GEOTECHNICAL ENGINEERING EXPLORATION
KAILUA ROAD INTERSECTION IMPROVEMENTS
VICINITY OF ULUOA STREET AND ULUMANU DRIVE
KAILUA, OAHU, HAWAII

SECTION 1.0 INTRODUCTION

We have performed a geotechnical engineering exploration for the *Kailua Road Intersection Improvements* project in Kailua on the Island of Oahu, Hawaii. The location of the project and general vicinity are shown on the Project Location Map, Plate 1.

The purpose of our exploration was to observe and evaluate the general subsurface conditions at accessible locations at the project site to formulate geotechnical recommendations to assist in the design of the project. This report summarizes the findings and presents our geotechnical recommendations resulting from our site reconnaissance, field exploration, laboratory testing, and engineering analyses for the project. The findings and recommendations presented herein are subject to the limitations noted at the end of this report.

1.1 PROJECT CONSIDERATIONS

The project generally involves traffic improvements at two intersections along Kailua Road in Kailua on the Island of Oahu, Hawaii. In general, the planned intersection improvements along Kailua Road are at Uluoa Street and Ulumanu Drive and generally involve the installation of new traffic signal systems. Layouts of the project sites are shown on the Site Plans, Plates 2.1 and 2.2.

Based on the information provided, we understand the new traffic signal systems will generally consist of traffic signals, underground ducts, poles, foundations, interconnect, controller hardware and other appurtenant equipment, such as CCTV camera(s). It is our understanding that Type 1 traffic signal structures and Type 2 traffic signal structures with 30-foot mast arms are planned for the intersections. The project structural engineer provided the following preliminary structural loading information for the foundation design analyses of the new traffic signal pole structures.

PRELIMINARY STRUCTURAL LOADING INFORMATION	
Type 1 Traffic Signal Structure	
Axial Load (y-direction)	0.2 kips
Horizontal Load (z-direction)	1 kips
Overturning Moment (z-direction)	6 kip-feet
Torsion (z-direction)	0.7 lb-feet
Type 2 Traffic Signal Structure with 30-foot Mast Arm	
Axial Load (y-direction)	3 kips
Horizontal Load (z-direction)	6 kips
Overturning Moment (z-direction)	22 kip-feet
Overturning Moment (x-direction)	99 kip-feet
Torsion (z-direction)	67 kip-feet

1.2 PURPOSE AND SCOPE OF WORK

The purpose of our services was to generally explore and evaluate the subsurface soil conditions at accessible locations at the project site to provide geotechnical recommendations to assist in the design of the project. The work was performed in general accordance with our fee proposal dated November 10, 2022. The scope of work for this exploration included the following items:

1. Research and review of available in-house soils boring data and other information for the project.
2. Coordination of boring stake-out and utility clearances at the proposed boring locations by our field engineer.
3. Mobilization and demobilization of a truck-mounted drill rig and two operators to the project site and back.
4. Drilling and sampling of four boreholes extending to depths ranging from about 8.5 to 21.5 feet below the existing ground surface.
5. Coordination of the field exploration and logging of the borings by our field engineer.

6. Laboratory testing of selected soil samples obtained during the field exploration as an aid in classifying the materials and evaluating their engineering properties.

7. Analyses of the field and laboratory data to formulate geotechnical recommendations to assist in the design of the project.

8. Preparation of this report summarizing our work on the project and presenting our findings and recommendations.

9. Coordination of our overall work on the project by our senior engineer.

10. Quality assurance and client/design team consultation by our principal engineer.

11. Miscellaneous work efforts such as drafting, word processing, and clerical support.

Detailed descriptions of our field exploration methodology are presented in the following section and the Logs of Borings are presented in Appendix A. Results of the laboratory tests performed are presented in Appendix B.

END OF INTRODUCTION

SECTION 2.0 SITE CHARACTERIZATION AND FINDINGS

2.1 GENERAL SITE GEOLOGY

The project site is generally located on the southeastern flank of the Koolau Volcano on the Island of Oahu. Based on the geologic maps of the Island of Oahu (Stearns, 1939 and Sherrod and others, 2007), the general area of the project site is underlain by Honolulu Volcanics Training School Lava Flows (Qol) and Older Alluvium (QTao). In general, the rocks associated with Qol are generally characterized by flows of jointed, dense vesicular basalt with interbedded thin clinker layers. In-situ weathering of these lava flows has occurred, forming a mantle of residual and saprolitic soils overlying the top of the basalt rock formation.

In general, saprolite is composed mainly of silty material that may exhibit a relict structure (vesicles, joints, etc.) from its parent rock, while residual soil tends to be more clayey and is usually “structureless.” Both residual and saprolitic soils are typical of the tropical weathering of volcanic rocks. The residual and saprolitic soils grade to basaltic rock formation with increased depth.

Erosional processes in the mountainous regions of Hawaii are dominated by the detachment of soil and rock masses from the valley walls which are transported downslope toward the axis of the valley primarily by gravity as colluvium. Colluvial accumulations often consist of material that is generally deposited by gravity fall, rain wash and mudflow. Once these materials reach the stream in the central portion of the valley, alluvial processes become dominant, and the sediments are transported and deposited as alluvium.

In general, all the big valleys in Hawaii have flat floors built by alluvium deposited by their streams. Stream flows in Hawaii are intermittent and flashy, such that the stream flows transmit large volumes of water for very short duration. Because of this, transport of sediments is intermittent, and the bulk of the stream's hydraulic load consists of a poorly-sorted mixture of boulders, cobbles, gravel, sands, and fines. When the erosional base levels change, these sediment loads are left as deposits.

The surface soils underlying the project site are classified as Pohakupu Silty Clay Loam (PkB) by the U.S. Soil Conservation Service in their publication "Soil Survey of Islands of Kauai, Oahu, Maui, Molokai and Lanai, State of Hawaii" (1972). The PkB soil type is described as dark reddish brown, sticky, plastic clay that formed in old alluvium derived from basic igneous rock. PkB is also described as having a moderate shrink-swell potential and a high shear strength. Mass grading work and development along Kailua Road have brought the project site to its present form.

2.2 SITE DESCRIPTION

The project site is located along Kailua Road at the intersections with Uluoa Street and Ulumanu Drive in Kailua on the Island of Oahu, Hawaii. In general, Kailua Road at these intersections is an existing four-lane divided roadway with two lanes in each direction along with several turning lanes, center median, and pedestrian walkways.

Based on our field observations, the project site appears to generally slope down from the southwest to the northeast. A topographic survey plan was not provided at the time this report was prepared. Based on Google Earth imagery, we anticipate existing ground surface elevations at the project site to range from about +104 to +90 feet Mean Sea Level (MSL) at the southwestern and northeastern portions of the project site, respectively.

At the time of our field exploration, portions of the site were generally covered by asphaltic concrete pavements, concrete walkways, and mown lawn grass. Exposed surface soils at the project site generally consisted of brown silty clay with varying amounts of sand and gravel.

2.3 FIELD EXPLORATION

We explored the subsurface conditions at the project site by drilling and sampling four borings, designated as Boring Nos. 1 through 4, extending to depths ranging from about 8.5 to 21.5 feet below the existing ground surface. The borings were drilled utilizing a truck-mounted drill rig equipped with continuous flight augers. The approximate boring locations are shown on the Site Plans, Plates 2.1 and 2.2.

Our engineer monitored the drilling operations on a near continuous (full-time) basis and classified the materials encountered in the borings by visual and textural examination in the field in general accordance with ASTM D2488. These classifications were further reviewed visually and by testing in the laboratory. Soils were classified in general accordance with ASTM D2487 and the Unified Soil Classification System.

Soil samples were obtained in general accordance with ASTM D1586 by driving a 2-inch OD standard penetration sampler with a 140-pound hammer falling 30 inches. In addition, relatively undisturbed soil samples were obtained in general accordance with ASTM D3550 by driving a 3-inch OD Modified California sampler using the same hammer and drop. The blow counts needed to drive the sampler the second and third 6 inches of an 18-inch drive are shown as the "Sampling Resistance" on the Logs of Borings at the appropriate sample depths. The blow counts may need to be factored to obtain the Standard Penetration Test (SPT) blow counts.

Pocket penetrometer tests were performed on selected cohesive soil samples retrieved in the field. The pocket penetrometer test provides an indication of the unconfined compressive strength of the sample. Pocket penetrometer test results are summarized on the Logs of Borings at the appropriate sample depths.

2.4 SUBSURFACE CONDITIONS

Our borings generally encountered pavement structures consisting of about 8 inches of asphaltic concrete and 4 inches of base material overlying surface fill materials, alluvial soils, and hard basalt rock formation extending down to the maximum depth explored of about 21.5 feet below the existing ground surface. The surface fill materials were encountered to depths ranging from about 2 to 5 feet below the existing ground surface and generally consisted of stiff to very stiff silty clay and medium dense silty sand with some gravel.

Alluvial soils were encountered underlying the surface fill materials to depths ranging from about 12 feet to the maximum depth explored of about 21.5 feet below the existing ground surface in Boring No. 1 and generally consisted of stiff to hard silty clay with some sand and gravel. In addition, boulders were encountered within the alluvial soil deposits in Boring No. 2 at depths of about 2 and 6 feet below the existing ground surface.

Hard basalt rock formation was encountered underlying the alluvial soils in Boring Nos. 3 and 4 only, and extended down to the maximum depth explored in these boring of about 15.3 feet. We did not encounter groundwater in the borings at the time of our field exploration. However, it should be noted that groundwater levels are subject to change due to rainfall, time of year, seasonal precipitation, surface water runoff, and other factors. Graphic representations of the materials encountered are presented on the Logs of Borings, Appendix A.

2.5 LABORATORY TESTING

Moisture Content (ASTM D2216) and Unit Weight (ASTM D2937) determinations were performed on selected samples as an aid in the classification and evaluation of soil properties. The test results are presented on the Logs of Borings at the appropriate sample depths.

Two Atterberg Limits tests (ASTM D4318) were performed on selected soil samples to evaluate the liquid and plastic limits. The samples tested generally had high Plasticity Indexes (PIs) of about 28 and 32 and plotted as high plasticity clay (CH) on a Standard Plasticity Chart. The test results are summarized on the Logs of Borings at the appropriate sample depths. Graphic presentations of the Atterberg Limits test results are provided on Plate B-1.

Two one-inch Ring Swell tests were performed on relatively undisturbed (natural) and remolded samples to evaluate the swelling potential of the on-site soils. Swell test results of about 2.1 and 8.9 percent were observed for the relatively undisturbed (natural) and remolded samples, respectively, indicating the on-site soils have a moderately high to high swelling potential when subjected to moisture fluctuations. The Ring Swell test results are summarized on Plate B-2.

One Unconfined Compression test (ASTM D2166) was performed on a selected in-situ cohesive soil sample to evaluate the unconfined compressive strength of the soil. The test resulted in an unconfined compressive strength of about 1.8 kips per square foot (ksf). The Unconfined Compression test results are presented on Plate B-3.

2.6 SEISMIC DESIGN CONSIDERATIONS

Based on the International Building Code, 2018 Edition (IBC 2018) and American Society of Civil Engineers Standard ASCE/SEI 7-16 (ASCE 7-16), the project site may be subject to seismic activity, and seismic design considerations will need to be addressed. Based on the subsurface materials encountered at the project site and the geologic setting of the area, we anticipate the project site may be classified from a seismic analysis standpoint as being a “Stiff Soil Profile” site corresponding to a Site Class D soil profile type based on Chapter 20 of ASCE 7-16.

Based on Site Class D, the following seismic design parameters were estimated and may be used for seismic analysis of the project.

SUMMARY OF SEISMIC DESIGN PARAMETERS	
Mapped MCE Spectral Response Acceleration, S_S	0.552g
Mapped MCE Spectral Response Acceleration, S_1	0.155g
Site Class	D
Site Coefficient, F_a	1.358
Site Coefficient, F_v	2.291
Design Spectral Response Acceleration, S_{DS}	0.500g
Design Spectral Response Acceleration, S_{D1}	0.236g
Peak Ground Acceleration, PGA	0.254g
Site Modified Peak Ground Acceleration, PGA_M	0.342g

Based on the subsurface conditions encountered, the phenomenon of soil liquefaction is not a design consideration for this project site.

END OF SITE CHARACTERIZATION AND FINDINGS

SECTION 3.0 DISCUSSION AND RECOMMENDATIONS

Based on the results of our field exploration, the project site is generally underlain by pavement structures consisting of about 8 inches of asphaltic concrete and 4 inches of base material overlying surface fill materials, alluvial soils, and hard basalt rock formation extending down to the maximum depth explored of about 21.5 feet below the existing ground surface. The surface fill materials were encountered to depths ranging from about 2 to 5 feet below the existing ground surface and generally consisted of stiff to very stiff silty clay and medium dense silty sand with some gravel.

Alluvial soils were encountered underlying the surface fill materials to depths ranging from about 12 feet to the maximum depth explored of about 21.5 feet below the existing ground surface in Boring No. 1 and generally consisted of stiff to hard silty clay with some sand and gravel. In addition, boulders were encountered within the alluvial soil deposits in Boring No. 2 at depths of about 2 and 6 feet below the existing ground surface.

Hard basalt rock formation was encountered underlying the alluvial soils in Boring Nos. 3 and 4 only, and extended down to the maximum depth explored in these boring of about 15.3 feet. We did not encounter groundwater in the borings at the time of our field exploration. However, it should be noted that groundwater levels are subject to change due to rainfall, time of year, seasonal precipitation, surface water runoff, and other factors. In addition, subterranean seepage may be encountered during construction due to high rainfall in the area, sloping terrain and relict structure in the alluvial soils and basalt rock formation encountered.

Based on the loading demands provided by the project structural engineer and the subsurface soil conditions encountered at the site, we believe the new traffic signal structures may be supported by a deep foundation system consisting of cast-in-place concrete drilled shafts. Detailed discussion of these items and our geotechnical recommendations for design of drilled shaft foundations and other geotechnical aspects of the project are further discussed in the following sections.

3.1 DRILLED SHAFT FOUNDATIONS

In order to develop the required bearing and lateral load resistances, we believe the new traffic signal structures may be supported by a deep foundation system consisting of cast-in-place concrete drilled shafts. In general, drilled shaft foundations are constructed by drilling a hole down into the bearing strata, placing reinforcing steel, and then pumping high slump concrete to fill up the hole. The result is a cast-in-place concrete drilled shaft for foundation support.

Based on the bolt circle diameter and the square bearing plate dimensions, we envision that drilled shaft foundations with minimum diameters of 24 and 42 inches may be used to support the new Type 1 and Type 2 traffic signal structures, respectively. Detailed discussions and recommendations for foundation design are presented in the following subsections.

3.1.1 LATERAL LOAD RESISTANCE

The lateral load resistance of drilled shafts is a function of the stiffness of the surrounding soil, the stiffness of the drilled shaft, allowable deflection at the top of the drilled shaft, and the induced moment in the drilled shaft. The lateral load analyses were performed using the program LPILE, which is a microcomputer adaptation of a finite difference, laterally loaded deep foundation program originally developed at the University of Texas at Austin.

The program solves for deflection and bending moment along a deep foundation under lateral loads as a function of depth. The analysis was carried out with the use of non-linear "p-y" curves to represent soil moduli. The lateral deflection was then computed using the appropriate soil moduli at various depths.

Based on the provided preliminary structural loads and the anticipated subsurface soil conditions at each site, we performed the lateral load analyses for the above drilled shaft foundations. The results of our analyses are summarized in the tables below.

In general, we recommend installing 24-inch diameter drilled shaft foundations with minimum embedment lengths of 6 feet below the design finished grade to support the new Type 1 traffic signal structures planned for the project. In addition, we recommend installing 42-inch diameter drilled shaft foundations with minimum embedment lengths of 12 feet below the design finished grade to support the new Type 2 traffic signal structures planned for the project. The project structural engineer should verify the drilled shaft structural capacity for the calculated induced stresses.

FOUNDATION ANALYSES FOR TYPE 1 TRAFFIC SIGNAL POLE STRUCTURE				
Minimum Drilled Shaft Diameter (inches)	**Minimum Drilled Shaft Length** (feet)	**Lateral Deflection** (inches)	**Maximum Induced Moment** (kip-foot)	**Depth to Maximum Moment** (feet)
24	6	0.1	6.4	1.6

FOUNDATION ANALYSES FOR TYPE 2 TRAFFIC SIGNAL POLE STRUCTURE WITH 30-FOOT MAST ARM				
Minimum Drilled Shaft Diameter (inches)	**Minimum Drilled Shaft Length** (feet)	**Lateral Deflection** (inches)	**Maximum Induced Moment** (kip-foot)	**Depth to Maximum Moment** (feet)
42	12	0.3	108.3	3.5

3.1.2 FOUNDATION SETTLEMENT

Settlement of the drilled shaft foundations will result primarily from elastic compression of the shaft member and subgrade response. We estimate the total settlement of the drilled shaft supported foundations to be 0.5 inches or less with differential settlements between drilled shafts not exceeding about one half of the total settlement. We believe these settlements are essentially elastic and should occur as the loads are applied.

3.1.3 DRILLED SHAFT CONSTRUCTION CONSIDERATIONS

In general, the performance of drilled shafts depends significantly upon the contractor's method of installation and construction procedures. The load bearing capacities of the drilled shafts depend, to a significant extent, on the friction between the shaft and the surrounding soils. Therefore, proper construction techniques, especially during the drilling operations, are important. The contractor should exercise care in drilling the shaft holes and placing concrete into the drilled holes.

It should be noted that cobbles, boulders, and hard basalt rock formation may be encountered within the depth of the drilled shafts. Therefore, some difficult drilling conditions may be expected. The drilled shaft subcontractor should have the appropriate equipment and tools to drill through these types of natural obstructions, where encountered. The drilled shaft subcontractor should demonstrate that the proposed drilling equipment will be capable of installing the drilled shafts to the recommended depths and dimensions.

Based on the estimated lengths of the drilled shafts, groundwater is generally not expected in the drilled hole during the shaft installation. However, concrete placement by tremie method is recommended during construction of the drilled shafts in lieu of free fall method. This is to reduce the potential for concrete from striking the steel reinforcement cage or shaft sidewalls during concrete placement.

A low-shrink concrete mix with high slump (7 to 9-inch slump range) should be used to provide close contact between the drilled shaft and the surrounding soils. Due to factors such as seasonal rainfall and perched water, groundwater may be encountered in the drilled hole. The concrete should be placed in a suitable manner to reduce the potential for segregation of the aggregates from the concrete mix. In addition, the concrete should be placed promptly after drilling (within 24 hours after drilling of the holes) to reduce the potential for softening of the sides of the drilled hole.

We recommend a specialty contractor experienced in the construction of drilled shaft foundations (minimum five projects) perform the installation of the drilled shafts. Due to the specialized nature of the drilled shaft foundation construction, observation and testing of the drilled shaft foundation system should be designated as a "Special Inspection" item. Therefore, a Kokua Geotech LLC representative (Special Inspector) should be present to observe the geotechnical aspects of the drilled shaft foundation construction.

3.2 UTILITY TRENCHES

We anticipate that new underground utility lines may be required for the project. As discussed above, all excavations should be made in accordance with applicable Occupational Safety and Health Administration (OSHA) and state regulations. The contractor should determine the method and equipment to be used for utility trench excavation, subject to practical limits and safety considerations. In addition, the trench excavations should comply with the applicable federal, state, and local safety requirements. The contractor should be responsible for trench shoring design and installation.

Based on our borings, trench excavations will likely encounter surficial fill materials and alluvial soils generally consisting of stiff to hard silty clay and medium dense silty sand with some gravel. In addition, cobbles, boulders, and basalt rock formation may be encountered within the depth of the excavations. It is anticipated that most of the material may be excavated with normal heavy excavation equipment. However, deep excavations and excavations encountering boulders may require the use of hoerams.

In general, we recommend providing granular bedding consisting of 6 inches of open-graded gravel, such as No. 3 Fine gravel (ASTM C33, No. 67 gradation), under the pipes for uniform support. In addition, open-graded gravel (ASTM C33, No. 67 gradation) should also be used for the initial trench backfill up to about 12 inches above the pipes to provide adequate support around the pipes. It is critical to use a free-draining material, such as open-graded gravel, to reduce the potential for formation of voids below the haunches of pipes and to provide adequate support for the sides of the pipes. Improper trench backfill could result in backfill settlement and pipe damage.

Trench subgrades should firm and unyielding prior to placing the minimum 6-inch thick layer of open-graded gravel below the pipes. Soft and/or loose materials encountered at the bottom of trench excavations should be over-excavated to expose the underlying firm materials. The over-excavation should be backfilled with general fill materials compacted to a minimum of 90 percent relative compaction. Before the placement of bedding material, a Kokua Geotech LLC representative should observe the excavated trench bottom to confirm that firm materials are exposed at the bottom of the trench.

Trench backfill material above the open-graded gravel may consist of general fill materials (on-site soils with rock fragments less than 3 inches in largest dimension). The backfill should be placed in maximum 8-inch level loose lifts and mechanically compacted to no less than 90 percent relative compaction to reduce the potential for appreciable future ground subsidence. The upper 2 feet below the finished grade in areas subjected to vehicular traffic should be compacted to a minimum of 95 percent relative compaction.

Relative compaction refers to the in-place dry density of soil expressed as a percentage of the maximum dry density of the same soil determined in accordance with ASTM D1557. Optimum moisture is the water content (percentage by dry weight) corresponding to the maximum dry density.

3.3 SITE DRAINAGE CONSIDERATIONS

The drainage condition around the new traffic signal structures is critical to maintaining proper foundation performance because ponded water could cause subsurface soil saturation and subsequent heaving or loss of strength. Finished grades outside the new structures should be sloped to shed water away from the slabs and foundations and to reduce the potential for ponding around the structure.

Drainage systems and finished grades for the project site should be designed by a Licensed Civil Engineer so that surface runoff is directed away from the structures. Drainage swales should be provided as soon as possible and should be maintained to drain surface water runoff away from the foundations.

3.4 DESIGN REVIEW AND CONSTRUCTION OBSERVATION SERVICES

The construction plans and specifications for the project should be forwarded to us for review to determine whether the recommendations contained in this report are adequately reflected in those documents. If this review is not made, Kokua Geotech LLC cannot assume responsibility for misinterpretation of our recommendations.

Kokua Geotech LLC should also be retained to monitor the drilled shaft foundation installation operations and other aspects of earthwork construction to determine whether the recommendations of this report are followed. The recommendations presented herein are contingent upon such observations. If the actual exposed subsurface soil conditions encountered during construction differ from those assumed or considered in this report, Kokua Geotech LLC should be contacted to review and/or revise the geotechnical recommendations presented herein.

END OF DISCUSSION AND RECOMMENDATIONS

SECTION 4.0 LIMITATIONS

This report has been prepared for the exclusive use of Community Planning and Engineering, Inc. and their project consultants for specific application to the design of the *Kailua Road Intersection Improvements, Vicinity of Uluoa Street and Ulumanu Drive* project in accordance with generally accepted geotechnical engineering principles and practices. No warranty is expressed or implied. If any part of the project concept is altered or if subsurface conditions differ from those described in this report, then the information presented herein shall be considered invalid, unless the changes are reviewed, and any supplemental or revised recommendations issued in writing by Kokua Geotech LLC.

The analyses and report recommendations are based in part upon information obtained from the field borings and the assumption that subsurface conditions do not vary significantly from those observed in the borings. Variations of the subsurface conditions between and beyond the field borings may occur, and the nature and extent of these variations may not become evident until construction is underway. If variations then appear evident, Kokua Geotech LLC should be notified so that we can re-evaluate the recommendations presented herein.

The owner/client should be aware that unanticipated soil conditions are commonly encountered. Unforeseen subsurface conditions, such as perched groundwater, soft deposits, hard layers or cavities, may occur in localized areas and may require additional probing or corrections in the field (which may result in construction delays) to attain a properly constructed project. Therefore, a sufficient contingency fund is recommended to accommodate these possible extra costs.

The field boring locations indicated herein are approximate, having been estimated by taping from visible features shown on the Traffic Signal Plan transmitted by Community Planning and Engineering, Inc. on October 6, 2023. A topographic survey plan was not provided at the time this report was prepared. Elevations of the borings were estimated from Google Earth imagery.

SECTION 4.0 LIMITATIONS

The field boring locations and elevations should be considered accurate only to the degree implied by the methods used.

The stratification breaks shown on the graphic representations of the borings depict the approximate boundaries between soil types and, as such, may denote a gradual transition. We did not encounter groundwater in the borings at the time of our field exploration. However, it should be noted that groundwater levels are subject to change due to rainfall, time of year, seasonal precipitation, surface water runoff, and other factors. These data have been reviewed and interpretations made in the formulation of this report.

This report has been prepared solely for the purpose of assisting the design engineers in the design of the project. Therefore, this report may not contain sufficient data, or the proper information, to serve as a basis for detailed construction cost estimates.

This geotechnical engineering exploration conducted at the project site was not intended to investigate the potential presence of hazardous materials existing at the project site. It should be noted that the equipment, techniques, and personnel used to conduct a geo-environmental exploration differ substantially from those applied in geotechnical engineering.

END OF LIMITATIONS