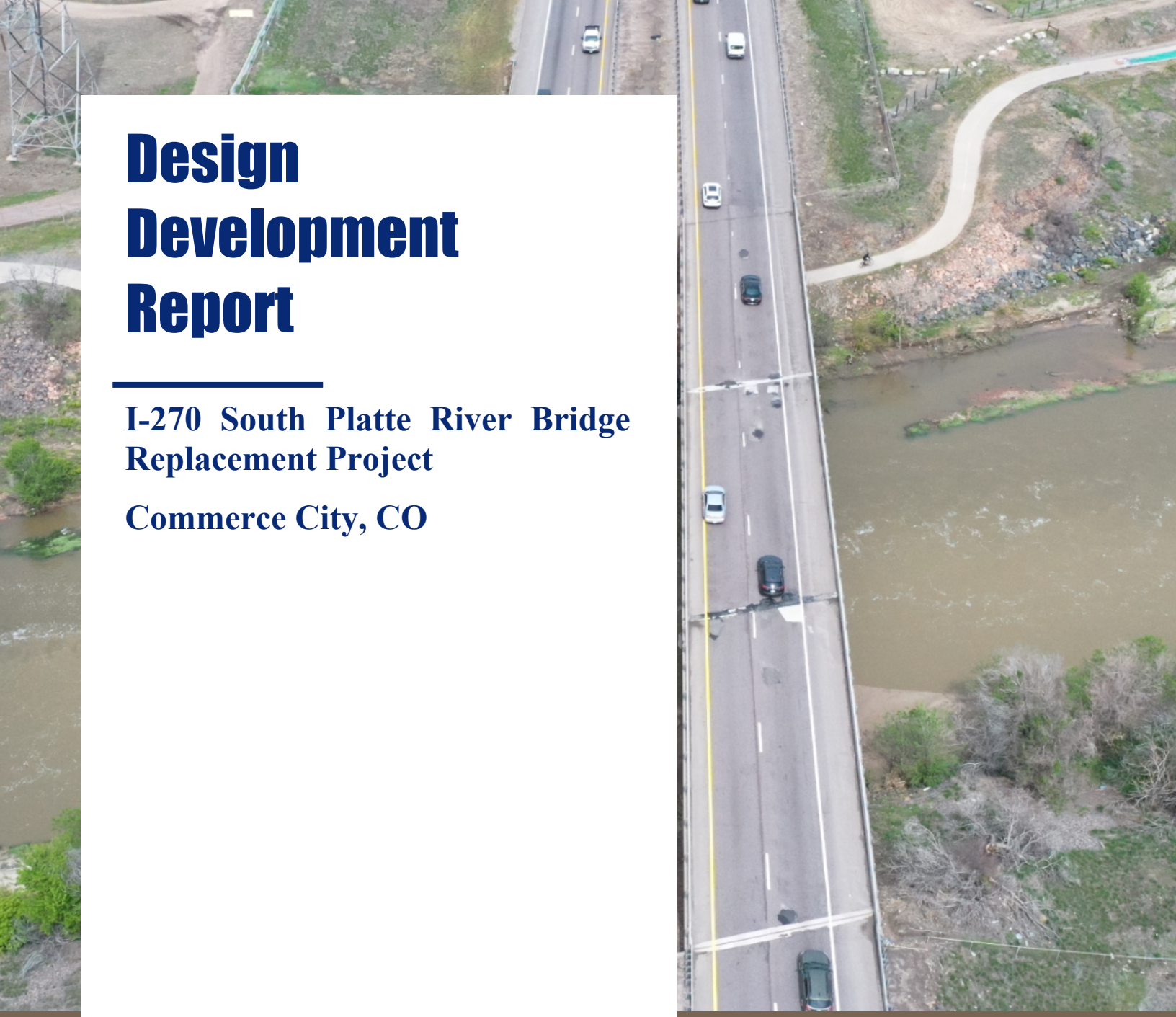

Design Development Report

I-270 South Platte River Bridge Replacement Project

Commerce City, CO

APRIL 25TH, 2024

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CVEN 4899 Senior Capstone Project
University of Colorado Boulder

ALPINE ENGINEERING

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Section 1: Introduction and Decision Matrix

1.1 Design Summary

Our final design solution is a 3-span bridge, with piers unevenly spaced, and a 2-inch isolation gap between each direction of traffic. This design solution will include 6 total piers, with 3 at each span end. Drilled shaft foundations will support each pier. Construction will take place in multiple phases, with phase zero and phase four being site mobilization and demobilization (section 5.10):

- Phase 1: Sawcut existing bridge and infill center with permanent structure.
- Phase 2: Demolish existing westbound bridge and build permanent structure.
- Phase 3: Demolish existing eastbound bridge and build permanent structure.

Our prospicient solution allows for convenient lane additions in both the eastbound and westbound directions in the future. Aside from the bridge structure itself, detailed water resources and geotechnical analyses were performed and features to protect the site from hazards such as flooding and landslides are proposed in this report. Some geotechnical and water resources elements of our design include:

- A mechanically stabilized retaining wall to protect the bike path from erosion and unsafe slopes.
- Riprap around the piers to reduce scour of the riverbed.
- Foundation elements are designed to withstand 10-100 year flood events.

The key elements of our bridge are

- 3-span
- Reinforced Concrete Bridge
- 2-inch isolation gap between eastbound and westbound traffic
- Total Estimated Cost ~ $34,408,000
- Estimated Start Date ~ 10/24/2024
- Estimated End Date ~ 6/22/2026

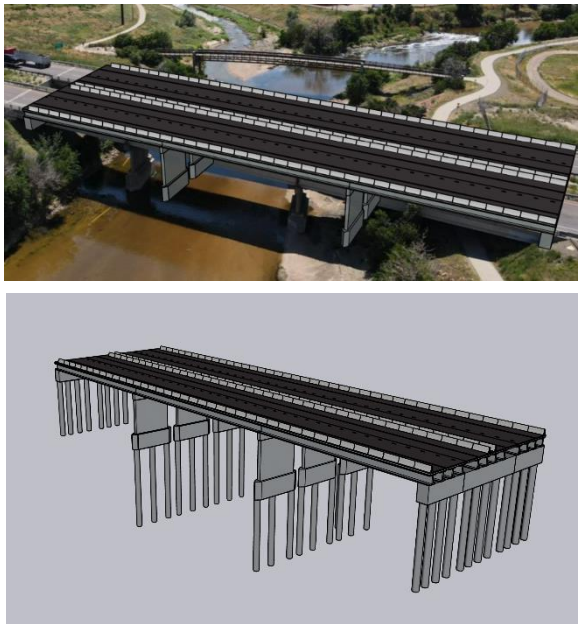
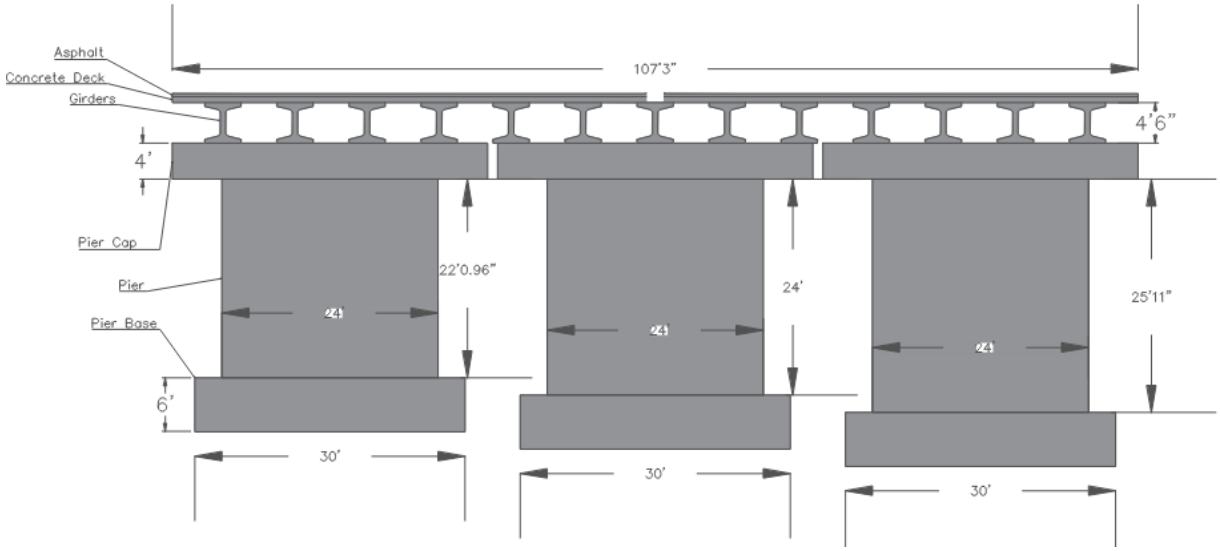
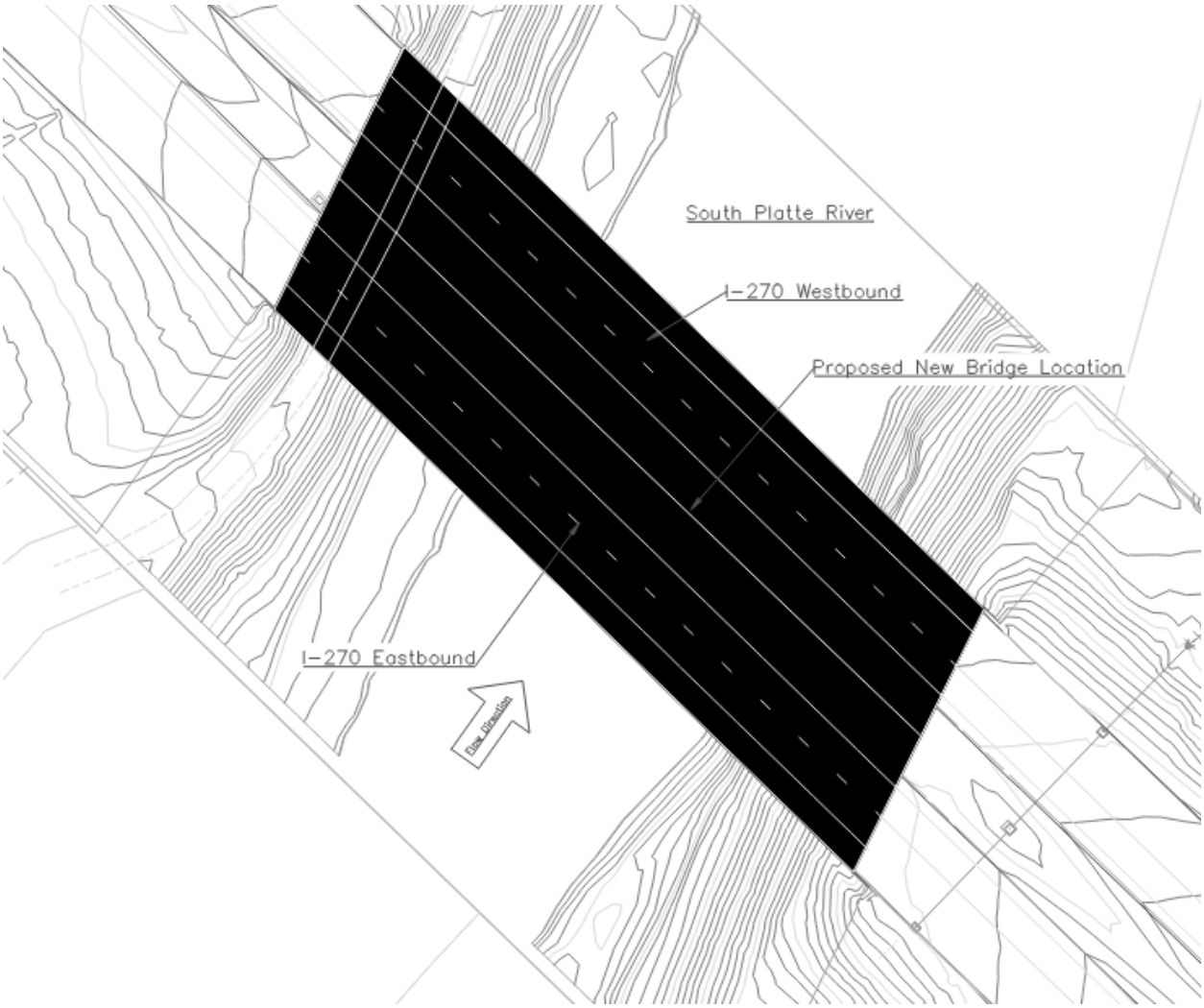


Figure 1.1 Bridge Model


Asphalt
Concrete Deck
Girders
107'3"
4'6"
4'
Pier Cap
Pier
Pier Base
22'0.96"
24'
25'11"
6'
30'
30'
30'


South Platte River
I-270 Westbound
Proposed New Bridge Location
I-270 Eastbound
Flow direction

1.2 Project Background

I-270 is a major highway in Commerce City, CO serving as a connection between I-25 and I-70. The bridge transports around 100,000 cars per day and avoids downtown Denver's traffic. I-270 crosses the South Platte River via separate Eastbound and Westbound bridges constructed in 1968. The I-270 crossing has been in service for over 50 years and requires frequent emergency repairs. Emergency repairs are costly, forcing CDOT to spend money to make rushed improvements and requiring lane closures which prevent the efficient flow of traffic.

Figure 1.2 Existing Bridge

1.3 Problem Statement and Constraints

The Colorado Department of Transportation (CDOT) is looking to replace the Eastbound and Westbound bridges crossing the South Platte River. CDOT is searching for an integrated design/build firm to replace and decommission all deteriorating structures, address concerns of the local community, reduce project costs, and limit impacts to the traveling public. Public health, safety, and welfare is paramount in this project, and the bridge must be replaced with the consideration of social, environmental, and economic factors. The design/build firm selected for this project must create an efficient and sustainable design, which cuts costs through innovative construction methods and materials sourcing.

CDOT is searching for a firm to provide structural design, geotechnical design, water resources, drainage design, and detailed construction plans. All designs must align with CDOT's approved practices and the new bridge should have a design life of 75 years. Disturbance to the surrounding community and the impact on the natural environment must be minimized.

There are several significant constraints in this project including, but not limited to:

- Two lanes open in each direction throughout construction
- Pedestrian/bike path along the river maintained
- $60M budget
- Keep flood plain elevations the same
- Dust must be controlled at all times
- Maintain habitats and vegetation (special consideration for swallows and native trees)
- Maintain water and air quality,

This project is crucial in improving I-270, enhancing the quality of the corridor for commuters and saving Coloradoan's money while architecting a more sustainable future for infrastructure in Colorado.

1.4 Design Alternatives & Final Selection

Two structural design options were proposed originally. The first design alternative was a 3-span bridge which was all one 4-lane single structure. The second design alternative was a 2-span bridge, however in this bridge, there was a 2-inch expansion gap between each direction of traffic, with the center-most pier being connected to both the East and Westbound lanes.

The final selection was based on multiple factors that were placed into a decision matrix. After analyzing these factors, it noted that the 3-span bridge choice was overall the better option, however, the 2-inch expansion gap was a benefit of the 2-span solution, and it could easily be incorporated into the 3-span alternative.

	Alternative Designs	Life Cycle Cost	Constructability	Time to Complete	Sustainability	Settlement	Public/Environmental Health	Safety	Impact on Traffic	Matching Existing Conditions	Force of Water on Piers	Average Score
Structure	(weight)	9	10	10	10	8	10	10	8	9	4	
	3-Span	9	9	8	9	8	9	10	9	9	8	78
	2-Span	10	7	8	9	9	9	10	9	9	4	76.1

Figure 1.3 Design Matrix

After considering the decision matrix and combining elements from both our first and second design alternatives, we selected a 3-span bridge with a 2-inch isolation gap between each direction of traffic.

The process which describes how the score for each factor was arrived at is expanded upon in more depth in Section 4: Structural Design.

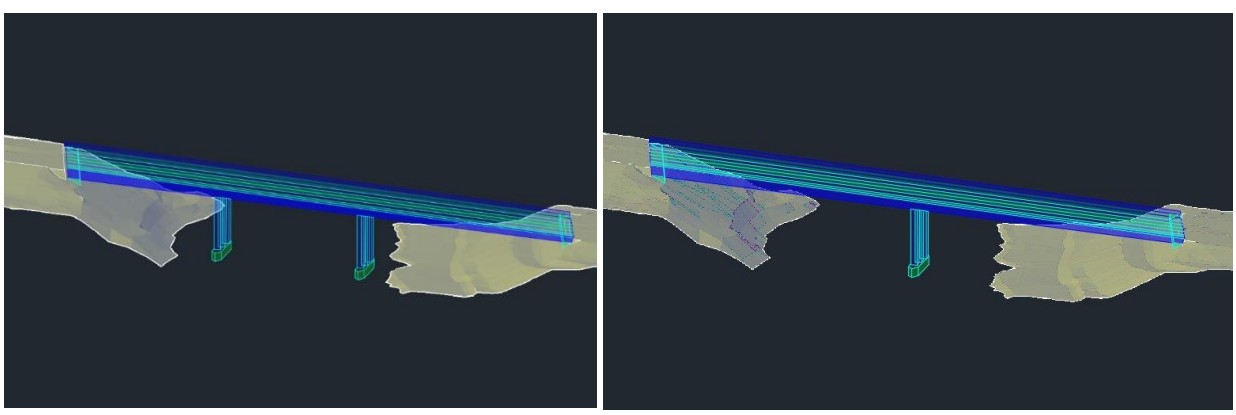


Figure 1.4 Conceptual Bridge Options

1.5 Resilience and Sustainability

The human and environmental impact of the bridge cannot be overlooked in its design and construction. Innovation, resilience, sustainability, and community building are incorporated into the core of the project. The bridge was structurally designed using CDOT's Tier 1 bridge strategies, developed to ensure the resilience and longevity of new bridges (CDOT, 2020).

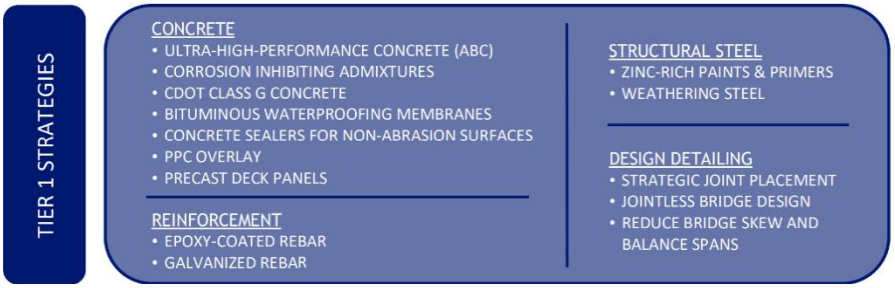


Figure 1.5 CDOT Tier 1 Bridge Strategies

Not only does a resilient bridge cut costs in the long term, but reducing the need for repairs also lowers lane closures and keeps traffic flowing, thus reducing accidents, lowering emissions, and supporting the movement of goods and people. Sustainability for each aspect of the bridge design (structural, geotechnical, etc) is discussed in detail in the body of the report. The geotechnical design incorporates MSE retaining walls and geogrids rather than relying on concrete structural capacity to decrease the carbon footprint of the bridge. The impact on the river flow and conditions was analyzed by our water resources engineer to inform design decisions. Some additional recommendations for CDOT to consider include alternatives to cementitious material, implementing a bridge health monitoring system and building a wildlife crossing beneath the bridge. Although structural steel was not used in this bridge, materials that have minimal environmental impact in the production process, such as class D concrete, were selected for this project.

Figure 1.6 Sustainable Geogrids (left) and Riprap to Prevent Scour and Erosion (right)

Sustainability does not only mean maintaining river quality and flow, preserving the swallow habitat and native trees on the site, preventing soil erosion, and reducing air pollution—all of which our design accomplishes– but also strengthening the community around the bridge. In 2020, Adams County began a series of projects on the South Platte River pedestrian and commuter trail

including “Murals for Hope” and “Love your Trails”. These projects employed several local artists from the culturally diverse neighborhoods of Commerce City to paint exterior murals on the path and surrounding infrastructure. One of the project founders and muralists who painted the path was Commerce City resident Sofi Ramirez (pictured left). Our proposed bridge will be named after Ramirez for her contributions to the local community and we would like to invite the local artists who participated in the Murals for Hope project to paint the side and bottom of our bridge when construction is complete.

Figure 1.6 Sofi Ramirez Painting the Trail

This will integrate the bridge into the existing project and surrounding community— which has historically been disproportionately impacted by the effects of poor infrastructure. Transforming it from another highway infrastructure project to a spot that bikers look forward to passing, and the surrounding community has a legitimate stake in. More detailed descriptions of sustainability as it relates to design and construction are included in the subsequent section of the report.

Section 2: Water Resources

2.1 Watershed Description

The I-270 South Platte River bridge is a part of the Big Dry Creek-South Platte River Watershed. The total area of (CDOT, 2019) this watershed is 4070.63 square miles. The primary land types on this watershed are evergreen forest which covers 33.55% of the area, and grasslands that cover 30.00% of the area (Watershed Analysis, 2024).

Type	NLCD Code	Area (sq mi)	Coverage (%)
Open Water	11	27.46	0.67
Perennial Ice/Snow	12	2.34	0.06
Developed, Open Space	21	165.77	4.07
Developed, Low Intensity	22	152.72	3.75
Developed, Medium Intensity	23	134.89	3.31
Developed, High Intensity	24	57.25	1.41
Barren Land (Rock/Sand/Clay)	31	19.11	0.47
Deciduous Forest	41	122.17	3
Evergreen Forest	42	1365.59	33.55
Mixed Forest	43	13.86	0.34
Shrub/Scrub	52	618.89	15.2
Grassland/Herbaceous	71	1221.29	30
Pasture/Hay	81	1.54	0.04
Cultivated Crops	82	13.54	0.33
Woody Wetlands	90	55.77	1.37
Emergent Herbaceous Wetlands	95	98.46	2.42
Total		4070.63	100

Table 2.1: Big Dry Creek-South Platte River Watershed Land Use (Watershed Analysis, 2024)

2.2 Flood Flow Assessment

Flooding Source & Location	Drainage Area (sq mi)	Peak Discharges (cfs)			
		10-percent	2-percent	1-percent	0.2-percent
South Platte River, at confluence with Clear Creek	4572.0	12700	27500	37600	66710

Table 2.2: Discharge Data (FEMA, 2021)

Clear Creek is the closest confluence to the I-270 bridge over the South Platte River. Using the data from FEMA, the 10-year flood flow is 12700 CFS, the 50-year flood flow is 27500 CFS, the 100-year flood flow is 37600 CFS, and the 500-year flood flow is 66710 CFS.

2.3 Geometric Data

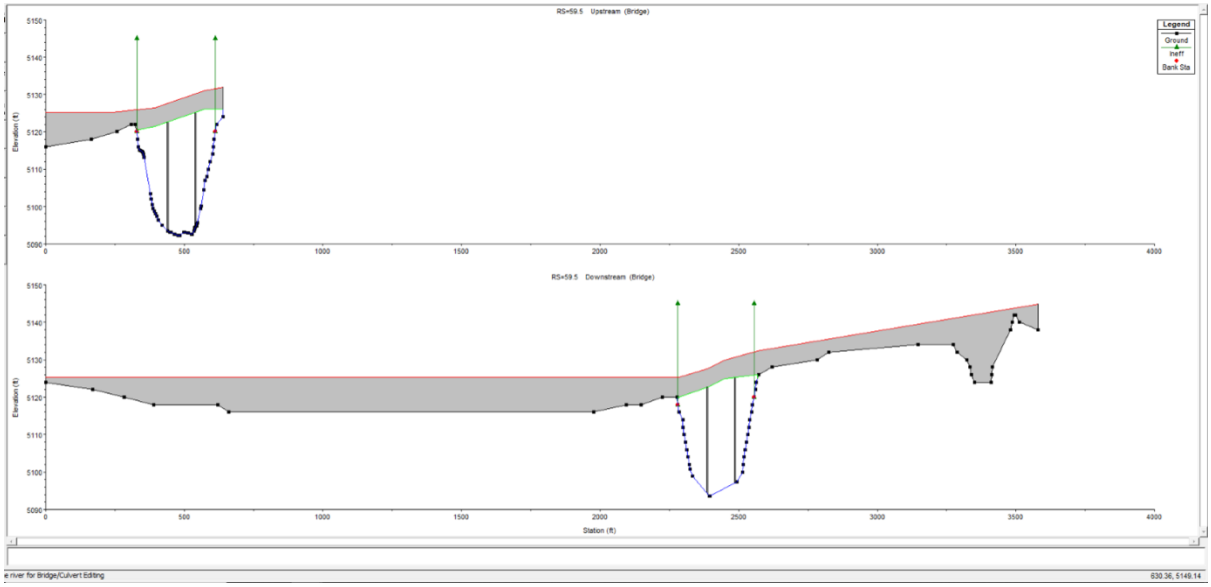


Figure 2.1: Corrected Bridge Geometry

HEC-RAS is used to smoothen out the river cross section and correct the mistakes in the bridge dimensions such as the piers being 0.225 ft when they are from 2 ft at the top to a 6 ft base. The river cross section previously had very sharp edges and inaccuracies that made the flood elevations have a 4-5 ft difference with the FEMA SRP. After smoothening out the cross sections, the flood elevations are within 0.03-0.37 ft of the FEMA SRP.

2.4 Water Surface Profiles

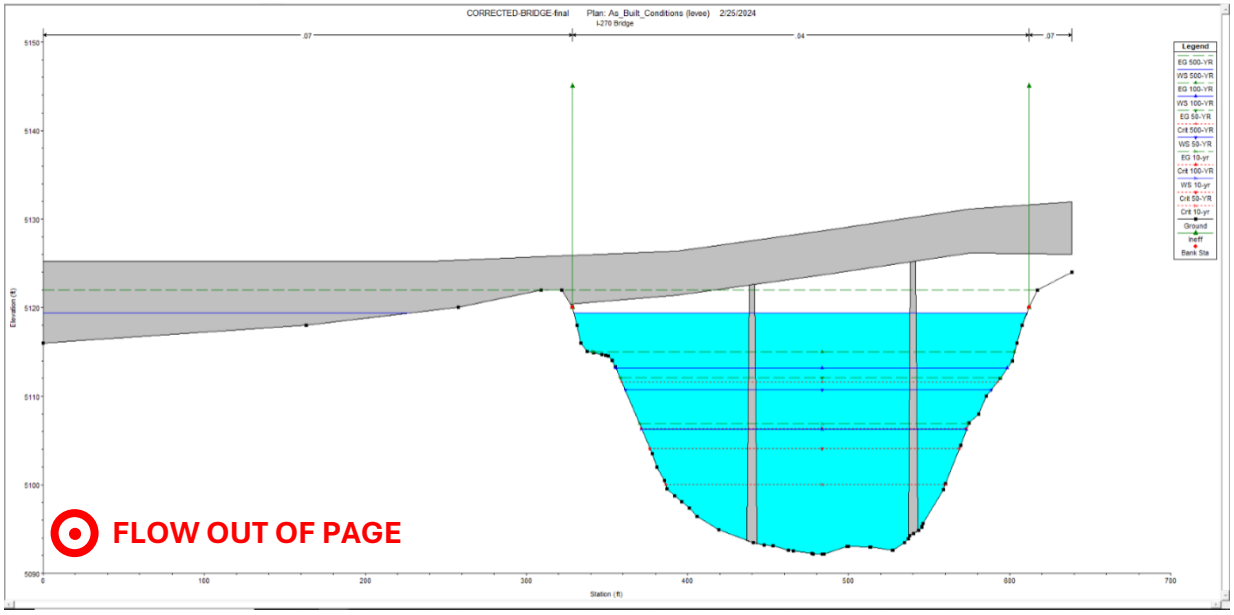


Figure 2.4: Water surface profile of the currently built I-270 bridge.

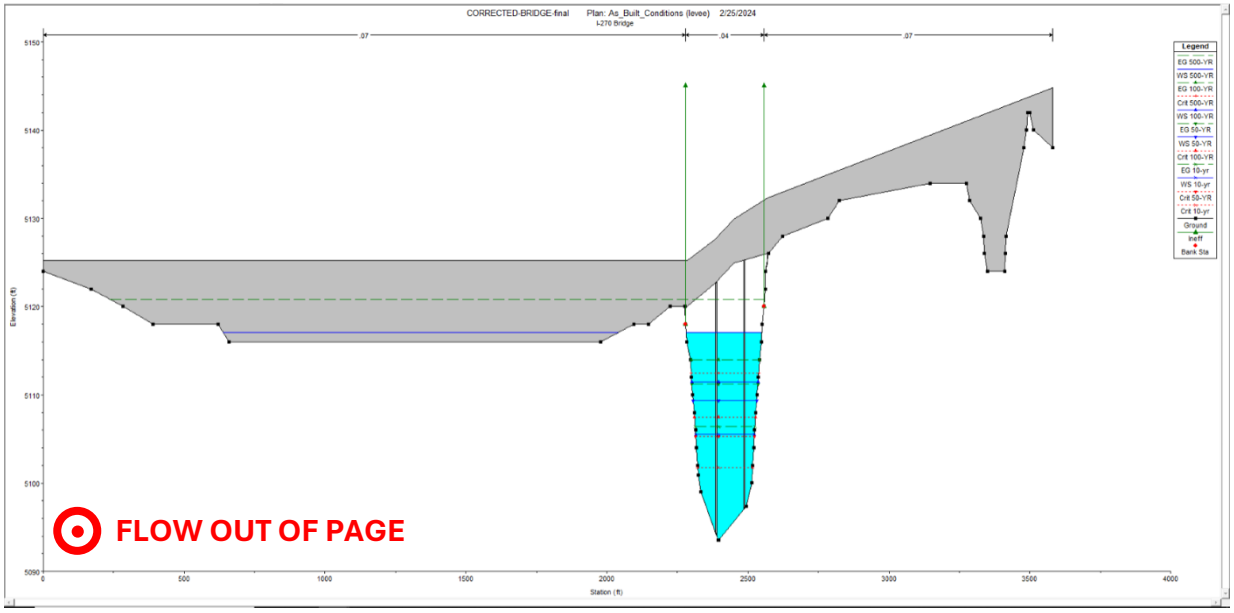


Figure 2.5: Water surface profile extended station view of the I-270 bridge.

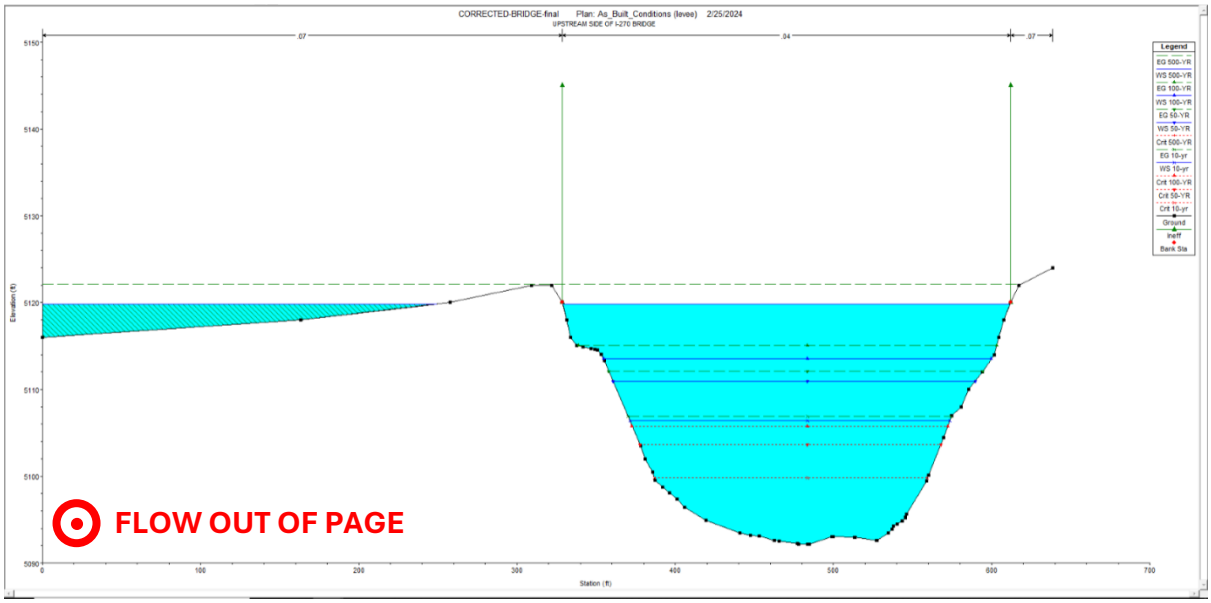


Figure 2.6: Water surface profile upstream of the I-270 bridge.

These are the surface profiles upstream of the bridge, at the bridge, and downstream of the bridge with the floodplain elevations. The floodplain elevations are obtained by doing a steady-flow analysis.

2.5 FEMA and HEC-RAS Model Comparison

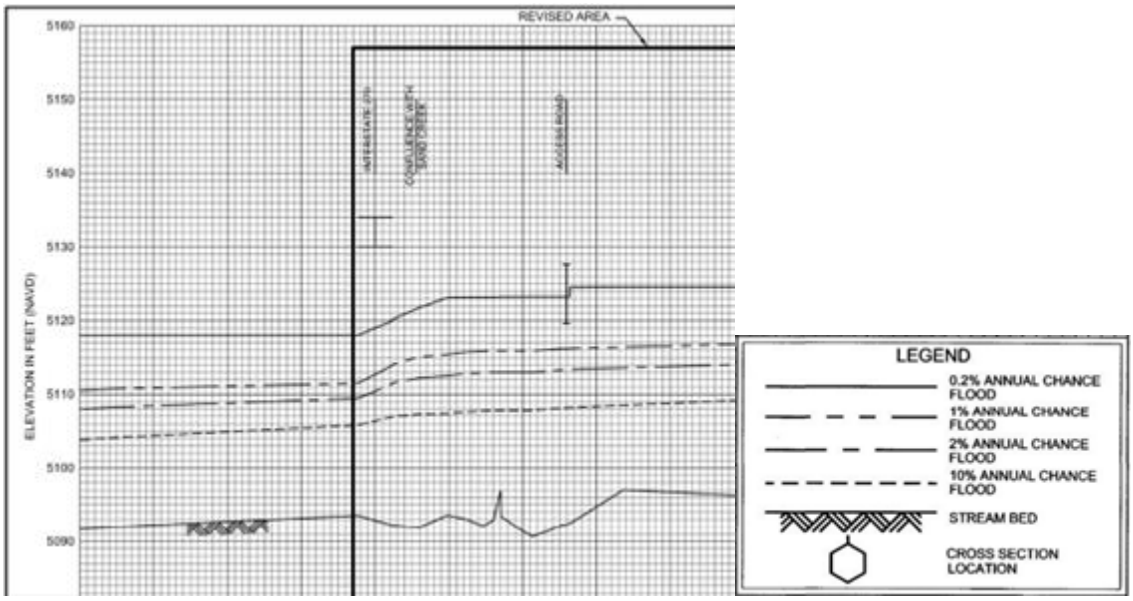


Figure 2.7: Flood Profiles of the South Platte River & Legend (FEMA, 2021).

	FEMA SRP Elev. (ft)	HEC-RAS Model Elev. (ft)	Difference (ft)
Ground	5092.75	5092.17	0.58
10-year	5106.25	5106.28	-0.03
50-year	5110.5	5110.72	-0.22
100-year	5113	5113.19	-0.19
500-year	5119	5119.37	-0.37

Table 2.3: FEMA vs. HEC-RAS Elev. Comparison.

2.6 Freeboard

Floodplain	HEC-RAS Model Elev. (ft)	Freeboard (ft)
10-year	5106.28	14.1
50-year	5110.72	9.66
100-year	5113.19	7.19
500-year	5119.37	1.01

Table 2.4: Freeboard for the different floodplain elevations.

The freeboard is calculated by finding the distance between the lowest elevation of the underside of the bridge's structure (5120.38 ft), and the floodplain elevations.

2.7 Sustainability in Water Resources

When designing the replacement bridge's piers, there will be lots of consideration as to how the newly designed piers affect the river's aquatic habitat. Minimizing disturbance to the fragile ecosystem of the river is a main priority. For our 3-span bridge design, there was coordination between the water resources and geotechnical engineering associates to optimize the river's flow and reduce sedimentation and erosion by assessing different methods of channel erosion protection for the structures and riverbanks. In addition, the reduced pier base widths improve the flood water elevations at the bridge and on the upstream side of the bridge, which reduces erosion. There has also been consideration for riprap along the piers to keep the river's cross section unaffected by scour.

2.8 Hydraulic Analysis of Alternative Design (2-Span)

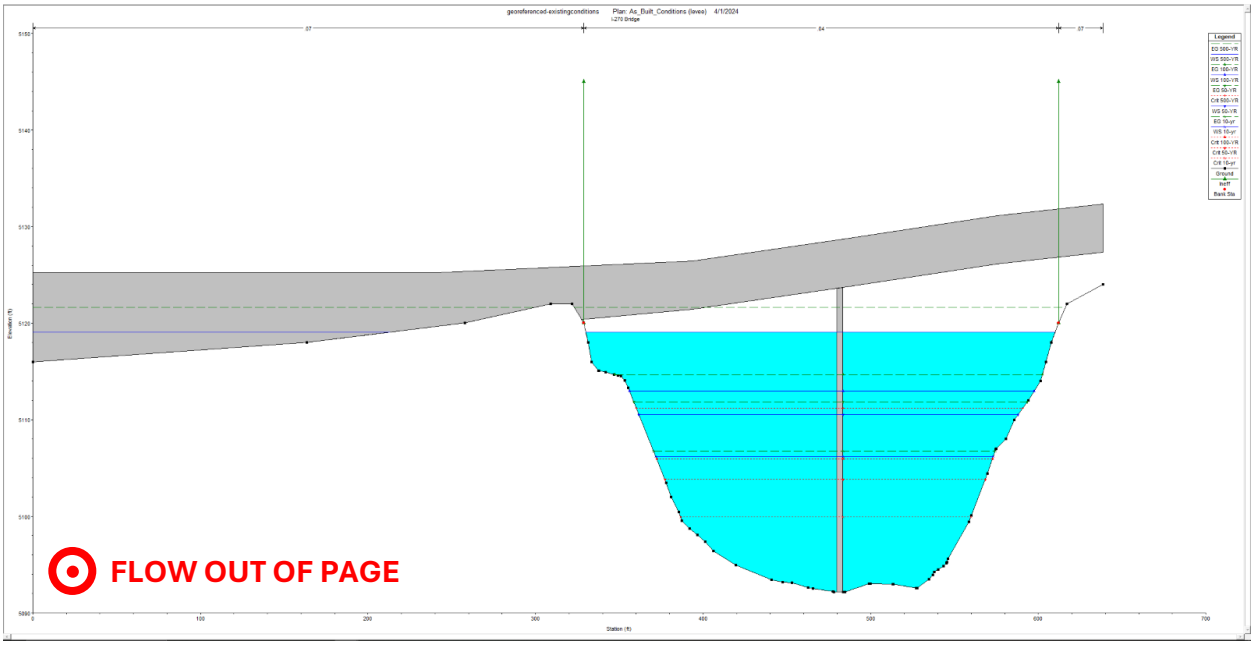


Figure 2.8: Water surface profile of the 2-span bridge

Floodplain	HEC-RAS Model Elev. (ft)	Difference w/ Current I-270 Elev. (ft)
10-year	5106.20	-0.08
50-year	5110.54	-0.18
100-year	5112.97	-0.22
500-year	5119.06	-0.31

Table 2.5: Water surface elevations of the 2-span bridge

From the water surface elevations obtained from the HEC-RAS model of the 2-span bridge design, there is a decrease in all water surface elevations from the current bridge elevations.

Floodplain	HEC-RAS Model Elev. (ft)	Freeboard (ft)
10-year	5106.20	14.57
50-year	5110.54	10.23
100-year	5112.97	7.80
500-year	5119.06	1.71

Table 2.6: Freeboard for the 2-span bridge

The low chord of the 2-span bridge is located at an elevation of 5120.77 ft.

2.9 Hydraulic Analysis of Chosen Design (3-Span)

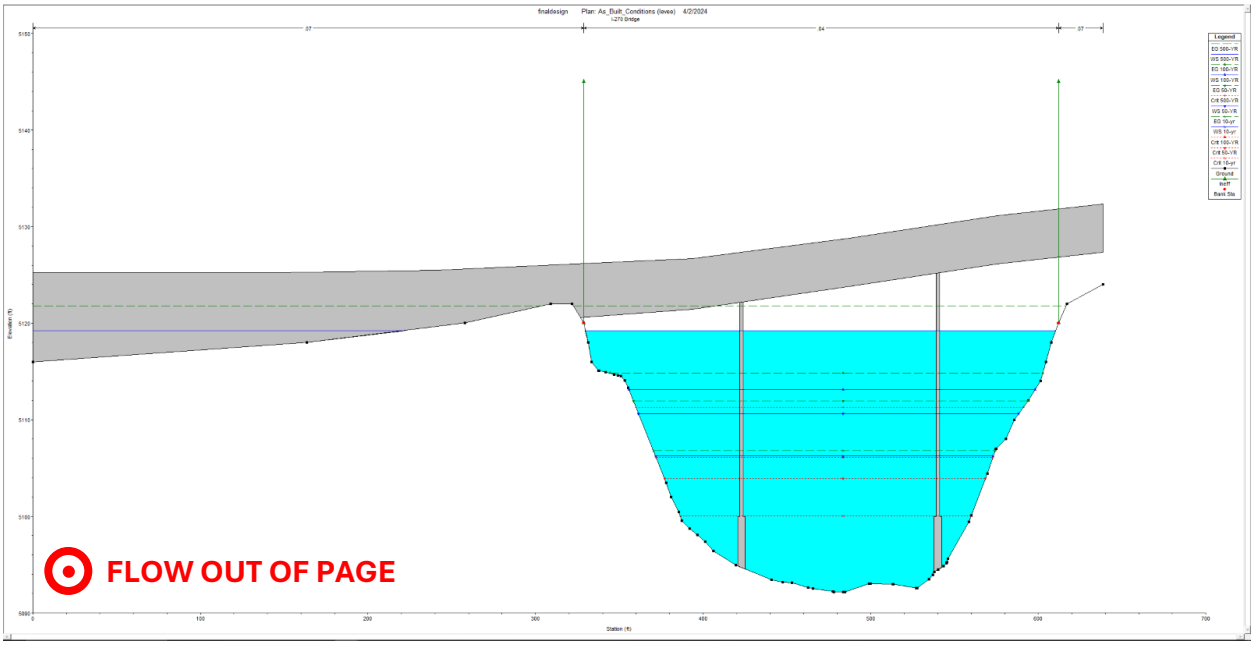


Figure 2.9: Water surface profile of the 3-span bridge

Floodplain	HEC-RAS Model Elev. (ft)	Difference w/ Current I-270 Elev. (ft)
10-year	5106.25	-0.03
50-year	5110.64	-0.08
100-year	5113.10	-0.09
500-year	5119.24	-0.13

Table 2.7: Water surface elevations of the 3-span bridge

Floodplain	HEC-RAS Model Elev. (ft)	Freeboard (ft)
10-year	5106.25	14.33
50-year	5110.64	9.94
100-year	5113.10	7.48
500-year	5119.24	1.34

Table 2.8: Freeboard for the 3-span bridge

The low chord of the 3-span bridge is located at an elevation of 5120.58 ft.

2.10 Scour Analysis for 3-Span Bridge

100-yr Design Storm:

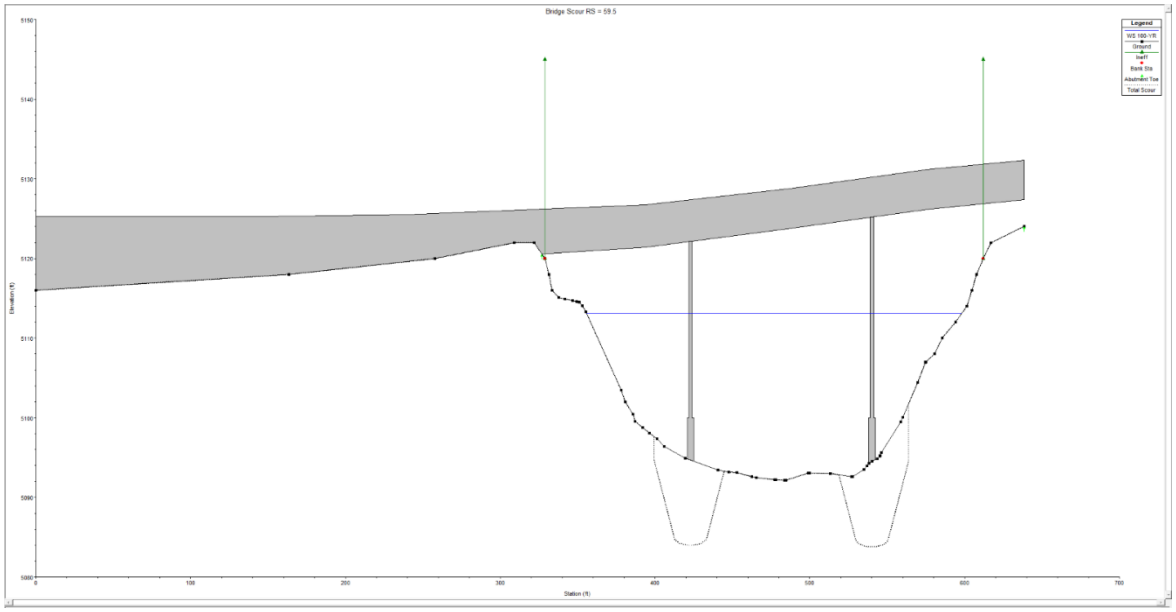


Figure 2.10: Local scour at the piers for 100-yr design storm

- *Scour depth for 100-yr design storm = 10.70 ft*

500-yr Storm:

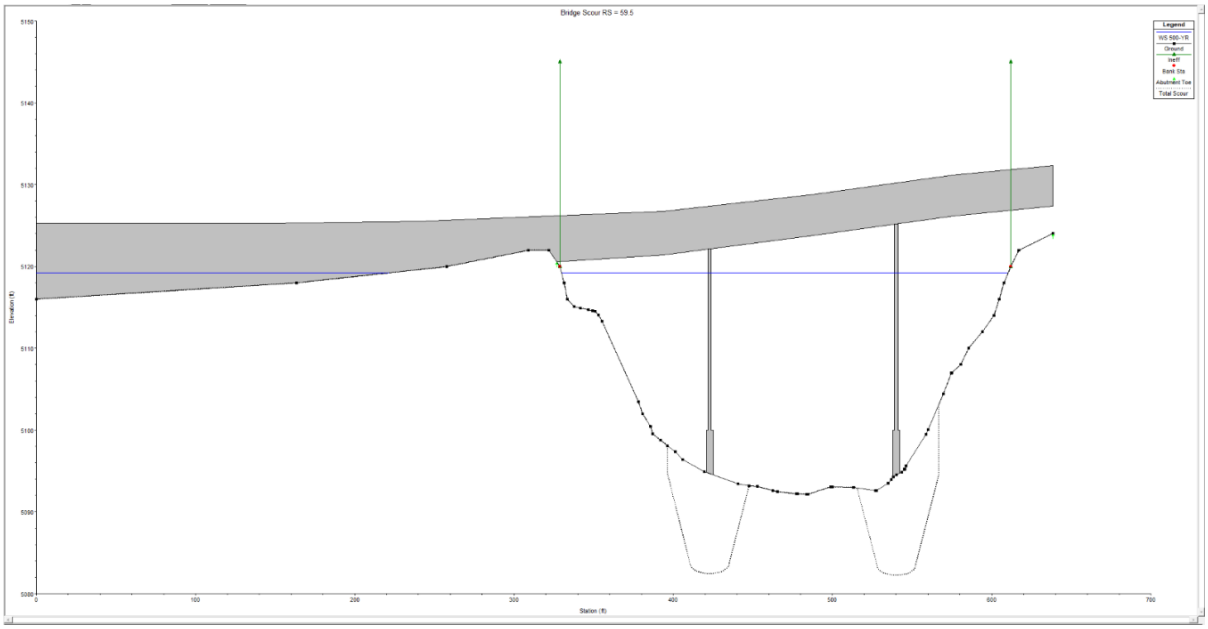


Figure 2.11: Local scour at the piers for 500-yr storm

- *Scour depth for the 500-yr storm = 12.22 ft*

Scour analysis was performed using the CSU equation with a sharp nose pier shape. Refer to Appendix A for further details.

The scour depth for both the 100-yr and 500-yr storms nearly reaches the bedrock. The minimum scour elevations of the 100-yr and 500-yr storms are 5083.79 ft and 5082.26 ft respectively. From the sub-surface profile of the river cross section, the bedrock elevation near the piers is at an elevation of around 5080 ft. The piers must be anchored to the bedrock to prevent failure due to scour.

2.11 Bridge Design Impacts on Channel Hydraulics

The 3-span bridge design contains piers with a 4.5 ft base and 2 ft wide piers and pier caps. Comparing this to the current I-270 bridge design which has piers with a 6 ft base and is 2 ft wide to the top, there is a decrease across all flood elevations ranging from a 0.03 ft decrease to a 0.13 ft decrease. There also are increases in the freeboard for all flood elevations, and since the freeboard for the 500-year floodplain is 1.34 ft, there would not be an incipient overtopping event (when the flood's water surface elevation occurs over the bridge). Furthermore, the 500-yr storm induces a scour depth of 12.22 ft which nearly reaches bedrock. Riprap must be placed around the piers to reduce scour. A HEC-RAS hydraulic design computation shows that the D_{50} (median rip rap stone size) is 11.033 in. Using standard CDOT gradations for rock riprap, the typical stone dimensions for riprap placed around the piers will be 21 in with a typical stone weight of 440 lbs. The layer thickness of the riprap will be twice D_{50}, so 22 in (or 1.83 ft). Moving on, channel hydraulics upstream of the bridge were also positively affected with decreases in water surface elevations (CDOT, 2019). These decreases could mainly be attributed to the decrease in the width of the pier base and the spacing of the piers where they are closer to the riverbanks.

2.12 Drag Force on Piers

The drag force due to water flowing through the pier was obtained by using equation 3.52 from the Federal Highway Administration's Hydraulic Design of Safe Bridges. The drag force was used by the Structural Engineering associate to check flexural stresses on the piers. The total drag force of the river's flow on the piers is 336 kips.

2.13 3-D Schematics of Flood Extent

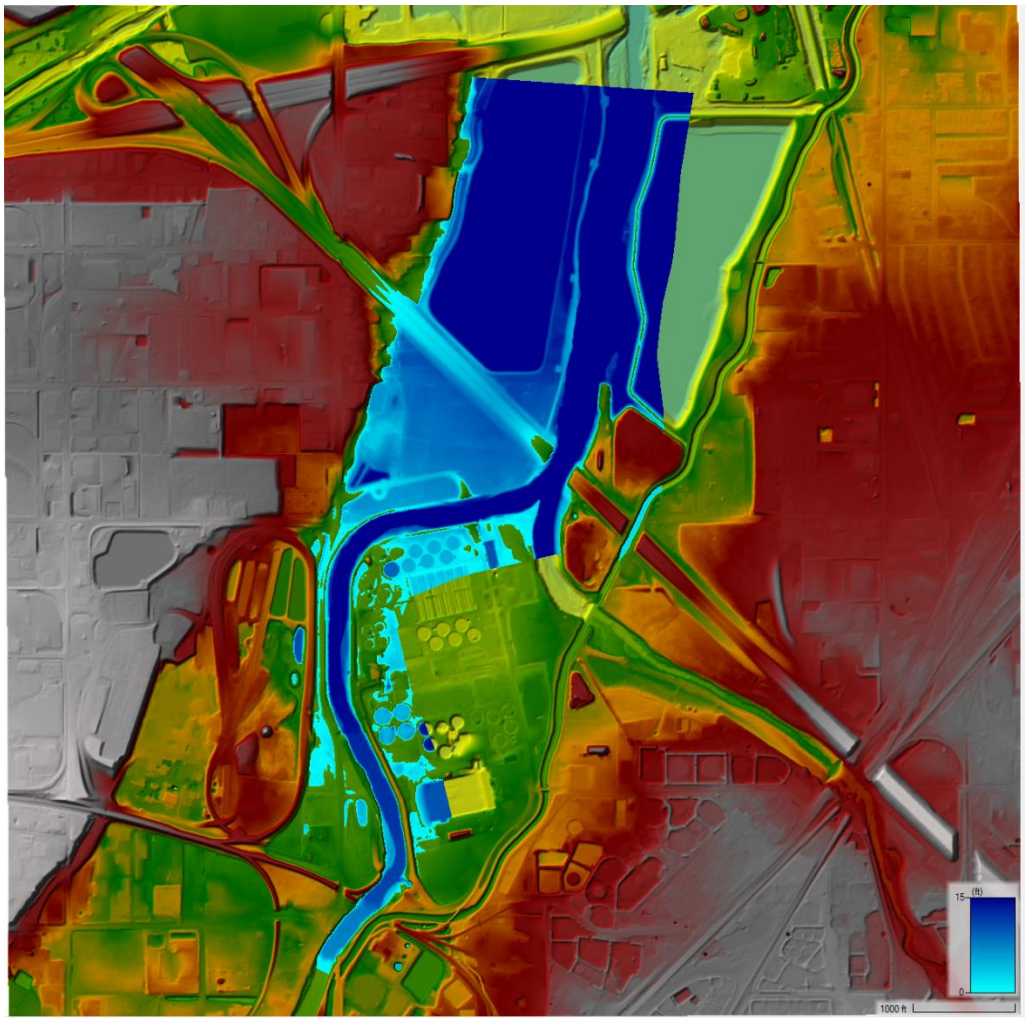


Figure 2.12: Flood extent schematics w/legend that shows depth of water.

This heat map shows the water depth using the FEMA peak discharge data for the 10-year, 50-year, 100-year, and 500-year floodplains, where the results of the steady flow analysis are represented as depth. To create the heatmap, LIDAR data from the Colorado Water Conservation Board was imported into the RAS Mapper feature in HEC-RAS and a new georeferenced hydraulic model of the river and bridge. The two dark blue areas on the left and right of the South Platte River are the Cat Lake and Miller Reservoir respectively. For the Miller Reservoir on the right, the sharp edges are the ends of the cross sections of the hydraulic model.

2.14 Deck Drainage

The Federal Highway Administration's HEC 21 Design of Bridge Deck Drainage was used to determine the drainage needs of our bridge design. The first item of concern was the rainfall intensity on the bridge deck. To find the rainfall intensity, an iterative method was used to match the sum of the overflow (t_o) and gutter (t_g) times of concentration to the duration of the rainfall intensity. In the final iteration a t_o of 1.09 min and a t_g of 9.44 min were found. This gives a rainfall intensity of 6.20 in/hr for the 100-yr design storm with a 10 min duration of rainfall intensity, where this intensity was obtained from the NOAA tabular data of rainfall intensities for Denver. Using this intensity, the full gutter flow is equal to 10.1 ft^3/s. Finally, the inlet spacing (L_0) was calculated to be 2190 ft which is longer than the total length of the bridge. Therefore, no inlets will be required on the bridge decks.

Since no inlets are required, there still needs to be a collection system off the bridge's structure. To determine pipe sizing for the bridge end treatments, the Spread Plus Rational Method was used to find the peak runoff rate, which was determined to be 1.41 ft^3/s. For this peak runoff, cast-iron pipes with an 8-in diameter are needed to be able to handle the 100-yr design storm. These pipes will then deliver the precipitation to the existing drainage ditches used for the current I-270 bridge. All calculation details, tables, and data used are located in Appendix A. Structural considerations when designing deck drainage include minimizing the use of inlets since they could interfere with the structural continuity of bridge decks. In our case, there are no inlets located on the decks, so there are no effects on the structural integrity of the bridge decks. Another structural consideration is ensuring that the water does not come in contact with exposed structural members and cause corrosion.

Drainage Design Summary:

- Asphalt at a 3% grade.
- No inlets required on bridge deck.
- 8-in diameter cast iron pipes for the collection system.
- Rainfall transported to existing drainage ditch.

Section 3: Geotechnical Analysis and Schematic Design

3.1 Existing Geotechnical Conditions

The project site is located at the I-270 - South Platte River crossing in Commerce City, Colorado. The broad geologic conditions are defined by the presence of a large amount of fill soil underneath I-270, as well as the Broadway Alluvium and Post-Piney Creek Alluvium formations to the East and West of the site, respectively (Yeh and Associates, Inc, 2022, "Preliminary Geotechnical Report").

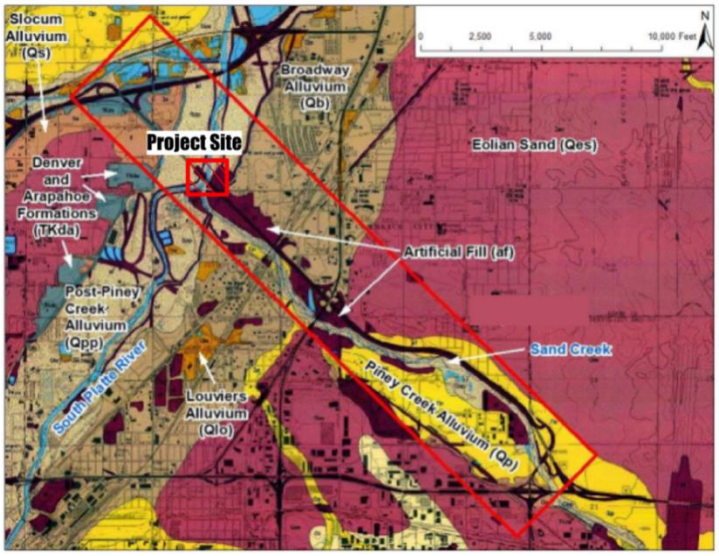


Figure 3.1 Regional Geologic Map, Adapted from the Geological Preliminary Geotechnical Investigation Report

Several borings were taken at or near the project site, which informed the geological cross section and geotechnical model of the site, the borings used in analysis are B-2, B-3, B-9, YA-B-01, YA-B-02, B-6, and YA-PV-110. Verbiage such as "borings" or "the borings" later in the report is referring to these borings.

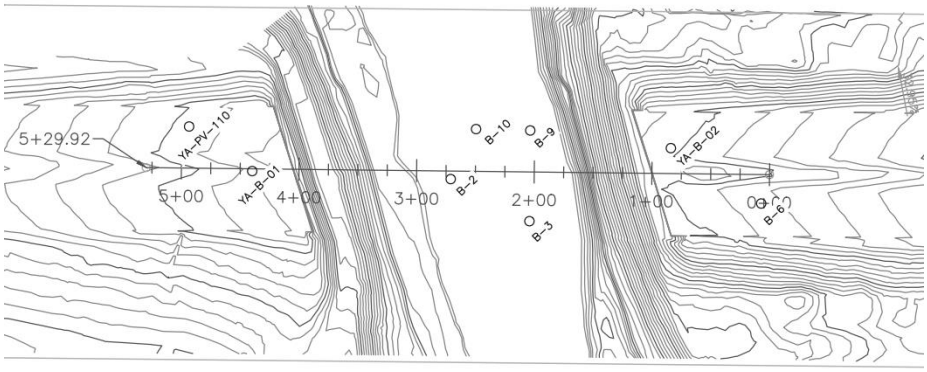


Figure 3.2 Boring Map Plan View with Stations

3.1.1 Cross Section of Existing Ground Conditions

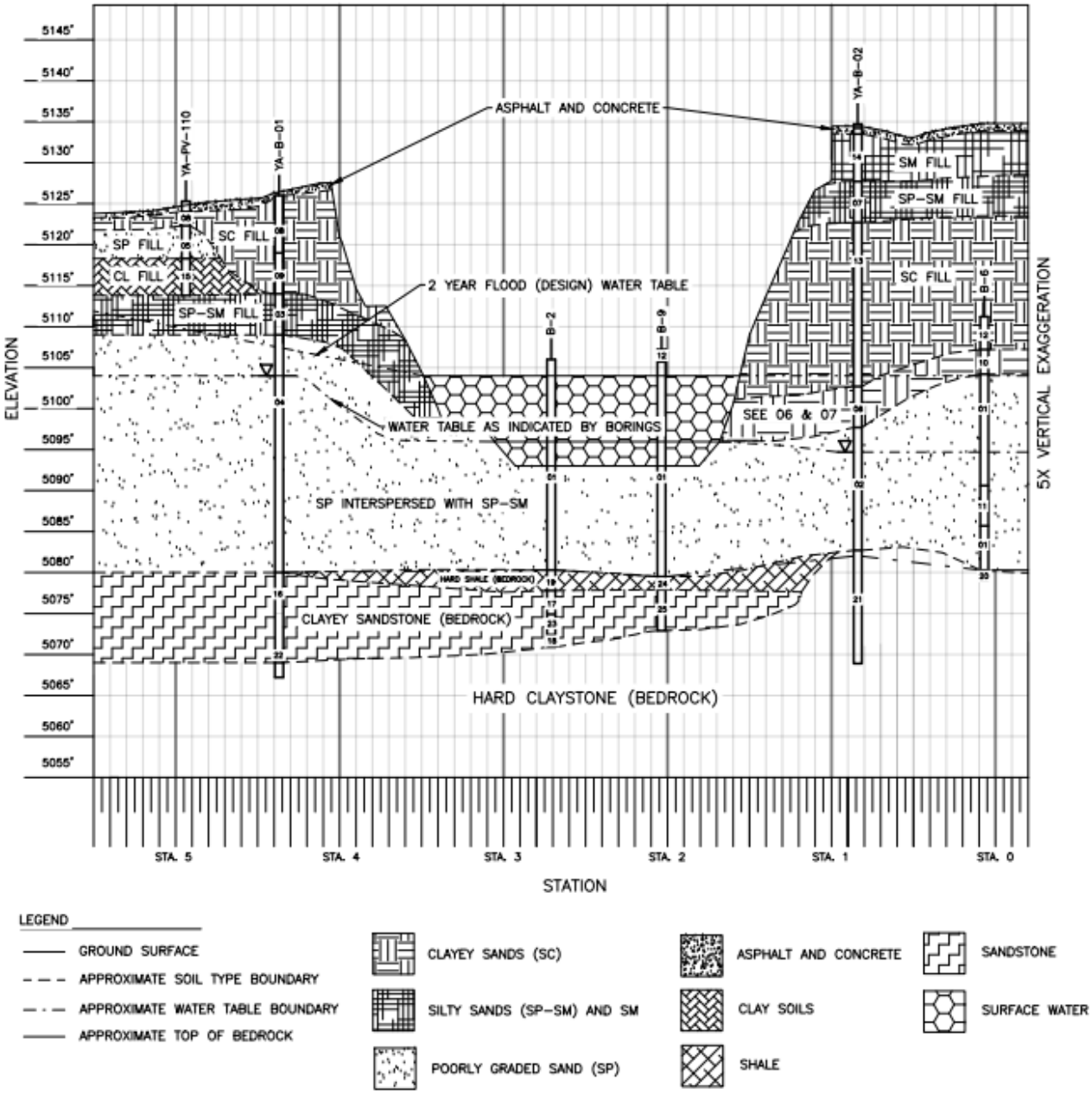


Figure 3.3 Soil Cross Section, Legend, and Table

NUMBER	SOIL SITE GROUP	DESCRIPTION FROM BORING
01	03	SAND, GRAVELLY, FEW COBBLES, TAN (SIMILAR TO 1–A)
02	02	POORLY GRADED SAND WITH SILT (SP–SM) AND SOME GRAVEL, BROWN, TAN, GRAY, PINK, NO PLASTICITY, MOIST TO WET, DENSE
03	02	POORLY GRADED SAND WITH SILT (SP–SM) AND SOME GRAVEL, BROWN, NO PLASTICITY, MOIST, MEDIUM DENSE
04	03	POORLY GRADED SAND (SP) WITH GRAVEL, BROWN, TAN, PINK WITH RUST, NO PLASTICITY, MOIST TO WET, LOOSE
05	03	POORLY GRADED SAND WITH GRAVEL (SP) (FILL), BROWN–TAN–WHITE, NO PLASTICITY, MOIST, MEDIUM DENSE
06	03	POORLY GRADED SAND WITH GRAVEL (SP) (FILL), BLACK AND GRAY, NO PLASTICITY, MOIST, POSSIBLE FILL, DIFFICULT TO DISTINGUISH
07	02	POORLY GRADED SAND WITH SILT (SP–SM) (FILL) AND SOME GRAVEL, BROWN, TAN, GRAY, PINK, NO PLASTICITY, MOIST TO WET, DENSE
08	01	CLAYEY SAND (SC) WITH SOME GRAVEL (FILL), BROWN, MEDIUM, PLASTICITY, MOIST, MEDIUM DENSE
09	01	CLAYEY SAND (SC) WITH MINOR GRAVEL (FILL), BROWN, LOW PLASTICITY, MOIST, MEDIUM DENSE, DRILLERS NOTED COBBLES 7 TO 9 FEET

NUMBER	SOIL SITE GROUP	DESCRIPTION FROM BORING
10	MISC.	SAND, VERY SILTY, INTERLAYERED WITH CLAY, SOFT, DARK GRAY
11	MISC.	SAND, SILTY, VERY FINE TO MEDIUM, MICACEOUS, TAN
12	MISC.	FILL
13	01	CLAYEY SAND (SC) WITH MINOR GRAVEL (FILL), BROWN, LOW PLASTICITY, MOIST, MEDIUM DENSE
14	02	SILTY SAND WITH GRAVEL (SM) (FILL), BROWN AND TAN, NO PLASTICITY, MOIST, MEDIUM DENSE
15	MISC.	SANDY LEAN CLAY WITH GRAVEL (CL) (FILL), BROWN, LOW PLASTICITY, MOIST, VERY STIFF
16	04	CLAYEY SANDSTONE, BLUE–GRAY TO GRAY, VERY HARD
17	04	SANDSTONE, SILTY, FRIABLE, VERY FINE TO MEDIUM GRAINED, LIGHT GREENISH GRAY
18	04	SANDSTONE, SILTY, FRIABLE VERY FINE TO MEDIUM GRAINED, LIGHT GREENISH GRAY
19	06	SHALE, CLAYEY, SILTY, VERTICALLY, AND HORIZONTALLY FRACTURED, MOIST GREENISH BLUE, HARD

NUMBER	SOIL SITE GROUP	DESCRIPTION FROM BORING
20	06	SHALE SILTY, HIGHLY FRACTURED (SIMILAR TO 1–B)
21	05	CLAYSTONE, GRAY, VERY HARD
22	05	SILTY CLAYSTONE, OLIVE–GRAY, VERY HARD
23	06	SHALE, HARD, DARK GRAY TO BLACK, HIGHLY FRACTURED, PLASTIC
24	06	SHALE, REWORKED, WEATHERED WITH SMALL COAL PARTICLE, AND MICA, GRAY
25	06	SHALE, SILTY, FRACTURED WITH LOCAL, THIN MEDIUM GRAINED SANDSTONE LENSES, DARK GRAY TO BLUE A–7–5(11) (9–B)

Figure 3.3 Soil Cross Section, Legend, and Table

The B-2, B-6, and B-9 borings were taken in 1967 and the ground surface elevation has changed since then. It is assumed that the bedrock's location in borings B-2 and B-9 did not change significantly, but the sandy soil (Number 01 soil in the cross section) eroded to the current riverbed elevation (5092'-5094'). It is also assumed that the soil information from the 1967 borings is still largely accurate. The exact boundaries between soil types are unknown except at the locations of the borings but were approximated in the cross section. The approximate top of bedrock elevation line was created from the depth listed in the Preliminary Geotechnical Report and the bedrock elevations found in the borings. Boring B-3 is not shown in the cross section because it is located at the same station as B-9, but further from the centerline of the bridge.

3.1.2 Site Soil Groupings

The borings revealed several different soil types near the site area and from these soil types, several site-specific soil groups were identified and labeled. These soil groups were formed based on their prevalence in borings around the site. There are sub-groups within the primary soil groups which have slightly differing properties, but due to uncertainty in the exact location of specific soil types, the conservative properties of the soil groups will be reported for use in design. Throughout the report the following soil site groups will be referenced. Note that the numberings of the groups are unrelated to AASHTO soil group numberings. Only borings B-2, B-3, B-9, YA-B-01, and YA-B-02 were used to create the soil groups as these are representative of the soils in which the foundation structure will be constructed. Including extra borings farther from the site (such as the

shallow PV borings found in the Geotechnical Report) would dilute the averages of parameters governing design.

Table 3.1 Soil Groupings

Soil Group 1: Clayey Sand with Gravel (SC) [Fill and Non-Fill]

Subgroup	Location	Description
1.1	Boring YA-B-01: Elev. 5119-5126	Clayey Sand (SC) with some gravel (Fill), brown, medium plasticity, moist, medium dense to very dense
1.2	Boring YA-B-01: Elev. 5114 - 5119 Boring YA-B-02: Elev. 5102.7 - 5122.7	Clayey SAND (SC) with minor gravel (Fill), brown, low plasticity, moist, medium dense
1.3	Boring YA-B-02: Elev. 5127.7 - 5133.5	Silty SAND with gravel (SM) (Fill), brown and tan, no plasticity, moist, medium dense.

Soil Group 2: Poorly Graded Sand with Silt (SP-SM) [Fill and Non-Fill]

Subgroup	Location	Description
2.1	Boring YA-B-01: Elev. 5109 - 5114 Boring YA-B-02: Elev. 5122.7 - 5127.7	Poorly graded SAND with silt (SP-SM) (Fill), with some gravel, brown, no plasticity, moist, medium dense.
2.2	Boring YA-B-02: Elev. 5082.7 - 5097.7	Poorly graded SAND with silt (SP-SM) and some gravel, brown, tan, gray, pink, no plasticity, moist to wet, dense.

Soil Group 3: Poorly Graded Sand with Gravel (SP) [Fill and Non-Fill]

Subgroup	Location	Description
3.1	Boring YA-B-01: Elev. 5080 - 5109	Poorly graded SAND (SP) with gravel, brown, tan, pink with rust, no plasticity, moist to wet, loose.
3.2	Boring YA-B-02: Elev. 5097.7 - 5102.7	Poorly graded SAND with gravel (SP) (Fill), black and gray,no

		plasticity, moist, Dense, possible fill, difficult to distinguish.
3.3	Boring B-2: Elev. 5080.3 - 5106 Boring B-3: Elev. 5106.4 - 5082.4 Boring B-9: Elev. 5083.2 - 5105.7	Sand, gravelly, few cobbles, tan (Similar to 1-A)

Soil Group 4: Sandstone Formation [Bedrock]

Subgroup	Location	Description
4.1	Boring 2: Elev. 5074.9 - 5077.8 Boring 2: Elev. 5070.9 - 5072.4	Sandstone, silty, friable, very fine to medium grained, light greenish gray.
4.2	Boring YA-B-01: Elev. 5069 - 5080	Clayey Sandstone, blue-gray to gray, very hard.

Soil Group 5: Claystone Formation [Bedrock]

Subgroup	Location	Description
5.1	Boring YA-B-01: Elev. 5067.2 - 5069	Silty Claystone, olive-gray, very hard.
5.2	Boring YA-B-02: Elev. 5068.9 - 5082.7	Claystone, gray, very hard.

Soil Group 6*: Shale Formations [Bedrock]

Subgroup	Location	Description
6.1	Boring 9: Elev. 5077 - 5081.8	Shale, reworked, weathered with small coal particles, and mica, gray
6.2	Boring 9: Elev. 5072.9 - 5077 Boring 7: Elev. 5072.6 - 5080 Boring 3: Elev. 5075.4 - 5082.5	Shale, sandy, silty, dense, blue. Shale, silty, fractured with local, thin medium grained sandstone lenses, dark gray to blue.
6.3	Boring 2: Elev. 5077.8 - 5080.3	Shale clayey, silty, vertically and horizontally fractured, moist, greenish blue, hard

6.4	Boring 2: Elev. 5072.4 - 5074.9	Shale, hard, dark gray to black, highly fractured plastic.

*The shale formations which comprise soil group 6 could be the same as the claystone in soil group 5, based on their location in the geological cross section. It is possible that the soil layer naming convention varied between the 1967 and 2020 borings.

3.2 Soil Parameters

3.2.1 Site Soil Unit Weights

Unit weight is critical for foundation design. Dry unit weights for the sandstone and claystone were included in the boring logs, but for soil groups 1-3, only an approximate unit weights were given in tables 6 and 7 in Appendix B. Water content of the soil types was included in borings YA-B-02 and YA-B-03 and an average water content was found for each soil type. The water content is useful in finding bulk unit weight from dry unit weight with the equation below.

$$\gamma = \gamma_d(1 + \omega)$$

Where γ is bulk unit weight, γ_d is dry unit weight, and ω is water content. In the table shown in Figure 3.3 below, the unit weights were calculated using AASHTO typical values (Federal Highway Administration, 2017). In Figure 3.4 these same values were obtained directly from tables 6 and 7 in the Preliminary Geotechnical Report (see Appendix B). The values from the Geotechnical Report are assumed to be more accurate than the AASHTO typical values, but due to the higher than expected dry unit weight for the sandy soils, in certain cases the more typical 105-110 PCF dry unit weight may be used for conservatism. Furthermore, a clear distinction was not made in the Geotechnical Report between the dry unit weight and total unit weight, but the listed value of 125 PCF for sandy soils aligns with a typical saturated unit weight rather than a dry unit weight. Above the groundwater table, the effective unit weight is assumed to be the same as dry unit weight. Water content was obtained from YA-B-01 and YA-B-02 (See Appendix B lab results).

Table 3.2 Soil Unit Weights Based on AASHTO Typical Values

Soil Site Group	USCS Classification	Typ. Dry Unit Weight (PCF)	Typ. Saturated Unit Weight (PCF)	Typ. Effective Unit Weight (Below WT) (PCF)	Water Content (%)	Total Unit Weight
1	SC	105	125	62.6	13.3	119.0
2	SP-SM	105	125	62.6	9.4	114.9
3	SP	109	130	67.6	9.4	119.2
4	SANDSTONE					
5	CLAYSTONE					
6	SHALE					

Table 3.3 Soil Unit Weights Based on Geotechnical Report Parameters

Soil Site Group	USCS Classification	Unit Weight (PCF) From Tables	Effective Unit Weight (PCF) From Tables (Below WT)	Dry Unit Weight (PCF) From Borings	Water Content (%) From Borings	Total Unit Weight (PCF)
1	SC	125	62.5			125
2	SP-SM	125	62.5			125
3	SP	125	62.5			125
4	SANDSTONE	135	135	105.5	17.1	123.5
5	CLAYSTONE	135	135	106.1	20.3	127.6
6	SHALE	125	62.5			125

3.2.2 Granular Non-Bedrock Soil Friction Angle Analysis

The Geotechnical Report outlines parameters for general soil types (see tables 7 and 8 in Appendix B). According to table 7 the friction angle for fill and native primarily sandy soils is 32°, and the cohesion of these soils is zero. However, standard penetration tests and Modified California SPT tests were performed on soil samples extracted from borings near the bridge site, which allow the calculation of more site-specific friction angles. It is expected that the soils in groups one, two, and three will exhibit a drained (effective stress) response, and the most important parameters for design in these soils is the effective friction angle and cohesion. The SPT test soil response can be considered drained based on the permeability of the soil and the allowance of volume change. Therefore, the friction angle calculated from the SPT results is assumed to be equal to the effective friction angle of the soil. The calculations of friction angle for site soil groups one, two, and three are shown below. The water table elevation for use in effective vertical stress calculations was approximated at an elevation of 5,106.25’, consistent with a 10-year flood event (See Section 3.4.1).

Table 3.4 Effective Friction Angle From SPT, Soil Group One (Row One Emitted from Average). Note that effective unit weight is taken as dry unit weight (120 PCF according to the Geotechnical Report) above the water table and taken as (saturated unit weight – unit weight of water) beneath the water table. 10 Year flood level water table was considered.

SPT Calculations Soil Group 1 Strength Limit State											
Boring	Effective Unit Weight (PCF)	Middle Depth (Ft)	Em	Cb	Cs	Cr	N	N60	Vertical Effective Stress (PSF)	Phi'	Su
YA-B-01	120	6	0.89	1	1	0.75	59	66	720	45	N/A
YA-B-01	120	11	0.89	1	1	0.75	16	18	1320	32	N/A
YA-B-02	120	15.5	0.85	1	1	0.85	11	13	1860	30	N/A
YA-B-02	120	21	0.85	1	1	0.95	26	35	2520	35	N/A
YA-B-02	120	25	0.85	1	1	0.95	15	20	3000	32	N/A
YA-B-02	120	30.75	0.85	1	1	1	18	26	3690	32	N/A
Average										32	N/A
Expected Phi Value (From Report) = 32 Degrees											

Table 3.5 Effective Friction Angle From SPT, Soil Group Two. Note that effective unit weight is taken as dry unit weight (120 PCF according to the Geotechnical Report) above the water table and taken as (saturated unit weight – unit weight of water) beneath the water table. 10 Year flood level water table was considered.

SPT Calculations Soil Group 2 Strength Limit State											
Boring	Effective Unit Weight (PCF)	Middle Depth (Ft)	Em	Cb	Cs	Cr	N	N60	Vertical Effective Stress (PSF)	Phi'	Su
YA-B-01	120	15.5	0.89	1	1	0.85	18	23	1860	32	N/A
YA-B-02	62.5	41	0.85	1	1	1	30	43	4198.4	36	N/A
YA-B-02	120	11	0.85	1	1	0.75	23	24	1320	34	N/A
Average										34	N/A
Expected Phi Value (From Report) = 32 Degrees											

Table 3.6 Effective Friction Angle From SPT, Soil Group Three. Note that effective unit weight is taken as dry unit weight (120 PCF according to the Geotechnical Report) above the water table and taken as (saturated unit weight – unit weight of water) beneath the water table. 10 Year flood level water table was considered.

SPT Calculations Soil Group 3 Strength Limit State											
Boring	Effective Unit Weight (PCF)	Middle Depth (Ft)	Em	Cb	Cs	Cr	N	N60	Vertical Effective Stress (PSF)	Phi'	Su
YA-B-01	120	20.5	0.89	1	1	0.95	10	14	2416.9	30	N/A
YA-B-02	62.5	35.5	0.85	1	1	1	33	47	4605.9	37	N/A
B-2	62.5	9.75	0.7	1	1	0.75	9	8	609.4	30	N/A
B-2	62.5	15.25	0.7	1	1	0.85	43	43	953.1	40	N/A
B-2	62.5	20.55	0.7	1	1	0.95	50	55	1284.4	42	N/A
Average										34	N/A
Expected Phi Value (From Report) = 32 Degrees											

N_{60} was found using $N_{60} = \frac{E_m C_B C_S C_R N}{.6}$ where N is the blow count, E_m is the efficiency, and C_b, C_s, C_R are the correction factors for diameter, sampler type, and depth. E_m was given for the YA-B-01 and YA-B-02 borings and assumed to be 0.70 for other borings. Peck's correlation between friction angle (φ') and N_{60} was used in the calculations as $(.3)N_{60} + 27$ (Coduto, 1999). The friction angle calculations were also cross-referenced with figure 3.4 below which yields somewhat similar results using the correlation between N_{60}, σ_z', and φ'. The effective soil weights were based on figures 3.3 and 3.4 above. For sandy soils, 120 PCF was used as effective unit weight above the water table and 62.5 PCF was used below the water table. The friction angles found from figure 3.4 were generally higher than the friction angles found with Peck's correlation.

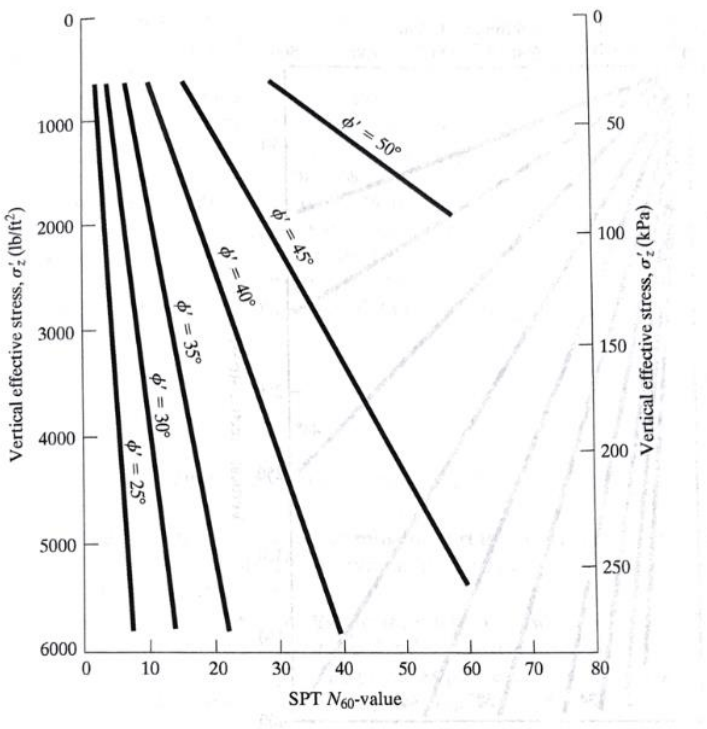


Figure 3.4 Empirical Correlation Between N_{60} and φ′ For Uncemented Sands. (Coduto, 1999)

Using the methodology described above, all soil groups' average friction angles from the SPT tests were slightly higher than the Geotechnical Report's 32° . Therefore, for general calculations, or calculations involving a large volume of sandy soil (I.E. retaining wall or slope calculations) the 32° is sufficient. The lowest friction angles were 29° to 31° , which will partially inform the factor of safety in design. Cohesion will be assumed to be zero in soil groups one through three, although due to the presence of clay and silt in soil groups one and two, some cohesion is expected.

3.2.3 Bedrock Strength and Drilled Shaft Design Parameters (Claystone, Shale, and Sandstone)

The strength design of a drilled shaft relies on the shaft and skin friction capacities. The bearing capacity of the drilled shaft will also need to be calculated. In the case of the site's primarily claystone bedrock, these capacities rely on the undrained shear strength of the soil, the tip resistance, and the side resistance. Due to the extremely high cohesion and the rock-like nature of the bedrock formations, the primary design parameter for the deep foundation design is the undrained shear strength of the bedrock. The undrained shear strength is equivalent to the undrained cohesion, estimated as 8,000 PSF in Table 7 from the Preliminary Geotechnical Report for the claystone and sandstone bedrock. Note that for these rock-like materials, unconfined compressive strength approximately equals undrained shear strength, which approximately equals cohesion. The 1967 borings reported that the shale was heavily fractured and weathered.

Table 3.7 Su Values by Soil Group

Soil Group	Su According to Table 8 (PSF)	Su Expected From Soil Type (PSF)	Boring Notes Regarding Su and UCS
4	8,000	1,500-15,000	
5	8,000	1,500-15,000	UCS Measured as 12,852 at Elev. 5072 in YA-B-01
6	3,000	1,500-15,000	UCS Measured as 29,600 at Elev. 5073 in B-9

The nominal tip and side resistance was also reported in the Preliminary Geotechnical Report in Table 6 (see Appendix B).

Table 3.8 Side and Tip Bedrock Resistance

Boring	Estimated Top of Bedrock (Ft)	Nominal Side Resistance (PSF)	Nominal Tip Resistance (PSF)
YA-B-01	5080	9.00	110
YA-B-02	5082	9.00	110

The 1967 borings (B-2, B-3, B-6, B-9) do not include nominal tip and side resistance of the bedrock, but due the similar values for YA-B-01 and YA-B-02, located on either side of the bridge, the non-weathered bedrock beneath the entire bridge will be assumed to have the same tip and side resistance as the bedrock in borings YA-B-01 and YA-B-02. Figure 3.5 displays the soil strengths on the cross section.

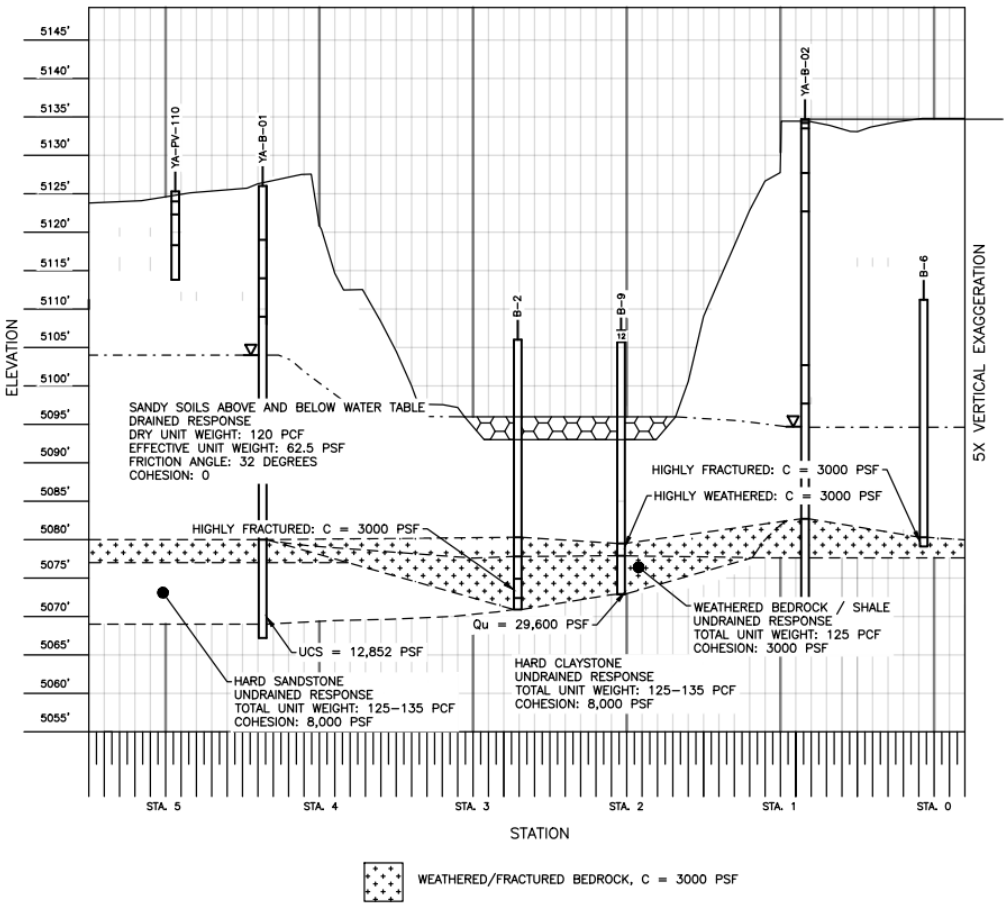


Figure 3.5 Diagram of Soil Parameters

The Geotechnical Report specifies that the top three feet of bedrock should be treated as weathered.

3.2.4 Serviceability Parameters

Serviceability limits will not be explicitly calculated in the foundation and retaining wall design, but some parameters that are used in deep foundation settlement analyses include dry unit weight, P-y modulus, initial void ratio, and over-consolidation ratio. Dry unit weights can be found in section 3.2.1. Constrained modulus from table 7 in Appendix B and typical void ratios are shown below (Geotechdata, 2013).

Table 3.9 Serviceability Parameters Non-Bedrock Soils

Soil Group	P-Y Modulus (PCI) (Above WT)	P-Y Modulus (PCI) (Below WT)	Est. Void ratio
1	90	60	0.17 - 0.59
2	90	60	0.33 - 0.98
3	90	60	0.30 - 0.75

3.3 Preliminary Discussion of Foundation Loads

All the loads acting on the bridge's substructure and superstructure are transferred into the foundation. Foundation structures will be constructed beneath each bridge pier and abutment. The factored loads from the Structural Analysis section of this report are used to estimate the vertical and horizontal loads induced on the foundation by the structure itself.

3.3.1 Vertical Structure-Induced Foundation Loads

In the three-span design option the vertical load on the West abutment is 2,714.1 Kips and the vertical load on the East abutment is 2,714.1 Kips. Most of the vertical load from the structure is translated into the piers which will feed into the deep foundation structures beneath. Each column carries 2,501 Kips and there are three columns per pier. Therefore, the total vertical load carried by each pier foundation is 7,503 Kips. There are two piers. One potential design will include four foundations, one beneath each pier and abutment. Due to the low loads on the abutments, and the large distance to bedrock, a shallow foundation might be allowable beneath the abutments, while deep foundations with drilled shafts are needed beneath the piers. Another option is deep foundations beneath both abutments and piers, which may be necessary to support the structure, and aligns with CDOT's recommendations for bridge design.

Table 3.10 Vertical Structure Loads 3-Span Solution (Recommended)

Element	Vertical Load (Kips) (-Z Direction⬇)
Foundation Beneath West Abutment	2,714.1
Foundation Beneath East Abutment	2,714.1
Foundation Beneath West Pier	7,503
Foundation Beneath East Pier	7.503

In the two-span design option, the abutments both experience a factored vertical load of 5,107.8 kips, while the pier columns beneath the single middle pier carry 3,413.5 kips each. There are three

columns in the single pier, so the foundation beneath the pier will carry a vertical load of 10,240.5 kips.

Table 3.11 Vertical Structure Loads 2-Span Solution

Element	Vertical Load (Kips) (-Z Direction⬇)
Foundation Beneath West Abutment	5,107.8
Foundation Beneath East Abutment	5,107.8
Foundation Beneath Pier	10,240.5

3.3.1 Lateral Structure-Induced Foundation Loads

Wind loading on the bridge is the primary concern for lateral loads on the foundation. However, the structural engineer has calculated only the dead load and vehicle live load, which will govern the structural design of the foundation. The final foundation will not be designed to account for a wind load. However, an estimate of wind load propagating into the foundation is shown below. Only lateral wind loads are estimated, not upward lift which will be insignificant to the foundation design for such a heavy structure. The equation used to estimate unfactored wind load is below (note this is not the equation used in the final structural design). The coefficients and wind speed were taken from CDOT's Bridge Design Manual (CDOT, 2022).

$$Unfactored\ Wind\ Load = (C_D)(Area)(Pressure\ Coefficient)(Wind\ Speed)$$

Table 3.12 Wind Load Coefficients

Lateral Structure Area	$(5.5\ ft)(306\ ft) = 1{,}683\ ft^2$
Wind	120 MPH
Wind Pressure coefficient	.00256
Drag Coefficient	2.0

$$Unfactored\ Wind\ Load = 124\ Kips$$

The maximum induced moment on a drilled shaft foundation cap will occur at the base of the longest pier. The maximum distance from the bridge superstructure (where the wind load is acting) to the pier base (top of foundation) is 60.5 feet (the East pier in the 3-Span solution).

$$Maximum\ Moment\ at\ Pier\ Base = (124\ Kips)(60.5\ Ft) = 7{,}502\ Kip - Ft$$

3.3.2 Water Lateral Loads and Earth Pressure Considerations

Exact hydrostatic loads on the piers will be calculated by the water resources engineer in the schematic design and lateral earth pressures will be calculated once design of geotechnical elements such as retaining walls begins. Some initial guidelines are provided in the Geotechnical Report. For earth pressures, a load factor of 1.5 is recommended for active and 1.35 for at-rest, passive is unlikely and will be neglected in design. Active conditions are characterized by walls which can deflect .1 to .2% of wall height. See reference AASHTO sections below.

Table 3.13 AASHTO Sections for Load Calculations

Lateral earth pressure induced by surcharge	AASHTO Section 3.11
Friction Angles Between Fill and Wall Materials	AASHTO LRFD Table C3.11.5.3-1
Lateral earth pressure for Structure Backfill	AASHTO LRFD specifications Section 3.11.5

In retaining wall design, hydrostatic and seepage pressures can develop in the soil, even if retaining wall designs allow for drainage. This will be accounted for by taking the saturated soil weight for the driving force of retaining wall calculations and considering the impact on the effective stress (I.E. resisting effective stress may decrease if dry soil becomes saturated). Drag force on the piers will need to be considered, which will be calculated by the water resources engineer for a 500-year flood event.

3.3.3 Serviceability Considerations

Foundation movement should be minimal, and if vertical deflection exceeds $\frac{Bridge\ Length}{1000}$ cracking of concrete, fatigue of reinforcement, and discomfort of those using the bridge may occur (Federal Highway Administration, 2021). Most of this deflection should occur in the superstructure of the bridge, and the foundation should deflect vertically roughly less than one inch. The settlement of foundation elements will not be calculated in this project, but making reasonable engineering decisions regarding settlement such as anchoring the foundation in bedrock is necessary.

3.3.4 Ultimate Limit State Criteria

The ultimate limit state criteria is primarily based on information found in Braja Das's Principles of Foundation Engineering (Das, Principles of Foundation Engineering Seventh Edition, 1983). Due to the nature of the site, only driven piles or drilled shafts are suitable for the foundation. Drilled shafts are more efficient from a cost and constructability standpoint, so the limit state criteria equations for drilled shafts in bedrock will dictate design. The following Equations (given in SI units) will be used in design.

Ultimate shaft capacity: $Q_u = Q_p + Q_s = q_p A_p + fpL$

End bearing capacity: $Q_p = q_p A_p = \left[4.38\left(\frac{q_u MN}{m^2}\right)^{.51}\right][A_p(m^2)]$

Side resistance capacity (smooth socket): $Q_s = fpL = \left[0.4\left(\frac{q_u MN}{m^2}\right)^{.5}\right][\pi D_s(m)][L(m)]$

Side resistance capacity (rough socket): $Q_s = fpL = \left[0.8\left(\frac{q_u MN}{m^2}\right)^{.5}\right][\pi D_s(m)][L(m)]$

These equations are optimal for the design of the foundation for this project as they describe the strength of drilled shafts using tip and side resistance, which are given by the Geotechnical Report. A factor of safety (F.S) of 3.0 will be applied to the nominal capacity to obtain the necessary design strength for driven piles. A F.S of 3.0 is standard in foundation design and provides the necessary conservatism. For drilled shafts, a F.S of 2.25 is acceptable according to CDOT's Bridge Design Manual.

3.4 Geotechnical Hazards

3.4.1 Water Level and Water Table Fluctuations

The most significant hazard on the project site is the presence of water and the potential for fluctuation of the river water and groundwater elevation. In Colorado, there is somewhat significant seasonal change in ground and surface level elevations. Generally, groundwater is at a lower elevation in the Winter months and higher elevation in the late Spring and Summer. The Geotechnical Report provided data from multiple groundwater monitoring wells, but these wells were somewhat far from the project site. To estimate the seasonal fluctuation of groundwater at the project site, the fluctuation between July groundwater depth (peak elevation, minimum depth) and December groundwater depth (minimum elevation, maximum depth) was calculated from table 3 in the Preliminary Geotechnical Report (see Appendix B). The groundwater monitoring wells all had data for both the summer and winter months, so the average difference in groundwater depth between seasons could be found. The groundwater fluctuation of the project site was estimated using the inverse distance method (equation below) to create a weighted average of the monitoring well results.

$$w_i = \frac{\left(\frac{1}{d_i^2}\right)}{\sum \frac{1}{d_i^2}}$$

Table 3.14 Groundwater Elevation Change

Monitoring Well	July Groundwater Depth (Ft)	November/December Groundwater Depth (Ft)	Difference (Ft)	Distance From Site (Ft)	Weight
YA-B-10	18.9	20	1.1	5850	0.0728
YA-B-11	10.8	30	19.2	20650	0.0058
YA-G-01	7.1	10	2.9	1700	0.8616
YA-W-05	13.4	14.5	1.1	6450	0.0598
Weighted Average Fluctuation (Inverse Distance Method)			2.76 Foot Fluctuation At Bridge Site		

The YA-B-02 and YA-B-03 borings noted that their groundwater table elevations were measured in December, so it is assumed that these groundwater measurements represent the minimum elevation of the water table and groundwater fluctuations could cause the water table to rise by around 2 feet, shown in the figure below. This is somewhat irrelevant to design, as for conservatism geotechnical elements will be designed assuming a 10 year flood event or in some cases an even more extreme flood event.

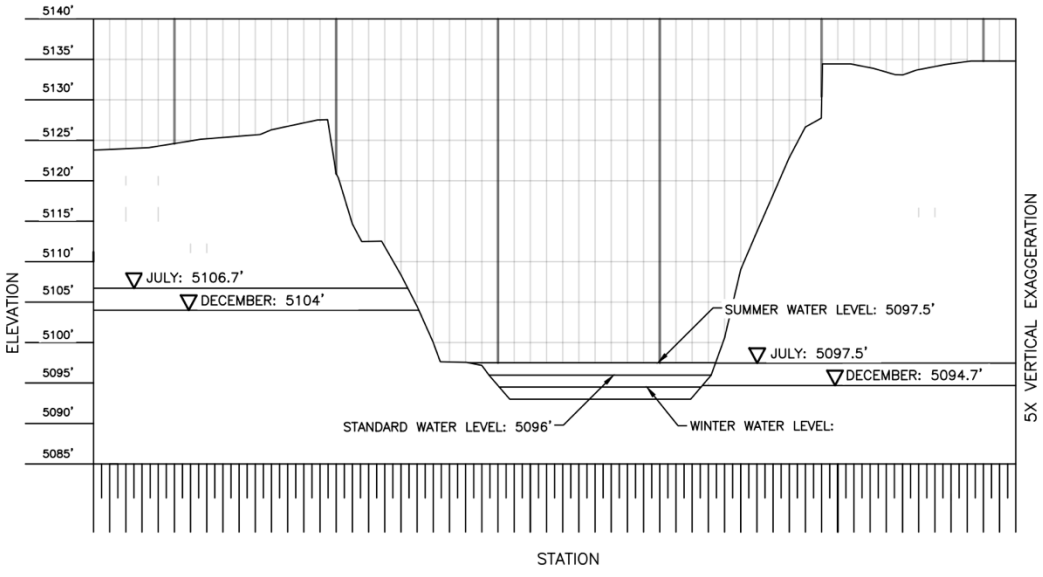
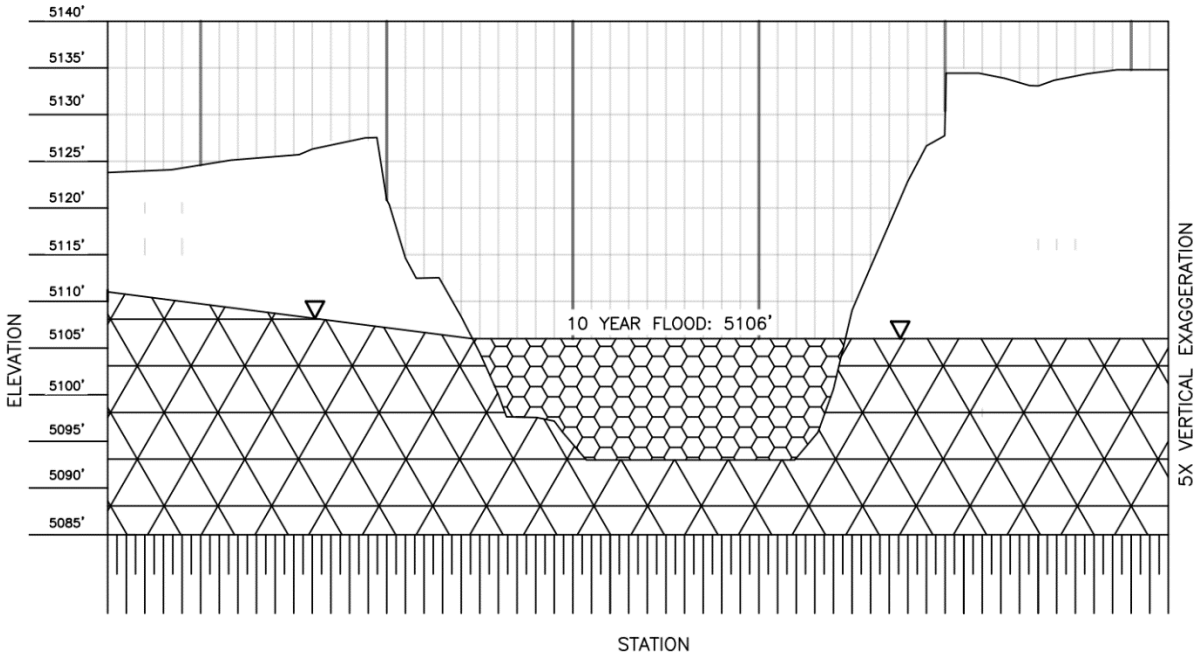
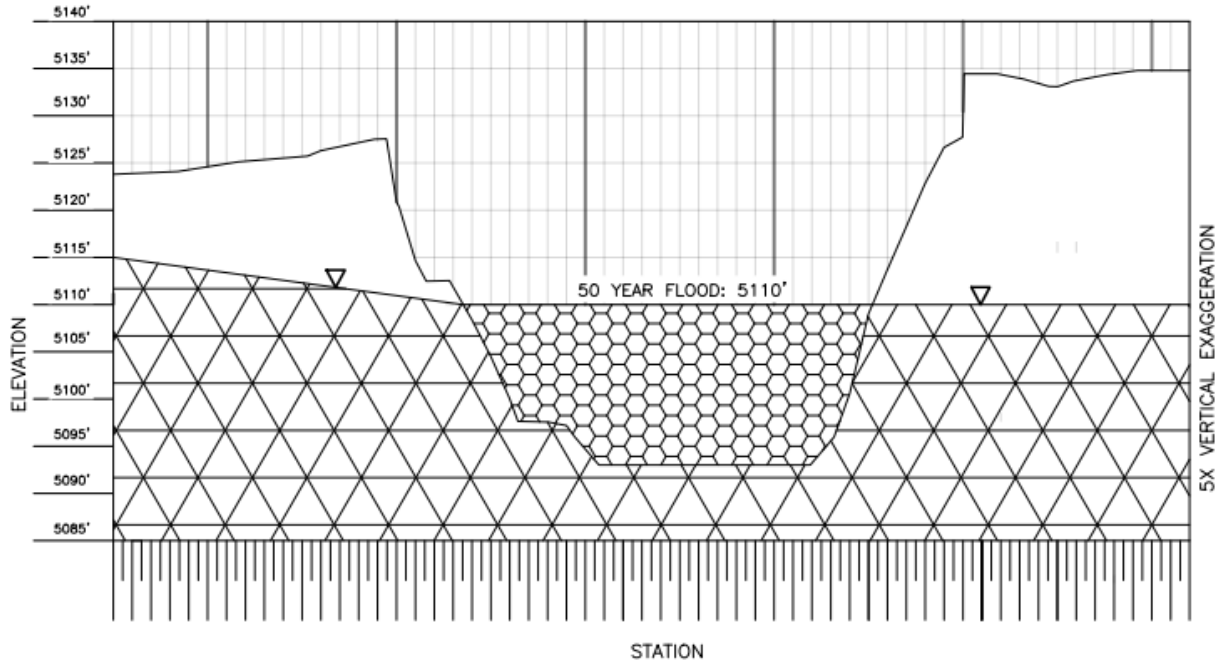


Figure 3.6 Seasonal Groundwater Table Fluctuation

While the seasonal fluctuations are minor, ground and surface water can change by several feet rapidly during floods. Below are conceptual plots of the surface water level and approximate groundwater level during flood events. The groundwater table may not look exactly like the conceptual estimates below, but for design, it will be assumed that the minimum water table

elevation during flood events is equal to the elevation of the surface water (although it might be lower due to excess runoff during storms).





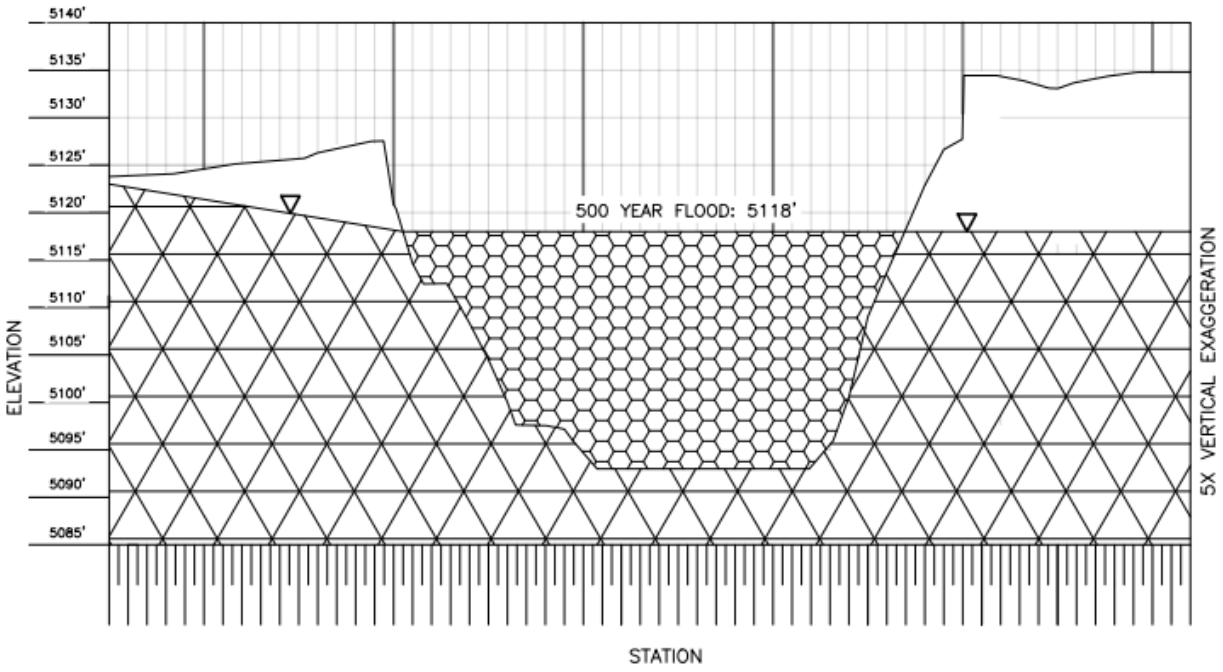


Figure 3.7 10, 50, and 500 Year Flood Surface and Groundwater Table Elevations

Finally, in the region around the South Platte River, the overall water table appears to be lowering slowly due to climate change and geological factors. According to USGS data from groundwater monitoring stations dating back over 50 years, the groundwater table is gradually lowering (Colorado Decision Support Systems, 2024). In this site, the groundwater table was estimated between 5,101.9' and 5,105.5' in 1967, but current groundwater data from borings YA-B-02 indicates that the groundwater table has lowered in elevation due to site changes.

3.4.2 Erosion and Scour

The riverbed has lowered substantially (approximately 10-14 feet) since 1967, indicating rapid erosion. Construction activities and development may have accelerated the erosion and settlement of the soil in this area, causing such a dramatic elevation change. The foundation must be designed with the consideration of potential future erosion. The design capacity therefore should rely on the bedrock rather than the soil above which is prone to erosion and settlement. According to the Geotechnical Report, additional embedment for soil stiffening elements such as MSE and soil nail walls may be required for this site. However, specific guidance for design is not given. According to the 1967 report, the effect of scour could extend up to 18 feet beneath the stream bed (it is notable that the stream bed has changed since then). Furthermore, a nearby railroad crossing failed

in 1965 due to the effects of scour, highlighting the importance of accounting for scour in the design. The approximate soil profile of the 1967 report is plotted with the current soil profile. The approximate profile was created using the boring elevations from 1967.

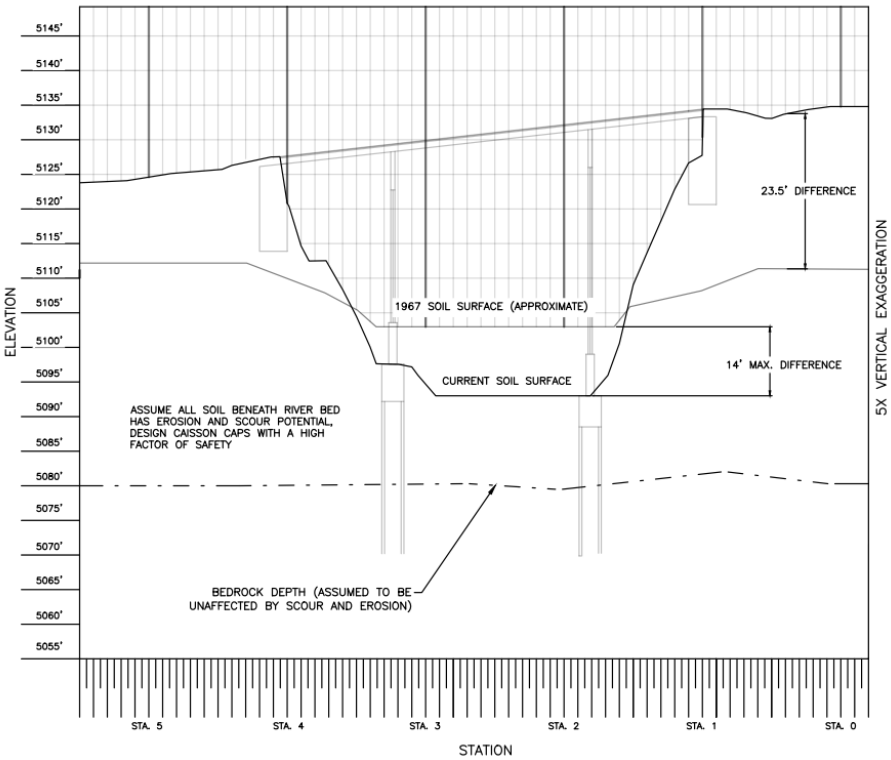


Figure 3.8 Change in Soil Surface Elevation 1967-2020

3.4.3 Historic Landfill Waste

One minor geotechnical concern for this site is the presence of landfill waste material near the project location. Continuous settlement has taken place over the last fifty years in certain sections of I-270 due to this landfill waste. In particular, the stretch of highway in between Brighton Boulevard and the Eastern side of the South Platte Overpass (our site area) has experienced significant settlement as a result of the landfill waste. Exploring the bridge site in detail reveals that the landfill waste may not be a large concern for the bridge foundation, as there are accessible strong soil layers and bedrock beneath the approximate bridge pier locations. However, the excessive settlement of the landfill waste could impact the access road, crane pad, and other construction elements further from the bridge itself. Furthermore, the exact properties and distribution of the landfill waste is unknown, and it varies substantially in depth along the highway (between 3 and 29 feet), which could result in differential settlement, especially if we need to construct an approach to the bridge which spans waste areas.

3.4.4 Settlement and Weak Soil

Soil stiffening may be required in certain sections of the site, particularly near slopes and retaining structures. The poorly graded sand was noted as "flowing sand" in YA-B-01, so weak sand is a concern in this project. Grout injections, geogrid reinforced platforms, dynamic compaction, and controlled modulus columns are all options for strengthening the soil. The Geotechnical Report recommends the use of concrete or geotextiles in soil stabilization. Geotextiles are the preferred option, but grout injection is the second-best and potentially more feasible solution. Settlement and soil shearing is a concern on any geotechnical project and will be mitigated through deep foundations anchored in the bedrock and should experience less settlement than foundations relying on the soil above.

3.4.5 Expansive Bedrock

Expansive bedrock is a potential geotechnical hazard but is unlikely to significantly impact this project. The Geotechnical report notes that some of the bedrocks in the project area are expansive, including claystone which is the predominant strong layer beneath the bridge site. In the laboratory section of the report, only one swell value is given in YA-B-02, which states that the claystone will expand 1.4 % under 1,000 PSF of pressure. This is a relatively low swell value compared to elsewhere along I-70 and will likely not dictate design. Swell is a larger concern with exposed or unsaturated bedrock.

3.4.6 Sustainability in Geotechnical Design

A final geotechnical hazard is the potential to disrupt the environment and community with the construction of geotechnical elements. All geotechnical elements should be designed with durability and minimal environmental impact in mind. Some design considerations to reduce the impact of the geotechnical design of the site are listed.

- Minimize soil excavation
  - Reduce the size of the foundation by using drilled shafts in strong soil rather than large shallow foundations or inefficient driven piles.
- Improving on site soil
  - Strategies such as MSE walls and localized grout injection should be used to improve soil properties to reduce the need to import off-site soil, which requires environmentally costly excavation and transport.
- Geosynthetics
  - If possible, geosynthetics and MSE walls should be used rather than heavily concrete-reliant structures for slope stabilization.
- Water Management

- Water must be controlled during the construction of the foundation and no geotechnical designs which negatively impact river flow or groundwater quality should be employed.

3.5 Preliminary Foundation Recommendations (Schematic Design)

3.5.1 Foundation Environment

New Foundations will be designed and constructed beneath each pier, while the foundations beneath the abutments will remain in their current state. The first step in designing the foundation is understanding the particular site geometry and the exact location of the foundation elements. The figure below shows the location of the foundation elements as well as the depth to bedrock, bedrock layers, and water table elevation used in design. Refer to section 3.1 for soil USCS classifications.

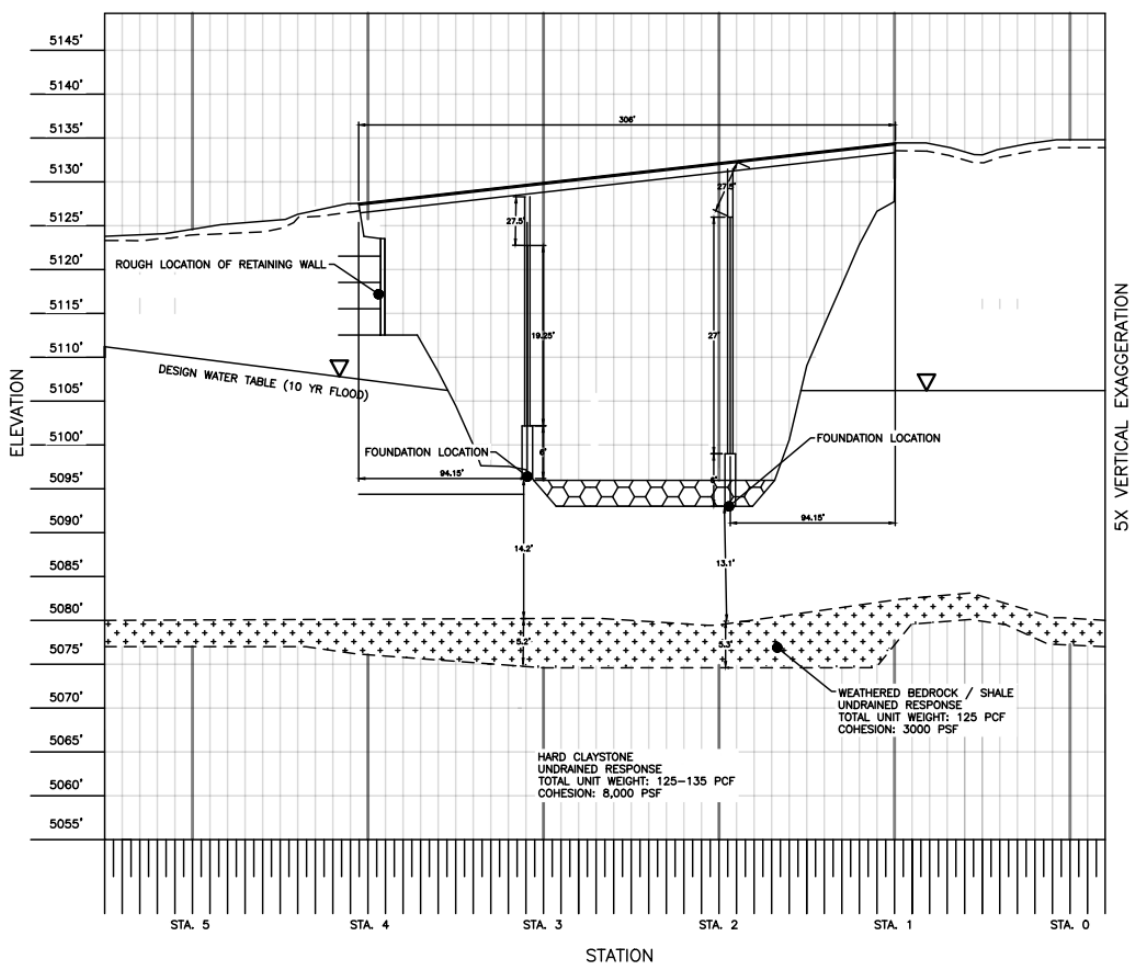


Figure 3.9 Foundation Environment Cross Section

The foundation is a part of the structural system of the bridge, not just an individual structure that must support arbitrary point loads. Therefore, a knowledge of bridge substructure geometry is essential before designing the foundation. There are two groups of three piers, one group on the Northwest bank of the river and one near the Southeast bank. The geometry of the piers and the pier footings is shown in the figure below.

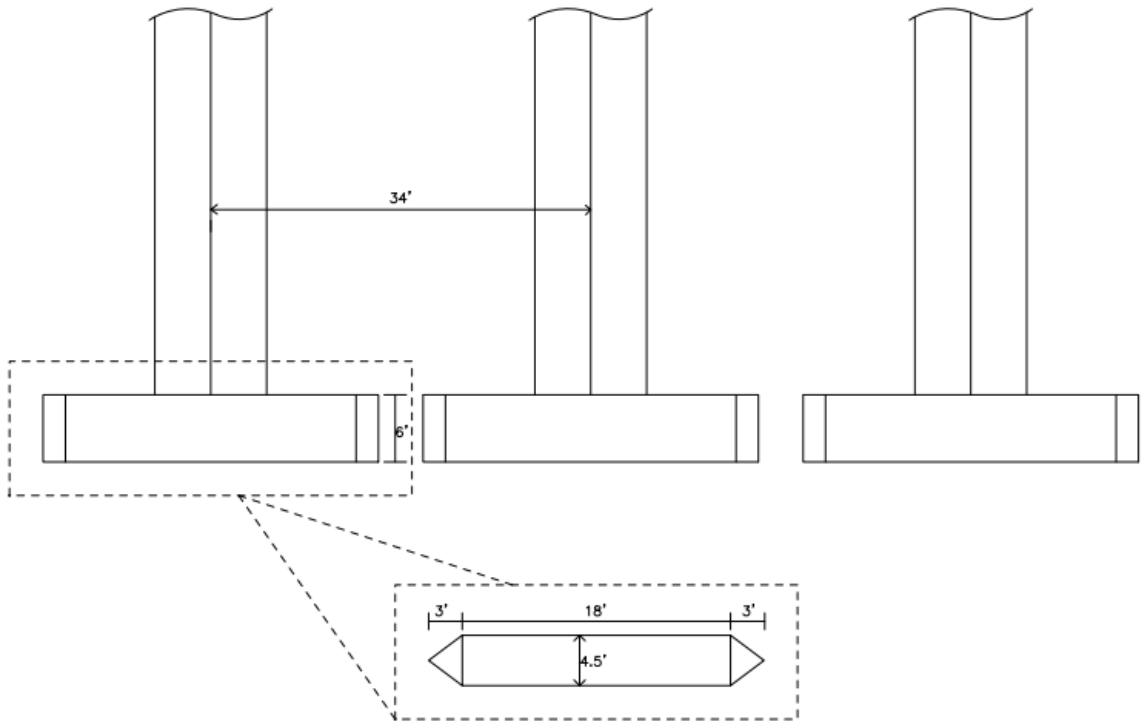


Figure 3.10 Piers and Footings (1 Pier Footing)

3.5.2 Driven Pile Foundation (H-Pile)

Driven piles will be considered as one option for the bridge foundation as they align with CDOT's tier-1 bridge design practices and were recommended as a foundation option in the preliminary geotechnical report.

3.5.2.1 Pile Cap Considerations, Pile Loads and Omissions, and Discussion of Service Limit State

According to the LRFD Bridge Design Manual (AASHTO, 2020), center-to-center pile spacing should not be less than 2.5 pile diameters (or equivalent diameters) and H piles must be at least 9

inches from the edge of the pile cap. Piles must be embedded at least 1 foot into the pile cap. These general guidelines will be used to design the pile cap, but the structural capacity of the cap itself is not considered in the schematic design phase (only the deep foundation capacity).

Uplift and frost heave will not be considered in the schematic design. Uplift is negligeable due to the small swell potential of the bedrock. The effect of pile-driving vibration on the placed piles will not be considered directly in design but should be monitored during construction. The top of cap and down drag loads should be considered as potential pile-driving hazards and precaution should be taken during construction. Another load which will be considered conceptually in the design of the foundation is the factored lateral load on the structure and lateral loads from soil, but the effects of lateral loads on the calculation will not be numerically calculated.

The primary load that will dictate the design of the driven pile foundation is the factored vertical structure load (2,501 Kips per pier).

The service limit state considers the settlement of the deep foundation elements. Down drag and the compression of soil beneath the foundation can cause settlement in addition to long-term structure loads. According to the Preliminary Geotechnical Report, settlement should be less than ½ inches for driven piles. Detailed settlement calculations are outside of the scope of this submittal, but the constructor should ensure the end bearing of the piles are not located in a weak or weathered bedrock pocket to minimize the potential for punching or local shear failure.

3.5.2.2 Constructability and Additional Considerations for Piles Constructed in Rock

On the bridge site, piles will be driven into soft rock (Su < 20,000 PSF). Therefore, ultimate limit strength will be calculated using the same procedures as highly cohesive soil. Tip protection is not necessary for pile driving on this site as the bedrock is not hard enough for concern (note the Preliminary Geotechnical Report suggests tip protection for driven piles but this should be disregarded) (AASHTO, 2020). According to the Preliminary Geotechnical Report and in-line with typical pile driving practices, the piles should be driven about 5-10 feet into the hard bedrock, and the resistance found in the field should be compared with that used in design. Constructors should expect to drive between 5-10 piles per day (AASHTO, 2020).

One significant consideration when using a driven pile foundation is the potential for change in resistance over time. Pile capacity should be checked several days after driving the pile, as soil relaxation or setup can affect the nominal resistance of a driven pile. Soil setup increases the pile capacity, while soil relaxation decreases the axial resistance. Restrikes should be used to test piles

for a change in resistance after construction. Restrikes should take place approximately 7 days after driving (AASHTO, 2020).

Potential issues such as groundwater level fluctuation and scour were avoided by conservatively omitting the capacity of the top layer of sand and using a conservative estimate for the width of the weathered bedrock layer. Omitting the sand layer completely from these calculations is suggested by the Geotechnical Report but is too conservative for an accurate design. However, because there is such a significant potential for scour, it is difficult to reliably use the resistance of the pile embedded in sand for a 75 year design life so the scour should be monitored closely over the design life of the structure.

Down drag and the pile driving force must be considered in the design of a driven pile foundation. The nominal pile driving resistance (equal to the minimum required driving force) is the sum of the nominal pile resistance and the additional side resistance during driving which should be measured during construction or during a dynamic load test. These calculations are omitted for this phase of design.

3.5.2.5 Driven Pile Foundation Design (Strength Limit State)

The static analysis alpha method was used in bedrock to design a foundation that meets the strength limit state. In the sand above the bedrock, the beta method was used to determine the additional capacity, although the contribution from the sand layer to pile capacity was very low. The equations and tables for this method can be found in Appendix B2.

Combining the alpha and beta method, the pile capacity was found. Some additional conservatism was included, such as not considering the top 6.5 feet of sand, using the resistance factors given in the geotechnical report, and taking perimeter of the H pile to be the box perimeter around the outside edges. Several different piles were considered and many different pile configurations meet the capacity requirements. However, due to the relatively high vertical pier loads (2,501 Kips per pier), and the F.S. of 3.0 the largest possible piles should be used to minimize the amount of piles to be driven. Therefore, HP460 Piles were used.

Tables 3.15 Driven Pile Capacity Tables

Hard Bedrock Capacity	
Surface Area Per Unit Depth(ft^2/ft)	6
Depth of Embedment (ft)	10
Total Side Area of Steel (ft^2)	60
Su (PSF)	8000
Alpha	0.4
qs (PSF)	3200
qp (PSF)	72000
Pile End Area (ft^2)	0.42
Rs (lbs)	192000
Rp (lbs)	30240
Rn (lbs)	222240
Rn (Kips)	222.24
Resistance Factor	0.65
Rr (Design Capacity) (Kips)	144.46

Weathered Bedrock Capacity	
Surface Area Per Unit Depth(ft^2/ft)	6
Depth of Embedment (ft)	5.2
Total Side Area of Steel (ft^2)	31.2
Su (PSF)	3000
Alpha	0.75
qs (PSF)	2250
qp (PSF)	27000
Pile End Area (ft^2)	0.42
Rs (lbs)	70200
Rp (lbs)	0
Rn (lbs)	70200
Rn (Kips)	70.2
Resistance Factor	0.65
Rr (Design Capacity) (Kips)	45.63

Sand Capacity (Beta Method)	
Delta (Radians)	0.45
Ks=K0	0.47
Phi' (Radians)	0.56
Surface Area Per Unit Depth(ft^2/ft)	6
Depth of Embedment (ft)	7
Total Side Area of Steel (ft^2)	42
Midpoint Vertical Stress (PSF)	1250
Beta	0.23
qs (PSF)	281.53
qp (PSF)	0
Pile End Area (ft^2)	0.42
Rs (lbs)	11824.31
Rp (lbs)	0
Rn (lbs)	11824.31
Rn (Kips)	11.82
Resistance Factor	0.65
Rr (Design Capacity) (Kips)	7.69

These capacities were used to determine the number of piles needed for each pier and the piles needed for the overall project. A summary table is shown below along with drawings detailing the pile caps and embedded piles. 4,500 PSI structural concrete shall be used in the pile cap.

Tables 3.16 Driven Pile Design Table

Pile Type	**HP460 (18X18)**
Nominal Surface Area Per Foot (ft^2/ft)	8.73
Surface Area Per Foot for Design (ft^2/ft)	6
End Area (ft^2)	0.42
Weight Per Foot (Lbs)	204
Material	A 572 Grade 60
Equivalent Diameter (ft)	0.73
Minimum Pile Spacing (ft)	2.19
Length (Northwest Piers) (ft)	30
Length (Southeast Piers) (ft)	29
Total Design Bearing Capacity Per Pile (Kips)	197.77
Piles Per Pier	38
F.S.	3
Total Piles Beneath Bridge Structure	228

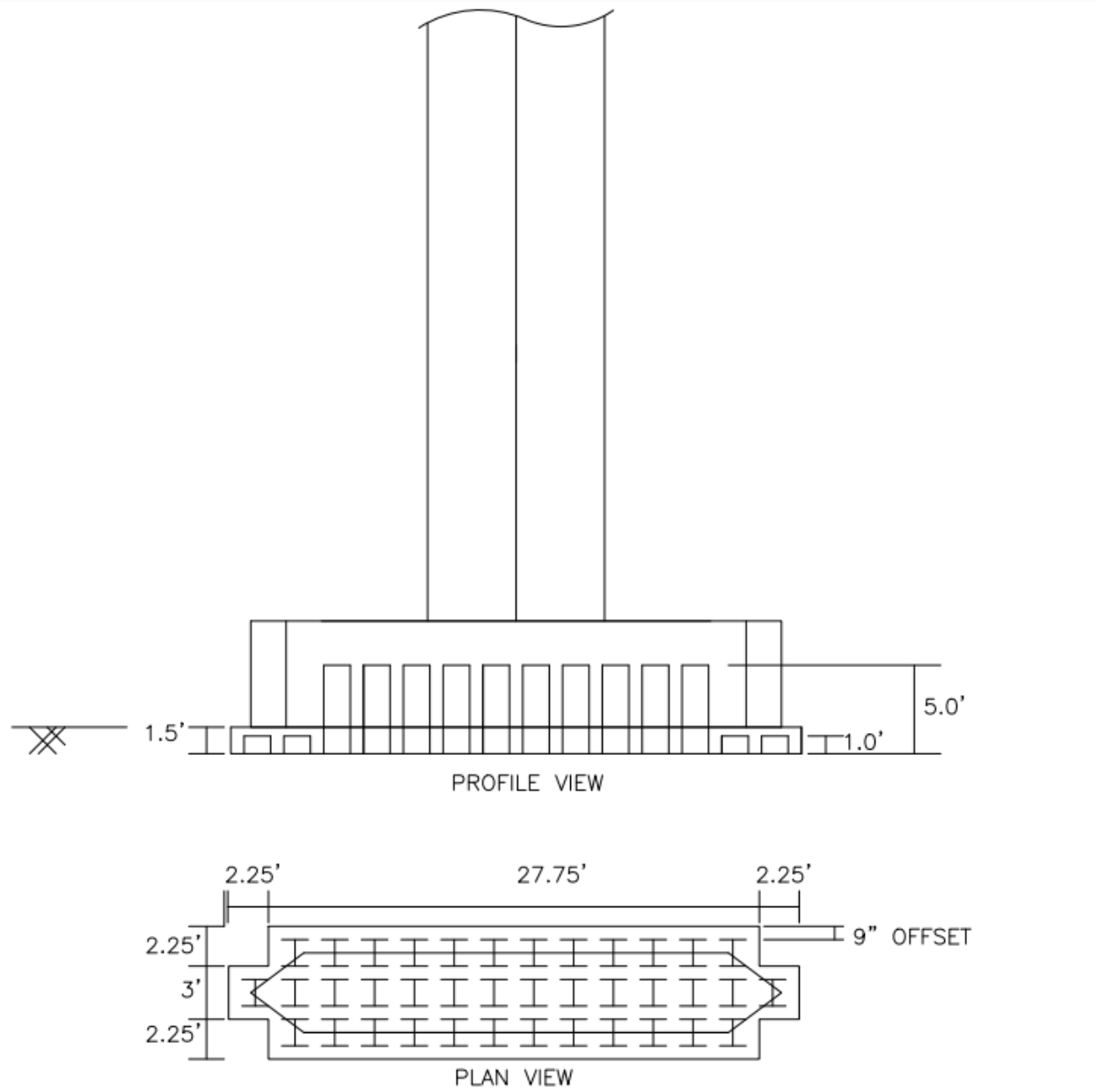


Figure 3.11 Foundation Plan Driven Piles

Piles are spaced at 2.25' center to center (meeting AASHTO design requirements), reinforcement for drilled shaft cap should be #7 bars spaced at 9 inches square grid (recommended by construction expert).

3.5.3 Drilled Shaft Foundation

Drilled shafts were also recommended as a foundation option in the geotechnical report and the CDOT bridge manual. Drilled shafts are not as common as driven piles for similar projects, but due to the high loads the foundation will experience drilled shafts might be a more economic and efficient design choice.

3.5.3.1 Drilled Shaft Design Considerations, Pile Loads and Omissions, and Serviceability Limit State Discussion

The geotechnical report outlines more specific guidelines for the design of drilled shafts than that of driven piles. Therefore, the calculation of drilled shaft capacity will be based on the findings of the preliminary geotechnical report rather than the standard LRFD approach. These results will be compared with the capacity found using the methodology laid out in the Foundation Design textbook (Das, Principles of Foundation Engineering Seventh Edition, 1983). According the Preliminary Geotechnical Report, the resistance of the overburden soil and weathered bedrock should be neglected entirely. Resistance factors of 0.60 should be generally be used for the side resistance and 0.55 used for the tip resistance. The nominal resistances corresponding to YA-B-01 and YA-B-02 in Table 6 in Appendix B2 will be used in design.

The spacing of drilled shafts must be at least 4 times the diameter of the shaft (from center to center) and the edge of the shaft cap must be at least 1 foot from the edge of the nearest drilled shaft. Required structural shaft embedment into the foundation cap is not calculated in this phase of design, but additional rebar is included in the design to tie the caissons (drilled shafts) into the foundation cap.

The loads considered on the foundation are the same as those considered for the driven piles (see section 3.1.2.1B). Drilled shafts have low deflection tolerances and are weaker under lateral or torsional loading than driven piles (AASHTO, 2020).

3.5.3.2 Drilled Shaft Construction Considerations

Shaft diameters should not exceed 4’ for convenience of acquiring drilling equipment. While drilled shafts can be drilled extremely deep into bedrock, it will become difficult from a constructability standpoint if they are deeper than 25’ in hard bedrock.

Steel casing will reduce the side resistance of the drilled shaft. If a permanent casing is required during construction the capacities calculated in this schematic design report may be an overestimation (AASHTO, 2020). Further site-specific construction considerations are discussed

later in this report. As per this design, no casing is required, but it may be determined necessary during construction.

3.5.3.3 Drilled Shaft Capacity and Design

Similar to driven piles, the drilled shaft foundation will be designed according to the structure vertical load but resistance to lateral loads should be considered conceptually. For simplicity of design and construction, all drilled shafts will have the same dimensions (with the exception in a slight difference in length between the shafts beneath the East piers and West piers due to different depth to bedrock).

According to CDOT's bridge design manual, a factor of safety of 2.25 may be used for drilled shaft foundations in claystone (like the bedrock found on site) as long as the analysis follows the Abu-Hejleh method (CDOT, 2022) (Abu-Hejleh et al., 2003). The capacity calculation methodology outlined in the Preliminary Geotechnical Report conservatively follows the Abu-Hejleh method.

Single Pier Vertical Load and Resistance		
Factored Vertical Load	2501	Kips
Foundation Required Resistance (F.S. = 2.25)	5627.25	Kips

This method (recommended by CDOT and the Preliminary Geotechnical Report) neglects the capacity contributed by non-bedrock soils (in this case the overlying sand), which is consistent with CDOT's guidelines which state that bridge foundations should be designed for a 100 year flood event. In a 100-year flood event, scour depths reach 10.7 feet below the river basin – nearly the entire depth of the sand layer. Thus, this sand layer cannot be relied on in a conservative foundation design.

The capacity table using the nominal resistances given in the Geotechnical Report is shown below along with drawings detailing the drilled shaft foundation plans. Calculations can be found in Appendix B2.

Table 3.17 Drilled Shaft Calculation Table Method 1

Hard Bedrock Calculation (Preliminary Geotechnical Report)	
Number of Drilled Shafts Per Pier	3
Embedment Into Hard Bedrock (ft)	22
Diameter (ft)	3.5
Surface Area Per Unit Depth (ft^2/ft)	11.00
Total Area of Side Resistance (ft^2)	241.90
End Area (ft^2)	9.62
qp (KSF)	110
qs (KSF)	9
Nominal Qp (Kips)	1058.32
Nominal Qs (Kips)	2177.12
Factored Qp (Kips)	582.08
Factored Qs (Kips)	1306.27
Qu (Ultimate Capacity 1 Drilled Shaft) (Kips)	1888.35
Qu (Total Ultimate Capacity Per Pier Foundation System) (Kips)	5665.06

The rebar must also be designed for the drilled shafts. For foundation caissons, minimum reinforcement is typically used as the frictional capacity is unaffected by reinforcement design. Calculations for reinforcement can be found in Appendix B2.

Table 3.18 Drilled Shaft Design Table

Drilled Shaft Specifications		
Diameter	3.5	Feet
shaft Length (Beneath Northwest Piers)	42	Feet
shaft Length (Beneath Southeast Piers)	41	Feet
Total Drilled Shafts Per Pier	3	Per Pier
Design Capacity (Per Drilled Shaft)	1888.35	Kips
Total Design Capacity (Per 5 Drilled Shafts)	5665.06	Kips
Concrete Strength	4500	PSI
Concrete Per Drilled Shaft (NothWest Pier Shafts)	14.97	CY
Total # of Northwest Pier Drilled Shafts	9	QTY
Total Concrete for Northwest Pier Drilled Shafts	134.70	CY
Concrete Per Drilled Shaft (SouthEast Pier Shafts)	14.61	CY
Total # of SouthEast Pier Drilled Shafts	9	QTY
Total Concrete SouthEast Pier Drilled Shafts	131.49	CY
Total Drilled Shaft Concrete (Whole Project)	266.18	CY
Longitudinal Reinforcement Area (in^2)	12.48	in^2
# of Bars	8	#11 Bars
Total Length of Longitudinal Rebar (Whole Project)	5976	LF of #11 Bars
Transverse Reinforcement Diameter	3	ft
Hoop Length	9.42	ft
# of Hoops NW Pier Drilled Shaft	57	QTY
# of Hoops SE Pier Drilled Shaft	56	QTY
Total Length of Transverse Reinf. (Whole Project)	9585.00	LF #3 Bars

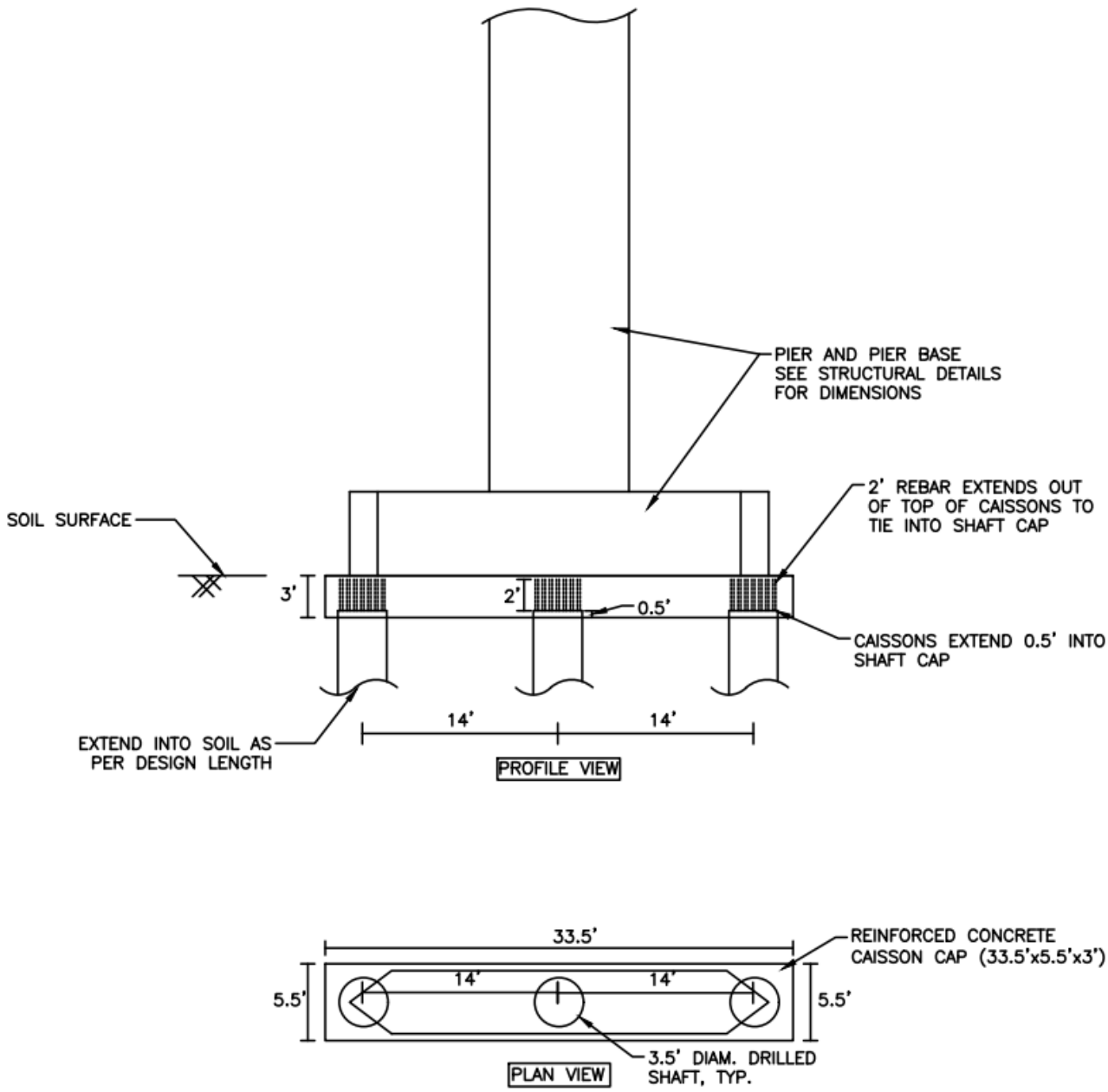


Figure 3.12 Drilled Shaft Plans

3.5.3.4 Drilled Shaft Alternative Capacity Calculation

The previous calculation was performed using the methodology outlined in the Preliminary Geotechnical Report. While this methodology is encouraged by CDOT, it is uncommon in the broader field of foundation design. To ensure this is an appropriate design, a more in-depth calculation using the equations in the Das Foundation Design textbook will be used to cross-verify capacity. Calculations can be found in Appendix B2.

Tables 3.19 Drilled Shaft In-Depth Calculation Tables

Hard Bedrock Calculation (Long Method) (Medium-Rough Socket)	
Unconfined Compression Strength (PSF)	14000
Number of Drilled Shafts Per Pier	3
Embedment Into Hard Bedrock (ft)	22
Diameter (ft)	3.5
Surface Area Per Unit Depth (ft^2/ft)	11.00
Total Area of Side Resistance (ft^2)	241.90
End Area (ft^2)	9.62
qp (KSF)	82.24
qs (KSF)	10.26
Nominal Qp (Kips)	791.24
Nominal Qs (Kips)	2481.21
Factored Qp (Kips)	435.18
Factored Qs (Kips)	1488.73
Qu (Ultimate Capacity 1 Drilled Shaft) (Kips)	1923.91
Qu (Total Ultimate Capacity Per Pier Foundation System) (Kips)	5771.72

Weathered Bedrock Calculation (Long Method) (Medium-Rough Socket)	
Unconfined Compression Strength (PSF)	6000
Number of Drilled Shafts Per Pier	3
Embedment Into Hard Bedrock (ft)	5.2
Diameter (ft)	3.5
Surface Area Per Unit Depth (ft^2/ft)	11.00
Total Area of Side Resistance (ft^2)	57.18
End Area (ft^2)	9.62
qp (KSF)	0
qs (KSF)	6.714821
Nominal Qp (Kips)	0.00
Nominal Qs (Kips)	383.93
Factored Qp (Kips)	0.00
Factored Qs (Kips)	230.36
Qu (Ultimate Capacity 1 Drilled Shaft) (Kips)	230.36
Qu (Total Ultimate Capacity Per Pier Foundation System) (Kips)	691.08

Sand Calculation (Long Method)	
Average Vertical Effective Stress (PSF)	715
Effective Friction Angle (Radians)	0.56
Unconfined Compression Strength (PSF)	6000
Number of Drilled Shafts Per Pier	3
Embedment Into Hard Bedrock (ft)	6
Diameter (ft)	3.5
Surface Area Per Unit Depth (ft^2/ft)	11.00
Total Area of Side Resistance (ft^2)	65.97
End Area (ft^2)	9.62
qp (KSF)	0
qs (KSF)	0.19
Nominal Qp (Kips)	0.00
Nominal Qs (Kips)	12.59
Factored Qp (Kips)	0.00
Factored Qs (Kips)	7.56
Qu (Ultimate Capacity 1 Drilled Shaft) (Kips)	7.56
Qu (Total Ultimate Capacity Per Pier Foundation System) (Kips)	22.67

Total Design Capacity Per 3 Drilled Shafts (Kips)	6485.47
Total Design Capacity Per Drilled Shaft (Kips)	1297.0942

The total capacity using the textbook method is higher than the capacity found using the method recommended in the Geotechnical Report. Therefore, the capacity using the Geotechnical Report method (following the Abu-Hejleh method) will be used for conservatism in design. Table 3.18 contains the final specifications for the foundation.

3.6 Foundation Selection and Justification

After producing the preliminary designs in the above sections, a foundation type will be selected. While driven piles are more sustainable from a material perspective, the quantity of piles needed to sufficiently support the bridge structure is unreasonable, and driving over 200 large piles would be difficult from a constructability standpoint and extremely expensive. Driving the piles is also slightly less safe and drilled shafts because there is potential for the vibrations to disrupt other structural elements or the existing abutment foundation

Drilled shafts on this site are also difficult to construct due to the presence of water on the site. The shafts are reasonable diameter (3 ft) and acquiring an appropriate drilling rig should not be challenging. The drilled shafts are substantially less expensive than the driven piles (about ½ the cost of the driven piles according to the rough estimate from the construction lead). However, the drilled shafts require a large amount of concrete (roughly 326 cubic yards total) which makes them less sustainable than the driven piles from a material perspective. However, when the community is considered, the long construction times, vibrations, and noise from driving all of the piles is much more disruptive than the construction of drilled shafts.

A weighted decision matrix was created to decide between driven piles and drilled shafts. Drilled shafts are the better option for this project and the final design of the bridge will consist of drilled shaft (caisson) foundation.

Table 3.20 Foundation Decision Matrix

	Category	Safety	Constructibility	Life Cycle Cost	Project Sustainability	Community Sustainability	Score
Foundation	Weight	9	8	8	6	4	
	Driven Piles	8	4	4	6	2	5.14
	Drilled Shafts	9	6	8	2	4	6.31

3.7 Slope Stability Analysis

Even though a retaining structure will support part of the slope, there are many slopes around the bridge site and all of these must be analyzed for stability to some extent. In this section, an initial rough factor of safety will be calculated for several slopes, and then the more detailed method of slices method will be used to analyze unsupported large slopes.

Rough Factor of Safety (Infinite Slope Method)

To determine necessary soil stabilization and retaining structures on the site, a rough slope stability analysis must be carried out. This analysis will not be numerically comprehensive but should give a strong estimate of which slopes will need to be stabilized, which slopes should not be changed during construction, and which slopes are safe to modify.

CDOT recommended using the infinite slope analysis method (see Appendix B2 for the equation) to determine the rough factor of safety of a cohesionless soil. This analysis method simply compares the friction angle of the soil to the angle of the slope and represents the soil as an infinite homogeneous slope. The slopes and their factors of safety from this initial check are shown below:

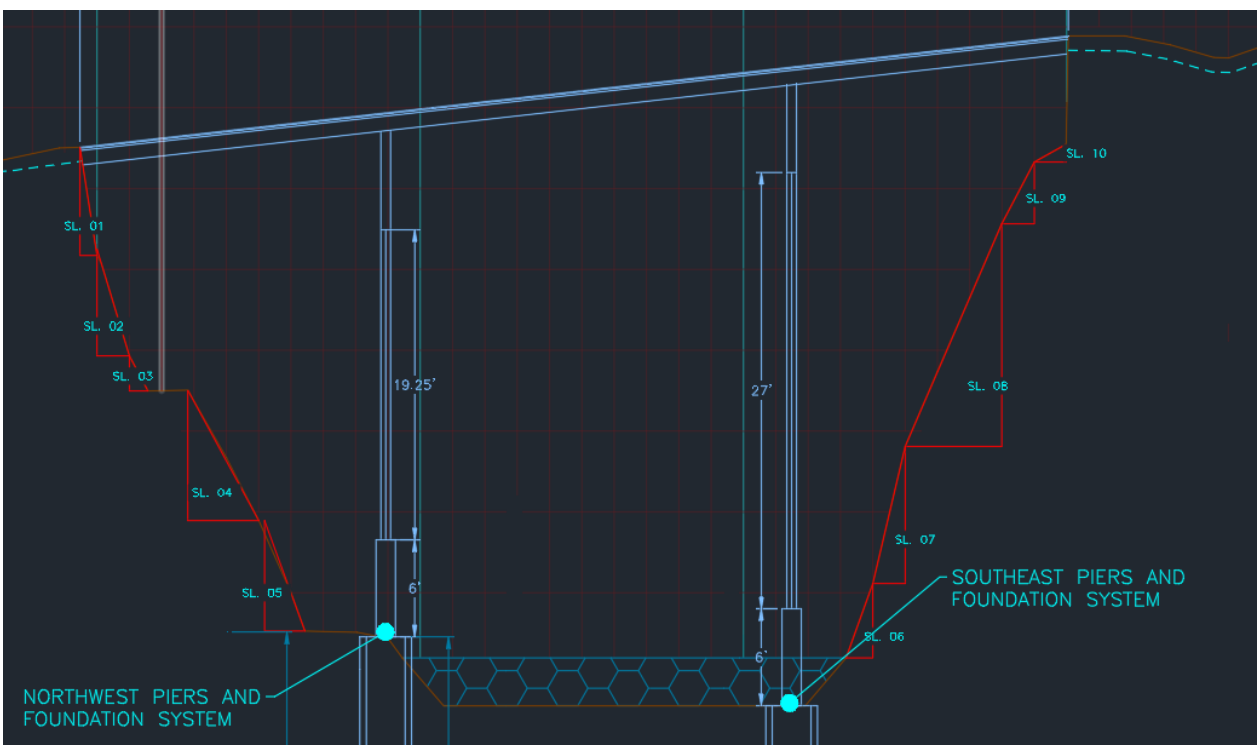


Figure 3.13 Initial Slope Delineation

Tables 3.21 Initial Slope Stability F.S. Check

SL. 01	
Height	6.69
Width	5.29
Beta	0.9
Beta	51.67
F.S.	0.49

SL. 03	
Height	2.01
Width	5.74
Beta	0.34
Beta	19.3
F.S.	1.78

SL. 02	
Height	6.21
Width	10.02
Beta	0.55
Beta	31.78
F.S.	1.01

SL. 04	
Height	8.07
Width	21.96
Beta	0.35
Beta	20.18
F.S.	1.7

SL. 05	
Height	6.83
Width	12.49
Beta	0.5
Beta	28.67
F.S.	1.14

SL. 07	
Height	8.48
Width	10.01
Beta	0.7
Beta	40.27
F.S.	0.74

SL. 09	
Height	3.82
Width	10.08
Beta	0.36
Beta	20.76
F.S.	1.65

SL. 06	
Height	4.6
Width	8.29
Beta	0.51
Beta	29.03
F.S.	1.13

SL. 08	
Height	13.77
Width	29.87
Beta	0.43
Beta	24.75
F.S.	1.36

SL. 10	
Height	1.1
Width	9.81
Beta	0.11
Beta	6.4
F.S.	5.57

From this analysis, the potentially problematic slopes are:

- Slope 1
- Slope 2
- Slope 5
- Slope 6
- Slope 7

Slope 1 may be dismissed because the abutment is located at the location of slope 1 and is assumed to be structurally stable. No additional reinforcement is needed apart from the abutment structure.

A retaining structure will be constructed at the location of slopes 2 and 3, so no further analysis of the stability of these slopes is need (only the retaining wall calculations).

Slopes 5 and 6 are heavily seeded, close to the river bank, and relatively far from construction activity. The proximity to the river makes it difficult to stabilize these slopes without damaging the river, while the seeding and distance from construction activity means that the slopes are likely stable despite a low factor of safety using the infinite slope method. However, precaution should be taken around these slopes during construction.

The slopes on the Southeast side of the river, including slope 7, are reinforced with railway ties and wire-wrapped rip rap and are covered with vegetation. Therefore, a low factor of safety is acceptable for these slopes because these stiffening elements were not accounted for.

Method of Slices

The above factors of safety were based on an extremely rough approximation. A few specific large slope faces were selected for a more in-depth "method of slices" failure analysis. The slopes from Southeast abutment to the river were analyzed for base failure and toe failure which could be catastrophic to the project. The Northwest side of the river was not analyzed in-depth for slope stability because the area of concern is already being replaced with a retaining structure. The soil layers were omitted and treated as one layer of cohesionless sand with $\phi' = 32°$.

The base failure for the Southeast slope was found to have a factor of safety of 2.63. Calculations and the Excel table can be found in Appendix B2.

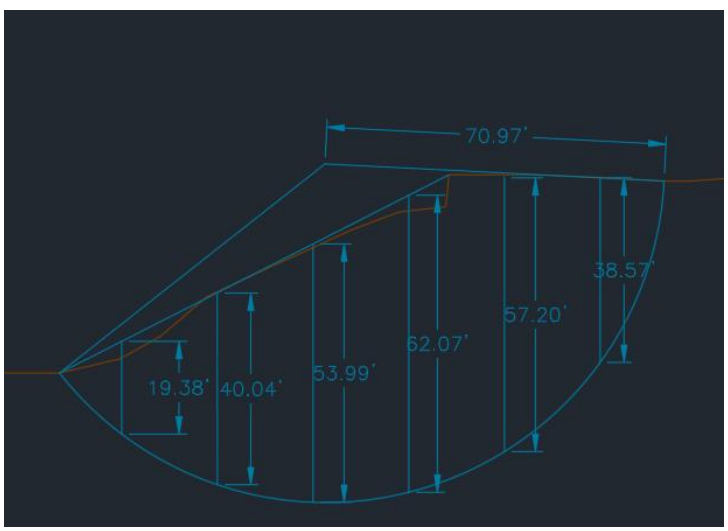


Figure 3.14 Method of Slices Diagram 1

The Toe failure for the Southeast slope was found to have a factor of safety of 1.11. Calculations and the Excel table can be found in Appendix B2.

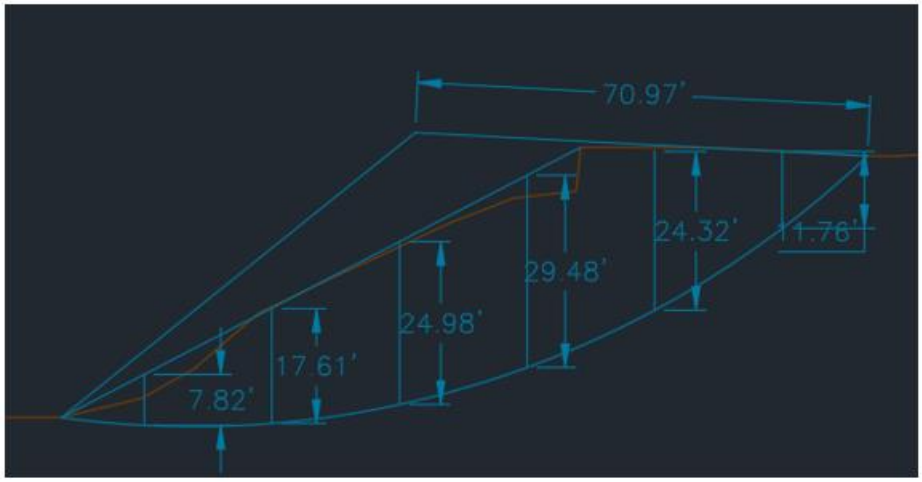


Figure 3.15 Method of Slices Diagram 2

3.8 Retaining Wall Design

A retaining structure must be designed along the bike path on the Northwest section of the site. CDOT prefers MSE retaining structures for tier-1 bridge designs.

3.7.1 Site Geometry, Parameter Review, and Basic Retaining Wall Dimensions

The retaining structure should be constructed adjacent to the bike path, and far enough away from the abutment such that it is not directly supporting abutment loads. Using the cross-section of the site, the rough preliminary profile view of the retaining structure is shown below.

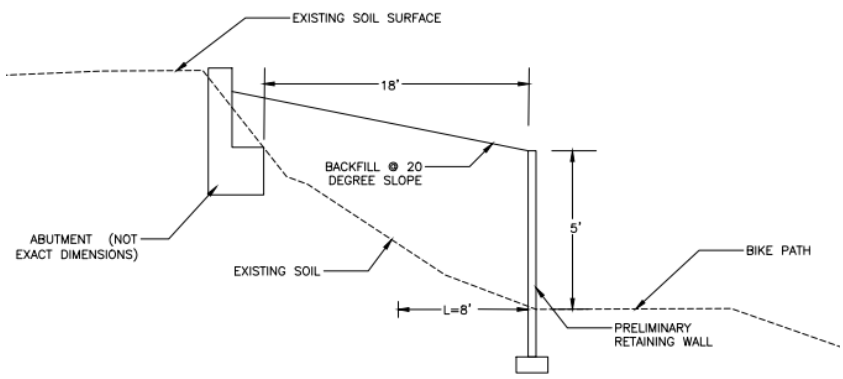


Figure 3.16 Retaining Wall Initial Estimate of Dimensions

The retaining wall should extend along the side of the bike path beneath the bridge, roughly 123 feet long. The ends of the retaining wall will be angled into the slope for aesthetic and containment purposes, but the retaining wall strength and stability will only be checked for the straight portion.

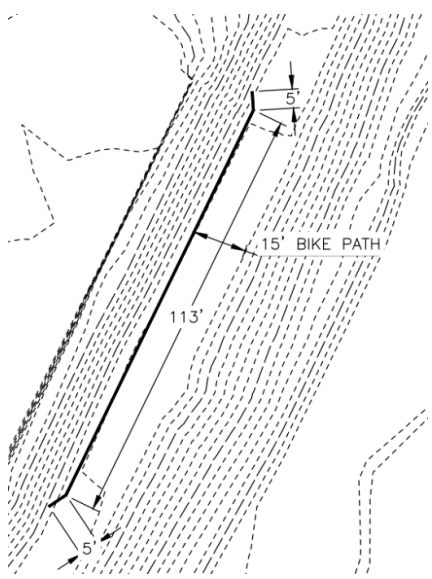


Figure 3.17 Retaining Wall Plan View

The initial retaining wall design dimensions were based on the site geometry and guidelines in the Federal Highway Administration's Mechanically Stabilized Earth Walls design guide (FHA, 2009).

Tables 3.22 Initial Dimensions and Parameters of Retaining Structure

Dimensions		
Height (H)	5	Feet
Reinforcement Length 1	8	Feet
Reinforcement Length 2	4.5	Feet
L	8	Feet
B	8.5	Feet
h	7.91	Feet
Unit Weight of Reinforced Soil	125	PCF
Geosynthetic Reinforcement	N/A	N/A
Spacing of Reinforcement	1	Feet

Site Parameters		
Parameter	Degrees	Radians
Effective Friction Angle of Retained Soil (Phi')	32	0.56
Slope of Backfill (Beta)	20	0.35
Friction Angle between backfill and reinforced soil = Beta	20	0.35
Angle of Wall (Theta)	90	1.57
Backfill Bulk Unit Weight	135	PCF

3.8.2 Detailed Retaining Wall Dimensions, Capacities, and Factor of Safety

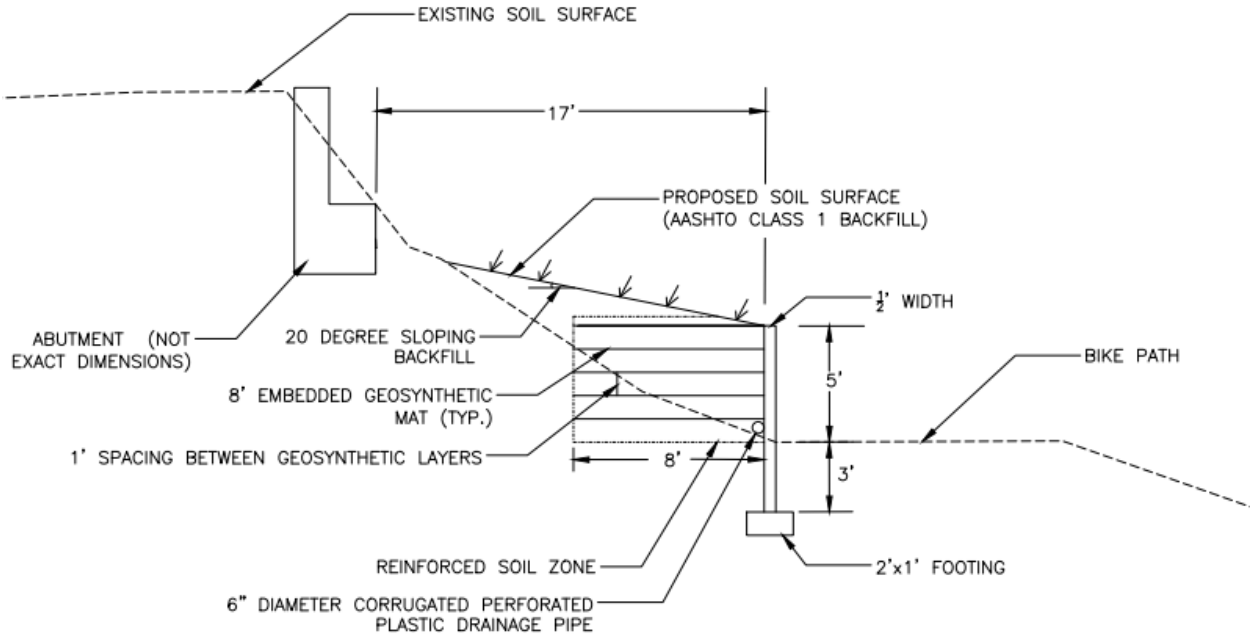
The retaining wall plan and a table of its factors of safety against external and internal modes of failures is shown below. Detailed calculations and additional narrative describing methodology can be found in Appendix B2.

Tables 3.23 F.S. Table for Retaining Structure

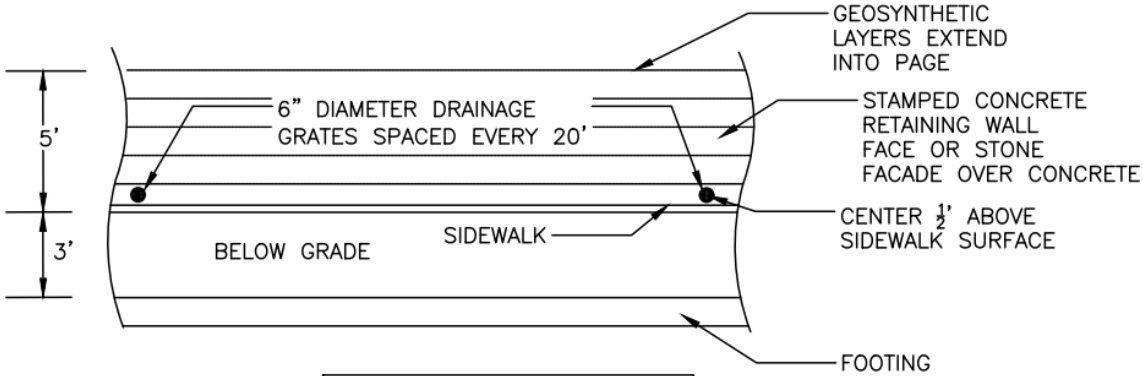
External Stability Checks		
Stability Check	Resistance Factor	Factor of Safety
Sliding	1	3.24
Eccentricity	0.65	22.22
General Bearing	0.65	10.30
Internal Stability Checks		
Stability Check	Resistance Factor	Factor of Safety
Tensile Resistance	Built into Calculations	1.55
Pullout Failure	Built into Calculations	6.35

Due to the resistance and design factors applied during the calculations, a factor of safety of greater than 1.5 for the checks is acceptable. This also follows from the F.S given in the Preliminary

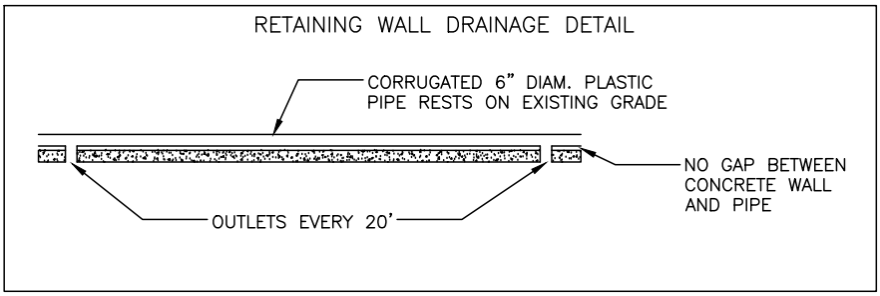
Geotechnical Report. The final retaining wall design is a 5 foot high wall, embedded 3 feet beneath existing grade as per FHWA requirements, (FHA, 2009). The retaining wall has 6 layers of Miragrid 2XT biaxial geogrid, with 1 foot spacing in between each layer. All layers of reinforcement extend 8 feet into the soil (the minimum embedment). Drainage features required for the wall are 6” diameter drainage grates spaced every 20 feet along the wall with a 6” diameter corrugated drainage pipe running behind the base of wall to transfer water out following standard drainage design for retaining walls of this size (Plumbing, 2024). Note that FHWA provides details for a typical swale, but this introduces unnecessary material costs for this size of retaining wall. The retaining wall drawings and drainage detail are shown below.



TYPICAL SIDE PROFILE OF RETAINING STRUCTURE



FACE OF RETAINING STRUCTURE



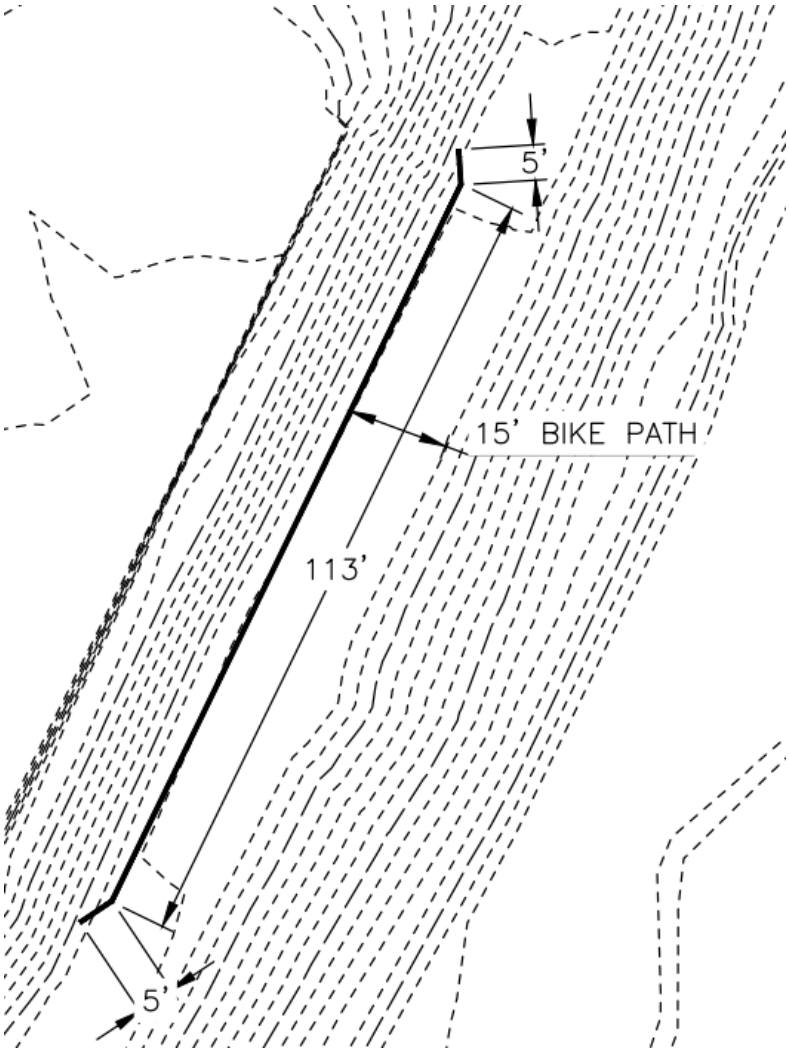


Figure 3.18 Retaining Structure Plans

3.9 Alternative Retaining Wall Options

Several other types of retaining walls were listed in CDOT's Bridge Design Manual including cantilever walls, drilled shaft (secant pile) walls, gravity walls, landscape walls, and soil nail walls (CDOT, 2022). Most of these alternative options are not suitable for this site for a variety of reasons. Soil nail walls also internally stabilize the soil similarly to the selected MSE wall, but soil nails are typically used in cohesive soils which can stand up unsupported during construction (CDOT, 2022). The backfill of the retaining wall on this site is primarily granular and thus a MSE wall is more appropriate. A gravity wall is not only inefficient from a sustainability perspective (requiring a large amount of structural concrete), but on this site it would be extremely difficult to construct a sufficiently large gravity wall adjacent to the bike path. A landscape wall retains less than 4 vertical feet of soil and is practically non-structural, which is not sufficient for this site. Secant pile walls are the most promising alternative, but CDOT specifically mentioned that they prefer MSE walls when possible. An MSE wall should also require less structural material than a secant pile wall and is therefore more efficient and sustainable in this case.

Another alternative option is using metal sheets to stabilize the soil in the MSE wall rather than geosynthetics. Metal sheets are less environmentally friendly and require more laborious construction (rather than just rolling out geosynthetics). However, the advantage of metal sheets is their properties are well understood and they are very durable. In this design, a geosynthetic appropriate for a 100 year design life was selected, which should be more or less equivalent to metal sheets in durability and maintenance requirements. Ultimately a mechanically stabilized earth wall constructed with geosynthetic stiffening elements was selected as it is sustainable, durable, and consistent with CDOT's preferences. The retaining wall was selected specifically for sustainability. Not only is an MSE wall more sustainable than a gravity wall (which requires a large volume of structural concrete and steel) but geosynthetics were selected over steel sheet soil stiffeners as they are far more sustainable in both construction and manufacturing. A table with the top three options of retaining wall types is shown below which was used to select the wall type.

Tables 3.24 Qualitative Pro/Con List of Retaining Wall Options

Retaining Wall Type	Pros	Cons
Secant Wall	• Reasonable Spatial requirements • Commonly Used • Reasonable Cost	• Not aesthetically appealing • Lots of steel and concrete • Non-permeable
MSE Wall	• Sustainable • Recommended by CDOT • Low material costs	• Difficult construction • Maintenance requirements • Obtaining geotextiles can be difficult
Soil Nail Wall	• Sustainable • Recommended by CDOT	• Not commonly used for granular soil

3.10 Resilience and Sustainability of Foundation

The selected drilled shaft (caisson) foundation system is resilient as it was constructed with a sufficiently large factor of safety and followed CDOT's tier 1 bridge recommendations and AASHTO specifications. Potential issues with the resilience of the foundation include instability under lateral loading and bending/tensile loads due to concrete's weak tensile capacity and the use of minimum steel reinforcement. Another potential issue is cracking which could occur due to temperature changes, differential settlement, or water infiltration. Cracking of the drilled shaft foundation could lower capacity and lead to corrosion and rust of the reinforcing steel. Overall, the foundation is resilient and designed to withstand all factored structural loads with a F.S. of 2.25. It is also constructed from well-researched materials using a well-understood design methodology, further adding to our confidence that this foundation will withstand the 75 year design life of the bridge. Furthermore, the foundation was designed with the geotechnical hazards in mind. Scour was considered as the capacity of the foundation comes from the hard bedrock rather than the scour-prone sand. The soil and bedrock around the foundation should always be saturated and pore water pressure for slope stability was overestimated to account for flooding – which adds to the resilience of the foundation design under flooding conditions.

The foundation system is not extremely sustainable. The drilled shafts scored lower than driven piles in sustainability (see section 3.5) due to the large quantity of reinforced concrete required. Concrete is not sustainable due to its CO2 emissions and difficulty to reuse. However, we chose to use a factor of safety of 2.25 (allowed by CDOT given our methodology) rather than 3.0, which significantly reduced the quantity of structural concrete. This increased efficiency resulting from a relatively low factor of safety makes our foundation more materially and economically sustainable. In order to make the design as sustainable as possible, minimal impact construction methods should be used during the construction of all geotechnical and foundation elements. This is why the diameters of the shafts were relatively small, preventing the need for extremely disruptive drilling equipment.

Section 4: Structural Design

4.1 Design Alternatives & Decision Matrix

Referring to section 1.4, which presents our design alternatives and the decision matrix, we now provide an in-depth summary of the process in which the scores were determined for the decision matrix.

	Alternative Designs	Life Cycle Cost	Constructability	Time to Complete	Sustainability	Settlement	Public/Environmental Health	Safety	Impact on Traffic	Matching Existing Conditions	Force of Water on Piers	Average Score
Structure	(weight)	9	10	10	10	8	10	10	8	9	4	
	3-Span	9	9	8	9	8	9	10	9	9	8	78
	2-Span	10	7	8	9	9	9	10	9	9	4	76.1

The 3-span bridge scored lower on lifecycle cost because the pier bases are prone to scour and degradation due to proximity to water and potential for cracking due to settlement. In the 3-span alternative, the additional piers (6 instead of 3) mean more piers will need to be maintained and there is a slightly higher probability of an issue with at least one pier.

In terms of constructability, the 3-span allows for smaller girders which are much easier to place. The piers are also closer to the riverbank than in the middle of the river as in a two-span solution.

Time to complete, sustainability, public health, safety, traffic impact, and existing conditions were roughly the same in both alternatives and scored accordingly. Both bridges were designed to maximize sustainability and reduce negative impacts on public/environmental health. Both bridges are designed with worker and construction safety in mind and designed to match existing conditions while minimizing impacts on traffic and it is impossible to assign one a higher score than the other in these categories.

The two-span experiences slightly more settlement because there are far more concentrated loads, leading to high vertical settlement commonly experienced by soil under large point-load foundations. Furthermore, the two-span solution has higher force of water on the piers due to the location of the piers in the center of the river and the large area of the pier bases transverse to the direction of flow.

Overall, the 3-span is a slightly better option than the 2-span design. Therefore, the 3-span bridge solution was selected as our final design. However, the two-inch isolation gap in the two-span solution allows for transverse expansion of the bridge, making it more resistant to thermal expansion due to temperature changes as well as lateral movements and differential settlement. Accounting for thermal expansion is particularly important in a 75-year bridge as the effects of climate change in the future are unpredictable and potentially extreme. This is an example of the intersectionality of sustainability, durability, and efficiency in our bridge's structural design.

4.1.1 Final Selection

Our design is a 3-span bridge with a 2-inch isolation gap between each direction of traffic that will be formed by applying closed-cell foam during the construction phase and stripping it when the project is complete. Therefore, the bridge deck will be two separate structures, but the centermost pier will be connected to both. Since the gap is so small, to drivers it will seem like the bridge is all one connected structure. The reason for doing this is it serves as a transverse expansion joint which decreases the lifecycle cost of the bridge while increasing its resiliency to temperature fluctuations, differential settlement, and natural hazards. This design will have 6 piers, with 3 spaced at the first span end and the other 3 at the next span end. Each set of 3 piers will be exactly parallel with each other. Having 3 separate piers allows for a simpler construction process, described in the phasing plan in Section 5 of this report. The spacing of the spans was determined from a recommendation from class speaker Cindy Otegui, PE, from Felsburg Holt & Ullevig that the middle span should be longer than the two outside spans. She provided equations to calculate each span length, and those lengths are shown below (See Appendix C, "Span Lengths" for full calculations).

$$L_2 = 117.7\ ft$$

$$L_1 = L_3 = 94.15\ ft$$

4.1.2 Risk Level and Objectives

The risk level on this bridge replacement is high because this is a main highway in Denver connecting several vital arterial roads and is extremely important to Denver's overall transportation system. We recognize that this is a very critical segment of the highway, and plan to conservatively design it accordingly. We plan to do this by ensuring that all materials being used are standard CDOT building materials, and by ensuring that all limit states are checked after design.

The intended use is to transport cars smoothly and safely. Some of the objectives of consideration include minimizing veering off the existing roadway in order to not cause a large curve which would change the traffic speed. Also, we plan to design the bridge to cause minimal impact to the bike path and keep it functioning during construction. We plan to be economically and materially efficient, minimizing the quantity of concrete and steel being used, all while minimizing construction schedule times.

4.2 Design Approach

4.2.1 Codes and Design Steps

The governing codes and standards that we are following include the AASHTO LRFD Bridge Design Specifications, 9th Edition, and the CDOT Bridge Design manual, 2022. Design steps include identifying the dimensions of the new structure, identifying the loads of different materials and vehicles from both the AASHTO and CDOT manuals, and multiplying the loads by their dimensions with the goal of getting them all into k/ft units. After this, the different loads were plugged into load combinations to determine the governing load combination that would be used to design the substructure. This governing load was used to plot the shear and moment diagrams, which displayed their respective maximum values. Using these values, reinforcement design was then conducted for the three primary elements; deck, girders, and columns. Lastly, the expansion behavior of the bridge was analyzed.

4.2.2 Limit States

The limit states that will apply include strength limit state, service limit state, and extreme event limit state. The strength limit state will evaluate the overall stability of the bridge. In the service limit state, the stress and deformation are checked. Since this bridge is over a flowing river, the extreme event limit state is important in extreme floods where high water elevations are expected.

4.2.3 Loads and Load Combinations

As stated earlier, the governing loads and load combinations were identified from both the AASHTO and CDOT manuals. After listening to presentations from current structural engineers, I learned that during conceptual design phase the primary loads that need to be considered are the dead and live load. The components of the Dead and Live Load that were considered are as follows:

- Dead Load
  - Asphalt Protective Layer
  - Concrete Deck
  - Metal Deck
  - Girders
- Live Load
  - Colorado Permit Vehicle (CDOT Bridge Design Manual 2022)
  - Design Truck (AASHTO LRFD Bridge Design Specs, 9th Edition)

These loads were identified from the two codes as shown in Appendix C, "Loads". Using the dimensions of our new bridge design, these loads were all converted into $\frac{k}{ft}$ units to be added together (See Appendix C, "Dead Load", "Live Load"). When total Dead Load and Live Load values were determined, they were plugged into AASHTO load combinations to determine the governing (maximum) one. The AASHTO load combination table that was used to govern the load on the bridge is Table 3.4.1-1 from the AASHTO LRFD Bridge Design Specifications, 9th edition (see Appendix C, "Governing Load Combinations").

This table accounts for many loads and events that weren't applicable in this conceptual design. Knowing that the loads being considered are dead and live, it can be seen from the table that either the Strength I or Strength II Limit States would govern.

As defined in the AASHTO manual, the Strength I Load Combination Limit State considers the design vehicle as the live load, and the Strength II Load Combination Limit State evaluates the permit vehicle specified by each state. Given this information, it was known that the live load values in Strength I and Strength II would differ. When considering Dead Loads, the only two that were applicable were DC and DW, which stand for Components and Attachments, and Wearing Surfaces and Utilities, respectively (AASHTO LRFD Bridge Design Specs, 9th Edition, 3.4.1-2). Both live load values include a design lane load, as defined in the AASHTO manual 3.6.1.2.4. (See Appendix C, "Governing Load Combinations for full calculations.)

All that was still needed to plug into the load combinations was the γ_p factor. The value of this factor was found in the AASHTO manual as 1.25 for DC and 1.50 for DW (AASHTO LRFD Bridge Design Specs, 9th Edition, 3.4.1-2). Now, I was ready to plug into the load combinations, and determine which one would govern.

Calculating Strength I and Strength II, we find that:

$$Strength\,I\ = 53.9\ \frac{k}{ft}$$

$$Strength\,II\ = 66.9\ \frac{k}{ft}$$

Since Strength II is greater, we know that Strength II governs, and that $66.9\ \frac{k}{ft}$ is the governing load.

4.2.4 Material Selection

The materials that were selected for this design include Class D Concrete, (150 pcf) and #4, #5, and #9 galvanized rebar. The reason for selecting the Class D concrete is due to reviewing CDOT Section 601 about structural concrete and seeing that Class D concrete offers high strength and is used in applications where load bearing capacity is critical. Also, because of its enhanced durability, structures built with Class D concrete require less frequent maintenance and repair. Lastly, Class D concrete has good sulfate resistance, and since this bridge is over a water source, there is a higher chance of sulfate exposure. One disadvantage of it is that the curing time of it is longer than some of its alternatives, like Class B, however this curing time has been accounted for in the schedule (see Appendix D).

The reinforcement sizing was determined from each element's design, and the reason that the reinforcement was chosen to be hot dip galvanized is because after reviewing the CDOT 100-year design life guidelines, it was seen that they provide corrosion resistance. These bars have historically been seen as a strategy to protect from damage due to bending, transport, or construction handling. These bars are used as a substitute for epoxy-coated rebar for Tier 1 applications.

The bridge expansion device, which offers 0 to 4 inches of expansion, is the selected expansion joint on one end of the bridge. Due to the thermal expansion calculations, there is an estimated 2.64 inches of expansion in the longitudinal direction (See Appendix C, "Longitudinal Expansion"), which is why one of these expansion devices was selected.

To conclude, the combination of concrete, rebar, and expansion joint selection make this bridge design the best possible design because they were chosen to minimize long-term maintenance, and avoid any problems that could cause long, expensive repairs.

4.3 Shear and Moment Diagrams

Using the SAP2000 12th edition modeling software, the 3-dimensional bridge was modeled, and shear and moment diagrams were generated. Shear and moment diagrams were generated for both the bridge's longitudinal and transverse sides. The longitudinal side, which spans a total of 306 feet in length is the 3-span bridge view with the span lengths shown as calculated above in section 4.1 ($L_1 = L_3 = 94.15\ ft$, $L_2 = 117.7\ ft$). The diagrams were modeled by applying the $66.9\ \frac{k}{ft}$ distributed load that was calculated above. The following maximum values were found from the diagrams (see Appendix C, "Shear and Moment" for diagrams).

Longitudinal View (306 ft span):

Shear:

$$Maximum\ Shear = \frac{4664\ kips}{13\ girders} = 358.8\ kips$$

Moment:

$$Maximum\ Moment = \frac{71654\ kips - ft}{13\ girders} = 5511.8\ kip - ft$$

The shear and moment diagrams are also presented for the transverse cross section of the bridge. This cut was taken at the piers, as opposed to at the end, because at the piers, there are only the supports from the pier, and no earth supports on the wide ends of the bridge, which caused this to have greater maximum shear and moment values compared to those at the start of the bridge, where it connects to the existing roadway. The transverse view has 3 evenly spaced piers at each span end, and this is what is being analyzed in the shear and moment diagrams (see Appendix C, "Shear and Moment").

Transverse View (106 ft span):

Shear:

$$Maximum\ Shear = \frac{4778.4\ kips}{13\ girders} = 367.6\ kips$$

Moment:

$$Maximum\ Moment = \frac{74740.9\ kip - ft}{13\ girders} = 5749.3\ kip - ft$$

After analyzing all these values, it was found that the maximum moment was, $M_{max} = 74740.96\ k - ft$ and the maximum reaction was $P_{max} = 8706.07\ k$. Since the SAP2000 model analyzed the 3D bridge as one complete structure, these values need to be divided by their respective bridge elements. The maximum moment will be divided by the number of girders, and the maximum reaction force will be divided by the number of columns that it is going into.

$$\frac{M_{max}}{\#\ of\ girders} = \frac{74740.96\ k}{13} = 5749.3\ k - ft$$

$$\frac{P_{max}}{\#\ of\ columns} = \frac{8706.07\ k}{3} = 2902\ k$$

4.4 Column in Flexure

Given that the bridge is above the South Platte River, the force of the water on the piers must be considered, since this force will put the column in flexure. After discussing with the water lead of the group, I was told that the force of the water on the piers could be narrowed down to a point load of 336 kips (see Appendix A for calculation). When a column is in flexure, the worst-case scenario is when the point loading in at the midpoint of the column. For this reason, the column was modeled in a second modeling software, RISA 3D, with the force of the water of 336 kips acting horizontally as a point load at the midpoint of the column, and the compression load of 2902 kips acting vertically downward on the top of the column. The shear and moment diagrams are shown in Appendix C, "Column in flexure". The maximum values from these diagrams are shown below.

Shear:

$$Maximum\ Shear\ from\ Flexure = 336\ kips$$

Moment:

$$Maximum\ Moment\ from\ Flexure = 6048\ kip - ft$$

This maximum moment value will be later used in the design of the column's rebar reinforcement calculations (Section 4.7, "Column Design").

4.5 Girder Design

The selection of girders is from the CDOT Standard Use girders, and the selected girder is the CDOT CBT 54 girder from the PCS Precast girder selection document. CDOT has standard girder depths that are usually used, and all other dimensions do not vary (See detail in Appendix C, "Girder Selection").

The girder spacing of 8 feet was determined by analyzing the span length graph and finding the girder spacing that intersects the CBT 54 line at the bridge's respective span length (See Appendix C, "Girder count").

After determining the girder spacing, the number of girders can be calculated.

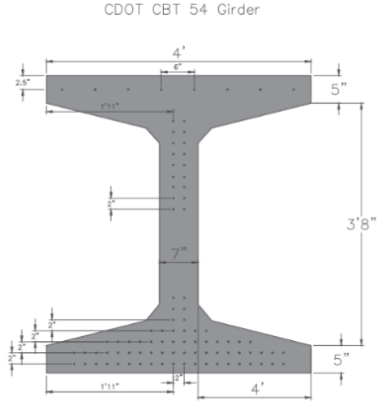
$$\frac{bridge\ width}{girder\ spacing} = \frac{107.25\ ft}{8\ ft} = 13.4\ \rightarrow\ 13\ girders$$

The girder depth was determined by utilizing Table 2.5.2.6.3-1 from the AASHTO bridge design manual (See Appendix C, "Girder depth"). From this table, the longest span length is multiplied by a factor to determine your minimum girder depth.

$$Minimum\ Girder\ Depth = 0.040\left(117.7\ ft \cdot \frac{12}{1}\frac{in}{ft}\right) - 12 = 44.5\ in$$

Since the minimum girder depth is 44.5 inches, and the next highest CDOT standard girder depth is 54 inches, choose 54 inches as the girder depth.

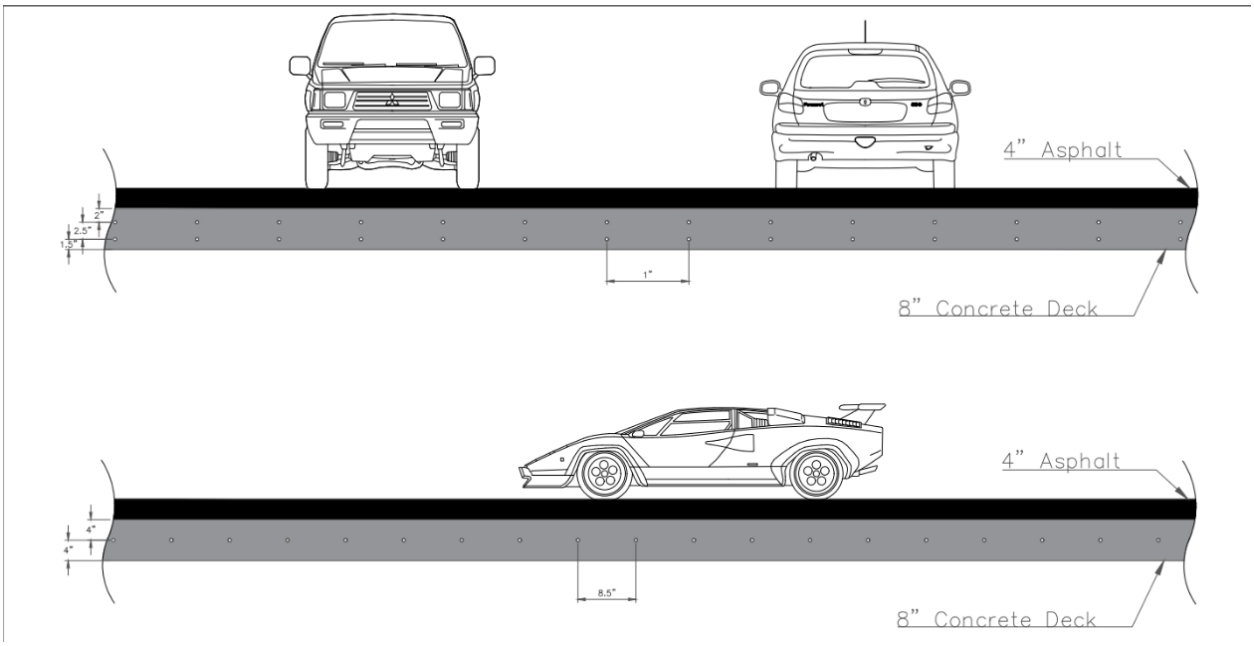
The rebar in the girder was calculated by following Example 6.4 from the *Design of Concrete Structures, 15th Edition* textbook. Following this example and calculating the required area of steel given a maximum moment, it was found that the girder would be reinforced with 85 #5 bars (See Appendix C, "Girder design"). The layout of the bars was determined by following the design example found in the CDOT PCS Precast Girder Sections and is shown below.



4.6 Deck Design

In order to design the rebar in the deck, Table 9-1 from the 2022 CDOT Bridge Design Manual was utilized (See Appendix C, “Deck Design”). Since it was previously determined that the girder spacing is 8 feet on center, and the concrete deck thickness was 8 inches, these values could be used to determine the transverse and longitudinal reinforcing of the deck.

The transverse reinforcing uses #5 rebar spaced at 8.5 inches, and the longitudinal reinforcing uses #4 rebar spaced at 12 inches in the top of the deck, and #5 rebar spaced at 12 inches in the bottom of the deck. A sketch of how this rebar will be laid out in the deck is shown below. Cars are shown in the images to define the direction of rebar placement.



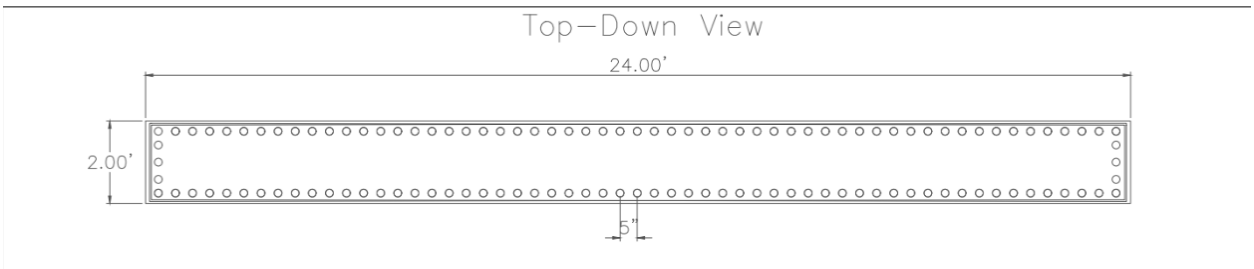
After selecting the rebar, check calculations were ran to check shear and to check reinforcement limits and minimums in both the longitudinal and transverse directions. These check calculations, which all passed, can be found in Appendix C, “Deck Design”.

4.7 Column Design

After selecting the column's dimensions, the interaction diagram that can be found in Appendix C, “Column Design” was used to design the rebar reinforcement in each column.

Using the maximum moment and compression values found from the RISA 3D model, where the column was put in flexure, we were able to go through a calculation using the interaction diagram to ultimately determine the necessary area of steel needed to support the combination of the moment and compression on the column (See Appendix C, “Column Design”).

These calculations found that there would be 128 #9 bars at 5-inch spacing in each column. A detail of this rebar layout is shown below.



This rebar design includes #4 stirrups at 12-inch vertical spacing, up the height of the columns (See Appendix C, “Column design”).

4.8 Expansion Behavior

Thermal expansion is something that must be considered during bridge design. Because thermal expansion can occur in both the longitudinal and transverse directions, both must be checked. After referencing CDOT bridge expansion joint example #5, which considers a strip seal expansion device which allows for 0 to 4 inches of expansion, we were able to perform our expansion calculations.

The following formula was used to calculate expansion.

$$\Delta L = l_o \cdot \alpha \cdot \Delta T$$

In this equation, l_o is the original length, α is the coefficient of thermal expansion, and ΔT is the change in temperature in Fahrenheit. Following the example, the coefficient of thermal expansion was found to be $\alpha = 6 \cdot 10^{-6}$ and the change in temperature was found to be $\Delta T = 120°F$.

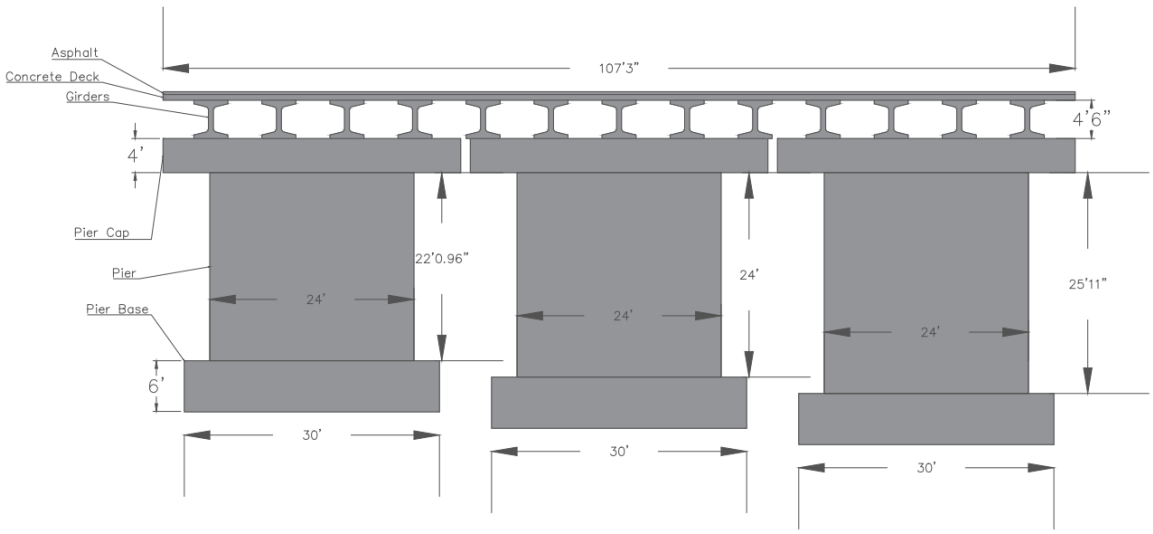
The l_o value was the length of the bridge when calculating for the longitudinal expansion, and the width of the bridge when calculating for the transversal expansion.

The calculations found that in the longitudinal direction, $\Delta L = 2.64\ inches$ and in the transversal direction, $\Delta L = 0.93\ inches$ (See Appendix C, "Expansion behavior" for full calculations).

The isolation gap that our design includes accounts for the lateral expansion of 0.93 inches, because that gap is 2 inches wide. The longitudinal expansion is accounted for with the expansion device that is placed at one end of the bridge, because it allows for up to 4 inches of expansion.

4.9 Structural Design Conclusion

This structure is a 3-span bridge, and at each span there are three piers. The bridge is 107.25 feet in width, and 306 feet in length. Working from top to bottom, there is a 4-inch asphalt protective layer, and an 8-inch concrete deck. Beneath that are 13 CDOT standard CBT 54” girders at 8-foot on center spacing. Then each of the 3 piers have a 4-foot tall pier cap, and 24-foot wide pier, and a 6-foot tall pier base. This design uses Class D concrete, and galvanized bars to minimize long-term maintenance. There is a 2-inch isolation gap between each direction of traffic, allowing for lateral expansion, and a 0 to 4 inch strip seal expansion device at one end of the bridge.



This bridge was designed to minimize construction costs while maximizing its lifetime before repair. This was done by selecting the optimal materials, spacing of spans, and number of piers at each span end. This is the best possible structural design choice for this bridge replacement.

Section 5: Construction Management

At Alpine Engineering, we pride ourselves on our commitment to excellence in construction management, with safety as our paramount concern. Our team is dedicated to surpassing industry standards and delivering exceptional results for our clients. Using the latest technology and innovative construction techniques, we strive to elevate every project to new heights.

The next steps for Alpine Engineering involve translating our design into construction documents to adjust the budget and schedule. Additionally, we will solidify lead times to ensure seamless coordination with subcontractors and suppliers. Simultaneously, we will pursue the necessary permits and navigate the review process efficiently. Following the award of the bid, our immediate focus will be towards initiating the 404 permit process to ensure a successful project execution.

5.1 Stakeholders

In the South Platte River, I-270 Bridge Replacement Project managed and owned by the Colorado Department of Transportation (CDOT), several stakeholders need to be involved or informed about traffic control and phasing plans. These stakeholders play various roles in the project, and all contribute to its successful execution.

Project Name	I-270 Bridge Replacement	**Date**	4/18/2024
Project Manager	Lauren Bone	**Project Owner/Client**	CDOT

Stakeholder Name	Title/ Project Role
CDOT	Project Owner
Alpine Engineering	Execution of Construction and Design
State of Colorado	Regulatory
Excel Energy	Identify and Relocate Utilities
Colorado Department of Public Health	Environmental Impact

Emergency Services	Public Safety
City of Commerce	Adjacent Property Owner
Public service CO of Colorado C/O Property and Local Taxes	Adjacent Property Owner
Christenson Justin D aka Shirley Justin D UND 3/4 INT/Klein Ben UND 1/4TH INT	Adjacent Property Owner
A T and W INC	Adjacent Property Owner
Lammers Harold R Jr	Adjacent Property Owner
Metropolitan Denver Sewage Disposal DIST NO.1	Adjacent Property Owner
Keesen Landscape	Adjacent Business- along 64th avenue
Pro Box Portable Storage	Adjacent Business- along 64th avenue
Animal Pest Control Specialists, Inc	Adjacent Business- along 64th avenue

Table 5.1: Stakeholder list register

5.2 Interruptions

Starting in January 2025, a significant project that we believe will greatly benefit the transportation infrastructure in the Commerce City area will commence. Our team at Alpine Engineering is prepared to collaborate with CDOT on replacing eastbound and westbound bridges on I-270 over the South Platte River.

Our proposal entails the construction of two bridges separated by an isolation gap, that are three spans to replace the existing structures, aiming to improve safety and efficiency along this vital transportation route.

To ensure minimal disruption to traffic flow during construction, we have developed a comprehensive plan:

- We will maintain two lanes of travel in each direction during peak hours, with necessary shoulder narrowing.
- Construction activities will be strategically scheduled behind concrete barriers on weekdays to minimize impact on commuters.

- Lane shifts will be implemented in both eastbound and westbound directions to facilitate construction operations.
- Intermittent nightly single lane closures from 10 p.m. to 5 a.m., Monday through Thursday, will allow for essential work to progress while minimizing disruptions to daytime traffic.

We understand the importance of transparency and effective communication throughout this process. Therefore, we recommend that the public utilize CDOT's website for regular updates on the project and its associated traffic impacts.

Furthermore, we are aware of potential impacts on the South Platte River path during construction:

- Pedestrian access will be maintained throughout the construction period.
- To ensure the safety of pedestrians and construction workers alike, intermittent flaggers will be deployed to oversee equipment movement and crane operations.

We are committed to executing this project with the utmost professionalism and efficiency, in alignment with CDOT's objectives for enhancing transportation infrastructure in Colorado.

5.3 Detailed Cost Estimate

At the design development stage, the overall cost of the I-270 South Platte River Bridge Replacement Project is **$34,408,000**. This cost estimate is based on the I-76 over York Street CDOT project, scaled to our project quantities (section 5.3.1) and inflated for the future years. From CDOT's 2023 cost index an average inflation rate of 1.6% was found between 2022 and 2023 (appendix D). Assuming an inflation rate of 3% for the following years, the value was compounded annually to find the item cost in 2025. The cost estimate is broken down into supervisory labor, different packages and markups (section 5.3.3).

DETAILED ESTIMATE

I-270 Bridge Replacement Project

Cost Summary

	Item		Total Cost
Cost of Work	Supervisory Labor		$2,093,779
	General Conditions		$3,376,296
	Existing Conditions		$2,085,945
	Substructure		$3,041,993
	Superstructure		$2,927,803
	Finishes/Specialties		$2,711,642
	Earthwork		$6,111,976
	Exterior Improvements		$2,723,128
	Utilities		$509,284.88
	Subtotal		$25,581,848
	Design Cost (10%)	10.00%	$2,558,185
	Contingency (15 %)	15.00%	$3,837,277
	Overhead & Profit (7%)	7.00%	$1,790,729
	Insurance (1 %)	1.00%	$255,818
	Bond (1.5 %)	1.50%	$383,728
		Total	$34,407,586

Figure 5.1: Overall Cost Estimate of South Platte River I-270 Bridge Replacement Project

5.3.1 Quantity Estimates

The quantity estimates were determined by design decision and analyzing the I-76 over York Street project scaled to our project. Concrete, reinforcing steel and earthwork quantities heavily influence the cost of the overall project and require considerable planning to successfully execute this project. The big-ticket items are listed below (figure 5.2) and for further breakdown of quantities see (appendix D).

Big Ticket Quantity Estimates		
Total Concrete	2852	CY
Concrete: W/ 3% Waste	**2938**	**CY**
Reinforcing Steel (Pier Cap (Caisson), Girders, Piers, Deck, and Caissons)	301	Tons
Account for Reinforcing Steel (Pier Caps, Pier Base, Retainning Wall and Abutments)	190	Tons
Reinforcing Steel: W/ 5% Waste and lap	**515**	**Tons**
Earthwork	**10993**	**CY**

Figure 5.2: Quantity Estimates for Big Ticket Items

The quantity of earthwork value comes from coordination with our geotechnical engineer. Using AutoCAD, the current soil profile was used to find the area that needed to be excavated such that the access road and temporary equipment access has a 6% slope. The road was created as a grading object in Civil 3D and projected over the existing surface. Another quantity that involves earthwork will be the installation of the abutments which will require excavating into the existing soil profile to create a flat surface for them to rest on. To effectively phase traffic, road alignment will also require fill material to create a level road in between the existing eastbound and westbound roads (appendix D).

5.3.2 Equipment List

Another major quantity estimate involved is heavy equipment estimates. Two smaller cranes will be used on both sides of the South Platte River to ease critical picks required for girder placement. Excavators, dozers and compactors will be required for earthwork on the project. According to the Geotech report (section 3.2), the soil is mainly sand therefore vibratory compactors will be most effective for the existing soil on site. The quantity for the concrete trucks comes from the total concrete estimate, assuming each concrete truck will hold ten cubic yards of concrete.

Equipment Quantity Estimate					
Cranes	**Excavators**	**Compactor**	**Telehandler**	**Dozers**	**Concrete Trucks**
2	2	2	1	2	294

Figure 5.3: Quantity Estimate for Heavy Equipment

For a project of this magnitude, many types of heavy equipment will be required as listed below.

Heavy Equipment List:

- Cranes
- Excavators
- Dozers
- Grader
- Loaders

- Dump Trucks
- Vibratory Compactors
- Smooth Drum Compactors
- Pavers
- Skid Steers
- Telehandlers
- Concrete Trucks
- Concrete Pumps
- Drilling Rig

5.3.3 Detailed Pricing

Supervisory labor breaks down the field construction management required for a project of this scale. Their base rate and hours worked are analyzed for each year for a total of **$2,093,779.** The base rates are based on an average salary by position in Colorado and assuming a 5% salary increase each year.

Alpine Engineering	2024								2025												2026																		
Discipline Position/Title	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec	Jan	Feb	Mar	Apr	May	Jun	Jul	Total Hours	2024 Salary Rate	2024 Base Rate	2025 Base Rate	2026 Base Rate	Total Cost
FIELD CONSTRUCTION MANAGEMENT																																							
Project Manager	80	80	86	86	86	86	173	173	173	173	173	173	173	173	173	173	173	173	173	173	173	173	173	173	173	173	173	5348	120000	$ 58	$ 61	$ 64	$325,181.39
General Superintendent	30	30	30	30	20	20	20	20	45	45	45	45	45	45	45	45	45	45	45	45	45	45	45	45	45	45	45	1370	150000	$ 72	$ 76	$ 80	$104,209.44
Superintendent	40	40	40	86	86	86	173	173	173	173	173	173	173	173	173	173	173	173	173	173	173	173	173	173	173	173	173	5222	140000	$ 67	$ 71	$ 74	$370,897.52
Assistant Superintendent	40	40	40	86	86	86	173	173	173	173	173	173	173	173	173	173	173	173	173	173	173	173	173	173	173	173	173	5222	110000	$ 53	$ 56	$ 58	$291,419.48
Assistant Project Manager	80	80	86	86	86	86	173	173	173	173	173	173	173	173	173	173	173	173	173	173	173	173	173	173	173	173	173	5348	100000	$ 48	$ 50	$ 53	$270,984.50
Project Engineer	30	30	30	30	30	30	173	173	173	173	173	173	173	173	173	173	173	173	173	173	173	173	173	173	173	173	173	5024	80000	$ 38	$ 40	$ 42	$204,326.06
Field Engineer	0	0	0	0	0	0	173	173	173	173	173	173	173	173	173	173	173	173	173	173	173	173	173	173	173	173	173	4844	75000	$ 36	$ 38	$ 40	$185,065.29
BIM/VDC	0	30	30	30	30	90	90	90	90	90	90	80	80	80	80	80	80	80	80	80	80	80	80	80	20	0	0	2280	100000	$ 48	$ 50	$ 53	$115,016.83
Safety Engineer	0	0	0	0	0	30	80	80	80	80	80	80	80	80	80	80	80	80	80	80	80	80	80	80	80	80	80	2270	100000	$ 48	$ 50	$ 53	$115,548.08
Admin	80	80	80	86	80	90	90	90	80	80	80	80	80	80	80	80	80	80	80	80	80	80	80	80	80	80	80	2756	80000	$ 38	$ 40	$ 42	$111,130.77
																												39,684					$2,093,779.36

Figure 5.4: Supervisory Labor Detailed Cost Estimate

The following packages were used to break down the cost into further detail. The line items come from I-76 over York Street and are separated into their corresponding construction packages (appendix D).

	Item	Total Cost	Percentage of Project Total Cost
	Supervisory Labor	$2,093,779	6.1%
Cost of Work	General Conditions	$3,376,296	9.8%
	Existing Conditions	$2,085,945	6.1%
	Substructure	$3,041,993	8.8%
	Superstructure	$2,927,803	8.5%
	Finishes/Specialties	$2,711,642	7.9%
	Earthwork	$6,111,976	17.8%
	Exterior Improvements	$2,723,128	7.9%
	Utilities	$509,284.88	1.5%

Figure 5.5: Construction Packages Detailed Cost Estimate

5.3.4 Project Alternatives/ Value Engineering

Value Engineering and exploring project alternatives are important to analyze as they may offer possible cost and schedule reduction without compromising quality or functionality. Moreover, this fosters innovation by encouraging creative problem-solving throughout the project to promote sustainability and long-term success.

Alternative	Cost Impact	Schedule Impact	Pros	Cons
Keep pavement between existing roadways	+$30,000 Higher cost to create more permanent road conditions	+ 2 Weeks Adds time to create more permanent road conditions	Allows future lanes to be added in the future	Increased cost and schedule
Metal Guardrail	- $820,000 Difference in price for metal instead of concrete	N/A Replace time allotted for guardrail	Reduced cost Absorbs energy from collision Lightweight and durable	Nighttime glare Corrosion
Led Lights for river path	+$7,500 Higher cost for lights and	N/A Time added is negligible due to	Improves visibility for pedestrians and bikers	Increased cost

	electrical materials	nearby power		
Stamped concrete retaining wall	+$10,000 Higher cost for craft work	N/A Time added is negligible	Enhances project appearance	Increased cost More difficult to repair
Bridge Structural Health Monitoring System	+ $50,000 Higher cost for sensors and installation	N/A Time added is negligible	Reduces repair cost Extends bridge lifespan 24/7 monitoring	Increased cost

Table 5.2: Value Engineering/Project Alternatives

5.4 Construction Schedule

To ensure successful completion the South Platte River 1-270 Bridge Replacement Project, additional time will be needed outside of the initial request for proposal. Alpine Engineering strongly advocates for the project to start on **October 24th, 2024**, with an anticipated completion date of **June 22nd, 2026**. With the project being over the South Platte River, substructure work needs to be completed outside of high-water season to ensure safety of the crew. By completing the project in June, this gives the project time for potential weather delays and possible unforeseen conditions. The full MS Project Schedule and all milestones can be found in appendix D.

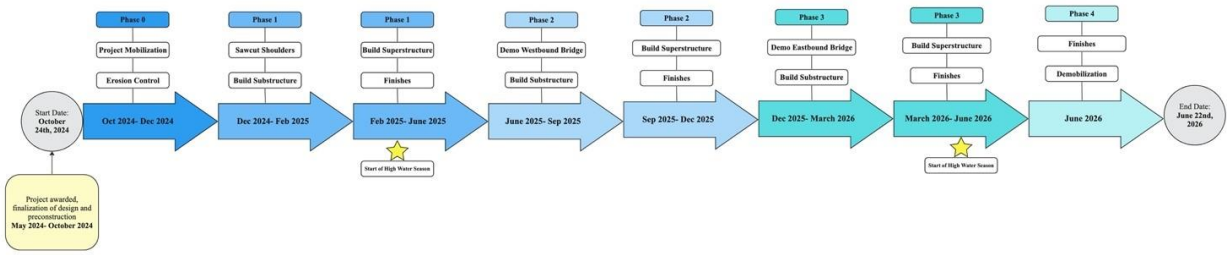


Figure 5.6: Schedule Graphic

The durations for activities were found by analyzing CDOT's I-76 over York Street project and scaled to our quantities and scope of work. The multiplier found was between one and two for each activity and was implemented in the detailed schedule (appendix D). Analyzing the I-76 project, this involves the reconstruction of two smaller bridges, an addition of an auxiliary lane and

reconstruction of an on-ramp. Our project involves just the reconstruction of a larger bridge which justifies how the duration factor is between one and two.

Assumptions:

- Working time is Monday-Friday, 7 a.m.-3:30 p.m. (including 1/2-hour lunch)
- Holidays: New Year's Day, Memorial Day, Independence Day, Labor Day, Thanksgiving Day and Christmas Day

5.4.1 Long Lead Time Items

Long lead times can affect the schedule and possibly cause delays if not properly planned for. These lead times can stem from high demand, limited production capacity and fluctuation in the raw material available.

Items with long lead times:

- Precast Concrete Girders
- Corrugated Deck Steel

Other non-material considerations that have long lead times include permits, specifically obtaining a 404 permit. This permit can take at least six months to acquire as it requires a thorough review of project plans, environmental impact assessment and consultations with regulatory agencies. The submittal and procurement process for these items have been accounted for in the schedule.

5.4.2 Applicable Permits

The South Platte River 1-270 Bridge Replacement Project is a large-scale construction project that requires numerous projects from federal, state and local authorities. These permits are in place to protect the environment, community and ensure compliance with sustainability mandated by issuing bodies. Below are the expected permits required on this project.

Permits Required:

<u>Federal:</u>

- Initial Plan of Operations
- NPDES- Water Discharges
- Corps 404- Wetlands
- SWPPP- Stormwater
- ESA- Consultation NOAA Fisheries
- Tribal Consultation
- Cultural/Historical

- Air Quality
- Waste Rock Handling
- Powerline R-O-W

State:

- Water Rights
- Cyanidation
- Plan of Operations
- Reclamation Plan
- Tailing Dam
- Steam Channel Alteration
- TMDL
- Ground Water
- Domestic Water Supply
- Wastewater Treatment

Local:

- Comprehensive Plan
- Zoning
- Local Health District
- Building Permits
- Road Use Authorization
- Conditional Use Permit

5.5 Formwork Design

Formwork design is crucial in construction projects to ensure the proper casting of concrete elements. Here are the needs for formwork design in the South Platte River I-270 Bridge Replacement Project.

The assumptions made in the formwork design are as follows:

- Use of Class D Concrete (unit weight: 150 pounds per cubic foot)
- Assume Concrete temperature is 60 degrees Fahrenheit (based on ASTM C 1064-86)
- Concrete placement rate of 1.8 ft/hour (see appendix D for calculation)
- Type I, II, or III cement without retarders is used
- All deflections are 1/360

Our plan for the cast-in-place, bridge abutment formwork system is as follows:

Bridge Abutment Formwork Design Decision			
	Type	Size	Spacing (in)
Sheathing	Douglas Fir	Plyform Class I 1 in	N/A
Studs	Douglas Fir	2x4	16
Wales (Doubled)	Douglas Fir	2x4	20
Ties	Snap Ties	2000 lb rate	20

Figure 5.7: Bridge Abutment Formwork Design Decision

Abutment Form Bracing Design Decision					
	Type	Size	Height from bottom of abutment to strut (ft)	Length from bottom of abutment to strut (ft)	Spacing (in)
Lateral Braces	Douglas Fir	3x4	7	6	12

Figure 5.8: Bracing Design Decision

5.5.1 Formwork Bill of Materials

The quantities listed in the figure below are for a single abutment since the formwork plans to be reused for the other abutment. Snap ties cannot be reused and therefore the quantity is for both the abutments in the project. The total quantity accounts for waste that will occur when constructing the formwork.

Bridge Abutment Formwork Bill of Materials									
Material	**Height**		**Width**		**Depth**		**Quantity**	**Waste**	**Total Quantity**
Plywood									
Sheathing	4	ft	8	ft	1	in	60	10%	66
Lumber									
Studs	8	ft	4	in	2	in	169	10%	186
Double Wales	8	ft	4	in	2	in	306	10%	337
Lateral Brace	10	ft	4	in	3	in	226	10%	248
Ties									
2000 lb Snap Ties							1380	5%	1449

Figure 5.9: Bridge Abutment Formwork Bill of Materials

5.6 Project Risks

On the I-270 Bridge Replacement Project over South Platte River, safety is paramount to protecting those on site and the general public. It is important to understand the risks present on the project and create a thorough plan to mitigate these risks which are as follows:

Risk	Mitigation Plans
Weather Conditions	• Build weather days into schedule • Builder's risk insurance for material damaged by weather • Plan concrete pours on suitable weather days (no rain, mild temperature, schedule for good weather months)
Environmental Impacts	• Consistent inspections on erosion control plan • Use of nets to catch falling debris from demolition • After swallow migration, place nets to prevent nests to form
Flood Risk	• Communication with water resource professional • Flood insurance • Use of sump pumps and drainage systems • Effective stormwater management
Differing Site Conditions	• Through site walk prior to start of construction • Review previous Geotech reports for understanding on the site • Review differing site conditions in contract
Security Risks	• Lock up equipment, trailers and Conex boxes when not in use • Sign-in list • Surveillance cameras
Falling Objects	• Use of whistles for objects overhead • Clear communication of critical picks • Use of radio system for crane services and different trades
Slips, Trips and Falls	• Use of handrails and guardrails • Secure ladders and ladder safety measures • Regular inspections to identify hazards • Good housekeeping practices to keep

	site clean of debris especially on walkways • Use slip resistant materials when possible • Use proper tie off techniques when applicable
Movement of Heavy Machinery	• Ensure equipment has working safety features such as warning signs, reflective markings and lights • Consideration of utility lines, weight restrictions and road conditions • Documentation of certified equipment operators
Hazardous Material	• Use of respiratory PPE during saw cutting • Proper storage and handling procedures • Use of proper PPE • Clear communication of potential hazardous materials on site
Unstable Soil Conditions	• Extensive site investigation and soil testing • Use of soil stabilization techniques • Ensure slope stability • Management of water
Proximity to Public	• Jersey barrier between traffic and construction • Proper signage • Community engagement • Dust and debris control • Noise management
Vehicular Accidents	• Detailed traffic management plan • Jersey barrier between construction and traffic • Install reflective tape and lights in low light conditions • Ensure work areas are well-lit
Collapse of Excavation	• Use of excavation support systems • Implement soil stabilization • Ensure proper access

	• Regular inspection of excavation and support systems
Design Change	• Thorough design planning and development • Clear communication between owner and contractor • Formalize change management processes • Weekly design meetings between owner and contractor
Cost Escalation	• Lock in price during buyout process • Allocation of contingency • Open communication to minimize cost impacts
Unqualified Subcontractor	• Use of field engineer to monitor daily work • Thorough review of subcontractor bids • Communication between contractor and subcontractors to ensure alignment of project goals
Missing/incomplete Drawings	• Collaboration with design team • Review drawings and design • Use of BIM • Perform site inspections to find any discrepancies between drawings and construction

Table 5.3: Project Risks

5.7 Erosion Control Plan

Choosing an erosion control system is a crucial aspect of project planning, to ensure the protection of local water bodies and storm drains from disruptions caused by on-site activities. The main goal is to maintain soil stability and prevent its displacement, and in this project, steep slopes, human activities, and the South Platte River pose significant erosion risks. Removing protective vegetation exposes soil to instability and increases its susceptibility to erosion. This risk is particularly high around the South Platte River after tree clearance and during excavation for the site.

To address these concerns and minimize environmental impacts, we are implementing a comprehensive erosion control plan. It is imperative to monitor resource damage, natural drainage,

runoff patterns, and foot traffic both on-site and around the I-270 South Platte Bridge replacement project. Temporary control measures will be employed based on the Colorado Department of Public Health and Environment best management practices, including:

- Silt Fence: Captures sediment at the bottom of slopes to ensure soil conservation. These will be placed near South Platte River and along contour of the slope.

Figure 5.10: Example of Silt Fence

- Riprap: Placed near crane pad along the riverbank to protect soil scour from erosion.

Figure 5.11: Example of Riprap

- Vehicle tracking pads: Placed at key vehicle access points to prevent soil and rocks displacement from the construction site.

Figure 5.12: Example of Vehicle Tracking Pad

Erosion Control blankets: Used on steep, exposed slopes as a barrier against soil erosion and protection of the soil surface

Figure 5.13: Example of Erosion Control Blanket

Other Considerations:

- Stockpile management
- Disposal of waste
- Material storage and delivery
- Demolition practices
- Dust control
- Concrete finishes and curing practices

All erosion control best management practices will receive regular maintenance to ensure environmental sustainability, regulatory compliance and long-term effectiveness. Erosion control drawings can be found on sheet C2.5.

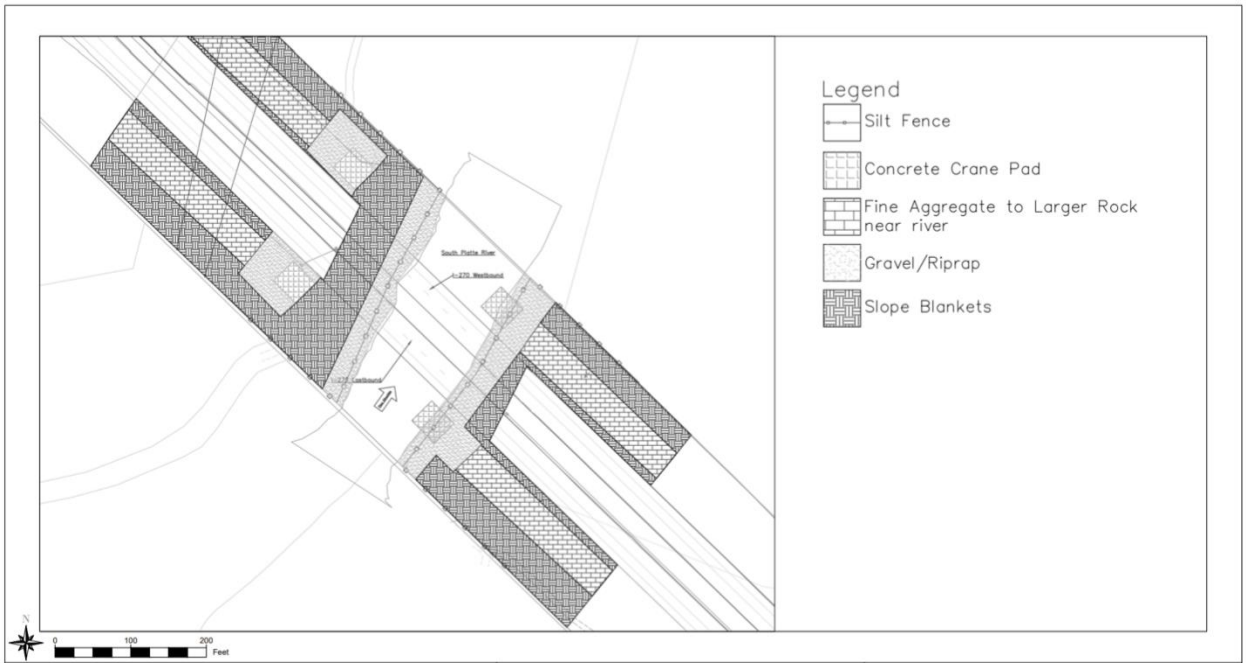


Figure 5.14: Erosion Control Plan

5.8 Traffic Engineering Plan

Creating a thorough traffic engineering plan is imperative for our project to optimize the movement of people and goods within an urban environment and ensure public safety. Our traffic engineering plan is dependent on the different phases (section 5.10) to ensure a seamless flow of vehicles during construction while prioritizing safety and sustainability. These traffic engineering plans can be found in drawing sheets TC1.1, TC1.2 and TC1.3.

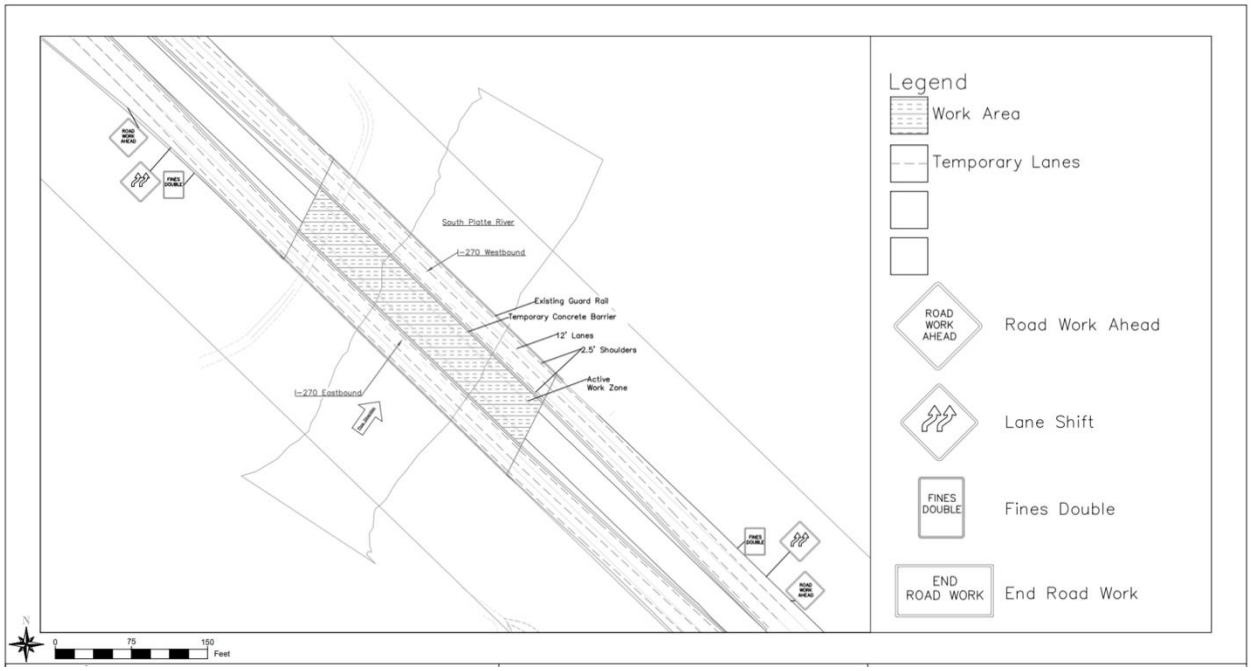


Figure 5.15: Phase One Traffic Engineering Plan

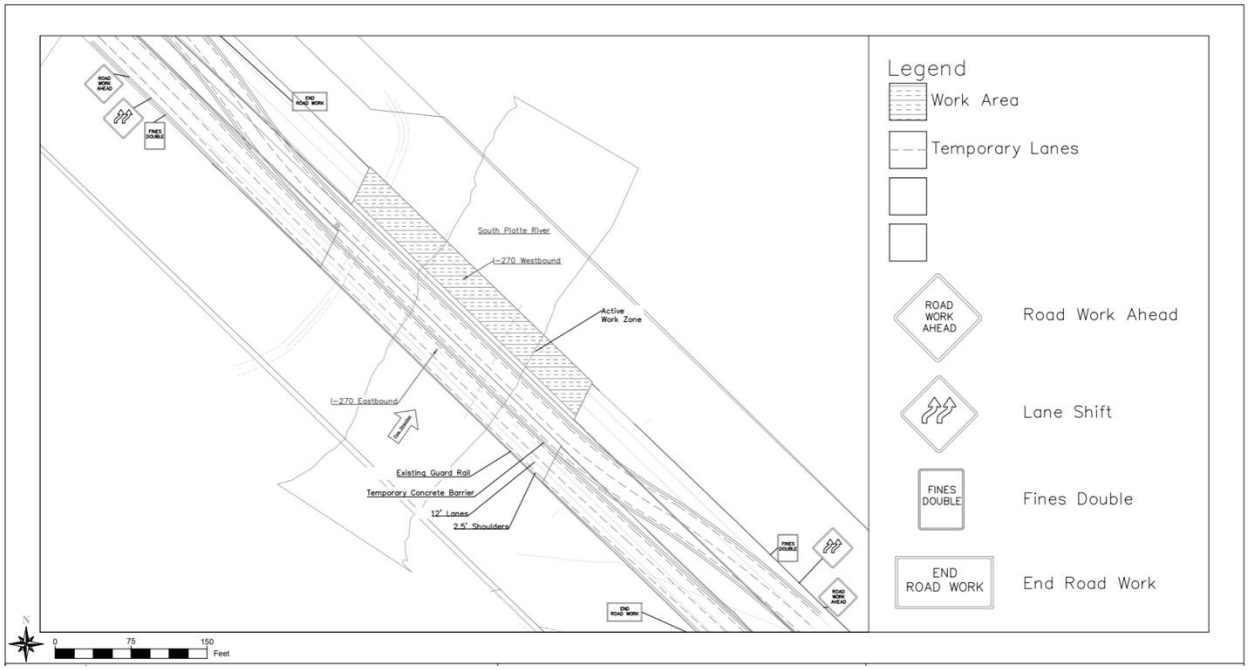


Figure 5.16: Phase Two Traffic Engineering Plan

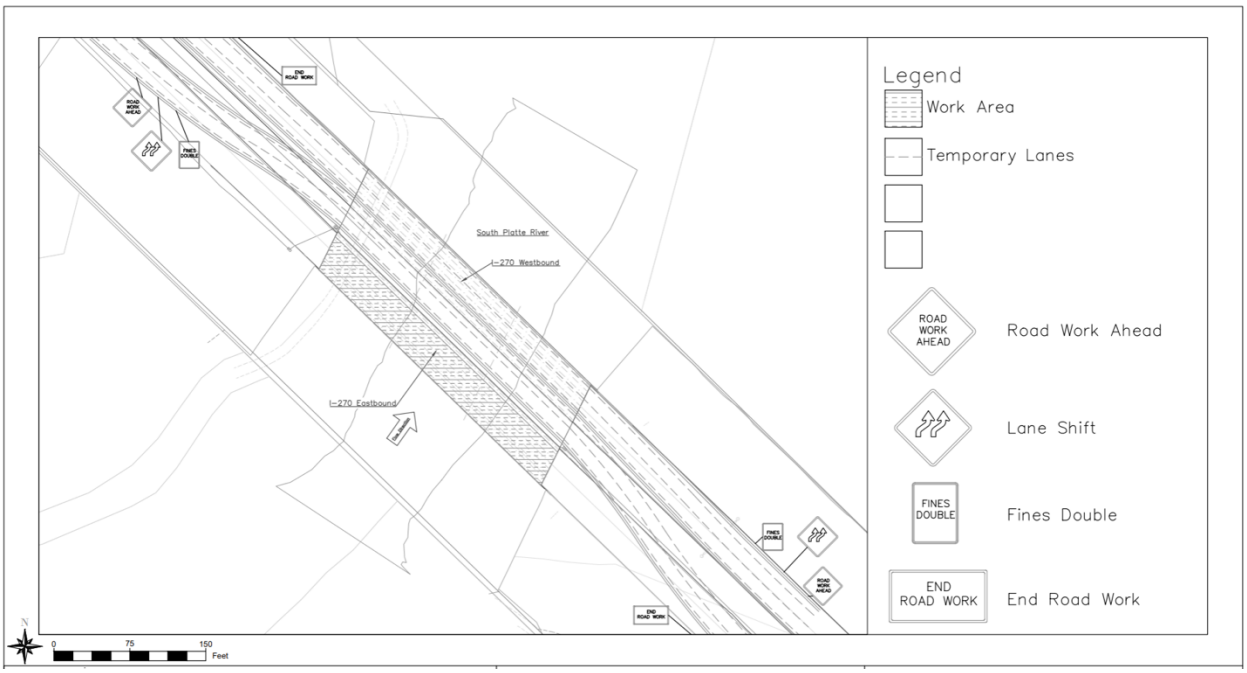


Figure 5.17: Phase Three Traffic Engineering Plan

5.9 Logistics Plan

Construction personnel can enter the access road via Colorado Boulevard (appendix D). Currently, along E 64th Avenue there is a small bridge that appears to not be in optimal condition. For the access road to support heavy equipment, this bridge will need to be reinforced to ensure structural stability.

Figure 5.18: Google Earth Image of Bridge Along E 64th Avenue

Along the access road there is also a light rail track (RTD N line) that provides plenty of vertical clearance for equipment. Coordination may be required with Denver RTD to acquire the actual height of the light rail track above E 64th Avenue.

Figure 5.19: Google Earth Image of Light Rail Along E 64th Avenue

For all crane pads not located along the access road, the soil has been graded to maintain less than a 6 percent slope for temporary equipment access (appendix D). Guardrails will be removed prior to the equipment's arrival with plenty of room to allow the equipment to maneuver.

5.9.1 Phase One Logistics Plan

A total of two cranes will be necessary, located on the north side of each embankment. These cranes are placed for optimal picking and laying of the girders and other heavy materials necessary for the first two phases of construction. During this phase of construction, temporary overnight lane closures will occur, following the phasing plans (section 5.10), to ensure public safety during the crane's times of use. With centralized pump and truck staging, we are strategically allowing for efficient delivery of concrete to various areas of the project and ensuring timely and precise pouring of concrete into forms or structures. The proximity to parking is important to facilitate easy access for workers, suppliers, and equipment, thereby minimizing downtime and improving overall productivity.

Figure 5.20: Phase One Site Logistics Plan

5.9.2 Phase Two Logistics Plan

Maintaining the crane positions in a consistent location for the second phase of the logistics plan proves to be optimal as the reconstruction of the westbound bridge commences. This strategic decision ensures continuity and efficiency in operations, streamlining the workflow and minimizing unnecessary adjustments. Since girder laydown and pump/truck staging occupy the same space, girder delivery will need to be strategically planned in schedule. Additionally, during Phase two, overnight lane closures will not be necessary, as the cranes will not need to traverse over traffic lanes, thus eliminating potential disruptions to road users and enhancing overall safety.

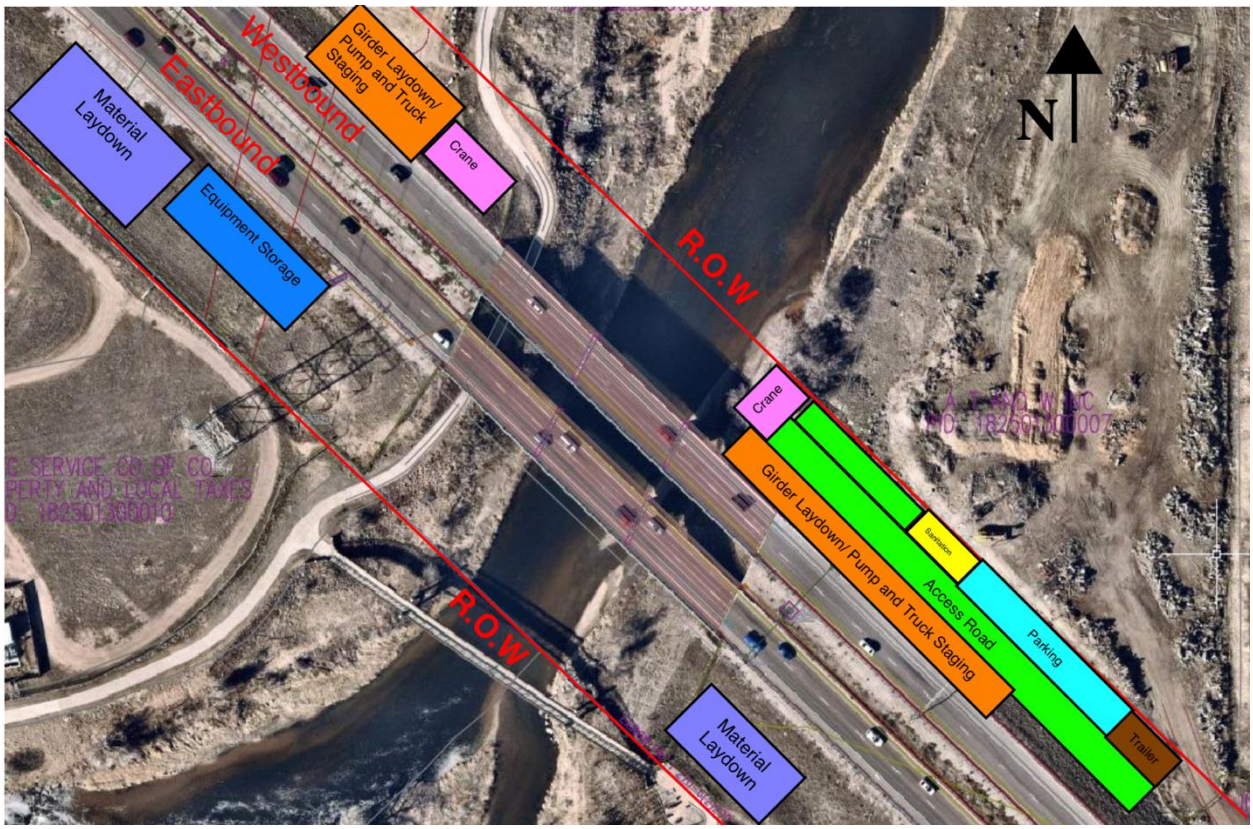


Figure 5.21: Phase Two Site Logistics Plan

5.9.3 Phase Three Logistics Plan

For the final construction phase, the two cranes will be strategically relocated to the south embankments of the bridge. This deliberate move is aimed at enhancing operational efficiency by facilitating optimal picking and placing of girders and other substantial materials, as well as eliminating the need for overnight lane closures during the final phase. Since girder laydown and pump/truck staging occupy the same space, girder delivery will need to be strategically planned in schedule. Furthermore, the relocation of the pump and truck staging area, along with the girder

laydown zone, will not only streamline crane access, but also ensure equitable access for pump trucks, thereby optimizing the overall construction workflow.

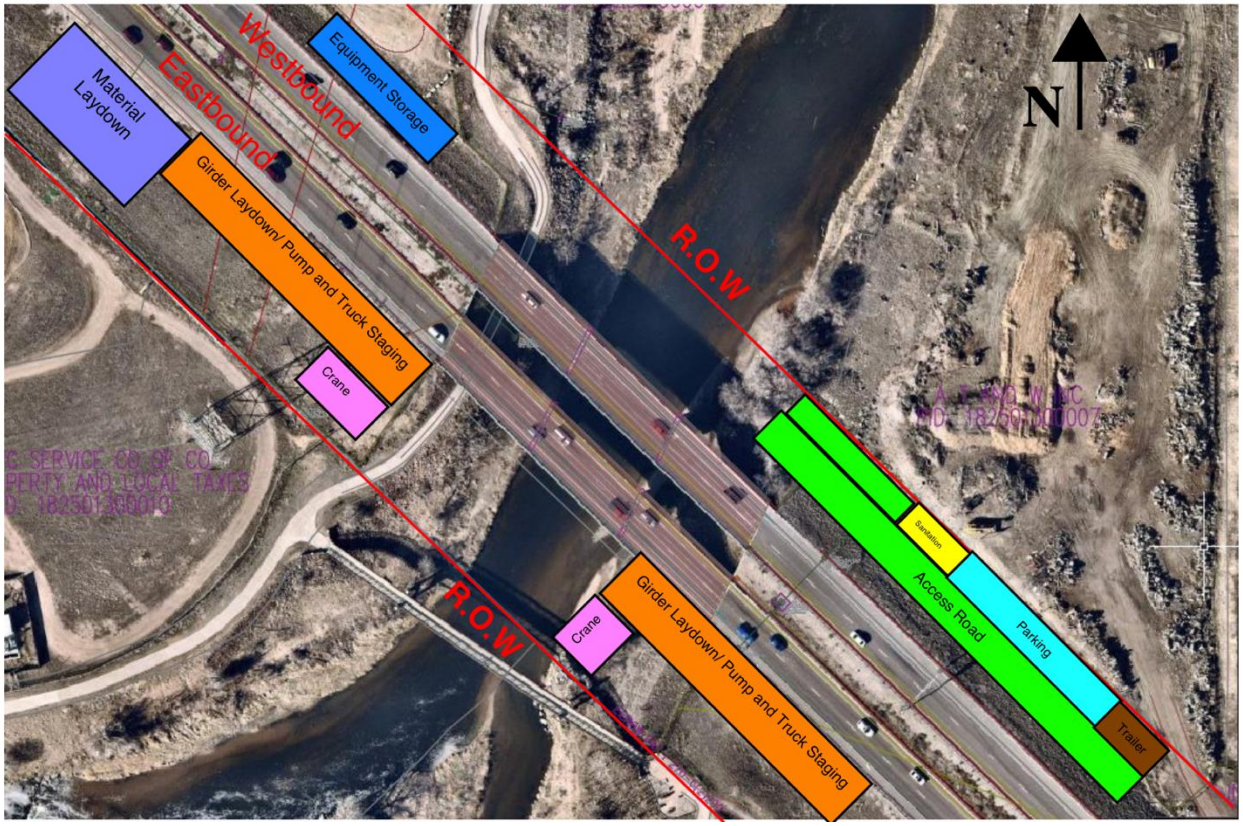


Figure 5.22: Phase Three Site Logistics Plan

5.10 Phasing Plan

Our phasing plan is strategic to reduce the amount of realignment needed in the final phase of the project. By starting construction in the middle of the two existing bridges, traffic shifts are minimal rather than rerouting traffic to the outside of the existing structures. Our site phasing plan is separated into five total phases, with phase zero being preconstruction activities and phase four being finishes and punch list items. Phases one through three align with the sections the new bridge will be built in along with the changes in traffic that will occur. With the small isolation gap in the middle of the bridges, steel plates sealed with asphalt will be used to ensure traffic can drive over safely without disturbing the integrity of the gap.

5.10.1 Phase One Phasing Plan

In the first phase of construction, 5’ of the eastbound shoulder and 5’ of the westbound shoulder will be sawcut off to make way for construction of the first section of the new, permanent bridge structure. A lane shift will be necessary on each side of the bridge to ensure that the CDOT standards for lane, shoulder, and barrier widths are met. Also, overnight lane closures will be necessary during specific points of this phase to ensure the public's safety, as cranes will need to traverse over the westbound traffic to place the new bridge girders. With construction starting in the center, a thorough safety plan is required to protect those working on site from moving traffic. Other considerations include saw cutting in sections with a net underneath to catch smaller debris to maintain the quality of water below.

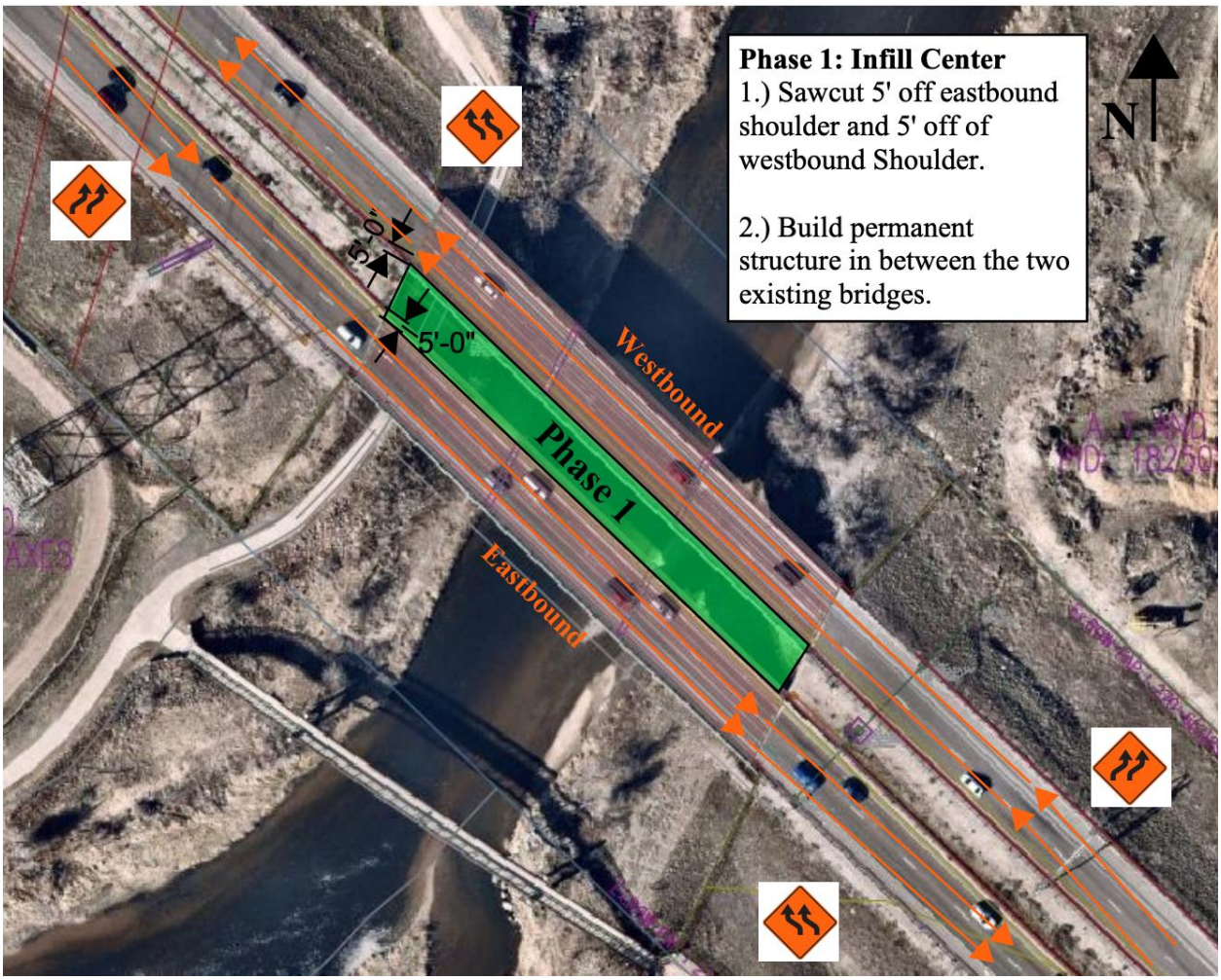


Figure 5.23: Phase One Phasing Plan and Traffic Control

5.10.2 Phase Two Phasing Plan

During the second phase of construction, the focus shifts towards the reconstruction of the previously demolished westbound portion of the bridge, seamlessly integrating it with the permanent structure erected in the initial phase. This intricate process entails not only rebuilding but also establishing structural connections to ensure the integrity and stability of the entire bridge system. Moreover, to accommodate the evolving construction dynamics, the westbound lanes will undergo rerouting onto the central section of the bridge, while the eastbound lanes will remain positioned in their designated locations as established in the preceding phase. This maneuver aims to optimize traffic flow, minimize disruptions, and uphold safety standards throughout the construction progression.

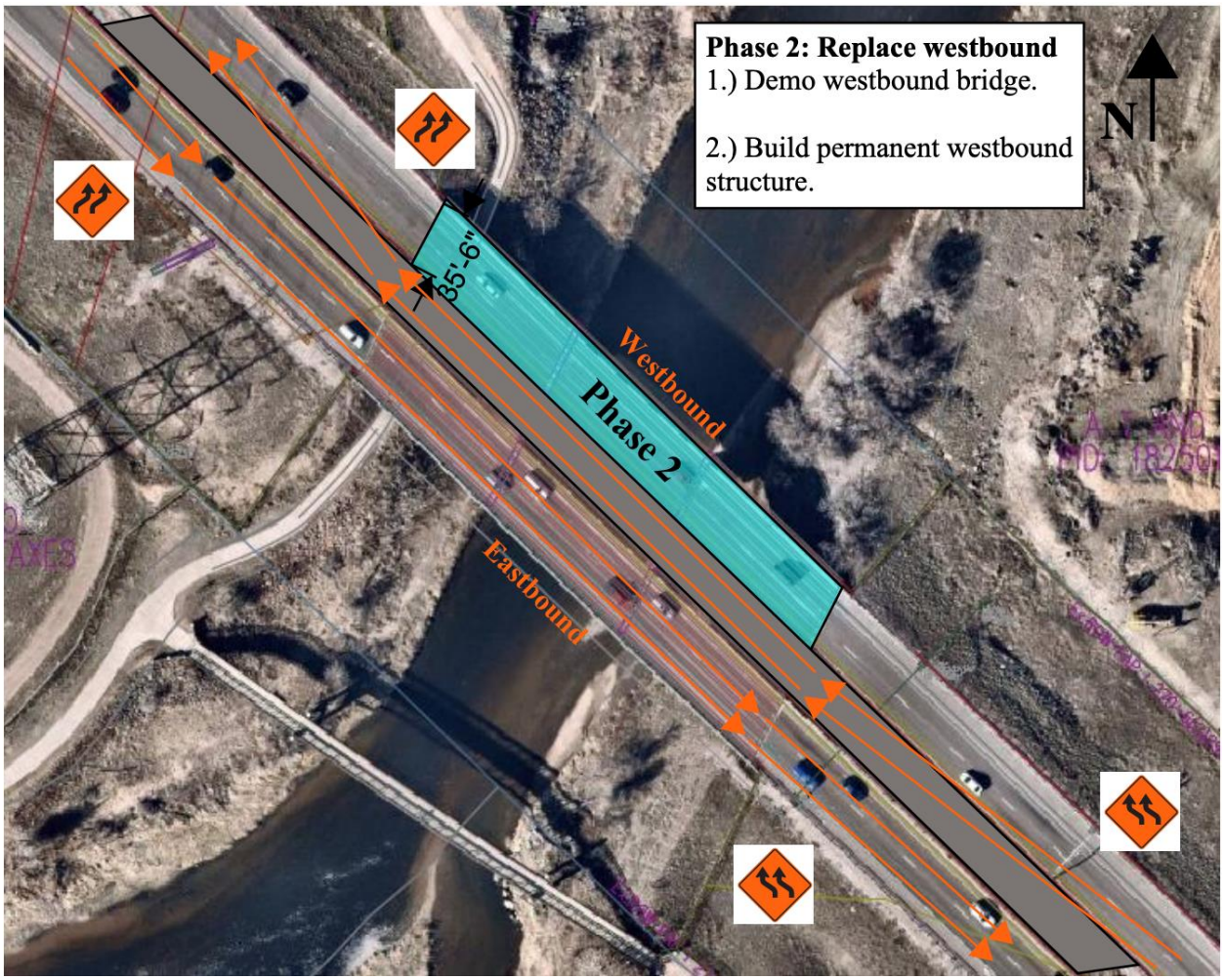


Figure 5.24: Phase Two Phasing Plan and Traffic Control

5.10.3 Phase Three Phasing Plan

In the final phase of construction, attention turns towards the reconstruction of the previously demolished eastbound segment of the bridge, meticulously integrating it with the existing permanent structure. This endeavor, similar to that in the construction of the westbound lanes, also encompasses the establishment of robust structural connections to ensure the seamless cohesion and durability of the entire bridge infrastructure. Furthermore, to accommodate construction and optimize traffic management, lane shifts will be implemented in the eastbound direction. These final adjustments and construction items are carefully orchestrated to facilitate the project's successful completion.

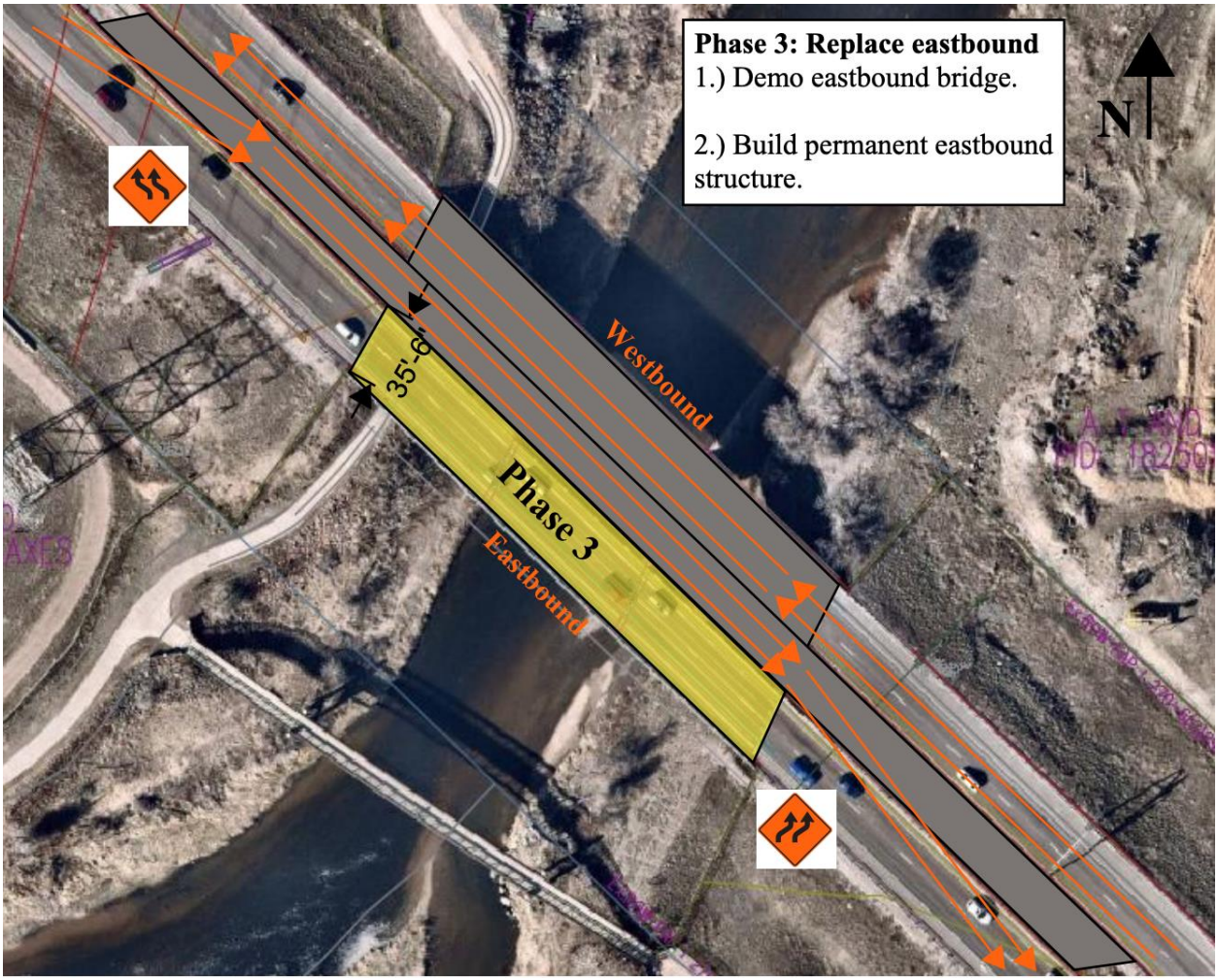


Figure 5.25: Phase Three Phasing Plan and Traffic Control

5.11 Major Haul Routes

There are many major access roads to the jobsite depending on the location of the haul. The major haul routes to and from the site include I-25 to I-270 and I-70 to I-270.

Figure 5.26: Major Haul Routes

5.12 Sustainability and Resiliency

Sustainability and resiliency in bridge construction has become more significant as urbanization and the environment continue to evolve. Below are some options to improve sustainability and resiliency in our design.

- **Recycled Materials:** Use of recycled concrete aggregate will reduce environmental impacts of sourcing raw materials.
- **Alternative Cementitious Materials:** Alternative cementitious materials include replacing a portion of the cement with silica fume or calcined clays. These alternatives can partially replace cement without compromising structural integrity while reducing carbon emissions associated with production of cement.
- **Higher Strength Concrete:** Using a higher strength concrete will improve the structural integrity of the bridge.
- **Bridge Structural Health Monitoring System:** Detects and diagnoses damage, deformation and fatigue through sensors to extend the lifespan of the bridge.
- **Carbon Capture and Storage:** Carbon capture and storage implements technologies to capture carbon dioxide emissions from cement production. This method captures carbon dioxide before it is released into the atmosphere and stores it underground.
- **Carbonation-Cured Concrete:** Carbonation curing involves using carbon dioxide from the atmosphere to cure concrete which reduces the carbon footprint.
- **Recycled Asphalt:** Use of recycled asphalt reduces the mining for raw materials which lowers energy consumption, greenhouse gas emissions and waste.
- **Permeable Pavements:** Use of permeable pavements on bridge surface or approach will allow precipitation to infiltrate to the ground rather than directly into the South Platte River.
- **Renewable energy:** Use of solar panels to generate electricity for lighting.
- **Native Grasses:** Native grasses absorb and filter precipitation runoff which will improve water quality.
- **Wildlife crossing:** Building a wildlife crossing beneath bridge to mitigate habitat fragmentation.

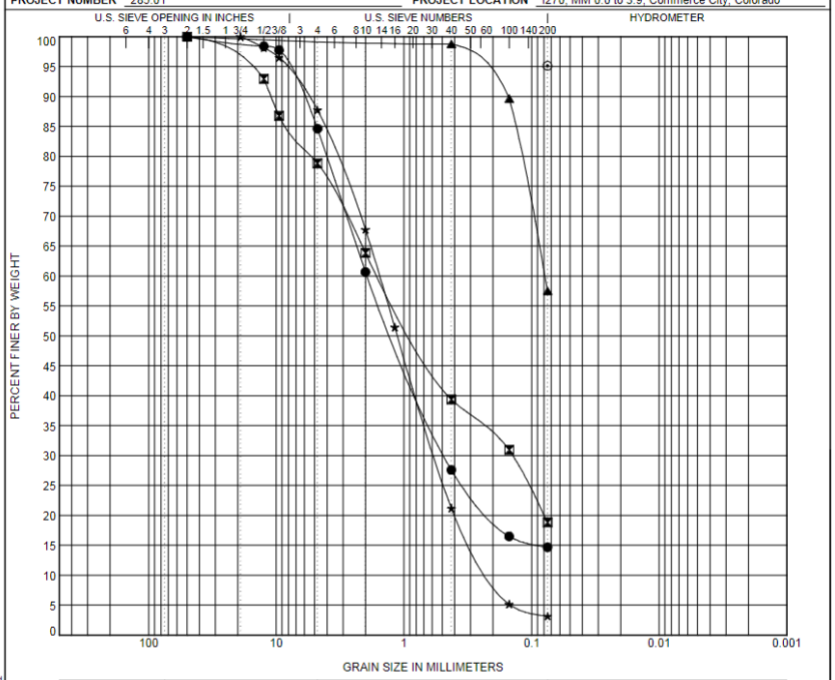
Appendix A: Water Resources

RockSol Consulting Group, Inc.
6510 W. 91st Ste. 130
Westminster, CO 80031
Telephone: 303-962-9300

GRAIN SIZE DISTRIBUTION

CLIENT CDOT Region 6 **PROJECT NAME** I- 270 Pavement Overlay Evaluation

PROJECT NUMBER 285.01 **PROJECT LOCATION** I270, MM 0.0 to 3.9, Commerce City, Colorado



COBBLES	GRAVEL		SAND			SILT OR CLAY
	coarse	fine	coarse	medium	fine	

	Specimen Identification		Classification	LL	PL	PI	Cc	Cu
●	BH-11	19.0	SILTY SAND with GRAVEL (SM)	NP	NP	NP		
■	BH-11	35.0	SILTY SAND					
▲	BH-11	40.0	SANDY LEAN CLAY (CL)	40	24	16		
★	BH-12	34.0	WELL-GRADED SAND (SW)	NP	NP	NP	1.03	7.58
⊙	BH-12	50.0	LEAN CLAY (CL)	45	26	19		

	Specimen Identification		D100	D60	D30	D10	%Gravel	%Sand	%Silt	%Clay
●	BH-11	19.0	50	1.937	0.475		15.4	69.9	14.7	
■	BH-11	35.0	50	1.564	0.142		21.1	60.0	18.8	
▲	BH-11	40.0	50	0.079			0.6	41.9	57.5	
★	BH-12	34.0	19	1.555	0.572	0.205	12.2	84.6	3.2	
⊙	BH-12	50.0	0.075						95.1	

GRADATION STANDARD 285.01.GPJ ROCKSOL TEMPLATE.GDT 5/2/12

Pier Scour

All piers have the same scour depth

Input Data

Pier Shape:	Sharp nose
Pier Width (ft):	4.50
Grain Size D50 (mm):	1.50000
Depth Upstream (ft):	20.61
Velocity Upstream (ft/s):	11.42
K1 Nose Shape:	0.90
Pier Angle:	0.00
Pier Length (ft):	185.00
K2 Angle Coef:	1.00
K3 Bed Cond Coef:	1.10
Grain Size D90 (mm):	7.50000
K4 Armouring Coef:	1.00

Results

Scour Depth Ys (ft):	10.70
Froude #:	0.44
Equation:	CSU equation

Pier Scour

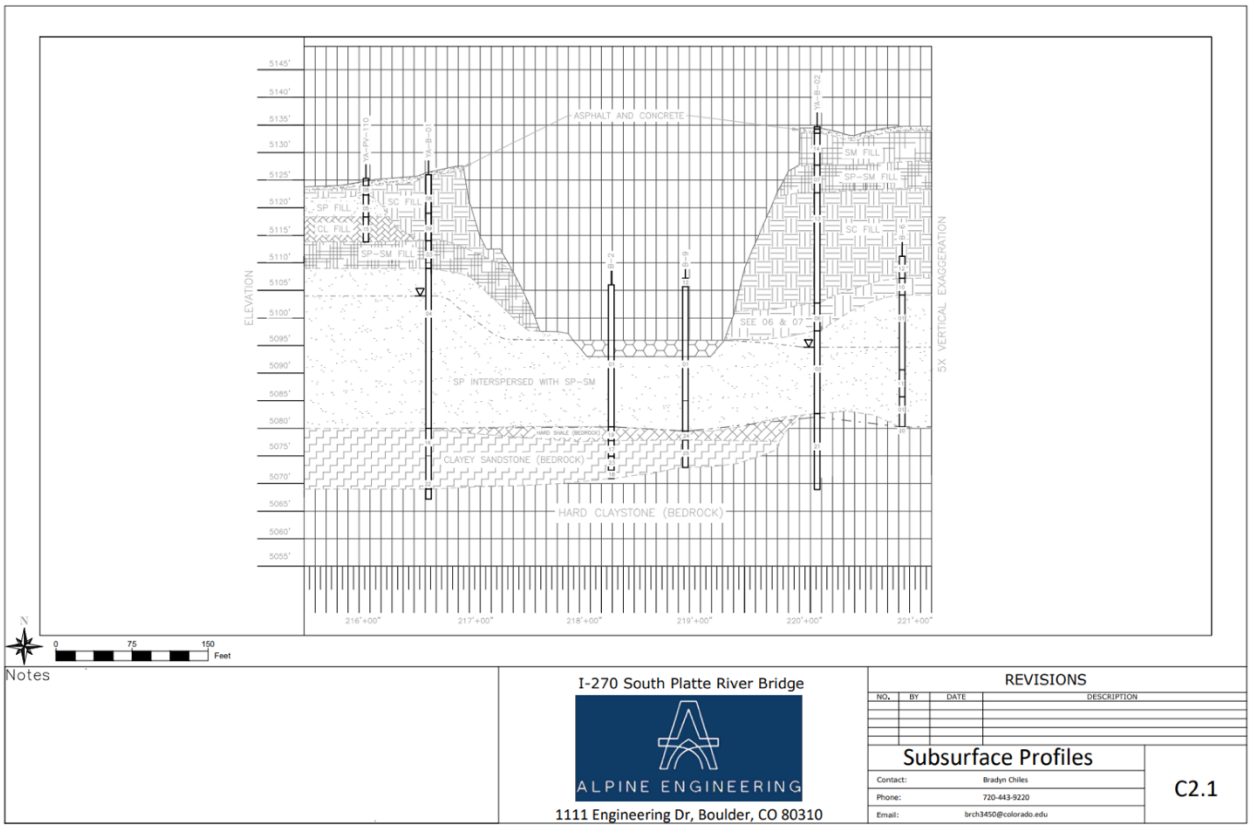
All piers have the same scour depth

Input Data

Pier Shape:	Sharp nose
Pier Width (ft):	4.50
Grain Size D50 (mm):	1.50000
Depth Upstream (ft):	26.82
Velocity Upstream (ft/s):	14.33
K1 Nose Shape:	0.90
Pier Angle:	0.00
Pier Length (ft):	185.00
K2 Angle Coef:	1.00
K3 Bed Cond Coef:	1.10
Grain Size D90 (mm):	7.50000
K4 Armouring Coef:	1.00

Results

Scour Depth Ys (ft):	12.22
Froude #:	0.49
Equation:	CSU equation


ASPHALT AND CONCRETE
SP FILL
SC FILL
CL FILL
SP-SM FILL
SM FILL
SP INTERSPERSED WITH SP-SM
CLAYEY SANDSTONE (BEDROCK)
HARD CLAYSTONE (BEDROCK)
SEE 06 & 07
ELEVATION
5X VERTICAL EXAGGERATION
Notes
I-270 South Platte River Bridge
ALPINE ENGINEERING
1111 Engineering Dr, Boulder, CO 80310
REVISIONS
Subsurface Profiles
C2.1

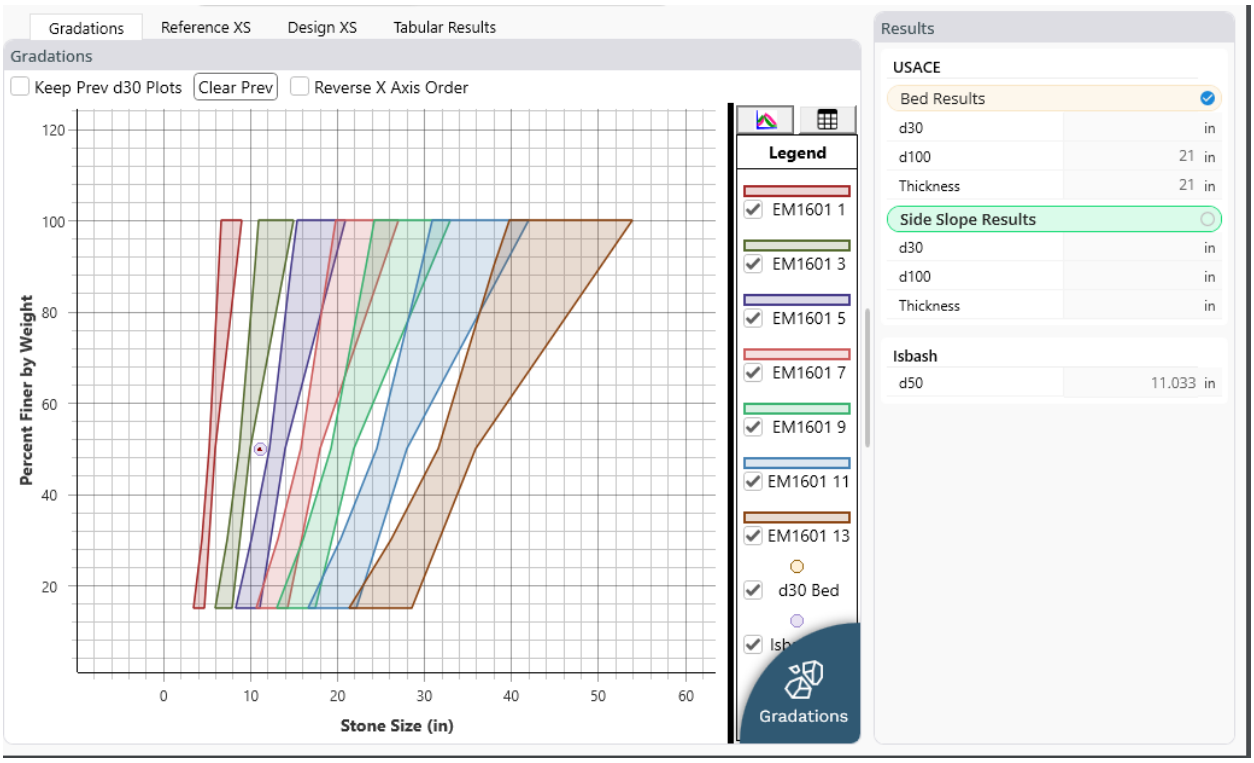

Gradations
Reference XS
Design XS
Tabular Results
Keep Prev d30 Plots
Clear Prev
Reverse X Axis Order
Percent Finer by Weight
Stone Size (in)
Legend
EM1601 1
EM1601 3
EM1601 5
EM1601 7
EM1601 9
EM1601 11
EM1601 13
d30 Bed
Results
USACE
Bed Results
d30
d100
21 in
Thickness
21 in
Side Slope Results
Isbash
d50
11.033 in

Table 17.1 Standard CDOT gradations for rock riprap.

Pay Item	Stone Size d50[1] (Inches)	Percent of Material Smaller Than Typical Stone[2]	Typical Stone Dimensions[3] (Inches)	Typical Stone Weight[4] (Pounds)
Riprap	6	70-100 50-70 35-50 2-10	12 9 6 2	85 35 10 0.4
Riprap	9	70-100 50-70 35-50 2-10	15 12 9 3	160 85 35 1.3
Riprap	12	70-100 50-70 35-50 2-10	21 18 12 4	440 275 85 3
Riprap	18	100 50-70 35-50 2-10	30 24 18 6	1280 650 275 10
Riprap	24	100 50-70 35-50 2-10	42 33 24 9	3500 1700 650 35

[1]d50 = nominal stone size
[2]based on typical rock mass
[3]equivalent spherical diameter
[4]based on a specific gravity = 2.5

$$F_D = \frac{1}{2} \rho v^2 C_D A$$

$\rho = 62.4 \ lb/ft^3$

$v = 11.45 \ ft/s$

$C_D = 1.39$ (triangular nose pier 60°)

$A = 59.1 \ ft^2$

$$F_D = \frac{1}{2}\left(62.4 \frac{lb}{ft^3}\right)\left(11.45 \frac{ft}{s}\right)^2 (1.39)(59.1 \ ft^2)$$

$= 336022 \ lb$

$= 336 \ kips$

Table 4. Gravity flow capacity of small diameter cast-iron pipes

Diameter (in)	Full Gravity Discharge at 2 percent Slope (ft^3/s)	Velocity at 10 percent Full (ft/s)
4	0.3	1.1
6	0.8	1.4
8	2.3	1.6
10	3.0	1.9
12	5.0	2.1

PDS-based precipitation frequency estimates with 90% confidence intervals (in inches/hour)[1]

Duration	Average recurrence interval (years)									
	1	2	5	10	25	50	100	200	500	1000
5-min	2.42 (1.91-3.07)	3.00 (2.36-3.80)	4.01 (3.14-5.09)	4.91 (3.83-6.25)	6.23 (4.73-8.23)	7.32 (5.40-9.73)	8.47 (6.02-11.5)	9.70 (6.61-13.4)	11.4 (7.48-16.1)	12.8 (8.12-18.1)
10-min	1.78 (1.40-2.25)	2.20 (1.73-2.78)	2.93 (2.30-3.73)	3.59 (2.80-4.57)	4.56 (3.46-6.03)	5.36 (3.95-7.13)	6.20 (4.42-8.39)	7.10 (4.84-9.80)	8.36 (5.47-11.8)	9.37 (5.95-13.2)
15-min	1.44 (1.14-1.83)	1.78 (1.41-2.26)	2.38 (1.87-3.03)	2.92 (2.28-3.72)	3.70 (2.81-4.90)	4.36 (3.22-5.79)	5.04 (3.59-6.82)	5.77 (3.94-7.96)	6.80 (4.45-9.56)	7.62 (4.84-10.8)
30-min	1.04 (0.818-1.31)	1.28 (1.01-1.62)	1.70 (1.33-2.15)	2.07 (1.61-2.63)	2.62 (1.99-3.46)	3.07 (2.27-4.09)	3.55 (2.53-4.81)	4.07 (2.77-5.61)	4.79 (3.13-6.73)	5.36 (3.40-7.58)
60-min	0.650 (0.513-0.824)	0.793 (0.625-1.00)	1.04 (0.820-1.33)	1.27 (0.993-1.62)	1.61 (1.22-2.14)	1.90 (1.40-2.53)	2.20 (1.57-2.98)	2.52 (1.72-3.48)	2.98 (1.95-4.20)	3.35 (2.13-4.74)
2-hr	0.391 (0.312-0.490)	0.473 (0.377-0.593)	0.621 (0.493-0.779)	0.755 (0.596-0.950)	0.957 (0.737-1.26)	1.13 (0.844-1.49)	1.31 (0.945-1.76)	1.51 (1.04-2.06)	1.78 (1.18-2.49)	2.01 (1.29-2.81)
3-hr	0.282 (0.227-0.351)	0.340 (0.273-0.423)	0.444 (0.355-0.553)	0.539 (0.429-0.674)	0.684 (0.531-0.893)	0.807 (0.609-1.06)	0.938 (0.683-1.25)	1.08 (0.753-1.47)	1.29 (0.859-1.78)	1.45 (0.940-2.02)
6-hr	0.166 (0.135-0.204)	0.200 (0.162-0.245)	0.260 (0.211-0.320)	0.315 (0.254-0.390)	0.399 (0.314-0.514)	0.470 (0.359-0.609)	0.545 (0.402-0.719)	0.628 (0.443-0.843)	0.745 (0.504-1.02)	0.839 (0.550-1.15)
12-hr	0.100 (0.082-0.121)	0.121 (0.099-0.147)	0.158 (0.129-0.192)	0.191 (0.156-0.233)	0.240 (0.190-0.304)	0.280 (0.216-0.358)	0.323 (0.241-0.420)	0.369 (0.263-0.489)	0.434 (0.297-0.587)	0.486 (0.323-0.660)
24-hr	0.061 (0.050-0.073)	0.073 (0.061-0.088)	0.095 (0.079-0.115)	0.114 (0.094-0.138)	0.142 (0.114-0.178)	0.165 (0.129-0.208)	0.189 (0.142-0.242)	0.214 (0.154-0.280)	0.249 (0.172-0.332)	0.277 (0.186-0.372)
2-day	0.035 (0.030-0.042)	0.043 (0.036-0.050)	0.055 (0.046-0.065)	0.066 (0.055-0.078)	0.081 (0.065-0.099)	0.093 (0.073-0.115)	0.106 (0.080-0.133)	0.119 (0.087-0.153)	0.137 (0.096-0.180)	0.151 (0.103-0.201)
3-day	0.025 (0.021-0.029)	0.030 (0.026-0.036)	0.040 (0.033-0.047)	0.047 (0.040-0.056)	0.058 (0.047-0.071)	0.067 (0.053-0.082)	0.076 (0.058-0.095)	0.085 (0.062-0.108)	0.097 (0.068-0.126)	0.106 (0.073-0.140)
4-day	0.019 (0.016-0.022)	0.024 (0.020-0.028)	0.031 (0.026-0.036)	0.037 (0.031-0.044)	0.046 (0.037-0.055)	0.053 (0.042-0.064)	0.059 (0.046-0.074)	0.066 (0.049-0.084)	0.075 (0.053-0.098)	0.082 (0.057-0.108)
7-day	0.013 (0.011-0.015)	0.015 (0.013-0.017)	0.019 (0.016-0.022)	0.023 (0.019-0.026)	0.027 (0.023-0.033)	0.031 (0.025-0.038)	0.035 (0.027-0.044)	0.039 (0.029-0.050)	0.045 (0.032-0.058)	0.049 (0.034-0.064)
10-day	0.010 (0.008-0.011)	0.011 (0.010-0.013)	0.014 (0.012-0.016)	0.017 (0.014-0.019)	0.020 (0.017-0.024)	0.023 (0.018-0.027)	0.025 (0.020-0.031)	0.028 (0.021-0.035)	0.032 (0.023-0.041)	0.035 (0.025-0.046)
20-day	0.006 (0.005-0.007)	0.007 (0.006-0.008)	0.009 (0.007-0.010)	0.010 (0.009-0.011)	0.012 (0.010-0.014)	0.013 (0.011-0.016)	0.014 (0.011-0.018)	0.016 (0.012-0.020)	0.018 (0.013-0.023)	0.020 (0.014-0.025)
30-day	0.005 (0.004-0.006)	0.006 (0.005-0.006)	0.007 (0.006-0.008)	0.008 (0.007-0.009)	0.009 (0.008-0.011)	0.010 (0.008-0.012)	0.011 (0.009-0.013)	0.012 (0.009-0.015)	0.014 (0.010-0.017)	0.015 (0.010-0.019)
45-day	0.004 (0.003-0.004)	0.004 (0.004-0.005)	0.005 (0.005-0.006)	0.006 (0.005-0.007)	0.007 (0.006-0.008)	0.008 (0.007-0.010)	0.009 (0.007-0.011)	0.010 (0.007-0.012)	0.011 (0.008-0.013)	0.011 (0.008-0.015)
60-day	0.003 (0.003-0.004)	0.004 (0.003-0.004)	0.005 (0.004-0.005)	0.005 (0.005-0.006)	0.006 (0.005-0.007)	0.007 (0.006-0.008)	0.008 (0.006-0.009)	0.008 (0.006-0.010)	0.009 (0.007-0.012)	0.010 (0.007-0.012)

[1] Precipitation frequency (PF) estimates in this table are based on frequency analysis of partial duration series (PDS).

Numbers in parenthesis are PF estimates at lower and upper bounds of the 90% confidence interval. The probability that precipitation frequency estimates (for a given duration and average recurrence interval) will be greater than the upper bound (or less than the lower bound) is 5%. Estimates at upper bounds are not checked against probable maximum precipitation (PMP) estimates and may be higher than currently valid PMP values.

Please refer to NOAA Atlas 14 document for more information.

Estimates from the table in CSV format: Precipitation frequency estimates Submit

Deck Drainage Design :

* Spread Plus Rational Method: $Q = kCiA$

where: Q = peak runoff rate off of deck

$k = 1$

$C = 0.9$ for bridge decks

i = rainfall intensity (in/hr)

A = drainage area (acres) $= \frac{306\,ft \cdot 36\,ft}{43560} = 0.2529$ acres

* Rainfall Intensity:

Duration of rainfall intensity = time of concentration, t_c

t_c = overland flow time of conc. + gutter flow time of conc.
$= t_o + t_g$

$$t_o = \frac{0.93(I_o n)^{0.6}}{(ci)^{0.4} S^{0.3}}, \quad t_g = \frac{484 S_x T^2}{ci W_p}$$

$I_o, W_p = 36$ ft $\qquad S, S_o = 0.02$

$n = 0.016$ for asphalt $\qquad T = 14$ ft

$C = 0.9$

Trial 1: Using 100yr 5-min $i = 8.47$ in/hr

$$t_o = \frac{0.93(36\,ft \cdot 0.016)^{0.6}}{(0.9 \cdot 8.47\,in/hr)^{0.4}(0.02)^{0.3}} = 0.96 \text{ min}$$

$$t_g = \frac{484(0.02)(14\,ft)^2}{0.9 \cdot 8.47\,in/hr \cdot 36\,ft} = 6.91 \text{ min}$$

Trial 2: Using 100yr 10-min $i = 6.20$ in/hr

$$t_o = \frac{0.93(36\,ft \cdot 0.016)^{0.6}}{(0.9 \cdot 6.2\,in/hr)^{0.4}(0.02)^{0.3}} = 1.09 \text{ min}$$

$$t_g = \frac{484(0.02)(14\,ft)^2}{0.9 \cdot 6.2\,in/hr \cdot 36\,ft} = 9.44 \text{ min}$$

Since in trial 2 we get a t_c = 10.53 min which is close to the 10-min duration used, our design intensity is 6.2 in/hr.

* Full Gutter Flow:

$$Q_f = \frac{0.56}{0.016}(0.02)^{1.67}(0.03)^{0.5}(14\,ft)^{2.67} = 10.1\ ft^3/s$$

* Inlet Requirement:

$$L_o = \frac{(43560)(10.1\,ft^3/s)}{(0.9)(6.2\,in/hr)(36\,ft)} = 2190\ ft \quad \therefore \text{INLETS NOT REQUIRED}$$

* Bridge-end treatments:

Peak runoff rate, $Q = (1)(0.9)(6.2\,in/hr)(0.2529\,acres) = 1.41\ ft^3/s$

* Design Summary:

At a 3% grade for the asphalt surface, no inlets located at the bridge are needed. However an inlet collection system at the end of the bridge is needed.

* Collection System:

cast-iron pipe w/ a diameter of 8 in at the end of the bridge to collect runoff and transport it to the existing drainage ditches.

Appendix B: Geotechnical

Tables

Table 3 – Groundwater Measurements

Boring ID	Surface Elevation (ft)	Date of Measurement	Groundwater	
			Depth (ft)	Elevation (ft)
YA-B-10	5157.1	12/9/2020	20	5137.1
		1/29/2021	20	5137.1
		3/2/2021	19.9	5137.2
		4/22/2021	19.4	5137.7
		7/12/2021	18.9	5138.2
YA-B-11	5164.1	11/30/2020	30	5134.1
		1/29/2021	11.5	5152.6
		3/2/2021	11.5	5152.6
		4/22/2021	10.7	5153.4
		7/12/2021	10.8	5153.3
YA-G-01	5127.5	12/2/2020	10	5117.5
		1/29/2021	7.4	5120.1
		3/2/2021	7.4	5120.1
		4/22/2021	6.6	5120.9
		7/12/2021	7.1	5120.4
YA-W-05	5163.8	12/4/2020	14.5	5149.3
		1/29/2021	14.6	5149.2
		3/2/2021	14.8	5149.0
		4/22/2021	13.8	5150.0
		7/12/2021	13.4	5150.4

Table 6 – Nominal Drilled Shaft Axial Resistances

Bridge Structure ID	Reference Borings	Estimated Top of Bedrock Elevation (feet)	Axial Resistance Strength Limit State	
			Nominal Side Resistance (ksf)	Nominal Tip Resistance (ksf)
E-17-IC[1]	B-1 B-2 B-3 B-8 B-9 B-10	5131 5127 5133 5133 5130 5134	4.9	60
E-17-ID E-17-IE	YA-B-01 YA-B-02	5080 5082	9.0	110
E-17-IF E-17-IG	YA-B-03 YA-B-04	5084 5107	8.3	101
E-17-IH E-17-II	YA-B-05 YA-B-06 YA-B-07	5109 5108 5110	10.4	128
E-17-IJ E-17-IK	YA-B-08 YA-B-09 YA-B-10	5130 5133 5134	9.4	125
E-17-AT	YA-B-11 YA-B-12	5146 5148	7.1	87
E-17-IN E-17-IO	YA-B-13 YA-B-14	5140 5138	9.0	110

[1]See CDOT Structure Memorandum "Foundation Investigation – Structure E-17-IC" dated July 7, 1967, for boring logs and details

Table 7 – LPILE Parameters

Soil Type	LPILE Soil Criteria	Effective Unit Weight (pcf)		Friction Angle, ϕ (deg.)	Undrained Cohesion, c (psf)	Strain Factor, ε_{50}	p-y modulus k_{static} (pci)	
		AGT[1]	BGT[2]				AGT[1]	BGT[2]
Class 1 Structure Backfill	Sand (Reese)	135	72.5	34	--	--	90	60
Fill and Native Clayey Soils (USCS Classification of CL or CH)	Soft Clay (Matlock)	120	57.5	--	800	0.010	--	--
Fill and Native Primarily Sandy Soils (USCS Classification of SC, SM, SP, SP-SM, or SW-SM)	Sand (Reese)	125	62.5	32	--	--	90	60
Debris Fill (Landfill)	Sand (Reese)	115	52.5	30	--	--	60	40
Weathered Bedrock	Stiff Clay w/o Free Water (Reese)	125	62.5	--	3,000	0.005	--	--
Hard to Very Hard Claystone and Sandstone Bedrock	Stiff Clay w/o Free Water (Reese)	135	135	--	8,000	0.004	--	--

[1]Above Groundwater Table
[2]Below Groundwater Table

Table 8 – Wall Nominal Bearing Resistance

Foundation Material	Soil Properties Used for Bearing Calculations	Nominal Bearing Resistance (ksf) – Level Toe Slope
Fill and Native Clay Soils	ϕ = 6° c = 400 psf γ = 120 pcf	3.0 + 0.02 B'
Fill and Native Sandy Soils	ϕ = 32° c = 0 psf γ = 125 pcf	4.4 + 0.9 B'
Weathered Bedrock	ϕ = 8° c = 1,000 psf γ = 125 pcf	7.9 + 0.03 B'
Claystone or Sandstone Bedrock	ϕ = 10° c = 2,000 psf γ = 135 pcf	17.3 + 0.04 B'

Borings

DEPARTMENT OF HIGHWAYS, STATE OF COLORADO
Materials Division
Form DOH 267
May 1967 Rev.

Project # I 270-6(1)
Location Commerce City West
Structure So. Platte River
Route I 270 County Adams
Date Drilled 5-24-67

FOUNDATION BORING LOG

Top Hole Elev. 5106.0 Geologist Card Station 89+32 12'Lt Boring No. 2

Elev.	Depth	Description of Material	"N" Blows	Remarks
5106.0	0-25.7	Sand, gravelly, few cobbles,		NX casing with
		tan (similar to 1-A)		3" tricone & water
5097.0	9.0-10.5	As above	9	
91.5	14.5-16.0	" "	43	
86.2	19.8-21.3	" "	50	
81.6	24.4-26.1	24.4-25.7 Same as above 25.7-26.1 Same as below	40	
80.3	25.7-28.2	Shale, clayey, silty, vertically		
		and horizontally fractured, moist		
		greenish blue, hard		
79.4	26.6-31.1	26.6-28.2 Same as above 28.2-31.3 Same as below		BX core bbl w/H20 Recovery=1.4'=31%
77.8	28.2-31.1	Sandstone, silty, friable, very		
		fine to medium grained, light		
		greenish gray		
74.9	31.1-33.6	31.1-33.6, shale, hard, dark gray		BX core BBL with water to 35.1
		to black, highly fractured		Recovery=2.7'=68%
		plastic		
	33.6-35.1	33.6-35.1, sandstone, silty, friable		
		very fine to medium grained, light		
		greenish gray		
5070.9	35.1	SIS		

Water level upon completion 0.9* Elev. 5105.1 Date 5-25-67 Time 1:37 PM
Water level (24 hr.) _____ Elev. _____ Date _____ Time _____
*Hole caved after casing was pulled.

jh 6/6/67

DEPARTMENT OF HIGHWAYS, STATE OF COLORADO
Materials Division
Form DOH 267
May 1967 Rev.

Project # I 270-6(1)
Location Commerce City West
Structure So. Platte River
Route I 270 County Adams
Date Drilled 5-25-67

FOUNDATION BORING LOG

Top Hole Elev. 5106.4 Geologist Card Station 89+77 36'Rt Boring No. 3

Elev.	Depth	Description of Material	"N" Blows	Remarks
5106.4	0-24.0	Sand, gravelly, few cobbles, tan		NX casing with 3"
		(similar to 1-A)		tricone & water
96.4	10.0-11.5	As above	13	
86.9	19.5-21.0	" "	81	
82.4	24.0-31.0	Shale, fractured greenish		
		blue (similar to 1-B)		
82.4	24.0-25.4	As above	79/.9	
80.4	26.0-31.0	" "		BX core bbl Recovery=2'=40%
5075.4	31.0	SIS		

Water level upon completion 1.0* Elev. 5105.4 Date 5-26-67 Time 10:00 AM
Water level (24 hr.) _____ Elev. _____ Date _____ Time _____

* Hole filled with sand.

jh 6/16/67

DEPARTMENT OF HIGHWAYS, STATE OF COLORADO
Materials Division
Form DOH 267
May 1967 Rev.

Project # I 270-6(1)
Location Commerce City West
Structure Over So. Platte
Route I 270 County Adams
Date Drilled 6-5-67

FOUNDATION BORING LOG

Top Hole Elev. 5111.2 Geologist Card Station 91+67 5'Rt Boring No. 6

Elev.	Depth	Description of Material	"N" Blows	Remarks
5111.2	0-4.0	Fill		Drilled w/4½" bit
07.2	4.0-7.0	Sand, very silty, interlayered with		using drilling mud
		clay, soft, dark gray		to 19.5'
04.2	7.0-19.5	Sand, gravelly, few cobbles, tan		
		(similar to 1-A)		
01.7	9.5-11.0	As above	12	
5091.7	19.5-21.0	19.5-20.6 as above 20.6-21.0 As below	20	Drove NX casing
90.6	20.6-25.5	Sand, silty, v.fine to medium,		
		micaceous, tan		
87.2	24.0-25.5	As above	57	
85.7	25.5-30.9	Sand, gravelly, few cobbles, tan		
		(similar to 1-A)		
82.2	29.0-30.5	As above	31	
80.3	30.9-39.0	Shale silty, highly fractured		
		(similar to 1-B)		
80.0	31.2-32.1	As above	100/.4	
5072.2	39.0	SIS		

Water level upon completion 0 Elev. _____ Date 6-6-67 Time 3:55 PM
Water level (24 hr.) 6.4 Elev. 5104.8 Date 6-7-67 Time 7:55 AM

jh 6/16/67

DEPARTMENT OF HIGHWAYS, STATE OF COLORADO
Materials Division
Form DOH 267
May 1967 Rev.

Project# I 270-6(1)
Location Commerce City West
Structure Over So. Platte
Route I 270 County Adams
Date Drilled 6-7-67

FOUNDATION BORING LOG

Top Hole Elev. 5107.2 Geologist Card Station 89+90 60'Lt Boring No. 9

Elev.	Depth	Description of Material	"N" Blows	Remarks
5107.2	0-1.5	Fill		NX casing and
05.7	1.5-27.7	Sand, gravelly, few cobbles		3" tricone bit
		(similar to 1-A)		with water
5082.0	25.2-26.7	As above	34	
79.5	27.7-29.3	Shale, reworked, weathered with		
		small coal particles and mica,		
		gray		
79.0	28.2-29.3	As above	104/.6	
77.9	29.3-34.3	Shale, silty, fractured with		BX core bbl Recovery=4.0'=80%
		local, thin medium grained sand-		
		stone lenses, dark gray to blue		
		A-7-5(11) (9-B)		Qu=14.8 tons/sq.ft.
5072.9	34.3	SIS		

Water level upon completion 1.2* Elev. 5106.0 Date 6-9-67 Time 3:00 PM
Water level (24 hr.) _____ Elev. _____ Date _____ Time _____

* Hole caved after casing was pulled.

fh 6/16/67

Yeh and Associates, Inc.
Geotechnical • Geological • Construction Services

Project Name: I-270 Environmental Assessment and Conceptual Design

PAGE 1 of 2

Project Number: 219-418 *Boring No.:* **YA-B-01**

Boring Began: 12/20/2020
Boring Completed: 12/20/2020
Drilling Method(s): Hollow-Stem Auger
Driller: Authentic Drilling
Drill Rig: CME 550X Buggy
Hammer: Automatic (hydraulic), ER: 89%

Total Depth: 58.8 ft
Ground Elevation: 5126.0 ft
Coordinates: N: 176053.1 E: 154734.8
Location:
Logged By: R. Desterhouse
Final By: L. Safari

Weather Notes: Cold, cloudy
Inclination from Horiz.: Vertical
Night Work: ☑

Groundwater Levels:

Symbol	▽		
Depth	22.0 ft	-	-
Date	12/20/20	-	-

Elevation (feet)	Depth (feet)	Sample Type/Depth	Drilling Method	Soil Samples: Blows per 6 in	Soil Samples: Penetration Resistance	Lithology	Material Description	Moisture Content (%)	Dry Density (pcf)	Gravel Content (%)	Sand Content (%)	Fines Content (%)	Atterberg Limits: Liquid Limit	Atterberg Limits: Plasticity Index	AASHTO & USCS Classifications	Field Notes and Other Lab Tests
							0.0 - 7.0 ft. Clayey SAND (SC) with some gravel (Fill), brown, medium plasticity, moist, medium dense to very dense.									
				7-18	25			*11.4*		*11.0*	*51.0*	*38.0*	*32*	*16*	*A-6 (2) SC*	**2.0 ft** - Small plastic debris in sample
5120				7-42-17	59											
							7.0 - 12.0 ft. Clayey SAND (SC) with minor gravel (Fill), brown, low plasticity, moist, medium dense, drillers noted cobbles 7 to 9 ft..									
	10			4-5-11	16			15.2		2.0	53.0	45.0	28	12	A-6 (2) SC	
							12.0 - 17.0 ft. Poorly graded SAND with silt (SP-SM) (Fill), with some gravel, brown, no plasticity, moist, medium dense.								
5110				8-10	18											
							17.0 - 46.0 ft. Poorly graded SAND (SP) with gravel, brown, tan, pink with rust, no plasticity, moist to wet, loose.								
	20			3-7	10											
	▽															**21.0 ft** - flowing sands to 46 ft.
5100																
	30															
5090																

BORING LOG 2019 - SPT CDOT STYLE 220_294 I270 (GEOTECH) GPJ 2019 YEH COLORADO TEMPLATE GDT 2019 YEH COLORADO LIBRARY GLB 7/7/22

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Project Name: I-270 Environmental Assessment and Conceptual Design

PAGE 2 of 2

Project Number: 219-418 *Boring No.:* **YA-B-01**

Elevation (feet)	Depth (feet)	Sample Type/Depth	Drilling Method	Soil Samples: Blows per 6 in	Soil Samples: Penetration Resistance	Lithology	Material Description	Moisture Content (%)	Dry Density (pcf)	Gravel Content (%)	Sand Content (%)	Fines Content (%)	Atterberg Limits: Liquid Limit	Atterberg Limits: Plasticity Index	AASHTO & USCS Classifi-cations	Field Notes and Other Lab Tests
							17.0 - 46.0 ft. Poorly graded SAND (SP) with gravel, brown, tan, pink with rust, no plasticity, moist to wet, loose.									
5080							**46.0 - 57.0 ft. CLAYEY SANDSTONE**, blue-gray to gray, very hard.									
	50			50:3"	50:3"			17.1	105.5	0.0	62.0	38.0	NV	NP	A-4 (0) SM	
5070				50:5"	50:5"											
				33-50:3"	83:9"		**57.0 - 58.8 ft. SILTY CLAYSTONE**, olive-gray, very hard.	21.5	103.3							UCCS=12852 psf

Bottom of Hole at 58.8 ft.

Yeh and Associates, Inc.
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Project Name: I-270 Environmental Assessment and Conceptual Design

PAGE 1 of 2

Project Number: 219-418 *Boring No.:* **YA-B-02**

Boring Began: 12/20/2020
Boring Completed: 12/20/2020
Drilling Method(s): Hollow-Stem Auger
Driller: Vine Laboratories
Drill Rig: CME 55 Truck
Hammer: Automatic (hydraulic), ER: 85%

Total Depth: 65.8 ft
Ground Elevation: 5134.7 ft
Coordinates: N: 175820.8 E: 155005.5
Location:
Logged By: L. Safari
Final By: L. Safari

Weather Notes: Cold, clear
Inclination from Horiz.: Vertical
Night Work: ☑

Groundwater Levels:			
Symbol	▽		
Depth	40.0 ft	-	-
Date	12/20/20	-	-

Elevation (feet)	Depth (feet)	Sample Type/Depth	Drilling Method	Soil Samples: Blows per 6 in	Soil Samples: Penetration Resistance	Lithology	Material Description	Moisture Content (%)	Dry Density (pcf)	Gravel Content (%)	Sand Content (%)	Fines Content (%)	Atterberg Limits: Liquid Limit	Atterberg Limits: Plasticity Index	AASHTO & USCS Classifi-cations	Field Notes and Other Lab Tests
							0.0 - 0.5 ft. ASPHALT (6 inches).									
							0.5 - 1.2 ft. CONCRETE (7 inches).									
				12-11-14	25		**1.2 - 7.0 ft. Silty SAND with gravel (SM) (Fill)**, brown and tan, no plasticity, moist, medium dense.	4.6		14.0	68.0	18.0	NV	NP	A-1-b (0) SM	
5130				13-13-13	26											
							7.0 - 12.0 ft. Poorly graded SAND with silt and gravel (SP-SM) (Fill), brown, no plasticity, moist, medium dense.									
	10			12-13-10	23											
							12.0 - 32.0 ft. Clayey SAND (SC) with minor gravel (Fill), brown, low plasticity, moist, medium dense.									
5120				5-4-7	11											
	20			6-8-18	26											pH=8.0 S=0.006% Chl=0.0245% Re=974ohm-cm
5110				6-6-9	15			13.4		2.0	63.0	35.0	26	8	A-2-4 (0) SC	
	30			2-5-13	18											
							32.0 - 37.0 ft. Poorly graded SAND with gravel (SP) (Fill), black and gray, no plasticity, moist, dense, possible fill, difficult to distinguish.									
5100				7-13-20	33											
							37.0 - 52.0 ft. Poorly graded SAND with silt (SP-SM) and some gravel, brown, tan, gray, pink, no plasticity, moist to wet, dense.									

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Project Name: I-270 Environmental Assessment and Conceptual Design

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Project Number: 219-418 *Boring No.:* **YA-B-02**

Elevation (feet)	Depth (feet)	Sample Type/Depth	Drilling Method	Soil Samples: Blows per 6 in	Soil Samples: Penetration Resistance	Lithology	Material Description	Moisture Content (%)	Dry Density (pcf)	Gravel Content (%)	Sand Content (%)	Fines Content (%)	Atterberg Limits: Liquid Limit	Atterberg Limits: Plasticity Index	AASHTO & USCS Classifi-cations	Field Notes and Other Lab Tests
				11-18-12	30		**37.0 - 52.0 ft. Poorly graded SAND with silt (SP-SM) and some gravel**, brown, tan, gray, pink, no plasticity, moist to wet, dense.	13.9		10.0	83.0	7.0	NV	NP	A-1-b (0) SP-SM	
5090																
	50						**52.0 - 65.8 ft. CLAYSTONE**, gray, very hard.									
5080				50:5"	50:5"			19.0	108.9	0.0	20.0	80.0	57	29	A-7-6 (25) CH	S/C=1.4%
	60			50:5"	50:5"											
5070				31-50:4"	81:10"											

Bottom of Hole at 65.8 ft.

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Project Name: I-270 Environmental Assessment and Conceptual Design

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Project Number: 219-418 *Boring No.:* **YA-PV-110**

Boring Began: 12/16/2020
Boring Completed: 12/16/2020
Drilling Method(s): Solid-Stem Auger
Driller: Vine Laboratories
Drill Rig: CME 55 Truck
Hammer: Automatic (hydraulic), ER: 85%

Total Depth: 11.5 ft
Ground Elevation: 5125.3 ft
Coordinates: N: 176118.6 E: 154732.3
Location:
Logged By: L. Safari
Final By: L. Safari

Weather Notes: Cold, clear
Inclination from Horiz.: Vertical
Night Work: ☑

Groundwater Levels: Not Observed			
Symbol			
Depth	-	-	-
Date	-	-	-

Elevation (feet)	Depth (feet)	Sample Type/Depth	Drilling Method	Soil Samples: Blows per 6 in	Soil Samples: Penetration Resistance	Lithology	Material Description	Moisture Content (%)	Dry Density (pcf)	Gravel Content (%)	Sand Content (%)	Fines Content (%)	Atterberg Limits: Liquid Limit	Atterberg Limits: Plasticity Index	AASHTO & USCS Classifi-cations	Field Notes and Other Lab Tests
							0.0 - 1.3 ft. ASPHALT (15 inches).									
				4-10-15	25		**1.3 - 3.0 ft. Clayey SAND (SC) with some gravel (Fill)**, brown, medium plasticity, moist, medium dense.	9.6		10.0	56.0	34.0	31	17	A-2-6 (1) SC	
5120				14-14-12	26		**3.0 - 7.0 ft. Poorly graded SAND with gravel (SP) (Fill)**, brown-tan-white, no plasticity, moist, medium dense.									
	10			8-9-10	19		**7.0 - 11.5 ft. Sandy lean CLAY with gravel (CL) (Fill)**, brown, low plasticity, moist, very stiff.									

Bottom of Hole at 11.5 ft.

ALPINE ENGINEERING

Laboratory Test Results

Project No: 219-418 / 220-294 Project Name: CDOT I-270 Date: 12/14/2020

Sample Location			Natural Moisture Content (%)	Natural Dry Density (pcf)	Gradation			Atterberg			PH	Water Soluble Sulfate %	Resistivity ohm.cm	Chloride %	% Swell (+) / Consoli-dation (-)	Uncon-Comp-Strength (psf)	R-Value	CLASSIFICATION	
Boring No. (final)	Depth (ft)	Sample Type			Gravel > #4 (%)	Sand (%)	Fines < #200 (%)	LL	PL	PI								AASHTO	USCS
YA-B-01	1-5	Bulk	11.4	–	11	51	38	32	16	16	–	–	–	–	–	–	–	A-6 (2)	SC
YA-B-01	10	SPT	15.2	–	2	53	45	28	16	12	–	–	–	–	–	–	–	A-6 (2)	SC
YA-B-01	50	CA	17.1	105.5	0	62	38	NV	NP	NP	–	–	–	–	–	–	–	A-4 (0)	SM
YA-B-01	58	CA	21.5	103.3	–	–	–	–	–	–	–	–	–	–	–	12852	–	–	–
YA-B-02	1-5	Bulk	4.6	–	14	68	18	NV	NP	NP	–	–	–	–	–	–	–	A-1-b (0)	SM
YA-B-02	20	SPT	–	–	–	–	–	–	–	–	8.0	0.006	974	0.0245	–	–	–	–	–
YA-B-02	25	SPT	13.4	–	2	63	35	26	18	8	–	–	–	–	–	–	–	A-2-4 (0)	SC
YA-B-02	40	SPT	13.9	–	10	83	7	NV	NP	NP	–	–	–	–	–	–	–	A-1-b (0)	SP-SM
YA-B-02	55	CA	19.0	108.9	0	20	80	57	28	29	–	–	–	–	1.4 (1ksf)	–	–	A-7-6 (25)	CH
YA-B-03	30	SPT	10.5	–	26	70	4	NV	NP	NP	–	–	–	–	–	–	–	A-1-b (0)	SW
YA-B-03	50	CA	16.3	111.8	–	–	–	–	–	–	7.6	0.003	1249	0.0008	–	–	–	–	–
YA-B-03	55	CA	16.2	113.4	–	–	–	–	–	–	–	–	–	–	–	28840	–	–	–
YA-B-04	15	SPT	8.1	–	21	72	7	NV	NP	NP	–	–	–	–	–	–	–	A-1-b (0)	SP-SM
YA-B-04	25	CA	14.2	118.5	0	7	93	42	21	21	–	–	–	–	1.7 (1ksf)	–	–	A-7-6 (21)	CL
YA-B-04	35	CA	13.8	115.4	–	–	–	–	–	–	–	–	–	–	–	18728	–	–	–
YA-B-05	0-5	Bulk	4.3	–	30	56	14	NV	NP	NP	–	–	–	–	–	–	–	A-1-b (0)	SM
YA-B-05	20	SPT	8.6	–	2	86	12	NV	NP	NP	–	–	–	–	–	–	–	A-1-b (0)	SW-SM

Project No: 219-418 / 220-294 Project Name: CDOT I-270 Date: 12/14/2020

Sample Location			Natural Moisture Content (%)	Natural Dry Density (pcf)	Gradation			Atterberg			PH	Water Soluble Sulfate %	Resistivity ohm.cm	Chloride %	% Swell (+) / Consoli-dation (-)	Uncon-Comp-Strength (psf)	R-Value	CLASSIFICATION	
Boring No. (final)	Depth (ft)	Sample Type			Gravel > #4 (%)	Sand (%)	Fines < #200 (%)	LL	PL	PI								AASHTO	USCS
YA-PV-110	1-5	Bulk	9.6	–	10	56	34	31	14	17	–	–	–	–	–	–	–	A-2-6 (1)	SC

Appendix B2: Geotechnical Schematic Design

Driven Pile Foundation Equations

Unless otherwise specified, all equations for driven pile foundation design were found in the AASHTO LRFD Bridge Design Manual (AASHTO, 2020).

The surface area per foot length of a driven H pile should be approximated as the perimeter of the outer dimensions (in this case approximate the H pile as an 18”X18” square) because soil inside the flanges may not provide sufficient resistance.

$$Surface\ Area\ Per\ Foot = (1.5\ ft)(4\ ft)/ft = 6\ ft^2/ft$$

The fundamental governing equation for pile capacity (resistance) is

$$R_R = \varphi R_n = \varphi_{stat} R_p + \varphi_{stat} R_s$$

$$R_p = q_p A_p$$

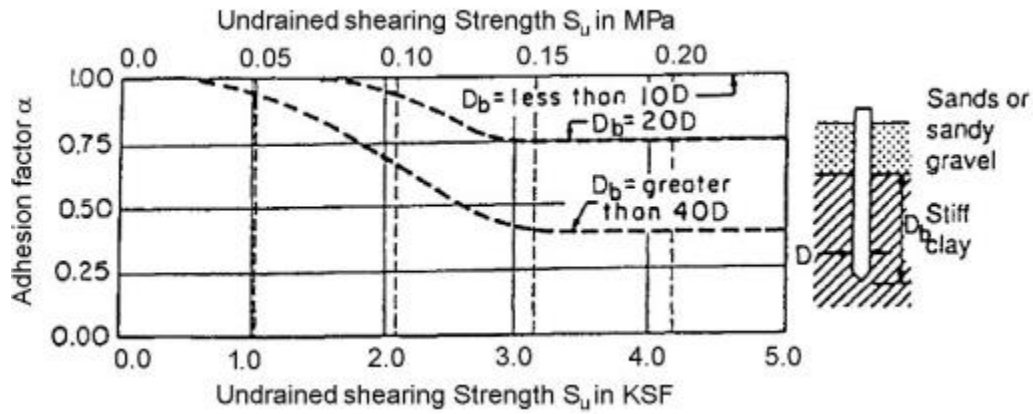
$$R_s = q_s A_s$$

φ_{stat} = resistance factor for the bearing resistance of a single pile specified in Article 10.5.5.2.3
R_p = pile tip resistance (kips)
R_s = pile side resistance (kips)
q_p = unit tip resistance of pile (ksf)
q_s = unit side resistance of pile (ksf)
A_s = surface area of pile side (ft^2)
A_p = area of pile tip (ft^2)

In this case, $\varphi_{stat} = 0.65$ as given in the Preliminary Geotechnical Report. The alpha method is a specific way to find the unit tip resistance of the pile using the equation

$$q_s = \alpha S_u$$

Where alpha is the adhesion factor. Alpha was determined to be 0.4 from the chart below.



The beta method was used to determine the frictional resistance of the pile-sand interface. The beta method equations in the LRFD were specific to clay, so equations from the foundation design textbook were used to calculate the capacity of the pile in sand (Das, Principles of Foundation Engineering Seventh Edition, 1983).

$$\beta = K_s \tan(\delta)$$

$$q_s = \beta(\sigma_0')$$

$$K_s = K_0 = (1 - \sin(\phi'))$$

The vertical stress at the midpoint of the sand (note should not be effective in the special case of sand beneath a body of water) is given by the equation: $\sigma_{v0}' = \frac{(width\ of\ sand\ layer)}{2}(125\ PSF)$

Drilled Shaft Foundation Equations

Table 6 – Nominal Drilled Shaft Axial Resistances

Bridge Structure ID	Reference Borings	Estimated Top of Bedrock Elevation (feet)	Axial Resistance Strength Limit State	
			Nominal Side Resistance (ksf)	Nominal Tip Resistance (ksf)
E-17-IC[1]	B-1 B-2 B-3 B-8 B-9 B-10	5131 5127 5133 5133 5130 5134	4.9	60
E-17-ID E-17-IE	YA-B-01 YA-B-02	5080 5082	9.0	110
E-17-IF E-17-IG	YA-B-03 YA-B-04	5084 5107	8.3	101
E-17-IH E-17-II	YA-B-05 YA-B-06 YA-B-07	5109 5108 5110	10.4	128
E-17-IJ E-17-IK	YA-B-08 YA-B-09 YA-B-10	5130 5133 5134	9.4	125
E-17-AT	YA-B-11 YA-B-12	5146 5148	7.1	87
E-17-IN E-17-IO	YA-B-13 YA-B-14	5140 5138	9.0	110

[1]See CDOT Structure Memorandum "Foundation Investigation – Structure E-17-IC" dated July 7, 1967, for boring logs and details

Calculations following the guidelines from the Preliminary Geotechnical Report:

$$Q_p = (A_p)(110\ KSF)$$

$$Q_s = (A_s)(9\ KSF)$$

Where A_s is the total side area of the shaft in hard bedrock. This method uses only the capacity of the shaft in hard bedrock and omits the capacity from embedment in sand and weathered bedrock.

$$A_s = \pi(Diameter)(Length\ of\ Embedment)$$

Calculations following the guidelines the Das Foundation Design textbook:

For the hard and weathered bedrock the following equations were used:

Note these equations are given for Metric units, conversions were built into the Excel table to convert into and out of Metric and English units.

$$Q_p(\mathrm{MN}) = q_p A_p = [4.83(q_u\ \mathrm{MN/m^2})^{0.51}][A_p(\mathrm{m^2})]$$

$$Q_s(\text{MN}) = fpL = [0.4(q_u\,\text{MN/m}^2)^{0.5}][\pi D_s(\text{m})][L(\text{m})] \quad \text{(for smooth socket)}$$

$$Q_s(\text{MN}) = fpL = [0.8(q_u\,\text{MN/m}^2)^{0.5}][\pi D_s(\text{m})][L(\text{m})] \quad \text{(for rough socket)}$$

In this case a medium-rough socket was used so the equation for unit side resistance is

$$q_s = (0.6)({q_u}^{0.5})$$

The side resistance from the sand requires different equations. For conservatism, the top 7 feet of sand were excluded from capacity because of the potential for scour. The equations used to calculate the capacity added by the pile friction with sand are below:

Estimation of Q_s

The frictional resistance at ultimate load, Q_s, developed in a drilled shaft may be calculated as

$$Q_s = \int_0^{L_1} pf\,dz \tag{12.20}$$

where

p = shaft perimeter = πD_s

f = unit frictional (or skin) resistance = $K\sigma'_o \tan\delta'$ (12.21)

K = earth pressure coefficient $\approx K_o = 1 - \sin\phi'$ (12.22)

σ'_o = effective vertical stress at any depth z

Thus,

$$Q_s = \int_0^{L_1} pf\,dz = \pi D_s(1 - \sin\phi')\int_0^{L_1} \sigma'_o \tan\delta'\,dz \tag{12.23}$$

The value of σ'_o will increase to a depth of about $15D_s$ and will remain constant thereafter, as shown in Figure 11.16.

For cast-in-pile concrete and good construction techniques, a rough interface develops and, hence, δ'/ϕ' may be taken to be one. With poor slurry construction, $\delta'/\phi' \approx 0.7$ to 0.8.

Good construction was assumed and thus $\delta' = \phi'$. The effective stress is linear with depth and thus the initial effective stress at the midpoint of the sand layer was used in the calculations as it represents the average initial effective stress.

Drilled shaft reinforcement calculations:

The drilled shaft reinforcement was designed per FHWA and AASHTO standards, which state that for vertical steel reinforcement $\frac{A_s f_y}{A_g f'_c} = 0.135$, solving this equation for A_s yields $A_s = \frac{(0.135)(9.6\ ft^2)(4.5\ KSI)}{60\ KSI} = 0.085\ ft^2 = 12.3\ in^2$. This required A_s can be achieved with 8 #11 bars (CDOT recommends maximum of #11 for vertical reinforcement) (AASHTO, 2020). These bars must extend 3 feet from the top of the caisson to form a connection with the cap. The shear reinforcement is designed per CDOT standards with a required concrete cover of 4 inches. The maximum AASHTO spacing for ties in a drilled shaft is 9 inches, and #3 bars are typical. Therefore, the reinforcing for each caisson is 3 ft diameter #3 rebar ties spaced at 9 inches. Note there is an extra tie (hoop) near the top and bottom of each caisson.

Table 5-5: Minimum Clear Cover for Drilled Shafts on Bridges

Drilled Shaft Diameter, D (ft.)	Minimum Concrete Cover (in.)
D ≤ 3	3
3 < D < 5	4
D ≥ 5	6

Slope Stability Equations and Tables

From the AASHTO LRFD Bridge Design Manual (AASHTO, 2020) and referenced by CDOT a rough factor of safety for cohesionless soils can be found with the equation below, $\phi = \phi' = 32°$.

$$FS = \frac{Tan\phi}{Tan\beta}$$

Where:

ϕ = the angle of internal friction for the soil

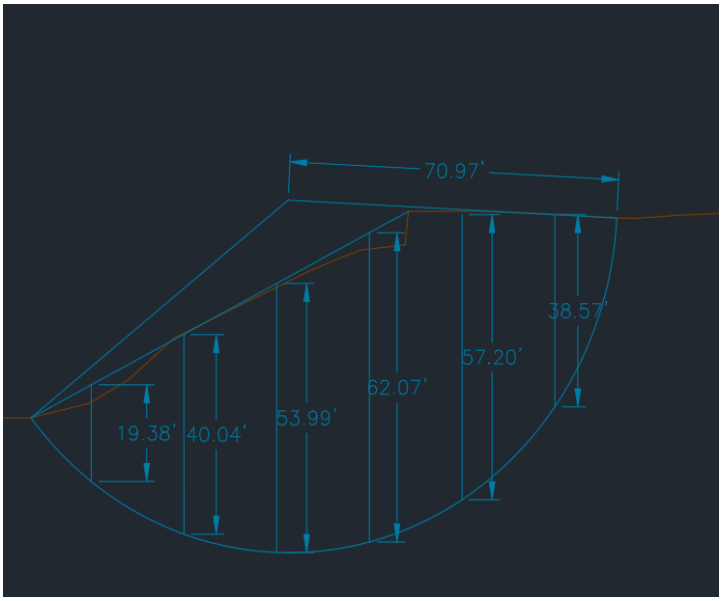
β = the slope angle relative to the horizontal

The tables below were used to compute the slope stability with the method of slices. All information for this methodology was found in the paper "Comparison of slope stability methods of analysis" (Krahn, 1977).

Base Failure South East Slope

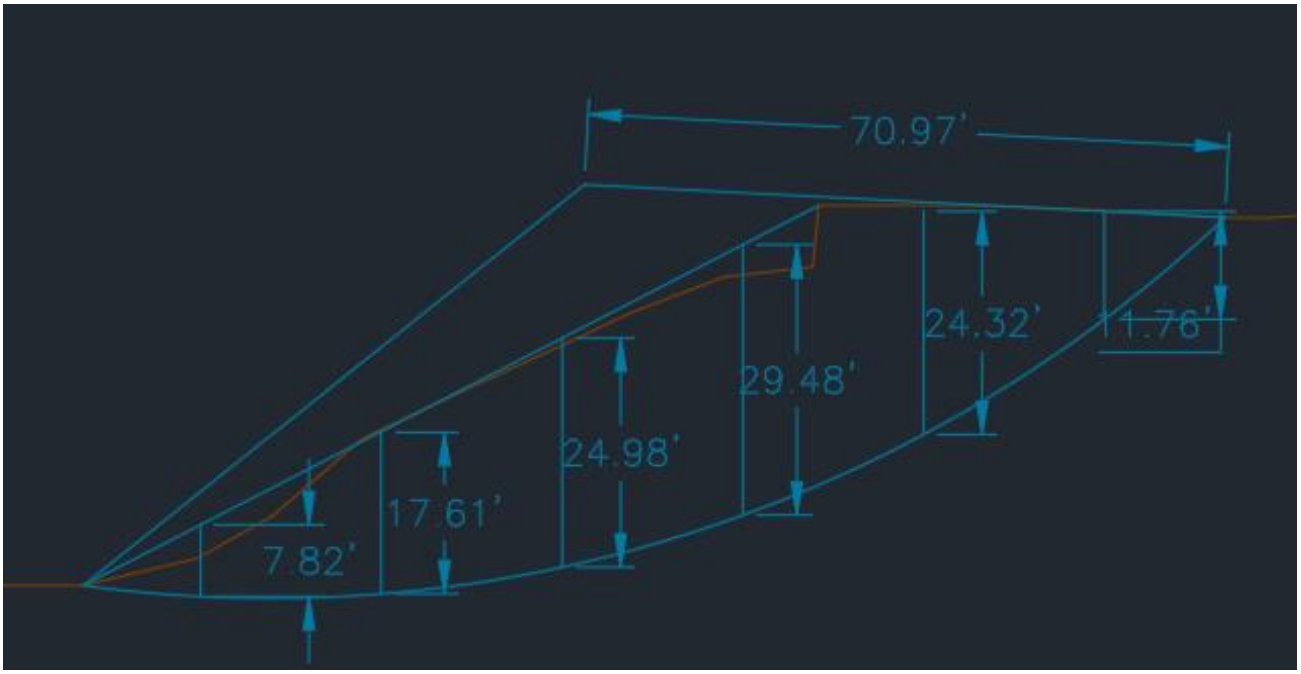
	Phi' (Degrees)	32
Sat. Unit Weight (PCF)	135	

Slice	Height Right (ft)	Height Left (ft)	Width (ft)	Area (ft^2	Weight (lbs)	Vertical Stress (midpoint) (PSF)	Resisting Shear Force (PSF)	Slice Bottom Slope (Radians)	Frictional Resisting Force (lbs/ft)	Tf (Resisting Force) (lbs/ft)	Driving Force (lbs/ft)
1	0.00	38.57	7.00	135.00	18224.33	1301.74	813.42	1.22	5570.94	3481.11	17125.26
2	38.57	57.20	10.00	478.85	64644.75	3232.24	2019.73	0.79	43361.98	27095.57	45710.74
3	57.20	62.07	10.00	596.35	80507.25	4025.36	2515.33	0.47	68807.39	42995.63	36549.53
4	62.07	53.99	10.00	580.30	78340.50	3917.03	2447.63	0.09	75196.02	46987.69	6827.82
5	53.99	40.04	10.00	470.15	63470.25	3173.51	1983.03	-0.09	60922.64	38068.69	-5531.80
6	40.04	19.38	10.00	297.10	40108.50	2005.43	1253.13	-0.79	26903.72	16811.31	-28360.99
7	19.38	0.00	5.00	48.45	6540.75	654.08	408.71	-0.79	4387.36	2741.53	-4625.01
									Sum	178181.5233	67695.55815
									F.S.	2.63210066	



Base Failure South East Slope		
	Phi' (Degrees)	32
Sat. Unit Weight (PCF)	135	

Slice	Height Right (ft)	Height Left (ft)	Width (ft)	Area (ft^2)	Weight (lbs)	Vertical Stress (midpoint) (PSF)	Resisting Shear Force (PSF)	Slice Bottom Slope	Frictional Resisting Force (lbs/ft)	Tf (Resisting Force) (lbs/ft)	Driving Force (lbs/ft)
1	0.00	11.76	7.00	41.16	5556.60	396.90	248.01	0.79	3727.22	2329.03	3929.11
2	11.76	24.32	10.00	180.40	24354.00	1217.70	760.90	0.79	16336.02	10207.88	17220.88
3	24.32	29.48	10.00	269.00	36315.00	1815.75	1134.61	0.61	28428.06	17763.82	20829.43
4	29.48	24.98	10.00	272.30	36760.50	1838.03	1148.53	0.52	30499.90	19058.45	18380.25
5	24.98	17.61	10.00	212.95	28748.25	1437.41	898.20	0.35	25970.00	16227.86	9832.48
6	17.61	7.82	10.00	127.15	17165.25	858.26	536.30	0.00	16541.58	10336.33	0.00
7	7.82	0.00	5.00	19.55	2639.25	263.93	164.92	-0.02	2542.96	1589.01	-46.06
									Sum	77512.3726	70146.0857
									F.S.	1.105013513	



For conservative design, flood conditions were considered, resulting in pore water pressure throughout the slope. Pore water pressure was calculated with:

$$u = h_w \gamma_w \cos^2(a_w)$$

Area and weight of each slice were calculated based on dimensions, angles and dimensions were found in CAD:

$$A_i = H_i * D_i$$

$$W_i = \gamma * A_i \ (kN/m)$$

The soil is assumed to be cohesionless (conservative assumption) so the only resisting force is frictional:

$$\overline{N_i} = W_i \cos(a) - u * L$$

The driving force was found using the equation:

$$W_{xi} = W_i sin(a)$$

where alpha is the inclination of the base of the slice.

The factor of safety was found using the equation:

$$\text{FoS} = \frac{\sum_{i=1}^{n} \text{Resisting Forces}}{\sum_{i=1}^{n} \text{Driving Forces}}$$

Retaining Structure Equations and methodology

All retaining structure calculations and guidelines were based on the Federal Highway Administration's Mechanically Stabilized Earth Walls design guide (FHA, 2009) unless otherwise stated.

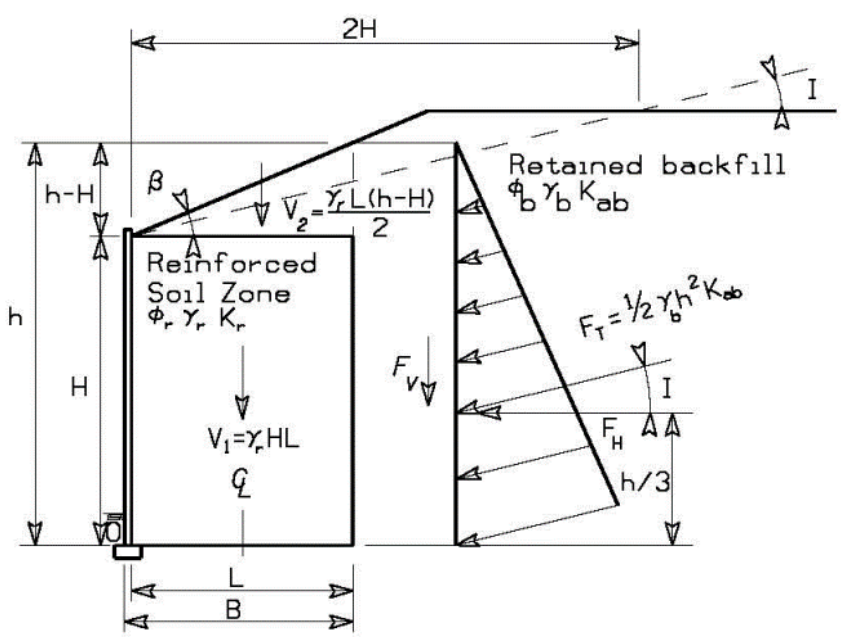
<u>Basic Geometry Equations and Diagram (FHA,2009):</u>

$$L = (length\ of\ reinforcement) = maximum\ of\ .7H\ or\ 8\ ft$$

$$h = (hieght\ of\ soil\ above\ base\ of\ wall) = H + L(\tan(\beta))$$

$$B = L + width\ of\ wall$$

where β is the slope of the backfill.



Earth Pressures and External Stability Checks:

The active earth pressure should be used for MSE wall calculations. From the Federal Highway Administration, the coefficient of active lateral earth pressure can be found from the equations

$$k_a = \frac{\sin^2(\theta + \phi'_f)}{\Gamma\,[\sin^2\theta \sin(\theta - \delta)]}$$

in which:

$$\Gamma = \left[1 + \sqrt{\frac{\sin(\phi'_f + \delta)\sin(\phi'_f - \beta)}{\sin(\theta - \delta)\sin(\theta + \beta)}}\right]^2$$

where

δ = friction angle between fill and wall (degrees)
β = angle of fill to the horizontal as shown in Figure 3.11.5.3-1 (degrees)
θ = angle of back face of wall to the horizontal as shown in Figure 3.11.5.3-1 (degrees)
ϕ'_f = effective angle of internal friction (degrees)

and the force of the backfill per unit height of the retaining wall is found with the following equation

$$F_T = \tfrac{1}{2} K_{ab}\,\gamma_b\,h^2$$

where:

K_{ab} = active earth pressure coefficient for the sloping backfill, see Eq. 4-2
h = total height of wall, H, and slope at the back of the reinforced zone
= H + L tan β

Inserting the site-specific parameters yields the following results:

Earth Pressures		
Γ	2.047029739	
Ka	0.373878936	
F_T	1579.723891	
F1	630.9207051	

The factors for the external stability checks were given by the Preliminary Geotechnical Report and are shown in the table below:

Resistance Factors	
Bearing	0.65
Sliding	1
Global Stability	0.65

<u>Sliding Stability:</u>

Excel Table:

Sliding Stability		
Nominal Retained Backfill Force	1579.72	lbs/ft
Nominal Horizontal Driving Force	1484.45	lbs/ft
REMOVED	REMOVED	REMOVED
EH Factor	1.50	N/A
EV Factor (Min)	1.00	N/A
EV Factor (Max)	1.35	N/A
Total Driving Force (Pd)	2226.68	lbs/ft
V1	5000.00	lbs/ft
V2	1455.88	lbs/ft
Nominal Resisting Force Per Unit Weight of Wall (R_R)	4813.86	lbs/ft
Factored Resisting Force Per Unit Weight of Wall (R_R)	4813.86	lbs/ft
Factor of Safety Sliding (R_R/Pd)	3.24	N/A

The sliding stability was found to have a factor of safety of 3.24. For the retaining wall, as long as the factor of safety for any stability check is greater than 1.2 the design is considered acceptable. The equations below were used to find the nominal horizontal driving force and total driving force.

For a sloping backfill condition:

$$\mathbf{F_H = F_T \cos\beta}$$

$$\mathbf{P_d = \gamma_{EH} F_H = \gamma_{EH} F_T \cos\beta}$$

Use the maximum EH load factor (= 1.50) in these equations because it creates the maximum driving force effect for the sliding limit state.

The vertical forces V_1 and V_2 were calculated using the equations shown in the diagram above. The nominal and factored resisting force are the same because the resistance factor is just 1.0. The resisting force was found with the equation below:

$$R_r = [\ \gamma_{EV}(V_1 + V_2) + \gamma_{EH}(F_T \sin\beta)] \times \mu$$

μ is the soil friction angle (32 degrees).

Finally, the factor of safety was found simply by dividing the resisting force of the wall by the driving force. The design is considered acceptable in sliding.

Eccentricity Check:

The eccentricity of the structure was found using the equation

$$e = \frac{\gamma_{EH\text{-}MAX} F_T \cos\beta \left(h/3\right) - \gamma_{EH\text{-}MAX} F_T \sin\beta \left(L/2\right) - \gamma_{EV\text{-}MIN} V_2 \left(L/6\right)}{\gamma_{EV\text{-}MIN} V_1 + \gamma_{EV\text{-}MIN} V_2 + \gamma_{EH\text{-}MAX} F_T \sin\beta}$$

This is essentially just a moment balance determining whether the structure will rotate too much at the base. According to the Federal Highway design guide, the eccentricity is considered acceptable as long as it is less than $\frac{1}{4}L$ which is equal to 2 feet in our case. The eccentricity for the structure is shown below:

Eccentricity Limit Check	
Eccentricity	0.09
Max Eccentricity	2

Bearing Resistance Check:

Bearing Resistance		
General Shear		
Eccentricity (Bearing)	0.09	N/A
Vertical Stress	640.18	PSF
Factored Bearing Stress q_vf	930.36	PSF
Nominal Bearing Resistance	14741.86	PSF
REMOVED	REMOVED	REMOVED
Factored Bearing Resistance	9582.21	PSF
Factor of Safety	10.30	N/A

The bearing resistance was found to be sufficient with a factor of safety of 10.30 between the factored resisting force and the driving force. The eccentricity from the eccentricity limit check was used although this is the maximum eccentricity (using the maximum e is not necessary for bearing checks). This results in a more conservative estimate of bearing. The factored bearing vertical stress was found using the equation

$$q_{V\text{-}F} = \frac{\gamma_{EV\text{-}MAX} V_1 + \gamma_{EV\text{-}MAX} V_2 + \gamma_{EH\text{-}MAX} F_T \sin\beta}{L - 2e_B}$$

And the nominal bearing resistance was found using the equation:

$$q_n = c_f N_c + 0.5L'\gamma_f N_\gamma \qquad (4\text{-}22)$$

where: c_f = the cohesion of the foundation soil

γ_f = the unit weight of the foundation soil

N_c and N_γ = dimensionless bearing capacity coefficients

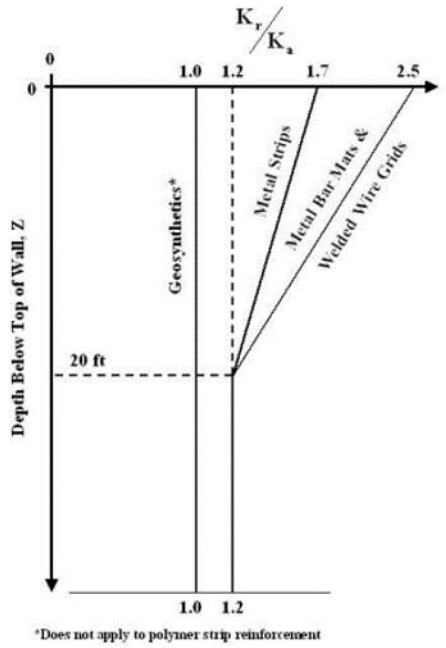
L' = effective foundation width, equal to $L - 2e_B$; set L' equal to L if e_B is a negative value

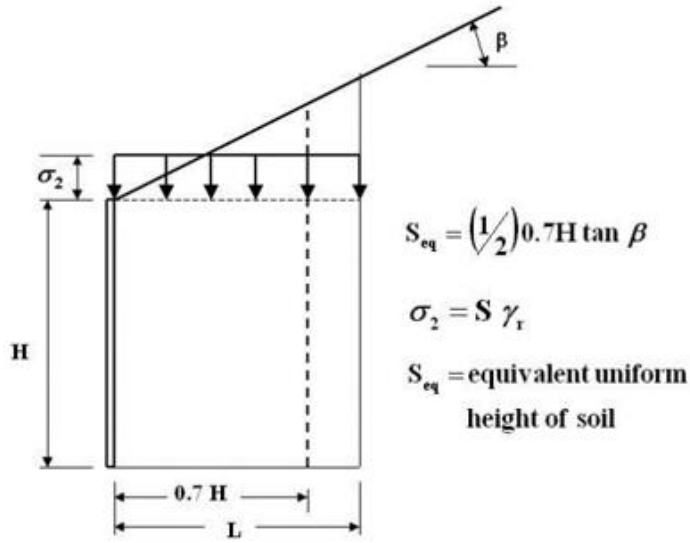
The bearing capacity coefficients were found from the table below.

ϕ	N_c	N_q	N_γ	ϕ	N_c	N_q	N_γ
0	5.14	1.0	0.0	23	18.1	8.7	8.2
1	5.4	1.1	0.1	24	19.3	9.6	9.4
2	5.6	1.2	0.2	25	20.7	10.7	10.9
3	5.9	1.3	0.2	26	22.3	11.9	12.5
4	6.2	1.4	0.3	27	23.9	13.2	14.5
5	6.5	1.6	0.5	28	25.8	14.7	16.7
6	6.8	1.7	0.6	29	27.9	16.4	19.3
7	7.2	1.9	0.7	30	30.1	18.4	22.4
8	7.5	2.1	0.9	31	32.7	20.6	25.9
9	7.9	2.3	1.0	32	35.5	23.2	30.2
10	8.4	2.5	1.2	33	38.6	26.1	35.2
11	8.8	2.7	1.4	34	42.2	29.4	41.1
12	9.3	3.0	1.7	35	46.1	33.3	48.0
13	9.8	3.3	2.0	36	50.6	37.8	56.3
14	10.4	3.6	2.3	37	55.6	42.9	66.2
15	11.0	3.9	2.7	38	61.4	48.9	78.0
16	11.6	4.3	3.1	39	37.9	56.0	92.3
17	12.3	4.8	3.5	40	75.3	64.2	109.4
18	13.1	5.3	4.1	41	83.9	73.9	130.2
19	13.9	5.8	4.7	42	93.7	85.4	155.6
20	14.8	6.4	5.4	43	105.1	99.0	186.5
21	15.8	7.1	6.2	44	118.4	115.3	224.6
22	16.9	7.8	7.1	45	133.9	134.9	271.8

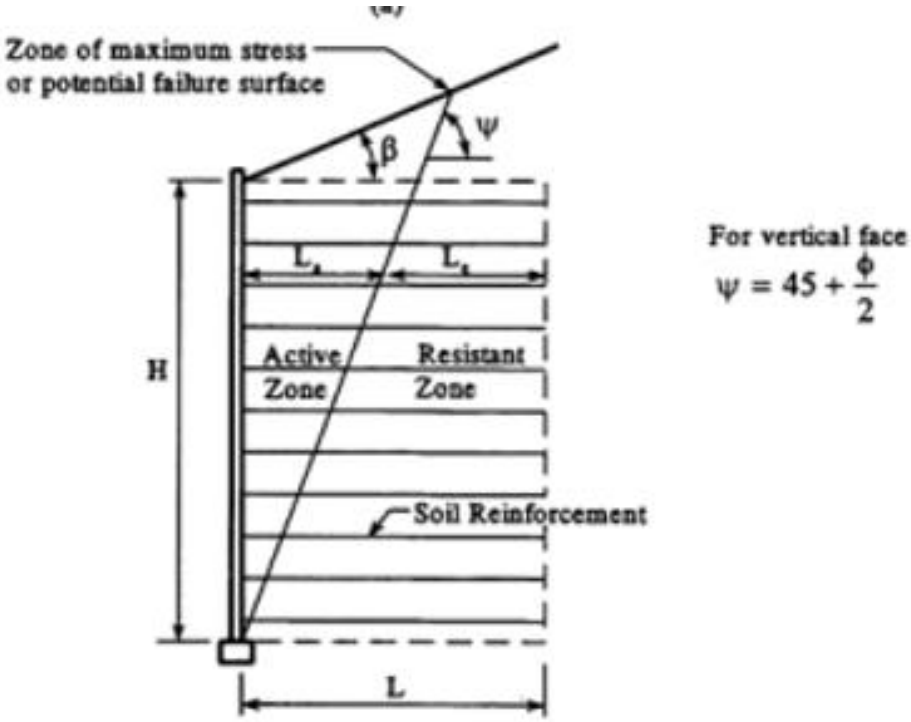
<u>Soil Reinforcement Resistance:</u>

The initial dimensions of the structure passed all external stability checks, so now the soil reinforcing layers must be designed. These provide the tensile capacity of the MSE wall. The first step is calculating the factored tensile stresses in the reinforcing layers. The Federal Highway guidelines specified that it is necessary to find the tensile stresses up to a depth of 0.8H, which is 8 feet in our case. This retaining wall will use geosynthetics to provide soil stability. The following diagrams were used to determine K_r and S_{eq}.





The simplified method was used for the internal resistance calculations which means that an assumed failure plane was taken by approximating the failure plane angle with the diagram below:



Internal Stability Check General Parameters		
Parameter	Degrees	Radians
Failure Angle (Psi)	61	1.06

Now, we can assemble the table to determine soil tensile stresses at the depths of reinforcement layers.

Factored Tensile Stresses in Reinforcement Layers			
Kr	0.37	N/A	
Seq	0.64	ft	Depth (z)
Horizontal Stress Z = 1 m	76.50	lbs/ft	1
Horizontal Stress Z = 2 m	123.24	lbs/ft	2
Horizontal Stress Z = 3 m	169.97	lbs/ft	3
Horizontal Stress Z = 4 m	216.71	lbs/ft	4
Horizontal Stress Z = 5 m	263.44	lbs/ft	5
Horizontal Stress Z = 6 m	310.18	lbs/ft	6

The equation to find the horizontal tensile stress is

$$\sigma_H = K_r\,[\gamma_r\,(Z + S_{eq})\,\gamma_{EV\text{-}MAX}]$$

These tensile stresses are essentially driving stresses. They are horizontal tensile stresses which must be resisted by the reinforcement layers. The selected geosynthetic material for the retaining structure is Miragrid 2XT. It was selected for its popularity, strength, and sustainability.

MIRAGRID 2XT

SOIL REINFORCEMENT

MIRAGRID® 2XT biaxial geogrid is composed of high molecular weight, high tenacity polyester multifilament yarns woven in tension and finished with a PVC coating. MIRAGRID 2XT geogrid is inert to biological degradation and resistant to naturally encountered chemicals, alkalis, and acids.

MIRAGRID 2XT geogrid is used as soil reinforcement in MSE structures such as segmental retaining walls, precast modular block walls, wire faced walls, geosynthetic wrapped faced walls and steepened slopes. MIRAGRID 2XT is also used in MSE stabilized platforms for voids bridging, embankments on soft soils, landfill veneer stability, reducing differential settlement and for foundation seismic stability.

TenCate Geosynthetics Americas (A Solmax Company) is accredited by Geosynthetic Accreditation Institute – Laboratory Accreditation Program (GAI-LAP).

MIRAFI 2XT meets Build America, Buy America Act, Pub. L. No. 117-58, div. G §§ 70901-52.

MECHANICAL PROPERTIES	TEST METHOD	UNIT	MINIMUM AVERAGE ROLL VALUE	
			MD	CD
Tensile Strength @ Ultimate	ASTM D6637(Method B)	lbs/ft (kN/m)	2300 (33.6)	2300 (33.6)
Tensile Strength @ 5% strain	ASTM D6637(Method B)	lbs/ft (kN/m)	1080 (15.8)	550 (8.0)

			MINIMUM ROLL VALUE	
			MD	CD
Creep Rupture Strength[1]	ASTM D5262/D6992	lbs/ft (kN/m)	1597 (23.3)	1597 (23.3)
Long Term Design Strength[2]		lbs/ft (kN/m)	1383 (20.2)	1383 (20.2)

PHYSICAL PROPERTIES	UNIT	ROLL SIZE
Roll Dimensions[3] (width x length)	ft (m)	4 x 50 (1.2 x 15) 6 x 150 (1.8 x 46) 12 x 150 (3.6 x 46)
Roll Area	yd^2 (m^2)	22 (18) 100 (84) 200 (167)
Estimated Roll Weight	lbs (kg)	25 (11) 50 (23) 109 (49)

[1] 75-year design life based on NTPEP Report REGEO-2016-01-062.
[2] Value based on Long Term Design Strength for sand, silt, clay. RF_{CR} = 1.44; RF_{ID} = 1.05; RF_D = 1.1
(Installation damage reduction factor for other soils available upon request)
[3] Special order roll lengths are available upon request.

Using the strength given in the specifications, the resistance can be found at the various depths.

Soil Reinforcement Resistance		
Resistance Factor	0.9	
Tult	2300	lbs/ft
RF_ID	1.5	
RF_CR	2.5	
RF_D	1.15	
Tal	533.33	lbs/ft
Resistance Factor	0.9	
Tr=Phi*Tal	480	
F.S. (Tr/T_Max)	1.54750413	

T_{Ult} was given in the specifications, and the various reduction factors were found from the tables below

Table 3-9. Installation Damage Reduction Factors.

Reduction Factor, RF_{ID}		
Geosynthetic	Type 1 Backfill Max. Size 4 in. (100mm) D_{50} about 1¼-in. (30 mm)	Type 2 Backfill Max. Size ¾ -in. (20mm) D_{50} about #30 (0.7 mm)
HDPE uniaxial geogrid	1.20 - 1.45	1.10 - 1.20
PP biaxial geogrid	1.20 - 1.45	1.10 - 1.20
PVC coated PET geogrid	1.30 - 1.85	1.10 - 1.30
Acrylic coated PET geogrid	1.30 - 2.05	1.20 - 1.40
Woven geotextiles (PP&PET)[a]	1.40 - 2.20	1.10 - 1.40
Non woven geotextiles (PP&PET)[a]	1.40 - 2.50	1.10 - 1.40
Slit film woven PP geotextile[a]	1.60 - 3.00	1.10 - 2.00
a. Minimum weight 8.0 oz/yd^2 (270 g/m^2).		

Typical ranges of RF_{CR} as a function of polymer type are:

Polymer Type	Creep Reduction Factors
Polyester (PET)	2.5 to 1.6
Polypropylene (PP)	5 to 4.0
High Density Polyethylene (HDPE)	5 to 2.6

Table 3-11. Durability (Aging) Reduction Factors for PET.

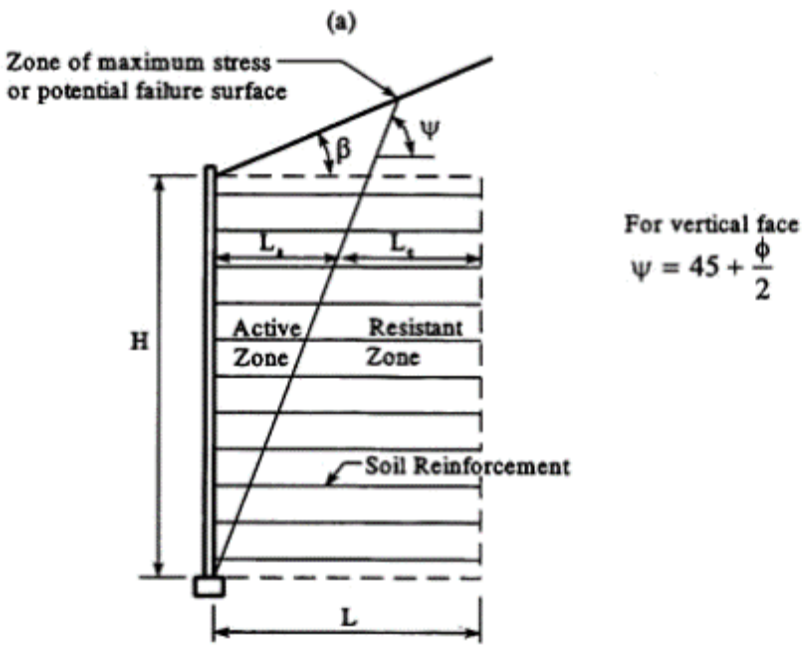
Product[a]	Durability Reduction Factor, RF_D	
	$5 \leq pH \leq 8$	$3^b < pH \leq 5$ $8 \leq pH < 9$
Geotextiles $M_n < 20{,}000$, $40 < CEG < 50$	1.6	2.0
Coated geogrids, Geotextiles $M_n > 25{,}000$, $CEG < 30$	1.15	1.3

M_n = number average molecular weight
CEG = carboxyl end group
Notes:
a. Use of materials outside the indicated molecular property range requires specific product testing. Use of products outside of $3 < pH < 9$ range is not recommended.
b. Lower limit of pH for permanent applications is 4.5 and lower limit for temporary applications is 3, per Article 11.10.6.4.2b (AASHTO, 2007).

Using these reduction factors, we can find $T_{Al} = \frac{T_{Ult}}{(RFD)(RFID)(RFCR)}$, the factor for tensile strength is 0.9, so the resisting tensile force is $0.9T_{Al}$. The maximum driving tensile stress should be used to determine the Factor of Safety. In this case, a factor of safety of 1.1 was found, which is acceptable because this is representative of the absolute least safe part of the structure.

Soil Reinforcement Pull Out Resistance and Required Embedment:

The resistance zone of the embedded soil reinforcement must be determined in order to find the required embedment. The resistance zone of the reinforcement is denoted L_e and it is equal to L (.8H = 8 feet) minus the length of the active zone, which depends on the failure plane and varies with depth.



The minimum L_e is 3 feet and the value of L_e is determined from the equation below

$$L_e \geq \frac{T_{MAX}}{\phi F^* \alpha \sigma_v C R_c} \geq 3 \text{ ft } (1 \text{ m})$$

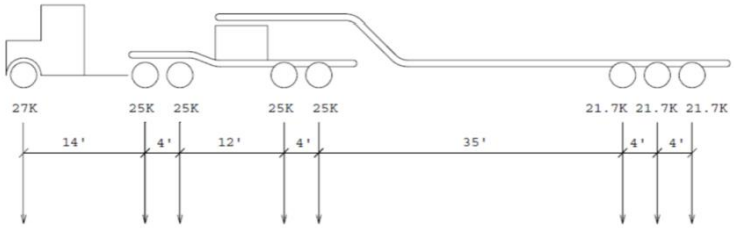
In this equation, T_{Max} is the maximum tensile force at the given depth (as determined above in section 3.4.2.4B). The pullout resistance factor (F), scale correction factor (alpha), and R_C constant all equal 1. C = 2, phi = 0.9, and the vertical stress is simply equal to the bulk unit weight of backfill multiplied by depth. Following this equation yields the following embedment depths for each geotextile layer:

Required Length of Embedment to Avoid Pullout Failure							
Reinforcement Layer	Z (Depth from top of structure) (ft)	Vertical Stress	Dist. From structure to failure plane (ft)	Required Le (ft)	Total Required Embedment (ft)	Designed Embedment (ft)	F.S.
1	1	125	2.22	3	5.22	8	1.53
2	2	250	1.66	3	4.66	8	1.72
3	3	375	1.11	3	4.11	8	1.95
4	4	500	0.55	3	3.55	8	2.25
5	5	625	0.00	3	3.00	8	2.67

Appendix C: Structural

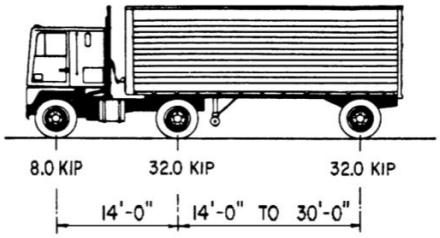
Loads:

- Asphalt, 146.67 $\frac{lb}{ft^3}$, (CDOT Bridge Design Manual 2022 3.4.2)
- Concrete, 145 $\frac{lb}{ft^3}$, (AASHTO LRFD Bridge Design Specs, 9th Edition, 3.5.1-1)
- Metal Deck, 5 $\frac{lb}{ft^2}$, (CDOT Bridge Design Manual 2022 3.4.1)
- Precast Girders, 173 $\frac{lb}{ft^3}$, (CDOT Bridge Design Manual 2022 3.4.1.1)
- Colorado Permit Vehicle, 192,000 lb , (CDOT Bridge Design Manual 3.6)



COLORADO PERMIT VEHICLE
192,000 LBS (96 Tons) on 8 Axles, 77 Feet Long

- Design Truck, 72,000 lb , (AASHTO LRFD Bridge Design Specs, 9th Edition, 3.6.1.2.2)



Dead Load

width of bridge = 106 ft = 1272 in

girder spacing = 81 in O.C.

# of girders = $\frac{1272\,in}{81\,in}$ = 13.7 => 13 girders

CS Area = $2 \times 240\,in^2 + 308\,in^2 = 788\,in^2 = 5.47\,ft^2$

Weight of girders = CS area × unit weight × # of girders

$= 5.47\,ft^2 \times 0.173\,\frac{k}{ft^2} \times 13$

Weight of girders = 15.15 k/ft

Asphalt weight = 0.1467 k/ft^3

Asphalt thickness = 4 in = 0.33 ft

Asphalt weight = width × unit weight × thickness

$= 106\,ft \times 0.1467\,\frac{k}{ft^3} \times 0.33\,ft$

Weight of asphalt = 5.18 k/ft

Concrete unit weight = 0.145 k/ft^3

Concrete thickness = 8 in = 0.667 ft

Weight of concrete = $106\,ft \times 0.145\,k/ft^3 \times 0.667\,ft$

Weight of concrete = 10.25 k/ft

Metal deck = $5 \frac{lb}{ft^2} = 0.005 \frac{k}{ft^2}$

Weight of metal deck = $106\ ft \times 0.005 \frac{k}{ft^2}$

Weight of metal deck = $0.53\ k/ft$

Live Load

CDOT Vehicle = 192,000 lb = 192 k

Tributary width = 10 ft

Vehicle Load = $\frac{192\ k}{10\ ft} = 19.2 \frac{k}{ft}$

Design Lane Load = $0.64\ k/ft$

Live Load = $19.2\ k/ft + 0.64\ k/ft$

Live Load = $19.84\ k/ft$ (for CDOT vehicle)

Normal vehicle = 72 k

Vehicle load = $\frac{72\ k}{10\ ft} = 7.2\ k/ft$

Design lane load = $0.64\ k/ft$

Live Load = $7.2\ k/ft + 0.64\ k/ft$

Live Load = $7.84\ k/ft$ (for normal vehicles)

Governing Load Combinations:

Table 3.4.1-1—Load Combinations and Load Factors

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	—	—	—	—	—	—	—	—	—	—	—	—

Note: For Service I, the load factor for *EV* equals 1.2 for Stiffness Method Soil Failure as shown in Table 3.4.1-2.

AASHTO LRFD Bridge Design Specifications, 9th edition Table 3.4.1-1

Governing Load Combination:

Strength I

$1.25DC + 1.5DW + 1.75LL_n$

Strength II

$1.25DC + 1.5DW + 1.35LL_c$

DC = Concrete deck + metal deck + girders

DC = 10.25 k/ft + 0.53 k/ft + 15.15 k/ft = 25.9 k/ft

DW = Asphalt weight

DW = 5.18 k/ft

LL_n = Live load (normal vehicle) = 7.2 k/ft + 0.64 k/ft = 7.84 k/ft

LL_c = Live load (CDOT vehicle) = 19.2 k/ft + 0.64 k/ft = 19.84 k/ft

Strength I

1.25(25.9 k/ft) + 1.5(5.18 k/ft) + 1.75(7.84 k/ft)

Strength I => 53.9 k/ft

Strength II

1.25(25.9 k/ft) + 1.5(5.18 k/ft) + 1.35(19.84 k/ft)

Strength II => 66.9 k/ft

Span Lengths:

Span Lengths

Total Bridge Length: 306 ft

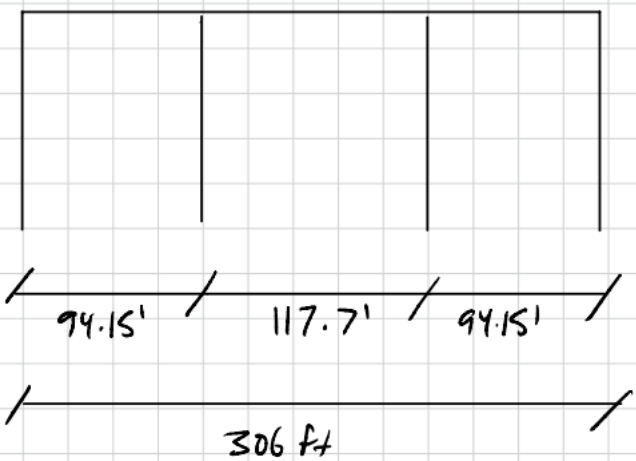
3-span bridge

$$2(0.8L_2) + L_2 = 306 \text{ ft}$$

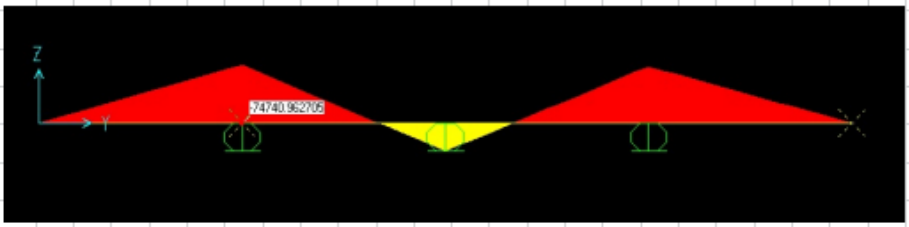
$$L_2 = 117.7 \text{ ft}$$

$$L_1 = L_3 = \frac{306 - 117.7}{2}$$

$$L_1 = L_3 = 94.15 \text{ ft}$$

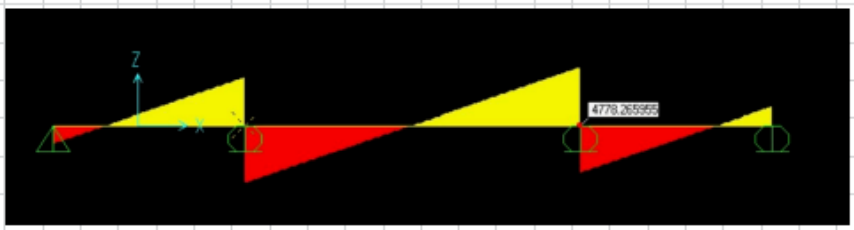


Shear and Moment:



$M_{max} = 74740.96$ k ft (from SAP)

$$\frac{M_{max}}{\#\text{ of girders}} = \frac{74740.96 \text{ k ft}}{13} = 5749.3 \text{ k-ft}$$



$V_{max} = 4778.3$ k

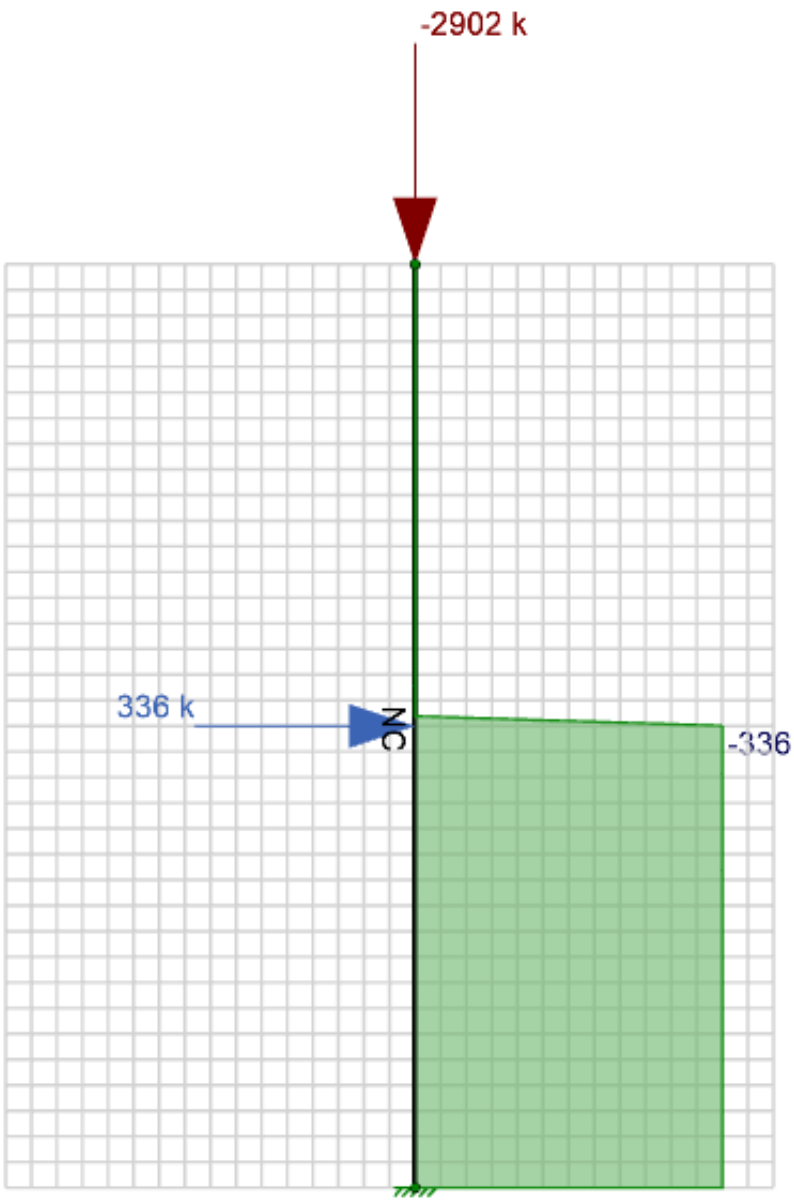
$$\frac{V_{max}}{\#\text{ of girders}} = \frac{4778.3 \text{ k}}{13} = 367.6 \text{ k}$$

$P_{max} = 8706.07$ k (from SAP)

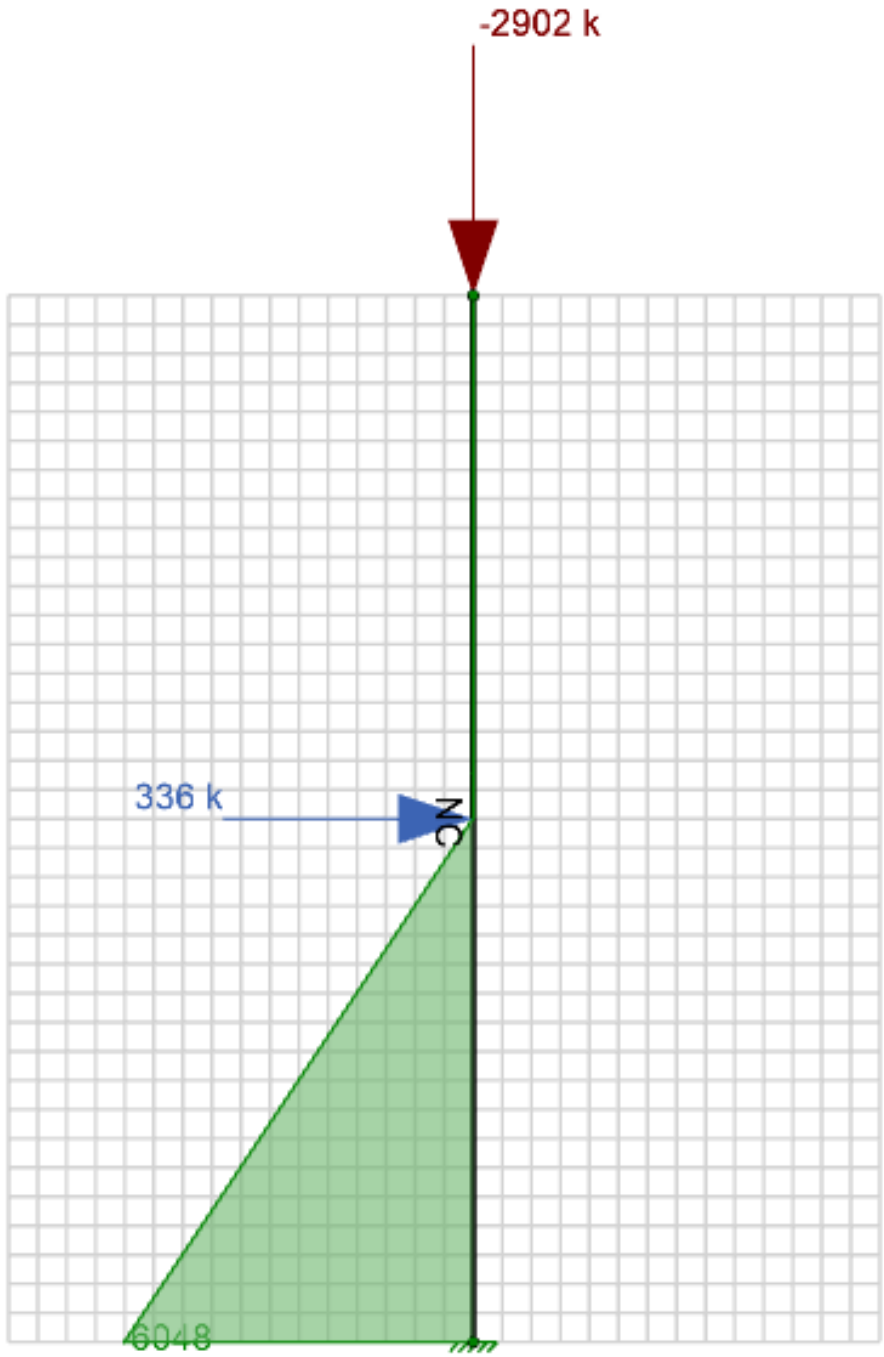
$$\frac{P_{max}}{\#\text{ of columns}} = \frac{8706.07 \text{ k}}{3} = 2902 \text{ k}$$

Columns in flexure:

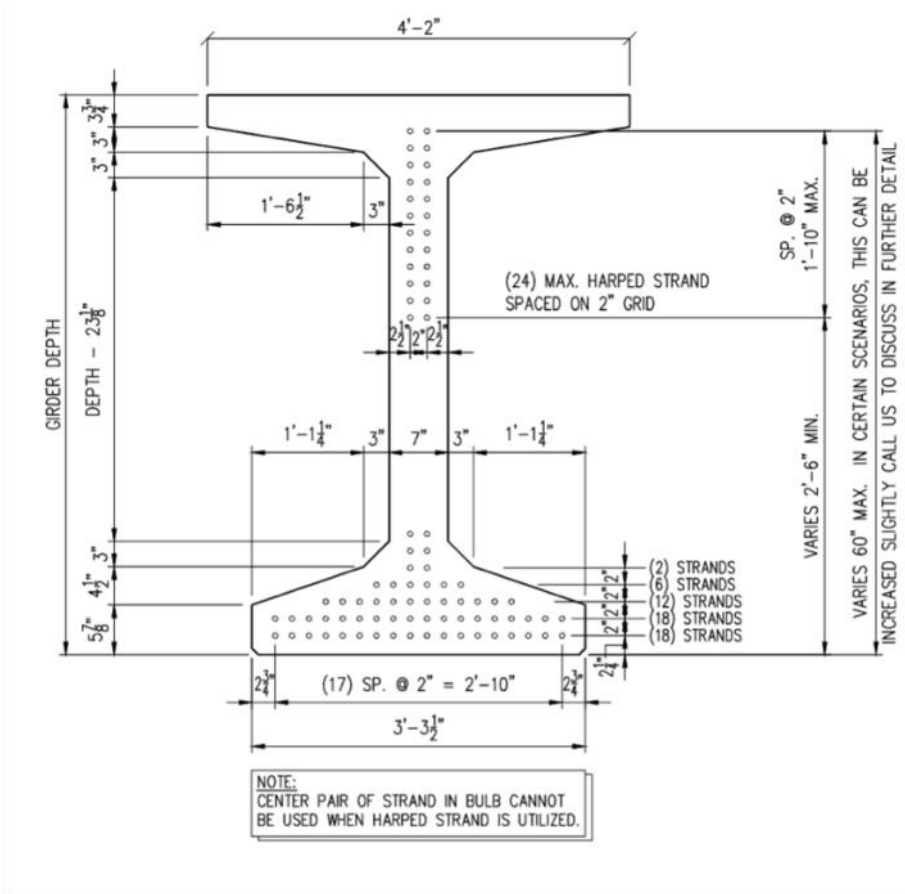
Shear diagram -



Moment diagram -



Girder Selection:



CDOT std. depths
37.5"
45"
54"
63"
72"
81"

Girder Depth:

Girder depth:

$$L = 117.7\,ft \times \frac{12\,in}{1\,ft} = 1412.4\,in$$

From Table 2.5.2.6.3-1 (AASHTO)

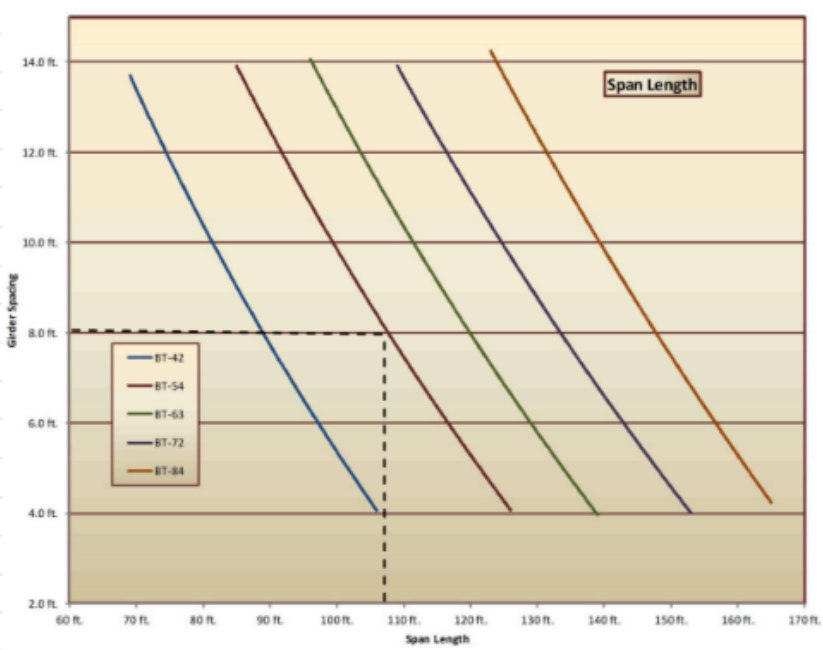
Use Minimum depth = 0.040L

$$0.040(1412.4\,in) - 12 = 44.5\,in$$

Conservatively round up to next girder depth

Girder depth = 54"

Girder count: $\frac{\text{bridge width}}{\text{girder spacing}} = \frac{107.25\,ft}{8\,ft} \Rightarrow$ 13 girders



Girder Depth:

Table 2.5.2.6.3-1—Traditional Minimum Depths for Constant Depth Superstructures

Superstructure		Minimum Depth (Including Deck) When variable depth members are used, values may be adjusted to account for changes in relative stiffness of positive and negative moment sections	
Material	Type	Simple Spans	Continuous Spans
Reinforced Concrete	Slabs with Main Reinforcement Parallel to Traffic	$\frac{1.2(S+10)}{30}$	$\frac{S+10}{30} \geq 0.54$ ft
	T-Beams	$0.070L$	$0.065L$
	Box Beams	$0.060L$	$0.055L$
	Pedestrian Structure Beams	$0.035L$	$0.033L$
Prestressed Concrete	Slabs	$0.030L \geq 6.5$ in.	$0.027L \geq 6.5$ in.
	CIP Box Beams	$0.045L$	$0.040L$
	Precast I-Beams	$0.045L$	$0.040L$
	Pedestrian Structure Beams	$0.033L$	$0.030L$
	Adjacent Box Beams	$0.030L$	$0.025L$
Steel	Overall Depth of Composite I-Beam	$0.040L$	$0.032L$
	Depth of I-Beam Portion of Composite I-Beam	$0.033L$	$0.027L$
	Trusses	$0.100L$	$0.100L$

Girder Design:

Rebar in girders:

$$A_s = \frac{M_u}{\phi f_y (d - a/2)}$$

Assume a = slab thickness for now

$$A_s = \frac{5749.3 \text{ kft} \times \frac{12''}{1ft}}{(0.9)(50 \text{ ksi})((8'+54'') - 8''/2)} = 26.43 \text{ in}^2$$

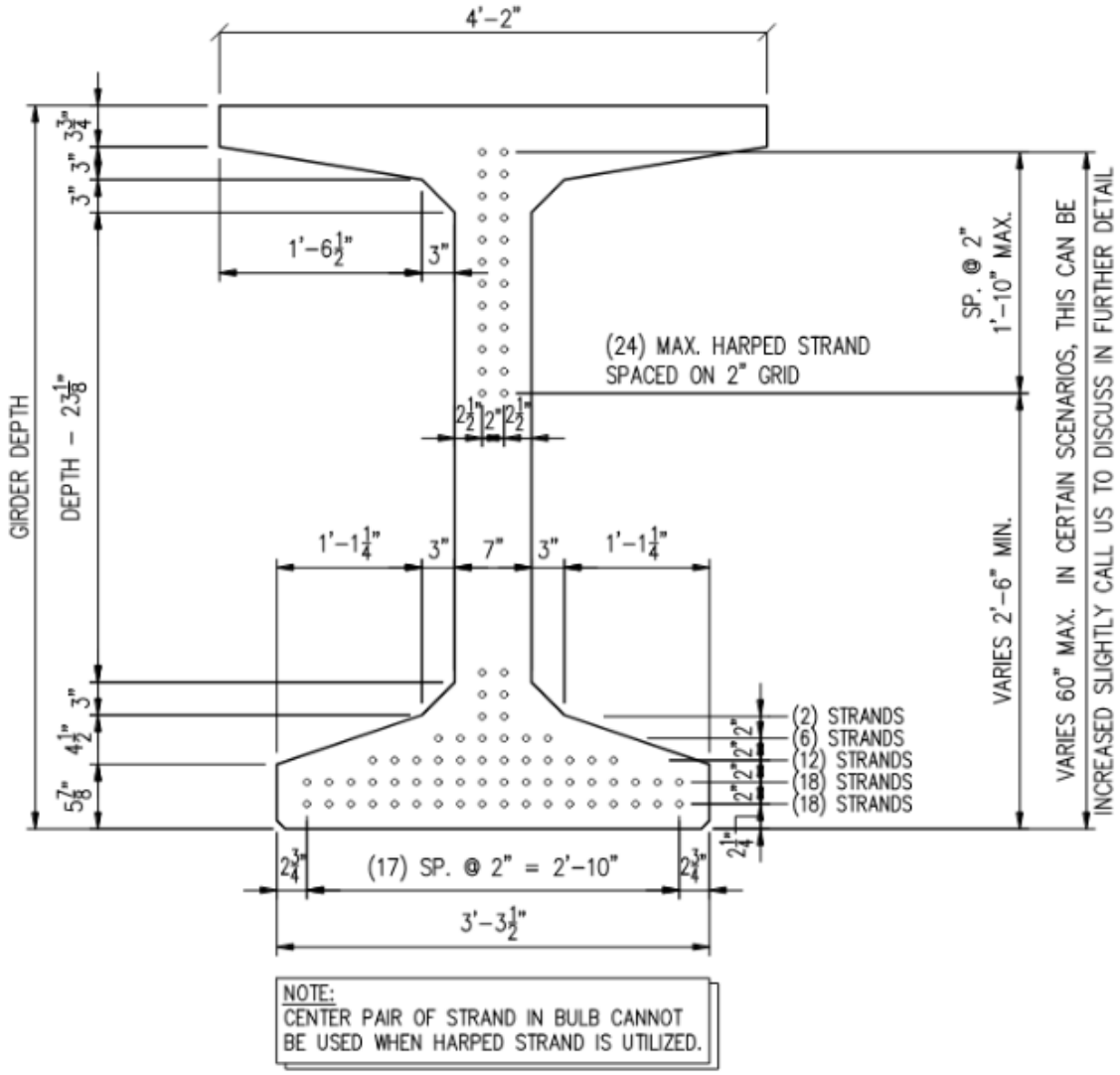
$$a = \frac{A_s f_y}{0.85 f'_c b} = \frac{(26.43 \text{ in}^2)(50 \text{ ksi})}{(0.85)(4.5 \text{ ksi})(48'')} = 7.19$$

Recalculate As w/ new a value

$$A_s = \frac{5749.3 \text{ kft} \times \frac{12 \text{ in}}{1 \text{ ft}}}{(0.9)(50 \text{ ksi})\left((8'+54'') - \frac{7.19}{2}\right)} = 26.25 \text{ in}^2$$

Area of #4 rebar = 0.2 in^2 $\qquad \frac{26.25 \text{ in}^2}{0.31} = 84.67$

, so use 85 #5 bars



NOTE:
CENTER PAIR OF STRAND IN BULB CANNOT BE USED WHEN HARPED STRAND IS UTILIZED.

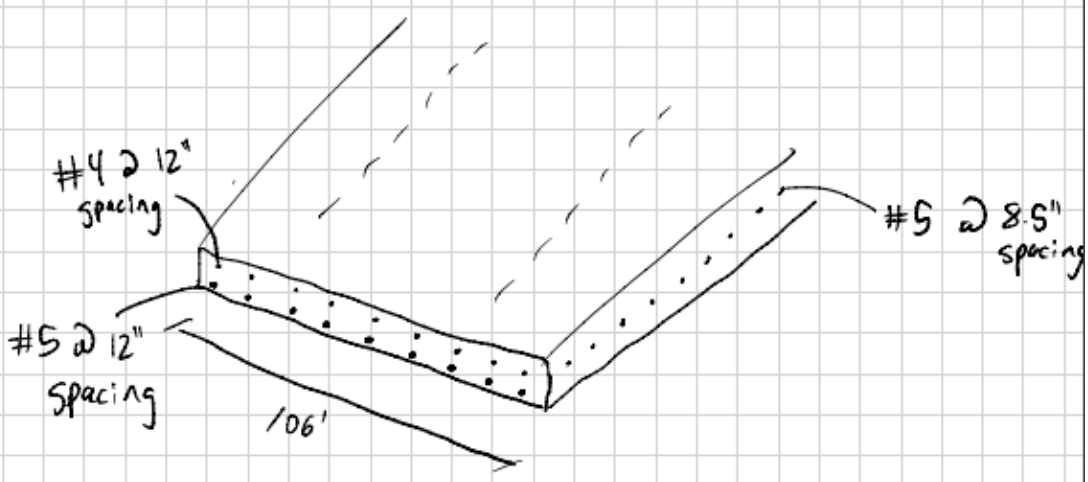
Deck Design:

Rebar in deck:

FROM Table 9-1 CDOT Bridge design manual 2022

Table 9-1: CDOT Standard CBT Girder Load and Resistance Factor Design (f_y = 60 ksi)

Girder spacing CL to CL	Concrete deck thick. (w/o haunch)	* Maximum overhang	Transverse reinforcing		Longitudinal reinforcing	
			Top / bot. mat slab reinforcing.		** Top mat (#4 min.)	Bot. mat/ "D" bar (#5)
(ft.)	(in.)	(ft. - in.)	Size	Max. Spacing (in.)	Max. Spacing (in.)	Max. Spacing (in.)
4.00	8.00	2' - 3.5"	#5	9	12	12
4.25	8.00	2' - 3.5"		9		12
4.50	8.00	2' - 3.5"		9		12
4.75	8.00	2' - 4.5"		9		12
5.00	8.00	2' - 6"		9		12
5.25	8.00	2' - 7.5"		9		12
5.50	8.00	2' - 9"		9		12
5.75	8.00	2' - 10.5"		9		12
6.00	8.00	3' - 0"		9		12
6.25	8.00	3' - 1.5"		9		12
6.50	8.00	3' - 3"		9		12
6.75	8.00	3' - 4.5"		9		12
7.00	8.00	3' - 6"		9		12
7.25	8.00	3' - 7.5"		9		12
7.50	8.00	3' - 9"		9		12
7.75	8.00	3' - 10.5"		9		12
8.00	8.00	4' - 0"		9		12
8.25	8.00	4' - 1.5"		8.5		12
8.50	8.00	4' - 3"		8.5		12
8.75	8.00	4' - 4.5"		8		11
9.00	8.00	4' - 6"		8		11



106 #4 longitudinally

106 #5 longitudinally

432 #5 transversely

$$A_{s_{longit.}} = (0.2\ in^2)(106) + (0.31\ in^2)(106) = 54.06\ in^2$$

$$A_{s_{trans.}} = (0.31\ in^2)(432) = 133.92\ in^2$$

$$A_{s_{Total}} = A_{s_{longit.}} + A_{s_{trans}} = 54.06\ in^2 + 133.92\ in^2 = 187.98\ in^2$$

Check Shear:

$$V_{max} = 367.6\ k$$

$$\phi V_n = \phi\ 2\lambda \sqrt{f'_c}\ b_w\ d \quad (h - 1")$$

$$= (0.75)(2)(1)\sqrt{4500_{psi}}\left(106' \times \frac{12"}{1'}\right)(8" - 1")$$

$$= 895948\ lb$$

$$\phi V_n = 895.9\ k$$

Check: $\phi V_n > V_{max}$ ✓ OK

$895.9\ k > 367.6\ k$ ✓ OK

Check limits & minimums

Longitudinally:

$$\rho = \frac{A_{s_{long}}}{b \cdot h} = \frac{54.06\ in^2}{\left(106' \times \frac{12''}{1'}\right)(8'')} = 0.0053$$

$\rho_{min} = 0.0018$

$$A_{s,min} = 0.0018\left(106' \times \frac{12''}{1'}\right)(8'') = 18.32\ in^2$$

$A_s > A_{s,min}$ ✓ Ok

$54.06\ in^2 > 18.32\ in^2$ ✓ Ok

Transversely:

$$\rho = \frac{A_{s_{trans}}}{b \cdot h} = \frac{133.92\ in^2}{\left(306' \times \frac{12''}{1'}\right)(8'')} = 0.0046$$

$\rho_{min} = 0.0018$

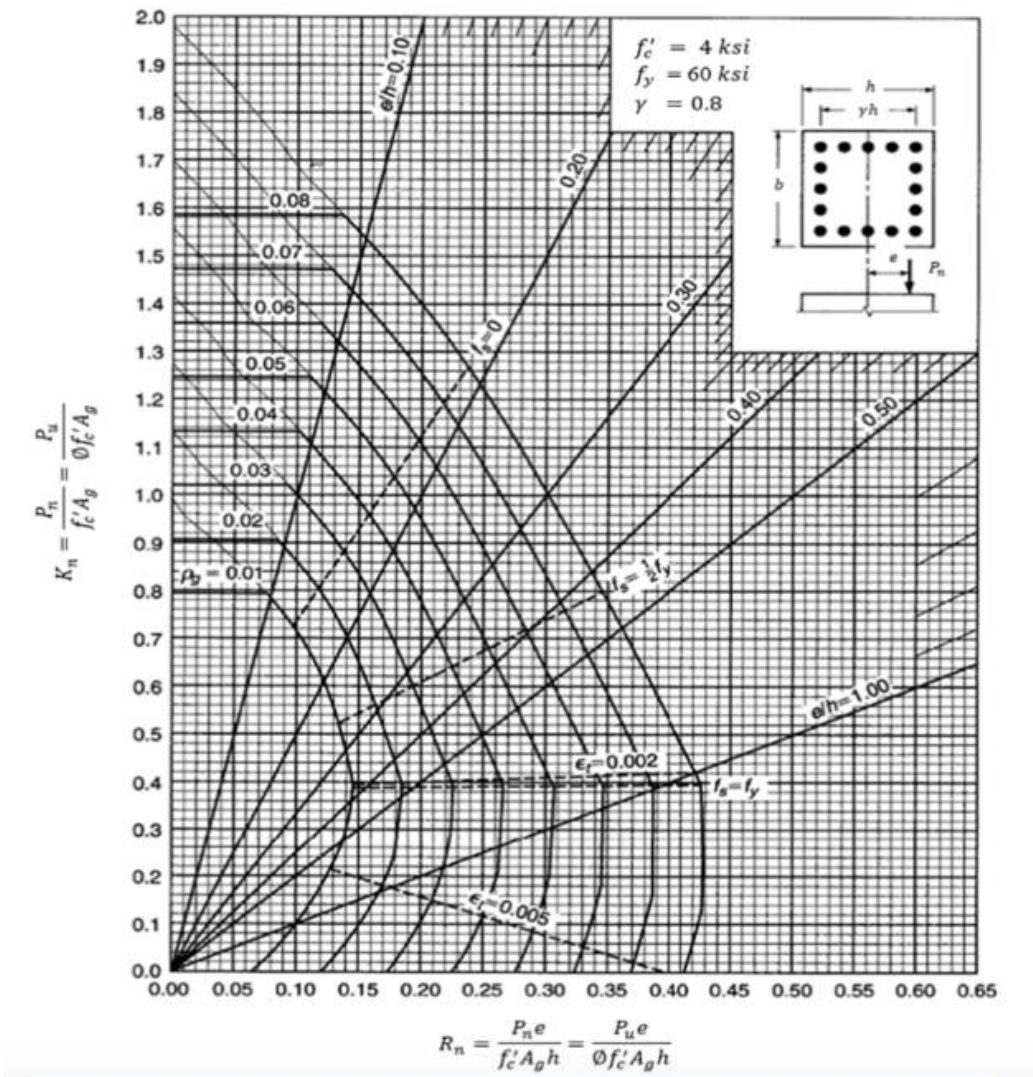
$$A_{s_{min}} = 0.0018\left(306' \times \frac{12''}{1'}\right)(8'') = 52.88\ in^2$$

$A_s > A_{s_{min}}$ ✓ Ok

$133.92\ in^2 > 52.88\ in^2$ ✓ Ok

Column Design:

Interaction Diagram -



Rebar in columns:

$$e = \frac{M_u}{P_u} = \frac{6048 \text{ kft}}{2902 \text{ k}} = \frac{72576 \text{ k in}}{2902 \text{ k}} = 25.01 \text{ in}$$

$$A_g = 2' \times 24' = 24 \text{ in} \times 288 \text{ in} = 6912 \text{ in}^2$$

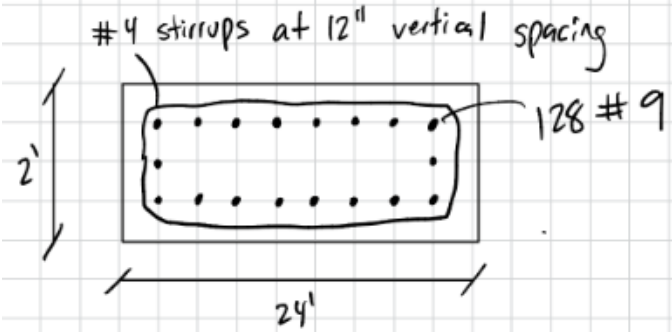
$$k_u = \frac{P_u}{\phi f'_c A_g} = \frac{2902 \text{ k}}{(0.65)(4.5 \text{ ksi})(6912 \text{ in}^2)} = 0.144$$

$$R_u = \frac{M_u}{\phi f'_c A_g h} = \frac{72576 \text{ k-in}}{(0.65)(4.5 \text{ ksi})(6912 \text{ in}^2)(24 \text{ in})} = 0.149$$

From Graph A.7, $\rho_g = .018$

$$A_s = \rho_g \cdot A_g = (.018)(6912 \text{ in}^2) = 124.4 \text{ in}^2$$

Use 128 #9 bars $\Rightarrow A_s = 128 \text{ in}^2$



Expansion behavior:

Transversal Expansion:

$$\varepsilon = \alpha \Delta T$$

$$\varepsilon = \frac{\Delta l}{l_0}$$

$$\Delta l = l_0 \cdot \alpha \cdot \Delta T$$

$$\Delta l = \left(107.25\,ft \times \frac{12\,in}{1\,ft}\right)\left(6\times10^{-6}\right)\left(110 - {}^{-}10\right)$$

$\Delta l = 0.93$ inches transversely

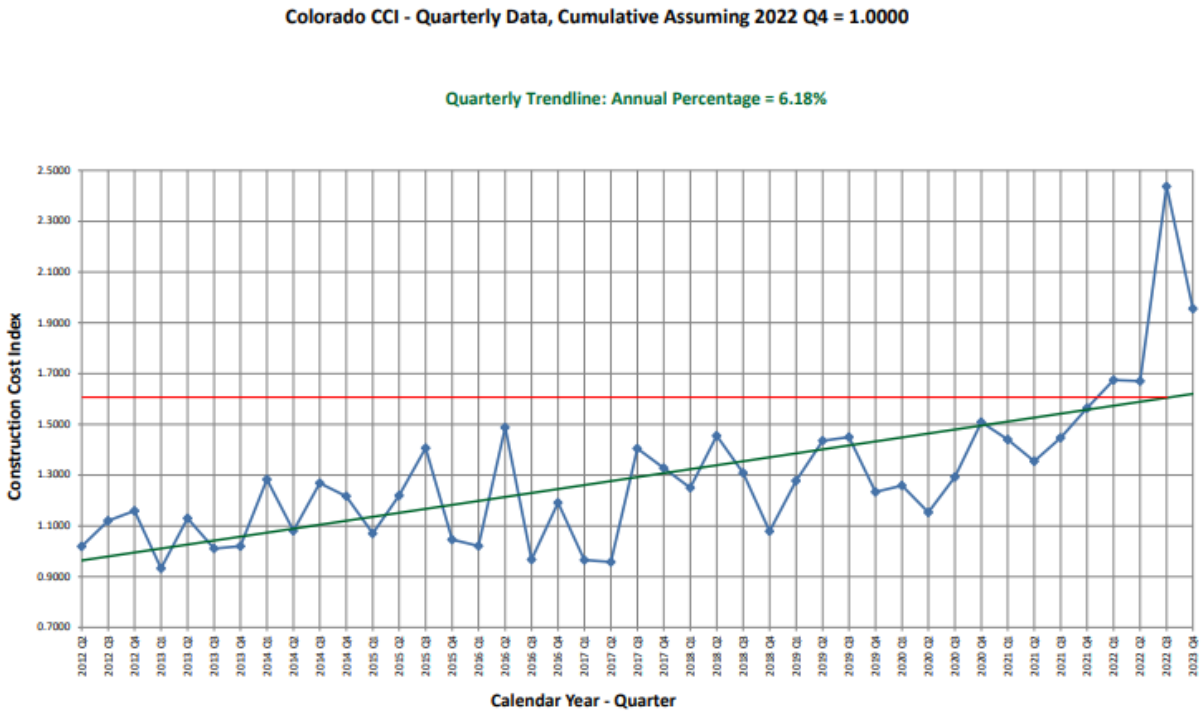
Longitudinal Expansion:

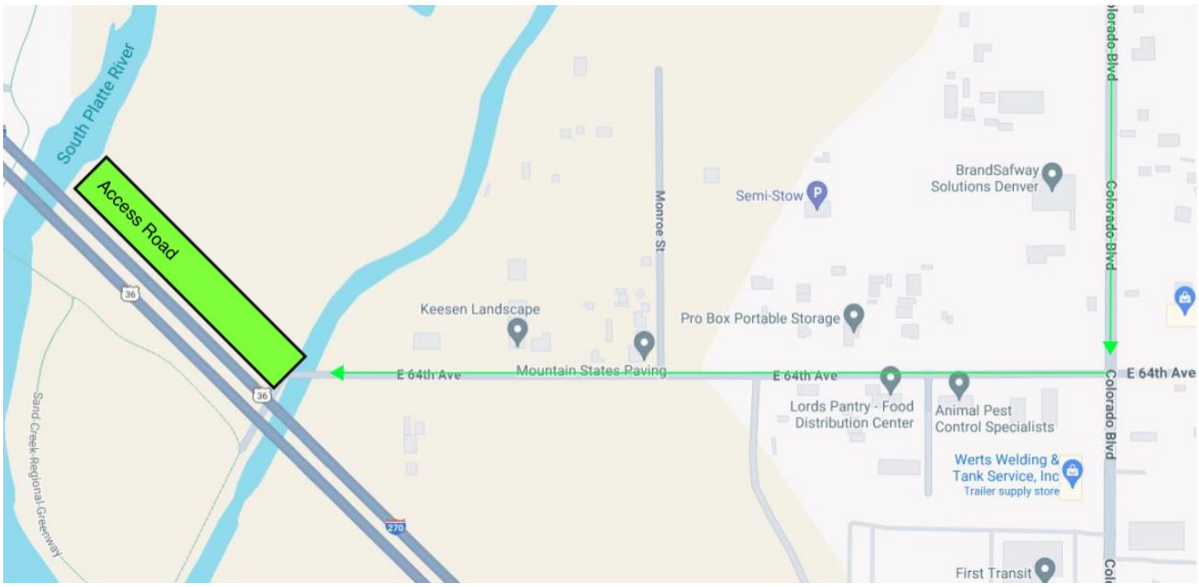
$$\Delta l = l_0 \cdot \alpha \cdot \Delta T$$

$$= \left(306\,ft \times \frac{12\,in}{1\,ft}\right) \times \left(6\times10^{-6}\right)\left(110 - {}^{-}10\right)$$

$\Delta l = 2.64$ inches longitudinally

Appendix D: Construction





Formwork Equations

Lateral concrete pressure when vertical rate of placement is less than 7 ft/h and the placement height is < 14 ft:

$$p = C_w C_c \left(150 + \frac{9000R}{T}\right) \frac{lb}{sq-ft} \qquad \text{Equation 1}$$

Where:
p = lateral pressure (lb/sq ft)
C_w: Unit weight of concrete coefficient
C_c: Chemistry coefficient
R = rate of vertical placement (ft/h)
T = concrete temperature (deg. F)
h = height of form (ft)
w = unit weight of concrete (lb/cu ft)
E = modulus of elasticity (lb/sq in)

Per ACI:
*Regardless, the minimum pressure that can be used is 600 x Cw lb/sq ft and the maximum pressure is w x h.
*When forms are vibrated externally, use 2 x the pressure obtained in equaitons 1 and 2
*When concrete is placed from the bottom up, use Equation 3 and add 25%
*If struts are used to support wall forms on only one side, the allowable stress for strut design will be the lowest of F_c, F_c', F_t

Unit Weight of Concrete	C_w
Under 140 lb/cu ft	$0.5\left(1 + \frac{w}{145}\right)$ but at least 0.80
[Under 2243 kg/m^3	$0.5\left(1 + \frac{w}{2323}\right)$ but at least 0.80]
140–150 lb/cu ft	1.0
[2243–2403 kg/m^3	1.0]
Over 150 lb/cu ft	$\left(\frac{w}{145}\right)$
[Over 2403 kg/m^3	$\left(\frac{w}{2323}\right)$]

Cement Type or Blend	C_c
Type I, II, or III without retarders	1.0
Type I, II, or III with a retarder	1.2
Other blends containing less than 70% slag or 40% fly ash without retarders	1.2
Other blends containing less than 70% slag or 40% fly ash with a retarder	1.4
Blends containing more than 70% slag or 40% fly ash	1.4

Species (No. 2 Grade, 4 x 4 [100 x 100 mm] or smaller)	Allowable Unit Stress (lb/sq in.)[kPa] (Moisture Content = 19%)					
	F_b	F_v	$F_{c\perp}$	F_c	F_t	E
Douglas fir—larch	1450 [9998]	185 [1276]	385 [2655]	1000 [6895]	850 [5861]	1.7×10^6 [11.7×10^6]
Hemlock—fir	1150 [7929]	150 [1034]	245 [1689]	800 [5516]	675 [4654]	1.4×10^6 [9.7×10^6]
Southern pine	1400 [9653]	180 [1241]	405 [2792]	975 [6723]	825 [5688]	1.6×10^6 [11.0×10^6]
California redwood	1400 [9653]	160 [1103]	425 [2930]	1000 [6895]	800 [5516]	1.3×10^6 [9.0×10^6]
Eastern spruce	1050 [7240]	140 [965]	255 [1758]	700 [4827]	625 [4309]	1.2×10^6 [8.3×10^6]
Reduction factor for wet conditions	0.86	0.97	0.67	0.70	0.84	0.97
Load duration factor (7-day load)	1.25	1.25	1.25	1.25	1.25	1.00

Thickness in. (mm)	Approximate Weight psf (kg/m²)	Face Grain Across Supports		
		EI	F_bKS	F_sIb/Q
		$10^6 \frac{lbin.^2}{ft}$	$10^3 \frac{lbin.}{ft}$	$10^3 \frac{lb}{ft}$
		$10^9 \frac{kPamm^4}{m}$	$10^3 \frac{Nmm}{m}$	$10^3 \frac{N}{m}$
Plyform Class I				
1/2 (12.7)	1.5 (7.3)	0.116 (1087)	0.517 (191)	0.371 (5.41)
5/8 (15.9)	1.8 (8.8)	0.195 (1836)	0.691 (256)	0.412 (6.01)
3/4 (19.1)	2.2 (10.7)	0.298 (2810)	0.878 (326)	0.517 (7.55)
7/8 (22.2)	2.6 (12.7)	0.444 (4180)	1.127 (418)	0.616 (8.99)
1 (25.4)	3.0 (14.6)	0.641 (6030)	1.422 (527)	0.675 (9.85)
1 1/8 (28.6)	3.3 (16.1)	0.831 (7824)	1.639 (607)	0.751 (10.96)

Nominal Size ($b \times d$)	Actual Size (S4S)		Area of Section A		Section Modulus S		Moment of Inertia I	
in.	in.	mm	in.²	10^3 mm²	in.³	10^5 mm³	in.⁴	10^6 mm⁴
1 × 3	0.75 × 2.5	19 × 64	1.875	1.210	0.7812	0.1280	0.9766	0.4065
1 × 4	0.75 × 3.5	19 × 89	2.625	1.694	1.531	0.2509	2.680	1.115
1 × 6	0.75 × 5.5	19 × 140	4.125	2.661	3.781	0.6196	10.40	4.328
1 × 8	0.75 × 7.25	19 × 184	5.438	3.508	6.570	1.077	23.82	9.913
1 × 10	0.75 × 9.25	19 × 235	6.938	4.476	10.70	1.753	49.47	20.59
1 × 12	0.75 × 11.25	19 × 286	8.438	5.444	15.82	2.592	88.99	37.04
2 × 3	1.5 × 2.5	38 × 64	3.750	2.419	1.563	0.2561	1.953	0.8129
2 × 4	1.5 × 3.5	38 × 89	5.250	3.387	3.063	0.5019	5.359	2.231
2 × 6	1.5 × 5.5	38 × 140	8.250	5.323	7.563	1.239	20.80	8.656
2 × 8	1.5 × 7.25	38 × 184	10.88	7.016	13.14	2.153	47.63	19.83
2 × 10	1.5 × 9.25	38 × 235	13.88	8.952	21.39	3.505	98.93	41.18
2 × 12	1.5 × 11.25	38 × 286	16.88	10.89	31.64	5.185	178.0	74.08
2 × 14	1.5 × 13.25	38 × 337	19.88	12.82	43.89	7.192	290.8	121.0
3 × 4	2.5 × 3.5	64 × 89	8.750	5.645	5.104	0.8364	8.932	3.718
3 × 6	2.5 × 5.5	64 × 140	13.75	8.871	12.60	2.065	34.66	14.43
3 × 8	2.5 × 7.25	64 × 184	18.12	11.69	21.90	3.589	79.39	33.04

Formwork Calculations

15 min Spacing

$$\frac{Cy}{hr} = \frac{10\ CY}{truck} \cdot \frac{4\ trucks}{hr} = 40\ \frac{CY}{hr}$$

Bridge abutment dimensions:

$$\underset{L}{107'3''} \times \underset{h}{8.5'} \times \underset{w}{5.5'}$$

Vertical Placement

$$40\ \frac{Cy}{hr} \cdot \frac{27\ ft^3}{1\ Cy} \cdot \frac{1}{107.25'} \cdot \frac{1}{5.5'} = \frac{1.8\ ft}{hr}$$

Lateral Concrete Pressure		
Type	Class D	**Unit**
Cw	1	N/A
Cc	1	N/A
R	1.8	ft/h
T	60	Deg. F
P	420	lb/sq ft

Sheathing		
Type	Plyform Class I	**Unit**
d	1	in
w	420	lb/ft
EI	641000	lb* Sq in/ft
FbKS	1422	lb*in/ft
Fslb/Q	675	lb/ft
Bending	20	in
Shear	34	in
Deflection	19	in
Minimum spacing (studs)	16	in

Studs		
Type	2x4 Douglas Fir	**Unit**
d	3.5	in
w	420	lb/ft
E	1700000	psi
I	5.4	in^4
A	5.25	sq in
S	3.063	cubic in
Fb	1400	psi
Fv	180	psi
Bending	35	in
Shear	37	in
Deflection	47	in
Minimum spacing (wales)	20	in

Wales		
Type	2x4 doubled wales Douglas Fir	**Unit**
d	3.5	in
w	840	lb/ft
E	1700000	psi
I	5.359	in^4
A	5.25	sq in
S	3.063	cubic in
Fb	1400	psi
Fv	180	psi
Bending	35	in
Shear	37	in
Deflection	47	in
Minimum spacing (ties)	20	in

Ties		
		Unit
w (wales)	840	lb/ft
s (ties)	1.7	ft
p	1400	lbs

Lateral Bracing		
Type	3x4 Douglas Fir	Unit
Fc	1000	psi
Ft	850	psi
E	1700000	psi
d	2.5	in

8-22 ft height		
		Unit
h	8.5	ft
wf	15	lb/sq ft
H	100	lb/ft
h'	7	ft
l'	6	ft
l	9.2	ft
p'	186.6	lb

Check Allowable Unit Stress (C)		
l/d	34.6	Pass
Allowable Compressive Force Per Braced Strut		
		Unit
fc'	426	Psi
Fc	1000	Psi
		Pass

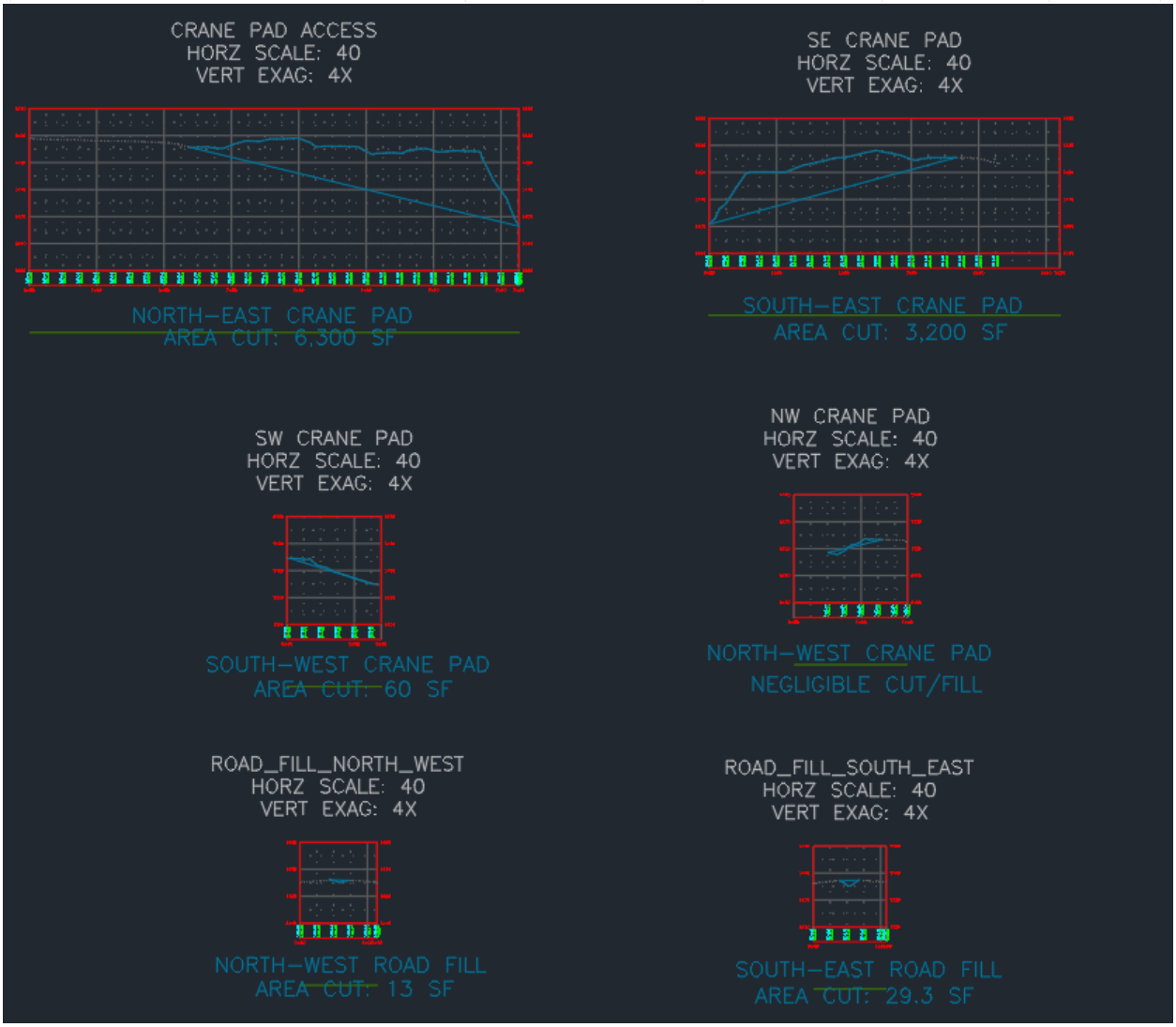
Max Spacing for Strut		
A	8.75	in^2
fc'	425.7	Psi
P	3724.7	lbs
S	12.4	in

Quantities Estimate Calculations

Corrugated Deck Metal		
Length	306	ft
Width	107.25	ft
Area	32819	SF

Asphalt Quantity Estimate		
Deck (4" Thick)	406	CY
Infill Between EB and WB (4" Thick)	74	CY

Earthwork Quantity Estimate				
	Width (ft)	Lateral Face (ft2)	Volume (CY)	
NE Corner	20	6300	4667	Cut
SE Corner	20	3200	2370	Cut
SW Corner	20	60	44	Cut
NW Corner	0	0	0	Negligible
Abutment	23.5	4134	3598	
NW Road Alignment	200	13	96	Fill
SE Road Alignment	200	29.3	217	Fill
Total Earthwork			**10993**	



Erosion Control Quantities		
Silt Fence	1414	LF
Erosion Control Blanket	81500	SF
Riprap/Gravel	567	CY

Gap Material Quantity Estimate		
Asphalt (Over Steel Plates) (4" Thick)	153	SF
Expanded Polystyrene foam (2" thick) 24" by 28' Sheets	3.00	Each
Steel Plates (1/4" Thick) 4"x 12"	310	Each

Guardrail Quantity Estimate		
Removal of Existing Guardrail (Bridge)	1224	LF
Removal of Existing Guardrail (Temp Work)	1600	LF
Total Removal of Guardrail	2824	LF
Concrete Barrier	2824	LF

Retaining Wall Quantity Estimate		
Concrete	59.2	CY
Geosynthetic Mat	3936	SF
Geosynthetic Mat Roll (12'x150') (Single)	1800	SF
Geosynthetic Mat Roll (12'x150')	3	EACH

Pier Concrete Quantity Estimate		
Piers		
Amount of Piers	6	EACH
Length	24	ft
Width	2	ft
Height (1)	22.08	ft
Height (2)	25.92	ft
Volume (1)	117.76	CY
Volume (2)	138.24	CY
Pier Cap		
Length	35	ft
Width	2	ft
Height	4	ft
Volume	62.2	CY
Pier Base		
Area	121.5	SF
Height	6	ft
Volume	162	CY
Total Volume	**480.2**	**CY**

Pier Reinforcing Steel Quantity Estimate		
128 #9	3.4	lb/ft
Amount of Piers	6	EACH
Height (1)	22.08	ft
Height (2)	25.92	ft
Total Length of Rebar	18432	ft
Total Tons (Longitudinal)	31.3	Tons
#4 Stirrups @12"	0.668	lb/ft
Length of stirrup	52.0	ft
Amount of stirrups	144	EACH
Total Tons (Longitudinal and Transversal)	**33.8**	**Tons**

Girder Concrete Quantity Estimate		
Cross Sectional Area	5.47	SF
Amount of Girders Per Span	12	EACH
Total Length	3665.64	LF
Span Length (1)	94.15	ft
Volume (Span 1)	229.0	CY
Span Length (2)	117.17	ft
Volume (Span 2)	285.0	CY
Span Length (3)	94.15	ft
Volume (Span 3)	229.0	CY
Total Volume	**742.9**	**CY**

Girder Reinforcing Steel Quantity Estimate		
85 #5 Bars	1.043	lb/ft
Span 1 (Length)	94.15	ft
Span 2 (Length)	117.17	ft
Span 3 (Length)	94.15	ft
Amount of Girders Per Span	12	EACH
Total Length	311579.4	ft
Total Tons (Longitudinal)	162.5	Tons
#3 Stirrups @12"	0.376	lb/ft
Length of Stirrup	8.83	ft
Total Amount of Stirrups	3666	EACH
Total Tons (Longitudinal and Transversal)	**168.6**	**Tons**

Deck Quantity Estimate		
Concrete		
Length	306	ft
Width	107.25	ft
Depth	0.67	ft
Volume	**809.1**	**CY**
Reinforcing Steel		
Top Bar (#4 @12")	0.668	lb/ft
Total Length (#4)	65270	ft
Bottom Bar (#5 @12")	1.043	lb/ft
Total Length (#5)	65270	ft
Transversal Reinforcment (#5 @ 8.5")		
Total Length	46245.8	ft
Total Reinforcing Steel	**80**	**Tons**

Caisson Quantity Estimate		
Concrete		
Diameter	3.50	ft
NW Piers Length	42.00	ft
SE Piers Length	41.00	ft
Total Amount Per Pier	3.00	EACH
Area	9.62	ft^2
Total LF of Caissons	747	LF
Volume (NW)	134.70	CY
Volume (SE)	131.49	CY
Reinforcing Steel		
#11 Rebar- Longitudinal		
Longitudinal Reinforcement Area	10.31	in^2
Amount of Longitudinal Reinforcement	8.00	EACH
Longitudinal Reinforcement Length (Total)	5976.00	LF
Longitudinal Reinforcement Weight	5.31	lb/ft
Total Longitudinal Reinforcement	15.88	tons
#3 Rebar (Hoops)- Transverse Reinforcement	0.38	lb/ft
Diameter	3.00	ft
Length per Hoop	9.42	ft
Amount of Hoops (NW)	57.00	Per Caisson
Amount of Hoops (SE)	56.00	Per Caisson
Total Transverse Reinforcement Length	9585.00	ft
Total Transverse Reinforcement	1.80	tons
Total Caisson Reinforcing Steel	**17.7**	**tons**
Total Caisson Concrete	**266.2**	**CY**

Abutment Concrete Quantity Estimate		
Concrete		
Width	8.5	ft
Length	107.25	ft
Depth	5.5	ft
Total Volume	**371.4**	**CY**

Pier Cap (Caisson) Quantity Estimate		
Concrete		
Area Pier Cap	184	ft^2
Depth	3.00	ft
Amount of Pier Caps	6.00	EACH
Total Volume	122.83	CY
Reinforcing Steel		
Top Bar- #7's @ 9"	3.4	lb/ft
Top Bar Weight	0.42	Tons
Bottom Bar- #7's @9"	3.4	lb/ft
Bottom Bar Weight	0.42	Tons
Total Pier Cap (Caisson) Reinforcing Steel	**0.835**	**Tons**

Construction Packages- Cost Estimate

General Conditions

Description	Quantity	Unit	(0) -EST- Engineer's Estimate		(1) 099H FLATIRON CONSTRUCTORS, INC.		Inflation (Engineer's Estimate)			Inflation (Flatiron Constructors)		
			Unit Price	Ext Amount	Unit Price	Ext Amount	2022-2023 (1.6%)	2023-2024 (3%)	2024-2025 (3%)	2022-2023 (1.6%)	2023-2024 (3%)	2024-2025 (3%)
Sweeping (Sediment Removal)	20	HOUR	150	3,000.00	100	2,000.00	3048.00	3139.44	3233.62	2032.00	2092.96	2155.75
Erosion Control Management	546	DAY	250.00	136,500.00	10.00	5,460.00	138684.00	142844.52	147129.86	5547.36	5713.78	5885.19
Monitoring Technician	30	HOUR	140.00	4,200.00	115.00	3,450.00	4267.20	4395.22	4527.07	3505.20	3610.36	3718.67
Health and Safety Officer	20	HOUR	160	3,200.00	125	2,500.00	3251.20	3348.74	3449.20	2540.00	2616.20	2694.69
Field Office (Special)	1	EACH	250000	250,000.00	433500	433,500.00	254000.00	261620.00	269468.60	440436.00	453649.08	467258.55
Field Laboratory (Class 2)	1	EACH	120,000.00	120,000.00	80,000.00	80,000.00	121920.00	125577.60	129344.93	81280.00	83718.40	86229.95
Sanitary Facility	2	EACH	5000	10,000.00	7000	14,000.00	10160.00	10464.80	10778.74	14224.00	14650.72	15090.24
Mobilization	1	LS	1,000,000.00	1,000,000.00	1,150,000.00	1,150,000.00	1016000.00	1046480.00	1077874.40	1168400.00	1203452.00	1239555.56
Public Information Management (Tier III)	500	DAY	120	60,000.00	125	62,500.00	60960.00	62788.80	64672.46	63500.00	65405.00	67367.15
Flagging	2000	HOUR	30	60,000.00	39	78,000.00	60960.00	62788.80	64672.46	79248.00	81625.44	84074.20
Traffic Control Inspection	206	DAY	400.00	82,400.00	170.00	35,020.00	83718.40	86229.95	88816.85	35580.32	36647.73	37747.16
Traffic Control Management	516	DAY	1,200.00	619,200.00	1,600.00	825,600.00	629107.20	647980.42	667419.83	838809.60	863973.89	889893.10
Uniformed Traffic Control Coordination	150	HOUR	35	5,250.00	11	1,650.00	5334.00	5494.02	5658.84	1676.40	1726.69	1778.49
Courtesy Patrol	500	DAY	870.00	435,000.00	1,490.00	745,000.00	441960.00	455218.80	468875.36	756920.00	779627.60	803016.43
Traffic Incident Management Plan Development	1	LS	4,000.00	4,000.00	3,300.00	3,300.00	4064.00	4185.92	4311.50	3352.80	3453.38	3556.99
Traffic Incident Management Plan Implementation	24	MON	800	19,200.00	450	10,800.00	19507.20	20092.42	20695.19	10972.80	11301.98	11641.04
Division Total									$ 3,030,928.92			$ 3,721,663.17
											Average Cost	$ 3,376,296.04

Existing Conditions

Description	Quantity	Unit	(0) -EST- Engineer's Estimate		(1) 099H FLATIRON CONSTRUCTORS, INC.		Inflation (Engineer's Estimate)			Inflation (Flatiron Constructors)		
			Unit Price	Ext Amount	Unit Price	Ext Amount	2022-2023 (1.6%)	2023-2024 (3%)	2024-2025 (3%)	2022-2023 (1.6%)	2023-2024 (3%)	2024-2025 (3%)
Clearing and Grubbing	1	ACRE	8,000.00	8,000.00	20,000.00	20,000.00	8128.00	8371.84	8623.00	20320.00	20929.60	21557.49
Removal of Structure	1	EACH	75,000.00	75,000.00	2,500.00	2,500.00	76200.00	78486.00	80840.58	2540.00	2616.20	2694.69
Removal of Inlet	3	EACH	1,200.00	3,600.00	1,800.00	5,400.00	3657.60	3767.33	3880.35	5486.40	5650.99	5820.52
Removal of Embankment Protector Type 3	5	EACH	1,000.00	5,000.00	850	4,250.00	5080.00	5232.40	5389.37	4318.00	4447.54	4580.97
Removal of Embankment Protector Type 5	2	EACH	500	1,000.00	1,050.00	2,100.00	1016.00	1046.48	1077.87	2133.60	2197.61	2263.54
Removal of Slope and Ditch Paving	1,252.00	SY	25	31,300.00	27	33,804.00	31800.80	32754.82	33737.47	34344.86	35375.21	36436.47
Removal of Riprap	209	SY	40	8,360.00	15	3,135.00	8493.76	8748.57	9011.03	3185.16	3280.71	3379.14
Removal of Pipe	782	LF	29	22,678.00	28	21,896.00	23040.85	23732.07	24444.04	22246.34	22913.73	23601.14
Removal of End Section	2	EACH	500	1,000.00	600	1,200.00	1016.00	1046.48	1077.87	1219.20	1255.78	1293.45
Removal of Electrical Conduit	612	LF	16	9,792.00	12	7,344.00	9948.67	10247.13	10554.55	7461.50	7685.35	7915.91
Removal of Delineator	34	EACH	10	340.00	20	680.00	345.44	355.80	366.48	690.88	711.61	732.95
Removal of Concrete Median Cover Material	200	SY	15	3,000.00	14	2,800.00	3048.00	3139.44	3233.62	2844.80	2930.14	3018.05
Removal of Concrete Pavement	38581	SY	21.00	810,201.00	7.00	270,067.00	823164.22	847859.14	873294.92	274388.07	282619.71	291098.31
Removal of Asphalt Mat	32400	SY	11.00	356,400.00	4.5	145,800.00	362102.40	372965.47	384154.44	148132.80	152576.78	157154.09
Removal of Pavement Marking	30,000.00	SF	1	30,000.00	1.5	45,000.00	30480.00	31394.40	32336.23	45720.00	47091.60	48504.35
Removal of Bridge	2	EACH	120000	240,000.00	100000	200,000.00	243840.00	251155.20	258689.86	203200.00	209296.00	215574.88
Removal of Bridge Railing	2824	LF	40	112,960.00	125	353,000.00	114767.36	118210.38	121756.69	358648.00	369407.44	380489.66
Removal of Portions of Present Structure	340	SY	800	272,000.00	230	78,200.00	276352.00	284642.56	293181.84	79451.20	81834.74	84289.78
Removal of Ground Sign	32	EACH	120	3,840.00	100	3,200.00	3901.44	4018.48	4139.04	3251.20	3348.74	3449.20
Removal of Sign Panel	4	EACH	80	320.00	600	2,400.00	325.12	334.87	344.92	2438.40	2511.55	2586.90
Removal of Pull Box	1	EACH	465	465.00	100	100.00	472.44	486.61	501.21	101.60	104.65	107.79
Removal of Impact Attenuator	4	EACH	1800	7,200.00	1800	7,200.00	7315.20	7534.66	7760.70	7315.20	7534.66	7760.70
Removal of Guardrail Type 3	2824	LF	6	16,944.00	4	11,296.00	17215.10	17731.56	18263.50	11476.74	11821.04	12175.67
Removal of Guardrail Type 7	6766	LF	18.00	121,788.00	13.00	87,958.00	123736.61	127448.71	131272.17	89365.33	92046.29	94807.68
Removal of End Anchorage	9	EACH	340	3,060.00	300	2,700.00	3108.96	3202.23	3298.30	2743.20	2825.50	2910.26
Removal and Disposal of Sediment (Labor)	40	HOUR	60	2,400.00	60	2,400.00	2438.40	2511.55	2586.90	2438.40	2511.55	2586.90
Removal and Disposal of Sediment (Equipment)	40	HOUR	140.00	5,600.00	100.00	4,000.00	5689.60	5860.29	6036.10	4064.00	4185.92	4311.50
Removal of Nests	90	HOUR	120	10,800.00	100	9,000.00	10972.80	11301.98	11641.04	9144.00	9418.32	9700.87
Construction Surveying	1	LS	192000	192,000.00	188000	188,000.00	195072.00	200924.16	206951.88	191008.00	196738.24	202640.39
Division Total									$ 2,538,445.95			$ 1,633,443.20
											Average Cost	$ 2,085,944.58

ALPINE ENGINEERING

Substructure

Description	Quantity	Unit	(0) -EST- Engineer's Estimate		(1) 099H FLATIRON CONSTRUCTORS, INC.		Inflation (Engineer's Estimate)			Inflation (Flatiron Constructors)		
			Unit Price	Ext Amount	Unit Price	Ext Amount	2022-2023 (1.6%)	2023-2024 (3%)	2024-2025 (3%)	2022-2023 (1.6%)	2023-2024 (3%)	2024-2025 (3%)
Concrete Class D	1340	CY	1,200.00	1,608,000.00	1,200.00	1,608,000.00	1633728	1682739.84	1733222.035	1633728	1682739.84	1733222.035
Structural Concrete Coating	1200	SF	12	14,400.00	1.10	1,320.00	14630.4	15069.312	15521.39136	1341.12	1381.3536	1422.794208
Reinforcing Steel (Epoxy Coated)	510000	LB	1.6	816,000.00	1.35	688,500.00	829056	853927.68	879545.5104	699516	720501.48	742116.5244
Drilled Shaft (42 Inch)	747	LF	600.00	448,200.00	550.00	410,850.00	455371.2	469032.336	483103.3061	417423.6	429946.308	442844.6972
Crosshole Sonic Logging Testing	6	EACH	2,800.00	16,800.00	2,500.00	15,000.00	17068.80	17580.86	18108.29	15240.00	15697.20	16168.12
Concrete Sidewalk (6 Inch)	124	SY	70.00	8,680.00	70.00	8,680.00	8818.88	9083.4464	9355.949792	8818.88	9083.4464	9355.949792
Division Total									$ 3,138,856.48			$ 2,945,130.12
											Average Cost	$ 3,041,993.30

Superstructure

Description	Quantity	Unit	(0) -EST- Engineer's Estimate		(1) 099H FLATIRON CONSTRUCTORS, INC.		Inflation (Engineer's Estimate)			Inflation (Flatiron Constructors)		
			Unit Price	Ext Amount	Unit Price	Ext Amount	2022-2023 (1.6%)	2023-2024 (3%)	2024-2025 (3%)	2022-2023 (1.6%)	2023-2024 (3%)	2024-2025 (3%)
Concrete Class D	1599	CY	1,200.00	1,918,800.00	1,200.00	1,918,800.00	1949500.8	2007985.8	2068225.399	1949500.8	2007985.824	2068225.399
Structural Concrete Coating (Deck)	1200	SY	19	22,800.00	10	12,000.00	23164.8	23859.744	24575.53632	12192	12557.76	12934.4928
Reinforcing Steel (Epoxy Coated)	522000	LB	1.6	835,200.00	1.35	704,700.00	848563.2	874020.1	900240.6989	715975.2	737454.456	759578.0897
18 Inch Reinforced Concrete End Section	2	EACH	1,400.00	2,800.00	1,900.00	3,800.00	2844.8	2930.144	3018.04832	3860.8	3976.624	4095.92272
24 Inch Reinforced Concrete End Section	2	EACH	1,600.00	3,200.00	2100	4,200.00	3251.2	3348.736	3449.19808	4267.2	4395.216	4527.07248
23x14 Inch Reinforced Concrete End Section Elliptical	1	EACH	4,000.00	4,000.00	2,250.00	2,250.00	4064	4185.92	4311.4976	2286	2354.58	2425.2174
Transition Type 3G	11	EACH	3,200.00	35,200.00	4,500.00	49,500.00	35763.20	36836.10	37941.18	50292.00	51800.76	53354.78
Transition Type GR9-GR3	1	EACH	5000	5,000.00	5900	5,900.00	5080.00	5232.40	5389.37	5994.40	6174.23	6359.46
Transition Type 3H	9	EACH	1,800.00	16,200.00	1,600.00	14,400.00	16459.20	16952.98	17461.57	14630.40	15069.31	15521.39
Transition Type BR9-GR3	4	EACH	2400	9,600.00	6000	24,000.00	9753.60	10046.21	10347.59	24384.00	25115.52	25868.99
Division Total									$ 3,003,820.38			$ 2,851,786.19
											Average Cost	$ 2,927,803.29

Finishes/Specialties

Description	Quantity	Unit	(0) -EST- Engineer's Estimate		(1) 099H FLATIRON CONSTRUCTORS, INC.		Inflation (Engineer's Estimate)			Inflation (Flatiron Constructors)		
			Unit Price	Ext Amount	Unit Price	Ext Amount	2022-2023 (1.6%)	2023-2024 (3%)	2024-2025 (3%)	2022-2023 (1.6%)	2023-2024 (3%)	2024-2025 (3%)
Waterproofing (Membrane)	3126	SY	22.00	68,772.00	19.00	59,394.00	69872.35	71968.52	74127.58	60344.30	62154.63	64019.27
Expanded Polystyrene Foam Sheets (2")	3	EACH	120.00	-	-	-	-	360.00	370.80	-	360.00	370.80
Bridge Expansion Device (0-4 Inch)	429	LF	380	163,020.00	350	150,150.00	165628.32	170597.17	175715.08	152552.40	157128.97	161842.84
Sign Panel (Class I)	51	SF	34	1,734.00	31	1,581.00	1761.74	1814.60	1869.03	1606.30	1654.48	1704.12
Sign Panel (Class II)	248	SF	36	8,928.00	33	8,184.00	9070.85	9342.97	9623.26	8314.94	8564.39	8821.32
Sign Panel (Class III)	563	SF	38	21,394.00	34	19,142.00	21736.30	22388.39	23060.04	19448.27	20031.72	20632.67
Optical Attenuator	4	EACH	500	2,000.00	125	500.00	2032.00	2092.96	2155.75	508.00	523.24	538.94
Communications Cabinet	1	EACH	16,000.00	16,000.00	16,000.00	16,000.00	16256.00	16743.68	17245.99	16256.00	16743.68	17245.99
Traffic Signal Vehicle Detector (Micro Wave Type)	2	EACH	12000	24,000.00	17300	34,600.00	24384.00	25115.52	25868.99	35153.60	36208.21	37294.45
Microwave Vehicle Radar Detector	1	EACH	8,000.00	8,000.00	12,800.00	12,800.00	8128.00	8371.84	8623.00	13004.80	13394.94	13796.79
Impact Attenuator	5	EACH	22,000.00	110,000.00	39,000.00	195,000.00	111760.00	115112.80	118566.18	198120.00	204063.60	210185.51
Construction Traffic Sign (Panel Size A)	281	EACH	50	14,050.00	110	30,910.00	14274.80	14703.04	15144.14	31404.56	32346.70	33317.10
Construction Traffic Sign (Panel Size B)	145	EACH	70	10,150.00	110	15,950.00	10312.40	10621.77	10940.43	16205.20	16691.36	17192.10
Construction Traffic Sign (Panel Size C)	72	EACH	90	6,480.00	110	7,920.00	6583.68	6781.19	6984.63	8046.72	8288.12	8536.77
Construction Traffic Sign (Special)	520	SF	30	15,600.00	28	14,560.00	15849.60	16325.09	16814.84	14792.96	15236.75	15693.85
Steel Signpost (W 12x26)	46	LF	160	7,360.00	90	4,140.00	7477.76	7702.09	7933.16	4206.24	4332.43	4462.40
Steel Sign Support (2-Inch Round)(Post & Socket)	20	LF	30	600.00	40	800.00	609.60	627.89	646.72	812.80	837.18	862.30
Steel Signpost (4 Inch Round) (Slipbase)	36	LF	60	2,160.00	83	2,988.00	2194.56	2260.40	2328.21	3035.81	3126.88	3220.69
Steel Sign Support (2-1/2 In Round Sch 80) (Post & Slipbase)	25	EACH	600	15,000.00	400	10,000.00	15240.00	15697.20	16168.12	10160.00	10464.80	10778.74
Steel Plates (1/4" Thick) 4"x 12"	310	EACH	15	-	-	-	-	4650	4789.50	-	4650	4789.50
Hot Mix Asphalt (Patching) (Asphalt)	670	TON	220	147,400.00	250.00	167,500.00	149758.40	154251.15	158878.69	170180.00	175285.40	180543.96
Stone Matrix Asphalt (Fibers)(Asphalt)	568	TON	140	79,520.00	185	105,080.00	80792.32	83216.09	85712.57	106761.28	109964.12	113263.04
Hot Mix Asphalt (Grading S) (100) (PG 6422)	104	TON	110	11,440.00	100.00	10,400.00	11623.04	11971.73	12330.88	10566.40	10883.39	11209.89
Barrier (Temporary)	30800	LF	43.00	1,324,400.00	25.00	770,000.00	1345590.40	1385958.11	1427536.86	782320.00	805789.60	829963.29
Bridge Rail Type 9 (Concrete Rail)	435	LF	420	182,700.00	370	160,950.00	185623.2	191191.896	196927.6529	163525.2	168430.956	173483.8847
Modified Epoxy Pavement Marking	500	GAL	70.00	35,000.00	150.00	75,000.00	35560.00	36626.80	37725.60	76200.00	78486.00	80840.58
Pavement Marking Paint (High Build)	2800	GAL	125	350,000.00	80	224,000.00	355600.00	366268.00	377256.04	227584.00	234411.52	241443.87
Preformed Plastic Pavement Marking (Type I) (Inlaid)	6008	SF	12.00	72,096.00	19.00	114,152.00	73249.54	75447.02	77710.43	115978.43	119457.78	123041.52
Preformed Plastic Pavement Marking (Contrast)(Inlaid)	3004	SF	12.00	36,048.00	18.50	55,574.00	36624.77	37723.51	38855.22	56463.18	58157.08	59901.79
Preformed Plastic Pavement Marking (Type II)(Inlaid)	352	SF	22	7,744.00	25	8,800.00	7867.90	8103.94	8347.06	8940.80	9209.02	9485.29
Preformed Thermoplastic Pavement Marking (Xwalk-Stop Line)	68	SF	44.00	2,992.00	18.00	1,224.00	3039.87	3131.07	3225.00	1243.58	1280.89	1319.32
Division Total									$ 2,963,481.44			$ 2,459,802.59
											Average Cost	$ 2,711,642.02

ALPINE ENGINEERING

Earthwork

Description	Quantity	Unit	(0) -EST- Engineer's Estimate		(1) 099H FLATIRON CONSTRUCTORS, INC.		Inflation (Engineer's Estimate)			Inflation (Flatiron Constructors)		
			Unit Price	Ext Amount	Unit Price	Ext Amount	2022-2023 (1.6%)	2023-2024 (3%)	2024-2025 (3%)	2022-2023 (1.6%)	2023-2024 (3%)	2024-2025 (3%)
Unsuitable Material	10993	CY	100	1,099,300.00	22	241,846.00	1116888.80	1150395.46	1184907.33	245715.54	253087.00	260679.61
Mechanical Reinforcement of Soil	10993	CY	22.00	241,846.00	20.00	219,860.00	245715.54	253087.00	260679.61	223377.76	230079.09	236981.47
Filter Material (Class C)	10993	CY	125	1,374,125.00	75	824,475.00	1396111.00	1437994.33	1481134.16	837666.60	862796.60	888680.50
Unclassified Excavation (Complete In Place)	10993	CY	30.00	329,790.00	18.00	197,874.00	335066.64	345118.64	355472.20	201039.98	207071.18	213283.32
Embankment Material (Complete In Place) (Special)	10993	CY	24.00	263,832.00	19.75	217,111.75	268053.31	276094.91	284377.76	220585.54	227203.10	234019.20
Topsoil (Onsite)	10993	CY	15.00	164,895.00	11.00	120,923.00	167533.32	172559.32	177736.10	122857.77	126543.50	130339.81
Topsoil (Offsite)	10993	CY	50	549,650.00	55	604,615.00	558444.40	575197.73	592453.66	614288.84	632717.51	651699.03
Relay Riprap	567	CY	48.00	27,216.00	35.00	19,845.00	27651.46	28481.00	29335.43	20162.52	20767.40	21390.42
Dewatering	1	LS	10,000.00	10,000.00	1,000.00	1,000.00	10160.00	10464.80	10778.74	1016.00	1046.48	1077.87
Biotic Soil Amendments (Hydraulically Applied)	650	LB	1	650.00	1.5	975.00	660.40	680.21	700.62	990.60	1020.32	1050.93
Humate	1300	LB	3.6	4,680.00	0.75	975.00	4754.88	4897.53	5044.45	990.60	1020.32	1050.93
Aggregate Base Course (Class 6)	16146	CY	60	968,760.00	41.00	661,986.00	984260.16	1013787.96	1044201.60	672577.78	692755.11	713537.76
Geotextile (Drainage) (Class 2)	22	SY	20	440.00	44.00	968.00	447.04	460.45	474.26	983.49	1012.99	1043.38
Pile Tip	37	EACH	200.00	7,400.00	300	11,100.00	7518.40	7743.95	7976.27	11277.60	11615.93	11964.41
Proof Rolling	750	HOUR	140	105,000.00	110	82,500.00	106680.00	109880.40	113176.81	83820.00	86334.60	88924.64
Blading	60	HOUR	150.00	9,000.00	130.00	7,800.00	9144.00	9418.32	9700.87	7924.80	8162.54	8407.42
Dozing	60	HOUR	160	9,600.00	135	8,100.00	9753.60	10046.21	10347.59	8229.60	8476.49	8730.78
Combination Loader	60	HOUR	150.00	9,000.00	100.00	6,000.00	9144.00	9418.32	9700.87	6096.00	6278.88	6467.25
Potholing	750	HOUR	280	210,000.00	185	138,750.00	213360.00	219760.80	226353.62	140970.00	145199.10	149555.07
Structure Excavation	10993	CY	22.00	241,846.00	27.00	296,811.00	245715.54	253087.00	260679.61	301559.98	310606.78	319924.98
Structure Backfill (Class 1)	10993	CY	44.00	483,692.00	43.00	472,699.00	491431.07	506174.00	521359.22	480262.18	494670.05	509510.15
Structure Backfill (Class 2)	10993	CY	40	439,720.00	43	472,699.00	446755.52	460158.19	473962.93	480262.18	494670.05	509510.15
Soil Riprap (9 Inch)	567	CY	160	90,720.00	160	90,720.00	92171.52	94936.67	97784.77	92171.52	94936.67	97784.77
Division Total									$ 7,158,338.51			$5,065,613.83
											Average Cost	$ 6,111,976.17

Exterior Improvements

Description	Quantity	Unit	(0) -EST- Engineer's Estimate		(1) 099H FLATIRON CONSTRUCTORS, INC.		Inflation (Engineer's Estimate)			Inflation (Flatiron Constructors)		
			Unit Price	Ext Amount	Unit Price	Ext Amount	2022-2023 (1.6%)	2023-2024 (3%)	2024-2025 (3%)	2022-2023 (1.6%)	2023-2024 (3%)	2024-2025 (3%)
Subgrade Soil Preparation	25091	SY	2.00	50,182.00	2.50	62,727.50	50984.91	52514.46	54089.89	63731.14	65643.07	67612.37
Geosynthetic Mat Roll	3	EACH	1,200.00	-	-	-	-	3600.00	3708.00	-	3600.00	3708.00
Aggregate Bag	90	LF	9	810.00	7.25	652.50	822.96	847.65	873.08	662.94	682.83	703.31
Organic Fertilizer	3900	LB	3	11,700.00	0.35	1,365.00	11887.20	12243.82	12611.13	1386.84	1428.45	1471.30
Compost (Mechanically Applied)	211	CY	50.00	10,550.00	39.00	8,229.00	10718.80	11040.36	11371.57	8360.66	8611.48	8869.83
Seeding (Native) Drill	3.25	ACRE	2,000.00	6,500.00	500	1,625.00	6604.00	6802.12	7006.18	1651.00	1700.53	1751.55
Seeding (Native) Hydraulic	3.25	ACRE	2,000.00	6,500.00	1,000.00	3,250.00	6604.00	6802.12	7006.18	3302.00	3401.06	3503.09
Seeding (Wetland) Drill	0.05	ACRE	5,000.00	250.00	15,500.00	775.00	254.00	261.62	269.47	787.40	811.02	835.35
Mulching (Weed Free Hay)	1.91	ACRE	4,000.00	7,640.00	1,100.00	2,101.00	7762.24	7995.11	8234.96	2134.62	2198.65	2264.61
Spray-on Mulch Blanket	0.29	ACRE	4,000.00	1,160.00	5,500.00	1,595.00	1178.56	1213.92	1250.33	1620.52	1669.14	1719.21
Mulch Tackifier	230	LB	5	1,150.00	3.5	805.00	1168.40	1203.45	1239.56	817.88	842.42	867.69
Landscape Boulder	180	EACH	300	54,000.00	195	35,100.00	54864.00	56509.92	58205.22	35661.60	36731.45	37833.39
Pre-fabricated Vehicle Tracking Pad	4	EACH	4,600.00	18,400.00	2,700.00	10,800.00	18694.40	19255.23	19832.89	10972.80	11301.98	11641.04
Water Control	1	LS	30000	30,000.00	1000	1,000.00	30480.00	31394.40	32336.23	1016.00	1046.48	1077.87
Erosion Log Type 1 (12 inch)	420	LF	6	2,520.00	5	2,100.00	2560.32	2637.13	2716.24	2133.60	2197.61	2263.54
Silt Fence	1414	LF	2.25	3,181.50	1.25	1,767.50	3232.40	3329.38	3429.26	1795.78	1849.65	1905.14
Soil Retention Blanket (Straw-Coconut) (Photodegradable Class 1)	81500	SY	3.2	260,800.00	2.5	203,750.00	264972.80	272921.98	281109.64	207010.00	213220.30	219616.91
Herbicide Treatment	40	HOUR	180	7,200.00	185	7,400.00	7315.20	7534.66	7760.70	7518.40	7743.95	7976.27
Wildlife Biologist	250	HOUR	160	40,000.00	120	30,000.00	40640.00	41859.20	43114.98	30480.00	31394.40	32336.23
Netting	3700	SY	32	118,400.00	10	37,000.00	120294.40	123903.23	127620.33	37592.00	38719.76	39881.35
Prairie Dog Management	1	LS	10,000.00	10,000.00	15,000.00	15,000.00	10160.00	10464.80	10778.74	15240.00	15697.20	16168.12
Reconditioning	49883	SY	1.4	69,836.20	2.2	109,742.60	70953.58	73082.19	75274.65	111498.48	114843.44	118288.74

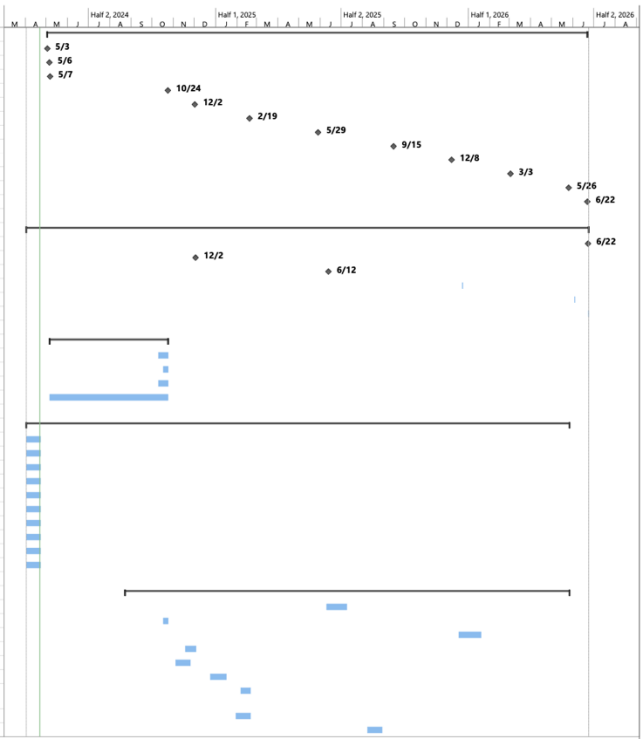
Concrete Safety Edge	4871	LF	6	29,226.00	4	19,484.00	29693.62	30584.42	31501.96	19795.74	20389.62	21001.30
Median Terminal	1	EACH	12,000.00	12,000.00	11,000.00	11,000.00	12192.00	12557.76	12934.49	11176.00	11511.28	11856.62
End Anchorage (Nonflared)	4	EACH	3600	14,400.00	4000	16,000.00	14630.40	15069.31	15521.39	16256.00	16743.68	17245.99
End Anchorage (Flared)	1	EACH	3,600.00	3,600.00	4,200.00	4,200.00	3657.60	3767.33	3880.35	4267.20	4395.22	4527.07
Fence (Plastic)	2390	LF	6.00	14,340.00	5.00	11,950.00	14569.44	15006.52	15456.72	12141.20	12505.44	12880.60
Fence Chain Link (Special) (36 Inch)	435	LF	44	19,140.00	80	34,800.00	19446.24	20029.63	20630.52	35356.80	36417.50	37510.03
Flashing Beacon (Portable)	61	EACH	1,000.00	61,000.00	1,050.00	64,050.00	61976.00	63835.28	65750.34	65074.80	67027.04	69037.86
Barricade (Type 3 M-A) (Temporary)	55	EACH	150	8,250.00	110	6,050.00	8382.00	8633.46	8892.46	6146.80	6331.20	6521.14
Vertical Panel (With Light) (Steady Burn)	250	EACH	40	10,000.00	55	13,750.00	10160.00	10464.80	10778.74	13970.00	14389.10	14820.77
Portable Message Sign Panel	10	EACH	5,500.00	55,000.00	11,000.00	110,000.00	55880.00	57556.40	59283.09	111760.00	115112.80	118566.18
Advance Warning Flasing or Sequencing Arrow Panel (C Type)	11	EACH	1,600.00	17,600.00	4,200.00	46,200.00	17881.60	18418.05	18970.59	46939.20	48347.38	49797.80
Drum Channelizing Device (With Light) (Flashing)	80	EACH	40.00	3,200.00	85.00	6,800.00	3251.20	3348.74	3449.20	6908.80	7116.06	7329.55
Drum Channelizing Device (With Light) (Steady Burn)	350	EACH	40	14,000.00	85	29,750.00	14224.00	14650.72	15090.24	30226.00	31132.78	32066.76
Portable Traffic Speed Monitor	2	EACH	6,000.00	12,000.00	4,200.00	8,400.00	12192.00	12557.76	12934.49	8534.40	8790.43	9054.14
Glare Screen (Temporary)	22580	LF	32.00	722,560.00	18.00	406,440.00	734120.96	756144.59	778828.93	412943.04	425331.33	438091.27
Traffic Cone	200	EACH	9	1,800.00	11	2,200.00	1828.80	1883.66	1940.17	2235.20	2302.26	2371.32
Imoact Attenuator (Temporary)	19	EACH	9000	171,000.00	5000	95,000.00	173736.00	178948.08	184316.52	96520.00	99415.60	102398.07
Pre-fabricated Concrete Washout Structure (Type 1)	6	EACH	3,000.00	18,000.00	1,100.00	6,600.00	18288.00	18836.64	19401.74	6705.60	6906.77	7113.97
HMA Safety Edge	360	LF	2.5	900.00	20	7,200.00	914.40	941.83	970.09	7315.20	7534.66	7760.70
Mobile Attenuator	200	DAY	800	160,000.00	510	102,000.00	162560.00	167436.80	172459.90	103632.00	106740.96	109943.19
Curb Type 6 (Section M)	1250	LF	16	20,000.00	53	66,250.00	20320.00	20929.60	21557.49	67310.00	69329.30	71409.18
Median Cover Material (Concrete)	1800	SF	13	23,400.00	10	18,000.00	23774.40	24487.63	25222.26	18288.00	18836.64	19401.74
Delineator (Type I)	350	EACH	28	9,800.00	33	11,550.00	9956.80	10255.50	10563.17	11734.80	12086.84	12449.45
Delineator (Type III)	5	EACH	50	250.00	38	190.00	254.00	261.62	269.47	193.04	198.83	204.80
Flashing Beacon	2	EACH	3400	6,800.00	5500	11,000.00	6908.80	7116.06	7329.55	11176.00	11511.28	11856.62
Detour Pavement	8810	SY	62.00	546,220.00	58.00	510,980.00	554959.52	571608.31	588756.55	519155.68	534730.35	550772.26
Night Work Lighting	1	LS	144000	144,000.00	80000	80,000.00	146304.00	150693.12	155213.91	81280.00	83718.40	86229.95
Division Total									$ 3,031,743.58			$ 2,414,513.25
											Average Cost	$ 2,723,128.42

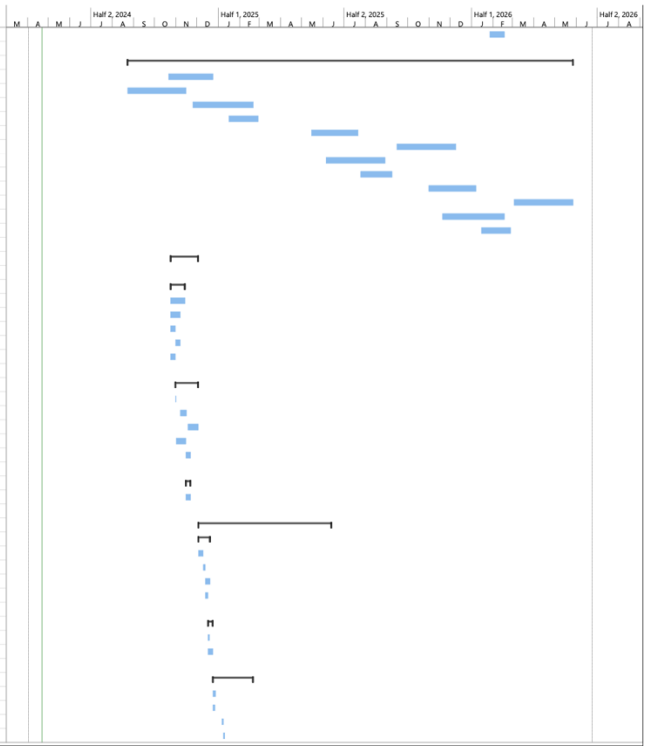
Utilities

Description	Quantity	Unit	(0) -EST- Engineer's Estimate		(1) 099H FLATIRON CONSTRUCTORS, INC.		Inflation (Engineer's Estimate)			Inflation (Flatiron Constructors)		
			Unit Price	Ext Amount	Unit Price	Ext Amount	2022-2023 (1.6%)	2023-2024 (3%)	2024-2025 (3%)	2022-2023 (1.6%)	2023-2024 (3%)	2024-2025 (3%)
Plug Culvert	1	EACH	3,820.00	3,820.00	2,500.00	2,500.00	3881.12	3997.55	4117.48	2540.00	2616.20	2694.69
Clean Culvert	5	EACH	1700	8,500.00	2000	10,000.00	8636.00	8895.08	9161.93	10160.00	10464.80	10778.74
Storm Drain Inlet Protection (Type I)	500	LF	19	9,500.00	10	5,000.00	9652.00	9941.56	10239.81	5080.00	5232.40	5389.37
Vane Grate Inlet (Double) (5 Foot)	1	EACH	9,000.00	9,000.00	3,900.00	3,900.00	9144.00	9418.32	9700.87	3962.40	4081.27	4203.71
Maintenance Hole Slab Base (15 Foot)	1	EACH	9,000.00	9,000.00	9100	9,100.00	9144.00	9418.32	9700.87	9245.60	9522.97	9808.66
Embankment Protector Type 3	5	EACH	2,400.00	12,000.00	2,200.00	11,000.00	12192.00	12557.76	12934.49	11176.00	11511.28	11856.62
Embankment Protector Type 5	1	EACH	2800	2,800.00	1400	1,400.00	2844.80	2930.14	3018.05	1422.40	1465.07	1509.02
Modify Inlet	1	EACH	3500	3,500.00	2500	2,500.00	3556.00	3662.68	3772.56	2540.00	2616.20	2694.69
Detour Drainage Pipe (Class 0)	1	LS	20000	20,000.00	7500	7,500.00	20320.00	20929.60	21557.49	7620.00	7848.60	8084.06
15 Inch Drainage Pipe (Class 7)	137	LF	75.00	10,275.00	60.00	8,220.00	10439.40	10752.58	11075.16	8351.52	8602.07	8860.13
24 Inch Drainage Pipe (Class 7)	143	LF	95	13,585.00	105	15,015.00	13802.36	14216.43	14642.92	15255.24	15712.90	16184.28
15 Inch Drainage Pipe (Class 9)	276	LF	160.00	44,160.00	59.00	16,284.00	44866.56	46212.56	47598.93	16544.54	17040.88	17552.11
18 Inch Drainage Pipe (Class 9)	163	LF	160.00	26,080.00	79.00	12,877.00	26497.28	27292.20	28110.96	13083.03	13475.52	13879.79
24 Inch Drainage Pipe (Class 9)	155	LF	220	34,100.00	105	16,275.00	34645.60	35684.97	36755.52	16535.40	17031.46	17542.41
18 Inch Equivalent Drainage Pipe Elliptical (Class 9)	589	LF	90.00	53,010.00	64.00	37,696.00	53858.16	55473.90	57138.12	38299.14	39448.11	40631.55
Reset Sign Panel	1	EACH	250.00	250.00	200.00	200.00	254.00	261.62	269.47	203.20	209.30	215.57
Reset End Anchorage	2	EACH	2000	4,000.00	1650	3,300.00	4064.00	4185.92	4311.50	3352.80	3453.38	3556.99
1 Inch Electrical Conduit	306	LF	22	6,732.00	30	9,180.00	6839.71	7044.90	7256.25	9326.88	9606.69	9894.89
2 Inch Electrical Conduit	306	LF	24	7,344.00	46	14,076.00	7461.50	7685.35	7915.91	14301.22	14730.25	15172.16
2 Inch Electrical Conduit (Bored)	306	LF	24	7,344.00	31	9,486.00	7461.50	7685.35	7915.91	9637.78	9926.91	10224.72
4 Inch Electrical Conduit	306	LF	40	12,240.00	46	14,076.00	12435.84	12808.92	13193.18	14301.22	14730.25	15172.16
4 Inch Electrical Conduit (Bored)	306	LF	15	4,590.00	38	11,628.00	4663.44	4803.34	4947.44	11814.05	12168.47	12533.52
1 Inch Electrical Conduit (Plastic)	306	LF	15	4,590.00	30	9,180.00	4663.44	4803.34	4947.44	9326.88	9606.69	9894.89
2 Inch Electrical Conduit (Plastic)	306	LF	6	1,836.00	33.5	10,251.00	1865.38	1921.34	1978.98	10415.02	10727.47	11049.29
Type One Pull Box	5	EACH	800	4,000.00	1,400.00	7,000.00	4064.00	4185.92	4311.50	7112.00	7325.36	7545.12
Type Three Pull Box	12	EACH	2,000.00	24,000.00	1,700.00	20,400.00	24384.00	25115.52	25868.99	20726.40	21348.19	21988.64
Type Four Pull Box	2	EACH	1,800.00	3,600.00	2,550.00	5,100.00	3657.60	3767.33	3880.35	5181.60	5337.05	5497.16
Wiring	1	LS	30,000.00	30,000.00	130000	130,000.00	30480.00	31394.40	32336.23	132080.00	136042.40	140123.67
Luminare Mounting Support System	1	LS	50,000.00	50,000.00	8,500.00	8,500.00	50800.00	52324.00	53893.72	8636.00	8895.08	9161.93
Lighting Control Center	1	EACH	9,000.00	9,000.00	7,300.00	7,300.00	9144.00	9418.32	9700.87	7416.80	7639.30	7868.48
1310nm SFP Optic Module	4	EACH	500	2,000.00	160.00	640.00	2032.00	2092.96	2155.75	650.24	669.75	689.84
Serial to IP Converter	1	EACH	1500	1,500.00	3900	3,900.00	1524.00	1569.72	1616.81	3962.40	4081.27	4203.71
Ramp Metering Controller	1	EACH	12,000.00	12,000.00	7,900.00	7,900.00	12192.00	12557.76	12934.49	8026.40	8267.19	8515.21
Fiber Optic Cable (Single Mode) (12 Fiber)	225	LF	20	4,500.00	8	1,800.00	4572.00	4709.16	4850.43	1828.80	1883.66	1940.17
Fiber Optic Termination Panel (12 Fiber)	1	EACH	1,400.00	1,400.00	1,600.00	1,600.00	1422.40	1465.07	1509.02	1625.60	1674.37	1724.60
Buffer Tube Fan Out Kit	1	EACH	200	200.00	190	190.00	203.20	209.30	215.57	193.04	198.83	204.80
Closed Circut Television	1	EACH	8,000.00	8,000.00	5,900.00	5,900.00	8128.00	8371.84	8623.00	5994.40	6174.23	6359.46
Test Fiber Optic Cable	1	LS	5,000.00	5,000.00	6,100.00	6,100.00	5080.00	5232.40	5389.37	6197.60	6383.53	6575.03
Breakaway Tapered ITS Pole Steel (30 Feet) xx	1	EACH	8000	8,000.00	8300	8,300.00	8128.00	8371.84	8623.00	8432.80	8685.78	8946.36
Splice Fiber Optic Cable (12 Strand)	1	EACH	2,600.00	2,600.00	1,250.00	1,250.00	2641.60	2720.85	2802.47	1270.00	1308.10	1347.34
Ethernet Switch Type II	1	EACH	8000	8,000.00	6400	6,400.00	8128.00	8371.84	8623.00	6502.40	6697.47	6898.40
Division Total									$ 519,595.82			$ 498,973.93
											Average Cost	$ 509,284.88

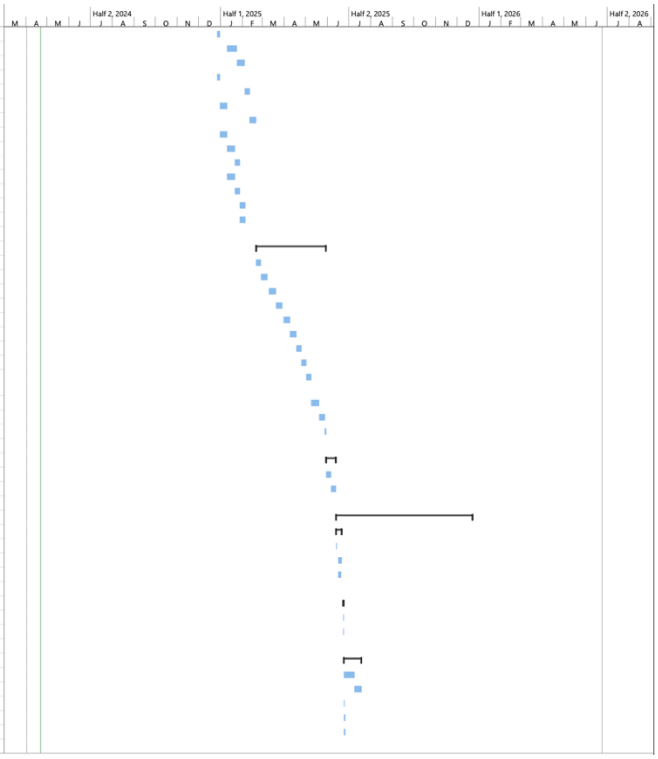
MS Schedule

ID	Task Name	Duration	Start	Finish	Predecessors
1	**Milestones**	**543 days**	**Fri 5/3/24**	**Mon 6/22/2**	
2	Contract Review/Contract Excecution	0 days	Fri 5/3/24	Fri 5/3/24	
3	Notice of Award	0 days	Mon 5/6/24	Mon 5/6/24	
4	Notice to Proceed	0 days	Tue 5/7/24	Tue 5/7/24	
5	Begin Contract Time	0 days	Thu 10/24/2	Thu 10/24/2	
6	Phase 0 Completion	0 days	Mon 12/2/24	Mon 12/2/24	
7	Phase 1 Substructure Completion	0 days	Wed 2/19/25	Wed 2/19/25	
8	Phase 1 Superstructure Completion	0 days	Thu 5/29/25	Thu 5/29/25	
9	Phase 2 Substructure Completion	0 days	Mon 9/15/25	Mon 9/15/25	
10	Phase 2 Superstructure Completion	0 days	Mon 12/8/25	Mon 12/8/25	
11	Phase 3 Substructure Completion	0 days	Tue 3/3/26	Tue 3/3/26	
12	Phase 3 Superstructure Completion	0 days	Tue 5/26/26	Tue 5/26/26	
13	Final Acceptance	0 days	Mon 6/22/2	Mon 6/22/2	
14					
15	**Project Management**	**568 days**	**Tue 4/2/24**	**Tue 6/23/26**	
16	Project Duration- To Substantial	0 days	Mon 6/22/2	Mon 6/22/2	244
17	Phase 0	0 days	Mon 12/2/24	Mon 12/2/24	67
18	Phase 1	0 days	Thu 6/12/25	Thu 6/12/25	86
19	Phase 2	1 day	Tue 12/23/2	Tue 12/23/2	135
20	Phase 3	1 day	Wed 6/3/26	Wed 6/3/26	190
21	Phase 4	1 day	Tue 6/23/26	Tue 6/23/26	244
22					
23	**Permits**	**120 days**	**Mon 5/6/24**	**Thu 10/24/2**	
24	Aquire Adams County Building Permit	10 days	Thu 10/10/2	Thu 10/24/2	67SF
25	Transfer CDPHE Stormwater Permit	5 days	Thu 10/17/2	Thu 10/24/2	67SF
26	Acquire State Demo Permit	10 days	Thu 10/10/2	Thu 10/24/2	67SF
27	Acquire 404 Permit	120 days	Mon 5/6/24	Wed 10/23/2	
28					
29	**Subs/Services**	**548 days**	**Tue 4/2/24**	**Wed 5/27/2**	
30	Prep Excecute Sub/Service Agreements- Traffic Control	15 days	Tue 4/2/24	Mon 4/22/24	
31	Prep/Excecute PO- Precast	15 days	Tue 4/2/24	Mon 4/22/24	
32	Prep/Excecute Sub/Service Agreements- Misc.	15 days	Tue 4/2/24	Mon 4/22/24	
33	Prep/Excecute PO- Aggregates	15 days	Tue 4/2/24	Mon 4/22/24	
34	Prep/Excecute Sub/Service Agreements- Erosion Contro	15 days	Tue 4/2/24	Mon 4/22/24	
35	Prep/Excecute PO- Paving Materials	15 days	Tue 4/2/24	Mon 4/22/24	
36	Prep/Excecute PO- Rebar	15 days	Tue 4/2/24	Mon 4/22/24	
37	Prep/Excecute Sub/Service Agreements- Pavement Mar	15 days	Tue 4/2/24	Mon 4/22/24	
38	Prep/Excecute Sub/Service Agreements- Asphalt Paving	15 days	Tue 4/2/24	Mon 4/22/24	
39	Prep/Excecute PO- Misc. Materials	15 days	Tue 4/2/24	Mon 4/22/24	
40					
41	**Submittals**	**448 days**	**Fri 8/23/24**	**Wed 5/27/2**	
42	Prep Submit Bridge Removal Plan Phase 2	20 days	Tue 6/10/25	Wed 7/9/25	147SF
43	Prep/Submit Misc Administrative Submittals	5 days	Thu 10/17/	Thu 10/24/2	67SF
44	Prep/Submit Bridge Removal Plan Phase 3	20 days	Thu 12/18/2	Mon 1/19/2	202SF
45	Prep/Submit Inital MHTs	10 days	Mon 11/18/2	Tue 12/3/24	87SF
46	Prep/Submit Girder Shop Drawings (ALL)	15 days	Mon 11/4/24	Mon 11/25/2	56SF
47	Prep/Submit Precast Deck Panel Shop Drawings (ALL)	15 days	Tue 12/24/2	Thu 1/16/25	57SF
48	Prep/Submit/Request Partial Closure MHT- WB I-270 for Phase 1 Girders	10 days	Thu 2/6/25	Thu 2/20/25	118SF
49	Prep/Submit Girder Erection Plan- Phase 1	15 days	Thu 1/30/25	Thu 2/20/25	118SF
50	Prep/Submit Girder Erection Plan- Phase 2	15 days	Fri 8/8/25	Fri 8/29/25	173SF
51	Prep/Submit Girder Erection Plan- Phase 3	15 days	Tue 1/27/26	Tue 2/17/26	228SF
52					
53	**Procurement**	**448 days**	**Fri 8/23/24**	**Wed 5/27/2**	
54	Bridge Rebar Procurement- Phase 1	45 days	Mon 10/21/2	Tue 12/24/2	98SF
55	Paving Material Procurement- Phase 1	60 days	Fri 8/23/24	Fri 11/15/24	84SF
56	Girder Procurement- Phase 1	60 days	Mon 11/25/2	Thu 2/20/25	118SF
57	Precast Deck Panel Procurement- Phase 1	30 days	Thu 1/16/25	Thu 2/27/25	119SF
58	Bridge Rebar Procurement- Phase 2	45 days	Thu 5/15/25	Mon 7/21/25	153SF
59	Paving Material Procurement- Phase 2	60 days	Mon 9/15/25	Tue 12/9/25	187SF
60	Girder Procurement- Phase 2	60 days	Thu 6/5/25	Fri 8/29/25	173SF
61	Precast Deck Panel Procurement- Phase 2	30 days	Fri 7/25/25	Mon 9/8/25	174SF
62	Bridge Rebar Procurement- Phase 3	45 days	Fri 10/31/25	Wed 1/7/26	208SF
63	Paving Material Procurement- Phase 3	60 days	Tue 3/3/26	Wed 5/27/2	242SF
64	Girder Procurement- Phase 3	60 days	Thu 11/20/2	Tue 2/17/26	228SF
65	Precast Deck Panel Procurement- Phase 3	30 days	Thu 1/15/26	Thu 2/26/26	229SF
66					
67	**Phase 0**	**27 days**	**Thu 10/24/2**	**Mon 12/2/2**	
68					
69	**Project Mobilization**	**15 days**	**Thu 10/24/2**	**Wed 11/13/2**	**27**
70	Prep/Excecute Office Lease Agreement	15 days	Thu 10/24/2	Wed 11/13/2	
71	Prep Site/Setup Trailer/Hookup Temp Power	10 days	Thu 10/24/2	Wed 11/6/24	
72	Move-In/Setup/Open Project Offices	5 days	Thu 10/24/2	Wed 10/30/2	
73	Instal Initial BMPs	5 days	Thu 10/31/2	Wed 11/6/24	72
74	Erosion Control	5 days	Thu 10/24/2	Wed 10/30/2	
75					
76	**Earthwork**	**22 days**	**Thu 10/31/2**	**Mon 12/2/2**	
77	Clearing and Grubbing	1 day	Thu 10/31/2	Thu 10/31/2	74
78	Exc/Emb/Prep RDWY Subgrade	7 days	Thu 11/7/24	Fri 11/15/24	73
79	Place/Finish ABC (Class 6)	10 days	Mon 11/18/2	Mon 12/2/24	78
80	Exc/Emb/Prep Site Subgrade	10 days	Fri 11/1/24	Thu 11/14/2	77
81	FPRS Crane Pad	5 days	Fri 11/15/24	Thu 11/21/2	80
82					
83	**Paving**	**5 days**	**Fri 11/15/24**	**Thu 11/21/2**	
84	Detour Paving (Lane Shift)	5 days	Fri 11/15/24	Thu 11/21/2	80
85					
86	**Phase 1- Construct Between Existing**	**135 days**	**Tue 12/3/24**	**Thu 6/12/25**	**67**
87	**Traffic Control/SWPPP**	**13 days**	**Tue 12/3/24**	**Thu 12/19/2**	
88	Set Initial Traffic Control	5 days	Tue 12/3/24	Mon 12/9/24	80
89	Restripe Project- Temporary	3 days	Tue 12/10/2	Thu 12/12/2	88
90	Install Initial BMPs- Phase 1	5 days	Fri 12/13/24	Thu 12/19/2	89
91	Set Barrier- Phase 1	2 days	Fri 12/13/24	Mon 12/16/2	89
92					
93	**Demo**	**5 days**	**Tue 12/17/2**	**Mon 12/23/2**	
94	Misc Removals (Guardrails, Signs, etc)	2 days	Tue 12/17/2	Wed 12/18/2	91
95	Misc Removals- Saw Cut	5 days	Tue 12/17/2	Mon 12/23/2	91
96					
97	**Substructure**	**40 days**	**Tue 12/24/2**	**Wed 2/19/25**	
98	Install Drilled Shafts (36")- Phase 1	3 days	Tue 12/24/2	Fri 12/27/24	95
99	Piling Strain Gauge Install- Abut1 & 2/Phase 1	2 days	Tue 12/24/2	Thu 12/26/2	95
100	CSL Testing - Phase 1	2 days	Mon 1/6/25	Tue 1/7/25	98FS+4 days
101	FPRS Pier Cap (Caissons)- Phase 1	2 days	Wed 1/8/25	Thu 1/9/25	100

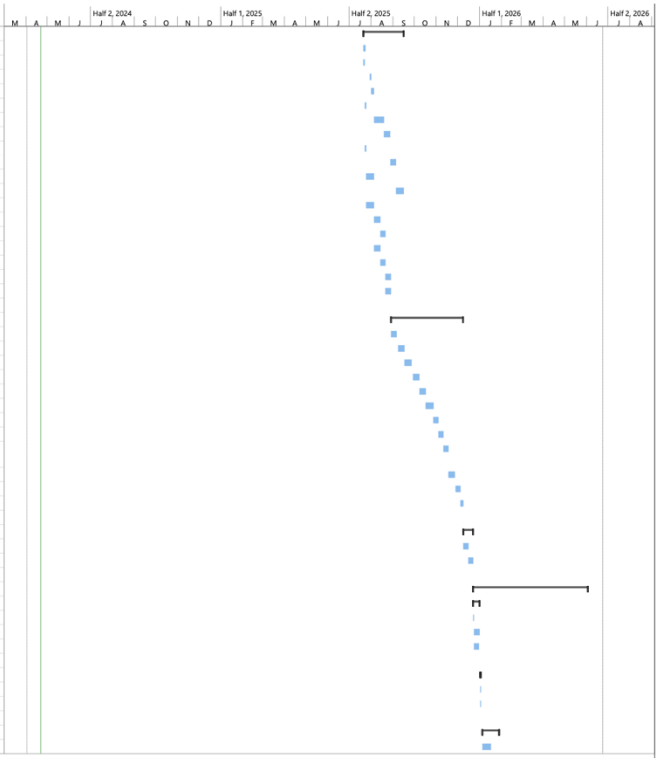




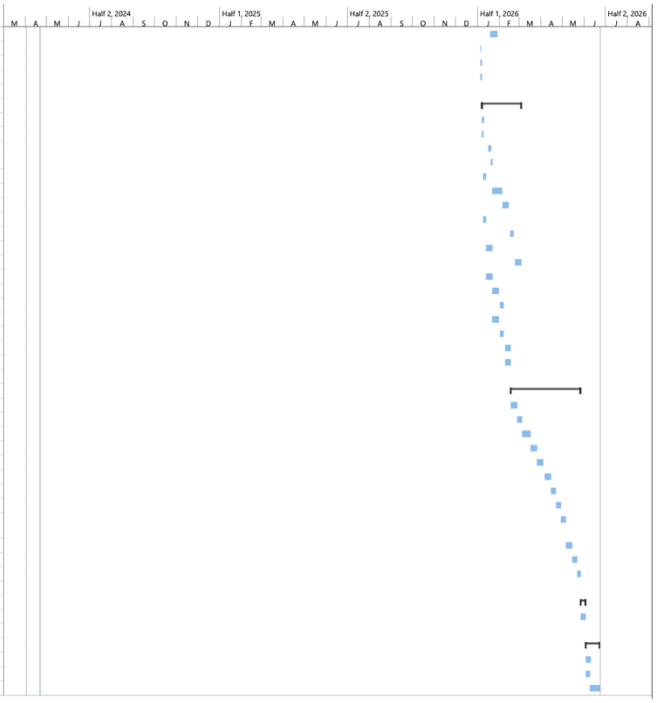
ID	Task Mode	Task Name	Duration	Start	Finish	Predecessors
102		Structure Exc- Abut1/Phase 1	2 days	Fri 12/27/24	Mon 12/30/2	99
103		FRPS Piers- Phase 1	10 days	Fri 1/10/25	Thu 1/23/25	101
104		Cure Pier- Phase 1	7 days	Fri 1/24/25	Mon 2/3/25	103
105		Structure Exc-Abut2/Phase 1	2 days	Fri 12/27/24	Mon 12/30/2	99
106		FRPS Pier Cap- Phase 1	5 days	Tue 2/4/25	Mon 2/10/25	104
107		FRPS Abutment to Bearing Seat (Abut1/Phase 1)	7 days	Tue 12/31/2	Thu 1/9/25	102
108		Cure Pier Cap- Phase 1	7 days	Tue 2/11/25	Wed 2/19/25	106
109		FRPS Abutment to Bearing Seat (Abut 2/Phase 1)	7 days	Tue 12/31/2	Thu 1/9/25	105
110		Cure Abutment 1- Phase 1	7 days	Fri 1/10/25	Mon 1/20/25	107
111		FRPS Slope Pavement- Abut 1/Phase 1	5 days	Tue 1/21/25	Mon 1/27/25	110
112		Cure Abutment 2- Phase 1	7 days	Fri 1/10/25	Mon 1/20/25	109
113		FRPS Slope Pavement- Abut 2/Phase 1	5 days	Tue 1/21/25	Mon 1/27/25	112
114		Structure Backfill- Abut 1/Phase 1	6 days	Tue 1/28/25	Tue 2/4/25	111
115		Structure Backfill- Abut 2/Phase 1	6 days	Tue 1/28/25	Tue 2/4/25	113
116						
117		**Superstructure**	**70 days**	**Thu 2/20/25**	**Thu 5/29/25**	**97**
118		Prep/Set Conc Girders- Phase 1	5 days	Thu 2/20/25	Wed 2/26/25	115
119		Set Precast Panel Deck btw Girders- Phase 1	7 days	Thu 2/27/25	Fri 3/7/25	118
120		Install Overhangs/Form Deck- Phase 1	8 days	Mon 3/10/25	Wed 3/19/25	119
121		Reinforce Deck- Phase 1	7 days	Thu 3/20/25	Fri 3/28/25	120
122		Prep/Pour Bridge Deck- Phase 1	7 days	Mon 3/31/25	Tue 4/8/25	121
123		Cure Deck- Phase 1	7 days	Wed 4/9/25	Thu 4/17/25	122
124		Strip EOD/Overhangs- Phase 1	5 days	Fri 4/18/25	Thu 4/24/25	123
125		FRPS Sleeper- Abutment 1 & 2- Phase 1	5 days	Fri 4/25/25	Thu 5/1/25	124
126		FRPS Approach Slabs with Expansion Device Abutment 1 & 2- Phase 1	5 days	Fri 5/2/25	Thu 5/8/25	125
127		Cure Approach Slabs Abutment 1 & 2- Phase 1	7 days	Fri 5/9/25	Mon 5/19/25	126
128		Waterproofing Membrane- Phase 1	5 days	Tue 5/20/25	Tue 5/27/25	127
129		3 HWM Overlay- Phase 1	2 days	Wed 5/28/25	Thu 5/29/25	128
130						
131		**Finishes**	**10 days**	**Fri 5/30/25**	**Thu 6/12/25**	
132		Road Paving- Phase 1	5 days	Fri 5/30/25	Thu 6/5/25	129
133		Temporary Striping and Signage- Phase 1	5 days	Fri 6/6/25	Thu 6/12/25	132
134						
135		**Phase 2- Reconstruct I-270 Westbound Bridge**	**134 days**	**Fri 6/13/25**	**Mon 12/22/2**	**86**
136		**Traffic Control/SWPPP**	**6 days**	**Fri 6/13/25**	**Fri 6/20/25**	
137		Traffic Switch onto Phase 1- Phase 2	1 day	Fri 6/13/25	Fri 6/13/25	
138		Install Initial BMPs- Phase 2	5 days	Mon 6/16/25	Fri 6/20/25	137
139		Reset/Set Barrier- Phase 2	4 days	Mon 6/16/25	Thu 6/19/25	137
140						
141		**Utilities**	**1 day**	**Mon 6/23/2**	**Mon 6/23/2**	
142		Drainage Pipe Class 9- Phase 2	1 day	Mon 6/23/25	Mon 6/23/25	138
143		Reroute Utilities- Phase 2	1 day	Mon 6/23/25	Mon 6/23/25	138
144						
145		**Demo**	**18 days**	**Tue 6/24/25**	**Fri 7/18/25**	
146		Milling of Existing Pavement	10 days	Tue 6/24/25	Tue 7/8/25	143
147		Bridge & Pier Removal	8 days	Wed 7/9/25	Fri 7/18/25	146
148		Clearing and Grubbing- Phase 2	1 day	Tue 6/24/25	Tue 6/24/25	143
149		Misc Removals (Guardrails, Signs, etc.)- Phase 2	2 days	Tue 6/24/25	Wed 6/25/25	143
150		Drainage Removals- Phase 2 (Inlets, Pipe, MHs, etc.)	2 days	Tue 6/24/25	Wed 6/25/25	142
151						



ID	Task Mode	Task Name	Duration	Start	Finish	Predecessors
152		**Substructure**	**40 days**	**Mon 7/21/2**	**Mon 9/15/2**	
153		Install Drilled Shafts (36")- Phase 2	3 days	Mon 7/21/25	Wed 7/23/25	145
154		Piling Strain Gauge Install- Abut1 & 2/Phase 2	2 days	Mon 7/21/25	Tue 7/22/25	145
155		CSL Testing - Phase 2	2 days	Wed 7/30/25	Thu 7/31/25	153FS+4 days
156		FRPS Pier Cap (Caissons)- Phase 2	2 days	Fri 8/1/25	Mon 8/4/25	155
157		Structure Exc- Abut1/Phase 2	2 days	Wed 7/23/25	Thu 7/24/25	154
158		FRPS Piers- Phase 2	10 days	Tue 8/5/25	Mon 8/18/25	156
159		Cure Pier- Phase 2	7 days	Tue 8/19/25	Wed 8/27/25	158
160		Structure Exc-Abut2/Phase 2	2 days	Wed 7/23/25	Thu 7/24/25	154
161		FRPS Pier Cap- Phase 2	5 days	Thu 8/28/25	Thu 9/4/25	159
162		FRPS Abutment to Bearing Seat (Abut1/Phase 2)	7 days	Fri 7/25/25	Mon 8/4/25	157
163		Cure Pier Cap- Phase 2	7 days	Fri 9/5/25	Mon 9/15/25	161
164		FRPS Abutment to Bearing Seat (Abut 2/Phase 2)	7 days	Fri 7/25/25	Mon 8/4/25	160
165		Cure Abutment 1- Phase 2	7 days	Tue 8/5/25	Wed 8/13/25	162
166		FRPS Slope Pavement- Abut 1/Phase 2	5 days	Thu 8/14/25	Wed 8/20/25	165
167		Cure Abutment 2- Phase 2	7 days	Tue 8/5/25	Wed 8/13/25	164
168		FRPS Slope Pavement- Abut 2/Phase 2	5 days	Thu 8/14/25	Wed 8/20/25	167
169		Structure Backfill- Abut 1/Phase 2	6 days	Thu 8/21/25	Thu 8/28/25	166
170		Structure Backfill- Abut 2/Phase 2	6 days	Thu 8/21/25	Thu 8/28/25	168
171						
172		**Superstructure**	**70 days**	**Fri 8/29/25**	**Mon 12/8/2**	
173		Prep/Set Conc Girders- Phase 2	5 days	Fri 8/29/25	Fri 9/5/25	170
174		Set Precast Panel Deck btw Girders- Phase 2	7 days	Mon 9/8/25	Tue 9/16/25	173
175		Install Overhangs/Form Deck- Phase 2	8 days	Wed 9/17/25	Fri 9/26/25	174
176		Reinforce Deck- Phase 2	7 days	Mon 9/29/25	Tue 10/7/25	175
177		Prep/Pour Bridge Deck- Phase 2	7 days	Wed 10/8/25	Thu 10/16/2	176
178		Cure Deck- Phase 2	7 days	Fri 10/17/25	Mon 10/27/2	177
179		Strip EOD/Overhangs- Phase 2	5 days	Tue 10/28/2	Mon 11/3/25	178
180		FRPS Sleeper- Abutment 1 & 2- Phase 2	5 days	Tue 11/4/25	Mon 11/10/2	179
181		FRPS Appraoch Slabs with Expansion Device Abutment 1 & 2- Phase 2	5 days	Tue 11/11/25	Mon 11/17/25	180
182		Cure Approach Slabs Abutment 1 & 2- Phase 2	7 days	Tue 11/18/2	Wed 11/26/2	181
183		Waterproofing Membrane- Phase 2	5 days	Fri 11/28/25	Thu 12/4/25	182
184		3 HWM Overlay- Phase 2	2 days	Fri 12/5/25	Mon 12/8/25	183
185						
186		**Finishes**	**10 days**	**Tue 12/9/25**	**Mon 12/22/2**	
187		Road Paving- Phase 2	5 days	Tue 12/9/25	Mon 12/15/2	184
188		Temporary Striping and Signage- Phase 2	5 days	Tue 12/16/2	Mon 12/22/2	187
189						
190		**Phase 3- Reconstruct I-270 Eastbound Bridge**	**113 days**	**Tue 12/23/2**	**Tue 6/2/26**	**135**
191		**Traffic Control/SWPPP**	**6 days**	**Tue 12/23/2**	**Wed 12/31/2**	
192		Traffic Switch onto Phase 2- Phase 3	1 day	Tue 12/23/2	Tue 12/23/2	188
193		Install Initial BMPs- Phase 3	5 days	Wed 12/24/2	Wed 12/31/2	192
194		Reset/Set Barrier- Phase 3	4 days	Wed 12/24/2	Tue 12/30/2	192
195						
196		**Utilities**	**1 day**	**Fri 1/2/26**	**Fri 1/2/26**	
197		Drainage Pipe Class 9- Phase 3	1 day	Fri 1/2/26	Fri 1/2/26	193
198		Reroute Utilities- Phase 3	1 day	Fri 1/2/26	Fri 1/2/26	193
199						
200		**Demo**	**18 days**	**Mon 1/5/26**	**Wed 1/28/2**	
201		Milling of Existing Pavement	10 days	Mon 1/5/26	Fri 1/16/26	198



ID	Task Mode	Task Name	Duration	Start	Finish	Predecessors
202		Bridge & Pier Removal	8 days	Mon 1/19/26	Wed 1/28/26	201
203		Clearing and Grubbing- Phase 3	1 day	Mon 1/5/26	Mon 1/5/26	198
204		Misc Removals (Guardrails, Signs, etc.)- Phase 3	2 days	Mon 1/5/26	Tue 1/6/26	198
205		Drainage Removals- Phase 3 (Inlets, Pipe, MHs, etc.)	2 days	Mon 1/5/26	Tue 1/6/26	197
206						
207		**Substructure**	**40 days**	**Wed 1/7/26**	**Tue 3/3/26**	
208		Install Drilled Shafts (36")- Phase 3	3 days	Wed 1/7/26	Fri 1/9/26	205
209		Piling Strain Gauge Install- Abut1 & 2/Phase 3	2 days	Wed 1/7/26	Thu 1/8/26	205
210		CSL Testing - Phase 3	2 days	Fri 1/16/26	Mon 1/19/26	208FS+4 days
211		FRPS Pier Cap (Caissons)- Phase 3	2 days	Tue 1/20/26	Wed 1/21/26	210
212		Structure Exc- Abut1/Phase 3	2 days	Fri 1/9/26	Mon 1/12/26	209
213		FRPS Piers- Phase 3	10 days	Thu 1/22/26	Wed 2/4/26	211
214		Cure Pier- Phase 3	7 days	Thu 2/5/26	Fri 2/13/26	213
215		Structure Exc-Abut2/Phase 3	2 days	Fri 1/9/26	Mon 1/12/26	209
216		FRPS Pier Cap- Phase 3	5 days	Mon 2/16/26	Fri 2/20/26	214
217		FRPS Abutment to Bearing Seat (Abut1/Phase 3)	7 days	Tue 1/13/26	Wed 1/21/26	212
218		Cure Pier Cap- Phase 3	7 days	Mon 2/23/26	Tue 3/3/26	216
219		FRPS Abutment to Bearing Seat (Abut 2/Phase 3)	7 days	Tue 1/13/26	Wed 1/21/26	215
220		Cure Abutment 1- Phase 3	7 days	Thu 1/22/26	Fri 1/30/26	217
221		FRPS Slope Pavement- Abut 1/Phase 3	5 days	Mon 2/2/26	Fri 2/6/26	220
222		Cure Abutment 2- Phase 3	7 days	Thu 1/22/26	Fri 1/30/26	219
223		FRPS Slope Pavement- Abut 2/Phase 3	5 days	Mon 2/2/26	Fri 2/6/26	222
224		Structure Backfill- Abut 1/Phase 3	6 days	Mon 2/9/26	Mon 2/16/26	221
225		Structure Backfill- Abut 2/Phase 3	6 days	Mon 2/9/26	Mon 2/16/26	223
226						
227		**Superstructure**	**70 days**	**Tue 2/17/26**	**Tue 5/26/26**	
228		Prep/Set Conc Girders- Phase 3	7 days	Tue 2/17/26	Wed 2/25/26	225
229		Set Precast Panel Deck btw Girders- Phase 3	5 days	Thu 2/26/26	Wed 3/4/26	228
230		Install Overhangs/Form Deck- Phase 3	8 days	Thu 3/5/26	Mon 3/16/26	229
231		Reinforce Deck- Phase 3	7 days	Tue 3/17/26	Wed 3/25/26	230
232		Prep/Pour Bridge Deck- Phase 3	7 days	Thu 3/26/26	Fri 4/3/26	231
233		Cure Deck- Phase 3	7 days	Mon 4/6/26	Tue 4/14/26	232
234		Strip EOD/Overhangs- Phase 3	5 days	Wed 4/15/26	Tue 4/21/26	233
235		FRPS Sleeper- Abutment 1 & 2- Phase 3	5 days	Wed 4/22/26	Tue 4/28/26	234
236		FRPS Approach Slabs with Expansion Device Abutment 1 & 2- Phase 3	5 days	Wed 4/29/26	Tue 5/5/26	235
237		Cure Approach Slabs Abutment 1 & 2- Phase 3	7 days	Wed 5/6/26	Thu 5/14/26	236
238		Waterproofing Membrane- Phase 3	5 days	Fri 5/15/26	Thu 5/21/26	237
239		3 HWM Overlay- Phase 3	2 days	Fri 5/22/26	Tue 5/26/26	238
240						
241		**Finishes**	**5 days**	**Wed 5/27/26**	**Tue 6/2/26**	
242		Road Paving- Phase 3	5 days	Wed 5/27/26	Tue 6/2/26	239
243						
244		**Phase 4**	**14 days**	**Wed 6/3/26**	**Mon 6/22/26**	**190**
245		Permanent Striping	5 days	Wed 6/3/26	Tue 6/9/26	242
246		Permanent Barrier	4 days	Wed 6/3/26	Mon 6/8/26	242
247		Demobilization	10 days	Tue 6/9/26	Mon 6/22/26	246



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