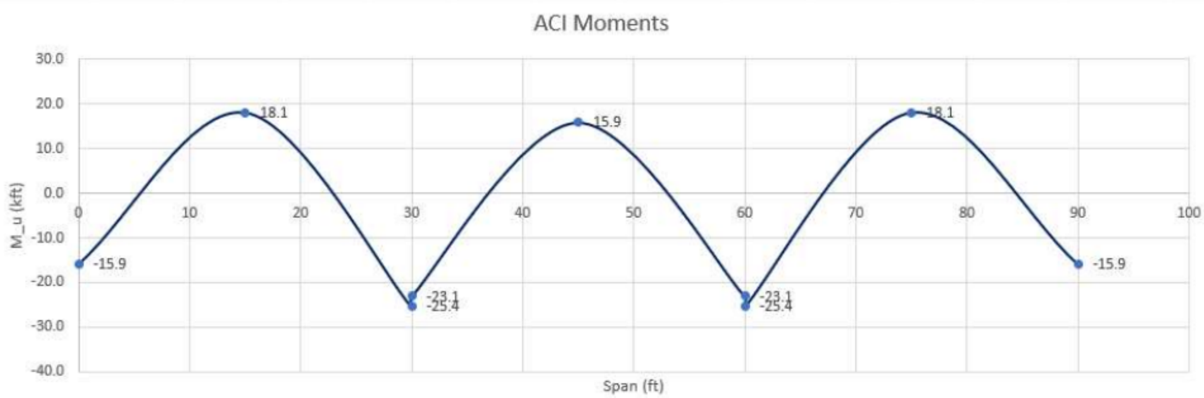
These are the maximum loads, using the ACI coefficient method, shown on the following page. The curves show **the maximum loading on the ramp for both shear and moment** . Thus, these curves represent the load envelope, as defined by ACI.

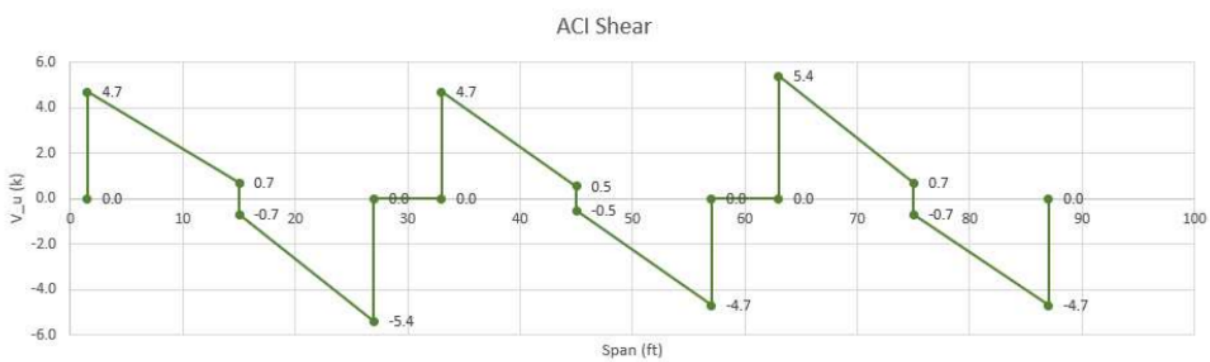
Loads using ACI Coefficient method

ACI Design Coefficients						
	D	157 psf		Type:	3	
	L	100 psf		Beam/Slab:	Slab	
	W_u	0.35 klf		Col/Beam width	3 ft	
	W_Lu	0.16 klf				
Criteria:				span #:	3	
L<=3*D:						
Spans<=1.2						

Type: Choose between
1. Discontinuous End Unrestrained,
2. Discontinuous End Restrained w Spandrel, or
3. Discontinuous End Restrained w Column

		Clear span 27 ft					Clear span 27 ft				Clear span 27 ft
		c/c spacing 30 ft					c/c spacing 30 ft				c/c spacing 30 ft
	End	Mid	End	End	Mid	End	End	Mid	End		
Cm	-0.06	0.07	-0.10	-0.09	0.06	-0.09	-0.10	0.07	-0.06		
Cv	1.00	0.15	1.15	1.00	0.11	1.00	1.15	0.15	1.00		
Flexure:											
l_n (ft)	27	27	27	27	27	27	27	27	27		
W_u*l_n^2	254	254	254	254	254	254	254	254	254		
M_u (kft)	-15.9	18.1	-25.4	-23.1	15.9	-23.1	-25.4	18.1	-15.9		
Shear:											
l_n (ft)	27	27	27	27	27	27	27	27	27		
W_u*l_n/2	4.7	4.7	4.7	4.7	4.7	4.7	4.7	4.7	4.7		
V_u (k)	4.7	0.7	-5.4	4.7	0.5	-4.7	5.4	0.7	-4.7		
		-0.7			-0.5			-0.7			





Designing the one-way slab:

Top of the slab:

$\varepsilon_{cu} = 0.003 \qquad \beta_1 = 0.85$

$f'_c \rightarrow 3\, ksi$

$f_y = 60{,}000\, psi$

$b_{slab} = 12\, in$

$d_{v,slab} = 13\, in$

$d_{slab} = 11\, in$

$A_{5bar} = 0.31\, in^2 \qquad A_{6bar} = 0.44\, in^2$

$$A_{s,slab} = A_{6bar} \cdot \left(\frac{b_{slab}}{slabspacing} \right) \rightarrow 0.59 in^2$$

$slabspacing = 9\, in$

$$a_{slab} = \frac{A_{s,slab} \cdot f_y}{0.85 \cdot f'_c \cdot b_{slab}} \rightarrow 1.15\, in$$

$$M_{n,slab} = A_{s,slab} \cdot f_y \cdot \left(d_{slab} - \frac{a_{slab}}{2} \right) \rightarrow 30.6\, kip \cdot ft$$

$$A_{smin,slab} = 0.0018 \cdot \left(b_{slab} \cdot d_{v,slab} \right) \rightarrow 0.28 in^2$$

$$c_{slab} = \frac{a_{slab}}{\beta_1} \rightarrow 1.35\, in$$

$$\varepsilon_t = \frac{\varepsilon_{cu} \cdot \left(d_{slab} - c_{slab} \right)}{c_{slab}} \rightarrow 0.021$$

$$\phi M_{n,slab} = 0.9 \cdot M_{n,slab} \rightarrow 27.5\, kip \cdot ft \quad > \quad -1 \cdot M_{u,ramp,negative} \rightarrow 25.4\, kip \cdot ft$$

GIVEN INFORMATION

f'_c	3000	psi	ß	0.85		Φ	0.9
f_y	60000	psi	a	1.15686			
A_s	0.590	in^2	c	1.36101			
b	12	in					
d	11	in	Bar size	6			
d_v	13	in	A_stot	0.590	in^2		0.88
ε_cu	0.003						
ε_ty	0.00207						
E	29000	ksi	ρ	0.00447			
cc	1.625	in	s	9	in		8

Reinforcement			Bar Diameter	
d_1	11	in	0.75	in
d_2	0	in	0	in
d_3	0	in	0	in

RESULTS

T	35400	lbs	35.4	kips
C	35400	lbs	35.4	kips
M_n	30743.63	lb-ft	30.7	kft
ΦM_n	2305.77	lb-ft	27.7	kft

REQUIREMENTS

ε_t	0.02124661	yielded?	YES	
Min net tensile strain	MET	ACI limit	0.00507	
Min. Rein	MET	ρ_min	0.00333	
Spacing	MET	s_max	10.9375	in

Bottom of the slab: $slabspacing_{bottom} = 8\ in$

$$A_{s,slab,bottom} = A_{5bar} \cdot \left(\frac{12\ in}{slabspacing_{bottom}} \right) \rightarrow 0.47 in^2$$

$$a_{slab,bottom} = \frac{A_{s,slab,bottom} \cdot f_y}{0.85 \cdot f'_c \cdot b_{slab}} \rightarrow 0.91\ in$$

$$M_{n,slabbottom} = A_{s,slab,bottom} \cdot f_y \cdot \left(d_{slab} - \frac{a_{slab,bottom}}{2} \right) \rightarrow 24.5\ kip \cdot ft$$

$$A_{smin,slabbottom} = 0.0018 \cdot \left(b_{slab} \cdot d_{slab} \right) \rightarrow 0.24 in^2$$

$$c_{slab,bottom} = \frac{a_{slab,bottom}}{\beta_1} \rightarrow 1.07\ in$$

$$\varepsilon_{t,bottom} = \frac{\varepsilon_{cu} \cdot \left(d_{slab} - c_{slab,bottom} \right)}{c_{slab,bottom}} \rightarrow 0.028$$

$$\phi M_{n,slabbottom} = 0.9 \cdot M_{n,slabbottom} \rightarrow 22.1\ kip \cdot ft \quad > \quad M_{u,ramp,positive} \rightarrow 18.1\ kip \cdot ft$$

GIVEN INFORMATION

f'_c	3000	psi	ß	0.85		Φ	0.9
f_y	60000	psi	a	0.92157			
A_s	0.470	in^2	c	1.0842			
b	12	in					
d	11	in	Bar size	5			
d_v	13	in	A_stot	0.470	in^2		0.62
ε_cu	0.003						
ε_ty	0.00207						
E	29000	ksi	ρ	0.00356			
cc	1.6875	in	s	8	in		8

Reinforcement			Bar Diameter	
d_1	11	in	0.625	in
d_2	0	in	0	in
d_3	0	in	0	in

RESULTS

T	28200	lbs	28.2	kips
C	28200	lbs	28.2	kips
M_n	24767.16	lb-ft	24.8	kft
ΦM_n	1857.54	lb-ft	22.3	kft

REQUIREMENTS

ε_t	0.02743723	yielded?	YES	
Min net tensile strain	MET	ACI limit	0.00507	
Min. Reinf	MET	ρ_min	0.00333	
Spacing	MET	s_max	10.7813	in

Checking shear:

Being a one-way slab, there are no shear reinforcement and the concrete must resist all shear loading:

$$V_{u,ramp} \rightarrow 5.4\ kips \quad < \quad 0.75 \cdot \sqrt{3{,}000} \cdot 12 \cdot 11 \rightarrow 5{,}422$$

Therefore, stirrups are not required.

$$V_c = 9.28\ kips$$

$$\phi V_c = 0.75 \cdot V_c \rightarrow 6.96 kips \quad > \quad V_{u,ramp} \rightarrow 5.4 kips$$

Therefore, shear is acceptable in this slab.

Slab deflections and serviceability:

According to ACI 318 section 7.3.2.1, immediate and long-term deflections do not need to be calculated if the slab meets minimum thickness requirements outlined in Table 7.3.1.1.

Table 7.3.1.1—Minimum thickness of solid nonprestressed one-way slabs

Support condition	Minimum $h^{[1]}$
Simply supported	$\ell/20$
One end continuous	$\ell/24$
Both ends continuous	$\ell/28$
Cantilever	$\ell/10$

[1]Expression applicable for normalweight concrete and f_y = 60,000 psi. For other cases, minimum h shall be modified in accordance with 7.3.1.1.1 through 7.3.1.1.3, as appropriate.

For a 30' one-way slab with both ends continuous: $h_{min} = \frac{30\,ft}{28} \rightarrow 12.9\,in$

This is compared to the actual depth of the slab: $h_{slab,use} \rightarrow 13\,in$

Therefore, immediate and long-term deflections do not need to be calculated, according to ACI 318.

Column Design

Axial load on the column supporting pedestrian ramp:

$$A_{trib,column} = 30\,ft \cdot Width \rightarrow 360ft^2$$

$$p_u = (1.2 \cdot D_{ramp}) + (1.6 \cdot L) \rightarrow 349\,psf$$

$$P_{u,column} = p_u \cdot A_{trib,column} \rightarrow 125\,kips$$

This is the ultimate axial load applied on the column from the ramp.

The moment applied to the column is calculated as follows:

A column diameter of 2.5 ft is initially selected $D_{column} = 2.5\,ft$

Weight of column: (using conservative 20ft height)

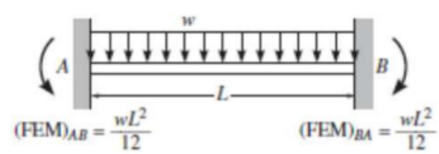
$$W_{column} = \frac{\gamma_{concrete} \cdot 20\,ft \cdot \pi \cdot D_{column}^{\;2}}{4} \rightarrow 14.2\,kips$$

To get the weight in plf, the total weight is divided by the projection (diameter)

$$w_{column} = \frac{W_{column}}{D_{column}} \rightarrow 5.69\,klf$$

$$F_{column1} = max(0.4 \cdot S_{DS} \cdot I \cdot w_{column}\,, 0.1 \cdot w_{column}) \rightarrow 2.33\,klf$$

It is desired that the pier caps and footings have greater moment capacity than the column, so that the plastic hinge forms in the column first. Thus, the columns are modeled as **fixed on both ends**. Using a fixed end moment table with distributed load, the moment due to seismic is:



https://engineering.stackexchange.com/questions/15040/how-to-determine-fixed-end-moment-in-beam

This is the seismic and wind moment at each end of the column.

The distributed load due to wind is calculated as:

$$wind_{column} = D_{column} \cdot p_{wind} \rightarrow 76\,plf$$

The loads on each column (C) are outlined below, corresponding to the column labels in Figure L.1

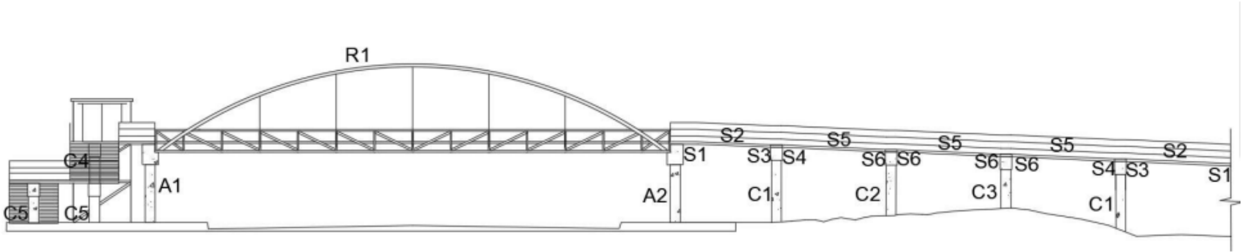


Figure L.1 - Labels for Structural Loads

Column C1

$$P_{1D} = A_{trib,column} \cdot D_{ramp} + \frac{\gamma_{concrete} \cdot \pi \cdot D_{column}^{2}}{4} \cdot 20\,ft \rightarrow 70.8\,kips$$

$$P_{1L} = A_{trib,column} \cdot L \rightarrow 36\,kips$$

$$P_{1S} = A_{trib,column} \cdot S \rightarrow 9.58\,kips$$

$$P_{1Ev} = -0.2 \cdot S_{DS} \cdot P_{1D} \rightarrow -14.5\,kips$$

Load			Combo	D	L	L_r/S/R	W	E_h	E_v	Result
D	70.8		1	1.4						99.1
L	36		2	1.2	1.6	0.5				147.4
L_r	0		3a	1.2	1	1.6				136.3
S	9.58		3b	1.2		1.6	0.5			100.3
L_r/S/R	9.58		3c	1.2		1.6	-0.5			100.3
R			4a	1.2	1	0.5	1			125.8
H			4b	1.2	1	0.5	-1			125.8
E			5a	0.9			1			63.7
E_h	0		5b	0.9			-1			63.7
E_v	14.45736		6a	1.2		0.2		1	1	101.3
F			6b	1.2		0.2		-1	1	72.4
T			7a	0.9				1	-1	78.2
W	0		7b	0.9				-1	-1	49.3

The maximum load effect is shown in red.

For the moments on the column:

$$M_{EColumn1} = \frac{F_{column1} \cdot (20\,ft)^{2}}{12} \rightarrow 77.5\,kip \cdot ft$$

$$M_{wColumn1} = \frac{(p_{wind} \cdot D_{column}) \cdot (20\,ft)^{2}}{12} \rightarrow 2.53\,kip \cdot ft$$

Load			Combo	D	L	L_r/S/R	W	E_h	E_v	Result
D	0		1	1.4						0.0
L	0		2	1.2	1.6	0.5				0.0
L_r	0		3a	1.2	1	1.6				0.0
S	0		3b	1.2		1.6	0.5			1.3
L_r/S/R	0		3c	1.2		1.6	-0.5			-1.3
R			4a	1.2	1	0.5	1			2.5
H			4b	1.2	1	0.5	-1			-2.5
E			5a	0.9			1			2.5
E_h	77.5		5b	0.9			-1			-2.5
E_v	0		6a	1.2		0.2		1	1	77.5
F			6b	1.2		0.2		-1	1	77.5
T			7a	0.9				1	-1	-77.5
W	2.53		7b	0.9				-1	-1	-77.5

Column C2

$$P_{2D} = A_{trib,column} \cdot D_{ramp} + \frac{\gamma_{concrete} \cdot \pi \cdot D_{column}^{2}}{4} \cdot 15\,ft \rightarrow 67.2\,kips$$

$$P_{2L} = A_{trib,column} \cdot L \rightarrow 36\,kips$$

$$P_{2S} = A_{trib,column} \cdot S \rightarrow 9.58\,kips$$

$$P_{2Ev} = -0.2 \cdot S_{DS} \cdot P_{2D} \rightarrow -13.7\,kips$$

Load			Combo	D	L	L_r/S/R	W	E_h	E_v	Result
D	67.2		1	1.4						94.1
L	36		2	1.2	1.6	0.5				143.0
L_r	0		3a	1.2	1	1.6				132.0
S	9.58		3b	1.2		1.6	0.5			96.0
L_r/S/R	9.58		3c	1.2		1.6	-0.5			96.0
R			4a	1.2	1	0.5	1			121.4
H			4b	1.2	1	0.5	-1			121.4
E			5a	0.9			1			60.5
E_h	0		5b	0.9			-1			60.5
E_v	13.72224		6a	1.2		0.2		1	1	96.3
F			6b	1.2		0.2		-1	1	68.8
T			7a	0.9				1	-1	74.2
W	0		7b	0.9				-1	-1	46.8

For the moments on the column:

$$M_{EColumn2} = \frac{F_{column1} \cdot (15\,ft)^{2}}{12} \rightarrow 43.6\,kip \cdot ft$$

$$M_{wColumn2} = \frac{(p_{wind} \cdot D_{column}) \cdot (15\,ft)^{2}}{12} \rightarrow 1.43\,kip \cdot ft$$

Load			Combo	D	L	L_r/S/R	W	E_h	E_v	Result
D	0		1	1.4						0.0
L	0		2	1.2	1.6	0.5				0.0
L_r	0		3a	1.2	1	1.6				0.0
S	0		3b	1.2		1.6	0.5			0.7
L_r/S/R	0		3c	1.2		1.6	-0.5			-0.7
R			4a	1.2	1	0.5	1			1.4
H			4b	1.2	1	0.5	-1			-1.4
E			5a	0.9			1			1.4
E_h	43.6		5b	0.9			-1			-1.4
E_v	0		6a	1.2		0.2		1	1	43.6
F			6b	1.2		0.2		-1	1	43.6
T			7a	0.9				1	-1	-43.6
W	1.43		7b	0.9				-1	-1	-43.6

Column C3

$$P_{3D}=A_{trib,column}\cdot D_{ramp}+\frac{\gamma_{concrete}\cdot \pi \cdot D_{column}^{\;2}}{4}\cdot 10\,ft \rightarrow 63.7\,kips$$

$$P_{3L}=A_{trib,column}\cdot L \rightarrow 36\,kips$$

$$P_{3S}=A_{trib,column}\cdot S \rightarrow 9.58\,kips$$

$$P_{3Ev}=-0.2\cdot S_{DS}\cdot P_{3D} \rightarrow -13\,kips$$

Load			Combo	D	L	L_r/S/R	W	E_h	E_v	Result
D	63.7		1	1.4						89.2
L	36		2	1.2	1.6	0.5				138.8
L_r	0		3a	1.2	1	1.6				127.8
S	9.58		3b	1.2		1.6	0.5			92.5
L_r/S/R	9.58		3c	1.2		1.6	-0.5			91.1
R			4a	1.2	1	0.5	1			118.7
H			4b	1.2	1	0.5	-1			115.8
E			5a	0.9			1			58.8
E_h	0		5b	0.9			-1			55.9
E_v	13.00754		6a	1.2		0.2		1	1	91.4
F			6b	1.2		0.2		-1	1	65.3
T			7a	0.9				1	-1	70.3
W	1.43		7b	0.9				-1	-1	44.3

For the moments on the column:

$$M_{EColumn3}=\frac{F_{column1}\cdot (10\,ft)^2}{12} \rightarrow 19.4\,kip\cdot ft$$

$$M_{wColumn3}=\frac{(p_{wind}\cdot D_{column})\cdot (10\,ft)^2}{12} \rightarrow 0.63\,kip\cdot ft$$

Load			Combo	D	L	L_r/S/R	W	E_h	E_v	Result
D	0		1	1.4						0.0
L	0		2	1.2	1.6	0.5				0.0
L_r	0		3a	1.2	1	1.6				0.0
S	0		3b	1.2		1.6	0.5			0.3
L_r/S/R	0		3c	1.2		1.6	-0.5			-0.3
R			4a	1.2	1	0.5	1			0.6
H			4b	1.2	1	0.5	-1			-0.6
E			5a	0.9			1			0.6
E_h	19.4		5b	0.9			-1			-0.6
E_v	0		6a	1.2		0.2		1	1	19.4
F			6b	1.2		0.2		-1	1	19.4
T			7a	0.9				1	-1	-19.4
W	0.63		7b	0.9				-1	-1	-19.4

Column C4

$$P_{4D}=\left(10\,ft\cdot 12\,ft\right)\cdot D_{ramp}+\frac{\gamma_{concrete}\cdot\pi\cdot D_{column}^{2}}{4}\cdot 20\,ft+\left(\frac{15\,in\cdot\gamma_{concrete}\cdot 17.25\,ft}{2}\cdot 12\,ft\right)\rightarrow 51.8\,kips$$

$$P_{4L}=\left(\left(10\,ft\cdot 12\,ft\right)+\left(\frac{17.25\,ft}{2}\cdot 12\,ft\right)\right)\cdot L\rightarrow 22.4\,kips$$

$$P_{4S}=\left(\left(10\,ft\cdot 12\,ft\right)+\left(\frac{17.25\,ft}{2}\cdot 12\,ft\right)\right)\cdot S\rightarrow 5.95\,kips$$

$$P_{4Ev}=-0.2\cdot S_{DS}\cdot P_{4D}\rightarrow -10.6\,kips$$

Load			Combo	D	L	L_r/S/R	W	E_h	E_v	Result
D	51.8		1	1.4						72.5
L	22.4		2	1.2	1.6	0.5				101.0
L_r	0		3a	1.2	1	1.6				94.1
S	5.95		3b	1.2		1.6	0.5			71.7
L_r/S/R	5.95		3c	1.2		1.6	-0.5			71.7
R			4a	1.2	1	0.5	1			87.5
H			4b	1.2	1	0.5	-1			87.5
E			5a	0.9			1			46.6
E_h	0		5b	0.9			-1			46.6
E_v	10.57756		6a	1.2		0.2		1	1	73.9
F			6b	1.2		0.2		-1	1	52.8
T			7a	0.9				1	-1	57.2
W	0		7b	0.9				-1	-1	36.0

For the moments, there are moments in the plane of Figure L.1 and out of plane. Due to the stairs adding more tributary area, the out of plane moments control.

$$M_{4D}=\left(\frac{17.25\,ft}{4}\right)\cdot\left(\frac{15\,in\cdot\gamma_{concrete}\cdot 17.25\,ft}{2}\cdot 12\,ft\right)\rightarrow 80.9\,kip\cdot ft$$

$$M_{4L}=\left(\frac{17.25\,ft}{4}\right)\cdot\left(\frac{L\cdot 17.25\,ft}{2}\cdot 12\,ft\right)\rightarrow 44.6\,kip\cdot ft$$

$$M_{4S}=\left(\frac{17.25\,ft}{4}\right)\cdot\left(\frac{S\cdot 17.25\,ft}{2}\cdot 12\,ft\right)\rightarrow 11.9\,kip\cdot ft$$

$$M_{EColumn4}=\frac{F_{column1}\cdot\left(20\,ft\right)^{2}}{12}\rightarrow 77.5\,kip\cdot ft$$

$$M_{Ecolumn4v}=0.2\cdot S_{DS}\cdot M_{4D}\rightarrow 16.5\,kip\cdot ft$$

$$M_{wColumn4}=\frac{\left(p_{wind}\cdot D_{column}\right)\cdot\left(20\,ft\right)^{2}}{12}\rightarrow 2.53\,kip\cdot ft$$

Load			Combo	D	L	L_r/S/R	W	E_h	E_v	Result
D	80.9		1	1.4						113.3
L	44.6		2	1.2	1.6	0.5				174.4
L_r	0		3a	1.2	1	1.6				160.7
S	11.9		3b	1.2		1.6	0.5			117.4
L_r/S/R	11.9		3c	1.2		1.6	-0.5			114.9
R			4a	1.2	1	0.5	1			150.2
H			4b	1.2	1	0.5	-1			145.1
E			5a	0.9			1			75.3
E_h	77.5		5b	0.9			-1			70.3
E_v	16.51978		6a	1.2		0.2		1	1	193.5
F			6b	1.2		0.2		-1	1	160.4
T			7a	0.9				1	-1	11.8
W	2.53		7b	0.9				-1	-1	-21.2

Column C5

$$P_{5D}=\left(\frac{27\,ft}{2}\cdot 12\,ft\right)\cdot D_{ramp}+\frac{\gamma_{concrete}\cdot \pi \cdot D_{column}^{\;2}}{4}\cdot 11\,ft+\left(\frac{15\,in\cdot \gamma_{concrete}\cdot 17.25\,ft}{2}\cdot 12\,ft\right)\rightarrow 52\,kips$$

$$P_{5L}=\left(\left(\frac{27\,ft}{2}\cdot 12\,ft\right)+\left(\frac{17.25\,ft}{2}\cdot 12\,ft\right)\right)\cdot L\rightarrow 26.6\,kips$$

$$P_{5S}=\left(\left(\frac{27\,ft}{2}\cdot 12\,ft\right)+\left(\frac{17.25\,ft}{2}\cdot 12\,ft\right)\right)\cdot S\rightarrow 7.06\,kips$$

$$P_{5Ev}=0.2\cdot S_{DS}\cdot P_{5D}\rightarrow 10.6\,kips$$

Load			Combo	D	L	L_r/S/R	W	E_h	E_v	Result
D	52		1	1.4						72.8
L	26.6		2	1.2	1.6	0.5				108.5
L_r	0		3a	1.2	1	1.6				100.3
S	7.06		3b	1.2		1.6	0.5			73.7
L_r/S/R	7.06		3c	1.2		1.6	-0.5			73.7
R			4a	1.2	1	0.5	1			92.5
H			4b	1.2	1	0.5	-1			92.5
E			5a	0.9			1			46.8
E_h	0		5b	0.9			-1			46.8
E_v	10.6184		6a	1.2		0.2		1	1	74.4
F			6b	1.2		0.2		-1	1	53.2
T			7a	0.9				1	-1	57.4
W	0		7b	0.9				-1	-1	36.2

As for column C4, there is biaxial bending on column C5. Because the stairs have a larger tributary area then the landing, the out of plane moments control.

$$M_{5D}=\left(\frac{17.25\,ft}{4}\right)\cdot\left(\frac{15\,in\cdot \gamma_{concrete}\cdot 17.25\,ft}{2}\cdot 12\,ft\right)\rightarrow 80.9\,kip\cdot ft$$

$$M_{5L}=\left(\frac{17.25\,ft}{4}\right)\cdot\left(\frac{L\cdot 17.25\,ft}{2}\cdot 12\,ft\right)\rightarrow 44.6\,kip\cdot ft$$

$$M_{5S}=\left(\frac{17.25\,ft}{4}\right)\cdot\left(\frac{S\cdot 17.25\,ft}{2}\cdot 12\,ft\right)\rightarrow 11.9\,kip\cdot ft$$

$$M_{EColumn5}=\frac{F_{column1}\cdot (11\,ft)^2}{12}\rightarrow 23.4\,kip\cdot ft$$

$$M_{Ecolumn5v}=0.2\cdot S_{DS}\cdot M_{5D}\rightarrow 16.5\,kip\cdot ft$$

$$M_{wColumn5}=\frac{(p_{wind}\cdot D_{column})\cdot (11\,ft)^2}{12}\rightarrow 0.77\,kip\cdot ft$$

Load			Combo	D	L	L_r/S/R	W	E_h	E_v	Result
D	80.9		1	1.4						113.3
L	44.6		2	1.2	1.6	0.5				174.4
L_r	0		3a	1.2	1	1.6				160.7
S	11.9		3b	1.2		1.6	0.5			116.5
L_r/S/R	11.9		3c	1.2		1.6	-0.5			115.7
R			4a	1.2	1	0.5	1			148.4
H			4b	1.2	1	0.5	-1			146.9
E			5a	0.9			1			73.6
E_h	23.4		5b	0.9			-1			72.0
E_v	16.51978		6a	1.2		0.2		1	1	139.4
F			6b	1.2		0.2		-1	1	106.3
T			7a	0.9				1	-1	65.9
W	0.77		7b	0.9				-1	-1	32.9

Now that the loads are established, the column is designed:

Use 2-#10 bars at each layer of reinforcement. 

$$rebar_{spacing} = \frac{360\,°}{6} \rightarrow 60\,° \qquad \theta = \frac{rebar_{spacing}}{2} \rightarrow 30\,°$$

$$rebarradius_{cover} = \frac{D_{column}}{2} - 2\,in \rightarrow 13\,in$$

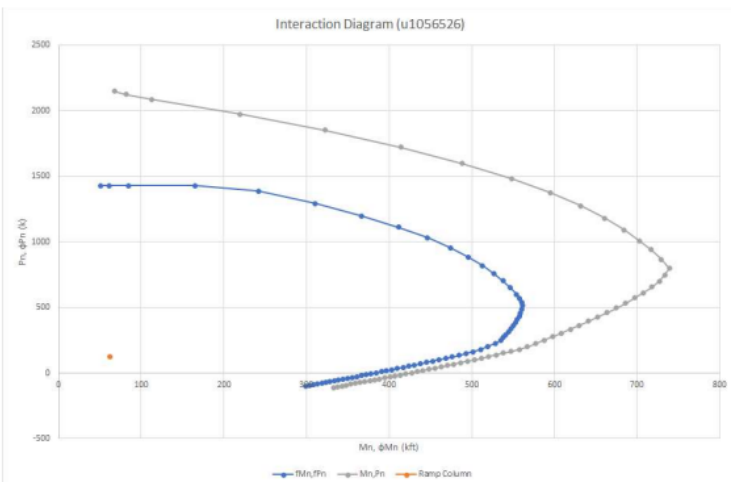
A clear cover of 2 inches on each side is used.

$$rebarradius = rebarradius_{cover} - \left(\frac{1.27\,in}{2}\right) \rightarrow 12.4\,in$$

$$rebardepth_1 = \frac{D_{column}}{2} + \left(rebarradius \cdot \cos(\theta)\right) \rightarrow 25.7\,in$$

$$rebardepth_2 = \frac{D_{column}}{2} + \left(rebarradius \cdot \cos\left(\theta + rebar_{spacing}\right)\right) \rightarrow 15\,in$$

$$rebardepth_3 = \frac{D_{column}}{2} - rebarradius \cdot \cos(\theta) \rightarrow 4.29\,in$$



This column appears to be too conservative for the loading. However, the minimum reinforcement and slenderness have not yet been checked.

$$\rho_{min,column} = 0.01$$

$$\rho_{column,10} = \frac{6 \cdot 1.27\,in^2}{\frac{\pi \cdot \left(D_{column}\right)^2}{4}} \rightarrow 0.011$$

This meets the minimum reinforcement ratio of 0.01. Using 6 #10 bars yields a ρ of 0.0085.

$$\rho_{column,9} = \frac{6 \cdot 1.00\,in^2}{\frac{\pi \cdot \left(D_{column}\right)^2}{4}} \rightarrow 0.0085$$

Next, slenderness is checked to see if a smaller column diameter is appropriate. The tallest column is 20 feet tall; this is the unbraced length. The effective length factor is taken as 0.8 because the column is idealized as fixed at the footing and top with no translation.

$$l_{u,column} = 20\,ft \qquad I_{column} = \frac{\pi \cdot \left(D_{column}\right)^4}{64} \rightarrow 1.92 ft^4$$

$$k_{column} = 0.65 \qquad A_{column} = \frac{\pi \cdot \left(D_{column}\right)^2}{4} \rightarrow 4.91 ft^2$$

$$r_{column} = \sqrt{\frac{I_{column}}{A_{column}}} \rightarrow 0.63\,ft$$

$$\frac{k_{column} \cdot l_{u,column}}{r_{column}} \rightarrow 20.8 \quad < \quad 34 - 12 \cdot \left(\frac{M_{EColumn1}}{M_{EColumn1}}\right) \rightarrow 22$$

A column diameter of 2 ft is now checked for slenderness to ensure a diameter of 2.5 feet is sufficient:

$$D_{column,1} = 2\,ft$$

$$I_{column,1} = \frac{\pi \cdot \left(D_{column,1}\right)^4}{64} \rightarrow 0.79 ft^4$$

$$A_{column,1} = \frac{\pi \cdot \left(D_{column,1}\right)^2}{4} \rightarrow 3.14 ft^2$$

$$r_{column,1} = \sqrt{\frac{I_{column,1}}{A_{column,1}}} \rightarrow 0.5\,ft$$

$$\frac{k_{column} \cdot l_{u,column}}{r_{column,1}} \rightarrow 26 \quad > \quad 34 - 12 \cdot \left(\frac{M_{EColumn1}}{M_{EColumn1}}\right) \rightarrow 22$$

Therefore, a 2.5 ft diameter column meets slenderness limits, unlike a 2 ft diameter column.

The applied axial load and the moment fits well within the interaction diagram.
Therefore, this column is sufficient.

$$P_{u,column} \rightarrow 125\,kips$$

$$M_{EColumn1} \rightarrow 77.5\,kip \cdot ft$$

The column spiral is now designed:

$$A_{g,column} = \frac{\pi \cdot D_{column}^{\;2}}{4} \rightarrow 4.91 ft^2$$

$$D_{column} \rightarrow 2.5\,ft$$

$$D_{core} = D_{column} - \left(2 \cdot 2\,in\right) \rightarrow 26\,in \qquad A_{core} = \frac{\pi \cdot \left(D_{core}\right)^2}{4} \rightarrow 531 in^2$$

$$C_{spiral} = \pi \cdot D_{core} \rightarrow 81.7\,in$$

$$\rho_{s,column} = \frac{0.45 \cdot \left(\frac{A_{g,column}}{A_{core}} - 1\right) \cdot f'_c}{60{,}000\,psi} \rightarrow 0.0075$$

This is the volumetric ratio of reinforcement required.

$$V_{c,column} = A_{g,column} \cdot 12\,in \rightarrow 8{,}482 in^3$$

The volume of concrete per 1 ft of length

$$V_{req} = \rho_{s,column} \cdot V_{c,column} \rightarrow 63.2 in^3$$

$$A_{4bar} = 0.2\,in^2$$

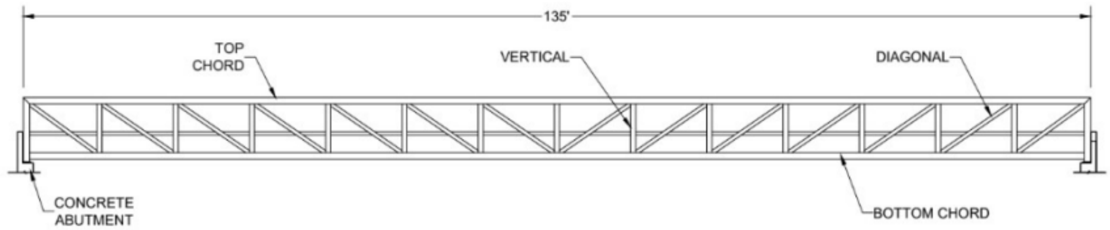
The required volume of reinforcement per 1 ft of length

$$s_{pitch} = \frac{12\,in}{V_{req}} \cdot A_{4bar} \cdot C_{spiral} \rightarrow 3.1\,in$$

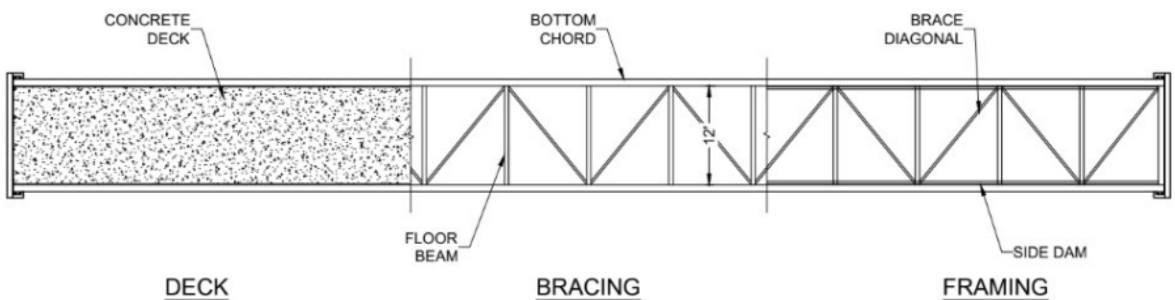
Therefore, choose a number 4 spiral with a pitch of 3 inches.

The total weight of the arch bridge is approximated to properly design the abutment:

BRIDGE SUMMARY
Connector Pedestrian Bridge 135' Span x 12' Width
Deck Type: Concrete
Bridge Finish: Weathering Steel



BRIDGE ELEVATION



BRIDGE PLAN

Taken from ContechES DYOB

Approximate weight of the truss:

Self weight of sections

$\omega_{8x8} = 25.82\, plf$

$\omega_{6x6} = 19.02\, plf$

$\omega_{10x10} = 32.63\, plf$

Number of members:

$NumberDiagonal = 28$

$NumberDiagonalBrace = 28$

$NumberVerticals = 15$

$NumberTopBraces = 5$

$NumberBeams = 30$

Length of members

$l_{vertical} = 7\, ft$

$l_{diagonal} = \sqrt{(10\, ft)^2 + (7\, ft)^2} \rightarrow 12.2\, ft$

$l_{diagonalbrace} = \sqrt{(12\, ft)^2 + (10\, ft)^2} \rightarrow 15.6\, ft$

$l_{approx} = \sqrt{(12\, ft)^2 + (20\, ft)^2} \rightarrow 23.3\, ft$

(This is the length of the top cross bracing.)

Total weight (dead load) of the truss is estimated as:

$$D_{trusssteel} = 4 \cdot \left(\omega_{8x8} \cdot 135\,ft \right) + \left(28 \cdot l_{diagonal} \cdot \omega_{6x6} \right) + \left(28 \cdot l_{diagonalbrace} \cdot \omega_{6x6} \right) + \left(30 \cdot l_{vertical} \cdot \omega_{8x8} \right) + \left(30 \cdot 12\,ft \cdot \omega_{8x8} \right) \rightarrow 43.5\,kips$$

The above calculation accounts for (in order) the top and bottom chords, the diagonals on the truss, the diagonal braces against lateral sway, vertical members, and floor beams.

Now, the weight of the concrete deck is estimated, using a 3" metal deck with 2" of concrete on top.

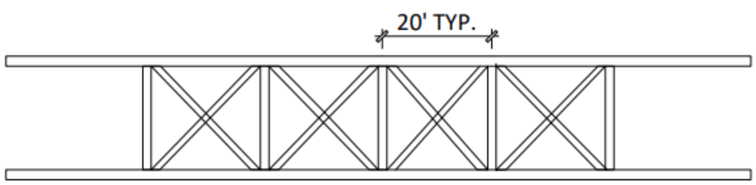
$$\gamma_{concrete} \rightarrow 145\,pcf$$

$$D_{concrete} = \left(2\,in \cdot \gamma_{concrete} \cdot 135\,ft \cdot 12\,ft \right) \cdot 1.5 \rightarrow 58.7\,kips$$

The dead load of the truss is then:

$$D_{truss} = D_{trusssteel} + D_{concrete} \rightarrow 102\,kips$$

Now, the weight of the arch ribs and top braces will be estimated:



PLAN VIEW OF ARCH RIBS AND BRACING

$$D_{ribs} = Length \cdot 2 \cdot \omega_{rib} \rightarrow 4.23\,kips$$

$$D_{arch} = D_{ribs} + \left(5 \cdot 12\,ft \cdot \omega_{6x6} \right) + \left(8 \cdot l_{approx} \cdot \omega_{6x6} \right) \rightarrow 8.92\,kips$$

This accounts for the 5 top braces and the 8 cross braces.

Then, the total structure weight is:

$$D_{StructureTotal} = D_{arch} + D_{truss} \rightarrow 111\,kips$$

The total live and snow loads on the structure are:

$$L_{Total} = L \cdot 12\,ft \cdot \left(135\,ft + 10\,ft + 15\,ft \right) \rightarrow 192\,kips \qquad S_{Total} = S \cdot 12\,ft \cdot \left(135\,ft + 10\,ft + 15\,ft \right) \rightarrow 51.1\,kips$$

On each end column, parts of the pedestrian ramp transfer load, and thus this dead load must be accounted for. The tributary width of 10 feet is used on the left abutment, while the tributary load of 15 feet is used on the right.

The loading on abutments 1 and 2 are given, according to the labels in Figure L.1

$$D_{Abut1} = \frac{D_{StructureTotal}}{2} + \left(D_{ramp} \cdot \left(10\,ft \right) \cdot 12\,ft \right) \rightarrow 74.4\,kips$$

$$D_{Abut2} = \frac{D_{StructureTotal}}{2} + \left(D_{ramp} \cdot 15\,ft \cdot 12\,ft \right) \rightarrow 83.8\,kips$$

$$L_{Abut1} = L \cdot 12\,ft \cdot \left(\frac{135\,ft}{2} + 10\,ft \right) \rightarrow 93{,}000\,lbs$$

$$L_{Abut2} = L \cdot 12\,ft \cdot \left(\frac{135\,ft}{2} + 15\,ft \right) \rightarrow 99{,}000\,lbs$$

$$S_{Abut1} = S \cdot 12\,ft \cdot \left(\frac{135\,ft}{2} + 10\,ft \right) \rightarrow 24{,}738\,lbs$$

$$S_{Abut2} = S \cdot 12\,ft \cdot \left(\frac{135\,ft}{2} + 15\,ft \right) \rightarrow 26{,}334\,lbs$$

There are also moments on the column due to eccentricities. The eccentricity of loads from the bridge *and* ramps are approximated as 2'. The applied moments from the bridge and ramp are in opposition to each other; therefore, the total moments are:

$e = 2\,ft \rightarrow 2\,ft$ $\qquad$ $e_{ramp} = 2\,ft \rightarrow 2\,ft$

$$M_{DAbut1} = \frac{D_{StructureTotal}}{2} \cdot e - D_{ramp} \cdot 10\,ft \cdot 12\,ft \cdot e_{ramp} \rightarrow 73.4\,kip \cdot ft$$

$$M_{DAbut2} = \frac{D_{StructureTotal}}{2} \cdot e - \left(D_{ramp} \cdot 15\,ft \cdot 12\,ft \right) \cdot e_{ramp} \rightarrow 54.6\,kip \cdot ft$$

$$M_{LAbut1} = L \cdot 12\,ft \cdot \left(\frac{135\,ft}{2} \right) \cdot e - L \cdot 12\,ft \cdot 10\,ft \cdot e_{ramp} \rightarrow 138\,kip \cdot ft$$

$$M_{LAbut2} = L \cdot 12\,ft \cdot \left(\frac{135\,ft}{2} \right) \cdot e - L \cdot 12\,ft \cdot 15\,ft \cdot e_{ramp} \rightarrow 126\,kip \cdot ft$$

$$M_{SAbut1} = S \cdot 12\,ft \cdot \left(\frac{135\,ft}{2} \right) \cdot e - S \cdot 12\,ft \cdot 10\,ft \cdot e_{ramp} \rightarrow 36.7\,kip \cdot ft$$

$$M_{SAbut2} = S \cdot 12\,ft \cdot \left(\frac{135\,ft}{2} \right) \cdot e - S \cdot 12\,ft \cdot 15\,ft \cdot e_{ramp} \rightarrow 33.5\,kip \cdot ft$$

The AASHTO LRFD Load Combinations are shown:

Load Combination Limit State	*DC* *DD* *DW* *EH* *EV* *ES* *EL* *PS* *CR* *SH*	*LL* *IM* *CE* *BR* *PL* *LS*	*WA*	*WS*	*WL*	*FR*	*TU*	*TG*	*SE*	Use One of These at a Time				
										EQ	*BL*	*IC*	*CT*	*CV*
Strength I (unless noted)	γ_p	1.75	1.00	—	—	1.00	0.50/1.20	γ_{TG}	γ_{SE}	—	—	—	—	—
Strength II	γ_p	1.35	1.00	—	—	1.00	0.50/1.20	γ_{TG}	γ_{SE}	—	—	—	—	—
Strength III	γ_p	—	1.00	1.00	—	1.00	0.50/1.20	γ_{TG}	γ_{SE}	—	—	—	—	—
Strength IV	γ_p	—	1.00	—	—	1.00	0.50/1.20	—	—	—	—	—	—	—
Strength V	γ_p	1.35	1.00	1.00	1.00	1.00	0.50/1.20	γ_{TG}	γ_{SE}	—	—	—	—	—
Extreme Event I	1.00	γ_{EQ}	1.00	—	—	1.00	—	—	—	1.00	—	—	—	—
Extreme Event II	1.00	0.50	1.00	—	—	1.00	—	—	—	—	1.00	1.00	1.00	1.00
Service I	1.00	1.00	1.00	1.00	1.00	1.00	1.00/1.20	γ_{TG}	γ_{SE}	—	—	—	—	—
Service II	1.00	1.30	1.00	—	—	1.00	1.00/1.20	—	—	—	—	—	—	—
Service III	1.00	γ_{LL}	1.00	—	—	1.00	1.00/1.20	γ_{TG}	γ_{SE}	—	—	—	—	—
Service IV	1.00	—	1.00	1.00	—	1.00	1.00/1.20	—	1.00	—	—	—	—	—
Fatigue I—*LL, IM & CE* only	—	1.75	—	—	—	—	—	—	—	—	—	—	—	—
Fatigue II—*LL, IM & CE* only	—	0.80	—	—	—	—	—	—	—	—	—	—	—	—

$$n_1 = 1.00$$

$$\gamma_D = 1.25$$

$$\gamma_L = 1.75$$

$$\gamma_{EQ} = 1.00$$

$$\gamma_S = 1.75$$

Using the Strength I and Extreme Event I combinations, the moments are obtained. The maximum load effects are given:

$$P_{aashtoabut1strength1} = n_1 \cdot \gamma_D \cdot D_{Abut1} + n_1 \cdot \gamma_L \cdot L_{Abut1} + n_1 \cdot \gamma_S \cdot S_{Abut1} - n_1 \cdot \gamma_{EQ} \cdot 0.2 \cdot S_{DS} \cdot D_{Abut1} \rightarrow 284\ kips$$

$$P_{aashtoAbut2strength1} = \left(n_1 \cdot \gamma_D \cdot D_{Abut2}\right) + \left(n_1 \cdot \gamma_L \cdot L_{Abut2}\right) + \left(n_1 \cdot \gamma_S \cdot S_{Abut2}\right) - \left(n_1 \cdot \gamma_{EQ} \cdot 0.2 \cdot S_{DS} \cdot D_{Abut2}\right) \rightarrow 307\ kips$$

$$P_{aashtoabut1EE1} = n_1 \cdot \gamma_D \cdot D_{Abut1} + n_1 \cdot \gamma_{EQ} \cdot L_{Abut1} + n_1 \cdot \gamma_S \cdot S_{Abut1} - n_1 \cdot \gamma_{EQ} \cdot 0.2 \cdot S_{DS} \cdot D_{Abut1} \rightarrow 214\ kips$$

$$P_{aashtoAbut2EE1} = \left(n_1 \cdot \gamma_D \cdot D_{Abut2}\right) + \left(n_1 \cdot \gamma_{EQ} \cdot L_{Abut2}\right) + \left(n_1 \cdot \gamma_S \cdot S_{Abut2}\right) - \left(n_1 \cdot \gamma_{EQ} \cdot 0.2 \cdot S_{DS} \cdot D_{Abut2}\right) \rightarrow 233\ kips$$

The maximum factored moments are:

$$M_{aashtoabut1strength1} = n_1 \cdot \gamma_D \cdot M_{DAbut1} + n_1 \cdot \gamma_L \cdot M_{LAbut1} + n_1 \cdot \gamma_S \cdot M_{SAbut1} + n_1 \cdot \gamma_{EQ} \cdot M_{EColumn1} \rightarrow 475\,kip \cdot ft$$

$$M_{aashtoabut2strength1} = n_1 \cdot \gamma_D \cdot M_{DAbut2} + n_1 \cdot \gamma_L \cdot M_{LAbut2} + n_1 \cdot \gamma_S \cdot M_{SAbut2} + n_1 \cdot \gamma_{EQ} \cdot M_{EColumn1} \rightarrow 425\,kip \cdot ft$$

$$M_{aashtoabut1EE1} = n_1 \cdot \gamma_D \cdot M_{DAbut1} + n_1 \cdot \gamma_{EQ} \cdot M_{LAbut1} + n_1 \cdot \gamma_S \cdot M_{SAbut1} + n_1 \cdot \gamma_{EQ} \cdot M_{EColumn1} \rightarrow 372\,kip \cdot ft$$

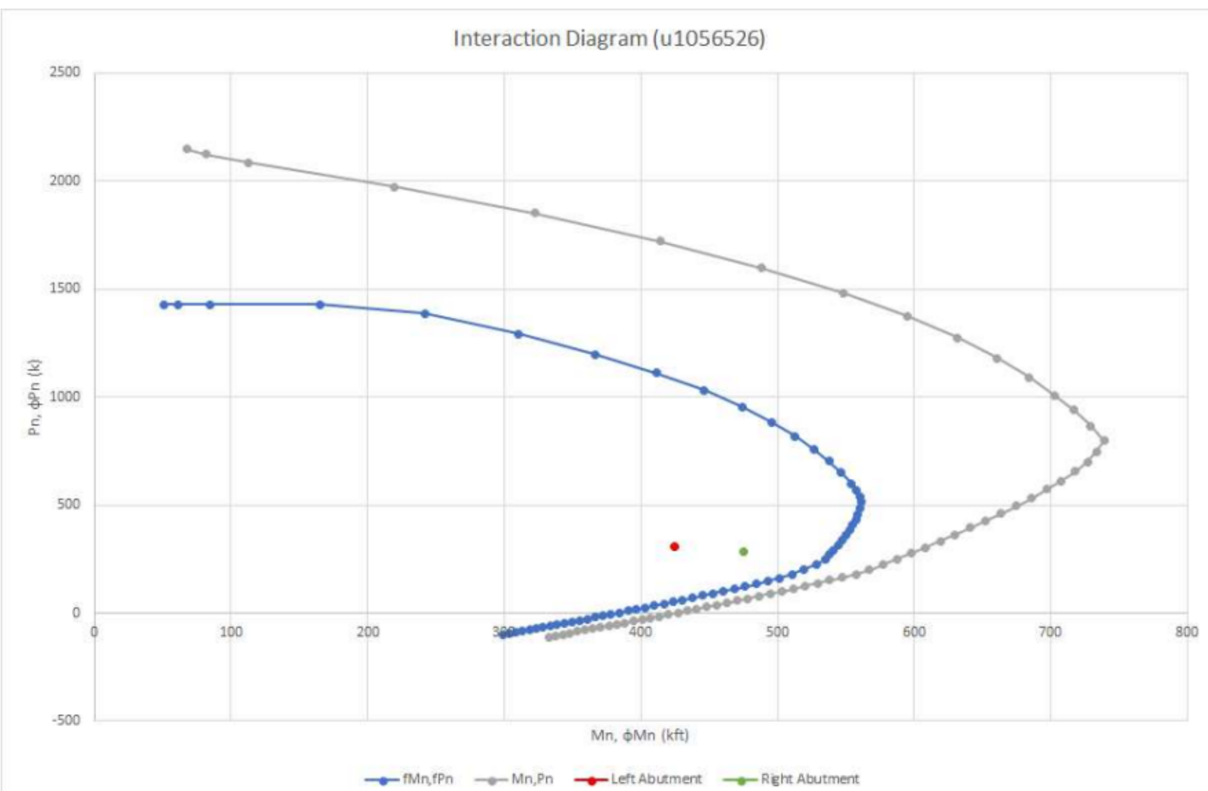
$$M_{aashtoabut2EE1} = n_1 \cdot \gamma_D \cdot M_{DAbut2} + n_1 \cdot \gamma_{EQ} \cdot M_{LAbut2} + n_1 \cdot \gamma_S \cdot M_{SAbut2} + n_1 \cdot \gamma_{EQ} \cdot M_{EColumn1} \rightarrow 330\,kip \cdot ft$$

The maximum values to be plotted on the interaction diagram are:

$$P_{aashtoAbut2strength1} \rightarrow 307\,kips$$
$$M_{aashtoabut2strength1} \rightarrow 425\,kip \cdot ft$$

$$P_{aashtoabut1strength1} \rightarrow 284\,kips$$
$$M_{aashtoabut1strength1} \rightarrow 475\,kip \cdot ft$$

Using the same column as previously designed (with the same reinforcement configuration and column height), the loads are plotted:



this loading fits within the interaction diagram; thus this column is sufficient.

FOUNDATIONS

All foundations will be piles.

These will likely be governed by seismic loads, which are outside the scope of this preliminary analysis.

The pile caps will be as thick as the column diameter:

$$t_{pilecap} = D_{column} \rightarrow 2.5ft$$

For this preliminary analysis the pile caps will have a 7' x 7' footprint. This is done to provide enough area for the piles, yet the size may be increased/decreased upon further pile calculations.

A.2 ISI Envision Analysis

Name: Justin Wilstead

Date: 09/08/22

Quality of Life

Does the project:

1. Improve health and safety for the broader community? 2
2. Preserve and enhance cultural resources? 2
3. Meet the needs and goals of the community? 1
4. Minimize negative impact on the surrounding community? 1
5. Follow a fair, equitable, and inclusive development process? 2
6. Is the project located near public transportation? 2

Discuss:

The addition of a grade separated crossing of Foothill Drive will improve safety and connectivity in this area. If care is taken during the design, the natural area can be preserved and/or enhanced. Nearby public transportation can become more desirable if access is achieved.

For each question, speculate as to:

+0 not applicable or no opportunity

+1 basic opportunity

+2 chance to go above and beyond for little cost

SCORE: 10

Leadership

1. Are there sustainability commitments from the project developers? 2
2. Is there a sustainability management plan in place? 1
3. Are stakeholders engaged? 2
4. Will the project stimulate economic development? 1
5. Are local residents employed on the project? 1
6. Is the project located near public transportation? 0

Discuss:

SLC is an Envision supported agency, meaning they have a high importance on sustainability. Following their lead, as well as the University's, should result in a responsible design that can enhance this community.

For each question, speculate as to:

+0 not applicable or no opportunity

+1 basic opportunity

+2 chance to go above and beyond for little cost

SCORE: 7

Resource Allocation

1. Is the project constructed from sustainable materials? 1
2. Does the project manage construction and operational waste? 1
3. Does the project reduce energy consumption and source renewable energy? 2
4. Does the project reduce water consumption and protect water resources? 2
5. Does the project monitor energy and water use? 1

Discuss:

Current stakeholders have a heavy emphasis on protecting the creek and preserving the natural landscape. This project has the opportunity to preserve those resources while creating a new recreational/commuter connection with Research Park and other trails in the area.

For each question, speculate as to:

+0 not applicable or no opportunity

+1 basic opportunity

+2 chance to go above and beyond for little cost

SCORE: 7

Natural World

Does the project:

1. Avoid sites of high ecological value? 1
2. Protect wetland and surface water quality? 2
3. Maintain hydrological functions? 2
4. Manage storm water? 1
5. Protect soil health? 1
6. Manage or eliminate invasive species? 1

Discuss:

Protection of the creek and surrounding area is very important to the project. Maintaining, if not bettering, the current system will be accounted for in the design. Opportunities to help preserve the area are: choosing a location that isn't impactful to the current creek area, considering the feel of the trail system and try to carry that into the crossing design and consider existing wildlife to include what their needs are.

For each question, speculate as to:

+0 not applicable or no opportunity

+1 basic opportunity

+2 chance to go above and beyond for little cost

SCORE: 8

Climate and Resilience

Does or is the project:

1. Reduce greenhouse gas emissions? 1
2. Reduce air pollutant emissions? 1
3. Avoid unsuitable sites? 0
4. Reduce climate change vulnerability? 1
5. Resilient and adaptable? 0

Discuss:

The inclusion of this crossing will help with the short trips from the Sunnyside Community to this area, which should help reduce emissions and improve air quality. However, we are pretty set in the general location where it should be located, and it would be hard to adjust the placement elsewhere, meaning we aren't very adaptable.

For each question, speculate as to:

+0 not applicable or no opportunity

+1 basic opportunity

+2 chance to go above and beyond for little cost

SCORE:3

Summary:

The key opportunities for this project are within the Quality of Life and Natural World areas, which is reflected in the scoring. This project has the ability to provide infrastructure that will allow for a safer and more comfortable crossing that can be integrated into the natural landscape to be used by recreational, student and commuting users.

A.3 Cost Estimate

Estimate for Stair and Elevator West side Ramp on the east				
Activities	Items	Unit	Unit Price	Amount
Mobilization	1	LS	$311,841.43	$311,841.43
Traffic Control	1	LS	$93,552.43	$93,552.43
Survey	1	LS	$34,353.22	$30,875.39
Environmental Study	1	LS	$100,000.00	$100,000.00
Clear and Grub	1	LS	$93,552.43	$93,552.43
Tree Removal	10	Each	$800.00	$8,000.00
Concrete Sidewalk removal	60	SF	$25.00	$1,500.00
Erosion Control	1	LS	$5,000.00	$5,000.00
SWPPP	1	LS	$10,000.00	$10,000.00
Inlet Protections	6	EA	$75.00	$450.00
Drive Pile 12"	2,560	ft	$80.00	$204,800.00
Roadway Excavation	400	CY	$30.00	$12,000.00
Prefabricated Bridge Truss (Contech)	1	LS	$750,000.00	$750,000.00
Structural Steel (Tied arch aesthetics)	1	LS	$40,000.00	$40,000.00
Structural Steel Cables (Tied arch aesthetics)	1	LS	$10,000.00	$10,000.00
Landscape Restoration	1	LS	$50,000.00	$50,000.00
Structural Concrete Ramp East (includes reinforcement)	1	EA	$400,000.00	$400,000.00
Stair Structure/ Bike Rail West side	1	EA	$150,000.00	$150,000.00
Elevator/ Elevator Structure West side	1	EA	$700,000.00	$700,000.00
8' Chain link fence	1	LS	$25,000.00	$25,000.00
MSE Retaining Wall	2,100	SF	$100.00	$210,000.00
4" Concrete Flat work	1,000	SF	$15.00	$15,000.00
Granular Backfill Borrow	250	CY	$95.00	$23,750.00
Power Pole Relocation	1	LS	$150,000.00	$150,000.00
Lighting/ Cameras	1	LS	$40,000.00	$40,000.00
Total + 30% Contingency				**$3,435,321.67**

Optional Stair and Elevator on Both sides				
Stair Structure East	1	EA	$150,000.00	$150,000.00
Elevator Structure East	1	EA	$400,000.00	$400,000.00
Elevator East	1	EA	$300,000.00	$300,000.00
Total				**$850,000.00**

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