ECS Southwest, LLP

Geotechnical Engineering Report

Proposed Accu-Sharp Development

2205 Downing Lane
Cedar Park, TX

ECS Project Number 17:4984

July 12th, 2018

ECS SOUTHWEST, LLP
"Setting the Standard for Service"
Geotechnical • Construction Materials • Environmental • Facilities
TX Registered Engineering Firm F-8461

July 12, 2018

Mr. Jay Cavanaugh
Downing Lane LLC
10312 Milky Way
Austin, TX 78730

ECS Project No. 17:4984

Reference: Geotechnical Engineering Report
Accu-Sharp Development
2205 Downing Lane
Cedar Park, Texas

Dear Mr. Cavanaugh:

ECS Southwest, LLP (ECS) has completed the subsurface exploration, laboratory testing, and geotechnical engineering analyses for the above-referenced project. ECS Southwest, LLP (ECS) conducted this subsurface exploration and geotechnical engineering evaluation in accordance with ECS Proposal No. 17-5688 dated June 11, 2018. This report presents our understanding of the geotechnical aspects of the project, the results of the field exploration and laboratory testing conducted, and our recommendations for design and construction.

It has been our pleasure to be of service during the design phase of this project. We would appreciate the opportunity to remain involved during the continuation of the design phase, and we would like to provide our services during construction phase operations as well to verify the subsurface conditions considered for this report. Should you have any questions concerning the information contained in this report, or if we can be of further assistance to you, please contact us.

Respectfully submitted,

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EXECUTIVE SUMMARY

The following summarizes the main findings of the exploration, particularly those that may have a cost impact on the planned development. Further, our principal geotechnical recommendations are summarized. Information gleaned from the executive summary should not be utilized in lieu of reading the entire geotechnical report.

- The underlying geology of the site consists of the Cedar Park, Bee Cave, and Bull Creek Members, undivided, of the Walnut Formation (Kwcpbc), which generally consists of limestone, argillaceous limestone, and marl. The site is designated as located within the Edwards Aquifer Contributing Zone.

- The borings completed for this study generally encountered two strata, which included a 5 to 9 foot thick, dark to light brown, stiff to very hard, lean clay, sandy lean clay, fat clay, and sandy fat clay overburden soils stratum, underlain by a very hard marlstone bedrock stratum. A 4 foot tall karstic solution feature was encountered within the bedrock in one of the borings. Groundwater was not encountered in the borings completed at the site during drilling operations.

- The predominant geotechnical and geological constraints that need to be addressed at the site are the potential for karstic solution cavity and cave features in the subsurface and the very hard bedrock anticipated in excavations.

- It should be noted that there is inherent risk associated with development over bedrock that contains karstic solution cavity and cave features. Certain measures such as pilot hole drilling, electrical resistivity imaging geophysical studies, reduced bearing pressures, drilled shaft foundations designed for skin friction only, etc. can mitigate the risk. ECS is pleased to provide a discussion of available services and options upon request.

- Where cuts, trenches or drilled shafts are anticipated into the bedrock, contractors should be prepared for the requirement for heavy duty rock excavation equipment and tooling.

- Several methods exist to evaluate swell potential of expansive clay soils. We have estimated potential heave in the proposed building areas utilizing the TxDOT PVR method (Tex 124-E). We estimate the existing PVR in the proposed building areas to be about 1 inch. Fills in the building areas are recommended to be made using select fill materials to maintain this level of PVR.

- The proposed buildings can be supported by drilled shaft foundation systems. An allowable skin friction value of 3,500 psf is recommended for the socketed length of the bedrock embedment. End bearing is not recommended due to the encounter with the karstic feature within the Edwards Aquifer Contributing Zone.

- Light duty pavements can consist of 2 inches asphaltic concrete on 7 inches base on a prepared subgrade, or 5 inches concrete on a prepared subgrade. Moderate duty pavements can consist of 2½ inches asphaltic concrete on 9 inches base on prepared subgrade, or 5½ inches of concrete on a prepared subgrade. Heavy duty pavements can consist of 6½ inches of concrete on a prepared subgrade.

1.0 INTRODUCTION

1.1 General

The purpose of this study was to provide geotechnical information for the design of foundations for the proposed commercial buildings, as well as associated pavements, retaining walls, utilities and appurtenances.

The recommendations developed for this report are based on project information provided by the Client. This report contains the results of our subsurface explorations and geotechnical laboratory testing programs, site characterization, engineering analyses, and recommendations for the design and construction of the proposed improvements.

1.2 Scope of Services

To obtain the necessary geotechnical information required for evaluation of subsurface conditions supporting the proposed structures, 4 borings were drilled and sampled at the site to depths of 25 feet each and 2 borings were drilled and sampled to depths of 20 feet each beneath the existing ground surface. The number of borings and the locations of the borings were selected by the ECS, and the borings were located in the field using a handheld GPS unit. A laboratory testing program was also implemented to characterize the physical and geotechnical engineering properties of the subsurface soils.

This report discusses our exploratory and testing procedures, presents our findings and evaluations, and includes the following:

- A brief review and description of our field and laboratory test procedures and the results of testing conducted.
- A review of surface features and site conditions.
- A review of site geologic conditions.
- A review of subsurface soil/rock stratigraphy with pertinent available physical properties.
- Logs of our geotechnical test borings.
- Recommendations for site preparation, grading and drainage.
- Recommendations for foundation design and construction.
- Recommendations for retaining wall design.
- Recommendations for pavement design.

The scope of services for this project did not include an environmental assessment for determining the presence or absence of wetlands, or corrosive, hazardous or toxic materials in the soil, bedrock, surface water, groundwater, or air on or below, or around this site. Any statements in this report or on the boring logs regarding odors, colors, and unusual or suspicious items or conditions are strictly for informational purposes.

ECS did not provide any service to investigate or detect the presence of moisture, mold or other biological contaminants in or around any structure, or any service that was designed or intended to prevent or lower the risk of the occurrence of the amplification of the same. Mold is ubiquitous to the environment with mold amplification occurring when building materials are impacted by moisture.

1.3 Authorization

Our services were provided in accordance with ECS Proposal No. 17-5688 dated June 11, 2018. This study was authorized on June 13, 2018 by Mr. Jay Cavanaugh, Managing Partner with Downing Lane LLC, via signature of the referenced proposal acceptance page.

2.0 PROJECT INFORMATION

2.1 Project Location

The project is located about 1,300 feet northwest of the intersection of Downing Lane and CR 180 in Cedar Park, Texas. The location is depicted in Figure 2.1.1 as shown below.

Figure 2.1.1 Site Location

2.2 Current Site Conditions

The subject site is approximately 10.6 acres and is currently developed with a small unoccupied residential building, and contains a driveway, some trees, underbrush and grassed areas. Elevations onsite vary from approximately EL. 904 feet at the southeastern end of the site, up to about EL. 924 feet at the northwestern end of the site. Based on a review of the TCEQ Edwards Aquifer Viewer, the site is mapped within the Edwards Aquifer Contributing Zone. Drainage channels are located relatively close to the site to the north and south.

2.3 Proposed Construction

The proposed construction will include three 25,080 square foot concrete tilt-wall buildings, a detention pond, pavements and associated utilities and appurtenances. Proposed finished floor elevations were not provided at the time of this report’s preparation. Structural loading information was also not available at the time of this report’s preparation.

3.0 FIELD EXPLORATION

3.1 Field Exploration Program

The field exploration was planned with the objective of characterizing the project site in general geotechnical and geological terms and to evaluate subsequent field and laboratory data to assist in the determination of geotechnical recommendations.

3.1.1 Test Borings

The subsurface conditions were explored by drilling a total of 6 test borings. Four (4) borings were drilled to depths of about 25 feet each in the proposed building areas and 2 borings were drilled to depths of about 20 feet each in the proposed building areas. Drilling was performed using a truck-mounted drill rig, utilizing air rotary drilling methodology.

The boring locations were determined by and identified in the field by ECS personnel using a handheld GPS unit. The approximate boring locations are shown on the Boring Location Plan attached in Appendix A.

Field logs of the soils/rock encountered in the borings were maintained by the drill crew. After recovery, each geotechnical sample was removed from the sampler and visually classified. Representative portions of each soil sample were then bagged in plastic and placed in boxes and transported to our laboratory for further visual examination and laboratory testing. After completion of the exploratory operations, the boreholes were backfilled with cuttings to the existing ground surface.

3.1.2 Penetration Tests and Sampling

Standard Penetration Tests (SPTs) were performed to obtain representative samples and penetration resistance measurements in general accordance with ASTM D 1586. Soil samples were obtained at various intervals with the 1.625-inch inside diameter, 2-inch outside diameter, Split Spoon sampler. The Split Spoon sampler was first seated 6 inches to penetrate any loose cuttings, and then was driven an additional 12 inches with blows of a 140-pound hammer falling 30 inches. The number of hammer blows required to drive the sampler each 6 inch increment was recorded. The penetration resistance “N-value” is defined as the number of hammer blows required to drive the sampler the final 12 inches and is indicated on the test boring logs. In very dense materials such as weathered rock material, the SPT test is usually stopped after 50 blows from the hammer and the measurement is recorded as 50 blows per distance penetrated (i.e. 50 over 3 inches).

3.2 Regional Geology

The *Geologic Map of the West Half of Taylor, Texas* indicates that the site is underlain by the Cedar Park, Bee Cave, and Bull Creek Members, undivided, of the Walnut Formation (Kwcpbc), which generally consists of limestone, argillaceous limestone, and marl. Individual members are about 30 to 50 feet thick. A fault is mapped approximately 0.6 miles southeast of the site. The location of the site on the geologic map is provided on Figure 3.2.1 on the following page.

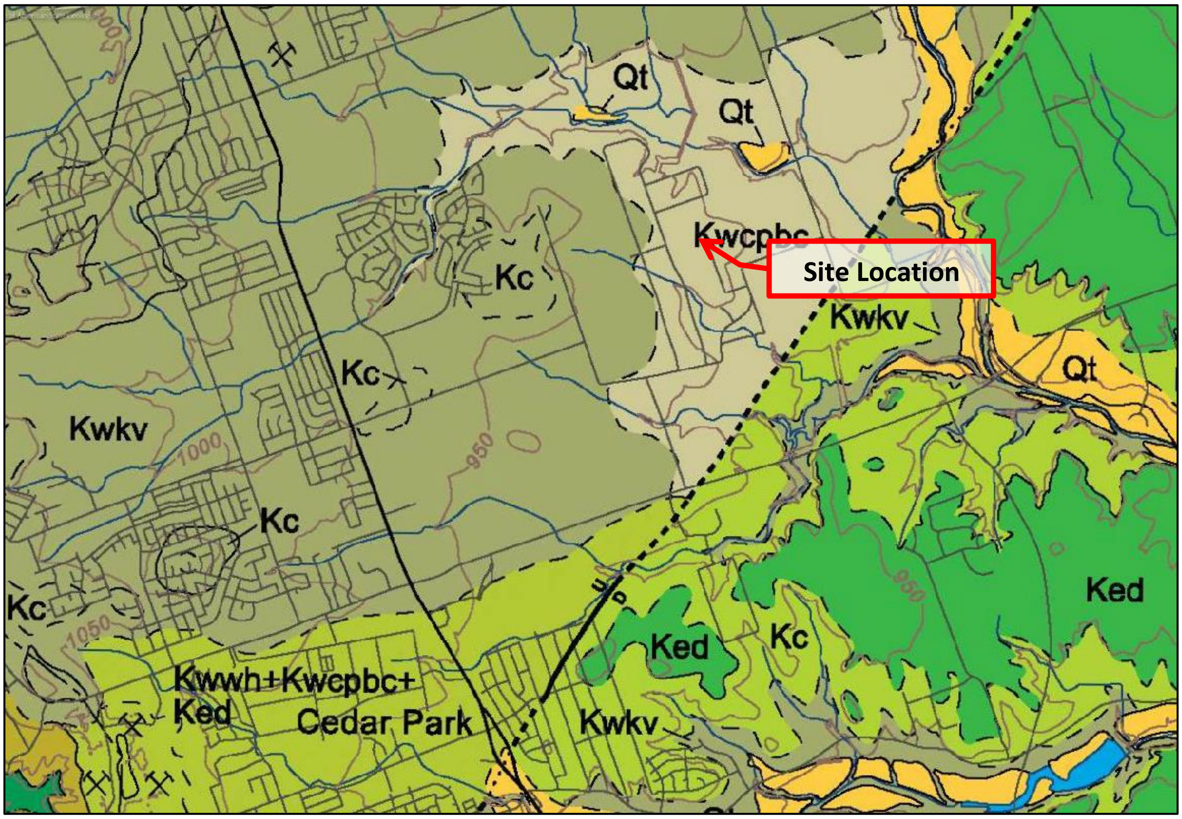


Figure 3.2.1

Map for Figure 3.2.1 obtained from the Geologic Map of the West Half of Taylor, Texas – 2005

3.3 Subsurface Characterization

The following table provides generalized characterizations of the soil and rock strata encountered during our subsurface exploration. For specific subsurface information, refer to the Boring Logs in Appendix B.

Information from the test borings indicates that the stratigraphy may generally consist of 2 distinguishable strata within the exploration depth of 25 feet. A general description of each stratum is included in the table on the following page.

STRATUM	**RANGE OF DEPTH (FT)**	**STRATUM DESCRIPTION AND CLASSIFICATION**	**WC RANGE**	**PI RANGE**	**N RANGE**
			WC AVG.	**PI AVG.**	**N AVG.**
I	0 – (5-9)	OVERBURDEN SOILS: (CL) Lean Clay and Sandy Lean Clay, and (CH) Fat Clay and Sandy Fat Clay, dark brown to light brown, stiff to very hard	10-21	8-59	11-58
			15	23	28
II	(5-9) – 25	MARLSTONE BEDROCK: Marl; light brown; very hard; Large void noted in Boring B-1 at a depth of 16' to 20'	--	--	50/5"-50/0"
			--	--	50/2"

Notes: **Depth**- Soil Stratum depth from existing ground surface at the time of our geotechnical exploration
WC- Water Content, %
PI- Plasticity Index
N- Standard Penetration Test (SPT) value, field blows per foot

Please refer to the attached boring logs, and laboratory data summary tables for a more detailed description of the subsurface conditions encountered, as the stratification descriptions above are generalized for presentation purposes.

3.4 Groundwater Observations

The borings were advanced using relatively dry techniques to their full depths, enabling the potential detection of the presence of groundwater during exploration operations. Groundwater was not encountered during or upon completion of drilling the borings at the site. Upon completion of field operations, the boreholes were backfilled with soil cuttings generated during our field operations.

It should be noted that water levels in open boreholes may require several hours to several days to stabilize depending on the permeability of the soils and that groundwater levels at the site may be subject to seasonal conditions, recent rainfall, drought or temperature effects. Clays, marls, and massive limestone are generally not conducive to the presence of groundwater; however, gravels, sands and silts, and open fractures and solution features; where present, can store and transmit "perched" groundwater flow or seepage. Therefore, groundwater conditions should be evaluated just prior to and during construction.

4.0 LABORATORY TESTING

Samples were transported to the ECS laboratory where they were examined and visually classified by an ECS geotechnical engineer using the Unified Soil Classification System (USCS) in general accordance with ASTM D 2488. To aid in classification of the soils and determination of their selected engineering characteristics, a testing program was conducted on selected samples in general accordance with the following standards:

LABORATORY TESTING	TEST STANDARD
Moisture Content	ASTM D 2216
Sieve Analysis	ASTM D 1140 and ASTM D 422
Atterberg Limits	ASTM D 4318

Results of the laboratory tests are included in the appendices on the boring logs and are presented on the laboratory test summary tables. Laboratory test results were used to classify the soils encountered as outlined by USCS in general accordance with ASTM D 2487. The USCS group symbols for each soil type are indicated in parentheses with the soil descriptions on the test logs. A brief explanation of the USCS is included in Appendix B.

All samples were returned to our laboratory in Austin, Texas. Samples not tested in the laboratory will be stored for a period of 60 days subsequent to submittal of this report and will be discarded after this period, unless we receive alternate instructions regarding their disposition.

5.0 DESIGN RECOMMENDATIONS

The following recommendations have been developed on the basis of the previously described project characteristics and subsurface conditions. If there are any changes to the project characteristics or if different subsurface conditions are encountered during construction, ECS should be consulted so that the recommendations of this report can be reviewed. We have considered that the finished floor elevations will be at or near the existing grades. If the finished grades deviate from existing grades, the recommendations provided below should be evaluated by our office.

5.1 Potential Vertical Rise

Structural damage and/or cosmetic/operational distress can be caused by volume changes in clay soils. The expansive soils found at this site are capable of swelling and shrinking in volume dependent on potentially changing soil water conditions during or after construction. Clays can shrink when they lose water and swell (grow in volume) when they gain water. The potential of expansive clays to shrink and swell is related to; amongst other things, the Plasticity Index (PI). Clays with a higher PI generally have a greater potential for soil volume changes due to moisture content variations.

Several methods exist to evaluate swell potential of expansive clay soils. We have estimated potential heave for this site utilizing the TxDOT PVR method (Tex 124-E). The Tex 124-E method provides an estimate of potential vertical rise (PVR) using the liquid limits, plasticity indices, and existing water contents for soils. The PVR is estimated in the seasonally active zone, which can be up to about 15 feet in the site vicinity, or to a depth of inert material such as marlstone bedrock.

Estimated PVR values are based upon assumed typical changes in soil moisture content from a dry (existing) to wet condition; however, soil movements in the field depend on the actual changes in moisture content. Thus, actual soil movements could be less than that calculated if little soil moisture variations occur, or the actual movement could exceed the estimated values if actual soil moisture content changes exceed the PVR methods assumed dry and wet limits. This condition is often the result of excessive droughts, flooding, “perched” groundwater infiltration, poor surface-drainage, excessive irrigation adjacent to building foundations, and/or leaking irrigation lines or plumbing.

We estimate the existing PVR in the proposed building areas to be about 1 inch. Therefore no remedial earthwork will be required to reduce the PVR to 1 inch. It is recommended that any and all fills placed in the building pad areas consist of select fill materials to maintain this level of PVR.

In this general area, most structural and geotechnical engineers consider a PVR of 1 inch to be within acceptable tolerances for properly designed drilled shaft foundation systems. However, this movement does not take into consideration the movement criteria required or perceived by the facility owner or occupants. These “operational” performance criteria may be, and often are, more restrictive than the structural criteria or tolerances.

5.2 Straight Shaft Drilled Pier Foundations

A straight shaft drilled pier foundation system can be utilized to support the proposed structures. Due to the potential for voids in the bedrock, piers are recommended to be designed for skin friction only. Piers at least 10 feet in length and embedded at least 4 feet into Stratum II marlstone bedrock and can be designed using an allowable skin friction value of 3,500 psf for the portion of the pier embedded into the undisturbed and cemented marlstone bedrock. The allowable skin friction can be applied in compression and uplift as required. The skin friction acting on the upper 5 feet of the pier shaft should be neglected in design.

Allowable skin friction incorporates a design safety factor of 2. The post-construction total and differential settlement (over a 40 foot distance) for properly constructed piers is anticipated to be approximately ½ inch and ¼ inch, respectively.

In the event that significant voids are encountered in the pier excavation sidewalls, the piers could require the use of permanent casing through the voided area, and then accumulation of skin friction commencing below the casing. Another possibility where minor voids are encountered, the shaft and void can alternately be filled with concrete, and then a straight shaft excavated the following day by drilling out the concrete in the shaft. ECS can assist with downhole video and lighting to help assess the voids and provide appropriate mitigation techniques. The Texas Commission on Environmental Quality also regulates activities having the potential for polluting the Edwards Aquifer, which can sometimes impact the construction of drilled shafts, etc. ECS can be consulted for further information on this topic.

5.2.1 Pier Foundation Lateral Resistance

Resistance to lateral loads and the expected pier behavior under the applied loading conditions will depend not only on the subsurface conditions, but also on loading conditions, the pier size, and the engineering properties of the pier. We recommend the designer use a performance based design methodology using non-linear soil support springs (“p-y curves”) to model the soil behavior. Several computer programs are commercially available for this purpose; we recommend LPILE (Ensoft, Inc.) since the software is relatively current and actively supported.

The graphical relationship between the soil resistance (p) and pile deflection (y) is commonly referred to as a “p-y curve”. Along the depth of the shaft, soil resistance (p) is expressed as a non-linear function of lateral shaft deflection (y). Various researchers developed “p-y” criteria for different kinds of soils. The “p-y” curves can be automatically generated by LPILE. Recommended design soil properties needed for generating “p-y” curves are provided in the table below.

Stratum	LPILE DESCRIPTION	EFFECTIVE UNIT WEIGHT (PCF)	C_U (PSF)	ε50
I	Stiff Clay w/o Free Water	105	1,500	0.008
II	Stiff Clay w/o Free Water	135	8,000	0.004

Notes: C_u - Undrained Cohesive Strength (psf); ε50 - strain corresponding to one-half the principle stress

The parameters presented in the above table are ultimate values. Laterally loaded foundations are typically designed according to a "limit state" and a factor of safety should be introduced into the analysis. The limit state criteria used for design should be selected by the project structural engineer.

5.2.2 Drilled Pier Group Effects Considerations

Piers should be spaced at least 3 shaft diameters, center to center apart, where the largest diameter of the piers in question is used. If piers will be placed within 3 pier diameters of each other, the skin friction acting on both piers will be affected. We recommend the values shown in the following table be used to reduce the available skin friction on all closely spaced piers for design purposes. Any groupings of more than 4 piers should be specifically reviewed by the geotechnical engineer for interaction and capacity analysis, and the reductions listed below shall not be considered valid.

CENTER TO CENTER PIER SPACING/PIER DIAMETER	ALLOWABLE SKIN FRICTION REDUCTION FACTOR
3.0	1.00
2.5	0.90
2.0	0.75
1.5	0.66
1.0	0.50

Note: If necessary, intermediate values may be interpolated linearly.

It should be noted that L-Pile analyses only covers single piles. Pile groups have an efficiency less than one when subjected to a lateral load, due to pile-soil-pile interaction in a group. For this reason, the use of p-multipliers (pm) in the L-Pile program is suggested. The AASHTO LRFD Bridge Design Specifications, 6th Edition (2013) and the FHWA Drilled Shaft Manual (2010) provide additional design guidance on this subject.

5.3 Non-Structural Slab-on-Grade Floors

The design of any grade-supported floor slab should take into consideration the interaction between the slab and the supporting soils in resisting moments and shears induced by applied loads. Several design methods use the modulus of subgrade reaction, k, to account for soil properties in design. The modulus of subgrade reaction is a spring constant that depends on the soil type, the degree of compaction and the moisture content. The k-value presented in the following table can be used for the design of flat, grade-supported floor slabs for this project. The k-value assumes that soil materials have been properly placed and compacted beneath the slab and that site drainage is good. Adequate construction joints and reinforcement should be provided to reduce the potential for cracking of the floor slabs due to differential movement.

SELECT FILL TYPE	K-VALUE, PCI
Compacted Native Soils or Select Fill	85
Compacted Native Soils, 6 Inches Compacted TxDOT Item 247 Type A, Grade 1 Base	125

Where moisture sensitive floor coverings or equipment will be installed, we recommend that at least a 10 mil vapor retarder be used beneath the slab. The vapor retarder should conform to ASTM E1745, Class C or better and shall have a maximum water vapor permeance of 0.044 when tested in accordance with ASTM E96. Consideration to specifying a thicker, more durable vapor retarder should also be made where anticipated construction traffic dictates. If a vapor retarder is considered to provide moisture protection, special attention should be given to the surface curing of the slabs to minimize uneven drying of the slabs and associated cracking and/or slab curling. Please refer to the latest edition of ACI 302.2R-06 Guide for Concrete Slabs that Receive Moisture-Sensitive Flooring Materials and ASTM E 1643 Standard Practice for Installation of Water Vapor Retarders Used in Contact with Earth or Granular Fill Under Concrete Slabs for additional guidance on this issue.

5.4 Seismic Design Considerations

For the purposes of seismic design, a Site Class C (Very Dense Soil and Soft Rock) as defined in the 2012 International Building Code (IBC) / ASCE 7 is recommended for use at the project site. The site class is based on our review of geologic maps and literature and the subsurface conditions encountered in our soil borings. Using this site class and the location of the project site (lat. 30.5468°, long. -97.8043°), probabilistic ground motion values were determined for this project and are shown in the following table:

PERIOD (SECONDS)	DESIGN SPECTRAL RESPONSE ACCELERATION PARAMETERS	SITE COEFFICIENT, FA	SITE COEFFICIENT, FV
0.2	0.050 (S_{DS})	1.2	---
1.0	0.039 (S_{D1})	---	1.7

It should be noted that our borings at the project site extended to maximum depths of 25 feet below the ground surface, whereas ASCE 7 site classifications are based on characterization of the upper 100 feet of the soil profile. The seismic parameters shown in the above table are based on the information provided in the IBC manual on Tables 1613.3.3(1) and 1613.3.3(2), the site classification, and mapped spectral response accelerations at the short and one (1) second time periods. The above parameters were developed using the United States Geological Survey geohazards webpage, and the 2012/2015 IBC design provisions.

5.5 Retaining Walls

The magnitude of the lateral earth pressures on retaining walls is dependent upon the in-situ material behind the wall; and if displaced, the type of material used to backfill the "active zone" behind the wall. The magnitude of the earth pressure is also dependent upon whether the active zone is allowed to drain water freely. The active zone can be considered as the area behind the structure within a boundary created by a 45 degree angle extending from the outside edge of the foundation heel upward to the ground surface.

The lateral earth pressures for drained, level soil backfill are expressed in terms of pounds per cubic foot (psf/ft.) "equivalent fluid" weight applied in a triangular distribution pattern as listed below. If the walls are free to deflect or rotate slightly at the top they may be designed using "active" lateral earth pressures. If the walls are laterally restrained at the top, "at-rest" lateral earth pressures should be used for the retaining wall design. Where multiple material types are used within the active zone, the higher values below should be used. The equivalent fluid weights shown in the table do not include any safety factors and do not account for any surcharges. Lateral loads from uniform surcharges on the wall backfill can calculated by multiplying the vertical surcharge by the below earth pressure coefficients and should be considered as rectangular loads acting on the full wall height. An increase of 1 pcf and 1.5 pcf should be added to the active and at-rest earth pressures; respectively, for each degree of inclination of backfill.

SOIL DESCRIPTION	TOTAL UNIT WEIGHT (PCF)	ACTIVE EARTH PRESSURE COEFFICIENT	AT-REST EARTH PRESSURE COEFFICIENT	DRAINED ACTIVE EARTH PRESSURE (PSF/FT)	DRAINED AT-REST EARTH PRESSURE (PSF/FT)
Undisturbed or Compacted Native Soil	120	0.39	0.56	47	67
Select Fill	120	0.36	0.53	43	64
Undisturbed/Cemented Marlstone Bedrock	145	0.24	0.38	34	53
ASTM C33 Size #56, #57 or #467 Stone	110	0.33	0.50	37	55
Compacted Manufactured Sand (< 8% Fines)	120	0.33	0.50	40	60
Compacted TxDOT Item 247, Type A or C, Grade 1 or 2 Base	135	0.26	0.41	35	56

For sliding resistance, a coefficient of friction of 0.32 is recommended between the base of the foundation elements and underlying soils. In addition, for footings cast directly against excavation sidewalls, a passive resistance equal to an equivalent fluid applying 250 pounds per cubic foot pressure may be used to resist lateral forces. The passive resistance should be neglected in the upper 12 inches unless the ground immediately in front of the footing is covered with concrete or other impervious pavement. The above values are ultimate values, and an appropriate safety factor should be used in design.

Retaining walls outside of the buildings pads can be supported by shallow foundations bearing on undisturbed soils or compacted fill using an allowable bearing capacity of 2,500 psf at the bearing surface. Footing excavations should have firm bottoms and be free from excessive slough prior to concrete or reinforcing steel placement. The geotechnical engineer should be allowed to observe foundation excavations prior to reinforcing steel or concrete placement to confirm anticipated ground conditions.

Retaining walls should be waterproofed as required by the project architect. Subdrain systems and/or drainboard composites are recommended to reduce hydrostatic pressures on retaining walls. A subdrain system can consist of 4 inch perforated pipe placed at the base of the retaining wall and surrounded by ASTM C33 Size #57 stone completely wrapped in Mirafi 140N or 160N filter fabric, or equivalent approved by the geotechnical engineer. The drainrock wrapped in fabric should be at least 12 inches wide and extend from the base of the wall to within two feet of the ground surface. The upper two feet of backfill should consist of compacted native soil or other impervious pavement. The retaining wall drainage system should be sloped to outlet pipes draining away from the foundations and pumped to the surface as grades require. The use of drainage openings through the base of the wall (weep holes, etc.) is not recommended where the seepage could be a nuisance or otherwise adversely impact the property adjacent to the base of the wall. The subdrain system should be inspected periodically to ensure functionality; failure of the subdrain system will affect the design lateral earth pressures and the retaining wall stability. If subdrain systems are determined to not be practical, full hydrostatic pressures should be incorporated into the wall design.

As an alternative to a stone and fabric backdrain, a prefabricated drainage composite (drainboard) such as MiraDRAIN 2000, or approved equivalent, can be used behind the retaining wall. The drainboard should extend from the base of the wall to within two feet of the ground surface, and should be installed in accordance with manufacturer specifications. A subdrain collector pipe surrounded with at least 5 cubic feet per foot size #57 stone (wrapped in filter fabric) should be installed at the base of the drainboard; or alternatively, an engineered system can be selected with sufficient capacity for direct connectivity to a closed pipe system. The groundwater should be conducted to an appropriate discharge or sump pump facility.

Where free-draining, clean granular materials will be used to backfill the walls, and where structures, pavements or other improvements will be located closely behind the retaining walls, it is recommended that all clean granular materials be separated from the soils and fills with the use of the above stated filter fabrics. The use of the filter fabric can greatly reduce the intrusion of the soils into the void spaces of the clean granular materials. Intrusion of the soils into the void spaces causes a net ground loss, and can cause settlement of the ground surface and overriding improvements.

The retaining wall backfill should be compacted and tested in maximum 8 inch lifts to be at least 95 percent of the standard Proctor maximum dry density (TxDOT 114-E) at moisture contents between optimum and plus three (+3) percentage points of the optimum moisture content.

5.6 Pavement Design

ECS has prepared the following recommendations for the design and construction of both flexible and rigid pavement systems for use on the subject project. The “AASHTO Guide for Design of Pavement Structures” published by the American Association of State Highway and Transportation Officials was used to develop the pavement thickness recommendations in this report. This method of design considers pavement performance, traffic, roadbed soil, pavement materials, environment, drainage and reliability. Each of these items is incorporated into the design methodology.

We have based our analysis on the following ESAL information and pavement-related subgrade design parameters, which are considered to be typical for the area. A CBR (California Bearing

Ratio) value of 3.5 percent was selected for design purposes. The CBR value was estimated based on ECS's knowledge and experience with similar soils and projects in this area.

RELIABILITY	70
INITIAL SERVICEABILITY INDEX, FLEXIBLE PAVEMENTS	4.2
INITIAL SERVICEABILITY INDEX, RIGID PAVEMENTS	4.5
TERMINAL SERVICEABILITY INDEX, ALL PAVEMENTS	2.0
STANDARD DEVIATION, FLEXIBLE PAVEMENTS	0.45
STANDARD DEVIATION, RIGID PAVEMENTS	0.35

Based on the design parameters listed above, we developed recommendations for "light duty," "moderate duty" and "heavy duty" pavement sections. "Light duty" pavements are intended for general parking areas with passenger vehicles only and have an approximate capacity of 20,000 ESAL. "Moderate duty" pavements are intended for areas subject to channelized traffic and delivery areas and have an approximate capacity of 80,000 ESAL. "Heavy duty" pavements are intended for areas subject to heavier vehicles with extensive turning, starting and stopping, such as pavement aprons associated with trash enclosures, and have an approximate capacity of 250,000 ESAL. If the owner or other members of the design team feel that the ESAL values used for design are not appropriate, ECS should be notified in writing, so any new information can be reviewed, and if necessary, the pavement recommendations revised accordingly.

The minimum recommended thickness for both hot mixed asphalt concrete (HMAC) and reinforced Portland cement concrete (PCC) pavement sections are presented in the following table for the described "light", "moderate" and "heavy" traffic conditions.

RECOMMENDED PAVEMENT SECTION OPTIONS						
COMPONENT	**LIGHT-DUTY 20,000 ESALS**		**MODERATE-DUTY 80,000 ESALS**		**HEAVY-DUTY 250,000 ESALS**	
	RIGID	**ASPHALT**	**RIGID**	**ASPHALT**	**RIGID**	**ASPHALT**
Portland Cement Reinforced Concrete (PCC)	5.0 in	--	5.5 in	--	6.5 in	--
Hot Mixed Asphalt Concrete (HMAC)	--	2.0 in	--	2.5 in	--	--
Crushed Limestone Base (CLB)	--	7.0 in	--	9.0 in	--	--

The pavement sections described above are considered suitable for general-purpose usage for the anticipated subgrade conditions and were designed using the AASHTO Pavement and Analysis

System. An aggressive maintenance program to keep joints and cracks sealed to prevent moisture infiltration will help extend the pavement life.

We recommend that rigid pavement sections be used in all heavy truck traffic areas. The concrete pavement should extend throughout the areas that require extensive turning and maneuvering of the delivery vehicles, etc. Waste dumpster pads, loading areas and other heavily loaded pavement areas that are not designed to accommodate these conditions often experience localized pavement failures, particularly if flexible pavement sections are used.

5.6.1 Pavement Materials

Recommendations regarding material requirements for the various pavement sections are summarized below:

> **Portland Cement Concrete** - Concrete used for paving should have a minimum compressive strength of 3,000 psi at 28-days. The air content at the point of placement should range from 2 to 4 percent. The concrete pavements should be reinforced and jointed per current ACI recommendations.
>
> **Hot Mix Asphalt Concrete (HMAC) Surface Course** - The asphalt concrete surface course should be plant mixed, hot laid Type D (Fine Graded Surface) or Type C (Coarse Graded Surface Course) meeting the specifications requirements of TxDOT Item 340 and specific criteria for the job mix formula. The mix should be compacted to between 92 and 97 percent of the maximum theoretical density as determined by TEX-227-F.
>
> **Crushed Limestone Base Course** - Crushed limestone base should be placed in maximum 6 inch compacted lifts. The base materials should be compacted to at least 98 percent of the maximum dry density as determined by TxDOT 113-E. Flexible base materials should be moisture conditioned to between minus two (-2) and plus three (+3) percentage points of the optimum moisture content during compaction. Flexible base materials should meet all requirements specified in 2004 TxDOT Standard Specification Item 247, Type A, Grade 1 or 2.

5.6.2 Rigid Pavement

Joints are typically placed in rigid pavements to control cracking, to facilitate construction, and to isolate a section of pavement from a structure or an adjacent pavement section. Joints used to control cracking are typically known as contraction or control joints as they are intended to control cracking that arises out of the shrinkage of concrete as it cures. Construction joints are used to provide clean breaks between pavement sections that result from the construction process. Isolation joints (or expansion joints) are used to separate the pavement from other structures or pavements and typically include the use of compressible materials in the joint as opposed to contraction or construction joints. Contraction joints should be spaced no greater than 15 feet between the nearest parallel joints with joint depths of at least one-quarter (¼) of the slab thickness. Contraction and construction joints should be no wider than one-eighth (⅛) of an inch whereas isolation joints may be up to one (1) inch wide.

Steel reinforcement of concrete is typically not necessary for concrete pavements with construction or isolation (expansion) joint spacing closer than 15 feet. When joints are spaced greater than 15 feet, steel reinforcement can be used to control the width of cracks that form between these joints, such the fracture faces that form in the concrete are held together. Steel reinforcement is also used where subgrade conditions are not likely to provide uniform support to the concrete pavement. Generally, sites with expansive soils present are often unable to provide such support to rigid pavement sections. Therefore, reinforcing steel can be used to span between construction and isolation (expansion) joints and should consist of at-minimum No. 3 bars spaced 18 inches on-centers each way. The rebar should be Grade 60 steel.

As with steel reinforcement, load transfer devices such as dowels are typically not necessary for most parking lots. However, in situations where heavy loads are present or the subgrade may not provide uniform support to the pavement, dowels are commonly used to transfer loads across joints. Smooth dowels can be used for this purpose and should be utilized as recommended in the following table.

DOWEL DESIGN INFORMATION				
SLAB THICKNESS, IN.	**DOWEL DIAMETER, IN.**	**MIN. DOWEL EMBEDMENT EACH SIDE, IN.**	**MIN. DOWEL LENGTH, IN.**	**DOWEL SPACING ON-CENTERS, IN**
5.0	⅝	5	12	12
5.5	¾	6	14	12
6.5	⅞	7	16	12

The joint and reinforcing design of a rigid pavement system is largely a function of geometry for the pavement area. The proper length of concrete panels (defined as the distance between discontinuous pavement sections; e.g. between construction or isolation joints, or a combination of the two) and the location of contraction, construction, and isolation (expansion) joints are not included as a function of the above concrete pavement guidelines. Rather, these features should be determined based on the geometry and construction sequencing of the pavement. Actual joint spacing should be based on actual pavement areas and final panel lengths so that joints are evenly spaced. Joints should be designed to form approximately square panels where geometrically feasible. The values provided herein are guidelines and the recommendations selected by the project civil engineer and any guidelines not provided or mentioned herein should not exceed the American Concrete Institute (ACI) 330R recommendations.

5.6.3 Pavement Drainage, Subdrainage, and Trenching

Longitudinal cracks and apparent distress due to expansive soils may appear in the pavement after construction and the introduction of landscape irrigation. These cracks and distress are not pavement failures with respect to traffic support, although they may be aesthetically undesirable. In addition, without regular maintenance, the cracks can allow additional moisture intrusion and rapid degradation of the pavement section. The pavement sections are primarily designed to support the traffic and will not resist the forces generated by swelling soils.

Positive drainage should be provided on and around pavement areas to prevent ponding of water. Irrigation of lawn and landscaped areas adjacent to the pavements should be moderate, with no excessive wetting or drying of soils adjacent to the pavements. If landscaped islands are provided, they should be designed to restrict excess water from migrating to the pavement subgrade by using self-contained beds, raised planter boxes, vertical moisture barriers, and/or edge drains. Curbs should extend through the base course and at least 4 inches into the underlying subgrade. Good perimeter surface drainage guiding surface water away from the pavement area is also recommended.

Utility trench backfill can act materially different than adjacent natural soils, even if properly placed and compacted. Differential movements may occur which can lead to crack development near the edges of utility trenches, riser structures, manholes, etc, with the more noticeable cracks appearing in deeper fill zones. This type of cracking is considered typical for this type of construction if special care is not taken to prevent it.

As an option to help mitigate the effects of differential soil movements, we recommend that fill placed at depths greater than 5 feet be compacted to no less than 98% of the maximum dry density between minus one (-1) and plus three (+3) percentage points of the optimum moisture content (TxDOT 114-E).

6.0 SITE PREPARATION, GRADING AND DRAINAGE

Preparation of the subgrade soils for areas to receive structures, fills or pavements should be conducted in accordance with the recommendations presented in the following sections.

6.1 General Site Preparation

Existing structures, foundations, vegetation, organic laden soil, loose or soft soils and any other deleterious materials must be removed from the proposed construction areas and properly disposed. Excavations resulting from the removals should be cleaned down to firm soils and backfilled with general fill in accordance with this report. Abandoned subsurface utilities and permeable backfills should be removed and/or grouted a sufficient distance from the proposed buildings to prevent the conduit of water beneath the proposed structure.

After stripping and any required cuts have been completed, the subgrade soils should be scarified, moisture conditioned and compacted to at least 95 percent of the maximum dry density as determined by TxDOT 114-E to a depth of at least 8 inches. The soils should be moisture conditioned to between optimum and plus four (+4) percentage points of the optimum moisture content just prior to compaction.

Proof-rolling should be performed where possible with a heavy (minimum 20 ton) rubber-tired vehicle such as a loaded dump truck. Soils that are observed to rut or deflect excessively under the moving load should be under-cut and replaced with compacted structural fill that meets the requirements of the section titled General Fill. All proof-rolling and under-cutting activities should be observed by ECS and should be performed during periods of dry weather.

After stripping, removals, subgrade preparation, proof-rolling and evaluation has been completed, fill placement may begin where required. Excavated soil that meets the material requirements in the General Fill section below may be used as compacted fill. If suitable fill soils have to be imported to the site, they must meet the material and compaction requirements of the General Fill section of this report.

6.2 Building Pad Grading

After stripping (as discussed in the General Site Preparation section) and the required cuts have been completed, the subgrade soils should be scarified, moisture conditioned and compacted to at least 95 percent of the maximum dry density as determined by Tex-114-E to a depth of at least 8 inches. The soils should be moisture conditioned to between optimum and plus four (+4) percentage points of the optimum moisture content just prior to compaction.

Proof-rolling should be performed where possible with a heavy (minimum 20 ton) rubber-tired vehicle such as a loaded dump truck. Soils that are observed to rut or deflect excessively under the moving load should be under-cut and replaced with compacted structural fill that meets the requirements of the section titled General Fill. All proof-rolling and under-cutting activities should be observed by ECS and should be performed during periods of dry weather.

After stripping, removals, subgrade preparation and evaluation has been completed, fill placement may begin. Fills in the building pad areas should consist of materials meeting the requirements of the Select Fill section below. Consideration should be given to creating an “all weather” working surface with the upper 6 inches of the select fill building pads. Such a working surface should consist of compacted TxDOT Item 247 Type A, Grade 1 or 2 Base material. The use of an “all weather” working surface can significantly improve the accessibility of the site to construction traffic during periods of wet weather.

The upper 18 inches of fill outside of the structures and adjoining concrete flatwork should consist of a properly compacted low permeability clay (CL) soil to reduce infiltration of moisture into the fill materials comprising the building pads. This clay layer may be replaced with asphalt or concrete pavement that extends to the edge of the structure foundation.

6.3 General Fill

General fill can consist of on-site or imported soils, provided they meet the requirements described below. All general fill materials should be clean of organics, construction debris, deleterious materials, and should be free of rocks larger than 4 inches in greatest dimension. General fill materials which are imported should have a PI of less than 30. Proposed general fill should be evaluated and tested by ECS prior to placement in the field.

ECS recommends that general fill be placed in horizontal loose lifts of not more than 8 inches in thickness. Lift thickness should be decreased when using light compaction equipment. General fill should be compacted to at least 95% of the maximum dry density at moisture contents within the range of optimum to plus four (+4) percentage points of the optimum moisture content (Tex-114-E).

6.4 Select Fill

Select fill materials should be clean of organics, construction debris, deleterious materials, and should be free of rocks larger than 4 inches in greatest dimension. Select fill should have a Plasticity Index of between 5 and 20. Select fill should be evaluated and tested by ECS prior to placement in the field.

ECS recommends that select fill be placed in horizontal loose lifts of not more than 8 inches in thickness. Select fill should be compacted to at least 95% of the maximum dry density at moisture contents within the range of minus one (-1) to plus three (+3) percentage points of the optimum moisture content (Tex-114-E).

6.5 Drainage

Water should not be allowed to collect in the foundation excavations, on foundation surfaces, or on prepared subgrades within the construction area either during or after construction. Undercut or excavated areas should be sloped toward one corner to facilitate removal of any collected rainwater, groundwater, or surface runoff. Final grading should be designed to promote positive drainage away from the structures and pavements. Soil areas within 10 feet of the buildings should slope at a minimum of 5 percent away from the structure. Adjacent pavements and concrete hardscape should slope at 1½ to 2 percent away from the structure. Roof leaders and downspouts should discharge onto paved surfaces sloping away from the structure or into a closed pipe system which outfalls to the street gutter pan or directly to the storm drain system.

7.0 CONSTRUCTION CONSIDERATIONS

7.1 Earthwork

Clayey soil is very sensitive to changes in moisture content. Subgrade support capacity will deteriorate when the moisture content increases. Effort should be made to keep fill, slab, pavement, and foundation subgrade areas properly drained and free of ponding water. Vehicle traffic on top of the subgrade should be prevented when the subgrade is visibly wet, and should be kept to a minimum at other times. Site grading and fill placement should preferably be performed during drier seasons of the year.

Fill materials should not be placed on soils that have been recently subjected to precipitation or saturation. All wet soils should be removed or allowed to dry prior to continuation of fill placement operations. Borrow fill materials, if required, should not contain wet materials at the time of placement.

If any problems are encountered during the earthwork operations, or if site conditions deviate from those encountered during our subsurface exploration, the Geotechnical Engineer should be notified immediately to determine the effect on recommendations expressed in this report.

Certain construction practices can reduce the magnitude of problems associated with moisture content increases of subgrade soil for slabs and areas to receive compacted fill. The contractor should seal exposed subgrade areas at the end of the work day with a smooth drum roller to reduce the potential for infiltration of water into the subgrade. Site grading should be continuously evaluated to assure that surface runoff will drain away from slab and fill areas.

7.2 Drilled Pier Foundations

To reduce the potential for arching within the shaft or casing, ECS recommends using a concrete mix with a slump of 6 inches, +/- 1 inch. “Tailgating” and the free-fall method might allow the concrete to strike reinforcing steel, casing, or shaft sidewalls, causing segregation and undesirable concrete strength properties. The free-fall method is acceptable for dry holes if the concrete is directed through a hopper and falls down the center of the shaft without striking the sides of the shaft or the reinforcing steel cage. A tremie pipe should be used to place concrete in any wet excavations.

Zones of groundwater may be encountered during pier construction. Depending on the amount of water, project specifications should include the use of casing. The removal of the casing should be performed with care to avoid mixing of soils and groundwater with the fresh concrete. The volume of concrete used should be checked as a quality control measure to reduce the probability for voids or necking within the pier.

Concrete placement in drilled shafts should comply with the American Concrete Institute (ACI) 318-99 Building Code Requirements for Structural Concrete, ACI 336.1-98 Standard Specification for Construction of Drilled Piers and ACI 336.3R-93 Suggested Design and Construction Procedures for Pier Foundations. The contractor must comply with local, state, and federal safety regulations concerning excavations and drilled shafts.

7.3 Utility Trench Construction

Utility trenches in the building pads should be backfilled above the utility bedding and shading materials with select fill, and general fill material outside the building pad areas. The backfill materials should be placed in lifts not to exceed 8 inches loose measure, or 6 inches compacted measure. Thinner lifts may be required when using hand held compaction equipment. Backfill materials should be moisture conditioned to between optimum and plus three (+3) percentage points of the optimum moisture content and compacted to at least 95 percent of the maximum dry density as determined by TxDOT 114-E.

Utility trenches should be sealed with lean concrete, lean clayey soil, controlled low-strength material or flowable fill where the utility approaches and enters the building pad areas. This would reduce the potential for migration of water beneath the buildings through the bedding and shading materials in the utility trench.

8.0 FIELD OBSERVATIONS & TESTING

Personnel from ECS should perform the field observations and testing recommended in this report because of our familiarity with the project and site conditions. The performance of foundations and pavements is primarily controlled by the quality of the construction. To prevent misinterpretation of our recommendations, ECS should be retained to perform full time quality control testing, inspection, and documentation during construction of the foundations and pavements.

The performance of slabs and pavements placed on new fill material is controlled by the quality of the compaction and the materials selection for the fill material. ECS should be retained to perform quality control testing and inspection during selection, placement, and compaction of the fill material.

8.1 Earthwork and Pavements

Field observations and testing should be performed during the earthwork operations to document proper construction. Stripping should be observed by the Geotechnical Engineer to help locate unsuitable materials that should be removed prior to placement of fill, slab, or pavement materials. Field observation and inspection should include final approval of subgrades prior to placement of compacted fill, slabs, or pavement. Proof-rolling should be performed by a heavy rubber-tired vehicle such as a loaded dump truck on slab and pavement subgrades. Appropriate laboratory tests such as Proctor moisture-density tests and Atterberg Limits should be performed on samples of fill material and pavement base course material. Field moisture-density tests and visual observation of lift thickness and material types should be performed during compaction operations to document that the construction satisfies material and compaction requirements. The frequency of field density tests should be at least 2 tests per lift per 10,000 sf of building area, at least 2 tests per lift per 10,000 sf of pavement area, and at least 1 test per lift per 200 linear feet of utility trench.

8.2 Drilled Pier Foundations

The performance of the foundation system is highly dependent on the quality of the installation. Therefore, the construction of drilled shafts should be observed by a Geotechnical Engineer or qualified Engineering Technician to document the drilling conditions encountered, the cleaning of the bottom of the shaft, the type of bearing material, the depth and diameter of shaft, the size, number, configuration, and grade of steel reinforcement, and the volume of concrete used. The suitability of the bearing soil will be based on a review of test boring data, physical observation of the material type excavated, and observation of the cleaning of the bottom of the shaft.

Concrete material should be sampled and tested for compressive strength, and placement operations should be monitored to record concrete slump, air content, temperature, and age at time of placement. If slurry is used, properties such as specific gravity and viscosity should be tested and documented. During concrete placement, the elevation of the concrete should be monitored and at no time should the relative elevation of the concrete increase during casing extraction. This event implies that the concrete is adhering to the casing, causing a vacuum condition and possible soil intrusion into the shaft. If any piers are contaminated in this way, coring should be performed to determine pier integrity.

9.0 EXCAVATIONS

Drilled shaft foundation and earthwork contractors should be prepared with high torque drill rigs and appropriate rock excavation tooling to excavate the very hard bedrock at the site.

Our comments on excavation are based on our experience in the project vicinity and examination of the recovered samples. Excavation depends on the contractor's equipment, capabilities, and experience. Therefore, it should be the contractor's responsibility to determine the most effective methods for excavation. The above comments are intended for informational purposes for the design team only and may be used to review the contractor's proposed excavation methods.

Excavations that will receive compacted fill should have vertical or benched sidewalls so that lifts of fill material will be placed and compacted on horizontal planes. Stockpiles of soil or materials, and heavy equipment should not be placed immediately above and adjacent to unbraced vertical excavation walls (trenches).

In Federal Register, Volume 54, No. 209 (October 1989), the United States Department of Labor, Occupational Safety and Health Administration (OSHA) amended its “Construction Standards for Excavations, 29 CFR, Part 1926, subpart P”. This document was issued to insure the safety of workmen entering trenches or excavations.

It is mandated by this federal regulation that all excavations such as utility trenches, basement excavation, or footing excavations be constructed in accordance with the new OSHA guidelines. These regulations are enforced.

The contractor is solely responsible for designing and constructing stable, temporary excavations and for shoring, sloping, or benching the sides of excavations as required to maintain stability of both the excavation sides and bottom. The contractor’s responsible person as defined in 29 CFR Part 1926 should evaluate the soil exposed in the excavations as part of the contractor’s safety procedures. In no case should slope height, slope inclination, or excavation depth exceed those specified in all local, state, and federal safety regulations.

We are providing this information solely as a service to our client. ECS does not assume responsibility for construction site safety or the contractor’s or other party’s compliance with local, state, and federal regulations.

10.0 LIMITATIONS

This report has been prepared to aid in the evaluation of subsurface conditions at this site and to assist design professionals in the geotechnical related design of this project. It is intended for use with regard to the specific project as described in this report. Any substantial changes or differences in understood building loads, building and pavement layouts, understood finished floor elevation, or understood site grading should be brought to our attention so that we may determine any effect on the recommendations provided in this report. It is recommended that all construction operations dealing with earthwork and foundations be reviewed by an experienced Geotechnical Engineer to provide information on which to base a decision as to whether the design requirements are fulfilled in the actual construction.

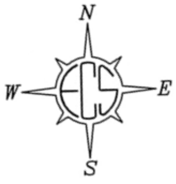
The opinions and conclusions expressed in this report are those of ECS and represent interpretation of the subsurface conditions based on tests and the results of our analyses. ECS is not responsible for the interpretation or implementation by others of recommendations provided in this report. This report has been prepared in accordance with generally accepted principles of geotechnical engineering practice and no warranties are included, expressed, or implied, as to the professional services provided under the terms of our agreement.

The analysis and recommendations submitted in this report are based upon the data obtained from the test borings and test pits performed at the locations indicated in the exploration location plan, and from other information described in this report. This report does not reflect any variations that may occur around the test borings and test pits. In the performance of the subsurface exploration, specific information is obtained at specific locations at specific times. However, it is a well known fact that variations in soil conditions and depth to rock exist on most sites between test boring and test pit locations, and conditions such as groundwater levels vary from time to time. The nature and extent of variations may not become evident until the course of construction. If variations then appear evident, after allowing ECS to perform on-site observations during the construction period and note characteristics and variations, a re-evaluation of the recommendations in this report will be necessary.

The client should consider including a “changed conditions” clause in the contract documents. Such a clause might permit contractors to provide a lower bid since they can consider lower costs to account for contingencies. The client should consider including an arbitration procedure in the contract documents for situations when the owner, contractor, and design professionals do not agree on the changed conditions and their effects at the moment they are disclosed.

APPENDIX A – Figures

Site Location Plan
Boring Location Plan
Site Geologic Map



Google Imagery Date: January 13, 2018

ECS-SOUTHWEST, LLP
14050 Summit Drive, Suite 101
Austin, Texas 78728

PROJECT No.: 17-4984

FIGURE: **1**

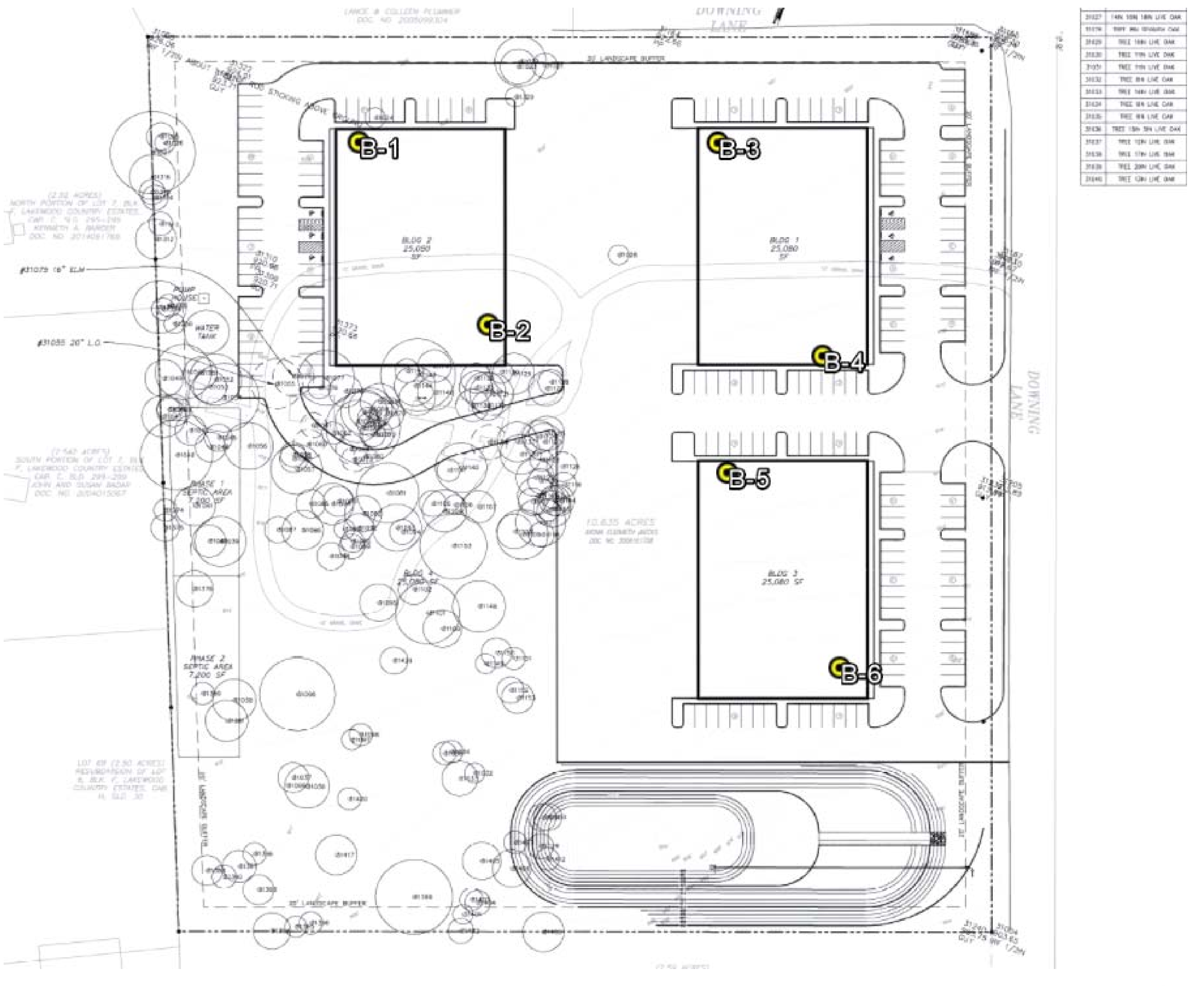
SCALE: NTS

DATE: JULY 2018

PM:MS

FIG 1: Site Location Plan

Proposed Accu-Sharp
Development
2205 Downing Lane
Cedar Park, TX



● - Approximate Boring Location

FIG 2: Boring Location Plan

Proposed Accu-Sharp
Development
2205 Downing Lane
Cedar Park, TX

ECS-SOUTHWEST, LLP
14050 Summit Drive, Suite 101
Austin, Texas 78728

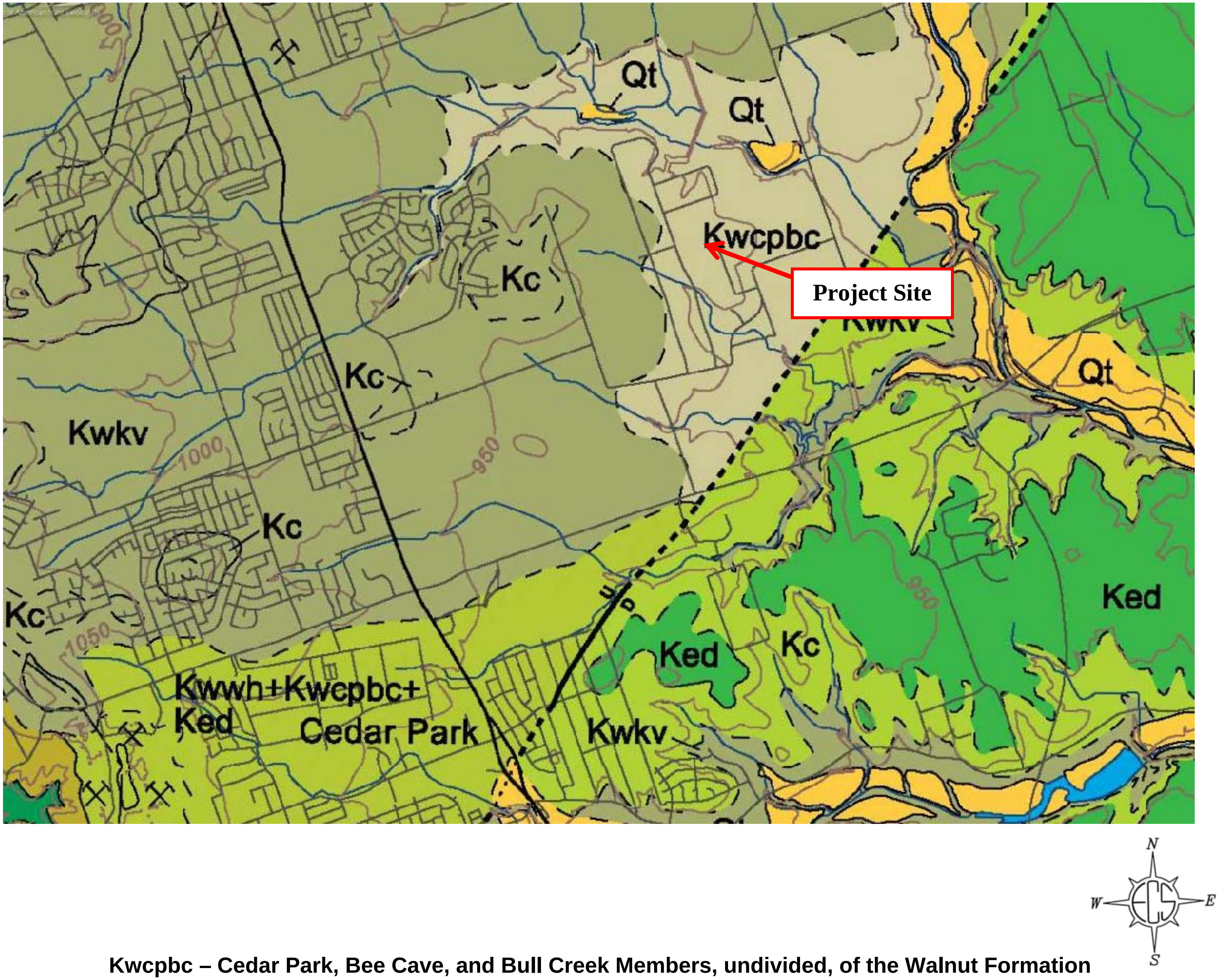
SCALE: NTS

PROJECT No.: 17-4984

DATE: JULY 2018

FIGURE: **2**

PM:MS



Kwcpbc – Cedar Park, Bee Cave, and Bull Creek Members, undivided, of the Walnut Formation

Geologic Map of the West Half of Taylor, Texas - Bureau of Economic Geology, The University of Texas at Austin, 2005

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14050 Summit Drive, Suite 101
Austin, Texas 78728

PROJECT No.: 17-4984

FIGURE: **3**

SCALE: NTS

DATE: JULY 2018

PM:MS

FIG 3: Site Geologic Map

Proposed Accu-Sharp
Development
2205 Downing Lane
Cedar Park, TX

APPENDIX B – Field Operations

Boring Logs B-1 to B-6
Reference Notes for Boring Logs
Unified Soil Classification System Information

CLIENT	Job #:	BORING #	SHEET
Downing Lane LLC	17:4984	B-1	1 OF 1

PROJECT NAME: Accu-Sharp Development

ARCHITECT-ENGINEER:

SITE LOCATION: 2205 Downing Lane, Cedar Park, TX

NORTHING: | EASTING: | STATION:

DESCRIPTION OF MATERIAL — ENGLISH UNITS

BOTTOM OF CASING ➤ LOSS OF CIRCULATION

SURFACE ELEVATION 923 Feet

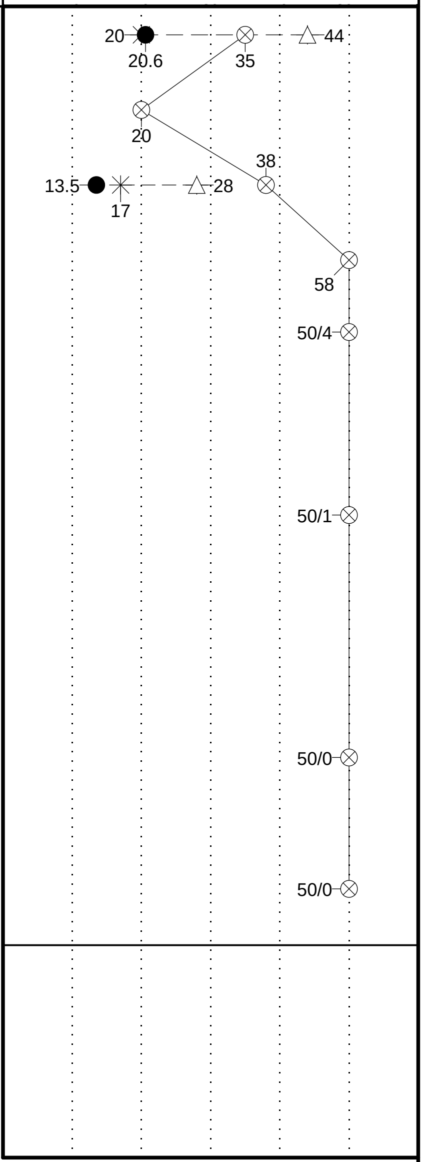
-○- CALIBRATED PENETROMETER TONS/FT^2

ROCK QUALITY DESIGNATION & RECOVERY
RQD% – – – REC% ———

PLASTIC LIMIT% ✕ — WATER CONTENT% ● — LIQUID LIMIT% △

⊗ STANDARD PENETRATION BLOWS/FT

DEPTH (FT)	SAMPLE NO.	SAMPLE TYPE	SAMPLE DIST. (IN)	RECOVERY (IN)	DESCRIPTION OF MATERIAL	WATER LEVELS	ELEVATION (FT)	BLOWS/6"
0	S-1	SS	18	18	(CL) LEAN CLAY WITH SAND, Light Brown to Light Reddish Brown, Very Stiff to Very Hard			10 15 20
	S-2	SS	18	18			920	5 8 12
5	S-3	SS	18	18				9 18 20
	S-4	SS	18	18				19 28 30
							915	
	S-5	SS	4	4	MARL, Light Brown, Very Hard			50/4
10								
							910	
	S-6	SS	1	1				50/1
15								
					Karstic Solution Cavity			
							905	
20	S-7	SS	0	0	MARL, Light Brown, Very Hard			50/0
							900	
	S-8	SS	0	0				50/0
25					END OF BORING @ 25'			
							895	
30								



THE STRATIFICATION LINES REPRESENT THE APPROXIMATE BOUNDARY LINES BETWEEN SOIL TYPES. IN-SITU THE TRANSITION MAY BE GRADUAL.

WL None WS ☐ WD ☒	BORING STARTED 06/25/18	CAVE IN DEPTH
WL(SHW) WL(ACR)	BORING COMPLETED 06/25/18	HAMMER TYPE Auto
WL	RIG CME 75 FOREMAN Austin Geo	DRILLING METHOD Air Rotary, SS

CLIENT
Downing Lane LLC

Job #: 17:4984

BORING # B-2

SHEET 1 OF 1

PROJECT NAME
Accu-Sharp Development

ARCHITECT-ENGINEER

SITE LOCATION
2205 Downing Lane, Cedar Park, TX

NORTHING

EASTING

STATION

DESCRIPTION OF MATERIAL — ENGLISH UNITS

BOTTOM OF CASING

LOSS OF CIRCULATION

SURFACE ELEVATION 920 Feet

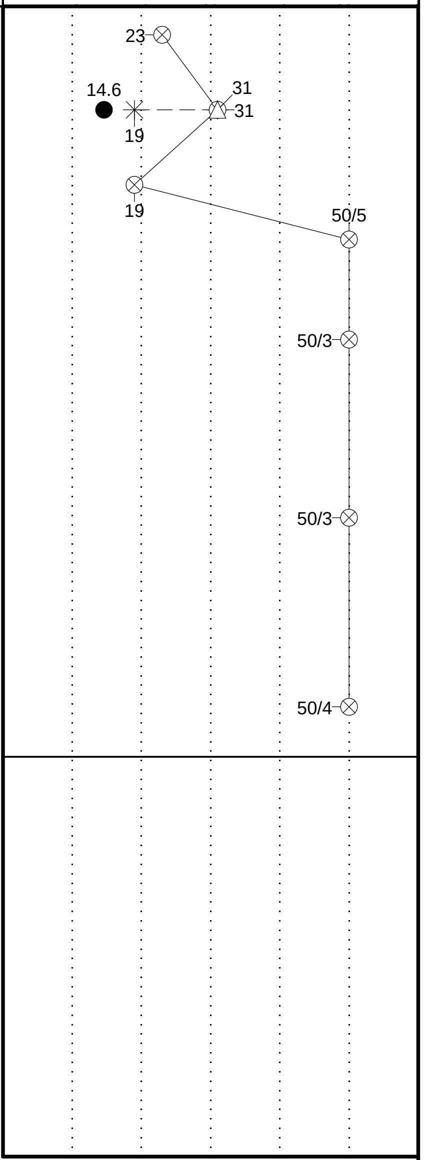
―○― CALIBRATED PENETROMETER TONS/FT^2

ROCK QUALITY DESIGNATION & RECOVERY
RQD% – – – REC% ―――

PLASTIC LIMIT% ✕ WATER CONTENT% ● LIQUID LIMIT% △

⊗ STANDARD PENETRATION BLOWS/FT

DEPTH (FT)	SAMPLE NO.	SAMPLE TYPE	SAMPLE DIST. (IN)	RECOVERY (IN)	DESCRIPTION OF MATERIAL	WATER LEVELS ELEVATION (FT)	BLOWS/6"
0	S-1	SS	18	18	(CL) LEAN CLAY WITH SAND, Light Brown, Very Stiff	920	5 14 9
	S-2	SS	18	18	(CL) LEAN CLAY, Light Brown, Hard to Very Stiff		13 15 16
5	S-3	SS	18	18		915	12 12 7
	S-4	SS	5	5	MARL, Light Brown, Very Hard		50/5
	S-5	SS	9	9			17 50/3
10						910	
	S-6	SS	3	3			50/3
15						905	
	S-7	SS	4	4			50/4
20					END OF BORING @ 20'	900	
25						895	
30						890	



THE STRATIFICATION LINES REPRESENT THE APPROXIMATE BOUNDARY LINES BETWEEN SOIL TYPES. IN-SITU THE TRANSITION MAY BE GRADUAL.

WL None WS ☐ WD ☒	BORING STARTED 06/25/18	CAVE IN DEPTH
WL(SHW) WL(ACR)	BORING COMPLETED 06/25/18	HAMMER TYPE Auto
WL	RIG CME 55 FOREMAN Austin Geo	DRILLING METHOD Air Rotary, SS

CLIENT	Job #:	BORING #	SHEET
Downing Lane LLC	17:4984	B-3	1 OF 1

PROJECT NAME: Accu-Sharp Development

ARCHITECT-ENGINEER:

SITE LOCATION: 2205 Downing Lane, Cedar Park, TX

NORTHING: EASTING: STATION:

DESCRIPTION OF MATERIAL — ENGLISH UNITS

BOTTOM OF CASING ▶ LOSS OF CIRCULATION

SURFACE ELEVATION 921 Feet

—○— CALIBRATED PENETROMETER TONS/FT^2

ROCK QUALITY DESIGNATION & RECOVERY

RQD% — — — REC% ———

PLASTIC LIMIT% × WATER CONTENT% ● LIQUID LIMIT% △

⊗ STANDARD PENETRATION BLOWS/FT

SAMPLE NO.	SAMPLE TYPE	SAMPLE DIST. (IN)	RECOVERY (IN)	BLOWS/6"
S-1	SS	18	18	6 8 6
S-2	SS	18	18	6 5 6
S-3	SS	18	18	5 7 6
S-4	SS	18	18	7 8 9
S-5	SS	8	8	35 50/2
S-6	SS	3	3	50/3
S-7	SS	4	4	50/4
S-8	SS	1	1	50/1

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

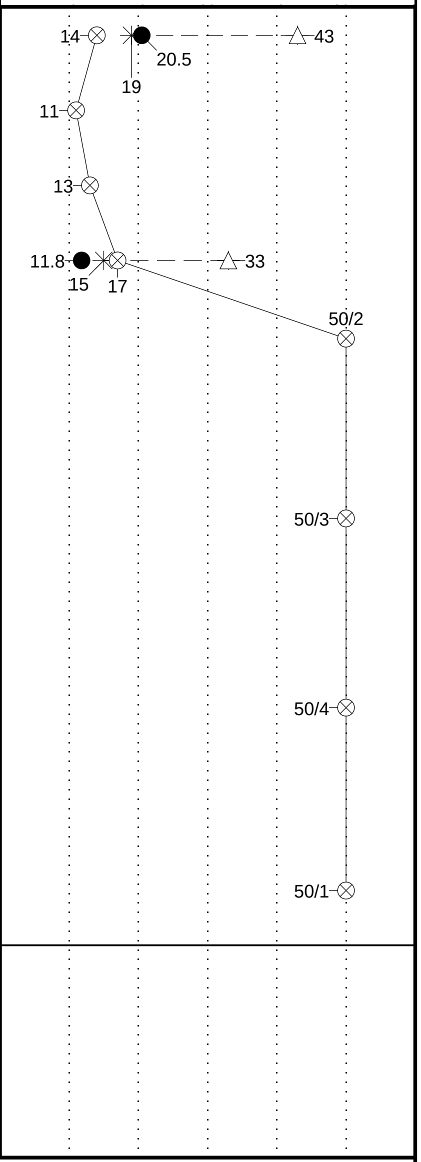
ELEVATION (FT): 920, 915, 910, 905, 900, 895

(CL) SANDY LEAN CLAY, Light Reddish Brown, Stiff

(CL) LEAN CLAY, Light Brown, Stiff to Very Stiff

MARL, Light Brown, Very Hard

END OF BORING @ 25'



THE STRATIFICATION LINES REPRESENT THE APPROXIMATE BOUNDARY LINES BETWEEN SOIL TYPES. IN-SITU THE TRANSITION MAY BE GRADUAL.

WL None WS ☐ WD ☒	BORING STARTED 06/25/18	CAVE IN DEPTH
WL(SHW) WL(ACR)	BORING COMPLETED 06/25/18	HAMMER TYPE Auto
WL	RIG CME 55 FOREMAN Austin Geo	DRILLING METHOD Air Rotary, SS

CLIENT Downing Lane LLC

Job #: 17:4984

BORING # B-4

SHEET 1 OF 1

PROJECT NAME Accu-Sharp Development

ARCHITECT-ENGINEER

SITE LOCATION 2205 Downing Lane, Cedar Park, TX

NORTHING

EASTING

STATION

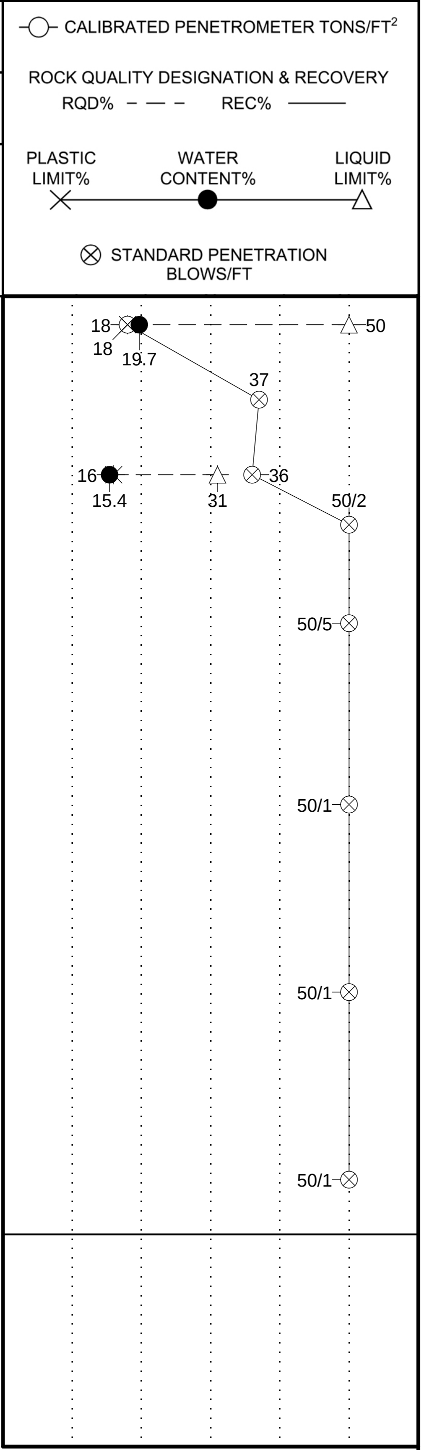
DESCRIPTION OF MATERIAL ENGLISH UNITS

BOTTOM OF CASING

LOSS OF CIRCULATION 100%

SURFACE ELEVATION 916 Feet

DEPTH (FT)	SAMPLE NO.	SAMPLE TYPE	SAMPLE DIST. (IN)	RECOVERY (IN)	DESCRIPTION OF MATERIAL	WATER LEVELS	ELEVATION (FT)	BLOWS/6"
0	S-1	SS	18	18	(CL/CH) SANDY LEAN TO FAT CLAY, Dark to Light Brown, Very Stiff		915	4 8 10
	S-2	SS	18	18	(CL) LEAN CLAY WITH SAND, Light Brown, Hard			15 16 21
5	S-3	SS	18	18				11 15 21
	S-4	SS	2	2	MARL, Light Brown, Very Hard		910	50/2
	S-5	SS	5	5				50/5
10								
							905	
	S-6	SS	1	1				50/1
15								
							900	
	S-7	SS	1	1				50/1
20								
							895	
	S-8	SS	1	1				50/1
25					END OF BORING @ 25'			
							890	
30								



THE STRATIFICATION LINES REPRESENT THE APPROXIMATE BOUNDARY LINES BETWEEN SOIL TYPES. IN-SITU THE TRANSITION MAY BE GRADUAL.

WL None WS ☐ WD ☒	BORING STARTED 06/25/18	CAVE IN DEPTH
WL(SHW) WL(ACR)	BORING COMPLETED 06/25/18	HAMMER TYPE Auto
WL	RIG CME 75 FOREMAN Austin Geo	DRILLING METHOD Air Rotary, SS

CLIENT Downing Lane LLC	Job #: 17:4984	BORING # B-5	SHEET 1 OF 1
PROJECT NAME Accu-Sharp Development	ARCHITECT-ENGINEER		

SITE LOCATION
2205 Downing Lane, Cedar Park, TX

NORTHING
EASTING
STATION

-○- CALIBRATED PENETROMETER TONS/FT^2

ROCK QUALITY DESIGNATION & RECOVERY
RQD% – — – REC% ———

PLASTIC LIMIT% ✕ WATER CONTENT% ● LIQUID LIMIT% △

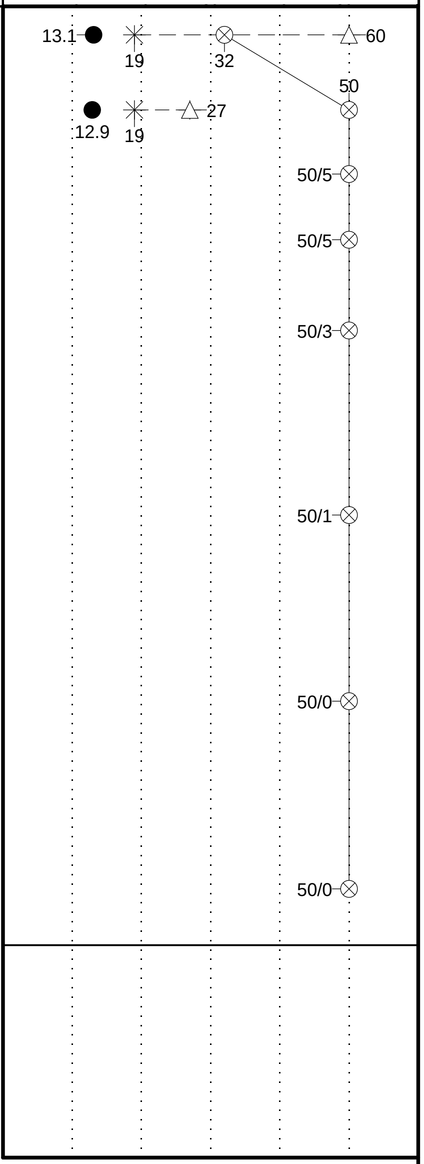
⊗ STANDARD PENETRATION BLOWS/FT

DESCRIPTION OF MATERIAL ENGLISH UNITS
BOTTOM OF CASING LOSS OF CIRCULATION
SURFACE ELEVATION 914 Feet

SAMPLE NO.	SAMPLE TYPE	SAMPLE DIST. (IN)	RECOVERY (IN)	BLOWS/6"	DESCRIPTION OF MATERIAL
S-1	SS	18	18	4 10 22	(CH) SANDY FAT CLAY WITH GRAVEL, Dark Brown to Light Reddish Brown, Hard
S-2	SS	18	18	10 25 25	(CL) LEAN CLAY WITH SAND, Light Brown, Very Hard
S-3	SS	11	11	17 50/5	
S-4	SS	5	5	50/5	MARL, Light Brown, Very Hard
S-5	SS	3	3	50/3	
S-6	SS	1	1	50/1	
S-7	SS	0	0	50/0	
S-8	SS	0	0	50/0	

END OF BORING @ 25'

DEPTH (FT): 0, 5, 10, 15, 20, 25, 30
ELEVATION (FT): 910, 905, 900, 895, 890, 885



THE STRATIFICATION LINES REPRESENT THE APPROXIMATE BOUNDARY LINES BETWEEN SOIL TYPES. IN-SITU THE TRANSITION MAY BE GRADUAL.

WL None WS ☐ WD ☒	BORING STARTED 06/25/18	CAVE IN DEPTH
WL(SHW) WL(ACR)	BORING COMPLETED 06/25/18	HAMMER TYPE Auto
WL	RIG CME 55 FOREMAN Austin Geo	DRILLING METHOD Air Rotary, SS

CLIENT Downing Lane LLC	Job #: 17:4984	BORING # B-6	SHEET 1 OF 1
PROJECT NAME Accu-Sharp Development	ARCHITECT-ENGINEER		
SITE LOCATION 2205 Downing Lane, Cedar Park, TX			
NORTHING	EASTING	STATION	

-○- CALIBRATED PENETROMETER TONS/FT^2

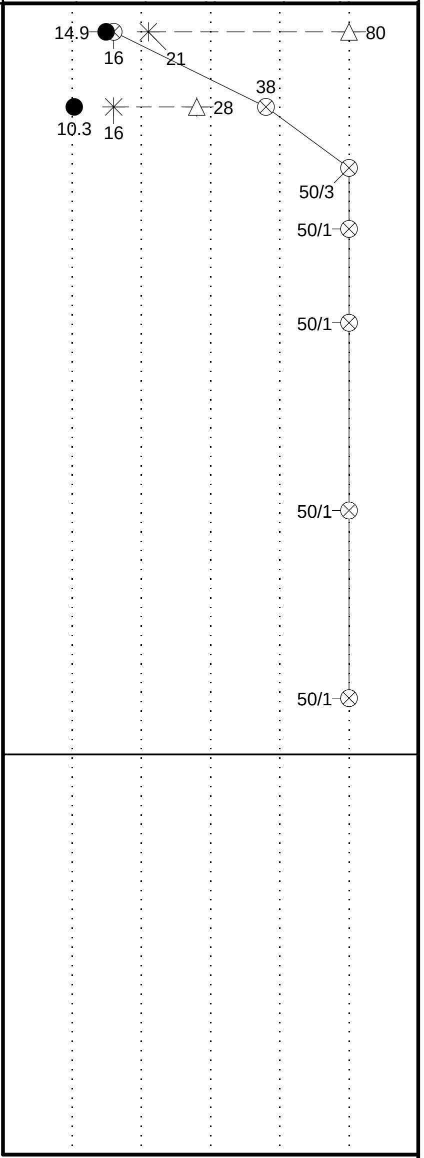
ROCK QUALITY DESIGNATION & RECOVERY
RQD% – — – REC% ———

PLASTIC LIMIT% × WATER CONTENT% ● LIQUID LIMIT% △

⊗ STANDARD PENETRATION BLOWS/FT

DESCRIPTION OF MATERIAL — ENGLISH UNITS
BOTTOM OF CASING — LOSS OF CIRCULATION
SURFACE ELEVATION 910 Feet

DEPTH (FT)	SAMPLE NO.	SAMPLE TYPE	SAMPLE DIST. (IN)	RECOVERY (IN)	DESCRIPTION OF MATERIAL	WATER LEVELS	ELEVATION (FT)	BLOWS/6"
0	S-1	SS	18	18	(CH) FAT CLAY WITH SAND, Dark to Light Brown, Very Stiff		910	3 6 10
	S-2	SS	18	18	(CL) LEAN CLAY WITH SAND, Light Brown, Hard			15 15 23
	S-3	SS	9	9				32 50/3
5					MARL, Light Brown, Very Hard		905	
	S-4	SS	0	0				50/0
	S-5	SS	0	0				50/0
10							900	
	S-6	SS	0	0				50/0
15							895	
	S-7	SS	0	0				50/0
20					END OF BORING @ 20'		890	
25							885	
30							880	



THE STRATIFICATION LINES REPRESENT THE APPROXIMATE BOUNDARY LINES BETWEEN SOIL TYPES. IN-SITU THE TRANSITION MAY BE GRADUAL.

WL None WS ☐ WD ☒	BORING STARTED 06/25/18	CAVE IN DEPTH
WL(SHW) WL(ACR)	BORING COMPLETED 06/25/18	HAMMER TYPE Auto
WL	RIG CME 75 FOREMAN Austin Geo	DRILLING METHOD Air Rotary, SS

REFERENCE NOTES FOR BORING LOGS

MATERIAL[1,2]

■		**ASPHALT**
░		**CONCRETE**
○		**GRAVEL**
▨		**TOPSOIL**
□		**VOID**
▦		**BRICK**
○		**AGGREGATE BASE COURSE**
▒	**FILL**[3]	**MAN-PLACED SOILS**
▓	**GW**	**WELL-GRADED GRAVEL** gravel-sand mixtures, little or no fines
▓	**GP**	**POORLY-GRADED GRAVEL** gravel-sand mixtures, little or no fines
▓	**GM**	**SILTY GRAVEL** gravel-sand-silt mixtures
▓	**GC**	**CLAYEY GRAVEL** gravel-sand-clay mixtures
░	**SW**	**WELL-GRADED SAND** gravelly sand, little or no fines
░	**SP**	**POORLY-GRADED SAND** gravelly sand, little or no fines
░	**SM**	**SILTY SAND** sand-silt mixtures
▨	**SC**	**CLAYEY SAND** sand-clay mixtures
▥	**ML**	**SILT** non-plastic to medium plasticity
▥	**MH**	**ELASTIC SILT** high plasticity
▨	**CL**	**LEAN CLAY** low to medium plasticity
▨	**CH**	**FAT CLAY** high plasticity
▧	**OL**	**ORGANIC SILT or CLAY** non-plastic to low plasticity
■	**OH**	**ORGANIC SILT or CLAY** high plasticity
▒	**PT**	**PEAT** highly organic soils

DRILLING SAMPLING SYMBOLS & ABBREVIATIONS

SS	Split Spoon Sampler	PM	Pressuremeter Test
ST	Shelby Tube Sampler	RD	Rock Bit Drilling
WS	Wash Sample	RC	Rock Core, NX, BX, AX
BS	Bulk Sample of Cuttings	REC	Rock Sample Recovery %
PA	Power Auger (no sample)	RQD	Rock Quality Designation %
HSA	Hollow Stem Auger	TCP	Texas Cone Penetrometer

PARTICLE SIZE IDENTIFICATION

DESIGNATION		PARTICLE SIZES
Boulders		12 inches (300 mm) or larger
Cobbles		3 inches to 12 inches (75 mm to 300 mm)
Gravel:	Coarse	¾ inch to 3 inches (19 mm to 75 mm)
	Fine	4.75 mm to 19 mm (No. 4 sieve to ¾ inch)
Sand:	Coarse	2.00 mm to 4.75 mm (No. 10 to No. 4 sieve)
	Medium	0.425 mm to 2.00 mm (No. 40 to No. 10 sieve)
	Fine	0.074 mm to 0.425 mm (No. 200 to No. 40 sieve)
Silt & Clay ("Fines")		<0.074 mm (smaller than a No. 200 sieve)

COHESIVE SILTS & CLAYS

UNCONFINED COMPRESSIVE STRENGTH, Q_P[4]	SPT[5] (BPF)	CONSISTENCY[7] (COHESIVE)
<0.25	<2	Very Soft
0.25 - <0.50	2 - 4	Soft
0.50 - <1.00	4 - 8	Medium Stiff
1.00 - <2.00	8 - 16	Stiff
2.00 - <4.00	16 - 30	Very Stiff
4.00 - 8.00	31 - 50	Hard
>8.00	>50	Very Hard

GRAVELS, SANDS & NON-COHESIVE SILTS

SPT[5]	DENSITY
<5	Very Loose
5 - 10	Loose
10 - 30	Medium Dense
31 - 50	Dense
>50	Very Dense

RELATIVE AMOUNT[7]	COARSE GRAINED (%)	FINE GRAINED (%)
Trace	<5	<5
Dual Symbol *(ex: SW-SM)*	10	10
With	15 - 20	15-25
Adjective *(ex: "Silty")*	25 - <50	30 - <50

WATER LEVELS[6]

▽	WL	Water Level (WS)(WD) (WS) While Sampling (WD) While Drilling
▽	SHW	Seasonal High WT
▼	ACR	After Casing Removal
▽	SWT	Stabilized Water Table
	DCI	Dry Cave-In
	WCI	Wet Cave-In

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[1] *Classifications and symbols per ASTM D 2488-09 (Visual-Manual Procedure) unless noted otherwise.*

[2] *To be consistent with general practice, "POORLY GRADED" has been removed from GP, GP-GM, GP-GC, SP, SP-SM, SP-SC soil types on the boring logs.*

[3] *Non-ASTM designations are included in soil descriptions and symbols along with ASTM symbol [Ex: (SM-**FILL**)].*

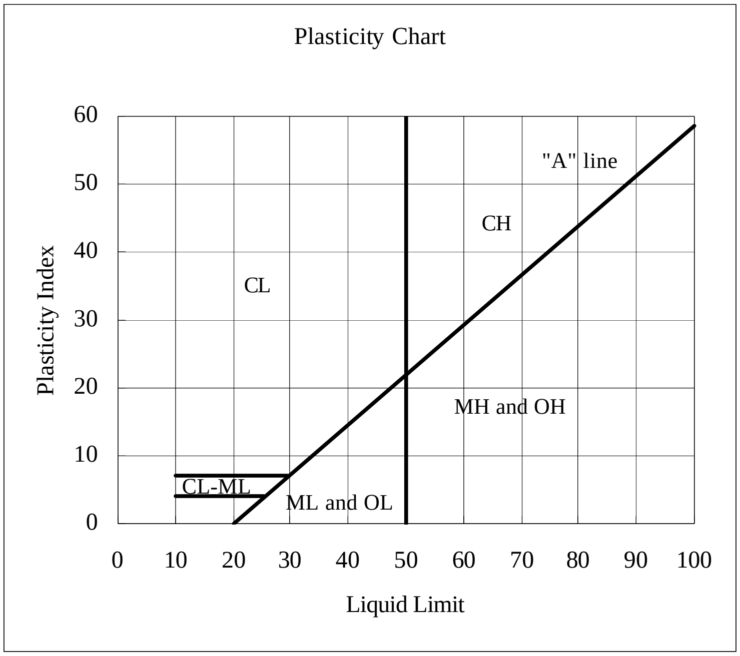
[4] *Typically estimated via pocket penetrometer or Torvane shear test and expressed in tons per square foot (tsf).*

[5] *Standard Penetration Test (SPT) refers to the number of hammer blows (blow count) of a 140 lb. hammer falling 30 inches on a 2 inch OD split spoon sampler required to drive the sampler 12 inches (ASTM D 1586). "N-value" is another term for "blow count" and is expressed in blows per foot (bpf).*

[6] *The water levels are those levels actually measured in the borehole at the times indicated by the symbol. The measurements are relatively reliable when augering, without adding fluids, in granular soils. In clay and cohesive silts, the determination of water levels may require several days for the water level to stabilize. In such cases, additional methods of measurement are generally employed.*

[7] *Minor deviation from ASTM D 2488-09.*

UNIFIED SOIL CLASSIFICATION SYSTEM (ASTM D 2487)

Major Divisions			Group Symbols		Typical Names	Laboratory Classification Criteria		
Coarse-grained soils (More than half of material is larger than No. 200 Sieve size)	Gravels (More than half of coarse fraction is larger than No. 4 sieve size)	Clean gravels (Little or no fines)	GW		Well-graded gravels, gravel-sand mixtures, little or no fines	Determine percentages of sand and gravel from grain-size curve. Depending on percentage of fines (fraction smaller than No. 200 sieve size), coarse-grained soils are classified as follows: Less than 5 percent GW, GP, SW, SP More than 12 percent GM, GC, SM, SC 5 to 12 percent Borderline cases requiring dual symbols [b]	$C_u = D_{60}/D_{10}$ greater than 4 $C_c = (D_{30})^2/(D_{10}xD_{60})$ between 1 and 3	
			GP		Poorly graded gravels, gravel-sand mixtures, little or no fines		Not meeting all gradation requirements for GW	
		Gravels with fines (Appreciable amount of fines)	GM[a]	d	Silty gravels, gravel-sand mixtures		Atterberg limits below "A" line or P.I. less than 4	Above "A" line with P.I. between 4 and 7 are borderline cases requiring use of dual symbols
				u				
			GC		Clayey gravels, gravel-sand-clay mixtures		Atterberg limits below "A" line or P.I. less than 7	
	Sands (More than half of coarse fraction is smaller than No. 4 sieve size)	Clean sands (Little or no fines)	SW		Well-graded sands, gravelly sands, little or no fines		$C_u = D_{60}/D_{10}$ greater than 6 $C_c = (D_{30})^2/(D_{10}xD_{60})$ between 1 and 3	
			SP		Poorly graded sands, gravelly sands, little or no fines		Not meeting all gradation requirements for SW	
		Sands with fines (Appreciable amount of fines)	SM[a]	d	Silty sands, sand-silt mixtures		Atterberg limits above "A" line or P.I. less than 4	Limits plotting in CL-ML zone with P.I. between 4 and 7 are borderline cases requiring use of dual symbols
				u				
			SC		Clayey sands, sand-clay mixtures		Atterberg limits above "A" line with P.I. greater than 7	
Fine-grained soils (More than half material is smaller than No. 200 Sieve)	Silts and clays (Liquid limit less than 50)		ML		Inorganic silts and very fine sands, rock flour, silty or clayey fine sands, or clayey silts with slight plasticity			
			CL		Inorganic clays of low to medium plasticity, gravelly clays, sandy clays, silty clays, lean clays			
			OL		Organic silts and organic silty clays of low plasticity			
	Silts and clays (Liquid limit greater than 50)		MH		Inorganic silts, micaceous or diatomaceous fine sandy or silty soils, elastic silts			
			CH		Inorganic clays of high plasticity, fat clays			
			OH		Organic clays of medium to high plasticity, organic silts			
	Highly Organic soils		Pt		Peat and other highly organic soils			

---

[a] Division of GM and SM groups into subdivisions of d and u are for roads and airfields only. Subdivision is based on Atterberg limits; suffix d used when L.L. is 28 or less and the P.I. is 6 or less; the suffix u used when L.L. is greater than 28.

[b] Borderline classifications, used for soils possessing characteristics of two groups, are designated by combinations of group symbols. For example: GW-GC, well-graded gravel-sand mixture with clay binder. (From Table 2.16 - Winterkorn and Fang, 1975)

APPENDIX C – Laboratory Testing

Laboratory Testing Summary
Grain Size Distributions

Laboratory Testing Summary

Sample Source	Sample Number	Depth (feet)	MC[1] (%)	Soil Type[2]	Atterberg Limits[3]			Percent Passing No. 200 Sieve[4]	Moisture - Density (Corr.)[5]		CBR Value[6]	Other
					LL	PL	PI		Maximum Density (pcf)	Optimum Moisture (%)		
B-1												
	S-1	0.00 - 1.50	20.6		44	20	24					
	S-3	4.00 - 5.50	13.5	CL	28	17	11	83.6				
B-2												
	S-2	2.00 - 3.50	14.6	CL	31	19	12	86.9				
B-3												
	S-1	0.00 - 1.50	20.5		43	19	24					
	S-4	6.00 - 7.50	11.8		33	15	18					
B-4												
	S-1	0.00 - 1.50	19.7		50	18	32					
	S-3	4.00 - 5.50	15.4	CL	31	16	15	81.1				
B-5												
	S-1	0.00 - 1.50	13.1	CH	60	19	41	66.6				
	S-2	2.00 - 3.50	12.9		27	19	8					
B-6												
	S-1	0.00 - 1.50	14.9	CH	80	21	59	75.5				
	S-2	2.00 - 3.50	10.3		28	16	12					

Notes: 1. ASTM D 2216, 2. ASTM D 2487, 3. ASTM D 4318, 4. ASTM D 1140, 5. See test reports for test method, 6. See test reports for test method

Definitions: MC: Moisture Content, Soil Type: USCS (Unified Soil Classification System), LL: Liquid Limit, PL: Plastic Limit, PI: Plasticity Index, CBR: California Bearing Ratio, OC: Organic Content (ASTM D 2974)

Project No. 17:4984
Project Name: Accu-Sharp Development
PM: Trevor Walker
PE: Michael Sorgenfrei
Printed On: Monday, July 09, 2018

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Grain Size Distributions

Boring	Depth	<#4	% Fines	% Gravel	% Sand	USCS Soil Type
B-1	4-5.5	95.1	83.6	4.9	11.5	(CL) LEAN CLAY WITH SAND
B-2	2-3.5	95.9	86.9	4.1	9.0	(CL) LEAN CLAY
B-4	4-5.5	94.8	81.1	5.2	13.7	(CL) LEAN CLAY WITH SAND
B-5	0-1.5	83.5	66.6	16.5	16.9	(CH) SANDY FAT CLAY WITH GRAVEL
B-6	0-1.5	93.0	75.5	7.0	17.5	(CH) FAT CLAY WITH SAND

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Project Number: 17:4984 | Date: July 2018