

REPORT OF
GEOTECHNICAL INVESTIGATION
PLAQUEMINES PARISH
PORT SULPHUR CONSOLIDATED COMMUNITY CENTER
278 CIVIC DR.
PORT SULPHUR (PLAQUEMINES PARISH), LA

FOR
PLAQUEMINES PARISH
C/O ALL SOUTH CONSULTING ENGINEERS, LLC.
METAIRIE, LOUISIANA



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11 June 2008

Plaquemines Parish
c/o All South Consulting Engineers, LLC.
110 Veterans Blvd.
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Attn: Mr. Jarret Bauer, E.I.

Re: Report of Geotechnical Investigation
Plaquemines Parish
Port Sulphur Consolidated Community Center
278 Civic Dr.
Port Sulphur (Plaquemines Parish), LA
AAI File: 08-L3135

Dear Jarret,

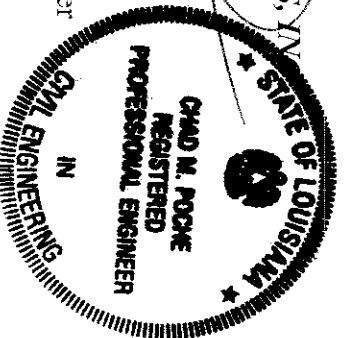
Transmitted are three (3) copies (two bound and one unbound) of our engineering report covering a geotechnical investigation for the subject project. Our findings, together with the analyses and conclusions based on them, are submitted in the attached report.


Thank you for asking us to perform these services. It has been a pleasure working with you on this project and we look forward to serving you again in the future.

Sincerely,

ARDAMAN & ASSOCIATES, INC.


CHAD M. POCHÉ, P.E.
Vice President/Branch Manager




ALEX JARAMILLO, E.I.
Assistant Project Engineer

(2) bound copies & (1) unbound copy

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FIGURES 1 THROUGH 4

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**REPORT OF
GEOTECHNICAL INVESTIGATION
PLAQUEMINES PARISH
PORT SULPHUR CONSOLIDATED COMMUNITY CENTER
278 CIVIC DR.
PORT SULPHUR (PLAQUEMINES PARISH), LOUISIANA**

1.0 INTRODUCTION AND LIMITATIONS

This report contains the results of a geotechnical investigation performed for a new consolidated community center located along Civic Dr. in Port Sulphur, Louisiana. The investigation was performed in accordance with AAI's accepted proposal dated 21 April 2008. All South Consulting Engineers, LLC (All South) are the program managers for the project.

The analyses and recommendations presented in this report are based on the provided project information and the results of the investigation. While it is not likely that conditions will differ greatly from those observed in the soil borings it is always possible that variations can occur away from the borehole locations. If it becomes apparent during construction that subsurface conditions differing significantly from those observed in our borings are being encountered, this office should be notified at once so their effects can be determined and any remedial measures necessary can be prescribed. Also, should the nature of the project change, the recommendations provided in this report may have to be re-evaluated.

This report has been prepared for the exclusive use of Plaquemines Parish, All South and the project designers for the purpose of constructing the proposed project features as generally described in this report. The recommendations provided in this report are site specific and are not intended for use at any other site or for any other facility. This report provides recommendations for design and construction and should not be used as construction specifications.

2.0 SCOPE

This investigation included drilling three (3) undisturbed soil borings to obtain samples of the various subsoils and to determine subsurface conditions and stratification. Soil mechanics laboratory tests were used to evaluate the physical properties of the subsoils.

Engineering analyses based on the soil borings and laboratory test results were made to determine recommendations regarding allowable pile load capacities for various embedments of driven treated timber piles and square, precast, concrete (SPC) piles, estimates of settlement, downdrag analyses, site preparation, and general construction recommendations. Additionally, paving recommendations were made for both rigid and flexible pavements for parking and driveway areas.



3.0 FIELD AND LABORATORY PROCEDURES

A detailed description of the field and laboratory procedures used for this project is contained in Appendix A. Details of the soil conditions encountered at the boreholes are provided on the individual boring logs in Appendix A.

For this project, three (3) soil borings were made. Two (2) borings were completed to the 100 foot depth and one (1) boring was completed to the 15 foot depth below ground surface. The locations of the soil borings at the project site are shown on Figure 1.

A series of soil mechanics laboratory tests were performed on the samples obtained from the soil borings. The results of the laboratory tests are summarized on the boring logs.

4.0 SITE CONDITIONS

In a geotechnical investigation of this nature, various aspects of site conditions must be taken into consideration. Subsurface conditions (soil and groundwater) have been investigated by performing soil borings. An understanding of site topography and the geology of the area are based upon observations made during the soil boring program and our experience in the general area. The following paragraphs provide a discussion of these various site conditions.

4.1 Location and Topography

The proposed project features will be constructed along Civic Dr. in Port Sulphur, Louisiana. There are currently buildings or temporary trailers on site. The remainder of the site is covered with grass.

4.2 Geology

Geologically, fill materials and deposits of Holocene Age appear to underlie the site to the borings' deepest termination depth of 100 feet.

4.3 Soil Conditions

Typically, there is medium stiff to stiff brown and gray silty clay with organic matter and shells to the approximate 4 to 6 foot depth. This is followed by interbedded layers of very soft to medium stiff gray and dark gray sandy clay, silty clay, and clay to the deepest boring termination depth of 100 feet.



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The generalized stratification previously described has been simplified and interpolated. This description is meant solely as an aid in visualizing the subsurface conditions and does not define the continuity of strata away from the borehole locations. For details of the conditions encountered at the boreholes, refer to the individual boring logs in Appendix A.

4.4 Ground Water

The soil borings were initially dry-augured in order to locate groundwater and observe its short-term rise characteristics. Where measured, free water was initially encountered at the approximate 4 foot depth. The ground water level rose to the approximate 3 ½ foot depth after a 15 minute waiting period.

The presence and depth to groundwater can fluctuate with rainfall or other seasonal variations. The depth to groundwater should be verified prior to beginning any construction operations affected by groundwater.

5.0 PROJECT CONSIDERATIONS

This section provides information regarding the project that is pertinent to the geotechnical investigation. This information includes a description of the project as provided to this office and recommendations regarding foundation support for the proposed structure.

5.1 Furnished Information

Mr. Jarret Bauer of All South provided the information for this project. We understand the project will consist of the construction of a new consolidated community center in Port Sulphur, LA. The complex will consist of a new structure with associated parking and driveway areas. The structure will be approximately 16,000 square feet in plan dimensions. No structural loading information was provided.

We understand that up to 3 feet of fill may be placed to raise the site above present grade. Foundation support for the structure is expected to consist of driven, treated timber piles, or square, precast, concrete (SPC) piles.



5.2 Foundation Recommendations

Due to highly compressible near surface soils at the site and relatively large calculated ground surface settlement cause by fill placement, we recommend the structure be supported on a deep foundation system consisting of driven treated timber piles or precast concrete square piles. For deep foundation support, all loads from the structure should be supported on piles having the same approximate tip embedment depth below the existing ground surface in order to minimize potential differential settlement.

Existing pile foundations, if present, should not be relied on for support. Existing piles should be cut off 2 feet below ground surface.

Pile caps should be poured integrally with the floor slabs and grade beams. Pile driving will cause vibrations that may affect nearby structures or pavements. AAI should be contacted to evaluate the effects of pile driving on adjacent structures.

Due to the placement of approximately 3 feet of fill at the site, downdrag (negative skin friction) will occur on piles that will result in reduced allowable capacities. Additionally, ground surface settlement will effect pavements and roadways. Routine maintenance, repairing, and possible overlaying of paved areas should be anticipated.

6.0 FLOOR SLABS

Structural loads carried by walls and columns will be transmitted to the supporting strata by deep foundations. Common types of slabs are reinforced slabs and post-tensioned slabs with or without interior ribs. Reinforced slabs or post-tensioned slabs can be ribbed to provide additional stiffness against differential movement and minimize slab cracking.

Either a reinforced slab or post-tensioned slab is recommended for this structure. Standard construction practice of using a vapor barrier and capillary breaker should be included in the design of the slab. A registered and qualified civil/structural engineer should design the slab.

The floor slab and interior grade beams should be a monolithic unit with no joints. In the case of shallow foundations, the slab should also be monolithic with the footings. If the concrete cannot be placed monolithically, it should be doweled for continuity and rigidity.



7.0 FILL PLACEMENT EFFECTS

The placement of approximately 3 feet of fill at the site will cause settlement of the ground surface and, in turn, will cause additional settlement of pile foundations due to downdrag. Reductions to the pile capacities are necessary because of negative skin friction caused by the fill.

7.1 Fill Settlement

Placement of fill will cause consolidation of the underlying soil deposits and settlement of the ground surface. We have calculated the estimated long-term settlement of the ground surface due to 3 feet of fill placed across the site. The results of our analyses are provided on the following table.

Fill Height (feet)	Estimated Center Settlement (inches)
3	4 to 5

Our settlement estimates are based on a unit weight of 110 pcf for the fill material and an assumed fill area of approximately 130 feet by 60 feet in plan dimensions.

7.2 Time Rate

The estimated settlement values provided in Section 7.1 are approximate long-term estimates from the time fill is placed. We estimate that approximately 50% of the ultimate settlement will occur within 2 years of fill placement and approximately 90% of the estimated ultimate settlement will occur within a time period of 5 to 8 years. It should be noted that time rate estimates are difficult to predict. Actual values may vary 20 to 30% from our predicted values.

8.0 DEEP FOUNDATIONS

8.1 Allowable Pile Load Capacities

Analyses have been made to determine the estimated allowable single pile load capacities for various depths of treated ASTM D 25 quality timber piles and SPC piles. Allowable pile load capacities for driven treated ASTM D 25 quality timber piles are provided on Figure 2. Figures 3 and 4 provide the allowable pile load capacities for SPC piles.



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The allowable pile load capacities provide for a 4-ft cutoff below the existing ground surface, assume the piles are vertical, and do not include the weight of the pile. The provided capacities contain appropriate estimated factors of safety against failure of a single pile through the soil. The capacities also include a limiting adhesion value based on load tests in geologically similar soils.

The analyses for pile capacities are based on a soil-pile relationship only. The structural capacity of the piles and their connections to transmit these loads should be determined by a structural engineer.

8.2 Pile Group Efficiency

Pile butt spacings are normally set to allow for typical construction tolerances in placement and vertical alignment. General practice in this area is to have the center-to-center spacing not less than either:

- A. 3 Pile Butt Diameters
- or
- B. 5% of the Pile Length

No reduction from individual pile capacities due to group effects will be necessary for small groups or widely spaced piles applicable to this project. However, the final layout should be checked by this office for group efficiency.

8.3 Estimated Settlement of Pile Foundations

We recommend the slab be cast monolithically with pile caps and grade beams to minimize the potential for differential settlements. The pile settlements provided on Figures 2 through 4 include all movements anticipated from fill and settlement loads.

Our settlement estimates do not include the elastic deformation of the piles, which should be added to the settlement estimates. Elastic deformation of the piles may be estimated at 67% of the static column strain of a pile acting as a column.

Our estimates of settlement are based on the assumption piles will be driven in small groups or widely spaced rows. We have also assumed the center to center spacing between rows of single piles will be no closer than 8 feet. All piles should be driven to the same approximate tip embedment in order to minimize differential foundation settlement.

In the event any of our assumptions are not valid, AAI should be contacted to evaluate the potential settlement of pile foundations.



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8.4 Pile Installation

Treated ASTM D 25 quality timber piles, as analyzed for this report, may be driven with an air operated single-acting hammer developing a manufacturer's rated energy of approximately 15,000 ft-lbs. per blow (Vulcan No. 1 or equivalent). Timber piles should be driven no harder than 25 blows per foot.

SPC piles, as analyzed for this report, may be driven with an air operated single-acting hammer developing a manufacturer's rated energy of approximately 26,000 ft-lbs. per blow (Vulcan No. 08 or equivalent).

It is important that inspection of piles be done by qualified geotechnical technicians be maintained so as to detect unexpected conditions as indicated by the driving resistance (hammer blows per foot) as well as any potential problems with breakage or driving difficulties.

Oak, nylon, and mica/ta/aluminum cushions are recommended. Dry plywood cushions are commonly used and can be effective, although frequent replacement is necessary. Should a pile be damaged during driving, it should be replaced and re-driven to the design depth.

8.5 Probe Piles and Pile Load Tests

Once the design pile type and length have been selected, it is recommended a series of probe piles be installed within the area of the proposed structure. The probe piles should be installed with the same equipment and methods that will be used to install the job piles.

The probe piles should be allowed to set for a period of 14 days and at least one of the probe piles should be load tested to failure in accordance with ASTM D 1143. The results of the pile load tests should be evaluated by AAI to verify the estimated pile load capacities.

8.6 Vibration Monitoring

Pile driving will cause vibrations that may affect nearby buildings and utilities. Pile driving should be monitored at any structure of concern during the driving of the job piles to record their magnitude of vibrations. Peak particle velocities of 0.25 in./sec are generally regarded as a vibration level uncomfortable to human perception.



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Sustained peak particle velocities of 0.5 in./sec measured at a structure may induce damage to the structure. Therefore, for sustained peak particle velocities in excess of 0.5 in./sec, pile driving operations should be terminated and consideration given to altering the pile installation criteria.

9.0 EARTHWORK CONSIDERATIONS

At a minimum, the site will require earthwork associated with grading and preparation for the new structure and paving. The following paragraphs provide pertinent recommendations associated with potential earthwork activities.

9.1 Site Preparation

Significant site preparation problems will develop unless good drainage is provided throughout the project duration. In order to prepare the building and pavement areas for fill or subgrade, the site should first be stripped of all vegetation, soft or loose surface soils, and deleterious material, etc.

Subsequent to the removal of these undesirable materials, the building areas should be proof-rolled. Proof-rolling should be performed using heavy wheeled vehicles, such as loaded dump trucks. The proof-rolling should be observed by a qualified geotechnical engineer or technician to detect pumping or yielding areas or other undesirable subgrade.

Such conditions, if present, may require undercutting and replacement with more competent fill. These recommended procedures are intended to serve as an aid to the construction of site fill and not as a method of providing "all weather" construction conditions.

Proper site drainage should be maintained during and after construction to divert the flow of surface waters from the area of the structure foundation. Providing drainage during the construction process will facilitate construction by reducing the potential for volumetric change in the near surface soils. In this regard, good roof and surface drainage should be assured with positive collection and runoff of these waters.

Maintaining drainage after construction will improve the life of the structure by avoiding water softening of the foundation soils. Also, a distance of at least one-half (1/2) the expected height of fully-grown trees should be maintained between any trees and the structure foundation.



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9.2 Fill Materials

Subsequent to the site preparation activities, the area should be brought to grade using a clean, select fill material free from debris or organic matter. A cohesionless soil described as a clean sand with less than 10% passing the U.S. No. 200 Sieve and less than 60% passing the U.S. No. 60 Sieve should be used as fill.

9.3 Fill Placement and Compaction

Cohesionless fill (including sand and lightweight aggregate) should be placed in 10 to 12 inch loose lifts. This fill should be compacted to a dry density equal to at least 95% of its maximum as determined by the Modified Proctor compaction test (ASTM D 1557) or to a minimum relative density of 75% as determined by ASTM D 4253 and D 4254.

9.4 Quality Control

The use of the correct fill materials and the proper placement and compaction are critical in any earthwork where subsequent construction of driveways, parking areas, or structures is planned. Construction monitoring by a qualified geotechnical engineer or technician is recommended to document that proper fill construction has been accomplished.

The responsibilities of the quality control representative generally include observation of excavations, proof rolling operations, documentation of material types, and fill placement and compaction techniques. The QC engineer or technician should make sure the fill is the proper material type, and is placed in the correct manner. Any deviation from the design should be reported to the design engineer.

10.0 PAVEMENTS

The site will be provided with paving for driveways and parking areas. Based upon our understanding of the proposed facility usage, we anticipate that the parking areas will be for automobiles and lightly loaded trucks. Driveways and service ways will require paving designed for access by waste collection and delivery vehicles in addition to the parking area traffic.



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Although no traffic loading information was furnished, we have defined two usage categories for the paving design as follows:

Usage Category	Loading	Daily Passes
Parking Areas	Automobiles and Light Trucks (2 kip front & rear single axles)	Up to 200
Driveways and Service Ways	Parking Areas Plus: Garbage Collection and Delivery Trucks (18 kip front single, 40 kip rear tandem axles)	Up to 10

The following paragraphs provide recommendations for flexible (asphaltic) and rigid (Portland cement concrete) paving.

10.1 Flexible Pavement

The subgrade should first be prepared in accordance with Section 9.0 of this report. The base courses should be constructed over the prepared subgrade. Base course material should consist of crushed stone aggregate (similar to LA DOTD Standard Specification, Section 1003.03(d), respectively, 2000 edition). Subbase should consist of cohesionless fill and compacted as described in Sections 9.2 and 9.3.

Crushed stone aggregate should be compacted to a relative density of 75%, or higher, according to the ASTM D 4253 and ASTM D 4254, or to a minimum dry density of 95 percent of its maximum dry density as determined by the Modified Proctor compaction test (ASTM D 1557).

The base and subbase materials should overlay a geotextile fabric to prevent pumping of finer grain soils. The geotextile should be a nonwoven fabric with an apparent opening size (AOS) smaller than a U.S. No. 70 sieve.

Based on the assumed traffic loading conditions, the following thicknesses are recommended for flexible pavement components. The recommended thicknesses are based on a subgrade CBR of 2.



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Recommended Flexible Paving Thicknesses (Inches)		
Item	Parking Areas	Driveways and Serviceways
Asphalt Pavement Course	3	5
Base Course	6	8
Subbase Course	10	12

We recommend the asphalt courses be placed as late as possible in the project so that the effects of settlement can be reduced. Proper drainage during and after construction is essential to the success of flexible asphaltic pavement systems.

Flexible pavements are susceptible to failures due to poor surface and subsurface drainage. Asphalt pavement generally requires surface sealing with a thin ($1/2$ inch) hot mix asphaltic concrete or an asphalt slurry seal at a 4 to 5 year interval to maintain a good pavement system because the local climate tends to weaken and oxidize the surface.

10.2 Rigid Pavement

Subgrade provisions are critical even though the concrete paving derives supporting capacity from the thickness, strength, and reinforcement of the concrete. Therefore, the recommendations provided in Section 9.0 should be followed for site preparation.

Upon completion of subgrade preparation, a six-inch (6) thick layer of crushed stone aggregate overlaying a geotextile fabric is recommended for base course.

The geotextile should be a nonwoven fabric with an apparent opening size (AOS) smaller than a U.S. No. 70 sieve. The stone should be compacted to a dry density at least equal to 95 percent of its maximum as determined by the Modified Proctor compaction test (ASTM D 1557).



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The crushed stone should meet the following gradation.

Sieve Size	% Passing
1½ inch	100
1 inch	90 - 100
½ inch	25 - 60
No. 4	0 - 10
No. 200	0 - 1

Rigid paving designs have been performed based on a modulus of subgrade reaction (k) of 50 pci. Additionally, the design assumes 3,000 psi concrete will be used. The design life of the pavement system is dependent upon periodic maintenance of the pavement. This maintenance includes, but is not limited to, cleaning and resealing joints, sealing cracks, and immediate repairs of damaged areas.

Based on design traffic information provided above, we recommend the following thicknesses for rigid concrete pavement.

Recommended Rigid Paving Thicknesses (Inches)		
Item	Parking Areas	Driveways & Serviceways
Rigid Concrete	5 inches	8 inches

The rigid concrete should conform similarly to the requirements of LA DOTD Standard Specifications, Section 601, 2000 Edition. Proper steel reinforcement (temperature and shrinkage), joint design, and installation are essential to satisfactory pavement performance.

The concrete pavement may contain adequate wire mesh reinforcement for structural strength and to reduce temperature expansion affects. Adequate joints should be provided for expansion and contraction. The location and configuration of joints is important to reduce the concrete panel edge stress. In addition, adequate load transfer at all joints is necessary to reduce the concrete panel edge stress. Panel sizes should not exceed 15 feet in any direction.



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Key-way joints with dowel bars are recommended. Primary use of saw-cut joints is not recommended. However, occasional use of saw-cut joints may be necessary where experience indicates the potential for cracking (e.g., corner distress). If saw-cut joints are used, they should contain steel dowel bars for load transfer. All joints should be initially sealed and maintained during the life of the pavement.

The strength of the concrete should be verified prior to allowing any traffic to traverse the pavement. In addition, no construction traffic should be allowed on the light traffic pavement areas. If the strength of the concrete is less than specified, no traffic should be allowed on the pavement until the strength is reached. If the concrete does not attain its design strength after a period of 28 days, it should be removed and replaced with adequate concrete.

In addition to performing compressive strength tests, consideration should be given to performing flexural strength tests since design is based on flexural strength. The flexural strength tests consist of casting beams instead of cylinders. The beams may be tested using the third-point loading method (ASTM C 78) or center-point loading method (ASTM C 293). As with compressive strength testing, the flexural strength should be verified prior to allowing any traffic to traverse the pavement. If the concrete does not attain the design strength after a period of 28-days, it should be removed and replaced with adequate concrete.

11.0 CONSULTATION

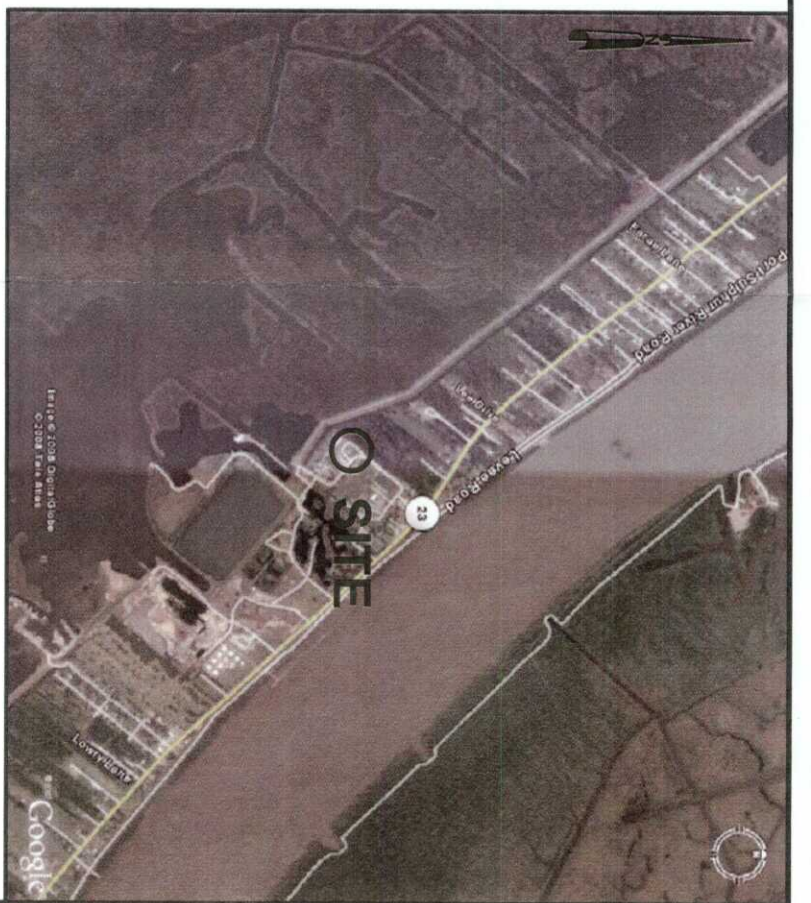
Often during final design and /or construction, questions can arise which are not specifically covered in the report. They can normally be handled by a brief phone call or conference with the designers.





NOT TO SCALE

REFERENCE:
PLAN BY GOOGLE.



VICINITY MAP

NOT TO SCALE

REFERENCE:
SITE IMAGE BY GOOGLE

NOTE:
BORING LOCATIONS ARE APPROXIMATE.

<p>PORT SULPHUR CONSOLIDATED COMMUNITY CENTER 278 CIVIC DR. PORT SULPHUR, LOUISIANA</p>			
<p>for ALL SOUTH CONSULTING ENGINEERS, LLC METAIRIE, LOUISIANA</p>			
<p>AAI Ardaman & Associates, Inc. Baton Rouge, LA</p>			
Project Engineer:	Drawn by:	Checked by:	
A. JARAMILLO	NMS	<i>[Signature]</i>	
File No.:	Date:	Figure No.:	
08-L3135	06/03/08	1	
<p>Title: BORING PLAN</p>			

**PLAQUEMINES PARISH
 PORT SULPHUR CONSOLIDATED COMMUNITY CENTER
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 PORT SULPHUR (PLAQUEMINES PARISH), LOUISIANA**

**ALLOWABLE PILE LOAD CAPACITIES
 (INCLUDING EFFECTS OF UP TO 3 FEET OF FILL)**

TREATED ASTM D 25 QUALITY TIMBER PILES

7-Inch Tip Diameter / 12-Inch Butt Diameter (minimum)

TOTAL ESTIMATED PILE MOVEMENT	PILE TIP EMBEDMENT BELOW EXISTING GROUND SURFACE IN FEET	ESTIMATED ALLOWABLE SINGLE PILE LOAD CAPACITIES IN TONS	
		COMPRESSION	TENSION
1 Inch or Less	55	3	1
	60	5	3
	65	7	5
1 to 2 Inches	40	2	1
	45	4	2
	50	6	4
	55	9	5
	60	11	6
	65	14	8

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**ALLOWABLE PILE LOAD CAPACITIES
 (INCLUDING EFFECTS OF UP TO 3 FEET OF FILL)**

SQUARE, PRECAST CONCRETE (SPC) PILES

12-Inch by 12-Inch

TOTAL ESTIMATED PILE MOVEMENT	PILE TIP EMBEDMENT BELOW EXISTING GROUND SURFACE IN FEET	ESTIMATED ALLOWABLE SINGLE PILE LOAD CAPACITIES IN TONS	
		COMPRESSION	TENSION
1 Inch or Less	50	1	1
	60	6	4
	70	10	6
	80	14	8
	90	18	10
1 to 2 Inches	100	22	12
	50	6	4
	60	11	6
	70	16	9
	80	21	11
	90	27	15
	100	33	18

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**ALLOWABLE PILE LOAD CAPACITIES
 (INCLUDING EFFECTS OF UP TO 3 FEET OF FILL)**

SQUARE, PRECAST CONCRETE (SPC) PILES

14-Inch by 14-Inch

TOTAL ESTIMATED PILE MOVEMENT	PILE TIP EMBEDMENT BELOW EXISTING GROUND SURFACE IN FEET	ESTIMATED ALLOWABLE SINGLE PILE LOAD CAPACITIES IN TONS	
		COMPRESSION	TENSION
1 Inch or Less	50	3	2
	60	8	5
	70	12	7
	80	17	9
	90	21	11
	100	25	13
1 to 2 Inches	50	9	5
	60	15	8
	70	20	11
	80	26	14
	90	32	17
	100	38	20

APPENDIX A

FIELD AND LABORATORY PROCEDURES

The following paragraphs describe the field and laboratory procedures used for this investigation. Soil Boring Logs are included with this appendix. The boring logs provide the field and laboratory data collected.

A.1 FIELD EXPLORATION

Three (3) soil borings were made for this project to investigate subsurface conditions. The borings were drilled on 8 through 12 May 2008. The approximate locations of the borings are shown on the Boring Plan, Figure 1. The locations were established by a geotechnical engineer and located in the field by the drill crew.

A.1.1 Drilling Methods

The borings were drilled with truck-mounted, rotary-type drilling equipment. The soil borings were advanced using a nominal four-inch diameter short flight auger. This technique allowed the proper borehole advancement to secure the appropriate samples (see "Sampling Procedures") and allowed the observation of the presence of free water in the borehole. Upon completion of the borings, the boreholes were backfilled or grouted full depth in accordance with Louisiana regulations.

A.1.2 Sampling Procedures

Soil samples were obtained continuously within the upper 10 feet. Continuous sampling was performed to provide detailed information for near surface stratigraphy. Below the 10 foot depth, the samples were obtained at three to five feet on center.

In these cohesive and semi-cohesive soils, relatively undisturbed samples were secured using a three-inch diameter, thin-wall steel tube sampler. In this sampling procedure, the borehole is advanced to the desired level, and the tube is lowered to the bottom of the boring. It is then pushed about two feet into the undisturbed soil in one continuous stroke. The sample and tube are retrieved from the borehole and detached from the drill string.

The sample is extruded by a hydraulic piston onto a rigid sample catcher to minimize disturbance. The sample is then visually classified. The classification includes description of soil color, strength estimates, identification of structural conditions (layering, seams, etc.) and variations (organics, oxide inclusions, etc.). A pocket penetrometer strength test is performed. Any disturbed portions are discarded, and the sample is sealed to minimize disturbance and moisture loss during transportation to the laboratory.



In the less cohesive materials, standard penetration tests were performed. These tests provide a measure of the in situ characteristics of the soil and secure a disturbed sample. In this test, a 2 inch OD, 1.37 ID, heavy-walled "split spoon" sampler is driven into the undisturbed soil at the bottom of the borehole with a drop hammer weighing 140 pounds and having a stroke of 30 inches. It is first seated 6 inches, then driven an additional two, six-inch increments. The "Penetration Resistance" is the number of such blows required to drive the spoon the last 12 inches. It is recorded on the boring log in the following manner:

24 b/f
(7-9-15)

where the figures in parenthesis indicate the number of blows required for each 6 inch increment.

A.2 LABORATORY PROCEDURES

Certain samples from the various strata were tested in the laboratory to determine their pertinent physical characteristics. The samples and types of tests performed were selected by a geotechnical engineer to develop information necessary for appropriate analyses. The testing program, conducted in general accordance with ASTM methods, is described below.

A.2.1 Strength Tests

The strength characteristics of the various soil strata are important for geotechnical engineering analyses. Twenty-five (25) unconfined compression tests were performed to develop this data. The testing procedures also include determination of the moisture content and wet and dry density of the sample.

The results of the compression tests are tabulated in the laboratory data portion of the soil boring logs under the column heading "Compressive Strength". The moisture content and dry density data are tabulated in the subsequent two columns within the laboratory data portion of the logs.

A.2.2 Classification Tests

In order to classify the soils more definitely than can be done by field methods, six (6) Atterberg limits tests were performed. The results of these tests are shown on the boring logs.



DESCRIPTION OF TERMS AND SYMBOLS USED ON SOIL BORING LOG

FIELD DATA		LABORATORY DATA					Soil Type	DESCRIPTION		
Depth (feet)	Samples	Field Test Results	Compressive Strength (tsf)	Water Content (%)	Dry Unit Weight (pcf)	Atterberg Limits				
						LL	PL	PI	Other	
40										<div style="border: 1px solid black; padding: 5px;"> <p>Description</p> <p>Classifications are based on visual observations by field & lab representatives as well as results of laboratory data (when available).</p> </div> <div style="border: 1px solid black; padding: 5px; margin-top: 5px;"> <p style="text-align: center;">Laboratory Data</p> <p>Compressive Strength</p> <p>Value based on peak compressive strength. Determined by unconfined compression test unless otherwise noted.</p> <p>Dry Unit Weight</p> <p>As determined by method similar to ASTM D-2937.</p> <p>Water Content</p> <p>As determined by pertinent portions of ASTM D-2216.</p> <p>Atterberg Limits</p> <p>LL : Liquid Limit PL : Plastic Limit PI : Plasticity Index (= Liquid Limit - Plastic Limit)</p> <p>Other</p> <p>Results of other tests such as consolidation, permeability, grain size or notes associated with testing program.</p> <p>Soil Type</p> <p>Graphical representation of soil type. In accordance with USCS Symbols.</p> </div>
35										
30										
25										
20										
15										
10										
5										
0										

Ground Water Levels

Long-Term Depth

Depth to water after boring is completed (time noted).

Short-Term Depth

Depth to water after initial water encountered prior to proceeding with boring (time noted).

Initially Encountered

Depth where free water was initially encountered during augering.

Sampling/Field Data

3.5 (P)

Undisturbed

3" dia. Tube sample

Pocket Penetrometer (P)

Penetration resistance (tons/sq. ft.)

Torvane (T)

Shearing resistance (tons/sq. ft.)

Spilt Spoon

Std. penetration test

13 b/f

(3-7-6)

Std. Penetration

No. of blows per foot (blows per each six inch increments).

Auger

Disturbed (auger) collected in accordance with ASTM D-1452.

No Recovery

Sampling attempted but no sample retrieved.

<p>Ground Water Level Data</p>	<p>Boring Advancement Method</p>
<p>Notes</p>	
<p>Boring Abandonment Method</p>	
<p>From LOGTERMS</p>	
<p>Strata Boundaries May Not Be Exact</p>	