SeaWolf Engineering 2015

University of Alaska Anchorage

Penguin Creek Crossing

DESIGN STUDY REPORT

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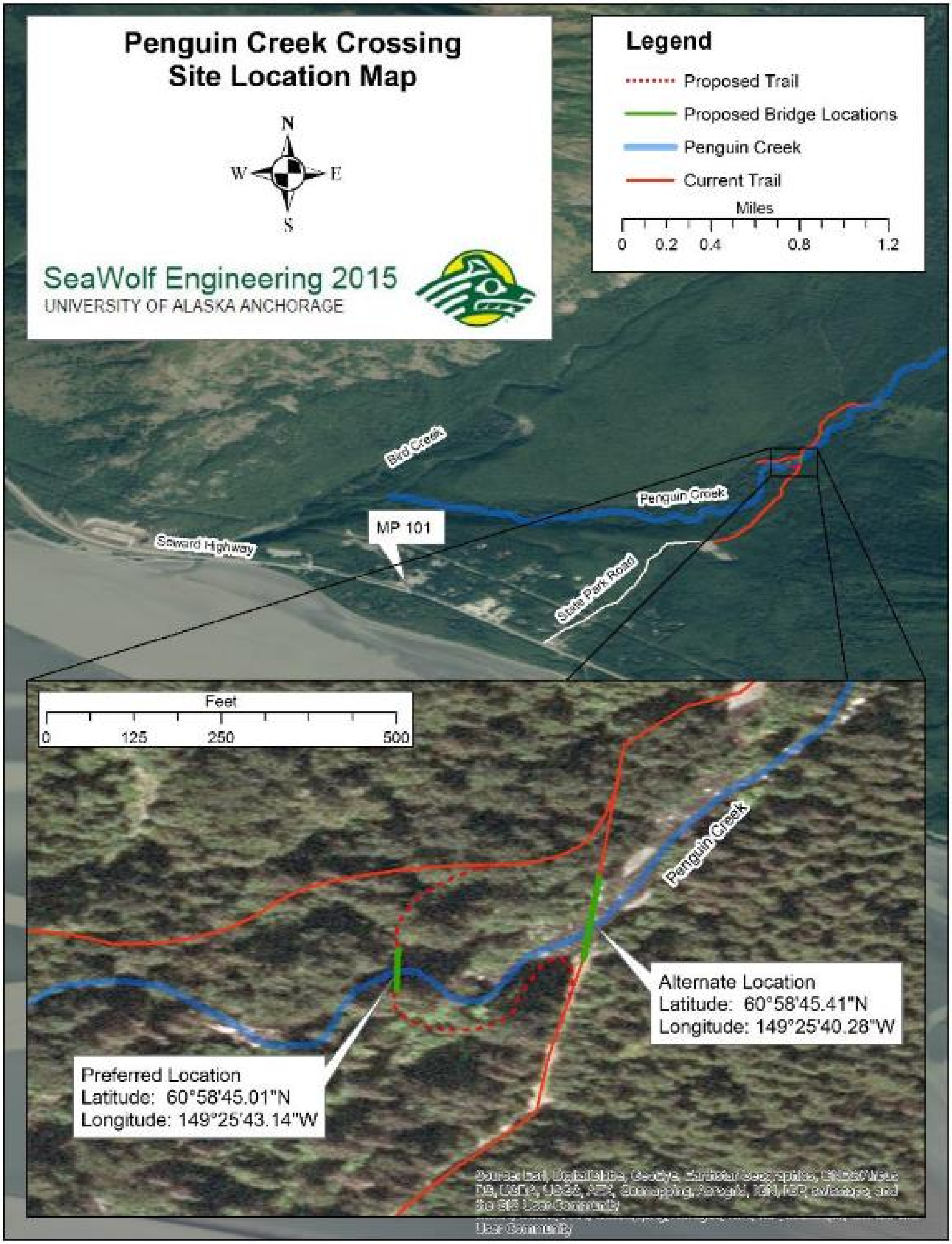

Source: Esri, DigitalGlobe, GeoEye, Earthstar Geographics, CNES/Airbus DS, USDA, USGS, AEX, Getmapping, Aerogrid, IGN, IGP, swisstopo, and the GIS User Community

User Community

Figure 1: Site Map

ABSTRACT

Looking at historical satellite imagery, it is evident that the path of Penguin Creek's flow has changed significantly over the past decade. It is important to take into consideration hydraulics and hydrology, determine an accurate 100 year flood depth to prevent the new bridge from washing away, and prevent scouring of the abutments during high flood events. Designing a safe, convenient, economical, and environmentally compatible bridge for Penguin Creek was the goal when selecting the site location, bridge, and foundation type. This design study report will explain the design process using knowledge from structural, geotechnical, hydraulics and hydrology, and environmental engineering.

INTRODUCTION

The purpose of this project is to construct a bridge over Penguin Creek along the Bird Valley Trailhead in order to remove a human interaction with this anadromous creek. Formerly there was a bridge in this location to allow movement from either side of the streambed. The original bridge spanned 70 feet with a width of 5 feet and timber abutments. Due to the susceptibility of flooding in this location the previous bridge washed away leaving small remnants of the timber foundation. Park users continue to cross Penguin Creek at this location causing un-needed disturbance of the fish and streambed. In order to prevent further disturbance of the anadromous creek, a bridge will be constructed to span the width.

The project is located near mile post 101 along the Old Seward Highway; 26 miles South of Anchorage and 12 miles North of Girdwood. The site is at latitude 60.9783 and longitude -149.4539. This report will discuss locations considered for the new bridge, hydraulic and hydrology analysis, foundation considerations and bridge assessments.

PROBLEM DEFINITION

Park users are regularly crossing Penguin Creek, an anadromous stream, while hiking and riding ATV’s along the Bird Valley Trailhead. The habitat disturbance caused by these crossings can have a negative impact on the spawning of Salmon and should be avoided wherever possible by providing for alternative means of crossing. The Alaska Department of Natural Resources, Division of Parks and Outdoor Recreation has requested a bridge be designed to accommodate ATVs and pedestrians at or near the original bridge site.

EXISTING CONDITIONS

The project is located approximately 0.5 miles from the Bird Valley Trailhead parking lot in the Chugach Mountains between Bird Peak and Penguin Peak. The area is of mountainous terrain and is densely forested around the trails and creek. Slopes within the project range from very steep to gradual running downward in a general east to west direction. Annual precipitation is estimated at 60 inches per year, while annual snowfall is estimated at 100 inches per year.

Throughout this report the creek bank sides will be referenced using the terms near and far. Near side bank refers to the South-East bank as approached from the Bird Valley Trailhead, whereas the far side bank refers to the North-West bank. Penguin Creek is a mild sloped meandering stream with some braiding throughout its path. The floodplain is very large, covering approximately 510 feet in width. Areas north of the original bridge location show signs of regular snow avalanches.

DESIGN CRITERIA

METHODOLOGY

Project construction is tentatively scheduled to for summer 2016, with total construction time estimated at 2 months. The structure will be designed in similar fashion to recent pedestrian bridges. Design loading will be consistent with ASCE 7-10.

For estimation purposes preliminary loads were developed. The raw loads were assumed of 120 psf snow load, 100 psf for dead load, and 100 psf for live load. Per DNR-DPOR's guidance, seismic load will be ignored for this project.

Since soils data was not available at the time this report was written, a soils profile was assumed. See Appendix D for a copy of the bore log annotating the assumed soil profile. The profile consisted of 2.0 feet silty/organic material, 23 feet of glacial till/gravel, followed by increasing grain sized gravel. It was assumed that the water table is 2 feet below the surface, and a frost active zone of 10 feet is present. See Appendix D for a detailed list of soil properties and calculations.

Because the previous bridge was scoured out around the abutments, DNR-DPOR has requested that the 100 year flood (Q_{100}) be used for design to protect against scour and washout. Penguin Creek is an ungagged site, and at the time this report was written no data was available for review. As such, regression equations provided by USGS and developed for different regions within Alaska and conterminous basins in Canada were used to calculate Q_{100}.

DATA COLLECTION

Data was collected from two separate site visits, the first included the whole team on 1/23/2015 in the presence of Jacob Gondek, P.E. with DNR-DROP and the second with three members of the design team. During the first site visit the team collected data for four potential bridge crossing sites. At each site the design team collected approximate bridge span length, number of spans, and general topography data of the surrounding area. The first site visit also included velocity flow estimations. The velocity was determined by noting the time a tennis ball took to float about 100 ft along the centerline of the stream. During the second site visit, the design team collected elevation data of the accessible streambed channel in order to better generate an idealized channel and floodplain cross-section of Penguin Creek.

PROPOSED LOCATIONS

SITE LOCATION A

One of the possible sites for the new bridge is the location of the original. The old bridge spanned approximately 70 ft, however due to bank erosion the new bridge would span between 90 and 110 ft. This span would be the longest of all four locations. The benefits of building at this location are that the cost of trail creation would be zero; park users are more likely to use the structure since it is located at the current point of crossing, and potential erosion is minimized by placing the bridge along a straight portion of the creek. Currently the trail leads directly to this location therefore only minor clearing and trail creation would need to be accomplished. Moreover, the locals are used to traveling this exact route and would not have to learn a new one. This would ensure usage of the bridge; making the construction purposeful and effective.

The challenges with this location are the span distance and the chance of another wash out. A minimum span of 90 feet introduces higher construction costs and a more involved design process. Since Penguin Creek is an anadromous stream no piers are allowed to be placed within the waters creating the requirement of a single span bridge. More detail would need to be gathered in order to properly design the long span within budget. Another obstacle is the west side bridge abutment would be placed directly within the floodplain. Of all the sites considered this is the only one where one of the banks is below the high water elevation. Thus this location would require fill to raise the existing ground.

SITE LOCATION B

The second location is approximately 350 feet upriver from Site A. At the time of our site visit excessive glaciation had caused braiding of the river requiring two separate bridge spans. The two spans would be 22 and 26 feet in length. One of the advantages to this site is that two smaller bridge spans are less expensive to construct. Besides the smaller bridge span other advantages include the higher ground on the near and far banks, increased channel depth, and a heavily vegetated island between the channels. Increasing the elevation of the abutments in relationship to the creek decreases the likely hood of bridge scour or abutment failure. Furthermore, increased channel depth and flow depth translates to lower water velocities further decreasing the chance of scour and erosion. The island between the two channels is heavily vegetated with trees and grass indicating a stable environment in regards to erosion.

The disadvantages with this location are steep slopes along the near side bank, and the distance from the current creek crossing. This upstream location introduces the risk that the park users may not travel the extra distance prior to crossing the creek. The greatest obstacle of Site B is that the slope on the near side bank is approximately 45 degrees, or 1:1. Because the site location is upstream, approximately 350 feet of trail would need to be created, and the minimum slope allowable according to AKDOT regulations are 2:1 necessitating slope stabilization efforts which would greatly increase the cost of construction.

SITE LOCATION C

Location C is the furthest location from the trail head at approximately 1500 feet upstream from Site A. This site, like location B, has an island in the middle which means there will be two bridges spanning smaller lengths. The two spans would be 35 ft and 24 ft as measured far side to near side of the creek. A few advantages to this location are the higher elevations, vegetation on the island, and a slow creek flow. The banks along Site C are steeper and higher up from the surface of the water. The higher ground creates a greater factor of safety against the bridge abutment scour. Like location B, there is vegetation on the island which increases the erosion stability. Moreover, the velocity of the water is slower. The slower velocity will cause less erosion to the stream bank.

The greatest disadvantage to this site is the location. The location is a problem for two reasons. First, this site is the furthest from the original bridge. With the location being so far upstream, there is a risk park users will not want to travel the increased distance to the bridge, and will continue to cross the stream at Site A. If this were the case, the construction of a new bridge would not serve the purpose of removing people from the stream. Another disadvantage with this location is that it is within a visible avalanche path. An avalanche could destroy the bridge or cause it to become inaccessible. Another disadvantage to this site is that the current trail terminates roughly after the original bridge. In addition to the avalanche danger, approximately 700 feet of new trail would have to be created to provide access for park users, greatly increasing the cost of construction.

SITE LOCATION D

Location D is approximately 120 feet downstream of Site A. The bridge would be a single span of 85 feet. The advantages to this site are: high ground, the shortest single span, higher depths of flow, and the site is visible from the existing trail. The average bank height from surface of water is approximately 5 feet providing for a freeboard of approximately 4 feet during high water events. The site would only require one single span bridge of 85 feet offering the lowest structure cost of all four sites. Deeper flow depths mean lower velocities which decrease the susceptibility of erosion. Additionally, the site is visible from the existing trail ensuring that park users will see and use the bridge.

The disadvantages to Site D include high trail creation costs and bank erosion susceptibility. The site would require approximately 120 feet of new trail creation within a limited amount of space. Adjacent to the existing trail there is a 20 foot elevation drop which would require large quantities of material to fill during trail creation. The site location is also at the beginning of a bend in the creek which could be more susceptible to erosion during high water events.

HYDROLOGY AND HYDRAULICS ANALYSIS

EVALUATING PEAK FLOW

An important consideration for site selection and bridge design is the peak flow during flood events. The USGS has developed a set of regression equations to evaluate peak flow for 5, 10, 25, 100, and 500 year flood events for un-gaged streams in Alaska. Q_{100} is an essential design parameter as the bridge is designed to withstand a 100 year flood. The USGS method assigns regression equations based on topographic zones. Penguin Creek is located in zone 4. The parameters needed to determine Q_{100} are drainage area, storage area, and mean annual precipitation. Drainage area for Penguin Creek was determined using Google Maps as 12.4 square miles. Mean annual precipitation was determined using the Water-Resources Investigations Report 93-4179 Plate 2. Girdwood also has a weather station which records mean annual precipitation but because precipitation tapers off from Girdwood to Anchorage the Water-Resources Investigations Report data was ideal. Storage area was estimated using Google Maps to be 1 percent of the total drainage area. Q_{100} was calculated to be 1200 cubic feet per second. See Appendix A for detailed calculations of the 100 year flood.

DATA COLLECTION

In order to determine appropriate channel assumptions for Penguin Creek a cross-section of the creek was measured using standard surveying equipment. The following cross-sectional data was collected every 25 feet for a total distance of 225 feet. At the near, middle and far side of the available stream, depth of flow and streambed elevations were recorded. The width of the stream was also recorded.

CHANNEL ASSUMPTIONS AND CALCULATIONS

The dimensions of the channel were idealized from the survey data. A trapezoidal channel with side slopes of 1:5 and a bottom width of 10 feet was used (Appendix C). A roughness coefficient of .060 was used to represent a rough natural channel with excessive overgrowth and obstructions. The slope of the channel was averaged using Google Earth to be 5%. Using

Manning's equation (Appendix C) the velocity and depth of flow were determined for both current conditions and 100 year peak flow conditions. The depth of flow calculated using Manning's equation was compared to the observed depth of flow measured during data collection and the percent difference was found to be 16 percent. This affirmed proper channel assumptions had been made. The depth of flow calculated for a 100 year flood event was 3.1 feet with an overflow of 1 inch into the floodplain. Since the banks on both sides of the bridge location are over 5 feet the bridge should be capable of withstanding a 100 year flood.

POTENTIAL SCOURING

Scouring can cause erosion around the banks of the bridge. Scouring was evaluated for both current stream conditions and conditions during a 100 year flood using equation from the FHWA Evaluating Scour at Bridges manual (Appendix C). The calculations showed that in current conditions particles of .5 inches and smaller will be carried downstream. This matched field observations in which only large stones were present in the streambed. During a 100 year flood particles 4.56 inches in diameter will be transported.

FOUNDATION CONSIDERATIONS

SOIL ASSUMPTIONS AND CALCULATIONS

After looking at soil conditions near the chosen site location, it is assumed the first 2 feet are silty/organic soils, with the remaining depth consisting of gravel. The glacial till or gravel will increase in grain size with depth. It is also assumed that the water table will be at 2 feet beneath the ground surface and an active permafrost zone 12 feet beneath the ground surface. According to *Principles of Geotechnical Engineering 7th Ed.* by Braja M. Das, the dry unit weight of glacial till is 134 lb/ft^2. According to the USCS, gravel has a conservative soil friction angle of 30˚. The diameter of the pipe is assumed to be 1 feet in diameter. For additional soil properties and calculations for negative skin friction, see Appendix D.

HELICAL PIERS

It is ideal to use helical piers when possible because of its many advantages. Helical piers are quick to install, are immediately loaded, have small installation equipment, are environmentally friendly, and can install in any weather. The ultimate capacity for this type of foundation can be as high as 200 kips in tension and 300 kips in compression (Fdncon, 2002). Helical piers are also very cost effective. Had the chosen site contained suitable soils, helical piers would have cost approximately $400/pile with installation.

Helical piers have limitations, primarily in soils that contain gravel and cobble, where the helical anchors cannot be drilled property into the ground. Upon further investigation, helical piers would not work at the Penguin Creek Crossing bridge site because the soil is assumed to be gravely for the entire pile length.

MICRO PILES

Micropiles are also cost effective. According to the FHWA Manual on Micropile Design and Construction Guidelines, the contract bid price range for micropiles in the United States is typically $46 - $92 per linear foot of pile. This type of foundation can be installed in any soil condition. Like helical piers, micropiles are environmentally friendly; they disturb minimal soil during installation and can be installed in access-restrictive areas with lightweight equipment. If necessary, micropiles can be installed in any angle below the horizontal. The structural capacity is obtained through high-capacity steel elements. The grout transfers the entire load through skin friction from the steel reinforcement to the ground in the bond zone. Micropiles are ideal at the Penguin Creek Crossing bridge site because of the reasons mentioned above.

Using the formula for granular soils within the water table by Das, the negative skin friction or frost heave is calculated to be 13 kips. Typically, the controlling design factor for micropiles, is the axial compression capacity. In order to calculate the axial capacity of micropiles, the cased length, the uncased length, and the geotechnical bond strength must be considered. For the cased portion of the micropile, the compressive strength of the grout, yield strength of the weaker steel (casing or rebar), and the areas of the bar and casing are considered. For the uncased portion of the micropile, only the compressive strength of the grout, yield strength of the rebar, and the area of the bar are considered. Finally, for the geotechnical bond strength, the grout-to-ground bond nominal strength, bond diameter (steel casing diameter + typical 2”), and the bond length are considered.

According to the Federal Highway Administration Manual on Micropile Design and Construction Guidelines, a grout-to-ground bond nominal strength is between 95-310 kPa for glacial till with a pressure grout (Type B). A design load of 111 kips/pile is obtained by dividing the overall loading caused by the superstructure by 4 piles. The allowable axial compression capacities for the upper cased portion, uncased portion, and the geotechnical bond portion, were 125, 280, and 117 kips respectively. These capacities are all greater than the design load of 111 kips; therefore, the micropile design works for the given loading. For in-depth micropile assumptions and calculations, see Appendix E.

The micropile will be 25 feet long. The standard continuous thread hollow bar anchor for the micropile length is 10 feet long. Two bars 10 feet length and a third bar 5 feet in length will be connected with a coupling system by Contec Systems. The top 6 feet will be encased within a 10 ¾ O.D. inch hollow steel pipe with 0.435 inch wall thickness with the remaining 19 feet being uncased. A continuous thread hollow bar anchor with a 2.047/1.024 O.D./I.D. will be at the center of the hollow steel pipe running along the entire length of the micropile. Grout will be pumped to fill any remaining space. All micropiles will be pressure grouted (Type B).

The pile ends will then be capped with a steel column cap then secured with hex nuts. Finally, the top of the column cap will be welded onto the bridge superstructure providing a fixed support.

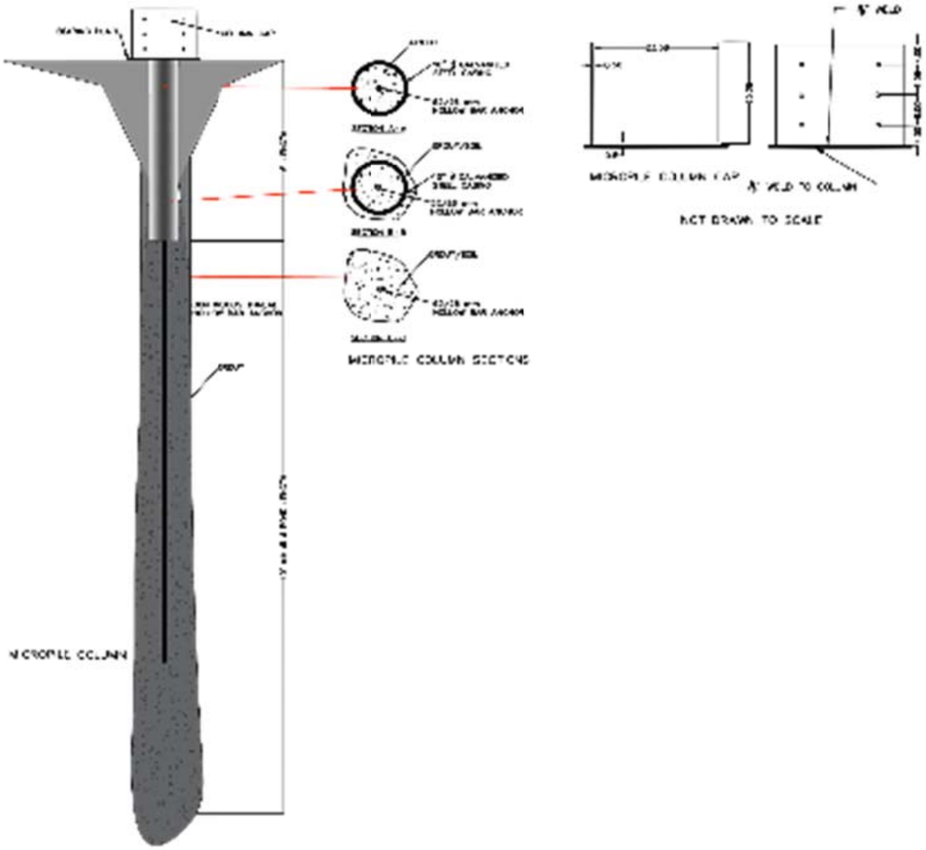

Figure 2: Micropile Columns and Sections

BRIDGE ALTERNATIVES

IN-HOUSE DESIGN

An in house design would consist of UAA SeaWolf Engineering preparing a custom design for the bridge. The components would include steel girders and transverse beams, a timber underlayment, timber decking, and a mesh hand-rail system. Aside from the architectural components of the bridge, UAA SeaWolf Engineering would perform all calculations and structural analysis on the bridge. Structural analysis would be done using RISA 3D. Hand calculations would be provided for all connections involved in the bridge system. One benefit of an in-house design is that it allows for a myriad of options for the appearance of the final product. Furthermore, it would be easier to supply the materials to the job-site.

One downfall involved with an in house design is the increased material and labor cost. The materials would be purchased separately and therefore several shipping costs would be incurred. Labor costs would also increase as the superstructure would need to be erected on site.

PREFABRICATED

A prefabricated bridge would be a supplied by one of several possible manufacturers. The prefab bridge would consist of a steel bridge superstructure with timber fascia including the deck. The bridge would be shipped to Anchorage in one or two pieces depending on the final determined span of the bridge superstructure. The bridge would be trucked on site and dragged into position using a large loader or dozer. One of the major benefits of a prefabricated bridge is the low cost. The entire structure including shipping to Seattle is approximately $40,000. This number will fluctuate based on the final determined span. Another reason to use a prefab bridge is the low cost of labor and materials. Though transportation of the bridge to the site and placement of the bridge will need to occur steel erection and timber work on site will not be necessary. The only work required after placement would be to secure the superstructure to the foundation.

CONCLUSIONS AND RECOMENDATIONS

In order to diminish the amount of disruption to the streambed and disturbance to the anadromous stream, a bridge needs to be constructed that provides access to the trails adjacent to Penguin Creek. Because the original bridge washed out, hydrology and hydraulics were one of the primary concerns in the project design. After conducting hydrological tests the depth of flow calculated for a 100 year flood was 3.1 feet with an overflow of 1 inch into the floodplain. In order to reduce the probability of a wash out, the location chosen would preferably have a higher stream bank. The micropiles are 25 feet long with the first 6 feet being encased within a 10 ¾ outer diameter steel casing and the next 19 feet uncased with grout in-between the continuous thread hollow bar anchor. The bridge is prefabricated with a span length of 85 feet.

After considering the four different site locations the recommendation is Site D with a prefabricated bridge and micropiles with gabion boxes. Site D requires the least amount of trail creation aside from the original location, has the shortest bridge span requirement, and the highest abutment elevation. Moreover, the location is closest to the original crossing. Therefore, with proper signage there is a high probability that the park users will use the new route and discontinue crossing the creek at site A. Also the ranger, Tom Crockett, stated that building downstream would be beneficial because it is further away from bear country. The shorter span length requires less material, which will reduce the cost of the bridge. This site also has high stream banks which reduce the chance of overflow to the trails and the bridge foundation. A prefabricated bridge is recommended per advice from the client. A prefabricated bridge reduces the inconveniences of constructability and reduces cost. Using an in-house design would require many logistical considerations because of the remote location and limited accessibility to the forested terrain. Micropiles were chosen due to their convenience of implementation, their ability to penetrate through glacial till, and provide sufficient allowable axial compression capacity. The accessibility to the site location inhibits the use of large equipment. However, micropiles can be installed with small drill rigs. Also, micropiles can be augured during the winter season. Working during the winter season minimizes impacts to the anadromous stream and existing vegetation. The alternative chosen decreases bridge and trail costs, is most convenient for park users, is further away from avalanche and bear zones, and considers the hydrological concerns.

In conclusion, it is recommended to go with site D, the location closest to the original site location. A prefabricated bridge is also recommended because of its cost savings over an in-house designed bridge. Micropiles are recommended because of its ability to work with the assumed soil conditions and ease of installation. With these recommendations, the total cost of the bridge is estimated to be $183,500.

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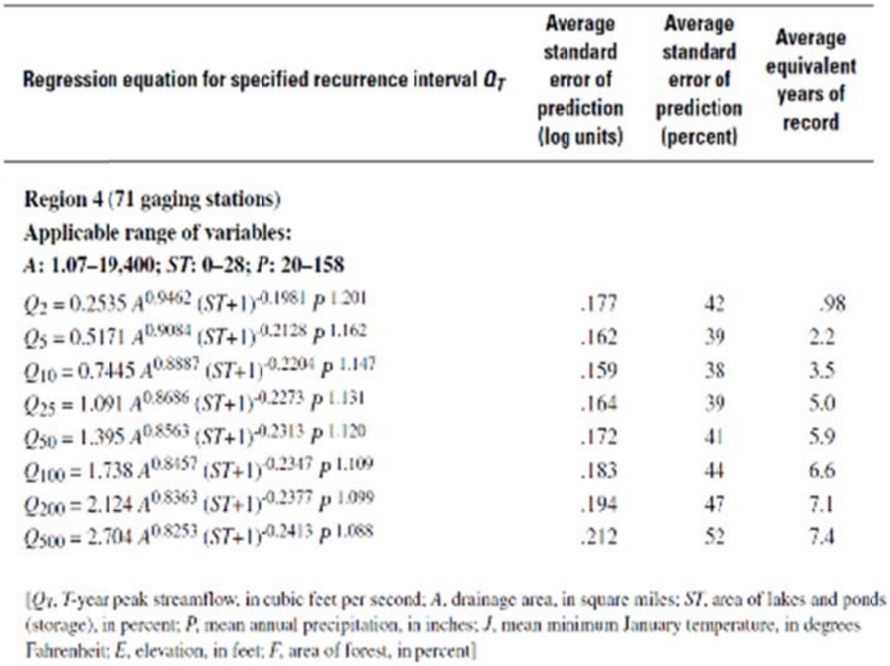
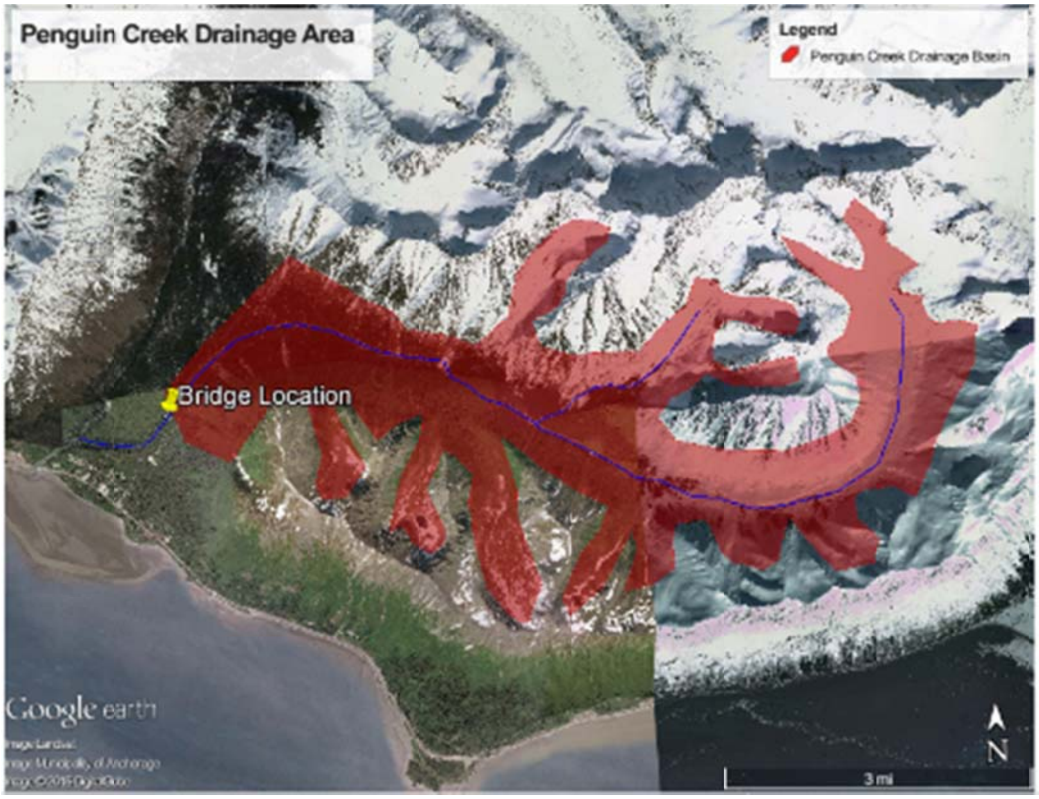
APPENDICES

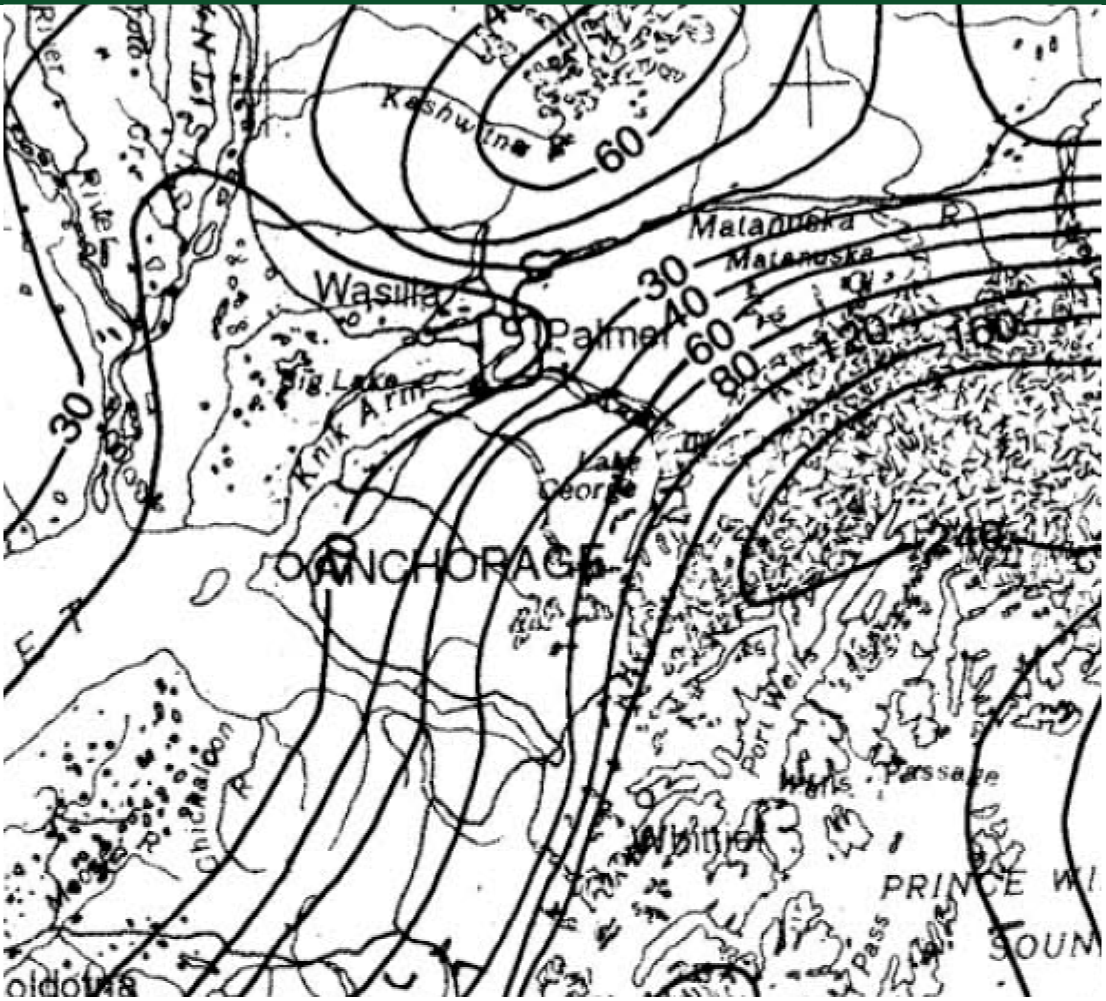
Appendix A: 100 Year Flood



Regression equation for specified recurrence interval Q_T	Average standard error of prediction (log units)	Average standard error of prediction (percent)	Average equivalent years of record
Region 4 (71 gaging stations)			
Applicable range of variables:			
A: 1.07–19,400; ST: 0–28; P: 20–158			
$Q_2 = 0.2535\, A^{0.9462}\, (ST+1)^{-0.1981}\, p^{1.201}$	.177	42	.98
$Q_5 = 0.5171\, A^{0.9084}\, (ST+1)^{-0.2128}\, p^{1.162}$	.162	39	2.2
$Q_{10} = 0.7445\, A^{0.8887}\, (ST+1)^{-0.2204}\, p^{1.147}$	.159	38	3.5
$Q_{25} = 1.091\, A^{0.8686}\, (ST+1)^{-0.2273}\, p^{1.131}$	.164	39	5.0
$Q_{50} = 1.395\, A^{0.8563}\, (ST+1)^{-0.2313}\, p^{1.120}$	.172	41	5.9
$Q_{100} = 1.738\, A^{0.8457}\, (ST+1)^{-0.2347}\, p^{1.109}$	.183	44	6.6
$Q_{200} = 2.124\, A^{0.8363}\, (ST+1)^{-0.2377}\, p^{1.099}$	.194	47	7.1
$Q_{500} = 2.704\, A^{0.8253}\, (ST+1)^{-0.2413}\, p^{1.088}$	.212	52	7.4

[Q_T, T-year peak streamflow, in cubic feet per second; A, drainage area, in square miles; ST, area of lakes and ponds (storage), in percent; P, mean annual precipitation, in inches; J, mean minimum January temperature, in degrees Fahrenheit; E, elevation, in feet; F, area of forest, in percent]




Kashwitna
60
Matanuska
Matanuska
Wasilla
Palmer
30
40
60
80
120
160
Big Lake
Knik Arm
30
Lake George
ANCHORAGE
240
Chickaloon R
Port Wells
Wells Passage
Whittier
PRINCE W
SOUN

Calculations

A	**12.4**	**Drainage Area, Square Miles**	
ST	**1**	**Storage Area of Lakes & Ponds, %**	
p	**60**	**Mean annual precipitation, inches**	
Q2	**326.9634233**	**Q50**	**1006.404011**
Q5	**511.6716637**	**Q100**	**1164.325711**
Q10	**655.8145184**	**Q200**	**1331 .117711**
Q25	**851.6002348**	**Q500**	**1571.801238**

Appendix B: Channel Assumptions and Calculations

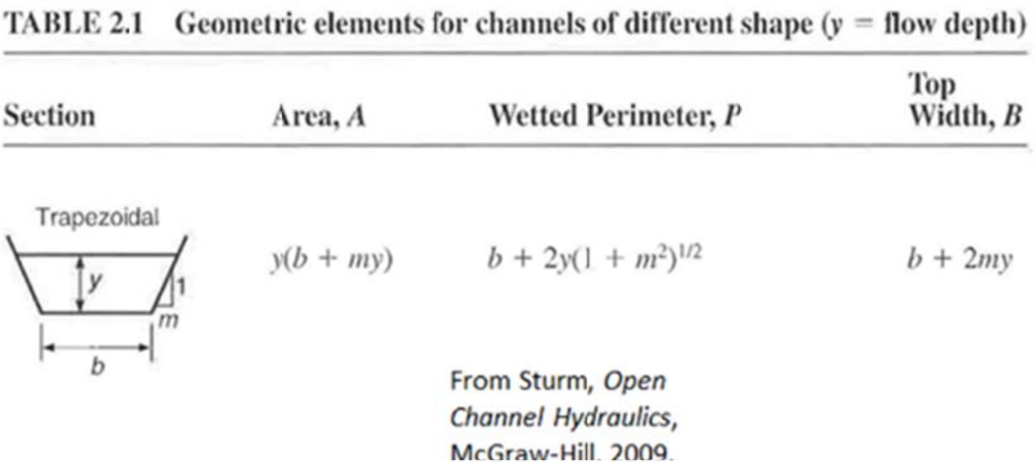
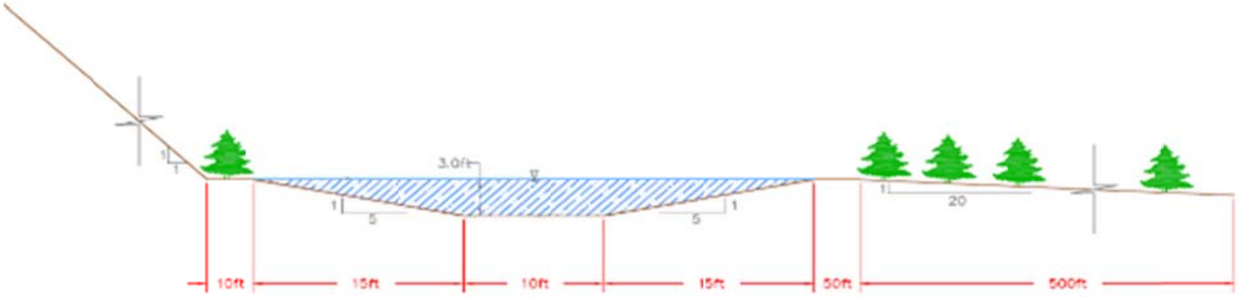
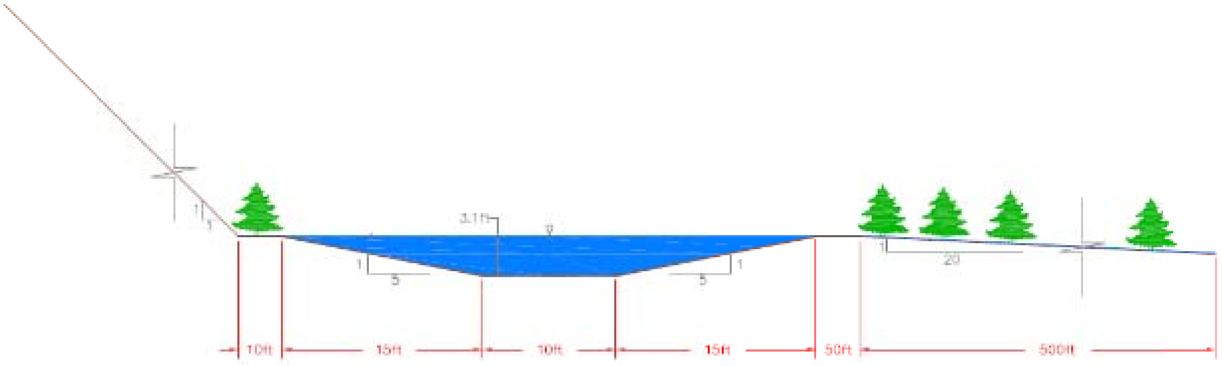


TABLE 2.1 Geometric elements for channels of different shape (y = flow depth)

Section	Area, A	Wetted Perimeter, P	Top Width, B
Trapezoidal	$y(b + my)$	$b + 2y(1 + m^2)^{1/2}$	$b + 2my$

From Sturm, *Open Channel Hydraulics*, McGraw-Hill, 2009.

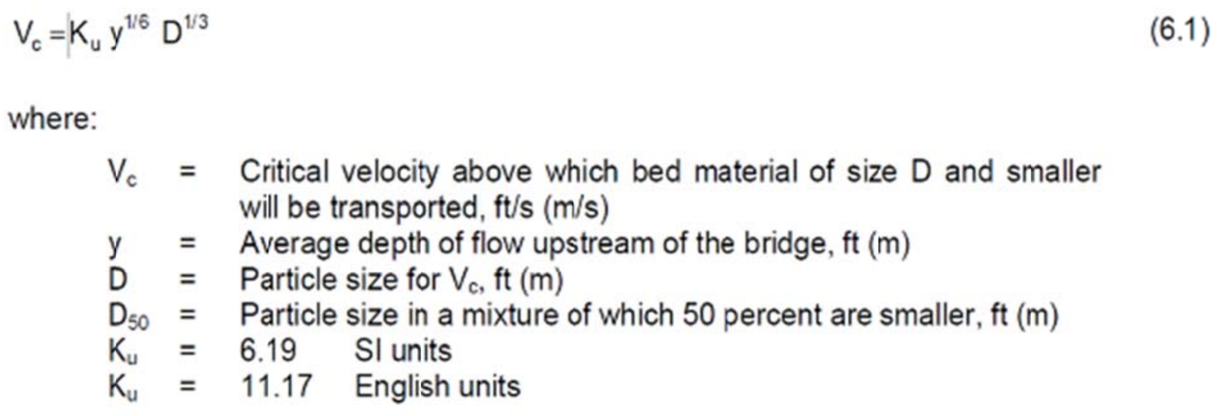




Distance: 100 feet					
T1	28.22	s	V1	3.54	ft/s
T2	27.45	s	V2	3.64	ft/s
T3	28.52	s	V3	3.51	ft/s
Vavg				3.56	ft/s

Trapezoidal Channel					
Normal Flow Depth from January 2015 Site Visit			**100 Year Flood Numbers**		
b	10	ft	b	10	ft
m	5		m	5	
y	0.65	ft	y	3.98	ft
S	0.05	ft/ft	S	0.05	ft/ft
n	0.060		n	0.060	
K_n	1.486		K_n	1.486	
A	8.55	ft^2	A	118.90	ft^2
P	16.59	ft	P	50.57	ft
B	16.46		B	49.78	
R	0.52	ft	R	2.35	ft
T	16.46	ft	T	49.78	ft
D	0.52	ft	D	2.39	ft
V	3.56	ft/s	V	9.79	ft/s
Q	30.4	ft^3/s	Q	1164.3	ft^3/s
Fraud Number from January Site Visit			**Fraud Number: 100 Year Flood**		
Fr	0.87		Fr	1.12	
Flow Type	Sub-Critical Flow		Flow Type	Super-Critical Flow	
Critical Depth: y_c			**Critical Depth: y_c**		
y_c	0.59	ft	y_c	4.19	ft
g	32.20	ft/s^2	g	32.20	ft/s^2
Q	30.4	ft^3/s	Q	1164.3	ft^3/s
A	7.7	ft^2	A	129.8	ft^2
V	3.9	ft/s	V	9.0	ft/s
T	15.9	ft	T	51.9	ft
D	0.5	ft	D	2.5	ft
sqrt(gD)	3.9		sqrt(gD)	9.0	
Fr	1.00		Fr	1.00	
Flow Type:	Sub-Critical Flow		Flow Type:	Super-Critical Flow	

Appendix C: Scouring Calculations



$$V_c = K_u\, y^{1/6}\, D^{1/3} \qquad (6.1)$$

where:

V_c = Critical velocity above which bed material of size D and smaller will be transported, ft/s (m/s)
y = Average depth of flow upstream of the bridge, ft (m)
D = Particle size for V_c, ft (m)
D_{50} = Particle size in a mixture of which 50 percent are smaller, ft (m)
K_u = 6.19 SI units
K_u = 11.17 English units

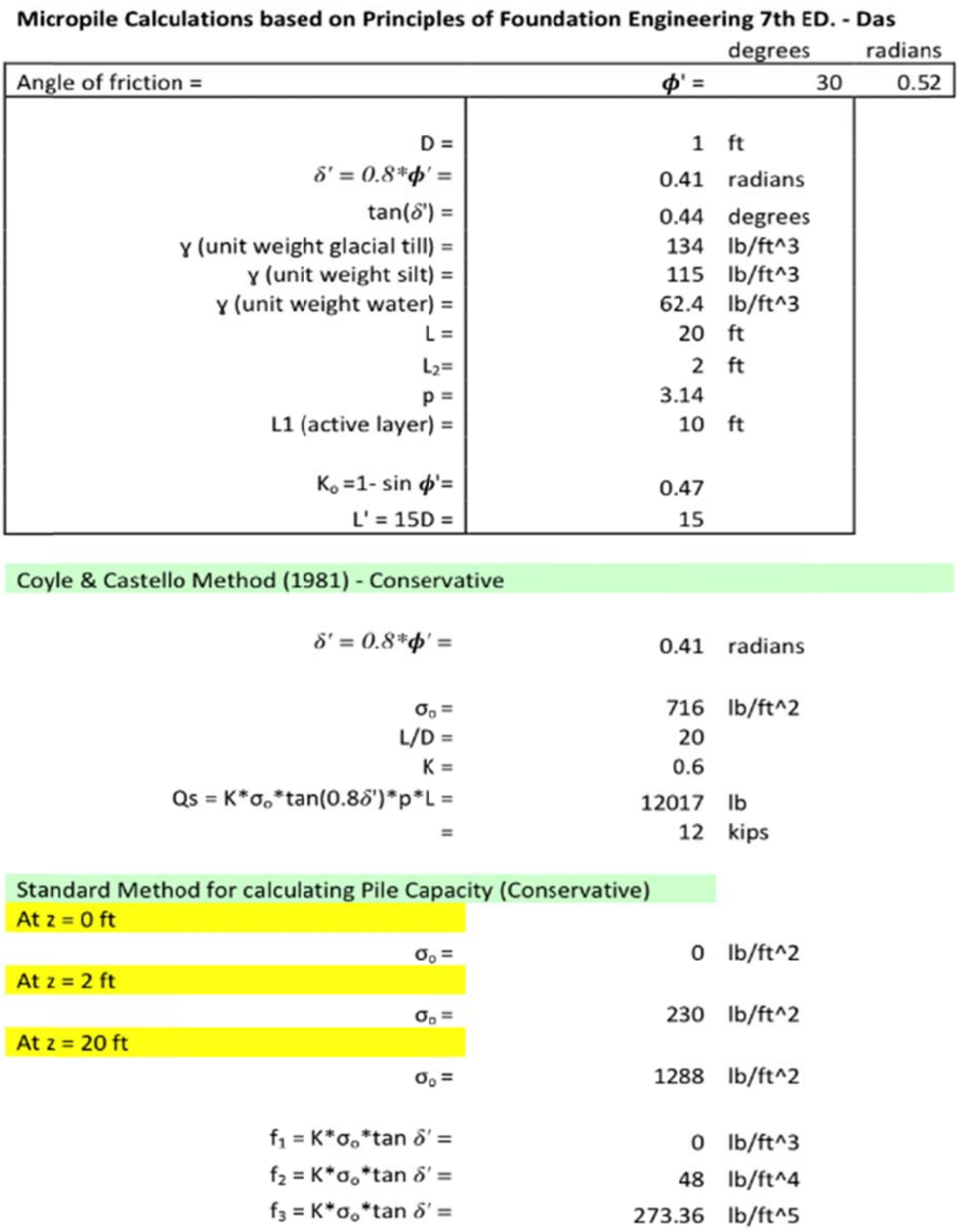
Stream in current conditions

Velocity(ft/s)	Y(ft)	D(ft)	Ku
3.56	0.65	0.04015446	11.17

Stream at 100 year flood

Velocity(ft/s)	Y(ft)	D(ft)	Ku
9.79	3.08	0.38363056	11.17

Appendix D: Uplift Calculation



Micropile Calculations based on Principles of Foundation Engineering 7th ED. - Das

		degrees	radians
Angle of friction =	ϕ' =	30	0.52

D = 1 ft

$\delta' = 0.8*\phi'$ = 0.41 radians

tan(δ') = 0.44 degrees

γ (unit weight glacial till) = 134 lb/ft^3

γ (unit weight silt) = 115 lb/ft^3

γ (unit weight water) = 62.4 lb/ft^3

L = 20 ft

L_2= 2 ft

p = 3.14

L1 (active layer) = 10 ft

K_o =1- sin ϕ'= 0.47

L' = 15D = 15

Coyle & Castello Method (1981) - Conservative

$\delta' = 0.8*\phi'$ = 0.41 radians

σ_o = 716 lb/ft^2

L/D = 20

K = 0.6

Qs = K*σ_o*tan(0.8δ')*p*L = 12017 lb

= 12 kips

Standard Method for calculating Pile Capacity (Conservative)

At z = 0 ft

σ_o = 0 lb/ft^2

At z = 2 ft

σ_o = 230 lb/ft^2

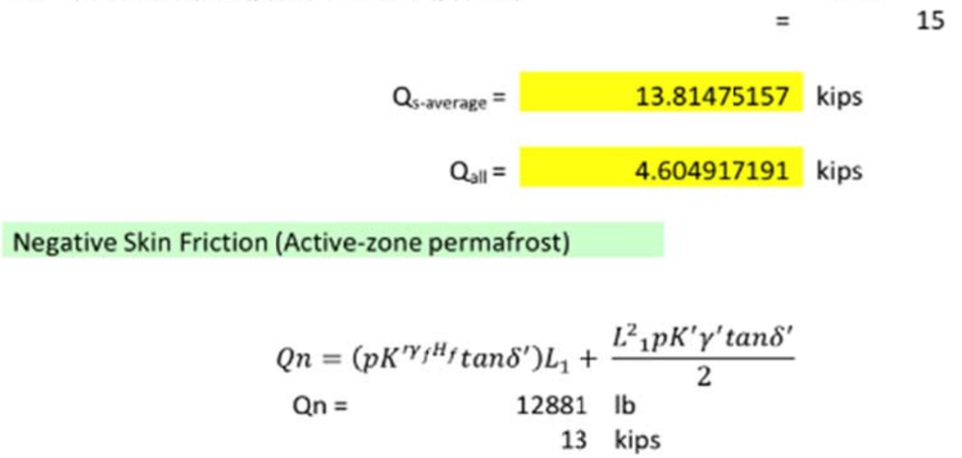
At z = 20 ft

σ_o = 1288 lb/ft^2

f_1 = K*σ_o*tan δ' = 0 lb/ft^3

f_2 = K*σ_o*tan δ' = 48 lb/ft^4

f_3 = K*σ_o*tan δ' = 273.36 lb/ft^5



Qs = $(f_{z=0} + f_{z=2ft})/2*(p)(L_2) + f_{z=20ft}*(p)(L-L_2)$ = 15611

= 15

$Q_{s\text{-average}}$ = 13.81475157 kips

Q_{all} = 4.604917191 kips

Negative Skin Friction (Active-zone permafrost)

$$Qn = (pK'^{\gamma_f}{}^{H_f} tan\delta')L_1 + \frac{L^2{}_1 pK'\gamma' tan\delta'}{2}$$

Qn = 12881 lb

13 kips

Appendix E: Micropile Calculations

Micropile Calculations based on FHWA Micropile Design (SA-97-070)

🟥 Design for Structural Strength Limit States

Axial Compression of Cased Length : The allowable compression load for the cased (free) length is:

$$P_{c-allowable} = \left[\frac{f'_{c-grout}}{FS_{grout}} \times A_{grout} + \frac{F_{y-steel}}{FS_{grout}}\left(A_{bar} + A_{casing}\right)\right] \times \frac{F_a}{\frac{F_{y-steel}}{FS_{y-steel}}}$$

f'c =	5	ksi	Casing_OD=	10.75	in
FS_g =	2.5	FHWA	Casing_ID=	9.88	in
A_g =	125	in^2	Grout_DIA=	12.72	in
$F_{y\text{-steel}}$ =	75	ksi	Q_required_c =	615528	lbs
FS $_{y\text{-steel}}$ =	0.47	FHWA		616	kips
A_{bar} =	1.56	in^2	Q_required_t	13	kips
A_{casing} =	14	in^2			
F_a =	35	ksi	# of piles =	4	

Bond Length is =	19	ft
Length that is cased in steel =	1	ft
Length that is Uncased =	19	ft
Plunge Length =	2.17	ft

Assumptions:
11.75" micropile diameter
#11 bar reinforcement, 75 ksi

$P_{c\text{-allowable}}$ = 160 kips

Allowable Tension of Cased Length

$$P_{t-allowable} = 0.55\,F_{y-steel} \times \left(A_{bar} + A_{casing}\right)$$

$P_{t\text{-allowable}}$ = 646

Axial Compression and Tension of Uncased Length

$$P_{c-allowable} = \left[0.4\, f'_{c-grout} \times A_{grout} + 0.47 \times F_{y-bar} \times A_{bar}\right]$$

$P_{c\text{-allowable}}$ = 306 kips

$$P_{t-allowable} = 0.55\, F_{y-bar} \times A_{bar}$$

$P_{t\text{-allowable}}$ = 64 kips

Ultimate Structural Capacity

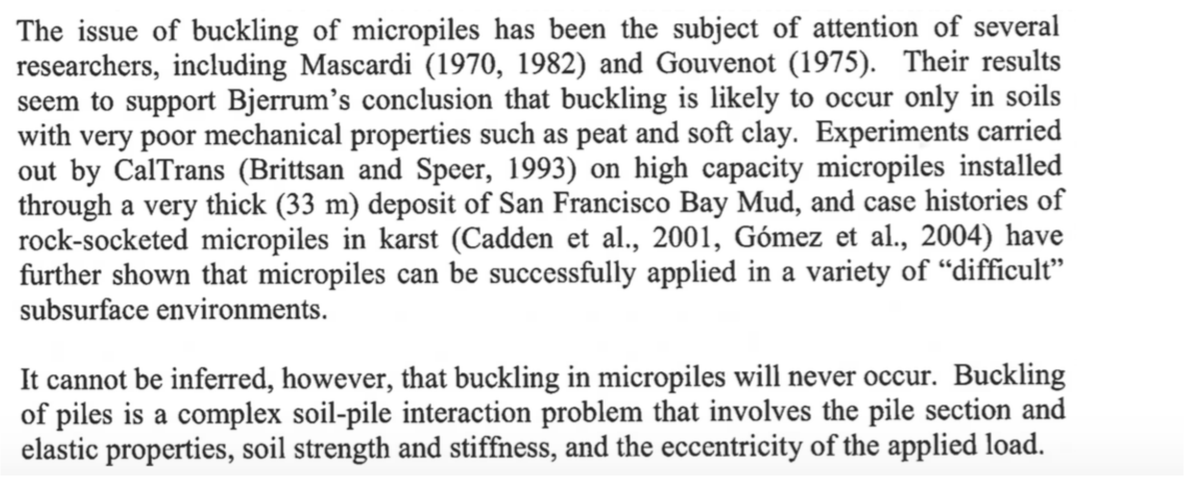
$$P_{ult-compression} = \left[0.85\, f'_{c-grout} \times A_{grout} + f_{y-casing} \times A_{casing} + f_{y-bar} \times A_{bar}\right]$$

$P_{ult\text{-compression}}$ = 1708 kips

$$P_{ult-tension} = \left[f_{y-casing} \times A_{casing} + f_{y-bar} \times A_{bar}\right]$$

$P_{ult\text{-tension}}$ = 1174 kips

Buckling of Micropiles



The issue of buckling of micropiles has been the subject of attention of several researchers, including Mascardi (1970, 1982) and Gouvenot (1975). Their results seem to support Bjerrum's conclusion that buckling is likely to occur only in soils with very poor mechanical properties such as peat and soft clay. Experiments carried out by CalTrans (Brittsan and Speer, 1993) on high capacity micropiles installed through a very thick (33 m) deposit of San Francisco Bay Mud, and case histories of rock-socketed micropiles in karst (Cadden et al., 2001, Gómez et al., 2004) have further shown that micropiles can be successfully applied in a variety of "difficult" subsurface environments.

It cannot be inferred, however, that buckling in micropiles will never occur. Buckling of piles is a complex soil-pile interaction problem that involves the pile section and elastic properties, soil strength and stiffness, and the eccentricity of the applied load.

Allowable Geotechnical Bond Load

$$P_{G\text{-design strength}} = \varphi_G \left[\alpha_{\text{bond nominal strength}} \right] \times 3.14 \times DIA_{\text{bond}} \times (\text{Bond Length})$$

$P_{G\text{-allowable}} =$ 158 kips

Bond Length = 221 in
18 ft
19 ft

$P_{G\text{-allowable}} >$ 160 OK kips

	compression	kips
Structural Upper Cased Length =	160	kips
Structural Lower Uncased Length =	306	
Geotechnical Bond Length =	158	kips

Allowable loads above are all greater than a design load of : 153.882

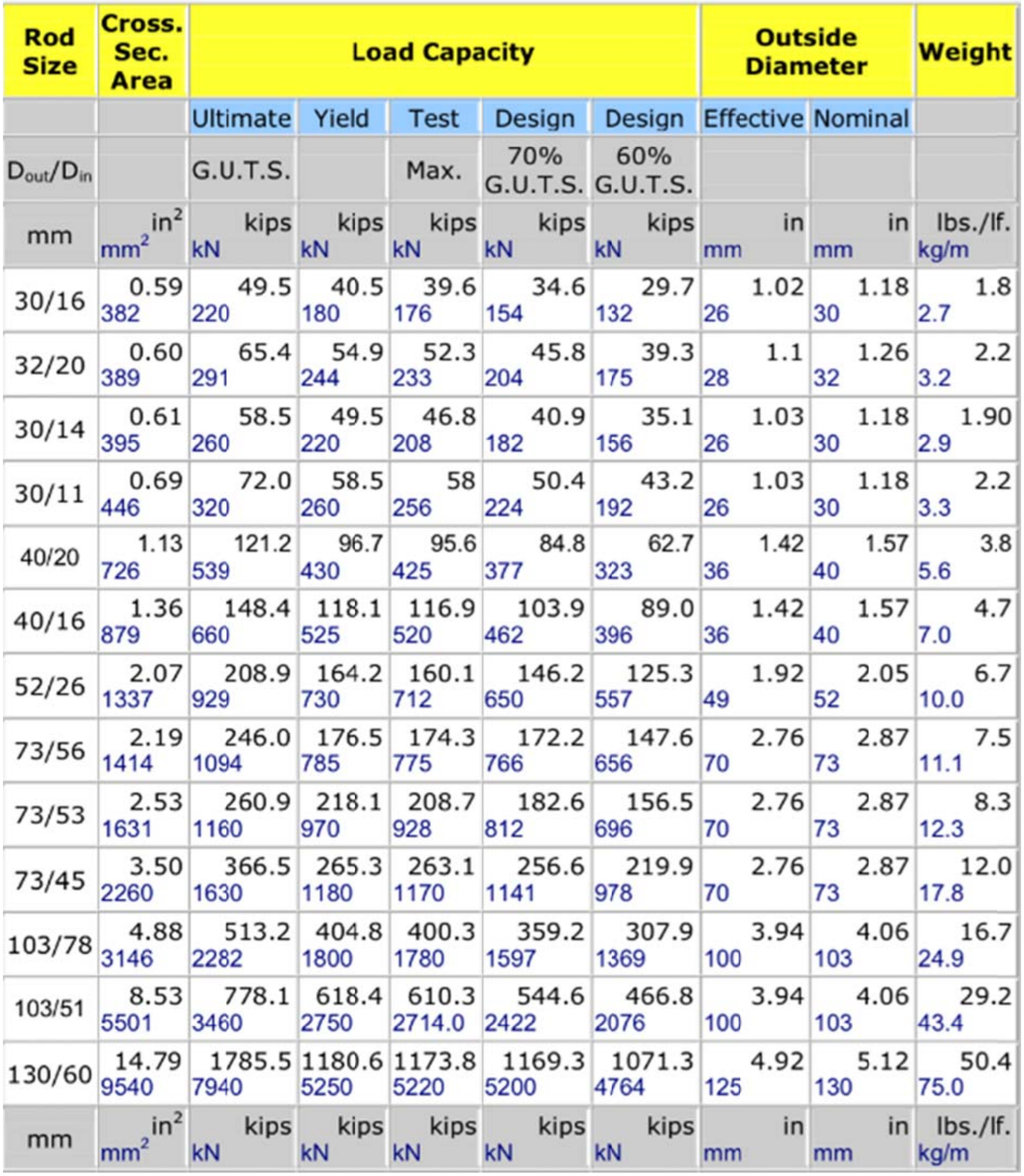
Plunge Length

Assumed Plunge length = 26 in

$P_{G\text{-allowable}} =$ 18 kips

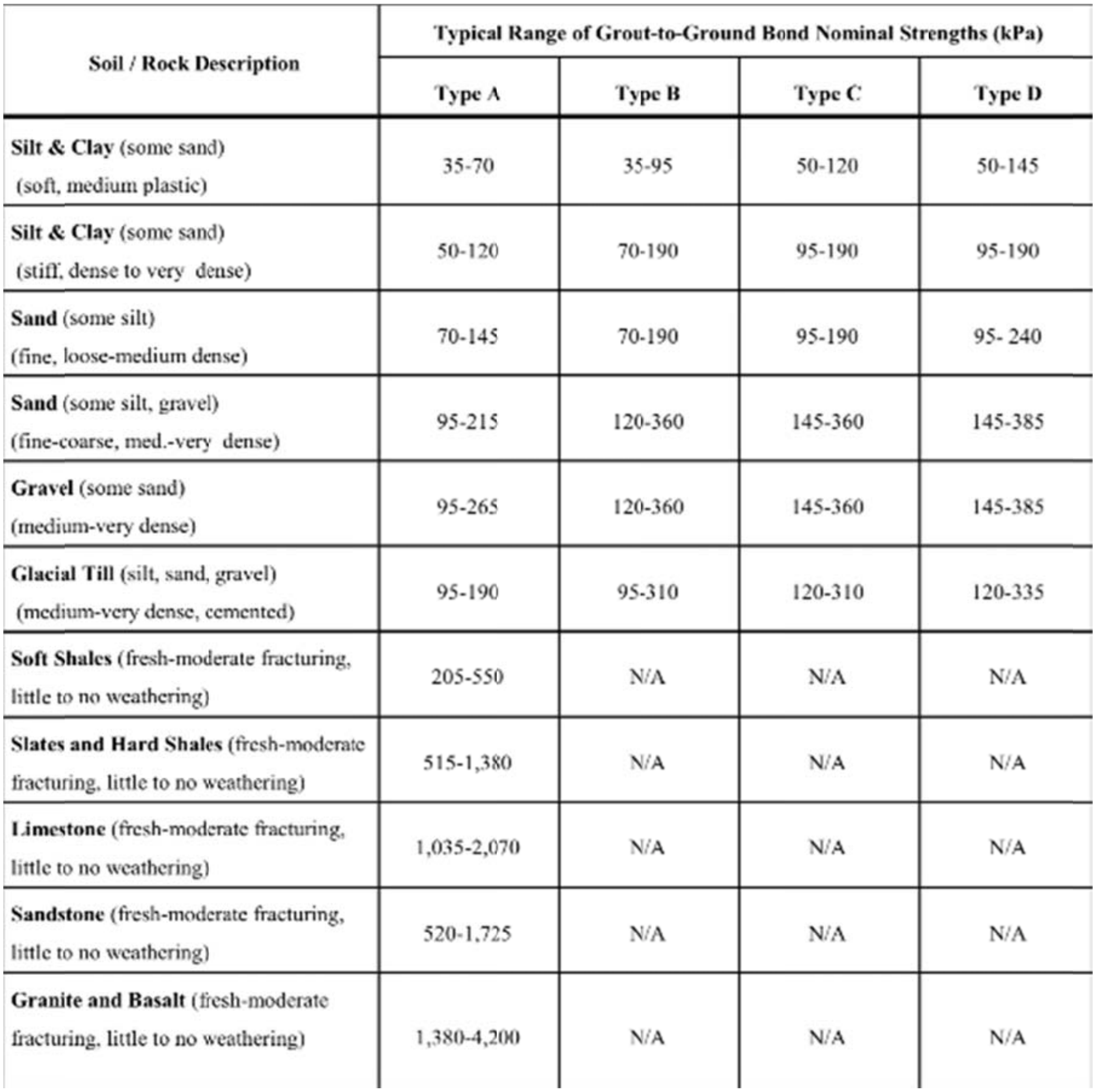
$P_{G\text{-allowable}}$ assumed to be at 50 kN or 11.24 kips.

18.07 kips > 11.24 kips, therefore, 26 inches is OK for plunge length.



Rod Size	Cross. Sec. Area	Load Capacity					Outside Diameter		Weight
		Ultimate	Yield	Test	Design	Design	Effective	Nominal	
D_{out}/D_{in}		G.U.T.S.		Max.	70% G.U.T.S.	60% G.U.T.S.			
mm	in^2 mm^2	kips kN	kips kN	kips kN	kips kN	kips kN	in mm	in mm	lbs./lf. kg/m
30/16	0.59 382	49.5 220	40.5 180	39.6 176	34.6 154	29.7 132	1.02 26	1.18 30	1.8 2.7
32/20	0.60 389	65.4 291	54.9 244	52.3 233	45.8 204	39.3 175	1.1 28	1.26 32	2.2 3.2
30/14	0.61 395	58.5 260	49.5 220	46.8 208	40.9 182	35.1 156	1.03 26	1.18 30	1.90 2.9
30/11	0.69 446	72.0 320	58.5 260	58 256	50.4 224	43.2 192	1.03 26	1.18 30	2.2 3.3
40/20	1.13 726	121.2 539	96.7 430	95.6 425	84.8 377	62.7 323	1.42 36	1.57 40	3.8 5.6
40/16	1.36 879	148.4 660	118.1 525	116.9 520	103.9 462	89.0 396	1.42 36	1.57 40	4.7 7.0
52/26	2.07 1337	208.9 929	164.2 730	160.1 712	146.2 650	125.3 557	1.92 49	2.05 52	6.7 10.0
73/56	2.19 1414	246.0 1094	176.5 785	174.3 775	172.2 766	147.6 656	2.76 70	2.87 73	7.5 11.1
73/53	2.53 1631	260.9 1160	218.1 970	208.7 928	182.6 812	156.5 696	2.76 70	2.87 73	8.3 12.3
73/45	3.50 2260	366.5 1630	265.3 1180	263.1 1170	256.6 1141	219.9 978	2.76 70	2.87 73	12.0 17.8
103/78	4.88 3146	513.2 2282	404.8 1800	400.3 1780	359.2 1597	307.9 1369	3.94 100	4.06 103	16.7 24.9
103/51	8.53 5501	778.1 3460	618.4 2750	610.3 2714.0	544.6 2422	466.8 2076	3.94 100	4.06 103	29.2 43.4
130/60	14.79 9540	1785.5 7940	1180.6 5250	1173.8 5220	1169.3 5200	1071.3 4764	4.92 125	5.12 130	50.4 75.0
mm	in^2 mm^2	kips kN	kips kN	kips kN	kips kN	kips kN	in mm	in mm	lbs./lf. kg/m

CTS/TITAN IBO Hollow Bar Anchors



Soil / Rock Description	Typical Range of Grout-to-Ground Bond Nominal Strengths (kPa)			
	Type A	Type B	Type C	Type D
Silt & Clay (some sand) (soft, medium plastic)	35-70	35-95	50-120	50-145
Silt & Clay (some sand) (stiff, dense to very dense)	50-120	70-190	95-190	95-190
Sand (some silt) (fine, loose-medium dense)	70-145	70-190	95-190	95- 240
Sand (some silt, gravel) (fine-coarse, med.-very dense)	95-215	120-360	145-360	145-385
Gravel (some sand) (medium-very dense)	95-265	120-360	145-360	145-385
Glacial Till (silt, sand, gravel) (medium-very dense, cemented)	95-190	95-310	120-310	120-335
Soft Shales (fresh-moderate fracturing, little to no weathering)	205-550	N/A	N/A	N/A
Slates and Hard Shales (fresh-moderate fracturing, little to no weathering)	515-1,380	N/A	N/A	N/A
Limestone (fresh-moderate fracturing, little to no weathering)	1,035-2,070	N/A	N/A	N/A
Sandstone (fresh-moderate fracturing, little to no weathering)	520-1,725	N/A	N/A	N/A
Granite and Basalt (fresh-moderate fracturing, little to no weathering)	1,380-4,200	N/A	N/A	N/A

Summary of Typical $\alpha_{\text{bond nominal strength}}$ Values (Grout-to-Ground Bond) for Preliminary Micropile Design that have been used in Practice.

Appendix F: Cost Estimate

Spec Section	Item	Quantity	Unit	Unit Price	Amount
201(3)	Clearing & Grubbing	All Req'd	Lump Sum	$ 10,000.	$ 10,000.
203	Excavation and Backfill	115	Cubic Yard	$ 5.	$ 575.
203	Borrow Type A	40	Cubic Yard	$ 40.	$ 1,600.
505	Furnish & Install Micropiles	4	Each	$ 3,000.	$ 12,000.
515	Pre-Manufactured Bridge, 85 Feet	All Req'd	Lump Sum	$ 51,400.	$ 51,400.
615	Furnish Standard Signage	All Req'd	Lump Sum	$ 1,500.	$ 1,500.
618	Furnish Seeding	0.1	Acre	$ 5,000.	$ 500.
620	Furnish topsoil	580	Square Yard	$ 5.	$ 2,900.
640(1)	Mobilization & Demobilization	All Req'd	Lump Sum	$ 12,000.	$ 12,000.
641(2)	Temporary Erosion, Sediment & Polution Control	All Req'd	Lump Sum	$ 2,500.	$ 2,500.
641(6)	ESCP Control Administration	All Req'd	Lump Sum	$ 5,000.	$ 5,000.
642(1)	Construction Surveying	All Req'd	Lump Sum	$ 5,000.	$ 5,000.
642(2)	Three Person Survey Party	8	Hour	$ 250.	$ 2,000.
650(21)	Barrier Rock	10	Each	$ 225.	$ 2,250.
-	Design Cost	All Req'd	Lump Sum	$ 16,50.	$ 16,50.
-	Contingency Fund	All Req'd	Lump Sum	$ 11,00.	$ 11,00.
-	Total Project Cost	-	-	$ 136,725.	**$ 136,725.**