Exhibit Y. Lake Charles Regional Airport Site Preliminary Geotechnical Engineering Report

SWLA ECONOMIC DEVELOPMENT ALLIANCE

CSRS

IMAGINE SHAPE DELIVER

SITE ENGINEERING, INC.
SELECTION ▪ INVESTIGATION ▪ TESTING ▪ ENGINEERING
650 ALBERTSON PARKWAY ▪ BROUSSARD, LA 70518
PHONE 337-981-1414
FAX 337-981-1402

Lake Charles Regional Airport Site Preliminary Geotechnical Engineering Report

March 27, 2018

Mr. George Swift
SWLA Economic Development Alliance
4310 Ryan Street
Lake Charles, Louisiana 70605

RE: **Preliminary Geotechnical Engineering Investigation**
Proposed Lake Charles Regional Airport Site
Gulf Highway
Calcasieu Parish, Louisiana
SITE Engineering Project: 18-G020-01

Dear Mr. Swift:

SITE Engineering, Inc. is pleased to transmit our Preliminary Geotechnical Engineering Investigation report for the above referenced project. This investigation was performed in general accordance with SITE Engineering Proposal Number 17-257G dated November 14, 2017. Authorization to proceed with the investigation was provided by Mr. George Swift, President of SWLA Economic Development Alliance, on February 20, 2018 by signing our proposal.

The purpose of this exploration was to investigate and analyze the existing subsurface conditions at the site to enable a general evaluation of foundation and pavement systems for the proposed project. This report includes the results of our field and laboratory testing and presents preliminary recommendations for site preparation, foundation and pavement design, and construction considerations.

We appreciate the opportunity to provide our services to your project and look forward to working with you in the future. If you have any questions pertaining to this report, or if we may be of further service, please do not hesitate to contact our office.

Sincerely,
SITE ENGINEERING, INC.

Clint S. McDowell, P.E.
President

Distribution: 3 – Above

SITE ENGINEERING, INC.

GEOTECHNICAL ENGINEERING SERVICES REPORT

PROPOSED LAKE CHARLES REGIONAL AIRPORT SITE
GULF HIGHWAY
CALCASIEU PARISH, LOUISIANA

SITE ENGINEERING REPORT NUMBER: 18-G020-01

Prepared For

Mr. George Swift
SWLA Economic Development Alliance
4310 Ryan Street
Lake Charles, Louisiana 70605

March 27, 2018

By

SITE ENGINEERING, INC.
650 Albertson Parkway
Broussard, Louisiana 70518
(337) 981-1414

Jarod J. Breaux, P.E. (#39061)
Project Engineer

Clint S. McDowell, P.E. (#27983)
President

TABLE OF CONTENTS

1.0 EXECUTIVE SUMMARY

SITE Engineering, Inc. has completed a preliminary exploration and evaluation of the subsurface conditions at the proposed Lake Charles Regional Airport site located on Gulf Highway in Calcasieu Parish, Louisiana. The project will consist of a commercial/industrial development encompassing approximately 156 acres. It should be noted that the project is in the extreme early stages of development and the actual types, sizes and locations of any proposed infrastructure have not been provided. Therefore, the recommendations provided in this report should be considered preliminary and general in nature. For final recommendations to be provided, additional borings will need to be performed.

The subsurface conditions were explored by the performance of soil test borings. As requested, our scope of services included drilling three (3) borings extending to depths ranging from 25 to 100 feet below the existing ground surface. The borings generally encountered approximately 8 to 10 inches of highly organic silty clay and lean clay topsoil followed by very stiff to stiff fat clay soils to depths ranging from 8 to 17 feet. These fat clays were underlain by stiff to soft lean clays, silty clays, and silty clayey sands to the boring completion depth of 25 feet within borings B-2 and B-3 and to a depth of about 17 feet within boring B-1. Below this depth, boring B-1 generally encountered stiff to very stiff fat clay soils with intermittent layers of firm to stiff lean clay soils to a depth of about 97 feet followed by clayey sand extending to a depth of at least 100 feet, the maximum depth explored.

Groundwater was initially encountered during the drilling operations at depths ranging from 13 to 15 feet below the exiting surface within the borings performed at this site. Immediately after drilling, the boreholes were plugged and abandoned. Therefore, subsequent delayed groundwater readings were not possible. The boring logs included in the appendix of this report should be reviewed for specific soil and groundwater information at each boring location.

The near surface soils encountered in the borings performed at this site are considered good in strength and support capabilities but are considered moderate to high in shrink/swell potential. As previously mentioned, site development information was not provided due to the extremely preliminary nature of this project. Therefore, this report will provide general recommendations for potential foundation types including shallow foundation systems such as typical spread and continuous footings as well as deep foundation systems such as drilled cast-in-place concrete shafts and driven timber piles.

General recommendations are also being provided for various flexible and rigid pavement systems. Preliminary recommendations and details related to site development, foundation and pavement design, and construction considerations are included in subsequent sections of this report. Again, the recommendations provided within this report should be considered preliminary in nature due to the limited number of borings performed in relation to the size of the subject site. It should be noted that the soil characteristics within an isolated construction area may be drastically different than those provided in this report and should be determined with additional soil borings once specific project information is ascertained.

2.0 PROJECT INFORMATION

2.1 Project Authorization

SITE Engineering, Inc. has completed a preliminary geotechnical investigation at the proposed Lake Charles Regional Airport site located on Gulf Highway in Calcasieu Parish, Louisiana. This investigation was performed in general accordance with SITE Engineering Proposal Number 17-257G dated November 14, 2017. Authorization to proceed with the investigation was provided by Mr. George Swift, President of SWLA Economic Development Alliance, on February 20, 2018 by signing our proposal.

2.2 Project Description

The project will consist of a commercial/industrial development encompassing approximately 156 acres of currently undeveloped land. It should be noted that the project is in an extremely early stage of development and the actual types, sizes and locations of proposed infrastructure have not been provided. Therefore, the recommendations provided in this report should be considered preliminary in nature. For final recommendations to be provided, additional borings will need to be performed.

The preliminary recommendations presented in this report are based on the subsurface materials encountered in the limited number of borings performed. SITE Engineering will not be responsible for the implementation of the recommendations presented in this report if not given the opportunity to provide a thorough investigation once the development plans are more complete.

2.3 Purpose and Scope of Services

The purpose of this preliminary geotechnical investigation was to explore the subsurface conditions at the site to enable an evaluation of various foundation and pavement systems. As requested, our scope of services was limited to the drilling of three (3) soil test borings to depths ranging from 25 to 100 feet below the existing ground surface. Our services also included select laboratory testing of the sampled subsurface soils and preparation of this geotechnical report. This report briefly outlines the testing procedures, presents available project information, describes the site and subsurface conditions, and presents general recommendations regarding the following:

- Foundation design recommendations including recommended bearing depths and load bearing values for shallow foundation elements;
- Allowable compression and tension capacities for various deep foundation types;
- Estimates of settlements for the recommended foundation types and estimates of settlement due to the weight of any structural fill required above existing grade to reach design elevation;
- Recommendations for utility trenches and excavations;
- Recommendations for design and construction of both rigid and flexible pavement systems, and;
- Recommendations for general site preparation including organic and unstable soil removal and structural fill criteria and compaction requirements.

Our scope of services did not include an environmental assessment for determining the presence or absence of wetlands, or hazardous or toxic materials in the soil, surface water, groundwater, or air on or below, or around this site. Any statements in this report or on the boring logs regarding odors, colors, and unusual or suspicious items or conditions are strictly for informational purposes.

3.0 SITE AND SUBSURFACE CONDITIONS

3.1 Project Location and Site Description

The proposed Lake Charles Regional Airport site is located on Gulf Highway in Calcasieu Parish, Louisiana. At the time of drilling, the majority of subject site was grass covered. The surface of site was generally dry and in a firm condition. Our all-terrain drilling rig and support pick-up truck experienced little to no difficulty in accessing the boring locations.

Existing site topographic information was not provided. However, based on visual observations, the subject property appeared to be relatively level with little elevation difference between high and low points.

3.2 Subsurface Conditions

As requested, the subsurface conditions were explored with three (3) soil test borings drilled to depths ranging from 25 to 100 feet below the existing ground surface. The number and depth of the borings were determined by CSRS and SWLA Economic Development Alliance. The locations of the borings were determined by SITE Engineering, Inc. The borings were located on the subject site by a representative of SITE Engineering using a surveyor's wheel and based on an aerial photograph provided by SWLA Economic Development Alliance. The approximate location of each boring can be seen on the Boring Location Diagram included in the appendix.

The borings were advanced utilizing continuous flight auger and wet rotary drilling techniques. Soil samples were obtained continuously in the upper ten feet of the borings and on five-foot centers thereafter to the boring completion depths. Drilling and sampling methods were accomplished in general accordance with ASTM procedures. Upon completion of the drilling, the borings were plugged and abandoned in accordance with the regulations of the Louisiana Department of Natural Resources.

Undisturbed samples of cohesive soils were obtained using thin-wall tube sampling procedures in general accordance with the procedures for "Thin-Walled Tube Geotechnical Sampling of Soils" (ASTM D 1587). These samples were extruded in the field with a hydraulic ram. Undisturbed samples were identified according to boring number and depth, were placed in polyethylene plastic wrapping to protect against moisture loss, and were transported to the laboratory in special containers to prevent disturbance.

In addition to the field exploration, a supplemental laboratory-testing program was conducted to evaluate additional pertinent engineering characteristics of the subsurface materials necessary in analyzing the behavior of the foundation system for the proposed project. The laboratory-testing program included supplementary visual classification and water content tests on all soil samples. In addition, selected samples were subjected to unconfined compressive strength testing, Atterberg Limits determinations, and percent passing a number 200 sieve analysis. Additional estimates of shear strength were also determined through the use of a hand torvane and pocket penetrometer.

The borings generally encountered approximately 8 to 10 inches of silty clay and lean clay topsoil followed by very stiff to stiff fat clay soils to depths ranging from 8 to 17 feet. These fat clay soils were underlain by stiff to soft lean clays, silty clays, and silty clayey sands to the boring completion depth of 25 feet within borings B-2 and B-3 and to a depth of about 17 feet within boring B-1. Below this depth, boring B-1 generally encountered stiff to very stiff fat clay soils with intermittent layers of firm to stiff lean clay soils to a depth of about 97 feet followed by clayey sand to a depth of 100 feet, the maximum depth explored.

The above subsurface description is of a generalized nature to highlight the major subsurface stratification features and material characteristics. The boring logs included in the appendix should be reviewed for specific subsurface information at individual boring locations. These records include soil descriptions, stratifications, locations of the samples and laboratory test data. The stratifications shown on the boring logs represent the conditions only at the actual boring locations. Variations may occur and should be expected between boring locations and elsewhere on the site. The stratifications represent the approximate boundary between subsurface materials and the actual transition may be gradual. The samples which were not altered by laboratory testing will be retained for 60 days from the date of this report and then discarded.

3.3 Groundwater Information

Groundwater was initially encountered during the drilling operations at depths ranging from 13 to 15 feet below the exiting surface within the borings performed at this site. Immediately after drilling, the boreholes were plugged and abandoned. Therefore, subsequent delayed groundwater readings were not possible. The boring logs included in the appendix of this report should be reviewed for specific groundwater information at each boring location.

The groundwater information provided above and on the boring logs were the levels recorded at the time of our field investigation. In addition, it may take several days for the groundwater level to become static in an open borehole. Therefore, it should be noted, that it is possible for a groundwater table to fluctuate depending upon climatic and rainfall conditions. We recommend that the Contractor determine the actual groundwater levels at the site at the time of the construction activities.

4.0 EVALUATION AND RECOMMENDATIONS

4.1 General

The type and depth of foundation suitable for a given structure primarily depends on several factors including the subsurface conditions, the function of the structure, the loads it may carry, the cost of the foundation, and the criteria set by the Design Engineer with respect to vertical and differential movement which the structure can withstand without damage. The near surface soils encountered in the borings performed at this site are considered good in strength and support capabilities but are considered moderate to high in shrink/swell potential. Provided the site preparation recommendations presented in this report are followed, lightly-loaded structures at this site may be supported on relatively shallow foundation systems consisting of isolated spread footings, continuous wall footings, and grade beams.

However, if shallow foundation systems do not provide adequate support or tolerable settlements, deep foundation systems should be utilized. Therefore, we have also provided recommendations for drilled cast-in-place concrete shafts and driven timber piles as common and cost-effective deep foundation alternatives. Specific details related to foundation design and construction considerations will be presented in subsequent paragraphs.

Once again, the recommendations provided within this report should be considered preliminary in nature due to the limited number of borings performed in relation to the size of the subject site. It should be noted that the soil characteristics within an isolated construction area may be drastically different than those represented in this report and should be determined with additional soils borings once specific project development plans are completed.

4.2 Potential Volumetric Change

Field and laboratory test results indicate that soils which exhibit a moderate to high potential for expansion (swelling) were present in the borings performed at this site. Swelling of these soils will likely occur with changes in moisture content. The estimated amount of vertical movement of a foundation or floor slab constructed on swelling clays or expansive soils is referred to as the Potential Vertical Rise (PVR). To reduce the potential for swelling of the site soils, it is important that consideration be given to reducing the potential for moisture changes of these soils. At a minimum, positive drainage away from the new building should be provided. If positive drainage is not provided, water will pond around or below the structure and excessive total and differential movements may occur.

Movements may also be caused by shrinkage of the underlying clay soils due to evapo-transpiration which is the loss of water due to evaporation and transpiration during periods of dry weather. In periods where the loss of moisture in the soil is greater than precipitation, drying of the soils will occur. As the highly plastic clay soils dry, they will also shrink. The amount of drying and subsequent shrinkage is generally greater at the edge of a building and less towards the center, thereby causing undesirable differential movements.

The PVR was determined using the method developed by the American Association of State Highway Transportation Officials (AASHTO). This method assumes a linear variation with depth where the percent volumetric change is maximum at the ground surface and zero at the bottom of the active depth. The active depth in the project area was considered to be about six (6) feet.

A PVR value of about 2 inches was calculated for the soil conditions encountered in the borings. This magnitude of volume change is considered high and is generally considered intolerable for grade-supported building structures. Based on our analyses, it is recommended that at least 3 feet of low plasticity structural fill be placed beneath new building slabs on this site. Providing 3 feet of low plasticity structural fill beneath the new structures will decrease the estimated PVR values to one (1) inch or less.

The 3 feet of low plasticity soil should extend a minimum of 5 feet beyond the perimeter of the new buildings. The required thickness of low plasticity material can be achieved by undercutting and replacing the upper 36 inches of naturally occurring soils, by raising the building construction site at least 36 inches above existing grade with low plasticity structural fill, or by a combination of these alternatives. Greater thicknesses of low plasticity soil placed beneath any floor slab will further reduce the potential volumetric change. However, it should be noted that excessive settlement may be induced by significant thickness of fill placed above existing grade. Settlement estimates for various thicknesses of fill placed above existing grade are provided in subsequent sections of this report.

As previously mentioned, swelling or shrinkage occurs in soils due to changes in moisture content. Ponding of water around the slab may also result in reduction of soil strength, thereby causing movements. It is important to minimize the possibility of moisture content changes by considering the following precautions:

1. Direct surface runoff away from the structure by sloping the subgrade away from the slab.

2. Extend paving or other impervious covering, such as sidewalks, to the slab edges.

3. Extend downspouts so that the discharge is at least 5 feet from the slab.

4. Further increase fill thickness under the slab.

5. If shrubs or bushes are placed next to the structure, an impervious membrane should be used to separate the slab from the shrubs to limit any infiltration of water under the slab. The minimum distance between a tree and the building slab should be about one-half the expected height of the tree.

6. Provide adequate irrigation during periods of drought to minimize excessive drying of the subgrade soils around the perimeter of the building.

7. Extend perimeter footings deeper into the natural subgrade soils to provide a vertical barrier against moisture transferal

4.3 Site Preparation

We recommend that all topsoil, organics, and any soft, loose or deleterious soils in the areas intended for construction and for a distance of at least 5 feet beyond the perimeter of any building and 2 feet beyond the perimeter of the any pavement area be stripped from the site and either wasted or stockpiled for later use in landscaping. Based on the borings performed, the depth of stripping necessary to ensure removal of all excessively organic or otherwise deleterious materials will be on the order of about 10 inches. However, due to the size of the subject site, the actual stripping depth will likely vary and should be verified and monitored by the geotechnical engineer to ensure adequate removal of deleterious materials.

Where trees or brush will be removed from the site, over-excavation of the root zones should continue until all roots greater than ½-inch in diameter are removed. Deep over-excavations required for the removal of root zones should be backfilled in thin lifts with adequately compacted structural fill meeting the material characteristics and compaction guidelines as described later in this report. If a tree will be allowed to remain in-place and a structure is to be placed within the drip line of the tree, consideration should be given to the placement of a root barrier adjacent to the new foundation.

After stripping and excavation to the proposed subgrade, all areas intended for construction should be proofrolled with a partially-loaded tandem axle dump truck or similar heavy rubber-tired vehicle/equipment weighing approximately 15 to 20 tons. Soils which are observed to rut or deflect excessively under the moving load should be undercut and replaced with properly compacted structural fill. The proof-rolling, undercutting and filling activities should be witnessed by a representative of the geotechnical engineer and should be performed during a period of dry weather.

It should be noted that the upper soils encountered within the borings performed at this site are considered moisture sensitive. If wet at the time of construction, it may be necessary to further undercut and replace the near surface soils prior to the placement of any required structural fill. In lieu of extensive undercutting and replacement, surficial soft, wet or otherwise unstable soils could be stabilized or chemically dried by the addition of lime, fly ash or cement. If a chemical stabilization option is considered, SITE Engineering should be contacted to provide additional recommendations.

After subgrade preparation and observation have been completed and a stable subgrade is verified or provided, structural fill placement may begin. The first layer of fill should be placed in a relatively uniform horizontal lift and be adequately keyed into the stripped and scarified subgrade soils. Fill soils should be free of organic or other deleterious materials, have a maximum particle size less than 2 inches, have a liquid limit of 42 or less, a plasticity index between 10 and 22, and classify as CL in accordance with the Unified Soil Classification System (ASTM D-2487). Soils which classify as ML (silt) are not recommended for use as structural fill. Soils with less than 60 percent fines (material passing a number 200 sieve) should also not be used as fill on this site due to their permeable nature and the volume change potential of the underlying highly plastic subgrade soils. More stringent plasticity requirements may be warranted in the pavement areas depending on the type of base chosen.

All structural fill within the proposed construction areas and for a distance of at least 5 feet beyond any new building perimeter and 2 feet beyond the edges of new pavements should be compacted to at least 95 percent of the standard Proctor maximum dry density as determined by ASTM Designation D698. Structural fill should be placed in maximum lifts of 8 to 9 inches of loose material and should be compacted within the range of one percentage point below (-1%) to three percentage points above (+3%) the optimum moisture content value.

Close moisture content control will be required to achieve the recommended degree of compaction. If water must be added, it should be uniformly applied and thoroughly mixed into the soil by disking or scarifying. Each lift of compacted structural fill should be tested by a qualified geotechnical engineer or his representative prior to placement of subsequent lifts. After the density of each lift has been verified, light scarification of the surface of the lift should be performed prior to placement of additional fill to ensure an adequate bond between lifts. The edges of compacted structural fill should extend at least 5 feet beyond the edges of buildings and 2 feet beyond the edges of pavements prior to sloping. Care should be taken to apply compactive effort throughout the structural fill and structural fill slope areas.

We also recommend that water not be allowed to collect in the foundation excavations, floor slab areas, or on prepared subgrades of the construction area either during or after construction. Undercut or excavated areas should be sloped toward one corner to facilitate removal of any collected rainwater, groundwater or surface runoff. Positive site surface drainage should be provided to reduce infiltration of surface water around the perimeter of the buildings and beneath the floor slabs.

4.4 Fill-Induced Settlement

Due to the preliminary nature of this project, topographic information including existing site grades and potential finished elevations was not provided. Therefore, the following table provides estimated settlements for various thicknesses of fill placed above existing site grade:

Settlement Due to the Weight of Fill* Placed Above Existing Grade	
Fill Thickness (feet)**	**Estimated Settlement (inches)**
1	¼
2	½
3	¾
4	1
5	1¼

* Assumed wet unit weight of 125 pcf
**Above Existing Grade

Values were derived from empirical equations using average soil characteristics from laboratory testing performed on samples of the subsurface soils of the borings performed at this site. Therefore, it is anticipated that settlements throughout the subject site will likely vary.

It should be noted that all subsequent foundation induced settlement estimates provided in this report do not include the settlement induced by the weight of the fill. The settlement due to the weight of the fill provided in the above table should be added to any settlements which were estimated for any proposed foundations.

If possible, we recommend placing any required fill at least 30 days prior to construction of the foundations. This will allow approximately 90 percent of the estimated fill-induced settlement to occur prior to construction of the foundation elements. If the above recommended time is not feasible, additional fill could be placed above the elevation of required fill for a temporary period to decrease the amount of time necessary for consolidation of the subgrade soils due to the weight of the required fill. This is referred to as a surcharge program. Recommendations for a surcharge program can be provided at your request.

4.5 Shallow Foundation Recommendations

Provided the site preparation recommendations given in this report are followed, lightly-loaded structures constructed at this site may be supported on a relatively shallow foundation system bearing at a minimum depth of 2 feet below final grade. Foundation elements bearing on existing naturally occurring clay soils or within newly imported compacted structural fill at the recommended depth can be proportioned utilizing a maximum net allowable soil bearing pressure of 2,000 pounds per square foot for isolated spread footings and 1,600 pounds per square foot for continuous (wall) footings.

The recommended bearing pressures include a factor of safety of 3.0 against bearing capacity failure. However, minimum dimensions of 18 inches for continuous footings and 24 inches for spread footings should be used for design, even if the resulting bearing pressure is less than the allowable bearing pressure, to minimize the possibility of a local bearing capacity failure.

Consolidation of the soils resulting from the foundation loads will result in measurable but tolerable increments of soil settlements. Based on the results of field and laboratory tests, and assuming the foundation elements will be loaded to the maximum net allowable bearing capacity provided above, it is estimated that the settlement of square footings up to 5 feet by 5 feet in plan dimension and continuous footings up to 3 feet in width will be less than one (1) inch. Differential settlement between adjacent columns or along a length of 25 feet of continuous footing should be on the order of 50 percent of the realized total settlement increment.

It should be noted that the aforementioned bearing capacities are maximum allowable bearing capacities. For isolated spread footings, a lower bearing capacity can be utilized in conjunction with a larger footing size. As a result, a higher applied point load can be supported with equal or lower settlements. The following table provides settlement estimates for various footing sizes and applied pressures.

ESTIMATED SETTLEMENT FOR SQUARE SPREAD FOOTINGS (INCHES)										
Square Footing Size (feet)		**3**	**3½**	**4**	**4½**	**5**	**5½**	**6**	**6½**	**7**
Actual Applied Pressure (psf)	**1,400**	0.56	0.63	0.69	0.74	0.79	0.84	0.88	0.93	0.97
	1,600	0.61	0.69	0.75	0.81	0.86	0.91	0.96	-	-
	1,800	0.66	0.74	0.81	0.88	0.93	0.99	-	-	-
	2,000	0.71	0.79	0.86	0.94	0.99	-	-	-	-

Note: A graphical representation of this table is provided in the appendix of the report. The values presented above are based on spread footings bearing at a depth of 2 feet below final grade.

The above table should be utilized to govern footing design only if the aforementioned maximum net allowable bearing capacity and corresponding limiting footing size does not provide adequate support of the anticipated structural loads. A single applied pressure should be chosen and used for the design of all spread footings within a given structure.

The settlements provided above are estimates. Values were derived from empirical equations using average soil characteristics from laboratory testing performed on samples of the subsurface soils of the borings performed at this site. Therefore, settlements throughout subject site will likely vary. The settlement estimates provided in the above table do not include the settlement induced by any proposed fill placed above existing grade. Therefore, proper time should be given to allow subgrade consolidation due to the weight of any fill to occur prior to foundation construction.

It should be noted that total settlements on the order of one (1) inch and differential settlement of ½-inch or less are generally considered moderate but tolerable for building structures. However, it is highly recommended that the design of masonry walls include provisions for liberally spaced, vertical control joints to minimize the effects of cosmetic “cracking”. Furthermore, it is recommended that good rigidity of the structure foundations be provided. This could consist of stiffening the slab with grade beams and tying the individual foundation elements together to form a “waffle” pattern or by the use of post-tensioned reinforcement.

The foundation excavations should be observed by a representative of SITE Engineering, Inc. prior to placement of reinforcing steel or concrete to assure that the foundation soils are consistent with the materials discussed in this report. Soft or loose soil zones encountered at the bottom of the footing excavations should be removed to the level of suitable bearing material and replaced with adequately compacted structural fill as directed by the Geotechnical Engineer.

After opening, the footing excavations should be observed and concrete placed as quickly as possible to avoid exposure of the footing bottoms to wetting and drying. Surface run-off water should be drained away from the excavations and not be allowed to pond. If it is required that footing excavations be left open for more than one day, they should be protected to reduce evaporation or entry of moisture.

The provided recommendations should be considered preliminary. The actual bearing capacity and estimated settlements should be determined utilizing additional subsurface soil characteristics obtained within each proposed structure area once more detailed development plans are established.

4.6 Uplift Resistance of Shallow Foundation Elements

Uplift resistance of shallow footings will be limited to the weight of the foundation concrete and the soil above the extensions of spread footings. For design purposes, the ultimate uplift resistance can be based on unit weights of 140 pcf for the concrete in the footings and 110 pcf for the soil directly above the footings. A factor of safety of at least 1.1 should be applied to the calculated uplift resistance to account for potential variations in the concrete and soil unit weights. The size and depth of foundation should be checked by the structural engineer to assure that it is capable of supporting the uplift forces.

If adequate uplift resistance cannot be achieved, consideration should be given to supporting the proposed building on a deep foundation system. Recommendations for the design of drilled cast-in-place concrete shafts and driven timber piles are provided in subsequent sections of this report.

4.7 Drilled Shaft Foundation System

Although shallow foundation elements may perform adequately for light to moderate building structural loads at this site, structures may also be supported on drilled cast-in-place concrete shafts. The following paragraphs provide preliminary recommendations for design and installation of drilled cast-in-place concrete shafts or piers. The shafts should be installed by contractors having adequate experience in the methods of installation in similar soil conditions. In addition, it should be noted that drilled shaft installation involves removing the existing soil. Consideration needs to be given to soil removal and disposal.

The axial compression capacities of drilled concrete shafts have been computed using a factor of safety of 2.0 against failure at the pile/soil interface (skin friction) and a factor of safety of 3.0 against end bearing failure. The following tables present the allowable compressive capacities of various diameter drilled shafts installed to various tip embedments below the existing ground surface elevation. <u>The provided compression capacities include the effective weight of the concrete in the shaft.</u>

ESTIMATED ALLOWABLE COMPRESSION CAPACITY OF DRILLED CAST-IN-PLACE CONCRETE SHAFTS IN KIPS (Factor of Safety = 2.0 for Skin Friction and 3.0 for End Bearing)

Installation Depth* (feet)	Shaft Diameter				
	18-inch	24-inch	30-inch	36-inch	42-inch
20	16	20	24	27	30
25	20	25	30	34	37
30	27	35	42	49	55
35	35	45	55	64	73

*Below existing grade

The following table presents the allowable uplift or tension capacities of various diameter drilled shafts installed to depths ranging from 20 to 35 feet below the existing ground surface elevation. The uplift or tension capacities of the shafts have been computed using a factor of safety of 2.5 against failure at the shaft/soil interface. The effective weight of the shaft has not been included in the allowable uplift capacities.

ESTIMATED ALLOWABLE UPLIFT CAPACITY OF DRILLED CAST-IN-PLACE CONCRETE SHAFTS IN KIPS (Factor of Safety = 2.5)

Installation Depth* (feet)	Shaft Diameter				
	18-inch	24-inch	30-inch	36-inch	42-inch
20	14	19	24	29	34
25	18	24	30	37	43
30	23	31	39	47	55
35	30	40	50	60	70

*Below existing grade

It should also be noted that the shaft capacity estimates were calculated using average strength values from laboratory testing performed on samples of the subsurface soils obtained from all the borings within the upper 25 feet and from boring B-1 only for depths below 25 feet. Therefore, the actual shaft capacities throughout the site will vary and should be determined utilizing additional subsurface soil characteristics obtained within each proposed construction area once the project development plans are more complete.

The capacities provided above are based on geotechnical properties and soil-shaft relationship only. Consideration should be given to the structural integrity of the shaft itself under the design load conditions. Again, the effective weight of the shaft has been included in the compression capacities and excluded in the uplift capacities provided above. As a conservative approach, the weight of the concrete in the shaft should not be added to the uplift capacities provided in the above table.

The values presented above assume each shaft is isolated from any influence of nearby foundation loading. Center-to-center spacing between shafts should be at least 3 shaft diameters. Settlement of the drilled shafts up to 42 inches in diameter designed in accordance with the recommendations provided above should be less than one (1) inch. Differential settlement across the foundation area should be slightly less than the realized total settlement of an individual shaft provided all shafts are installed to the same tip elevation.

Due to the potential hydrostatic heave associated with the clayey sands and sandy silty clays encountered below the depths of about 8 to 17 feet and the fact that these soils are in a somewhat soft condition, installation of shafts below these depths will likely require the use of a drilling slurry and/or casing during augering followed by placement of concrete with a closed tremie. To determine the necessity for the utilization of a drilling slurry and/or casing during augering, several test shaft excavations should be drilled near the subject installation area. The test holes should be the same diameter and extend at least 3 feet deeper than the final tip elevation of the proposed production shafts.

During installation, the slurry level in the shaft, if used, should be maintained even with the ground surface. As concrete is being placed the tremie should be kept at least three feet below the top of the concrete in the shaft. Regardless of the installation method used, concrete for shaft construction should be placed with a slump range of six (6) to eight (8) inches and be designed to achieve the required strength at the recommended slump.

Installation of the shafts should be carried out in accordance with the National Highway Institute Course No. 132014 entitled “Drilled Shafts: Construction Procedures and LRFD Design Methods”, Publication Number FHWA-NHI-10-016 dated May 2010. Care should be taken to ensure concrete is not allowed to strike the sides of the shaft excavation. We recommend that a geotechnical engineer or qualified technician observe the installation of the shafts to verify that, among other things, 1) subsurface conditions are as anticipated from the boring, 2) the shafts are constructed to the proper diameter, penetration and plumbness, 3) reinforcing is properly placed and centered in the open shaft, and 4) a tremie is properly used for concrete placement. These critical items are fundamental to proper performance of shafts in accordance with design recommendations.

4.8 Driven Timber Piles

Driven timber piles may also be an economical alternative for deep foundation support of structures at this site. Our analyses have been limited to treated timber piles. However, should alternative pile types such as pre-cast concrete, steel pipe, or steel H-piles be desired, SITE Engineering, Inc. should be contacted to provide additional recommendations. Based on the soils encountered in the borings performed at this site, the recommended driven lengths and corresponding estimated allowable compressive and tensile capacities for timber piles are presented in the following table:

ESTIMATED ALLOWABLE SINGLE PILE LOAD CAPACITY IN KIPS
(Factor of Safety = 2.0 in Compression and 3.0 in Tension)

Tip Installation Depth* (feet)	Small Timber Piles (6” tip and 8” butt)		Large Timber Piles (7” tip and 12” butt)	
	Compression	Tension	Compression	Tension
30	13	8	--	--
35	18	11	23	14
40	26	15	33	19
45	34**	22	41	25
50	--	--	46**	31

*Below existing grade.
**Maximum Allowable Stress of Pile Material (Southern Pine)

Driven treated timber piles should conform to ASTM D25 with minimum tip and butt dimensions of six (6) and eight (8) inches for the recommended small timber piles and seven (7) and twelve (12) inches for the large timber piles, respectively. The piles should be treated in accordance to AWPA Specification C-3.

Pile length indicated in the above table is below the existing ground surface elevation prior to fill placement. However, a pile cutoff of up to 3 feet should have little effect on the provided capacities. It should also be noted that the provided capacities are based on soil-pile relationship only. Therefore, consideration should be given to the structural integrity of the pile member under the design load conditions as well as during handling and driving.

The estimated pile capacities include a factor of safety of at least 2.0 in compression and 3.0 in tension. It should also be noted that the pile capacity estimates were calculated using average strength values from laboratory testing performed on samples of the subsurface soils obtained from all the borings within the upper 25 feet and from boring B-1 only for depths below 25 feet. Therefore, the actual piles capacities throughout the site will vary and should be determined utilizing additional subsurface soil characteristics obtained within each proposed structure once additional project information is ascertained.

Using the recommended pile load capacities, it is estimated that settlement of single isolated piles or pile groups of up to 9 piles with minimum center-to-center spacing between piles of at least three pile butt diameters will be less than one (1) inch. While settlement of this magnitude is generally considered tolerable for structures of the type proposed, it is recommended that masonry wall design include provisions for liberally-spaced vertical control joints to minimize the effects of cosmetic cracking.

Pile driving hammers used to drive foundation piles should be selected according to pile type, length, size, and weight of pile, as well as potential vibrations resulting from pile driving operations. Care should be taken to assure that the hammer selected is capable of achieving the desired penetration without causing damage to the piles or causing excessive vibrations which could damage existing, nearby structures. Hammers having a rated energy in the range of 7,500 to 12,000 foot-pounds for the small timber piles and 12,000 to 16,000 foot-pounds for the large timber piles should be satisfactory.

The pile capacities provided in the above tables assume that penetration to the stated depth can be achieved. It is recommended that the pile driving operations be monitored by the geotechnical engineer or his representative. Sometimes premature refusal occurs due to poor performance of the hammer rather than from soil resistance. Although not anticipated, pre-drilling may be required to achieve the design tip elevation. If pre-drilling is used, the diameter of the drill bit should not exceed 80 percent of the pile tip diameter. The pre-drilled depth should be limited to no deeper than 5 feet above of the pile tip design elevation.

Each pile should be driven to the desired tip elevation and driving resistance without interruption in the driving operations. Driving of the center piles in a pile cluster first will better facilitate driving operations. Accurate records of the final tip elevation and driving resistances should be obtained during the pile driving operations. Some pile heaving may be experienced during installation of adjacent displacement type piles. It is therefore recommended that the elevation of the piles be recorded and if heave of the pile butt in excess of ½-inch is noted after driving of subsequent piles, provisions must be made for reseating them.

4.9 Load Testing of Deep Foundation Elements

The load carrying capacity of deep foundation elements utilized at this site should be verified by field load tests. The piles or shafts should be tested in compression as outlined by ASTM D1143. The installed test piles or shafts should be allowed to “rest” for a period of at least 7 days after installation or until proper concrete strength is achieved prior to commencing the load test. The load tests should be performed under the guidance of the Geotechnical Engineer so that the data may be interpreted and the recommended capacities adjusted, if necessary, according to the load test results.

4.10 Lateral Capacity of Deep Foundations

For deep foundations, the lateral loads are resisted by the soil as well as the rigidity of the pile or shaft. Analyses can be performed by methods ranging from chart solutions to finite difference methods. It is recommended that our office be contacted to perform lateral load analysis for the proposed foundation system once the pile or shaft sizes and group dimensions are determined.

4.11 Spacing and Group Efficiency of Deep Foundation Elements

The spacing of deep foundation elements is normally set to allow for typical construction tolerances in placement and vertical alignment. Center-to-center spacing should not be less than either three (3) times the largest diameter of the pile/shaft or five (5) percent of the pile/shaft length whichever produces the greater spacing. For closer spacing, the capacities should be checked using the “Perimeter Shear Formula.” Information on this procedure can be provided upon request.

A reduction of individual capacities due to group effects should not be necessary for groups of up to 9 piles or shafts spaced as suggested above. However, it is recommended that SITE Engineering, Inc. be contacted to determine if a reduction in pile capacity will be necessary based on the planned pile groupings.

4.12 Other Foundation Types

It should be noted that foundation types other than those discussed in this report could be used for support of the structures at this site. These foundation systems include but are not limited to auger cast-in-place piles, driven steel or concrete piles, and screwed helical piles. Ground improvement techniques such as aggregate piers (stone columns) or rigid inclusions may also offer an increase in bearing capacity while minimizing settlements without the expense of a typical deep foundation system. Some of these foundation types and ground improvement systems are patented and should be designed by the manufacturer or distributor. SITE Engineering, Inc. can provide recommendations for various foundation alternatives at your request.

4.13 Floor and Grade-Supported Slab Recommendations

As previously discussed, due to the moderately plastic nature of the near surface clays encountered in the borings performed at this site, a soil supported floor slab is expected to experience movement associated with volume changes within these soils due to changes in moisture content. Therefore, it is recommended that the construction area be prepared in accordance with the recommendations provided in previous sections of this report.

Floor slab loads are commonly distributed to grade (either existing or finished soil grade) by slab-on-grade type construction. Otherwise, a structural floor is used to carry the floor loads independent of the grade. Common types of slabs-on-grade are reinforced slabs, which may or may not include interior ribs, and post-tensioned slabs. The ribbed slab and post-tensioned slab provide rigidity against differential movement and minimize slab cracking. Recommendations for a ribbed slab and post-tensioned slab are provided in the following paragraphs in the event they are preferred over a structural concrete slab.

Ribbed Floor Slab: The ribbed slab should be designed by a registered and qualified structural engineer. However, certain design criteria are suggested. Interior grade beams should be at least 18 inches deep from the top of the slab. The spacing of the ribs should be determined by the structural engineer based on the thickness of the slab but should in no case be greater than 20 feet. Where practical, these ribs should be arranged to coincide with non-load bearing interior walls. A minimum beam width of 12 inches is recommended to allow adequate bearing area. The floor slab and interior grade beams should be a monolithic unit with no joints. If concrete cannot be placed monolithically, it should be doweled to provide continuity and good rigidity.

Post-Tensioned Floor Slab: An alternative to a reinforced ribbed slab foundation is post-tensioned reinforcement. Post-tensioning involves providing tensile steel reinforcement in the slab system by stressing high strength steel tendons after the concrete has achieved sufficient strength. A post-tension ribbed slab is a specialized structural design and should be designed by a qualified structural engineer who is competent and familiar with this type of reinforcement.

In either case, soil supported floor slabs for this project can be designed utilizing a modulus of subgrade reaction (spring constant), k, of 75 psi per inch for the adequately stripped and proofrolled, naturally occurring lean clay soils or compacted structural fill. If a higher modulus of subgrade reaction is required, a k value of 110 pci can be obtained by provided a minimum of 4 inches of clean sand (less than 10 percent fines) directly beneath the floor slab. A k value of 145 pci may be required for design of interior floor slabs where forklift traffic is anticipated. This may be achieved by the placement of a minimum of 4 inches of crushed limestone, crushed concrete or washed gravel.

If moisture sensitive floor coverings are used on the interior slab, consideration should be given to the use of barriers (either polyethylene or a thin sand, graded gravel, or limestone) to minimize potential vapor rise through the slab. Other design and construction considerations, as outlined in the American Concrete Institute (ACI) Design manual, Section 302.1R are recommended. Positive separations and/or isolation joints should be provided between the grade slab and all foundations and walls/columns to allow independent movement.

5.0 PAVEMENT RECOMMENDATIONS

We have evaluated both rigid and flexible pavement systems for this project. Although specific traffic information was not provided to us, we have assumed that traffic in the proposed light duty areas will consist mainly of passenger vehicles (cars and small trucks) with occasional passes of medium to large trucks for deliveries, etc. The heavy-duty pavement areas will likely experience heavy truck loads and possibly some forklift traffic.

It should be noted that the recommended pavement thicknesses presented below are considered preliminary for the assumed parameters in the general site area. The actual pavement thicknesses should be determined utilizing specific traffic information and additional subsurface soil characteristics obtained within each proposed construction area once project plans are more complete. In addition, local municipal ordinances should be reviewed as pavement section thicknesses greater than provided in this report may be required.

The general pavement design information presented in this report is based on information published by AASHTO and the Portland Cement Association as well as past experience in this area. The published information was utilized in conjunction with the available field and laboratory test data to develop general pavement recommendations.

Although extensive evaluation, including California Bearing Ratio (CBR) testing of the near surface soils or potential sources of imported structural fill was not performed, a CBR value of 3.0 and a modulus of subgrade reaction, k, of 75 psi/inch for the adequately stripped and proofrolled naturally occurring soils or compacted structural fill were used for the design of the pavement sections. Therefore, it is assumed that the site preparation criteria presented in the report will be followed and all topsoil and any isolated soft or loose areas encountered during proofrolling of the subgrade will be removed and replaced with compacted fill or be chemically stabilized as previously discussed. Specific design parameters considered in the pavement analyses are as follows:

CBR	3.0
Modulus of Subgrade Reaction, k	75 pci
Reliability	85%
Concrete Modulus of Elasticity	3.99×10^6
Deviation	0.45 Asphalt, 0.35 Rigid
Initial Serviceability	4.2 Asphalt, 4.5 Rigid
Terminal Serviceability	2.5
Modulus of Rupture (concrete)	630 psi
Load Transfer	3.0 Dowels or Keys
Drainage Coefficient	1.0
Layer Coefficients (Asphalt Pavements)	0.42 Asphalt 0.14 Base Course 0.06 Compacted Fill

LIGHT-DUTY PAVEMENTS

(Areas not subject to repetitive 3-axle vehicle loads)

LIGHT-DUTY FLEXIBLE PAVEMENT		
Pavement Materials	Minimum Thickness (Inches)	
	Parking Stalls	Drives
Asphaltic Concrete Wearing Course	2½	3½
Compacted Crushed Limestone Base	10	12
Geotextile Fabric Separator	YES*	YES*
Adequately Stripped & Proofrolled Subgrade or Compacted Structural Fill	--	--

*Note: If the pavement supporting soils are treated with cement the fabric separator may be omitted.

The compacted crushed limestone base for light-duty flexible pavements may be replaced with a cement stabilized base course. The thickness of the soil-cement layer and percentage of cement will vary depending on grading plans and the type of material to be stabilized. However, it is estimated that a soil-cement layer approximately 12 inches in thickness stabilized with approximately 8 percent cement by volume should be sufficient. The actual amount of cement should be determined in the field at the time of construction based on the type of soil to be stabilized.

LIGHT-DUTY RIGID PAVEMENT		
Pavement Materials	Minimum Thickness (Inches)	
	Parking Stalls	Drives
Portland Cement Concrete	5	6
Compacted Granular Base	4	4
Adequately Stripped & Proofrolled Subgrade or Compacted Structural Fill	--	--

The compacted granular base for light-duty rigid pavements should consist of crushed limestone or crushed concrete meeting the 2016 Edition of the Louisiana Standard Specifications for Roads and Bridges (LSSRB) Section 1003.03.01 or 1003.03.02, or relatively clean sands with less than 15 percent fines (material passing a number 200 sieve). Granular base for rigid pavements should be compacted to at least 98 percent of the maximum dry density as determined by ASTM D-698 at moisture contents within 2 percent of optimum.

HEAVY-DUTY PAVEMENTS

(Truck Drives/Parking & Dumpster Areas)

HEAVY-DUTY FLEXIBLE PAVEMENT		
Pavement Materials	**Minimum Thickness (Inches)**	
	Option #1	**Option #2**
Asphaltic Concrete Wearing Course	2	2
Asphaltic Concrete Base Course	2	2
Compacted Crushed Limestone Base	12	--
Geotextile Fabric Separator	YES	NO
Cement Stabilized Base (treated with 8% cement by volume)	--	12
Adequately Stripped & Proofrolled Subgrade or Compacted Structural Fill	--	--

Note: An asphalt pavement section is not recommended for pavements that will experience forklift traffic.

HEAVY-DUTY RIGID PAVEMENT*		
Pavement Materials	**Minimum Thickness (Inches)**	
	Option #1	**Option #2**
Adequately Reinforced Portland Cement Concrete (4,000 psi)	7	7
Compacted Crushed Limestone or Crushed Portland Cement Concrete Base	8	--
Geotextile Fabric Separator	YES	NO
Cement Stabilized Base (treated with 8% cement by volume)	--	12
Adequately Stripped & Proofrolled Subgrade or Compacted Structural Fill	--	--

*Thicker sections than noted above may be required where forklifts will utilize the pavement system. A structural engineer should be consulted regarding forklift pavement sections.

Soils to be cement treated should have a plasticity index (PI) of 15 or less. If the pavement base soils have a PI greater than 15, then lime treatment will be necessary to lower the PI prior to cement stabilization. The thickness of lime treatment, if necessary, should be at least 12 inches. The amount of lime necessary to lower the PI of the fill soils will depend on the plasticity index of the soils to be treated and should be determined at the time of construction. If grading plans require at least 12 inches of structural fill to reach final grade in the pavement areas, lime stabilization will not be required if the imported fill has a PI of 15 or less.

It should be noted that soil cement base has a tendency to shrink similar to concrete causing tension cracks that can reflect up through the surface of asphaltic concrete pavements. The surface cracks will require additional maintenance and sealing to maintain the design life of the pavement. Higher percentages of cement than recommended above will further increase the frequency and severity of the hydration/shrinkage cracks.

Pavements fill materials and base material/construction should meet the requirements of the Louisiana Standard Specifications for Roads and Bridges (LSSRB), 2016 edition. Structural fill utilized in the pavement areas should be compacted to 95 percent of the maximum dry density as determined by ASTM D698 (standard Proctor) at a moisture content within 2 percent of the optimum value.

Proper finishing of concrete pavement requires the use of appropriate construction joints to reduce the potential for cracking. Construction joints should be designed in accordance with current Portland Cement Association and the American Concrete Institute guidelines. Joints should be sealed to reduce the potential for water infiltration into pavement joints and subsequent infiltration into the supporting soils.

Load transfer devices at the pavement joints should be designed in accordance with accepted codes. The concrete should have a minimum compressive strength of 3,500 psi at 28 days (unless otherwise noted in the above tables). The concrete should also be designed with 5±1 percent entrained air to improve workability and durability. Asphaltic concrete pavement materials should meet the requirements of the LSSRB Section 502 and should be compacted to a minimum of 95 percent of the density of the laboratory molded specimen or 92 percent of the theoretical maximum dry density.

In addition, water should not be allowed to pond behind curbs and saturate the pavement base. In down grade areas, granular base should extend through the slope to allow any water entering the base a path to exit. The subgrade or fill soils beneath the pavement base course should be sloped to facilitate drainage. Landscape areas within the pavement system or next to the building should not be allowed to drain under the pavement system or into the pavement base.

The recommended crushed limestone or crushed recycled portland cement concrete base should meet the material requirements of LSSRB Section 1003.03.01 or 1003.03.02, respectively, and be compacted to at least 95 percent of the maximum dry density as determined by ASTM D-1557 (modified Proctor) at moisture contents within 2 percent of optimum.

It is also recommended that a geotextile fabric separator meeting the requirements of LSSRB Section 1019 be placed on the compacted fill prior to placement of crushed stone base. The purpose of the separator is to limit migration of the crushed aggregate into the fine grained soils below during periods of wet weather. If the pavement subbase soils are cement treated, the fabric separator may be omitted. In addition, if a sand base is utilized under the light-duty rigid pavements, the geotextile fabric may be omitted; however, placement of a strip of fabric separator approximately 18 to 24 inches in width under each pavement joint is recommended to minimize migration of the sand into the pavement joints.

If unstable soils are encountered during proofrolling of the pavement subgrade, a geogrid soil reinforcement product may be utilized to minimize undercutting or stabilization of soft soils. If desirable, SITE Engineering should be contacted to provide alternative pavement sections which include the placement of geogrid reinforcement.

In addition, water should not be allowed to pond behind curbs and saturate the pavement base. In down grade areas, granular base should extend through the slope to allow any water entering the base a path to exit. The subgrade or fill soils beneath the pavement base course should be sloped to facilitate drainage. Landscape areas within the pavement system or next to the buildings should not be allowed to drain under the pavement system or into the pavement base. It is highly recommended that weep holes be constructed or installed in catch basins at the bottom of the aggregate base layer to allow a drainage path for any water that enters the base materials. SITE Engineering can provide details related to weep hole design and placement once the pavement plans are more complete.

6.0 CONSTRUCTION CONSIDERATIONS

6.1 Construction Testing and Inspection

Many problems can be avoided or solved in the field if proper inspection and testing services are provided. It is recommended that the site preparation, foundation and floor slab construction, and pavement area construction be monitored by the geotechnical engineer or his representative.

Density tests should be performed to verify compaction and moisture content in the fill and base material. Each lift of fill or base material should be tested and approved by the soils engineer prior to placement of subsequent lifts. As a guideline, it is recommended that field density tests be performed at a frequency of not less than one test per 2,500 and 5,000 square feet of surface area per lift in the building and pavement areas, respectively, with a minimum of three tests per lift per structure.

Inspection should be performed prior to and during concrete placement. Foundation excavations should be observed by the soils engineer or his representative to verify that the exposed materials are suitable for support of the foundations.

It is recommended that SITE Engineering, Inc. be retained to provide observation and testing of construction activities involved in the foundations and pavements, earthwork, and related activities of this project. SITE Engineering, Inc. cannot accept any responsibility for any conditions which deviated from those described in this report, nor for the performance of the foundations and pavements if not engaged to also provide construction observation and testing for this project.

6.2 Utility Lines

It is recommended that all utility pipes be bedded in firmly placed and compacted bedding materials. The bedding should be at least 8 inches in thickness and should extend one-half of the pipe diameter beyond the edge of either side of the pipe or a minimum of 12 inches, whichever is greater. The pipe should be side bedded to the mid-height of the pipe or to the pipe spring line if arch pipe is used. The bedding material should consist of well graded, free draining stone or a sand gravel mixture consisting of approximately 35 percent clean sand with less than 5 percent fines and approximately 65 percent pea gravel with a maximum aggregate size of ½ inch, compacted to at least 70 percent relative density as determined by ASTM D4253 and ASTM D4254 or to at least 90 percent of the maximum density as determined by ASTM D698 (standard Proctor). If utility piping that does not include water-tight joints is used, a geotextile fabric should be placed around the pipe at each joint to reduce potential migration of the fines in the fill or base into the joints of the pipe.

The trench excavations should be backfilled to the surface with granular fill or excavatable flowable fill. Granular backfill should consist of limestone or sand with less than 20 percent fines and should be placed in lifts not exceeding 8 inches in thickness. The backfill should be compacted to a minimum of 95 percent of the maximum dry density as determined by ASTM D698. Flowable fill should meet the requirements of LSSRB Section 710. Where utility excavations traverse the pavement system, the upper 12 inches of utility trench backfill should consist of structural fill soils meeting the classification requirements provided in the Site Preparation section of this report.

For utility lines that are not placed beneath building structures, do not traverse the pavement system, and are not installed within five (5) feet of the perimeter of the buildings or within two (2) feet of the edges of pavements, backfill of the utility trenches may consist of previously excavated soils placed in lifts not exceeding 12 inches in thickness and compacted to at least 90 percent of the standard Proctor maximum dry density.

6.3 Moisture Sensitive Soils/Weather Related Concerns

The upper soils encountered at this site are expected to be sensitive to changes in moisture content and may lose strength if allowed to become wet. During wet weather periods, increases in the moisture content of the soil can cause significant reduction in the soil strength and support capabilities. In addition, soils that become wet may be slow to dry and thus significantly retard the progress of grading and compaction activities. It will, therefore, be advantageous to perform earthwork and foundation construction activities during dry weather. If the upper soils are allowed to become saturated and the construction schedule does not allow for drying of the soils naturally, removal and replacement or chemical stabilization will likely be required.

6.4 Drainage and Groundwater Concerns

Water should not be allowed to collect in the foundation excavations or floor slab areas, or on prepared subgrades of the construction area either during or after construction. Undercut or excavated areas should be sloped toward one corner to facilitate removal of any collected rainwater, groundwater, or surface runoff. Positive site surface drainage should be provided to reduce infiltration of surface water around the perimeter of the buildings and beneath the floor slabs.

Groundwater was initially encountered during the drilling operations at depths ranging from 13 to 15 feet below the exiting surface within the borings performed at this site. It should be noted, that it is possible for a groundwater table to fluctuate depending upon climatic and rainfall conditions. It is recommended that the Contractor determine the actual groundwater levels at the site at the time of the construction activities.

It is recommended that the site be graded in anticipation of wet weather periods to help prevent water from “ponding” within the construction areas and/or flowing into excavations. Filtered sump pumps placed in the bottoms of excavations, or other conventional dewatering techniques, such as drainage swales or other methods approved by the geotechnical engineer, are expected to be suitable for control of surface or runoff water. However, if uncontrollable groundwater infiltration into the excavations is experienced during construction, SITE Engineering should be contacted.

6.5 Excavations

In Federal Register, Volume 54, No. 209 (October 1989), the United States Department of Labor, Occupational Safety and Health Administration (OSHA) amended its "Construction Standards for Excavations, 29 CFR, part 1926, Subpart P". This document was issued to better insure the safety of workmen entering trenches or excavations. It is mandated by this federal regulation that excavations, whether they be utility trenches, basement excavation or footing excavations, be constructed in accordance with the new OSHA guidelines. It is our understanding that these regulations are being strictly enforced and if they are not closely followed, the owner and the contractor could be liable for substantial penalties.

The contractor is solely responsible for designing and constructing stable, temporary excavations and should shore, slope, or bench the sides of the excavations as required to maintain stability of both the excavation sides and bottom. The contractor's "responsible person", as defined in 29 CFR Part 1926, should evaluate the soil exposed in the excavations as part of the contractor's safety procedures. In no case should slope height, slope inclination, or excavation depth, including utility trench excavation depth, exceed those specified in local, state, and federal safety regulations.

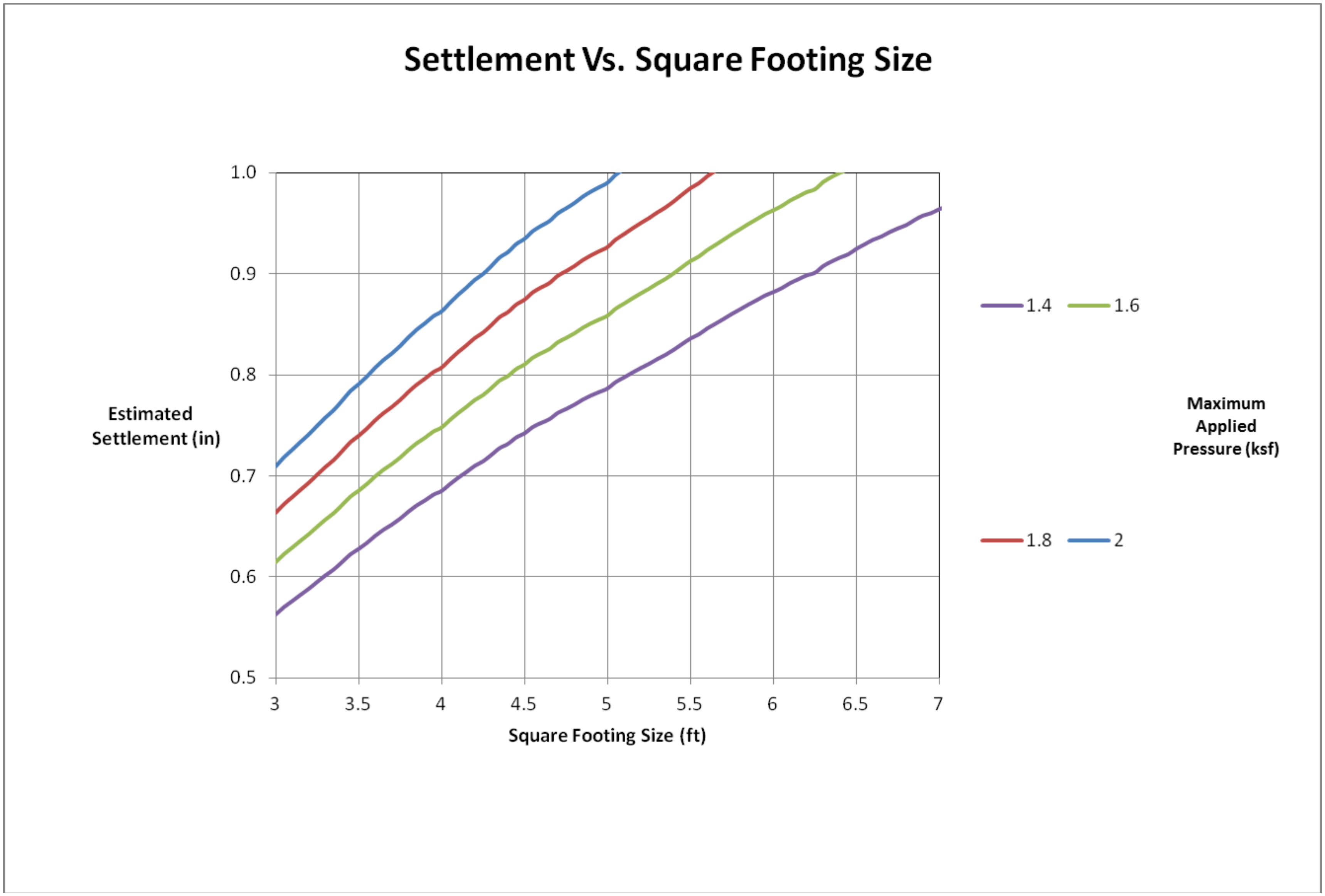
We are providing this information solely as a service to our client. SITE Engineering, Inc. does not assume responsibility for construction site safety or the contractor's or other parties' compliance with local, state, and federal safety or other regulations.

7.0 REPORT LIMITATIONS

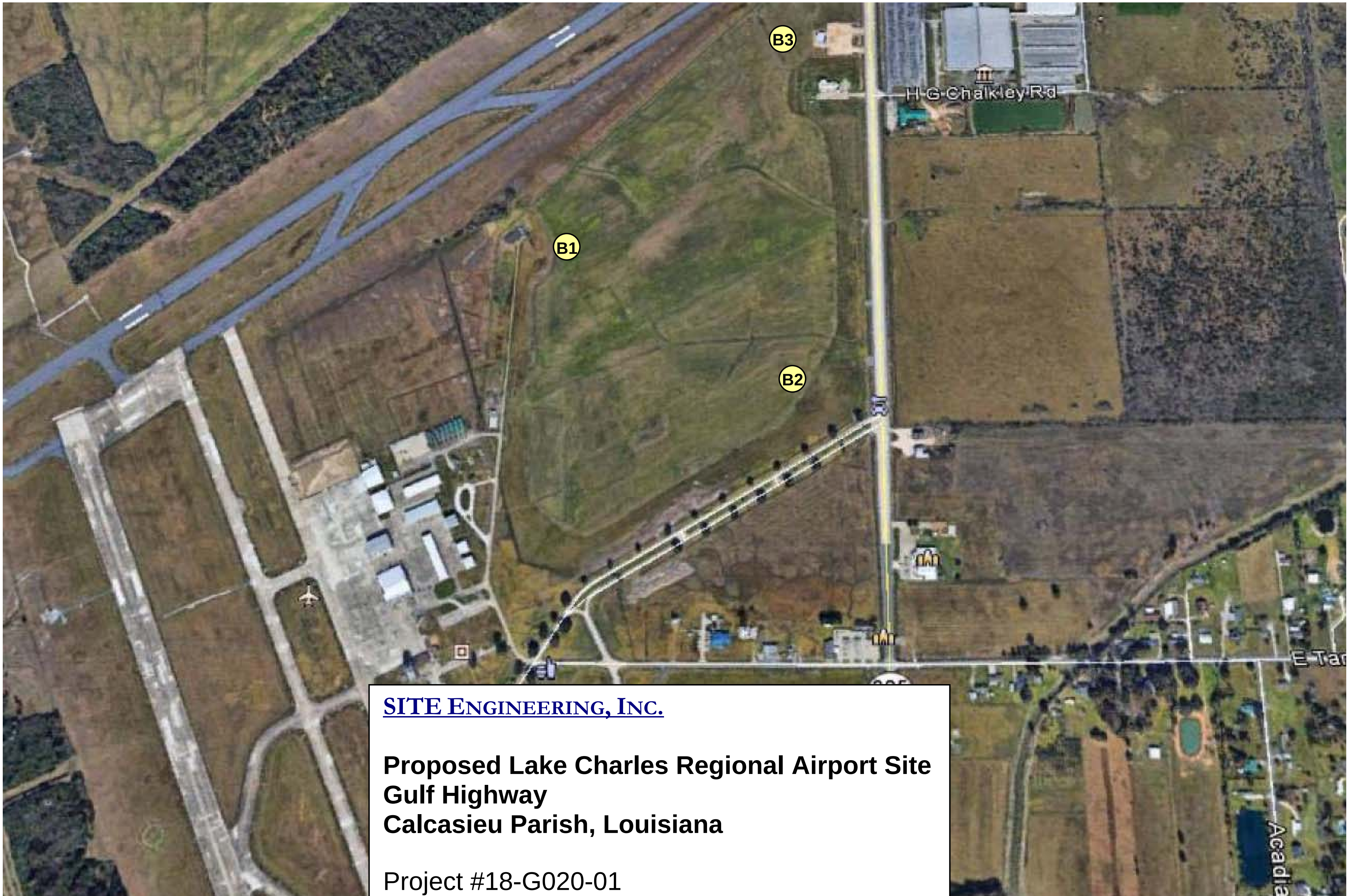
The recommendations submitted, in this report, are based on the available subsurface information obtained by SITE Engineering and are considered extremely preliminary in nature. Once final development details and project information is established, additional borings should be performed to provide specific recommendations.

The geotechnical engineer warrants that the findings, recommendations, specifications, or professional advice contained herein have been made in accordance with generally accepted professional geotechnical engineering practices in the local area. No other warranties are implied or expressed. This report has been prepared for the exclusive use of SWLA Economic Development Alliance or their assigns for the proposed Lake Charles Regional Airport site to be constructed at the referenced location in Calcasieu Parish, Louisiana.

APPENDIX


Settlement Vs. Square Footing Size
Estimated Settlement (in)
1.0
0.9
0.8
0.7
0.6
0.5
3
3.5
4
4.5
5
5.5
6
6.5
7
Square Footing Size (ft)
1.4
1.6
1.8
2
Maximum Applied Pressure (ksf)

Boring Location Diagram



SITE ENGINEERING, INC.

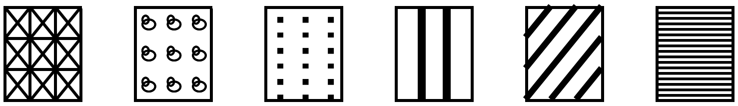
Proposed Lake Charles Regional Airport Site
Gulf Highway
Calcasieu Parish, Louisiana

Project #18-G020-01
Date: March 27, 2018

\# = Approximate Boring Location

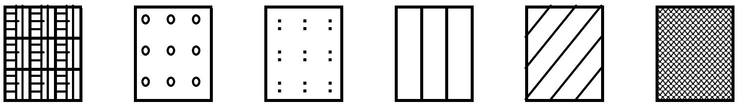
KEY TO TERMS AND SYMBOLS USED ON LOGS

SOIL TYPE



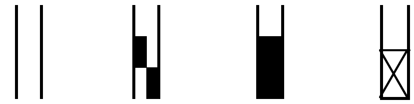
ROCK | GRAVEL | SAND | SILT | CLAY | PEAT

MODIFIERS

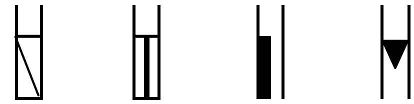


STONE or CONCRETE | GRAVELY | SANDY | SILTY | CLAYEY | FILL or TOPSOIL

SAMPLE TYPE



NO SAMPLE | AUGER SAMPLE | SHELBY TUBE | SPLIT SPOON



NO RECOVERY | ROCK CORE | 2" SHELBY TUBE | TXDOT CONE

UNIFIED SOIL CLASSIFICATION SYSTEM - ASTM D 2487-98

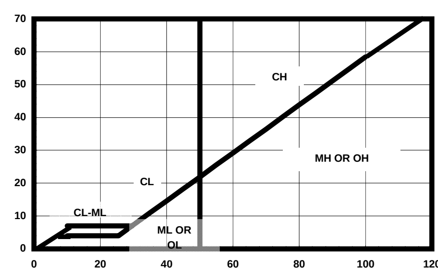
MAJOR DIVISIONS			LETTER SYMBOL	TYPICAL DESCRIPTIONS
COARSE GRAINED SOILS (LESS THAN 50% PASSING NO. 200 SIEVE)	GRAVEL & GRAVELY SOILS LESS THAN 50% PASSING NO. 4 SIEVE	CLEAN GRAVEL (LITTLE OR NO FINES	GW	WELL GRADED GRAVEL, GRAVEL-SAND MIXTURES WITH LITTLE OR NO FINES
			GP	POORLY GRADED GRAVEL, GRAVEL-SAND MIXTURES WITH LITTLE OR NO FINES
		W/ APPRECIABLE FINES	GM	SILTY GRAVEL, GRAVEL-SAND-SILT MIXTURES
			GC	CLAYEY GRAVELS, GRAVEL-SAND-CLAY MIXTURES
	SANDS MORE THAN 50% PASSING NO. 4 SIEVE	CLEAN SANDS LITTLE FINES	SW	WELL GRADED SAND, GRAVELY SAND (LITTLE FINES)
			SP	POORLY GRADED SANDS, GRAVELY SAND (L.FINES)
		SANDS WITH APPREA. FINES	SM	SILTY SANDS, SAND-SILT MIXTURES
			SC	CLAYEY SANDS,SAND-CLAY MIXTURES
FINE GRAINED SOILS (MORE THAN 50% PASSING NO. 200 SIEVE)	SILTS AND CLAYS LIQUID LIMIT LESS THAN 50		ML	INORGANIC SILTS & VERY FINE SANDS,ROCK FLOUR SILTY OR CLAYEY FINE SANDS OR CLAYEY SILT W/ LOW PI
			CL	INORGANIC CLAY OF LOW TO MEDIUM PI LEAN CLAY GRAVELY CLAYS, SANDY CLAYS, SILTY CLAYS
			OL	ORGANIC SILTS & ORGANIC SILTY CLAYS OF LOW PI
	SILTS AND CLAYS LIQUID LIMIT GREATER THAN 50		MH	INORGANIC SILTS, MICACEOUS OR DIATOMACEOUS FINE SANDY OR SILTY SOILS, ELASTIC SILTS
			CH	INORGANIC CLAYS OF HIGH PLASTICITY FAT CLAYS
			OH	ORGANIC CLAYS OF MED TO HIGH PI, ORGANIC SILT
HIGHLY ORGANIC SOIL			PT	PEAT AND OTHER HIGHLY ORGANIC SOILS
UNCLASSIFIED FILL MATERIALS				ARTIFICIALLY DEPOSITED AND OTHER UNCLASSIFIED SOILS AND MAN-MADE SOIL MIXTURES

CONSISTENCY OF COHESIVE SOILS

CONSISTENCY	UNCONFINED COMPRESSIVE STRENGTH IN TONS/FT^2
VERY SOFT	0.0 TO 0.25
SOFT	0.25 TO 0.50
FIRM	0.50 TO 1.0
STIFF	1.0 TO 2.0
VERY STIFF	2.0 TO 4.0
HARD	> 4.0 OR 4.0+

RELATIVE DENSITY - GRANULAR SOILS

CONSISTENCY	N-VALUE (BLOWS/FOOT)
VERY LOOSE	0-4
LOOSE	4-9
MEDIUM DENSE	10-29
DENSE	30-49
VERY DENSE	> 50 OR 50+



ABBREVIATIONS

Qp - HAND PENETROMETER
Qt - TORVANE
MV - MINIATURE VANE

Qu - UNCONFINED COMPRESSION TEST
UU - UNCONSOLIDATED UNDRAINED TRAIXIAL
CU - CONSOLIDATED UNDRAINED

▼ GROUNDWATER FIRST ENCOUNTERED

∇ (# HRS) DELAYED GROUNDWATER READING W/ ELAPSED TIME

CLASSIFICATION OF GRANULAR SOILS

U.S. STANDARD SIEVE SIZE(S)

	6"	3"	3/4"	4	10	40	200	
BOUL-DERS	COBBLES	GRAVEL COARSE	GRAVEL FINE	SAND COARSE	SAND MEDIUM	SAND FINE	SILT OR CLAY	CLAY
152	76.2	19.1	4.76	2.0	0.42	0.074		0.002

GRAIN SIZE IN MM

LOG OF BORING B-1

Proposed Lake Charles Regional Airport Site

Gulf Highway

Calcasieu Parish, Louisiana

TYPE OF BORING: Solid Flight Auger to 15' then Wet Rotary | SITE Project #: 18-G020

DEPTH, FT.	SOIL TYPE	SAMPLE TYPE	SOIL DESCRIPTION **SURFACE ELEVATION**: Existing Grade	N-VALUE, blows per foot	UNCONFINED COMPRESSIVE STRENGTH (Qu), tsf	HAND PENETROMETER (Qp), tsf	TORVANE (Qt), tsf	UNIT DRY WEIGHT pcf	MOISTURE CONTENT, %	LIQUID LIMIT	PLASTICITY INDEX	% PASSING #200 SIEVE
			8" Lean Clay topsoil / Stiff brown lean CLAY (CL) with silt		1.04	1.5		103	19			
			Stiff gray and light brown fat CLAY (CH) with ferrous nodules		1.43	2.0		96	23			
					1.53	2.0		104	23	53	38	
5					1.20	2.0		93	30			
					1.62	2.5		101	23	52	37	
10			Stiff reddish brown and gray lean CLAY (CL) with silt and trace sand			2.0			24			98
			Reddish brown silty clayey SAND (SC-SM) ▼									
15									23			30
			Stiff reddish brown fat CLAY (CH)									
20						2.0			26			
25					1.63	2.5		90	32	73	50	
			Firm brown and gray sandy lean CLAY (CL) with silt and shell									
30					0.72		0.35	100	27			66
			Firm gray and reddish brown fat CLAY (CH) with pockets of sand									
35					0.87		0.45	97	27			84
			Stiff to very stiff gray fat CLAY (CH)									
40					1.39	2.0		94	31			
			- becoming very stiff at 42 feet									
45					3.29	4.5		110	19			
			- becoming reddish brown and gray at 47 feet									
50					2.93	4.0		101	23			

(continued on page 2)

DEPTH OF BORING: 100 Feet Below Existing Grade | **DEPTH TO GROUNDWATER**: 13 Feet Below Existing Grade

DATE OF BORING: March 7, 2018

LOG OF BORING B-1 (continued)

Proposed Lake Charles Regional Airport Site

Gulf Highway

Calcasieu Parish, Louisiana

SITE Project #: 18-G020

DEPTH, FT.	SOIL TYPE	SAMPLE TYPE	SOIL DESCRIPTION	N-VALUE, blows per foot	UNCONFINED COMPRESSIVE STRENGTH (Qu), tsf	HAND PENETROMETER (Qp), tsf	TORVANE (Qt), tsf	UNIT DRY WEIGHT pcf	MOISTURE CONTENT, %	LIQUID LIMIT	PLASTICITY INDEX	% PASSING #200 SIEVE
			(continued from page 1)									
			Stiff to very stiff gray fat CLAY (CH)									
55					1.87	2.5		85	37			
			- with pockets of silt from 57 to 62 feet									
60					1.96	3.0		108	20			
			- becoming very stiff at 62 feet									
65					3.27	4.5		95	29			
			Stiff gray lean CLAY (CL) with silt and sand									
70						3.0			19			76
			Very stiff to stiff gray fat CLAY (CH)									
75					2.51	3.5		108	20			
80					2.47	2.5 4.5		84	39 50			
			- becoming stiff at 82 feet									
85					1.97	3.0		94	30			
90					1.05	1.5		96	29			
95						2.0			30			
			Brown and gray clayey SAND (SC)									
100									24			49
			Boring Terminated at 100 Feet Below Existing Grade									

DEPTH OF BORING: 100 Feet Below Existing Grade

DATE OF BORING: March 7, 2018

LOG OF BORING B-2

Proposed Lake Charles Regional Airport Site

Gulf Highway

Calcasieu Parish, Louisiana

TYPE OF BORING: Solid Flight Auger | SITE Project #: 18-G020

DEPTH, FT.	SOIL TYPE	SAMPLE TYPE	SOIL DESCRIPTION	N-VALUE, blows per foot	UNCONFINED COMPRESSIVE STRENGTH (Qu), tsf	HAND PENETROMETER (Qp), tsf	TORVANE (Qt), tsf	UNIT DRY WEIGHT pcf	MOISTURE CONTENT, %	LIQUID LIMIT	PLASTICITY INDEX	% PASSING #200 SIEVE
			SURFACE ELEVATION: Existing Grade									
			10" Silty Clay topsoil / Stiff gray SILTY CLAY (CL-ML)			2.0			24			
			Stiff gray and light brown fat CLAY (CH) with ferrous nodules		1.88	2.5		107	19	54	40	96
					1.98	3.0		107	20			
5					1.82	2.5		103	22	54	37	
					1.11	2.0		103	20			
			Firm to soft reddish brown sandy lean CLAY (CL) with silt									
10					0.55		0.30	110	21			59
			- becoming soft at 12 feet									
15							0.20		24			70
			Firm reddish brown SILTY CLAY (CL-ML) with trace sand									
20							0.25		23			96
25							0.40		24			96
			Boring terminated at 25 feet below grade									
30												
35												
40												
45												
50												

DEPTH OF BORING: 25 Feet Below Existing Grade

DEPTH TO GROUNDWATER: 13 Feet Below Existing Grade

DATE OF BORING: March 8, 2018

LOG OF BORING B-3

Proposed Lake Charles Regional Airport Site
Gulf Highway
Calcasieu Parish, Louisiana

TYPE OF BORING: Solid Flight Auger

SITE Project #: 18-G020

SURFACE ELEVATION: Existing Grade

DEPTH, FT.	SOIL DESCRIPTION	N-VALUE, blows per foot	UNCONFINED COMPRESSIVE STRENGTH (Qu), tsf	HAND PENETROMETER (Qp), tsf	TORVANE (Qt), tsf	UNIT DRY WEIGHT pcf	MOISTURE CONTENT, %	LIQUID LIMIT	PLASTICITY INDEX	% PASSING #200 SIEVE
	10" Silty Clay topsoil / Stiff brown SILTY CLAY (CL-ML)			2.0			20			
	Very stiff gray and reddish brown fat CLAY (CH) with ferrous nodules		2.24	3.0		104	21	62	45	
			3.61	4.5		110	19			
5			3.78	4.5		113	19			
			2.83	4.0		109	19			
10	Stiff gray and light brown fat CLAY (CH)		1.74	2.5		99	25			
15	▼		1.66	2.5		97	27			
	Soft reddish brown sandy SILTY CLAY (CL-ML)									
20					0.15		24			63
	Soft reddish brown SILTY CLAY (CL-ML) with trace sand									
25					0.15		26			93
	Boring terminated at 25 feet below grade									
30										
35										
40										
45										
50										

DEPTH OF BORING: 25 Feet Below Existing Grade

DEPTH TO GROUNDWATER: 15 Feet Below Existing Grade

DATE OF BORING: March 8, 2018

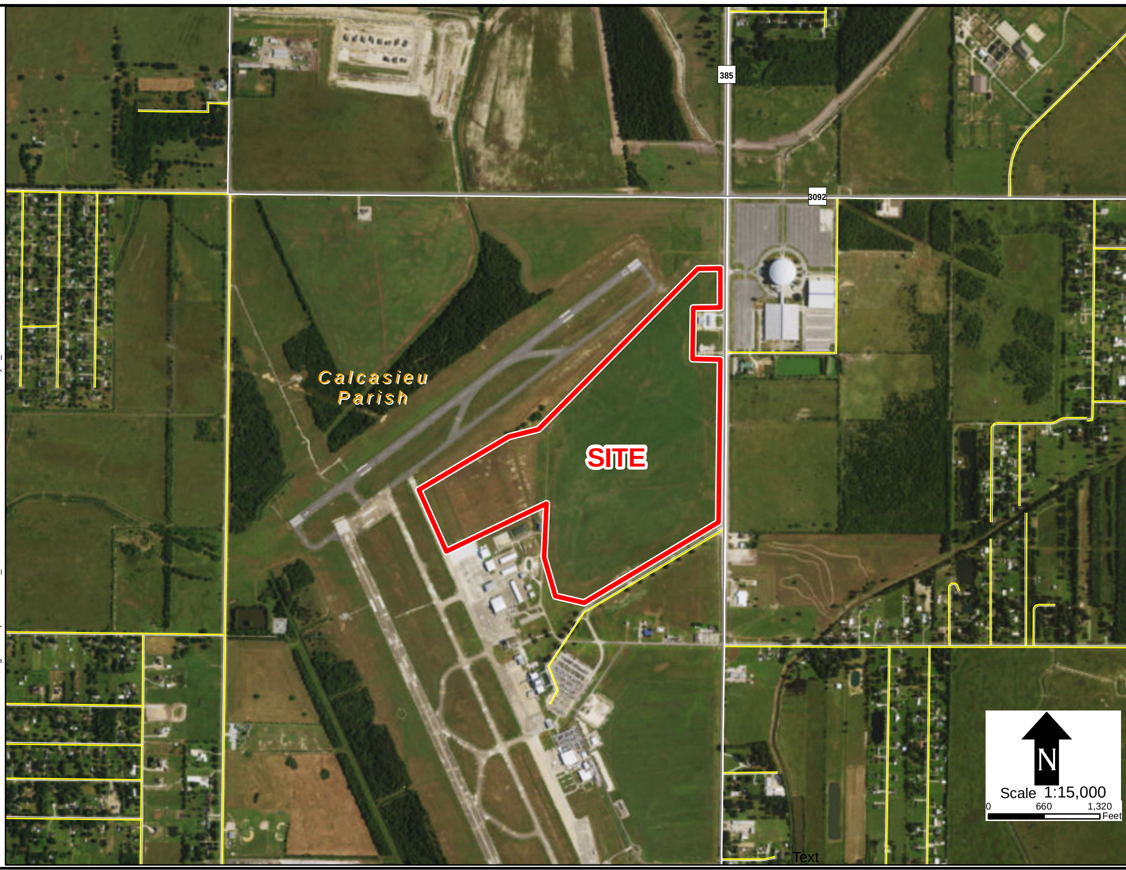
SITE Engineering, Inc.

Geotechnical Report Questionnaire

Site Name: Lake Charles Regional Ariport

CSRS Project ID: 214135

P:\214135\SWLA Certified Sites\Lake Charles Regional Airport Site\RFP_Letters\RFP Letter Common Folder\Attachments\Flyer_Aerial.mxd



Date:	03/26/2018
Provider Name:	SITE Engineering, Inc.
Address:	650 Albertson Parkway
City:	Broussard
State:	Louisiana

Zip Code:	70518
Name:	Jarod J. Breaux, PE
Phone:	(337) 981-1414
Email:	jarod@site-eng.com
Title:	Project Engineer

Does the study indicate that this site is compatible with industrial development? (●) Yes ( ) No

Does the study indicate that soils are suitable for building foundation and/or construction of on-site roadways?

(●) Yes ( ) No

Will soil augmentation be required for the construction of a “typical” 100,000 sq. ft. industrial manufacturing building?

(●) Yes ( ) No

If yes, state reasons augmentation will be required.

See Geotechnical Report (SITE Engineering Report #18-G020-01)

What is the depth (feet) to groundwater? 13 to 15 feet

Submit Form