TRANSMITTAL FORM

ADDENDUM TO THE CONTRACT DOCUMENTS	Page Number 1	No. of Pages 97
Addendum No. One (1)	**Date Addendum Issued**: January 26, 2021	
Issuing Office Alaska State Parks, Design & Construction Section 550 West 7th Avenue, Suite 1340 Anchorage, Alaska 99501 Phone: (907) 269-8731 Fax: (907) 269-8917	**Previous Addenda Issued** None.	
Project: Eagle Rock KRSMA Caretaker Cabin **Project No.**: 78036-5	**Date & Hour of Quotes Due**: February 3, 2021 2:00 PM prevailing time	

NOTICE TO BIDDERS

Bidder must acknowledge receipt of this addendum prior to the hour and date set for the quotes being due by one of the following methods:

(a) By acknowledging receipt of this addendum on the quote submitted.
(b) By telegram or telefacsimile which includes a reference to the project and addendum number.

The bid documents require acknowledgment individually of all addenda to the drawings and/or specifications. This is a mandatory requirement and any quote received without acknowledgment of receipt of addenda may be classified as not being a responsive bid. If, by virtue of this addendum it is desired to modify a quote already submitted, such modification may be made by telegram or telefacsimile provided such a telegram or telefacsimile makes reference to this addendum and is received prior to the opening hour and date specified above.
****

The Bid Documents are modified as follows:

Replace **Table of Contents** with the modified **Table of Contents** (Addendum No. 1 – Attachment A)

The Technical Specifications are modified as follows:

Add Appendix F – Special Reports *to the Technical Specifications.* (Addendum No. 1 – Attachment B)

Bidders are required to acknowledge this addendum on the proposal form or by FAX prior to the quotes being due.

****
Addendum Number One (1) received.

____
Name/Title Date

____
Firm

END OF ADDENDUM

TABLE OF CONTENTS

(State Funded)

APPENDIX F
SPECIAL REPORTS

March 2, 2017 NGE-TFT Project # 4597-16(A)

Alaska Department of Natural Resources
Division of Parks and Outdoor Recreation
550 W. 7th Avenue, Suite 300
Anchorage, Alaska 99501

Attn: Jacob Gondek, P.E.

RE: GEOTECHNICAL ASSESSMENT OF THE PROPOSED IMPROVEMENTS TO THE EAGLE ROCK BOAT LAUNCH IN KENAI, ALASKA (AK DNR PROJECT #78036-1), ADDENDUM #1: UPDATED PILE FOUNDATION RECOMMENDATIONS

Jacob,

We (Northern Geotechnical Engineering, Inc. *d.b.a.* Terra Firma Testing) are pleased to present this Addendum to our geotechnical report for AK DNR Project # 78035-1. In our original geotechnical report, we indicated that we would be able to provide updated pile foundation recommendations after we were provided updated loading conditions for the planned pile foundations. The load information presented in Table 1 of this Addendum was provided to us by Roxanne Risse with the Division of Parks & Outdoor Recreation (DPOR).

Table 1: Pile Loading Conditions

FOUNDATION I.D.	MAXIMUM PILE STICKUP (FT)	AXIAL LOAD (KIPS)	LATERAL LOAD (KIPS)
ELEVATED WALKWAY PILES	2.5	6	0.3
FLOATING DOCK PILES	10	-	12

Roxanne indicated that DPOR would prefer to use 6-in pipe piles for the elevated walkway and 8-in pipe piles for the floating dock. The loads and preferred pile foundation for the elevated walkways fall within the range of acceptable loads as presented in our original geotechnical report. No additional analysis is required. However, because of the magnitude of the lateral load and pile stickup, the floating dock piles required additional analysis.

Based on our analysis, which was conducted using the computer program AllPile produced by CivilTech Software, an 8-in pile with 10 feet of stickup will experience excessive deflection under the design lateral load of 12 kips. We used AllPile to calculate the deflection of multiple pile sizes with an ultimate lateral load of 12 kips acting at 10 feet above the mudline. Table 2 of this addendum presents the results of our analysis.

Table 2: Calculated Deflections for a 12-kip Lateral Load 10 feet Above the Mudline

NOMINAL PILE DIAMETER (INCHES)	8	10	12	16	18	20	24
NOMINAL WALL THICKNESS (INCHES)	0.5	0.5	0.5	0.5	0.5	0.5	0.5
DEFLECTION (INCHES)	17.2	8.7	5.3	2.8	2.1	1.6	1.0

We are not in a position to recommend which pile size the DPOR should choose for the floating dock. However, we have provided the data in Table 2 of this addendum so that the DPOR may work with their designers to determine which pile size is appropriate based on the allowable deflection of the floating dock system.

We conducted this evaluation following the standard of care expected of professional undertaking similar work in the State of Alaska under similar conditions. No warranty, expressed or implied, is made.

Sincerely,
Northern Geotechnical Engineering, Inc. *d.b.a.* Terra Firma Testing,

Cody J. Kreitel, P.E.
Senior Project Engineer

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FINAL GEOTECHNICAL REPORT
for the proposed
EAGLE ROCK BOAT LAUNCH IMPROVEMENTS
KENAI, ALASKA
part of the
KENAI AREA GEOTECHNICAL CONTRACT
(AK DNR Project #78036-1)

Prepared for:

Alaska Department of Natural Resources
Division of Parks and Outdoor Recreation
550 West 7th Avenue, Suite 1380
Anchorage, Alaska 99501

Prepared by:

Northern Geotechnical Engineering, Inc. *d.b.a.* Terra Firma Testing

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JANUARY 2017

January 17, 2017 NGE-TFT Project # 4597-16(A)

Alaska Department of Natural Resources
Division of Parks and Outdoor Recreation
550 W. 7th Avenue, Suite 300
Anchorage, Alaska 99501

Attn: Jacob Gondek, P.E.

RE: GEOTECHNICAL ASSESSMENT OF THE PROPOSED IMPROVEMENTS TO THE EAGLE ROCK BOAT LAUNCH IN KENAI, ALASKA (AK DNR PROJECT #78036-1)

Jacob,

We (Northern Geotechnical Engineering, Inc. *d.b.a.* Terra Firma Testing) have completed geotechnical engineering assessment of the aforementioned project. Our assessment suggests that the project site is suitable for the proposed improvements provided that the conclusions and recommendations that we present in the following report are considered during the design and construction process.

All of our explorations encountered layers or peat and organic soils to depths of approximately 6 to 10 feet below the ground surface. Only one of the boreholes, KENB2, was advanced through the existing gravel fill pad. KENB2 revealed approximately five feet of granular fill above the native peat soils. We cannot be certain if this depth of fill is representative of the entire existing fill pad. In the following report, we present our recommendations for pavement sections above the organic soils. Because there were no explorations advanced within the existing gravel fill pad, it will be the responsibility of the owner to determine if the existing fill pad meets or exceeds the recommendations presented in this report.

While pavement sections may be "floated" above the peat and organic soils, the organics are not suitable for supporting traditional shallow foundations. Excavation of the peat and replacement with structural fill will be required for any shallow foundations. Alternatively, deep foundation systems, such as driven steel piling or helical piers, are a suitable foundation option that will not require the excavation and replacement of organic soils. We provide recommendations for both traditional shallow concrete foundations and deep foundations in the following report.

With the existence of peat soils to depths of approximately 10 feet bgs, it is impractical to design a boat ramp section/surface that will prevent/resist all of the sources for potential ground movements. Therefore, the boat ramp surface design should be modular (i.e., a series of interconnected concrete planks, pads, mats, etc.), so that individual boat ramp surface modules (BRSMs) are not rigidly connected to one another. This will allow for some movement to occur beneath individual BRSMs without impacting adjacent BRSMs and can allow for localized

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maintenance/repairs to damaged/displaced BRSMs without impacting adjacent BRSMs. We provide additional details regarding the recommended design of the boat ramp in the following report.

We greatly appreciate the opportunity to provide you with our professional service. Please contact us directly with any questions or comments you may have regarding the information that we present in this report, or if you have any other questions, comments, and/or requests.

Sincerely,
Northern Geotechnical Engineering, Inc. *d.b.a.* Terra Firma Testing,

Cody J. Kreitel, P.E.
Senior Project Engineer

Keith F. Mobley, P.E.
President

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Table of Contents

List of Figures

List of Tables

List of Appendices

1.0 INTRODUCTION

In this report, we (Northern Geotechnical Engineering, Inc. *d.b.a.* Terra Firma Testing) present the results of our geotechnical assessment, conducted at the Eagle Rock Boat Launch (ERBL) located on the north bank of the Kenai River at the end of Eagle Rock Place in Kenai, Alaska.; hereafter referred to solely as "the project site". We provided our professional service in accordance with the scope of service that we detail in our response to Request for Proposals (RFP) #78036-1 issued by the Alaska Department of Natural Resources (AKDNR) Division of Parks and Outdoor Recreation (DPOR) on October 20, 2016. We submitted our response to RFP #78036-1 to the DPOR on September 1, 2016 and we received a notice to proceed from the DPOR (Agreement No: 78036-1) on December 2, 2016. This report does not address the Old Kasilof Landing State Recreational Site (KLSRS) also covered under the Kenai Area Geotechical Contract (#78036-1). Information regarding our geotechnical assessment of the KLSRS can be found in our geotechnical report #4597-16(B)

DPOR contracted us to conduct a geotechnical evaluation of the proposed improvements at the project site. In this report, we provide a summary of our field exploration effort and laboratory testing, as well as provide our engineering conclusions and design and construction recommendations related to the geotechnical aspects of the proposed improvements.

2.0 PROJECT OVERVIEW

The project site is located at 4306 Eagle Rock Drive, Kenai, Alaska, as shown in Figure 1 of this report. The legal description provided by the Kenai Peninsula Borough (KPB) Assessing Department is Tract A of the Poore Subdivision, Kenai, Alaska.

At the time of our field explorations, the project site was developed with an outhouse structure, boat launch, and gravel parking area accessed by a gravel road at the end of Eagle Rock Place. The undeveloped portion of the project site is vegetated with moderately dense stands of spruce and birch with the ground covered with moss, leaves, and grass. The area near the proposed host parking space is wetlands with sparse spruce trees, brush and grasses. The portion of the project site planned for the parking area improvements is relatively flat with a gradual downhill grade from east to west. The entrance road, which approaches the parking area near the northeast end of the parking lot is steeper with approximately 45 feet of vertical elevation change from Eagle Rock Place to the parking area.

The proposed improvements to the site include the construction of parking facilities, restroom facilities, a double wide boat launch, a caretakers RV parking pad (or a cabin), an elevated walkway, and boat mooring.

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3.0 PROJECT SITE ACTIVITIES

3.1 Subsurface Exploration

We contracted Discovery Drilling, Inc. (DDI) of Anchorage, Alaska to provide the drilling services for our subsurface explorations. DDI mobilized a track mounted CME 55 drill rig to the project site to advance a series of five boreholes (designated KENB1 through KENB5) across the project site. Figure 2 of this report provides the approximate borehole locations. From December 15 to December 16, 2016, DDI advanced the five boreholes to depths ranging from approximately 21 feet below the ground surface (bgs) to 31.5 feet bgs. A geotechnical engineer from our office was presented during the entire exploration program to determine the final exploration locations (which were coordinated in the field with Roxanne Risse from the DPOR), observe drilling progress, log the geology of the boreholes, and collect appropriate soil sample for laboratory analysis.

Under our direction, DDI performed a Modified Penetration Test (MPT) at regular intervals during the drilling of each borehole. A MPT can be used to assess the consistency of a soil interval and to collect representative soil samples. A MPT is performed by driving a 3.0-inch O.D. (2.4-inch I.D.) split-spoon sampler at least 18 inches past the bottom of the advancing augers with blows from a 340-lb drop-hammer, free-falling 30 inches onto an anvil attached to the top of the drill rod stem. Our field representative recorded the hammer blows required to drive the modified split-spoon sampler the entire length of each sample interval, or until sampler refusal was encountered. We have provided the field blow count data for each sample interval (in six-inch increments) on the graphical borehole logs in Appendix A of this report.

During the course of our subsurface exploration at the project site, we encountered a common sampling phenomenon known as “sand-heave”. Sand-heave typically occurs when sampling saturated sand deposits with hollow stem augers, as the increased hydrostatic pressure outside of the hollow-stem augers forces a sand slurry up into the hollow auger when the drill rods are removed (to allow for sampling). At times, sand-heave can be significant; filling the inside of the augers with several feet of sand. As a result, sand-heave disturbs the in-situ density of the sand deposit and leads to unrepresentative blow count data (soil resistance measurements). Approximately two feet of sand have was observed at approximately 15 feet bgs in borehole KENB3.

Sand-heave can typically be controlled by filling the inside of the augers with an appropriate drilling fluid (e.g., water, drill mud, etc.) which equalizes the hydrostatic pressures inside and outside of the augers. Smaller diameter drill rods and SPT samplers can further help to reduce the effect of sand heave by reducing the potential for sand particles to bind downhole tooling inside of the hollow-stem augers. We have noted on our borehole logs when it was necessary to control sand-heave, along with the methods that DDI used to control the sand heave.

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We corrected the field blow count data for all five boreholes for standard confining pressure, drill rod length, and drop-hammer operation procedure to estimate a standard $(N_1)_{60}$ value for each sample interval. $(N_1)_{60}$ values are a measure of the relative density (compactness) and consistency (stiffness) of cohesionless or cohesive soils, respectively. Our estimate of the $(N_1)_{60}$ values is based on the drop-hammer blows required to drive the spilt-spoon sampler the final 12-inches of an 18-inch MPT. We have provided our estimated $(N_1)_{60}$ values for each sample interval on the graphical borehole logs in Appendix A of this report. The automatic drop-hammer that DDI used for this project is not standard, so a correction factor of 1.1 was applied to the $(N_1)_{60}$ values to account for the efficiency of the automatic drop-hammer that DDI used for the project. We have provided a graphical plot of the field blow count corrections that we used to correct for confining pressure and drill rod length in Figure 3 of this report.

We did not report the $(N1)_{60}$ values on the borehole logs where sand-heave occurred, as the $(N1)_{60}$ values obtained for those sample intervals are not representative of the in-situ material.

Our field representative photographed each split-spoon sample that they collected during the exploration program. A photograph of each split-spoon sample that we collected during our subsurface exploration program is provided in Appendix B of this report. We sealed each sample that was collected during our subsurface exploration program inside of an air-tight bag and/or container, to help preserve the moisture content of each sample, and then submitted each sample to our laboratory for further identification and analysis.

Once the exploration activities were complete, we directed DDI to backfill the annulus of each exploration with its respective drill cuttings.

4.0 LABORATORY TESTING

We collected a total of 36 soil samples from the five boreholes that DDI advanced at the project site and submitted all of the soil samples to our laboratory for further identification and geotechnical analysis. We tested select soil samples in accordance with the respective ASTM standard test methods including:

- moisture content analysis (ASTM D-2216);
- determination of fines content (a.k.a. P200 – ASTM D-1140);
- grain size sieve and hydrometer analysis (ASTM D-6913 & D-422);
- organic content (ASTM D2974); and

It is important to note that ASTM test method D-6913 requires that any soil sample specimen which is to be submitted for gradational analysis (by ASTM D-422 or other methods) must satisfy a minimum mass requirement based on the maximum particle size of the sample specimen. Split-spoon sampling techniques (standard or modified), as well as other small-diameter soil sampling techniques (e.g., macro-core, etc.), typically recover anywhere from approximately 1 to 10 pounds of sample specimen. The amount of sample specimen recovered can be influenced by (amongst other variables) the soil gradation, soil density, sample interval, sampler tooling, and soil moisture content. As a result, samples of coarse-grained soils (with individual soil particles greater than approximately 0.75 inches in diameter) collected with small-diameter sampling methods (e.g., split-spoons, macro-core, etc.) may not meet the minimum mass requirement specified by Table 2 of ASTM D-6913. This may result in inaccurate gradational and frost classification results. The use of small-diameter sampling devices in coarse-grained soils (e.g., sand and gravel) can result in the collection of unrepresentative samples due to: the exclusion of oversized particles (larger than the opening of the sampler) from the sample; and the mechanical breakdown/degradation of coarse-grained particles by the sampling process (producing an unrepresentative increase in smaller-diameter particles in the sample). Both of these sampling biases can skew laboratory test results towards the fine-grained end of the gradational spectrum.

The laboratory test results, along with the observations we made during our subsurface exploration efforts, aid in our evaluation of the subsurface conditions at the project site and help us to assess the suitability of the subsurface materials located at the project site to support the proposed improvements. The results of our geotechnical laboratory analyses are provided on the graphical exploration logs contained in Appendix A of this report and on the laboratory data sheets contained in Appendix B of this report.

5.0 DESCRIPTION OF SUBSURFACE CONDITIONS

We compiled our field observations with the results from our laboratory analyses to produce graphical logs of each subsurface exploration (Appendix A). The graphical exploration logs depict the subsurface conditions that we identified at each exploration location and help us to interpret/extrapolate the subsurface conditions for areas adjacent to, and immediately surrounding, each exploration location across the project site.

5.1 General Subsurface Profile

Borehole KENB2 was drilled within the footprint of the existing gravel boat launch. All of the other boreholes were drilled outside of the footprint of the existing developments at the project site.

In KENB2, we observed a layer of approximately five feet of loose to medium dense fill above the native organics. In the laboratory the samples of the fill were classified as ranging from (SP-SM) poorly-graded sand with silt and gravel to (GP-GM) poorly graded gravel with silt and sand. The samples of the fill material which we collected had 100% of the material passing the 1.5” sieve. Below the fill we observed native peat with some thin sand layers to approximately 10 feet bgs.

In the other boreholes, organic soils were observed from the surface to depths of 5.5 to 10 feet bgs. The organic soils ranged from mineral soils with trace amounts of organics to fibrous peat.

Below the organic soils in all of the boreholes, we observed various layers of medium dense to dense riverine deposits with variable particle size distributions.

5.2 Groundwater

We observed the groundwater table from eight feet bgs in borehole KENB3 to 13 feet bgs in KENB2. We did not observe the water table in boreholes KENB1 or KENB5. Due to the proximity to the river, we anticipate groundwater levels across the site to be influence significantly by the stage of the Kenai River.

5.3 Frozen Soils

At the time of our field explorations, seasonally frozen soils were observed to depths of 1.5 to 2.5 feet bgs. Permafrost was not observed in any of our subsurface explorations and is not expected to occur across the project site.

6.0 THERMAL ANALYSIS

We conducted a thermal analysis of the site soils in order to estimate the approximate maximum seasonal frost penetration that can reasonably be expected to occur at the project site. The primary purpose of this analysis is to provide guidance for minimum pile penetration depths to prevent frost jacking. We tested the thermal conductivity of two soils samples in order to make the analysis more representative of the site conditions. We then modeled the frost penetration using TEMP/W and BERG2.

6.1 Thermal Conductivity Testing

We performed the thermal conductivity testing using the KD2 Pro thermal properties analyzer manufactures by Decagon Devices. The KD2 Pro is capable of measuring thermal conductivity to within ±10%. We tested two near surface samples for thermal conductivity: samples KENB1-S2 and KENB3-B3. The samples were taken from the boreholes located near the proposed pile supported walkway. KENB1-S2 consisted of silt with sand with a moisture content of approximately 80% by weight. KENB3-S3 consisted of peat with a moisture content of approximately 510% by weight. The measured thermal conductivity of each sample is presented in Table 1 of this report. The measurements were taken by placing the soil in a two-inch diameter PVC mold. The soil was lightly compacted in the mold to approximate the soft conditions observed in the field.

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Table 1: Thermal Conductivity Testing Results

Sample ID	Measured Thermal Conductivity, *k* (BUT/hr-ft-°F)
KENB1-S2	**0.56**
KENB3-S3	**0.33**

6.2 TEMP/W (GeoStudio 2012)

We used the numerical modelling software TEMP/W to perform a thermal analysis to estimate frost penetration at the project site. TEMP/W is a two-dimensional, finite-element analysis software program that can model thermal changes in the subsurface due to a variety of environmental factors. TEMP/W can also be used to compute the transient distribution of subsurface temperatures (*i.e.*, temperature change with respect to time).

We constructed two subsurface models that approximate the conditions observed in boreholes KENB1 and KENB3 in TEMP/W's graphical user interface. Each of these subsurface models was then used to perform thermal analysis using two annual temperature models. The first temperature model used the daily 30-yr normal temperatures. The second used the daily 2012 temperature record for Kenai (representing the coldest year of the 30 year record). The analyses were conducted assuming zero snow cover (which will produce a conservatively deep calculated frost penetration).

Figures 4-7 of this report present a graphical view of the results of the four analyses. Table 2 of this report presents the maximum calculated frost penetration depth for each individual TEMP/W modeling scenario.

Table 2: FROST PENETRATION CALCULATED BY TEMP/W

MODELING SCENARIO	CALCUALTED FROST PENETRATION
KENB1 – NORMAL TEMPERATURES	7.2
KENB1 – 2012 TEMERATURES	12.9
KENB3 – NORMAL TEMPERATURES	3.8
KENB3 – 2012 TEMPERATURES	8.0

6.3 BERG2

BERG2 is program that simply solves the modified Berggren equation for multiple soils layers. The calculations uses the thermal properties of the individual soils layers and a design freezing index to calculate the frost penetration. For the purposes of this project we used a design freezing index of 3000 °F-days. We built two soil layer profiles in BERG2 – one approximating the conditions observed in KENB1 and the other the conditions observed in KENB3. In building the soil profile, we used the thermal conductivity measurements we took for the layers considered representative of the soils samples and allowed BERG2 to use the default values for the other layers. We ran a single calculation for each soil layer profile. Figures 8-9 of this report present the results of the BERG2 calculations. The calculations assumed zero snow cover. BERG2 calculated an approximate frost penetration of 6.9 feet bgs for the KENB1 soil profile and 5.4 feet for the KENB3 soil profile.

6.4 Conclusions

The design frost penetration for piles to be constructed for the elevated walkway can be reasonably expected to range from 6 to 8 feet bgs. The 12.9-ft penetration calculated for the 2012 temperatures at KENB1 should be considered overly conservative considering that the coldest year of the 30-yr record was applied to a subsurface profile with zero snow cover. Actual frost penetration depths will vary depending on a wide range of variables including but not limited to: seasonal weather conditions, snow cover, and soil moisture content variations.

7.0 ENGINEERING CONCLUSIONS

7.1 General Site Conclusions

Based on the findings of our field explorations, laboratory testing, and engineering analysis, it is our conclusion that the native riverine deposits which we observed across the project site are generally suitable to support the proposed improvements; provided that our concerns and recommendations that we present in this report are addressed by the design and construction processes.

The near surface organics are unsuitable for supporting any foundations or gravity fed utilities. However, properly proportioned pavement sections may be designed to “float” above the organic soils.

7.2 Earthworks

Any shallow foundations planned at the project site will require the excavation of the unsuitable organic materials. The organic materials were observed to depths of approximately 10 feet bgs. Properly proportioned pavement sections may be “floated” above the organic materials using a geotextile fabric.

7.3 Foundations

Shallow foundations will require the excavation of the organic materials that are not suitable for foundation support. DPOR has indicated that none of the foundations planned for this project will be continuously heated. Given the depth of the organic materials (up to 10 feet bgs) and the lack of continuous heating, deep foundations, such as driven piles or helical piers, may be a more economical option.

Deep foundations are planned for the elevated walkway near boreholes KENB1 and KENB3. This approach is appropriate especially considering the organic soils located at the ground surface in these areas.

7.4 Underground Utilities

The organic soils observed at the project site are not suited for supporting gravity fed utilities. Any gravity fed utilities will require the excavation of the existing organic materials. The utilities may then be founded on the underlying native mineral soils or properly placed structural fill.

7.5 Pavement

The pavement section for the parking areas may be "floated" above the organic soils. Currently, there is a gravel parking area located within the footprint of the proposed parking improvements. The only borehole we advanced within the existing gravel fill is KENB2 which was located near the top of the existing gravel boat launch. At KENB2, the fill material was approximately five feet thick and consisted of approximately equal part sand and gravel with approximately 6 to 8 % fines. If this material is representative of the entire existing parking area, the pavement section may be constructed on top of the existing fill. The thickness and gradation of the existing parking area fill should be confirmed before construction to determine a suitable pavement section. More detailed recommendations regarding pavement sections are presented in Section 8.4 of this report.

7.6 Settlements

Settlements for shallow foundations should be within tolerable limits, provided that they are placed directly onto properly placed structural fill which has been placed directly above the undisturbed mineral soils. If organic materials are left in place below foundations, settlements may be significantly higher and less predictable. We anticipate a total settlement for shallow concrete foundations placed on either the undisturbed native mineral soils and/or properly place structural structural fill (as we discuss in Section 8.1 of this report) to be less than three-quarters (3/4) of an inch, with differential settlements comprising about one-half (1/2) of the total anticipated settlement. Settlement amounts could increase substantially if the structural fill material used to bring any foundation pads to grade is not properly compacted or if any organic materials are not removed from the foundation footprint. Most of the settlements should occur as the building loads are applied, such that additional long-term settlements should be relatively small and within tolerable limits. Settlements for deep foundations (as we discuss in Section 8.3 of this report) should be negligible.

Settlements under driveways, parking areas, and street sections are expected to be vary more than under any buildings, especially where utility trenches are located. Proper earthwork is

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necessary to help reduce the settlement potential. The settlement potential can be reduced by performing all utility excavation and backfill efforts as early in the construction schedule as possible and placing any pavement as last in the construction schedule as possible.

7.7 Seismic Design Parameters

We have assumed that the International Building Code (IBC) 2012 will be used for the design of the proposed structure. The seismic site classification for the project site is *D* based on the $(N_1)_{60}$ values that we calculated for the that occur at the project site. We utilized the United States Geological Survey (USGS) Seismic Design Maps tool (which can be found at the following URL: http://earthquake.usgs.gov/designmaps/us/application.php) to calculate the seismic design parameters for the project site, which are F_a = 1.0 (S_s = 1.298) and F_v = 1.6 (S_1 = 0.486). A copy of the USGS Design Maps report for the project site is contained in Appendix C of this report.

Due to the relatively dense riverine deposits observed below the water table, we expect there to be a low potential for soil liquefaction at the site.

7.8 Boat Launch

Near the existing boat launch, borehole KENB2 revealed approximately five feet of granular fill material overlying approximately five feet of peat with some interbedded sand layers. The fill materials range in frost classification from PFS to S1 (slightly frost susceptible. It is likely that, as explained in 4.0 that Modified Penetration Test sampling procedure skewed the gradational results to the fine grained end of the spectrum (i.e. resulted in a high frost classification). As such, the material may be NFS. It is our professional opinion that five feet of granular fill that has been in place above the peat for a number of years, is suitable for supporting concrete modular boat ramp surfacing with a reduced risk of differential movement. However, some differential movement may still occur as a result of the underlying peat. The only way to eliminate this risk, would be to completely excavated the peat soils (to approximately 10 feet bgs) and replace with properly compacted structural fill. This approach is not only cost prohibitive, but is not entirely necessary given the light loads and slow speeds associated with small boat ramps. Regular maintenance to repair any differential movements is a more appropriate approach.

The boat ramp surface design should be modular (i.e., a series of flexibly-connected concrete planks, pads, mats, etc.), so that individual boat ramp surface modules (BRSMs) are not rigidly tied to one another. This will allow for some movement to occur beneath individual BRSMs without impacting adjacent BRSMs and can allow for localized maintenance/repairs to damaged/displace BRSMs without impacting adjacent BRSMs.

8.0 DESIGN RECOMMENDATIONS

We have presented our design recommendations in the general order that the project site will most likely be developed. Our design recommendations can be used in parts (as needed) for the final design configuration.

8.1 Earthworks

Our recommendations assume that any shallow foundations (i.e., poured-concrete footings) will be founded either directly onto the undisturbed native mineral soils or compacted structural fill pads constructed directly above the undisturbed mineral soils and that all organic materials will be excavated from any foundation footprint prior to foundation construction. Any structural fill materials used on-site should be compacted to a minimum of 95 percent of the modified Proctor density.

Any material removed during the initial site grading and excavation activities, which does not contain any organic/deleterious material, and has relatively low silt content (less than 15 percent passing the #200 sieve), can be re-used on-site as structural fill. Proper placement and compaction techniques need to be applied during the backfill process (see Section 9.1 of this report for more details). Additional laboratory testing may be required to verify the frost susceptibility of any excavated soil for use in shallow fill applications.

All earthworks should be completed with quality control inspection, including: bottom-of-hole inspections; fill gradation classification; and in-situ compacting testing. A bottom-of-hole inspection should be conducted by a qualified geotechnical engineer, geologist, or special inspector following site excavation activities (and before any foundation construction begins) in order to visually confirm the findings of this report and provide recommendations for any non-conforming conditions encountered during the excavation activities.

8.2 Shallow Foundations

For the purposes of this report, a shallow foundation can be considered any foundation which will require over-excavation of the existing organic-rich soils prior to structural fill placement and/or foundation construction. The excavation of the organic materials should extend a minimum of 10 feet past the perimeter of any shallow foundations. All of the recommendations regarding shallow foundations presented in this section of the report assume that all organic materials will be excavated and replaced with properly placed structural fill for a minimum of 10 feet laterally beyond the footprint of the foundations and that only unheated foundations are planned for this project.

8.2.1 Soil Bearing Capacity

Concrete foundations placed on on structural fill pads (constructed directly above the undisturbed mineral soils) may be designed for an allowable soil bearing capacity of 2,500

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pounds per square foot (psf). The soil bearing capacity may be increased by one-third (1/3) to accommodate short-term wind and/or seismic loads. Larger footings (smallest dimension greater than two feet in plan dimension) may be designed for greater bearing capacities at a rate of 300 psf for every additional horizontal linear foot of footing up to a maximum value of 4,000 psf.

8.2.2 Continuous Strip Footings and Spread Footings

Continuous strip footings and/or spread footings can be founded directly onto either: 1) the undisturbed native mineral soils (below the near surface organic layers), or 2) properly placed structural fill (located directly above the undisturbed mineral soils). There is no minimum requirement for structural fill thickness for this project. The minimum horizontal dimension for continuous strip footings should be 16 inches. The minimum horizontal dimension for spread footings should be 24 inches. Shallow foundation footings should extend laterally a minimum of one-eighth (1/8) of the footing width beyond any foundation walls to help resist any anticipated uplift/overturning forces (Figure 10). We discuss the effects of various uplift and lateral forces on foundations in more detail in Sections 8.2.4 and 8.2.5 of this report.

8.2.3 Thickened Edge Slab Foundations and Floor Slabs

Given the thickness of the organic materials found at the site, we assume any floor slabs will be constructed on properly placed structural fill placed directly above the undisturbed mineral soils following the excavation of the organic materials. The thickened edge (i.e., perimeter footing) of any thickened edge slab foundation should extend a minimum of 16 inches below the exterior finished grade to achieve the recommended allowable soil bearing capacity and help resist any lateral forces.

As we mention in Section 8.1 of this report, the upper structural fill material (at or above the footing grade) used to construct the structural pad for a building should be relatively free draining (sands and gravels) with less than 15% of the fill material passing through a #200 sieve. Furthermore, the top four to six inches of the structural pad located beneath the slab should be free draining, with less than 3% passing the #200 sieve. This “blanket” will serve as a capillary break to help maintain a dry slab.

Concrete slabs constructed on properly constructed granular fill pads (located directly above the undisturbed mineral soils), as we described above, may be designed using a modulus of subgrade reaction of k_1=200 pci (k_1 is the value for a 1-ft × 1-ft rigid plate). For this project, the following equations can be used (with standard English units) to calculate the appropriate modulus of subgrade reaction for slabs bearing on structural fill placed directly above the undisturbed mineral soils:

$$k_{(B\,x\,B)} = k_1 \left(\frac{B+1}{2B}\right)^2 \qquad (1)$$

where:

B = the slab width of a square slab in feet
k_1 = the modulus of subgrade reaction for a 1-ft × 1-ft rigid plate in pci
$k_{(B\ x\ B)}$ = the modulus of subgrade reaction for a square slab of width B in pci

The following equation (2) can be used for a rectangular slab having the dimensions $B \times L$ (in feet) with similar bearing soils as the slab loading equation above (1):

$$k_{(B\ x\ L)} = \frac{k_{(B\ x\ B)}\left(1+0.5\frac{B}{L}\right)}{1.5} \tag{2}$$

where:

$k_{(B\ x\ B)}$ = the modulus of subgrade reaction for a $B \times B$ square slab
$k_{(B\ x\ L)}$ = the modulus of subgrade reaction for $B \times L$ rectangular slab
B = the least horizontal dimension of a rectangular slab
L = the larger horizontal dimension of a rectangular slab

8.2.4 Footing Uplift

Shallow foundations should be buried sufficiently deep so as to resist any anticipated uplift/overturning forces (e.g. wind, seismic, frost jacking, etc.). The uplift capacity of a foundation is a function of its weight, configuration, and depth. The ultimate uplift capacity can be calculated by using 80 percent of the weight of the foundation plus 80 percent of the weight of the effective soil mass located above the footing. Figure 10 of this report illustrates the impact that effective soil mass has on the uplift capacity of a shallow foundation footing. An effective unit weight of 130 pcf can be used for granular structural backfill material. The ultimate uplift load includes any short-term load factors, so no increase in uplift capacity should be added for short-term loading.

8.2.4.1 Frost Heaving and Frost Protection

Frost heaving forces can generate significant footing uplift loads. Furthermore, it can be difficult to predict the depth of frost penetration and extent of ice lens formation at a given site. As such, footings need to be buried sufficiently deep so as to resist any anticipated frost heaving uplift forces. We have provided a schematic detailing our recommended uninsulated shallow foundation configurations is in Figure 11 of this report. For this project, only unheated foundations are planned. The minimum burial depth for any uninsulated shallow foundation footings should be 60 inches (D_3 in Figure 11) for cold footings (measured from the bottom of the footing to the lowest elevation of either the interior or exterior finished grade – including floor slabs). The cold foundation depth of D_3 (60 inches) can be reduced to 42 inches if the foundation is placed on a 5-foot thick structural pad constructed of non-frost susceptible (NFS) fill. NFS material should have less than six percent of the material passing a #200 sieve. The

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NFS structural pad thickness may be reduced by using insulation at a rate of one inch of insulation to one foot of NFS material. Any insulation used should conform to the specifications in Section 8.6 of this report. A minimum of 18 inches of NFS material is required between the footing and insulation as shown in Figure 12 (Configuration A). Below the insulation, proper bedding material should be used to provide a flat, smooth surface for the insulation.

The risk of ice lens formation and frost heaving beneath foundations may be reduced through the proper use of artificial insulation. We have presented our recommended insulation and footing configurations for various shallow foundation and floor slab combinations in Figure 12 of this report. For this project site, we recommend using insulation configuration A for unheated shallow foundation with stem walls and floor slabs and configuration D for unheated thickened edge slab foundations.

8.2.5 Lateral Loads for Foundation and Retaining Walls

Retaining walls (such as perimeter foundation stem walls for buildings with basements or crawl spaces) must be designed to resist lateral earth pressures. The magnitude of the pressure exerted on a retaining wall is dependent upon several factors, including:

1) whether the wall is allowed to deflect after placement of backfill;
2) the type of backfill used;
3) compaction effort; and
4) wall drainage provisions.

Any foundation stem walls that are not designed to carry lateral loads should be backfilled on both sides simultaneously to prevent differential lateral loading of the foundation stem wall. We developed the unit weights provided in Table 3 of this report assuming that structural fill (containing less than ten percent fines) is used as backfill, and that the fill is compacted to at least 90 percent of the modified Proctor density.

An active-earth pressure condition will prevail (under static loading) if a retaining wall is allowed to deflect or rotate a minimum of 0.001 times by the wall height. An at-rest pressure condition will prevail if a retaining wall is restrained at the top and cannot move at least 0.001 times the wall height. Lateral forces exerted by wind or seismic activity may be resisted by passive-earth pressures against the sides of the foundation footings, exterior walls (below grade), and grade beams.

In order to prevent water accumulation against the outside of any foundation or retaining wall, the wall must have a perimeter drainage system connected to an outlet that will not freeze closed at any time of the year. The top of the drainage piping must be located below the top of the footing for the foundation and/or retaining wall. Backfill used against the wall (and extending a

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minimum of one foot beyond the wall) must be free-draining with less than three percent fines. The top one-foot of backfill against the outside of a foundation and/or retaining wall should consist of relatively impermeable (fine-grained) material and be tightly compacted such that surface water is directed away from the foundation and/or retaining wall. A permeable geotextile fabric may be useful to prevent mixing of the impermeable (fine-grained) overburden and underlying free-draining (coarse-grained) backfill. Furthermore, the finished surface should slope away from any foundation and/or retaining wall with a grade between 1 to 2 percent, such that surface water is directed away from the foundation and/or retaining wall.

Seismic loading on foundation and/or retaining walls generally increases the lateral pressures on the wall and decreases the passive resistance. For foundation systems where the building foundation is continuous, the differential lateral movement between the soil and foundation is very small, and as such, essentially no excess lateral loading on the foundation wall is experienced. Foundation walls with a differential in backfill heights of over six feet (basements, crawl spaces, etc.) will experience seismic lateral loading from the inertial effects of seismic waves passing through the foundation.

The lateral soil pressures can be represented by equivalent fluid pressures. The pressure distribution is a function of wall restraint, seismic loading, and drainage conditions. Figure 13 presents the distribution diagrams for various loading conditions. Table 2 presents the unit weights to be used with Figure 13 for this project.

Table 3: Equivalent Fluid Specific Weight for Lateral Loading Design

LOADING CONDITION	DRAINED EQUIVALENT FLUID SPECIFIC WEIGHT		UN-DRAINED EQUIVALENT FLUID SPECIFIC WEIGHT	
	SPECIFIC WEIGHT (pcf)	**SYMBOL**	**SPECIFIC WEIGHT (pcf)**	**SYMBOL**
ACTIVE	35	t_1	24	t_2
AT-REST	55	t_3	38	t_4
PASSIVE	400	t_5	280	t_6
SEISMIC	16	t_7	9	t_8

Lateral forces may also be resisted by friction between the concrete foundations and the underlying soil. The frictional resistance may be calculated using a coefficient of friction of 0.4 between the concrete and soil.

8.3 Deep Foundations

For the purposes of this report, a deep foundation can be considered any foundation which transfers foundation loads (both bearing and uplift) through the existing organic soils to the deeper, more competent mineral soils (with limited foundation excavation effort required). Deep foundation systems, however, are often only employed when unsuitable subsurface conditions persist at a site (e.g., excessive thicknesses of non-structural fill or peat, shallow groundwater, etc.) which makes the construction of a conventional shallow foundation unpractical and/or uneconomic. It is our experience that deep foundations start to become cost effective in scenarios where there is at least 10 to 12 feet of unsuitable soils across a large portion of the site and/or where the unsuitable soils extend more than 1 to 2 feet below the groundwater table. In some instances, a combination of both a shallow and deep foundation may be employed to help reduce overall construction cost. This project falls right at the boundary of these criteria with approximately 5.5 to 10 feet of unsuitable materials at the surface.

8.3.1 Steel Pipe Piles

The most common type of deep foundation system in the Southcentral Alaska consists of driven steel pipe piling. Steel pipe piling can be obtained in a variety of diameters and wall thicknesses to accommodate a wide-range of applications, and is relatively inexpensive and readily available. Steel pipe piles are typically installed open-ended so that the soil can penetrate the inside of the pile, which helps facilitate efficient pile driving activities. Open-ended steel pipe pile can be driven with or without the use of a re-enforced/hardened drive shoe; which protects the end of the pile from damage during the driving activities. Steel pipe piles can also be installed close-ended, which helps to increase pile bearing capacities in soft, fine-grained soils. Any pile installation should be completed with quality control inspection to verify the pile configuration and final penetration rate. The final penetration rate is used to determine that the individual piles have the required axial capacity.

8.3.2 Pile Bearing Capacity

For this project, we recommend open-ended driven steel piles. Figure 14 of this report presents the allowable bearing capacity of a function of driven depth bgs. We based our calculations on the assumption that any piles installed at the project site would be installed near boreholes KENB1, KENB3, KENB4 (the areas planned for the elevated walkway and caretakers cabin). We can refine the pile recommendations, under our original contract, once the foundation loads are known and a preferred pile diameter/size has been selected.

8.3.2.1 Pile Uplift Capacity

Cold pile foundations (pile foundations where the soils surrounding individual piles are allowed to freeze) will need to be installed to greater depths than what would typically be required for continuously heated spaces in order to resist frost jacking uplift forces. A minimum pile embedment of 18 feet bgs is required for any cold piles installed at the project site in order to resist frost jacking forces. The short-term uplift capacity of each pile may be taken as one-half (1/2) of the long-term bearing capacity as we detail in Figure 14 of this report. The uplift capacity may not be increased for short term loading. When multiple piles are installed in close

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proximately to one another, then pile group efficiency should be considered. We discuss pile group efficiency in further detail in Section 8.3.2.3 of this report.

8.3.2.2 Lateral Pile Capacity

We used the computer program ALLpile7 (developed by CivilTech software) to analyze the lateral capacity for each of the pile diameters/sizes presented in Figure 14 of this report. We assumed a free-head condition for the piles (i.e., the pile head is allowed to rotate/deflect) with the pile head level with the ground surface (i.e., no pile stickup). The ultimate and allowable lateral loads for each pile diameter/size at the ground surface (with no pile stickup) are listed in Table 4 of this report. The allowable lateral loads are ½ of the ultimate lateral loads. We can recalculate the lateral loads, under our original contract, once the pile head elevation and connection design has been defined, as it is not feasible for us to provide an analysis for multiple design options. We anticipate that the piles planned for the elevated walkway will have significant stickup above grade. It should be noted that the lateral pile capacities significantly decrease as the pile stickup (above grade) increases. The lateral capacity of the boardwalk piles can be increased with lateral bracing (which should be designed by a structural engineer). When multiple piles are installed in close proximately to one another, then pile group efficiency should be considered. We discuss group efficiency in Section 8.3.2.3 of this report.

Table 4: Free-Head Lateral Pile Capacity

PILE TYPE	MAX. DEFLECTION (in)	MIN. DEPTH (ft)	ULTIMATE CAPACITY (kips)*	ALLOWABLE CAPACITY (kips)*
6-in SCH. 40	1	18	1.8	0.9
8-in SCH. 40	1	18	3.2	1.6
10-in SCH. 40	1	18	5.6	2.8
12-in SCH. 40	1	18	8.0	4.0

*Lateral pile capacities calculated with pile head at grade (i.e., no pile stickup above grade)

8.3.2.3 Pile Group Efficiency

Group efficiency of steel pipe piles is a function of the spacing of the individual piles. In Table 5 of this report, we present pile group efficiency parameters (as a function of pile diameter and spacing). The allowable pile capacities provided in Figure 14 of this report should be adjusted as necessary according to the spacing of individual piles.

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Table 5: Axial Pile Group Efficiency Values

PILE SPACING(S)	3B	4B	5B	6B	≥8B
GROUP EFFICIENCY (G_e)	0.70	0.75	0.85	0.90	1.00

*B = Largest Diameter of Pile

In Table 6 of this report we provide pile group efficiency parameters for lateral loads (as a function of pile diameter and spacing). The allowable capacities provided in Table 4 of this report should be adjusted as necessary according the spacing of individual piles.

Table 6: Lateral Pile Group Efficiency Values

PILE SPACING(S)	3B	4B	5B	6B	≥8B
GROUP EFFICIENCY (G_e)	0.50	0.60	0.68	0.70	1.00

*B = Diameter of Pile

8.3.2.4 Pile Foundations with Connecting Structural Members

Cold pile foundations are not recommended with the use of any grade-level structural members as frost heaving forces can damage the structural members and/or result in failures at connections between pile foundations and structural members. We recommend that a minimum air gap of 12 inches be maintained between the ground surface and any structural members that span between cold pile foundations. We should be consulted in the event that the structural design cannot accommodate a sub-structural member air gap so that we can evaluate any frost heaving pressures that may develop, so that they can be accounted for by the structural design.

8.3.3 Helical Piers

Helical piers are an alternative deep foundation system which have a relatively comparable price-point to steel piles, are fairly easy to install, and provide relatively high bearing and uplift resistance, with relatively shallow embedment.

Helical piers come in numerous sizes and configurations. For a site with moderately dense soils at depth (as exist at the project site), it is typical to have only a single helix on each helical pier. However, multiple helices can be used in order to distribute the foundation loads over a longer vertical section of the soil profile. Helical piers can also carry significant uplift loads (such as frost jacking) with less penetration than driven steel piles. Furthermore, the portions of any helical piers which are located above grade will typically need to be braced to help distribute any anticipated lateral loading.

For this project a helical pier, such as Techno Metal Post model P3 with an 8-in diameter helix, would be an acceptable product for the anticipated bearing loads. However, given the very soft organic soils near the surface, lateral bracing designed by a structural engineer will likely be required to resist any significant lateral loads. Typically, helical piers do not frost jack if installed to the required bearing capacity. Some manufacturer's/contractors (such as Techno Metal Post – Alaska) provide a guarantee against frost jacking. We can provide helical pier sizing and installation criteria once the proposed building/walkway loads and the proposed pile shaft stick up have been established. Furthermore, the ultimate capacity of a helical pier can be verified by the torque resistance measured during installation. This torque provides verification of the design and greatly increases the reliability of the foundation, and reduces the potential for differential movements.

Because it is not practical to provide a specific helical pier design without knowing the design structural loads or pier stickup, if helical piers are the desired option, we recommend that DPOR contact a local helical pier contractor and review which products they have available. We can then provide an analysis of selected products to determine their suitability for the design loads and stickup under our original contract.

8.4 Pavement Sections

The exploration locations chosen by DPOR were all outside of the footprint of the existing gravel parking area. Therefore, we do not have data that represents the current fill thickness and gradation. As such, the pavement section recommendations in this report assume a pavement section constructed above similar soils as those encountered in our explorations. DPOR should confirm whether or not the existing gravel parking area meets the pavement section criteria presented in this report. We assume that the parking areas will only be subjected to relatively light loads at load speeds.

Table 7 of this report presents a "floating" pavement section that is suitable for the highly organic soils we encountered in our explorations. A Type B, Class 2 geotextile should be used tor the "floating" section. The material specifications for the geotextile can be found in Table 8 of this report.

Table 7: Suitable Pavement Section Construction above the Existing Organic Material

SECTION THICKNESS	MATERIAL
2 INCHES MIN.	ASPHALT
2 INCHES MAX.	NFS D-1 BASE COURSE (A.K.A. "D-1")
24 INCHES	SELECTED MATERIAL, TYPE A
N/A	GEOTEXTILE FABRIC – TYPE B, CLASS 2
N/A	FROST SUSCEPTIBLE OR ORGANIC SOILS (NATIVE OR FILL)

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Any base course used should be NFS in order to maintain a low potential for ice lens development within the leveling course. It is our experience that the D-1 base course material currently available in Southcentral Alaska may not be NFS following compaction, because the compaction with a vibratory compactor further increases the frost susceptibility of the leveling course by increasing the percentage of fine-grained material (due to degradation of the soil particles from the impact of the compaction equipment). As such, the leveling course thickness should be kept to two inches or less to reduce the potential for ice lens formation in the leveling course. All of these materials should be placed in thin lifts and each lift should be compacted to a minimum of 95 % of the modified Proctor density. As an alternative to D-1, recycled asphalt pavement (RAP) can be used. The residual oil in the RAP greatly reduces the frost susceptibility.

Table 8: Type B, Class 2 Geotextile Fabric Strengths

FABRIC PROPERTY	ASTM STANDARD USED TO DETERMINE STRENGTH	WOVEN FABRIC STRENGTH	NON-WOVEN FABRIC STRENGTH
GRAB STRENGTH	D4632	250	160
SEWN SEAM STRENGTH	D4632	225	140
TEAR STRENGTH	D4533	90	56
PUNCTURE STRENGTH	D6241	495	310

Note: Units in lbs per foot.

8.5 Boat Launch

Boat ramp surfaces (e.g., pre-cast concrete planks, pads, mats, etc.) placed directly onto the relatively dense, native coarse-grained sand/gravel soils or onto a structurally-reinforced fill pad constructed directly above the existing peat soils (as we describe in Section 8.1 of this report) can be designed for an allowable dynamic (i.e., short-term) bearing capacity of 1000 pounds per square foot (psf). The dimensional and structural reinforcement criteria for concrete boat ramp surfaces will be a function of the anticipated boat ramp loads and should be evaluated by a structural engineer as a part of the ramp design.

Some differential movements should be expected beneath of the completed boat ramp surface, especially where it extends below MLLW, due to the presence of relatively soft silt soils and/or river scouring, etc. Some differential movements may also be expected beneath the proposed boat ramp surface (above MLLW) during winter months as a result of frost heaving forces, especially if frost susceptible materials (fill or native) are present beneath of the proposed boat ramp surface. It is impractical to try and design a boat ramp section/surface for this project site that will prevent/resist all of the potential sources for ground movements. Therefore, the boat ramp surface design should be modular (i.e., a series of flexibly-connected concrete planks, pads, mats, etc.), so that individual BRSMs are not structurally tied to one another. This will allow for some movement to occur beneath individual BRSMs without impacting adjacent BRSMs and can allow for localized maintenance/repairs to damaged/displaced BRSMs without impacting adjacent BRSMs.

8.6 Insulation

Any subsurface insulation should consist of extruded polystyrene such as DOW Styrofoam™ Highload or UC Industries Foamular. Any subsurface insulation used under pavement sections or structural slabs should be closed cell, board stock with a minimum compressive strength of 60 psi at five percent deflection. Subsurface insulation around foundations should have a minimum compressive strength of 25 psi at five percent deflection. The insulation should not absorb more than two percent water per ASTM Test Method C-272. The thermal conductivity (*k*) of the insulation should not exceed 0.25 BTU-in/hr-ft^2-°F when tested at 75°F.

8.7 Surface Drainage

After the property is brought to grade it should be relatively flat, such that storm water will tend to accumulate and flow off the site slowly. Water accumulation will have a detrimental effect on foundations, retaining structures, and pavement sections. Provisions should be included in the design to collect runoff and divert it away from any foundations, retaining structures, and pavement sections. The ground surface surrounding the proposed developments should be graded such that surface runoff is channeled away from foundations, retaining walls, and pavement sections. The soils on the surface should be tightly compacted to help reduce surface runoff infiltration. Roof, parking lot, and driveway drainage should be directed away from foundations. If storm sewer is available, tight-line connections from roof drain collectors should be made.

9.0 CONSTRUCTION RECOMMENDATIONS

We have presented our construction recommendations in the general order that the project site will most likely be developed. Our construction recommendations are intended to aid the construction contractor(s) during the construction process.

9.1 Earthwork

Any and all fill material used should be placed at 95 percent of the modified Proctor density as determined by ASTM D-1557, unless specifically stated otherwise in other sections of this report. The thickness of individual lifts will be determined based on the equipment used, the soil type, and existing soil moisture content. Typically, fill material will need to be placed in lifts of less than one-foot in thickness. All earthworks should be completed with quality control inspection.

In our professional experience, structural fill should have less than approximately 10 to 15 percent passing the #200 sieve for ease of placement. Soils with higher silt contents can be used within the foundation footprint. However, the effort required to achieve proper compaction of silt-rich soils may be more costly than purchasing better grade materials. The time of year, existing moisture content, rainfall, air temperature, and fill temperature can all have an impact on the effort required to adequately compact silt-rich material.

Any excavated fill or native mineral soils (which are free of organic material and have relatively low silt contents) which are stockpiled on-site (for later use as structural backfill) should be protected from additional moisture inputs (precipitation, etc.) through the use of plastic tarps, etc. Additional moisture inputs can have detrimental effects on the effort needed to achieve proper compaction rates.

9.2 Shallow Foundations

Care should be taken during foundation excavation activities to limit the disturbance of the bottom of any foundation excavations. The bottom of any foundation excavation should be moisture conditioned and proof-rolled as necessary to return the exposed soils to their original in-situ density.

In general, the soils in which the proposed foundation pads are to be constructed vary from silt with sand to with gravel. As such, any surface water (*e.g.,* from precipitation, snowmelt, etc.) that enters into foundation excavations may or may not infiltrate easily. Excess water will have a negative impact on any backfill and compaction efforts. Therefore, if surface water does accumulate in any open foundation excavations it can be controlled by excavating a shallow drainage trench around the perimeter of the excavation. The drainage trench will collect surface water and direct it to a sump area, which should be located outside of the foundation footprint. The excess water can then be pumped from the sump area and be discharged at an appropriate location away from the excavation and any other existing foundations.

9.3 Unheated Shallow Foundations

Because shallow foundations will require the excavation of the existing organic materials and replacement with properly placed structural fill, the frost susceptibility of the underlying native mineral soils is of little consequence for shallow foundations. It is important that any fill used to bring the foundation pad to grade be NFS. As we mention in Section 8.2.4.1of this report, the minimum cold foundation burial depth (60 inches) can be reduced to 42 inches, if the foundation is placed on a five-foot thick structural pad constructed of NFS fill. The NFS structural pad thickness may be reduced by using insulation at a rate of one inch of insulation to one foot of NFS material.

9.4 Deep Foundations

A drive shoe is not required if the steel pipe pile wall thickness used is sufficient to help reduce the potential for buckling. Any drive shoe used during pipe pile installation should have an outside diameter smaller than the outside diameter of the pile so that it does not oversize the pile annulus and reduce the skin friction on the pile. Once the pile size, pile loading, and pile hammer are chosen, we can perform a pile analysis to determine a final driving rate for the allowable load

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required. The installation of any driven piles should be observed by a qualified engineer or special inspector to confirm that each pile has reached the require design capacity.

Piles may be allowed to freeze and/or be installed in frozen soils, if they are driven to a minimum depth of 18 feet for cold pile foundations (assuming no grade-level structural members are connect adjoin pile foundations – See Section 8.3.2.4 of this report for more detail).

Any helical piers should also be inspected by a qualified engineer or special inspector to confirm that each helical pier has reached the appropriate axial capacity. This is typically verified with a combination of installation torque and installed depth.

9.5 Pavement

All of the earthwork within any areas to be paved should be completed as early in the construction schedule as possible, and the pavement placed as late in the construction schedule as possible. This will give the subgrade soils time to settle, compress, and stabilize prior to placement of the pavement. Any structural fill used should be placed in thin lifts (less than one foot in thickness) and each lift should be compacted to a minimum of 95 percent of the modified Proctor density. Prior to paving, any surface fill material should be re-leveled and re-compacted. All backfill and paving materials should be inspected and tested for material specification compliance and compaction.

Underground utility piping should be installed prior to construction of any pavement sections such that trenching is done through the subgrade soils only. This will help ensure that a uniform pavement section is maintained, which will reduce the potential for differential settlements along underground utility trench alignments.

The minimum thickness for any asphalt pavement surfaces is two inches. The minimum thickness of any concrete pavement surfaces will be a function of the reinforcement required. All applicable ACI and IBC standards should be followed.

9.6 Insulation

The satisfactory performance of any subsurface insulation is in part controlled by the details of construction including: 1) the care taken to ensure that the board stock lies flat on a smooth, level surface; and 2) the adjoining ends of the insulation are closely butted together. Any vertical joints should be staggered where more than one layer of insulation is used.

9.7 Winter Construction

Proper placement and compaction of structural fill is not possible when fill material is frozen, and as such, frozen fill material should never be used for structural support unless it has been subsequently thawed and compacted to 95 percent of the modified Proctor density (throughout its vertical extent). Furthermore, subgrade soils (fill or native) need to be completely thawed prior to the placement and compaction of additional lifts of thawed fill material. In our professional experience, ambient soil temperatures need to be above 37 °F in order to achieve efficient compaction. It is extremely difficult to achieve compaction levels equal to 95 percent of the modified Proctor density in fill material that is between 32 °F to 37 °F.

10.0 THE OBSERVATIONAL METHOD

A comprehensive geoprofessional service (e.g., geotechnical, geological, civil, and/or environmental engineering, etc.) should consist of an interdependent, two-part process comprised of:

Part I - pre-construction site assessment, engineering, and design; and

Part II - continuous construction oversight and design support.

This process, commonly referred to in the geoprofessional industry as "The Observational Method", was developed to reduce the costs required to complete a construction project, while simultaneously reducing the overall risk associated with the design and construction of the project.

In geotechnical engineering, Part I of the Observational Method (OM) begins with a geotechnical assessment of the site, which typically consists of some combination of literature research, site reconnaissance, subsurface exploration, laboratory testing, and geotechnical engineering. These efforts are usually documented in a formal report (e.g., such as this report) that summarizes the findings of the geotechnical assessment, and presents provisional geotechnical engineering recommendations for design and construction. Geotechnical assessment reports (and the findings and recommendations contained within) are considered provisional due to the fact that their contents are typically based primarily on limited subsurface information for a site. Most conventional geotechnical exploration programs only physically characterize a very small percentage of a given site, as it is typically cost prohibitive to conduct extensive (i.e. high density/frequency) exploration programs. As an alternative, geoprofessionals use the subsurface information available for a site to extrapolate subsurface conditions between exploration locations and develop appropriate provisional recommendations based on the inferred site conditions. As a result, the geoprofessional of record cannot be certain that the provisional recommendations will be wholly applicable to the site, as subsurface conditions other than those identified during the geotechnical assessment may exist at the site which could present obstacles and/or increased risk to the proposed design and construction.

Part II of the OM is employed by geoprofessionals to help reduce the risk associated with unidentified and/or unexpected subsurface conditions. Geoprofessionals accomplish Part II of the OM by providing construction oversight (e.g., construction observation, inspection, and testing). Part II of the OM is a valuable service, as the geoprofessional of record is available if unexpected conditions are encountered during the construction process (e.g., during excavation, fill

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placement, etc.) to make timely assessments of the unexpected conditions and modify their design and construction recommendations accordingly; thus reducing considerable cost resulting from potential construction delays and reducing the risk of future problems resulting from inappropriate design and construction practices.

Oftentimes, a client may be persuaded to use an alternative geoprofessional firm to conduct Part II of the OM for a given project; as some geoprofessional firms offer the same services at discounted prices in order to help them obtain the overall construction materials engineering and testing (CoMET) commission. The geoprofessional industry as a whole recommends against this practice. An alternative geoprofessional firm cannot provide the same level of service as the geoprofessional of record. The geoprofessional of record has (amongst other things) a unique familiarity with the project including; an intimate understanding of the subsurface conditions, the proposed design, and the client's unique concerns and needs, as well as other factors that could impact the successful completion of a construction project. An alternative geoprofessional firm is not aware of the inferences made and the judgment applied by the geoprofessional of record in developing the provisional recommendations, and may overlook opportunities to provide extra value during Part II of the geoprofessional service.

Clients that prevent the geoprofessional of record from performing a complete service can be held solely liable for any complications stemming from engineering omissions as a result of unidentified conditions. The geoprofessional of record may not be liable for any resulting complications that occur, as the geoprofessional of record was not able to complete their services. Furthermore, the replacement geoprofessional firm may also be found to have no liability for the same reasons.

We are available at any time to discuss the OM in more detail, or to provide you with an estimate for any additional construction observation and testing services required.

11.0 CLOSURE

We (Northern Geotechnical Engineering, Inc. d.b.a. Terra Firma Testing) prepared this report exclusively for the use of DPOR and their consultants/contractors/etc. for use in the design and construction of the proposed improvements. We should be notified if significant changes are to occur in the nature, design, or location of the proposed improvements in order that we may review our conclusions and recommendations that we present in this report and, if necessary, modify them to satisfy the proposed changes.

This report should always be read and/or distributed in its entirety (including all figures, exploration logs, appendices, etc.) to ensure that all of the pertinent information has been adequately disseminated. Otherwise, an incomplete or misinterpreted understanding of the site conditions and/or our engineering recommendations may occur. Our recommended best practice is to make this report accessible, in its entirety, to any design professional and/or contractor working on the project. Any part of this report (e.g., exploration logs, calculations, material values, etc.) which is presented in the design/construction plans and/or specifications for the project should have an adequate reference which clearly identifies where the report can be obtained for further review.

Due to the natural variability of earth materials, variations in the subsurface conditions across the project site may exist other than those we identified during the course of our geotechnical assessment. Therefore, a qualified geotechnical engineer, geologist, and/or special inspector be on-site during construction activities to provide corrective recommendations for any unexpected conditions revealed during construction (see our discussion of the Observational Method in Section 10.0 of this report for more detail). Furthermore, the construction budget should allow for any unanticipated conditions that may be encountered during construction activities.

We conducted this evaluation following the standard of care expected of professionals undertaking similar work in the State of Alaska under similar conditions. No warranty, expressed or implied, is made.

REPORT

FIGURES

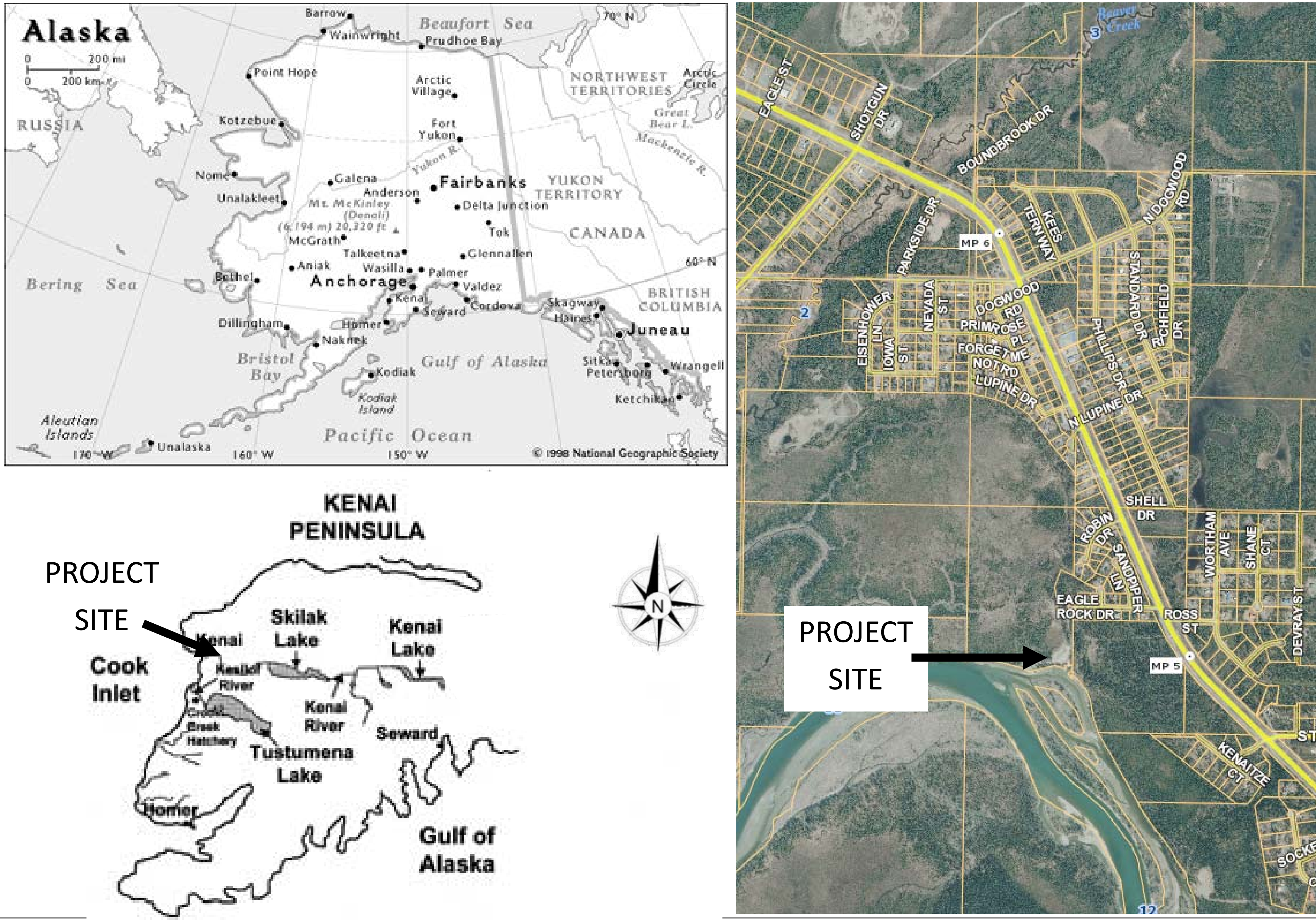

FIGURE TITLE:
PROJECT SITE LOCATION MAP

PROJECT NAME:
EAGLE ROCK BOAT LAUNCH IMPROVEMENTS

PROJECT ID:
4597-16

PROJECT LOCATION:
KENAI, AK

FIGURE NUMBER:
1

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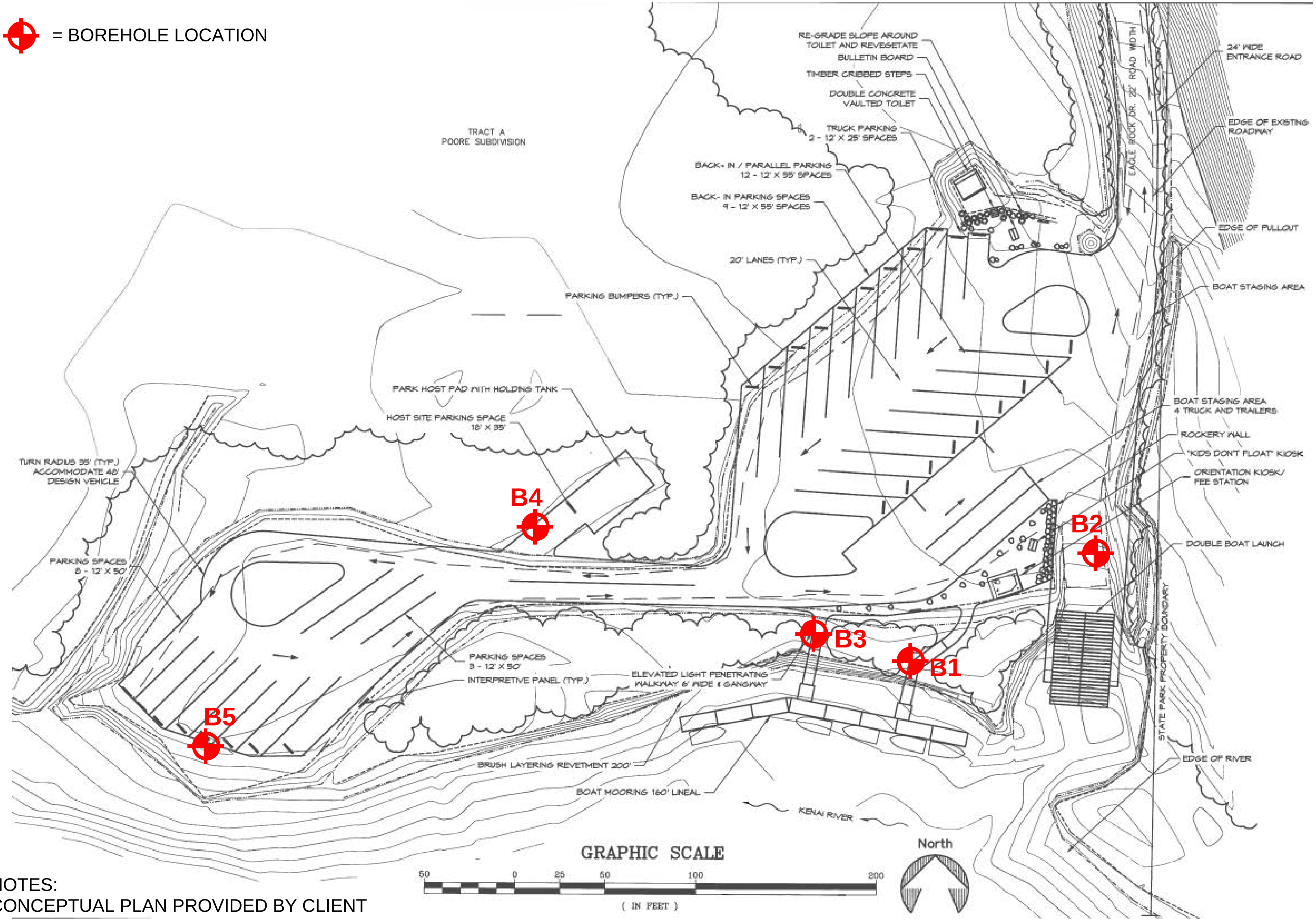
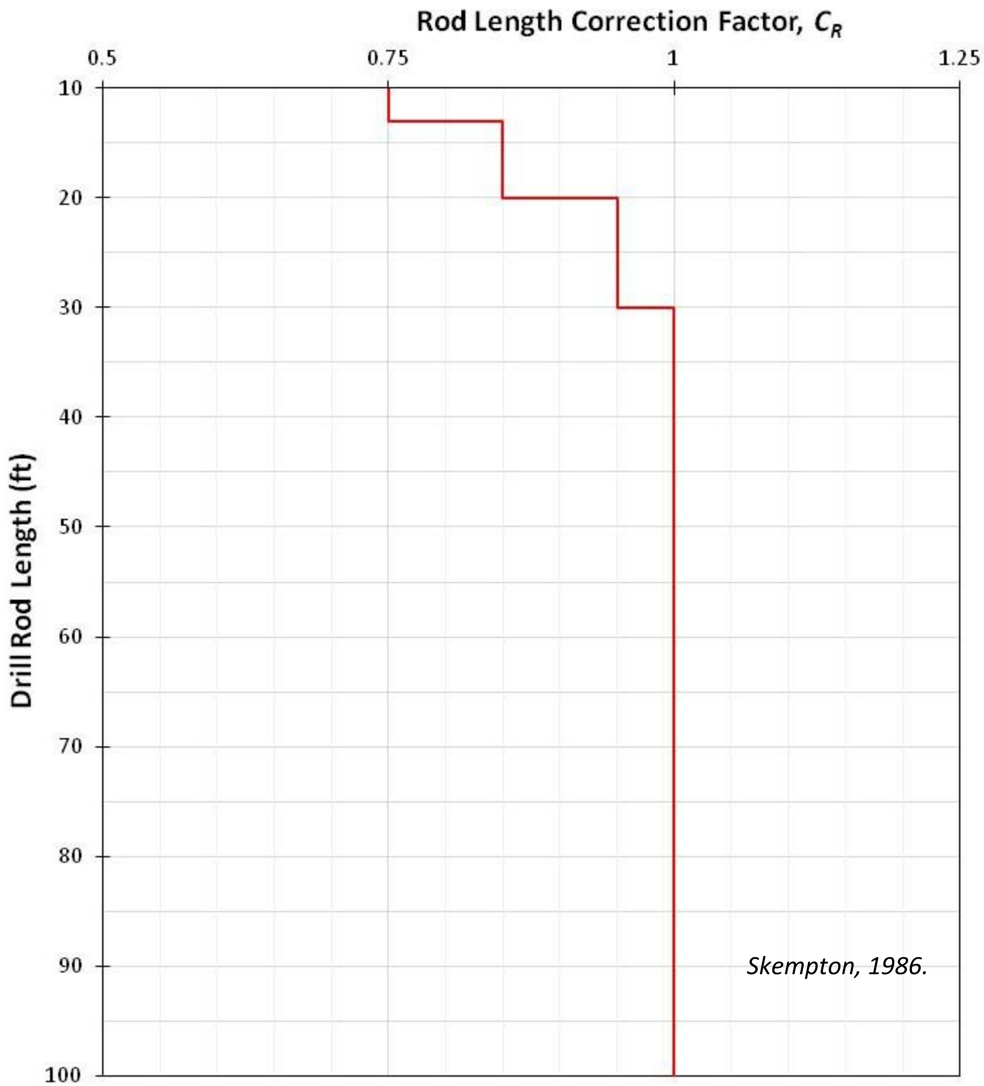
= BOREHOLE LOCATION

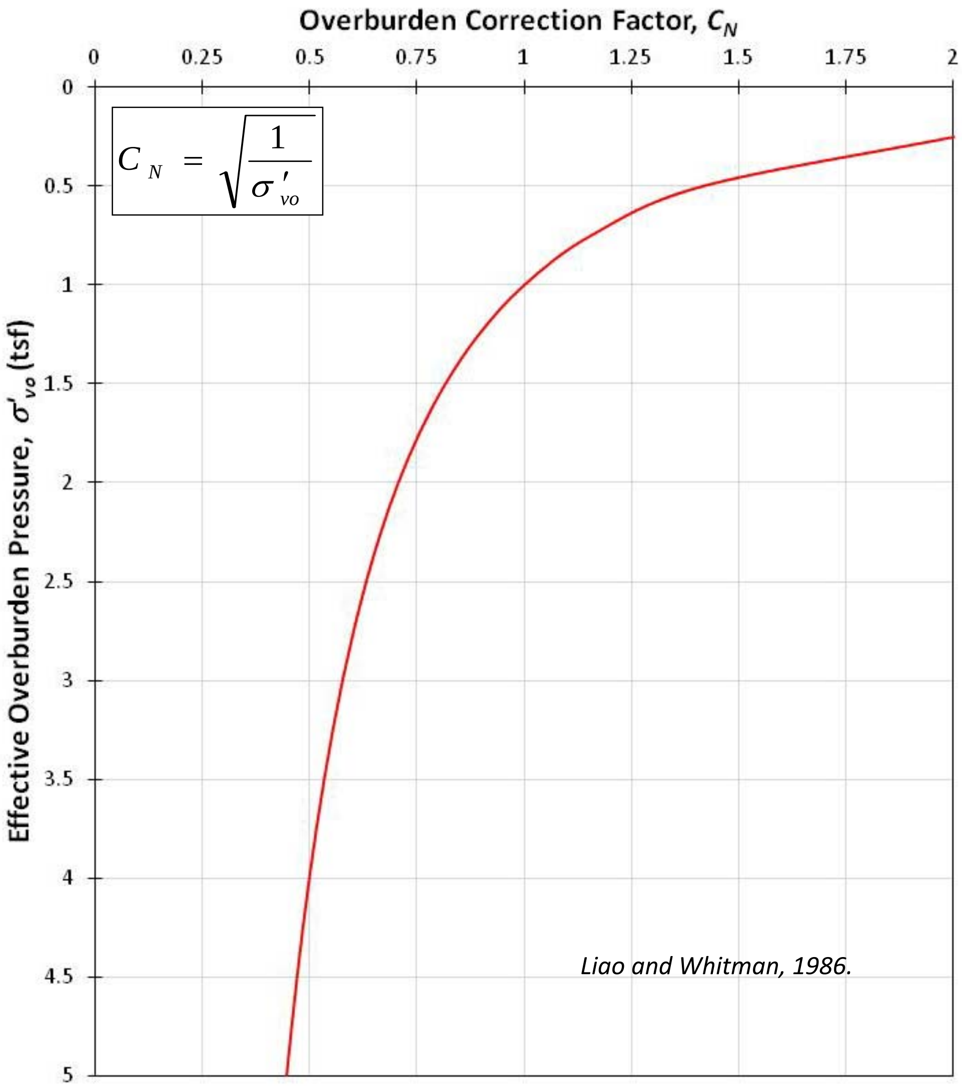


NOTES:
CONCEPTUAL PLAN PROVIDED BY CLIENT

NORTHERN GEOTECHNICAL ENGINEERING, INC.
TERRA FIRMA TESTING

FIGURE TITLE: APPROXIMATE BOREHOLE LOCATIONS	
PROJECT NAME: EAGLE ROCK BOAT LAUNCH IMPROVEMENTS	PROJECT ID: 4597-16
PROJECT LOCATION: KENAI, AK	FIGURE NUMBER: 2





Notes:

- Overburden correction factor is used only for cohesionless soils
- C_N is the ratio of the measured blow count to what the blow count would be at an overburden pressure of 1 ton/ft^2
- σ'_{vo} is the effective overburden pressure at the point of measurement (ton/ft^2)

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FIGURE TITLE: BLOW COUNT CORRECTIONS	
PROJECT NAME: EAGLE ROCK BOAT LAUNCH IMPROVEMENTS	PROJECT ID: 4597-16
PROJECT LOCATION: KENAI, AK	FIGURE NUMBER: 3

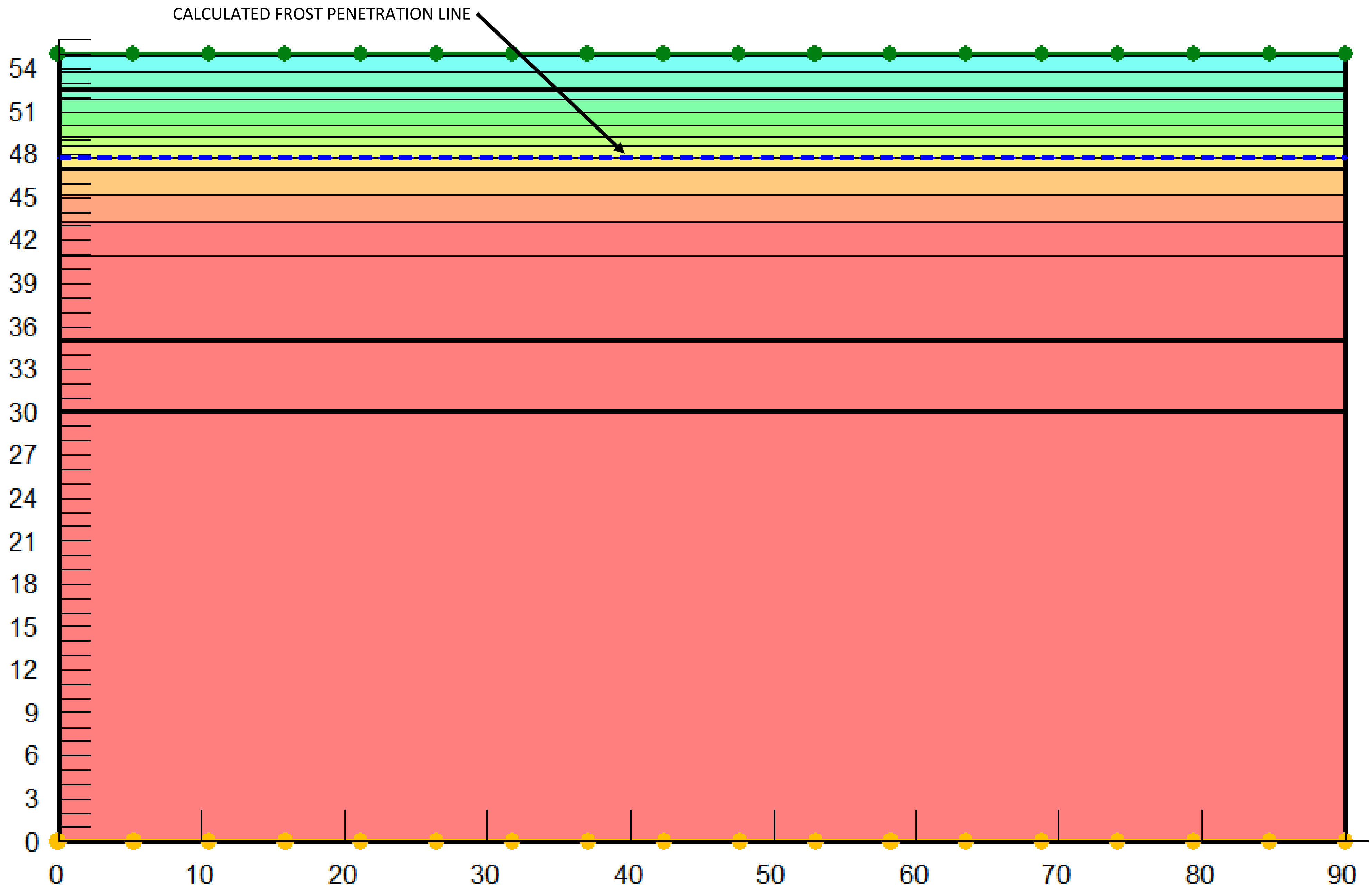

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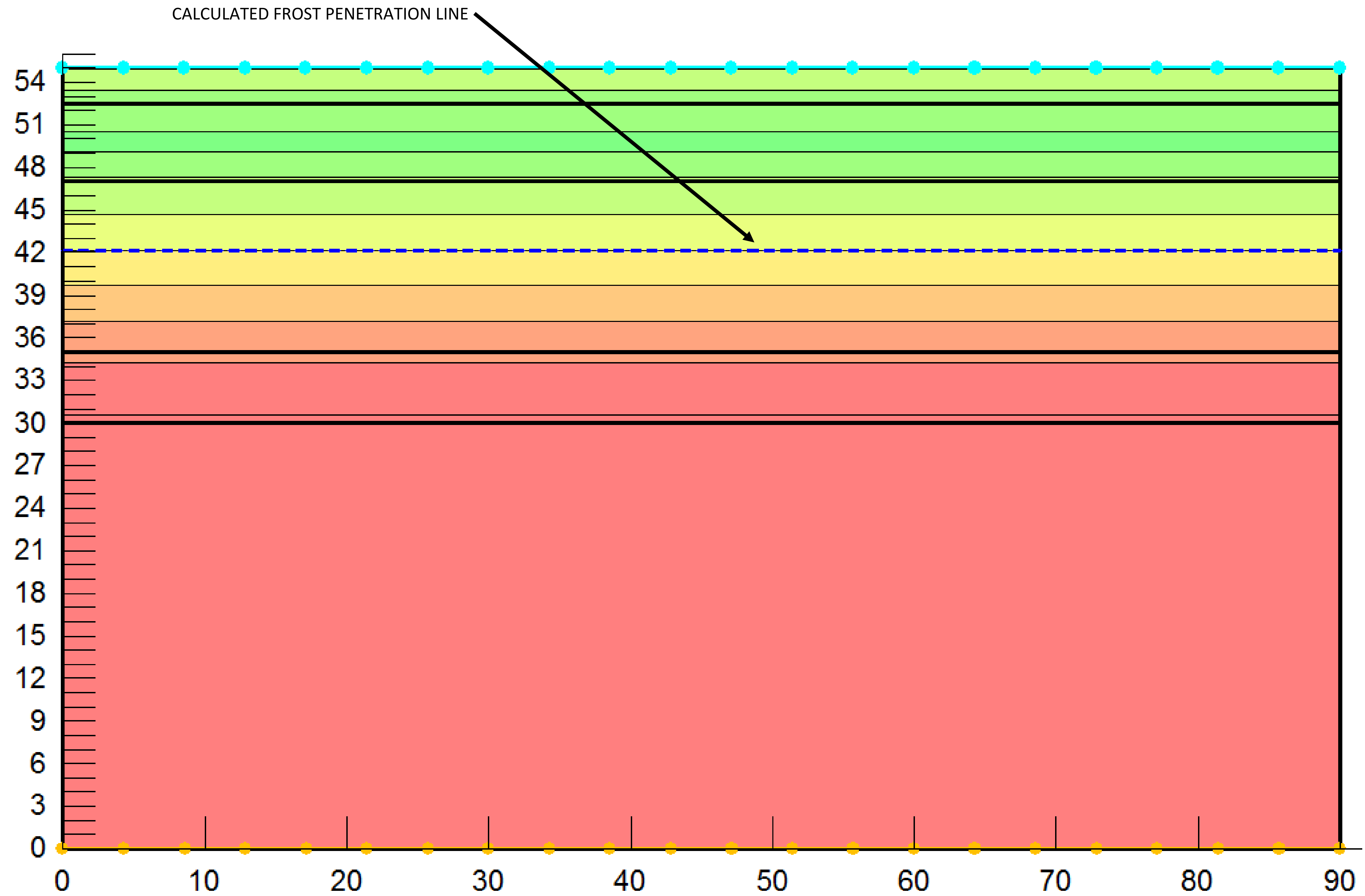
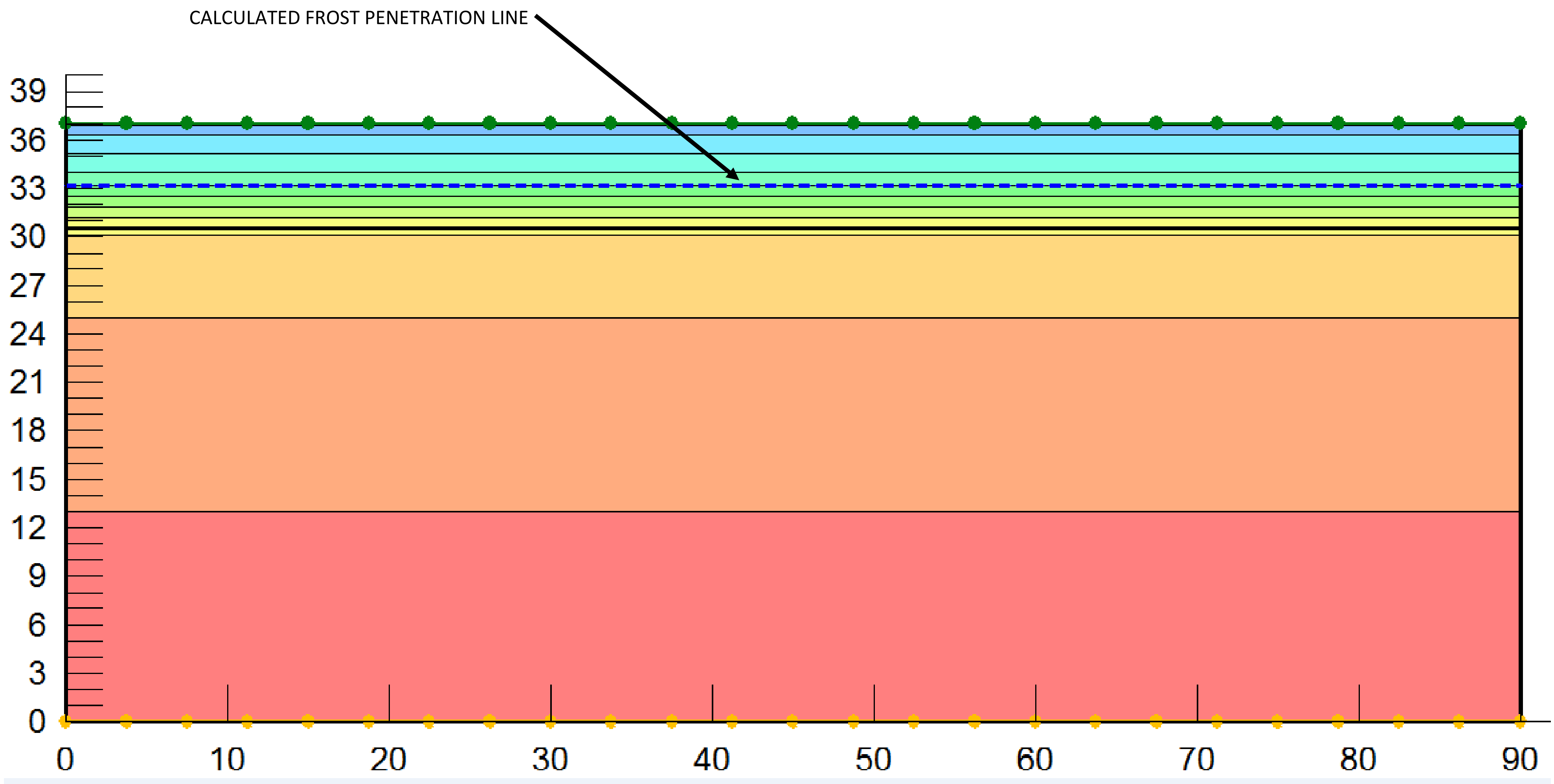
FIGURE TITLE: TEMP/W ANALYSIS RESULTS FOR KENB1—NORMAL TEMPERATURES	
PROJECT NAME: EAGLE ROCK BOAT LAUNCH IMPROVEMENTS	PROJECT ID: 4597-16
PROJECT LOCATION: KENAI, AK	FIGURE NUMBER: 4



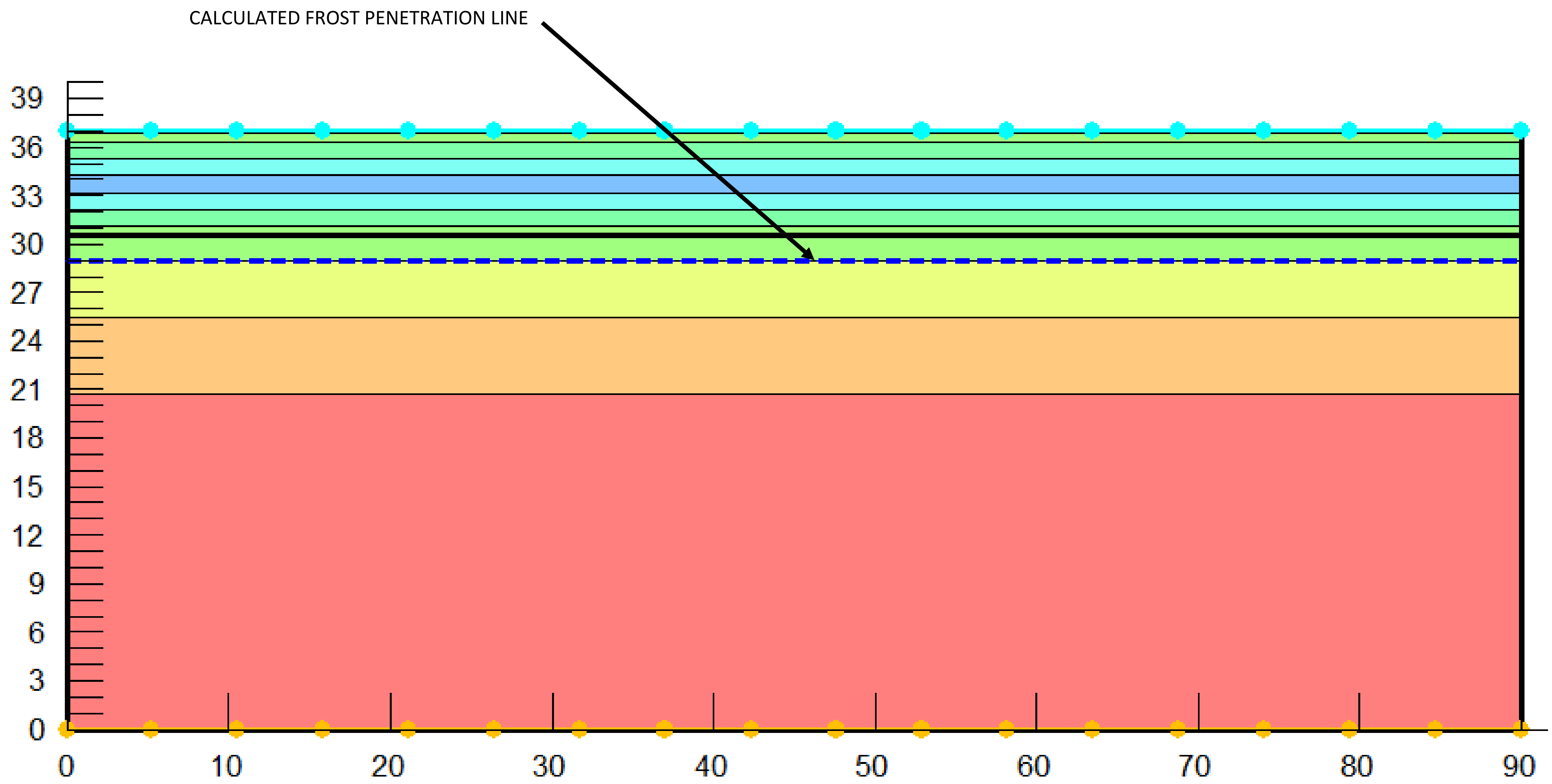
NORTHERN GEOTECHNICAL ENGINEERING, INC.
TERRA FIRMA TESTING

FIGURE TITLE: TEMP/W ANALYSIS RESULTS FOR KENB1—2012 TEMPERATURES	
PROJECT NAME: EAGLE ROCK BOAT LAUNCH IMPROVEMENTS	PROJECT ID: 4597-16
PROJECT LOCATION: KENAI, AK	FIGURE NUMBER: 5



NORTHERN GEOTECHNICAL ENGINEERING, INC.
TERRA FIRMA TESTING

FIGURE TITLE: TEMP/W ANALYSIS RESULTS FOR KENB3—NORMAL TEMPERATURES	
PROJECT NAME: EAGLE ROCK BOAT LAUNCH IMPROVEMENTS	PROJECT ID: 4597-16
PROJECT LOCATION: KENAI, AK	FIGURE NUMBER: 6



NORTHERN GEOTECHNICAL ENGINEERING, INC.
TERRA FIRMA TESTING

FIGURE TITLE: TEMP/W ANALYSIS RESULTS FOR KENB3—2012 TEMPERATURES	
PROJECT NAME: EAGLE ROCK BOAT LAUNCH IMPROVEMENTS	PROJECT ID: 4597-16
PROJECT LOCATION: KENAI, AK	FIGURE NUMBER: 7

LOCATION	THAW N	FREZ N	MAAT	THAW °F DAY	FREZ °F DAY	THAW DAYS	FREZ DAYS
ANCHORAG	1.70	1.00	36	4000	3000	200	165

THAW CYCLE	1	2	3	4	5	6
FROZEN % MOIS.	17.0	28.0	17.0	7.0	8.0	17.0
FROZEN DENS.	110.0	90.0	110.0	130.0	110.0	110.0
LATENT HEAT	2693	3629	2693	1310	1267	2693
FROZEN HEAT CAP	28.05	27.90	28.05	26.65	23.10	28.05
FROZEN COND.	2.00	1.12	2.00	1.79	1.03	2.00
THAWED % MOIS.	17.0	28.0	17.0	7.0	8.0	17.0
THAWED DENS.	110.0	90.0	110.0	130.0	110.0	110.0
THAWED HEAT CAP	37.40	40.50	37.40	31.20	27.50	37.40
THAWED COND.	1.32	0.56	1.32	1.65	1.08	1.32
INITIAL THICK	2.50	5.50	12.00	5.00	5.00	10.00
AMOUNT THAWED	2.50	4.89	0.00	0.00	0.00	0.00
CONSOLIDATION	----	----	----	----	----	----
FINAL THICK	2.50	5.50	12.00	5.00	5.00	10.00

FREEZE CYCLE	1	2	3	4	5	6
LATENT HEAT	2693	3629	2693	1310	1267	2693
FROZEN DENS.	110.0	90.0	110.0	130.0	110.0	110.0
FROZEN HEAT CAP	28.05	27.90	28.05	26.65	23.10	28.05
FROZEN COND.	2.00	1.12	2.00	1.79	1.03	2.00
INITIAL THICK	2.50	5.50	12.00	5.00	5.00	10.00
AMOUNT FROZEN	2.50	4.38	0.00	0.00	0.00	0.00

ESTIMATED THAW= 7.39 FREEZE= 6.88 PRINT LOCATION SOIL QUIT

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FIGURE TITLE:	
BERG2 RESULTS—KENB1	
PROJECT NAME: EAGLE ROCK BOAT LAUNCH IMPROVEMENTS	PROJECT ID: 4597-16
PROJECT LOCATION: KENAI, AK	FIGURE NUMBER: 8

LOCATION	THAW N	FREZ N	MAAT	THAW °F DAY	FREZ °F DAY	THAW DAYS	FREZ DAYS
ANCHORAG	1.70	1.00	36	4000	3000	200	165

THAW CYCLE	1	2
FROZEN % MOIS.	45.0	15.0
FROZEN DENS.	60.0	110.0
LATENT HEAT	3888	2376
FROZEN HEAT CAP	23.70	26.95
FROZEN COND.	0.98	1.79
THAWED % MOIS.	45.0	15.0
THAWED DENS.	60.0	110.0
THAWED HEAT CAP	37.20	35.20
THAWED COND.	0.33	1.28
INITIAL THICK	6.50	20.00
AMOUNT THAWED	4.49	0.00
CONSOLIDATION	----	----
FINAL THICK	6.50	20.00

FREEZE CYCLE	1	2
LATENT HEAT	3888	2376
FROZEN DENS.	60.0	110.0
FROZEN HEAT CAP	23.70	26.95
FROZEN COND.	0.98	1.79
INITIAL THICK	6.50	20.00
AMOUNT FROZEN	5.43	0.00

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FIGURE TITLE: BERG2 RESULTS—KENB3

PROJECT NAME: EAGLE ROCK BOAT LAUNCH IMPROVEMENTS

PROJECT ID: 4597-16

PROJECT LOCATION: KENAI, AK

FIGURE NUMBER: 9

UPLIFT CAPACITY = 0.8 x (EFFECTIVE SOIL WEIGHT + WEIGHT OF FOUNDATION)

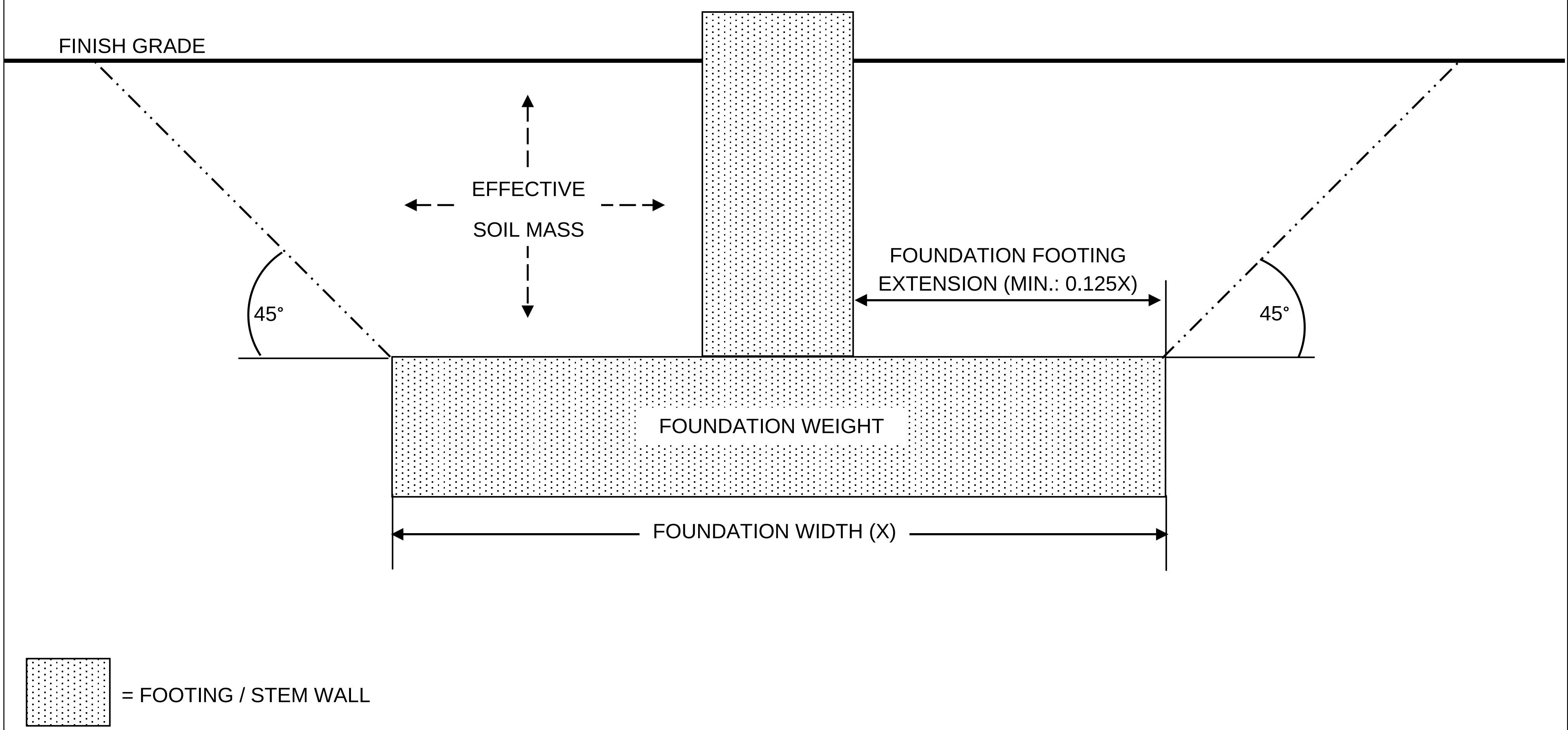
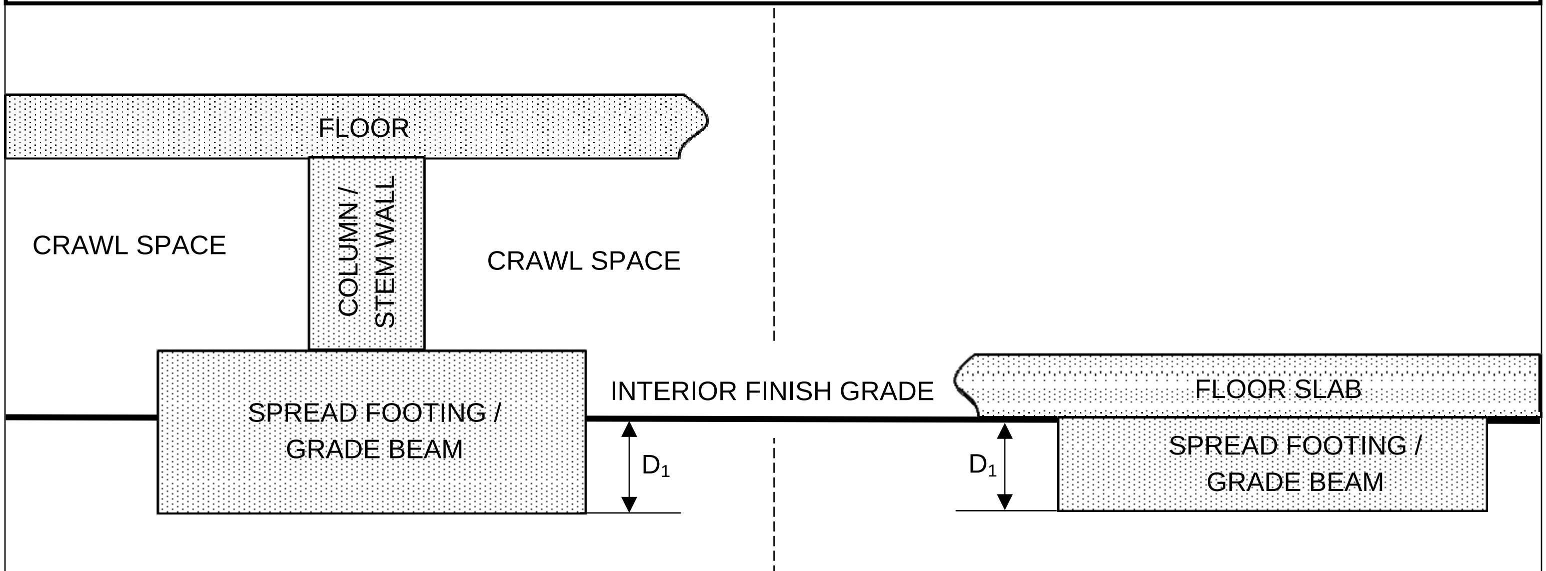


DIAGRAM NOT TO SCALE

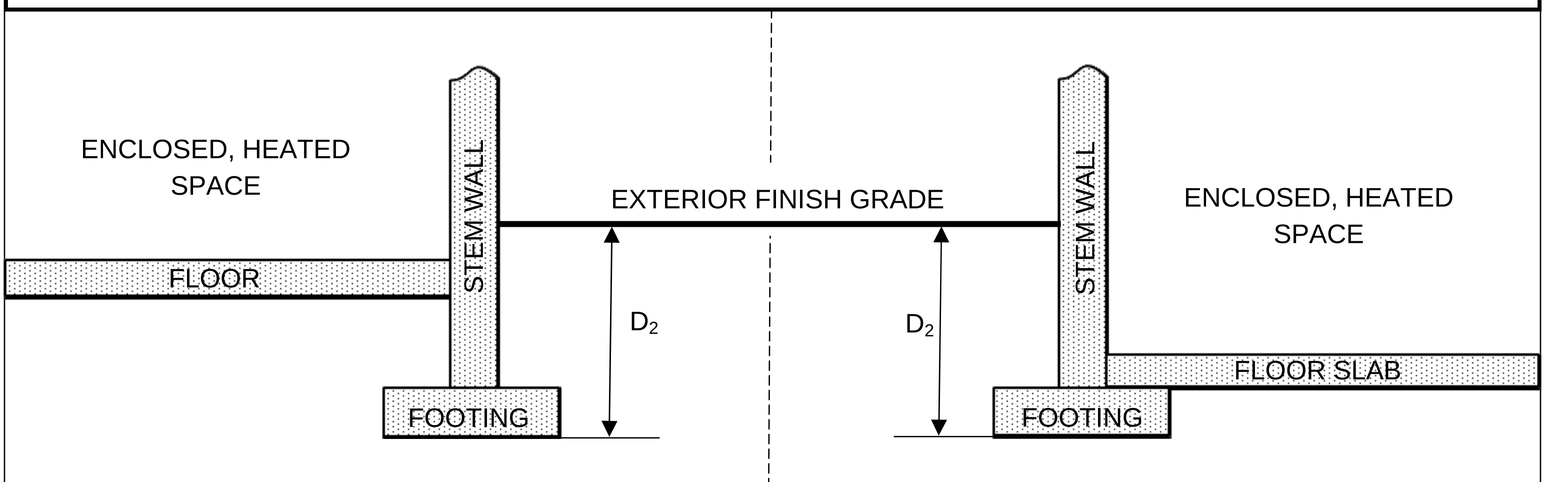
NORTHERN GEOTECHNICAL ENGINEERING, INC.
TERRA FIRMA TESTING

FIGURE TITLE: UPLIFT CAPACITY DIAGRAM	
PROJECT NAME: EAGLE ROCK BOAT LAUNCH IMPROVEMENTS	PROJECT ID: 4597-16
PROJECT LOCATION: KENAI, AK	FIGURE NUMBER: 10

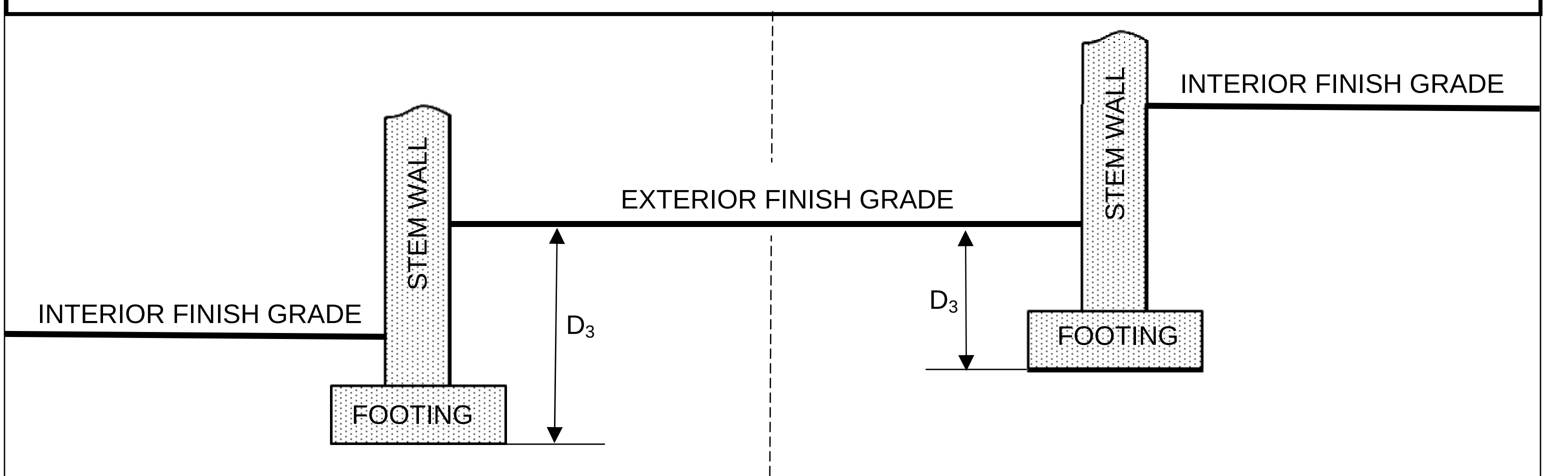
SHALLOW FOUNDATION FOOTING LOCATED ENTIRELY WITHIN AN ENCLOSED, CONTINUOUSLY HEATED SPACE



SHALLOW FOUNDATION FOOTING LOCATED ALONG THE PERIMETER OF AN ENCLOSED, CONTINUOUS HEATED SPACE



SHALLOW FOUNDATION FOOTING LOCATED ENTIRELY WITHIN AN ENCLOSED, CONTINUOUSLY HEATED SPACE



= FOOTING / FLOOR SLAB

SCHEMATIC NOT TO SCALE

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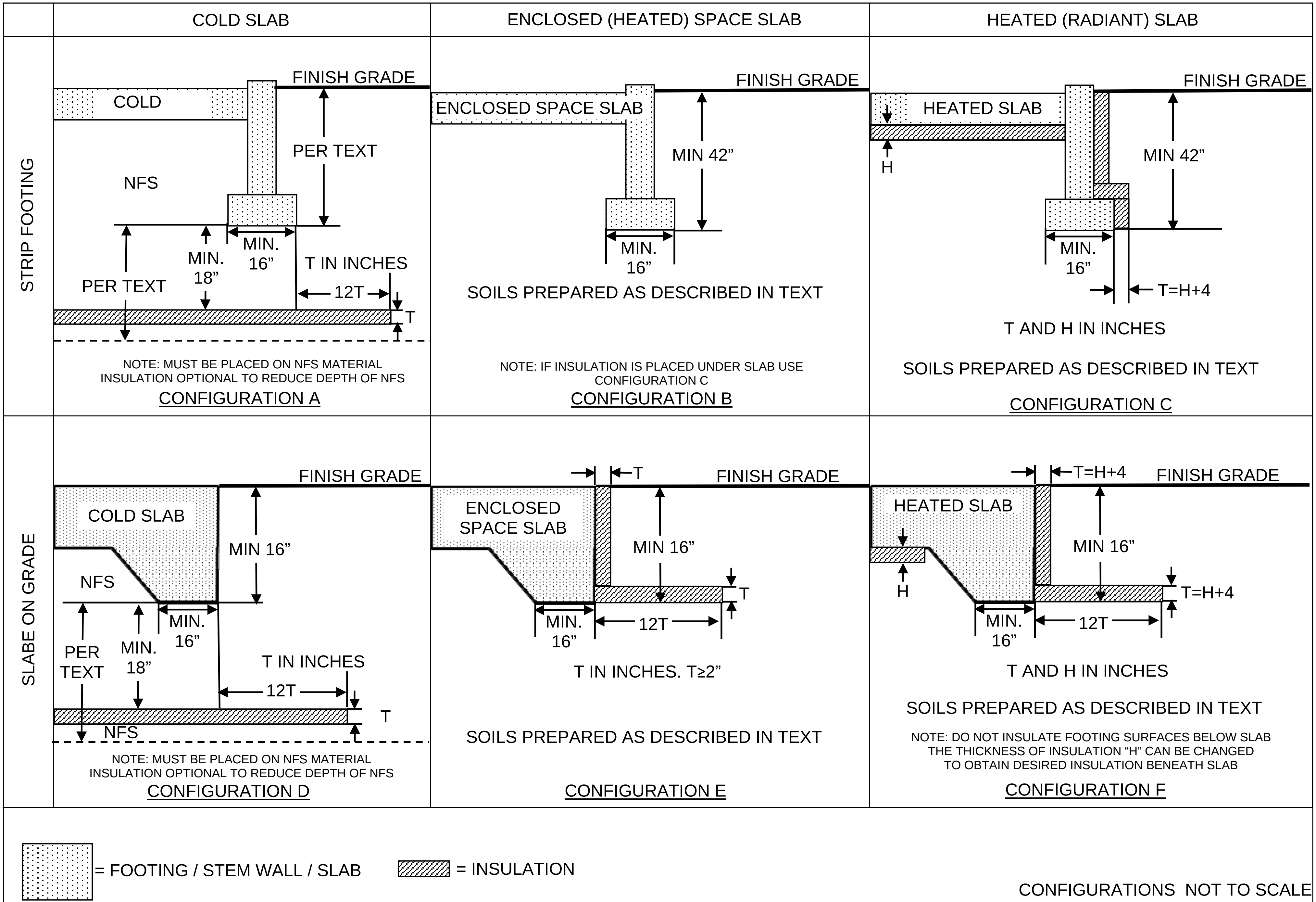
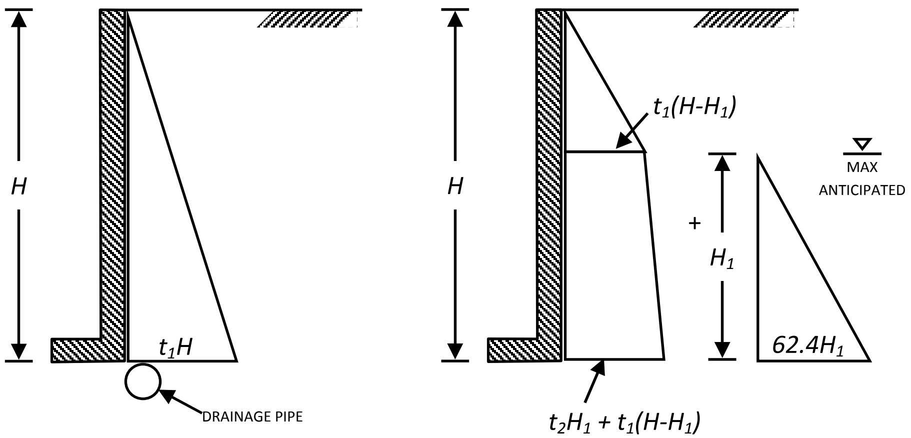
FIGURE TITLE: UNINSULATED SHALLOW FOUNDATION SCHEMATICS		
PROJECT NAME: EAGLE ROCK BOAT LAUNCH IMPROVEMENTS	PROJECT ID: 4597-16	
PROJECT LOCATION: KENAI, AK	FIGURE NUMBER: 11	



NORTHERN GEOTECHNICAL ENGINEERING, INC.
TERRA FIRMA TESTING

FIGURE TITLE: FOUNDATION INSULATION CONFIGURATIONS	
PROJECT NAME: EAGLE ROCK BOAT LAUNCH IMPROVEMENTS	PROJECT ID: 4597-16
PROJECT LOCATION: KENAI, AK	FIGURE NUMBER: 12

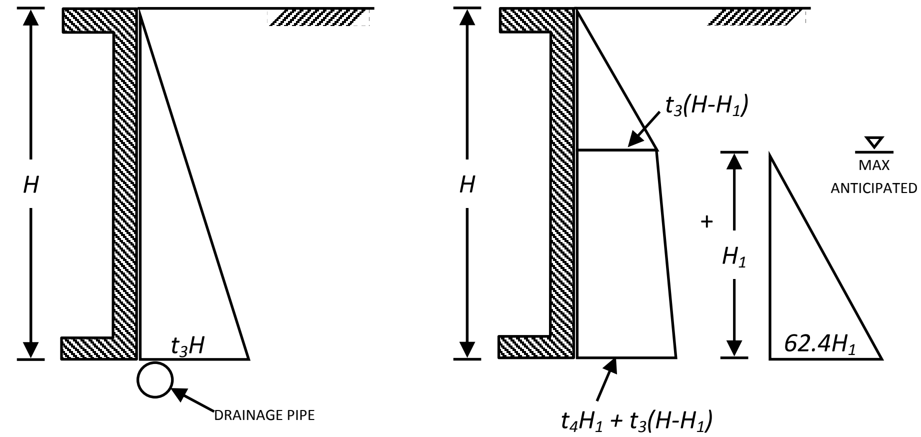
ACTIVE PRESSURE CONDITION



WITH DRAINAGE

WITHOUT DRAINAGE

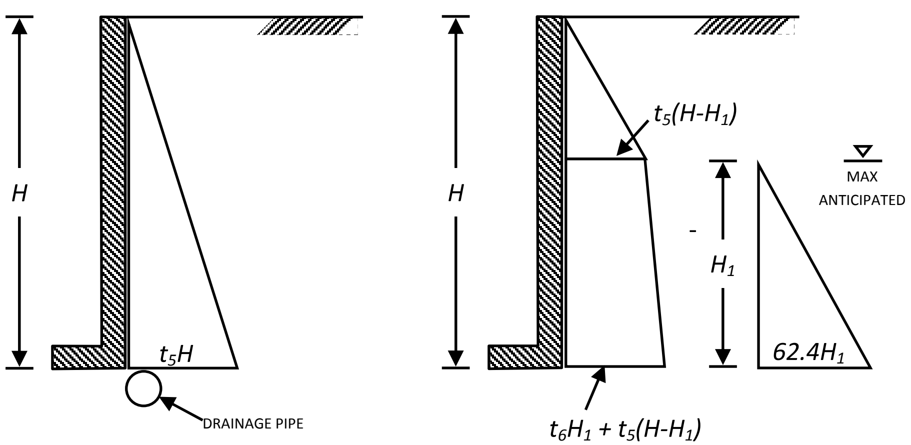
AT-REST PRESSURE CONDITION



WITH DRAINAGE

WITHOUT DRAINAGE

PASSIVE PRESSURE CONDITION

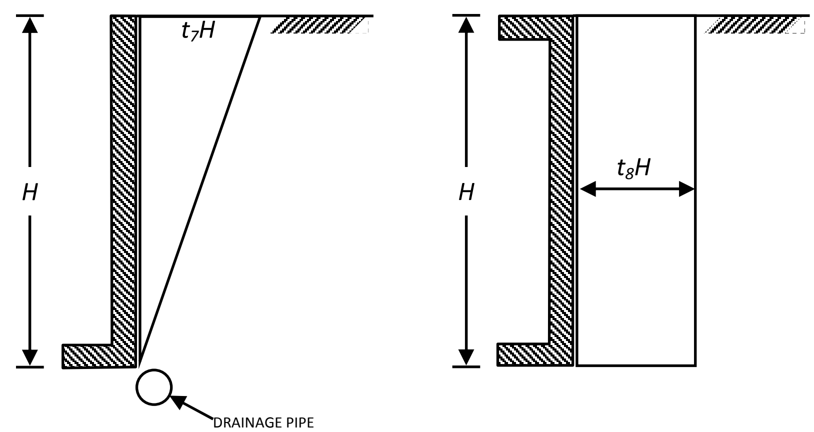


WITH DRAINAGE

WITHOUT DRAINAGE

NOTE: WALLS CAN BE EITHER FREE OR RESTRAINED AT THE TOP FOR THE PASSIVE PRESSURE CONDITION. EQUATIONS ARE ONLY VALID FOR UNITS OF $t_{1\text{-}8}$ (PCF) AND H-H_1(FT).

SEISMIC



ACTIVE

AT-REST

NOTE: SEISMIC LOADS ARE VALID FOR WALLS RETAINING LESS THAN 8 FEET VERTICAL OF EARTH. THE SEISMIC LOAD IS ADDED TO ACTIVE & AT-REST CONDITIONS AND IS SUBTRACTED FROM PASSIVE CONDITIONS.

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FIGURE TITLE: LATERAL RETAINING WALL PRESSURES	
PROJECT NAME: EAGLE ROCK BOAT LAUNCH IMPROVEMENTS	PROJECT ID: 4597-16
PROJECT LOCATION: KENAI, AK	FIGURE NUMBER: 13

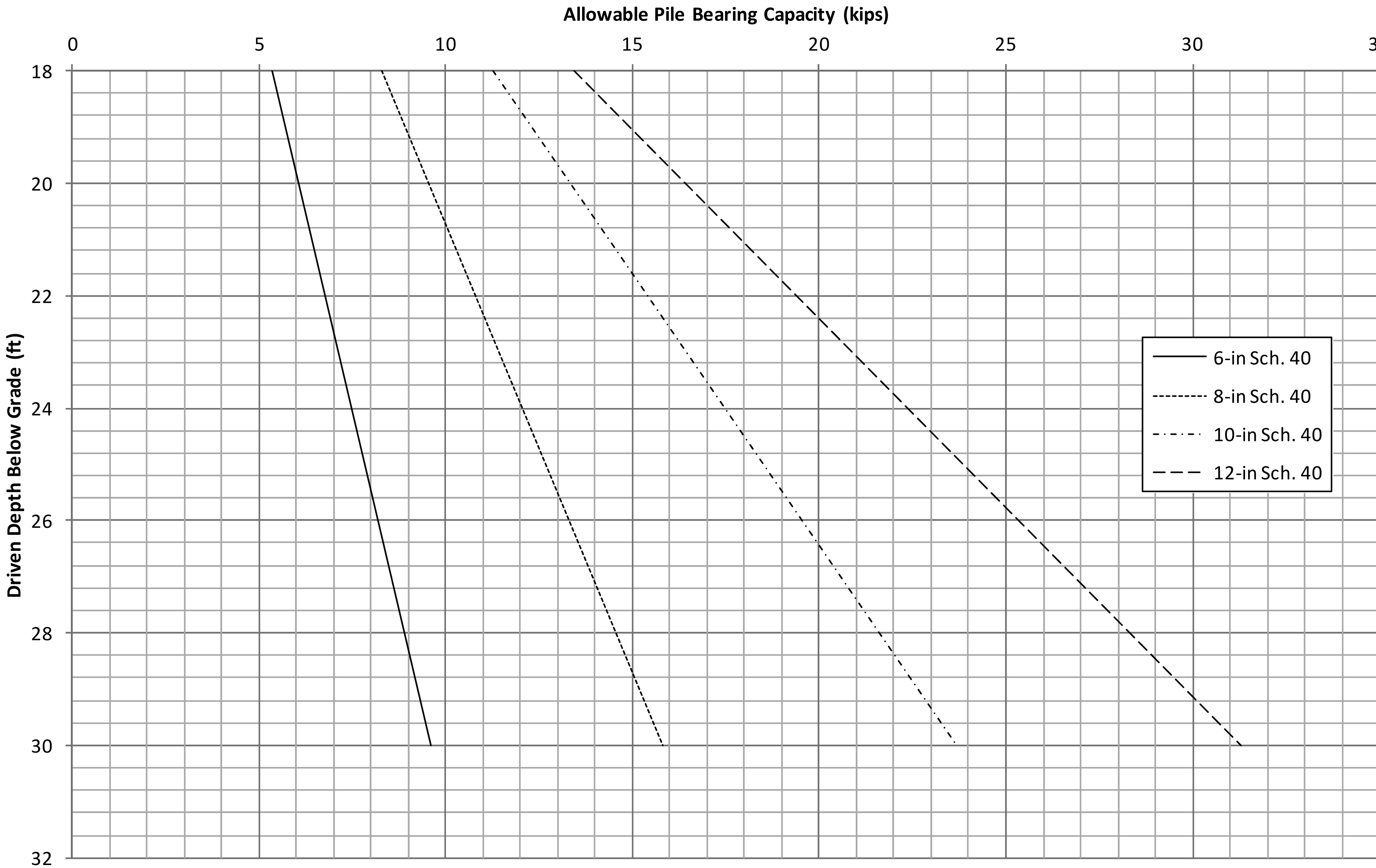

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FIGURE TITLE: ALLOWABLE AXIAL PILE CAPACITIES	
PROJECT NAME: EAGLE ROCK BOAT LAUNCH IMPROVEMENTS	PROJECT ID: 4597-16
PROJECT LOCATION: KENAI, AK	FIGURE NUMBER: 14

APPENDIX A

GRAPHICAL EXPLORATION LOGS AND SPLIT-SPOON SAMPLE PHOTOGRAPHS

Northern Geotechnical Engineering Inc.
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Fax: 907-344-5993

EXPLORATION KENB1

NGE-TFT PROJECT NAME: Kenai & Kasilof Boat Ramps
NGE-TFT PROJECT NUMBER: 4597-16 (A)
PROJECT LOCATION: Kenai & Kasilof, AK
EXPLORATION CONTRACTOR: Discovery Drilling, Inc.
EXPLORATION EQUIPMENT: Track-mounted CME 55
EXPLORATION METHOD: Hollow Stem Auger w/ Center Drill Rods
SAMPLING METHOD: Modified Split-spoon w/ 340lb autohammer
LOGGED BY: C. Banzhaf
DATE/TIME STARTED: 12/15/2016 @ 9:30:00 AM
DATE/TIME COMPLETED: 12/15/2016 @ 11:20:00 AM
EXPLORATION LOCATION: Kenai River
GROUND ELEVATION: Approx. 18 ft amsl
▽ **GROUNDWATER (ATD):** Approx. 9.8 ft bgs
▼ **GROUNDWATER ():** N/A
EXPLORATION COMPLETION: Backfilled with cuttings
WEATHER CONDITIONS: Cloudy, 16° F

DEPTH (ft)	MATERIAL DESCRIPTION	SAMPLE NUMBER	RECOVERY (in)	FIELD BLOWS	$(N_1)_{60}$	LAB RESULTS	REMARKS/NOTES
0	Dark brown, organic mat **SAND WITH SILT** (SP), trace organics, gray and brown	S1	18	9 8 2	N/A	S1 MC = 53.2%	6 in of snow cover.
	SILT WITH SAND (ML), soft, gray, moist **SILT WITH PEAT** (ML), soft, fibrous, rootlets	S2	12	1 1 1	2	S2 MC = 80.4%	
5	**SILT** (ML), trace organics, soft, gray, moist	S3	12	1 1 2	2	S3 MC = 46.3% P200 = 91.3%	
	SANDY SILT (ML), trace organics, medium dense, gray, moist ▽	S4	9	2 7 12	22	S4 MC = 26.1% 0.7% gravel, 48.5% sand, 50.8% silt	
10	**SAND** (SP), medium dense, gray, medium grained	S5	18	5 6 11	18	S5 MC = 23.8%	
	SILT WITH SAND (ML), gray, fine grained **SAND WITH GRAVEL** (SP), dense, gray, medium grained, subangular to subrounded gravel, fine gravel						
15		S6	6	12 15 14	30	S6 MC = 10.2%	
20	**GRAVEL** (GP), with sand, dense, gray	S7	12	15 18 22	42	S7 MC = 6.6%	
25	**GRAVEL WITH SAND** (SP), medium dense, gray, medium grained, subrounded gravel, fine gravel	S8	12	5 9 12	20	S8 MC = 7.9% 51.6% gravel, 45.0% sand, 3.4% silt	
30	**SAND** (SP), dense, gray, fine grained	S9	6	12 24 17	39	S9 MC = 22.5%	

Bottom of borehole at 31.5 ft bgs.

Always refer to our complete geotechnical report for this project for a more detailed explanation of the subsurface conditions at the project site and how they may affect any existing and/or prospective project site development.

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Fax: 907-344-5993

PHOTO APPENDIX

CLIENT State of Alaska Department of Natural Resources

PROJECT NAME Kenai & Kasilof Boat Ramps

PROJECT NUMBER 4597-16 (A)

PROJECT LOCATION Kenai & Kasilof, AK

Exploration KENB1 Sample S1
Sample Interval 0 - 1.5 ft bgs

Exploration KENB1 Sample S2
Sample Interval 2.5 - 4 ft bgs

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PHOTO APPENDIX

CLIENT State of Alaska Department of Natural Resources

PROJECT NAME Kenai & Kasilof Boat Ramps

PROJECT NUMBER 4597-16 (A)

PROJECT LOCATION Kenai & Kasilof, AK

Exploration KENB1 Sample S3
Sample Interval 5 - 6.5 ft bgs

Exploration KENB1 Sample S4
Sample Interval 7.5 - 9 ft bgs

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PHOTO APPENDIX

CLIENT State of Alaska Department of Natural Resources **PROJECT NAME** Kenai & Kasilof Boat Ramps

PROJECT NUMBER 4597-16 (A) **PROJECT LOCATION** Kenai & Kasilof, AK

Exploration KENB1 Sample S5
Sample Interval 10 - 11.5 ft bgs

Exploration KENB1 Sample S6
Sample Interval 15 - 16.5 ft bgs

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PHOTO APPENDIX

CLIENT State of Alaska Department of Natural Resources

PROJECT NAME Kenai & Kasilof Boat Ramps

PROJECT NUMBER 4597-16 (A)

PROJECT LOCATION Kenai & Kasilof, AK

Exploration KENB1 Sample S7
Sample Interval 20 - 21.5 ft bgs

Exploration KENB1 Sample S8
Sample Interval 25 - 26.5 ft bgs

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PHOTO APPENDIX

CLIENT State of Alaska Department of Natural Resources
PROJECT NAME Kenai & Kasilof Boat Ramps
PROJECT NUMBER 4597-16 (A)
PROJECT LOCATION Kenai & Kasilof, AK

Exploration KENB1 Sample S9
Sample Interval 30 - 31.5 ft bgs

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Anchorage, AK 99515
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Fax: 907-344-5993

EXPLORATION KENB2

NGE-TFT PROJECT NAME: Kenai & Kasilof Boat Ramps
NGE-TFT PROJECT NUMBER: 4597-16 (A)
PROJECT LOCATION: Kenai & Kasilof, AK
EXPLORATION CONTRACTOR: Discovery Drilling, Inc.
EXPLORATION EQUIPMENT: Track-mounted CME 55
EXPLORATION METHOD: Hollow Stem Auger w/ Center Drill Rods
SAMPLING METHOD: Modified Split-spoon w/ 340lb autohammer
LOGGED BY: C. Banzhaf
DATE/TIME STARTED: 12/15/2016 @ 11:35:00 AM
DATE/TIME COMPLETED: 12/15/2016 @ 12:45:00 PM
EXPLORATION LOCATION: Kenai River
GROUND ELEVATION: Approx. 19 ft amsl
▽ **GROUNDWATER (ATD):** Approx. 13.0 ft bgs
▼ **GROUNDWATER ():** N/A
EXPLORATION COMPLETION: Backfilled with cuttings
WEATHER CONDITIONS: Cloudy, 18° F

DEPTH (ft)	MATERIAL DESCRIPTION	SAMPLE NUMBER	RECOVERY (in)	FIELD BLOWS	$(N_1)_{60}$	LAB RESULTS	REMARKS/NOTES
0	**SAND WITH SILT AND GRAVEL TO GRAVEL WITH SILT AND SAND** (SP-SM), loose, brown, moist, medium grained, subrounded gravel, fine gravel, FILL	S1	16	17 18 9	N/A	S1 MC = 4.5% 44.3% gravel, 48.9% sand, 6.8% silt P0.02 = 4.8% FC = PFS	6 in of snow cover.
		S2	18	11 2 3	8	S2 MC = 7.6% 46.3% gravel, 46.1% sand, 7.6% silt P0.02 = 5.0% FC = S1	
5	**PEAT** (PT), dark brown, moist, NATIVE	S3	18	1 0 1	1	S3 MC = 510.2% OC = 73.0%	
	SAND (SP), very loose, gray, moist						
	PEAT (PT), dark brown **SAND** (SP), gray, medium grained **PEAT** (PT), soft, dark brown, moist	S4	18	0 1 0	1	S4 MC = 364.3% OC = 62.6%	
10	**SILT WITH SAND** (ML), and organics, medium dense, gray, moist ▽	S5	18	1 1 1	2	S5 MC = 39.4% P200 = 85.8%	
15	**SAND WITH GRAVEL** (SP), medium dense, gray, medium grained, subrounded gravel	S6	12	3 6 7	13	S6 MC = 9.7%	
20		S7	12	13 11 11	23	S7 MC = 7.8%	

Bottom of borehole at 21.5 ft bgs.

Always refer to our complete geotechnical report for this project for a more detailed explanation of the subsurface conditions at the project site and how they may affect any existing and/or prospective project site development.

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PHOTO APPENDIX

CLIENT State of Alaska Department of Natural Resources

PROJECT NAME Kenai & Kasilof Boat Ramps

PROJECT NUMBER 4597-16 (A)

PROJECT LOCATION Kenai & Kasilof, AK

Exploration KENB2 Sample S1
Sample Interval 0 - 1.5 ft bgs

Exploration KENB2 Sample S2
Sample Interval 2.5 - 4 ft bgs

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CLIENT State of Alaska Department of Natural Resources

PROJECT NAME Kenai & Kasilof Boat Ramps

PROJECT NUMBER 4597-16 (A)

PROJECT LOCATION Kenai & Kasilof, AK

Exploration KENB2 Sample S3
Sample Interval 5 - 6.5 ft bgs

Exploration KENB2 Sample S4
Sample Interval 7.5 - 9 ft bgs

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CLIENT State of Alaska Department of Natural Resources **PROJECT NAME** Kenai & Kasilof Boat Ramps

PROJECT NUMBER 4597-16 (A) **PROJECT LOCATION** Kenai & Kasilof, AK

Exploration KENB2 Sample S5
Sample Interval 10 - 11.5 ft bgs

Exploration KENB2 Sample S6
Sample Interval 15 - 16.5 ft bgs

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CLIENT State of Alaska Department of Natural Resources

PROJECT NAME Kenai & Kasilof Boat Ramps

PROJECT NUMBER 4597-16 (A)

PROJECT LOCATION Kenai & Kasilof, AK

Exploration KENB2 Sample S7
Sample Interval 20 - 21.5 ft bgs

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EXPLORATION KENB3

NGE-TFT PROJECT NAME: Kenai & Kasilof Boat Ramps
NGE-TFT PROJECT NUMBER: 4597-16 (A)
PROJECT LOCATION: Kenai & Kasilof, AK
EXPLORATION CONTRACTOR: Discovery Drilling, Inc.
EXPLORATION EQUIPMENT: Track-mounted CME 55
EXPLORATION METHOD: Hollow Stem Auger w/ Center Drill Rods
SAMPLING METHOD: Modified Split-spoon w/ 340lb autohammer
LOGGED BY: C. Banzhaf
DATE/TIME STARTED: 12/15/2016 @ 1:10:00 PM
DATE/TIME COMPLETED: 12/15/2016 @ 2:15:00 PM
EXPLORATION LOCATION: Kenai River
GROUND ELEVATION: Approx. 19 ft amsl
GROUNDWATER (ATD): Approx. 8.0 ft bgs
GROUNDWATER (): N/A
EXPLORATION COMPLETION: Backfilled with cuttings
WEATHER CONDITIONS: Cloudy, 18° F

DEPTH (ft)	MATERIAL DESCRIPTION	SAMPLE NUMBER	RECOVERY (in)	FIELD BLOWS	$(N_1)_{60}$	LAB RESULTS	REMARKS/NOTES
0	Dark brown, organic mat **PEAT** (PT), very loose, dark brown, moist	S1	6	1 2 0	N/A	S1 MC = 405.2%	6 in of snow cover.
		S2	4	0 1 0	2	S2 MC = 401.4%	
5		S3	18	0 0 1	2	S3 MC = 533.3%	
	SAND (SP), medium dense, gray, wet, medium grained						
	▽	S4	18	1 5 5	13	S4 MC = 24.0% 0.0% gravel, 97.4% sand, 2.6% silt	
10	**SAND WITH GRAVEL** (SP), medium dense, gray, medium grained, subrounded gravel	S5	18	5 9 11	23	S5 MC = 9.6% 44.4% gravel, 51.4% sand, 4.2% silt	
15			0		N/A		2 ft of sand heave in augers.
20		S6	6	19 12	N/A	S6 MC = 10.3%	Thought sampler was full.

Bottom of borehole at 21.5 ft bgs.

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CLIENT State of Alaska Department of Natural Resources

PROJECT NAME Kenai & Kasilof Boat Ramps

PROJECT NUMBER 4597-16 (A)

PROJECT LOCATION Kenai & Kasilof, AK

Exploration KENB3 Sample S1
Sample Interval 0 - 1.5 ft bgs

Exploration KENB3 Sample S2
Sample Interval 2.5 - 4 ft bgs

Exploration KENB3 Sample S3
Sample Interval 5 - 6.5 ft bgs

Exploration KENB3 Sample S4
Sample Interval 7.5 - 9 ft bgs

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CLIENT State of Alaska Department of Natural Resources

PROJECT NAME Kenai & Kasilof Boat Ramps

PROJECT NUMBER 4597-16 (A)

PROJECT LOCATION Kenai & Kasilof, AK

Exploration KENB3 Sample S5
Sample Interval 10 - 11.5 ft bgs

Exploration KENB3 Sample S6
Sample Interval 20 - 21.5 ft bgs

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EXPLORATION KENB4

NGE-TFT PROJECT NAME: Kenai & Kasilof Boat Ramps
NGE-TFT PROJECT NUMBER: 4597-16 (A)
PROJECT LOCATION: Kenai & Kasilof, AK
EXPLORATION CONTRACTOR: Discovery Drilling, Inc.
EXPLORATION EQUIPMENT: Track-mounted CME 55
EXPLORATION METHOD: Hollow Stem Auger w/ Center Drill Rods
SAMPLING METHOD: Modified Split-spoon w/ 340lb autohammer
LOGGED BY: C. Banzhaf
DATE/TIME STARTED: 12/15/2016 @ 2:50:00 PM
DATE/TIME COMPLETED: 12/15/2016 @ 4:15:00 PM
EXPLORATION LOCATION: Kenai River
GROUND ELEVATION: Approx. 18 ft amsl
GROUNDWATER (ATD): N/E
GROUNDWATER (): N/A
EXPLORATION COMPLETION: Backfilled with cuttings
WEATHER CONDITIONS: Cloudy, 18° F

DEPTH (ft)	MATERIAL DESCRIPTION	SAMPLE NUMBER	RECOVERY (in)	FIELD BLOWS	$(N_1)_{60}$	LAB RESULTS	REMARKS/NOTES
0	Dark brown, organic mat **PEAT WITH SILT** (PT), very loose, gray, moist	S1	12	4 2 1	N/A	S1 MC = 210.9% OC = 18.8%	6 in of snow cover.
	PEAT (PT), very loose to loose, brown, moist, fibrous	S2	16	1 1 0	2	S2 MC = 358.6% OC = 81.4%	
5		S3	18	1 0 5	8	S3 MC = 13.0%	
	SILTY SAND (SM), gray, moist **SAND WITH GRAVEL** (GP), dense, gray, moist, medium to coarse grained, subrounded gravel						
		S4	18	8 12 19	38	S4 MC = 8.9% 53.8% gravel, 42.6% sand, 3.6% silt	
10		S5	16	12 13 16	31	S5 MC = 7.6%	
15		S6	16	17 15 22	38	S6 MC = 8.8%	Fractured rock. Rocky/cobbles drilling from 17 - 20 ft bgs.
20		S7	2	7 13 25 2"	N/A	S7 MC = 13.4%	

Bottom of borehole at 21.2 ft bgs.

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CLIENT State of Alaska Department of Natural Resources **PROJECT NAME** Kenai & Kasilof Boat Ramps

PROJECT NUMBER 4597-16 (A) **PROJECT LOCATION** Kenai & Kasilof, AK

Exploration KENB4 Sample S1A
Sample Interval 0 - 1.5 ft bgs

Exploration KENB4 Sample S1B
Sample Interval 0 - 1.5 ft bgs

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CLIENT State of Alaska Department of Natural Resources

PROJECT NAME Kenai & Kasilof Boat Ramps

PROJECT NUMBER 4597-16 (A)

PROJECT LOCATION Kenai & Kasilof, AK

Exploration KENB4 Sample S2
Sample Interval 2.5 - 4 ft bgs

Exploration KENB4 Sample S3
Sample Interval 5 - 6.5 ft bgs

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PROJECT NAME Kenai & Kasilof Boat Ramps

PROJECT NUMBER 4597-16 (A)

PROJECT LOCATION Kenai & Kasilof, AK

Exploration KENB4 Sample S4
Sample Interval 7.5 - 9 ft bgs

Exploration KENB4 Sample S5
Sample Interval 10 - 11.5 ft bgs

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PROJECT NAME Kenai & Kasilof Boat Ramps

PROJECT NUMBER 4597-16 (A)

PROJECT LOCATION Kenai & Kasilof, AK

Exploration KENB4 Sample S6
Sample Interval 15 - 16.5 ft bgs

Exploration KENB4 Sample S7
Sample Interval 20 - 21.2 ft bgs

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EXPLORATION KENB5

NGE-TFT PROJECT NAME: Kenai & Kasilof Boat Ramps
NGE-TFT PROJECT NUMBER: 4597-16 (A)
PROJECT LOCATION: Kenai & Kasilof, AK
EXPLORATION CONTRACTOR: Discovery Drilling, Inc.
EXPLORATION EQUIPMENT: Track-mounted CME 55
EXPLORATION METHOD: Hollow Stem Auger w/ Center Drill Rods
SAMPLING METHOD: Modified Split-spoon w/ 340lb autohammer
LOGGED BY: C. Banzhaf
DATE/TIME STARTED: 12/16/2016 @ 9:30:00 AM
DATE/TIME COMPLETED: 12/16/2016 @ 10:40:00 AM
EXPLORATION LOCATION: Kenai River
GROUND ELEVATION: Approx. 19 ft amsl
GROUNDWATER (ATD): N/E
GROUNDWATER (): N/A
EXPLORATION COMPLETION: Backfilled with cuttings
WEATHER CONDITIONS: Snowing, 25°F

DEPTH (ft)	GRAPHIC LOG	FROZEN SOILS	MATERIAL DESCRIPTION	SAMPLE TYPE	SAMPLE COLLECTED	SAMPLE NUMBER	RECOVERY (in)	FIELD BLOWS	$(N_1)_{60}$	LAB RESULTS	REMARKS/NOTES
0			Dark brown, organic mat **SILTY SAND WITH GRAVEL** (SM), brown, medium grained, subangular to subrounded gravel			S1	18	17 23 28	N/A	S1 MC = 9.4% 19.0% gravel, 65.6% sand, 15.4% silt P0.02 = 9.9% FC = F2	10 in of snow cover.
			SILT WITH SAND (ML), trace organics, gray, moist, medium grained			S2	18	7 1 2	N/A	S2 MC = 77.8% P200 = 70.3%	
5			**SILT** (ML), trace organics, and roots, soft, gray, damp to moist, fine grained, 1/2 in diameter stick debris			S3	18	2 2 2	3	S3 MC = 30.9% P200 = 95.3%	
			PEAT (PT), very loose, brown, moist to damp, fibrous			S4	16	0 1 1	1	S4 MC = 364.3% OC = 72.8%	
10			**SILT WITH SAND** (ML), gray, moist, fine grained **SAND** (SP), medium dense, gray, moist, medium grained			S5	12	1 4 22	21	S5 MC = 35.2%	Fractured rock in shoe.
15			**GRAVEL WITH SILT AND SAND** (GP-GM), medium dense, gray, medium grained, subangular to subrounded gravel			S6	16	11 12 11	22	S6 MC = 7.5% 61.4% gravel, 33.6% sand, 5.0% silt	
20						S7	10	15 10 10	19	S7 MC = 8.0%	

Bottom of borehole at 21.5 ft bgs.

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CLIENT State of Alaska Department of Natural Resources

PROJECT NAME Kenai & Kasilof Boat Ramps

PROJECT NUMBER 4597-16 (A)

PROJECT LOCATION Kenai & Kasilof, AK

Exploration KENB5 Sample S1
Sample Interval 0 - 1.5 ft bgs

Exploration KENB5 Sample S2
Sample Interval 2.5 - 4 ft bgs

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CLIENT State of Alaska Department of Natural Resources

PROJECT NAME Kenai & Kasilof Boat Ramps

PROJECT NUMBER 4597-16 (A)

PROJECT LOCATION Kenai & Kasilof, AK

Exploration KENB5 Sample S3
Sample Interval 5 - 6.5 ft bgs

Exploration KENB5 Sample S4
Sample Interval 7.5 - 9 ft bgs

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CLIENT State of Alaska Department of Natural Resources

PROJECT NAME Kenai & Kasilof Boat Ramps

PROJECT NUMBER 4597-16 (A)

PROJECT LOCATION Kenai & Kasilof, AK

Exploration KENB5 Sample S5
Sample Interval 10 - 11.5 ft bgs

Exploration KENB5 Sample S6
Sample Interval 15 - 16.5 ft bgs

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Exploration KENB5 Sample S7
Sample Interval 20 - 21.5 ft bgs

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EXPLORATION LEGEND

CLIENT State of Alaska Department of Natural Resources

NGE-TFT PROJECT NAME Kenai & Kasilof Boat Ramps

NGE-TFT PROJECT NUMBER 4597-16 (A)

PROJECT LOCATION Kenai & Kasilof, AK

LITHOLOGIC SYMBOLS (Unified Soil Classification System)

- GP: USCS Poorly-graded Gravel
- GP-GM: USCS Poorly-graded Gravel with Silt
- GPS: Sandy Gravel
- ML: USCS Silt
- MLS: Sandy Silt
- PT: USCS Peat
- SM: USCS Silty Sand
- SP: USCS Poorly-graded Sand
- SPG: Gravelly Sand
- SP-SM: USCS Poorly-graded Sand with Silt
- TOPSOIL: Topsoil

SAMPLER SYMBOLS

- Modified Penetration Test
- No Recovery

WELL CONSTRUCTION SYMBOLS

ABBREVIATIONS

LL - LIQUID LIMIT (%)
PI - PLASTIC INDEX (%)
MC - MOISTURE CONTENT (%)
DD - DRY DENSITY (PCF)
NP - NON PLASTIC
P200 - PERCENT PASSING NO. 200 SIEVE
P0.02- PERCENT PASSING 0.02mm SIEVE
PP - POCKET PENETROMETER (tons/ft^2)
S/U - CASING STICK-UP

TV - TORVANE
PID - PHOTOIONIZATION DETECTOR
UC - UNCONFINED COMPRESSION
ppm - PARTS PER MILLION
Water Level at Time Drilling, or as Shown
Water Level After 24 Hours, or as Shown

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SOIL CLASSIFICATION CHART

CLIENT State of Alaska Department of Natural Resources

PROJECT NAME Kenai & Kasilof Boat Ramps

NGE-TFT PROJECT NUMBER 4597-16 (A)

PROJECT LOCATION Kenai & Kasilof, AK

MAJOR DIVISIONS			SYMBOLS		TYPICAL DESCRIPTIONS
			GRAPH	LETTER	
COARSE GRAINED SOILS MORE THAN 50% OF MATERIAL IS LARGER THAN NO. 200 SIEVE SIZE	GRAVEL AND GRAVELLY SOILS MORE THAN 50% OF COARSE FRACTION RETAINED ON NO. 4 SIEVE	CLEAN GRAVELS (LITTLE OR NO FINES)		GW	WELL-GRADED GRAVELS, GRAVEL - SAND MIXTURES, LITTLE OR NO FINES
				GP	POORLY-GRADED GRAVELS, GRAVEL - SAND MIXTURES, LITTLE OR NO FINES
		GRAVELS WITH FINES (APPRECIABLE AMOUNT OF FINES)		GM	SILTY GRAVELS, GRAVEL - SAND - SILT MIXTURES
				GC	CLAYEY GRAVELS, GRAVEL - SAND - CLAY MIXTURES
	SAND AND SANDY SOILS MORE THAN 50% OF COARSE FRACTION PASSING ON NO. 4 SIEVE	CLEAN SANDS (LITTLE OR NO FINES)		SW	WELL-GRADED SANDS, GRAVELLY SANDS, LITTLE OR NO FINES
				SP	POORLY-GRADED SANDS, GRAVELLY SAND, LITTLE OR NO FINES
		SANDS WITH FINES (APPRECIABLE AMOUNT OF FINES)		SM	SILTY SANDS, SAND - SILT MIXTURES
				SC	CLAYEY SANDS, SAND - CLAY MIXTURES
FINE GRAINED SOILS MORE THAN 50% OF MATERIAL IS SMALLER THAN NO. 200 SIEVE SIZE	SILTS AND CLAYS	LIQUID LIMIT LESS THAN 50		ML	INORGANIC SILTS AND VERY FINE SANDS, ROCK FLOUR, SILTY OR CLAYEY FINE SANDS OR CLAYEY SILTS WITH SLIGHT PLASTICITY
				CL	INORGANIC CLAYS OF LOW TO MEDIUM PLASTICITY, GRAVELLY CLAYS, SANDY CLAYS, SILTY CLAYS, LEAN CLAYS
				OL	ORGANIC SILTS AND ORGANIC SILTY CLAYS OF LOW PLASTICITY
	SILTS AND CLAYS	LIQUID LIMIT GREATER THAN 50		MH	INORGANIC SILTS, MICACEOUS OR DIATOMACEOUS FINE SAND OR SILTY SOILS
				CH	INORGANIC CLAYS OF HIGH PLASTICITY
				OH	ORGANIC CLAYS OF MEDIUM TO HIGH PLASTICITY, ORGANIC SILTS
HIGHLY ORGANIC SOILS				PT	PEAT, HUMUS, SWAMP SOILS WITH HIGH ORGANIC CONTENTS

NOTE: DUAL SYMBOLS ARE USED TO INDICATE BORDERLINE SOIL CLASSIFICATIONS.
DIAGONAL LINES INDICATE UNKNOWN DEPTH OF SOIL TRANSITION.

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EXPLORATION LOG KEY

CLIENT State of Alaska Department of Natural Resources
PROJECT NAME Kenai & Kasilof Boat Ramps
NGE-TFT PROJECT NUMBER 4597-16 (A)
PROJECT LOCATION Kenai & Kasilof, AK

SAMPLER SYMBOLS

SPT w/ 140# Hammer
30" Drop and 2.0" O.D. Sampler

Modified SPT w/ 340# Hammer
30" Drop and 3.0 O.D. Sampler

Grab Sample

Shelby Tube Sample

Rock Core Sample

Direct Push Sample

No Recovery

N/E Not Encountered

COMPONENT DEFINITIONS

COMPONENT	SIZE RANGE
Boulders	Larger than 12 in
Cobbles	3 in to 12 in
Gravel	3 in to No. 4 (4.5mm)
Coarse gravel	3 in to 3/4 in
Fine gravel	3/4 in to No. 4 (4.5 mm)
Sand	No. 4 (4.5 mm) to No. 200
Coarse sand	No. 4 (4.5 mm) to No. 10 (2.0 mm)
Medium sand	No. 10 (2.0 mm) to No. 40 (0.42 mm)
Fine sand	No. 40 (0.42 mm) to No. 200 (0.074 mm)
Silt and Clay	Smaller than No. 200 (0.074 mm)

COMPONENT PROPORTIONS

DESCRIPTIVE TERMS	RANGE OF PROPORTION
Trace	1-5%
Few	5-10%
Little	10-20%
Some	20-35%
And	35-50%

WELL SYMBOLS

1" Slotted Pipe
Backfilled with Silica Sand

1" PVC Pipe
Backfilled with Auger Cuttings

1" PVC Pipe
with Bentonite Seal

Capped Riser

MOISTURE CONTENT

DRY	Absence of moisture, dusty, dry to the touch
DAMP	Some perceptible moisture; below optimum
MOIST	No visible water; near optimum moisture content
WET	Visible free water, usually soil is below water table

RELATIVE DENSITY OR CONSISTENCY VERSUS SPT N-VALUE

COHESIONLESS SOILS			COHESIVE SOILS		
DENSITY	N (BLOWS/FT)	APPROXIMATE RELATIVE DENSITY (%)	CONSISTENCY	N (BLOWS/FT)	APPROXIMATE UNDRAINED SHEAR STRENGTH (PSF)
VERY LOOSE	0-4	0-15	VERY SOFT	0-1	< 250
LOOSE	5-10	15-35	SOFT	2-4	250-500
MEDIUM DENSE	11-25	35-65	MEDIUM STIFF	5-8	500-1000
DENSE	26-50	65-85	STIFF	9-15	1000-2000
VERY DENSE	> 50	85-100	VERY STIFF	16-30	2000-4000
			HARD	> 30	> 4000

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EXPLORATION LOG KEY

CLIENT State of Alaska Department of Natural Resources

PROJECT NAME Kenai & Kasilof Boat Ramps

NGE-TFT PROJECT NUMBER 4597-16 (A)

PROJECT LOCATION Kenai & Kasilof, AK

FROST DESIGN SOIL CLASSIFICATION

FROST GROUP (USACOE)	FROST GROUP (M.O.A.)	SOIL TYPE	% FINER THAN 0.02mm BY MASS	TYPICAL SOIL TYPES UNDER UNIFIED SOIL CLASSIFICATION SYSTEM
NFS*	NFS*	(A) GRAVELS CRUSHED STONE CRUSHED ROCK (B) SANDS	0 - 1.5 0 - 3	GW, GP SW, SP
PFS+	NFS*	(A) GRAVELS CRUSHED STONE CRUSHED ROCK	1.5 - 3	GW, GP
	F2	(B) SANDS	3 - 10	SW, SP
S1	F1	GRAVELLY SOILS	3 - 6	GW, GP, GW-GM, GP-GM
S2	F2	SANDY SOILS	3 - 6	SW, SP, SW-SM, SP-SM
F1	F1	GRAVELLY SOILS	6 - 10	GM, GW-GM, GP-GM
F2	F2	(A) GRAVELLY SOILS (B) SANDS	10 - 20 6 - 15	GM, GW-GM, GP-GM SM, SW-SM, SP-SM
F3	F3	(A) GRAVELLY SOILS (B) SANDS, EXCEPT VERY FINE SILTY SANDS (C) CLAYS, PI>12	Over 20 Over 15 - - - - - -	GM, GC SM, SC CL, CH
F4	F4	(A) ALL SILTS (B) VERY FINE SILTY SANDS (C) CLAYS, PI<12 (D) VARVED CLAYS AND OTHER FINE GRAINED, BANDED SEDIMENTS	- - - - - - Over 15 - - - - - - - - - - - -	ML, MH SM CL, CL-ML CL & ML; CL, ML, & SM; CL, CH, & ML; CL, CH, ML, & SM

*Non-frost susceptible
+Possibly frost susceptible, but requires lab testing to determine frost design soils classification.

ICE CLASSIFICATION SYSTEM

GROUP	ICE VISIBILITY	DESCRIPTION		SYMBOL	
N	SEGREGATED ICE NOT VISIBLE BY EYE	POORLY BONDED OR FRIABLE		Nf	
		WELL BONDED	NO EXCESS ICE	Nb	Nbn
			EXCESS MICROSCOPIC ICE		Nbe
V	SEGREGATED ICE IS VISIBLE BY EYE AND IS ONE INCH OR LESS IN THICKNESS	INDIVIDUAL ICE CRYSTALS OR INCLUSIONS		Vx	
		ICE COATINGS ON PARTICLES		Vc	
		RANDOM OR IRREGULARY ORIENTED ICE		Vr	
		STRATIFIED OR DISTINCTLY ORIENTED ICE		Vs	
		UNIFORMLY DISTRIBUTED ICE		Vu	
ICE	ICE IS GREATER THAN ONE INCH IN THICKNESS	ICE WITH SOILS INCLUSIONS		ICE + Soil Type	
		ICE WITHOUT SOILS INCLUSIONS		ICE	

APPENDIX B

LABORATORY TEST DATA

Summary of Laboratory Test Results

Eagle Rock Boat Launch

NGE-TFT Project #:4597-16

Exploration ID	Sample Number	Depth Interval (ft) Top	Depth Interval (ft) Bottom	Moisture Content ASTM D2216 (% By Dry Mass)	Particle Size Analysis ASTM C136/D422/D6913 (% By Mass) Gravel	Sand	Silt/Clay	Passing #200 ASTM D1140 (% By Mass)	Passing 0.02mm ASTM D422 (% By Mass)	Frost Class.	Organic Content (ASTM D2974) (% By Mass)	Unified Soil Classification ASTM D2487
KENB1	S1	0.0	1.5	53.2								
KENB1	S2	2.5	4.0	80.4								
KENB1	S3	5.0	6.5	46.3				91.3				
KENB1	S4	7.5	9.0	26.1	0.7	48.5	50.8		N/A	N/A		(ML) Sandy silt
KENB1	S5	10.0	11.5	23.8								
KENB1	S6	15.0	16.5	10.2								
KENB1	S7	20.0	21.5	6.6								
KENB1	S8	25.0	26.5	7.9	51.6	45.0	3.4		N/A	N/A		(GW) Well-graded gravel w/ sand
KENB1	S9	30.0	31.5	22.5								
KENB2	S1	0.0	1.5	4.5	44.3	48.9	6.8		4.8	PFS		(SP-SM) Poorly-graded sand w/ silt and gravel
KENB2	S2	2.5	4.0	7.6	46.3	46.1	7.6		5.0	S1		(GP-GM) Poorly-graded gravel w/ silt and sand
KENB2	S3	5.0	6.5	510.2							73.0	
KENB2	S4	7.5	9.0	364.3							62.6	
KENB2	S5	10.0	11.5	39.4				85.8				
KENB2	S6	15.0	16.5	9.7								
KENB2	S7	20.0	21.5	7.8								
KENB3	S1	0.0	1.5	405.2								
KENB3	S2	2.5	4.0	401.4								
KENB3	S3	5.0	6.5	533.3								
KENB3	S4	7.5	9.0	24.0	0.0	97.4	2.6		N/A	N/A		(SP) Poorly-graded sand
KENB3	S5	10.0	11.5	9.6	44.4	51.4	4.2		N/A	N/A		(SP) Poorly-graded sand w/ gravel
KENB3	S6	20.0	1.5	10.3								
KENB4	S1	0.0	1.5	210.9							18.8	
KENB4	S2	2.5	4.0	358.6							81.4	
KENB4	S3	5.0	6.5	13.0								
KENB4	S4	7.5	9.0	8.9	53.8	42.6	3.6		N/A	N/A		(GW) Well-graded gravel w/ sand
KENB4	S5	10.0	11.5	7.6								
KENB4	S6	15.0	16.5	8.8								
KENB4	S7	20.0	21.5	13.4								
KENB5	S1	0.0	1.5	9.4	19.0	65.6	15.4		9.9	F2		(SM) Silty sand w/ gravel
KENB5	S2	2.5	4.0	77.8				70.3				
KENB5	S3	5.0	6.5	30.9				95.3				
KENB5	S4	7.5	9.0	364.3							72.8	
KENB5	S5	10.0	11.5	35.2								
KENB5	S6	15.0	16.5	7.5	61.4	33.6	5.0		N/A	N/A		(GP-GM) Poorly-graded gravel w/ silt and sand
KENB5	S7	20.0	21.5	8.0								

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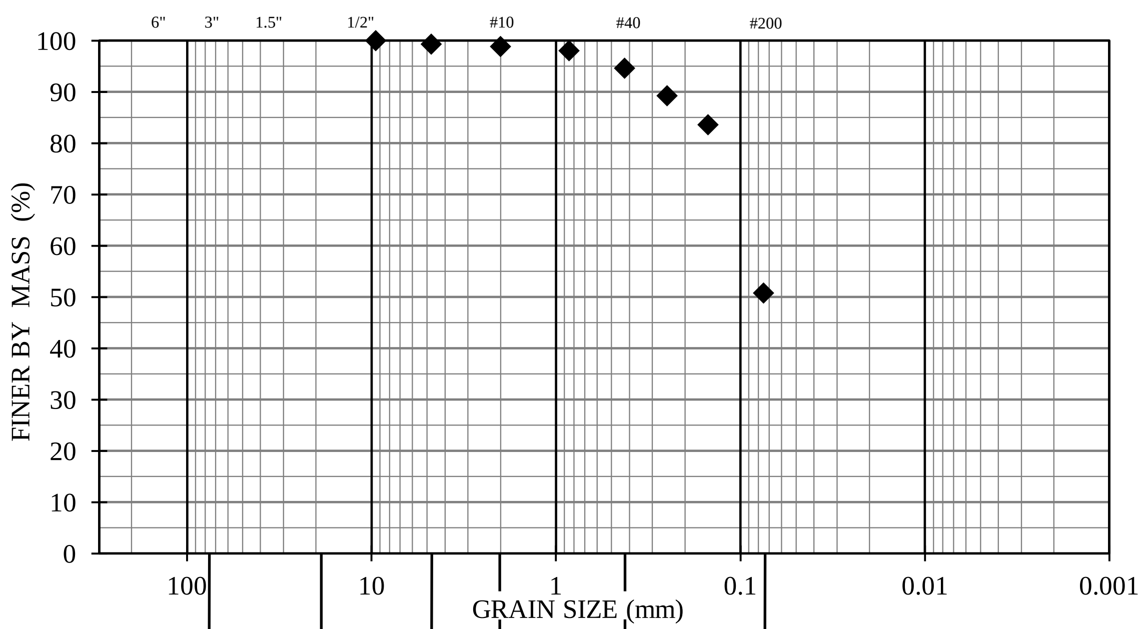
PROJECT CLIENT:	**AK DNR - DPOR**
PROJECT NAME:	**Eagle Rock Boat Launch**
PROJECT NO.:	**4597-16**
SAMPLE LOC.:	**KENB1**
NUMBER/ DEPTH:	**S4 / 7.5 - 9'**
DESCRIPTION:	**Sandy silt**
DATE RECEIVED:	**12/19/2016**
TESTED BY:	**CJK/XG**
REVIEWED BY:	**CJK**

% GRAVEL	**0.7**	USCS	**ML**
% SAND	**48.5**	USACOE FC	**N/A**
% SILT/CLAY	**50.8**	% PASS. 0.02 mm	**N/A**
% MOIST. CONTENT	**26.1**	% PASS. 0.002 mm	**N/A**
UNIFORMITY COEFFICIENT (C_u)	**UNKNOWN**		
COEFFICIENT OF GRADATION (C_c)	**UNKNOWN**		
ASTM D1557 (uncorrected)	**N/A**		
ASTM D4718 (corrected)	**N/A**		
OPTIMUM MOIST. CONTENT. (corrected)	**N/A**		

PARTICLE SIZE ANALYSIS ASTM D422 / C136

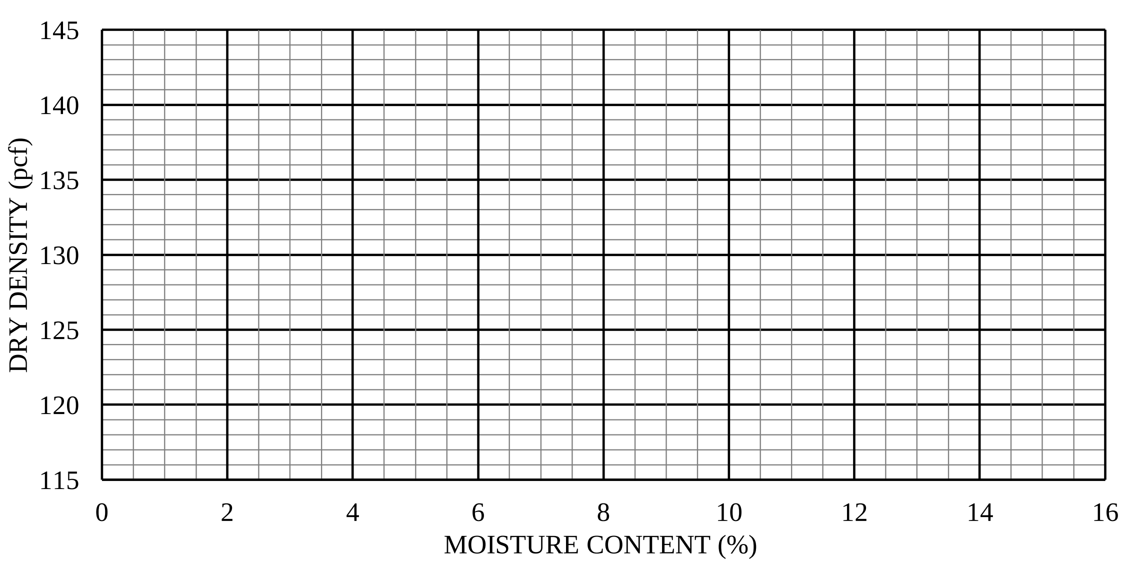


COBBLES	GRAVEL		SAND			SILT or CLAY
	Coarse	Fine	Coarse	Medium	Fine	

SIEVE ANALYSIS RESULT

SIEVE SIZE (mm)	SIEVE SIZE (U.S.)	TOTAL % PASSING	SPECIFICATION (% PASSING)
9.50	3/8"	**100**	
4.75	#4	**99**	
2.00	#10	**99**	
0.85	#20	**98**	
0.43	#40	**95**	
0.25	#60	**89**	
0.15	#100	**84**	
0.075	#200	**50.8**	

MOISTURE-DENSITY RELATIONSHIP ASTM D1557



HYDROMETER RESULT

ELAPSED TIME (MIN)	DIAMETER (mm)	TOTAL % PASSING
0		
0.5		
1		
2		
4		
8		
15		
30		
60		
250		
1440		

HYDRAULIC COND. (ASTM D2434)	**N/A**
DEGRADATION (ATM T-313)	**N/A**
PLASTICITY INDEX ASTM 4318	**N/A**

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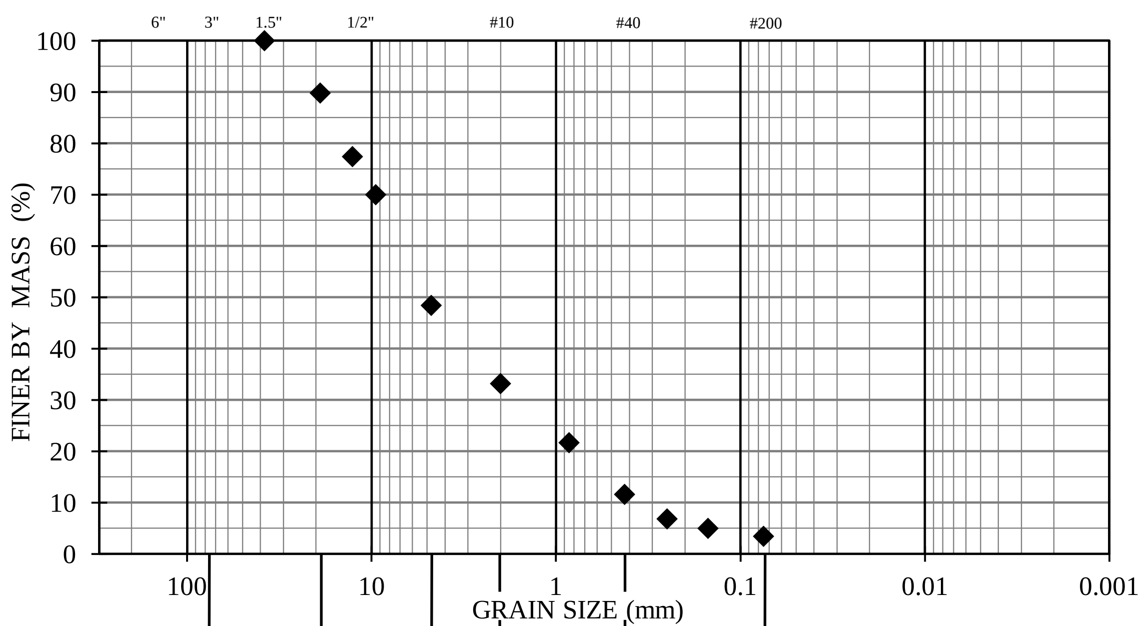
PROJECT CLIENT:	**AK DNR - DPOR**
PROJECT NAME:	**Eagle Rock Boat Launch**
PROJECT NO.:	**4597-16**
SAMPLE LOC.:	**KENB1**
NUMBER/ DEPTH:	**S8 / 25 - 26.5'**
DESCRIPTION:	**Well-graded gravel w/ sand**
DATE RECEIVED:	**12/19/2016**
TESTED BY:	**CJK/XG**
REVIEWED BY:	**CJK**

% GRAVEL	**51.6**	USCS	**GW**
% SAND	**45.0**	USACOE FC	**N/A**
% SILT/CLAY	**3.4**	% PASS. 0.02 mm	**N/A**
% MOIST. CONTENT	**7.9**	% PASS. 0.002 mm	**N/A**
UNIFORMITY COEFFICIENT (C_u)		**19.9**	
COEFFICIENT OF GRADATION (C_c)		**1.1**	
ASTM D1557 (uncorrected)		**N/A**	
ASTM D4718 (corrected)		**N/A**	
OPTIMUM MOIST. CONTENT. (corrected)		**N/A**	

PARTICLE SIZE ANALYSIS ASTM D422 / C136

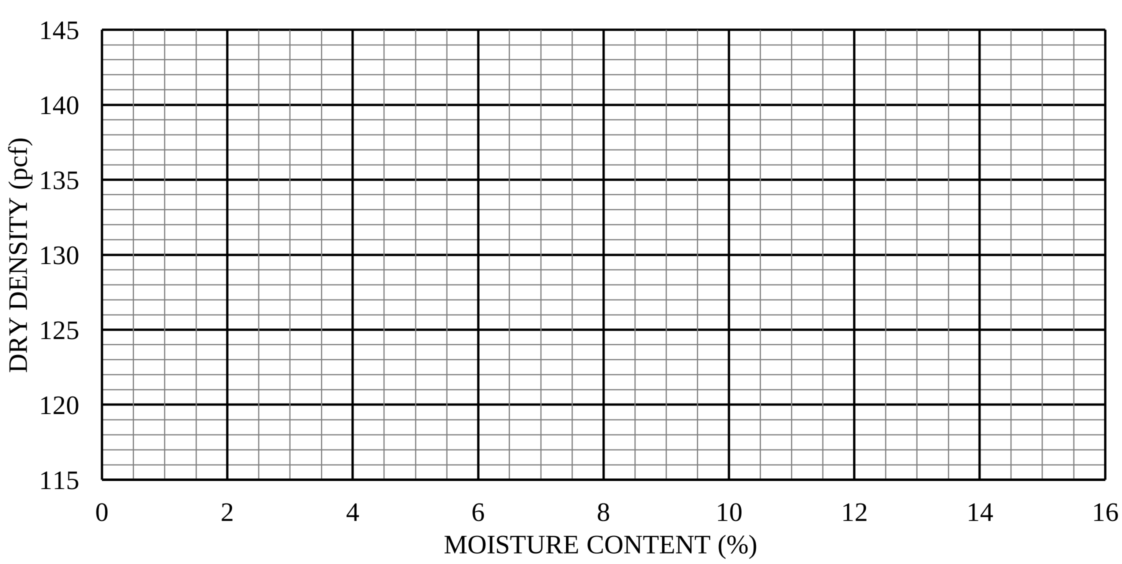


COBBLES	GRAVEL		SAND			SILT or CLAY
	Coarse	Fine	Coarse	Medium	Fine	

SIEVE ANALYSIS RESULT

SIEVE SIZE (mm)	SIEVE SIZE (U.S.)	TOTAL % PASSING	SPECIFICATION (% PASSING)
38.10	1.5"	**100**	
19.00	3/4"	**90**	
12.70	1/2"	**77**	
9.50	3/8"	**70**	
4.75	#4	**48**	
2.00	#10	**33**	
0.85	#20	**22**	
0.43	#40	**12**	
0.25	#60	**7**	
0.15	#100	**5**	
0.075	#200	**3.4**	

MOISTURE-DENSITY RELATIONSHIP ASTM D1557



HYDROMETER RESULT

ELAPSED TIME (MIN)	DIAMETER (mm)	TOTAL % PASSING
0		
0.5		
1		
2		
4		
8		
15		
30		
60		
250		
1440		

HYDRAULIC COND. (ASTM D2434)	**N/A**
DEGRADATION (ATM T-313)	**N/A**
PLASTICITY INDEX ASTM 4318	**N/A**

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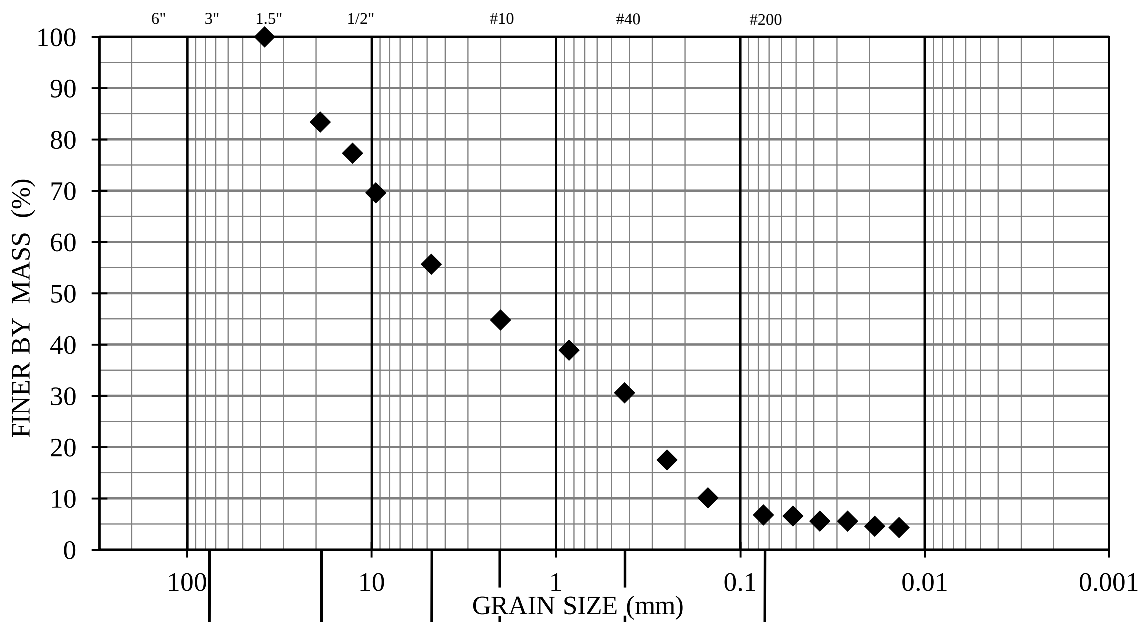
PROJECT CLIENT:	**AK DNR - DPOR**
PROJECT NAME:	**Eagle Rock Boat Launch**
PROJECT NO.:	**4597-16**
SAMPLE LOC.:	**KENB2**
NUMBER/ DEPTH:	**S1 / 0 - 1.5'**
DESCRIPTION:	**Poorly-graded sand w/ silt and gravel**
DATE RECEIVED:	**12/19/2016**
TESTED BY:	**CJK**
REVIEWED BY:	**CJK**

% GRAVEL	**44.3**	USCS	**SP-SM**
% SAND	**48.9**	USACOE FC	**PFS**
% SILT/CLAY	**6.8**	% PASS. 0.02 mm	**4.8**
% MOIST. CONTENT	**4.5**	% PASS. 0.002 mm	**N/A**
UNIFORMITY COEFFICIENT (C_u)	**42.2**		
COEFFICIENT OF GRADATION (C_c)	**0.2**		
ASTM D1557 (uncorrected)	**N/A**		
ASTM D4718 (corrected)	**N/A**		
OPTIMUM MOIST. CONTENT. (corrected)	**N/A**		

PARTICLE SIZE ANALYSIS ASTM D422 / C136



COBBLES	GRAVEL		SAND			SILT or CLAY
	Coarse	Fine	Coarse	Medium	Fine	

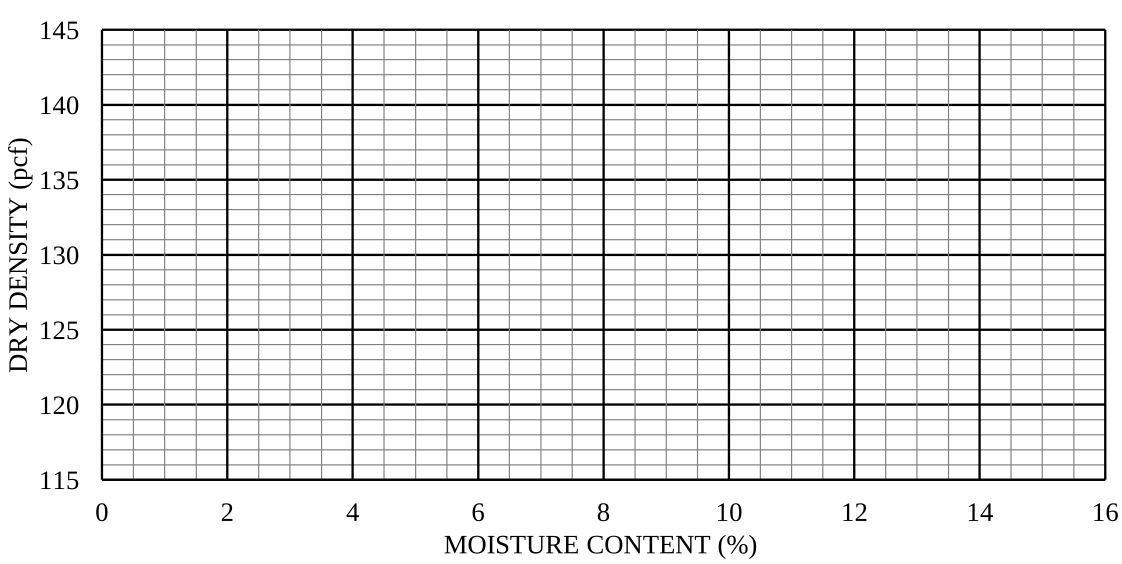
SIEVE ANALYSIS RESULT

SIEVE SIZE (mm)	SIEVE SIZE (U.S.)	TOTAL % PASSING	SPECIFICATION (% PASSING)
38.10	1.5"	**100**	
19.00	3/4"	**83**	
12.70	1/2"	**77**	
9.50	3/8"	**70**	
4.75	#4	**56**	
2.00	#10	**45**	
0.85	#20	**39**	
0.43	#40	**31**	
0.25	#60	**17**	
0.15	#100	**10**	
0.075	#200	**6.8**	

HYDROMETER RESULT

ELAPSED TIME (MIN)	DIAMETER (mm)	TOTAL % PASSING
0		
0.5		
1	0.0519	6.6
2	0.0371	5.6
4	0.0262	5.6
8	0.0187	4.6
15	0.0138	4.3
30		
60		
250		
1440		

MOISTURE-DENSITY RELATIONSHIP ASTM D1557



HYDRAULIC COND. (ASTM D2434)	**N/A**
DEGRADATION (ATM T-313)	**N/A**
PLASTICITY INDEX ASTM 4318	**N/A**

The testing services reported herein have been performed to recognized industry standards, unless otherwise noted. No other warranty is made. Should engineering interpretation or opinion be required, NGE-TFT will provide upon written request.

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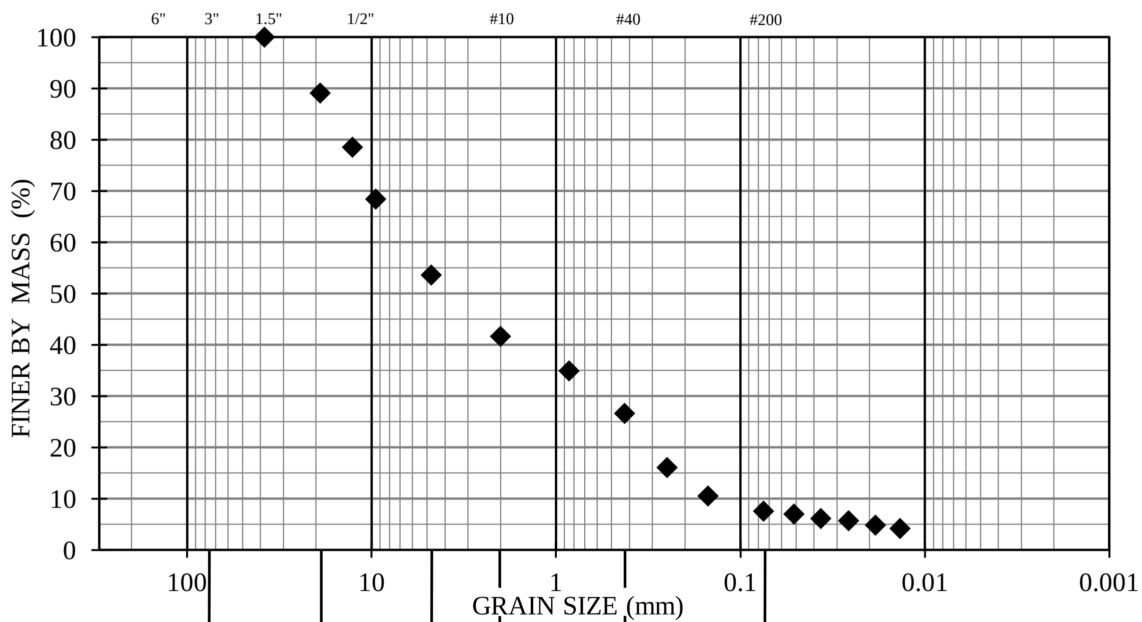
PROJECT CLIENT:	**AK DNR - DPOR**
PROJECT NAME:	**Eagle Rock Boat Launch**
PROJECT NO.:	**4597-16**
SAMPLE LOC.:	**KENB2**
NUMBER/ DEPTH:	**S2 / 2.5 - 4'**
DESCRIPTION:	**Poorly-graded gravel w/ silt and sand**
DATE RECEIVED:	**12/19/2016**
TESTED BY:	**CJK**
REVIEWED BY:	**CJK**

% GRAVEL	**46.3**	USCS	**GP-GM**
% SAND	**46.1**	USACOE FC	**S1**
% SILT/CLAY	**7.6**	% PASS. 0.02 mm	**5.0**
% MOIST. CONTENT	**7.6**	% PASS. 0.002 mm	**N/A**

UNIFORMITY COEFFICIENT (C_u)	**49.7**
COEFFICIENT OF GRADATION (C_c)	**0.4**
ASTM D1557 (uncorrected)	**N/A**
ASTM D4718 (corrected)	**N/A**
OPTIMUM MOIST. CONTENT. (corrected)	**N/A**

PARTICLE SIZE ANALYSIS ASTM D422 / C136



COBBLES	GRAVEL		SAND			SILT or CLAY
	Coarse	Fine	Coarse	Medium	Fine	

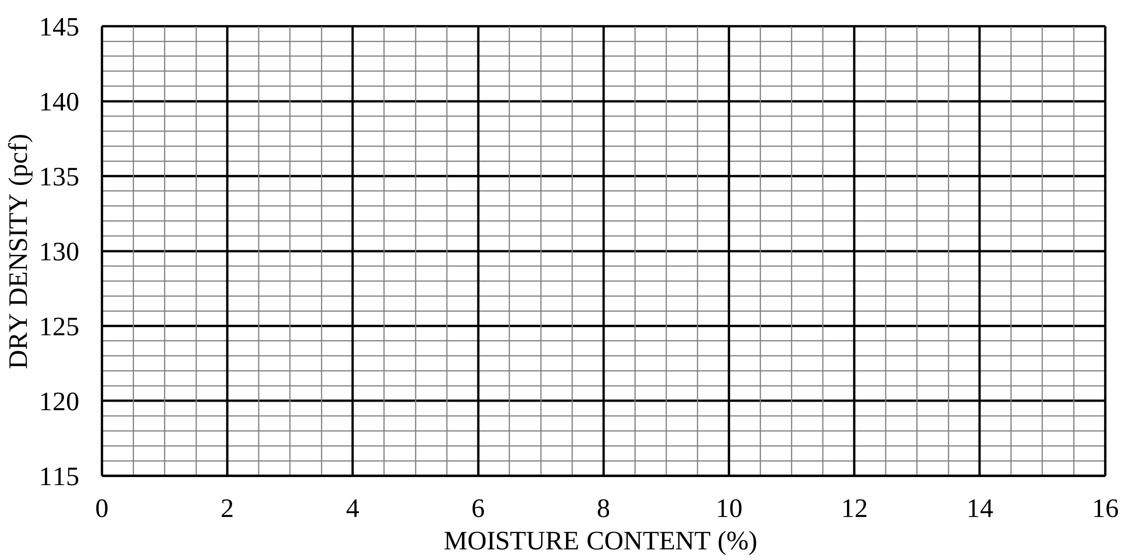
SIEVE ANALYSIS RESULT

SIEVE SIZE (mm)	SIEVE SIZE (U.S.)	TOTAL % PASSING	SPECIFICATION (% PASSING)
38.10	1.5"	**100**	
19.00	3/4"	**89**	
12.70	1/2"	**79**	
9.50	3/8"	**68**	
4.75	#4	**54**	
2.00	#10	**42**	
0.85	#20	**35**	
0.43	#40	**27**	
0.25	#60	**16**	
0.15	#100	**11**	
0.075	#200	**7.6**	

HYDROMETER RESULT

ELAPSED TIME (MIN)	DIAMETER (mm)	TOTAL % PASSING
0		
0.5		
1	0.0513	7.0
2	0.0367	6.1
4	0.0259	5.7
8	0.0185	4.8
15	0.0136	4.2
30		
60		
250		
1440		

MOISTURE-DENSITY RELATIONSHIP ASTM D1557



HYDRAULIC COND. (ASTM D2434)	**N/A**
DEGRADATION (ATM T-313)	**N/A**
PLASTICITY INDEX ASTM 4318	**N/A**

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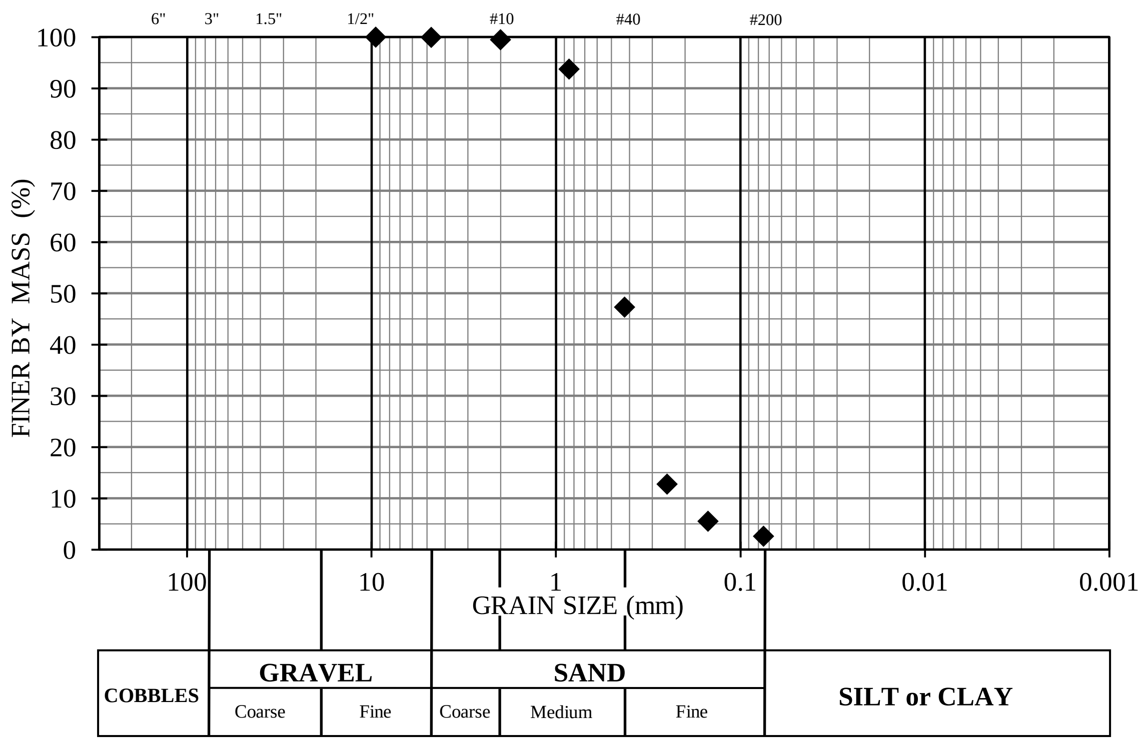
PROJECT CLIENT:	**AK DNR - DPOR**
PROJECT NAME:	**Eagle Rock Boat Launch**
PROJECT NO.:	**4597-16**
SAMPLE LOC.:	**KENB3**
NUMBER/ DEPTH:	**S4 / 7.5 - 9'**
DESCRIPTION:	**Poorly-graded sand**
DATE RECEIVED:	**12/19/2016**
TESTED BY:	**CJK/XG**
REVIEWED BY:	**CJK**

% GRAVEL	**0.0**	USCS	**SP**
% SAND	**97.4**	USACOE FC	**N/A**
% SILT/CLAY	**2.6**	% PASS. 0.02 mm	**N/A**
% MOIST. CONTENT	**24.0**	% PASS. 0.002 mm	**N/A**
UNIFORMITY COEFFICIENT (C_u)	**2.6**		
COEFFICIENT OF GRADATION (C_c)	**1.0**		
ASTM D1557 (uncorrected)	**N/A**		
ASTM D4718 (corrected)	**N/A**		
OPTIMUM MOIST. CONTENT. (corrected)	**N/A**		

PARTICLE SIZE ANALYSIS ASTM D422 / C136



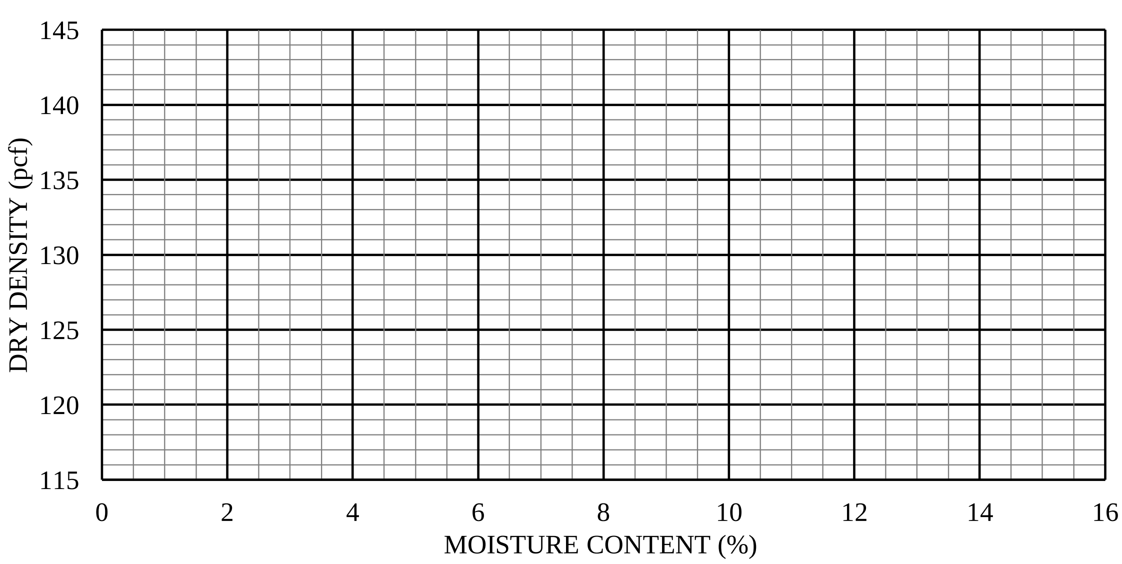
SIEVE ANALYSIS RESULT

SIEVE SIZE (mm)	SIEVE SIZE (U.S.)	TOTAL % PASSING	SPECIFICATION (% PASSING)
9.50	3/8"	**100**	
4.75	#4	**100**	
2.00	#10	**99**	
0.85	#20	**94**	
0.43	#40	**47**	
0.25	#60	**13**	
0.15	#100	**6**	
0.075	#200	**2.6**	

HYDROMETER RESULT

ELAPSED TIME (MIN)	DIAMETER (mm)	TOTAL % PASSING
0		
0.5		
1		
2		
4		
8		
15		
30		
60		
250		
1440		

MOISTURE-DENSITY RELATIONSHIP ASTM D1557



HYDRAULIC COND. (ASTM D2434)	**N/A**
DEGRADATION (ATM T-313)	**N/A**
PLASTICITY INDEX ASTM 4318	**N/A**

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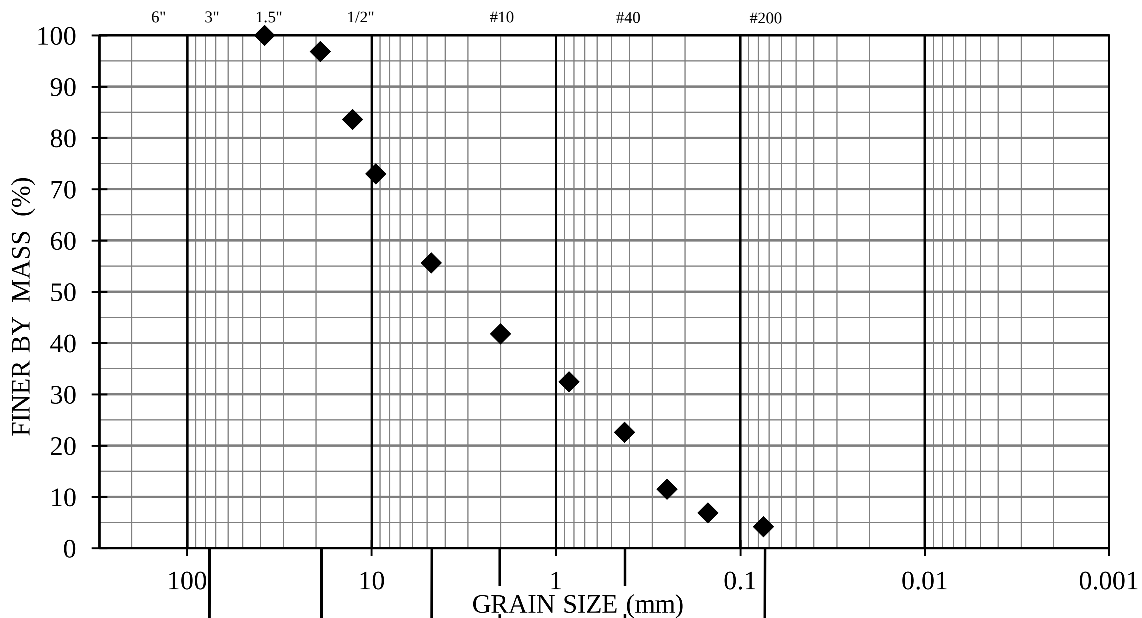
PROJECT CLIENT:	**AK DNR - DPOR**
PROJECT NAME:	**Eagle Rock Boat Launch**
PROJECT NO.:	**4597-16**
SAMPLE LOC.:	**KENB3**
NUMBER/ DEPTH:	**S5 / 10 - 11.5'**
DESCRIPTION:	**Poorly-graded sand w/ gravel**
DATE RECEIVED:	**12/19/2016**
TESTED BY:	**CJK/XG**
REVIEWED BY:	**CJK**

% GRAVEL	**44.4**	USCS	**SP**
% SAND	**51.4**	USACOE FC	**N/A**
% SILT/CLAY	**4.2**	% PASS. 0.02 mm	**N/A**
% MOIST. CONTENT	**9.6**	% PASS. 0.002 mm	**N/A**
UNIFORMITY COEFFICIENT (C_u)	**27.3**		
COEFFICIENT OF GRADATION (C_c)	**0.4**		
ASTM D1557 (uncorrected)	**N/A**		
ASTM D4718 (corrected)	**N/A**		
OPTIMUM MOIST. CONTENT. (corrected)	**N/A**		

PARTICLE SIZE ANALYSIS ASTM D422 / C136



COBBLES	GRAVEL		SAND			SILT or CLAY
	Coarse	Fine	Coarse	Medium	Fine	

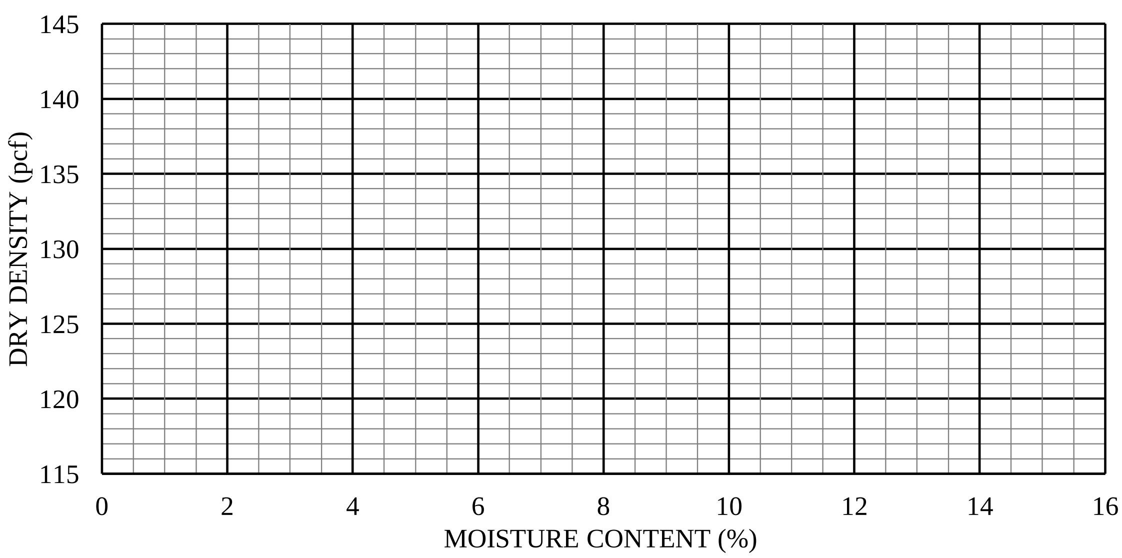
SIEVE ANALYSIS RESULT

SIEVE SIZE (mm)	SIEVE SIZE (U.S.)	TOTAL % PASSING	SPECIFICATION (% PASSING)
38.10	1.5"	**100**	
19.00	3/4"	**97**	
12.70	1/2"	**84**	
9.50	3/8"	**73**	
4.75	#4	**56**	
2.00	#10	**42**	
0.85	#20	**32**	
0.43	#40	**23**	
0.25	#60	**11**	
0.15	#100	**7**	
0.075	#200	**4.2**	

HYDROMETER RESULT

ELAPSED TIME (MIN)	DIAMETER (mm)	TOTAL % PASSING
0		
0.5		
1		
2		
4		
8		
15		
30		
60		
250		
1440		

MOISTURE-DENSITY RELATIONSHIP ASTM D1557



HYDRAULIC COND. (ASTM D2434)	**N/A**
DEGRADATION (ATM T-313)	**N/A**
PLASTICITY INDEX ASTM 4318	**N/A**

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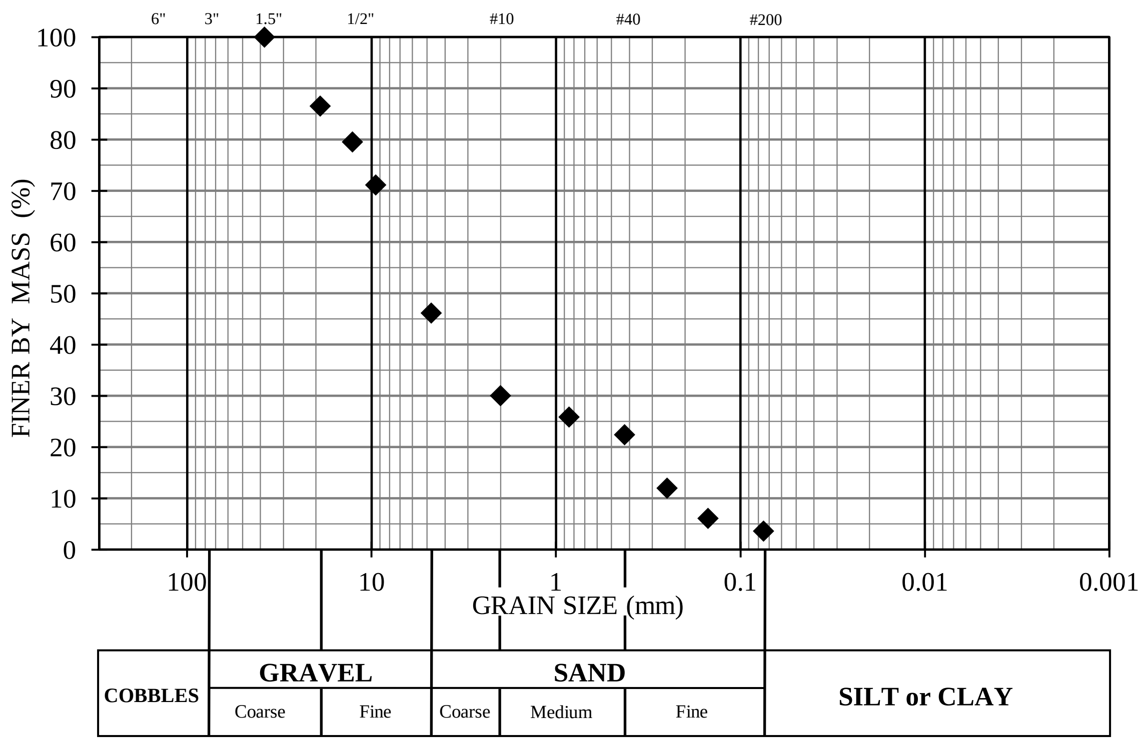
PROJECT CLIENT:	**AK DNR - DPOR**
PROJECT NAME:	**Eagle Rock Boat Launch**
PROJECT NO.:	**4597-16**
SAMPLE LOC.:	**KENB4**
NUMBER/ DEPTH:	**S4 / 7.5 - 9'**
DESCRIPTION:	**Well-graded gravel w/ sand**
DATE RECEIVED:	**12/19/2016**
TESTED BY:	**CJK/XG**
REVIEWED BY:	**CJK**

% GRAVEL	**53.8**	USCS	**GW**
% SAND	**42.6**	USACOE FC	**N/A**
% SILT/CLAY	**3.6**	% PASS. 0.02 mm	**N/A**
% MOIST. CONTENT	**8.9**	% PASS. 0.002 mm	**N/A**
UNIFORMITY COEFFICIENT (C_u)	**34.1**		
COEFFICIENT OF GRADATION (C_c)	**2.5**		
ASTM D1557 (uncorrected)	**N/A**		
ASTM D4718 (corrected)	**N/A**		
OPTIMUM MOIST. CONTENT. (corrected)	**N/A**		

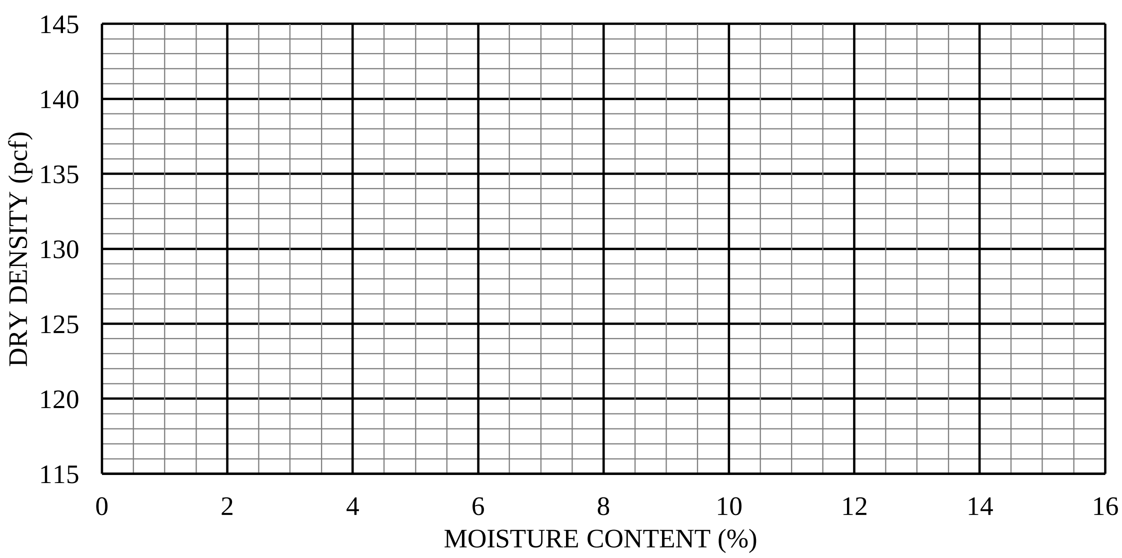
PARTICLE SIZE ANALYSIS ASTM D422 / C136



SIEVE ANALYSIS RESULT

SIEVE SIZE (mm)	SIEVE SIZE (U.S.)	TOTAL % PASSING	SPECIFICATION (% PASSING)
38.10	1.5"	**100**	
19.00	3/4"	**87**	
12.70	1/2"	**80**	
9.50	3/8"	**71**	
4.75	#4	**46**	
2.00	#10	**30**	
0.85	#20	**26**	
0.43	#40	**22**	
0.25	#60	**12**	
0.15	#100	**6**	
0.075	#200	**3.6**	

MOISTURE-DENSITY RELATIONSHIP ASTM D1557



HYDROMETER RESULT

ELAPSED TIME (MIN)	DIAMETER (mm)	TOTAL % PASSING
0		
0.5		
1		
2		
4		
8		
15		
30		
60		
250		
1440		

HYDRAULIC COND. (ASTM D2434)	**N/A**
DEGRADATION (ATM T-313)	**N/A**
PLASTICITY INDEX ASTM 4318	**N/A**

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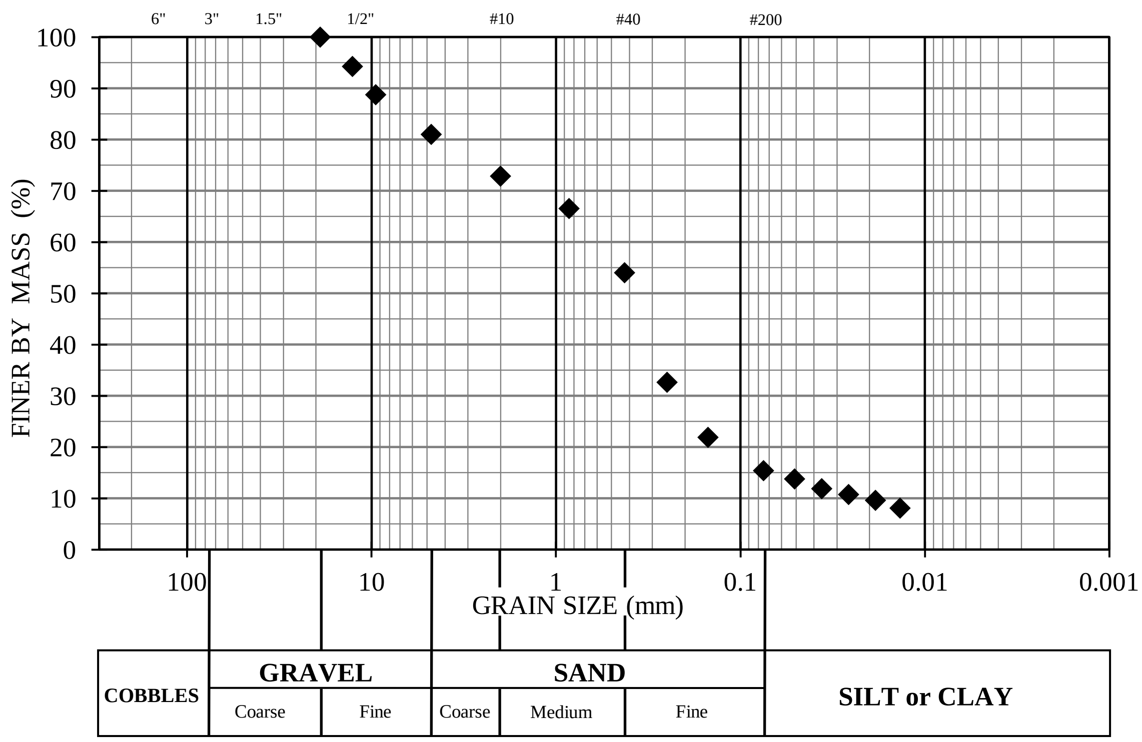
PROJECT CLIENT:	**AK DNR - DPOR**
PROJECT NAME:	**Eagle Rock Boat Launch**
PROJECT NO.:	**4597-16**
SAMPLE LOC.:	**KENB5**
NUMBER/ DEPTH:	**S1 / 0 - 1.5'**
DESCRIPTION:	**Silty sand w/ gravel**
DATE RECEIVED:	**12/19/2016**
TESTED BY:	**CJK**
REVIEWED BY:	**CJK**

% GRAVEL	**19.0**	USCS	**SM**
% SAND	**65.6**	USACOE FC	**F2**
% SILT/CLAY	**15.4**	% PASS. 0.02 mm	**9.9**
% MOIST. CONTENT	**9.4**	% PASS. 0.002 mm	**N/A**
UNIFORMITY COEFFICIENT (C_u)		**29.7**	
COEFFICIENT OF GRADATION (C_c)		**3.8**	
ASTM D1557 (uncorrected)		**N/A**	
ASTM D4718 (corrected)		**N/A**	
OPTIMUM MOIST. CONTENT. (corrected)		**N/A**	

PARTICLE SIZE ANALYSIS ASTM D422 / C136



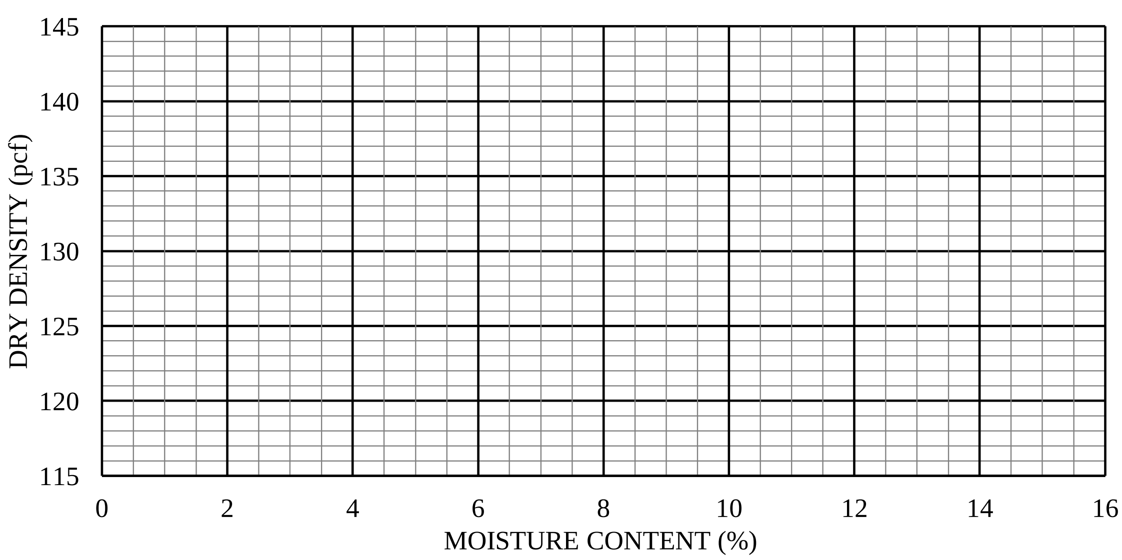
SIEVE ANALYSIS RESULT

SIEVE SIZE (mm)	SIEVE SIZE (U.S.)	TOTAL % PASSING	SPECIFICATION (% PASSING)
19.00	3/4"	**100**	
12.70	1/2"	**94**	
9.50	3/8"	**89**	
4.75	#4	**81**	
2.00	#10	**73**	
0.85	#20	**67**	
0.43	#40	**54**	
0.25	#60	**33**	
0.15	#100	**22**	
0.075	#200	**15.4**	

HYDROMETER RESULT

ELAPSED TIME (MIN)	DIAMETER (mm)	TOTAL % PASSING
0		
0.5		
1	0.0509	13.8
2	0.0363	11.9
4	0.0259	10.7
8	0.0185	9.6
15	0.0136	8.1
30		
60		
250		
1440		

MOISTURE-DENSITY RELATIONSHIP ASTM D1557



HYDRAULIC COND. (ASTM D2434)	**N/A**
DEGRADATION (ATM T-313)	**N/A**
PLASTICITY INDEX ASTM 4318	**N/A**

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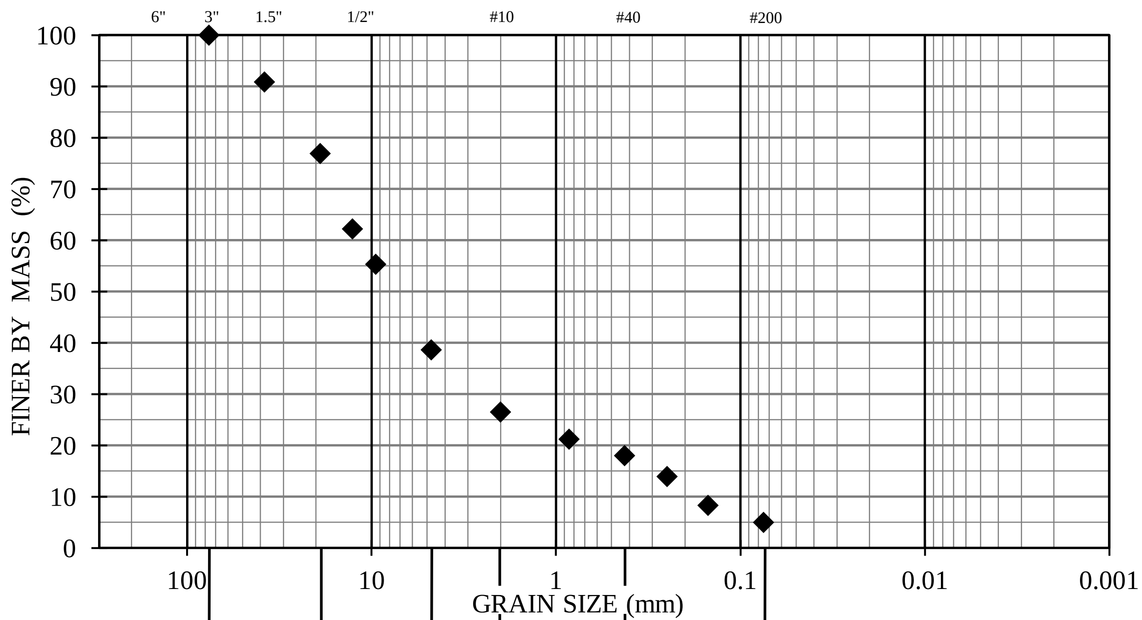
PROJECT CLIENT:	**AK DNR - DPOR**
PROJECT NAME:	**Eagle Rock Boat Launch**
PROJECT NO.:	**4597-16**
SAMPLE LOC.:	**KENB5**
NUMBER/ DEPTH:	**S6 / 15 - 16.5'**
DESCRIPTION:	**Poorly-graded gravel w/ silt and sand**
DATE RECEIVED:	**12/19/2016**
TESTED BY:	**CJK/XG**
REVIEWED BY:	**CJK**

% GRAVEL	**61.4**	USCS	**GP-GM**
% SAND	**33.6**	USACOE FC	**N/A**
% SILT/CLAY	**5.0**	% PASS. 0.02 mm	**N/A**
% MOIST. CONTENT	**7.5**	% PASS. 0.002 mm	**N/A**
UNIFORMITY COEFFICIENT (C_u)	**64.7**		
COEFFICIENT OF GRADATION (C_c)	**3.7**		
ASTM D1557 (uncorrected)	**N/A**		
ASTM D4718 (corrected)	**N/A**		
OPTIMUM MOIST. CONTENT. (corrected)	**N/A**		

PARTICLE SIZE ANALYSIS ASTM D422 / C136



COBBLES	GRAVEL		SAND			SILT or CLAY
	Coarse	Fine	Coarse	Medium	Fine	

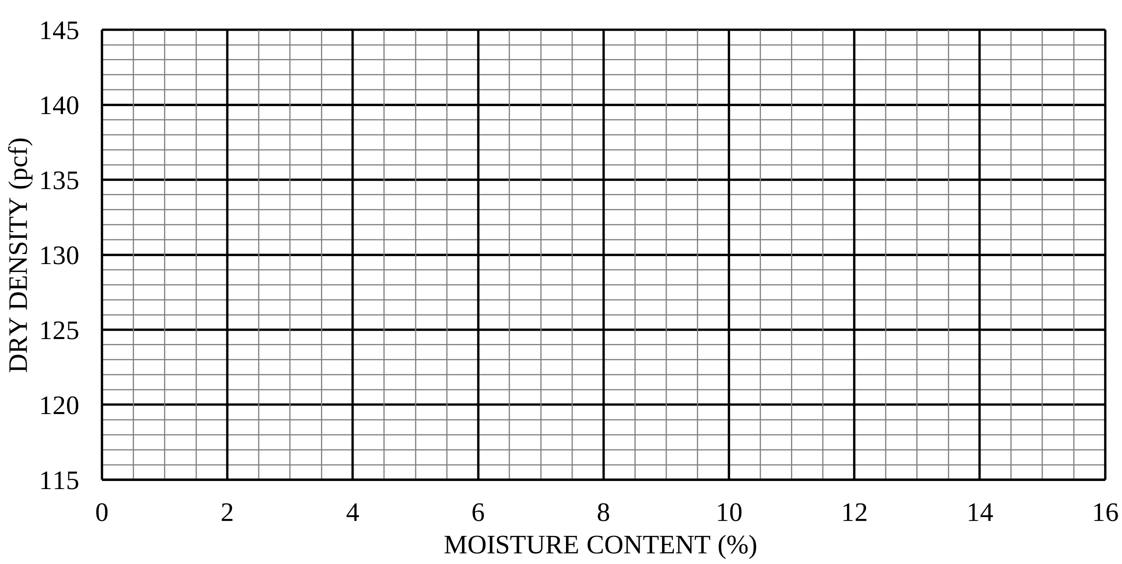
SIEVE ANALYSIS RESULT

SIEVE SIZE (mm)	SIEVE SIZE (U.S.)	TOTAL % PASSING	SPECIFICATION (% PASSING)
76.20	3"	**100**	
38.10	1.5"	**91**	
19.00	3/4"	**77**	
12.70	1/2"	**62**	
9.50	3/8"	**55**	
4.75	#4	**39**	
2.00	#10	**27**	
0.85	#20	**21**	
0.43	#40	**18**	
0.25	#60	**14**	
0.15	#100	**8**	
0.075	#200	**5.0**	

HYDROMETER RESULT

ELAPSED TIME (MIN)	DIAMETER (mm)	TOTAL % PASSING
0		
0.5		
1		
2		
4		
8		
15		
30		
60		
250		
1440		

MOISTURE-DENSITY RELATIONSHIP ASTM D1557



HYDRAULIC COND. (ASTM D2434)	**N/A**
DEGRADATION (ATM T-313)	**N/A**
PLASTICITY INDEX ASTM 4318	**N/A**

The testing services reported herein have been performed to recognized industry standards, unless otherwise noted. No other warranty is made. Should engineering interpretation or opinion be required, NGE-TFT will provide upon written request.

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APPENDIX C

USGS DESIGN MAPS REPORT

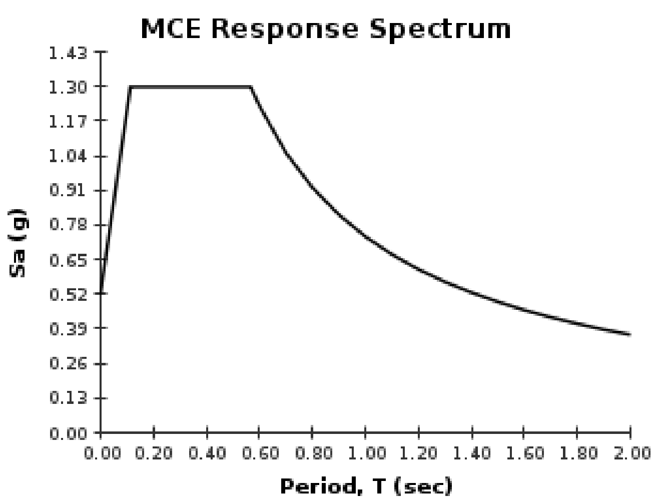
Design Maps Summary Report

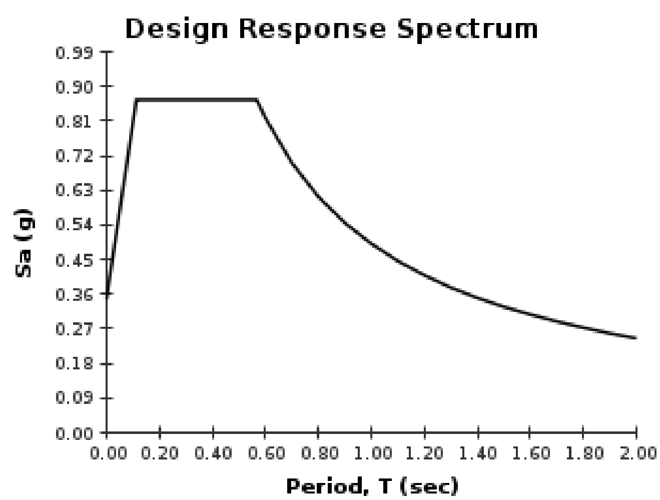
User–Specified Input

Report Title	Eagle Rock Boat Launch Wed January 11, 2017 22:10:14 UTC
Building Code Reference Document	2006/2009 International Building Code (which utilizes USGS hazard data available in 2002)
Site Coordinates	60.54955°N, 151.10923°W
Site Soil Classification	Site Class D – "Stiff Soil"
Occupancy Category	I/II/III

USGS–Provided Output

S_S = 1.298 g	S_{MS} = 1.298 g	S_{DS} = 0.865 g
S_1 = 0.486 g	S_{M1} = 0.736 g	S_{D1} = 0.491 g





Design Maps Detailed Report

2006/2009 International Building Code (60.54955°N, 151.10923°W)

Site Class D – "Stiff Soil", Occupancy Category I/II/III

Section 1613.5.1 — Mapped acceleration parameters

Note: Maps in the 2006 and 2009 International Building Code are provided for Site Class B. Adjustments for other Site Classes are made, as needed, in Section 1613.5.3.

From Figure 1613.5(11) [1] $S_S = 1.298$ g

From Figure 1613.5(12) [2] $S_1 = 0.486$ g

Section 1613.5.2 — Site class definitions

SITE CLASS	SOIL PROFILE NAME	Soil shear wave velocity, $\bar{v}_s$, (ft/s)	Standard penetration resistance, $\bar{N}$	Soil undrained shear strength, $\bar{s}_u$, (psf)
A	Hard rock	$\bar{v}_S > 5{,}000$	N/A	N/A
B	Rock	$2{,}500 < \bar{v}_S \leq 5{,}000$	N/A	N/A
C	Very dense soil and soft rock	$1{,}200 < \bar{v}_S \leq 2{,}500$	$\bar{N} > 50$	>2,000 psf
D	Stiff soil profile	$600 \leq \bar{v}_S < 1{,}200$	$15 \leq \bar{N} \leq 50$	1,000 to 2,000 psf
E	Stiff soil profile	$\bar{v}_S < 600$	$\bar{N} < 15$	<1,000 psf
E	—	Any profile with more than 10 ft of soil having the characteristics: 1. Plasticity index *PI* > 20, 2. Moisture content $w \geq 40\%$, and 3. Undrained shear strength $\bar{s}_u < 500$ psf		
F	—	Any profile containing soils having one or more of the following characteristics: 1. Soils vulnerable to potential failure or collapse under seismic loading such as liquefiable soils, quick and highly sensitive clays, collapsible weakly cemented soils. 2. Peats and/or highly organic clays (*H* > 10 feet of peat and/or highly organic clay where *H* = thickness of soil) 3. Very high plasticity clays (*H* > 25 feet with plasticity index *PI* > 75) 4. Very thick soft/medium stiff clays (*H* > 120 feet)		

For SI: 1ft/s = 0.3048 m/s 1lb/ft² = 0.0479 kN/m²

Section 1613.5.3 — Site coefficients and adjusted maximum considered earthquake spectral response acceleration parameters

TABLE 1613.5.3(1)
VALUES OF SITE COEFFICIENT F_a

Site Class	Mapped Spectral Response Acceleration at Short Period				
	$S_S \leq 0.25$	$S_S = 0.50$	$S_S = 0.75$	$S_S = 1.00$	$S_S \geq 1.25$
A	0.8	0.8	0.8	0.8	0.8
B	1.0	1.0	1.0	1.0	1.0
C	1.2	1.2	1.1	1.0	1.0
D	1.6	1.4	1.2	1.1	1.0
E	2.5	1.7	1.2	0.9	0.9
F	See Section 11.4.7 of ASCE 7				

Note: Use straight–line interpolation for intermediate values of S_S

For Site Class = D and S_s = 1.298 g, F_a = 1.000

TABLE 1613.5.3(2)
VALUES OF SITE COEFFICIENT F_v

Site Class	Mapped Spectral Response Acceleration at 1–s Period				
	$S_1 \leq 0.10$	$S_1 = 0.20$	$S_1 = 0.30$	$S_1 = 0.40$	$S_1 \geq 0.50$
A	0.8	0.8	0.8	0.8	0.8
B	1.0	1.0	1.0	1.0	1.0
C	1.7	1.6	1.5	1.4	1.3
D	2.4	2.0	1.8	1.6	1.5
E	3.5	3.2	2.8	2.4	2.4
F	See Section 11.4.7 of ASCE 7				

Note: Use straight–line interpolation for intermediate values of S_1

For Site Class = D and S_1 = 0.486 g, F_v = 1.514

In the equations below, the equation number corresponding to the 2006 edition is listed first, and that corresponding to the 2009 edition is listed second.

Equation (16-37; 16-36): $S_{MS} = F_a S_S = 1.000 \times 1.298 = 1.298$ g

Equation (16-38; 16-37): $S_{M1} = F_v S_1 = 1.514 \times 0.486 = 0.736$ g

Section 1613.5.4 — Design spectral response acceleration parameters

Equation (16-39; 16-38): $S_{DS} = ⅔\, S_{MS} = ⅔ \times 1.298 = 0.865$ g

Equation (16-40; 16-39): $S_{D1} = ⅔\, S_{M1} = ⅔ \times 0.736 = 0.491$ g

Section 1613.5.6 — Determination of seismic design category

TABLE 1613.5.6(1)
SEISMIC DESIGN CATEGORY BASED ON SHORT-PERIOD RESPONSE ACCELERATION

VALUE OF S_{DS}	OCCUPANCY CATEGORY		
	I or II	**III**	**IV**
$S_{DS} < 0.167g$	A	A	A
$0.167g \le S_{DS} < 0.33g$	B	B	C
$0.33g \le S_{DS} < 0.50g$	C	C	D
$0.50g \le S_{DS}$	D	D	D

For Occupancy Category = I and S_{DS} = 0.865 g, Seismic Design Category = D

TABLE 1613.5.6(2)
SEISMIC DESIGN CATEGORY BASED ON 1-SECOND PERIOD RESPONSE ACCELERATION

VALUE OF S_{D1}	OCCUPANCY CATEGORY		
	I or II	**III**	**IV**
$S_{D1} < 0.067g$	A	A	A
$0.067g \le S_{D1} < 0.133g$	B	B	C
$0.133g \le S_{D1} < 0.20g$	C	C	D
$0.20g \le S_{D1}$	D	D	D

For Occupancy Category = I and S_{D1} = 0.491 g, Seismic Design Category = D

Note: When S_1 is greater than or equal to 0.75g, the Seismic Design Category is **E** for buildings in Occupancy Categories I, II, and III, and **F** for those in Occupancy Category IV, irrespective of the above.

Seismic Design Category ≡ "the more severe design category in accordance with Table 1613.5.6(1) or 1613.5.6(2)" = D

Note: See Section 1613.5.6.1 for alternative approaches to calculating Seismic Design Category.

References

1. *Figure 1613.5(11)*: http://earthquake.usgs.gov/hazards/designmaps/downloads/pdfs/IBC-2006-Figure1613_5(11).pdf
2. *Figure 1613.5(12)*: http://earthquake.usgs.gov/hazards/designmaps/downloads/pdfs/IBC-2006-Figure1613_5(12).pdf