GEOTECHNICAL FINDINGS REPORT

on

FOUNDATION SUBGRADE & APPROACH EMBANKMENT CONDITIONS

at

ARRC BRIDGE 127.5
EAGLE RIVER, ALASKA

Prepared for:
Alaska Railroad Corporation
327 W. Ship Creek Avenue
Anchorage, AK 99501

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

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FEBRUARY 2021

NORTHERN GEOTECHNICAL ENGINEERING, INC. / TERRA FIRMA TESTING

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

February 3, 2021 NGE-TFT Project #5894-20

Alaska Railroad Corporation
327 W. Ship Creek Avenue
Anchorage, AK 99501

Attn: Dave Kabella – Senior Project Manager

RE: GEOTECHNICAL FOUNDATION SUBGRADE & APPROACH EMBANKMENT CONDITIONS AT ARRC BRIDGE 127.5 – EAGLE RIVER, ALASKA

Dave,

We (Northern Geotechnical Engineering, Inc. *d.b.a.* Terra Firma Testing) have completed a geotechnical engineering assessment of existing foundation subgrade and bridge approach embankment conditions at ARRC Bridge 127.5. Our assessment suggests that the project site is underlain by relatively shallow sedimentary bedrock which is suitable for supporting the foundations for the planned replacement bridge. The type of foundation(s) ultimately selected to support the replacement bridge will likely be a function of both cost and constructability. The fill which comprises the existing bridge approach embankments appears to have a relatively loose consistency, however, we do not expect that the loose nature of the existing approach embankment fill will negatively affect the performance of the new approach embankments for the planned replacement bridge, and additional fill can be placed directly onto the existing approach embankments following the guidelines that we outline in the following report.

The following report contains a summary of our field and laboratory testing activities, as well as our conclusions regarding the geotechnical aspects of the project site as they pertain to the preliminary design of the planned replacement bridge foundations and approach embankments. Additional geotechnical engineering evaluation and analysis will be required once a final bridge design and anticipated pier foundation loadings have been developed.

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

Andrew C. Smith, CPG
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PHOTO: Drilling of Exploration B3 at north pier site of Bridge 127.5

1.0 INTRODUCTION

In this report, we (Northern Geotechnical Engineering, Inc. *d.b.a.* Terra Firma Testing) present the results of a geotechnical engineering assessment that we conducted at the site of Alaska Railroad Corporation (ARRC) Bridge 127.5, which we hereafter refer to as "the project site". We provided our professional service in accordance with our service fee proposal #20-216 which we submitted to our client, ARRC, on October 15, 2020. ARRC authorized our proposed scope of service on October 21, 2020 via Task No. 001 of Purchase Order 001 (under Term Contract 117848).

ARRC contracted us to characterize bridge pier foundation subgrade and approach embankment conditions at the project site to assess the suitability of the existing materials to support a new bridge (and approach embankments) which are to replace the existing span. In this report, we provide a summary of our field and laboratory testing activities, as well as provide our conclusions regarding the geotechnical aspects of the project site as they pertain to the preliminary design of the planned replacement bridge and approach embankments.

2.0 PROJECT OVERVIEW

The ARRC plans to replace the existing rail bridge located at Mile 127.5 of their main rail alignment between Seward and Fairbanks, Alaska. The existing rail bridge (which we hereafter refer to as Bridge 127.5) spans Eagle River, approximately two river miles downstream of the collateral Alaska Department of Transportation and Public Facilities Glenn Highway bridges, which also span Eagle River. We detail the approximate location and configuration of Bridge 127.5 in Figure 1 of this report.

According to historic drawings provided by ARRC, Bridge 127.5 (as it currently exists) was constructed circa 1923 and consists of a total of five steel deck girder spans supported by two central steel truss piers and two reinforced concrete abutment piers: for a total bridge span length of 308 feet. Each leg of the two central truss piers (four legs per pier) are founded on individual reinforced concrete footings which extend approximately 12 to 16 feet below the existing grade and likely bear onto either bedrock or competent soils (or both). Each bridge abutment pier consists of a large reinforced concrete spread footing, which supports a vertical cast concrete buttress and backwall, all of which are encased inside of earthen fill that forms the approach embankment at either end of the existing bridge. Portions of the previous (abandoned) timber bridge trestles are also reportedly encased within the existing approach embankments. Both abutment pier footings also likely bear onto either bedrock or competent soils (or both). We have included a copy of the 1922 construction bid drawing for Bridge 127.5 in Figure 2 of this report for reference.

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As we detail in Figure 2 of this report, Bridge 127.5 spans Eagle River at a location where the river has cut down through existing glacially-deposited sediments which overlie the surrounding area and forms a relatively deep ravine approximately 1000 feet in width and 60 to 70 feet in depth; with steep side slopes that enclose a relatively narrow meander belt (i.e., historic flood plain and abandoned and active river channels). The rail alignment crosses the ravine formed by Eagle River at an approximately 40 degree angle to the axis of the ravine and the two bridge approach embankments extend out into the ravine approximately 300 to 500 feet before they tie into the abutment piers for Bridge 127.5.

The ARRC plans to replace Bridge 127.5 with a new three-span bridge. The new (i.e., replacement) bridge will be located immediately downstream of existing Bridge 127.5, and the existing bridge will remain in service until the replacement bridge has been constructed: after which time the existing bridge will be disassembled and removed from the project site.

The design of the replacement bridge has not yet been finalized. However, ARRC's preliminary design concept for the replacement bridge includes:

- one, 70-ft long south approach span;
- one, 165-ft long main (i.e., central) span, and;
- one 165-ft long north approach span;

for a total bridge length of approximately 400 feet. The three new bridge spans will be supported by two abutment piers and two central piers. The abutment piers for the replacement bridge will be located an unknown distance behind the existing abutment piers (i.e., further away from, and upslope of, Eagle River) and the central piers for the replacement bridge will be located on opposing banks of Eagle River (adjacent to the central truss piers of the existing bridge). The central piers for the replacement bridge will likely be located above the mean seasonal high-water level of Eagle River but may be situated partially and/or wholly within the current flood plain of Eagle River.

Preliminary pier foundation concepts for the replacement bridge include drilled shaft/pile foundations (i.e., concrete caissons or driven steel pipe piling) and/or large poured concrete footings with tension anchors. The earthen approach embankments for the replacement bridge will likely incorporate portions of the existing bridge approach embankments, as the ARRC right-of-way (ROW) is not wide enough at the project site to allow for the construction of separate bridge approach embankments.

3.0 GEOLOGIC SETTING

The project site is located within the physiographic province *(i.e.,* geomorphic province) referred to by the United States Geological Survey (USGS) as the *Cook Inlet-Lower Susitna* province (Wahrhaftig, 1965). This province is characterized by glaciated lowlands dominated by ground moraine and stationary glacial landform deposits underlain by poorly consolidated coal-bearing rocks (i.e., bedrock) of Tertiary age (i.e., Paleogene to Neogene period or 2.6 to 66 million MA).

More precisely, the project site is located within an informal subunit of the *Cook Inlet-Lower Susitna* province known as the *Anchorage Lowland* (Schmoll & others, 1984), which is characterized mainly by low to moderate-relief hills of glacial drift (*i.e.,* glacial moraine deposits) having intervening meltwater channels, many of which are surfaced by boggy ground (Yehle & Schmoll, 1989).

The glacial deposits located within the *Anchorage Lowland* are Pleistocene in age and the predominant glacial landforms/deposits surrounding the project site are associated with the Elmendorf ground moraine. Elmendorf ground moraine deposits generally consist of massive diamicton (*i.e.,* glacial till) consisting of varying mixtures of unsorted to poorly-sorted silty sand and gravel sediments (with occasional large boulders) which are moderately to well compacted (Yehle & Schmoll, 1989). The Elmendorf ground moraine deposits are variably incised/overlain by glaciofluvial deposits of sand and gravel (deposited in glacial melt-water channels) and fine-grained/organic glaciolacustrine deposits (deposited in small ponds and lakes). The entire area is subsequently overlain by a thin blanket of aeolian (wind-deposited) silt/sand and (where undisturbed) a thin organic mat consisting of decaying vegetative matter and root masses. Deposits of peat often occur locally in low-lying wetland/bog areas.

The glacial deposits which overlie the *Anchorage Lowland* are underlain at depth by Mesozoic-aged metamorphic rocks of the *Kenai-Chugach Mountains*. However, adjacent to the Chugach Mountain front, these metamorphic rocks are overlain by relatively soft, sedimentary rocks (e.g., continental sandstone, siltstone, claystone, and minor coal) of Tertiary age, which most likely belong to the *Tyonek Formation* (a subunit of the *Kenai Group)*. These sedimentary rocks only outcrop in a few areas across the *Anchorage Lowland*; mainly along the Eagle River drainage west of the Glenn Highway and at scattered localities north of the community of Eagle River, Alaska (Yehle & Schmoll, 1989).

Rocks of the *Tyonek Formation* are believed to be Oligocene in age (23 to 33.9 MA) and constitute the middle portion of the *Kenai Group.* The *Tyonek Formation* generally consists of poorly indurated/lithified, massively bedded sandstones and lignitic coal beds which grade to sandy siltstone, claystone, and coal to the northeast of its aerial extent and are considered to be a poor water-bearing unit (Calderwood & Fackler, 1972 and Hartman et al, 1972). Existing isopach maps of the *Tyonek Formation* prepared by the Alaska Department of Geological and Geophysical Surveys suggest that these rocks range up to approximately 1000 feet in thickness in the area of the project site (Hartman et al, 1972 – Plate 7). Outcrops of rocks resembling those belonging to the *Tyonek Formation* have been identified at locations along the incised river channel of Eagle River (immediately upstream and downstream of the project site) by Clark (1970), Schmoll (1971), and Yehle (1989).

4.0 SUBSURFACE CHARACTERIZATION ACTIVITIES

We conceived, coordinated, and directed a subsurface exploration program at the project site in an effort to characterize the nature of:

1. the foundation subgrade adjacent to the existing bridge piers; and
2. the existing bridge approach embankments.

ARRC provided a rail-mounted crane, flat car, hi-rail pickups, and two-man support crew (crane operator and site foreman) to deploy our field personnel and exploration equipment to/from the project site and provide on-site lift support and track authority. ARRC allocated these resources and provided for site access for a total of five consecutive days between November 2nd and 6th, 2020. A qualified representative from our office was present on-site during the entire exploration program to select the exploration locations, direct the exploration activities, log the geology of each exploration, and collect representative samples for further identification and laboratory analysis.

We subcontracted Discovery Drilling, Inc. (DDI) of Anchorage, Alaska to provide the necessary geotechnical exploration services. Under our direction DDI advanced a total of three subsurface explorations (B1, B2, and B3) at the project site to depths ranging from approximately 70 to 88 feet below the existing ground surface (bgs) using a Geoprobe 7822 DT drill rig. DDI advanced explorations B1 and B2 using conventional hollow-stem auger drilling and split-spoon sampling methods, whereas DDI advanced exploration B3 using a HWT casing advancement system, which allows for the collection of both conventional split-spoon samples and continuous rock core samples.

4.1 Soil Borings

Under our direction, DDI performed a Modified Penetration Test (MPT) at regular intervals within the soil column during the drilling of all three explorations. 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 tool string 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 contained in Appendix A of this report.

We corrected the field blow count data for all three 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 contained in Appendix A of this report. The automatic drop-hammer that DDI used for this project is not standard, so we applied a correction factor of 1.1 to the $(N_1)_{60}$ values to account for the efficiency of the automatic drop-hammer used. 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.

Our field representative photographed each split-spoon sample that they collected during our exploration program and we have included these photographs in Appendix A of this report. Our field representative sealed each soil sample that they collected during our subsurface exploration program inside of an air-tight plastic bag, to help preserve the moisture content of each sample, and then submitted each sample to our laboratory for further identification and analysis.

We directed DDI to install one-inch diameter, open-ended PVC casing from the ground surface down to the bottom of explorations B1 and B2 to provide conduits (i.e., monitoring wells) for future groundwater level monitoring purposes (if necessary). As per our instruction, DDI hand-slotted a section of each monitoring well casing prior to installation and then backfilled the annulus of each monitoring well borehole with prescribed amounts of drill cuttings and engineered backfill. We have included construction diagrams for each groundwater monitoring that DDI installed at the project site on the graphical borehole logs contained in Appendix A of this report.

4.2 Bedrock Coring

DDI advanced the rock coring at exploration B3 using a HWT casing advancement system and a 5-ft long HQ core barrel with a diamond core bit. DDI retrieved continuous bedrock core samples from exploration B3 and our field representative sequentially placed each rock core sample into labeled/numbered waxed cardboard core boxes and returned each core box to our laboratory for further identification and analysis. Our field representative photographed each complete core box and we have included these photographs in Appendix A of this report. Once the rock coring effort at exploration B3 was complete, DDI backfilled the borehole with a Portland cement slurry up to the ground surface using tremie placement methods.

5.0 LABORATORY TESTING

We collected a total of 24 discrete soil samples and approximately 73 feet of continuous rock core from the three explorations that DDI advanced at the project site and submitted all of the samples to our laboratory for further identification and geotechnical analysis.

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 bridge improvements.

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5.1 Soil Testing

We tested select soil samples in accordance with the respective ASTM standard test methods including:

- moisture content analysis (ASTM D-2216);
- grain size sieve and hydrometer analysis (ASTM D-6913 & D-7928); and
- determination of fines content (a.k.a. P200 – ASTM D-1140).

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-7928 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 gradational results which are not representative of the actual (i.e., in-situ) soil gradation. 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.

We have included the results of our soil testing program 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.

5.2 Rock Testing

We tested select sections of the rock core in accordance with the respective ASTM standard test methods including:

- Unconfined compressive strength (ASTM D7012 – Method C);
- Splitting Tensile Strength (ASTM D3967); and
- Specific Gravity (ASTM C127).

We have included the results of our rock testing program 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.

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6.0 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). These 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.

6.1 General Subsurface Profile

In general, the project site is overlain by native deposits of relatively dense, glacially-derived sediments and/or alluvial sediments deposited by Eagle River; all of which rest unconformably upon sedimentary bedrock which we interpret to be a part the *Tyonek Formation* of the *Kenai Group* (See Sections 3.0 and 6.1.1 of this report for more detail regarding the *Tyonek Formation*). Along the existing track alignment, the native materials are overlain by a considerable thickness of imported fill which comprises the existing rail bed and bridge approach embankments and encases the existing bridge abutment piers.

Bedrock depths/overburden thicknesses generally increase perpendicular to the channel of Eagle River and we expect the surface of the bedrock to be relatively flat, with a slight dip to the northwest. The channel of Eagle River appears to be incised into the bedrock, and we observed a small bedrock outcrop in a cut bank approximately 100 yards downstream of the existing bridge (along the north bank of Eagle River – See Figures 2 & 4 of this report). The bedrock within the existing river channel is likely overlain by a relatively thin section of alluvial sediments consisting of differing mixtures of silt, sand, gravel, and cobbles (with occasional boulders). These alluvial sediments likely thin towards the distal boundaries of the flood plain where they likely transition into glacially-derived sediments. We have prepared a generalized geologic cross section of the project site (based on the information that we collected during our subsurface exploration program and existing 1922 construction bid drawings) and have included a copy of our cross section in Figure 5 of this report.

6.1.1 Bridge Piers

The area immediately surrounding the existing central truss piers appears to be overlain by a relatively thin section of alluvial sediments and/or fill (generally less than approximately 10 to 15 feet in total thickness); including minor amounts of armor stone/rip-rap approximately 2 to 4 feet in diameter (which primary occur around the base of the southern truss pier). Conversely, the areas immediately surrounding the two abutment piers are overlain by approximately 65 to 75 feet of embankment fill and native alluvial sediments.

The fill/alluvial sediments are underlain by sedimentary bedrock which appears to grade from a massively-bedded sandstone to thinly bedded siltstone with depth and contains intermittent carbonaceous siltstone layers and lignitic coals beds ranging from a few inches in thickness to over 13 feet in thickness.

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The bedrock appears to be poorly to moderately indurated/lithified, with a relative hardness ranging from soft to moderately hard, and the Rock Quality Designation (RQD) of the bedrock mass tends to decrease with depth (and decreasing bedding spacing). RQD values are highest in the upper sandstone intervals (i.e., top 50 feet of bedrock), with RQD values ranging from approximately 92 to 48, whereas the deeper siltstone and coal intervals have very low RQD values (approaching zero). Bedding planes within the bedrock exhibit an apparent dip of approximately 15 to 20 degrees and the dominant discontinuities in the rock core are bedding plane separations. We did not observe any significant discontinuities in the rock core (i.e., structural joints and/or fractures) which would suggest that the bedrock is bisected by any regional or systematic joint sets or other structural planes of weakness. As such, it is our professional opinion that the RQD values for the rock core are not necessarily representative of the inherent strength or quality of the in-situ rock mass.

Our laboratory testing suggests that the bedrock has unconfined compressive strengths ranging from approximately 3,600 psi to 10,200 psi and unconfined tensile strengths ranging from approximately 150 psi to 655 psi. In general, bedrock strength appears to increase with depth (and decreasing rock grain size). The silt-rich bedrock intervals (i.e., the siltstone and silty sandstone intervals) appear to exhibit the highest strength characteristics, whereas the upper sandstone interval appears to be weaker. Our laboratory testing also indicates that the bulk specific gravity (i.e., bulk density) of the bedrock increases with depth (and decreasing rock grain size), which suggests that the increase in bedrock strength with depth is primarily a function of increased cementation between individual rock grains (and an overall decrease in effective porosity). Bedrock strengths may also be linked (to a lesser degree) to rock grain size/shape/gradation and the overall degree of rock weathering and/or lithification.

6.1.2 Bridge Approach Embankments

The existing bridge approach embankments appear to consist primarily of imported fill which is similar in composition to the regional native glacial deposits (i.e., silty sand and gravel). The $(N_1)_{60}$ values that we determined from field blow count data suggest that the fill is relatively loose throughout its vertical extent, and the action of the drilling equipment that we observed during our exploration effort (e.g., auger chatter, drilling resistance, etc.) suggest that the fill contains varying fractions of cobble-sized particles. The thickness of the fill decreases with distance from either bridge abutment and overlies alluvial sediments deposited by Eagle River and/or native glacially-deposited sediments, all of which are underlain by the sedimentary bedrock which we describe in detail in Section 6.1.1 of this report.

6.2 Groundwater

We did not observe any visual indications of groundwater during our subsurface exploration effort. The existing bedrock and overlying silt-rich glacial soils exhibit poor water transmission properties, and as such, we expect the majority of any groundwater flow to be confined to the bedrock/soil contact (within the alluvial sediments adjacent to Eagle River). This flow may be sporadic and of varying volume. Due to the general lack of structural fracturing, and assumed low porosity within the bedrock, we expect groundwater storage/transmission in the bedrock to be low.

6.3 Frozen Soils

We did not observe any indications of frozen subgrade conditions during our subsurface exploration effort and we do not expect permafrost to underline any portion of the proposed bridge alignment.

7.0 GEOTECHNICAL ENGINEERING CONCLUSIONS

Based on the findings of our subsurface exploration and laboratory testing efforts, it is our conclusion that the existing subgrade at Bridge 127.5 is generally suitable to support the proposed replacement bridge and approach embankments; provided that our concerns and recommendations that we present in this report are addressed by the design and construction processes.

ARRC has yet to establish any specific design criteria for the replacement bridge, and as such, our conclusions are based on ARRC's preliminary design concept, which we detail in Section 2.0 of this report. Additional geotechnical engineering evaluation and analysis will be required once a final bridge design and anticipated pier foundation loadings have been developed (See Section 8.0 of this report).

7.1 Replacement Bridge Piers

Preliminary foundation concepts for the replacement bridge piers (both abutment piers and central span piers) include both shallow poured concrete foundation footings and deep foundation systems such as drilled concrete caissons and/or driven steel pipe piling; or some combination of all the above. Given the presence of shallow bedrock at the project site, and the relatively large, anticipated bridge pier loads, it is likely that all bridge pier foundations for the planned replacement bridge (regardless of foundation type) will bear directly onto/into bedrock. The foundation type(s) ultimately selected for the replacement bridge will likely be a function both cost and constructability. However, given the proximity of the planned replacement bridge alignment to the existing bridge alignment, it is likely that both abutment piers for the planned replacement bridge will need to be founded on some form of deep foundation system, as the existing bridge approach embankments will likely make it difficult to construct shallow foundation footings at the planned abutment locations (due to an excessive thickness of embankment fill). Conversely, the central piers for the planned replacement bridge will likely be situated outside of the footprint of the existing bridge approach embankments (and outside of the existing river channel) and therefore can consist of either shallow or deep foundation systems (or some combination of both).

Minimum bedrock embedment depths for any deep foundation systems will be primarily a function of the anticipated lateral and uplift loads, as our laboratory testing suggests that the bedrock can support bearing loads on the order of approximately 3,600 psi to 10,200 psi without any risk of differential settlement or other vertical rock loading failure. However, we expect that the minimum

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bedrock embedment for any deep foundations constructed for the two new abutment piers will be significantly less than those required for the two new central piers, as portions (or all) of the two new abutment piers will likely be encased in earthen fill as part of the construction of the new approach embankments (similar to the existing abutment piers), and the embankment fill will carry the majority (if not all) of the lateral and uplift loads for the new abutment piers.

Pilot hole drilling will be required for any pile foundation system constructed at the project site, as the existing bedrock (while poorly lithified) will likely prove too hard to allow for effective pile driving without risking damage to individual piling and/or initiating pile refusal before minimum embedment depths can be achieved. Drilling resistance in the existing bedrock, however, should be relatively low, and conventional drilling techniques (e.g., solid flight augers, air rotary, etc.) will likely suffice to advance boreholes of any diameter in the existing bedrock. Furthermore, boreholes drilled into the bedrock should remain relatively open and free from excessive slough and/or collapse without the need for protective casing, etc.

Shallow foundations placed directly onto the bedrock will likely require some form of uplift resistance to carry the anticipated lateral and uplift loads for the planned replacement bridge piers; which we expect to be fairly large. The easiest way to resist these uplift forces will likely be to use some form of steel anchor(s) grouted into the underlying bedrock and structurally connected to the pier footings (under tension). The anchors can be installed prior to the construction of the pier footings (and then cast into the footing) or be drilled through previously constructed footings and into the underlying bedrock. The number, size, spacing, and embedment of any foundation anchors required will be a function of the anticipated lateral and uplift loads for the replacement bridge as well as the tensile strength of the bedrock interval grouted, and the strength of the anchors/grout used. Our laboratory testing suggests that the bedrock has maximum tensile strengths ranging from approximately 150 psi to 655 psi, which should allow for a relatively coarse anchor spacing and/or shallow to moderate anchor embedment. We should be consulted once individual footing uplift loads have been established so that we can develop appropriate foundation anchor design criteria.

As we mention in Section 6.1.1. of this report, the bedrock does not appear to contain any significant structural jointing or other fracturing, and the dominant discontinuities in the bedrock appear to consist of bedding plane separations, which primarily manifest themselves once the bedrock becomes exposed in core/cut/outcrop (i.e., unconfined). The bedding planes within the bedrock are roughly perpendicular to the anticipated vertical bridge loads (both bearing and uplift), so we do not expect bedding plane separations within the bedrock mass to reduce the overall strength of the bedrock (as it pertains to bridge support). Furthermore, due to the general lack of bedrock fracturing, we do not expect much grout penetration into the bedrock formation during any foundation anchor installation or other bedrock grouting activities (conducted at normal injection pressures).

Additionally, the general lack of bedrock jointing/fracturing suggests that groundwater seepage/transmission through the bedrock will be relatively low: except near the bedrock surface where the bedrock has been variably weathered and is close to the local groundwater source (i.e., Eagle River). As such, the upper 10 to 15 feet of any boreholes drilled into the bedrock (e.g., caisson/pile pilot holes, foundation anchor bores, etc.) may need to be cased to prevent groundwater seepage into the boreholes (if groundwater seepage will negatively affect construction activities).

The central piers for the planned replacement bridge will likely be situated within the flood plain of Eagle River. However, assuming that the new pier foundations will bear directly onto/into bedrock, we do not anticipate that river scour (during potential flood events) will negatively impact the new pier foundations, as the bedrock subgrade will be relatively resistant to erosive forces (over the life expectancy of the planned replacement bridge).

7.2 Replacement Bridge Approach Embankments

As we mention in Section 2.0 of this report, the approach embankments for the planned replacement bridge will likely incorporate large sections of the existing bridge approach embankments due to an overall lack of ROW width at the project site. The existing bridge approach embankments extend up to approximately 60 to 70 feet above the native ground surface at the existing bridge abutments, and (at an estimated 1V:1.5H embankment slope) stretch laterally approximately 90 to 105 feet to either side of the track alignment (at the bridge abutment piers), and then narrow as they trend away from the bridge in either direction. As such, the edges of the existing approach embankments are already close to (or at) the boundary of the existing ARRC ROW. As such, the portions of the embankments for the replacement bridge approaches may extend beyond the boundaries of the ARRC ROW. As such, some form of retaining structure may be necessary to restrain the additional embankment fill on the downstream side of the new approach embankments and keep the additional fill entirely within the ARRC ROW. We can provide recommendations for such approach embankment retaining structures (if necessary) once the new track alignment and embankment grading plan have been developed. We have provided a generalized drawing in Figure 6 of this report demonstrating this concept.

As we mention in Section 6.2.1 of this report, the existing approach embankment fill appears to have a relatively loose consistency: despite being in-place for approximately 100 years and being subjected to repeated heavy train loads and at least two large magnitude (>7.0), long-duration seismic events (i.e., the 1964 Good Friday Earthquake and the 2018 Anchorage Earthquake); and it is likely that the embankment fill was placed without any formal compactive effort. It is not possible to ascertain how much (if any) settlement has occurred within the existing bridge approach embankments since their initial construction. However, we do not expect that the additional load imparted to the existing approach embankments (by the new approach embankments for the planned replacement bridge) will induce any significant settlements within the existing approach embankment that could negatively impact the performance of either the existing track alignment or the new track alignment for the planned replacement bridge. Any minor settlements which may occur within either the existing or planned approach embankments (due to consolidation of any loose/non-compact fill) can be repaired using ballast fill and/or additional embankment fill placed at the rail bed surface, which is part of routine track maintenance.

Furthermore, we do not anticipate that future pile driving activities within the existing bridge approach embankments (e.g., associated with the construction of the abutment piers for the planned replacement bridge) will significantly affect the condition (i.e., compactness) of the existing embankment fill. Some localized embankment settlements may occur directly adjacent to any pile foundations that get driven during the construction of the new abutment piers (as a result of any ground shaking generated during pile installation activities), however, we expect these potential settlements to be relatively small and have a relatively small radius of lateral influence (up to maximum of approximately 2 to 3 times the diameter of any piles being driven). Ultimately, the impact that any future pile driving activities may have on the existing rail bed will depend on several variables including (but not limited to): the size and proximity of the piles being driven; the type of pile driving equipment being used; the expertise of the pile driving contractor; etc. As a preventative precaution, it may be beneficial to monitor the elevation of the adjacent rail bed during any abutment pile driving activities to help ensure that the existing rail bed is not negatively impacted by any pile driving activities, and so that pile driving methods can be modified if the pile driving activities do result in any differential settlements within the existing rail bed.

Any fill placed along the slope of the existing approach embankments should be keyed into the existing fill slope in a tiered, stair-step manner as we detail in the conceptual drawing in Figure 6 of this report. The height of individual fill tiers/benches will likely be driven primarily by the size of the equipment used to cut the tier/bench and the maximum height that existing fill will stand in a vertical cut. Any organic material present at the ground surface should be removed to its vertical and lateral extents (within the footprint of the proposed fill) prior to any fill placement.

7.3 Seismic Design Parameters

We estimate that the seismic site classification for the project site is *C* (very dense soil and/or soft bedrock) and we estimate a peak ground acceleration for the project site of 0.6g. We expect the overall seismic liquefaction risk at the project site to be low given that the foundations for the planned replacement bridge will likely all bear onto bedrock. Some seismic liquefaction could potentially occur in the alluvial sediments associated with Eagle River (which underlie portions of the existing/planned bridge approach abutments; however, we expect this risk to be relatively low given the coarse-fraction and in-situ density of the alluvial sediments and the relatively thin vertical extent of these deposits.

8.0 DESIGN SUPPORT AND COORDINATION

The information contained within this report is intended to be used to help develop a preferred design approach for the planned replacement bridge, as it is not feasible to provide detailed geotechnical engineering recommendations without an established foundation design concept or anticipated foundation loads. As such, additional geotechnical engineering evaluation and analysis will be required once a preferred bridge design and anticipated pier foundation loadings have been developed. We should be consulted during all phases of the design process to help ensure that the foundation design is suitable for the subsurface conditions that we expect to occur at the project site and to provide detailed recommendations for the design and construction of the new bridge pier foundations and approach embankments; including (but not limited to):

- Pile/caisson design recommendations (sizing, spacing, embedment criteria, material requirements, etc.);
- Shallow foundation and anchor recommendations (sizing, spacing, embedment criteria, material requirements, etc.);
- Temporary shoring requirements for any excavations/borings;
- Pile and/or anchor grouting requirements;
- Embankment construction; and
- Slope stability assessments, etc.

9.0 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

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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. 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.

10.0 CLOSURE

We (Northern Geotechnical Engineering, Inc. d.b.a. Terra Firma Testing) prepared this report exclusively for the use of the Alaska Railroad Corporation and their consultants/contractors/etc. for use in the design of the proposed replacement bridge. 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.

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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 9.0 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.

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11.0 REFERENCES CITED

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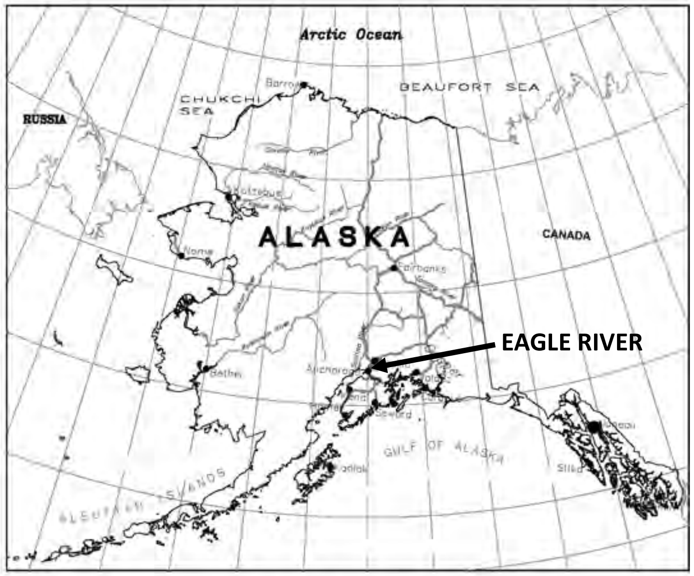
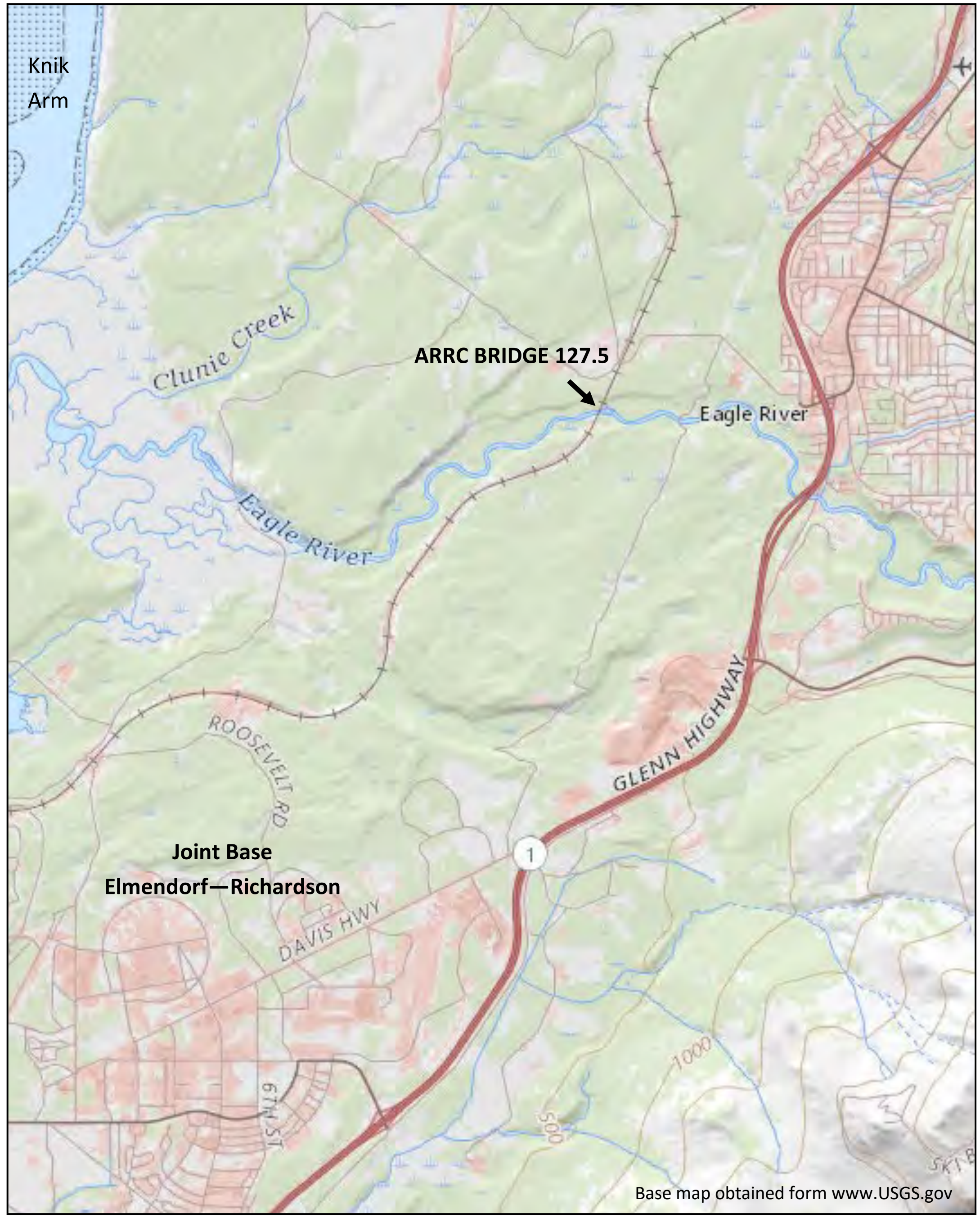
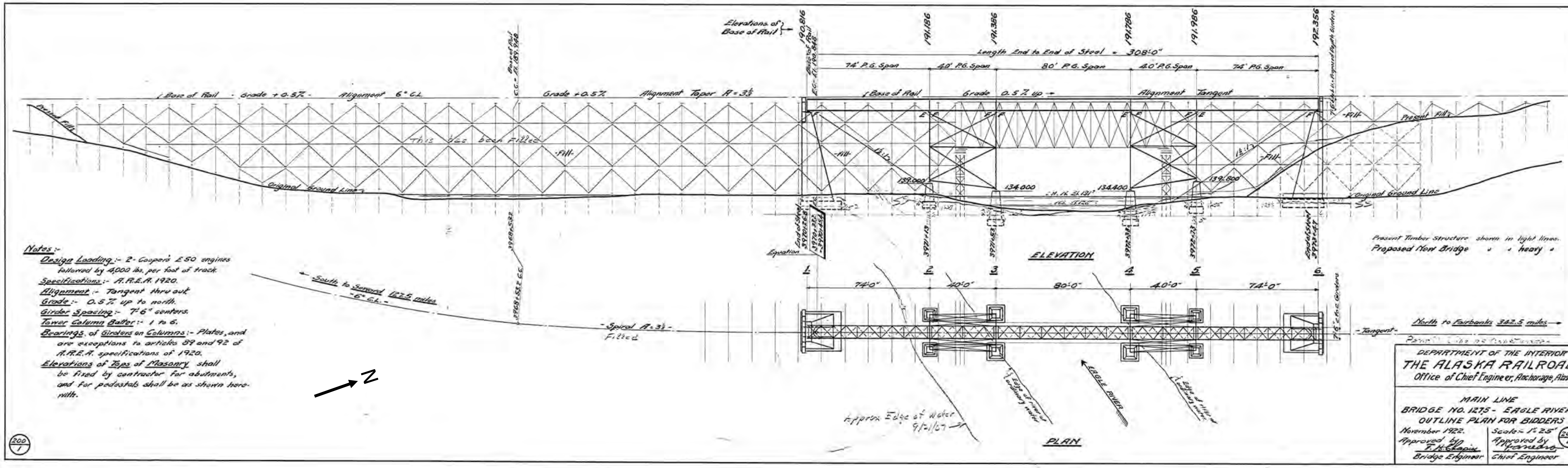
REPORT FIGURES

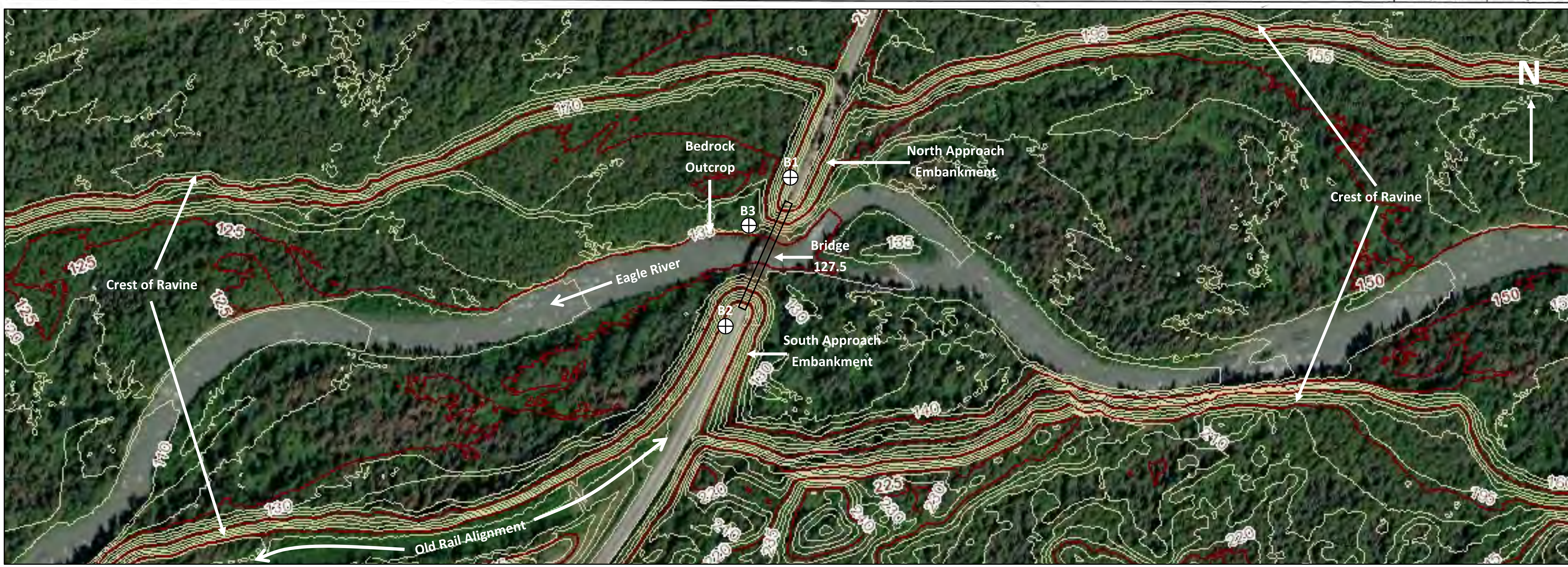




NORTHERN GEOTECHNICAL ENGINEERING, INC.
TERRA FIRMA TESTING

FIGURE TITLE: PROJECT SITE LOCATION	
PROJECT NAME: ARRC BRIDGE 127.5 (EAGLE RIVER)	PROJECT ID: 5894-20
PROJECT LOCATION: EAGLE RIVER, ALASKA	FIGURE NUMBER: 1





⨁ = Approximate location of subsurface exploration

Aerial imagery and topography obtained from Municipality of Anchorage (https://muniorg.maps.arcgis.com/apps/Profile/)

FIGURE TITLE: BRIDGE 127.5 LAYOUT, SITE TOPOGRAPHY, & EXPLORTAION LOCATIONS

PROJECT NAME: ARRC BRIDGE 127.5 (EAGLE RIVER)

PROJECT LOCATION: EAGLE RIVER, ALASKA

PROJECT ID: 5894-20

FIGURE NUMBER: 2

NORTHERN GEOTECHNICAL ENGINEERING, INC.
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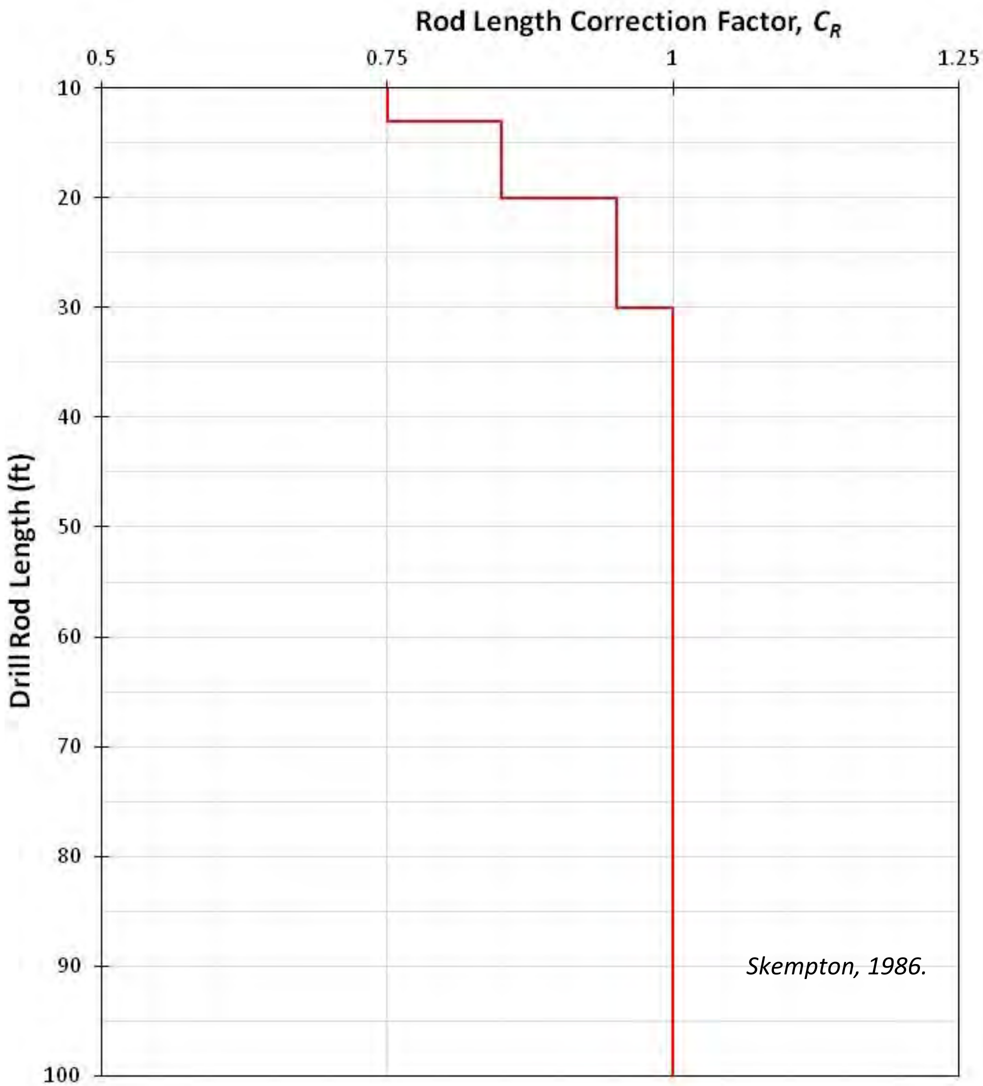

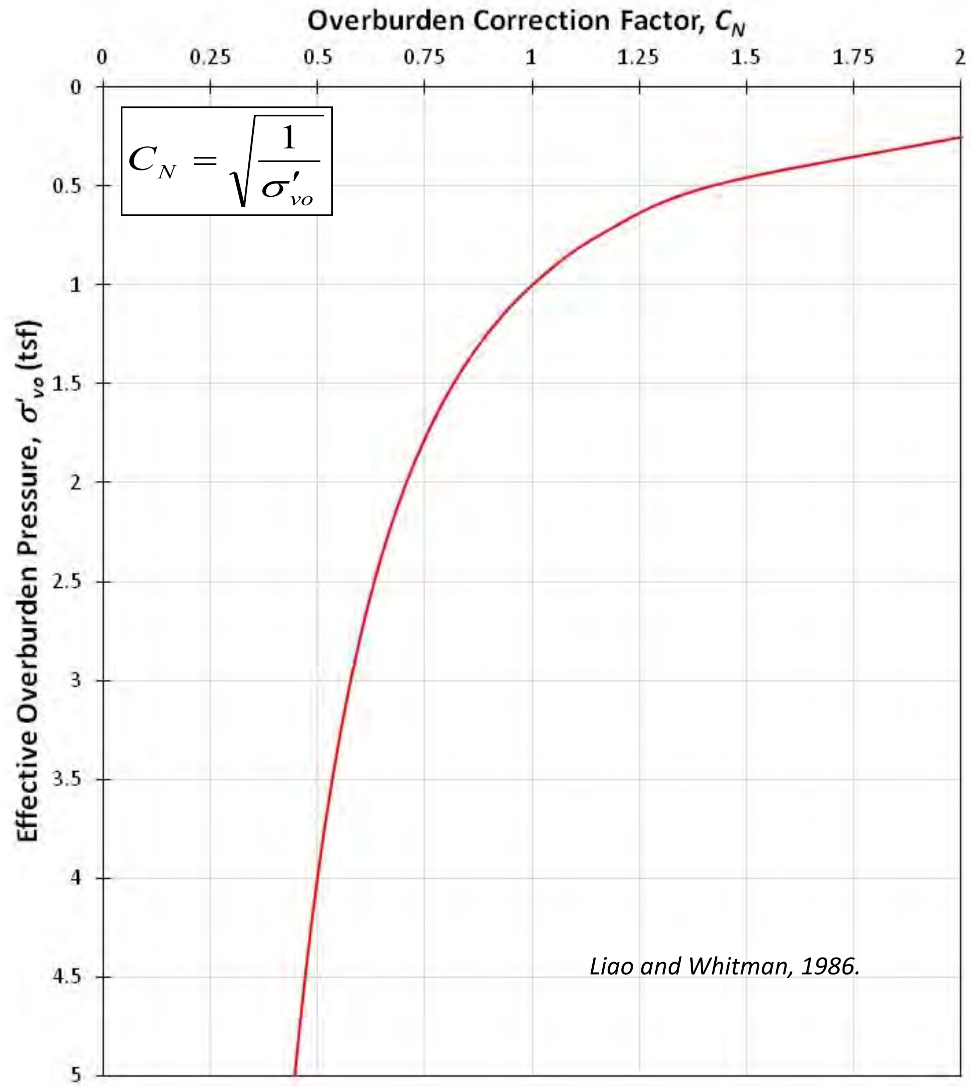

Notes:

- 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)

NORTHERN GEOTECHNICAL ENGINEERING, INC.
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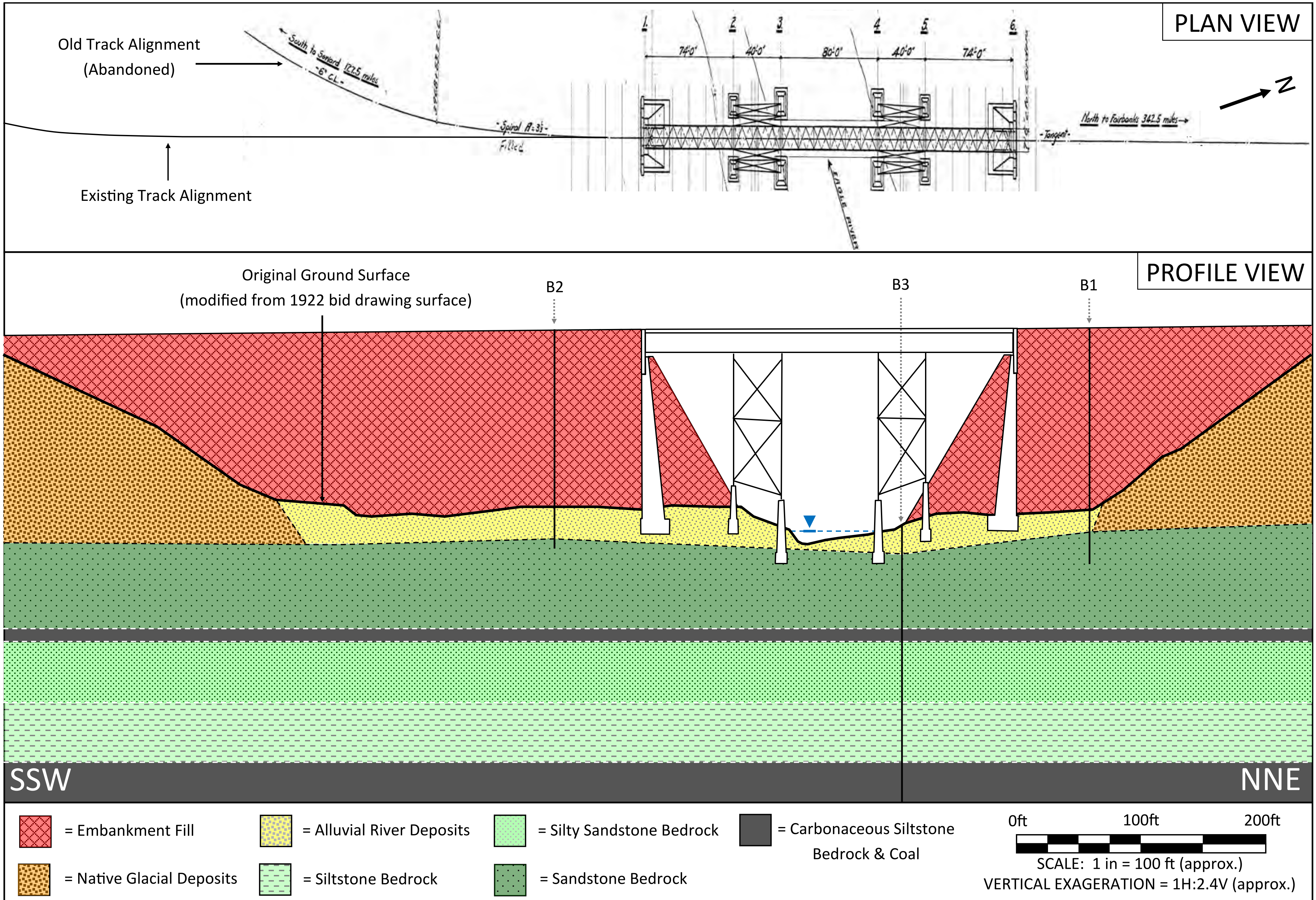
FIGURE TITLE: BLOW COUNT CORRECTIONS	
PROJECT NAME: ARRC BRIDGE 127.5 (EAGLE RIVER)	PROJECT ID: 5894-20
PROJECT LOCATION: EAGLE RIVER, ALASKA	FIGURE NUMBER: 3

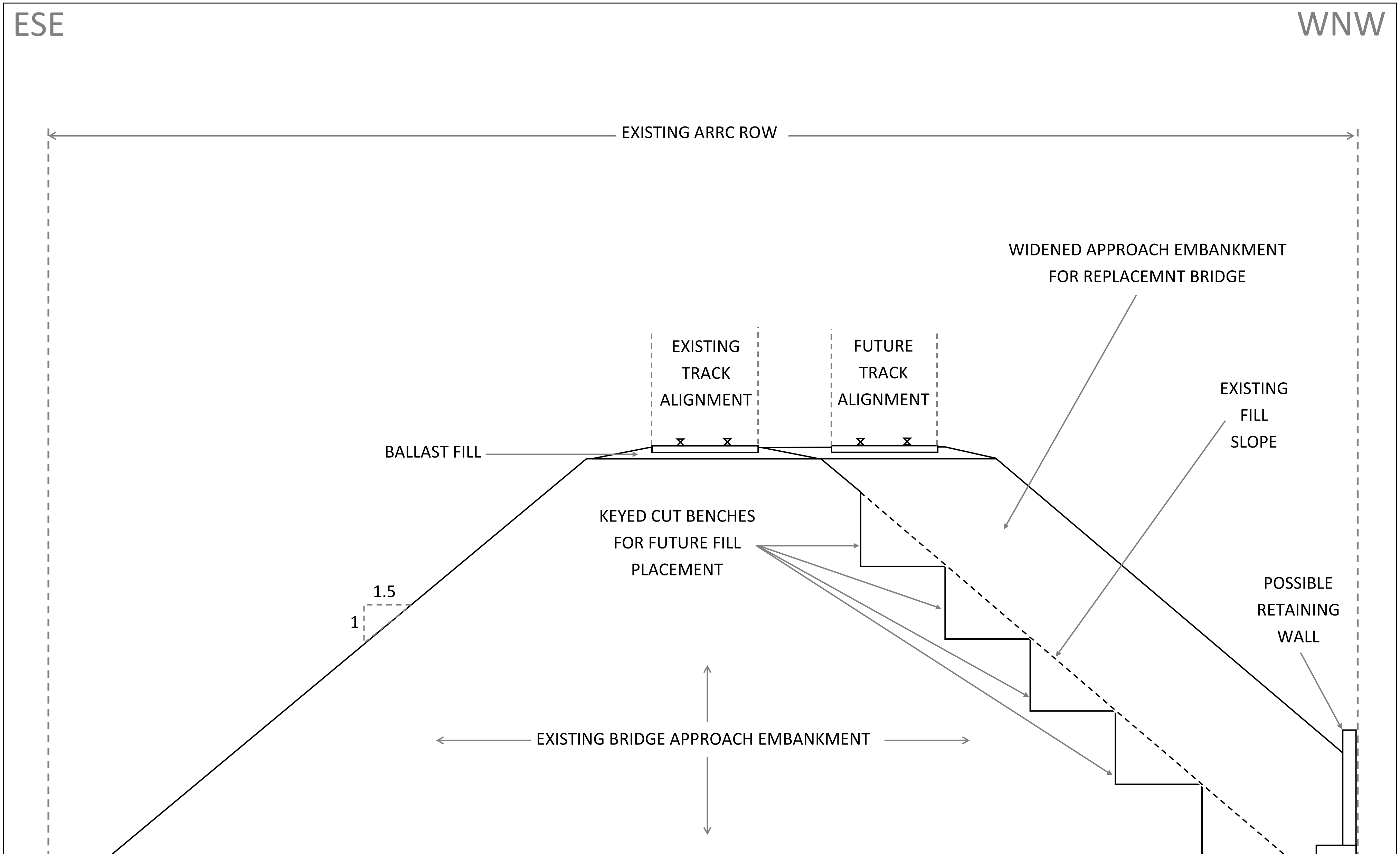
NORTHERN GEOTECHNICAL ENGINEERING, INC.
TERRA FIRMA TESTING

FIGURE TITLE: BEDROCK OUTCROP DOWNSTREAQM OF BRIDGE 127.5 (NORTH BANK)	
PROJECT NAME: ARRC BRIDGE 127.5 (EAGLE RIVER)	PROJECT ID: 5894-20
PROJECT LOCATION: EAGLE RIVER, ALASKA	FIGURE NUMBER: 4



NORTHERN GEOTECHNICAL ENGINEERING, INC.
TERRA FIRMA TESTING

FIGURE TITLE: GENERALIZED GEOLOGIC CROSS SECTION	
PROJECT NAME: ARRC BRIDGE 127.5 (EAGLE RIVER)	PROJECT ID: 5894-20
PROJECT LOCATION: EAGLE RIVER, ALASKA	FIGURE NUMBER: 5



DRAWING NOT TO SCALE

NORTHERN GEOTECHNICAL ENGINEERING, INC.
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FIGURE TITLE: GENERAL BRIDGE APPROACH EMBANKMENT WIDENING CONCEPTS	
PROJECT NAME: ARRC BRIDGE 127.5 (EAGLE RIVER)	PROJECT ID: 5894-20
PROJECT LOCATION: EAGLE RIVER, ALASKA	FIGURE NUMBER: 6

APPENDIX A

GRAPHICAL EXPLORATION LOGS & SAMPLE PHOTOGRAPHS

Northern Geotechnical Engineering, Inc.
and Terra Firma Testing
11301 Olive Lane
Anchorage, AK 99515
Telephone: 907-344-5934

EXPLORATION B1

NGE-TFT PROJECT NAME: ARRC Bridge 127.5
NGE-TFT PROJECT NUMBER: 5894-20
PROJECT LOCATION: Eagle River, AK
EXPLORATION CONTRACTOR: Discovery Drilling, Inc.
EXPLORATION EQUIPMENT: Geoprobe 7822DT
EXPLORATION METHOD: Hollow Stem Auger
SAMPLING METHOD: MPT w/ 340lb autohammer
LOGGED BY: A. Smith
DATE/TIME STARTED: 11/2/2020 @ 12:30:00 PM
DATE/TIME COMPLETED: 11/3/2020 @ 1:00:00 PM
EXPLORATION LOCATION: Apx. 52 ft NNE of N. Bridge Abutment Backwall and 12 ft W of CL of Track
GROUND ELEVATION: Approx. 2 ft below top of rail
GROUNDWATER (ATD): None observed
GROUNDWATER (): N/A
EXPLORATION COMPLETION: See comments at end of log
WEATHER CONDITIONS: Clear, calm, 0-10°F

DEPTH (ft)	MATERIAL DESCRIPTION	FIELD SAMPLE ID	RECOVERY (in)	FIELD BLOWS	$(N_1)_{60}$	LAB SAMPLE ID	LAB RESULTS	REMARKS/NOTES
0	**RR Ballast** - crushed rock 4"-6"							
	FILL, **SILTY GRAVEL WITH SAND** (GM), some cobbles, loose, olive brown, damp, gravel up to 3" in diameter							
5		S1	8	3 3 3	7	S1	S1 MC = 3.0%	
10		S2	0	6 10 10	18	S2		No recovery - pushing cobbles.
15		S3	12	5 4 3	6	S3	S3 MC = 4.4% 45.4% gravel, 40.6% sand, 14.0% silt	
20		S4	6	6 7 5	10	S4	S4 MC = 2.9%	Poor recovery. Cobbles blocking end of sampler.
25		S5	12	4 4 3	5	S5	S5 MC = 7.1%	
30		S6	6	5 5 6	8	S6	S6 MC = 5.9%	Poor recovery, pushing cobbles ahead of sampler.
35								

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.

(Continued Next Page)

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EXPLORATION B1

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GROUND ELEVATION: Approx. 2 ft below top of rail
▽ **GROUNDWATER (ATD):** None observed
▼ **GROUNDWATER ():** N/A
EXPLORATION COMPLETION: See comments at end of log
WEATHER CONDITIONS: Clear, calm, 0-10°F

DEPTH (ft)	MATERIAL DESCRIPTION	FIELD SAMPLE ID	RECOVERY (in)	FIELD BLOWS	$(N_1)_{60}$	LAB SAMPLE ID	LAB RESULTS	REMARKS/NOTES
35	*FILL*, **SILTY GRAVEL WITH SAND** (GM), some cobbles, loose, olive brown, damp, gravel up to 3" in diameter *(continued)*	S7	12	7 6 9	10	S7	S7 MC = 6.5% 43.8% gravel, 37.7% sand, 18.5% silt	
40		S8	8	6 6 5	7	S8	S8 MC = 4.6%	Cobble blocking end of sampler.
45		S9	8	6 5 4	6	S9	S9 MC = 4.9%	
50		S10	12	4 4 5	5	S10	S10 MC = 8.9%	
55		S11	11	6 4 4	4	S11	S11 MC = 5.7% 52.7% gravel, 30.8% sand, 16.5% silt	
60	**ORGANICS** (Original ground surface) *NATIVE*, **SILTY SAND** (SM), medium dense, medium gray, damp	S12	14	11 5 4	10	S12	S12 MC = 18.9% P200 = 85.0%	Some woody debris in sampler.
65	*BEDROCK* **SANDSTONE**, light gray to dark gray, thinly bedded, soft, unfractured to slightly fractured, fractures consist of separations along bedding planes	S13	14	30 50 3"	N/A	S13	S13 MC = 8.7% P200 = 40.9%	Significantly increased drilling resistance below 66 ft bgs.
70								

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.

(Continued Next Page)

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GROUND ELEVATION: Approx. 2 ft below top of rail
GROUNDWATER (ATD): None observed
GROUNDWATER (): N/A
EXPLORATION COMPLETION: See comments at end of log
WEATHER CONDITIONS: Clear, calm, 0-10°F

DEPTH (ft)	GRAPHIC LOG	FROZEN SOILS	MATERIAL DESCRIPTION	SAMPLE TYPE	FIELD SAMPLE ID	RECOVERY (in)	FIELD BLOWS	$(N_1)_{60}$	SAMPLE INT. COLLECT	LAB SAMPLE ID	LAB RESULTS	REMARKS/NOTES	WELL DIAGRAM
70			*BEDROCK* **SANDSTONE**, light gray to dark gray, thinly bedded, soft, unfractured to slightly fractured, fractures consist of separations along bedding planes *(continued)*		S14	10	106	N/A		S14			

Bottom of borehole at 73.0 ft bgs.
Set 1" PVC casing to BOH. Hand slotted bottom 50 ft of casing. Backfilled with cuttings up to approx. 5 ft bgs, bentonite chips from 2-5 ft bgs, and drill cuttings up to ground surface

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 LOG
EXPLORATION B1

CLIENT Alaska Railroad Corporation

PROJECT NAME ARRC Bridge 127.5

PROJECT NUMBER 5894-20

PROJECT LOCATION Eagle River, AK

Exploration B1 Sample S2
Sample Interval 5 - 6.5 ft bgs

Exploration B1 Sample S3
Sample Interval 15 - 16.5 ft bgs

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PHOTO LOG
EXPLORATION B1

CLIENT Alaska Railroad Corporation

PROJECT NAME ARRC Bridge 127.5

PROJECT NUMBER 5894-20

PROJECT LOCATION Eagle River, AK

Exploration B1 Sample S4
Sample Interval 20 - 21.5 ft bgs

Exploration B1 Sample S5
Sample Interval 25 - 26.5 ft bgs

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PHOTO LOG
EXPLORATION B1

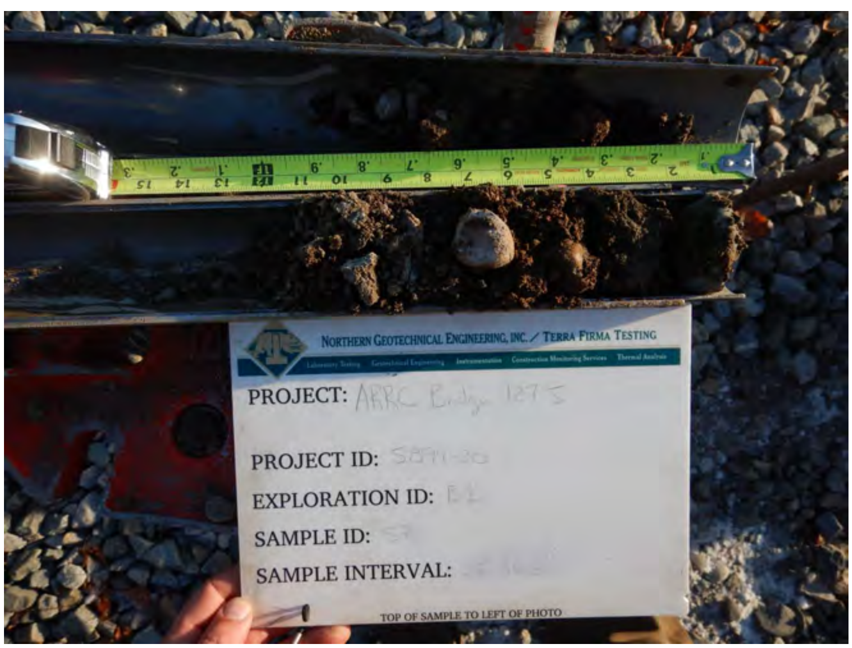
CLIENT Alaska Railroad Corporation

PROJECT NAME ARRC Bridge 127.5

PROJECT NUMBER 5894-20

PROJECT LOCATION Eagle River, AK

Exploration B1 Sample S
Sample Interval 30 - 31.5 ft bgs



Exploration B1 Sample S7
Sample Interval 35 - 36.5 ft bgs

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PHOTO LOG
EXPLORATION B1

CLIENT Alaska Railroad Corporation
PROJECT NAME ARRC Bridge 127.5
PROJECT NUMBER 5894-20
PROJECT LOCATION Eagle River, AK

Exploration B1 Sample S8
Sample Interval 40 - 41.5 ft bgs

Exploration B1 Sample S9
Sample Interval 45 - 46.5 ft bgs

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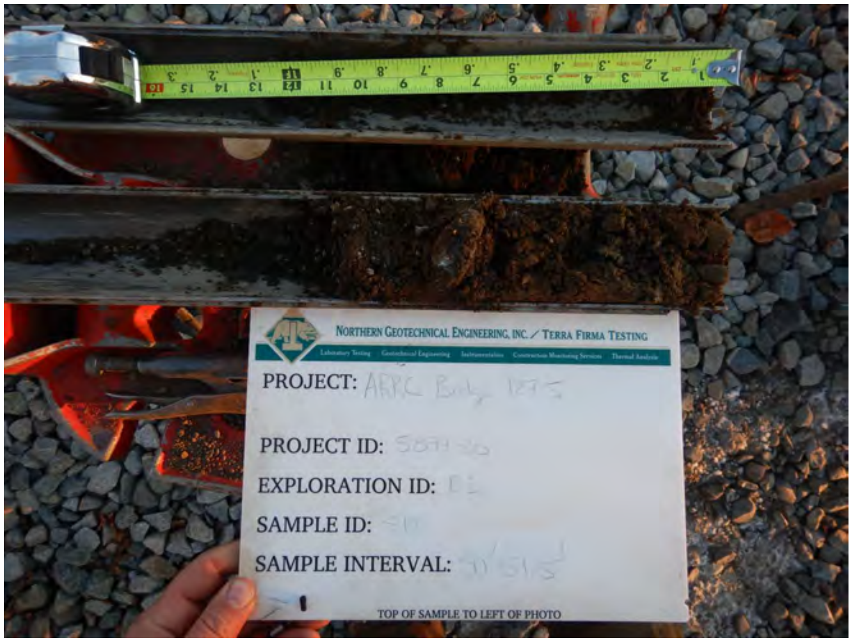
PHOTO LOG
EXPLORATION B1

CLIENT Alaska Railroad Corporation

PROJECT NAME ARRC Bridge 127.5

PROJECT NUMBER 5894-20

PROJECT LOCATION Eagle River, AK



Exploration B1 Sample S10
Sample Interval 50 - 51.5 ft bgs

Exploration B1 Sample S11
Sample Interval 55 - 56.5 ft bgs

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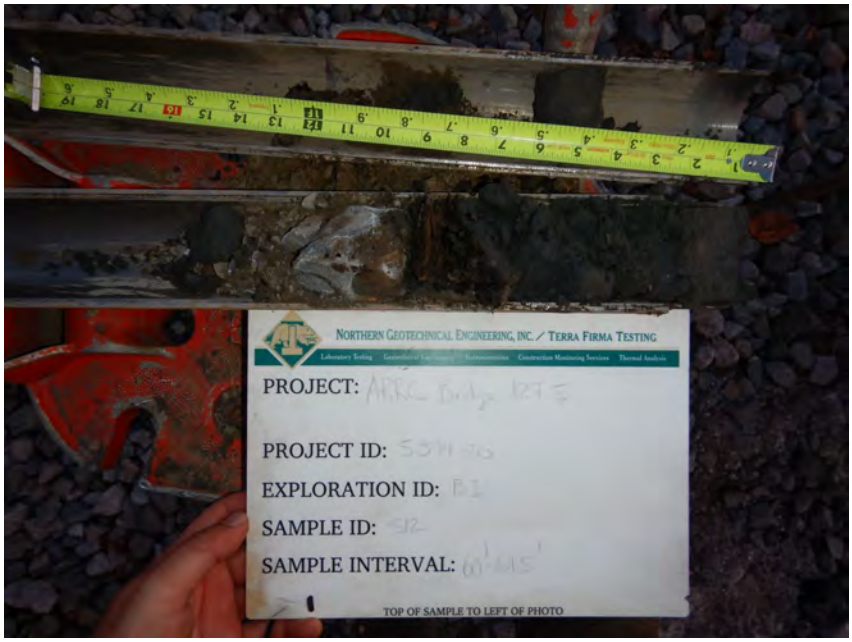
PHOTO LOG
EXPLORATION B1

CLIENT Alaska Railroad Corporation

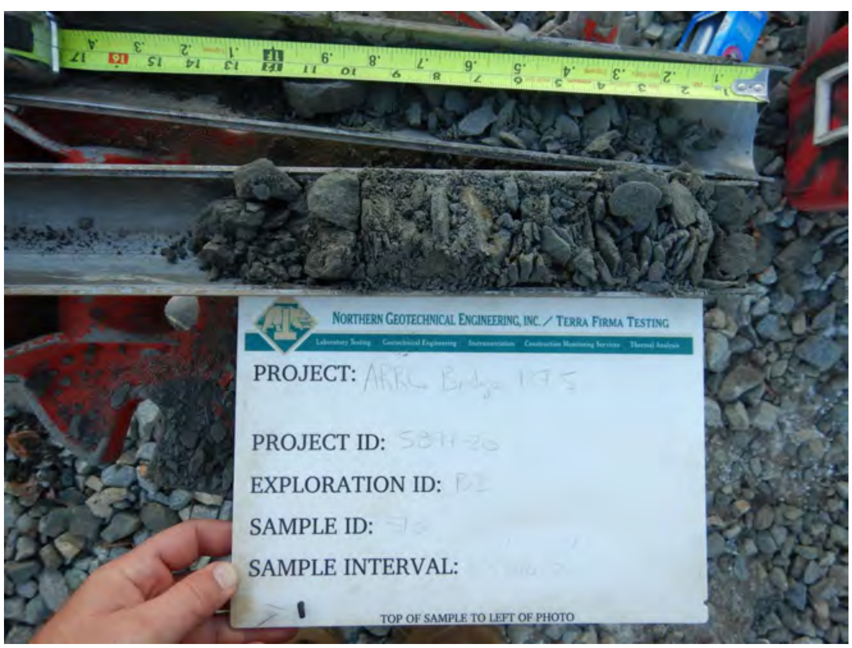
PROJECT NAME ARRC Bridge 127.5

PROJECT NUMBER 5894-20

PROJECT LOCATION Eagle River, AK



Exploration B1 Sample S12
Sample Interval 60 - 61.5 ft bgs



Exploration B1 Sample S13
Sample Interval 65 - 66.5 ft bgs

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PHOTO LOG
EXPLORATION B1

CLIENT Alaska Railroad Corporation

PROJECT NAME ARRC Bridge 127.5

PROJECT NUMBER 5894-20

PROJECT LOCATION Eagle River, AK

Exploration B1 Sample S14
Sample Interval 72.5 - 73 ft bgs

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EXPLORATION B2

NGE-TFT PROJECT NAME: ARRC Bridge 127.5

NGE-TFT PROJECT NUMBER: 5894-20

PROJECT LOCATION: Eagle River, AK

EXPLORATION CONTRACTOR: Discovery Drilling, Inc.

EXPLORATION EQUIPMENT: Geoprobe 7822DT

EXPLORATION METHOD: Hollow Stem Auger

SAMPLING METHOD: MPT w/ 340lb autohammer

LOGGED BY: A. Smith

DATE/TIME STARTED: 11/3/2020 @ 2:00:00 PM

DATE/TIME COMPLETED: 11/4/2020 @ 3:15:00 PM

EXPLORATION LOCATION: Apx. 70 ft SSW of S. Bridge Abutment Backwall and 13 ft W of CL of Track

GROUND ELEVATION: Approx. 3 ft below top of rail

▽ **GROUNDWATER (ATD):** None Observed

▼ **GROUNDWATER ():** N/A

EXPLORATION COMPLETION: See comments at end of log

WEATHER CONDITIONS: Clear, calm, 0-10°F

RR Ballast (0 ft)

FILL, **SILTY GRAVEL WITH SAND** (GM), loose, olive gray, gravel up to 3" in diameter, some cobbles 3"-6" in diameter

DEPTH (ft)	SAMPLE TYPE	FIELD SAMPLE ID	RECOVERY (in)	FIELD BLOWS	$(N_1)_{60}$	SAMPLE INT. COLLECT	LAB SAMPLE ID	LAB RESULTS	REMARKS/NOTES
5		S1	8	2 2 2	5		S1	S1 MC = 4.0%	
10		S2	12	1 1 4	5		S2	S2 MC = 7.8% 44.0% gravel, 42.9% sand, 13.1% silt	
15		S3	7	15 14 8	19		S3		Blows not representative due to cobbles. Increased drill chatter from 15-20 ft bgs.
20		S4	9	12 6 8	12		S4	S4 MC = 4.0%	Blows not representative due to cobbles.
25		S5	8	25 11 11	16		S5		Mostly broken rock/cobbles in sampler. Blows not representative.
30		S6	9	11 7 21 3"	N/A		S6	S6 MC = 5.4%	Refusal on cobble at approx. 31.25 ft bgs.
35									

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.

(Continued Next Page)

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EXPLORATION B2

NGE-TFT PROJECT NAME: ARRC Bridge 127.5

NGE-TFT PROJECT NUMBER: 5894-20

PROJECT LOCATION: Eagle River, AK

EXPLORATION CONTRACTOR: Discovery Drilling, Inc.

EXPLORATION EQUIPMENT: Geoprobe 7822DT

EXPLORATION METHOD: Hollow Stem Auger

SAMPLING METHOD: MPT w/ 340lb autohammer

LOGGED BY: A. Smith

DATE/TIME STARTED: 11/3/2020 @ 2:00:00 PM

DATE/TIME COMPLETED: 11/4/2020 @ 3:15:00 PM

EXPLORATION LOCATION: Apx. 70 ft SSW of S. Bridge Abutment Backwall and 13 ft W of CL of Track

GROUND ELEVATION: Approx. 3 ft below top of rail

GROUNDWATER (ATD): None Observed

GROUNDWATER (): N/A

EXPLORATION COMPLETION: See comments at end of log

WEATHER CONDITIONS: Clear, calm, 0-10°F

DEPTH (ft)	MATERIAL DESCRIPTION	FIELD SAMPLE ID	RECOVERY (in)	FIELD BLOWS	$(N_1)_{60}$	LAB SAMPLE ID	LAB RESULTS	REMARKS/NOTES
35	*FILL*, **SILTY GRAVEL WITH SAND** (GM), loose, olive gray, gravel up to 3" in diameter, some cobbles 3"-6" in diameter *(continued)*	S7	2	13 13 14	18	S7		No recovery. Cobble blocking end of sampler.
40	*FILL*, **SILTY SAND WITH GRAVEL** (SM), some cobbles, medium dense, olive brown, damp, gravel up to 2" in diameter	S8	10	6 7 7	15	S8	S8 MC = 4.9% 28.4% gravel, 53.1% sand, 18.5% silt	
45		S9	0	12 11 16	15	S9		No recovery. Cobble blocking end of sampler.
50		S10	14	17 9 10	21	S10	S10 MC = 4.8%	
55		S11	0	12 13 14	30	S11		No recovery.
60		S12	9	15 26 3"	N/A	S12	S12 MC = 4.2% 17.6% gravel, 60.8% sand, 21.6% silt	
65	*ALLUVIUM*, **GRAVEL** (No sample obtained)							Increased drill chatter at 65 ft suggests increased gravel content.
	BEDROCK **SANDSTONE**, light gray to dark gray, thinly bedded, soft, unfractured to slightly fractured, fractures consist of separations along bedding planes							
70								

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.

(Continued Next Page)

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EXPLORATION B2

NGE-TFT PROJECT NAME: ARRC Bridge 127.5
NGE-TFT PROJECT NUMBER: 5894-20
PROJECT LOCATION: Eagle River, AK
EXPLORATION CONTRACTOR: Discovery Drilling, Inc.
EXPLORATION EQUIPMENT: Geoprobe 7822DT
EXPLORATION METHOD: Hollow Stem Auger
SAMPLING METHOD: MPT w/ 340lb autohammer
LOGGED BY: A. Smith
DATE/TIME STARTED: 11/3/2020 @ 2:00:00 PM
DATE/TIME COMPLETED: 11/4/2020 @ 3:15:00 PM
EXPLORATION LOCATION: Apx. 70 ft SSW of S. Bridge Abutment Backwall and 13 ft W of CL of Track
GROUND ELEVATION: Approx. 3 ft below top of rail
GROUNDWATER (ATD): None Observed
GROUNDWATER (): N/A
EXPLORATION COMPLETION: See comments at end of log
WEATHER CONDITIONS: Clear, calm, 0-10°F

DEPTH (ft)	GRAPHIC LOG	FROZEN SOILS	MATERIAL DESCRIPTION	SAMPLE TYPE	FIELD SAMPLE ID	RECOVERY (in)	FIELD BLOWS	$(N_1)_{60}$	SAMPLE INT. COLLECT	LAB SAMPLE ID	LAB RESULTS	REMARKS/NOTES	WELL DIAGRAM
70					S13	9	127	N/A		S13	S13 MC = 7.0% P200 = 39.9%		

Bottom of borehole at 70.5 ft bgs.
Set 1" PVC casing to BOH. Hnad slotted botom 50 ft of casing. Backiflled with drill cuttings and p-gravel up to approx. 5 ft bgs, bentonite chips from 2-5 ft bgs, and then p-gravel up to ground surface

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PHOTO LOG
EXPLORATION B2

CLIENT Alaska Railroad Corporation
PROJECT NAME ARRC Bridge 127.5
PROJECT NUMBER 5894-20
PROJECT LOCATION Eagle River, AK

Exploration B2 Sample S1
Sample Interval 5 - 6.5 ft bgs

Exploration B2 Sample S2
Sample Interval 10 - 11.5 ft bgs

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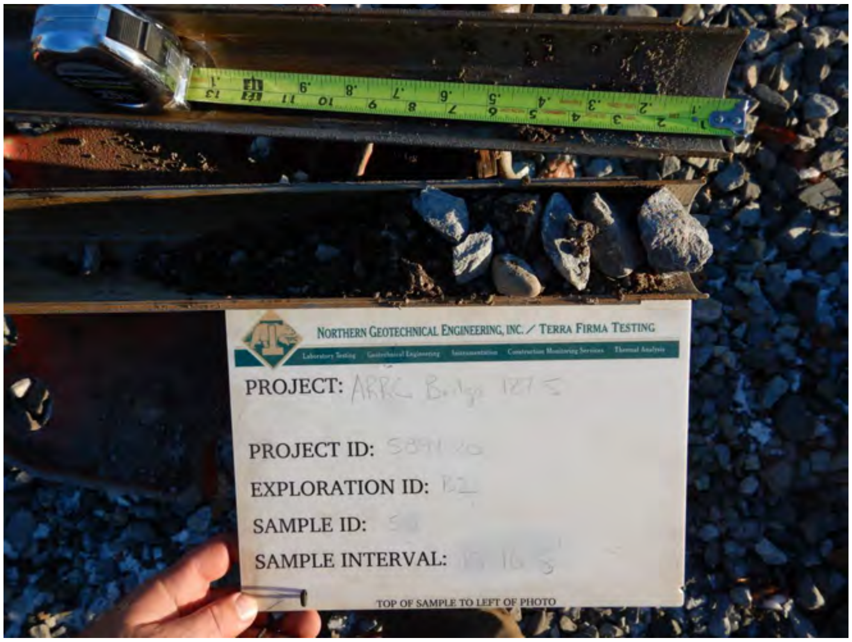
PHOTO LOG
EXPLORATION B2

CLIENT Alaska Railroad Corporation

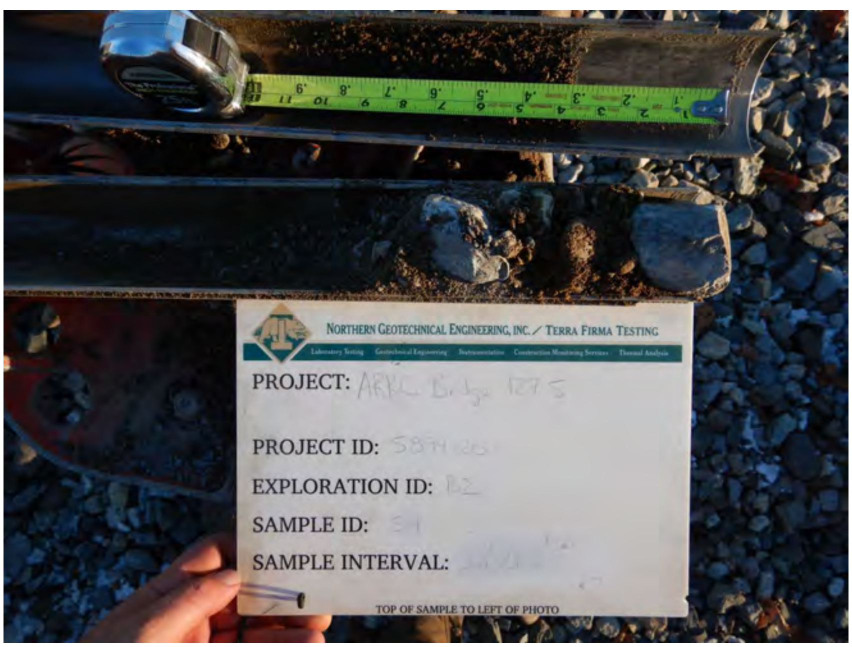
PROJECT NAME ARRC Bridge 127.5

PROJECT NUMBER 5894-20

PROJECT LOCATION Eagle River, AK



Exploration B2 Sample S3
Sample Interval 15 - 16.5 ft bgs



Exploration B2 Sample S4
Sample Interval 20 - 21.5 ft bgs

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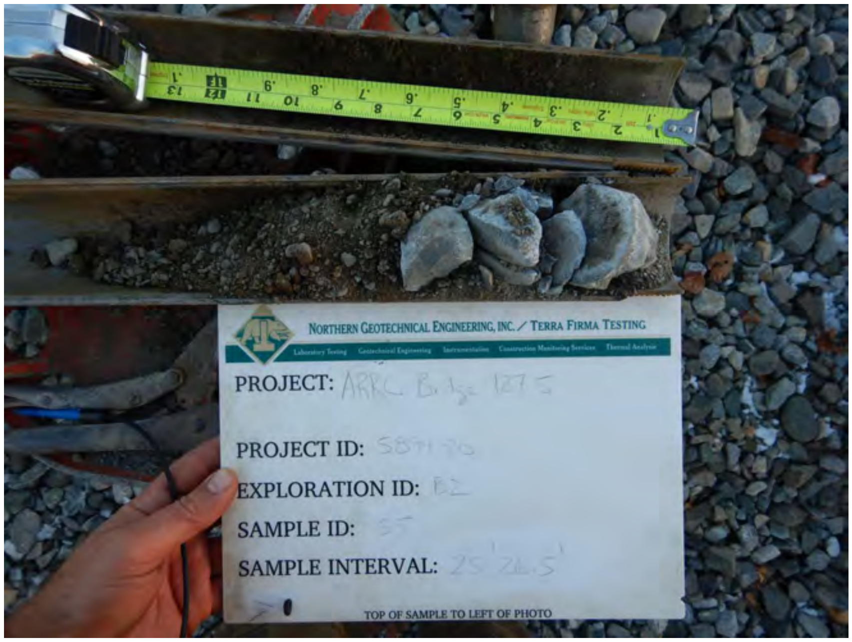
PHOTO LOG
EXPLORATION B2

CLIENT Alaska Railroad Corporation

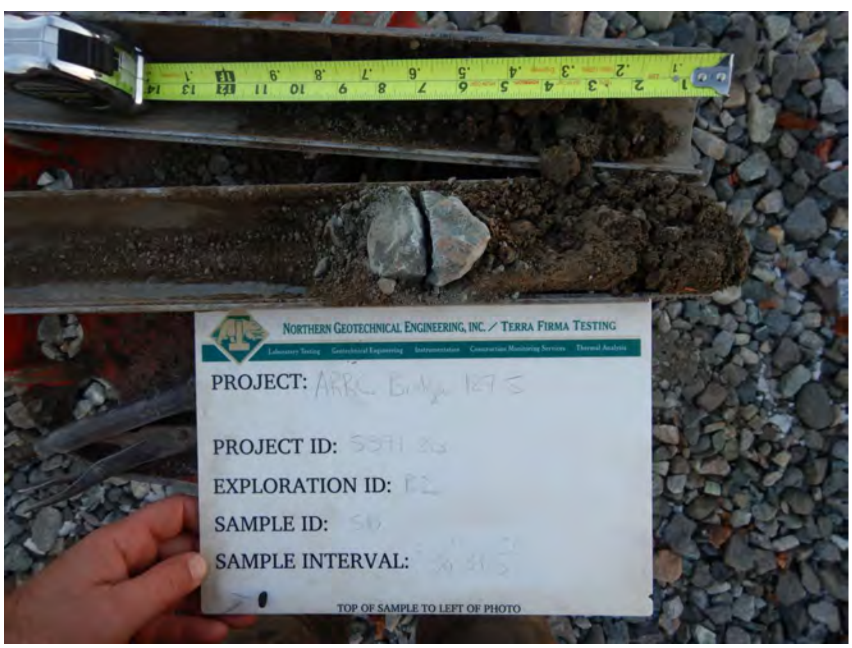
PROJECT NAME ARRC Bridge 127.5

PROJECT NUMBER 5894-20

PROJECT LOCATION Eagle River, AK



Exploration B2 Sample S5
Sample Interval 25 - 26.5 ft bgs



Exploration B2 Sample S6
Sample Interval 30 - 31.5 ft bgs

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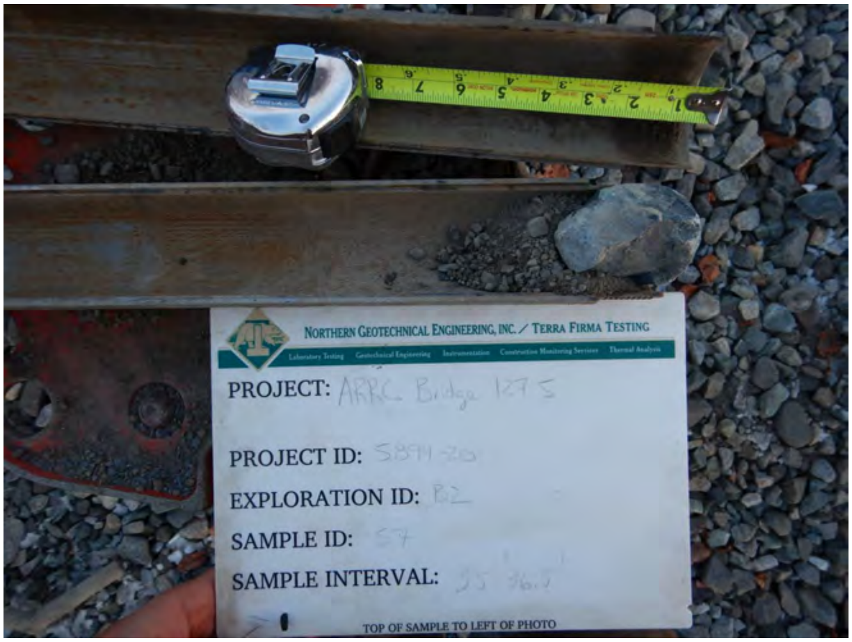
PHOTO LOG
EXPLORATION B2

CLIENT Alaska Railroad Corporation

PROJECT NAME ARRC Bridge 127.5

PROJECT NUMBER 5894-20

PROJECT LOCATION Eagle River, AK



Exploration B2 Sample S7
Sample Interval 35 - 36.5 ft bgs

Exploration B2 Sample S8
Sample Interval 40 - 41.5 ft bgs

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PHOTO LOG
EXPLORATION B2

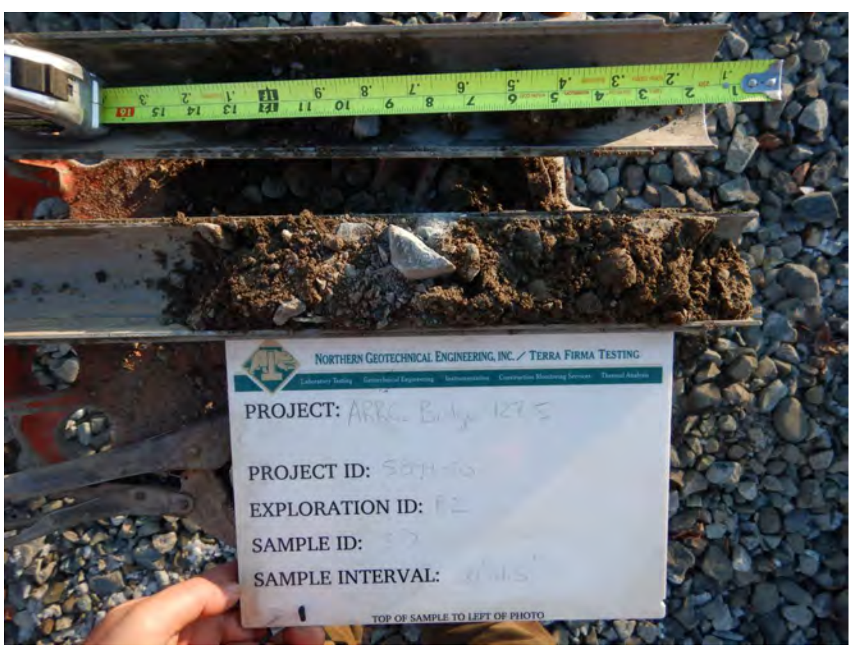
CLIENT Alaska Railroad Corporation

PROJECT NAME ARRC Bridge 127.5

PROJECT NUMBER 5894-20

PROJECT LOCATION Eagle River, AK

Exploration B2 Sample S9
Sample Interval 45 - 46.5 ft bgs



Exploration B2 Sample S10
Sample Interval 50 - 51.5 ft bgs

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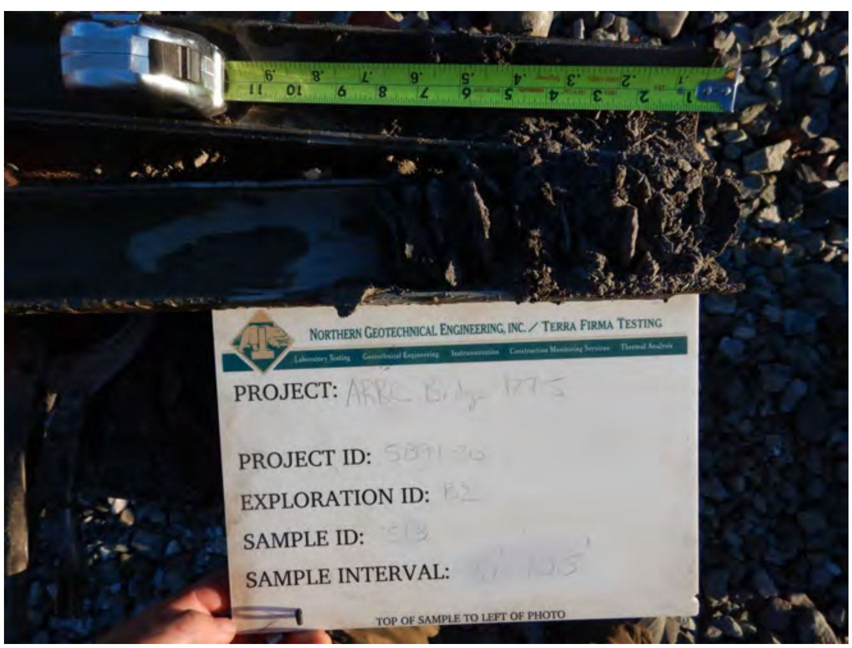
PHOTO LOG
EXPLORATION B2

CLIENT Alaska Railroad Corporation **PROJECT NAME** ARRC Bridge 127.5

PROJECT NUMBER 5894-20 **PROJECT LOCATION** Eagle River, AK

Exploration B2 Sample S12
Sample Interval 60 - 60.75 ft bgs



Exploration B2 Sample S13
Sample Interval 70 - 70.5 ft bgs

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EXPLORATION B3

NGE-TFT PROJECT NAME: ARRC Bridge 127.5
NGE-TFT PROJECT NUMBER: 5894-20
PROJECT LOCATION: Eagle River, AK
EXPLORATION CONTRACTOR: Discovery Drilling, Inc.
EXPLORATION EQUIPMENT: Geoprobe 7822DT
EXPLORATION METHOD: HWT Casing Advacement System w/ Tricone Bit & HQ Diamond Core Bit
SAMPLING METHOD: MPT w/ 340lb autohammer and 5' HQ corebarrel
LOGGED BY: A. Smith
DATE/TIME STARTED: 11/5/2020 @ 8:40:00 AM
DATE/TIME COMPLETED: 11/6/2020 @ 3:30:00 PM
EXPLORATION LOCATION: Apx. 20' WNW of N. Bridge Pier
GROUND ELEVATION: Approx. 63 ft below top of rail
▽ GROUNDWATER (ATD): None observed
▼ GROUNDWATER (): N/A
EXPLORATION COMPLETION: Backfilled with cement slurry
WEATHER CONDITIONS: Overcast, light snow, 10-20°F

DEPTH (ft) bgs	MATERIAL DESCRIPTION	FIELD SAMPLE ID	RECOVERY (in)	FIELD BLOWS	$(N_1)_{60}$	RUN (BOX #)	RUN START (STOP)	%REC / (RQD)	LAB SPECIMEN ID	LAB RESULTS	REMARKS/NOTES
0	**SURFICIAL ORGANICS**										
	POORLY GRADED GRAVEL WITH SAND (GP), some cobbles up to approx. 10 inches in diameter										
	SILTY SAND (SM), dense, olive gray to dark gray, damp, thin silt layers >1cm in thickness interbedded throughout										
5		S1	16	12 15 18	27				S1	MC = 10.4% 0.0% gravel, 72.7% sand, 27.3% silt	
10		S2	14	18 44 50 3"	N/A				S2	MC = 8.9% P200 = 34.6%	45 min to drill from 10-12.5'. Very hard.
		S3	8	49	N/A				S3	MC = 7.4% P200 = 30.6%	Bedrock fragments in sampler.
	BEDROCK **SANDSTONE**, medium gray to light gray, thinly bedded, hard to hard, unfractured to very slightly fractured, fractures consist exclusively of separations along bedding planes at an apparant dip of 15-20°										
15		R1	11			1 (1)	14:20 (14:30)	44 (31)			Switch to HQ coring at 15 ft bgs.
		R2	58			2 (1)	14:50 (15:05)	97 (76)			
20									S4T	TS = 175 psi	
									S4C.1	CS = 3663 psi	
									S4C.2	CS = 3591 psi	
		R3	60			3 (1-2)	15:55 (16:10)	100 (70)			
25											
		R4	60			4 (2)	9:15 (9:25)	100 (92)			
30											

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.

(Continued Next Page)

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EXPLORATION B3

NGE-TFT PROJECT NAME: ARRC Bridge 127.5
NGE-TFT PROJECT NUMBER: 5894-20
PROJECT LOCATION: Eagle River, AK
EXPLORATION CONTRACTOR: Discovery Drilling, Inc.
EXPLORATION EQUIPMENT: Geoprobe 7822DT
EXPLORATION METHOD: HWT Casing Advacement System w/ Tricone Bit & HQ Diamond Core Bit
SAMPLING METHOD: MPT w/ 340lb autohammer and 5' HQ corebarrel
LOGGED BY: A. Smith
DATE/TIME STARTED: 11/5/2020 @ 8:40:00 AM
DATE/TIME COMPLETED: 11/6/2020 @ 3:30:00 PM
EXPLORATION LOCATION: Apx. 20' WNW of N. Bridge Pier
GROUND ELEVATION: Approx. 63 ft below top of rail
GROUNDWATER (ATD): None observed
GROUNDWATER (): N/A
EXPLORATION COMPLETION: Backfilled with cement slurry
WEATHER CONDITIONS: Overcast, light snow, 10-20°F

DEPTH (ft) bgs	GRAPHIC LOG	MATERIAL DESCRIPTION	SAMPLE TYPE/LENGTH	FIELD SAMPLE ID	RECOVERY (in)	FIELD BLOWS	$(N_1)_{60}$	RUN (BOX #)	RUN START (STOP)	%REC / (RQD)	LAB SPECIMEN INT.	LAB SPECIMEN ID	LAB RESULTS	REMARKS/NOTES
30		*BEDROCK* **SANDSTONE**, medium gray to light gray, thinly bedded, hard to hard, unfractured to very slightly fractured, fractures consist exclusively of separations along bedding planes at an apparant dip of approx. 15-20° *(continued)*												
		CARBONACEOUS SILTSTONE, dark gray to black, thinly bedded, moderately soft to soft, some coal layers, lignitic to bituminous		R5	48			5 (2-3)	9:35 (9:47)	80 (0)				
35														
		SILTY SANDSTONE, light gray with dark gray, very thinly bedded, hard to soft, unfractured to slightly fractured, fractures consist of separations along bedding planes at an apparant dip of approx. 10-20°												
				R6	60			6 (3)	10:00 (10:13)	100 (65)				
40														
				R7	60			7 (3-4)	10:25 (10:38)	100 (75)				
45														
				R8	59			8 (4)	11:03 (11:15)	90 (80)				
												S5C.1	CS = 6178 psi	
50												S5C.2	CS = 6730 psi	
												S5T	TS = 312 psi	
				R9	56			9 (4-5)	11:22 (11:35)	93 (48)				
55														
		SILTSTONE, light gray with medium gray, completely weathered, very thinly bedded, soft, moderately to intensely fractured, fractures consist of separatoins along bedding planes at an apparant dip of 15-20°												
				R10	60			10 (5)	11:45 (11:58)	100 (0)				
60														

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(Continued Next Page)

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EXPLORATION B3

NGE-TFT PROJECT NAME: ARRC Bridge 127.5
NGE-TFT PROJECT NUMBER: 5894-20
PROJECT LOCATION: Eagle River, AK
EXPLORATION CONTRACTOR: Discovery Drilling, Inc.
EXPLORATION EQUIPMENT: Geoprobe 7822DT
EXPLORATION METHOD: HWT Casing Advacement System w/ Tricone Bit & HQ Diamond Core Bit
SAMPLING METHOD: MPT w/ 340lb autohammer and 5' HQ corebarrel
LOGGED BY: A. Smith
DATE/TIME STARTED: 11/5/2020 @ 8:40:00 AM
DATE/TIME COMPLETED: 11/6/2020 @ 3:30:00 PM
EXPLORATION LOCATION: Apx. 20' WNW of N. Bridge Pier
GROUND ELEVATION: Approx. 63 ft below top of rail
▽ **GROUNDWATER (ATD):** None observed
▼ **GROUNDWATER ():** N/A
EXPLORATION COMPLETION: Backfilled with cement slurry
WEATHER CONDITIONS: Overcast, light snow, 10-20°F

DEPTH (ft) bgs	MATERIAL DESCRIPTION	FIELD SAMPLE ID	RECOVERY (in)	FIELD BLOWS	$(N_1)_{60}$	RUN (BOX #)	RUN START (STOP)	%REC / (RQD)	LAB SPECIMEN ID	LAB RESULTS	REMARKS/NOTES
60	**SILTSTONE**, light gray with medium gray, completely weathered, very thinly bedded, soft, moderately to intensely fractured, fractures consist of separatoins along bedding planes at an apparant dip of 15-20° *(continued)*										
		R11	62			11 (5-6)	12:23 (12:34)	103 (23)			
65									S6C.1	CS = 12,147 psi	
									S6T	TS = 655 psi	
		R12	60			12 (6-7)	12:39 (12:52)	100 (28)			
70									S6C.2	CS = 8252 psi	
		R13	55			13 (7)	13:00 (13:15)	92 (0)			
75	**CARBONACEOUS SILTSTONE**, dark gray to black, thinly bedded, some lignitic to bituminous coal layers										Color of fluid return changed from gray to brown at approx. 75 ft bgs.
		R14	53			14 (7-8)	13:20 (13:31)	88 (0)			
80											
		R15	51			15 (8)	13:35 (13:49)	85 (0)			
85											

Bottom of borehole at 88.0 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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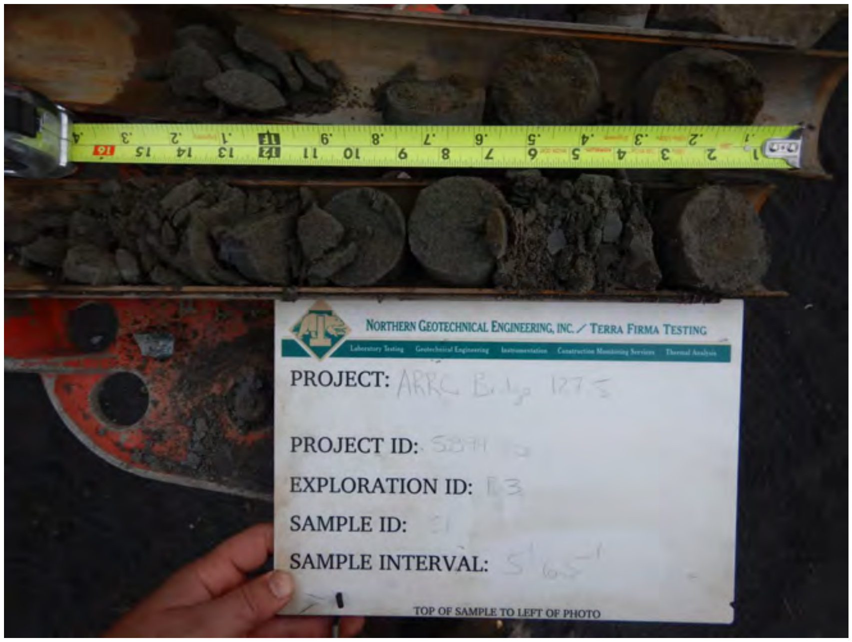
PHOTO LOG
EXPLORATION B3

CLIENT Alaska Railroad Corporation

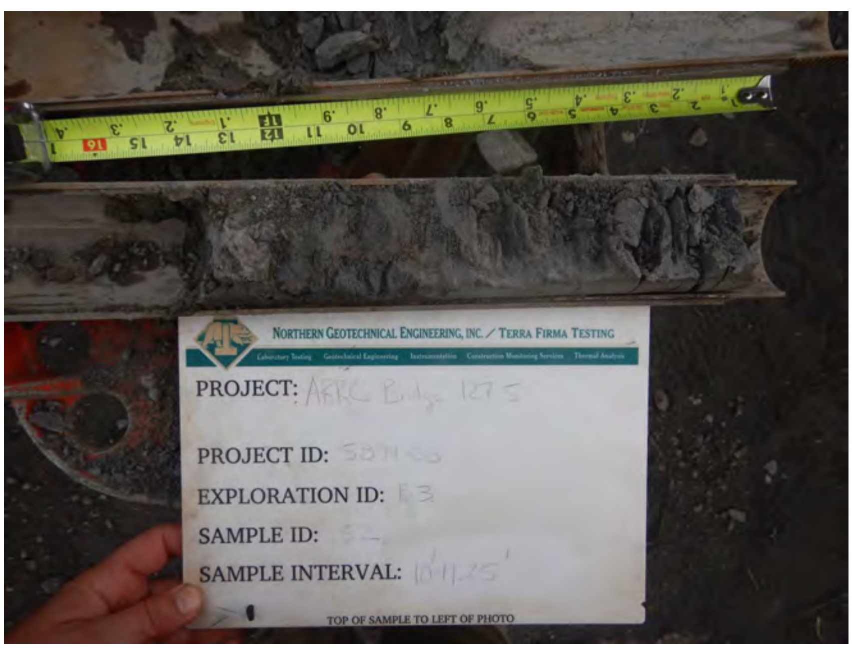
PROJECT NAME ARRC Bridge 127.5

PROJECT NUMBER 5894-20

PROJECT LOCATION Eagle River, AK



Exploration B3 Sample S1
Sample Interval 5 - 6.5 ft bgs



Exploration B3 Sample S2
Sample Interval 10 - 11.25 ft bgs

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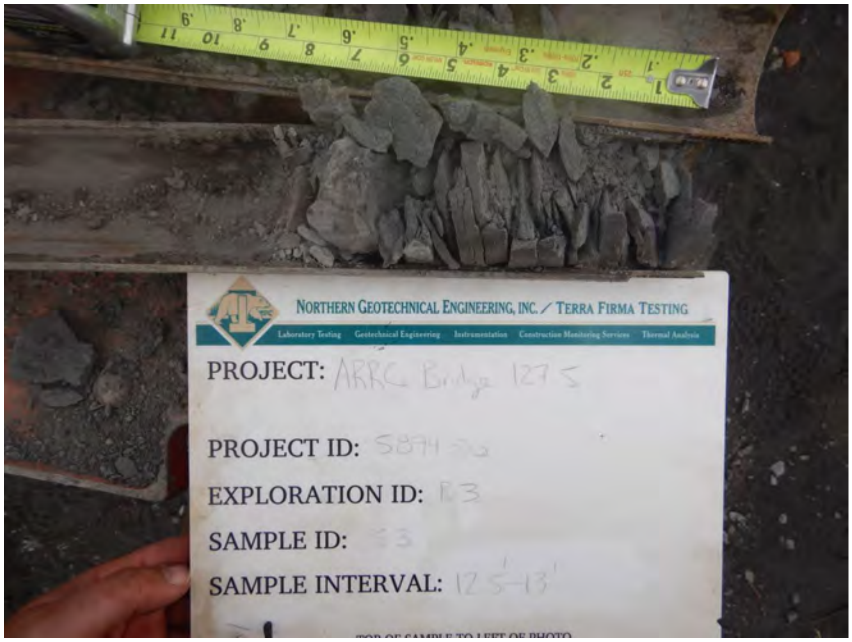
PHOTO LOG
EXPLORATION B3

CLIENT Alaska Railroad Corporation

PROJECT NAME ARRC Bridge 127.5

PROJECT NUMBER 5894-20

PROJECT LOCATION Eagle River, AK



Exploration B3 Sample S3
Sample Interval 12.5 - 13 ft bgs

Core Box 1

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PHOTO LOG
EXPLORATION B3

CLIENT Alaska Railroad Corporation

PROJECT NAME ARRC Bridge 127.5

PROJECT NUMBER 5894-20

PROJECT LOCATION Eagle River, AK

Corebox 2

Corebox 3

Corebox 4

Corebox 5

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Telephone: 907-344-5934

PHOTO LOG
EXPLORATION B3

CLIENT Alaska Railroad Corporation

PROJECT NAME ARRC Bridge 127.5

PROJECT NUMBER 5894-20

PROJECT LOCATION Eagle River, AK

Corebox 6

Corebox 7

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PHOTO LOG
EXPLORATION B3

CLIENT Alaska Railroad Corporation **PROJECT NAME** ARRC Bridge 127.5

PROJECT NUMBER 5894-20 **PROJECT LOCATION** Eagle River, AK

Corebox 8

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EXPLORATION LEGEND

CLIENT Alaska Railroad Corporation
NGE-TFT PROJECT NAME ARRC Bridge 127.5
NGE-TFT PROJECT NUMBER 5894-20
PROJECT LOCATION Eagle River, AK

LITHOLOGIC SYMBOLS
(Unified Soil Classification System)

- SANDSTONE
- CARBONACEOUS SILTSTONE
- SILTSTONE
- GM: USCS Silty Gravel
- GP: USCS Poorly-graded Gravel
- GPF: Poorly graded gravel - Fill (made ground)
- OH: USCS High Plasticity Organic silt or clay
- SILTY SANDSTONE:
- SM: USCS Silty Sand

SAMPLER SYMBOLS

- Modified Penetration Test
- No Recovery
- Rock Core

WELL CONSTRUCTION SYMBOLS

- Bentonite Seal
- Pipe backfilled with pea gravel
- Slough Backfill
- Slotted Pipe Backfilled with Slough

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
TS - TENSILE STRENGTH

CS - COMPRESSIVE STRENGTH
TV - TORVANE
PID - PHOTOIONIZATION DETECTOR
UC - UNCONFINED COMPRESSION
ppm - PARTS PER MILLION
N/E - NOT ENCOUNTERED

∇ Water Level at Time Drilling, or as Shown
▼ Water Level After 24 Hours, or as Shown

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and Terra Firma Testing
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Telephone: 907-344-5934

SOIL CLASSIFICATION CHART

CLIENT Alaska Railroad Corporation

PROJECT NAME ARRC Bridge 127.5

NGE-TFT PROJECT NUMBER 5894-20

PROJECT LOCATION Eagle River, AK

MAJOR DIVISIONS			SYMBOLS GRAPH	SYMBOLS LETTER	TYPICAL DESCRIPTIONS
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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and Terra Firma Testing
11301 Olive Lane
Anchorage, AK 99515
Telephone: 907-344-5934

EXPLORATION LOG KEY

CLIENT Alaska Railroad Corporation
PROJECT NAME ARRC Bridge 127.5
NGE-TFT PROJECT NUMBER 5894-20
PROJECT LOCATION Eagle River, 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

WELL SYMBOLS

1" Slotted Pipe
Backfilled with Silica Sand

1" PVC Pipe
Backfilled with Auger Cuttings

1" PVC Pipe
with Bentonite Seal

Capped Riser

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%

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 Alaska Railroad Corporation **PROJECT NAME** ARRC Bridge 127.5

NGE-TFT PROJECT NUMBER 5894-20 **PROJECT LOCATION** Eagle River, 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 RESULTS

Summary of Laboratory Test Results

ARRC Bridge 127.5

Eagle River, Alaska

NGE-TFT Project #:5894-20

Exploration ID Number	Sample Number	Depth Interval (ft) Top	Depth Interval (ft) Bottom	SOIL SPECIMENS: Moisture Content ASTM D2216 (% By Dry Mass)	Particle Size Analysis ASTM C136/D7928/D6913 (% By Mass) Gravel	Sand	Silt/Clay	Passing #200 ASTM D1140 (% By Mass)	Unified Soil Classification ASTM D2487	ROCK CORE SPECIMENS: Splitting Tensile Strength (avg. p.s.i.) ASTM D3967	Unconfined Compressive Strength (avg. p.s.i.) ASTM D7012	Bulk Specific Gravity ASTM C127
B1	S1	5.00	6.50	3.0	--	--	--	--	--	--	--	--
B1	S3	15.00	16.50	4.4	45.4	40.6	14.0	--	(GM) Silty gravel w/ sand	--	--	--
B1	S4	20.00	21.50	2.9	--	--	--	--	--	--	--	--
B1	S5	25.00	26.50	7.1	--	--	--	--	--	--	--	--
B1	S6	30.00	31.50	5.9	--	--	--	--	--	--	--	--
B1	S7	35.00	36.50	8.1	43.8	37.7	18.5	--	(GM) Silty gravel w/ sand	--	--	--
B1	S8	40.00	41.50	4.6	--	--	--	--	--	--	--	--
B1	S9	45.00	46.50	4.9	--	--	--	--	--	--	--	--
B1	S10	50.00	51.50	8.9	--	--	--	--	--	--	--	--
B1	S11	55.00	26.50	5.7	52.7	30.8	16.5	--	(GM) Silty gravel w/ sand	--	--	--
B1	S12a	60.50	61.50	18.9	--	--	--	85.0	--	--	--	--
B1	S13	65.00	65.75	8.7	--	--	--	40.9	--	--	--	--
B1	S14	72.50	73.00	8.5	--	--	--	30.9	--	--	--	--
B2	S1	5.00	6.50	4.0	--	--	--	--	--	--	--	--
B2	S2	10.00	11.50	7.8	44.0	42.9	13.1	--	(GM) Silty gravel w/ sand	--	--	--
B2	S4	20.00	21.50	4.0	--	--	--	--	--	--	--	--
B2	S6	30.00	31.25	5.4	--	--	--	--	--	--	--	--
B2	S8	40.00	41.50	4.9	28.4	53.1	18.5	--	(SM) Silty sand w/ gravel	--	--	--
B2	S10	50.00	51.50	4.8	--	--	--	--	--	--	--	--
B2	S12	60.00	60.75	4.2	17.6	60.8	21.6	--	(SM) Silty sand w/ gravel	--	--	--
B2	S13	70.00	70.50	7.0	--	--	--	39.9	--	--	--	--
B3	S1	5.00	6.50	10.4	0.0	72.7	27.3	--	(SM) Silty sand	--	--	--
B3	S2	10.00	11.25	8.9	--	--	--	34.6	--	--	--	--
B3	S3	12.50	13.00	7.4	--	--	--	30.6	--	--	--	--
B3	S4	20.00	22.50	--	--	--	--	--	--	175	3,627	2.21
B3	S5	49.25	51.50	--	--	--	--	--	--	312	6,454	2.34
B3	S6	64.75	72.25	--	--	--	--	--	--	655	10,200	2.44

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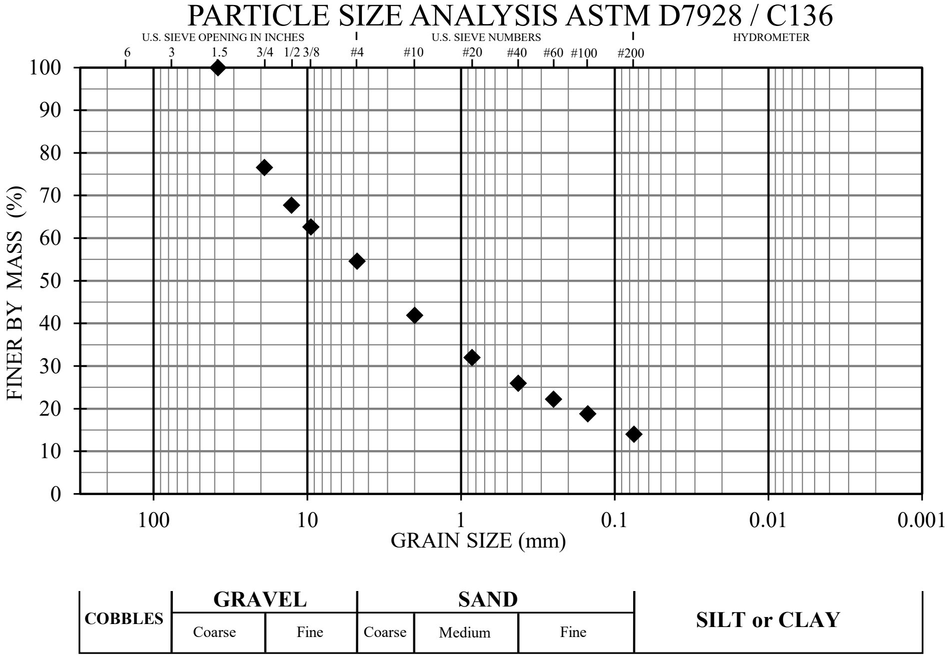
PROJECT CLIENT:	Alaska Rail Road Corp.
PROJECT NAME:	ARRC Bridge 127.5
PROJECT NO.:	5894-20
SAMPLE LOC.:	B1
NUMBER/ DEPTH:	S3 / 15 - 16.5'
DESCRIPTION:	Silty gravel w/ sand
DATE RECEIVED:	11/13/2020
TESTED BY:	EA
REVIEWED BY:	ACS

% GRAVEL	45.4	USCS	GM
% SAND	40.6	USACOE FC	N/A
% SILT/CLAY	14.0	% PASS. 0.02 mm	N/A
% MOIST. CONTENT	4.4	% 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

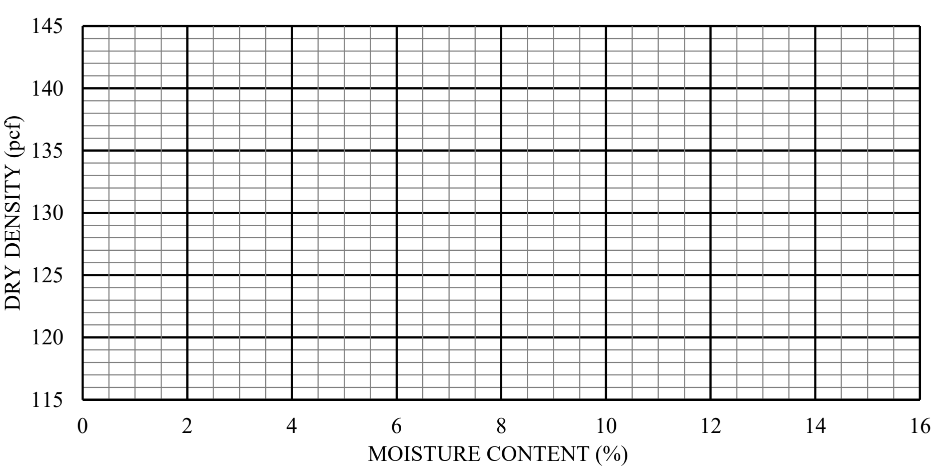
PARTICLE SIZE ANALYSIS ASTM D7928 / C136



SIEVE ANALYSIS RESULT

SIEVE SIZE (mm)	SIEVE SIZE (U.S.)	TOTAL % PASSING	SPECIFICATION (% PASSING)
152.40	6"		
76.20	3"		
38.10	1.5"	**100**	
19.00	3/4"	**77**	
12.70	1/2"	**68**	
9.50	3/8"	**63**	
4.75	#4	**55**	
2.00	#10	**42**	
0.85	#20	**32**	
0.43	#40	**26**	
0.25	#60	**22**	
0.15	#100	**19**	
0.075	#200	**14.0**	

MOISTURE-DENSITY RELATIONSHIP ASTM D1557



HYDROMETER RESULT

ELAPSED TIME (MIN)	DIAMETER (mm)	TOTAL % PASSING
0		
1		
2		
5		
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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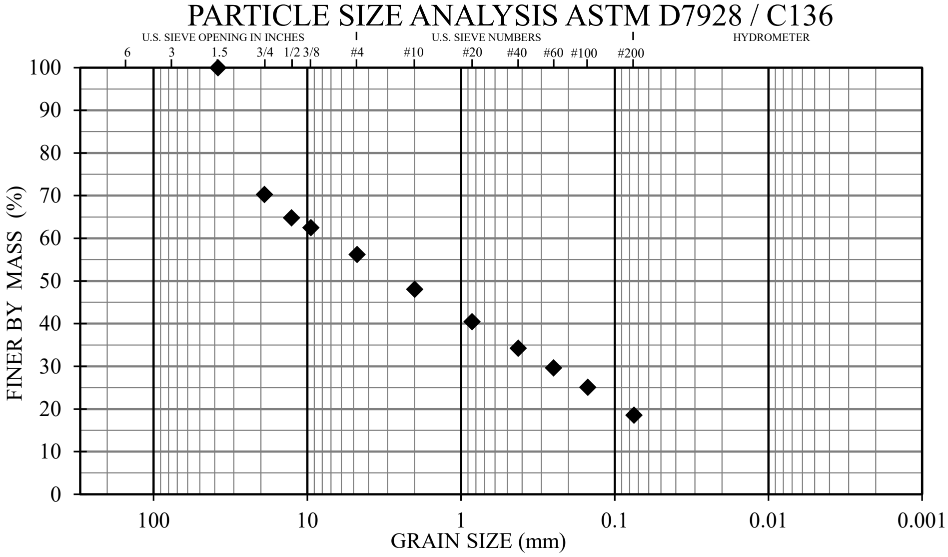
PROJECT CLIENT:	**Alaska Rail Road Corp.**
PROJECT NAME:	**ARRC Bridge 127.5**
PROJECT NO.:	**5894-20**
SAMPLE LOC.:	**B1**
NUMBER/ DEPTH:	**S7 / 35 - 36.5'**
DESCRIPTION:	**Silty gravel w/ sand**
DATE RECEIVED:	**11/13/2020**
TESTED BY:	**EA**
REVIEWED BY:	**ACS**

% GRAVEL	**43.8**	USCS	**GM**
% SAND	**37.7**	USACOE FC	**N/A**
% SILT/CLAY	**18.5**	% PASS. 0.02 mm	**N/A**
% MOIST. CONTENT	**6.5**	% 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 D7928 / C136

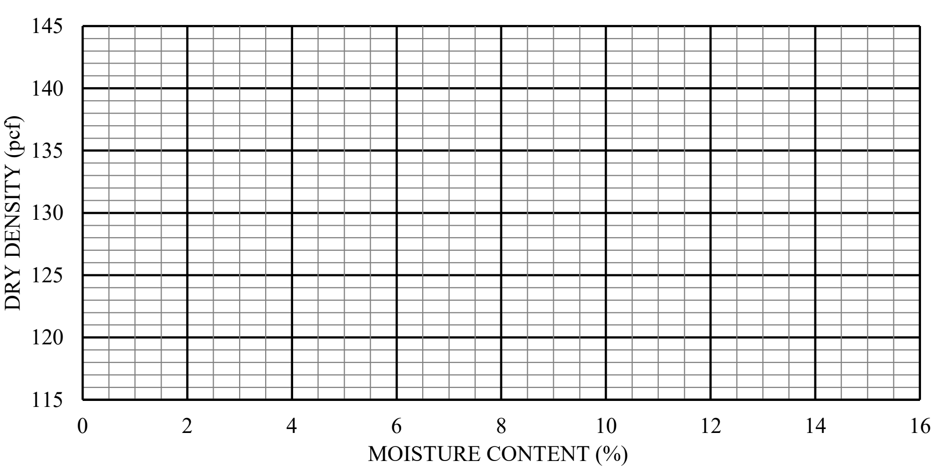


COBBLES	GRAVEL		SAND			SILT or CLAY
	Coarse	Fine	Coarse	Medium	Fine	

SIEVE ANALYSIS RESULT

SIEVE SIZE (mm)	SIEVE SIZE (U.S.)	TOTAL % PASSING	SPECIFICATION (% PASSING)
152.40	6"		
76.20	3"		
38.10	1.5"	**100**	
19.00	3/4"	**70**	
12.70	1/2"	**65**	
9.50	3/8"	**63**	
4.75	#4	**56**	
2.00	#10	**48**	
0.85	#20	**40**	
0.43	#40	**34**	
0.25	#60	**30**	
0.15	#100	**25**	
0.075	#200	**18.5**	

MOISTURE-DENSITY RELATIONSHIP ASTM D1557



HYDROMETER RESULT

ELAPSED TIME (MIN)	DIAMETER (mm)	TOTAL % PASSING
0		
1		
2		
5		
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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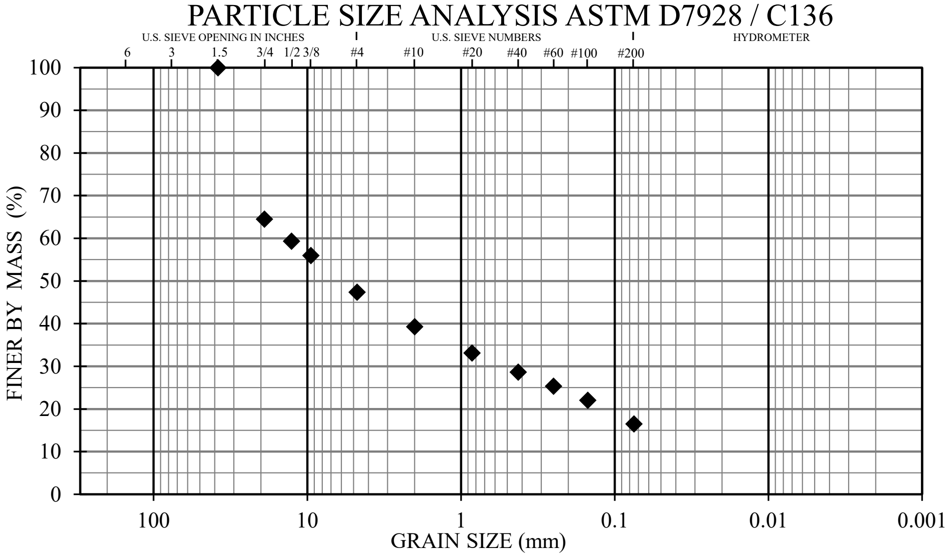
PROJECT CLIENT:	**Alaska Rail Road Corp.**
PROJECT NAME:	**ARRC Bridge 127.5**
PROJECT NO.:	**5894-20**
SAMPLE LOC.:	**B1**
NUMBER/ DEPTH:	**S11 / 55 - 26.5'**
DESCRIPTION:	**Silty gravel w/ sand**
DATE RECEIVED:	**11/13/2020**
TESTED BY:	**EA**
REVIEWED BY:	**ACS**

% GRAVEL	**52.7**	USCS	**GM**
% SAND	**30.8**	USACOE FC	**N/A**
% SILT/CLAY	**16.5**	% PASS. 0.02 mm	**N/A**
% MOIST. CONTENT	**5.7**	% 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 D7928 / C136



COBBLES	GRAVEL		SAND			SILT or CLAY
	Coarse	Fine	Coarse	Medium	Fine	

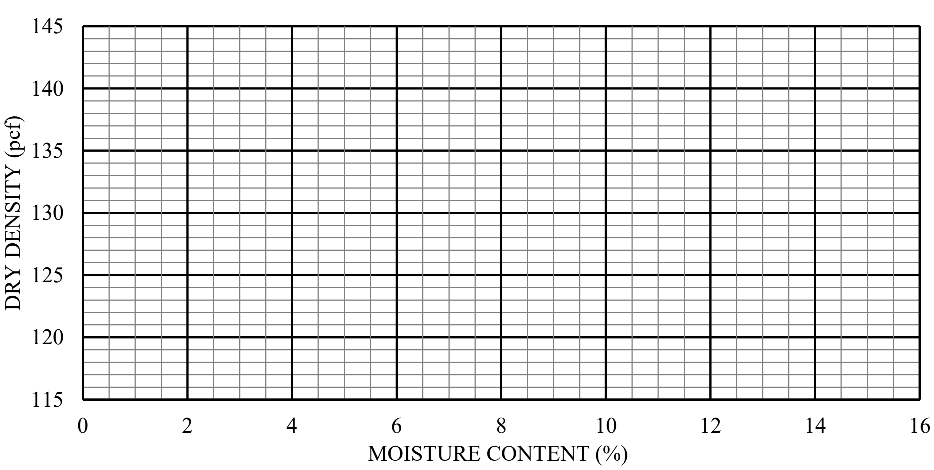
SIEVE ANALYSIS RESULT

SIEVE SIZE (mm)	SIEVE SIZE (U.S.)	TOTAL % PASSING	SPECIFICATION (% PASSING)
152.40	6"		
76.20	3"		
38.10	1.5"	**100**	
19.00	3/4"	**64**	
12.70	1/2"	**59**	
9.50	3/8"	**56**	
4.75	#4	**47**	
2.00	#10	**39**	
0.85	#20	**33**	
0.43	#40	**29**	
0.25	#60	**25**	
0.15	#100	**22**	
0.075	#200	**16.5**	

HYDROMETER RESULT

ELAPSED TIME (MIN)	DIAMETER (mm)	TOTAL % PASSING
0		
1		
2		
5		
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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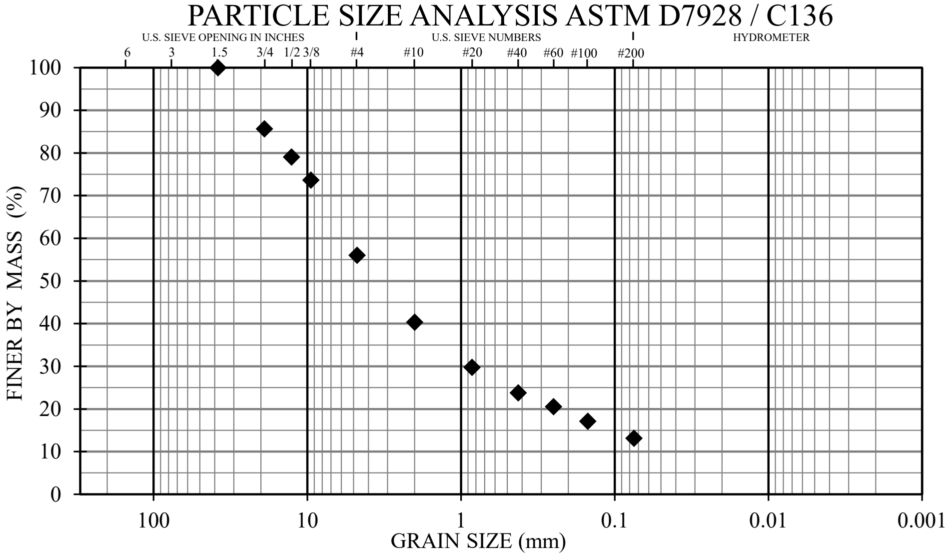
PROJECT CLIENT:	**Alaska Rail Road Corp.**
PROJECT NAME:	**ARRC Bridge 127.5**
PROJECT NO.:	**5894-20**
SAMPLE LOC.:	**B2**
NUMBER/ DEPTH:	**S2 / 10 - 11.5'**
DESCRIPTION:	**Silty gravel w/ sand**
DATE RECEIVED:	**11/13/2020**
TESTED BY:	**EA**
REVIEWED BY:	**ACS**

% GRAVEL	**44.0**	USCS	**GM**
% SAND	**42.9**	USACOE FC	**N/A**
% SILT/CLAY	**13.1**	% PASS. 0.02 mm	**N/A**
% MOIST. CONTENT	**7.8**	% 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 D7928 / C136



COBBLES	GRAVEL		SAND			SILT or CLAY
	Coarse	Fine	Coarse	Medium	Fine	

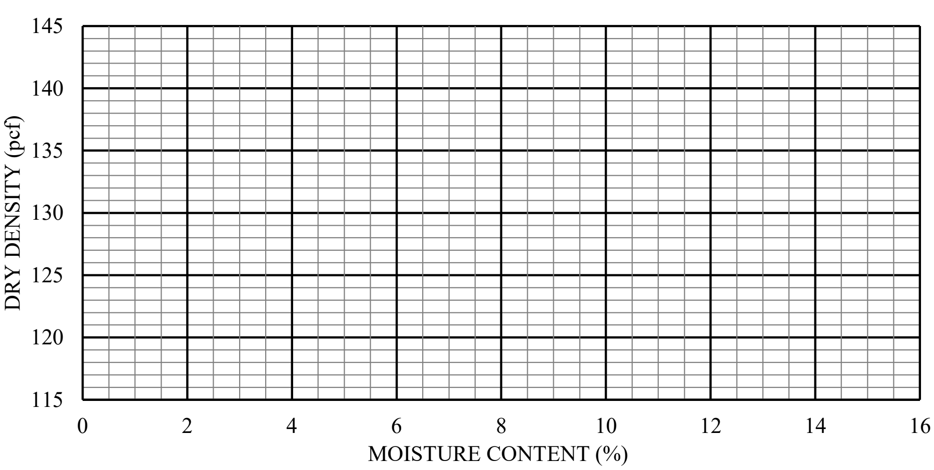
SIEVE ANALYSIS RESULT

SIEVE SIZE (mm)	SIEVE SIZE (U.S.)	TOTAL % PASSING	SPECIFICATION (% PASSING)
152.40	6"		
76.20	3"		
38.10	1.5"	**100**	
19.00	3/4"	**86**	
12.70	1/2"	**79**	
9.50	3/8"	**74**	
4.75	#4	**56**	
2.00	#10	**40**	
0.85	#20	**30**	
0.43	#40	**24**	
0.25	#60	**21**	
0.15	#100	**17**	
0.075	#200	**13.1**	

HYDROMETER RESULT

ELAPSED TIME (MIN)	DIAMETER (mm)	TOTAL % PASSING
0		
1		
2		
5		
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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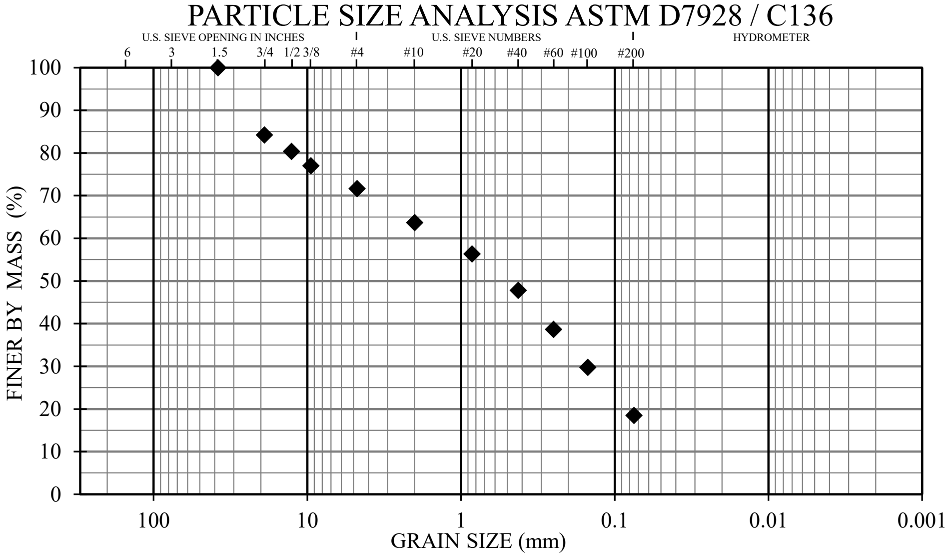
PROJECT CLIENT:	**Alaska Rail Road Corp.**
PROJECT NAME:	**ARRC Bridge 127.5**
PROJECT NO.:	**5894-20**
SAMPLE LOC.:	**B2**
NUMBER/ DEPTH:	**S8 / 40 - 41.5'**
DESCRIPTION:	**Silty sand w/ gravel**
DATE RECEIVED:	**11/13/2020**
TESTED BY:	**EA**
REVIEWED BY:	**ACS**

% GRAVEL	**28.4**	USCS	**SM**
% SAND	**53.1**	USACOE FC	**N/A**
% SILT/CLAY	**18.5**	% PASS. 0.02 mm	**N/A**
% MOIST. CONTENT	**4.9**	% 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 D7928 / C136



COBBLES	GRAVEL		SAND			SILT or CLAY
	Coarse	Fine	Coarse	Medium	Fine	

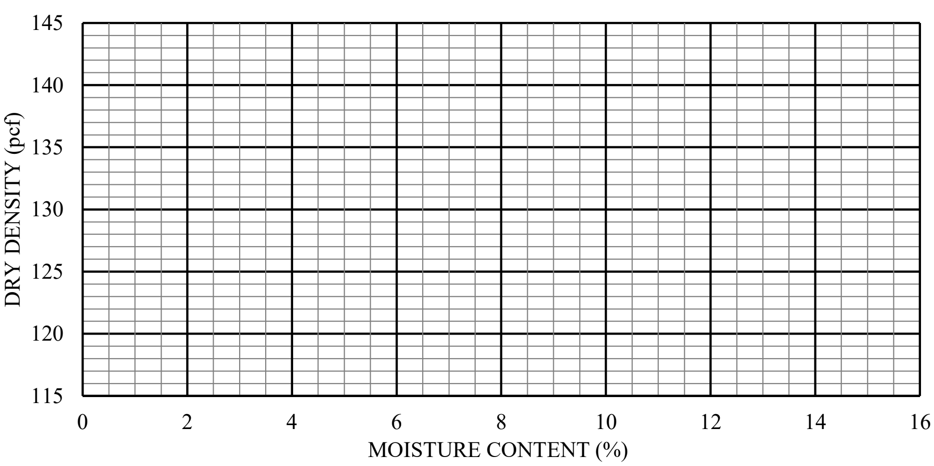
SIEVE ANALYSIS RESULT

SIEVE SIZE (mm)	SIEVE SIZE (U.S.)	TOTAL % PASSING	SPECIFICATION (% PASSING)
152.40	6"		
76.20	3"		
38.10	1.5"	**100**	
19.00	3/4"	**84**	
12.70	1/2"	**80**	
9.50	3/8"	**77**	
4.75	#4	**72**	
2.00	#10	**64**	
0.85	#20	**56**	
0.43	#40	**48**	
0.25	#60	**39**	
0.15	#100	**30**	
0.075	#200	**18.5**	

HYDROMETER RESULT

ELAPSED TIME (MIN)	DIAMETER (mm)	TOTAL % PASSING
0		
1		
2		
5		
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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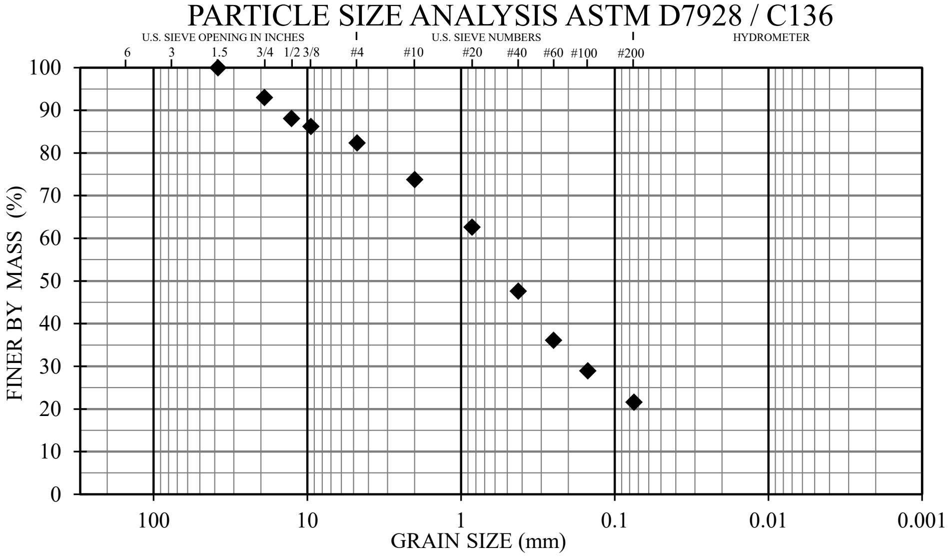
PROJECT CLIENT:	**Alaska Rail Road Corp.**
PROJECT NAME:	**ARRC Bridge 127.5**
PROJECT NO.:	**5894-20**
SAMPLE LOC.:	**B2**
NUMBER/ DEPTH:	**S12 / 60 - 60.75'**
DESCRIPTION:	**Silty sand w/ gravel**
DATE RECEIVED:	**11/13/2020**
TESTED BY:	**EA**
REVIEWED BY:	**ACS**

% GRAVEL	**17.6**	USCS	**SM**
% SAND	**60.8**	USACOE FC	**N/A**
% SILT/CLAY	**21.6**	% PASS. 0.02 mm	**N/A**
% MOIST. CONTENT	**4.2**	% 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 D7928 / C136



COBBLES	GRAVEL		SAND			SILT or CLAY
	Coarse	Fine	Coarse	Medium	Fine	

SIEVE ANALYSIS RESULT

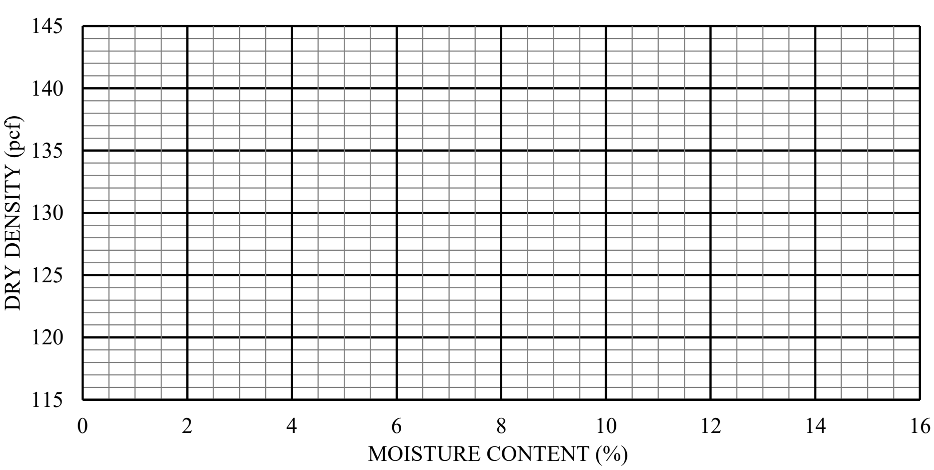
SIEVE SIZE (mm)	SIEVE SIZE (U.S.)	TOTAL % PASSING	SPECIFICATION (% PASSING)
152.40	6"		
76.20	3"		
38.10	1.5"	**100**	
19.00	3/4"	**93**	
12.70	1/2"	**88**	
9.50	3/8"	**86**	
4.75	#4	**82**	
2.00	#10	**74**	
0.85	#20	**63**	
0.43	#40	**48**	
0.25	#60	**36**	
0.15	#100	**29**	
0.075	#200	**21.6**	

HYDROMETER RESULT

ELAPSED TIME (MIN)	DIAMETER (mm)	TOTAL % PASSING
0		
1		
2		
5		
8		
15		
30		
60		
250		
1440		

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

MOISTURE-DENSITY RELATIONSHIP ASTM D1557



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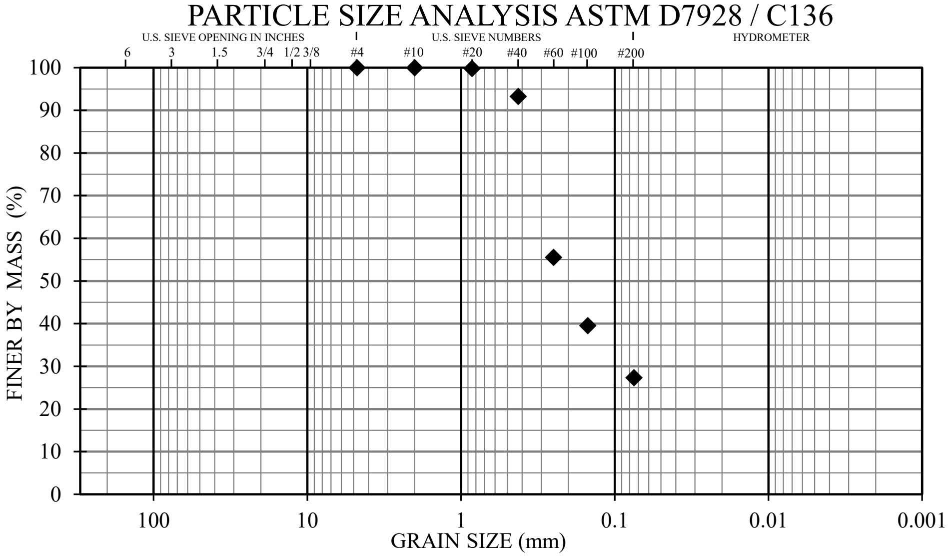
PROJECT CLIENT:	**Alaska Rail Road Corp.**
PROJECT NAME:	**ARRC Bridge 127.5**
PROJECT NO.:	**5894-20**
SAMPLE LOC.:	**B3**
NUMBER/ DEPTH:	**S1 / 5 - 6.5'**
DESCRIPTION:	**Silty sand**
DATE RECEIVED:	**11/13/2020**
TESTED BY:	**EA**
REVIEWED BY:	**ACS**

% GRAVEL	**0.0**	USCS	**SM**
% SAND	**72.7**	USACOE FC	**N/A**
% SILT/CLAY	**27.3**	% PASS. 0.02 mm	**N/A**
% MOIST. CONTENT	**10.4**	% 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 D7928 / C136



COBBLES	GRAVEL		SAND			SILT or CLAY
	Coarse	Fine	Coarse	Medium	Fine	

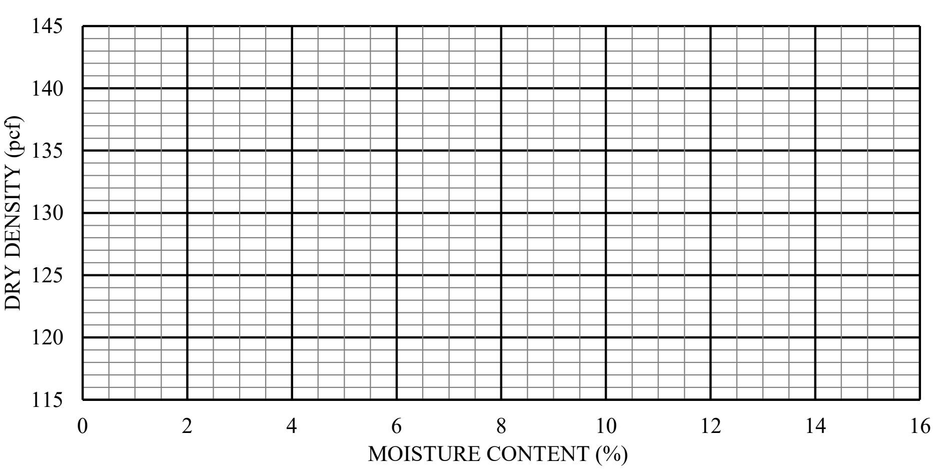
SIEVE ANALYSIS RESULT

SIEVE SIZE (mm)	SIEVE SIZE (U.S.)	TOTAL % PASSING	SPECIFICATION (% PASSING)
152.40	6"		
76.20	3"		
38.10	1.5"		
19.00	3/4"		
12.70	1/2"		
9.50	3/8"		
4.75	#4	**100**	
2.00	#10	**100**	
0.85	#20	**100**	
0.43	#40	**93**	
0.25	#60	**55**	
0.15	#100	**40**	
0.075	#200	**27.3**	

HYDROMETER RESULT

ELAPSED TIME (MIN)	DIAMETER (mm)	TOTAL % PASSING
0		
1		
2		
5		
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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ASTM D3967 STANDARD TEST METHOD FOR SPLITTING TENSILE STRENGTH OF INTACT ROCK CORE SPECIMENS

PROJECT NO:	5894-20	CLIENT:	Alaska Rail Road Corporation
PROJECT NAME:	ARRC Bridge 127.5 (Eagle River)	SAMPLE NO.:	B3 - S4T
TESTED BY:	A. Fortt	DATE RECEIVED:	11/13/2020
PROJ. MANAGER:	A. Smith	DATE TESTED:	1/11/2021

BOREHOLE	SAMPLE #	LENGTH	DIAMETER	FAILURE LOAD	TENSILE STRENGTH	TENSILE STRENGTH	FAILURE MODE
		[in]	[in]	[lb]	[psi]	[MPa]	
B3	S4T-1	1.48	2.37	1114	202	1.39	Failure plane perp. to core axis, parrallel to load axis.
B3	S4T-2	1.46	2.38	1192	220	1.51	Failure plane perp. to core axis, parrallel to load axis.
B3	S4T-3	1.46	2.38	1138	209	1.44	Failure plane perp. to core axis, parrallel to load axis.
B3	S4T-4	1.48	2.35	751	138	0.95	Failure plane perp. to core axis, parrallel to load axis.
B3	S4T-5	1.48	2.32	669	124	0.86	Failure plane perp. to core axis, parrallel to load axis.
B3	S4T-6	1.47	2.36	886	163	1.12	Failure plane perp. to core axis, parrallel to load axis.
B3	S4T-7	1.48	2.36	1006	184	1.27	Failure plane perp. to core axis, parrallel to load axis.
B3	S4T-8	1.47	2.36	801	147	1.02	Failure plane perp. to core axis, parrallel to load axis.
B3	S4T-9	1.48	2.37	1104	202	1.39	Failure plane perp. to core axis, parrallel to load axis.
B3	S4T-10	1.48	2.37	888	162	1.12	Failure plane perp. to core axis, parrallel to load axis.

	FAILURE LOAD	TENSILE STRENGTH [psi]	TENSILE STRENGTH [MPa]
AVERAGE	955	175	1.21
STDEV	181	33	0.23

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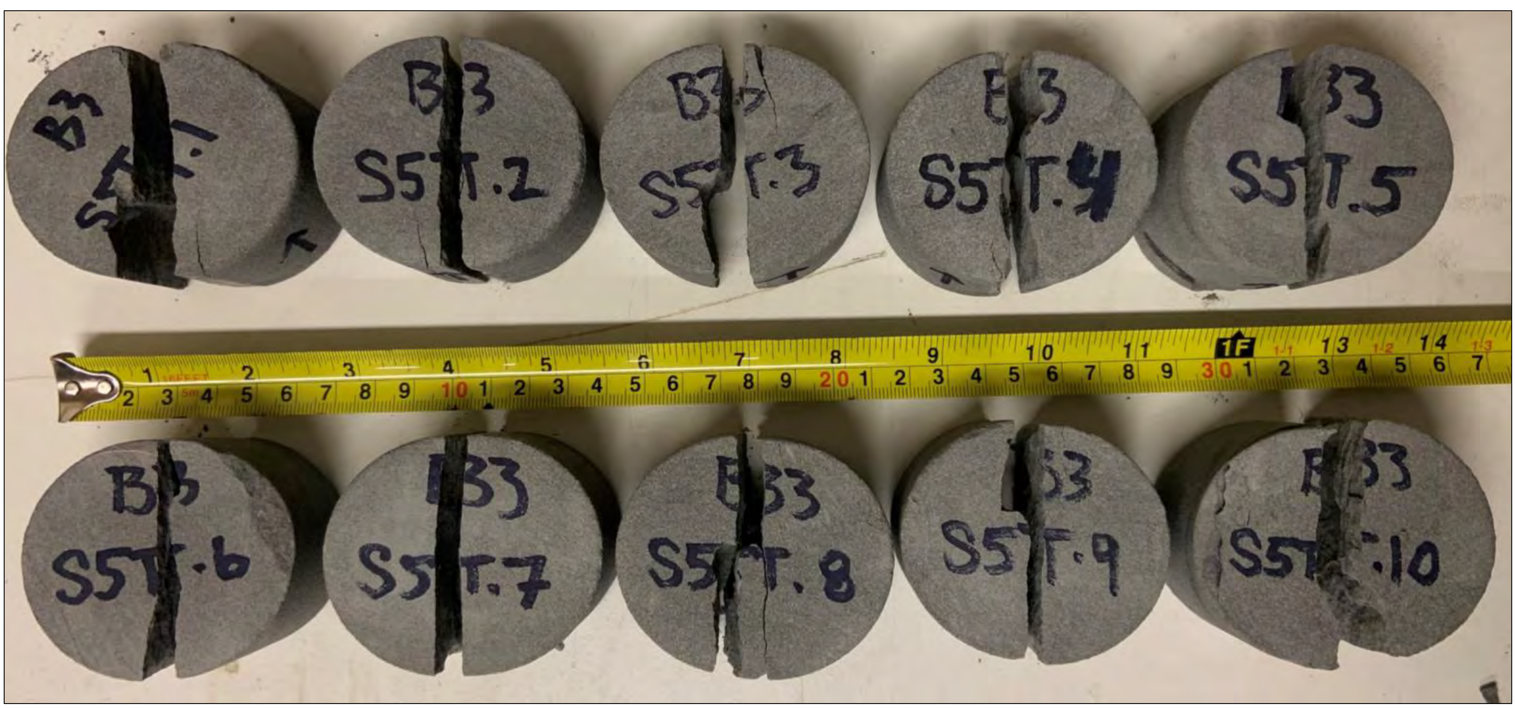
ASTM D3967 STANDARD TEST METHOD FOR SPLITTING TENSILE STRENGTH OF INTACT ROCK CORE SPECIMENS

PROJECT NO:	5894-20	CLIENT:	Alaska Rail Road Corporation
PROJECT NAME:	ARRC Bridge 127.5 (Eagle River)	SAMPLE NO.:	B3 - S5T
TESTED BY:	A. Fortt	DATE RECEIVED:	11/13/2020
PROJ. MANAGER:	A. Smith	DATE TESTED:	1/11/2021

BOREHOLE	SAMPLE #	LENGTH [in]	DIAMETER [in]	FAILURE LOAD [lb]	TENSILE STRENGTH [psi]	TENSILE STRENGTH [MPa]	FAILURE MODE
B3	S5T-1	1.49	2.36	2036	370	2.55	Failure plane perp. to core axis, parrallel to load axis.
B3	S5T-2	1.48	2.36	1752	320	2.21	Failure plane perp. to core axis, parrallel to load axis.
B3	S5T-3	1.49	2.36	2124	385	2.66	Failure plane perp. to core axis, parrallel to load axis.
B3	S5T-4	1.48	2.36	1487	273	1.88	Failure plane perp. to core axis, parrallel to load axis.
B3	S5T-5	1.47	2.36	1382	255	1.76	Failure plane perp. to core axis, parrallel to load axis.
B3	S5T-6	1.48	2.35	1623	298	2.05	Failure plane perp. to core axis, parrallel to load axis.
B3	S5T-7	1.46	2.36	1237	229	1.58	Failure plane perp. to core axis, parrallel to load axis.
B3	S5T-8	1.47	2.36	1902	348	2.40	Failure plane perp. to core axis, parrallel to load axis.
B3	S5T-9	1.48	2.36	1809	332	2.29	Failure plane perp. to core axis, parrallel to load axis.
B3	S5T-10	1.47	2.35	1688	310	2.14	Failure plane perp. to core axis, parrallel to load axis.

	FAILURE LOAD [lb]	TENSILE STRENGTH [psi]	TENSILE STRENGTH [MPa]
AVERAGE	1704	312	2.15
STDEV	282	50	0.34



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ASTM D3967 STANDARD TEST METHOD FOR SPLITTING TENSILE STRENGTH OF INTACT ROCK CORE SPECIMENS

PROJECT NO:	**5894-20**	CLIENT:	**Alaska Rail Road Corporation**
PROJECT NAME:	**ARRC Bridge 127.5 (Eagle River)**	SAMPLE NO.:	**B3 - S6T**
TESTED BY:	**A. Fortt**	DATE RECEIVED:	**11/13/2020**
PROJ. MANAGER:	**A. Smith**	DATE TESTED:	**1/11/2021**

BOREHOLE	SAMPLE #	LENGTH [in]	DIAMETER [in]	FAILURE LOAD [lb]	TENSILE STRENGTH [psi]	TENSILE STRENGTH [MPa]	FAILURE MODE
B3	S6T-1	1.48	2.38	5477	989	6.82	Failure plane perp. to core axis, parrallel to load axis.
B3	S6T-2	1.54	2.34	3856	681	4.70	Failure plane perp. to core axis, parrallel to load axis.
B3	S6T-3	1.38	2.36	2359	464	3.20	Failure plane perp. to core axis, parrallel to load axis.
B3	S6T-4	1.47	2.36	2961	543	3.74	Failure plane perp. to core axis, parrallel to load axis.
B3	S6T-5	1.46	2.35	4336	804	5.55	Failure plane perp. to core axis, parrallel to load axis.
B3	S6T-6	1.48	2.35	3720	681	4.70	Failure plane perp. to core axis, parrallel to load axis.
B3	S6T-7	1.48	2.38	3269	594	4.10	Failure plane perp. to core axis, parrallel to load axis.
B3	S6T-8	1.43	2.38	2846	533	3.67	Failure plane perp. to core axis, parrallel to load axis.
B3	S6T-9	1.47	2.37	3460	631	4.35	Failure plane perp. to core axis, parrallel to load axis.
B3	S6T-10	1.50	2.38	3524	630	4.34	Failure plane perp. to core axis, parrallel to load axis.

	FAILURE LOAD	TENSILE STRENGTH [psi]	TENSILE STRENGTH [MPa]
AVERAGE	**3581**	**655**	**4.52**
STDEV	**869**	**151**	**1.04**

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PROJECT NO:	5894-20
PROJECT NAME:	ARRC Bridge 127.5 (Eagle River)
CLIENT:	Alaska Rail Road Corporation
PROJ. MANAGER:	A. Smith
TESTED BY:	A. Fortt
DATE TESTED:	1/11/2021

ASTM D7012 (METHOD C) - UNCONFINED COMPRESSIVE STRENGTH OF INTACT ROCK CORE SPECIMENS

NO.	BOREHOLE I.D.	SAMPLE NUMBER	DEPTH (ft)		DIA. (in)	HEIGHT (in)	AREA (in^2)	VOLUME (in^3)	MASS (lb)	WET DENSITY (lb/in^3)	M. CONTENT (%)	DRY DENSITY (lb/in^3)	MAX. LOAD (lb)	MAX. STRESS (psi)	ROCK TYPE	FAILURE MODE	
			TOP	BOT.												TYPE	DEG
1	B3	S4C.1			2.35	4.47	4.33	19.37	N/A	N/A	N/A	N/A	15,876	3,663		2	
2	B3	S4C.2			2.36	4.48	4.37	19.54	N/A	N/A	N/A	N/A	15,683	3,591		2	
3	B3	S5C.1			2.36	4.49	4.36	19.54	N/A	N/A	N/A	N/A	26,909	6,178		6	
4	B3	S5C.2			2.36	4.48	4.37	19.59	N/A	N/A	N/A	N/A	29,413	6,730		6	
5	B3	S6C.1			2.38	4.48	4.46	19.98	N/A	N/A	N/A	N/A	54,223	12,147		6	
6	B3	S6C.2			2.38	4.47	4.43	19.82	N/A	N/A	N/A	N/A	36,557	8,252		6	

FAILURE MODES

(1) - DIAGONAL SHEAR PLANE(S) **(2)** - VERTICAL FRACTURE(S) **(3)** - VERTICAL SPLITTING **(4)** SHEAR ALONG FOLIATION/DISCONTINUITY **(5)** - CONICAL **(6)** - SPALLING **(7)** OTHER

NOTE: (DEG) MEASURED FROM CORE AXIS

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