Exhibit Y - Acadiana Regional Airport P1 Geotechnical Study

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October 4, 2016

Mr. Jim Bourgeois
One Acadiana
804 E. St. Mary Boulevard
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RE: **Preliminary Geotechnical Engineering Investigation**
Acadiana Regional Airport Site Classification
Industrial Drive
New Iberia, Louisiana
SITE Engineering Project: 16-G071-01

Gentlemen:

SITE Engineering, Inc. is pleased to transmit our Preliminary Geotechnical Engineering Investigation for the above referenced project. This investigation was performed in general accordance with SITE Engineering Proposal Number 16-170G dated September 26, 2016. Authorization to proceed with the investigation was provided by Mr. Jim Bourgeois, Executive Director of One Acadiana on September 29, 2016 by signing our proposal.

The purpose of this investigation was to provide a general evaluation of various foundation systems for potential use on the proposed project. This report presents preliminary recommendations for site preparation, foundation 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

ACADIANA REGIONAL SITE CLASSIFICATION
INDUSTRIAL DRIVE
NEW IBERIA, LOUISIANA

SITE ENGINEERING REPORT NUMBER: 16-G071-01

Prepared For

Mr. Jim Bourgeois
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October 4, 2016

By

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TABLE OF CONTENTS

1.0 EXECUTIVE SUMMARY 2

2.0 PROJECT INFORMATION 3

2.1 PROJECT AUTHORIZATION 3
2.2 PROJECT DESCRIPTION 3
2.3 PURPOSE AND SCOPE OF SERVICES 3

3.0 SITE AND SUBSURFACE CONDITIONS 4

3.1 PROJECT LOCATION AND SITE DESCRIPTION 4
3.2 SUBSURFACE CONDITIONS AND GROUNDWATER INFORMATION 4

4.0 EVALUATION AND RECOMMENDATIONS 5

4.1 GENERAL 5
4.2 SITE PREPARATION 5
4.4 FILL-INDUCED SETTLEMENT 7
4.5 SHALLOW FOUNDATION RECOMMENDATIONS 8
4.6 UPLIFT RESISTANCE OF SHALLOW FOUNDATION ELEMENTS 9
4.7 DRILLED SHAFT FOUNDATION SYSTEM 10
4.8 DRIVEN PILE FOUNDATIONS 11
4.9 SETTLEMENT OF PILES 13
4.10 SPACING AND GROUP EFFICIENCY OF PILES 13
4.11 DRIVEN PILE INSTALLATION 14
4.12 LOAD TESTING OF DEEP FOUNDATION ELEMENTS 14
4.13 LATERAL CAPACITY OF DEEP FOUNDATIONS 14
4.14 OTHER FOUNDATION TYPES 15
4.15 FLOOR AND GRADE-SUPPORTED SLAB RECOMMENDATIONS 15

5.0 CONSTRUCTION CONSIDERATIONS 16

5.1 CONSTRUCTION TESTING AND INSPECTION 16
5.2 UTILITY LINES 16
5.3 MOISTURE SENSITIVE SOILS/WEATHER RELATED CONCERNS 17
5.4 DRAINAGE AND GROUNDWATER CONCERNS 17
5.5 EXCAVATIONS 17

6.0 REPORT LIMITATIONS 19

APPENDIX ESTIMATED SETTLEMENT VS. SQUARE FOOTING SIZE

1.0 EXECUTIVE SUMMARY

SITE Engineering, Inc. has completed a preliminary evaluation of the subsurface conditions at the proposed site located adjacent to the Acadiana Regional Airport on Industrial Drive in New Iberia, Louisiana. We understand that the project is in the very early stages of development and the actual types and sizes of proposed infrastructure have not been provided. Therefore, the recommendations presented in this report should be considered preliminary and general in nature. For specific recommendations to be provided, detailed information regarding the size, type and locations of structures would be needed as well as additional borings within the footprints of the proposed structures.

Furthermore, it should be noted that the recommendations presented herein are based on subsurface soil characteristics obtained during a previous geotechnical investigation (SITE Engineering Report 13-G041-01) performed for Berard, Habetz & Associates, Inc. Authorization to access and utilize this prior information was provided in an email from Mr. Ted Habetz of Berard, Habetz & Associates, Inc. on October 3, 2016.

As part of the original investigation, one (1) boring to a depth of 100 feet and three (3) borings to a depth of 50 feet below the existing ground surface elevation were performed. The borings generally encountered 8 to 10 inches of lean clay topsoil followed by very stiff to soft lean clay soils to depths ranging from 22 to 27 feet. These lean clay soils were underlain by very stiff to firm fat clay soils to the boring completion depth of 50 feet within borings B-2, B-3 and B-4 and to a depth of about 62 feet within boring B-1. Below this depth, boring B-1 encountered firm lean clay soils to a depth of about 82 feet followed by stiff fat clay soils to a depth of about 85 feet. These soils were underlain by layers of sandy lean clays and clayey sands to a depth of about 97 feet followed by very stiff lean clay soils extending to a depth of at least 100 feet, the maximum depth explored.

Groundwater was initially encountered at depths ranging from 11 to 18 feet below the existing ground surface within the borings performed at this site. After completion of the drilling and prior to demobilization of our drilling equipment, the boreholes were grouted with a cement-bentonite mixture as required by state regulations. Therefore, subsequent delayed groundwater measurements were not possible.

The near surface soils encountered in the borings performed at this site are considered fair to good in strength and support capabilities and are considered low 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 consisting of isolated spread footings, continuous wall footings, and grade beams, and deep foundation systems such as drilled cast-in-place concrete shafts and driven piles of various materials.

Again, the recommendations provided within this report should be considered preliminary in nature. Soil characteristics within an isolated construction area may be drastically different than those provided in this report and should be determined with supplemental borings once additional project information is ascertained.

2.0 PROJECT INFORMATION

2.1 Project Authorization

SITE Engineering, Inc. has completed a preliminary geotechnical investigation at the proposed site located on Industrial Drive in New Iberia, Louisiana. This investigation was performed in general accordance with SITE Engineering Proposal Number 16-170G dated September 26, 2016. Authorization to proceed with the investigation was provided by Mr. Jim Bourgeois, Executive Director of One Acadiana on September 29, 2016 by signing our proposal.

As previously mentioned, the recommendations presented herein are based on subsurface soil characteristics obtained during a previous geotechnical investigation (SITE Engineering Report 13-G041-01) performed for Berard, Habetz & Associates, Inc. Authorization to utilize this information was provided in an email from Mr. Ted Habetz of Berard, Habetz & Associates, Inc. on October 3, 2016.

2.2 Project Description

We understand that the project is in the very early stages of development and the actual types and sizes of proposed infrastructure have not been provided. Therefore, the recommendations that will be provided should be considered preliminary and general in nature. For final recommendations to be provided, additional borings will need to be performed once detailed information regarding the size, type and locations of structures is ascertained.

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 perform additional borings once the development plans are more complete.

2.3 Purpose and Scope of Services

The purpose of this preliminary geotechnical investigation was to evaluate various foundation systems. Our scope of services included preparation of this geotechnical report. This report presents available project information and presents general recommendations regarding the following:

- Preliminary and general recommendations for various types of foundation systems including shallow foundation elements, drilled cast-in-place concrete shafts, driven timber piles, driven square pre-cast concrete piles, and driven open-ended steel pipe piles;

- Estimates of settlement associated with the recommended foundation type(s), and;

- Construction considerations including potential groundwater concerns, presence of deleterious material, stripping depths, subgrade preparation, and material and compaction recommendations for fill and backfill.

Our 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 site is located on Industrial Drive in New Iberia, Louisiana. At the time of the original geotechnical investigation (SITE Engineering Report Number 13-G041-01), the majority of the subject site was being utilized as an agricultural crop (sugar cane) field. The furrows throughout the site were approximately 12 to 18 inches in height. However, it should be noted that the borings performed at this site were not drilled within the existing furrows. The borings were drilled in adjacent head lands. Furthermore, two (2) drainage features (ditches) traversed the subject site generally in an east-west direction.

3.2 Subsurface Conditions and Groundwater Information

The borings generally encountered 8 to 10 inches of lean clay topsoil followed by very stiff to soft lean clay soils to depths ranging from 22 to 27 feet. These lean clay soils were underlain by very stiff to firm fat clay soils to the boring completion depth of 50 feet within borings B-2, B-3 and B-4 and to a depth of about 62 feet within boring B-1. Below this depth, boring B-1 encountered firm lean clay soils to a depth of about 82 feet followed by stiff fat clay soils to a depth of about 85 feet. These soils were underlain by layers of sandy lean clays and clayey sands to a depth of about 97 feet followed by very stiff lean clay soils extending to a depth of at least 100 feet, the maximum depth explored.

Groundwater was initially encountered at depths ranging from 11 to 18 feet below the existing ground surface within the borings performed at this site. After completion of the drilling and prior to demobilization of our drilling equipment, the boreholes were grouted with a cement-bentonite mixture in accordance with the requirements of the Louisiana Department of Natural Resources. Therefore, subsequent delayed groundwater measurements were not possible.

Once again, the aforementioned groundwater and subsurface soil characteristics were obtained from the previous geotechnical investigation (SITE Engineering Report 13-G041-01) performed in May of 2013. The boring logs included in the aforementioned report should be reviewed for specific soil and groundwater information at the boring locations.

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 fair to good in strength and support capabilities and are considered low in shrink/swell potential. Provided the site preparation recommendations presented in this report are followed and the allowable bearing capacities and estimated settlements are deemed sufficient, structures on 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 elements should be utilized. Therefore, we have also provided recommendations for drilled cast-in-place concrete shafts, driven timber piles, driven square pre-cast concrete piles, and driven open-ended pipe piles as feasible foundation alternatives. Specific details related to foundation design and construction considerations will be presented in subsequent paragraphs.

4.2 Site Preparation

As previously mentioned, at the time of the field exploration, the majority of the subject site was being utilized as an agricultural crop (sugar cane) field. The furrows throughout the site were approximately 12 to 18 inches in depth. Based on our experience with similar sites, it is anticipated that once the crop rows are leveled, the actual depth of stripping necessary to ensure removal of all excessively organic or otherwise deleterious materials will be on the order of one-half of the existing row height plus a few inches. For bidding purposes, stripping on the order of 10 to 12 inches after leveling of the rows should be anticipated. However, the actual stripping depth should be determined and verified 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 proof-rolled with a loaded tandem axle dump truck or similar heavy rubber-tired vehicle 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 soils encountered 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 or wet 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 confirmed, 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.

Generally, all structural fill within the proposed construction areas and for a distance of at least 5 feet beyond the building perimeters and 2 feet beyond the edges of new pavements should be compacted to at least 95 percent of the maximum dry density as determined by ASTM D-698 (standard Proctor). Higher percentages of compaction may be recommended depending on the type of structures planned and the anticipated loads. 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. The edges of compacted structural fill should extend at least 5 feet beyond the edge of the buildings 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.

4.3 Backfilling of Existing Drainage Features

As previously mentioned, two (2) drainage features (large ditches) traverse the subject property and generally runs in an east-west direction. In the event that these elements require relocation, it is recommended that all vegetation and soft soils at the bottom and on the sides of the existing ditches be over-excavated or “mucked out” to a level of firm, undisturbed soil as verified by the geotechnical engineer. The cleaned drainage features should then be backfilled with structural fill meeting the material requirements provided in the “Site Preparation” section of this report. Backfilling of the ditches should be performed as soon as possible to allow adequate time for “self-weight consolidation” of the newly placed fill prior to subsequent construction over these areas. Settlement within the newly placed fill section should approximate about one percent of the thickness of the backfill. However, approximately 90 percent of this consolidation settlement is expected to occur within about 60 to 90 days after the time of placement.

If the duration discussed above for self-weight consolidation is considered to be excessive, then the drainage features could be partially backfilled with relatively clean sands (less than 20 percent passing a number 200 sieve). The sand should be placed in maximum 8-inch loose lifts and compacted to at least 95 percent of the standard Proctor maximum dry density at moisture contents within 2 percent of the optimum value. If sand backfill is utilized, it should be terminated at a maximum elevation of 3 feet below the bottom of the lowest overlying foundation element elevation where shallow foundations are planned or 2 feet below the bottom of an overlying pavement system.

4.4 Fill-Induced Settlement

As previously mentioned, 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 fill thicknesses placed above existing grade.

Settlement Due to the Weight of Potential Fill Placed Above Existing Grade	
Fill Thickness (feet)*	**Estimated Settlement (in)**
1	½
2	¾
3	1½
4	2
5	3

*Above Existing Grade

The estimates provided above were derived from empirical equations using <u>average</u> 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 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 structures being constructed on the aforementioned fill thicknesses.

If possible, we recommend placing the fill at least 6 to 8 months prior to construction of the foundations. This will allow approximately 80 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 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 1,800 pounds per square foot for isolated spread footings and 1,400 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 settlement of square footings up to 4 feet by 4 feet in plan dimension and continuous footings up to 3 feet in width will be less than one (1) inch. Differential settlement across the foundation should be less than ½-inch.

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 anticipated footing sizes and maximum applied pressures.

ESTIMATED SETTLEMENT FOR SQUARE SPREAD FOOTINGS (INCHES)								
Square Footing Size (ft)		**3**	**3½**	**4**	**4½**	**5**	**5½**	**6**
Actual Applied Pressure (psf)	**1,200**	0.63	0.70	0.77	0.82	0.88	0.93	0.98
	1,400	0.70	0.78	0.85	0.91	0.97	-	-
	1,600	0.77	0.85	0.93	1.00	-	-	-
	1,800	0.82	0.91	1.00	-	-	-	-

Note: A graphical representation of the preceding table is provided in the appendix of this 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. Furthermore, if utilized, maximum anticipated structural loads should be used for design of all spread footings.

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, it is anticipated that settlements throughout subject site will likely vary. Furthermore, the settlement estimates provided in the above table do not include the settlement induced by the proposed fill. The estimates are additive to the estimated settlement due to the proposed fill. Therefore, proper time will be required to be given to allow the proposed fill to consolidate before foundation construction occurs, as discussed in the previous section of this report.

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 structures of the type proposed. 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 construction area once further project information is 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 footings. For design purposes, the ultimate uplift resistance can be based on unit weights of 145 pcf for the concrete in the footings and 110 pcf for the soil above the footing. 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 are provided in subsequent sections of this report.

4.7 Drilled Shaft Foundation System

Although shallow foundation elements may perform adequately for the proposed project, 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 for support of the structures at this site. 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. The provided capacities include the effective weight of the shaft.

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
15	13	17	21	25	29
20	17	22	26	30	34
25	24	31	38	45	51
30	35	47	60	73	85
35	44	58	73	88	103

*Below existing grade

The following table presents the allowable uplift or tension capacities of various diameter drilled shafts installed to depths ranging from 15 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
15	11	15	19	23	26
20	16	21	26	31	37
25	21	28	35	42	49
30	27	37	46	55	64
35	35	46	58	70	82

*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 from all of the borings performed at this site. Therefore, the actual shafts capacities throughout the site will vary and should be determined utilizing additional subsurface soil characteristics obtained within each proposed construction area once further project information is established.

Furthermore, 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 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 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.

Although not anticipated, installation of shafts may require the use of a drilling slurry and/or casing during augering followed by placement of concrete with a closed tremie. The installation of shafts at this site will likely require the use of casing and/or a drilling slurry during augering followed by placement of concrete with a closed tremie. During installation, the slurry level in the shaft, if required, 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. Concrete 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 Pile Foundations

A driven pile foundation may also be an affordable option for support of the anticipated structures. Therefore, recommendations with regards to driven treated timber piles, driven square pre-cast concrete piles, and driven open-ended pipe piles are being provided. The allowable capacities provided in the subsequent tables are soil-pile related. Therefore, consideration should be given to 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. The following tables provide allowable single pile load capacities for each aforementioned pile type installed to various tip elevations.

Treated Timber Piles

ESTIMATED ALLOWABLE SINGLE PILE LOAD CAPACITY FOR TREATED TIMBER PILES KIPS (Factor of Safety = 2.0 in Compression and 3.0 in Tension)				
Installation Depth* (feet)	Class 5 Small Timber Piles (6" tip and 8" butt)		Class B Large Timber Piles (7" tip and 12" butt)	
	Compression	Tension	Compression	Tension
25	11	6	--	--
30	16	10	--	--
35	22	14	28	17
40	28	17	35	22
45	--	--	44	29
50	--	--	46**	34

*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.

Square Pre-Cast Concrete Piles

ESTIMATED ALLOWABLE SINGLE PILE LOAD CAPACITY FOR SQUARE PRE-CAST CONCRETE PILES KIPS (Factor of Safety = 2.0 in Compression and 3.0 in Tension)						
Installation Depth* (feet)	14" x 14"		16" x 16"		18" x 18"	
	Compression	Tension	Compression	Tension	Compression	Tension
30	49	27	57	31	65	34
35	62	37	71	42	80	47
40	79	45	91	51	104	57
45	94	58	107	66	120	73
50	107	67	121	75	136	84
55	120	75	136	85	152	94
60	130	84	148	95	165	105
65	139	90	157	101	176	112
70	147	95	167	107	186	119

*Below Existing Grade

Open-Ended Steel Pipe Piles

ESTIMATED ALLOWABLE SINGLE PILE LOAD CAPACITY FOR OPEN-ENDED STEEL PIPE PILES KIPS (Factor of Safety = 2.0 in Compression and 3.0 in Tension)

Installation Depth* (feet)	12" (⅜" Wall Thickness)		14" (⅜" Wall Thickness)		16" (⅜" Wall Thickness)		18" (⅜" Wall Thickness)	
	Compression	Tension	Compression	Tension	Compression	Tension	Compression	Tension
30	29	16	34	18	39	20	45	23
35	36	22	42	25	48	28	55	31
40	47	27	55	30	63	34	71	38
45	55	34	64	39	72	44	81	49
50	63	39	73	45	82	50	92	56
55	70	44	81	51	92	57	103	63
60	76	49	89	57	100	64	112	71
65	82	53	95	61	107	69	120	76
70	87	56	101	65	115	73	128	82

*Below Existing Grade

It should be noted that the pile capacity estimates were calculated using average strength values from laboratory testing performed on samples of the subsurface soils from all soil borings. Therefore, it is anticipated that pile capacities for individual piles at varying locations throughout the structure may vary.

4.9 Settlement of Piles

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. Differential settlement across the foundation area(s) should be slightly less than the realized total settlement of an individual shaft provided all shafts are installed to the same tip elevation.

4.10 Spacing and Group Efficiency of Piles

The spacing of deep foundation elements is normally set to allow for typical construction tolerances in placement and vertical alignment. However, center-to-center spacing of the piles should not be less than either three (3) times the butt diameter of the pile or five (5) percent of the pile 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 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.

4.11 Driven Pile Installation

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 small timber piles (6" tip-8" butt) and 12,000 to 16,000 foot-pounds for the large timber piles (7" tip-12" butt) should be satisfactory. Hammers having a rated energy in the range of 24,000 to 42,000 foot-pounds for both the square pre-cast concrete piles and open-ended steel pipe piles should be satisfactory.

It is further recommended that dynamic monitoring by the use of PDA (Pile Driving Analyzer) methods be performed during installation of the pre-cast concrete or steel pipe probe piles and/or test piles. PDA monitoring should conform to the Standard Test Method for High-Strain Dynamic Testing of Piles (ASTM D-4945). This monitoring will ensure that the allowable stresses are not exceeded during driving and provide documentation regarding ultimate capacities. A pre-construction wave equation analysis should also be performed to optimize driving conditions and hammer energy, and to ensure the driving stresses induced by the hammer system will not cause structural damage to the piles.

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 tip 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.

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. Pre-drilling may be utilized to facilitate driving of the piles. If pre-drilling is used, the drill bit should not exceed 80 percent of the pile tip diameter. Furthermore, the pre-drilled depth should be limited to no deeper than 5 feet above of the pile tip design elevation.

4.12 Load Testing of Deep Foundation Elements

The load carrying capacity of deep foundation elements utilized at this site should be verified by a field load test(s) performed in accordance with ASTM D1143. The installed test shaft(s) shall be allowed to "rest" for a period of at least 14 days after installation or until proper concrete strength is achieved prior to commencement of 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.

4.13 Lateral Capacity of Deep Foundations

For deep foundations, the lateral loads are resisted by the soil as well as the rigidity of the pier 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 shaft sizes and group dimensions are determined.

4.14 Other Foundation Types

It should be noted that foundation types other than those discussed in this report could be used for support of the structure at this site. These foundation systems include but are not limited to auger cast-in-place piles, driven piles of other materials, 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.

4.15 Floor and Grade-Supported Slab Recommendations

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. Where deep foundations are utilized, the floor slab loads are commonly transferred to the foundation elements and do not rely on the soil for support. Recommendations for a ribbed slab and post-tensioned slab are provided in the following paragraphs.

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.

Furthermore, 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 CONSTRUCTION CONSIDERATIONS

5.1 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 construction, and floor slab 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 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 5,000 square feet of surface area per lift in the building areas, with a minimum of three tests per lift.

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, 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.

5.2 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 the building structure, 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.

5.3 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.

5.4 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.

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.

5.5 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.

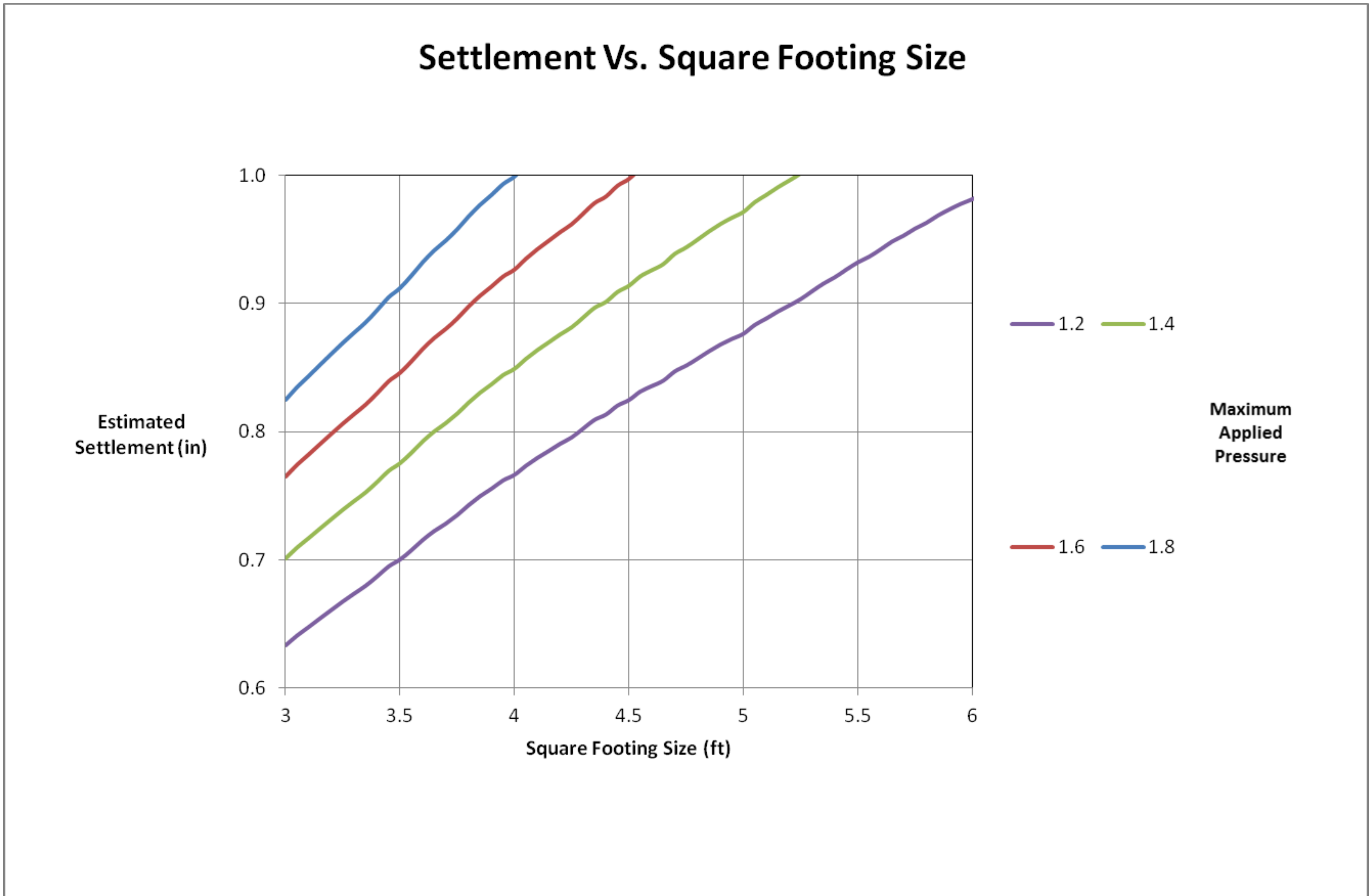
The contractor is solely responsible for designing and constructing stable, temporary excavations and should shore, slope, or bench the sides of the excavations as required to maintain stability of both the excavation sides and bottom. The contractor's "responsible person", as defined in 29 CFR Part 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.

6.0 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 further 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 One Acadiana or their assigns for the site located adjacent to the Acadiana Regional Airport in New Iberia, Louisiana.

APPENDIX


Settlement Vs. Square Footing Size
Estimated Settlement (in)
1.0
0.9
0.8
0.7
0.6
3
3.5
4
4.5
5
5.5
6
Square Footing Size (ft)
1.2
1.4
1.6
1.8
Maximum Applied Pressure