PRELIMINARY
SUBSURFACE EXPLORATION AND GEOTECHNICAL ENGINEERING EVALUATION
AERO PARK SITE
SHREVEPORT REGIONAL AIRPORT
SHREVEPORT, LOUISIANA
P.O. NO. 2015-00004850

PREPARED
FOR:
SHREVEPORT AIRPORT AUTHORITY
5103 HOLLYWOOD AVENUE
SUITE 300
SHREVEPORT, LOUISIANA 71109

PREPARED
BY:
ARDAMAN & ASSOCIATES, INC.
7222 GREENWOOD ROAD
SHREVEPORT, LOUISIANA 71119

ARDAMAN PROJECT NO.: 113-15-94-8609
SHREVEPORT FILE NO.: 15.94.062
AUGUST 17, 2015

Ardaman & Associates, Inc.

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Ardaman & Associates, Inc.

Geotechnical, Environmental and
Materials Consultants

August 17, 2015

Shreveport Airport Authority
5103 Hollywood Avenue
Suite 300
Shreveport, Louisiana 71109

Attention: Mr. Steven Price

Reference: Preliminary Geotechnical Investigation
Aero Park Site
Shreveport Regional Airport
Shreveport, Louisiana
Ardaman Project No.: 113-15-94-8609
Shreveport File No.: 15.94.062
P.O. NO. 2015-00004850

Gentlemen:

Attached is Ardaman & Associates, Inc. (AAI) Preliminary Geotechnical Investigation Report for the above referenced site. AAI would be pleased to assist you further by furnishing any subsequent site or client specific geotechnical studies that may be needed on the property in the future. It has been a pleasure to perform this work for you. If we can be of any further assistance, please do not hesitate to call on us.

Very truly yours,

ARDAMAN & ASSOCIATES, INC.

James M. Belt

James M. Belt, P.E.
Branch Manager
Shreveport Area Operations

STATE OF LOUISIANA
JAMES M. BELT
REG. No. 27126
REGISTERED
PROFESSIONAL ENGINEER
8-17-15
IN
CIVIL ENGINEERING

cc: Mr. Dennis Dean, P.E.
Aillet, Fenner, Jolly, & McClelland, Inc.

PRELIMINARY
SUBSURFACE EXPLORATION AND GEOTECHNICAL ENGINEERING EVALUATION
AERO PARK SITE
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GENERAL

AAI's proposal P15.26.062, dated May 21, 2015 was verbally approved on behalf of the Shreveport Airport Authority (SAA) by Steven Price and subsequently confirmed by receipt of the City of Shreveport's Purchase Order Number 2015-00004850. The purposes of the study were to (1) explore the subsurface conditions present at this site, (2) determine the pertinent engineering properties of the materials encountered, (3) characterize site soil and groundwater conditions, and (4) determine if unfavorable soil conditions exist at the site. Item 4 is general in nature and each boring has brief analysis and conditions presented.

PROJECT DESCRIPTION

The SAA's Site Certification Master Plan map provided to AAI indicates the Warehouse district property consists of about 93 acres. The property is located on the east side of the Regional Airport's property near the city of Shreveport's Airport Park property. The SAA is seeking to have this property certified for the Louisiana Economic Development (LED) "Certified Site" program. This report is, in part, a requirement of the program.

FIELD OPERATIONS

The geotechnical investigation consisted of a total of 3 test borings, more or less randomly located on the property. All test borings were advanced to a depth of approximately 25 feet below the existing ground surface. This investigation was conducted on July 16, 2015. Boring locations were pre-selected by the geotechnical engineer, but not staked in the field. Our drill crew estimated boring placement using the maps provided, referencing identifiable landmarks, and visually estimating positions on the property. Minor adjustments to the pre-selected map locations may have been made where site access was difficult or obstructions may have existed. A copy of the client provided site map along with a *Google™earth* aerial map indicating the approximate location of the test borings are included in appendix A of this report.

The test borings were advanced utilizing continuous-flight, solid stem augers and samples were obtained for laboratory evaluation in general accordance with provisions of *ASTM D1586* and *ASTM D1587*. Standard, thin-walled, seamless Shelby tube samplers were used to obtain specimens of cohesive materials. These specimens obtained at intervals of 5 feet as the borings were advanced.

Soils which contained enough cohesionless material or were sufficiently dense to prevent recovery of undisturbed specimens with Shelby Tube samplers were evaluated by means of the Standard Penetration test. This test consists of determining the number of blows required by a 140 pound hammer dropped 30 inches to achieve one foot penetration of the soil. This number is then related to "in situ" density of the material.

All samples obtained were logged, sealed and packaged in the field to protect them from disturbance and maintain their in situ moisture content during transportation to our laboratory. The results of our boring program (Logs of Boring) are included as Appendix "A" of this report.

LABORATORY TESTING

Upon return to our laboratory selected samples were subjected to standard laboratory tests under the supervision of a soils engineer. The Atterberg Limits, in situ unit weights, percent of material passing a #200 sieve, and moisture contents of the different subsurface soils were determined. These soil properties were used to classify the soils and evaluate their potential for volumetric change. Standard Penetration testing performed in the field or unconfined compression tests performed in laboratory was used to evaluate the shear strength of the different subsurface materials. The results of our testing program are included on the Logs of Boring in Appendix "A" of this report.

SOIL CONDITIONS

Soil conditions described in this section are of a generalized nature and intended to emphasize key features and characteristics. For a more detailed description of the subsurface materials encountered refer to the soil profile on each Log of Boring in Appendix "A". Strata contacts indicated on our Logs are approximate. Actual transitions may be gradual in nature. The soils described are at the specific boring locations within the depths explored. Soils at other locations or depths may be different than those encountered during this exploration. Considering the substantial distances between the boring locations, soil conditions are summarized for each individual test boring location.

Test Boring B-1.

Geotechnical laboratory testing performed on selected samples collected from this location indicates generally sandy soils with medium to high plasticity. The surface soils are sandy clays that classify *sandy fat clay* per *ASTM D2487, Standard Practice for Classification of Soils for Engineering Purposes (Unified Soils Classification System).* Strength consistency of this surficial stratum is stiff. Between the depths of 3 and 15 feet, low to medium plastic sands were encountered. These sand strata classify *clayey sand (SC), clayey silty sand (SC-SM), and* silty *sand (SM).* Relative density varies between medium dense and dense. Below the 15 foot depth, *fat clays (CH)* and *clayey sand (SC)* were again encountered. The boring was terminated in *clayey sand (SC)* at a depth of 25 feet below the existing ground surface (BGS).

Test Boring B-2.

Geotechnical laboratory testing performed on selected samples collected from this location indicates sandy soils with highly variable plasticity. The surface soils are slightly plastic sands that classify *clayey silty sand (SC-SM)* per *ASTM D2487, Standard Practice for Classification of Soils for Engineering Purposes (Unified Soils Classification System).* Relative density of this surficial stratum is medium dense. Between the depths of 3 and 10 feet, high to medium plastic clays were encountered. These clay strata classify *sandy fat clay (CH) and sandy lean clay (CL).* Strength consistency of the clays are medium stiff to stiff. Below the 10 foot depth, *clayey sand (SC)* was encountered. The boring was terminated in *clayey sand (SC)* at a depth of 25 feet BGS.

Test Boring B-3.

Geotechnical laboratory testing performed on selected samples collected from this location indicates sandy soils with highly variable plasticity. The surface soils are non-plastic silt that classifies *sandy silt (ML)* per *ASTM D2487, Standard Practice for Classification of Soils for Engineering Purposes (Unified Soils Classification System).* Relative density of this surficial stratum is medium dense. Between the depths of 3 and 15 feet, high to medium plastic clays were encountered. These clay strata classify *sandy fat clay (CH) and sandy lean clay (CL).* Strength consistency of the clays vary from very soft to very stiff. Below the 15 foot depth, very dense *clayey sand (SC)* and hard *silty clay (CL-ML)* were encountered. The boring was terminated in *silty clay (CL-ML) stratum* at a depth of 25 feet BGS.

GROUNDWATER

Shallow groundwater was encountered in all of the test boring locations during the drilling operations. Our water level observations, in feet BGS, are summarized in Table 1. These levels represent our estimation of initial encounter depth and should not be construed as static or equilibrium levels. Static levels may in fact be higher (or lower) than our initial observations. It must be understood shallow groundwater levels will fluctuate with site elevation, climatic conditions/seasons of the year, and the levels of any nearby streams and ponds. Based on the stratigraphy encountered at these sites, shallow groundwater will not likely adversely impact surface construction activities. However excavation operations below a depth of about 10 feet may encounter water seepage and construction activities would be impacted.

TABLE 1.

GROUNDWATER OBSERVATIONS

Test Boring Location	Groundwater Depth (feet BGS)
B-1	22
B-2	13.5
B-3	12.5

GENERAL SUBGRADE PREPARATION

Although none are expected, prior to subsequent construction activity any existing foundations, buried structures, or pavements in the proposed construction area should be demolished and removed. Any utilities with in the proposed construction area that are to be abandoned should be completely removed or plugged in-place. Top soil stripping in grassed areas should be expected. The contractor should anticipate stripping and de-grassing will require removal of the upper 4 to 6 inches of surface material. The contractor should provide drainage of the exposed subgrade by sloping grades and ditching away from the construction site.

After any required demolition and rough site grading are complete, the exposed surface of areas where fill or paving are to be placed should be proof rolled to identify any weak areas. Proofrolling can be accomplished with a loaded dump truck or similarly weighted equipment under the observation of the geotechnical engineer or his designated representative. Weak areas should be investigated, removed, and/or repaired under the supervision of the geotechnical engineer prior to subsequent construction activity. After verification of a stable subgrade layer, the exposed subgrade should be scarified to a minimum of 8 inches, the moisture content adjusted to within one 1 percent below to 3 percent above optimum and recompacted to 90 percent of the laboratory maximum as determined by *ASTM D1557, Laboratory Compaction Characteristics of Soil Using Modified Effort (56,000 ft-lbf/ft^3)* prior to placement of any fill or base materials.

It is imperative the contractor understand the importance of establishing and maintaining proper site drainage to maintain the prepared subgrade's suitability for subsequent construction activity. Failure to follow good site water control practices may result in construction delays and additional costs for reconditioning or replacing saturated materials. Earthwork performed during wet periods of the climatic cycle may warrant special considerations. The use of hydrated lime, fly ash or Portland cement stabilization should be considered to provide a working platform if work is expected to occur during these periods. The need for such techniques is dependent upon earthwork scheduling with respect to weather patterns and good site management of drainage during the construction phase. The suitability of any area left exposed to the elements should be verified by proofroll prior to subsequent construction activity.

GENERAL FILL RECOMMENDATIONS

Where fill materials may be required to achieve the desired finished grade elevations of any future projects, the material should be placed in controlled lifts. Lifts should be placed in thin horizontal layers not exceeding 8 inches *compacted* thickness. In sloped areas, horizontal lifts should be benched or stair stepped into the slope as the wedge of fill is constructed. Each lift of select fill should be moisture conditioned to 2 percentage points of optimum moisture and compacted to a minimum of 95 percent of the laboratory maximum as determined by ASTM D1557.

All imported fill material should be *"select"*. Select materials classify *clayey sand (SC) or sandy lean clay (CL)* in accordance with ASTM D2487. Select fill materials placed below grade should be sandy lean clay with liquid limits no greater than 35, plasticity indices (PI) between 8 and 18, and have no more than 60 percent passing the U.S. Standard No. 200 Sieve.

Onsite soils classifying CL, SC, SC-SM, SM, or SP-SM, free of organic materials or construction debris, are suitable for use as fill with adequate processing and moisture conditioning. Soils classifying ML, CH or those otherwise of suitable classification, but containing organics, construction debris or excessive moisture are not recommended for reuse as fill under structures or pavements. Typical specifications for compaction of sandy clay and clayey sand type soils are included in Appendix "B" or this report.

GENERAL FOUNDATION RECOMMENDATIONS

The soils discovered at the test boring locations on this site are of fair bearing quality but generally have slight to moderate potential for shrink and swell. Some of the near surface soils are considered active or *expansive*. Some clay soils that would be considered expansive were encountered near the existing ground surface in the locations of test borings B-1 and B-2. These soils exist within 5 feet of the existing ground surface. A soft layer was encountered in the location of test boring B-3 below the 3 foot depth. These soils will likely fail the required proofroll and require removal and replacement with density controlled select or suitable on site soils.

As such, with attention to the fore mentioned expansive clays and soft soils, or with certain site modifications, light to moderately loaded future developments could be supported on shallow foundation systems. Our preliminary estimate of allowable bearing capacities for continuous and spread footings placed in the native undisturbed soils or density controlled select fill, at depths between 2 and 4 feet BGS, are summarized in Table 2 for each of the test boring locations. Capacities include a minimum factor of 3 against shear failure of the bearing stratum. Where the sites are properly prepared and maintained, and allowable bearing capacities are not exceeded, total settlement from consolidation should be limited to an inch or less for typical spread footings with widths less than 5 feet and continuous footings with widths less than 3 feet.

To utilize a conventionally reinforced shallow foundation system, AAI recommends a minimum 4 feet of density controlled select fill or inactive native soil beneath the bottom of all footings and/or grade beams and any underlying active or expansive clay soil. A minimum of 6 feet is recommended beneath any *non-load bearing* floor slab sections. To meet these criteria, the proposed finished floor elevation can be raised, any existing expansive soils can be removed, or a combination of both procedures can be used to achieve an acceptable finished floor elevation with the required thickness of inactive fill beneath it.

For the soil profiles at B-1 and B-2, it appears the criteria could be met with a minimal amount of site modifications. AAI strongly recommends that our office review the preliminary grading plans so that we can finalize our recommendations regarding treatment of the fat clay soils with in the proposed building area. The Geotechnical Engineer should retained to inspect the exposed subgrade and verify the assumptions of this report before proceeding with fill material placement.

For heavy loading, where consolidation settlements may be critical, a deep foundation system would be more suitable. Probably the most economical deep foundation system to support heavily loaded structures is with an auger-cast-in-place (ACIP) pile system. The test boring sites are ideal for utilizing ACIP piles to support heavier loads or to minimize settlement. All areas explored can support heavily loaded structures on an ACIP pile foundation. Although generally less economical, driven timber piles, concrete piles, or steel piles can also be used. For moderate loads, straight sided drilled and cast-in-place concrete caissons (drilled shafts) or helical screw anchor type piers can also be utilized.

A seismic site classification of Class D, as defined in the International Building Code Section 1613, should be assumed for this site due to the lack of specific soil data to a depth of one hundred (100) feet.

TABLE 2.

ALLOWABLE BEARING CAPACITIES FOR SHALLOW FOUNDATIONS[1]

Test Boring Location	Allowable Net Bearing for Continuous Footings (psf)	Allowable Net Bearing for Spread Footings (psf)
B-1	3000	3500
B-2	3000	3500
B-3	3000	3500

The recommendations contained in this section are our opinion based on the limited amount of information available and are intended for preliminary planning purposes only. Prior to any final design, a thorough site/project specific geotechnical study should be made to finalized design parameters for specific areas of the Technology Park property.

[1] Table values assume site modifications including undercutting unsuitable soils and replacement with density controlled fill will be required at all test boring locations.

GENERAL PAVEMENT INFORMATION

The pavement section recommendations for this site are based upon subsurface conditions implied by the test borings and the assumption use will be limited to commercial or light industrial use. No specific pavement analyses was performed as there is no known specific usage at this time. Recommendations outlined here in are in our opinion, "typical" for the assumed site use and intended for preliminary planning purposes only.

The existing sandy clay and clayey sand subgrade soils, prepared and maintained as recommended in the Subgrade Preparation Section of this report should have a laboratory soaked California Bearing Ratio (CBR) values in the order of 5 to 10 or Modulus of Subgrade Reaction (k_s) in the order of 100 to 150 PCI.

Generalized specifications for recommended crushed aggregate base materials and geotechnical fabrics are included in Appendix "B" of this report. Aggregate base course layers in excess of four (4) inches in thickness should be compacted to not less than 98% of the laboratory maximum as determined by *ASTM D1557, Method C.* Layers of 4 inches or less can be compacted by establishing a rolling pattern under the direction of the geotechnical engineer that produces the maximum density.

Where a soil-cement sub-base is considered, the type soils recommended for use as *select* fill or the prepared subgrade can be readily stabilized with Type I Portland cement. The cement stabilized soil or "soil-cement" sub-base layer should achieve a minimum unconfined compressive strength of 300 PSI at 7 days of age. Eight percent by volume cement can be used for cost estimation for cement stabilization of select soils. The actual quantity required should be verified by the geotechnical engineer during the construction phase of the project in accordance with Louisiana Department of Transportation and Development Test Method TR 432.

Construction of the soil-cement subgrade layer should be in accordance with the provisions outlined in Section 303 of the *Louisiana Standard Specifications for Roads and Bridge, 2006 Edition.* Compaction of the finished subbase layer should not be less than 95% of the maximum laboratory density as determined by LDOTD TR 418. Heavy construction traffic should not utilize cement stabilized areas until the materials have cured sufficiently to obtain minimum specified strength.

Rigid Pavement - AAI recommends rigid pavement be considered for all heavy duty pavement applications. However we understand the need to make cost effective decisions for the facility. At a minimum turnout aprons, trash collection areas, and any parking aprons adjacent to the truck docks be constructed of rigid pavement.

Minimum flexural strength of the concrete should be 650 pounds per square inch (PSI) at twenty-eight (28) days of age or have compressive strength value of 4,000 PSI. AAI recommends the use of air entrainment chemicals that improve workability of the concrete mix and improve durability of the pavement surface. Control joint spacing should not exceed 15 feet for un-reinforced pavement of the thicknesses outlined below. All concrete paving should include provisions to mechanically control temperature induced shrinkage cracking and provide for load transfer across construction joints. Rigid pavement sections suggested for this site are summarized in Table 3.

TABLE 3.

RIGID PAVEMENT SECTIONS

Pavement Layer	Light Duty Auto Parking	Medium Duty Channelized Auto Drives	Heavy Duty Truck Access Drive, Dumpster Pads, & Turnout Aprons
Portland Cement Concrete	5.0"	6.0"	8.0"
Base Course Layer	4" crushed stone base material	4" crushed stone base material	8" crushed stone base material
Subbase Course Layer	Density controlled fill per Fill Section of this report	Density controlled fill per Fill Section of this report	Density controlled fill per Fill Section of this report
Subgrade Layer	Density controlled subgrade prepared per the Subgrade Section of this report	Density controlled subgrade prepared per the Subgrade Section of this report	Density controlled subgrade prepared per the Subgrade Section of this report

Flexible Pavement – Flexible paving structurally similar to the above rigid sections are provided for your cost comparison. Hot mixed asphaltic concrete (HMAC) mixtures should meet applicable requirements for materials, production, placement and acceptance as outlined in the *Louisiana Standard Specifications for Roads and Bridges, 2000 Edition*, Section 501 for Marshall mixtures or *LSSRB, 2006,* Section 502 for level 1 Superpave mixtures. For parking lot and light duty drive applications we recommend utilizing the ½ inch Nominal HMAC mix of either type. This mix produces a more aesthetic surface finish and generally holds up well under automobile parking lot use. Flexible pavement sections suggested for this site are summarized in Table 4.

TABLE 4.

FLEXIBLE PAVEMENT SECTIONS

Pavement Layer	Light Duty Auto Parking	Medium Duty Auto Drives	Heavy Duty Parking
Hot Mixed Asphaltic Concrete Wearing and Binder Course	2.0" WC	1.5" WC 1.5" BC	1.5" WC 3.0" BC
Base Course	8" crushed stone base material or 8" soil-cement base materials	8" crushed stone base material or 8" soil-cement base materials	12" crushed stone base material or 12" soil-cement base materials
Geotechnical Fabric Layer Requirement	Required under aggregate base material	Required under aggregate base material	Required under aggregate base material
Subbase Course	Density controlled fill per Fill Section of this report	Density controlled fill per Fill Section of this report	Density controlled fill per Fill Section of this report
Subgrade Layer	Density controlled subgrade prepared per the Subgrade Section of this report	Density controlled subgrade prepared per the Subgrade Section of this report	Density controlled subgrade prepared per the Subgrade Section of this report

If a soil-cement base layer is to be considered, be aware soil-cement materials develop tension cracks during the curing process and these cracks "reflect" through the overlying HMAC paving over time. A general rule of thumb for crack propagation is about one (1) vertical inch per year (takes about 2 years to show up through a 2 inch overlay). Although not initially structurally detrimental to the pavement system, the cracks must be periodically sealed to minimize moisture infiltration into the base system. Failure to preform regular maintenance of the cracks can lead to saturated subgrade soils and premature base failures in the pavement.

Reflective cracking cannot be prevented; however a separation layer can be used to minimize the propagation of reflective cracking. There are commercially available engineered fabrics (underlayment) that claim to reduce crack propagation and a thin layer of crushed aggregate base layer can also be used between the base layer and binder course layers to reduce reflective cracking. Both approaches have pros and cons and the benefit of either must be weighed against installation costs.

CONSTRUCTION CONCERNS

The upper soils at the site are fine-grained materials composed of significant silt and clay fractions. Silty and/or clayey soils are subject to changes in shear strength with varying moisture conditions. If construction is initiated during wetter seasons of the year, it may be difficult to move equipment about the site. Once these type soils become saturated, compaction operations can be hampered by a tendency of the silt to "pump" and the clay to "shear".

Consequently it is recommended, adequate site drainage be established prior to, during, and following construction operations to prevent water ponding on or adjacent to construction areas. Compaction operations may be expedited by using light compaction equipment and thin lifts of soil. Rolling only as necessary to obtain compaction is advisable because further repetitive loading may cause the subgrade to "pump" or fail. Once soils begin to pump, it is usually necessary to either start the moisture conditioning process over or remove and replace the saturated material. AAI can provide experience soils technicians to monitor the contractor's compaction operations and assist in expediting the site work.

Compaction operations and installation of the foundations should be supervised by a qualified soils technician under the supervision of the Geotechnical Engineer. All foundation excavations should be inspected to verify cleanliness and adequate bearing. Concrete should be placed in foundation excavations as soon as practical after forming and final clean-up have been approved, to avoid prolonged exposure of the bearing stratum and possible disturbance due to standing water, desiccation or other construction operations.

LIMITATIONS

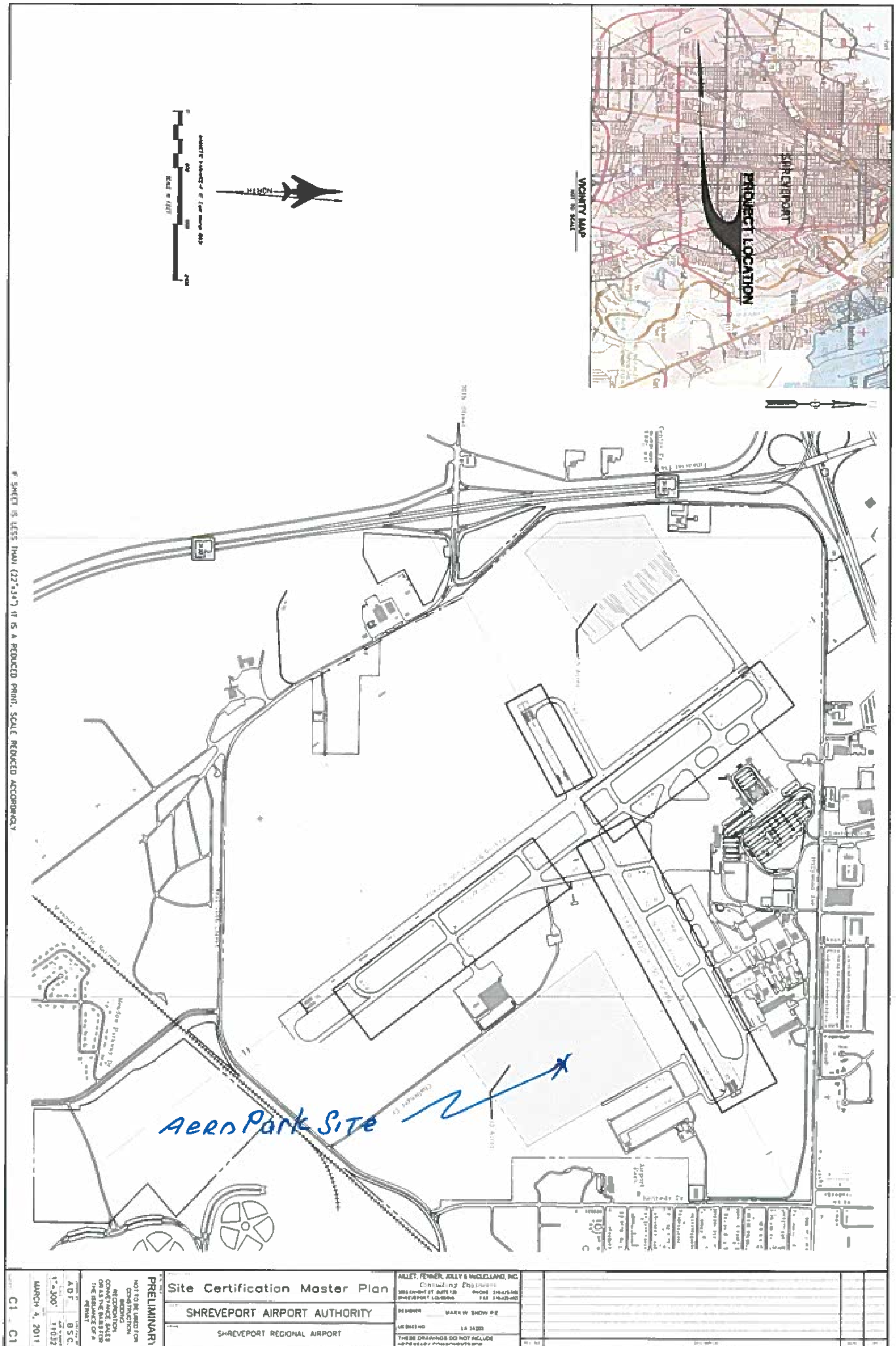
This study has been prepared in accordance with generally accepted preliminary geotechnical engineering principles and practices in this area at this time. We make no other warranty either express or implied.

The conclusions and recommendations submitted in this report are based upon the data obtained from the preliminary exploratory borings drilled at the location(s) indicated in Appendix A, the different possible type of construction, and our experience in the area. Our findings include interpolation and extrapolation of the subsurface conditions identified at the exploratory boring(s) and variations in the subsurface conditions may not become evident until excavations are performed.

This study has been prepared for the exclusive use by our client for preliminary purposes only. We are not responsible for technical interpretations by others of our exploratory information, which has not been described or documented in this report. As the site is eventually developed additional borings should be taken. Significant design changes could be required or modifications of the recommendations presented herein. We recommend on-site observation of excavations and foundation bearing strata by a representative of the geotechnical engineer.

APPENDIX A

SITE MAP
AND
LOGS OF BORING



SHREVEPORT
PROJECT LOCATION
VICINITY MAP
NOT TO SCALE
NORTH
Aero Park Site
IF SHEET IS LESS THAN (22"x34") IT IS A REDUCED PRINT. SCALE REDUCED ACCORDINGLY
PRELIMINARY
NOT TO BE USED FOR CONSTRUCTION BIDDING RECORDATION CONVEYANCE SALES OR AS THE BASIS FOR THE ISSUANCE OF A PERMIT
Site Certification Master Plan
SHREVEPORT AIRPORT AUTHORITY
SHREVEPORT REGIONAL AIRPORT
ALLET, FENNER, JOLLY & McCLELLAND, INC.
Consulting Engineers
MARK W. SNOW P.E.
LA 24203
THESE DRAWINGS DO NOT INCLUDE
ADF
BSC
1"=300'
11032
MARCH 4, 2011
C1 - C1
REVISIONS

TEST BORING LOCATIONS

AERO PARK SITE
SHREVEPORT REGIONAL AIRPORT
SHREVEPORT, LOUISIANA

LOG OF BORING NO. B-1

PROJECT: Shreveport Aero Park | **SHEET** 1 of 1

CLIENT: City of Shreveport Engineering Dept. | **LOCATION:** See Remarks

DATE: 7/16/15 | **SURFACE ELEV:** 231' +/-

DRILLING METHOD(S): Auger

GROUNDWATER INFORMATION: Water encountered at twenty-three (23) feet after 20 minutes stayed at twenty-two (22) feet

FIELD DATA				LABORATORY DATA									
SOIL & ROCK SYMBOL	DEPTH (FT)	SAMPLE TYPE	N: SPT, BLOWS/FT T: THD, BLOWS/FT P: HAND PEN, TSF	MOISTURE CONTENT, %	DRY DENSITY POUNDS/CU.FT	LIQUID LIMIT, %	PLASTIC LIMIT, %	PLASTICITY INDEX, %	MINUS NO. 200 SIEVE, %	COMPRESSIVE STRENGTH, KSF	FAILURE STRAIN (%)	CONFINING PRESSURE PSI	DESCRIPTION OF STRATUM
	0–3.0	Split-spoon	N=18	17		51	21	30	61				Stiff red and tan sandy fat clay (CH) — 3.0
	3.0–6.5	Split-spoon	N=19	14		25	20	5	40				Medium dense clayey silty sand (SC-SM) — 6.5
	6.5–11.5	Split-spoon	N=31	18		38	18	20	42				Dense red and tan clayey sand (SC) — 11.5
	11.5–16.5	Split-spoon	N=18	27									Medium dense tan sandy silt (ML) — 16.5
	16.5–21.5	Split-spoon	N=17	31		60	27	33	90				Stiff gray fat clay (CH) with fine sand partings — 21.5
	21.5–25.0	Split-spoon	N=46	27									Dense tan clayey sand (SC) — 25.0
	25.0												Bottom of boring at 25 feet

TUBE SAMPLE	AUGER SAMPLE	SPLIT-SPOON	ROCK CORE	THD CONE PEN.	NO RECOVERY

REMARKS: N 32 degrees 26 58.43 W 93 degrees 49 03.71

15.94.062

ARDAMAN & ASSOCIATES, INC.

LOG OF BORING NO. B-2

PROJECT: Shreveport Aero Park — SHEET 1 of 1

CLIENT: City of Shreveport Engineering Dept. — LOCATION: See Remarks

DATE: 7/16/15 — SURFACE ELEV: 230' +/-

DRILLING METHOD(S): Auger

GROUNDWATER INFORMATION: Water encountered at fifteen (15) feet after 20 minutes stayed at thirteen and one-half (13.5) feet

FIELD DATA				LABORATORY DATA										
SOIL & ROCK SYMBOL	DEPTH (FT)	SAMPLE TYPE	N: SPT, BLOWS/FT T: THD, BLOWS/FT P: HAND PEN, TSF	MOISTURE CONTENT, %	DRY DENSITY POUNDS/CU.FT	LIQUID LIMIT, %	PLASTIC LIMIT, %	PLASTICITY INDEX, %	MINUS NO. 200 SIEVE, %	COMPRESSIVE STRENGTH, KSF	FAILURE STRAIN (%)	CONFINING PRESSURE PSI	DESCRIPTION OF STRATUM	
			N=10	11		20	16	4	52				Medium dense red and tan clayey silty sand (SC-SM)	3.0
	5		N=9	23		64	24	40	67				Medium stiff red and tan sandy fat clay (CH)	6.5
	10		N=16	26		45	19	26	71				Stiff gray sandy lean clay (CL)	11.5
	15		N=6	32		34	20	14					Loose tan and gray clayey sand (SC)	
	20		N=12	34									--Medium dense	
	25		N=18	32					21					25.0
	30												Bottom of boring at 25 feet	

TUBE SAMPLE	AUGER SAMPLE	SPLIT-SPOON	ROCK CORE	THD CONE PEN.	NO RECOVERY

REMARKS: N 32 degrees 26 47.37
W 93 degrees 49 07.45

LOG OF BORING NO. B-3

PROJECT: Shreveport Aero Park | SHEET 1 of 1

CLIENT: City of Shreveport Engineering Dept. | LOCATION: See Remarks

DATE: 7/16/15 | SURFACE ELEV: 227' +/-

DRILLING METHOD(S): Auger

GROUNDWATER INFORMATION: Water encountered at twelve and one-half (12.5) feet depth

DEPTH (FT)	N: SPT, BLOWS/FT; T: THD, BLOWS/FT; P: HAND PEN, TSF	MOISTURE CONTENT, %	DRY DENSITY POUNDS/CU.FT	LIQUID LIMIT, %	PLASTIC LIMIT, %	PLASTICITY INDEX, %	MINUS NO. 200 SIEVE, %	COMPRESSIVE STRENGTH, KSF	FAILURE STRAIN (%)	CONFINING PRESSURE PSI	DESCRIPTION OF STRATUM
0–3.0	N=13	10		18	16	2	57				Medium dense tan sandy silt (ML) — 3.0
3.0–6.5	N=3	29		49	22	27	66				Very soft red and gray sandy lean clay (CL) — 6.5
6.5–11.5	N=15	36		89	33	56	81				Medium stiff gray fat clay (CH) with sand — 11.5
11.5–16.5	N=27	31									Very stiff tan and gray sandy lean clay (CL) — 16.5
16.5–21.5	N=44	29									Dense gray clayey sand (SC) — 21.5
21.5–25.0	N=54	26		27	23	4	87				Hard gray silty clay (CL-ML) with sand — 25.0
25.0											Bottom of boring at 25 feet

Sample types: TUBE SAMPLE | AUGER SAMPLE | SPLIT-SPOON | ROCK CORE | THD CONE PEN. | NO RECOVERY

REMARKS: N 32 degrees 26 49.28
W 93 degrees 48 54.90

15.94.062

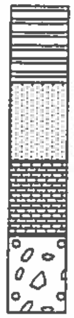
ARDAMAN & ASSOCIATES,INC.

KEY TO SOIL CLASSIFICATION TERMS AND SYMBOLS

SOIL OR ROCK TYPES


SAND
SILT
CLAY
FILL


SANDY
SILTY
CLAYEY
ORGANIC


SHALE
SANDSTONE
LIMESTONE
GRAVEL

SAMPLER TYPES

SHELBY TUBE	DISTURBED (AUGER)	SPLIT SPOON	NO RECOVERY
DENISON	PISTON	PITCHER	ROCK CORE

CONSISTENCY OF COHESIVE SOILS (MAJOR PORTION PASSING NO. 200 SIEVE)

DESCRIPTIVE TERM	UNDRAINED SHEAR STRENGTH, KIPS/SQ FT
VERY SOFT	LESS THAN 0.25
SOFT	0.25 TO 0.5
FIRM	0.5 TO 1.0
STIFF	1.0 TO 2.0
VERY STIFF	2.0 TO 4.0
HARD	GREATER THAN 4.0

RELATIVE DENSITY OF GRANULAR SOILS (MAJOR PORTION RETAINED ON NO. 200 SIEVE)

DESCRIPTIVE TERM	RELATIVE DENSITY,%
VERY LOOSE	LESS THAN 15
LOOSE	15 TO 35
MEDIUM DENSE	35 TO 65
DENSE	65 TO 85
VERY DENSE	GREATER THAN 85

WATER LEVELS

▼ - DEPTH GROUNDWATER FIRST ENCOUNTERED DURING DRILLING

▽ - GROUNDWATER LEVEL AFTER 24 HOURS (UNLESS OTHERWISE NOTED)

TERMS DESCRIBING SOIL STRUCTURE

Parting: paper thin in thickness

Seam: 1/8" - 3" in thickness

Layer: greater than 3" in thickness

Calcareous: containing appreciable quanties of calcium carbonate

Ferrous: containing appreciable quantities of iron

Well-graded: having wide range in grain size & similar proportions of all intermediate sizes

Poorly graded: predominately one grain size or having a range of sizes with few or no particles of some intermediate sizes

Fissured: containing shrinkage cracks, frequently filled with fine sand or silt, usually more or less vertical

Interbedded: composed of alternate layers of different soil types

Laminated: composed of thin layers of varying color and texture

Slickensided: having inclined planes of weakness that are slick & glossy in appearance

NOTE: Clays possessing slickensided or fissured structure may exhibit lower measured shear strength than indicated by the described consistency. The consistency of such soil is interpreted using the measured shear strength along with pocket penetrometer results.

APPENDIX B

MATERIAL PROCEDURES AND SPECIFICATIONS

B.1 GENERALIZED SPECIFICATIONS FOR COMPACTION

Sandy Clay and Clayey Sand Soils

The thickness of lifts used should be no more than the height of the teeth on sheepsfoot rollers. Generally, for a forty-eight (48) inch diameter or smaller drum roller, the maximum compacted lift thickness acceptable is six (6) inches. For rollers with drums of sixty (60) inches in diameter and larger with teeth about nine (9) inches long, a nine (9) inch final compacted lift thickness will be acceptable. The sole determination of the thickness of a lift will be the capability of the contractor's equipment to obtain the required compaction.

When obtaining the average density of a lift to determine its conformance to specifications, the lift should be immediately rejected if any density is more than 2% below the required average.

Generally, sheepsfoot rollers are most suitable for compaction of sandy clay and clayey sand soils, the contractor may use spiketooth rollers, rubber tired rollers, or any fill compaction equipment that has sufficient mass to compact the soil. Generally, the drums of sheepsfoot rollers should be filled with water or for additional weight with both water and sand. Tractors or other vehicles used primarily for hauling WILL NOT be allowed as fill compaction equipment. The contractor should also have smooth wheel rollers to seal the working area at the end of the day's operations so overnight rains will not saturate the soil and delay his work. These rollers should also be used to seal the surface whenever rainfall is imminent.

The soil engineer or his representative will perform density tests and will accept or reject a lift within two (2) hours after being tested. No material will be placed on any lift that has not been accepted by the engineer.

B.2 GENERALIZED COARSE AGGREGATE SPECIFICATIONS

Crushed Stone Crushed Concrete	Crushed stone base course shall be composed of crusher-run broken stone. The material shall be crushed and consist of durable particles of stone mixed with approved soil binder material.
Gradation	The base material shall meet the following requirements:

Pass #1-1/2"	100%
Pass #1"	90-100%
Pass #3/4"	70-100%
Pass #4	35-65%
Pass #40	12-32%
Pass #200	5-12%

Soil Binder	Material passing the No. 40 sieve shall be known as "soil binder" and shall meet the following requirements: Plasticity Index < 15
Compaction	Compaction shall be obtained by a minimum of 12 passes of a 5,000 pound sheepsfoot roller 3 to 4 feet wide. Surface shall be finished rolled by sufficient passes of a steel wheel roller to provide a smooth surface for application of the surface course.
Note	Extra binder material may be added with the approval of the geotechnical or design engineer. Soundness and Los Angeles abrasion tests should meet Louisiana Department of Transportation Specifications.

B.3 GENERIZED GEOTEXTILE FABRIC SPECIFICATIONS

The following proven woven Geotextile Fabrics are approved:

1. Amoco Pro Pex 2006
2. Beltech Style 980
3. ConTech C300
4. Mirafi 600X
5. Hanes (Terra Tex) HD

If alternate geotextile fabric from above is requested, the following qualifications should be met:

SPECIFICATIONS

Property	**Test Method**	**Minimum Requirements**
Fabric Structure	-	Woven
Polymer Composition	-	Polypropylene
Fabric Width	-	12½', 15', 17½'
Weight	ASTM D-3776C	5 oz. /yd.
Grab Strength	ASTM D-4632	300 x 300 lbs.
Elongation	ASTM D-4632	20%
Trap Tear Strength	ASTM D-4533	115 lbs. x 115 lbs.
Burst Strength	ASTM D-3786	575 psi.
Puncture	ASTM D-4833	120 lbs.
UV Resistance	ASTM D-4355	> 70%
A.O.S.	ASTM D-4751	35

NOTE:

1. Requires Mill Certification from manufacturer.

2. Minimum requirements are not minimum average values. Minimum average values per roll are not an acceptable specification.