GEOTECHNICAL REPORT

FY19 Special Project P-520
Distribution Switchgear ECIP
NAS JRB New Orleans, LA

Contract: N62470-15-D-4002

Task Order: N69450-20-F0050

May 19, 2021

CDM Smith

12400 Coit Road, Suite 400
Dallas, Texas 75251
tel: 214-345-2800

June 2, 2021

Indira Pjetrovic, P.E.
Project Manager
CDM Smith, Inc.
12400 Coit Road, Suite 400
Dallas, Texas 75251

Subject: Geotechnical Verification Letter – Final Submittal
Naval Facilities Engineering Command – Southeast
Distribution Switchgear ECIP Project
NAS JRB NOL – Belle Chasse, Louisiana

Dear Ms. Pjetrovic:

In accordance with the Department of Defense's Facilities Criteria (FC) Design Manual FC 1-300-09N – Chapter 16, Section 2.1, the Geotechnical Designer of Record (DOR) is required to submit a verification letter as part of the Final Submittal, as it relates to the design of the subject project. This letter is intended to provide this verification for the Distribution Switchgear ECIP project at the Naval Air Station (NAS) Joint Reserve Base (JRB) New Orleans Louisiana (NOL) Belle Chasse site.

After completing my review of the Ready to Advertise plans and specifications for this project by June 2, 2021, I find these documents are in general conformance with the findings and design recommendations included in the most recent Geotechnical Report for the project, prepared by CDM Smith, Inc. and dated May 19, 2021. Furthermore, the findings and recommendations provided in this latest version of the geotechnical report remain valid based on my current understanding of the project.

Please let me know if you have any questions or concerns.

Sincerely,

Jeffrey D. Van Pelt, P.E.
Senior Geotechnical Engineer
CDM Smith Inc.

cc: File

Distribution Switchgear ECIP

Belle Chasse, Louisiana

Naval Air Station
Joint Reserve Base New Orleans (NAS JRB NOL)

Geotechnical Design Report

May 19, 2021

Prepared By:

Aravind Pedarla
Geotechnical Engineer

Reviewed By:

STATE OF LOUISIANA
JEFFREY DONOVAN VAN PELT
License No. 42770
PROFESSIONAL ENGINEER
IN
CIVIL ENGINEERING

5-19-2021

Jeffrey D. Van Pelt, P.E.
Senior Geotechnical Engineer

CDM Smith

Table of Contents

Figures

Tables

Appendices

Acronyms

1H:1V	1 horizonal to 1 vertical
ACIP	Augered Cast-In-Place
ATV	All-terrain vehicle
bgs	below the ground surface
CDM Smith	CDM Smith, Inc.
Code	2018 International Building Code
ECIP	Energy Conservation Investment Program
El.	elevation
FEMA	Federal Emergency Management Agency
FFE	finish floor elevation
KV	kilovolt
HWY	highway
NAVFAC	Naval Facilities Engineering Command Southeast
NASJRB	Naval Air Station Joint Reserve Base
NAVD 88	North American Vertical Datum of 1988
OSHA	Occupational Safety and Health Administration
pcf	pounds per cubic foot
PIT	pile integrity testing
psf	pounds per square foot
SPT	standard penetration test
TIP	thermal integrity profiling
TWE	Tolunay-Wong Engineers, Inc.
USCS	Unified Soil Classification System
WOH	Weight of Hammer

Section 1
Introduction

CDM Smith, Inc. (CDM Smith) has been retained by the Naval Facilities Engineering Command Southeast (NAVFAC) to provide professional engineering services for the proposed Distribution Switchgear - Energy Conservation Investment Program (ECIP) project, located at the Naval Air Station Joint Reserve Base (NASJRB) in Belle Chasse, Louisiana. This geotechnical report summarizes subsurface conditions encountered in the test borings and provides geotechnical design recommendations and construction considerations for the proposed location for the Distribution Switchgear ECIP.

1.1 Project Description and Location

The NAS JRB Belle Chasse facility is located north of highway (HWY) 23 on Russell Drive in Belle Chasse, Louisiana. At this facility, an existing 15 kilovolt (KV) switchgear is located south of Enterprise Street and east of Russell Drive. It is understood that this switchgear is proposed to be demolished and a new 15 KV switchgear is to be constructed at a location further interior to the facility, along with the addition of miscellaneous equipment pads across the site. Trenchless installation of electrical utility lines is also understood to be proposed at the site, however geotechnical recommendations for this installation method was not included in CDM Smith's scope.

The 15 KV Distribution Switchgear is proposed to be installed on an at-grade concrete pad, approximately 16 feet by 29 feet in plan dimension, and have a maximum bearing pressure of 450 pounds per square foot (psf). The miscellaneous at-grade exterior equipment pads across the site are understood to be 3 feet x 8 feet in plan dimension, and have a maximum bearing pressure of 200 psf.

The location of the proposed improvements is shown on **Figure 1-1**.

1.2 Datum

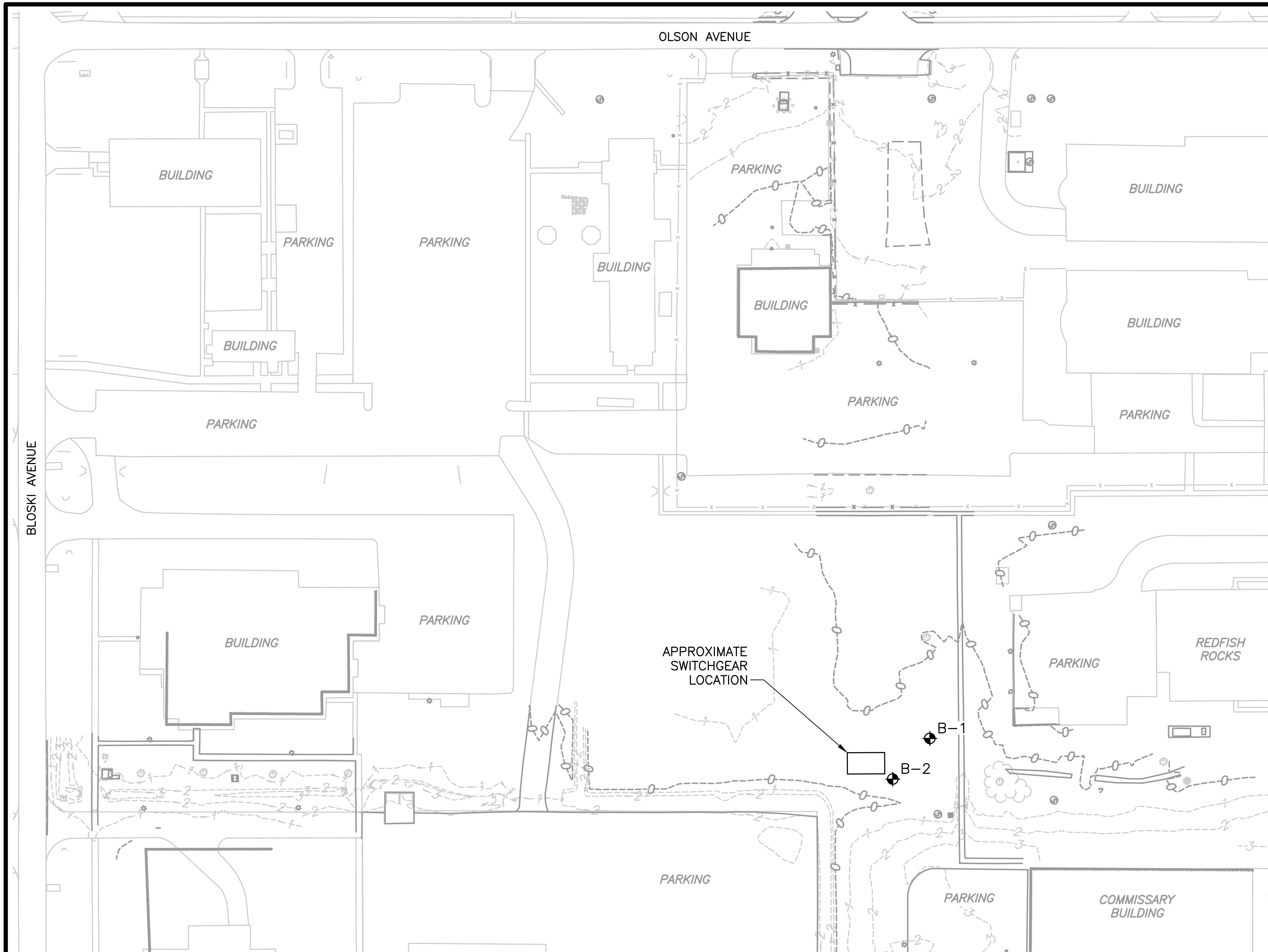
Elevations noted in this report are measured in feet and referenced to the North American Vertical Datum of 1988 (NAVD 88).

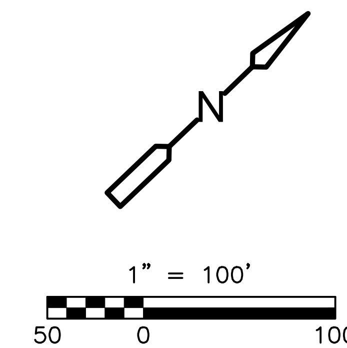
1.3 Purpose and Scope

The purpose of this geotechnical report is to summarize the subsurface conditions encountered in the test borings for the proposed site and provide geotechnical engineering recommendations for design and construction of the new 15 KV Distribution Switchgear.

The scope of the completed work for this project includes the following:

- Reviewing available geological and geotechnical information for the vicinity of the NAS JRB Belle Chasse, Louisiana facility to evaluate geologic features that could impact foundation design for the proposed 15 KV Distribution Switchgear and miscellaneous exterior equipment pads.





LEGEND:

B–1 DESIGNATION AND APPROXIMATE LOCATION OF TEST BORING DRILLED BY TOLUNAY–WONG ENGINEERS, INC. OF ELMWOOD, LOUISIANA ON MAY 19, 2020.

NOTES:

1. BASE PLAN FROM A SURVEY PREPARED BY MARTINEZ COUCH AND ASSOCIATES, LLC OF ROCKY HILL, CONNECTICUT IN JUNE 2020. GIS INFORMATION WAS ADDED OUTSIDE OF SURVEYED AREA.
2. THE AS–DRILLED TEST BORING LOCATIONS WERE LOCATED BY SURVEYOR NOTED ABOVE.
3. ELEVATIONS NOTED HEREIN ARE IN FEET AND REFERENCED TO THE NORTH AMERICAN VERTICAL DATUM OF 1988 (NAVD88).
4. TEST BORINGS WERE OBSERVED AND LOGGED BY TOLUNAY–WONG ENGINEERS, INC.

CDM Smith

NAVAL AIR STATION JRB
BELLE CHASSE, LOUISIANA
DISTRIBUTION SWITCHGEAR ECIP

FIGURE 1-1
BORING LOCATION PLAN
JULY 2020

- Advancing two (2) test borings at the proposed 15 KV Distribution Switchgear site to obtain information on subsurface conditions and collect geotechnical soil samples for laboratory testing,
- Conducting geotechnical laboratory tests on selected soil samples to assist with classification and estimation of engineering properties of these materials,
- Preparing test boring logs to characterize the general site subsurface conditions encountered,
- Performing engineering analyses and developing geotechnical engineering recommendations for the design of the proposed structure foundation; and
- Preparing this geotechnical report presenting CDM Smith's geotechnical recommendations for the proposed improvements associated with the NAS JRB Belle Chasse 15 KV Distribution Switchgear project, including all data used as part of the analyses and recommendations.

1.4 Report Limitations

This geotechnical report has been prepared for the NAS JRB Belle Chase, Louisiana's proposed 15 KV Distribution Switchgear, as understood at this time and described in this report. This report has been prepared in accordance with generally accepted engineering practices. No other warranty, express or implied, is made.

The conclusions and recommendations contained herein should be considered valid design recommendations for the improvements listed herein, unless changes in the design or location of the structures occur, at which time the content of this report should be verified in writing by CDM Smith.

Section 2

Site and Subsurface Conditions

2.1 Existing Site Conditions

2.1.1 Site Location and Description

NAS JRB Belle Chasse was constructed in the 1950s and covers more than 2,000 acres and includes facilities for the U.S. Air Force, U.S. Army, U.S. Navy, U.S. Marines, U.S. Coast Guard, and Louisiana Air National Guard. Existing utilities provide services for approximately 120 facilities spread across approximately 550 acres of the military installation. The NAS JRB Belle Chasse facility is in New Orleans, inside of Plaquemines Parish, and is protected from flood and storm surge by a system of levees and seawalls. The existing 15 KV switchgear is located south of Enterprise Street and east of Russell Drive, just past the main entrance gate.

2.1.2 Site Topography, Features and Boundaries

At the time of the geotechnical field investigation, the NAS JRB Belle Chasse site was comprised of military installations typical of a naval air station, areas of maintained grass, and undeveloped land in the southern portion of the facility. The topography at the site of the proposed 15 KV Distribution Switchgear is relatively flat, with grades ranging between elevation (El.) 0 and El. +1 feet. The switchgear site is generally bounded by paved parking lots and is located approximately 500 feet northwest of the Commissary Building.

2.2 Geotechnical Subsurface Explorations

A subsurface exploration program was conducted to investigate the subsurface conditions at the proposed 15 KV Distribution Switchgear project site at the NAS JRB Belle Chasse facility. The drilling program for the investigation consisted of two (2) test borings, B-1 and B-2, drilled to depths of 100 feet and 30 feet below the ground surface (bgs), respectively. The test borings were drilled by Tolunay-Wong Engineers, Inc. (TWE) of Elmwood, Louisiana, using an all-terrain vehicle (ATV) drill rig on May 19, 2020.

Details of the test boring program, including performance of in-situ geotechnical testing and the final test boring logs are included in TWE's Subsurface Investigation Report, dated June 15, 2020. This report is included in **Appendix A**. The as-drilled test boring locations are shown on Figure 1-1.

2.3 Geotechnical Laboratory Testing

Soil samples obtained from the NAS JRB Belle Chasse 15 KV Distribution Switchgear test boring program were transported to TWE's geotechnical laboratory located in Elmwood, Louisiana. Geotechnical laboratory tests were performed on select soil samples obtained from the test borings following CDM Smith's review of the field test boring logs.

Additional information on the laboratory testing program, including detailed laboratory test results from the recent subsurface investigation, is included in TWE's Subsurface Investigation Report, dated June 15, 2020, in **Appendix A**.

2.4 Subsurface Conditions

In general, the subsurface conditions encountered during the recent TWE test boring program consisted of surficial clayey soils underlain by interbedded layers consisting of peat, clay and sand. A summary of the subsurface conditions encountered is included in **Table 2-1**.

Table 2-1 Summary of Subsurface Investigations

				Approx. Strata Thickness (ft)					Groundwater (ft)	
Test Boring Number	Drill Date	Approx. Ground Surface Elevation[1]	Exploration Depth (ft)	Upper Clay	Peat	Middle Clay	Lower Clay	Lower Sand	Initial Depth Reading[2]	Approx. Groundwater Elevation[1]
B-1	5/19/2020	0.4	100.0	18.0	5.0	25.0	30.0	22	2.5	-2.0
B-2	5/19/2020	1.0	30.0	6.0	NE	24.0	-	-	4	-3.0

Notes:

1. Elevations are in feet and referenced to the North American Vertical Datum of 1988 (NAVD 88).
2. Groundwater depth was recorded after 15 minutes at the time of drilling.

Abbreviations:

NE Indicates strata was not encountered.

2.4.1 Upper Clay

A naturally deposited Upper Clay layer was encountered in both of the test borings drilled. This stratum was encountered at the ground surface and extended to approximately 18 feet bgs in test boring B-1 and until 6 feet bgs in test boring B-2. The soils comprising this stratum generally consisted of moist, very soft to very stiff, brown to gray, CLAY. The Unified Soil Classification System (USCS) classification symbols for the soils generally encountered in this stratum are CH and CL. Based on laboratory strength testing, the undrained shear strength of these soils was measured to be approximately 530 pounds per square foot (psf).

2.4.2 Peat

A Peat stratum was encountered in test boring B-1 directly beneath the Upper Clay stratum. The Peat was encountered in only one soil sample and was approximated to be 5 feet thick. The soils comprising this stratum generally consisted of wet, firm, dark gray, Organic Clay with silt seams. The USCS classification symbol for this type of soils is OH.

2.4.3 Middle Clay

Middle clay stratum was encountered directly beneath the Peat stratum in test boring B-1 and extended to about 48 feet bgs. This stratum was encountered directly beneath the Upper Clay stratum in test boring B-2 and extended till boring termination depths. The soils comprising this stratum generally consisted of wet, very soft, gray, CLAY with varying amounts of fines. The USCS classification symbol for the soils generally encountered in this stratum is CL. Based on laboratory strength testing, the undrained shear strengths of these soils ranged between approximately 350 to 520 psf.

2.4.4 Lower Clay

A Lower Clay stratum was encountered beneath the Middle Clay stratum in boring B-1. This stratum is approximately 30 feet in thickness and encountered in test boring B-1. The soils comprising this stratum generally consisted of wet, soft, gray, Clay with varying amounts of sand. The USCS classification symbols for the soils generally encountered in this stratum are CL and CH. Based on laboratory strength testing, the undrained shear strengths of these soils ranged between approximately 720 to 950 psf.

2.4.5 Lower Sand

A Lower Sand layer was encountered below the Lower Clay layer in boring B-1 and was encountered to be approximately 22 feet thick. Test boring B-1 was terminated in this layer. The soils comprising the Lower Sand stratum generally consisted of moist, loose to very loose, gray, SAND with varying amounts of fines. The USCS classification symbols for the soils generally encountered in this stratum are SM. All standard penetration tests (SPT) performed in this stratum measured blow counts (N-values) as weight-of-hammer (WOH), indicating the sampler penetrated 18 inches into the soil under the weight of only the sampler, drill rods and hammer.

2.5 Groundwater Conditions

Groundwater levels were measured by TWE personnel in the test borings during drilling. The water levels were measured when first encountered in the borings and again after a period of 15 minutes. Based on the water level readings in the test borings, the groundwater level was first encountered between approximately 5 and 7 feet bgs (El. -4.5 to -6.0) and subsequently rose to between approximately 2.5 and 4 feet bgs (El. -2.0 and -3.0).

2.6 Expected Variations in Subsurface Conditions

Interpretation of general subsurface conditions presented herein is based on soil and groundwater conditions observed at the test boring locations reviewed as part of this study. Subsurface conditions may vary between test boring locations. If conditions encountered during construction vary from those encountered in the test borings, the recommendations contained in this report should be re-evaluated by CDM Smith and confirmed in writing.

Water levels measured in the open boreholes should not necessarily be considered to represent stabilized groundwater levels. Water levels can be expected to fluctuate with season, tidal action, water levels in nearby canals, temperature, climate, construction in the area, and other factors. Actual conditions during construction may be different from those observed at the time of the explorations.

Section 3

Geotechnical Engineering Evaluation and Foundation Design Recommendations

3.1 Geotechnical Engineering Evaluations

This section describes CDM Smith's geotechnical engineering evaluation and foundation design recommendations for the proposed Distribution Switchgear ECIP project. These evaluations are based on the results of the subsurface investigation program discussed herein, published correlations, and the minimum requirements of the 2018 International Building Code (Code). In addition, recommended design criteria are based on performance tolerances, such as allowable settlement, as understood to relate to similar structures.

3.2 Geotechnical Design Considerations

The primary geotechnical consideration related to the design of the proposed 15 KV Distribution Switchgear is foundation movements related to consolidation/settlement of the underlying compressible soils under the loads of the new structure.

Based on the results of the test borings performed as part of the subsurface investigation, the subsurface soils consist of soft and compressible clays, very loose to, loose sands and a layer of Peat. These soils are expected to consolidate/settle under new applied loads. We anticipate that consolidation/settlement of shallow foundations in the native compressible soils due to new loading will be excessive (i.e., greater than 4 inches) for the proposed 15 KV Distribution Switchgear.

3.3 Foundation Design Recommendations

3.3.1 General

Based on the proposed Distribution Switchgear size/loading, base elevations relative to surrounding grades, and considering the subsurface soil conditions, local practice and construction considerations, the switchgear structure is recommended to be supported on deep foundations, with the lightly loaded miscellaneous equipment pads recommended to be supported on mat foundations.

3.3.2 Mat Foundations

The proposed miscellaneous exterior equipment pads associated with the electrical improvements are understood to be constructed at-grade across the project site. Based on their small plan footprint and a maximum bearing pressure of 200 psf, the pads should be supported using mat foundations bearing on a minimum 12-inch-thick layer of compacted select material overlying undisturbed native soils.

Around the perimeter of all mat foundations, a minimum 2-feet-wide and 6-inches-thick layer of properly placed and compacted impervious fill should be placed to lessen water infiltration in to the underlying granular select material.

Movements of the proposed mat foundations supported on properly placed and compacted select material as recommended herein should be limited to one (1) inch. Differential foundation movements are anticipated to be on the order of one-half (1/2) inch.

3.3.3 Deep Foundations

3.3.3.1 Augered Cast-In-Place Piles

The proposed switchgear improvement should be supported on deep foundations consisting of 18- or 24-inch diameter Augered Cast-In-Place (ACIP) piles. The ACIP piles should bear in the lower sand stratum to develop their load-carrying capacity in the middle, lower clay and sand stratum where consolidation settlements will have reduced effect on the structure.

The analysis to evaluate the allowable capacities of the ACIP piles was performed using procedures outlined in the Federal Highway Department Geotechnical Engineering Circular No. 8 (FHWA GEC 8), Design and Construction of Continuous Flight Auger Piles, dated April 2007. As part of the pile design methodology, the factor of safety to be applied in the analysis is based on if pile load testing is to be performed during construction to confirm the capacities used in the analysis. Based on the relatively small size of the proposed Distribution Switchgear and anticipated limited quantity of piles, the pile capacities provided herein assume no pile load testing is to be performed. All ACIP piles should penetrate a minimum of at least two (2) pile diameters into the Middle Sand or Lower Clay bearing strata.

A summary of the allowable ACIP pile capacities for 18- and 24-inch diameters, based on pile top and tip elevations is provided in **Table 3-1**:

Table 3-1 Allowable ACIP Pile Capacities

Structure	Estimated Top-of-Pile El. (ft)	Diameter (in)	Pile Length[1] (ft)	Approx. Min. Pile Tip El.[1] (ft)	Allowable Pile Capacities (kips)	
					Compression	Uplift
15 KV Distribution Switchgear	El. 0	18	85	-85	46.3	46.3
			90	-90	54.0	54.0
			95	-95	62.1	62.1
		24	80	-80	52.2	52.2
			85	-85	61.8	61.8
			90	-90	72.0	72.0

1. Existing/Finished grades in the vicinity of the structure may vary. All piles should extend to the minimum pile lengths indicated below finished ground surface.

Building loads should be transferred to the piles with a system of pile caps, grade beams and/or a structural slab foundation capable of bridging loads across the span between piles. ACIP piles should be embedded into the pile caps or mat, and pile connections should be designed in accordance with applicable building Code. Center-to-center spacing between piles should be at least three (3) pile diameters to limit group interaction. If spacing of less than three (3) pile diameters is used, CDM Smith must evaluate the system to determine the group efficiency and provide revised recommendations.

3.3.3.2 Foundation Movement

Properly constructed ACIP piles, installed to the minimum lengths below finished ground surface, with loads at or below the recommended capacities (provided above) will be subject to total settlements of less than 1 inch.

3.3.4 Earthquake Considerations

For the purposes of determining design earthquake forces for the structures in accordance with the Code, the site should be considered as a Site Class "E".

3.3.5 Resistance to Unbalanced Lateral Loads

Structures could be subjected to unbalanced lateral loads. These loads should be resisted by the ACIP piles. Lateral load capacity for the ACIP piles should be evaluated using LPILE™ software (or equivalent) once the design lateral loads, and preferred pile diameter are known.

When the design lateral loads exceed the lateral capacity of the piles, passive resistance provided by the pile caps, grade beams or slab foundations with an equivalent fluid pressure of 150 pcf can be utilized. This equivalent fluid pressure assumes the foundations are backfilled with structural fill that is compacted to a density of at least 98 percent of the maximum dry density as determined by laboratory test ASTM D698.

3.3.6 Design Groundwater

The groundwater levels measured during drilling in the test borings ranged between approximately El. -2.0 and El. -3.0. However, higher groundwater levels may be encountered during wet periods and/or flood events. Based on correspondence with NAVFAC personnel, the NAS JRB Belle Chasse facility is situated within a flood control protection system consisting of levees and seawalls. Federal Emergency Management Agency (FEMA) guidance for these protected locations is understood to recommend the finish floor elevation (FFE) for proposed structures be at least 3 feet above adjacent grades.

3.3.7 Buoyancy Considerations

The structures must be waterproofed and designed to resist buoyant forces after construction is complete. If needed, buoyant forces can be resisted by a variety of methods, including the dead load of the structure, foundation extensions beyond the perimeter of the structure, and uplift/tension resistance from deep foundations. The foundation extensions will provide additional downward force from the weight of the soil directly above these foundation extensions. A total unit weight of 115 pound per cubic foot (pcf) should be used for soil weight directly above foundation extensions.

Section 4

Construction Considerations

4.1 General

The purpose of this section is to discuss issues related to geotechnical aspects of construction as required for development of the contract documents. Included are anticipated methods of construction required to achieve the recommendations presented herein and identification of potential construction-related issues. The Contractor will be required to base their construction methods and cost estimates on an independent interpretation of the subsurface conditions.

4.2 Excavation and Excavation Support

Excavations for the proposed 15 KV Distribution Switchgear and miscellaneous equipment pads are anticipated to extend up to several feet bgs. The soils anticipated to be encountered include native soils consisting of clay, sand, and a combination thereof. Conventional earth moving equipment is expected to be suitable for excavation of these materials.

The Contractor will be responsible for conducting any excavation work in accordance with the applicable federal and state laws and regulations, including Occupational Safety and Health Administration (OSHA). Where open excavations are feasible, the side slopes should be designed in accordance with OSHA regulations. The Contractor should be responsible for selection and the design of the means and methods for excavation and excavation support such as opencut with stable side slopes, trench box, etc.

Use of excavation support may limit the amount of excavation spoils and serve to protect adjacent structures, utilities, roadways, and slopes. Selection of the excavation support systems will likely be dependent upon subsurface soils, groundwater conditions, adjacent structures, surcharge loading, etc. The Contractor should develop an excavation plan, including excavation support systems designed by a Professional Engineer licensed in the State of Louisiana. Excavation support systems should be utilized when excavations extend into the zone of influence of adjacent structure or utility foundations. The zone of influence is defined as extending 2 feet beyond the bottom exterior edge of the existing foundation then down and away at a one horizontal to one vertical (1H:1V) or at a 1H:1V slope from the springline of the utilities.

Additional design considerations may be required based on the Contractor's planned construction methods.

4.3 Dewatering

The Contractor will be responsible to design and implement a dewatering and drainage system that maintains a stable, undisturbed subgrade that is free of groundwater and surface water during all construction activities. It is anticipated that excavations may extend below the groundwater table based on the groundwater levels encountered during the drilling of the test borings. The design of the dewatering system should be performed by a Professional Engineer licensed in the State of Louisiana. To avoid disturbance of the subgrade, the water level in all excavations should be maintained to at least at the lowest excavation subgrade elevation during the entire period of excavation and fill placement.

Where applicable, the dewatering system should be designed in conjunction with an excavation support system selected by the Contractor. Depending on the depth of the excavation and excavation support system selected, pumping from open sumps within the excavation may be required. The dewatering system should be adequately filtered to reduce the potential for loss of fines. The site should be graded to direct surface runoff away from excavations.

The Contractor must be prepared to operate the dewatering system continuously, as required to complete the work and avoid floatation or uplift prior to completion of the work. During periods where failure of the system would adversely impact work completed, the Contractor should provide a back-up system to ensure continuous operation.

The Contractor must design the dewatering system to not adversely impact adjacent structures, utilities or site features. All dewatering, handling and disposal of pumped water and any special testing, including that associated with the contaminated areas, should be conducted in accordance with federal, state, and local regulations, permits and specified requirements.

4.4 Deep Foundations

4.4.1 General

The ACIP piles should be drilled using a continuous flight hollow stem auger. All ACIP piles should be installed under the direct supervision of a qualified geotechnical engineer or designated representative with knowledge of all installation requirements.

The augering operation should be performed in a manner such that the amount of soil removed is consistent with the rate of auger advancement. The spoils should be monitored to ensure that the amount of soil excavated does not exceed the volume of the augured hole by more than 15 percent. Each pile hole shall be drilled and filled with grout in an uninterrupted continuous operation, except in cases where the auger withdrawal is required or directed by the Engineer.

During auger withdrawal/grout injection, a “removing pressure” must be maintained with continuous positive grout pressure to prevent collapse of the surrounding soils into the pile excavation. Rate of grout injection and rate of auger withdrawal from the soil shall be coordinated, monitored, and recorded, to maintain a positive pressure in the grout at, all times during auger withdrawal. The magnitude of this positive pressure and performance of other augering and grouting procedures, such as rate of augering, rate of grout injection, and control of grout return around the auger flight are at the option of the Contractor, subject to review by the Engineer. The Contractor must account for the soil conditions and equipment capability in determination of appropriate grout procedures.

Positive rotation of the auger should be maintained throughout placement of the grout. Grout should be pumped into the hollow portion of the auger as the augers are withdrawn, allowing the grout to completely fill the hole. The minimum inside diameter of the hollow shaft of the auger flight shall be 1-1/4 inches. The total volume of grout injected into the augered hole must be at least 15 percent greater than the theoretical volume of the hole for each 5-foot segment of pile. If less grout is placed than the net volume required for any 5-foot increment, or grout pressures is interrupted during auger withdrawal, the auger must be reinserted into the grout and the piles shall be reinstalled by rotating the auger to the bottom of the pile, followed by a second, more-controlled removal and grout injection.

If the auger jumps upward during withdrawal, it shall be reinserted to the original tip elevation and the rate of withdrawal decreased to prevent further jumping. A grout head of at least 10-feet above the injection point should be maintained around the perimeter of the auger flights during raising of the auger so that the grout has a displacing action removing any loose material from the hole.

Reinforcing should be placed into the pile immediately after grouting and while the grout is still fluid. No pile shall be left partially completed overnight but must be completely grouted and protected at the termination of each day's operation.

All ACIP pile installation operations should be supervised by experienced personnel to assure that proper installation procedure is followed, and accurate construction records are kept.

4.4.2 Equipment and Materials

Grout injection equipment shall be provided with a grout pressure gauge in clear view of the equipment operator to verify the continuous presence of positive pressure (removing pressure) on the bottom of the flight auger. The grout pump shall be a positive displacement pump of an approved design. The pump discharge capacity shall be calibrated in strokes per cubic foot, or revolutions per cubic foot, by a method approved by the Engineer. Oil or other rust inhibitors shall be removed from the mixing drums and pressure grout pumps prior to mixing and pumping. The auger hoisting equipment shall be capable of withdrawing the auger smoothly and at a constant rate. An automatic measurement and recording system, such as Pile Installation Recorder, to record auger rotation, depth, torque, volume of grout and withdrawal rate should be available for all pile installation.

Grout shall consist of a mixture of Portland cement, a pozzolanic material (when approved), fluidifier, sand and water. This mixture will be proportioned and mixed to produce a grout capable of being pumped with an ultimate compressive strength as designed by the structural engineer. Other admixtures shall not be used.

Aggregate shall meet the requirements of ASTM C33, for fine aggregate, except as to grading. The sand shall consist of hard, dense, durable, uncoated rock fragments and shall be free from injurious amounts of silt, lumps, loam, soft, or flaky particles, shale, alkali, organic matter, mica, and other deleterious substances. If washed, the method shall not remove other desirable fines and the sand shall be permitted to drain until the residual free moisture is reasonably uniform and stable. Sand grading shall be reasonably consistent and shall conform to the following requirements as delivered to the grout mixer:

U.S. Standard Sieve	Percent Passing by Weight
8	100
16	95 – 100
30	55 – 80
50	30 – 55
100	10 – 30
200	0 - 10

4.4.3 Non-Destructive Pile Testing

To confirm the installed pile integrity, non-destructive pile integrity testing (PIT) is recommended to be performed on at least 15 percent of the production ACIP piles. The PIT testing should be performed using a low-strain dynamic method in accordance with ASTM D5882.

Alternatively, and where the pile length (L) divided by the pile diameter (B), is greater than 30, thermal integrity profiling (TIP) may be performed to assess the installed integrity of the same percentage of production piles. TIP should be performed in accordance with ASTM D7949.

4.5 Protection and Preparation of Subgrades

Care should be taken to avoid excess traffic on the excavated subgrades prior to placement of fill materials or concrete. The exposed soil subgrades should be adequately dewatered and protected against precipitation. Excessive drying and/or freezing of the subgrade should not be allowed. Structure subgrades consisting of granular soils (sand and gravel) should be, proof-rolled with at least four (4) passes with a vibratory compactor prior to placement of structural fill. Proof-rolling should not be conducted where the subgrade consists of cohesive soil (silt or clay), however, a smooth edge bucket should be used for final excavation in such soil. Any unsuitable material present at the subgrade level should be removed and replaced with compacted structural fill as recommended herein. Unsuitable materials include trash, wood, loam, construction debris, large rock or concrete (greater than 6 inches in maximum dimension), organics and other materials that degrade over time.

A geotechnical engineer should be present during foundation excavation to confirm that suitable subgrade conditions are present. Subgrade excavation and foundation preparation should be performed in the dry.

4.6 Backfill Materials and Compaction Requirements

4.6.1 General

All, topsoil, fill, and deleterious materials must be removed from the areas proposed for construction. On-site excavated material that is stockpiled should be segregated, placed and graded to prevent the stockpiled material from becoming saturated. However, it is likely that the water content of the stockpiled material will increase slightly during stockpiling. It should be anticipated that stockpiled soils will have to be disked, dried and further segregated prior to use as backfill. If water must be added, it should be uniformly applied and thoroughly mixed into the soil by disking or scarifying.

Prior to the start of any backfilling work, grain size analysis, Atterberg limits (plasticity), natural moisture content, and laboratory compaction testing should be conducted on each type of fill materials, including both on-site and imported materials, to evaluate fill suitability, confirm the properties used in the design, and for use in construction quality control activities. esting should be performed for each type of material used periodically during fill placement for quality assurance and for any changes noted during construction.

Each lift of compacted fill should be tested prior to placement of subsequent lifts. At a minimum, field density and moisture content testing should consist of one in-place density and moisture content test per 1,000 square feet of fill placed, with at least one test conducted per lift of bulk earthwork placed each day. Additional field testing should be conducted for changes in material type. Representative areas of all filling and compaction work should be tested. During the course of the backfilling and compaction work, one laboratory compaction test, one Atterberg limits test (if the material is cohesive), and one grain-size analysis should be conducted for every 10 in-place density tests and/or each change in material, whichever is more frequent.

At the end of each workday, new fill should be sloped to promote positive drainage and be sealed with a smooth drum roller to limit surface water infiltration. Prior to resuming fill placement, the fill surface should be scarified by tracking over it with a piece of tracked equipment and moisture conditioned if needed.

4.6.2 Common Fill

Common fill used as backfill to restore the site grades following the anticipated excavations, around structures where passive pressure is not relied on, and in landscaped areas should consist of granular soil free from organic material, loam, debris, frozen soil, or other deleterious material. It should contain stones no larger than 6 inches and have no more than 50 percent of material passing the No. 200 sieve. Common fill should have a plasticity index less than 25.

Common fill should be placed in layers not to exceed 12 inches, as placed, and compacted with suitable compaction equipment to at least 95 percent of maximum dry density as determined by ASTM D698. Lift thickness should be reduced to 6 inches in confined areas accessible only to hand-guided compaction equipment.

Based on the available lab test data, some portions of the on-site soils meet the above criteria and therefore may be suitable for use as common fill. However, the Contractor should make an independent evaluation of the suitability for reuse of these soils and submit samples of the material to be used as common fill for testing.

4.6.3 Select Material

Granular fill used as Select material below slabs and as backfill against foundations should consist of a mineral soil, free of organic material, loam, debris, frozen soil, or other deleterious material which may be, compressible or which cannot be properly compacted. Select material should conform to the following gradation requirements:

U.S. Standard Sieve	Percent Passing by Weight
1 ½ inches	100
No. 4	20-70
No. 40	5-35
No. 200	0-10

Select material should have a liquid limit less than 40 and a plasticity index not to exceed 10. Select material should be placed in layers no thicker than 8 inches, as placed, and compacted with suitable compaction equipment to at least 98 percent of maximum dry density as determined by ASTM D698. The moisture content at the time of compaction should be in the range of –2 to +2 percent of the optimum value as defined by ASTM D698. Lift thickness should be reduced to 4 inches in confined areas accessible only to hand guided compaction equipment.

Based on the available lab test data, native soils excavated as part of this work do not meet the above criteria and therefore are not suitable for use as select material.

4.6.4 Impervious Fill

Impervious fill should consist of cohesive soil free from organic material (less than 5 percent organic content in accordance with ASTM D2974), loam, debris, frozen soil, or other deleterious material. It should be classified as a CL or CH soil according to the Unified Soil Classification System in accordance with ASTM D2488. Impervious fill should have 100 percent of material passing the 1.5-inch sieve and have at least 50 percent of material passing the No. 200 sieve. Impervious fill should have a liquid limit of at least 30 and a plasticity index between 20 and 60.

Based on the available geotechnical laboratory test data, soils anticipated to be excavated as part of this work may meet the above criteria and therefore could be used as impervious fill. However, final determination of the reuse potential of onsite excavated materials should be determined by the contractor during construction.

4.7 Construction Monitoring

It is recommended that a qualified Geotechnical Engineer or experienced technician under the direction of the Geotechnical Engineer be present during construction to confirm that the Contractor complies with the intent of these recommendations. Specifically, the field representative would undertake the following responsibilities:

- Observe that the subgrade preparation for structures is performed as recommended herein for adequate support.
- Observe, test and document placement and compaction of fill and backfill materials where appropriate. Samples of fill should be submitted to the Engineer for approval prior to starting work.
- Observe that proper dewatering methods are employed, if required.
- Observe the proper installation of ACIP piles.

In addition, the field representative would be present to identify and provide response should conditions encountered differ from those assumed during preparation of this report.

4.8 Closing

These recommendations have been prepared as part of the NAS JRB Belle Chasse 15 KV Distribution Switchgear project located in Belle Chasse, Louisiana as understood at this time and described in this report. These recommendations have been prepared in accordance with generally accepted engineering practices. No other warranty, express or implied, is made. In the event, that changes in the design or location of the alignment occur, the conclusions and recommendations contained herein should not be considered valid unless verified in writing by CDM Smith.

Appendix A

Tolunay-Wong Engineers, Inc.

Subsurface Investigation Report
NAS JRB Belle Chasse

Belle Chasse, Louisiana

CDM Smith

Tolunay-Wong Engineers, Inc.

SUBSURFACE INVESTIGATION REPORT
NAS JRB BELLE CHASSE
BELLE CHASSE, LOUISIANA

Prepared for:

CDM Smith
12400 Coit Road, Suite 400
Dallas, Texas 75251

Prepared by:

Tolunay-Wong Engineers, Inc.
524 Elmwood Park Boulevard, Ste. 135
Elmwood, Louisiana 70123

June 15, 2020

TWE Project No. 20.33.421 / Report No. 25621

Tolunay-Wong Engineers, Inc.

524 Elmwood Park Boulevard, Ste 135 • Elmwood, Louisiana 70123 • Phone (504) 467-6009

June 15, 2020

CDM Smith
12400 Coit Road, Suite 400
Dallas, TX 75251

Attn: Jeff Van Pelt, PE
vanpeltjd@cdmsmith.com

Ref: Subsurface Investigation Report
NAS JRB Belle Chasse
Belle Chasse, Louisiana
TWE Project No. 20.33.421/ Report No. 25621

Dear Mr. Pelt,

Tolunay-Wong Engineers, Inc. (TWE) is pleased to submit this report of our subsurface investigation performed for the NAS JRB facility in Belle Chasse, Louisiana. This report contains a detailed description of the field and laboratory program performed for this study, as well as soil boring logs with tabulated field and laboratory test results.

We appreciate the opportunity to work with you on this phase of the project and look forward to the opportunity of providing additional services as the project progresses. If you have any questions or comments regarding this report or if we can be of further assistance, please contact us.

Sincerely,

TOLUNAY-WONG ENGINEERS, INC.
LAPELS - Registration No. EF.0003024

Lindsey B. Burst
Lindsey B. Burst
Staff Professional

Dustin S. Walker
Dustin S. Walker, P.E.
Regional Manager

STATE OF LOUISIANA
Dustin S. Walker
License No. 38444
PROFESSIONAL ENGINEER
IN
CIVIL ENGINEERING
6-15-20

TABLE OF CONTENTS

TABLES AND APPENDICES

TABLES

APPENDICES

1 INTRODUCTION AND PROJECT DESCRIPTION

1.1 Introduction

This report presents the results of our geotechnical subsurface investigation for the NAS JRB facility in Belle Chasse, Louisiana. This study was conducted in general accordance with TWE Proposal P20-GZL099, dated March 31, 2020, and authorized by CDM Federal Programs Corporation Contract No. N62470-15-D-4002, dated April 22, 2020.

1.2 Project Description

The project consists of the relocation of the 15kV service entrance to a more interior location at the NAS JRB facility in Belle Chasse, Louisiana. A site investigation is desired to determine subsurface soil conditions in the area of the planned relocation.

2 PURPOSE AND SCOPE OF SERVICES

The purpose of this geotechnical investigation was to provide geotechnical information needed to assist the Client with assessing subsurface soil and groundwater conditions at the planned relocation site of the 15kV service entrance, at the existing NAS JRB Facility in Belle Chasse, Louisiana.

The scope of our geotechnical services consisted of:

1. Drilling two (2) conventional soil borings at the locations requested to investigate subsurface soil and ground water conditions; one (1) soil boring to a depth of 100-ft below the existing subsurface and one (1) soil boring to a depth of 30-ft below the existing subsurface;
2. Performing geotechnical laboratory tests on recovered soil samples to assist in classification of the soils encountered and to evaluate selected engineering properties of the subsurface materials; and,
3. Providing a written report detailing our field and laboratory programs, and the results of our testing.

The scope of services did not include any environmental assessment for the presence or absence of wetlands or of hazardous or toxic materials within or on the soil, air, or water at this site. Any statements in this report or on the boring logs regarding odors, colors, unusual or suspicious items and conditions are strictly for the information of the Client.

3 FIELD PROGRAM

3.1 Summary of Field Program

TWE conducted an exploration of subsurface soil and groundwater conditions at the site on May 19, 2020 by drilling two (2) conventional soil borings; one (1) to a depth of 100-ft and one (1) to a depth of 30-ft below the existing ground surface. The approximate test locations are shown on TWE Drawing No. 20.33.421-1 in Appendix A.

3.2 Soil Borings

3.2.1 Drilling Methods

Field operations were performed in general accordance with *Standard Practice for Soil Investigation and Sampling by Auger Borings* [American Society for Testing and Materials (ASTM) D1452]. The soil borings were drilled using an ATV-mounted drilling rig equipped with a rotary head. The boreholes were advanced using dry-auger drilling methods until conditions required wet-rotary drilling. Samples were obtained continuously at intervals of 2-ft from the ground surface to a depth of 12-ft, at 13-ft to 15-ft, and at 5-ft depth intervals thereafter until the boring completion depth was achieved.

3.2.2 Soil Sampling

Fine-grained cohesive soil samples were recovered from the soil boring by hydraulically pushing a 3-in diameter, thin-walled tube to approximately 24-in. The field sampling procedures were conducted in general accordance with the *Standard Practice for Thin-Walled Tube Sampling of Soils* (ASTM D 1587). TWE's geotechnician visually classified the recovered soils and obtained a penetration resistance measurement of the recovered soils using a calibrated pocket penetrometer. The samples were extruded in the field, wrapped in foil, placed in moisture sealed plastic bags and protected from disturbance prior to transport to the laboratory. The recovered soil sample depths and pocket penetrometer measurements are presented on the boring logs in Appendix B.

Coarse-grained, cohesionless and semi-cohesionless soil samples were collected with the standard penetration test (SPT) sampler driven 18-in by blows from a 140-lb hammer falling 30-in in accordance with the Standard Test Method for Standard Penetration Test (SPT) and Spilt-Barrel Sampling of Soils (ASTM D1586). The number of blows required to advance the sampler three (3) consecutive 6-in depths are recorded for each corresponding sample on the boring log. The N-value, in blows per foot, is obtained from SPTs by adding the last two (2) blow count numbers. The compactness of cohesionless and semi-cohesionless samples and the consistency of cohesive samples are inferred from the N-value. The samples obtained from the split-barrel sampler were visually classified, placed in moisture sealed containers and transported to our laboratory.

3.2.3 Boring Logs

Our interpretation of general subsurface soil and groundwater conditions at the soil boring locations are included on the boring logs. The interpretations of the soil types throughout the boring depths and the locations of strata changes were based on visual classifications during field sampling and laboratory testing using ASTM D2487, *Unified Soil Classification System*, and ASTM D 2488, *Description and Identification of Soils*. The boring logs include the type and interval depth for each sample along with the corresponding pocket penetrometer readings for cohesive soils. The boring logs and a key to terms and symbols used on boring logs are presented in Appendix B.

3.2.4 Groundwater Measurements

At test boring B-1, groundwater was encountered at a depth of 5-ft and rose to a depth of 2.5-ft after fifteen (15) minutes. At test boring B-2, groundwater was encountered at a depth of 7-ft and rose to a depth of 4-ft after fifteen (15) minutes.

Groundwater levels at the project site could fluctuate with climatic and seasonal variations. Accurate determination of static groundwater levels is typically made with standpipe piezometers. Installation of standpipe piezometers to evaluate long-term groundwater conditions within the project site was not included in our scope of services for this project.

4 LABORATORY SERVICES

4.1 General

A laboratory testing program was conducted on selected soil samples obtained from the test borings to assist in classification and evaluation of the physical and engineering properties of the soils encountered at the project site. Types and depths of testing were selected by the Client. Laboratory tests were performed in general accordance with ASTM International standards. The types and brief descriptions of the laboratory tests performed are presented in Table 4-1 below.

Table 4-1: Geotechnical Laboratory Testing Program

Test Description	Test Method
Water (Moisture) Content of Soil	ASTM D2216
Liquid Limit, Plastic Limit and Plasticity Index of Soils	ASTM D4318
Organic Content of Soil	ASTM D2974
Amount of Material in Soils Finer than No. 200 Sieve	ASTM D1140
Unconfined Compressive Strength of Cohesive Soil	ASTM D2166
Unconsolidated-Undrained Triaxial Shear Strength of Soil	ASTM D2850
Density (Unit Weight) of Soil Specimens	ASTM D7263
One-Dimensional Consolidation of Soil	ASTM D2435
One-Dimensional Swell of Soil	ASTM D4546

Standard geotechnical laboratory test results are presented on the logs of test borings in Appendix B.

5 SITE AND SUBSURFACE CONDITIONS

5.1 General

Our interpretations of soil and groundwater conditions within the project site are based on information obtained at the soil boring locations. Subsurface conditions could vary at areas within the project site not explored by the test explorations.

5.2 Site Description and Surface Conditions

The project site is located in an undeveloped area with surrounding buildings and parking lots. The existing ground surface at the locations of borings B-1 and B-2 is grass covered.

5.3 Subsurface Soil Stratigraphy

The generalized soil profile in the project borings primarily consists of cohesive soils classified as lean clays (CL) and fat clays (CH) with intermittent layers of cohesionless and semi-cohesionless soils classified as silty sands (SM) and clayey sands (SC).

Detailed descriptions of the soils encountered along with the tabulated laboratory test results at the exploration locations are presented on the boring logs in Appendix B.

5.4 Subsurface Soil Properties

Cohesive Soils: Results of Atterberg limits tests on selected non-organic cohesive soil samples from the TWE project borings indicated liquid limits (LL) ranging from 32 to 122 with corresponding plasticity indices (PI) ranging from 9 to 79. In-situ moisture contents of the cohesive soils ranged from 14% to 83%.

Undrained shear strengths determined from laboratory unconfined compressive strength testing ranged from 0.18-tsf to 0.48-tsf with corresponding dry unit weights ranging from 51-pcf to 84-pcf. Based on the soil strength data obtained in the field and laboratory, the cohesive soils recovered from the project borings were generally inferred to be of very soft to very stiff consistencies.

Cohesionless and Semi-Cohesionless Soils: The recorded SPT N-values from the cohesionless and semi-cohesionless soil strata encountered were recorded as weight of hammer (WOH), indicating very loose densities of the silt and sand strata. In-situ moisture content of the cohesionless and semi-cohesionless soil samples ranged from 26% to 34%.

Tabulated laboratory test results at the recovered sample depths are presented on the boring logs in Appendix B.

5.5 Consolidation Testing

Two (2) one-dimensional consolidation tests were performed for the project on selected samples. One (1) at a depth of 18-ft to 20-ft and one (1) at a depth of 53-ft to 55-ft at boring B-1 in accordance with ASTM D2435. The consolidation tests were performed to determine the magnitude of consolidation of soil when it is restrained laterally and drained axially while subjected to incrementally applied controlled-stress loading. Time-deformation readings were obtained on all load increments. Successive load increments were applied after 100% primary consolidation of each load increment was reached. The laboratory consolidation test reports are presented in Appendix C.

5.6 Swell Testing

One (1) one-dimensional wetting-induced swell test was performed for the project on a selected sample at a depth of 4-ft to 6-ft at boring B-2 in accordance with ASTM D4546, Method B. The swell test was performed to determine the magnitude of swell or deformation when a sample is restrained laterally and subjected to the approximate vertical in-situ overburden pressure. The laboratory swell test report is presented in Appendix D.

6 LIMITATIONS

This report has been prepared for the exclusive use of CDM Smith and their design team for specific application to the relocation, design, and construction of the 15kV service entrance at the NAS JRB facility in Belle Chasse, Louisiana. Our report has been prepared in accordance with the generally accepted geotechnical engineering practice common to the local area. No other warranty, express or implied is made.

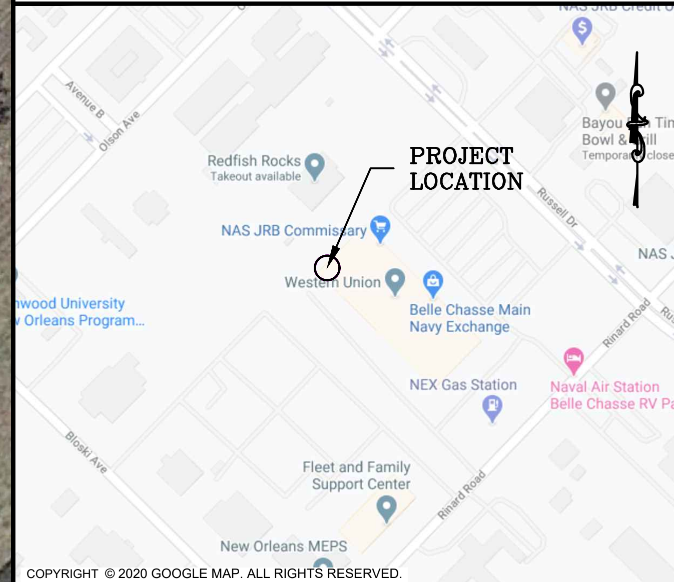
The information contained in this report is based on the data obtained from the referenced subsurface explorations within the project site. The soil borings indicate subsurface conditions only at the specific locations and times performed and only to the depths penetrated. The soil borings do not necessarily reflect strata variations that may exist at other locations within the project site. The validity of the information provided in this report is based in part on assumptions about the stratigraphy made by the Geotechnical Engineer. Such assumptions may be confirmed during construction.

TWE is not responsible for any claims, damages or liability associated with interpretation or reuse of the subsurface data without the expressed written authorization of TWE.

APPENDIX A

SOIL BORING LOCATION PLAN

VICINITY MAP



SOIL BORING COORDINATES

BORING	DEPTH	LONGITUDE	LATITUDE
B–1	100'	29° 49' 30.66" N	90° 01' 20.58" W
B–2	30'	29° 49' 30.22" N	90° 01' 20.67" W

LEGEND

SYMBOL	DESCRIPTION
⊕	APPROXIMATE SOIL BORING LOCATION

Tolunay-Wong Engineers, Inc.

PROPOSED SOIL BORING LOCATION PLAN
NAS JRB BELLE CHASSE
BELLE CHASSE, LOUISIANA

Drawn	*A.J.C.*	*04–23–2020*
Checked	*D.S.W.*	*04–23–2020*
Approved	*D.S.W.*	*04–23–2020*
Scale	*N.T.S.*	

TWE DRAWING NO. 20.33.421–1

APPENDIX B

SOIL BORING LOGS AND A KEY TO TERMS AND SYMBOLS

LOG OF BORING B-1

PROJECT: NAS-JRB
Belle Chase, Louisiana

CLIENT: CDM Smith
Dallas, Texas

COORDINATES: N 29° 49'0.66"
W 90° 01'0.58"

SURFACE ELEVATION: N.A.

DRILLING METHOD:
Dry Augered: 0 to 6'
Wash Bored: 6' to 100'

ELEVATION (FT) / DEPTH (FT): 0, 5, 10, 15, 20, 25, 30, 35

MATERIAL DESCRIPTION	(P) POCKET PEN (tsf) (T) TORVANE (tsf)	STD. PENETRATION TEST BLOWCOUNT	MOISTURE CONTENT (%)	DRY UNIT WEIGHT (pcf)	LIQUID LIMIT (%)	PLASTICITY INDEX (%)	COMPRESSIVE STRENGTH (tsf)	FAILURE STRAIN (%)	CONFINING PRESSURE (psi)	PASSING #200 SIEVE (%)	OTHER TESTS PERFORMED
Very stiff brown FAT CLAY (CH)	(P)2.50		21								
-becomes firm at 2'	(P)0.75		31								
Very soft gray LEAN CLAY (CL)	(P)0.25		33		38	17					
Very loose tan SILTY SAND (SM)			34							39	
Very loose gray CLAYEY SAND (SC)			33		30	9					
			30								
Firm dark gray ORGANIC CLAY (OH) -with silt seams	(P)1.50		83	51	122	79	0.61	4			
Very soft gray LEAN CLAY WITH SAND (CL)	(P)0.25										
	(P)0.25		30		32	9					
-becomes soft at 33'	(P)0.25		40	80	41	19	0.35	15	27		

COMPLETION DEPTH: 100 ft
DATE BORING STARTED: 05-19-20
DATE BORING COMPLETED: 05-19-20
LOGGER: D. Welch
PROJECT NO.: 20.33.421

NOTES: Groundwater was encountered at a depth of 5.0' during auger drilling and rose to a depth of 2.5' after a period of 15 minutes. The borehole was backfilled with a cement-bentonite grout mixture upon completion of drilling and sampling.

LOG OF BORING B-1

PROJECT: NAS-JRB
Belle Chase, Louisiana

CLIENT: CDM Smith
Dallas, Texas

COORDINATES: N 29° 49'0.66" W 90° 01'0.58"
SURFACE ELEVATION: N.A.
DRILLING METHOD:
Dry Augered: 0 to 6'
Wash Bored: 6' to 100'

DEPTH (FT)	MATERIAL DESCRIPTION	(P) POCKET PEN (tsf) (T) TORVANE (tsf)	STD. PENETRATION TEST BLOWCOUNT	MOISTURE CONTENT (%)	DRY UNIT WEIGHT (pcf)	LIQUID LIMIT (%)	PLASTICITY INDEX (%)	COMPRESSIVE STRENGTH (tsf)	FAILURE STRAIN (%)	CONFINING PRESSURE (psi)	PASSING #200 SIEVE (%)	OTHER TESTS PERFORMED
35	Soft gray LEAN CLAY WITH SAND (CL)											
38-40		(P)0.25										
43-45		(P)0.25		37	81	43	21	0.41	15	35		
48-50	Very soft gray FAT CLAY (CH)	(P)0.25										
53-55	-becomes firm at 53'	(P)0.50		58	65	84	52	0.74	6	43		
58-60		(P)0.50										
63-65		(P)0.50		58	68	87	52	0.95	5	51		
68-70		(P)0.75										

COMPLETION DEPTH: 100 ft
DATE BORING STARTED: 05-19-20
DATE BORING COMPLETED: 05-19-20
LOGGER: D. Welch
PROJECT NO.: 20.33.421

NOTES: Groundwater was encountered at a depth of 5.0' during auger drilling and rose to a depth of 2.5' after a period of 15 minutes. The borehole was backfilled with a cement-bentonite grout mixture upon completion of drilling and sampling.

TOLUNAY-WONG ENGINEERS, INC.

LOG OF BORING B-1

PROJECT: NAS-JRB
Belle Chase, Louisiana

CLIENT: CDM Smith
Dallas, Texas

COORDINATES: N 29° 49'0.66"
W 90° 01'0.58"

SURFACE ELEVATION: N.A.

DRILLING METHOD:
Dry Augered: 0 to 6'
Wash Bored: 6' to 100'

DEPTH (FT)	MATERIAL DESCRIPTION	(P) POCKET PEN (tsf) (T) TORVANE (tsf)	STD. PENETRATION TEST BLOWCOUNT	MOISTURE CONTENT (%)	DRY UNIT WEIGHT (pcf)	LIQUID LIMIT (%)	PLASTICITY INDEX (%)	COMPRESSIVE STRENGTH (tsf)	FAILURE STRAIN (%)	CONFINING PRESSURE (psi)	PASSING #200 SIEVE (%)	OTHER TESTS PERFORMED
	Firm gray FAT CLAY (CH)											
75	Firm gray LEAN CLAY WITH SAND (CL)	(P)0.50		38	82			0.72	7	59		
80	Loose gray SILTY SAND (SM)			26							13	
85	-becomes very loose at 83'		WOH									
90			WOH	27							33	
95			WOH									
100			WOH									
	Bottom @ 100'											
105												

COMPLETION DEPTH: 100 ft
DATE BORING STARTED: 05-19-20
DATE BORING COMPLETED: 05-19-20
LOGGER: D. Welch
PROJECT NO.: 20.33.421

NOTES: Groundwater was encountered at a depth of 5.0' during auger drilling and rose to a depth of 2.5' after a period of 15 minutes. The borehole was backfilled with a cement-bentonite grout mixture upon completion of drilling and sampling.

LOG OF BORING B-2

PROJECT: NAS-JRB
Belle Chase, Louisiana

CLIENT: CDM Smith
Dallas, Texas

COORDINATES: N 29° 49' 30.22" W 90° 01' 20.67"
SURFACE ELEVATION: N.A.
DRILLING METHOD:
Dry Augered: 0 to 10'
Wash Bored: 10' to 30'

DEPTH (FT)	MATERIAL DESCRIPTION	(P) POCKET PEN (tsf) (T) TORVANE (tsf)	STD. PENETRATION TEST BLOWCOUNT	MOISTURE CONTENT (%)	DRY UNIT WEIGHT (pcf)	LIQUID LIMIT (%)	PLASTICITY INDEX (%)	COMPRESSIVE STRENGTH (tsf)	FAILURE STRAIN (%)	CONFINING PRESSURE (psi)	PASSING #200 SIEVE (%)	OTHER TESTS PERFORMED
0	Hard brown FAT CLAY (CH)	(P)4.50		14								
	-stiff from 2' to 4' -becomes brown and gray at 2'	(P)1.50		17								
5	-becomes firm at 4'	(P)0.25		38	84	50	26	0.53	15			
	Very soft brown and gray LEAN CLAY (CL)	(P)0.25		34								OC= 0.7%
	Very soft brown SANDY LEAN CLAY (CL)	(P)0.25										
10	-becomes gray at 10'	(P)0.25		38							64	
	Very soft gray LEAN CLAY WITH SAND (CL)	(P)0.25										
15												
			WOH	58							85	
20												
		(P)0.25										
25												
	-becomes firm at 28'	(P)0.25		44	78	46	26	0.52	15	21		
30	Bottom @ 30'											
35												

COMPLETION DEPTH: 30 ft
DATE BORING STARTED: 05-19-20
DATE BORING COMPLETED: 05-19-20
LOGGER: D. Welch
PROJECT NO.: 20.33.421

NOTES: Groundwater was encountered at a depth of 7.0' during auger drilling and rose to a depth of 4.0' after a period of 15 minutes. The borehole was backfilled with a cement-bentonite grout mixture upon completion of drilling and sampling.

TOLUNAY-WONG ENGINEERS, INC.

KEY TO SYMBOLS AND TERMS USED ON BORING LOGS FOR SOIL

Most Common Unified Soil Classifications System Symbols

- Lean Clay (CL)
- Lean Clay w/ Sand (CL)
- Sandy Lean Clay (CL)
- Fat Clay (CH)
- Fat Clay w/ Sand (CH)
- Sandy Fat Clay (CH)
- Silty Clay (CL-ML)
- Sandy Silty Clay (CL-ML)
- Silty Clayey Sand (SC-SM)
- Clayey Sand (SC)
- Sandy Silt (ML)
- Silty Sand (SM)
- Silt w/ Sand (ML)
- Well Graded Sand (SW)
- Well Graded Sand w/ Gravel (SW-GM)
- Poorly Graded Sand (SP)
- Poorly Graded Sand w/ Silt (SP-SM)
- Silt (ML)
- Elastic Silt (MH)
- Elastic Silt w/ Sand (MH-SP)
- Silty Gravel (GM)
- Clayey Gravel (GC)
- Well Graded Gravel (GW)
- Well Graded Gravel w/ Sand (SP-GM)
- Poorly Graded Gravel (GP)
- Peat

Miscellaneous Materials

- Fill
- Concrete
- Asphalt and/or Base

Sampler Symbols	**Meaning**
	Pavement core
	Thin - walled tube sample
	Standard Penetration Test (SPT)
	Auger sample
	Sampling attempt with no recovery
	TxDOT Cone Penetrometer Test

Field Test Data

2.50	Pocket penetrometer reading in tons per square foot
(T)1.13	Torvane Measurement in tons per square foot
8/6"	Blow count per 6 - in. interval of the Standard Penetration Test
	Observed free water during drilling
	Observed static water level

Laboratory Test Data

Wc (%)	Moisture content in percent
Dens. (pcf)	Dry unit weight in pounds per cubic foot
Qu (tsf)	Unconfined compressive strength in tons per square foot
UU (tsf)	Compressive strength under confining pressure in tons per square foot
Str. (%)	Strain at failure in percent
LL	Liquid Limit in percent
PI	Plasticity Index
#200 (%)	Percent passing the No. 200 mesh sieve
( )	Confining pressure in pounds per square inch
*	Slickensided failure
**	Did not fail @ 15% strain

RELATIVE DENSITY OF COHESIONLESS & SEMI-COHESIONLESS SOILS

The following descriptive terms for relative density apply to cohesionless soils such as gravels, silty sands, and sands as well as semi-cohesive and semi-cohesionless soils such as sandy silts, and clayey sands.

Relative Density	Typical N_{60} Value Range*
Very Loose	0-4
Loose	5-10
Medium Dense	11-30
Dense	31-50
Very Dense	Over 50

* N_{60} is the number of blows from a 140-lb weight having a free fall of 30-in. required to penetrate the final 12-in. of an 18-in. sample interval, corrected for field procedure to an average energy ratio of 60% (Terzaghi, Peck, and Mesri, 1996).

CONSISTENCY OF COHESIVE SOILS

The following descriptive terms for consistency apply to cohesive soils such as clays, sandy clays, and silty clays.

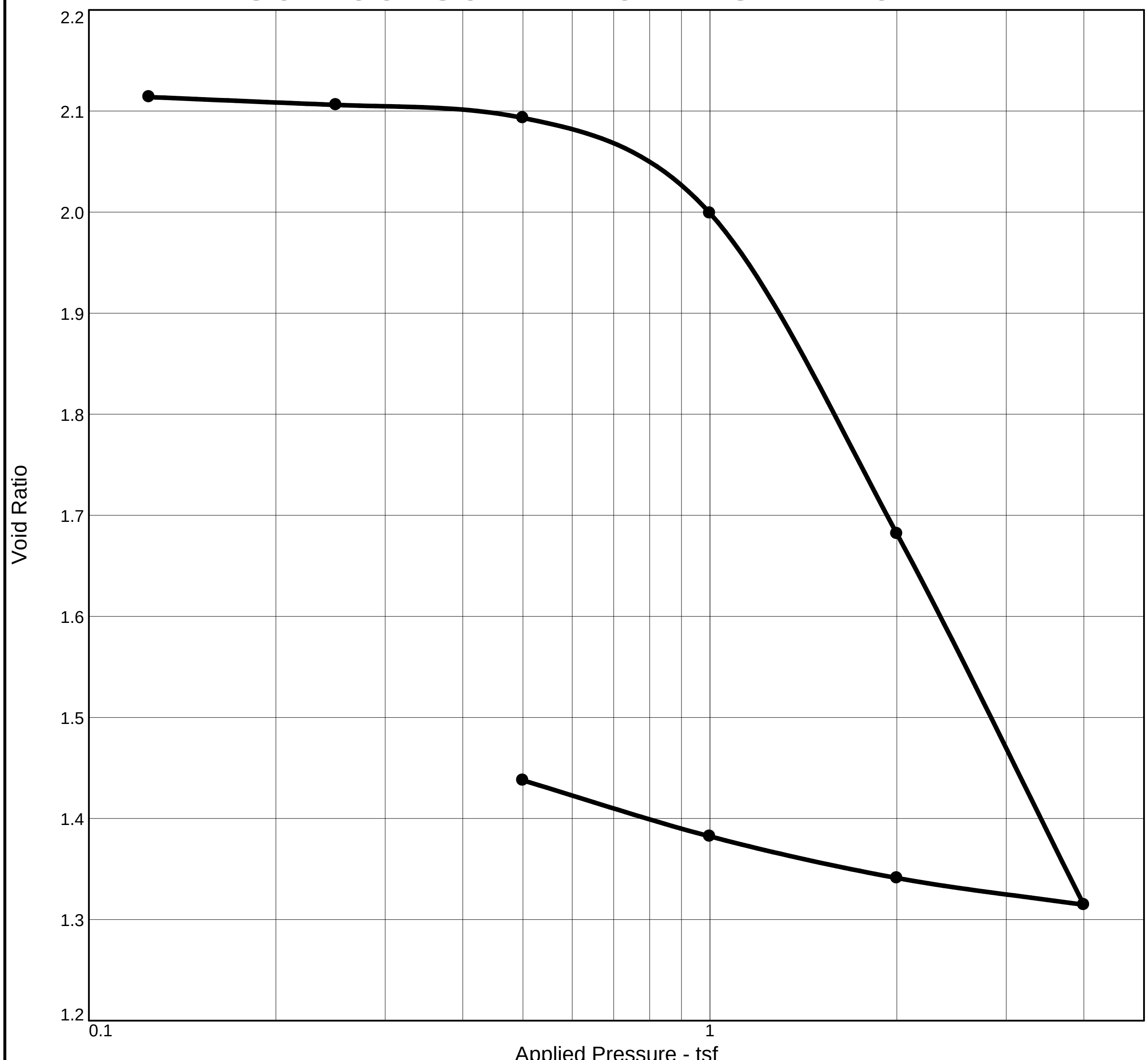
Typical Compressive Strength (tsf)	Consistency	Typical SPT "N_{60}" Value Range**
$q_u < 0.25$	Very soft	≤ 2
$0.25 \leq q_u < 0.50$	Soft	3-4
$0.50 \leq q_u < 1.00$	Firm	5-8
$1.00 \leq q_u < 2.00$	Stiff	9-15
$2.00 \leq q_u < 4.00$	Very Stiff	16-30
$q_u \geq 4.00$	Hard	≥ 31

** An "N_{60}" value of 31 or greater corresponds to a hard consistency. The correlation of consistency with a typical SPT "N_{60}" value range is approximate.

APPENDIX C

ONE-DIMENSIONAL CONSOLIDATION TEST REPORTS

SOIL CONSOLIDATION TEST REPORT



Natural		Dry Dens. (pcf)	LL	PI	Sp. Gr.	USCS	AASHTO	Initial Void Ratio
Saturation	Moisture							
100.0 %	79.4 %	53.4	122	79	2.67	CH		2.119

MATERIAL DESCRIPTION

Firm dark gray ORGANIC CLAY (OH)

Project No. 20.33.421 **Client:** CDM Smith

Project: NAS-JRB
Belle Chase, Louisiana

Source of Sample: B-1 **Depth:** 18

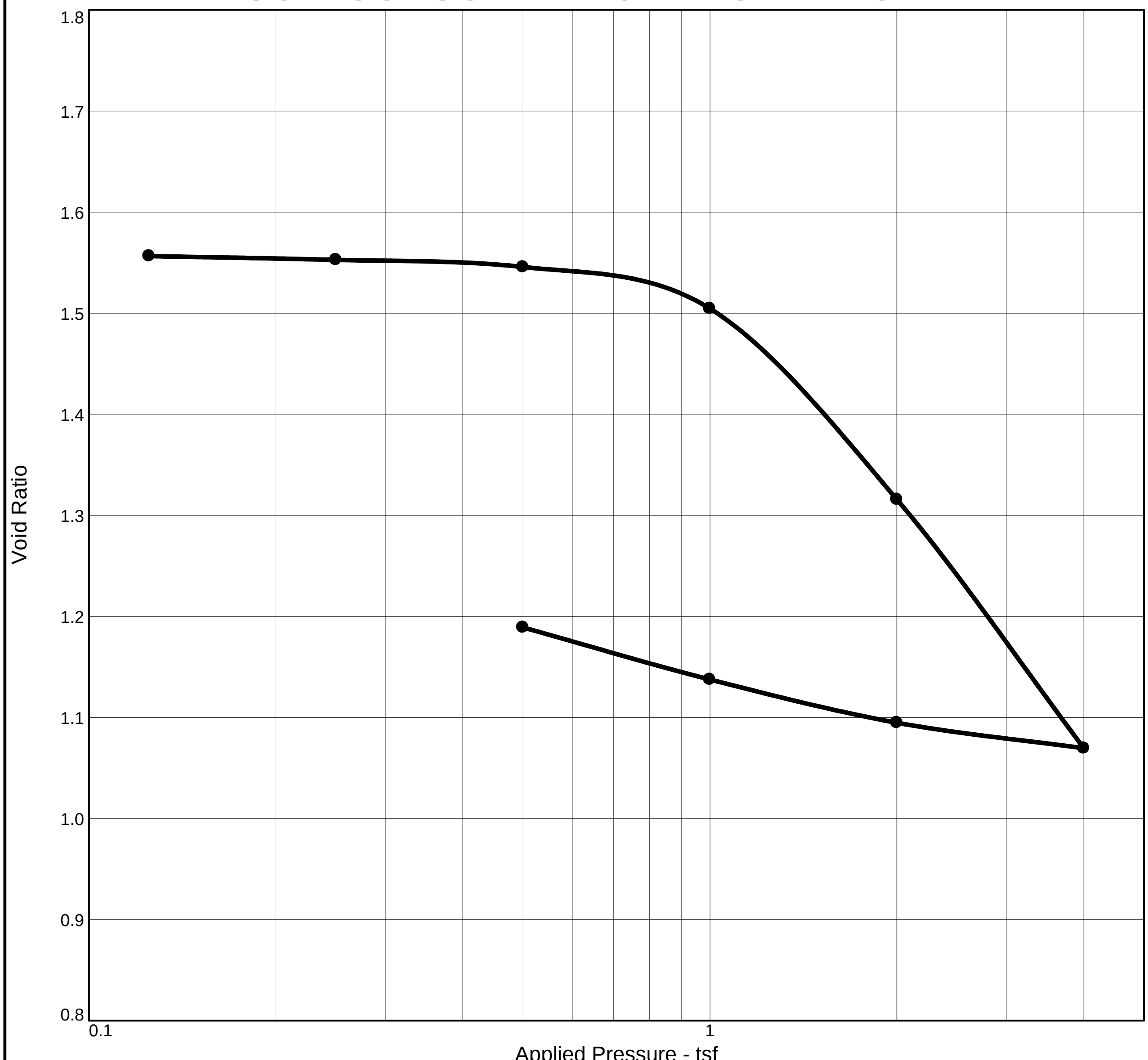
Tolunay-Wong Engineers, Inc.

Gonzales, LA

Remarks:
Specific gravity: Assumed

Figure

SOIL CONSOLIDATION TEST REPORT



Natural		Dry Dens. (pcf)	LL	PI	Sp. Gr.	USCS	AASHTO	Initial Void Ratio
Saturation	Moisture							
100.0 %	57.8 %	65.8	84	52	2.698	CH		1.559

MATERIAL DESCRIPTION

Firm gray FAT CLAY (CH)

Project No. 20.33.421 **Client:** CDM Smith

Project: NAS-JRB
Belle Chase, Louisiana

Source of Sample: B-1 **Depth:** 53

Tolunay-Wong Engineers, Inc.

Gonzales, LA

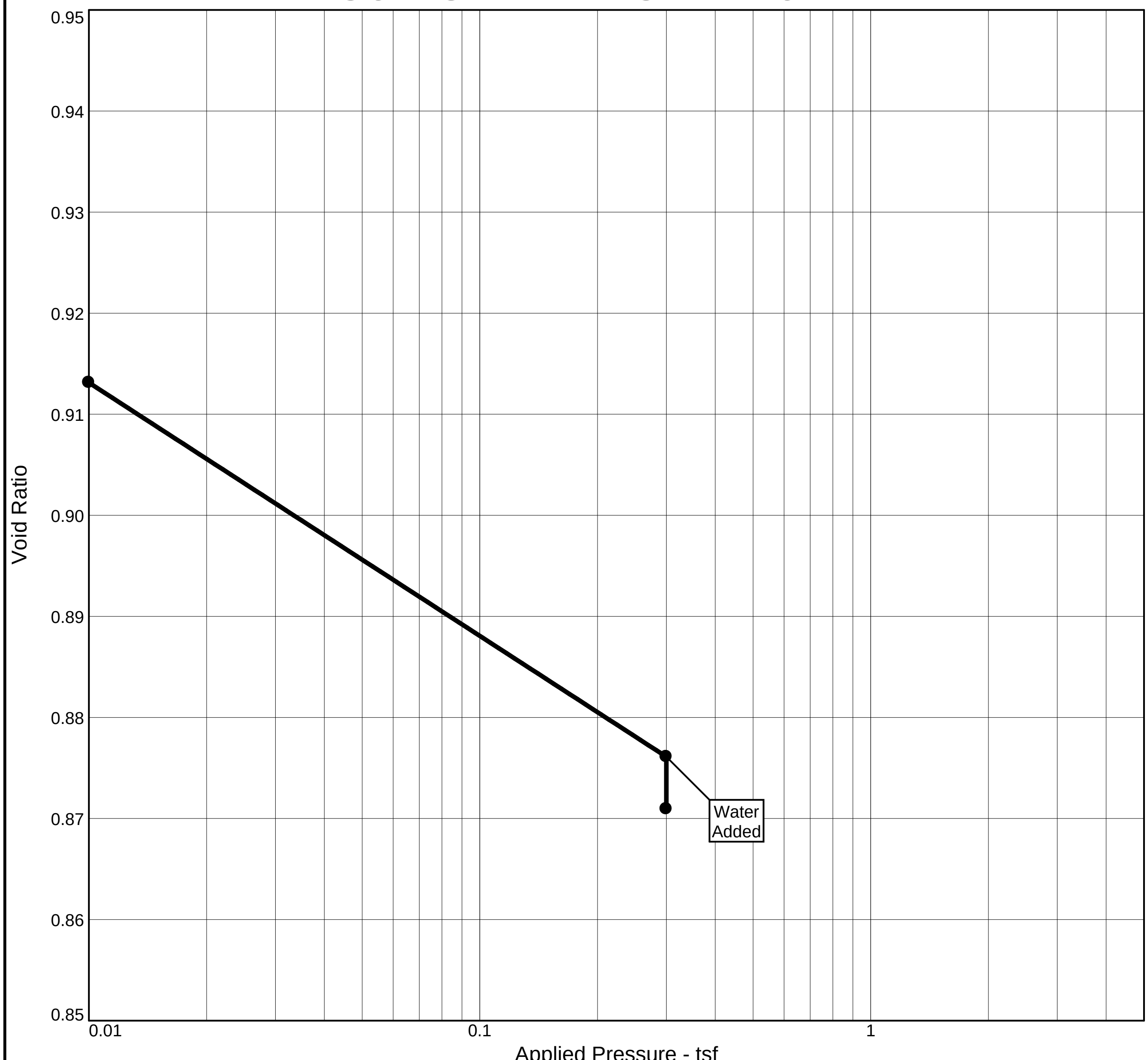
Remarks:
Specific gravity: Assumed

Figure

APPENDIX D

ONE-DIMENSIONAL SWELL TEST REPORT

SOIL SWELL TEST REPORT



Natural		Dry Dens. (pcf)	LL	PI	Sp. Gr.	USCS	AASHTO	Initial Void Ratio
Saturation	Moisture							
100.0 %	34.0 %	87.5	50	26	2.683	CH		0.913

MATERIAL DESCRIPTION

Firm brown and gray FAT CLAY (CH)

Project No. 20.33.421 **Client:** CDM Smith

Project: NAS-JRB
Belle Chase, Louisiana

Source of Sample: B-2 **Depth:** 4

Tolunay-Wong Engineers, Inc.

Gonzales, LA

Remarks:
Specific gravity: Assumed

Figure

CDM Smith®

cdmsmith.com