GEOTECHNICAL ENGINEERING ASSESSMENT
of the
CHRISTENSEN DRIVE / W 1ST AVE REPAIR
located at
ANCHORAGE, ALASKA

Prepared for:

Alaska Railroad Corporation
PO Box 107500
Anchorage, AK 99510-7500

Prepared by:

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

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DECEMBER 2021

NORTHERN GEOTECHNICAL ENGINEERING, INC. / TERRA FIRMA TESTING

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

December 17, 2021 NGE-TFT Project # 5136-18

Alaska Railroad Corporation
PO Box 107500
Anchorage, AK 99510-7500

RE: GEOTECHNICAL ENGINEERING ASSESSMENT OF STREET REPAIR OF CHRISTENSEN DRIVE / WEST 1ST AVENUE - ANCHORAGE, ALASKA.

Paul,

We (Northern Geotechnical Engineering, Inc. *d.b.a.* Terra Firma Testing) have completed our geotechnical engineering assessment for the above referenced project site. The soils underneath the existing street consist of gravels and sands that are suitable for supporting the proposed site improvements provided that proper engineering controls are incorporated into the design and construction of the proposed site improvements.

We recommend matching the existing street section to reduce the potential for differential settlement. We provide more details regarding the design and construction of our recommendations as well as a summary of our field and laboratory testing programs in the following report.

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

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

Josselynn P. Schneider-Curry, EIT
Project Engineer

Keith F. Mobley, P.E.
President

STATE OF ALASKA
49TH
17 Dec. 2021
Keith F. Mobley
CE - 5066
REGISTERED PROFESSIONAL ENGINEER

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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 Christensen Drive and West 1st Avenue in Anchorage, Alaska; which we hereafter refer to as "the project site". We provided our professional service in accordance with our service fee proposal #21-208 which we submitted to Mr. Paul Farnsworth of the Alaska Railroad Corporation (ARRC) (our client) on October 13, 2021. Our client authorized our proposed scope of service via email on or about October 29, 2020.

ARRC contracted us to characterize the subsurface conditions across the project site in an effort to assess the suitability of the subgrade to support the proposed site repairs. In this report, we provide a summary of our subsurface exploration and laboratory testing efforts, as well as provide our conclusions and recommendations for the design and construction of the proposed pavement section.

2.0 PROJECT OVERVIEW

As shown in Figure 1 of this report, the street section for repair is located on the south side of Depot drive at the intersection with Christensen Drive of West First Avenue.

A sheet pile retaining wall was constructed in 2019, based upon a geotechnical report that we provided to LCG Lantech, Inc., on October 26, 2018 and titled "*Recommendations For A Sheet Pile Retaining Wall For The Proposed West 1ST Avenue Access Road Extension In Anchorage, Alaska"*. A summary of the events and the wall stability are included in Appendix A of this report.

As the retaining wall is now no longer experiencing continued non-uniform deflection along most of its length, we feel it is appropriate to begin the repair of the street section mentioned above. In order to provide site-specific geotechnical engineering recommendations to address needed structural section of the street repair, we conducted a subsurface exploration program with soil borings located behind the existing retaining wall in order to help characterize the subsurface conditions.

3.0 SITE CHARACTERIZATION ACTIVITIES

3.1 Subsurface Exploration

We conceived, coordinated, and directed a subsurface exploration program at the project site in an effort to characterize the subsurface conditions of the project site as they currently exist. We subcontracted Discovery Drilling, Inc. (DDI) to provide the necessary geotechnical exploration services. 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

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laboratory analysis. Under our direction DDI advanced a total of two soil borings at the project site on November 23, 2021, to a depth approximately 16.5 feet below the existing ground surface (bgs) using conventional hollow-stem auger drilling and split-spoon sampling methods. We detail the approximate locations of the soil borings in Figure 2 of this report.

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

We corrected the field blow count data for both 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 B 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 sealed each sample that they collected during our subsurface exploration program inside of an air-tight bag and/or container, to help preserve the moisture content of each sample, and then submitted each sample to our laboratory for further identification and analysis.

Once the exploration activities were complete, we directed DDI to backfill the annulus of each exploration with its respective drill cuttings and repair the asphalt pavement surface at each soil boring location using cold-patch asphalt mix.

4.0 LABORATORY TESTING

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

- moisture content analysis (ASTM D-2216);

- grain size sieve and hydrometer analysis (ASTM D-6913 & D-7928); and
- Atterberg limits (ASTM D-4318).

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 and frost classification results which are not representative of the actual (i.e., in-situ) soil gradation and/or frost classification. The use of small-diameter sampling devices in coarse-grained soils (e.g., sand and gravel) can result in the collection of unrepresentative samples due to: the exclusion of oversized particles (larger than the opening of the sampler) from the sample; and the mechanical breakdown/degradation of coarse-grained particles by the sampling process (producing an unrepresentative increase in smaller-diameter particles in the sample). Both of these sampling biases can skew laboratory test results towards the fine-grained end of the gradational spectrum.

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

5.0 DESCRIPTION OF SUBSURFACE CONDITIONS

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

5.1 General Subsurface Profile

Underneath the existing road surface is a well graded gravel with silt and sand that extends to approximately 4.5 to 5 feet bgs, which is likely the current pavement section. The gravel is underlain by native silty sand with gravel to poorly graded sand with silt and gravel to approximately 7.5 to 10 feet bgs. This sandy layer is underlain by silty clay to clay to approximately 12 to 15.5 feet bgs. Poorly graded sand was identified underneath the clay in boring B1 and sandy silt underneath the silty clay in boring B2 to the bottom of the exploration (approximately 16.5 feet bgs).

5.2 Groundwater

No groundwater was encountered during the borehole explorations. In past locations and reports groundwater has been reported to be approximately 22 to 25 bgs.

5.3 Frozen Soils

We observed indications of seasonally frozen soils down to approximately 5 feet bgs in the borings advanced behind the retaining structure. We do not expect permafrost to occur anywhere across the project site.

6.0 ENGINEERING CONCLUSIONS

Based on the findings of our field and laboratory testing efforts along with our engineering analysis, it is our conclusion that the gravel and sand soils which we observed across the project site are generally suitable to support the proposed repairs; provided that our concerns and recommendations that we present in this report are addressed by the design and construction processes.

Based on our soil explorations, the existing street section differs from the as-built plans. The pavement was approximately five inches thick in our soil borings and no insulation was encountered.

Based on our laboratory testing efforts the near surface gravel with silt and sand classifies as non-frost susceptible to moderately frost susceptible (NFS to F2) on the Municipality of Anchorage (MOA) frost classification scale. The underlying silty sand with gravel classifies as moderately frost susceptible (F2). While the increased frost susceptibility of the subgrade soils would typically warrant a properly engineered pavement section, a different pavement section from the existing pavement section along Christensen Drive and West First Avenue will cause differential settlement. As such, we recommend that the pavement section for the repairs matches the existing pavement section (and not the as-built plans). We do, however, recommend removing a portion of the existing pavement section and replacing it with properly compacted NFS material as a part of the repair effort.

During excavation, the contractor should confirm that there is no insulation in the pavement section. Per the as-built plans, the insulation should be located one foot below the levelling course. If insulation is encountered, it should be removed and replaced.

From a geotechnical standpoint, only half of the roadway needs to be repaired. We provide more detailed recommendations in Sections 7.2 and 8.2 of this report.

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7.0 DESIGN RECOMMENDATIONS

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

7.1 Earthworks

Any structural fill materials used on-site should be compacted to a minimum of 95 percent of the modified Proctor density. Any exposed material should be proof rolled prior to placing any structural fill.

Any NFS material removed during the initial excavation activities, which does not contain any organic/deleterious material can be re-used on-site as structural fill. Proper placement and compaction techniques need to be applied during the backfill process (see Section 8.1 of this report for more details). Additional laboratory testing is required to verify the frost susceptibility of any excavated soil for reuse.

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

7.2 Pavement Section

As we discuss in Section 6.0 of this report, we do not recommend a different pavement section for the repairs of Christensen Drive/W 1st Avenue due to the increased risk of differential settlement. We recommend removing the pavement, levelling course, and 12 inches of the Type II-A section and replacing those materials. If insulation is encountered during excavation, we recommend replacing it with new insulation. We detail the new portion of the pavement section in Table 1 of this report. We have provided an insulated pavement section that satisfies MOA design requirements in Section 7.2.1 of this report for reference.

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Table 1: NGE-TFT Recommended Pavement Section Repairs for F2 subgrade

SECTION THICKNESS	MATERIAL
MATCH EXISTING	ASPHALT PAVEMENT
2 INCHES	LEVELING COURSE (A.K.A. "D-1")
12 INCHES	MOA TYPE II-A (NFS)
N/A	F2 SUBGRADE (NATIVE OR FILL)

7.2.1 MOA Pavement Section

The proposed street extension, once completed, will be maintained by the MOA. Therefore, the MOA requires that the structural pavement section be designed to reduce the potential for future frost-related pavement damage (as a result of ice lens development and subsequent thaw-related settlements) and prolong the life of the pavement surface. To accomplish this task, the 2007 MOA Design Criteria Manual (DCM) requires that all municipally-maintained roadways constructed by private parties be designed using either the Complete Protection Method or Limited Subgrade Frost Penetration (LSFP) Method. The LSFP Method is the most common design approach used for MOA-regulated pavement section designs, and is the design method that we have utilized to satisfy MOA design requirements

As part of the LSFP Method design, the MOA requires that a thermal analysis be conducted for all subgrade materials that exhibit an MOA frost class designation of F2, F3, or F4. The MOA's preferred methodology for the thermal analysis is to use a software program known as BERG2, which uses the modified Berggren equation to calculate frost-depth penetration. In order to fulfill MOA design requirements we performed the required BERG2 analyses and we have included the results of our BERG2 analyses (for pavement sections with insulation) in Figure 4 of this report.

We used the following average soil parameters for the of the F2 subgrade in our BERG2 analysis:

- Moisture Content = 6.6%,
- Unit Weight = 130 pcf;

These average soil parameters values are based on the findings of our subsurface exploration and laboratory testing programs. We detail the pavement sections that the MOA recommends (based on our BERG2 analyses and the LSFP Method) for construction at the project site in Table 2 of this report.

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Table 2: MOA Recommended Pavement Section for F2 subgrade (with Insulation)

SECTION THICKNESS	MATERIAL
2 INCHES	ASPHALT PAVEMENT
2 INCHES	LEVELING COURSE (A.K.A. “D-1”)
16 INCHES	MOA TYPE II-A
2 INCHES	CLOSED-CELL FOAM BOARD INSULATION
20 INCHES	MOA TYPE II
N/A	F2 SUBGRADE (NATIVE OR FILL)

Section 1.10F of the MOA DCM provides an acceptable guideline of a minimum of 18 inches of gravel fill cover over any insulation used in a pavement section to protect the insulation from heavy wheel loads during construction and to minimize frost formation on the pavement surface.

7.2.2 Insulation

If insulation is found in the existing section, then it will need to be replaced. Any subsurface insulation should consist of extruded polystyrene such as DOW Styrofoam™ Highload or UC Industries Foamular. Any subsurface insulation used under pavement sections should be closed cell, board stock with a minimum compressive strength of 60 psi at five percent deflection. The insulation should not absorb more than two percent water per ASTM Test Method C-272. The thermal conductivity (k) of the insulation should not exceed 0.25 BTU-in/hr-ft2-°F when tested at 75°F.

7.2.3 Confirmation Testing and Material Specifications

Any differing soil conditions from our recommended pavement section should have confirmation frost classification testing.

A permeable geotextile fabric is optional, but not required for this project. Any geotextile fabric used should meet the specifications in the 2015 Municipality of Anchorage Standard Specifications (MASS), Section 20.25. For the project site, we recommend a Type A, Class 2 (i.e., separation) geotextile fabric. The geotextile fabric may be either: 1) woven, or 2) non-woven with perforations. We have provided the various strengths for both a woven and non-woven Type A, Class 2 geotextile fabric in Table 3 of this report.

Table 3: Type A, Class 2 Geotextile Fabric Strengths

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

Note: Units in lbs per foot.

The leveling course, Type II, and Type II-A materials used should conform to the specifications we provide in Figure 5 of this report. Any leveling course used should be NFS in order to maintain a low potential for ice lens development within the leveling course. It is our experience that the "D-1" leveling course material currently available in Anchorage area may not be NFS following compaction, because the compaction with a vibratory compactor further increases the frost susceptibility of the leveling course by increasing the percentage of fine-grained material (due to degradation of the soil particles from the impact of the compaction equipment). As such, we recommend the use of two inches of recycled asphalt pavement (RAP) for the leveling course, as RAP has a low frost susceptibility. Otherwise, the leveling course thickness should be kept to two inches or less to reduce the potential for ice lens formation in the leveling course. Type II-A materials can be used as a substitute for Type II materials, as Type II materials are becoming difficult to procure in the Anchorage area. However, no Type II materials should be placed within 12 inches of any pavement surfaces to help reduce the risk of pavement dimpling (from oversized particles contained within the Type II material). All of these materials should be placed in thin lifts and each lift should be compacted to a minimum of 95 % of the modified Proctor density.

7.3 Surface Drainage

Water accumulation behind the retaining will have a detrimental effect on the stability of the wall. Provisions should be included in the design to collect runoff and divert it away from the wall. We recommend placing topsoil along the slope between the sidewalk and the retaining wall to raise the grade. Vegetation should be planted vegetation to reduce the surface infiltration.

8.0 CONSTRUCTION RECOMMENDATIONS

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

8.1 Earthwork

Any and all fill material used should be placed at 95 percent of the modified Proctor density as determined by ASTM D-1557, unless specifically stated otherwise in other sections of this report.

The thickness of individual lifts will be determined based on the equipment used, the soil type, and existing soil moisture content. Typically, fill material will need to be placed in lifts of less than one-foot in thickness. All earthworks should be completed with quality control inspection.

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

8.2 Pavement

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

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

8.3 Insulation

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

8.4 Winter Construction

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

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 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.

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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 and construction of the proposed improvements. We should be notified if significant changes are to occur in the nature, design, or location of the proposed improvements in order that we may review our conclusions and recommendations that we present in this report and, if necessary, modify them to satisfy the proposed changes.

This report should always be read and/or distributed in its entirety (including all figures, exploration logs, appendices, etc.) 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.

REPORT
FIGURES

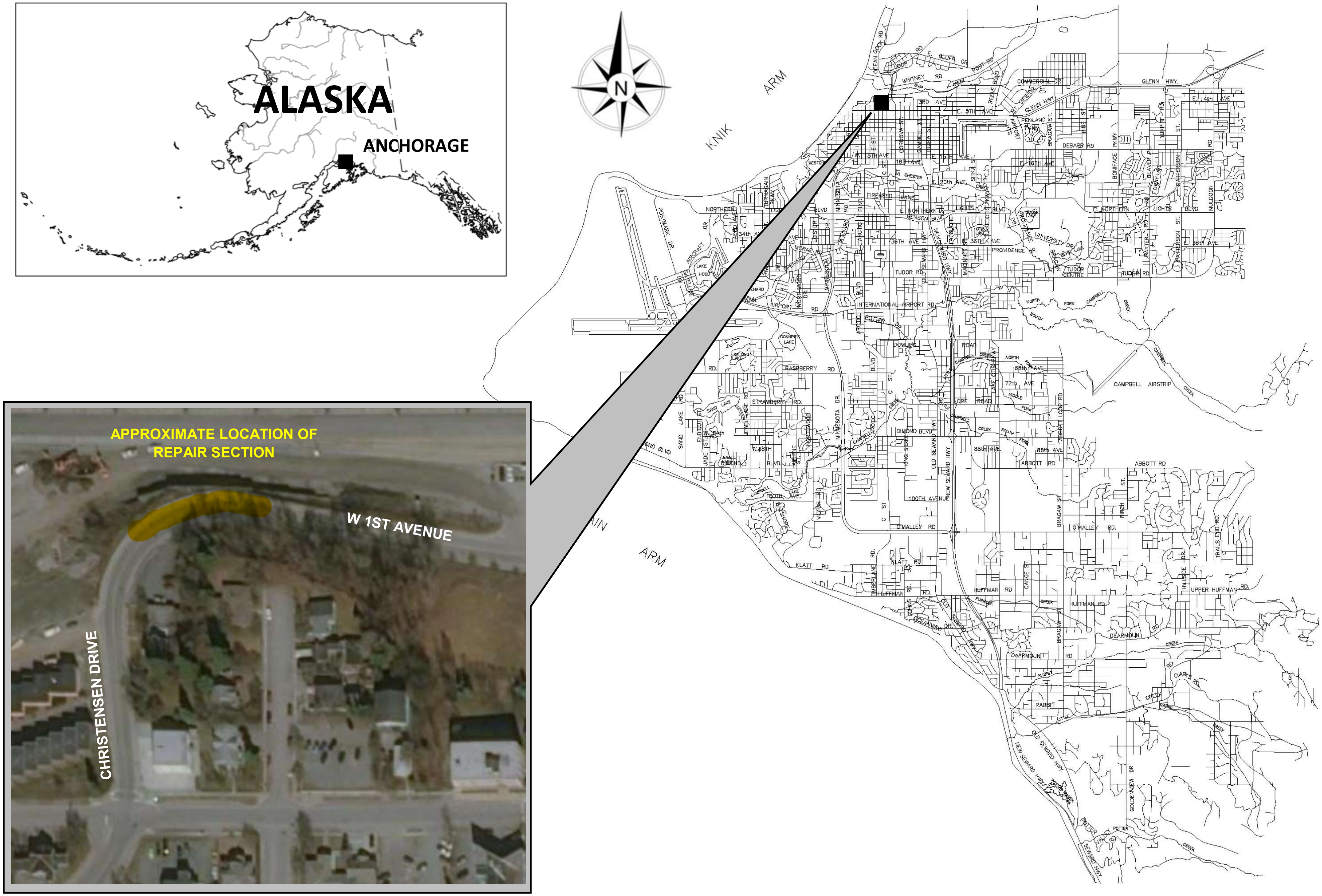

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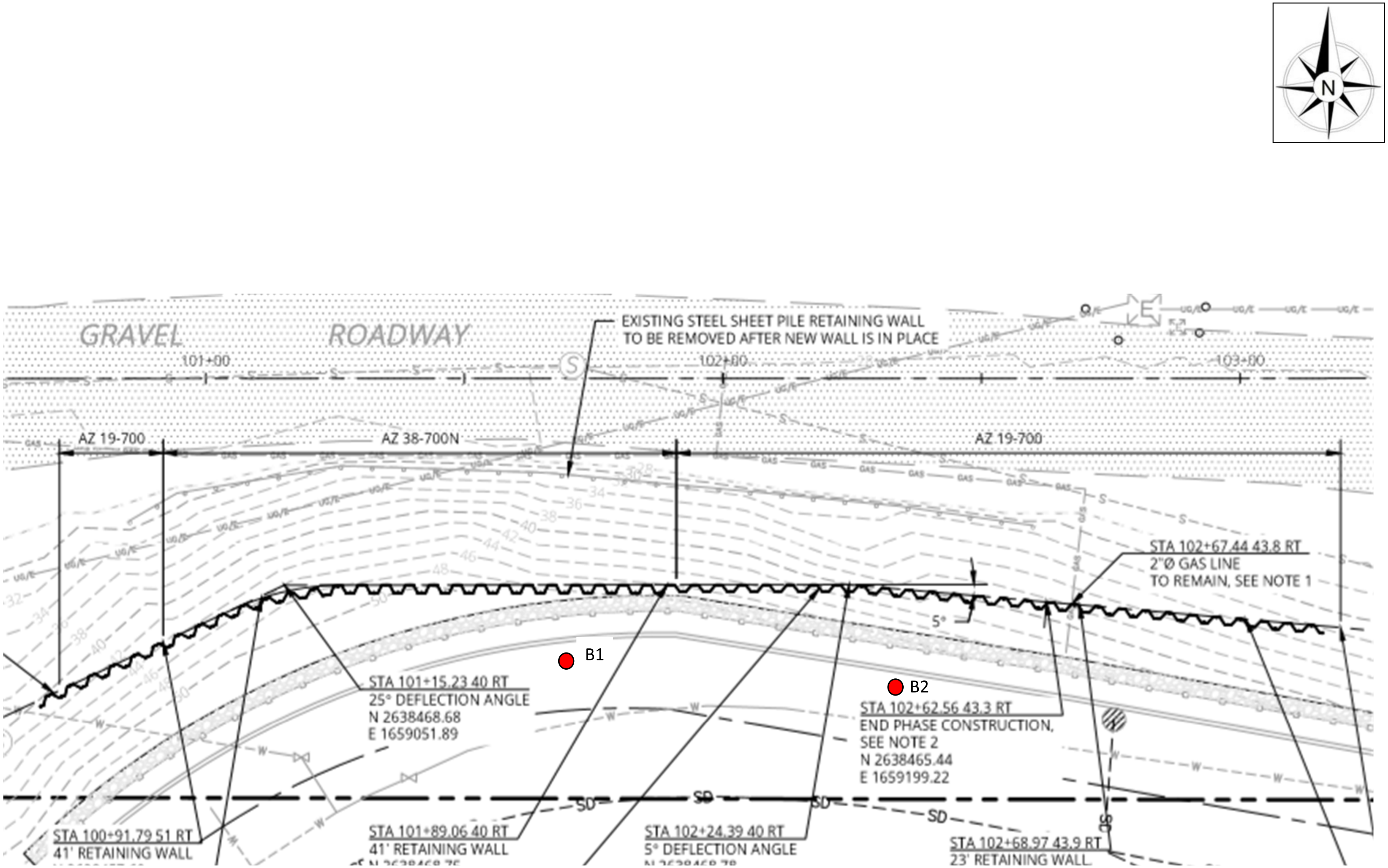
FIGURE TITLE: PROJECT SITE LOCATION MAP	
PROJECT NAME: CHRISTENSEN STREET REPAIR	PROJECT ID: 5136-18
PROJECT LOCATION: ANCHORAGE, ALASKA	FIGURE NUMBER: 1



= Approximate Borehole Locations

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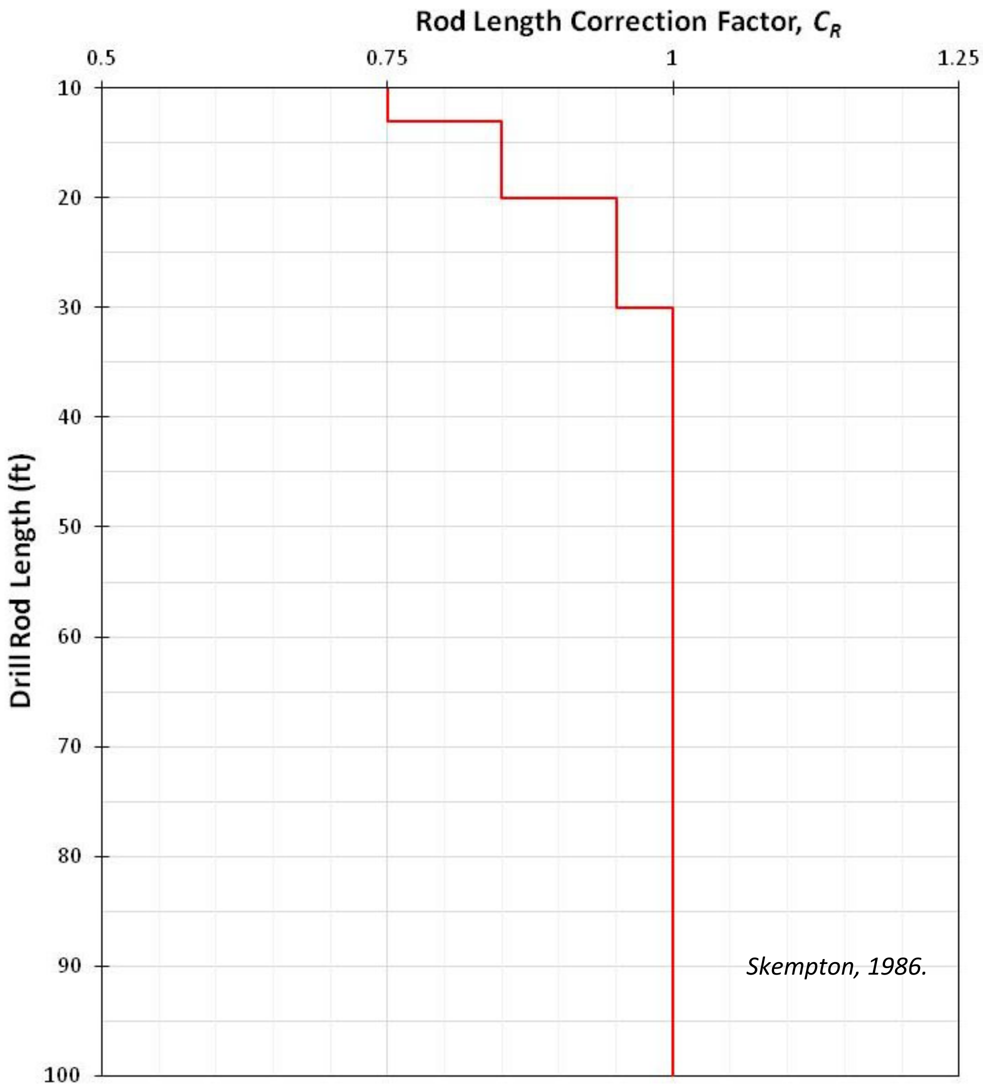
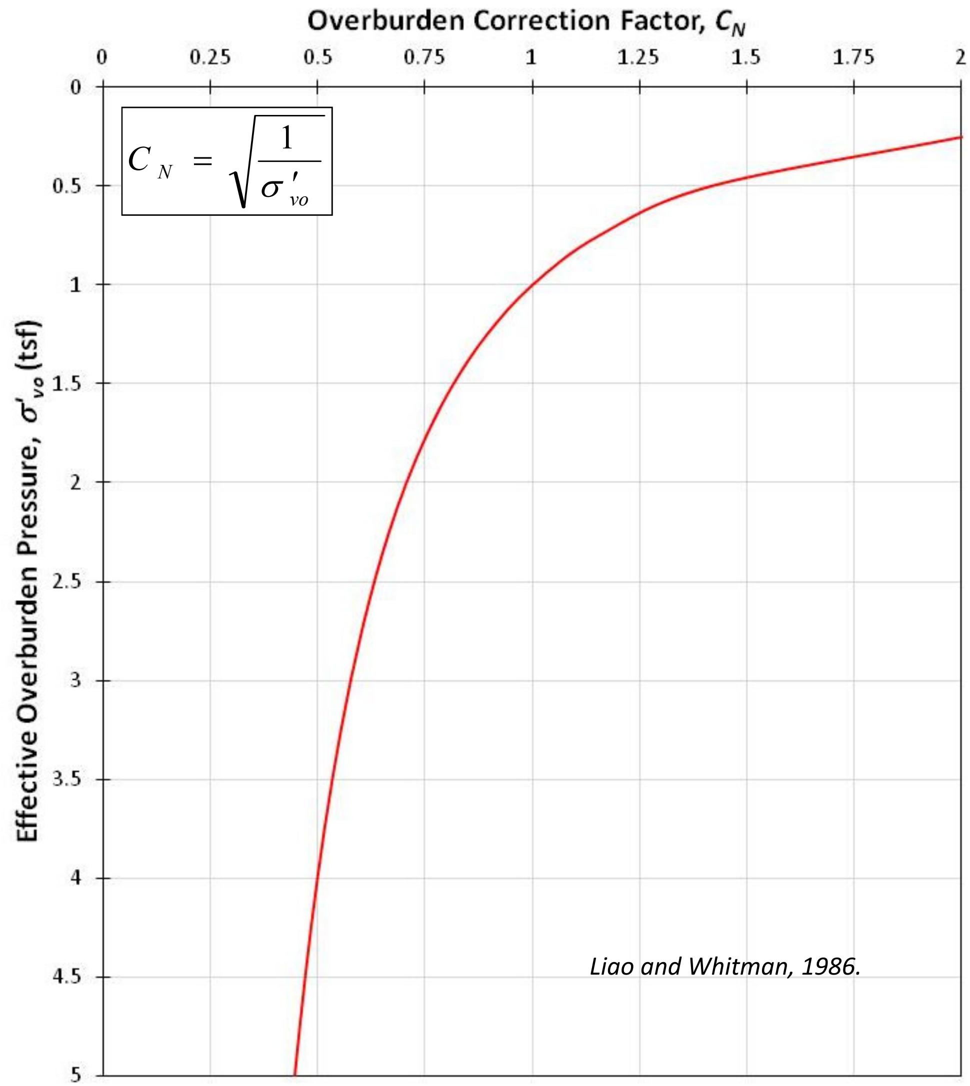
FIGURE TITLE: EXPLORATION LOCATION MAP	
PROJECT NAME: CHRISTENSEN STREET REPAIRS	PROJECT ID: 5136-18
PROJECT LOCATION: ANCHORAGE, ALASKA	FIGURE NUMBER: 2





Notes:

- OVERBURDEN CORRECTION FACTOR IS USED ONLY FOR COHESIONLESS SOILS
- C_N IS THE RATIO OF THE MEASURED BLOW COUNT TO WHAT THE BLOW COUNT WOULD BE AT AN OVERBURDEN PRESSURE OF 1 TON/FT^2
- Σ'_{VO} IS THE EFFECTIVE OVERBURDEN PRESSURE AT THE POINT OF MEASUREMENT (TON/FT^2)

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FIGURE TITLE: BLOW COUNT CORRECTIONS	
PROJECT NAME: CHRISTENSEN STREET REPAIR	PROJECT ID: 5136-18
PROJECT LOCATION: ANCHORAGE ALASKA	FIGURE NUMBER: 3

LOCATION	THAW N	FREZ N	MAAT	THAW °F DAY	FREZ °F DAY	THAW DAYS	FREZ DAYS
ANCHORAG	1.90	0.90	34	4000	3200	192	173

		1	2	3	4	5	6
THAW CYCLE	FROZEN % MOIS.	0.0	7.0	5.5	0.0	5.5	6.6
	FROZEN DENS.	152.0	137.0	135.0	2.0	135.0	130.0
	LATENT HEAT	0	1381	1069	0	1069	1236
	FROZEN HEAT CAP	28.00	28.09	26.66	3.00	26.66	26.39
	FROZEN COND.	0.86	2.25	1.73	0.02	1.73	1.70
	THAWED % MOIS.	0.0	7.0	5.5	0.0	5.5	6.6
	THAWED DENS.	152.0	137.0	135.0	2.0	135.0	130.0
	THAWED HEAT CAP	28.00	32.88	30.38	3.00	30.38	30.68
	THAWED COND.	0.86	1.94	1.71	0.02	1.71	1.62
	INITIAL THICK	0.17	0.17	1.33	0.17	1.63	10.00
	AMOUNT THAWED	0.17	0.17	1.33	0.17	1.63	4.76
	CONSOLIDATION	----	----	----	----	----	----
	FINAL THICK	0.17	0.17	1.33	0.17	1.63	10.00
FREEZE CYCLE	LATENT HEAT	0	1381	1069	0	1069	1236
	FROZEN DENS.	152.0	137.0	135.0	2.0	135.0	130.0
	FROZEN HEAT CAP	28.00	28.09	26.66	3.00	26.66	26.39
	FROZEN COND.	0.86	2.25	1.73	0.02	1.73	1.70
	INITIAL THICK	0.17	0.17	1.33	0.17	1.63	10.00
	AMOUNT FROZEN	0.17	0.17	1.33	0.17	1.63	0.31

ESTIMATED THAW= 8.23 FREEZE= 3.78 PRINT LOCATION SOIL QUIT

LAYER #	MATERIAL TYPE	LAYER THICKNESS
1	ASPHALT	2 IN.
2	LEVELING COURSE	2 IN.
3	TYPE II/II A	16 IN.
4	INSULATION	2 IN.
5	TYPE II	20 IN.
6	NATIVE SOILS (F2)	N/A

LEVELING COURSE

SIEVE SIZE	% BY MASS PASSING
1”	100
3/4”	70-100
3/8”	50-80
#4	35-65
#8	20-50
#50	8-28
#200	2-6
0.02	0-3

TYPE II

SIEVE SIZE	% BY MASS PASSING
8”	100
3”	70-100
1-1/2”	55-100
3/4”	45-85
#4	20-60
#10	12-50
#40	4-30
#200	*2-6
0.02	0-3

*IN ADDITION TO THE GRADING LIMITS LISTED ABOVE, THE FRACTION OF MATERIAL PASSING THE #200 SIEVE SHALL NOT BE GREATER THAN FIFTEEN PERCENT (15 %) OF THAT FRACTION PASSING THE #4 SIEVE.

TYPE II - A

SIEVE SIZE	% BY MASS PASSING
3”	100
3/4”	50-100
#4	25-60
#10	15-50
#40	4-30
#200	*2-6
0.02	0-3

*IN ADDITION TO THE GRADING LIMITS LISTED ABOVE, THE FRACTION OF MATERIAL PASSING THE #200 SIEVE SHALL NOT BE GREATER THAN TWENTY PERCENT (20 %) OF THAT FRACTION PASSING THE #4 SIEVE.

APPENDIX A

CHRISTENSEN DRIVE / W 1ST AVENUE WALL REPAIRS ANALYSIS

December 17, 2021 NGE-TFT Project # 5136-18

Alaska Railroad Corporation
PO Box 107500
Anchorage, AK 99510-7500

Attn: Paul Farnsworth

RE: EVAULATION OF THE ARRC RETAINING WALL LOCATED ON CHRISTENSEN DRIVE / W FIRST AVENUE - ANCHORAGE ALASKA

Paul,

We (Northern Geotechnical Engineering, Inc. *d.b.a.* Terra Firma Testing) have completed our evaluation of the sheet pile retaining wall repairs and alterations that were performed along the curve of Christensen Drive and W. 1st Avenue to the north of 2nd Avenue in downtown Anchorage.

1.0 History

A wall was installed to provide an access road at the base for the new development west of Christensen Street. Prior to construction of the wall, the sidewalk on 2nd avenue had moved north, away from the gutter and asphalt.

1.1 Original Construction

The sheet pile retaining wall was constructed in 2019, based upon a geotechnical report that we provided to LCG Lantech, Inc., on October 26, 2018, and titled "*Recommendations For A Sheet Pile Retaining Wall For The Proposed West 1ST Avenue Access Road Extension In Anchorage, Alaska''*. At the client's request (due to time constraints), we did not perform a geotechnical exploration at the project site, and therefore the report was based upon non-site-specific geotechnical information.

After completion of the initial design for the sheet pile wall along Christensen Street, additional information became available about the sheet pile material ARRC had available. The design was adjusted to accommodate the available sheets as best as possible while recognizing they would not perform as well as the original intended sheet design. This was done in an effort to expedite the

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construction of the retaining wall. Additional reinforcement was added to the wall with the H piles that were driven in front of the installed sheets due to the sheets being shorter than required. The top of some of the H piles were subsequently cut off to accommodate the installation of the helical anchors.

1.2 Movements

After installation of the sheets and additional reinforcements, survey monitoring of the wall was conducted to measure the long-term performance of the substituted sheets. From site observations conducted during the winter and spring of 2020, it was noted that water had accumulated behind the wall as evidenced by seepage through the sheet pile interlocks. When the water levels lowered (no more evidence of seepage through the interlocks), the wall movement ceased and rebounded slightly. In our professional opinion, the retaining wall deflections occurred primarily as a result of the seasonal accumulation of groundwater in the soils behind the retaining wall. After this was discovered, additional geotechnical boreholes were advanced, and water monitoring was added to the site.

This water appears to be seasonally impounded due to the formation of ice on, and around the outer surface of the retaining wall which creates an impermeable barrier to the flow of subsurface water. This condition resulted in an (unanticipated) additional mass (from the accumulated water) to act upon the sheet pile retaining wall, causing the wall to deflect under the increased load.

1.3 Anchor Design

Given the complex soil profile, variable water, laboratory strength data, and the noted movements, calculations were completed to match the empirical data with the site observations. Numerous iterations were completed to assess the sensitivity of the variables and to approach a factor of safety equal to 1. The most sensitive element of the analysis was the height of the water behind the wall.

With the wall constructed, existing utilities, and limitations of what could be done in front of the wall, a tie back anchor system was considered the most appropriate method to stabilize the wall. The sizes of the anchors installed were slightly smaller than designed due to limited locally available helical anchor flights during time of installation. The anchors were pull tested and installed per plan, but three areas failed to meet the required resistance. An additional three anchors were installed to achieve the final needed added resistance for wall stability.

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1.4 Wick Drains

Wick drains were incorporated into the repair design in order to help drain the accumulated water behind the wall. Horizontal wick drains have been used successfully for stabilization of landslides and retaining walls in past projects. One horizontal wick drain was installed per pair of sheets in the affected area with heat trace wiring inside to facilitate draining during winter months.

1.5 Construction

After the installation of the anchors, completion of the pull tests, and significant enough passage of time to allow the wick drains to mitigate the level of water behind the wall, additional calculations were performed to determine if more anchors were needed to stabilize the wall in the event of an earthquake. Additional anchors were added in Fall 2021 to finalize the stability of the wall.

2.0 Wall Evaluation

The original design of the wall anchors utilized the worst-case scenario with respect to wall height, impounded water, anchor influence area, and soil strengths. The actual anchor locations with respect to neighboring anchors were reviewed for potential excess capacity in some anchors to compensate for the few anchors that did not meet the preconstruction strengths. We determined the actual required capacity for each anchor considering the wall heights, sheet piles used, H pile heights (where located and cutoff), anticipated maximum water height affected by the installed PVC drains, seismic loading, a range of geotechnical conditions, anchor depth, anchor inclination, and the anticipated surcharge loading (street and traffic).

For the geotechnical analysis, seven cross sections of the wall with distinct features were analyzed throughout the wall. The soils behind the wall are generally consistent based on the boreholes used from the surrounding area and fall within the range of values explored in the analysis.

2.1 Assumptions

In order to complete the analysis some assumptions were made. Water levels, soil parameters, and seismic loading of the wall are all unknown variables. To account for these assumptions a range of values were explored with the most conservative values used to determine the final needed capacities for the installed anchor system. The ranges of values for the pertinent variables for analysis were chosen based on available bore logs in the area.

2.2 Results

Using the behavior of the wall seen over the last two years leads to the conclusion that the existing factor of safety of the wall before the anchors and drains were installed to be slightly less than 1 based on the slight movement of the wall at all measured points. This was used to verify the range of values used to calculate the stability of the wall. Based on our calculations all sheets when analyzed now meet or exceed a factor of safety of 1.2.

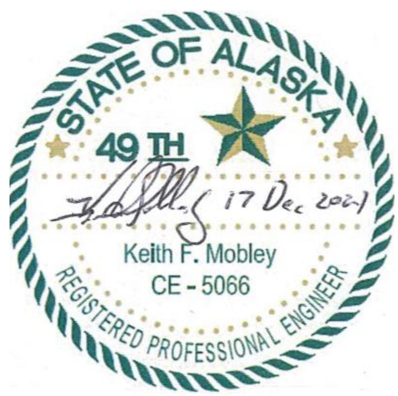
3.0 Conclusions

It is our professional opinion that the wall is currently stabile with all the additional reinforcement that has been installed.

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 letter, or if you have any other questions, comments, and/or requests.

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

Keith F. Mobley, P.E.
President



APPENDIX B

GRAPHICAL EXPLORATION BOREHOLE LOGS

Northern Geotechnical Engineering, Inc. and Terra Firma Testing 11301 Olive Lane Anchorage, AK 99515 Telephone: 907-344-5934	# EXPLORATION B-1 PAGE 1 OF 1

NGE-TFT PROJECT NAME: Christensen Street Retaining Wall
NGE-TFT PROJECT NUMBER: 5136-18
PROJECT LOCATION: Anchorage, AK
EXPLORATION CONTRACTOR: Discovery Drilling, Inc.
EXPLORATION EQUIPMENT: Truck-mounted CME 75
EXPLORATION METHOD: Hollow Stem Auger
SAMPLING METHOD: MPT w/ 340lb autohammer
LOGGED BY: S. Murphy
DATE/TIME STARTED: 11/23/2021 @ 9:30:00 AM
DATE COMPLETED: 11/23/2021
EXPLORATION LOCATION: See report Figure 2
GROUND ELEVATION: Not Known
▽ **GROUNDWATER (ATD):** N/E
▼ **GROUNDWATER ():** N/A
EXPLORATION COMPLETION: See comments at end of log
WEATHER CONDITIONS: N/A

DEPTH (ft bgs)	MATERIAL DESCRIPTION	FIELD SAMPLE ID	RECOVERY (in)	FIELD BLOWS	$(N_1)_{60}$	LAB SAMPLE ID	LAB RESULTS	REMARKS/NOTES
0	**WELL GRADED GRAVEL WITH SILT AND SAND** (GW-GM), brown, dry	S1	17	43 47 32	N/A	S1	S1 MC = 3.3% 46.8% gravel, 46.8% sand, 6.5% silt P0.02 = 1.9% FC = NFS	Approx. 5 in of pavement.
		S2	16	10 7 7	N/A	S2	S2 MC = 3.6% 43.0% gravel, 48.6% sand, 8.4% silt P0.02 = 5.3% FC = F2	
5	**SILTY SAND WITH GRAVEL** (SM), medium dense, brown, dry	S3	12	7 4 4	11	S3	S3 MC = 3.7% 34.5% gravel, 52.1% sand, 13.4% silt P0.02 = 9.0% FC = F2	
		S4	6	7 7 5	14	S4	S4 MC = 5.2%	
10	**CLAY** (CL), stiff, blue - gray, damp	S5	16	3 7 7	12	S5	S5 MC = 21.3% LL = 30 PL = 21 PI = 9	
15	**POORLY GRADED SAND** (SP), loose, brown, damp, fine grained	S6		6 5 4	8	S6	S6 MC = 24.2%	

Bottom of borehole at 16.5 ft bgs.
Backfilled with cuttings to 1 ft bgs, pea gravel with asphalt patch at the top.

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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and Terra Firma Testing
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Anchorage, AK 99515
Telephone: 907-344-5934

EXPLORATION B-2

NGE-TFT PROJECT NAME: Christensen Street Retaining Wall
NGE-TFT PROJECT NUMBER: 5136-18
PROJECT LOCATION: Anchorage, AK
EXPLORATION CONTRACTOR: Discovery Drilling, Inc.
EXPLORATION EQUIPMENT: Truck-mounted CME 75
EXPLORATION METHOD: Hollow Stem Auger
SAMPLING METHOD: MPT w/ 340lb autohammer
LOGGED BY: S. Murphy
DATE/TIME STARTED: 11/23/2021 @ 11:00:00 AM
DATE COMPLETED: 11/23/2021
EXPLORATION LOCATION: See report Figure 2
GROUND ELEVATION: Not Known
▽ **GROUNDWATER (ATD):** N/E
▼ **GROUNDWATER ():** N/A
EXPLORATION COMPLETION: See comments at end of log
WEATHER CONDITIONS: N/A

DEPTH (ft bgs)	MATERIAL DESCRIPTION	FIELD SAMPLE ID	RECOVERY (in)	FIELD BLOWS	$(N_1)_{60}$	LAB SAMPLE ID	LAB RESULTS	REMARKS/NOTES
0	**WELL GRADED GRAVEL WITH SILT AND SAND** (GW-GM), brown - gray, dry	S1	15	10 27 13	N/A	S1	S1 MC = 3.5% 52.9% gravel, 39.6% sand, 7.5% silt P0.02 = 3.9% FC = F1	Approx. 5 in of pavement.
		S2	4	12 9 16	N/A	S2	S2 MC = 15.3%	
5	**POORLY GRADED SAND WITH SILT** (SP-SM), medium dense, brown, dry	S3	3	7 7 6	17	S3	S3 MC = 7.2%	
	SILTY CLAY (CL-ML), medium stiff, brown, damp	S4	16	3 3 3	5	S4	S4 MC = 16.4% LL = 31 PL = 24 PI = 7	
10		S5	17	2 4 4	7	S5	S5 MC = 17.1%	
	SANDY SILT (ML), stiff, brown, damp							
15		S6	18	1 6 7	12	S6	S6 MC = 19.0%	

Bottom of borehole at 16.5 ft bgs.
Backfilled with cuttings to 1 ft bgs, pea gravel with asphalt patch at the top.

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EXPLORATION LEGEND

CLIENT Alaska Railroad Corporation

NGE-TFT PROJECT NAME Christensen Street Retaining Wall

NGE-TFT PROJECT NUMBER 5136-18

PROJECT LOCATION Anchorage, AK

LITHOLOGIC SYMBOLS
(Unified Soil Classification System)

CL: USCS Low Plasticity Clay

CL-ML: USCS Low Plasticity Silty Clay

GW-GM: USCS Well-graded Gravel with Silt

MLS: Sandy Silt

SM: USCS Silty Sand

SP: USCS Poorly-graded Sand

SP-SM: USCS Poorly-graded Sand with Silt

SAMPLER SYMBOLS

WELL CONSTRUCTION SYMBOLS

ABBREVIATIONS

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

▽ Water Level at Time Drilling, or as Shown

▼ Water Level After 24 Hours, or as Shown

TV - TORVANE
PID - PHOTOIONIZATION DETECTOR
UC - UNCONFINED COMPRESSION
ppm - PARTS PER MILLION
N/E - NOT ENCOUNTERED
NR - NOT REPRESENTATIVE
N/A - NOT APPLICABLE

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SOIL CLASSIFICATION CHART

CLIENT Alaska Railroad Corporation

PROJECT NAME Christensen Street Retaining Wall

NGE-TFT PROJECT NUMBER 5136-18

PROJECT LOCATION Anchorage, 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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Anchorage, AK 99515
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EXPLORATION LOG KEY

CLIENT Alaska Railroad Corporation

PROJECT NAME Christensen Street Retaining Wall

NGE-TFT PROJECT NUMBER 5136-18

PROJECT LOCATION Anchorage, 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 Christensen Street Retaining Wall

NGE-TFT PROJECT NUMBER 5136-18

PROJECT LOCATION Anchorage, 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 C

LABORATORY RESULTS

Summary of Laboratory Test Results

Christensen Street Repair

NGE-TFT Project #:5136-18

Exploration ID	Sample Number	Depth Interval (ft) Top	Depth Interval (ft) Bottom	Moisture Content ASTM D2216 (% By Dry Mass)	Atterberg Limits ASTM D4318 LL	PL	PI	Particle Size Analysis ASTM C136/D7928/D6913 (% By Mass) Gravel	Sand	Silt/Clay	Passing 0.02mm ASTM D7928 (% By Mass)	Frost Class.	Unified Soil Classification ASTM D2487
B1	S1	0.0	1.5	3.3				46.8	46.8	6.5	1.9	NFS	(SW-SM) Well-graded sand w/ silt and gravel
B1	S2	2.5	4.0	3.6				43.0	48.6	8.4	5.3	F2	(SW-SM) Well-graded sand w/ silt and gravel
B1	S3	5.0	6.5	3.7				34.5	52.1	13.4	9.0	F2	(SM) Silty sand w/ gravel
B1	S4	7.5	9.0	5.2									
B1	S5	10.0	11.5	21.3	30.0	21.0	9.0						
B1	S6	15.0	16.5	24.2									
B2	S1	0.0	1.5	3.5				52.9	39.6	7.5	3.9	F1	(GW-GM) Well-graded gravel w/ silt and sand
B2	S2	2.5	4.0	15.3									
B2	S3	5.0	6.5	7.2									
B2	S4	7.5	9.0	16.4	31.0	24.0	7.0						
B2	S5	10.0	11.5	17.1									
B2	S6	15.0	16.5	19.0									

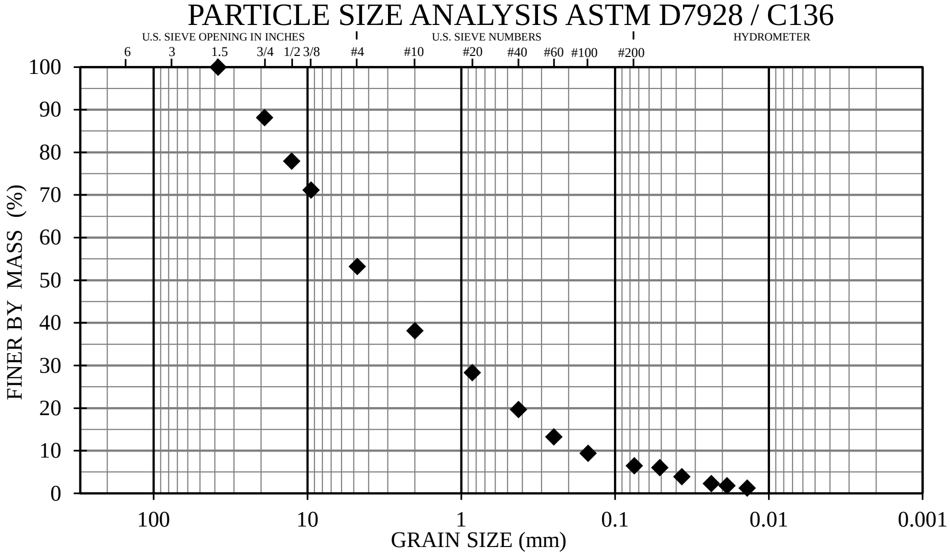
NORTHERN GEOTECHNICAL ENGINEERING, INC. / TERRA FIRMA TESTING

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

PROJECT CLIENT:	**ARRC**
PROJECT NAME:	**Christensen Street Repair**
PROJECT NO.:	**5136-18**
SAMPLE LOC.:	**B1**
NUMBER/ DEPTH:	**S1 / 0 - 1.5'**
DESCRIPTION:	**Well-graded sand w/ silt and gravel**
DATE RECEIVED:	**11/23/2021**
TESTED BY:	**Erik Boatwright**
REVIEWED BY:	**SPM**

% GRAVEL	**46.8**	USCS	**SW-SM**
% SAND	**46.8**	MOA FC	**NFS**
% SILT/CLAY	**6.5**	% PASS. 0.02 mm	**1.9**
% MOIST. CONTENT	**3.3**	% PASS. 0.002 mm	**N/A**
UNIFORMITY COEFFICIENT (C_u)	**39.4**		
COEFFICIENT OF GRADATION (C_c)	**1.0**		
ASTM D1557 (uncorrected)	**N/A**		
ASTM D4718 (corrected)	**N/A**		
OPTIMUM MOIST. CONTENT. (corrected)	**N/A**		

PARTICLE SIZE ANALYSIS ASTM D7928 / C136



COBBLES	GRAVEL		SAND			SILT or CLAY
	Coarse	Fine	Coarse	Medium	Fine	

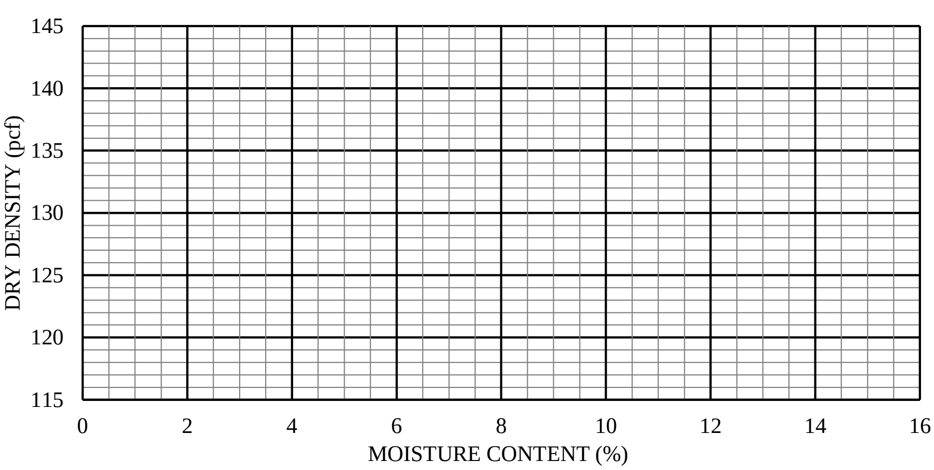
SIEVE ANALYSIS RESULT

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"	**88**	
12.70	1/2"	**78**	
9.50	3/8"	**71**	
4.75	#4	**53**	
2.00	#10	**38**	
0.85	#20	**28**	
0.43	#40	**20**	
0.25	#60	**13**	
0.15	#100	**9**	
0.075	#200	**6.5**	

HYDROMETER RESULT

ELAPSED TIME (MIN)	DIAMETER (mm)	TOTAL % PASSING
0		
1	0.0512	6.0
2	0.0368	3.9
5	0.0237	2.3
8	0.0187	1.8
15	0.0138	1.3
30		
60		
250		
1440		

MOISTURE-DENSITY RELATIONSHIP ASTM D1557



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

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NORTHERN GEOTECHNICAL ENGINEERING, INC. / TERRA FIRMA TESTING

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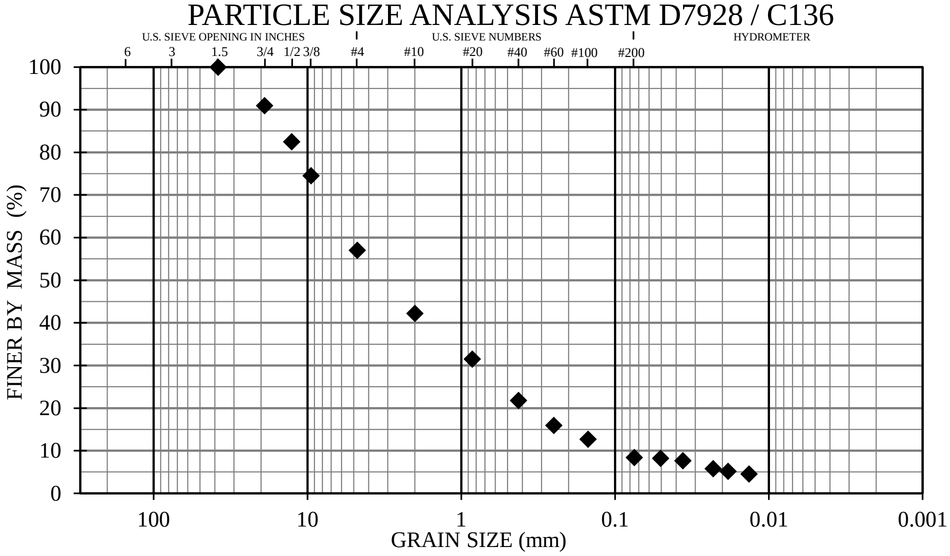
PROJECT CLIENT:	**ARRC**
PROJECT NAME:	**Christensen Street Repair**
PROJECT NO.:	**5136-18**
SAMPLE LOC.:	**B1**
NUMBER/ DEPTH:	**S2 / 2.5 - 4'**
DESCRIPTION:	**Well-graded sand w/ silt and gravel**
DATE RECEIVED:	**11/23/2021**
TESTED BY:	**Erik Boatwright**
REVIEWED BY:	**SPM**

% GRAVEL	**43.0**	USCS	**SW-SM**
% SAND	**48.6**	MOA FC	**F2**
% SILT/CLAY	**8.4**	% PASS. 0.02 mm	**5.3**
% MOIST. CONTENT	**3.6**	% PASS. 0.002 mm	**N/A**

UNIFORMITY COEFFICIENT (C_u)	**53.9**
COEFFICIENT OF GRADATION (C_c)	**1.1**
ASTM D1557 (uncorrected)	**N/A**
ASTM D4718 (corrected)	**N/A**
OPTIMUM MOIST. CONTENT. (corrected)	**N/A**

PARTICLE SIZE ANALYSIS ASTM D7928 / C136



COBBLES	GRAVEL		SAND			SILT or CLAY
	Coarse	Fine	Coarse	Medium	Fine	

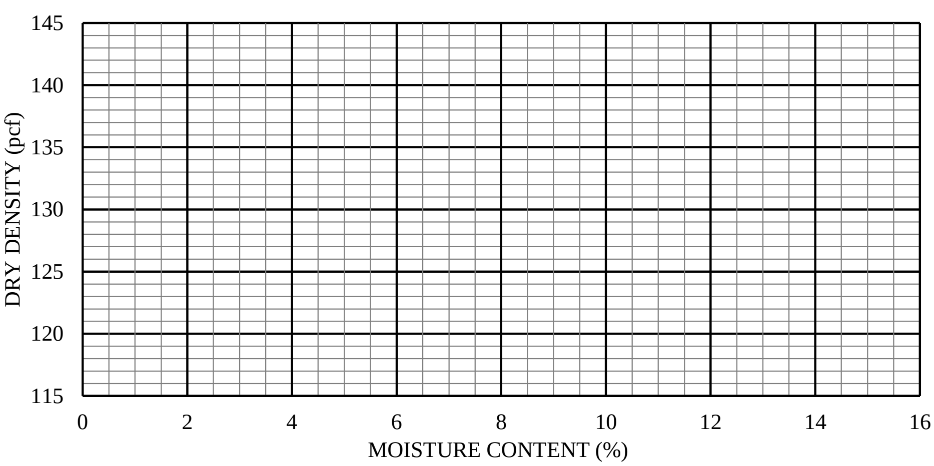
SIEVE ANALYSIS RESULT

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"	**91**	
12.70	1/2"	**83**	
9.50	3/8"	**74**	
4.75	#4	**57**	
2.00	#10	**42**	
0.85	#20	**31**	
0.43	#40	**22**	
0.25	#60	**16**	
0.15	#100	**13**	
0.075	#200	**8.4**	

HYDROMETER RESULT

ELAPSED TIME (MIN)	DIAMETER (mm)	TOTAL % PASSING
0		
1	0.0506	8.2
2	0.0362	7.6
5	0.0230	5.8
8	0.0184	5.2
15	0.0134	4.5
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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NORTHERN GEOTECHNICAL ENGINEERING, INC. / TERRA FIRMA TESTING

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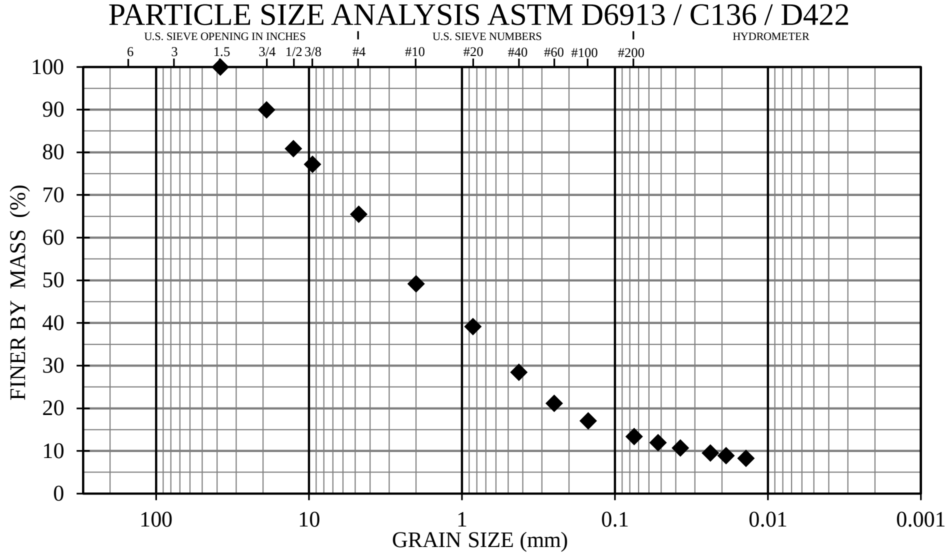
PROJECT CLIENT:	**ARRC**
PROJECT NAME:	**Christensen Street Repair**
PROJECT NO.:	**5136-18**
SAMPLE DESC.:	**B1-S3, 5-6.5'**
NGE-TFT ID #:	
CLASSIFICATION:	**Silty sand w/ gravel**
DATE RECEIVED:	**12/13/2021**
TESTED BY:	**Erik Boatwright**
REVIEWED BY:	**JSC**

% GRAVEL	**34.5**	USCS	**SM**
% SAND	**52.1**	USACOE FC	**F2**
% SILT/CLAY	**13.4**	% PASS. 0.02 mm	**9.0**
% MOIST. CONTENT	**4.6**	% PASS. 0.002 mm	**N/A**
UNIFORMITY COEFFICIENT (C_u)		**130.3**	
COEFFICIENT OF GRADATION (C_c)		**2.1**	
ASTM D1557 (uncorrected)		**N/A**	
ASTM D4718 (corrected)		**N/A**	
OPTIMUM MOIST. CONTENT. (corrected)		**N/A**	

PARTICLE SIZE ANALYSIS ASTM D6913 / C136 / D422



COBBLES	GRAVEL		SAND			SILT or CLAY
	Coarse	Fine	Coarse	Medium	Fine	

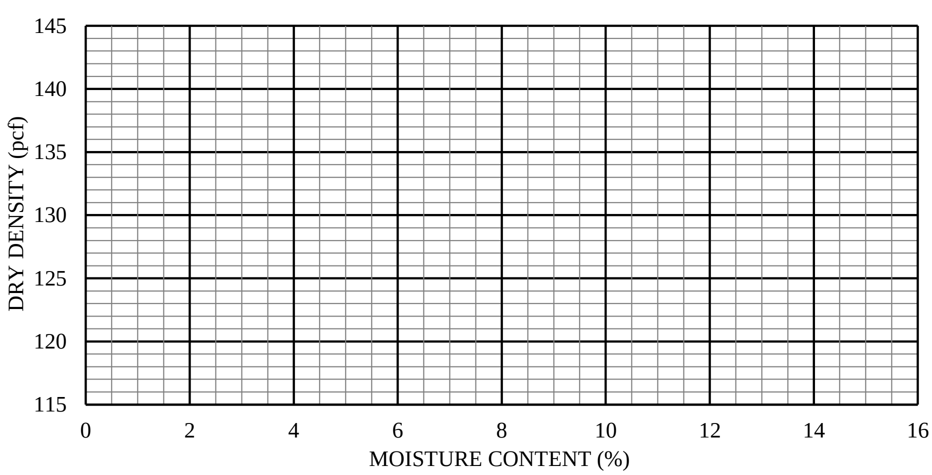
SIEVE ANALYSIS RESULT

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"	**90**	
12.70	1/2"	**81**	
9.50	3/8"	**77**	
4.75	#4	**65**	
2.00	#10	**49**	
0.85	#20	**39**	
0.43	#40	**28**	
0.25	#60	**21**	
0.15	#100	**17**	
0.075	#200	**13.4**	

HYDROMETER RESULT

ELAPSED TIME (MIN)	DIAMETER (mm)	TOTAL % PASSING
0		
1	0.0523	11.9
2	0.0374	10.7
5	0.0238	9.5
8	0.0188	8.9
15	0.0139	8.3
30		
60		
250		
1440		

MOISTURE-DENSITY RELATIONSHIP ASTM D1557



SOUNDNESS OF AGG. (ASTM C88)	**N/A**
DEGRADATION (ATM T-313)	**N/A**
LA ABRASION (ASTM C131/C535)	**N/A**
SP. GRAV. COARSE AGG. (ASTM C127)	**N/A**

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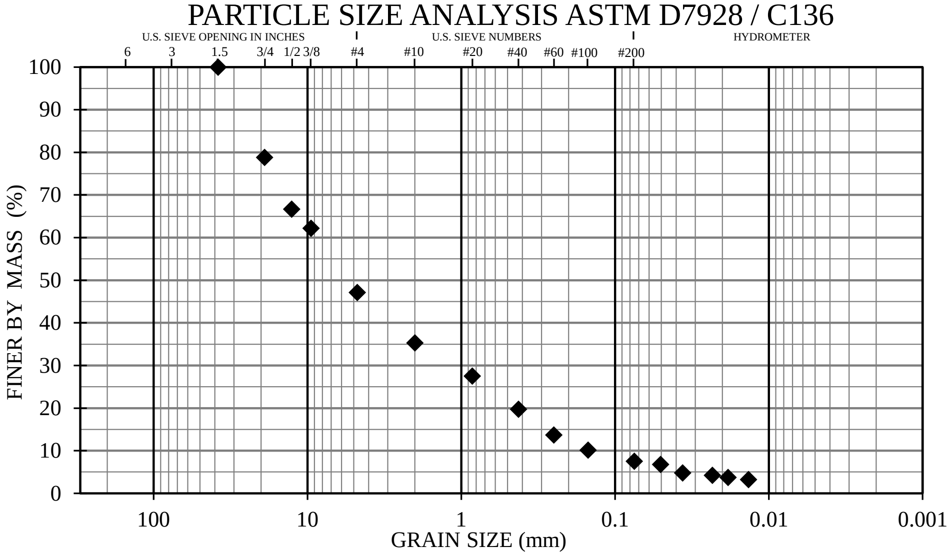
PROJECT CLIENT:	**ARRC**
PROJECT NAME:	**Christensen Street Repair**
PROJECT NO.:	**5136-18**
SAMPLE LOC.:	**B2**
NUMBER/ DEPTH:	**S1 / 0 - 1.5'**
DESCRIPTION:	**Well-graded gravel w/ silt and sand**
DATE RECEIVED:	**11/23/2021**
TESTED BY:	**Erik Boatwright**
REVIEWED BY:	**SPM**

% GRAVEL	**52.9**	USCS	**GW-GM**
% SAND	**39.6**	MOA FC	**F1**
% SILT/CLAY	**7.5**	% PASS. 0.02 mm	**3.9**
% MOIST. CONTENT	**3.5**	% PASS. 0.002 mm	**N/A**

UNIFORMITY COEFFICIENT (C_u)	**60.2**
COEFFICIENT OF GRADATION (C_c)	**1.2**
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	

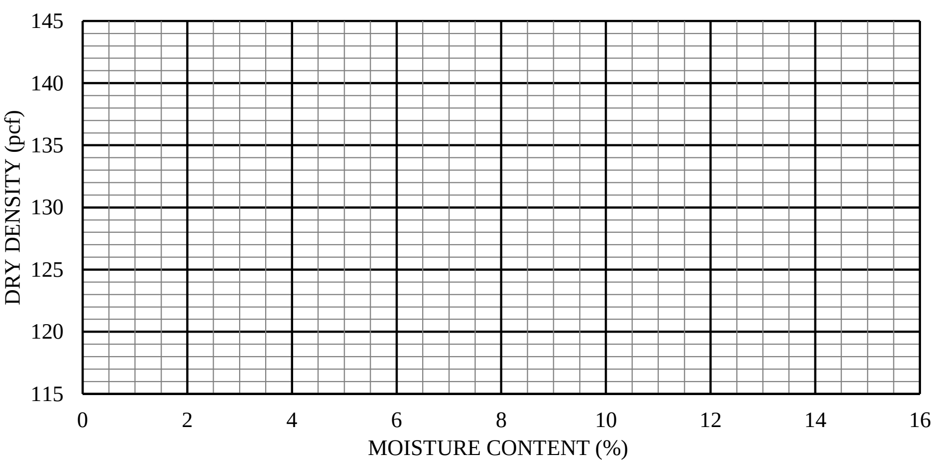
SIEVE ANALYSIS RESULT

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"	**79**	
12.70	1/2"	**67**	
9.50	3/8"	**62**	
4.75	#4	**47**	
2.00	#10	**35**	
0.85	#20	**27**	
0.43	#40	**20**	
0.25	#60	**14**	
0.15	#100	**10**	
0.075	#200	**7.5**	

HYDROMETER RESULT

ELAPSED TIME (MIN)	DIAMETER (mm)	TOTAL % PASSING
0		
1	0.0506	6.8
2	0.0364	4.8
5	0.0233	4.2
8	0.0184	3.7
15	0.0135	3.2
30		
60		
250		
1440		

MOISTURE-DENSITY RELATIONSHIP ASTM D1557



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

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