Preliminary
Geotechnical Engineering Report

Chennault Sites 2 & 2A - 185 Acre Tract
Chennault International Airport
Lake Charles, Louisiana

for

SJB Group, LLC
P.O. Box 1751
Baton Rouge, LA 70821

prepared by

Daniel J. Holder, P.E., Inc.
Consulting Civil / Geotechnical Engineer
2767 Scarborough Drive
Lake Charles, LA 70615

DJH File 14-109
22 December 2014

Daniel J. Holder, P.E., Inc.
Consulting Civil / Geotechnical Engineer

2767 Scarborough Drive
Lake Charles, LA 70615
dan@danholderpe.com
337-274-4125

22 December 2014

SJB Group, LLC
P.O. Box 1751
Baton Rouge, LA 70821

Attn: Mr. Michael L. Thompson, P.E., CET

RE: Preliminary Geotechnical Engineering Report
Chennault Sites 2 & 2A - 185 Acre Tract
Chennault International Airport
Lake Charles, Louisiana
DJH File 14-109

Dear Mr. Thompson:

I have completed the Preliminary Geotechnical Engineering Report for the referenced project, and am submitting the same herewith. This work was performed in general accordance with my written scope of work dated 30 January 2014, and was authorized by you in a telephone conversation on 07 November 2014.

Please advise if you have any questions regarding this information, or if I may be of any additional assistance. It has been a pleasure working with you on this project.

Sincerely,

Daniel J. Holder

Daniel J. Holder, P.E.
Louisiana P.E. Reg. No. 26532

STATE OF LOUISIANA
DANIEL J. HOLDER
NO 26532
REGISTERED
PROFESSIONAL ENGINEER
IN
CIVIL ENGINEERING

Report Distribution:

2 copies; 1 electronic file (.PDF)

Preliminary Geotechnical Engineering Report

Chennault Sites 2 & 2A - 185 Acre Tract
Chennault International Airport
Lake Charles, Louisiana

DJH File 14-109; 22 December 2014

PROJECT INFORMATION

1. Description of Project. Based on the information provided, it is understood that this project will consist of the preliminary geotechnical evaluation of a 185 acre property for the purpose of initial planning for the economic development of this site. No specific development plans are available for the property at present. Thus, the intent of this study is to make a number of widely spaced soil borings at representative locations to provide an overview of soils and ground water conditions and discuss probable earthwork issues, possible foundation types, and provide preliminary geotechnical recommendations for the general economic development of this site. It is understood that additional studies will be made for specific foundation recommendations once more detailed design information is available.

The 185 acre tract is an irregularly shaped parcel situated west of the main airport runway and north of East Prien Lake Road, in Lake Charles, Louisiana. Refer to the Site Vicinity Map (Figure 1) and Google Earth Aerial Photograph and Boring Location Plan (Figure 2) in the Appendix.

RESULTS OF INVESTIGATION

2. General. This investigation included the following work activities.

- a review of available geologic information,
- a site reconnaissance by the project engineer,
- two (2) soil borings to the 25 foot depth (B-1 and B-2), intended to supplement three (3) others made for a previous investigation at this site (B-3 through B-5, DJH File 11-045, dated 26 August 2011),
- laboratory testing of selected soil samples,
- engineering analyses and evaluations, and,
- the preparation of this report by the Geotechnical Engineer.

The approximate boring locations are shown on Figure 2 in the Appendix to this report.

3. Site Conditions. The 1,000 acre property is essentially a large pasture that is bounded by a large drainage channel on the north and east sides, by East Prien Lake Road on part of the east side and part of the south side, and the Pacific – Missouri

Railroad on much of the south side. East Prien Lake Road and the railroad, in addition to forming part of the boundaries of the tract, also cut through the tract. Overall, the site appeared to be relatively flat and level, with poor drainage. Crawfish holes were also observed all over the site.

According to the Geologic Map of Louisiana (*Pope, et al, 1984*), the site is underlain by the Prairie Formation of Pleistocene Age. These soils are described as *"Light gray to light brown clay, sandy clay, silt, sand, and some gravel."* A portable GPS unit indicated that the center of the site is located at an approximate latitude and longitude of N30^{o} 12' 06.5" and W 93^{o} 09' 06.7", respectively. The appropriate U.S.G.S. Topographic Map indicates that the site is at an elevation of about +15 MSL. Refer to Figures 1 and 2 in the Appendix

4. Soil and Ground Water Conditions. In general, the soils encountered in the borings made at this site may be summarized as follows.

Generalized Soil Stratification

Depth (ft)	**Soil Description**
0 to 2	Firm dark grayish brown SILTY CLAY (CL) to CLAYEY SILT (CL-ML), w/ roots
2 to 8	Stiff light gray & tan SILTY or SANDY CLAY (CL), w/ brown oxides, silt streaks, & crawfish holes
8 to 17	Stiff reddish brown & light gray SILTY CLAY (CL) & CLAY (CH), w/ black oxides & sand layers
17 to 25	Firm to stiff dark gray CLAY (CH), w shells

The borings were initially advanced using dry augering methods to determine the presence of and the hydrostatic conditions of ground water in the boreholes. Ground water was first encountered at about the 3 to 4½ foot depth (9 to 10 foot in 2011), and was not observed to rise significantly during a brief (about 15 minute) observation period. The depth to ground water can fluctuate with seasonal variations in rainfall and evaporation, etc.; the shallower ground water depths observed in 2014 are probably due to "perched" conditions. The actual depth to ground water should be determined more accurately at the time of construction, but may be assumed to be at a depth of about 6 to 8 feet at this site.

It should be emphasized that the actual soil and ground water conditions encountered in the relatively few soil borings made for this preliminary investigation varied widely. The information contained in this section has been generalized from the data obtained from all of the soil borings made for this investigation, and is meant to provide with a general overview of the soil and ground water conditions. For more specific information, refer to the Boring Logs in the Appendix.

GEOTECHNICAL RECOMMENDATIONS

5. General Considerations. The soil conditions encountered in the very widely spaced soil borings made for this investigation consisted of about 2 feet (or more) of silty surface soils, followed by firm to stiff natural clayey soils and some sandy soils to the limit of the exploration at about the 25 foot depth.

These soil conditions should be suitable for a wide variety of development options, including single story metal buildings to wood or steel frame buildings of several stories. Conventional shallow spread footings or reinforced slab-on-grade foundations should be suitable for the support of these structures, or drilled, cast-in-place concrete shafts may be considered for relatively heavy buildings or where settlement movements are less tolerable.

Typical site preparation and earthwork procedures (e.g., stripping the top 2 feet or more of silty soils and placing select fill to achieve the desired subgrade) should be expected at this site.

Although recommendations for foundations, etc., for specific buildings is beyond the scope of this preliminary investigation, typical recommendations for Site Preparation and Earthwork, Shallow Foundations, and Drilled Shaft Foundations are provided in Sections 6, 7, and 8, respectively. It is understood that additional study, including more field exploration and laboratory testing will be required to provide detailed design information once more specific building information is available.

6. Site Preparation and Earthwork Activities. Typically, all vegetation, organic matter, and roots, etc., is removed from the site to expose the firm to stiff clayey subgrade. An undercut of about 2 feet or so should be anticipated at this site, with deeper undercuts in some areas. The exposed subgrade surface should be inspected to ensure that a suitable surface exists upon which to place select fill. This inspection may include proofrolling the subgrade with a loaded, tandem-axle dump truck or other means as determined by the inspector. Any areas that are determined to be unsuitable for fill placement should be further undercut or stabilized to achieve a stable subgrade surface. Proper subgrade preparation and inspection is essential for the development of this project.

Once a firm subgrade exists upon which to conduct fill operations, select fill may be placed to achieve the desired building pad elevation, if required. Select fill should consist of a silty or sandy clay with a Liquid Limit of 30 to 42 and a Plasticity Index of 12 to 22. The fill should be placed in 6 inch thick loose lifts or less and compacted to 95% of the Standard Proctor Maximum Dry Density at $\pm$2% of the Optimum Moisture Content (ASTM D 698). Each lift should be tested to ensure compliance with these recommendations prior to placing subsequent lifts. A minimum testing frequency of one test per 2,500 square feet, but not less than 3 tests, per lift is recommended. All subgrade preparation and earthwork activities should be observed and tested by qualified personnel experienced in earthwork inspection.

Good surface drainage should be established prior to and during the earthwork activities. Standing water on the subgrade or in any excavations should be promptly drained or pumped off.

7. Shallow Foundations. The shallow soils at this site or properly placed and compacted select fill should be suitable for the support of shallow foundations for lightly loaded buildings. The following general recommendations for shallow foundations can be used for planning purposes for this site.

7.1 Reinforced Slab (or “Ribbed”) Foundation. Typically, a reinforced slab foundation is used for lightly loaded buildings in this area to help accommodate normal soil movements. A reinforced slab foundation consists of a monolithic slab-on-grade with turned-down edges (perimeter grade beams); interior grade beams may be included if required by the building loads and/or stiffness considerations. The perimeter grade beams function as shallow foundations to carry the exterior wall loads and serve to cutoff moisture fluctuations in the soils supporting the slab from the surrounding environment. Interior grade beams serve to stiffen the slab system, allowing it to better accommodate movements in the supporting soils. Interior grade beams should be located beneath any load bearing interior walls and/or columns, in which case they should be designed as a shallow foundation. In general, interior grade beams should be spaced at distances of 15 feet or less (each way). Adequate reinforcement, as determined by the structural engineer, should be provided in the slab-on-grade foundation and grade beams. The entire slab system should be placed monolithically (in one pour), or dowelled to provide equivalent rigidity.

The slab foundation may be reinforced with conventional reinforcing steel (rebar) or post tensioned steel tendons (i.e., a post-tensioned slab). The slab and grade beam dimensions and reinforcement of either foundation system should be determined by a qualified design professional knowledgeable in the design of slabs-on-grade.

The slab section should be underlain by a suitable polyethylene vapor barrier (e.g., Visqueen) and a granular leveling layer. The vapor barrier should extend beneath the grade beams and/or shallow foundation elements; the granular layer is typically located just beneath the slab-on-grade.

7.2 Bearing Capacity and Settlement Estimates. Shallow foundations should bear within the undisturbed, stiff clayey soils or properly placed and compacted select fill at a depth of at least 2 feet. Typically, a net allowable soil bearing pressure of 2,000 pounds per square foot (psf) is recommended for continuous footings, and 2,600 psf for isolated column footings in the stiff, shallow natural soils and/or properly placed and compacted fills in this area.

The allowable bearing pressures recommended in the preceding paragraph are net values, which means that the weight of the footing and overlying backfill has already been accounted for. Regardless of the computed footing width, a minimum footing width of 18 inches and 24 inches is recommended for continuous and isolated footings, respectively, to minimize the possibility of localized “shear punch” failure.

The long term settlement of shallow foundations is typically on the order of 1 inch or less for foundations designed for the recommended bearing pressures.

7.3 Rectangular Footings and Overturning. Capacities for rectangular footings may be increased according to the following formula:

$$q_r = q_w (1 + 0.3 B/L)$$

where
- q_r = net allowable bearing pressure for rectangular footings (psf)
- q_w = net allowable bearing pressure for continuous footings given in Section 7.2 (psf)
- B = footing width
- L = footing length (L>B)

Resistance to overturning loads should only consider the ***effective*** footing area, i.e., the portion of the footing centered beneath and effective in carrying the load. The equivalent footing dimensions B’ and L’ of the effective footing area are defined as:

$$B' = B - 2e_B \quad \text{and} \quad L' = L - 2e_L$$

where e_B and e_L are the eccentricity in each direction. Eccentricity is defined as the moment (M) divided by the axial load (P), or

$$e_B = M_B / P_B \quad \text{and} \quad e_L = M_L / P_L$$

<u>7.4 Lateral Loads.</u> Lateral loads on foundations will be resisted by lateral earth pressure against the side of the foundation and skin friction (or adhesion) between the base of the foundation and the underlying soil. The lateral earth pressure resistance should be neglected for shallow (i.e., 4 feet deep or less) foundations, and, in any case, the sliding resistance should be more than adequate for the anticipated lateral loads. The allowable sliding resistance may typically be taken as 250 psf for foundations bearing on undisturbed natural soils. This value includes a factor of safety of about 2 against shear failure of the foundation soils.

<u>7.5 Construction Considerations.</u> Shallow (i.e., less than about 4 to 6 feet deep) excavations in clayey soils should remain stable (i.e., not cave) for short periods of time in the absence of surface or ground water. The reinforcing steel and concrete should be placed expeditiously following the completion of the excavation. The excavation should not be permitted to stand open any longer than necessary. Any water that may accumulate in the excavation should be pumped out immediately.

The foundation excavations should be inspected by a qualified representative of the geotechnical engineer to ensure that the bearing surface is properly prepared prior to placing the reinforcing steel or concrete for the foundation. The soils at this site can become significantly weaker if wetted or disturbed during the construction operations. Traffic in the excavation should be prohibited, and drainage should be provided to direct surface and ground water (if any) away from the excavation. If the concrete for the foundation will not be placed on the same day as the excavation, a “mud mat” of lean concrete should be placed to protect the bearing surface.

According to OSHA regulations (CFR 1926.650 through 1926.652, and Appendix A to Subpart P), the contractor is responsible for developing and maintaining the appropriate safety systems for excavations on the project. The soils should be classified as Type C for this purpose. Recommendations for temporary slopes and/or shoring are beyond the scope of this investigation, but can be provided upon request once more specific design details are available.

After the foundation is placed, the excavation should be properly backfilled. The on-site soils should be suitable for this purpose, following some processing (e.g., mixing and moisture control, etc.) to achieve the specifications previously provided in this section. The fill should be placed in thin lifts (6 inches thick or less before compaction) and compacted thoroughly (to at least 95% of the Standard Proctor Dry Density value) before the next lift is placed. All backfill operations should be monitored and approved by the geotechnical engineer’s representative as part of the Construction Inspection Services.

8. Drilled Shaft Foundations. The deeper, natural soils at this site should be suitable for the support of drilled shaft foundations for relatively heavily loaded buildings or those that have strict settlement criteria. Drilled shafts are especially suitable for resisting the relatively large axial and shear loads and overturning moments typical of steel frame structures. As long as the site preparation and earthwork activities described in Section 6 are followed, grade supported reinforced floor slabs should be able to be used with the drilled shafts. The following general recommendations for shallow foundations can be used for planning purposes for this site.

Straight-sided drilled shafts should be utilized at this site; belled (underreamed) shafts will experience construction difficulties due to the presence of sandy soils and ground water at this site, particularly between about the 8 to 12 foot depth. Excavations for drilled shafts will require the use of full depth drilling slurry and/or temporary steel casing to maintain the sides of the excavations (i.e., prevent caving). Temporary steel casing should be effective if it is extended into the deeper clayey soils and used to "seal off" the shallow water bearing sandy soils. The contractor should be thoroughly experienced with the use of these drilling techniques or significant construction difficulties and/or inadequate shaft sections could result. Refer to Section 8.5 for construction considerations.

8.1 Axial Capacity. The compressive axial capacity of drilled shafts is derived from skin friction at the soil-shaft interface and end bearing. Uplift resistance is provided by skin friction and the buoyant weight of the shaft.

Numerous shaft diameters and embedment depths may be considered in order to allow the project designer to select the most suitable shaft geometry for the specific loading conditions. Representative values for drilled shafts in this area are tabulated below. The allowable shaft capacities include factors of safety of 2 and 2.5 for skin friction and end bearing in compression, respectively, and 2.5 for skin friction in uplift. The buoyant unit weight of the shaft is also included in the provided uplift capacities, along with a factor of safety of 1.1. Capacities for intermediate diameters and/or depths may be interpolated from the table. Extrapolation beyond the specified diameters and depths is not recommended without further consultation.

Typical Allowable Compression/Uplift Loads for Single Drilled Shaft Foundations (kips)

Depth* (ft)	18 Inch Diameter	24 Inch Diameter	30 Inch Diameter	36 Inch Diameter
10	18 / 12	27 / 17	36 / 22	47 / 27
15	33 / 22	47 / 30	64 / 38	82 / 48
20	47 / 34	66 / 46	87 / 59	110 / 73

* Depth refers to depth below existing site grades.

All shaft capacities cited above are based on good quality construction procedures being utilized. Sufficient full depth reinforcement, as determined by the structural engineer, is required to develop the full tensile capacity of the shaft.

8.2 Settlement. Total settlements for drilled shaft foundations designed and constructed in accordance with these recommendations are estimated to be about one-quarter inch or less. Differential settlements between adjacent shafts should be about one-half to three-quarters of the observed total settlement.

8.3 Lateral Loads and Overturning Moments. It is not known if the tops of the drilled shafts will be subject to lateral loads and/or overturning moments, or if these forces will be resisted by the structure itself. The evaluation of lateral loading and overturning moments on drilled shafts can be complex and time consuming for a large number of shaft geometries, such as that provided in Section 8.1. Once the final loading conditions on the drilled shafts are known, this office should be contacted for further evaluation.

8.4 Shaft Spacing and Group Effects. Shafts should be spaced a minimum of 2.5 to 3 diameters center-to-center or 5% of the shaft length, whichever is greater. Large groups of shafts are not anticipated; however, if groups of 5 or more shafts are utilized, the Geotechnical Engineer should be permitted to evaluate group efficiencies.

8.5 Construction Considerations. Excavations for drilled shafts will require the use of full depth drilling slurry and/or temporary steel casing to maintain the sides of the excavations (i.e., prevent caving). Temporary steel casing should be effective if it is extended into the deeper clayey soils and used to “seal off” the shallow water bearing sandy soils. The contractor should be thoroughly experienced with the use of these drilling techniques or significant construction difficulties and/or inadequate shaft sections could result.

Drilling slurry, if utilized, should be introduced into the excavation immediately upon drilling, and maintained at full depth during the drilling and concreting operations. The excavation and concrete placement should proceed as expeditiously as possible. Once the excavation is started, it should be completed and concrete placed without delay. The slurry should be premixed and brought to the proper consistency, etc., before introducing into the excavation. The drilling tools (augers) should be designed such that the slurry can pass freely around or through the tool as the auger is withdrawn, and the auger should be operated slowly enough that suction does not develop beneath the auger and cause caving. The bottom of the excavation should be cleaned out with an air lift pump or similar device; a clean-out bucket is not recommended. Prior to cleanout, the slurry should be allowed to stand undisturbed for about 15 to 30 minutes to allow all suspended solids to settle out.

The reinforcing steel and concrete for the shaft should be placed immediately after the clean out operations are complete. The reinforcing cage should be fixed in place with centralizers or other means so that it is not disturbed by the concrete placement. If temporary steel casing is used achieve a dry excavation, the concrete may be dropped freely through the excavation, provided it is not permitted to strike any obstructions on the way down and does not land in standing water. If this cannot be achieved, a full depth tremie should be utilized to place the concrete. A “head” of concrete of at least 5 feet above the bottom of the casing should be maintained while the temporary casing is withdrawn.

If drilling slurry is utilized, the concrete should be placed by means of a full depth, water-tight tremie with a valve or other means of separating the slurry from the concrete (e.g., a pig). The concrete should be proportioned so that it has the proper strength as determined by the project designers, while maintaining a slump of 6 to 8 inches at the time of placement. This is critical to ensure that the slurry is completely displaced, and that no voids remain within the completed shaft. All drilling and concreting operations should be observed by qualified personnel experienced in drilled shaft inspection techniques.

8.6 Floor Slabs. The floor slabs should consist of ground supported slab-on-grade placed monolithically with exterior and interior grade beams. The grade beams should be designed to rest upon and span across the drilled shaft foundations. The exterior grade beams should extend to a minimum depth of 2 feet below exterior finished grade to help minimize moisture fluctuations of the soils supporting the floor slab. The interior grade beams may be placed at any convenient depth as required by the structural considerations for the floor slab system. Sufficient reinforcement (for both positive and negative moments) and control joint spacing, as determined by the Structural Engineer, should be utilized.

OTHER GEOTECHNICAL CONSIDERATIONS

9. Drainage. Proper long term drainage should be provided to direct surface water away from the completed building foundations. Gutters and downspouts, as well as positive site grading, should be utilized for this purpose as required.

10. Additional Consulting Services. The Geotechnical Engineer should be kept informed of and permitted to address all aspects of the soils-related aspects of the project. Often, concerns may arise that are not specifically addressed by the Geotechnical Engineering Report. A brief conference can often address any such concerns, and can identify any other issues not anticipated by the design team.

Upon completion of design, and prior to the start of construction, the Geotechnical Engineer should be provided with the opportunity to review the design drawings and specifications to assure compliance with the Geotechnical Engineering Report. Such review is considered to be an integral part of the recommendations of this report.

11. Construction Inspection Services. Construction inspection services for this project are essential to assure that the soil conditions do not vary from that assumed in this report and to ensure that the recommendations in this report are followed. These services should be retained by the owner to assure that unbiased reporting is provided. The Geotechnical Engineer should be provided with timely copies of all test results.

12. Limitations. This report is based upon the information provided by the owner's representative, as well as the soil and ground water conditions encountered during the field investigation. Variations may occur away from or between the borehole locations. If such variations become apparent, or if the nature of the project changes significantly, the Geotechnical Engineer should be consulted for additional recommendations. It is understood that additional study will be required to provide specific foundation recommendations once more detailed design information is available.

The recommendations in this report pertain only to the soils-related aspects of the project. The structural design of the building foundations is beyond the scope of these services. Likewise, this report does not address the environmental aspects of the project. We would be pleased to assist with these additional services if requested.

APPENDIX

U.S.G.S. Topographic Map / Site Vicinity Map (Figure 1)

Google Earth Aerial Photograph and Boring Location Plan (Figure 2)

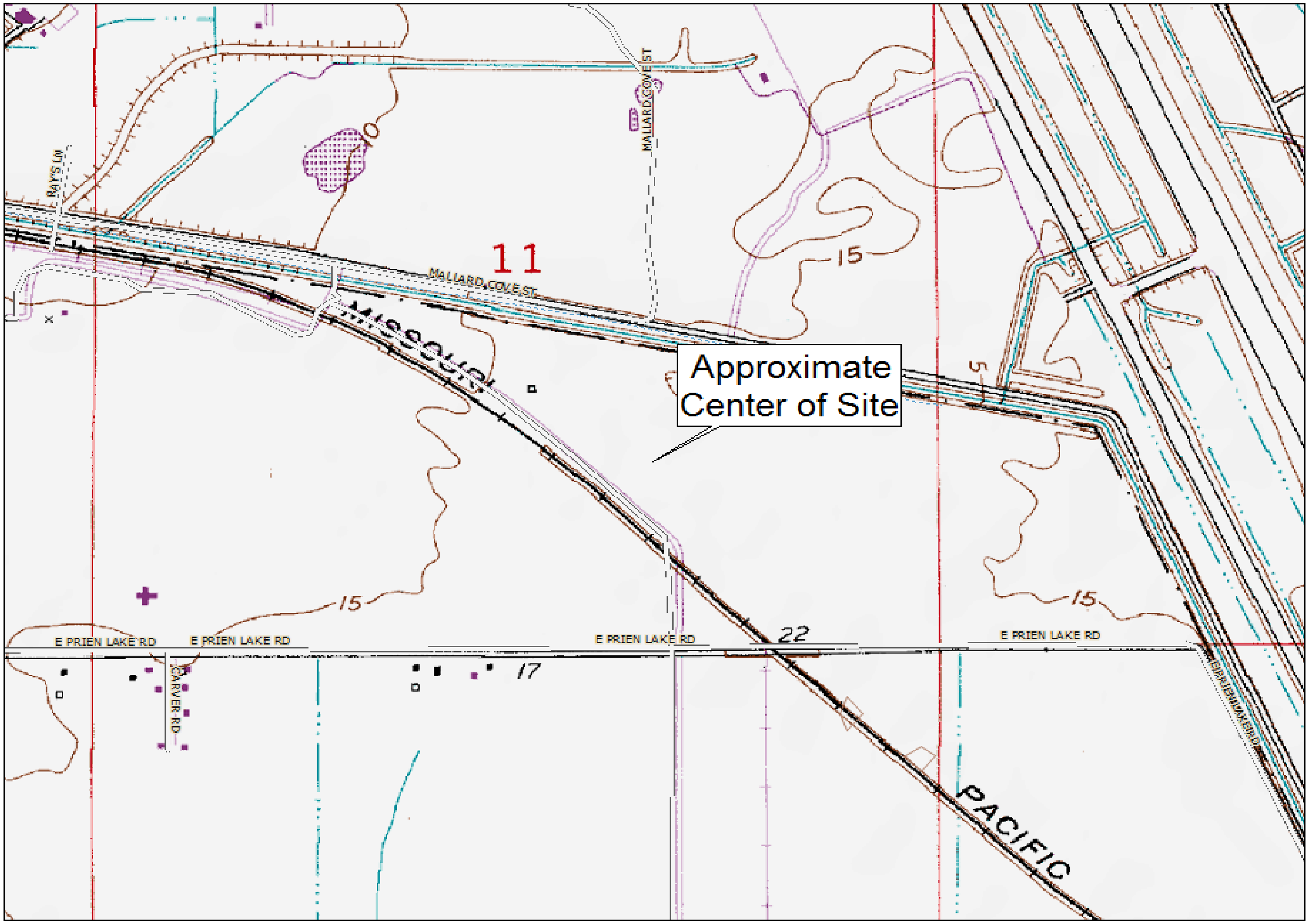
Soil Boring Logs (5)

Particle Size Analyses (2)
(Figures PSA-1 and PSA-2)

Description of Field and Laboratory Testing Procedures

DeLORME

Topo North America™ 9



MN (1.4° E)

0 500 1000 1500 ft

Data Zoom 14-3

Daniel J. Holder, P.E., Inc. Consulting Civil / Geotechnical Engineer 2767 Scarborough Drive Lake Charles, LA 70615 (337) 274-4125 dan@danholderpe.com	Chennault Sites 2 & 2A Lake Charles, Louisiana for SJB Group, LLC Baton Rouge, Louisiana	Project Engineer: DJH Drawn By: dan Checked By: DJH	DJH File No. 14-109 Date: 22 Dec 14 Figure No. 1
		Site Vicinity Map / U.S.G.S. Topographic Map	

Source: U.S.G.S. 7.5 Minute Topographic Map, 1999 (3-D TopoQuads, DeLorme)

NOTE: Borings B-1 and B-2 Made For This Investigation;
Borings B-3, B-4, and B-5 Made For DJH File 11-045

Daniel J. Holder, P.E., Inc. Consulting Civil / Geotechnical Engineer 2767 Scarborough Drive Lake Charles, LA 70615 (337) 274-4125 dan@danholderpe.com	Chennault Sites 2 & 2A Lake Charles, Louisiana for SJB Group, LLC Baton Rouge, Louisiana	Project Engineer: DJH Drawn By: dan Checked By: DJH	DJH File No. 14-109 Date: 22 Dec 14 Figure No. 2
		Google Earth® Aerial Photograph and Boring Location Plan	

Source: Site Plan Provided By SJB Group, LLC

SOIL BORING LOG

Boring No. B-1

Project: Chennault Sites 2 / 2A
Location: East Prien Lake Road
Lake Charles, Louisiana
Client: SJB Group, LLC
Baton Rouge, Louisiana

DJH File No: 14-109
Date Drilled: 11/24/2014
Logged By: Mike Fogarty
Drilled By: Masa Drilling, Inc.
Equipment: Ardco Top Drive (Buggy)

Depth (ft)	Field Tests: Sample Type	Field Tests: Penetrometer (tsf) or SPT (bpf)	Field Tests: Ground Water	Laboratory Tests: Qu / UU (tsf)	Dry Density, γd (pcf)	Moisture Content, w (%)	Atterberg Limits: Liquid Limit, %	Atterberg Limits: Plastic Limit, %	Atterberg Limits: Plasticity Index, %	Notes / Other Tests	Symbol	Description
1	ST	1½ tsf	▽ (after 15 min)			22	46	16	30	PSA 1		FILL - Firm brown & dark brown very SILTY CLAY (CL), w/ roots & tan & light gray silty clay pockets
2												
3	ST	2 tsf	▽ (first encountered)	1.3	110	20	51	16	35	ε_f = 6.4%		Stiff light gray & tan CLAY (CH), w/ brown oxides & light gray silt streaks
4												
5	ST	1¾ tsf		1.1	113	19				ε_f = 7.1%		- ditto, w/ large calcium nodules @ 5'
6												
7	ST	1½ tsf		0.7	113	19				ε_f = 3.6%		- ditto, firm
8												
9	ST	1¼ tsf										Firm gray w/ brown SILTY fine SAND (SM) w/ brown oxides, wet
10												
11	SS	34 bpf 9-18-16										- dense, brown
12												
13												Firm reddish brown w/ light gray CLAY (CH), w/ gray silt lenses / fractures & slickensides
14	ST	2¼ tsf		0.9	98	29	65	24	41	ε_f = 1.1%		
15												
16												
17												
18												
19	ST	2½ tsf										- ditto
20												
21												
22												Stiff dark brown w/ light gray CLAY (CH), w/ gray silt lenses / fractures & sand lenses & small shells
23												
24	ST	1½ tsf		1.5	89	35	83	30	53	ε_f = 2.9%		
25												Boring Completed at 25' Depth

Boring Data

Boring Advancement:
Dry Auger: 0 - 2'
Rotary Wash: 2' - 25'
Boring Abandonment:
Boring Backfilled w/ Soil Cuttings Upon Completion

Ground Water Data

▽ First Encountered: 3'
▽ After 15 Minutes: ½'
Boring Caved to 6' After 15 Mins.

Sample Type:
ST: Shelby Tube (ASTM D 1587)
SS: Split Spoon (ASTM D 1586)

Notes / Other Tests

ε_f = Failure Strain
PSA = Particle Size Analysis (ASTM D 422)
(refer to Figure PSA 1 in Appendix)

Soil Stratification is Approximate

Daniel J. Holder, P.E., Inc.
Consulting Civil / Geotechnical Engineer
2767 Scarborough Drive
Lake Charles, LA 70615
(337) 274-4125
dan@danholderpe.com

SOIL BORING LOG

Boring No. B-2

Project: Chennault Sites 2 / 2A
Location: East Prien Lake Road
Lake Charles, Louisiana
Client: SJB Group, LLC
Baton Rouge, Louisiana

DJH File No: 14-109
Date Drilled: 11/24/2014
Logged By: Mike Fogarty
Drilled By: Masa Drilling, Inc.
Equipment: Ardco Top Drive (Buggy)

Depth (ft)	Sample Type (Field Tests)	Penetrometer (tsf) or SPT (bpf) (Field Tests)	Ground Water (Field Tests)	Qu / UU (tsf) (Laboratory Tests)	Dry Density, γd (pcf)	Moisture Content, w (%)	Liquid Limit, % (Atterberg Limits)	Plastic Limit, %	Plasticity Index, %	Notes / Other Tests	Symbol	Description
1	ST	1½ tsf				19	36	15	21	PSA 2		Firm dark gray w/ brown SILTY CLAY (CL), w/ brown oxides
2												
3	ST	2 tsf		1.1	106	22	54	17	37	ε_f = 9.3%		Stiff light gray & tan CLAY (CH), w/ brown oxides, black oxide nodules, & gray silt streaks
4			▽ ▽									
5	ST	2 tsf		2.0	116	18	36	15	21	ε_f = 10%		Stiff to very stiff light gray & tan SILTY CLAY (CL), w/ brown oxides, large black oxide nodules & gray silt streaks
6												
7	ST	2¼ tsf										- ditto @ 7'
8												
9	ST	2¾ tsf		1.7	100	26				ε_f = 6.4%		
10												Stiff reddish brown CLAY (CH) w/ tan sand lenses
11	ST	3 tsf		1.4	97	28	60	23	37	ε_f = 3.6%		- reddish brown w/ light gray @ 11'
12												
13												
14	ST	3 tsf										- ditto
15												
16												
17												Stiff medium to dark gray CLAY (CH), w/ brown oxides
18												
19	ST	1¼ tsf		1.1	91	33	50	22	28	ε_f = 5.7%		
20												
21												
22												
23												
24	ST	1 tsf										- ditto
25												Boring Completed at 25' Depth

Boring Data

Boring Advancement:
Dry Auger: 0 - 4'
Rotary Wash: 4' - 25'
Boring Abandonment:
Boring Backfilled w/ Soil Cuttings Upon Completion

Ground Water Data

▽ First Encountered: 4½'
▽ After 15 Minutes: 4½'
Boring Caved to 5' After 15 Mins.

Sample Type:
ST: Shelby Tube (ASTM D 1587)
SS: Split Spoon (ASTM D 1586)

Notes / Other Tests

ε_f = Failure Strain
PSA = Particle Size Analysis (ASTM D 422)
(refer to Figure PSA 2 in Appendix)

Soil Stratification is Approximate

Daniel J. Holder, P.E., Inc.
Consulting Civil / Geotechnical Engineer
2767 Scarborough Drive
Lake Charles, LA 70615
(337) 274-4125
dan@danholderpe.com

SOIL BORING LOG

Boring No. B-3

Project: 343 Acre Tract - Southwest Property
Location: Chennault International Airport
Lake Charles, Louisiana
Client: Chennault International Airport Authority
Lake Charles, Louisiana

DJH File No: 11-045
Date Drilled: 7/22/2011
Logged By: Dan Holder
Drilled By: Triangle Resources, Inc.
Equipment: Ardco Top Drive (Buggy)

Boring Location: N 30° 12.103'; W 93° 08.579'

Depth (ft)	Field Tests: Sample Type	Field Tests: Penetrometer (tsf) or SPT (bpf)	Field Tests: Ground Water	Laboratory Tests: Qu / UU (tsf)	Dry Density, γd (pcf)	Moisture Content, w (%)	Atterberg Limits: Liquid Limit, %	Atterberg Limits: Plastic Limit, %	Atterberg Limits: Plasticity Index, %	Notes / Other Tests	Symbol	Description
1	ST	2 tsf										Firm gray CLAYEY SILT (CL-ML), w/ roots
2		4½+ tsf										Very stiff light gray w/ tan very SILTY CLAY (CL), dry
3	ST	2¾ tsf										Stiff light gray & tan SANDY CLAY (CL), w/ silt pockets
4												
5	ST	1½ tsf		1.4	108	22				ε_f = 10%		
6		3¾ tsf										
7	ST	2½ tsf										-ditto
8		1½ tsf										Stiff reddish brown & gray SILTY CLAY (CL), w/ sand layers & oxides
9	ST	2½ tsf	▽									
10												
11	ST	2¼ tsf		1.4	98	29				ε_f = 3.6%		-ditto, grading to CLAY (CH)
12												
13												
14												Stiff gray CLAY (CH), w/ lots of shells
15	ST	2¾ tsf										
16												
17												
18												
19	ST	1¼ tsf										-ditto
20												
21												
22												
23												
24	ST	1 tsf		0.8	93	30	43	15	28	ε_f = 10%		-ditto
25												Boring Completed at 25' Depth

Boring Data

Boring Advancement:
Dry Auger: 0 - 10'
Rotary Wash: 10' - 25'
Boring Abandonment:
Boring Backfilled w/ Soil Cuttings Upon Completion

Ground Water Data

▽ First Encountered: 10'
▽ After 15 Minutes: dry
Boring Caved to 9' After 15 Mins

Sample Type:

ST: Shelby Tube (ASTM D 1587)
SS: Split Spoon (ASTM D 1586)

Notes / Other Tests

ε_f = Failure Strain

Soil Stratification is Approximate

Daniel J. Holder, P.E., Inc.
Consulting Civil / Geotechnical Engineer
2767 Scarborough Drive
Lake Charles, LA 70615
(337) 274-4125
dan@danholderpe.com

SOIL BORING LOG

Boring No. B-4

Project: 343 Acre Tract - Southwest Property
Location: Chennault International Airport
Lake Charles, Louisiana
Client: Chennault International Airport Authority
Lake Charles, Louisiana

DJH File No: 11-045
Date Drilled: 7/22/2011
Logged By: Dan Holder
Drilled By: Triangle Resources, Inc.
Equipment: Ardco Top Drive (Buggy)

Boring Location:
N 30° 12.157'; W 93° 08.970'

Depth (ft)	Sample Type	Penetrometer (tsf) or SPT (bpf)	Ground Water	Qu / UU (tsf)	Dry Density, γd (pcf)	Moisture Content, w (%)	Liquid Limit, %	Plastic Limit, %	Plasticity Index, %	Notes / Other Tests	Description
1	ST	1¾ tsf									Firm brown SANDY SILT (ML), w/
2		½ tsf									roots
3	ST	1¾ tsf		2.6	114	18				ε_f = 10%	Stiff light gray & tan SANDY CLAY
4											(CL), w/ black oxides
5	ST	2 tsf									-ditto
6											
7	ST	1½ tsf									-ditto, very sandy
8											
9	ST	1 tsf									Stiff reddish brown w/ light gray very
10			▽								SILTY CLAY (CL), w/ black oxides
11	ST	1½ tsf		1.2	97	30	47	19	28	ε_f = 2.9%	-SILTY CLAY (CL) & CLAY (CH)
12											
13											
14											
15	ST	1¼ tsf									-ditto
16		No Test									Stiff gray CLAY (CH) & SHELLS
17											
18		5 bpf									
19	SS	2-2-3									-firm dark gray CLAY (CH), w/ shells
20											
21											
22											
23											
24	ST	½ tsf		0.9	87	36				ε_f = 5.0%	-ditto
25											Boring Completed at 25' Depth

Boring Data

Boring Advancement:
Dry Auger: 0 - 10'
Rotary Wash: 10' - 25'
Boring Abandonment:
Boring Backfilled w/ Soil
Cuttings Upon Completion

Ground Water Data

▽ First Encountered: 10'
▽ After 5 Minutes: dry
Boring Caved to 9' After 5 Mins

Sample Type:

ST: Shelby Tube (ASTM D 1587)
SS: Split Spoon (ASTM D 1586)

Notes / Other Tests

ε_f = Failure Strain

Soil Stratification is Approximate

Daniel J. Holder, P.E., Inc.
Consulting Civil / Geotechnical Engineer
2767 Scarborough Drive
Lake Charles, LA 70615
(337) 274-4125
dan@danholderpe.com

SOIL BORING LOG

Boring No. B-5

Project:	343 Acre Tract - Southwest Property	DJH File No:	11-045
Location:	Chennault International Airport	Date Drilled:	7/22/2011
	Lake Charles, Louisiana	Logged By:	Dan Holder
Client:	Chennault International Airport Authority	Drilled By:	Triangle Resources, Inc.
	Lake Charles, Louisiana	Equipment:	Ardco Top Drive (Buggy)

Boring Location:
N 30° 11.980'; W 93° 09.174'

Depth (ft)	Field Tests: Sample Type	Field Tests: Penetrometer (tsf) or SPT (bpf)	Field Tests: Ground Water	Laboratory Tests: Qu / UU (tsf)	Laboratory Tests: Dry Density, γd (pcf)	Laboratory Tests: Moisture Content, w (%)	Atterberg Limits: Liquid Limit, %	Atterberg Limits: Plastic Limit, %	Atterberg Limits: Plasticity Index, %	Notes / Other Tests	Symbol	Description
1	ST	<¼ tsf										Soft, wet, gray CLAYEY SILT (CL-ML)
		2 tsf										
2												Stiff light gray & tan SANDY CLAY
3	ST	2 tsf										(CL), w/ black oxides
4												
5	ST	1 tsf		0.9	111	19				ε_f = 10%		-ditto, very sandy
6												
7	ST	1½ tsf										-firm, very sandy
8		11 bpf										Medium dense light gray to reddish
9	SS	6-6-5										brown SILTY fine SAND (SM)
10			▽									
11	ST	1½ tsf										Stiff reddish brown CLAY (CH), w/
12												silty clay layers, sand lenses &
13												black oxides
14												
15	ST	1½ tsf		1.8	95	31	59	21	38	ε_f = 3.6%		-ditto
16												Firm reddish brown CLAYEY fine
17												SAND (SC), w/ shells
18		½ tsf										-ditto
19	ST	1¼ tsf										Stiff gray CLAY (CH), w/ some shells
20												
21												
22												
23												
24	ST	½ tsf		1.1	86	37				ε_f = 3.6%		-stiff, dark gray, w/ occasional shells
25												Boring Completed at 25' Depth

Boring Data

Boring Advancement:
- Dry Auger: 0 - 10'
- Rotary Wash: 10 - 25'

Boring Abandonment:
- Boring Backfilled w/ Soil Cuttings Upon Completion

Ground Water Data

- ▽ First Encountered: 10'
- After 5 Minutes: dry

Boring Caved to 8½' After 5 Mins

Sample Type:

ST: Shelby Tube (ASTM D 1587)
SS: Split Spoon (ASTM D 1586)

Notes / Other Tests

ε_f = Failure Strain

Soil Stratification is Approximate

Daniel J. Holder, P.E., Inc.
Consulting Civil / Geotechnical Engineer
2767 Scarborough Drive
Lake Charles, LA 70615
(337) 274-4125
dan@danholderpe.com

Particle Size Analysis (ASTM D 422)

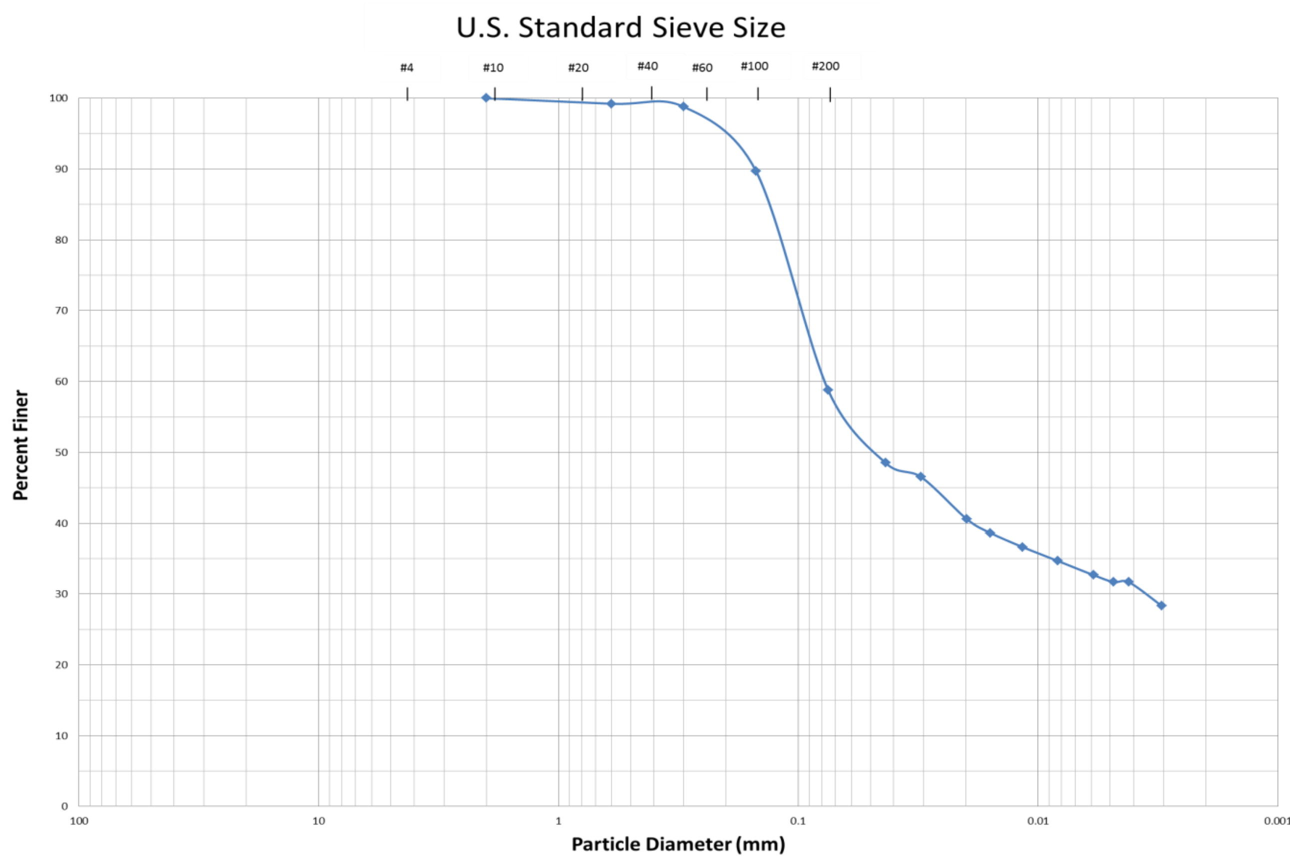
Sample Location: B-1, 0' - 2'

Sample Description: FILL - Firm brown & dark brown very SILTY CLAY (CL), w/ roots & light gray silty clay pockets

Particle Size (Sieve)	Particle Size (mm.)	Percent Finer By Wt.
1½"	38.100	
¾"	19.050	
⅜"	9.525	
#4	4.750	
#10	2.000	100
#30	0.600	99
#50	0.300	99
#100	0.150	90
#200	0.075	59
	0.043	49
	0.031	47
	0.020	41
	0.016	39
	0.012	37
	0.008	35
	0.006	33
	0.005	32
	0.004	32
	0.003	28

Atterberg Limits	
Liquid Limit, LL:	46
Plastic Limit, PL:	16
Plasticity Index, PI:	30
Moisture Content, w:	22

Gravel	Sand		Silt	Clay
	Coarse to Medium	Fine		



Daniel J. Holder, P.E., Inc. Consulting Civil / Geotechnical Engineer 2767 Scarborough Drive Lake Charles, LA 70615 (337) 274-4125 dan@danholderpe.com	Chennault Sites 2 & 2A Lake Charles, Louisiana for SJB Group, LLC Baton Rouge, Louisiana	Project Engineer: DJH Drawn By: Checked By: DJH	DJH File No. 14-109 Date: 22 Dec 14 Figure No. PSA-1
		Particle Size Analysis B-1, 0' - 2'	

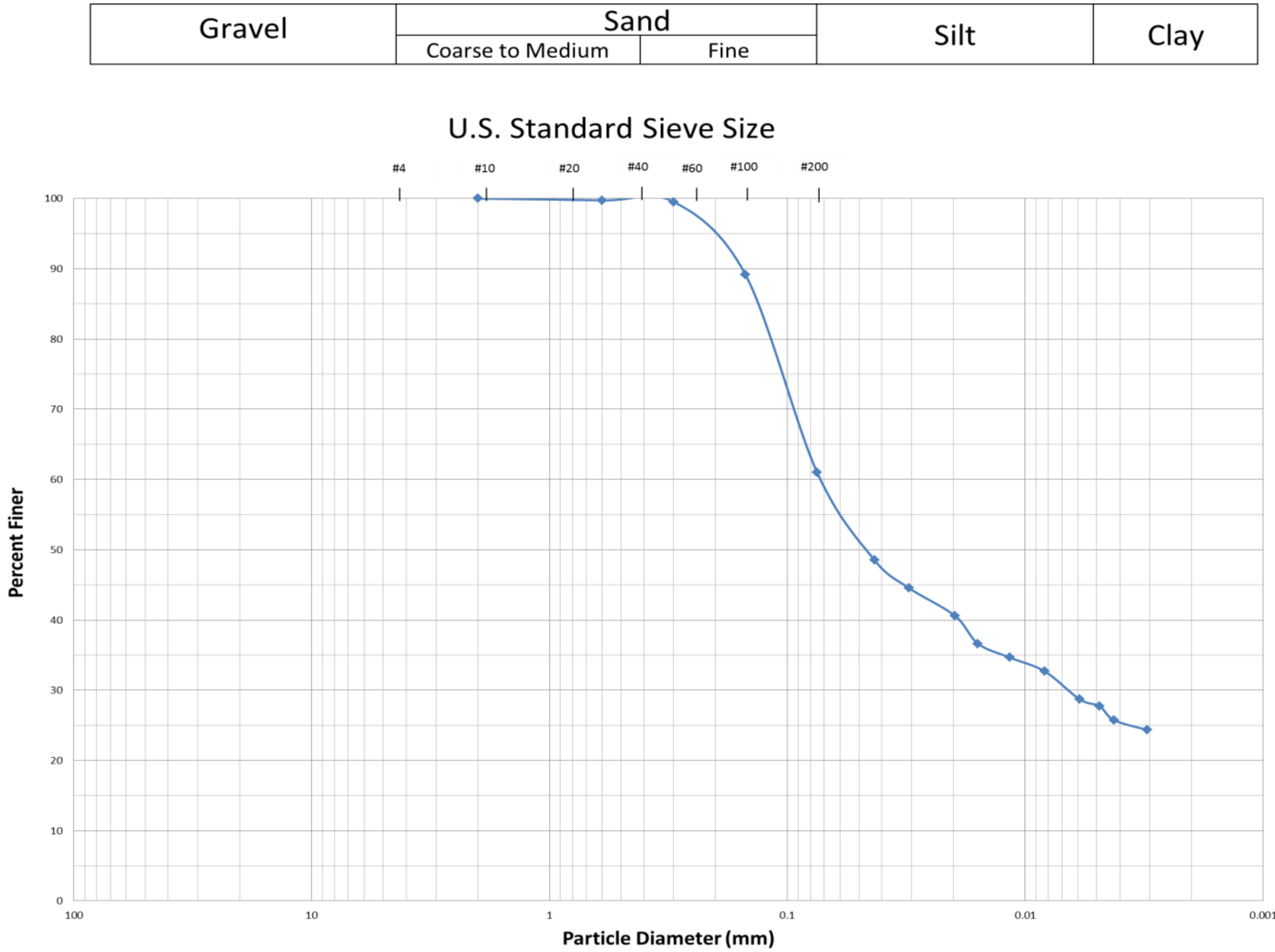
Particle Size Analysis (ASTM D 422)

Sample Location: B-2, 0' - 2'

Sample Description: Firm dark gray w/ brown SILTY CLAY (CL), w/ brown oxides

Particle Size (Sieve)	Particle Size (mm.)	Percent Finer By Wt.
1½"	38.100	
¾"	19.050	
⅜"	9.525	
#4	4.750	
#10	2.000	100
#30	0.600	100
#50	0.300	100
#100	0.150	89
#200	0.075	61
	0.043	49
	0.031	45
	0.020	41
	0.016	37
	0.012	35
	0.008	33
	0.006	29
	0.005	28
	0.004	26
	0.003	24

Atterberg Limits	
Liquid Limit, LL:	36
Plastic Limit, PL:	15
Plasticity Index, PI:	21
Moisture Content, w:	19



Daniel J. Holder, P.E., Inc.
Consulting Civil / Geotechnical Engineer
2767 Scarborough Drive
Lake Charles, LA 70615
(337) 274-4125 dan@danholderpe.com

Chennault Sites 2 & 2A
Lake Charles, Louisiana
for
SJB Group, LLC
Baton Rouge, Louisiana

Project Engineer: DJH	DJH File No. 14-109
Drawn By:	Date: 22 Dec 14
Checked By: DJH	Figure No. PSA-2

Particle Size Analysis
B-2, 0' - 2'

Description of Field and Laboratory Testing Procedures

Field Testing Procedures. The borings were (initially) advanced using dry augering methods. Soil samples were obtained continuously in the upper 10 foot and on 5 foot centers thereafter. The sample depths and types are recorded on the soil boring logs.

In general, relatively undisturbed "Shelby" tube samples (ASTM D 1587) were taken in clays and silty clays. Undisturbed soil samples are required for strength and density tests, and other properties that are dependent upon the soil being close to its natural state. In this procedure, the boring is advanced to the desired sampling depth, then a 3 inch diameter, thin-walled "Shelby" tube is inserted into the borehole. The tube is then pushed hydraulically about 2 feet into the undisturbed soil. The tube is withdrawn, and the sample extruded with a hydraulic piston. The sample is visually classified and tested with a spring loaded penetrometer, which provides a crude estimate of the unconfined compressive strength. The penetrometer test result is recorded on the soil boring log, and a representative portion of the sample is secured for transport to the laboratory.

In sands and silts, Standard Penetration Tests (ASTM D 1586) are generally made. This test provides a measure of the in-situ density or stiffness of the soil and provides a relatively disturbed sample that may be used for classification testing. In this procedure, the boring is advanced to the desired sampling depth, and a relatively heavy walled "split spoon" sampler is inserted into the borehole. The sampler is driven into the soil using a 140 pound "drop" hammer with 30 inch strokes. The number of blows required to drive each 6 inch increment is recorded. The first increment is a seating drive; the number of blows required to drive the second and third increments are added together to determine the "N-value," which has units of blows per foot (bpf). The N-value and the number of blows per increment are recorded on the soil boring log. The sample is visually classified, and a representative portion secured for transport to the laboratory.

Laboratory Testing Procedures. Representative samples from the field investigation were selected by the project engineer for laboratory testing to determine their relevant engineering characteristics. These tests generally fall into one of the following categories.

Strength Tests. Strength tests generally consist of the Unconfined Compressive Strength, or Qu Test, (ASTM D 2166), and the Unconsolidated, Undrained Triaxial Compressive Strength, or UU Test, (ASTM D 2850). In each of these tests, a cylindrical sample of undisturbed soil is subjected to an axial load until failure occurs, yielding the compressive strength of the soil. The principal difference between the two tests is that the Qu is not confined laterally, which can lead to premature failure, and thus, lower compressive strength values. The UU test is confined laterally in a triaxial cell, typically to the lateral stress that the in-situ soil sample was subject to. The compressive strength and axial strain at failure (ε_f) are recorded on the soil boring log. The confining stress of UU tests is also recorded.

Classification Tests. Common classification tests include the Atterberg Limit Tests and Particle Size Analyses. Atterberg Limit Tests (ASTM D 4318) are performed to determine the consistency (or "clayeyness") of a soil. The Atterberg limits consist of the Liquid Limit (LL) and the Plastic Limit (PL), and the Plasticity Index (PI), which is the difference between the LL and the PL. These values are recorded on the soil boring log.

The Particle Size Analysis Test (ASTM D 422) is performed to determine the distribution of the individual particle sizes of a soil sample. The test is typically performed using mechanical sieves for soils containing gravel and sands, or a "hydrometer" for clayey and silty soils. The results of the Particle Size Analysis are typically plotted on a log scale.

Physical Tests. Common physical tests include the Moisture Content Test (ASTM D 2216) and the Dry Density Test. As the names indicate, these tests determine the moisture content and dry density (or dry unit weight) of a soil sample.