GEOTECHNICAL ENGINEERING REPORT
HACIENDA CAR WASH
RIDGE ROAD (FM 740) NORTH OF WHITE HILLS DRIVE
ROCKWALL, TEXAS

Prepared For:

Stacy Technology and Design
412 South Jefferson
Pilot Point, Texas, 76258

Attn: Mr. Mark Stacy

March 2018

PROJECT NO. 18-22539

www.roneengineers.com

March 15, 2018

Mr. Mark Stacy
Stacy Technology and Design
412 South Jefferson
Pilot Point, Texas 76258

Re: Geotechnical Engineering Report
Hacienda Car Wash
Ridge Road (FM 740) North of White Hills Drive
Rockwall, Texas
Rone Report No. 18-22539

Dear Mr. Stacy:

Rone Engineering Services, Ltd. (Rone) is pleased to submit our Final Geotechnical Engineering Report for the above referenced project. The geotechnical engineering services performed for this study were carried out in general accordance with Rone Proposal No. P-24779-18, dated February 9, 2018.

This report presents engineering analyses and recommendations for site grading, foundations and pavements with respect to available project characteristics. Results of our field exploration and laboratory testing are shown in detail in the appendix section of the study.

We appreciate the opportunity to be of service to you on this project and look forward to providing additional Geotechnical Engineering and Construction Materials Testing services as the project progresses through the detailed design and construction phases. Please contact us if you have any questions or if we can be of further assistance.

Respectfully Submitted,

STATE OF TEXAS
RICHARD L. SANDERS
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LICENSED
PROFESSIONAL ENGINEER
03.15.2018

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GEOTECHNICAL ENGINEERING REPORT

1 INTRODUCTION

The proposed project is located on Ridge Road (FM 740) north of White Hills Drive in Rockwall, Texas. The site consists of Lot 4, Block A as shown and described on the Replat of The Woods at Rockwall Addition prepared by Douphrate & Associates, Inc. dated August 30, 2017. We understand the project consists of developing a 5,000 square foot single story car wash facility, with associated paved staging and drive lane areas. The site plan referenced above shows that the facility building consists of a lobby area, drive through car wash, equipment room with a covered canopy entrance and several covered parking spaces.

A site vicinity map and geology map are attached as Plates A.1 and A.2, respectively. The general location and orientation of the site are shown on the Boring Location Diagram, Plate A.3, in Appendix A of this study.

2 PURPOSES AND SCOPE OF STUDY

The principal purposes of this study are to evaluate the general soil and rock conditions at the proposed site and to develop geotechnical recommendations for the design and construction of foundations and pavement. To accomplish its intended purposes, the study was conducted in the following phases:

- Borings were drilled and sampled to evaluate the soil and rock conditions at the boring locations and to obtain soil and rock samples.
- Laboratory tests were conducted on selected samples recovered from the borings to establish the pertinent engineering characteristics of the foundation soils and rock.
- Engineering analyses were performed using field and laboratory data, to develop foundation and pavement design recommendations.

3 FIELD OPERATIONS AND LABORATORY TESTING

The borings were located in the field by Rone Engineering personnel with measurements taken from site landmarks and using an aerial of the site. These locations were not surveyed. The provided locations are accurate only to the extent implied by the technique used in their determination.

Soil and rock conditions were evaluated by completing two borings advanced to a depth of about 25 feet below existing grade using a track mounted drill rig. Borings were drilled in March 2018. The approximate boring locations are shown on Plate A.3, Boring Location Diagram. Sample depth, description of soils, and classification (based on the Unified Soil Classification System) are presented on the Logs of Boring, Plates A.4 and A.5. Keys to terms and symbols used on the logs are shown on Plates A.6 and A.7. Presented below are the proposed structures, the borings that were drilled and their depths.

Laboratory soil tests were performed on selected samples recovered from the borings to confirm visual classification and determine the pertinent engineering properties of the soils encountered. Classification test results are presented on the Logs of Boring. Swell test were performed on selected soil samples and the results are tabulated and presented in the Appendix section report on Plate A.8.

Descriptions of the procedures used in the field and laboratory phases of this study are presented in the Appendix of this report.

4 GENERAL SITE CONDITIONS

The site has a grade change varying from about elevation 525 feet at the northeast corner to elevation 545 feet at the south side of the site. The footprint of the proposed car wash building is located within an area which the elevation change varies from about elevation 538 feet to about 545 feet (about 7 feet). The finish floor elevations have not been provided at this time. Rone should be provided with the final grading plan for review to determine if any changes need to be made to our recommendations. Building construction details, type and materials have not been provided at this time.

4.1 Site Geology

Based on the subsurface conditions encountered in the borings and the Geologic Atlas of Texas Dallas Sheet (published by the Bureau of Economic Geology), the site appears to be mapped within the Marlbrook Marl (Kmb) formation. The USGS Mineral Resources On-Line Spatial Data reference contains the following description of the Marlbrook Marl formation:

4.1.1 Marlbrook Marl Formation

The Marlbrook Marl ("upper Taylor marl") generally consists of gray clay that is often calcareous. The residual clays can be silty, especially near the surface (upper 50 feet). The clays are highly expansive and undergo large volumetric changes with climatic cycles.

Please note that the geologic mapping was originally performed using aerial photography. Local variations and anomalies do occur.

4.2 Subsurface Soil Conditions

The various strata and their approximate depths and thickness are shown on the Logs of Boring. The stratification boundaries shown on the Logs of Boring represent the approximate locations of changes in types of soil and rock; in-situ, the transition between material types may be gradual and indistinct. A brief summary of the stratigraphy indicated by the borings is given below. Materials described as limestone may, in fact, have been taken from zones of competent limestone, marl, or even dolomite. This study was not performed in an effort to provide the contractor with information guidance in evaluating the rippability or excavatability of the subsurface materials at this site, and may lead to incorrect conclusions if used for that purpose.

The encountered subsurface conditions can be generalized as firm to very hard dark gray to gray fat clay (CH) to a depth of about 2 to 4 feet followed by very hard, tan, lean clay (CL) to depths of about 4 to 6 feet. Tan Marl was encountered at depths of about 4 to 6 feet followed by gray to dark gray Marl at depths of about 14 feet to the termination depth of 25 feet.

The Plasticity Index of the samples tested varied from 28 to 38, indicating moderate to high soil plasticity. A high plasticity Index is generally associated with a high potential for the active clayey soils to shrink and swell with changes in moisture content.

The unconfined compression strength of the cohesive soil tested was 8,604 pounds per square foot (psf). The pocket penetrometer values varied from 1.5 to more than 4.5 (tsf) in the soils. The Standard Penetration Test, N values vary between 40 blows per foot (bpf) and 50 blows for 5 inches in the tan Marl. The Texas Cone Penetrometer, TCP, values vary between 100 blows for 0.75 inches to 1.5 inches in the gray to dark gray Marl.

4.3 Groundwater

The borings were advanced using flight augers to observe the potential for water seepage during and after drilling. Free water was not observed in boring B-1 during or upon completion of drilling; however, free water was observed in B-2 at a depth of about 8 feet during drilling, but was not observed at completion. The scope of work did not include long term observations of ground water or perched water conditions. In addition, it is difficult to accurately predict the magnitude of subsurface water fluctuations that might occur following periods of inclement weather. Groundwater can be encountered above any of the less permeable soil or rock at this site, creating a temporary perched water condition, particularly during wet periods of the year. Groundwater levels should be expected to fluctuate throughout the year with variations in precipitation, runoff, irrigation, site topography, utilities and the water levels in nearby surface water features and other factors not evident at the time of the field services.

These observations have been made during the course of the field exploration, as indicated on the Logs of Boring. A groundwater study has not been performed. Long-term observations would be necessary to more accurately evaluate the groundwater levels and fluctuations. If these services are desired, Rone would be pleased to provide groundwater level monitoring as an additional scope of services.

4.4 Existing Site Conditions

A cursory review of the site in previous and current Google Earth® and our field observations revealed that the site is presently a wooded and appears to have in that condition since 1996. The site is bounded on the north by a vacant wooded lot, on the south by a grass coved sloped open area that contains a Wal-Mart property outfall which appears to be a storm water detention/retention area, on the south by Big D Auto & Lube facility which contains a short concrete wall at the rear of the facility parking and a retail office building. The site is bounded on the west by Ridge Road (FM 704) followed by a vacant tract to the west.

5 ANALYSIS AND RECOMMENDATIONS

5.1 Seismic Site Class

The site class for seismic design is based on several factors that include soil profile (soil or rock), shear wave velocity, density, relative hardness, and strength, averaged over a depth of 100 feet. The borings for this project did not extend to a depth of 100 feet; therefore, we assumed the soil and rock conditions below the depth of the borings to be similar to those encountered at the termination depth of the borings. Based on Section 1613.5.2 of the 2015 International Building Code, we recommend using **Site Class C (very stiff soil / soft rock)** for seismic design.

5.2 Potential Vertical Rise

At the time of our field exploration, the soils at the site were generally found to be in a moist condition. The calculated Potential Vertical Rise (PVR), using the TxDOT method, within a 12 foot deep active zone, is currently estimated to be between 2 and 2½ inches based on the encountered soils in a dry moisture condition. Results of free swell tests are reported on Plate A.8 and range between 0.1 and 0.8 percent. Soil moisture contents do not remain constant over time. Based on the calculated PVR, we recommend that a PVR of 2½ inches be adopted for design.

5.3 Excavation Safety Considerations

Please note that in accordance with Texas State Law, the design and maintenance of excavation safety systems is the sole responsibility of the contractor. Attention is drawn to OSHA Standards 29 CFR – 1926 Subpart P, including Appendices A and B, for guidance in the design of such systems.

5.4 Foundation Recommendations

Based on the conditions encountered in our borings and anticipated loading conditions, the structural loads of the proposed single-story car wash building may be supported by a ground supported conventionally reinforced beam and slab foundation system or a post-tensioned slab foundation system, provided some floor movements can be tolerated and subgrade treatment is performed to reduce the PVR to an acceptable design value of less than 3 inches.

5.4.1 Slab Foundation

The proposed structures may be supported on ground-supported foundations consisting of a conventionally reinforced beam and slab system or a post-tensioned slab foundation system provided some floor movements can be tolerated. As discussed earlier, a PVR on the order of 2½ inches is possible at this site and subgrade improvement will be required to reduce the PVR to a tolerable level of 1 inch, or as desired by the client. The foundations should be designed with exterior and interior grade beams adequate to provide sufficient rigidity to the foundation system to sustain the vertical soil movements expected at this site. The following recommendations are based upon the site requiring not more than 2 feet of cut or fill for the desired final pad elevation. Since the building footprint has an elevation change on the order of 7 feet Rone will need the final grading plan to determine if modification to this report will be required. It is recommended that the depth of moisture conditioned soil below the slab be uniform in depth and that the building slab is not supported on cut and fill within the footprint of the building.

A net allowable soil bearing pressure of 1,500 pounds per square foot (psf) may be used for design of all grade beams bearing in moisture-conditioned soils or 2,000 psf in select fill or native soils. Grade beams should be founded a minimum of 18 inches into compacted and tested moisture conditioned fill/natural soil/select fill.

The bottom of the beam trenches should be free of any loose or soft material prior to the placement of the concrete. All grade beams and floor slabs should be adequately reinforced with steel to minimize cracking as normal movements occur in the foundation soils. Moist soil conditions should be maintained within at least 5 feet of the foundation during their service life.

The PTI parameters are calculated based on the method described in the Post-Tensioning Institute (PTI), manual 3rd edition for designing slab-on-grade foundation systems. The effective PI for a conventionally reinforced concrete slab foundation should be taken as 32. Recommended PTI parameters for foundation design for PVR values of 1, 1½ and 2 inches and a Thornthwaite Moisture Index (TMI) of 0 is as follows:

Table 1: PTI Criteria

Design PVR = 1 inches			
Edge Moisture Variation Distance		**Differential Swell**	
Center Lift	7.5 feet	**Center Lift**	0.8 inches
Edge Lift	3.4 feet	**Edge Lift**	1.2 inches

Design PVR = 1½ inches			
Edge Moisture Variation Distance		**Differential Swell**	
Center Lift	7.5 feet	**Center Lift**	1.1 inches
Edge Lift	3.4 feet	**Edge Lift**	1.4 inches

Design PVR = 2 inches			
Edge Moisture Variation Distance		**Differential Swell**	
Center Lift	7.5 feet	**Center Lift**	1.4 inches
Edge Lift	3.4 feet	**Edge Lift**	1.6 inches

A moisture barrier should be used beneath the slab foundation.

The Post Tensioning Institute (PTI) method incorporates numerous design assumptions associated with the derivation of required variables needed to determine the soil design criteria. The PTI method of predicting differential soil movement is applicable when site moisture conditions are controlled by the climate alone on well-graded building pads (i.e. no improper drainage, percolation of water in unlined landscaped areas, utility water leaks or other free water sources). Soil moisture increases within the supporting soils beneath concrete slabs, particularly when the space above the floor slab is enclosed and air-conditioned. As soil moisture increases, the soils may swell, and the PTI design method is intended to provide stiffened foundation systems that can perform well under typical natural changes in soil moisture. The resulting differential foundation movements resulting from seasonal soil moisture content changes are typically much lower than upward movements that can occur due to free water sources near or beneath the residence, which are not directly addressed by the PTI design method.

5.5 Subgrade Treatments to Reduce Soil Movement

Several options are available for use in preparing the building pad for the proposed building. Each approach has advantages and disadvantages, typically relating to risk, cost and schedule. For this site, the method recommended for subgrade treatment is moisture conditioning due to the cut and fill that will be required to achieve final pad elevation. We believe the moisture conditioning option will yield a more uniform treatment and also prove to be schedule and cost effective. Guidelines for moisture treatment are provided below. The client has requested that water injection be considered; however, we believe this option is not suitable for this site due to the dense clay and marl that will make uniform application of water difficult, and the clay and marl can be difficult to penetrate with injection rods.

When considering the various treatment options, it is important to keep in mind that the soil/rock conditions, which resulted in the calculated PVR values in the borings, performed within the footprint of the building may not be uniformly present beneath the building. Some allowance for variable support should be incorporated in the foundation designs for the structure on this site.

5.5.1 Moisture Conditioning

Subgrade treatment may consist of removal and replacement of onsite clays and reworking with moisture and density control to the depth below the final pad grade as indicated in the table presented below. Recommendations for subgrade treatment are provided below. Please note that reworking the subgrade at this site may require excavation of some relatively strong ground, including limestones and marls. Removal and reconditioning of these materials may be difficult in portions of the site. However, the preparation of a uniform fill pad beneath the building is a crucial aspect of the moisture conditioning approach to site development.

Table 2: Moisture Conditioning Depth, Feet

Building	Design "As Is" PVR, Inches	Target PVR After Treatment, Inches		
		2	1½	1
Car Wash Building	2½	4 feet*	5 feet*	6 feet*

- * Depth measured from top of finished pad, including any fill used to raise grades. In addition, it is intended that to achieve the 1 inch design PVR, the pad be excavated to top of the marl. The depth reflected in the table above is based on approximately 2 feet of fill in the building pad area.

Reworking of the existing clays is performed to increase the moisture levels of the clays to a level that reduces their ability to absorb additional water that could result in post-construction heave in these soils. The reworked clays should extend at least 5 feet outside the perimeter of the proposed structures or other perimeter features sensitive to differential movement. Some post-construction drying and settlement of the fill should be expected.

The subgrade should be excavated to the required depth below the depth measured from top of finished pad, including any fill used to raise grades. Any deleterious materials or rock fragments greater than 4 inches in diameter encountered within the soils should be removed and discarded. The subgrade to receive moisture-conditioned clay should be scarified to a depth of 6 inches, and compacted as specified in Table 4 for moisture conditioning. The clays to be used as fill can then be placed in loose lifts less than 9 inches compacted to as specified in Table 4 for moisture conditioning. Any fill required above the existing grade to reach the desired elevations should be moisture conditioned as described above 4 or select fill can be used and compacted as required in Table 4.

The treated subgrade materials are prone to drying out after the treatment process is complete. The treated subgrade materials should be kept moist prior to slab concrete placement.

Moisture conditioned clay subgrade should be monitored and tested on a full-time basis by Rone Engineers to confirm conditions are as anticipated and to confirm the fill is suitable and placed with the proper moisture content and degree of compaction. Density tests should be performed on each lift of reworked clay.

5.6 Retaining Wall Design Parameters

Structural retaining walls should be designed by the Structural Engineer to resist anticipated earth and surcharge loads. External and global stabilities of the retaining walls should be evaluated in order to ensure the stability of the walls. Retaining wall profiles were not finalized at the time of this report. To perform the stability checks, retaining wall profiles should be provided to Rone once they become available. Provisions should be included for positive drainage of the backfill and slopes in the vicinity of the walls, consistent with the design assumptions.

Gravity retaining walls should be backfilled with clean, free-draining sand or gravel material. Meeting the material specifications recommended in section **5.6.1 Drainage Material**. Granular backfill should extend at least 2 feet behind the walls and should be included in the zone that extends back 30 degrees from vertical from the base of the backfill. Granular material should be compacted as specified in section **5.6.2 Wall Backfill**. The granular material should be capped with a minimum 2-foot layer of compacted clay, sloped to drain surface water. A suitable non-woven geotextile filter fabric, such as Mirafi 160N, should be used to separate the granular material from surrounding soils.

A perimeter drain should be provided at the base of the walls to prevent hydrostatic pressure on the walls. The drain should be surrounded by a suitable geotextile filter fabric, such as Mirafi 160N, to prevent the intrusion of fines into the drain line. The drain line should be sloped to provide positive gravity drainage.

We recommend a minimum active “Coulomb” equivalent fluid earth pressure of 45 psf per foot of depth be used to design retaining walls and that are backfilled with free-draining granular material. This pressure assumes that the wall will be constructed with a back slope of 6H:1V and a ground slope no steeper than 4H:1V in front of the wall. These pressures do not include any loading due to surcharge, which must be considered separately.

The lateral earth pressures cited above are based on drained wall backfill conditions, and do not include additional lateral loads due to seepage forces or surcharge loading (such as, but not limited to, sloping backfill, structural loads, and vehicle loads). If adequate drainage is not provided, hydrostatic pressure must be added to the lateral pressure. Also, surcharge loads should be included in the wall design. Lateral wall loads due to surcharge can be computed as 0.6 times the magnitude of the vertical surcharge pressure at the surface, applied laterally and uniformly over the full height of the wall.

The foundation elements of the retaining walls must be cast directly against competent limestone to generate the passive pressures. If the retaining wall is to be supported on footing foundations, the footings could be designed using an allowable bearing pressure of 2,000 psf below a minimum embedment depth of 24 inches in competent clay or marl. This bearing pressure assumes the grade at the toe of the wall is no steeper than 4H:1V for a distance at least equal to the wall height. The footings excavations should be observed by the geotechnical engineer to confirm the bearing conditions. A frictional coefficient of 0.25 may be used for footings founded in competent clay or marl. Additional resistance can be developed by generating the passive pressures of the competent clay or marl when the foundation of a retaining wall beneath the base of the retaining wall foundation is in direct contact with undisturbed, competent clay or marl. A uniform allowable passive pressure of 300 psf can be used on the retaining wall footing foundation to resist lateral forces for competent clay or marl on the foundation key, provided the edge of the footing is at least 5 feet from a slope with a maximum declination of 4:1 (horizontal:vertical.) These pressures are based on the footing foundation bearing directly against competent clay or marl.

The bottom of the face of the footing for retaining walls should be placed a minimum of 18 inches below the final grade on the lower side of the wall, assuming that the slope at the toe of the wall is relatively flat. For footings that are situated only 12 to 18 inches below grade the allowable bearing capacity should be reduced to 1,500 psf. For walls in excess of 4 feet in height, the minimum footing depth for the back face of the foundation element should be 2 feet.

All retaining walls should have regularly spaced weep holes. If the area behind the wall will be difficult to access once construction is complete, consideration should be given to installing a slotted drain system behind the wall in addition to the weep holes. It is not uncommon for weep holes in this region to become clogged or biologically fouled over time, resulting in their becoming ineffective. The installation of a perforated pipe in areas that cannot be easily excavated in the future will provide a relatively inexpensive supplement to the weep holes and will reduce the risk of the wall becoming overstressed and damaged by unexpected seepage pressures.

5.6.1 Drainage Material

The drainage system should consist of free-draining, clean, granular fill. This material should be compatible with ASTM C33, sizes 4 through 9. The drainage layer should extend at least 24-inches from the back face of the wall. A suitable non-woven geosynthetic wrap is strongly recommended to enclose any granular backfill to reduce the infiltration of fines. The granular backfill material should be capped with a minimum of 2 feet of clay materials with a Plasticity Index of 25 or more, compacted to at least 95 percent of Standard Moisture Density Relation test (ASTM D 698), at a moisture content of at least three percentage points (+3%) above the optimum moisture content and extend a distance of 5-feet beyond the wall excavation limits and sloped to reduce the surface water infiltration into the underlying fill. If the fill will be completely covered with concrete walkways or decking, the clay cap requirement can be waived provided the pavement joints are sealed with an appropriate elastomeric sealant.

5.6.2 Wall Backfill

Free-draining backfill soils should be placed in maximum lifts of 1-foot and lightly consolidated by use of a vibrating plate or sleds, light hand-held compactors or other appropriate methods to adequately compact the backfill. If on-site clayey soils are used, these materials should be placed in maximum 6-inch lifts and properly compacted to between 90 and 93 percent of the maximum dry density, as determined by Standard Moisture Density Relation test (ASTM D 698), and at a moisture content of at least three percentage points (+3%) above the optimum moisture content.

5.6.3 Wall Construction Considerations

Heavy compactors and grading equipment should not be allowed to operate within 15-feet of the crest of the wall. This recommendation is intended to reduce the risk of developing excessive additional temporary or long-term lateral soil pressures during construction.

Wall on footing excavation may expose shallow marl, which can be difficult to excavate. The selected contractor should have experience in construction and excavation within this formation. It should be noted that this study did not include evaluating the rippability or excavatability of the subsurface materials at this site. The contractor should use his own experience in the area when forming conclusions regarding appropriate means and methods to accomplish the planned construction.

5.7 Pavement Design Recommendations

This report includes recommendations for rigid pavements. Rigid pavements tend to be more durable and require less maintenance after construction, and rehabilitation/reconstruction of the pavement section is not typically considered a part of the pavement life cycle.

5.7.1 Rigid Pavements

When designing proposed pavement sections for driveways and parking areas, subgrade conditions must be considered, along with expected traffic use/frequency, pavement type, and design period. For this project, traffic loading and frequency conditions were assumed for various conditions as no specific traffic information was provided. The following information and assumptions were used in our analysis:

1) A design life of 20 years;
2) Initial serviceability, p_o, of 4.2 and a terminal serviceability, p_t, of 2.0 for concrete pavements;
3) A k-value of 100 pci for subgrade consisting of clay soils and 150 pci for lime-treated subgrade; and
4) Reliability of 80 percent, combined standard error of 0.4, Young's modulus equal to $57{,}000\sqrt{f'_c}$, 0.75 drainage factor, and load transfer coefficient of 3.8.

The pavement thickness determinations were performed in accordance with the "1993 AASHTO Guide for the Design of Pavement Structures" guidelines[1]. The minimum pavement sections are presented in the table below. These pavement sections are estimates based on assumed traffic volumes. A more precise design can be made with detailed traffic loading information.

Table 3: Minimum Pavement Sections and Calculated Traffic

Traffic Use	Portland Cement Concrete Thickness (inches)	Calculated ESAL Count For Flexural/Compressive Strength (psi):		
		530/3,500	580/4,000	627/4,500
Parking Areas for Autos and Light Trucks	5	24,000	31,000	40,000
Drive Lanes for Autos and Light Trucks/Fire Lanes*	6	59,000	79,000	101,000
Light Semi-Truck Traffic/Dumpster Areas	7	134,000	180,000	232,000

The concrete minimum 28-day compressive strength should be selected based on the expected traffic. As a minimum, reinforcing steel should consist of #3 bars spaced at a maximum of 18 inches on center in each direction.

Lime treatment of the pavement subgrade is recommended for PCC pavements subjected to heavy truck traffic (7-inch pavement section). In small localized areas (dumpster pads, etc.), it may not be practical to perform lime treatment. In these areas, the concrete thickness may be increased by one (1) inch and lime treatment omitted. Increased periodic maintenance (i.e. sealing of cracks/joints) is critical to the long-term performance of the pavement in areas without subgrade treatment. Lime treatment will improve pavement performance for the 5- and 6-inch

1 http://www.pavementinteractive.org/1993-aashto-rigid-pavement-structural-design/

section and is recommended. Periodic maintenance (i.e. sealing of cracks and joints) should be performed to prevent water intrusion into the underlying clay subgrade. The pavement surface should be contoured such that surface water drains off and away from the pavement or into inlets. Water allowed to pond on or adjacent to pavement surfaces could saturate the subgrade soils leading to premature pavement failure.

Pavement recommendations are based on the assumed loading conditions and commonly accepted design procedures that should provide satisfactory performance for the design life of 20 years for the assumed traffic loadings. The concrete pavement should have between 4 and 6 percent entrained air. Hand-placed concrete should have a maximum slump of six inches. A sand-leveling course should not be permitted beneath pavements. All steel reinforcement, dowel spacing/diameter, and pavement joints should conform to applicable city standards.

5.8 Pavement Subgrade Preparation

All topsoil, vegetation, and any unsuitable materials should be removed. The pavement subgrade should be proofrolled with a fully loaded tandem axle dump truck or similar pneumatic-tire equipment to locate areas of loose subgrade. In areas to be cut, the proofroll should be performed after the final grade is established. In areas to be filled, the proofroll should be performed prior to placement of engineered fill and after subgrade construction is complete. Areas of loose or soft subgrade encountered in the proofroll should be removed and replaced with engineered fill, or moisture conditioned (dried or wetted, as needed) and compacted in place.

Grading and compaction of pavement subgrade should follow the recommendations in Section **6 Site Preparation and Fill Placement** section. The final grades must be such that drainage is facilitated, and access of surface water to the subgrade materials is limited.

The existing soils are plastic and can undergo some volume change when subjected to moisture variations. If the moisture contents of these upper soils decrease, they may shrink and cracks may develop. If the moisture content of these materials increases, they could swell and lose strength. Shrinkage, swelling, or strength loss could be detrimental to the proper function of the pavement. Lime treatment will provide more uniform subgrade support and improve these soil's strength characteristics. If lime treatment is used, we recommend a minimum of 8 percent lime (by dry soil weight) to a depth of 6 inches. Lime stabilization should be performed in accordance with Item 260, current Standard Specifications for Construction of Highways, Streets, and Bridges, Texas Department of Transportation (TxDOT) or applicable standards. Lime stabilizing the upper 6 inches of the subgrade soils will improve subgrade support, but will not reduce the normal seasonal shrinking and swelling of the subgrade. Therefore, some differential vertical movements of the pavements should be expected.

The amount and type of stabilization should be determined when the site is graded and the pavement subgrade exposed. This can be done by standard lime series tests.

Water can be introduced beneath the pavement through granular materials used for aggregate bases and utility line embedment, and can cause differential movement of the pavement. Care should be exercised in detailing and constructing any flexible pavement sections to limit the opportunities for water intrusion into the pavement section, and all utilities should have clay plugs substituted for granular embedment material at the edges of the pavement to reduce the risk of moisture access and possible swelling.

6 SITE PREPARATION AND FILL PLACEMENT

The following recommendations for site preparation and fill placement may contain elements that do not appear to apply to the presently known conditions at the project site. These items have been included in this appendix since our experience has been that unforeseen obstacles are encountered on some project sites, and progress can be delayed while written guidance is prepared. While we cannot cover every possible circumstance, we have attempted to address the most frequently occurring issues in this report section.

6.1 General

All grade-supported slabs should be designed to accommodate anticipated potential movements as presented in the section **5.2 Potential Vertical Rise** earlier in this study.

Every attempt should be made to limit the extreme wetting or drying of the subsurface soils because swelling and shrinkage of these soils will result. Standard construction practices of providing good surface water drainage should be used. All grading should provide positive drainage away from the paving and should prevent water from collecting near the edge of pavements and structures. Also, ditches or swales should be provided to carry the run-off water both during and after construction. Lawn areas should be watered moderately, without allowing the clay soils to become too dry or too wet. Roof runoff should be collected by gutters and downspouts, and should discharge away from the building.

Backfill for utility lines or along the perimeter beams should consist of site-excavated soil. If the backfill is too dense or too dry, it will swell and a mound will form along the trench line. If the backfill is too loose or too wet, it will settle and a depression will form along the trench line. Backfill should be compacted as recommended in the section **6 Site Preparation and Fill Placement** below.

If granular material is used for embedment in utility trenches we recommend placing a clay plug, as a replacement for the granular embedment, at the location where the city line is located, at the location where the utility enters the structure, and at other connections. The intent is to stop any free moisture from passing through the granular embedment and entering the soil beneath the structure.

Root systems from trees and shrubs can draw a substantial amount of water from the clay soils at this site, causing the clays to dry and shrink. This could cause settlement beneath grade-supported slabs such as floors, walks, and paving. Trees and large bushes should be located a distance equal to at least one-half their anticipated mature height away from grade slabs.

All excavations should be sloped, shored, or shielded in accordance with OSHA requirements.

6.2 Earthwork

6.2.1 Site Preparation

Preparation of the site, for any future construction, should include the removal and proper disposal of any obstructions that would hinder construction. These obstructions should include all abandoned structures, foundations, debris, water wells, septic tanks and loose material. It is the intent of these recommendations to provide for the removal and disposal of all obstructions not specifically provided for elsewhere by the plans and specifications.

In general, we recommend that all active utilities that would extend beneath the building, and are not intended to provide service to the structure, be rerouted around the building footprint. Any abandoned lines should be removed and disposed of properly. All abandoned utilities within the building that are not removed represent a risk to future building performance; if the lines are abandoned in place, they must be fully grouted and capped so that the pipes do not provide a ready conduit for water.

This study was not prepared for use in evaluating the rippability or excavatability of the subsurface materials at this site, or for use in estimating the number of trucks needed to haul away excavation spoils based on the expected volume of excavated materials. The contractor must use his or her own experience in the area of this site when forming conclusions regarding appropriate means and methods to accomplish the planned construction, specifically including excavation tools, excavation rates, and number of trucks. Excavations at this site are likely to expose shallow rock, which can be difficult to excavate. The selected contractor should have experience in construction and excavation within this formation.

All excavations should be sloped, shored, or shielded in accordance with OSHA requirements.

All concrete, trees, stumps, brush, abandoned structures, roots, vegetation, rubbish and any other undesirable matter should be removed and disposed of properly. It is the intent of these recommendations to provide a loose surface with no features that would tend to prevent uniform compaction by the equipment to be used.

All areas to be filled should be disced or bladed until uniform and free from large clods. Soils should be brought to the proper moisture content and compacted as indicated in Table 4, Fill Placement Criteria, below.

Table 4: Fill Placement Criteria

Item	Description	Plasticity Requirement	Compaction Standard	Density Requirement	Moisture Requirement
On-site soils	General grading	None	ASTM D698	95% to 100% of maximum dry density	Optimum moisture to 4% above optimum moisture
Imported general fill	General grading	Liquid Limit less than 60	ASTM D698	95% to 100% of maximum dry density	Optimum moisture to 4% above optimum moisture
Moisture conditioned on-site soils	Structural fill	None	ASTM D698	92% to 96% of maximum dry density	At least 4% above optimum moisture
Select fill (soils)	Structural fill	5≤ PI ≤ 15; LL ≤ 35	ASTM D698	95% to 100% of maximum dry density	Minus 2% to Plus 2% of optimum moisture
Select fill (crushed rock or concrete)	Structural fill	Per TxDOT Item 247, Type A, C, or D Grade 2 or 3	ASTM D1557	95% to 100% of maximum dry density	Minus 2% to Plus 2% of optimum moisture
Select fill (crushed rock or concrete)	Structural fill – fill body cap (over soil fill body)	Per TxDOT Item 247, Type A, C, or D Grade 2 or 3	ASTM D698	95% to 100% of maximum dry density	Minus 2% to Plus 2% of optimum moisture
Lime Treated subgrade	Pavement support	See report text.	ASTM D698	95% to 100% of maximum dry density	Minus 2% to Plus 2% of optimum moisture

6.2.2 Select Fill

Select fill should consist of a clean, natural soil meeting the criteria listed in Table 4. The fill should have a moisture content within the specified range, be placed in loose lifts less than 9 inches thick, and compacted as indicated in Table 4. Lime-treated, on-site soils may also be used as the select fill cap, provided the PI of the material meets the specifications for select fill. The quantity of lime needed to achieve the PI requirement for select fill is not known. The actual percentage of lime should be determined once soils have been stockpiled and sampled.

Crushed recycled concrete can also be used as select fill. The crushed recycled concrete should meet the criteria listed in Table 4. The material should have a moisture content within the specified range, be placed in loose lifts less than 6 inches thick, and compacted as indicated in Table 4.

The fill material should be placed in level, uniform layers, which, when compacted, should have a moisture content and density conforming to the stipulations called for herein. Each layer should be thoroughly mixed during spreading to provide uniformity of the layer. The fill thickness should not exceed 6-inch loose lifts.

6.2.3 Site Grading

Site grading operations, where required, should be performed in accordance with the recommendations provided in this report. The site grading plans and construction should strive to achieve positive drainage around all sides of the proposed building. Inadequate drainage around structures built on-grade will cause excessive vertical differential movements to occur.

6.2.4 Utility Backfill

If on-site clayey soils are used as backfill, these materials should be placed in maximum 6-inch lifts and properly compacted to between 95 and 100 percent of the maximum dry density, as determined by Standard Moisture Density Relation test (ASTM D 698), and at a moisture content of at least two percentage points (min +2%) above the soils optimum moisture content. In instances where utility lines are more than 10 feet deep, the backfill below 10 feet should be compacted to 100 percent of the maximum dry density, as determined by Standard Moisture Density Relation test (ASTM D 698), and at a moisture content of within two percentage points (-2 to +2%) of the soils optimum moisture content.

It is typical for fills over 10 foot in depth to experience settlement. On an average, fills over 10 feet will experience between 1 and 2 percent settlement. This should be considered when designing utility lines beneath pavements, flatwork or any structure.

6.2.5 Density Tests

Field density tests should be made by the geotechnical engineer or his representative. Density tests should be taken in each layer of the compacted material below the disturbed surface. If the materials fail to meet the density specified, the course should be reworked as necessary to obtain the specified moisture content and compaction.

The specified moisture content and compaction must be maintained until placement of the overlying lift, or construction of overlying flatwork. Failure to maintain the moisture content and compaction could result in excessive soil movement and can have a detrimental effect on overlying structure such as shallow foundations and floor slabs. The contractor must provide some means of controlling the moisture content and compaction (such as water hoses, water trucks, etc.). Maintaining subgrade moisture and compaction is always critical, but will require extra effort during warm, windy and/or sunny conditions. Density and moisture testing is recommended to provide some indication that adequate earthwork is being provided. However, the quality of the fill is the sole responsibility of the contractor. Satisfactory testing is not a guarantee of the quality of the contractor's earthwork operations.

6.2.6 Tree Removal

Special attention should be given to any areas where trees were once present or will be removed from the residential pad/foundation slab area. It is possible that dry soil could be contained within the root zones of the trees that will be removed during the development of the site. When the dry soil zones become wet, they could swell and experience greater swell movement than that estimated in this report. Therefore, the existing roots and soil within the root zone of these trees should be entirely excavated from beneath the footprint of the slab foundation and replaced with similar but adequately moisture-conditioned, compacted soil as described in Table 6, **Fill Placement Criteria**.

6.3 Construction Observations

In any geotechnical investigation, the design recommendations are based on a limited amount of information about the subsurface conditions. In the analysis, the geotechnical engineer must assume the subsurface conditions are similar to the conditions encountered in the borings. However, during construction quite often anomalies in the subsurface conditions are revealed. The potential for the presence of varied geologic formations and significantly different support conditions at this site, which could result in changes in our design recommendations, increases the risk of damaging soil movements at this site. Therefore, it is recommended that Rone Engineering Services, Ltd. be retained to observe earthwork and foundation installation and perform materials evaluation and testing during the construction phase of the project. This enables the geotechnical engineer to stay abreast of the project and to be readily available to evaluate unanticipated conditions, to conduct additional tests if required and, when necessary, to recommend alternative solutions to unanticipated conditions. Until these construction phase services are performed by the project geotechnical engineer, the recommendations contained in this report on such items as final foundation bearing elevations, final depth of undercut of expansive soils for non-expansive earth fill pads, and other such subsurface-related recommendations should be considered as preliminary.

It is proposed that construction phase observation and materials testing commence by the project geotechnical engineer at the outset of the project. Experience has shown that the most suitable method for procuring these services is for the owner to contract directly with the project geotechnical engineer. This results in a clear, direct line of communication between the owner and the owner's design engineers, and the geotechnical engineer.

7 STUDY CLOSURE

The analyses, conclusions and recommendations contained in this study are based on site conditions as they existed at the time of the field exploration and further on the assumption that the exploratory borings are representative of the subsurface conditions throughout the site; that is, the subsurface conditions everywhere are not significantly different from those disclosed by the borings at the time they were completed. If during construction, different subsurface conditions from those encountered in our borings are observed, or appear to be present in excavations, we must be advised promptly so that we can review these conditions and reconsider our recommendations where necessary. If there is a substantial lapse of time between submission of this study and the start of the work at the site, if conditions have changed due either to natural causes or to construction operations at or adjacent to the site, or if structure locations, structural loads or finish grades are changed, we urge that we be promptly informed and retained to review our study to determine the applicability of the conclusions and recommendations, considering the changed conditions and/or time lapse.

Further, it is urged that Rone Engineering Services, Ltd. be retained to review those portions of the plans and specifications for this particular project that pertain to earthwork and foundations as a means to determine whether the plans and specifications are consistent with the recommendations contained in this study. In addition, we are available to observe construction, particularly the compaction of structural fill, or backfill and the construction of foundations as recommended in the study, and such other field observations as might be necessary.

This study has been prepared for the exclusive use of CLIENT and their designated agents for specific application to design of this project. We have used that degree of care and skill ordinarily exercised under similar conditions by reputable members of our profession practicing in the same or similar locality. No warranty, expressed or implied, is made or intended.

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APPENDIX A

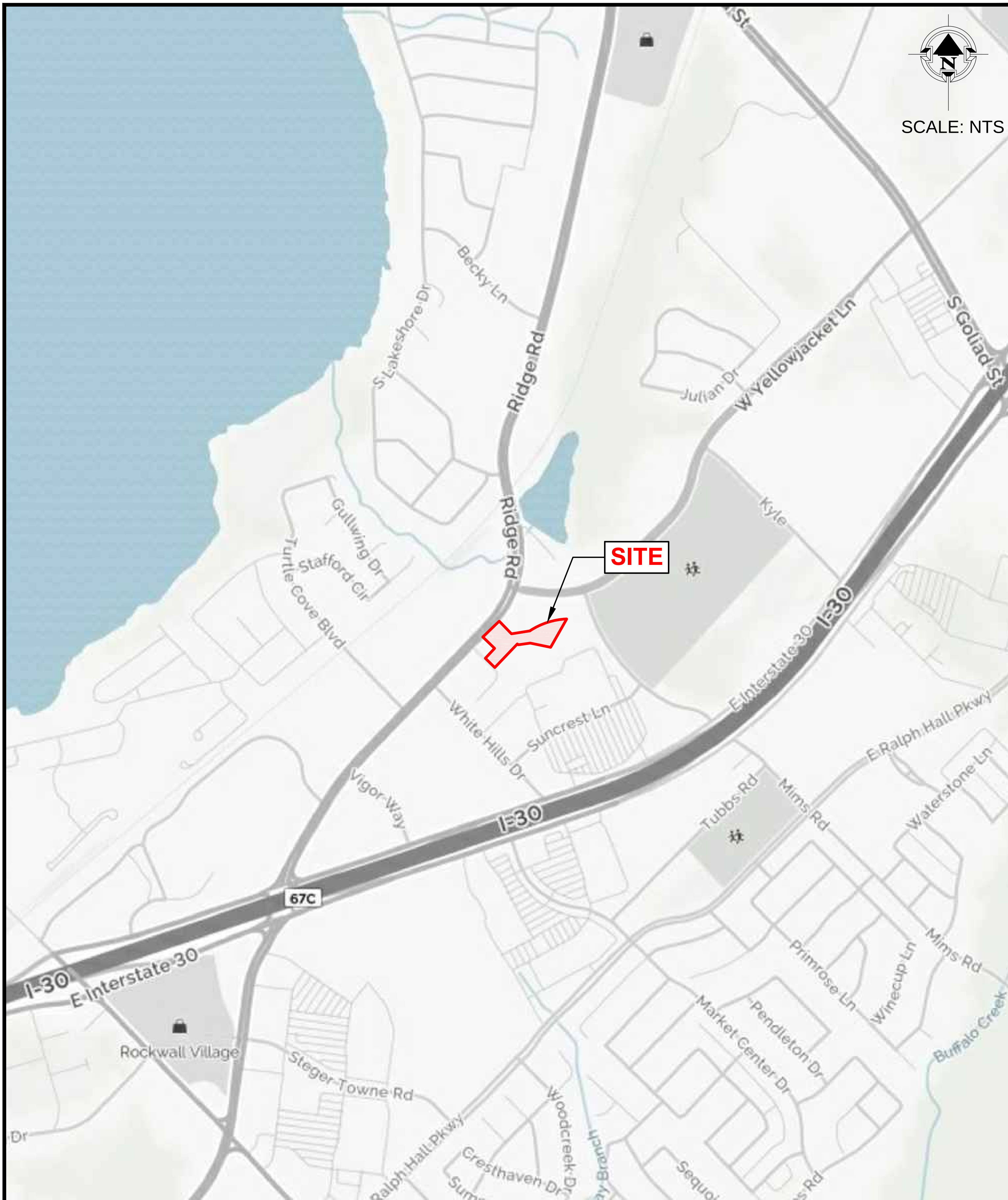

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PLATE A.1

VICINITY MAP

HACIENDA CAR WASH
RIDGE ROAD (FM 740) NORTH OF WHITE HILLS DRIVE
ROCKWALL, TEXAS

PROJECT NO:	18-22539		
FILE NAME:	1822539.DWG		
DRAWN BY:	CM	DATE:	03-09-2018
REVISED BY:		DATE:	
REVISED BY:		DATE:	
APPROVED BY:	RS	DATE:	03-09-2018

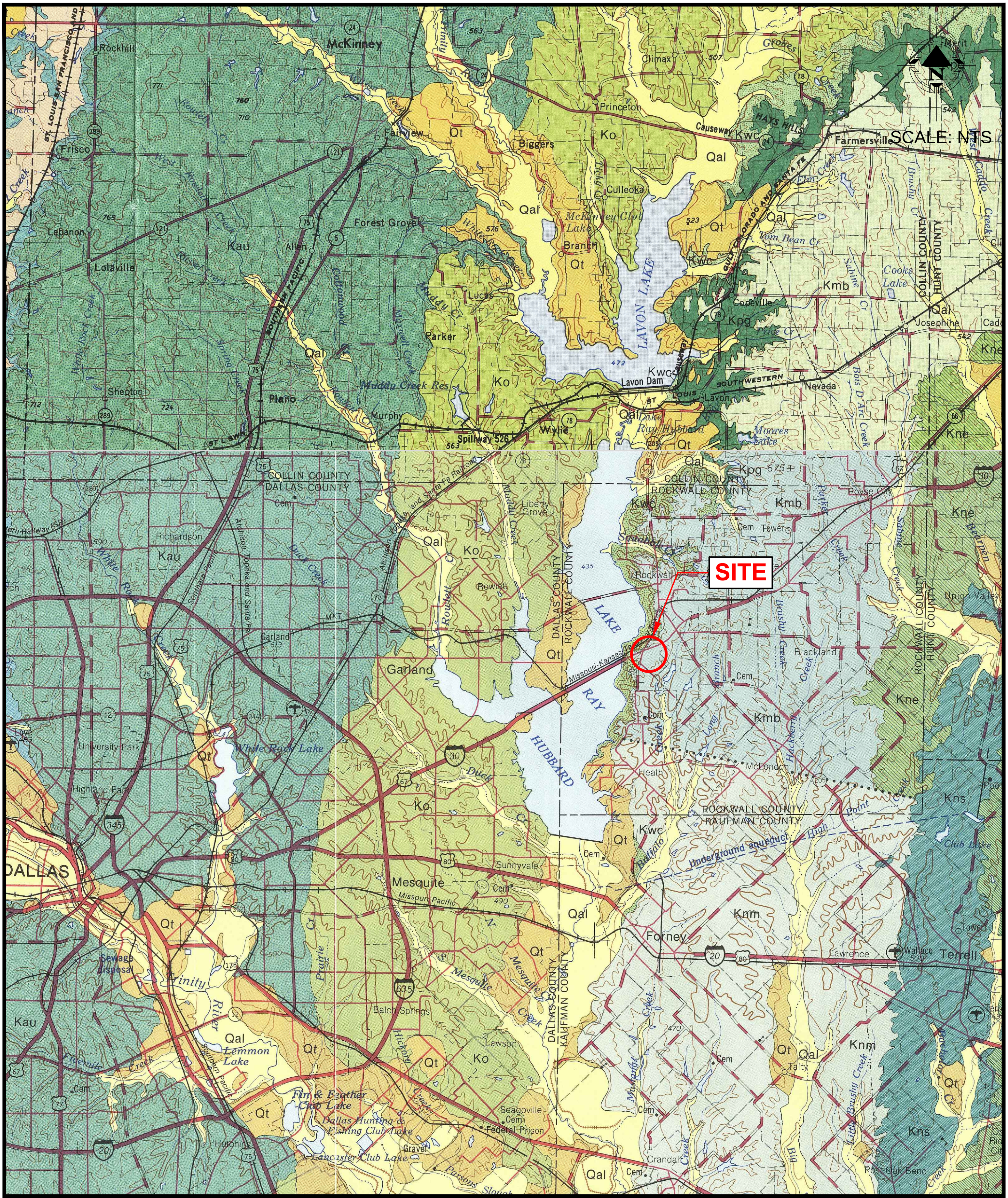

	PLATE A.2
	GEOLOGY MAP
	HACIENDA CAR WASH
	RIDGE ROAD (FM 740) NORTH OF WHITE HILLS DRIVE
	ROCKWALL, TEXAS

RONE ENGINEERING

PROJECT NO:	18-22539		
FILE NAME:	1822539.DWG		
DRAWN BY:	CM	DATE:	03-09-2018
REVISED BY:		DATE:	
REVISED BY:		DATE:	
APPROVED BY:	RS	DATE:	03-09-2018

RONE ENGINEERING

PLATE A.3

BORING LOCATION DIAGRAM

HACIENDA CAR WASH
RIDGE ROAD (FM 740) NORTH OF WHITE HILLS DRIVE
ROCKWALL, TEXAS

PROJECT NO:	18-22539		
FILE NAME:	1822539.DWG		
DRAWN BY:	CM	DATE:	03-09-2018
REVISED BY:		DATE:	
REVISED BY:		DATE:	
APPROVED BY:	RS	DATE:	03-09-2018

Project No.	Boring No.	
18-22539	B- 1	Hacienda Car Wash Rockwall, Texas

Location

Water Observations (feet)	
While Drilling	Not Encountered
At Boring Completion	Not Encountered
End of Day After Boring Completion	Not Measured

Completion Depth 25.0' | Completion Date 3-6-18

Surface Elevation 539 | Type CFA

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Stratum Description

Depth, Ft.	Elev.	Stratum Description	REC %	Penetrometer Reading, TSF	SPT - Blows/Foot TCP - Blows/Inch	Passing No. 200 Sieve, %	Liquid Limit, %	Plastic Limit, %	Plasticity Index	Moisture Content, %	Dry Unit Weight pcf	Unconfined Compression psf
0		**FAT CLAY (CH)** - firm, gray, with trace of roots		1.50						30		
2	536.5	**LEAN CLAY (CL)** - very hard, tan		4.5+		97	49	19	30	18		
4	534.5	**MARL** - tan		4.5+						17	117	8604
6.5					N=41	96	49	18	31	16		
8.5					N=50/5"					15		
14		- gray to dark gray below 14 feet			100/1.5"					13		
19					100/1.5"					12		
24					100/1.5"					12		
25	513.5	Boring Terminated at 25 feet.										

LOG OF BORING NO. **B- 1**

Plate A.4

Project No.	Boring No.		
18-22539	B- 2	**Hacienda Car Wash** **Rockwall, Texas**	
Location		Water Observations (feet)	
		While Drilling	8
Completion Depth 25.0'	Completion Date 3-6-18	At Boring Completion	Not Encountered
		End of Day After Boring Completion	Not Measured
Surface Elevation 542	Type CFA		

RONE ENGINEERING

Depth, Ft.	Elevation	Stratum Description	REC %	Penetrometer Reading, TSF	SPT - Blows/Foot TCP - Blows/Inch	Passing No. 200 Sieve, %	Liquid Limit, %	Plastic Limit, %	Plasticity Index	Moisture Content, %	Dry Unit Weight pcf	Unconfined Compression psf
		FAT CLAY (CH) - hard to very hard, dark gray to gray, with trace of roots		2.00						31		
				2.00		98	57	19	38	27		
	538.0	**LEAN CLAY (CL)** - very hard, tan, calcareous, with traces of gyosum		4.5+						19		
5	536.0	**MARL** - tan		4.5+		97	47	19	28	17		
		▽										
10					N=74/11"					16		
15		- gray to dark gray below 14 feet			100/1.5"					12		
20					100/1"					12		
25	517.0				100/0.75"					12		
		Boring Terminated at 25 feet.										

RONE ENGINEERING

SOIL OR ROCK TYPES

CLAY	SAND-*WELL GRADED*
FAT CLAY	LIMESTONE-WEATHERED
LEAN CLAY	CONCRETE
SANDY CLAY	FILL
LIMESTONE	GRAVEL
CLAYEY SAND	CLAYEY GRAVEL
SHALE	MARL
SAND-*POORLY GRADED*	SILT

Shelby Tube | Auger | Split Spoon

Rock Core | Cone Pen | No Recovery

TERMS DESCRIBING CONSISTENCY, CONDITION, AND STRUCTURE OF SOIL

Fine Grained Soils (More than 50% Passing No. 200 Sieve)

Consistency	Penetrometer Reading, (tsf)	Unconfined Compression, (psf)
Very Soft	$\leq$ 0.5	$\leq$ 1000
Soft	0.5 to 1.0	1000 to 2000
Firm	1.0 to 2.0	2000 to 4000
Hard	2.0 to 4.0	4000 to 8000
Very Hard	> 4.0	> 8000

Coarse Grained Soils (More than 50% Retained on No. 200 Sieve)

Penetration Resistance (Blows / Foot)	Descriptive Item	Relative Density
0 to 4	Very Loose	0 to 20%
4 to 10	Loose	20 to 40%
10 to 30	Medium Dense	40 to 70%
30 to 50	Dense	70 to 90%
Over 50	Very Dense	90 to 100%

Soil Structure

Calcareous	Contains appreciable deposits of calcium carbonate; generally nodular
Slickensided	Having inclined planes of weakness that ate slick and glossy in appearance
Laminated	Composed of thin layers of varying color or texture
Fissured	Containing cracks, sometimes filled with fine sand or silt
Interbedded	Composed of alternated layers of different soil types, usually in approximately equal proportions

TERMS DESCRIBING PHYSICAL PROPERTIES OF ROCK

Hardness and Degree of Cementation

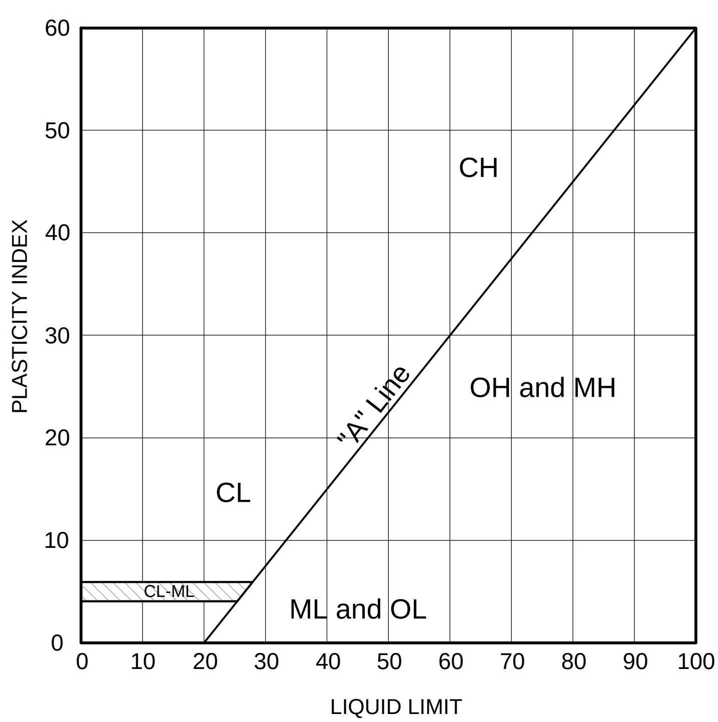
Very Soft or Plastic	Can be remolded in hand; corresponds in consistency up to hard in soils
Soft	Can be scratched with fingernail
Moderately Hard	Can be scratched easily with knife; cannot be scratched with fingernail
Hard	Difficult to scratch with knife
Very Hard	Cannot be scratched with knife
Poorly Cemented or Friable	Easily crumbled
Cemented	Bound together by chemically precipitated material; Quartz, calcite, dolomite, siderite, and iron oxide are common cementing materials.

Degree of Weathering

Unweathered	Rock in its natural state before being exposed to atmospheric agents
Slightly Weathered	Noted predominantly by color change with no disintegrated zones
Weathered	Complete color change with zones of slightly decomposed rock
Extremely Weathered	Complete color change with consistency, texture, and general appearance approaching soil

KEY TO CLASSIFICATION AND SYMBOLS **PLATE A.6**

Major Divisions			Grp. Sym.	Typical Names	Laboratory Classification Criteria		
Coarse - Grained Soils (more than half of the 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 form 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	$C_u = \frac{D_{60}}{D_{10}}$ greater than 4: $C_c = \frac{(D_{30})^2}{D_{10} \times D_{60}}$ 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	Silty gravels, gravel - sand - silt mixtures		Liquid and Plastic limits below "A" line or P.I. greater than 4	Liquid and plastic limits plotting in hatched zone between 4 and 7 are borderline cases requiring use of dual symbols
			GC	Clayey gravels, gravel - sand - clay mixtures		Liquid and Plastic limits above "A" line with P.I. greater 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		$C_u = \frac{D_{60}}{D_{10}}$ greater than 6: $C_c = \frac{(D_{30})^2}{D_{10} \times D_{60}}$ 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	Silty sands, sand silt mixtures		Liquid and Plastic limits below "A" line or P.I. less than 4	Liquid and plastic limits plotting between 4 and 7 are borderline cases requiring use of dual symbols
			SC	Clayey sands, sand clay mixtures		Liquid and Plastic limits above "A" line with P.I. greater than 7	
Fine - Grained Soils (more than half of the 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, and 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			



PLASTICITY CHART

UNIFIED SOIL CLASSIFICATION SYSTEM

PLATE A.7

SWELL TEST RESULTS

Geotechnical Engineering Report
Hacienda Car Wash
Rockwall, Texas
Rone Project Number: 18-22539

Boring	Depth (ft)	Liquid Limit	Plastic Limit	Plasticity Index	Initial MC (%)	Final MC (%)	Load (psf)	Swell (%)
B- 1	2-4	49	19	30	19	20	375	0.2
B- 2	2-4	57	19	38	24	25	375	0.1
B- 2	6-8	47	19	28	17	18	875	0.8

APPENDIX B

FIELD EXPLORATION

Subsurface conditions were defined by two sample borings located as shown on the Boring Location Diagram, Plate A.3. The borings were completed at locations staked in field by Rone personnel. The borings were advanced between sample intervals using continuous flight auger drilling procedures with a track mounted drill rig. The results of each boring are shown graphically on the Logs of Boring. Sample depth, description, and soil classification based on the Unified Soil Classification System are shown on the Logs of Borings. Keys to the symbols and terms used on the Logs of Borings are presented in the appendix section of the report.

Relatively undisturbed samples of cohesive soils were obtained with nominal 3-inch diameter tube samplers at the locations shown on the logs of borings. The tube sampler consists of a steel tube with a sharp cutting edge connected to a head equipped with a ball valve threaded for rod connection. The tube is pushed into the soil by the hydraulic pulldown of the drilling rig. The soil specimens were extruded from the tube in the field, logged, tested for consistency with a hand penetrometer, sealed and packaged to limit loss of moisture.

The consistency of cohesive soil samples was evaluated in the field using a calibrated hand penetrometer. In this test a 0.25-inch diameter piston is pushed into the undisturbed sample at a constant rate to a depth of 0.25-inch. The results of these tests are tabulated at respective sample depths on the logs. When the capacity of the penetrometer is exceeded, the value is tabulated as 4.5+.

Samples were obtained using split-barrel sampling procedures in general accordance with ASTM D1586. In the split-barrel procedure, a disturbed sample is obtained in a standard 2 inch OD split barrel-sampling spoon driven into 18 inches into the ground using a 140-pound hammer falling freely 30 inches. The number of blows for the last 12 inches of a standard 18-inch penetration is recorded as the Standard Penetration Test resistance (N-value). The N-values are recorded on the boring logs at the depth of sampling. The samples were sealed and returned to our laboratory for further examination and testing.

The encountered rock and rock-like materials were evaluated with a modified version of the Texas Cone Penetration test. Texas Department of Transportation (TX-DOT) Test Method Tex-132-E specifies driving a 3-inch diameter cone with a 170-pound hammer freely falling 24 inches. This results in 340 foot-pounds of energy for each blow. This method was modified by utilizing a 140-pound hammer freely falling 30 inches. This results in 350 foot-pounds of energy for each hammer blow. In relatively soft materials, the penetrometer cone is driven 1 foot and the number of blows required for each 6-inch penetration is tabulated at respected test depths, as blows per 6 inches on the log. In hard materials (rock or rock-like), the penetrometer cone is driven with the resulting penetrations, in inches, recorded for the first and second 50 blows, a total of 100 blows. The penetration for the total 100 blows is recorded at the respective testing depths on the boring logs.

Groundwater observations during and after completion of the boring are shown on the upper right of the boring log. Upon completion of the boring, the boreholes were backfilled with auger cuttings to ground level.

LABORATORY TESTING

General

Laboratory tests were performed on selected samples, retrieved from the borings, to evaluate the engineering characteristics of the subsurface materials encountered and to provide data for developing engineering design parameters. The subsurface materials recovered during the field exploration were described by an engineering geologist or senior staff member in the field and/or the laboratory and were later refined based on results of the laboratory tests performed.

Classification Tests

Visual classification of soils was verified by natural moisture content determinations, Atterberg limits determinations, and gradation tests (percent passing the No. 200 U.S. Standard Sieve). These tests were performed in general accordance with American Society for Testing and Materials (ASTM) procedures as follows:

All recovered soil samples were classified and described, in part, using the Unified Soil Classification System (USCS). To determine soil characteristics and to aid in classifying the soils, index property and classification testing was performed on selected samples of the soils.

Testing was performed in general accordance with the following ASTM standards, as applicable.

Atterberg Limits	ASTM D 4318
Percentage of Particles Passing the No. 200 Sieve	ASTM D 1140
Moisture Content	ASTM D 2216
Dry Unit Weight	ASTM D 2167
Unconfined Compressive Strength	ASTM D 2166
Free Swell Test	ASTM D 4546 Method B

Free Swell Tests

Selected samples of the near-surface cohesive soils were subjected to free swell tests. In the free swell test, a sample is placed in a consolidometer and subjected to the estimated overburden pressure. The sample is then inundated with water and allowed to swell. Moisture contents are determined both before and after completion of the test. Test results are recorded as the percent swell, with initial and final moisture content. Free swell test results are presented on Plate A.8.

Unconfined Compression Strength Tests - Soil

Unconfined compression tests were performed on selected samples of cohesive soils. In the unconfined compression test, a cylindrical specimen is subjected to axial load at a constant rate of strain until failure occurs. Test procedures were in general accordance with ASTM D 2166. Strengths determined by this test are tabulated at their respective sample depths on the logs of borings. Results of natural moisture content and dry unit weight determinations are also tabulated at the respective sample depths on the logs.

APPENDIX C

Important Information about This

Geotechnical-Engineering Report

Subsurface problems are a principal cause of construction delays, cost overruns, claims, and disputes.

While you cannot eliminate all such risks, you can manage them. The following information is provided to help.

The Geoprofessional Business Association (GBA) has prepared this advisory to help you – assumedly a client representative – interpret and apply this geotechnical-engineering report as effectively as possible. In that way, clients can benefit from a lowered exposure to the subsurface problems that, for decades, have been a principal cause of construction delays, cost overruns, claims, and disputes. If you have questions or want more information about any of the issues discussed below, contact your GBA-member geotechnical engineer. Active involvement in the Geoprofessional Business Association exposes geotechnical engineers to a wide array of risk-confrontation techniques that can be of genuine benefit for everyone involved with a construction project.

Geotechnical-Engineering Services Are Performed for Specific Purposes, Persons, and Projects

Geotechnical engineers structure their services to meet the specific needs of their clients. A geotechnical-engineering study conducted for a given civil engineer will not likely meet the needs of a civil-works constructor or even a different civil engineer. Because each geotechnical-engineering study is unique, each geotechnical-engineering report is unique, prepared *solely* for the client. *Those who rely on a geotechnical-engineering report prepared for a different client can be seriously misled.* No one except authorized client representatives should rely on this geotechnical-engineering report without first conferring with the geotechnical engineer who prepared it. *And no one – not even you – should apply this report for any purpose or project except the one originally contemplated.*

Read this Report in Full

Costly problems have occurred because those relying on a geotechnical-engineering report did not read it *in its entirety*. Do not rely on an executive summary. Do not read selected elements only. *Read this report in full.*

You Need to Inform Your Geotechnical Engineer about Change

Your geotechnical engineer considered unique, project-specific factors when designing the study behind this report and developing the confirmation-dependent recommendations the report conveys. A few typical factors include:

- the client's goals, objectives, budget, schedule, and risk-management preferences;
- the general nature of the structure involved, its size, configuration, and performance criteria;
- the structure's location and orientation on the site; and
- other planned or existing site improvements, such as retaining walls, access roads, parking lots, and underground utilities.

Typical changes that could erode the reliability of this report include those that affect:

- the site's size or shape;
- the function of the proposed structure, as when it's changed from a parking garage to an office building, or from a light-industrial plant to a refrigerated warehouse;
- the elevation, configuration, location, orientation, or weight of the proposed structure;
- the composition of the design team; or
- project ownership.

As a general rule, *always* inform your geotechnical engineer of project changes – even minor ones – and request an assessment of their impact. *The geotechnical engineer who prepared this report cannot accept responsibility or liability for problems that arise because the geotechnical engineer was not informed about developments the engineer otherwise would have considered.*

This Report May Not Be Reliable

Do not rely on this report if your geotechnical engineer prepared it:

- for a different client;
- for a different project;
- for a different site (that may or may not include all or a portion of the original site); or
- before important events occurred at the site or adjacent to it; e.g., man-made events like construction or environmental remediation, or natural events like floods, droughts, earthquakes, or groundwater fluctuations.

Note, too, that it could be unwise to rely on a geotechnical-engineering report whose reliability may have been affected by the passage of time, because of factors like changed subsurface conditions; new or modified codes, standards, or regulations; or new techniques or tools. *If your geotechnical engineer has not indicated an "apply-by" date on the report, ask what it should be*, and, in general, *if you are the least bit uncertain* about the continued reliability of this report, contact your geotechnical engineer before applying it. A minor amount of additional testing or analysis – if any is required at all – could prevent major problems.

Most of the "Findings" Related in This Report Are Professional Opinions

Before construction begins, geotechnical engineers explore a site's subsurface through various sampling and testing procedures. *Geotechnical engineers can observe actual subsurface conditions only at those specific locations where sampling and testing were performed.* The data derived from that sampling and testing were reviewed by your geotechnical engineer, who then applied professional judgment to form opinions about subsurface conditions throughout the site. Actual sitewide-subsurface conditions may differ – maybe significantly – from those indicated in this report. Confront that risk by retaining your geotechnical engineer to serve on the design team from project start to project finish, so the individual can provide informed guidance quickly, whenever needed.

This Report's Recommendations Are Confirmation-Dependent

The recommendations included in this report – including any options or alternatives – are confirmation-dependent. In other words, *they are not final*, because the geotechnical engineer who developed them relied heavily on judgment and opinion to do so. Your geotechnical engineer can finalize the recommendations *only after observing actual subsurface conditions* revealed during construction. If through observation your geotechnical engineer confirms that the conditions assumed to exist actually do exist, the recommendations can be relied upon, assuming no other changes have occurred. *The geotechnical engineer who prepared this report cannot assume responsibility or liability for confirmation-dependent recommendations if you fail to retain that engineer to perform construction observation.*

This Report Could Be Misinterpreted

Other design professionals' misinterpretation of geotechnical-engineering reports has resulted in costly problems. Confront that risk by having your geotechnical engineer serve as a full-time member of the design team, to:

- confer with other design-team members,
- help develop specifications,
- review pertinent elements of other design professionals' plans and specifications, and
- be on hand quickly whenever geotechnical-engineering guidance is needed.

You should also confront the risk of constructors misinterpreting this report. Do so by retaining your geotechnical engineer to participate in prebid and preconstruction conferences and to perform construction observation.

Give Constructors a Complete Report and Guidance

Some owners and design professionals mistakenly believe they can shift unanticipated-subsurface-conditions liability to constructors by limiting the information they provide for bid preparation. To help prevent the costly, contentious problems this practice has caused, include the complete geotechnical-engineering report, along with any attachments or appendices, with your contract documents, *but be certain to note conspicuously that you've included the material for informational purposes only*. To avoid misunderstanding, you may also want to note that "informational purposes" means constructors have no right to rely on the interpretations, opinions, conclusions, or recommendations in the report, but they may rely on the factual data relative to the specific times, locations, and depths/elevations referenced. Be certain that constructors know they may learn about specific project requirements, including options selected from the report, *only* from the design drawings and specifications. Remind constructors that they may perform their own studies if they want to, and *be sure to allow enough time* to permit them to do so. Only then might you be in a position to give constructors the information available to you, while requiring them to at least share some of the financial responsibilities stemming from unanticipated conditions. Conducting prebid and preconstruction conferences can also be valuable in this respect.

Read Responsibility Provisions Closely

Some client representatives, design professionals, and constructors do not realize that geotechnical engineering is far less exact than other engineering disciplines. That lack of understanding has nurtured unrealistic expectations that have resulted in disappointments, delays, cost overruns, claims, and disputes. To confront that risk, geotechnical engineers commonly include explanatory provisions in their reports. Sometimes labeled "limitations," many of these provisions indicate where geotechnical engineers' responsibilities begin and end, to help others recognize their own responsibilities and risks. *Read these provisions closely*. Ask questions. Your geotechnical engineer should respond fully and frankly.

Geoenvironmental Concerns Are Not Covered

The personnel, equipment, and techniques used to perform an environmental study – e.g., a "phase-one" or "phase-two" environmental site assessment – differ significantly from those used to perform a geotechnical-engineering study. For that reason, a geotechnical-engineering report does not usually relate any environmental findings, conclusions, or recommendations; e.g., about the likelihood of encountering underground storage tanks or regulated contaminants. *Unanticipated subsurface environmental problems have led to project failures*. If you have not yet obtained your own environmental information, ask your geotechnical consultant for risk-management guidance. As a general rule, *do not rely on an environmental report prepared for a different client, site, or project, or that is more than six months old.*

Obtain Professional Assistance to Deal with Moisture Infiltration and Mold

While your geotechnical engineer may have addressed groundwater, water infiltration, or similar issues in this report, none of the engineer's services were designed, conducted, or intended to prevent uncontrolled migration of moisture – including water vapor – from the soil through building slabs and walls and into the building interior, where it can cause mold growth and material-performance deficiencies. Accordingly, ***proper implementation of the geotechnical engineer's recommendations will not of itself be sufficient to prevent moisture infiltration.*** Confront the risk of moisture infiltration by including building-envelope or mold specialists on the design team. ***Geotechnical engineers are not building-envelope or mold specialists.***

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