GEOTECHNICAL ASSESSMENT

for the proposed improvements at

ALASKA RAILROAD CORPORATION
BRIDGE 25.7
TRAIL RIVER, ALASKA

Prepared for:

Wilson and Company, Inc., Engineers & Architects
11516 Miracle Hills Drive, Suite #102
Omaha, NE 68154

Prepared by:

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

---

OCTOBER 2019

NORTHERN GEOTECHNICAL ENGINEERING, INC. / TERRA FIRMA TESTING

Laboratory Testing | Geotechnical Engineering | Instrumentation | Construction Monitoring Services | Thermal Analysis

October 30, 2019 NGE-TFT Project #5566-19

Wilson and Company, Inc., Engineers & Architects
11516 Miracle Hills Drive, Suite #102
Omaha, NE 68154

Attn: Brandon Buckman, P.E

RE: GEOTECHNICAL ENGINEERING FOUNDATION RECOMMENDATIONS FOR THE PROPOSED IMPROVEMENTS AT ARRC BRIDGE MILE 25.7 LOCATED ON TRAIL RIVER, ALASKA

Brandon,

We (Northern Geotechnical Engineering, Inc. *d.b.a.* Terra Firma Testing) have completed geotechnical engineering assessment of the referenced project to aid in the design process. Proposed improvements include replacing the existing ARRC Bridge Mile 25.7 and abutments. Our assessment suggests that the existing, on-site native soils and bedrock are generally suitable to support the foundations of the proposed bridge replacement; provided that appropriate design and construction practices are implemented. We have provided details of our findings, along with our conclusions and engineering recommendations in the following report.

Once the pile driving equipment, design loads, and pile specifications have been determined, we recommend that a wave equation analysis be performed in an effort to refine our pile capacity and embedment depth estimates.

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

Clinton J. Banzhaf, P.E.
Senior Project Engineer

STATE OF ALASKA
49 TH
Clinton J. Banzhaf
CE - 14856
10/30/19
REGISTERED PROFESSIONAL ENGINEER

Keith F. Mobley, P.E.
President

11301 Olive Lane Anchorage, Alaska 99515 · Phone: (907) 344-5934 · Fax: (907) 344-5993 · Website: www.nge-tft.com

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 our foundation recommendations for the proposed improvements at the Alaska Railroad Corporation (ARRC) Bridge Mile 25.7; hereafter referred to as "the project site". We provided our professional service in accordance with our service fee proposal #19-229(G) which we submitted to our client, Wilson & Company, Inc. Engineers and Architects (Wilson & Co.), on August 19, 2019. Wilson & Co. authorized our proposed scope of service via signed Subconsultant Agreement on October 1, 2019.

Wilson & Co. contracted us to evaluate the subsurface conditions based on previous subsurface explorations near the existing bridge in an effort to develop pile foundation recommendations to aid in the design of the proposed bridge replacement. We have also included earthworks recommendations for the bridge abutment and railway work for the bridge replacement.

In this report, we provide our conclusions regarding the suitability of the subsurface conditions to support the proposed bridge replacement foundations. We also provide our engineering recommendations regarding the design and construction of the proposed bridge replacement foundations.

2.0 PROJECT OVERVIEW

As we detail in Figure 1 of this report, the project site is located at Mile 25.7 of the ARRC main rail alignment, near Lower Trail Lake, Alaska. A multi-span ARRC rail bridge resting on concrete abutments and piles is located at the project site at the location where the rail alignment crosses Trial River. Proposed improvements to the project site include the replacement of the existing bridge, which we expect to include a new bridge structure, foundation, bridge abutments and replacement of the railways. Currently, there are 30 percent preliminary drawings for the proposed bridge replacement.

Similar to other ARRC bridges, we expect that the bridge structure and abutment will be founded on piles.

3.0 PREVIOUS SUBSURFACE EXPLORATIONS

Alaska Department of Transportation and Public Facilities (AKDOT) coordinated and directed two subsurface exploration programs and produced two reports titled "Geology Foundation Report" dated October 2003 and "Supplemental Geology Foundation Report" dated May 2004. Between the two exploration programs, three test pits, nine penetrometers, and 18 test holes were completed adjacent to the existing ARRC Bridge 25.7.

11301 Olive Lane Anchorage, Alaska 99515 · Phone: (907) 344-5934 · Fax: (907) 344-5993 · Website: www.nge-tft.com

4.0 DESCRIPTION OF SUBSURFACE CONDITIONS

We reviewed the subsurface exploration logs presented by the AKDOT for this project site, to determine our general subsurface profile.

4.1 General Subsurface Profile

The project site is overlain by material that ranges from sand to gravel with sand and cobbles. This sandy material varies in thickness from approximately 40 feet below ground surface (bgs) at the southern end of the proposed bridge to approximately 4 feet bgs at the northern end of the proposed bridge. On the southern two-thirds of the project site, the sandy material is underlain by a sandy silt to silt to at least 92 feet bgs. The silt grades out towards the northern third of the project site. The silt located on the southern end of the project site and the sandy material on the northern end of the project site is underlain by competent bedrock. The location of bedrock increases in elevation quickly from the southern end to the northern end of the proposed bridge and becomes shallow bedrock.

4.2 Groundwater

In the previous explorations, groundwater was observed at varying depths. We expect groundwater to occur at similar to Trail River surface and vary seasonally with the seasonal fluctuations.

4.3 Frozen Soils

The explorations did not show, nor do we expect, permafrost to occur anywhere across the project site.

5.0 ENGINEERING CONCLUSIONS

5.1 General Site Conclusions

Based on the findings of the previous field exploration and laboratory testing programs, it is our conclusion that the existing silty sand and sandy silt sediments (located below depths of approximately 35 feet) or shallow bedrock are generally suitable to support the proposed bridge foundations; provided that our concerns and recommendations that we present in this report are addressed by the design and construction processes.

5.2 Earthworks

We expect the proposed bridge surface to remain at a similar elevation as the existing bridge surface. New and/or temporary embankments placed at the site will need to consider consolidation and settlement protentional. Any structural fill necessary to bring the project site to final grade should be placed per Section 6.1 of this report.

5.3 Foundations

The railway should be founded on non-frost susceptible structural fill founded on the existing structural fill or bridge structure. We recommend a deep foundation system for the proposed bridge abutments and bridge structure, which transfers the foundation loads though the surficial deposits to the deeper deposits (providing the required pile length for increased load capacity) and/or bedrock.

5.4 Settlement

Embankments placed on existing gravels are expected to experience 1 to 2 inches of settlement under a 10-foot thick fill pad. Most of the settlements should occur as the initial loads are applied, such that additional long-term settlements should be relatively small and within tolerable limits. Proper earthwork is necessary to help reduce the settlement potential. The settlement potential can be reduced by performing all fill efforts as early in the construction schedule as possible. Settlements for properly constructed deep foundations (as we discuss in Section 6.2 of this report) should be negligible.

5.5 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 *B* based on the subsurface explorations that occur at the project site. We utilized the Structural Engineers Association of California (SEAOC) Seismic Design Maps tool (https://seismicmaps.org/) to calculate the seismic design parameters for the project site, which are $F_a = 1.0$ ($S_s = 1.5$ g) and $F_v = 1.5$ ($S_1 = 0.715$ g). A copy of the USGS Design Maps report for the project site is contained in Appendix A of this report.

Given the gradation of the existing coarse-grained material and its relatively dense consistency, we expect there to be a remote to low potential for soil liquefaction, earthquake-induced lateral spreading, and/or pressure ridge development at the project site.

6.0 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.

6.1 Earthworks

We recommend that any railway structural fill pad be constructed directly above the undisturbed gravel (free of any organics), bedrock, or existing structural fill (free of any organics). Any structural fill materials used on-site should be compacted to a minimum of 95 percent of the modified Proctor density. We recommend fill slopes not to be steeper than 2H:1V slope.

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 7.1 of this report for more details). Additional laboratory testing may be required to verify the frost susceptibility of any excavated material for use in fill directly below the abutments and railroad.

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 structural fill begins) in order to visually confirm the findings of this report and provide recommendations for any non-conforming conditions encountered during the excavation activities.

6.2 Deep Foundations

The most common type of deep foundation system in 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. In areas of shallow bedrock, the piles will need to be set into the bedrock by drilling and grouting the pile in place (socketed).

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.

6.2.1 Pile Bearing Capacity

For open-ended and close-ended piles embedded into the native silty material, we present estimated allowable bearing capacities versus embedment depth for two different pile sizes in Figure 3 of this report. The embedment depths presented in Figure 3 of this report assumes that bedrock is not encountered. If the minimum embedment depth and bedrock is encountered, the allowable pile bearing capacity is 435 and 575 kips for a 20-inch 1/2-inch wall and 24-inch 1/2-inch wall pipe pile respectively. The driven piles should be installed to a minimum embedment of 35 feet below grade into the native silty material. The closed-ended piles should have a conical tip to promote the drivability of the pile in granular material however closed-ended piles are not recommended as the pile may be damaged if bedrock is encounter before meeting the minimum embedment depth.

For areas of shallow bedrock (less than minimum embedment of 35 feet bgs), the piles will need to be socketed 10 feet into competent bedrock. The allowable pile bearing capacity for a 20-inch 1/2-inch wall and 24-inch 1/2-inch wall pipe pile socketed 10 feet into competent bedrock is 435 and 575 kips, respectively.

Final embedment depths should be verified utilizing a wave equation analysis to confirm that the allowable bearing capacity has been achieved. We can provide this service once the pile driving equipment, design load, and pile specifications are known.

When multiple piles are installed in close proximately to one another, then pile group efficiency should be considered. Piles socketed into the bedrock do not need to consider group efficiency. We discuss pile group efficiency in further detail in Section 6.2.4 of this report.

6.2.2 Pile Uplift Capacity

As we discuss in Section 6.2.1 of this report, the proposed piles will need to be installed to a minimum embedment of 35 feet below the existing grade or socketed 10 feet into competent bedrock. The short-term uplift capacity of each pile may be taken as one-half (1/2) of the open-ended pile long-term bearing capacity as we detail in Figure 3 of this report. The 20-inch 1/2-inch wall and 24-inch 1/2-inch wall piles socketed into competent bedrock have a short-term uplift capacity of 285 and 340 kips, respectively. Our recommendations include a typical one-third (1/3) increase for short-term wind and seismic loading.

When multiple piles are installed in close proximately to one another, then pile group efficiency should be considered. Piles socketed into the bedrock do not need to consider group efficiency. We discuss pile group efficiency in further detail in Section 6.2.4 of this report.

6.2.3 Lateral Pile Capacity

We used the computer program ALLpile7 (developed by CivilTech software) to analyze the lateral capacity of the proposed piling for this project. We assumed a free-head condition for the piles (i.e., the pile head is allowed to rotate/deflect) with zero feet of pile stickup (above grade). We have detailed the allowable lateral loads in Table 1 of this report for 35 feet of embedment. The allowable lateral loads are ½ of the ultimate lateral loads. The ultimate lateral load is the load that results in a 1-inch deflection.

Table 1: Free-Head Lateral Pile Capacity – No Bed Rock

PILE TYPE	MAX. DEFLECTION (in)	MIN. EMBEDMENT BELOW EXISTING GRADE (ft)	ALLOWABLE LATERAL CAPACITY (kips)
20-in ½-in WALL	1	35	14
24-in ½-in WALL	1	35	18

We can recalculate the lateral loads 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. It should be noted that the lateral pile capacities significantly decrease as the pile stickup (above grade) increases. When multiple piles are installed in close proximately to one another, then pile group efficiency should be considered. We discuss group efficiency in Section 6.2.4 of this report.

6.2.4 Pile Group Efficiency

Group efficiency of steel pipe piles is a function of the spacing of the individual piles. In Table 2 of this report, we present pile group efficiency parameters (as a function of pile diameter). The allowable pile capacities that we provide in Figure 3 of this report should be adjusted as necessary according to the spacing of individual piles by adding the capacity of each pile in the group, then multiplying the sum by the factor in Table 2.

Table 2: 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
Pile Spacing = Distance between Pile Centers

In Table 3 we provide pile group efficiency parameters for lateral loads. The allowable capacities that we provide in Table 1 of this report should be adjusted as necessary according the spacing of individual piles using the same calculations presented in for the vertical group efficiency loads.

Table 3: 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 = Largest Diameter of Pile
Pile Spacing = Distance between Pile Centers

7.0 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.

7.1 Earthworks

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. Any excavated fill or native sand (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.

7.2 Pile Foundations

Open-ended steel pipe pile with or without an inside re-enforced/hardened drive shoe or conical tip pile can be driven here while using our recommendations in Section 6.2 of this report. 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. Piles installed in bedrock should be socketed into the bedrock using a corehole 6-inches larger than the diameter of the pile and the anulus between the pile and bedrock should be minimum of 1.5 inches when installed as shown in Figure 4 of this report. The anulus should be backfilled with 4,000 psi grout.

All pile installation must be completed with quality control inspection to verify the pile configuration and final penetration rate. We will perform a wave equation analysis using the pile driving equipment data before pile installation. During construction, the pile installation must be inspected to ensure the proper resistance is achieved based on the results of our wave equation analysis.

8.0 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 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.

11301 Olive Lane Anchorage, Alaska 99515 · Phone: (907) 344-5934 · Fax: (907) 344-5993 · Website: www.nge-tft.com

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.

9.0 CLOSURE

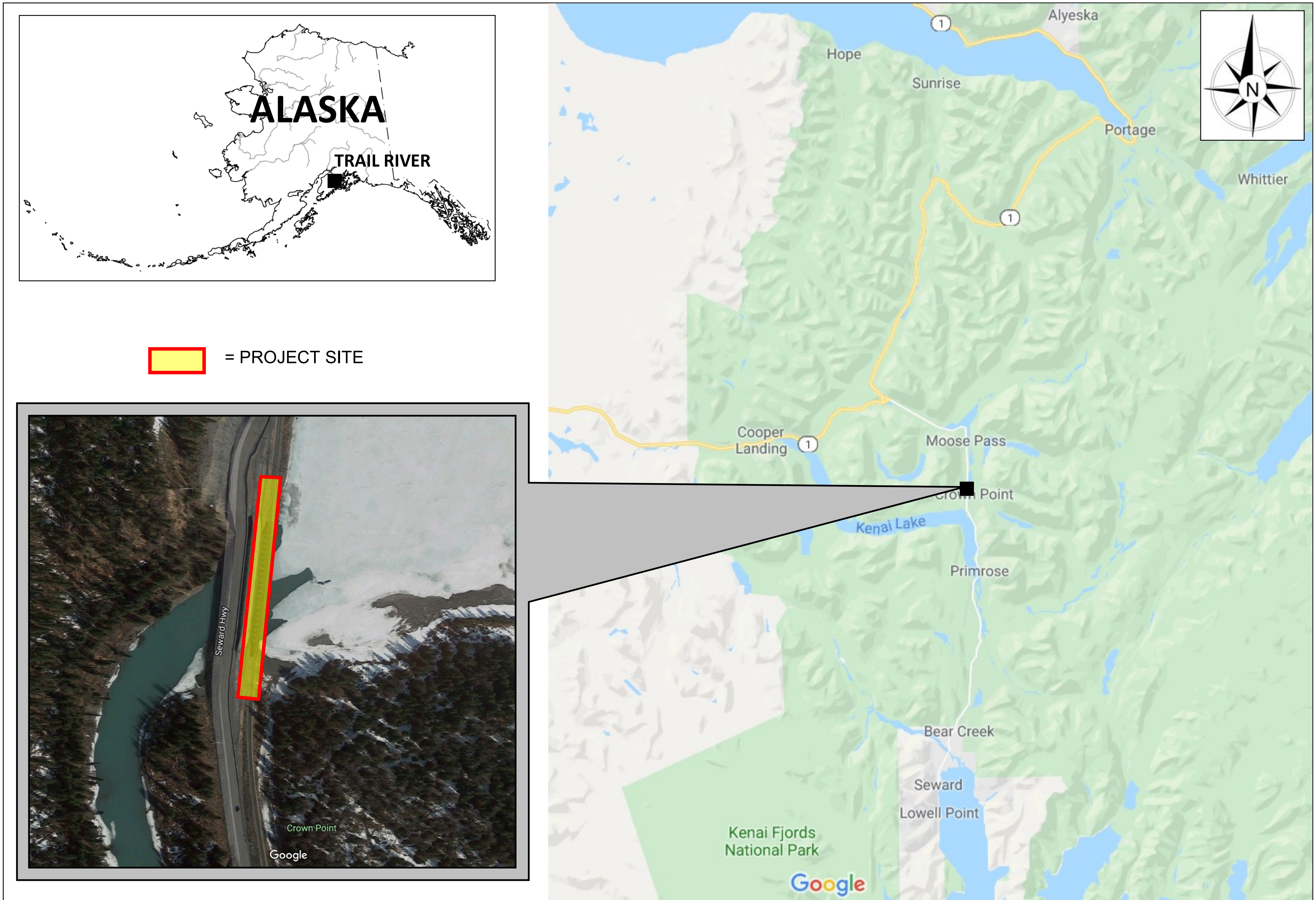
We (Northern Geotechnical Engineering, Inc. d.b.a. Terra Firma Testing) prepared this report exclusively for the use of the *Wilson and Company, Inc., Engineers & Architects* and their client/consultants/contractors/etc. for use in the design and construction of the proposed bridge improvements. We should 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.) so that all of the pertinent information contained within is effectively 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 8.0 of this report for more detail). Furthermore, the construction budget should allow for any unanticipated conditions that may be encountered during construction activities.

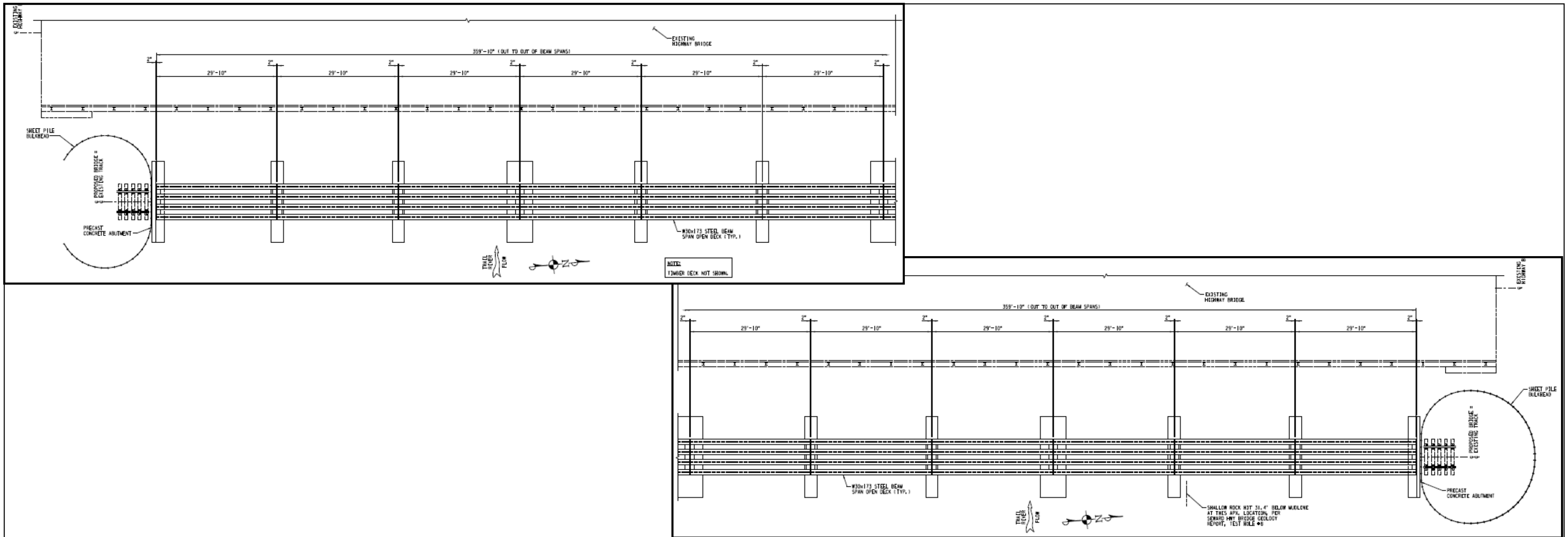
We conducted this evaluation following the standard of care expected of professionals undertaking similar work in the State of Alaska under similar conditions. No warranty, expressed or implied, is made.

REPORT
FIGURES

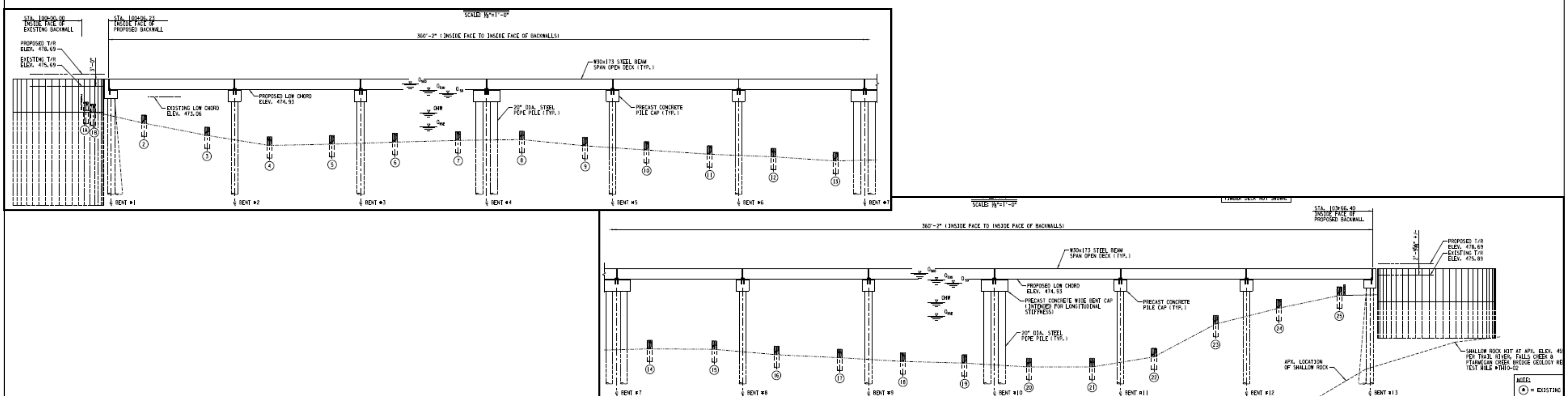


NORTHERN GEOTECHNICAL ENGINEERING, INC.
TERRA FIRMA TESTING

FIGURE TITLE: PROJECT SITE LOCATION MAP	
PROJECT NAME: ALASKA RAILROAD CORPORATION BRIDGE 25.7	PROJECT ID: 5566-19
PROJECT LOCATION: TRAIL RIVER, ALASKA	FIGURE NUMBER: 1



PLAN VIEWS

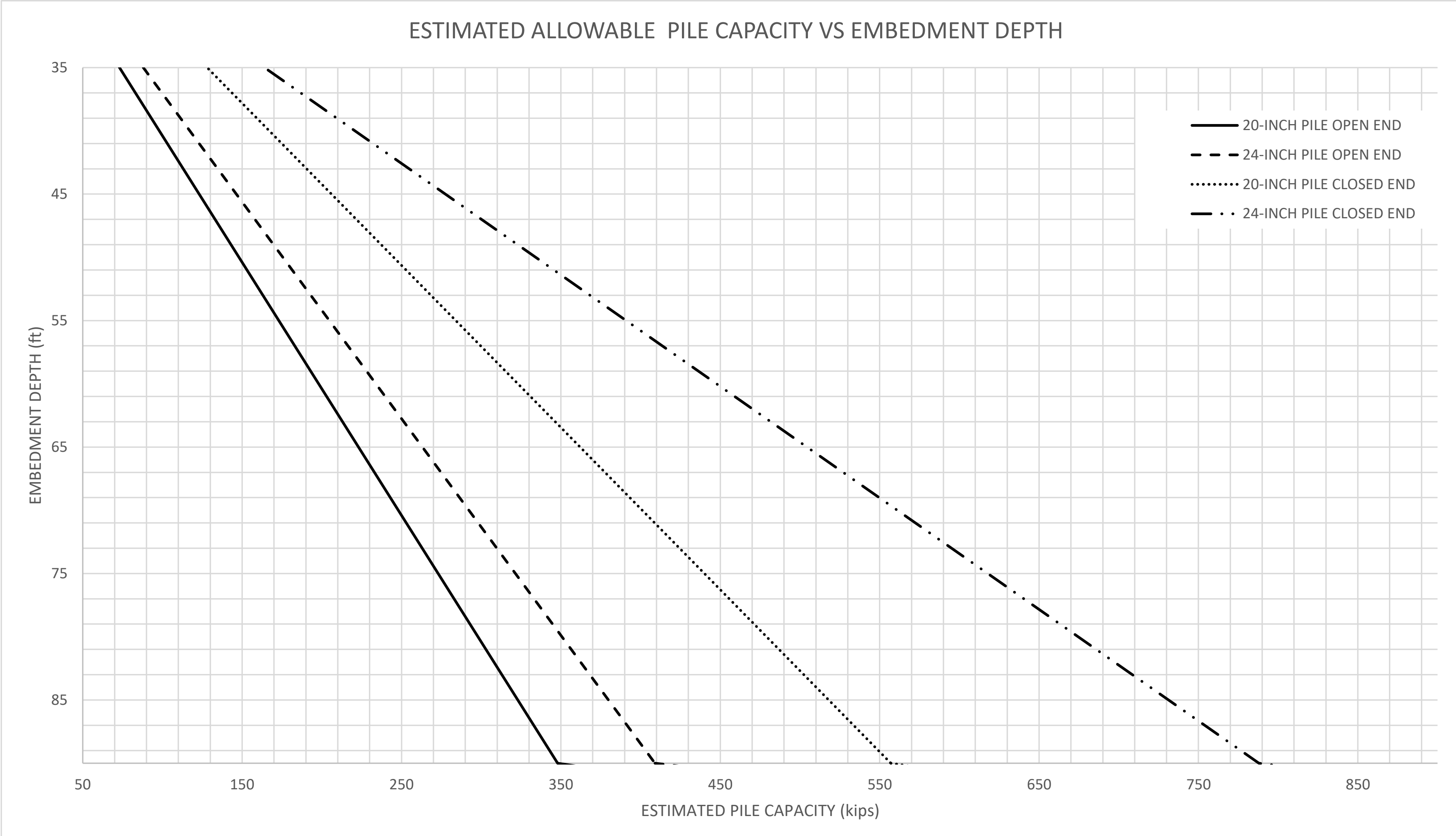


DRAWINGS MODIFIED FROM SHEET S02 (GENERAL ARRANGEMENT) AND SHEET S03 (GENERAL ARRANGEMENT) OF 30% PRELIMINARY OF BR. 25.70 REPLACEMENT (JULY 2019).

ELEVATION VIEWS

NORTHERN GEOTECHNICAL ENGINEERING, INC.
TERRA FIRMA TESTING

FIGURE TITLE: PROPOSED SITE LAYOUT	
PROJECT NAME: ALASKA RAILROAD CORPORATION BRIDGE 25.7	PROJECT ID: 5566-19
PROJECT LOCATION: TRAIL RIVER, ALASKA	FIGURE NUMBER: 2



NOTES:

FACTOR OF SAFETY = 2

MINIMUM EMBEDMENT OF 35 FEET BGS

IF BEDROCK IS ENCOUNTERED AFTER MINIMUM EMBEDMENT DEPTH IS MET THE ALLOWABLE BEARING CAPACITY IS 435 AND 575 KIPS FOR A 20-INCH 1/2 INCH WALL PILE AND 24-INCH 1/2 INCH WALL PILE RESPECTIVELY

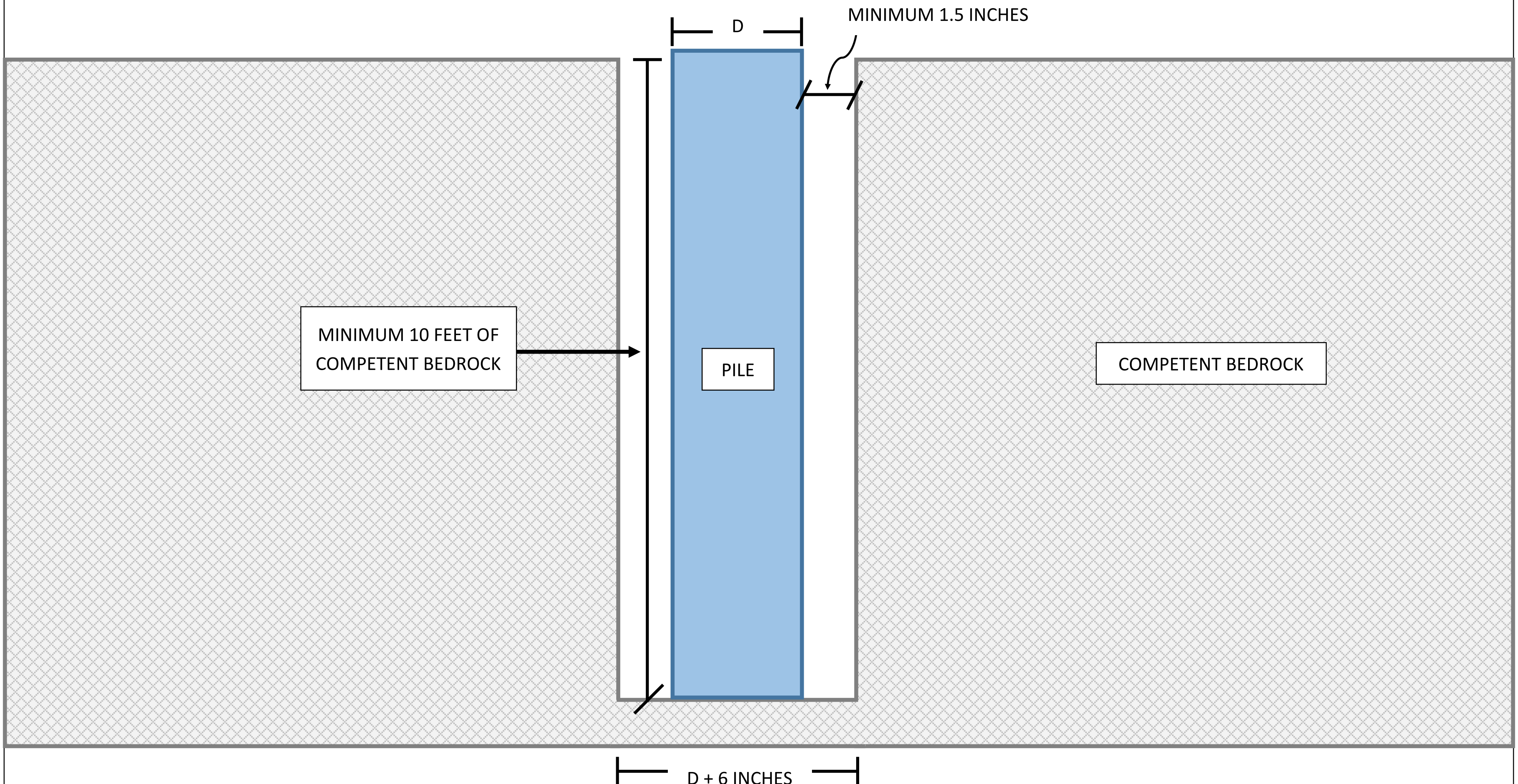
ALLOWABLE BEARING LOADS MAY BE INCREASED BY 1/3 FOR SHORT-TERM WIND AND SEISMIC LOADS

ALLOWABLE UPLIFT LOADS = 1/2 OF THE ALLOWABLE OPEN END ALLOWABLE BEARING LOADS

DO NOT ADD 1/3 FOR SHORT–TERM WIND AND SEISMIC LOADS FOR UPLIFT CONDITIONS

NORTHERN GEOTECHNICAL ENGINEERING, INC.
TERRA FIRMA TESTING

FIGURE TITLE: ESTIMATED ALLOWABLE PILE CAPACITY VS. EMBEDMENT DEPTH	
PROJECT NAME: ALASKA RAILROAD CORPORATION BRIDGE 25.7	PROJECT ID: 5566-19
PROJECT LOCATION: TRAIL RIVER, ALASKA	FIGURE NUMBER: 3



NOTE: ANNULUS BETWEEN PILE AND BEDROCK
SHOULD BE BACKFILLED WITH 4,000 PSI GROUT.

DRAWING NOT TO SCALE.

NORTHERN GEOTECHNICAL ENGINEERING, INC.
TERRA FIRMA TESTING

FIGURE TITLE: SOCKETED PILE SCHEMATIC	
PROJECT NAME: ALASKA RAILROAD CORPORATION BRIDGE 25.7	PROJECT ID: 5566-19
PROJECT LOCATION: TRAIL RIVER, ALASKA	FIGURE NUMBER: 4

APPENDIX A

USGS SEISMIC SITE CLASSIFICATION REPORTS

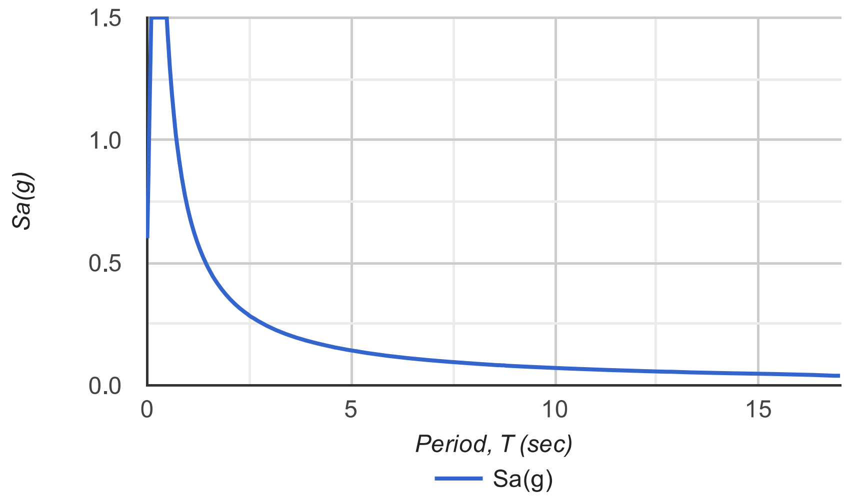
Latitude, Longitude: 60.43546248, -149.37188173

Date	10/29/2019, 11:00:02 AM
Design Code Reference Document	ASCE7-10
Risk Category	II
Site Class	B - Rock

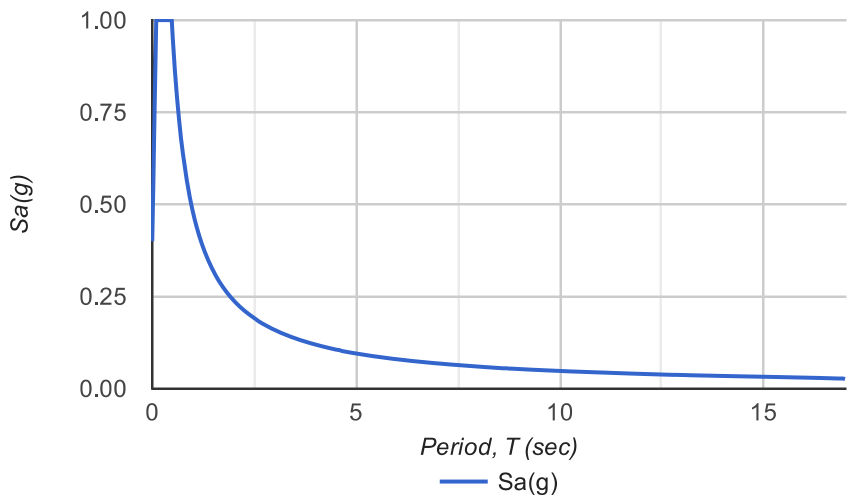
Type	Value	Description
S_S	1.5	MCE_R ground motion. (for 0.2 second period)
S_1	0.715	MCE_R ground motion. (for 1.0s period)
S_{MS}	1.5	Site-modified spectral acceleration value
S_{M1}	0.715	Site-modified spectral acceleration value
S_{DS}	1	Numeric seismic design value at 0.2 second SA
S_{D1}	0.477	Numeric seismic design value at 1.0 second SA

Type	Value	Description
SDC	D	Seismic design category
F_a	1	Site amplification factor at 0.2 second
F_v	1	Site amplification factor at 1.0 second
PGA	0.5	MCE_G peak ground acceleration
F_{PGA}	1	Site amplification factor at PGA
PGA_M	0.5	Site modified peak ground acceleration
T_L	16	Long-period transition period in seconds
SsRT	1.887	Probabilistic risk-targeted ground motion. (0.2 second)
SsUH	1.722	Factored uniform-hazard (2% probability of exceedance in 50 years) spectral acceleration
SsD	1.5	Factored deterministic acceleration value. (0.2 second)
S1RT	0.888	Probabilistic risk-targeted ground motion. (1.0 second)
S1UH	0.866	Factored uniform-hazard (2% probability of exceedance in 50 years) spectral acceleration.
S1D	0.715	Factored deterministic acceleration value. (1.0 second)
PGAd	0.5	Factored deterministic acceleration value. (Peak Ground Acceleration)
C_{RS}	1.096	Mapped value of the risk coefficient at short periods
C_{R1}	1.026	Mapped value of the risk coefficient at a period of 1 s

MCER Response Spectrum



Design Response Spectrum



DISCLAIMER

While the information presented on this website is believed to be correct, SEAOC /OSHPD and its sponsors and contributors assume no responsibility or liability for its accuracy. The material presented in this web application should not be used or relied upon for any specific application without competent examination and verification of its accuracy, suitability and applicability by engineers or other licensed professionals. SEAOC / OSHPD do not intend that the use of this information replace the sound judgment of such competent professionals, having experience and knowledge in the field of practice, nor to substitute for the standard of care required of such professionals in interpreting and applying the results of the seismic data provided by this website. Users of the information from this website assume all liability arising from such use. Use of the output of this website does not imply approval by the governing building code bodies responsible for building code approval and interpretation for the building site described by latitude/longitude location in the search results of this webstie.