GEOTECHNICAL ENGINEERING REPORT

CYPRESS ROSEHILL BUSINESS PARK

CYPRESS, TEXAS

GEOTECHNICAL ENGINEERING REPORT

Cypress Rosehill Business Park

Cypress, Texas

Prepared by:

Riner Engineering, Inc., a UES Company

Prepared for:

Project Montana, LLC
15903 San Saba Canyon Circle
Cypress, Texas 77429

Attention: Mr. Kevin Smith, Jr.

June 26, 2023

RINER Project No. 23-0242

TEXAS ENGINEERING FIRM REGISTRATION NO. F-17076

June 26, 2023

Mr. Kevin Smith, Jr.
Project Montana, LLC
15903 San Aba Canyon Circle
Cypress, Texas 77429

Re: GEOTECHNICAL ENGINEERING REPORT
Cypress Rosehill Business Park
Cypress, Texas
RINER Project No. 23-0242

Dear Mr. Smith:

Riner Engineering, Inc. (RINER) is pleased to submit this Geotechnical Engineering Report for the referenced project. We appreciate the opportunity of working with you. Please contact us if you have any questions or require additional services.

Respectfully submitted,

Andres Mexquitic Jr.
Project Geologist

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Engineering Manager

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GEOTECHNICAL ENGINEERING REPORT

Cypress Rosehill Business Park
Cypress, Texas

1.0 INTRODUCTION

Project Location. The project is located at 16602 Cypress Rosehill Road in Cypress, Texas. The general location and orientation of the site are provided in Appendix A - Project Location Diagrams.

Project Description. The project consists of proposed two to three office/warehouse buildings (footprint totaling 6,000 - 12,000 square foot (SF), each), a detention pond, parking ,drive areas, and a turn lane along Cypress Rosehill Road.

Project Authorization. This geotechnical study was authorized by Mr. Kevin Smith, Jr. with Project Montana, LLC and performed in accordance with RINER Proposal No. P23-0445, Revision 1 dated April 26, 2023.

Purpose and Methodology. The principal purposes of this study were to evaluate the general soil conditions at the proposed site and to develop geotechnical engineering design recommendations. To accomplish its intended purposes, this study was conducted in the following phases:

1. Drill sample borings to evaluate the soil conditions at the boring locations and to obtain soil samples;
2. Conduct laboratory tests on selected samples recovered from the borings to establish the pertinent engineering characteristics of the soils; and
3. Perform engineering analyses, using field and laboratory data, to develop design criteria.

Required Review. Detailed design plans were not available at the time of preparation of this report. Recommendations in our report are contingent upon RINER reviewing and approving in writing the following design items prior to construction:

- Site grading plan, and
- Foundation plan, details, and related structural loads.

Cautionary Statement Regarding Use of this Report. As with any geotechnical engineering report, this report presents technical information and provides detailed technical recommendations for civil and structural engineering design and construction purposes. RINER, by necessity, has assumed the user of this document possesses the technical acumen to understand and properly utilize the information and recommendations provided herein.

RINER strives to be clear in its presentation and, like the user, does not want potentially detrimental misinterpretation or misunderstanding of this report. Therefore, we encourage any user of this report with questions regarding its content to contact RINER for clarification. Clarification will be provided verbally and/or issued by RINER in the form of a report addendum, as appropriate.

Report Specificity. This report was prepared to meet the specific needs of the client for the specific project identified. Recommendations contained herein should not be applied to any other project at this site by the client or anyone else without the explicit approval of RINER.

This Report is NOT a Specification. Recommendations in this report are not specifications. Geotechnical engineering requires significant experience and professional judgment. Conditions vary in the field which require and/or allow modification to recommendations provided herein at the discretion of the Geotechnical Engineer of Record.

2.0 FIELD STUDY

Subsurface study. The subsurface study for this project is summarized in the following table. Boring locations are provided in Appendix B - Boring Location Diagram.

Boring Nos.	Depth, feet bgs[1]	Date Drilled	Location[2]
B-01 to B-03	20	5/8/2023	Building Area
B-04	15	5/8/2023	Detention Pond Area
B-05 to B-06	10	5/8/2023	Paving Area
B-07	10	5/8/2023	Turn Lane

Notes:
1. bgs = below ground surface
2. Boring locations provided in Appendix B - Boring Location Diagram were not surveyed and should be considered approximate. Borings were located by recreational hand-held GPS unit. Horizontal accuracy of such units is typically on the order of 20-feet.

Boring Logs. Subsurface conditions were defined using the sample borings. Boring logs generated during this study are included in Appendix C - Boring Logs and Laboratory Results. Borings were advanced between sample intervals using continuous flight auger drilling procedures.

Cohesive Soil Sampling. Cohesive soil samples were generally obtained using Shelby tube samplers in general accordance with American Society for Testing and Materials (ASTM) D1587. The Shelby tube sampler consists of a thin-walled 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 undisturbed soils by the hydraulic pulldown of the drilling rig. The soil specimens were extruded from the tube in the field, logged, tested for consistency using a hand penetrometer, sealed and packaged to maintain "in situ" moisture content.

Consistency of Cohesive Soils. 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 the respective sample depths on the boring logs. When the capacity of the penetrometer is exceeded, the value is tabulated as 4.5+.

Granular Soil Sampling. Granular soil samples were generally 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 outside diameter (OD) split barrel sampling spoon driven 18-inches into the ground using a 140-pound (lb) 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. Samples were sealed and returned to our laboratory for further examination and testing.

Groundwater Observations. Groundwater observations are shown on the boring logs.

Borehole Plugging. Upon completion of the borings, the boreholes were backfilled from the top and plugged at the surface.

3.0 LABORATORY TESTING

RINER performs visual classification and any of a number of laboratory tests, as appropriate, to define pertinent engineering characteristics of the soils encountered. Tests are performed in general accordance with ASTM or other standards and the results included at the respective sample depths on the boring logs or separately tabulated, as appropriate, and included in Appendix C - Boring Logs and Laboratory Results. Laboratory tests and procedures routinely utilized, as appropriate, for geotechnical studies are tabulated in the following table.

Test Procedure	Description
ASTM D7928	Standard Test Method for Particle-Size Distribution (Gradation) of Fine-Grained Soils Using the Sedimentation (Hydrometer) Analysis
ASTM D698	Standard Test Methods for Laboratory Compaction Characteristics of Soil Using Standard Effort
ASTM D1140	Standard Test Methods for Amount of Material in Soils Finer than the No. 200 (75-µm) Sieve
ASTM D1557	Standard Test Methods for Laboratory Compaction Characteristics of Soil Using Modified Effort
ASTM D1883	Standard Test Method for CBR (California Bearing Ratio) of Laboratory-Compacted Soils
ASTM D2166	Standard Test Method for Unconfined Compressive Strength of Cohesive Soil
ASTM D2216	Standard Test Method for Laboratory Determination of Water (Moisture) Content of Soil and Rock by Mass
ASTM D2217	Standard Practice for Wet Preparation of Soil Samples for Particle-Size Analysis and Determination of Soil Constants

Test Procedure	Description
ASTM D2434	Standard Test Method for Permeability of Granular Soils (Constant Head)
ASTM D2435	Standard Test Methods for One-Dimensional Consolidation Properties of Soils Using Incremental Loading
ASTM D2487	Standard Classification of Soils for Engineering Purposes (Unified Soil Classification System)
ASTM D2488	Standard Practice for Description and Identification of Soils (Visual-Manual Procedure)
ASTM D2850	Standard Test Method for Unconsolidated-Undrained Triaxial Compression Test on Cohesive Soil
ASTM D2937	Standard Test Method for Density of Soil in Place by the Drive-Cylinder Method
ASTM D4220	Standard Practices for Preserving and Transporting Soil Samples
ASTM D4318	Standard Test Methods for Liquid Limit, Plastic Limit and Plasticity Index of Soils
ASTM D4546	Standard Test Methods for One-Dimensional Swell or Settlement Potential of Cohesive Soils
ASTM D4643	Standard Test Method for Determination of Water (Moisture) Content of Soil by the Microwave Oven Method
ASTM D4644	Standard Test Method for Slake Durability of Shales and Similar Weak Rocks
ASTM D4647	Standard Test Method for Identification and Classification of Dispersive Clay Soils by the Pinhole Test
ASTM D4718	Standard Practice for Correction of Unit Weight and Water Content for Soils Containing Oversize Particles
ASTM D4767	Standard Method for Consolidated Undrained Triaxial Compression Test for Cohesive Soils
ASTM D4972	Standard Test method for pH of Soils
Manufacturer's Instructions	Soil Strength Determination Using a Torvane
Tex-145-E	Determining Sulfate Content in Soils - Colorimetric Method

4.0 SITE CONDITIONS

4.1 General

Review of Aerial Photographs. Historical aerial photographs of the site were reviewed for potential past alterations to the site which could impact geotechnical design conditions. Specifically, aerial photographs were reviewed to visually assess obvious areas of significant past fill on site. Aerial photographs reviewed for this study are identified in the following table and are included in Appendix D - Aerial Photographs.

Aerial Photographs Reviewed	
Year	Observations Since Prior Aerial Photograph
1944	The site was undeveloped. A dirt road was noted at the west of the site.
1978	Structures were noted at the west of the site.
1989	No visible changes were noted.
1995	Cypress Rosehill Road was noted to the west of the site.
2002	No visible changes.
2004	No visible changes.
2006	No visible changes.

2010	No visible changes.
2011	Earthwork and grading activities were noted to the west portion of the site.
2012	No visible changes.
2013	No visible changes.
2014	Rough grading activities were noted to west of the site.
2015	Rough grading activities were noted to the western half of the site.
2016	Earthwork activities were noted across the site.
2017	No visible changes.
2019	No visible changes.
2020	No visible changes.
2022	Rough grading activities were noted on the west half of the site.

Site Fills. Due to previous earthwork activities, we would expect surficial disturbance of site soils. Our review revealed areas of fill on-site. **Existing fill recommendation are provided in Section 5.10**

Potential Existing Foundations. Demolition considerations for the potential existing foundations are provided in Section 5.6.

Limitations. Due to the intermittent nature and relatively low resolution of aerial photographs, as well as our lack of detailed information regarding the past land use of the site, our review should not be interpreted as eliminating the possibility of cuts and/or fills on site which could detrimentally affect future construction.

Topography. A United States Geological Survey (USGS) topographic map of the site is provided in Appendix E - USGS Topographic Map. The map indicates the site slopes is relatively flat.

Site Photographs. Representative photographs of the site at the time of this study are provided in "Appendix F - Site Photographs". Photographed conditions are consistent with the aerial photographs and topographic map.

4.2 Geology

Geologic Formation. Based on available surface geology maps and our experience, it appears this site is located in the Lissie Formation near the map contact of the Willis Formation. A geologic atlas and USGS formation description are provided in "Appendix G - Geologic Information". Soils within the Lissie Formation can generally be characterized as sand, silt, and clay. Soils within the Willis Formation can generally be characterized as clay, silt, sand, and gravel.

Geologic Faults. A review of the attached geologic map indicates the nearest geologic fault is about 13-miles southeast of the project site. A geologic fault study was beyond the scope of this study.

4.3 Soil

Stratigraphy. Descriptions of the various strata and their approximate depths and thickness per the Unified Soil Classification System (USCS) are provided on the boring logs included in "Appendix C - Boring Logs and Laboratory Results". Terms and symbols used in the USCS are presented in "Appendix H - Unified Soil Classification System". A brief summary of the stratigraphy indicated by the borings is provided in the following table.

Generalized Subsurface Conditions at Proposed Building Location (Borings B-01 to B-03)			
Nominal Depth, feet bgs (Except as Noted)		General Description	Detailed Description of Soils/Materials Encountered
Top of Layer	Bottom of Layer		
0	**2**	**SILTY CLAY AND SILTY SAND**	**Hard SILTY CLAY (CL-M), and SILTY SAND (SM).**
2 to 4	10	SILTY CLAY AND LEAN CLAY	Stiff to hard SANDY LEAN CLAY (CL) / LEAN CLAY WITH SAND (CL), and stiff SILTY CLAY (CL-ML).
10	20	SAND	Medium dense SILTY SAND (SM).
Note: Boring Termination Depth = 20 feet bgs.			

Generalized Subsurface Conditions at Proposed Detention Pond Location (Borings B-04)			
Nominal Depth, feet bgs (Except as Noted)		General Description	Detailed Description of Soils/Materials Encountered
Top of Layer	Bottom of Layer		
0	2	SAND	SILTY SAND (SM).
2	10	LEAN CLAY	Very stiff to hard SANDY LEAN CLAY (CL).
10	20	SAND	Medium Dense SILTY SAND (SM).
20	25	LEAN CLAY	Hard SANDY LEAN CLAY (CL).
25	30	FAT CLAY	Very stiff FAT CLAY WITH SAND (CH).
Note: Boring Termination Depth = 20 feet bgs.			

Generalized Subsurface Conditions at Proposed Paving Location (Borings B-05 to B-06)			
Nominal Depth, feet bgs (Except as Noted)		General Description	Detailed Description of Soils/Materials Encountered
Top of Layer	Bottom of Layer		
0	2 to 4	SAND AND LEAN CLAY	Hard SANDY LEAN CLAY (CL) FILL, very stiff CLAYEY SAND (SC) FILL, and SILTY SAND (SM)
2 to 4	10	LEAN CLAY	Stiff to hard SANDY LEAN CLAY (CL) / LEAN CLAY WITH SAND (CL)
Note: Boring Termination Depth = 10 feet bgs.			

Generalized Subsurface Conditions at Proposed Turn Lane Location (Borings B-07)			
Nominal Depth, feet bgs (Except as Noted)		General Description	Detailed Description of Soils/Materials Encountered
Top of Layer	Bottom of Layer		
0	6	FAT CLAY	Hard SANDY LEAN CLAY (CL) FILL
6	10	LEAN CLAY	Very stiff to hard SANDY LEAN CLAY (CL)
Note: Boring Termination Depth = 10 feet bgs.			

Moisture Change Susceptibility of Near Surface Soils. The sandier/siltier soils encountered at and near the ground surface at this site are very susceptible to changes in moisture. The presence of surface water due to precipitation or groundwater may result in a decrease in the ability to compact and work with the soil. It is common for these soils to pump when subjected to high levels of moisture. In addition, these soils located at and near the ground surface will allow surface water to infiltrate until the water becomes perched on a less permeable layer at depth. Soils of this type are especially prone to requiring the implementation of wet weather/soft subgrade recommendations provided in this report.

Swell Potential based on Atterberg Limits. Atterberg (plastic and liquid) limits were performed on 4 shallow soil samples obtained at depths between 0- and 8-feet bgs. The plasticity index of the samples was between 4 and 34 with an average of 18 indicating that the soils have a moderate potential for shrinking and swelling with changes in soil moisture content.

Swell Tests. Swell tests were performed on selected clay soil samples. Swell test details are provided in “Appendix C - Boring Logs and Laboratory Results”. The results of the tests are summarized in the following table.

Boring No.	Avg. Depth (ft.)	Moisture Content, w, %	Liquid Limit, LL	Plasticity Index, PI	Applied Overburden Stress (psi)	Swell (%)
B-01	1	12	15	4	0.9	0.00
B-02	3	18	22	4	2.6	0.31
B-03	5	16	49	34	4.3	0.04
B-04	5	13	48	30	4.3	0.00

4.4 Groundwater

Groundwater Levels. The borings were advanced using auger drilling and intermittent sampling methods in order to observe groundwater seepage levels. Groundwater levels encountered in the borings during this study are identified in the following table.

Boring No.	Depth Groundwater Initially Encountered (feet, bgs)	Groundwater Depth after 15 Minutes (feet, bgs)
B-04	18	13.1
All other borings	Not Encountered	Not Encountered

Long-term Groundwater Monitoring. B-04 was converted to a piezometer after drilling completion at the site on May 26, 2023.

Piezometer No.	Location	Total Depth of Piezometer (feet, bgs)	Screen Length (feet, bgs)	Riser Length (feet, bgs)
PZ-01 (B-04)	B-05	20.0	10.0	10.0

Groundwater level measurements in the piezometers are presented below.

Piezometer No.	Date Measured	Groundwater Depth (feet, bgs)
PZ-01 (B-04)	5/26/2023	9.1
	6/27/2022	9.8

Long-term monitoring can reveal groundwater levels materially different than those encountered during measurements taken while drilling the borings and in the piezometer. *The piezometer installed as part of this study should be monitored over a period to assess groundwater fluctuations.*

Groundwater Fluctuations. Future construction activities may alter the surface and subsurface drainage characteristics of this site. It is difficult to accurately predict the magnitude of subsurface water fluctuations that might occur based upon short-term observations. The groundwater level should be expected to fluctuate throughout the years with variations in precipitation.

5.0 ANALYSIS AND RECOMMENDATIONS

5.1 Seismic Site Classification

The seismic site classification is based on the 2018 International Building Code (IBC) and is a classification of the site based on the type of soils encountered at the site and their engineering properties. Per Table 20.3-1 of ASCE 7-10, the seismic site classification for this site is D.

5.2 Potential Vertical Rise (PVR)

Potential Vertical Rise. Potential Vertical Rise, PVR, is the calculated upward heave of the ground surface due to expansive soils related to weather-related changes in soil moisture in the active zone. PVR only applies to upward movement. The term settlement applies to downward movement related to loads on the soil.

Problem Discussion. Most clay soils swell when subjected to increases in moisture content. Swelling clay soils exert an outward pressure that can easily exceed 5,000 psf when subjected to moisture increases. Swell potential and swell pressures are a function of several factors including clay mineralogy and antecedent moisture condition. Generally, for a given clay soil, the drier the soil the greater its potential to swell and the higher its swell pressure. Conversely, wetter soils generally have a lower potential to swell and have lower swell pressures. The potential for a clay soil to swell is a variable and cannot be separated from its moisture condition.

The overburden pressure at a given depth above the groundwater table is calculated as the unit weight of the soil times the depth. For a soil with a unit weight of 125 pcf, the overburden pressure at 10-feet would be 1250 psf (125 pcf x 10-feet). Thus, the swell pressure can exceed the overburden at depths of over 40-feet. This means soils at 40-feet exposed to changes in moisture can impact movements at the ground surface.

For a clay soil to swell or shrink, it must be subjected to increases or decreases in moisture content, respectively. The predominant way clay soils are subjected to increases or decreases in moisture content is the weather. As would be expected, extended periods of wet weather cause soil to get wetter and extended dry weather cause soil to get drier. The longer the period of wet or dry weather, the deeper the influence of the weather. Vegetation also causes variations in soil moisture content. Shallow rooted grass and bushes have a shallower impact, deep rooted trees have a deeper impact.

For a clay soil at a given depth to influence surface heave, two things must happen: (1) the soil must be subjected to an increase in moisture, and (2) the swell pressure of the soil must exceed the overburden pressure. Swell is typically calculated by assuming an "active" zone, a depth of soil impacted by weather which predominantly affects surface movements due to soil swell. Expansive soils below the active zone are typically ignored as they are assumed to be exposed to lower increases in moisture, experience higher overburden pressures, and have a less significant impact on the surface heave than the soils in the active zone.

"Deep-seated" soil movement is swelling of the clay soils below the active zone and above the equilibrium depth. The equilibrium depth is the depth at which the overburden pressure and clay swell pressure are equal. Deep-seated soil movement is caused by changes in moisture that are typically not related to weather or vegetation. They can be caused by man-made influences such as leaking deep water or sewer lines. They can be caused by natural influences such as fluctuations in soil moisture content or groundwater levels. They are notoriously hard to accurately predict and may or may not actually occur. Unless stated otherwise, we have not included the effects of deep-seated soil movement in our Potential Vertical Rise (PVR) calculation. The inclusion of deep-seated soil movement drastically increases the depth of the building pad preparation required and may make a slab-on-grade target PVR of 1-inch theoretically unattainable. The inclusion or exclusion of deep-seated soil movement is a matter of professional opinion, on which there is no consensus among consultants. It is also a matter of risk tolerance and cost, of which, the user of this report is being made aware.

As evidenced in this discussion, calculation of PVR is based on soil data, model assumptions, experience, and professional judgment. PVR is a calculated estimate and should not be construed to be an absolute number or a guarantee of performance. PVR can be higher or lower depending on actual site conditions. The PVR estimate we provide is our best estimate of what will be encountered and the user of this report with doubts is encouraged to get another professional opinion prior to using this report. However, based on this discussion, the reader understands variations between the model and reality can introduce significant differences in calculated PVR. The user of this report understands and accepts this risk. If this risk is intolerable, the user of this report should be prepared to utilize a structural slab suspended adequately above the subgrade surface and supported on deep foundations.

Differential swelling of clay soil is generally most pronounced around the perimeter of slabs or pavement where weather and/or vegetative influences are greatest. Unstiffened slabs or paving are generally prone to cracking around 5- to 10-feet from and parallel to the slab edge due to differential soil movements. If this expected cracking is unacceptable or needs to be minimized, the structural engineer should consider slab stiffening using grade beams and/or a flexible slab/wall connection design. We should be consulted by the structural engineer for clarifications and input regarding this type of slab movement if it is deemed critical.

Maintaining a consistent moisture content in the soil is the key to minimizing both heave and shrinkage related structural problems. Therefore, building maintenance and control of water are paramount in the performance of a slab-on-grade and shallow foundations.

PVR or Equivalent Calculations. The PVR or its equivalent can be estimated several ways. RINER utilizes the TxDOT method, swell tests, and a Volflo analysis to provide the best possible understanding of expected PVR and its variability.

Calculated PVR using TxDOT Method Tex-124-E. PVR calculations were performed in general accordance with the Texas Department of Transportation (TxDOT) Method Tex-124-E. The Tex-124-E method is empirical and is based on the Atterberg limits and moisture content of the subsurface soils. The calculated PVR is an empirical estimate of a soil's potential for swell based upon the soil's plasticity index, applied loading (due to structures or overburden), and antecedent moisture condition. The PVR calculated using TxDOT Method Tex-124-E is about 1-inch assuming an average antecedent moisture condition. The calculated PVR is consistent with soil moisture conditions at the time this study was conducted. An 8-feet zone of seasonal moisture variation was used in our analysis based on local experience.

Calculated PVR using Swell Test Results. The equivalent PVR based on the swell test results is about 1-inch. The PVR based on swell test results is dependent on the moisture conditions at the time of testing. An 8-feet zone of seasonal moisture variation was used in our analysis based on local experience.

Calculated PVR using Volflo Analysis. The equivalent PVR based on the Volflo analysis results is about 1-inch. The calculated PVR based on the Volflo analysis is dependent on the moisture conditions at the time of testing. A 8-feet zone of seasonal moisture variation was used in our analysis based on local experience.

Soil Moisture Confirmation Prior to Construction. The calculated PVR can vary considerably with prolonged wet or dry periods. We recommend the moisture content for the upper 8-feet (active zone) of soils within the building pad be assessed for consistency with this report prior to construction if: (1) an extended period of time has elapsed between the performance of this study and construction of the foundation, or (2) unusually wet or dry weather is experienced between the performance of this study and construction of the foundation.

5.3 Construction Excavations

Applicability. Recommendations in this section apply to short-term construction-related excavations for this project.

Sloped Excavations. All sloped short-term construction excavations on-site should be designed in accordance with Occupational Safety and Health Administration (OSHA) excavation standards. Borings from this study indicated that the soils may be classified per OSHA regulations as Type C from the ground surface to a depth of 10-feet bgs. Short-term construction excavations may be constructed with a maximum slope of 1.5:1, horizontal to vertical (H:V), to a depth of 10-feet bgs. If excavations are to be deeper than 10-feet, we should be contacted to evaluate the excavation. Recommendations provided herein are not valid for any long-term or permanent slopes on-site.

Shored Excavations. As an alternative to sloped excavations, vertical short-term construction excavations may be used in conjunction with trench boxes or other shoring systems. Shoring systems should be designed using an equivalent fluid weight of 75 pounds per cubic foot (pcf) above the groundwater table and 100 pcf below the groundwater table. Surcharge pressures at the ground surface due to dead and live loads should be added to the lateral earth pressures where they may occur. Lateral surcharge pressures should be assumed to act as a uniform pressure along the upper 10-feet of the excavation based on a lateral earth coefficient of 0.5. Surcharge loads set back behind the excavation at a horizontal distance equal to or greater than the excavation depth may be ignored. We recommend that no more than 200-feet of unshored excavation should be open at any one time to prevent the possibility of failure and excessive ground movement to occur. We also recommend that unshored excavations do not remain open for a period of time longer than 24-hours.

Limitations. Recommendations provided herein assume there are no nearby structures or other improvements which might be detrimentally affected by the construction excavation. Before proceeding, we should be contacted to evaluate construction excavations with the potential to affect nearby structures or other improvements.

Excavation Monitoring. Construction excavations and their related safety are the responsibility of the Contractor. Excavations should be monitored and documented by a competent professional to confirm site soil conditions consistent with those encountered in the borings drilled as part of this study. Discrepancies in soil conditions should be brought to the attention of RINER for review and revision of recommendations, as appropriate.

5.4 Groundwater Control

Groundwater was encountered in the piezometers at depths as shallow as 9.1-feet bgs in the proposed detention pond boring B-04. If groundwater is encountered during excavation, dewatering to bring the groundwater below the bottom of excavations may be required. Dewatering could consist of standard sump pits and pumping procedures, which may be adequate to control seepage on a local basis during excavation. Supplemental dewatering will be required in areas where standard sump pits and pumping is not effective. Supplemental dewatering could include submersible pumps in slotted casings, well points, or eductors. The contractor should submit a groundwater control plan, prepared by a licensed engineer experienced in that type of work.

5.5 Earthwork

5.5.1 Site Preparation

In the area of improvements, all concrete, trees, stumps, brush, debris, septic tanks, abandoned structures, roots, vegetation, rubbish and any other undesirable matter should be removed and properly disposed. All vegetation should be removed and the exposed surface should be scarified to an additional depth of at least 6 inches. 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.

5.5.2 Proofroll

Building pad and paving subgrades 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 fill placement. Areas of loose or soft subgrade encountered in the proofroll should be removed and replaced with engineered fill, moisture conditioned (dried or wetted, as needed) and compacted in place.

5.5.3 Grading and Drainage

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. A positive slope of the ground away from any foundation should be provided. Ditches or swales should be provided to carry the run-off water both during and after construction. Stormwater runoff should be collected by gutters and downspouts and should discharge away from the buildings.

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.

Lawn areas should be watered moderately, without allowing the clay soils to become too dry or too wet.

5.5.4 Wet Weather/Soft Subgrade

Soft and/or wet surface soils may be encountered during construction, especially following periods of wet weather. Wet or soft surface soils can present difficulties for compaction and other construction equipment. If specified compaction cannot be achieved due to soft or wet surface soils, one of the following corrective measures will be required:

1. Removal of the wet and/or soft soil and replacement with select fill,
2. Chemical treatment of the wet and/or soft soil to improve the subgrade stability, or
3. If allowed by the schedule, drying by natural means.

Chemical treatment is usually the most effective way to improve soft and/or wet surface soils. RINER should be contacted for additional recommendations if chemical treatment is planned due to wet and/or soft soils.

5.5.5 Fill

Select Fill. Any fill placed in building pad areas should consist of select fill. Select fill should consist of soil with a liquid limit less than 35 and a Plasticity Index between 7 and 20. The select fill should be placed in loose lifts not exceeding 8-inches and should be compacted to at least 95 percent maximum dry density (per ASTM D-698) and at a moisture content between optimum and 4 percent above optimum moisture content. The subgrade to receive select fill should be scarified to a depth of 6 inches and compacted to 92 to 96 percent of the material’s maximum standard Proctor dry density (ASTM D-698) at a workable moisture level at least 4 percentage points above optimum.

Lime-treated Native Clay Soil. Based on the laboratory testing conducted for this study, the native clay on-site soils will not meet requirements for select fill outlined in the section titled “Fill”. As an alternative to importing select fill, the native clay soil may be blended with lime to reduce the plasticity index to meet select fill requirements. Based on our experience, we expect that it will require between 4- and 6-percent lime (by dry unit weight) to reduce the plasticity index of the native clay soils to select fill requirements. Prior to selecting this alternative, lime series tests should be performed to assess the amount of lime required.

General Fill. General fill may be placed in improved areas outside of building pad areas. General fill should consist of material approved by the Geotechnical Engineer with a liquid limit less than 50. General fill should be placed in loose lifts not exceeding 8-inches and should be uniformly compacted to a minimum of 95 percent maximum dry density (per ASTM D-698) and within ±2 percent of the optimum moisture content.

Fill Restrictions. Select fill and general fill should consist of those materials meeting the requirements stated. Select fill and general fill should not contain material greater than 4-

inches in any direction, debris, vegetation, waste material, environmentally contaminated material, or any other unsuitable material.

Unsuitable Materials. Materials considered unsuitable for use as select fill or general fill include low and high plasticity silt (ML and MH), silty clay (CL-ML), organic clay and silt (OH and OL) and highly organic soils such as peat (Pt). These soils may be used for site grading and restoration in unimproved areas as approved by the Geotechnical Engineer. Soil placed in unimproved areas should be placed in loose lifts not exceeding 10-inches and should be compacted to at least 92 percent maximum dry density (per ASTM D-698) and at a moisture content within ±4 percentage points of optimum.

Cautionary Note. It is extremely important that select fill placed within building pads be properly characterized using one or more representative proctor samples. The use of a proctor sample which does not adequately represent the select fill being placed can lead to erroneous compaction (moisture and density) results which can significantly increase the potential for swelling of the select fill. The plasticity index of select fill soils placed during construction should be checked every day to confirm conformance to the project requirements and consistency with the proctor being utilized.

5.5.6 Testing

Required Testing and Inspections. Field compaction and classification tests should be performed by RINER. Compaction tests should be performed in each lift of the compacted material. We recommend the following minimum soil compaction testing be performed: one test per lift per 2,500 square feet (SF) in the area of the building pad, one test per lift per 5,000 SF outside the building pad, and one test per lift per 100 linear feet of utility backfill. If the materials fail to meet the density or moisture content specified, the course should be reworked as necessary to obtain the specified compaction. Classification confirmation inspection/testing should be performed daily on select fill materials (whether on-site or imported) to confirm consistency with the project requirements. The testing frequency recommended herein can be altered (increased or decreased) at the discretion of the geotechnical engineer of record.

Liability Limitations. Since proper field inspection and testing are critical to the design recommendations provided herein, RINER cannot assume responsibility or liability for recommendations provided in this report if construction inspection and/or testing is performed by another party.

5.6 Demolition Considerations

Applicability. Recommendations in this section apply to the removal of any existing foundations, utilities or pavement which may be present on this site.

General. Special care should be taken in the demolition and removal of existing floor slabs, foundations, utilities and pavements to minimize disturbance of the subgrade. Excessive disturbance of the subgrade resulting from demolition activities can have serious detrimental effects on planned foundation and paving elements.

Existing Foundations. Existing foundations are typically slabs, shallow footings, or drilled piers. If slab or shallow footings are encountered, they should be completely removed. If drilled piers are encountered, they should be cut off at an elevation at least 24-inches below proposed grade beams or the final subgrade elevation, whichever is deeper. The remainder of the drilled pier should remain in place. Foundation elements to remain in place should be surveyed and superimposed on the proposed development plans to determine the potential for obstructions to the planned construction. RINER should be contacted if drilled piers are to be excavated and removed completely. Additional earthwork activities will be required to make the site suitable for new construction if the piers are to be removed completely.

Existing Utilities. Existing utilities and bedding to be abandoned should be completely removed. Existing utilities and bedding may be abandoned in place if they do not interfere with planned development. Utilities which are abandoned in place should be properly pressure-grouted to completely fill the utility.

Backfill. Excavations resulting from the excavation of existing foundations and utilities should be backfilled in accordance with Section 5.5.5 - Fill.

Other Buried Structures. Other types of buried structures (wells, cisterns, etc.) could be located on the site. If encountered, RINER should be contacted to address these types of structures on a case-by-case basis.

5.7 Loading on Buried Structures

Uplift. Buried water-tight structures are subjected to uplift forces caused by differential water levels adjacent to and within the structure. Soils with any appreciable silt or sand content will likely become saturated during periods of heavy rainfall and the effective static water level will be at the ground surface. For design purposes, we recommend the groundwater level be assumed at the ground surface. Resistance to uplift pressure is provided by soil skin friction and the dead weight of the structure. Skin friction should be neglected for the upper 3 feet of soil. A skin friction of 200 pounds per square foot (psf) may be used below a depth of 3 feet.

Lateral Pressure. Lateral pressures on buried structures due to soil loading can be determined using an equivalent fluid weight of 100 pounds per cubic foot (pcf). This includes hydrostatic pressure but does not include surcharge loads. The lateral load produced by a surcharge may be computed as 50 percent of the vertical surcharge pressure applied as a constant pressure over the full depth of the buried structure. Surcharge loads located a horizontal distance equal to or greater than the buried structure depth may be ignored.

Vertical Pressure. Vertical pressures on buried structures due to soil loading can be determined using an equivalent fluid weight of 125 pcf. This does not include surcharge loads. The vertical load produced by a surcharge may be computed as 100 percent of the vertical surcharge pressure applied as a constant pressure over the full width of the buried structure.

5.8 Retaining Structures

Applicability. RINER was not notified of any specific retaining structures in conjunction with this project. Recommendations provided in this section are applicable to structures 5-feet or less in height. Retaining structures more than 5-feet should be brought to the attention of RINER for a more detailed assessment. *It is imperative that global stability be reviewed by RINER on any retaining structure more than 5-feet in height.*

Lateral Pressure. Lateral pressures on retaining structures due to soil loading can be determined using an equivalent fluid weight of 75 pcf if fill behind the wall is free-draining and above the groundwater table and 100 pcf if fill behind the wall is not free draining or is below the groundwater table. This does not include surcharge loads. This also assumes a horizontal ground surface behind the structure. The lateral load produced by a surcharge may be computed as 50 percent of the vertical surcharge pressure applied as a constant pressure over the full depth of the buried structure. Surcharge loads set back behind the retaining structure at a horizontal distance equal to or greater than the structure height may be ignored.

Lateral Resistance. Resistance to lateral loads may be provided by the soil adjacent to the structure. We recommend using an equivalent fluid weight of 200 pcf for lateral resistance. The passive resistance should be ignored if the material in front of the wall will be excavated at any time in the future. A coefficient of sliding friction of 0.3 between the retaining structure concrete footings and underlying soil may be combined with the passive lateral resistance. Appropriate safety factor should be utilized by the structural engineer for external stability analyses of the retaining structures.

Bearing Capacity. Assuming a minimum embedment depth of 24-inches, an allowable bearing capacity of 2,000 psf may be used for retaining structure footings (using a Factor of Safety of 3).

5.9 Buried Pipe

Applicability. Recommendations in this section are applicable to the design of buried piping placed by open cut methods associated with this project.

Pressure on Buried Pipe. Design recommendations provided in the "Loading on Buried Structures" section of this report apply to buried piping.

Thrust Restraints. Resistance to lateral forces at thrust blocks will be developed by friction developed along the base of the thrust block and passive earth pressure acting on the vertical face of the block. We recommend a coefficient of base friction of 0.3 along the base of the thrust block. Passive resistance on the vertical face of the thrust block may be calculated using the allowable passive earth pressures presented in the following table.

Allowable Passive Earth Pressure by Material Type	
Material	Allowable Passive Pressure (psf)
Sand	100 x Depth in Feet
Native Clay and Clayey Sand	2,000
Compacted Clay Fill	1,500
Note: Passive resistance should be neglected for any portion of the thrust block within 3 feet of the final site grade. The allowable passive resistance for native clays and clayey sand is based on the thrust block bearing directly against vertical, undisturbed cuts in these materials.	

Bedding and Backfill. Pipe bedding and pipe-zone backfill for the water and sanitary sewer piping should be in accordance with TxDOT standard specification Item 400 or the local equivalent. The pipe-zone consists of all materials surrounding the pipe in the trench from six (6) inches below the pipe to 12 inches above the pipe.

Trench Backfill. Excavated site soils will be utilized to backfill the trenches above the pipe-zone. Backfilled soil should be placed in loose lifts not exceeding 8-inches and should be compacted to at least 95 percent maximum dry density (per ASTM D-698) and at a moisture content between optimum and 4 percent above optimum moisture content.

Trench Settlement. Settlement of backfill should be anticipated. Even for properly compacted backfill, fills are still subject to settlements over time of up to 2 percent of the total fill thickness. This level of settlement can be significant for fills beneath streets. Therefore, close coordination and monitoring should be performed to reduce the potential for future movement.

5.10 Existing Fill

Existing fill was encountered at boring locations during the subsurface investigation. The existing fill extended to depths of about 2 feet bgs in the building pad areas. For the purpose

of this report, we assumed the existing fill was placed under engineered supervision. If there is no record indicating that the fill was placed and compacted in a controlled manner (engineered fill), it will be necessary to confirm appropriate compaction or excavate all the existing fill in the building pad area and recompact it in accordance with select fill placement requirements prior to construction of the building pads. Any existing fill within the proposed paving area may remain in-place provided the proofroll is passed.

5.11 Foundation System

Appropriate Foundation Types. The following foundation types are appropriate to the site based on the geotechnical conditions encountered:

- Shallow footings, or
- Underreamed drilled piers.

Foundation Determination. We have assumed that structural loads will be typical for the type and size of building proposed. Recommendations for the foundation types are presented below. Final determination of the foundation type to be utilized for this project should be made by the Structural Engineer based on loading, economic factors and risk tolerance.

Avoidance of Mixing Foundation Types. Mixing of foundation types for a given building should be avoided. Where mixing of shallow footings and underreamed drilled piers is required for a given building, we should be contacted to review the foundation plans prepared by the Structural Engineer prior to construction. Shallow footing foundations and underreamed drilled pier foundations can have incompatible movement characteristics.

Foundations Adjacent to Slopes. Foundations placed too close to adjacent slopes steeper than 5H:1V may experience reduced bearing capacities and/or excessive settlement. Recommendations provided herein assume foundations are not close enough to adjacent slopes in excess of 5H:1V to be detrimentally affected. Therefore, foundations closer than 5 times the depth of adjacent slopes, pits, or excavations in excess of 5H:1V should be brought to our attention in order that we may review the appropriateness of our recommendations.

Assumed Maximum Cut/Fill Depth. We have also assumed that cut/fill of less than 1-foot will be required to bring the site to grade. In the event cut/fill in the building pad exceed 1-foot, we should be notified and allowed to review the design to assess the suitability of the foundation recommendations provided. ***RINER must be allowed to review the finalized grading plan to assess the appropriateness of our recommendations.***

Foundation Plans Review. Our office should be contacted to review the foundation plans, details and related structural loads, prior to finalizing the design to check conformance with the recommendations presented herein.

5.11.1 Shallow Footings

General Requirement. Shallow strip and spread footing foundations may be used for support of the proposed structure provided that recommendations in the section entitled "Slab-on-Grade" are followed.

Foundation Depth. Shallow strip and spread footing foundations should bear on native soil or select fill at a minimum depth of 2-feet below the surrounding grade.

Bearing Capacity. Continuous strip footings can be proportioned using a net dead load plus sustained live load bearing pressure of 2,000 psf or a net total load bearing pressure of 3,000 psf, whichever condition results in a larger bearing surface. Individual spread footings can be proportioned using a net dead load plus sustained live load bearing pressure of 2,600 psf or a net total load bearing pressure of 3,900 psf, whichever condition results in a larger bearing surface. These bearing pressures are based on a safety factor of 3 and 2, respectively.

Geometry. Individual spread footings should be at least 30 inches wide and continuous strip footing foundations should be at least 16 inches wide.

Settlement. Settlement of footing foundations is influenced by several factors, including load (pressure), soil consolidation properties, depth to groundwater, geometry (width and length), depth, spacing, and quality of construction. Although a detailed settlement analysis is beyond the scope of this study, settlement for foundations, with a maximum horizontal dimension of 10-feet, constructed as described above should be about 1 inch or less.

Lateral Resistance. Resistance to lateral loads may be provided by the soil adjacent to the footings. We recommend using an equivalent fluid weight of 200 pcf for lateral resistance. An allowable coefficient of sliding friction of 0.3 (using a Factor of Safety of 2) between the concrete footings and underlying soil may be combined with the passive resistance.

Construction and Observation. The geotechnical engineer should monitor foundation construction to verify conditions are as anticipated and that the materials encountered are suitable for support of foundations. Soft or unsuitable soils encountered at the foundation bearing level should be removed to expose suitable, firm soil. Foundation excavations should be dry and free of loose material. Excavations for foundations should be filled with concrete before the end of the workday or sooner if necessary to prevent deterioration of the bearing surface. Prolonged exposure or inundation of the bearing surface with water will result in changes in strength and compressibility characteristics. If delays occur, the excavation should be deepened as necessary and cleaned, in order to provide a fresh bearing surface. If more than 24 hours of exposure of the bearing surface is anticipated in the excavation, a "mud slab" should be used to protect the bearing surfaces. If a mud slab is used, the foundation excavations should initially be over-excavated by approximately 4 inches and a lean concrete mud slab of approximately 4 inches in thickness should be placed in the bottom of the excavation immediately following exposure of the bearing surface by excavation. The mud slab will protect the bearing surface, maintain more uniform moisture in the subgrade, facilitate dewatering of excavations if required and provide a working surface for the placement of formwork and reinforcing steel.

5.11.2 Underreamed Drilled Piers

General. Underreamed drilled pier foundations bearing in native soil may be utilized at this site for the proposed office/warehouse buildings provided that recommendations in the section entitled "Slab-on-Grade" are followed.

Foundation Depth. We recommend that underreamed piers should bear in native soil at a depth of 8-feet below the existing grade.

Bearing Capacity. The piers may be proportioned using a net dead load plus sustained live load bearing pressure of 3,000 psf or a net total load pressure of 4,500 psf, whichever condition results in a larger bearing surface. These bearing pressures are based on a safety factor of 3 and 2, respectively, against shear failure of the foundation bearing soils.

Cautionary Note: Silty sand was encountered at depths as shallow as 10-feet bgs during the subsurface investigation. If silty sand is encountered within the pier installation depths, adjustment in pier depth may be required in some areas to maintain the bottom of the piers above any caving soils. Adjustments in the depths of the piers should be observed in the field by RINER. In addition, drilled, straight-shaft piers (with a straight-shaft diameter equal to the planned underream pier diameter) may need to be substituted in lieu of underreamed piers inisolated locations due to possibility of caving soils. Field observations by RINER during construction will be required to determine areas where, underreaming is not possible.

Settlement. Settlement of underreamed drilled pier foundations is influenced by several factors, including load (pressure), soil consolidation properties, depth to groundwater, geometry (width and length), depth, spacing, and quality of construction. Although a detailed settlement analysis is beyond the scope of this study, soil related settlement for foundations, 6-feet in diameter or less, constructed as described above should be about 1 inch or less. We should be allowed to review piers greater than 6-feet in diameter to assess their settlement. However, pier foundation settlement is heavily affected by construction quality and, as a result, oftentimes exceeds 1 inch. Our settlement estimate assumes that proper construction practices are followed and there are no overlapping stresses due to adjacent piers. To mitigate any overlapping stresses due to adjacent piers, we recommend a minimum clear spacing of one bell diameter (larger bell diameter) between adjacent piers.

Lateral Capacity. Because of the potential for the upper two feet of the soil to shrink and pull away from drilled piers during dry periods, we recommend soil resistance to lateral loads on drilled piers be ignored in the upper 2-feet of the soil profile. For resistance of lateral loads on drilled piers, we recommend the following LPILE design parameters.

Depth (feet) [1]	Soil Type	Effective Soil Unit Weight (pcf) [2]	Allowable Cohesion, c (psf) [3]	Angle of Internal Friction, ϕ (degrees)	Strain at ½ Peak Strength, ε_{50}	Soil Modulus Parameter, k (for lateral loads) (pci)
0 - 2	Sand	120	0	0	NA	NA
2 - 10	Clay	120	700	0	0.007	300

Notes:

1. Depth below existing grade.
2. Effective soil unit weight based on assumed groundwater depth greater than 10-feet.
3. Factor of safety 3 is included in the recommended cohesion parameter.

Uplift. The uplift force on the piers due to swelling of the active clays can be approximated by assuming a uniform uplift pressure of 1,000 psf acting over the perimeter of the shaft to a depth of 8 feet. The shafts should contain sufficient full length reinforcing steel to resist uplift forces.

Uplift Resistance. The uplift resistance provided by an underreamed drilled pier is the sum of resistance provided by the shear strength of the soil, the weight of the soil above the bell and the weight of the drilled pier itself. The following equation may be used to calculate the allowable uplift resistance:

$$F_a = R_F c N_u A_u + \frac{W_s}{FS_1} + \frac{W_c}{FS_2}$$

Where: F_a = allowable uplift resistance, lbs
c = allowable cohesion, psf
N_u = bearing capacity factor = $\frac{3.5D_b}{B_b} \leq 9$
R_F = reduction factor for closer pier spacings (discussed below)
D_b = depth to base of bell, ft
B_b = diameter of the bell, ft
A_u = projected area of bell, ft^2 = $\frac{\pi(B_b^2 - B_s^2)}{4}$
B_s = diameter of the drilled pier shaft, ft
W_s = weight of soil above bell, lbs
W_c = weight of drilled pier, lbs
FS_1, FS_2 = safety factors

A safety factor of 3 has been applied to the allowable cohesion value and is appropriate for sustained loading conditions. However, the allowable cohesion values may be increased by

50 percent, resulting in a safety factor of 2, for transient loading conditions. We recommend a soil weight safety factor, FS_1, of 1.2 and a drilled pier weight safety factor, FS_2, of 1.1.

Shaft/Diameter Ratio. The piers should be provided with an underream diameter to shaft diameter ratio not less than 2 to 1 and not greater than 3 to 1. **There is an inherent risk of bell collapse during construction. Unforeseen sand and silt pockets/seams and/or laminated/slickensided structures in clays or variable groundwater conditions can cause significant loss of tensile strength resulting in bell collapse. Therefore, RINER recommends test piers with underreams be constructed prior to finalizing the foundation design to assess the risk of bell collapse**.

Pier Spacing. For uplift considerations, piers should not be spaced closer than two underream diameters (edge to edge) based on the diameter of the larger underream. Closer pier spacings may result in reduced uplift capacity. We should be contacted to review closer pier spacings on a case-by-case basis.

Reduction Factor (R_F) for Closer Pier Spacings. A reduction in uplift resistance will be required for piers spaced closer than two underream diameters (edge to edge) as discussed above. The reduction factor is dependent on the number of piers in close proximity to the pier in question. The following table shows the recommended Reduction Factor (R_F) values based on number of piers in close proximity to the pier in question:

Numbers of Piers in Close Proximity to a Given Pier	Reduction Factor (R_F)
Piers > 2 Diameters Edge-to-Edge Spacing	1
1	0.6
2	0.4
3	0.2
Greater than or equal to 4	0

There will be no reduction in uplift resistance contribution from the weight of soil above bell and weight of drilled pier. RINER should be contacted to review the final foundation plans for review.

Construction Observation. The construction of all piers should be observed as a means to verify compliance with design assumptions and to verify:

1. the bearing stratum;
2. underream size;
3. the removal of all smear zones and cuttings;
4. that groundwater seepage, when encountered, is correctly handled; and
5. that the shafts are vertical (within acceptable tolerance).

We should be contacted for further evaluation and recommendations if soils other than those anticipated to be encountered at the design foundation bearing level, or if groundwater seepage and/or underream collapse occurs.

Groundwater. Groundwater was initially encountered at depths as shallow as 18-feet bgs in borings during drilling and rose to depths as shallow as 13.1-feet within 15-minutes. However, groundwater may be encountered during pier excavation and the risk of groundwater seepage is increased during or after periods of precipitation. Submersible pumps may be capable of controlling seepage in the pier excavation to allow for concrete placement.

Applicable TxDOT Standards. Drilled pier foundations should be constructed in accordance with the requirements of TxDOT Item 416 (standard specification for construction of drilled pier foundations).

Concrete Placement. Concrete should be placed in the shafts immediately after excavation to reduce the risk of significant groundwater seepage, deterioration of the foundation-bearing surface and underream collapse. Concrete should have a slump of 5 to 7 inches and should not be allowed to strike the shaft sidewall or steel reinforcement during placement.

5.12 Slab-on-Grade

Site Grading Plan. The site grading plan was not available at the time of writing this report. We have assumed that cut/fill of less than 1-foot will be required to bring the site to grade. In the event cut/fill in the building pad exceeds 1-foot, we should be notified and allowed to review the design to assess the suitability of the recommendations provided in this section. ***RINER must be allowed to review the finalized grading plan to assess the appropriateness of our recommendations.***

Potential Vertical Slab Movements. Based on the information gathered during this investigation, a slab constructed on-grade will be subject to potential vertical slab movements of about 1-inch.

Subgrade Treatment. Based on the calculated PVR of the onsite soils, no additional subgrade improvement is required to reduce the PVR to 1-inch. *The sandier soils encountered at and near the ground surface at this site are prone to requiring the implementation of wet weather/soft subgrade recommendations provided in this report. Any fill placed in building pad areas should consist of select fill.*

Subgrade Treatment at Exterior Doorways. Subgrade treatment should extend beneath sidewalk areas that abut exterior doorways to the building. Failure to perform subgrade treatment in these areas can increase the probability of differential heaving between exterior sidewalks and doorways, resulting in exterior doors that will not or have difficulty opening outward due to “sticking” caused by heaving sidewalk slabs.

Subgrade Moisture. The slab subgrade is prone to drying after being exposed and should be kept moist prior to slab placement.

Moisture Barrier. A moisture barrier should be used beneath the slab foundation in areas where floor coverings will be utilized (such as, but not limited to, wood flooring, tile, linoleum, and carpeting).

Slab Deflection Analysis*. Coefficient of subgrade reaction, k, values are soil, load, and settlement dependent. Upon request by the Structural Engineer for this project, k value recommendations will be provided for the specific loading application in question.*

Fill Related Slab Settlement. Any slab constructed on fill will settle under the weight of the fill. Therefore, post-construction slab settlement should be expected. A properly constructed fill will generally settle between 0.5% and 1.5% if the fill thickness due to its own weight and independent of external loads. That settlement begins as soon as lift placement begins. The time required for settlement to occur is a function of soil, pore water, and drainage path conditions and therefore can vary widely. As a result, fill related settlement should be expected after construction of the slab.

Load Related Slab Settlement. Slabs on grade will settle when subjected to load. Slab settlement is a function of soil type, load intensity, load geometry, and other factors. Upon request by the Structural Engineer for this project, settlement estimates will be provided for the specific loading application in question.

Movement Risk. Recommendations have been provided to mitigate the effects of soil movement. Some soil movement and related structural cracking and floor unevenness should be expected even after following recommendations in this report. The elimination of risk related to soil movement is typically not feasible. We would be happy to discuss other, more expensive, movement-related risk mitigation alternatives upon request.

5.13 Pavement

Recommendations for rigid pavement and preparation of the pavement subgrade are provided in the following sections. A traffic study indicating the number and type of vehicles on which to base the pavement design was not provided. Therefore, our recommendations are based upon our experience with similar projects assuming normal vehicular loading. Any unusual loading conditions should be brought to our attention prior to finalizing the pavement design so that we may assess and modify our recommendations as necessary.

5.13.1 Private Rigid Pavement

Portland cement concrete (PCC) with a minimum 28-day compressive strength of 3,500 pounds per square inch (psi) should be utilized for rigid pavement. Grade 60 reinforcing steel should be utilized in the transverse and longitudinal directions. The following pavement thicknesses and reinforcing are recommended:

Paving Use	Thickness (inches)	Reinforcing
Parking Areas for Automobiles and Light Trucks	5	No. 3 bars spaced on 22-inch intervals
Drive Lanes and Areas Subjected to Light to Medium Trucks	6	No. 3 bars spaced on 18-inch intervals
Areas Receiving Heavy Trucks and Dumpsters	7	No. 3 bars spaced on 16-inch intervals

Alternate Pavement Thickness. Concrete pavement thicknesses provided above can be increased an extra 1-inch (corresponding reinforcing requirements must be changed) as a substitution for stabilization of the pavement subgrade, provided a passing proof-roll is achieved prior to placement of reinforcing steel at the pavement subgrade areas.

Pavement Joints. Contraction joints should be spaced at about 24 times the pavement thickness up to a maximum of 15 feet in any direction. Saw cut control joints should be cut within 6 to 12 hours of concrete placement. Expansion joints should be spaced at locations as necessary and should be placed where the pavement abuts any structure. Dowels should have a diameter equal to $^1/_8$ the slab thickness, be spaced on 12-inch intervals, and be embedded at least 9-inches. Appropriate joint sealant is recommended to keep water from saturating the pavement subgrade and to prevent the introduction of incompressible material into the joints. Routine monitoring and maintenance of joint sealants are recommended. Where not specified herein, concrete pavement should comply with Texas Department of Transportation (TxDOT) Standard Specifications, Item 360, "Concrete Pavement", or local equivalent.

5.13.2 Turn Lane Pavement

Design Parameters: Based on the Harris County Major Thoroughfare and Freeway Plan (MTFP), we understand the exiting Cypress Rosehill Road is classified as Major Thoroughfare. A traffic study indicating the number and type of vehicles on which to base the pavement design was not provided. Any unusual loading conditions should be brought to our attention prior to finalizing the pavement design so that we may assess and modify our recommendations as necessary. The following design parameters were assumed in our analysis for the proposed roadway improvement. The assumed values are based on the AASHTO Pavement Design Guide and our experience with similar projects.

Rigid Pavement Pavement Design Parameters	
Parameter	Value
Pavement Life	20 Years
Native Soils	Sandy Soil
Subgrade Soils	Chemical Stabilized (8 inches)
Initial Serviceability	4.5
Terminal Serviceability	2.25
Reliability	95%
Standard Deviation (Rigid)	0.35
Drainage Coefficient (Rigid) [1]	1.0
Load Transfer Coefficient (Rigid) [2]	3.2
Concrete Compressive Strength	3,500
Concrete Modulus of Elasticity (psi)	3,600,000
Concrete Flexural Strength (psi)	580
Composite Modulus of Subgrade Reaction (k)	200 psi/in
Note: 1. Drainage coefficient for sub-base assumes that fair drainage quality prevails over the life of the pavement and that the pavement structure is exposed 5 to 25 percent of the time to moisture levels approaching saturation.	

Pavement Thickness and Reinforcement: The following pavement thicknesses and reinforcing are recommended. Grade 60 reinforcing steel should be utilized in the transverse and longitudinal directions.

Paving Use/Anticipated Traffic	Equivalent Single Axle Loads (ESALs) [1]	Thickness (inches)	Longitudinal Reinforcing	Transverse Reinforcing
Roadway Receiving up to about 60 Loaded Heavy (18-wheel) Trucks per day Per Direction	1,583,293	8.0	No. 5 bars spaced on 24.75-inch intervals	No. 5 bars spaced on 36-inch intervals
Roadway Receiving up to about 122 Loaded Heavy (18-wheel) Trucks per day Per Direction	3,219,363	9.0	No. 5 bars spaced on 21.75-inch intervals	No. 5 bars spaced on 36-inch intervals
Roadway Receiving up to about 235 Loaded Heavy (18-wheel) Trucks per day Per Direction	6,201,233	10.0	No. 5 bars spaced on 18.5-inch intervals	No. 5 bars spaced on 36-inch intervals
Note: 1. Fully loaded 18-wheel truck with gross weight of 80,000 pounds was assumed in our pavement analysis.				

Pavement Joints. Pavement joints should be designed and constructed per “Harris County Engineering Department Standards”.

Where not specified herein, concrete pavement should comply with "**Harris County Engineering Department Standards*****".***

5.13.3 Gravel RV Parking

Recommended Pavement Section. We recommend the following crushed aggregate pavement sections:

Paving Use	Aggregate Base Thickness Over Geogrid (inches)	Geogrid Specification and Location[1]	Subgrade Treatment
Gravel RV Parking Area	8	Tensar Triax TX5 Geogrid or equivalent Under the Base	Not Required
Gravel RV Parking Area	8	Not Required	6
Note: Proofroll should be performed prior to Geogrid placement .			

The crushed aggregate base should comply with TxDOT Standard Specifications, Item 247, Type D, Grade 1-2 or local equivalent. 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 fill placement.

***Note**: Routine maintenance of the crushed aggregate pavement will be required and often increases the design life significantly. Surface water drainage will be crucial to the performance of pavements. Standard construction practices of providing good surface water drainage should be used. A positive slope of the ground away from any pavement should be provided. Ditches, swales, or French drains should be provided to carry the run-off water both during and after pavement construction.*

5.13.4 Pavement Subgrade

Potential Vertical Soil Movements. We have assumed that site treatment as recommended in Section 5.12 - Slab-on-Grade will not be performed within the pavement areas for this project. As a result, pavements will be subjected to the calculated PVR for this site. Based on the information gathered during this study, a pavement constructed on-grade will be subject to potential vertical movements of about 1-inch. Because heave is generally associated with a source of water, it can occur differentially. Edge lift, excessive cracking, corner breaks, and poor ride quality are just a few of the many examples of pavement issues that can occur when in-situ PVR values are high. We should be contacted to provide PVR mitigation strategies to help reduce potential movements if desired. Strategies available for reducing potential soil movements include soil stabilization with lime or cement, removal of the on-site expansive soils and replacement with select fill or moisture conditioned soils.

Subgrade Preparation. Sandy soil is expected to be encountered or exposed at pavement subgrade. The pavement subgrade should be placed in loose lifts not exceeding 8-inches and should be uniformly compacted to a minimum of 95 percent maximum dry density (per ASTM D-698) and within ±2 percent of the optimum moisture content. We recommend the subgrade be stabilized using either of the following:

Reagent	**Application Rate (pounds per square yard)**	**Application Depth (inches)**
Portland Cement	23	6
70% Flyash/30% Lime Blend	36	6

Cement stabilization should be performed in accordance with TxDOT Standard Specifications, Item 275, "Portland Cement Treated Materials" or local equivalent, and lime-fly ash stabilization should be performed in accordance with TxDOT Standard Specifications, Item 265, "Lime-Fly Ash Treatment of Materials Used as Subgrade" or local equivalent.

5.14 Detention Pond

Soil Conditions. Borings B-04 was drilled in the area of the proposed detention basin. Soil conditions encountered in the vicinity of the proposed detention basin are summarized in the following table.

Generalized Subsurface Conditions at Proposed Detention Pond Location (Borings B-04)			
Nominal Depth, feet bgs (Except as Noted)		General Description	Detailed Description of Soils/Materials Encountered
Top of Layer	Bottom of Layer		
0	2	SAND	SILTY SAND (SM)
2	10	LEAN CLAY	Very stiff to hard SANDY LEAN CLAY (CL)
10	20	SAND	Medium Dense SILTY SAND (SM)
20	25	LEAN CLAY	Hard SANDY LEAN CLAY (CL)
25	30	FAT CLAY	Very stiff FAT CLAY WITH SAND (CH)
Note: Boring Termination Depth = 20 feet bgs.			

Groundwater Conditions. Groundwater was encountered in the piezometers at depths as shallow as 9.1-feet bgs in the proposed detention pond boring B-04.

Recommended Geometry. Based on the subsurface conditions, we recommend that the detention basin slopes be constructed at slopes no steeper than 3H:1V and with a maximum excavation depth not to exceed 10-feet bgs.

Excavated Soil Usage. The borings indicate that the excavated soil from the detention basin will not generally meet the requirements for select fill but will generally meet the requirements for general fill. For the clays encountered, we expect that the addition of between 4- and 6-percent lime would reduce the soil plasticity to that required for use as select fill. We recommend that a lime-series determination test be performed to determine the required amount of lime.

6.0 General Comments

Data Assumptions. By necessity, geotechnical engineering design recommendations are based on a limited amount of information about subsurface conditions. In the analysis, the geotechnical engineer must assume subsurface conditions are similar to those encountered in the borings. The analyses, conclusions and recommendations contained in this report are based on site conditions as they existed at the time of the field study and 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. As a result, estimated movements provided in this study are not guarantees of performance. Actual movements may be more or less than estimates provided in this study.

Subsurface Anomalies. Anomalies in subsurface conditions are often revealed during construction. 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.

Change of Conditions. If there is a substantial lapse of time between submission of this report 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 should be promptly informed and retained to review our report to determine the applicability of the conclusions and recommendations, considering the changed conditions and/or time lapse.

Design Review. Recommendations in our report are contingent upon RINER reviewing and approving in writing the following design items prior to construction:

- Site grading plan, and
- Foundation plan, details and related structural loads.

Construction Materials Testing and Inspection. RINER should be retained to observe earthwork and foundation installation and perform materials evaluation and testing during the construction phase of the project. This enables RINER's 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. It is proposed that construction phase observation and materials testing commence by the project geotechnical engineer (RINER) at the outset of the project. Experience has shown that the most suitable method for procuring these services is for the owner to contact 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.

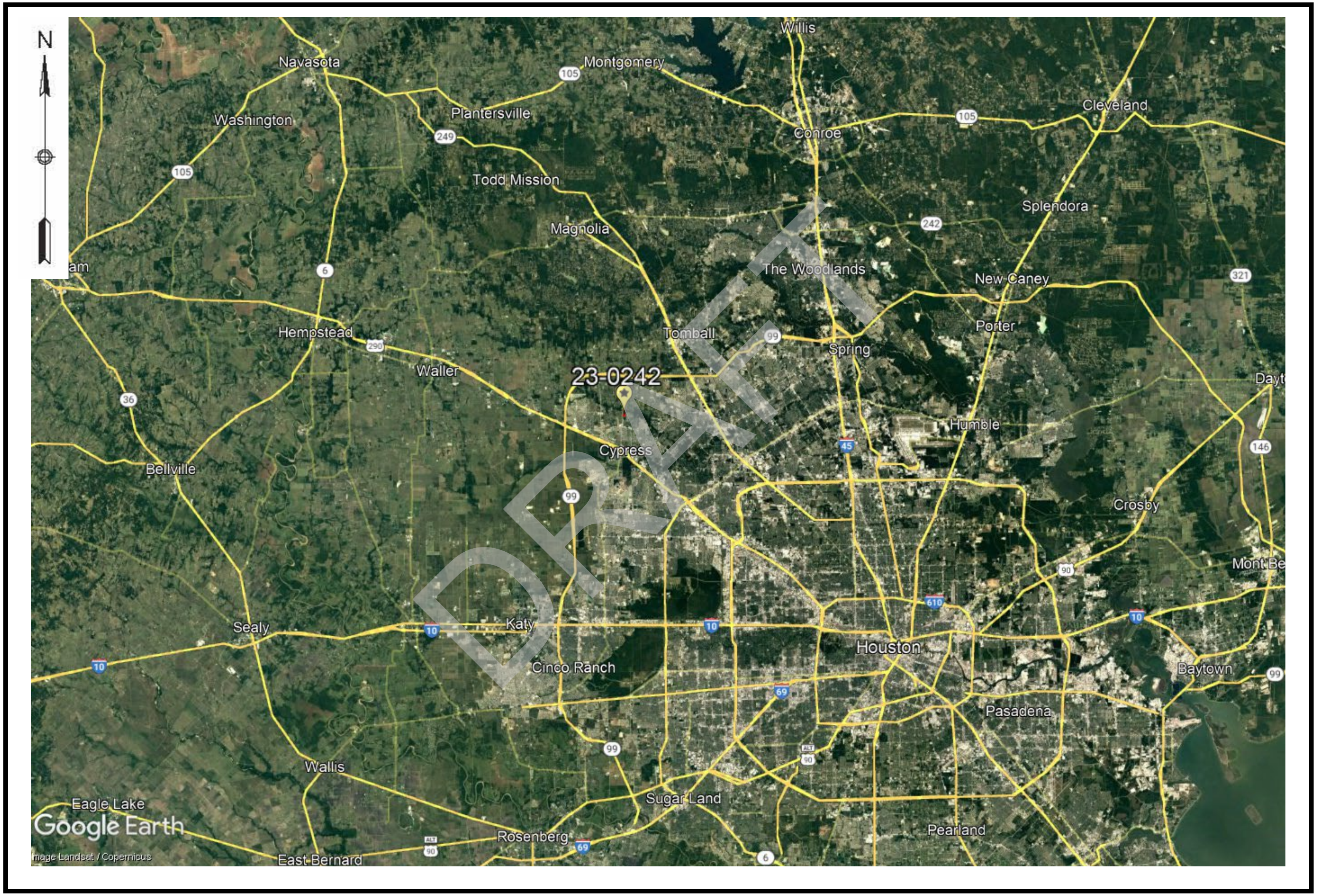
<u>Report Recommendations are Preliminary</u>. Until the recommended construction phase services are performed by RINER, the recommendations contained in this report on such items as final foundation bearing elevations, final depth of undercut of expansive soils for non-expansive earth fill pads and other such subsurface-related recommendations should be considered as preliminary.

<u>Liability Limitation</u>. RINER cannot assume responsibility or liability for recommendations provided in this report if construction inspection and/or testing recommended herein is performed by another party.

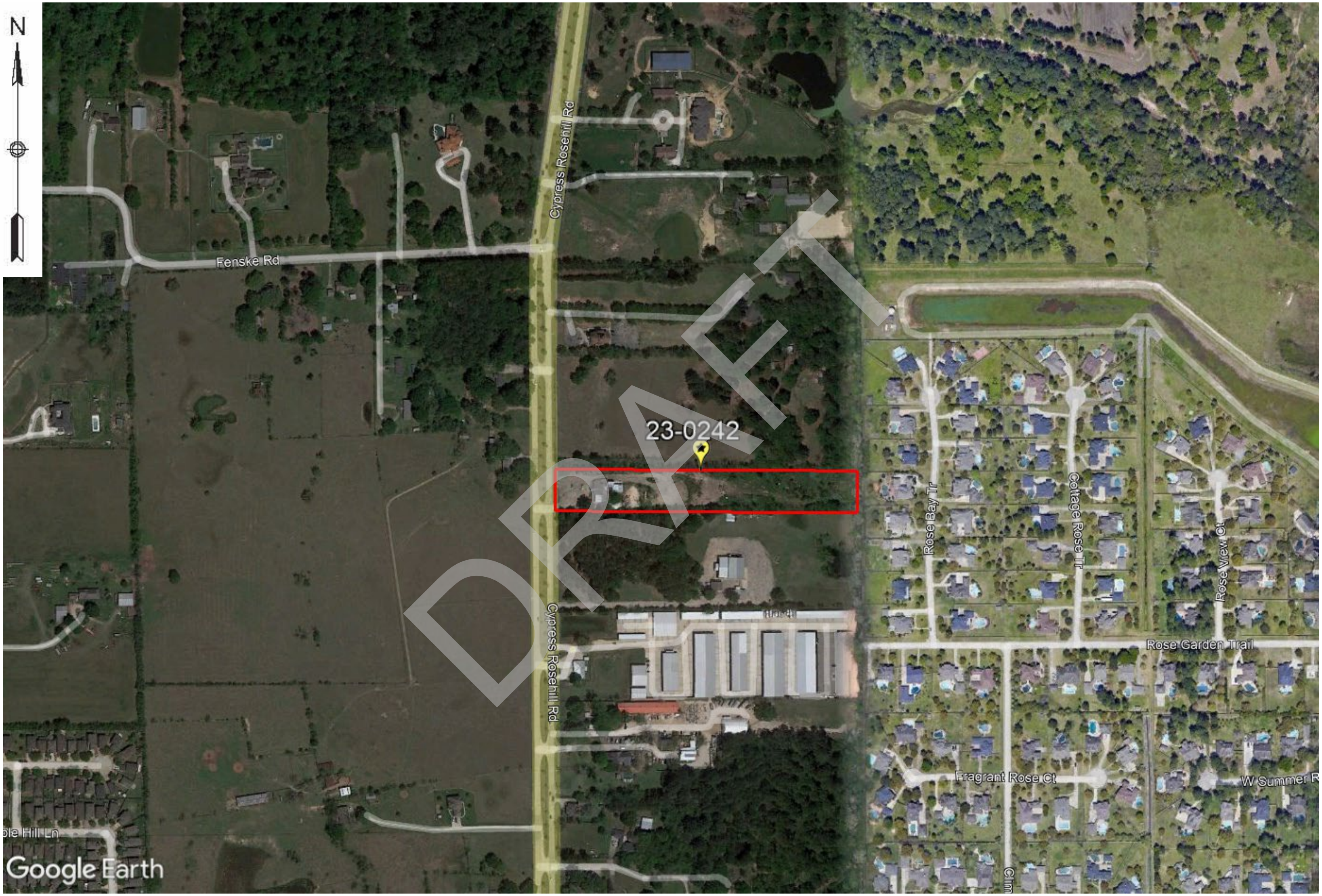
<u>Warranty</u>. This report has been prepared for the exclusive use of the Client and their designated agents for specific application to design of this project. We have used that degree of care and skill ordinarily exercised under similar conditions by reputable members of our profession practicing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

Appendix A - Project Location Diagrams

PROJECT LOCATION DIAGRAM - GENERAL

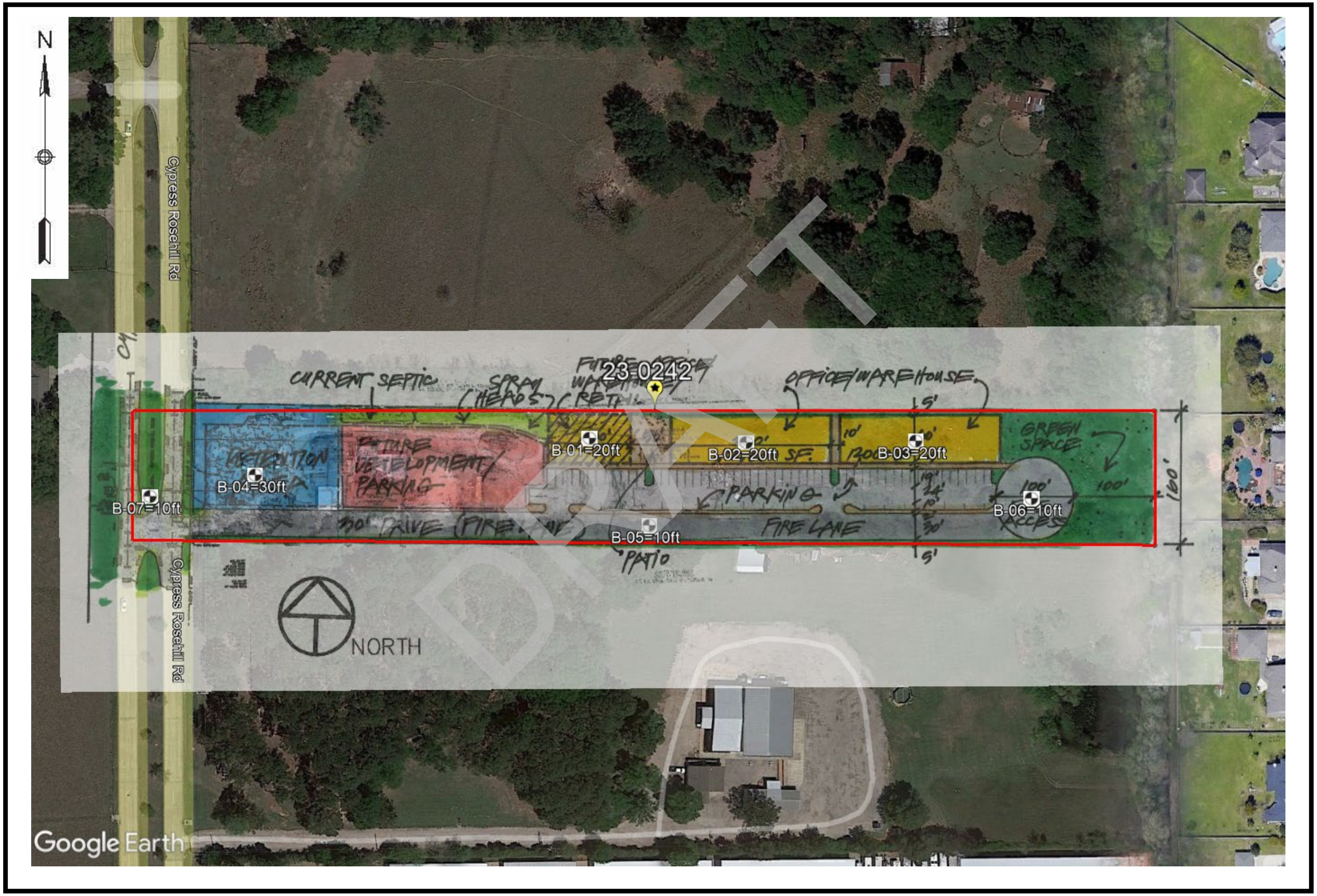


PROJECT LOCATION DIAGRAM - LOCAL



Appendix B - Boring Location Diagram

BORING LOCATION DIAGRAM W Overlay



RINER Project No. 23-0242 Cypress Rosehill Business Park

Appendix C - Boring Logs and Laboratory Results

Riner Engineering, Inc.
4641 Kennedy Commerce Drive, Houston, TX 77032
Telephone: 281-469-3347; Fax: 281-469.3594

BORING NUMBER B-01

CLIENT Project Montana, LLC
PROJECT NAME Cypress Rosehill Business Park
PROJECT NUMBER 23-0242
PROJECT LOCATION Cypress, Texas
DATE STARTED 5/8/23 **COMPLETED** 5/8/23
GROUND ELEVATION **NORTHING**
CONTRACTOR Diamond Drilling
GROUND WATER LEVELS: **EASTING**
METHOD Auger 0 - 20 feet
INITIALLY ENCOUNTERED Not Encountered
LOGGED BY A.P. **CHECKED BY** A.M.
AFTER 15 MIN. Not Measured
NOTES
AFTER ---

DEPTH (ft)	GRAPHIC LOG	MATERIAL DESCRIPTION	SAMPLE TYPE NUMBER	RECOVERY % (RQD)	BLOW COUNTS (N VALUE)	POCKET PEN. (tsf)	TORVANE (tsf)	Compressive Strength (tsf)	Confining Pressure (psi)	DRY UNIT WT. (pcf)	MOISTURE CONTENT (%)	ATTERBERG LIMITS: LIQUID LIMIT	ATTERBERG LIMITS: PLASTIC LIMIT	ATTERBERG LIMITS: PLASTICITY INDEX	FINES CONTENT (%)
0		SILTY CLAY (CL-ML) FILL - Hard, tan, and with roots from 0 to 4 feet.	ST			4.50+	1.4				10	15	11	4	53
		SANDY LEAN CLAY (CL) - Stiff to hard, light gray, and tan, with sand seams from 4 to 8 feet.	ST			3.00	1.5								
5			ST			3.00	1.6				15				
			ST			4.50+	1.9								
			ST			4.00	1.6	1.6		117	17				
10		SILTY SAND (SM) - Medium dense, and tan													
15			ST												
		Tan, and orange tan from 18 to 20 feet.	SS		7-11-12 (23)						16				
20															

Bottom of hole at 20.0 feet.

DRAFT

Riner Engineering, Inc.
4641 Kennedy Commerce Drive, Houston, TX 77032
Telephone: 281-469-3347; Fax: 281-469.3594

BORING NUMBER B-02

CLIENT Project Montana, LLC
PROJECT NAME Cypress Rosehill Business Park
PROJECT NUMBER 23-0242
PROJECT LOCATION Cypress, Texas
DATE STARTED 5/8/23 **COMPLETED** 5/8/23
GROUND ELEVATION **NORTHING**
CONTRACTOR Diamond Drilling
GROUND WATER LEVELS: **EASTING**
METHOD Auger 0 - 20 feet
INITIALLY ENCOUNTERED Not Encountered
LOGGED BY A.P. **CHECKED BY** A.M.
AFTER 15 MIN. Not Measured
NOTES
AFTER ---

DEPTH (ft)	GRAPHIC LOG	MATERIAL DESCRIPTION	SAMPLE TYPE NUMBER	RECOVERY % (RQD)	BLOW COUNTS (N VALUE)	POCKET PEN. (tsf)	TORVANE (tsf)	Compressive Strength (tsf)	Confining Pressure (psi)	DRY UNIT WT. (pcf)	MOISTURE CONTENT (%)	ATTERBERG LIMITS LIQUID LIMIT	ATTERBERG LIMITS PLASTIC LIMIT	ATTERBERG LIMITS PLASTICITY INDEX	FINES CONTENT (%)
0		SILTY SAND (SM) FILL- Brown, and with roots from 0 to 2 feet.	ST												
		SILTY CLAY (CL-ML) - Stiff, light gray, and light brown.	ST			3.00	1.5				17	22	18	4	57
5		LEAN CLAY WITH SAND (CL) / SANDY LEAN CLAY (CL) - Very stiff to hard, tan, and with iron nodules from 4 to 6 feet.	ST			4.00	1.4				17				
		Tan, and reddish brown from 6 to 8 feet	ST			4.50+	1.8								
		Tan, and yellowish tan from 8 to 10 feet.	ST			4.50+	1.5				13				
10		SILTY SAND (SM) - Medium dense, tan, with clay pockets from 13 to 20 feet.													
15			ST								15				
		Tan, and yellowish tan from 18 to 20 feet.	SS		4-7-7 (14)										
20															

Bottom of hole at 20.0 feet.

Riner Engineering, Inc.
4641 Kennedy Commerce Drive, Houston, TX 77032
Telephone: 281-469-3347; Fax: 281-469.3594

BORING NUMBER B-03

CLIENT Project Montana, LLC
PROJECT NAME Cypress Rosehill Business Park
PROJECT NUMBER 23-0242
PROJECT LOCATION Cypress, Texas
DATE STARTED 5/8/23 **COMPLETED** 5/8/23
GROUND ELEVATION **NORTHING**
CONTRACTOR Diamond Drilling
GROUND WATER LEVELS: **EASTING**
METHOD Auger 0 - 20 feet
INITIALLY ENCOUNTERED Not Encountered
LOGGED BY A.P. **CHECKED BY** A.M.
AFTER 15 MIN. Not Measured
NOTES
AFTER ---

DEPTH (ft)	GRAPHIC LOG	MATERIAL DESCRIPTION	SAMPLE TYPE NUMBER	RECOVERY % (RQD)	BLOW COUNTS (N VALUE)	POCKET PEN. (tsf)	TORVANE (tsf)	Compressive Strength (tsf)	Confining Pressure (psi)	DRY UNIT WT. (pcf)	MOISTURE CONTENT (%)	ATTERBERG LIMITS: LIQUID LIMIT	ATTERBERG LIMITS: PLASTIC LIMIT	ATTERBERG LIMITS: PLASTICITY INDEX	FINES CONTENT (%)
0		SILTY SAND (SM) FILL - Brown, and with roots from 0 to 2 feet.	ST								9				
		SANDY LEAN CLAY (CL) / LEAN CLAY WITH SAND (CL) - Stiff to hard, light gray, tan, and reddish brown.	ST			3.00	1.4								
5		With iron noduiles from 4 to 8 feet.	ST			4.00	1.1				15	49	15	34	50
			ST			4.50+	1.5								
10			ST			4.50+	1.5	2.4		124	13				
		SILTY SAND (SM) - Medium dense, tan, and yellowish tan.													
15			ST								16				
		Light gray from 18 to 20 feet.	SS		5-9-9 (18)										
20															

Bottom of hole at 20.0 feet.

Riner Engineering, Inc.
4641 Kennedy Commerce Drive, Houston, TX 77032
Telephone: 281-469-3347; Fax: 281-469.3594

BORING NUMBER B-04

CLIENT Project Montana, LLC
PROJECT NAME Cypress Rosehill Business Park
PROJECT NUMBER 23-0242
PROJECT LOCATION Cypress, Texas
DATE STARTED 5/8/23 **COMPLETED** 5/8/23
GROUND ELEVATION **NORTHING**
CONTRACTOR Diamond Drilling
GROUND WATER LEVELS: **EASTING**
METHOD Auger 0 to 20 feet, Rotary Wash 20 to 30 feet.
▽ **INITIALLY ENCOUNTERED** 18.0 ft
LOGGED BY A.P. **CHECKED BY** A.M.
▼ **AFTER 15 MIN.** 13.1 ft
NOTES
AFTER ---

DEPTH (ft)	MATERIAL DESCRIPTION	SAMPLE TYPE NUMBER	RECOVERY % (RQD)	BLOW COUNTS (N VALUE)	POCKET PEN. (tsf)	TORVANE (tsf)	Compressive Strength (tsf)	Confining Pressure (psi)	DRY UNIT WT. (pcf)	MOISTURE CONTENT (%)	ATTERBERG LIMITS: LIQUID LIMIT	ATTERBERG LIMITS: PLASTIC LIMIT	ATTERBERG LIMITS: PLASTICITY INDEX	FINES CONTENT (%)
0	SILTY SAND (SM) FILL - Brown.	ST								12				
	SANDY LEAN CLAY (CL) - Very stiff to hard, light brown, and gray, with sand seams from 2 to 8 feet.	ST			4.00	1.5								
5		ST			4.50+	1.5				14	48	18	30	53
	Tan, and yellowish tan from 6 to 10 feet, with iron seams from 6 to 8 feet.	ST			4.50+	1.6				13				
		ST			4.50+	1.8				13	34	13	21	55
10	SILTY SAND (SM) - Medium Dense, light gray, tan, and yellowish tan.													
15		ST												
		SS		3-8-10 (18)						19				
20	SANDY LEAN CLAY (CL) - Hard, light gray, tan, and yellowish tan.													
25		ST			4.50+	1.9								
	FAT CLAY WITH SAND (CH) - Very stiff, light gray, tan, and reddish brown, with sand seams from 28 to 30 feet.													
30		ST			4.00	1.9				23				

Bottom of hole at 30.0 feet.

Riner Engineering, Inc.
4641 Kennedy Commerce Drive, Houston, TX 77032
Telephone: 281-469-3347; Fax: 281-469.3594

BORING NUMBER B-05

CLIENT Project Montana, LLC
PROJECT NAME Cypress Rosehill Business Park
PROJECT NUMBER 23-0242
PROJECT LOCATION Cypress, Texas
DATE STARTED 5/8/23 **COMPLETED** 5/8/23
GROUND ELEVATION **NORTHING**
CONTRACTOR Diamond Drilling
GROUND WATER LEVELS: **EASTING**
METHOD Auger 0 - 10 feet
INITIALLY ENCOUNTERED Not Encountered
LOGGED BY A.P. **CHECKED BY** A.M.
AFTER 15 MIN. Not Measured
NOTES
AFTER ---

DEPTH (ft)	GRAPHIC LOG	MATERIAL DESCRIPTION	SAMPLE TYPE NUMBER	RECOVERY % (RQD)	BLOW COUNTS (N VALUE)	POCKET PEN. (tsf)	TORVANE (tsf)	Compressive Strength (tsf)	Confining Pressure (psi)	DRY UNIT WT. (pcf)	MOISTURE CONTENT (%)	ATTERBERG LIMITS LIQUID LIMIT	PLASTIC LIMIT	PLASTICITY INDEX	FINES CONTENT (%)
0.0		CLAYEY SAND (SC) FILL - Very stiff, with roots from 0 to 2 feet.	ST			3.50	1.4				11	21	11	10	48
2.5		SANDY LEAN CLAY (CL) FILL - Hard, light gray, tan, and reddish brown, with sand seams from 2 to 4 feet.	ST			4.50+	1.5								
5.0		SANDY LEAN CLAY (CL / LEAN CLAY WITH SAND (CL) - Hard, light gray, tan, and reddish brown, with iron nodles from 4 to 6 feet.	ST			4.50+	1.5				17				
7.5			ST			4.50+	1.5								
10.0			ST			4.50+	1.5				13				

Bottom of hole at 10.0 feet.

Riner Engineering, Inc.
4641 Kennedy Commerce Drive, Houston, TX 77032
Telephone: 281-469-3347; Fax: 281-469.3594

BORING NUMBER B-06

CLIENT Project Montana, LLC
PROJECT NAME Cypress Rosehill Business Park
PROJECT NUMBER 23-0242
PROJECT LOCATION Cypress, Texas
DATE STARTED 5/8/23 **COMPLETED** 5/8/23
GROUND ELEVATION **NORTHING**
CONTRACTOR Diamond Drilling
GROUND WATER LEVELS: **EASTING**
METHOD Auger 0 - 10 feet
INITIALLY ENCOUNTERED Not Encountered
LOGGED BY A.P. **CHECKED BY** A.M.
AFTER 15 MIN. Not Measured
NOTES
AFTER ---

DEPTH (ft)	GRAPHIC LOG	MATERIAL DESCRIPTION	SAMPLE TYPE NUMBER	RECOVERY % (RQD)	BLOW COUNTS (N VALUE)	POCKET PEN. (tsf)	TORVANE (tsf)	Compressive Strength (tsf)	Confining Pressure (psi)	DRY UNIT WT. (pcf)	MOISTURE CONTENT (%)	ATTERBERG LIMITS: LIQUID LIMIT	ATTERBERG LIMITS: PLASTIC LIMIT	ATTERBERG LIMITS: PLASTICITY INDEX	FINES CONTENT (%)
0.0		SILTY SAND (SM) FILL - Brown, and with roots from 0 to 2 feet.	ST								13				40
2.5		SANDY LEAN CLAY (CL) / LEAN CLAY WITH SAND (CL) - Stiff to hard, light gray, tan, and reddish brown, with iron nodules from 2 to 4 feet, and sand seams from 2 to 8 feet.	ST			3.00	1.8								
5.0			ST			4.50+	1.4				14	35	14	21	
7.5			ST			4.00	1.6								
10.0			ST			4.50+	1.6				13				

Bottom of hole at 10.0 feet.

Riner Engineering, Inc.
4641 Kennedy Commerce Drive, Houston, TX 77032
Telephone: 281-469-3347; Fax: 281-469.3594

BORING NUMBER B-07

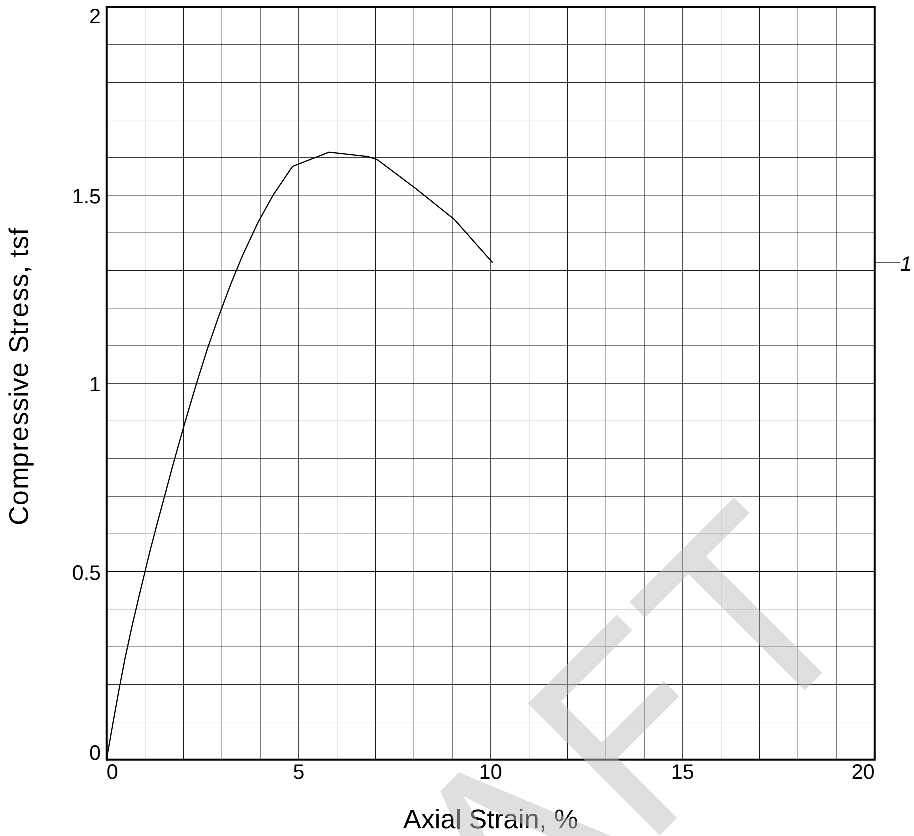
CLIENT Project Montana, LLC
PROJECT NAME Cypress Rosehill Business Park
PROJECT NUMBER 23-0242
PROJECT LOCATION Cypress, Texas
DATE STARTED 5/8/23 **COMPLETED** 5/8/23
GROUND ELEVATION **NORTHING**
CONTRACTOR Diamond Drilling
GROUND WATER LEVELS: **EASTING**
METHOD Auger 0 - 10 feet
INITIALLY ENCOUNTERED Not Encountered
LOGGED BY A.P. **CHECKED BY** A.M.
AFTER 15 MIN. Not Measured
NOTES
AFTER ---

DEPTH (ft)	GRAPHIC LOG	MATERIAL DESCRIPTION	SAMPLE TYPE NUMBER	RECOVERY % (RQD)	BLOW COUNTS (N VALUE)	POCKET PEN. (tsf)	TORVANE (tsf)	Compressive Strength (tsf)	Confining Pressure (psi)	DRY UNIT WT. (pcf)	MOISTURE CONTENT (%)	ATTERBERG LIMITS LIQUID LIMIT	ATTERBERG LIMITS PLASTIC LIMIT	ATTERBERG LIMITS PLASTICITY INDEX	FINES CONTENT (%)
0.0		SANDY LEAN CLAY (CL) FILL - Hard, light brown, and light gray, with roots from 0 to 2 feet.	ST			4.50+	1.3				13	32	12	20	51
2.5		With sand seams from 2 to 6 feet.	ST			4.50+	1.8								
5.0			ST			4.50+	1.3				16				
		SANDY LEAN CLAY (CL) - Very stiff to hard, tan, and reddish brown, with sand seams from 6 to 8 feet	ST			4.50+	1.6								
7.5		With iron nodules from 8 to 10 feet.	ST			3.50	1.4				14				
10.0															

Bottom of hole at 10.0 feet.

ABSORPTION SWELL TEST (ASTM D4546) RESULTS						
Boring No.	B-01	B-02	B-03	B-04		
Average Sample Depth (ft)	1	3	5	5		
Sample Height (in)	0.8	0.8	0.8	0.8		
Sample Diameter (in)	2.5	2.5	2.5	2.5		
Initial Sample Volume (cu in)	3.93	3.93	3.93	3.93		
Initial Sample Weight (gr)	128.5	131.7	127.2	141.5		
Initial Moisture (%)	12	18	16	13		
Final Moisture (%)	13	19	17	16		
Initial Wet Unit Weight (pcf)	125	128	123	137		
Initial Dry Unit Weight (pcf)	112	108	106	121		
Applied Over Burden (psi)	0.9	2.6	4.3	4.3		
Initial Dial Reading (in)	0.0149	0.0170	0.0120	0.0218		
Final Dial Reading (in)	0.0135	0.0195	0.0123	0.0218		
Swell (%)	0.00	0.31	0.04	0.00		

UNCONFINED COMPRESSION TEST



Sample No.	1			
Unconfined strength, tsf	1.6145			
Undrained shear strength, tsf	0.8073			
Failure strain, %	5.8			
Strain rate, %/min.	1.00			
Water content, %	16.6			
Wet density, pcf	135.9			
Dry density, pcf	116.6			
Saturation, %	98.6			
Void ratio	0.4566			
Specimen diameter, in.	2.78			
Specimen height, in.	5.75			
Height/diameter ratio	2.07			

Description: Light gray and tan SANDY LEAN CLAY (CL)

LL =	PL =	PI =	Assumed GS= 2.72	Type: Shelby Tube

Project No.: 23-0242
Date Sampled: 05/08/2023
Remarks:

Figure ________

Client: Project Montana, LLC

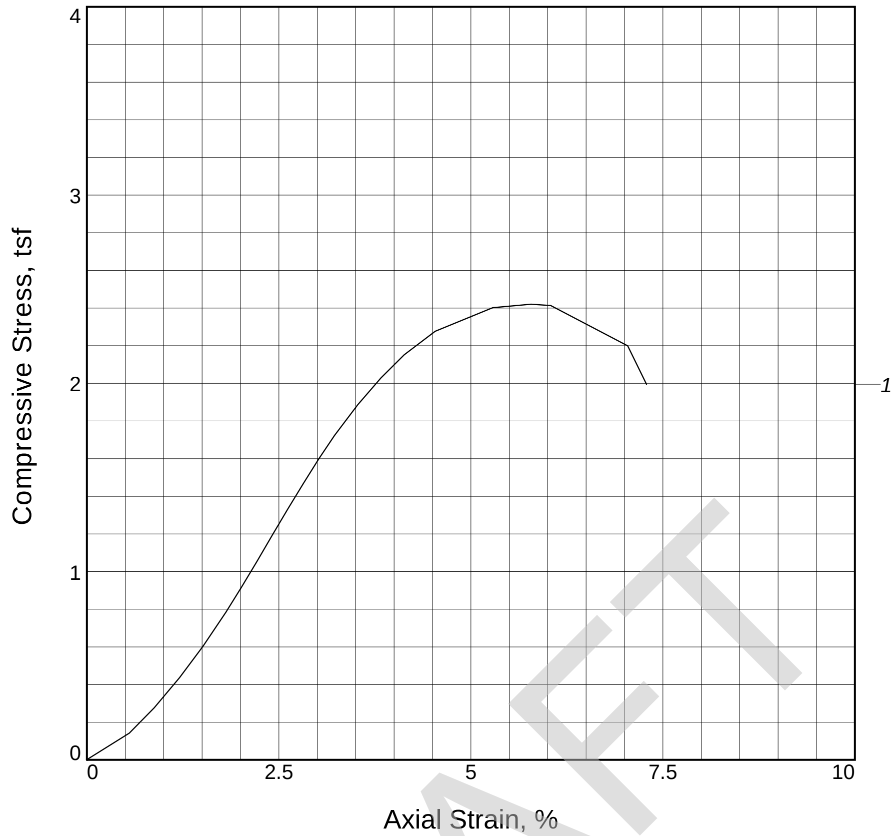
Project: Cypress Rosehill Business Park

Location: Boring B-01
Sample Number: 5 **Depth:** 8'-10'

UNCONFINED COMPRESSION TEST
Riner Engineering, Inc.
Houston, Texas

Tested By: A.C. **Checked By:** A.M.

UNCONFINED COMPRESSION TEST



Sample No.	1			
Unconfined strength, tsf	2.4204			
Undrained shear strength, tsf	1.2102			
Failure strain, %	5.8			
Strain rate, %/min.	1.00			
Water content, %	13.5			
Wet density, pcf	140.9			
Dry density, pcf	124.2			
Saturation, %	99.7			
Void ratio	0.3671			
Specimen diameter, in.	2.74			
Specimen height, in.	5.75			
Height/diameter ratio	2.10			

Description: Light gray, tan, and reddish brown SANDY LEAN CLAY (CL)

LL =	**PL =**	**PI =**	**Assumed GS=** 2.72	**Type:** Shelby Tube

Project No.: 23-0242
Date Sampled: 05/08/2023
Remarks:

Figure _______

Client: Project Montana, LLC

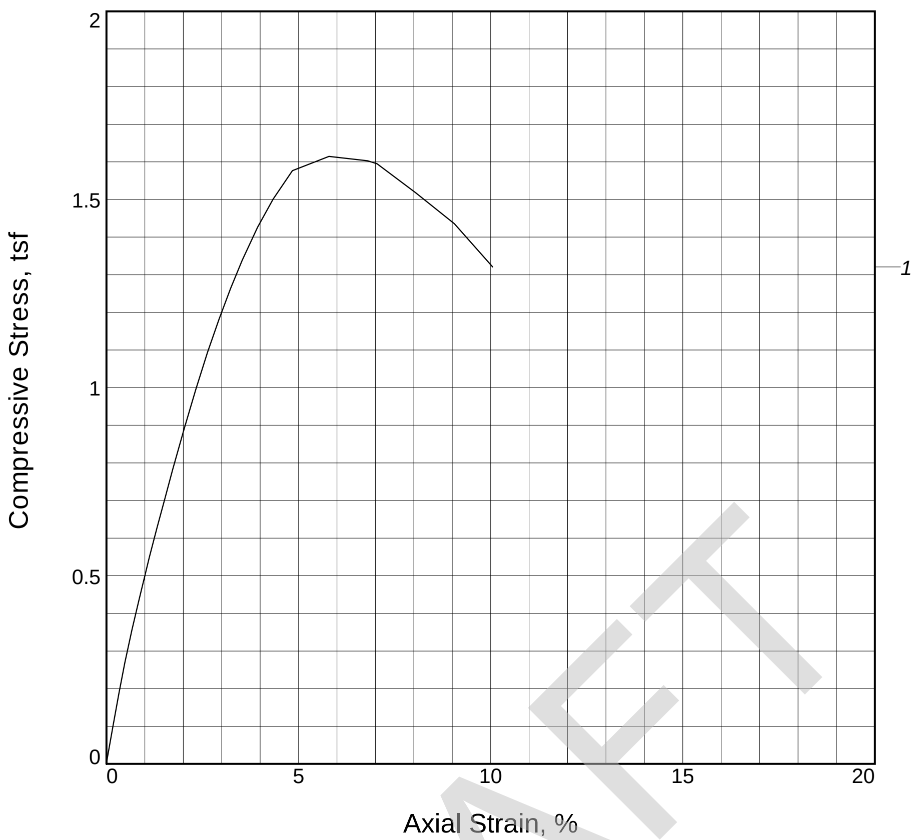
Project: Cypress Rosehill Business Park

Location: Boring B-03
Sample Number: 5 **Depth:** 8'-10'

UNCONFINED COMPRESSION TEST
Riner Engineering, Inc.
Houston, Texas

Tested By: A.C. **Checked By:** A.M.

UNCONFINED COMPRESSION TEST



Sample No.	1			
Unconfined strength, tsf	1.6145			
Undrained shear strength, tsf	0.8073			
Failure strain, %	5.8			
Strain rate, %/min.	1.00			
Water content, %	16.6			
Wet density, pcf	135.9			
Dry density, pcf	116.6			
Saturation, %	98.6			
Void ratio	0.4566			
Specimen diameter, in.	2.78			
Specimen height, in.	5.75			
Height/diameter ratio	2.07			

Description: Light gray and tan SANDY LEAN CLAY (CL)

LL =	PL =	PI =	Assumed GS= 2.72	Type: Shelby Tube

Project No.: 23-0242
Date Sampled: 05/08/2023
Remarks:

Figure _______

Client: Project Montana, LLC

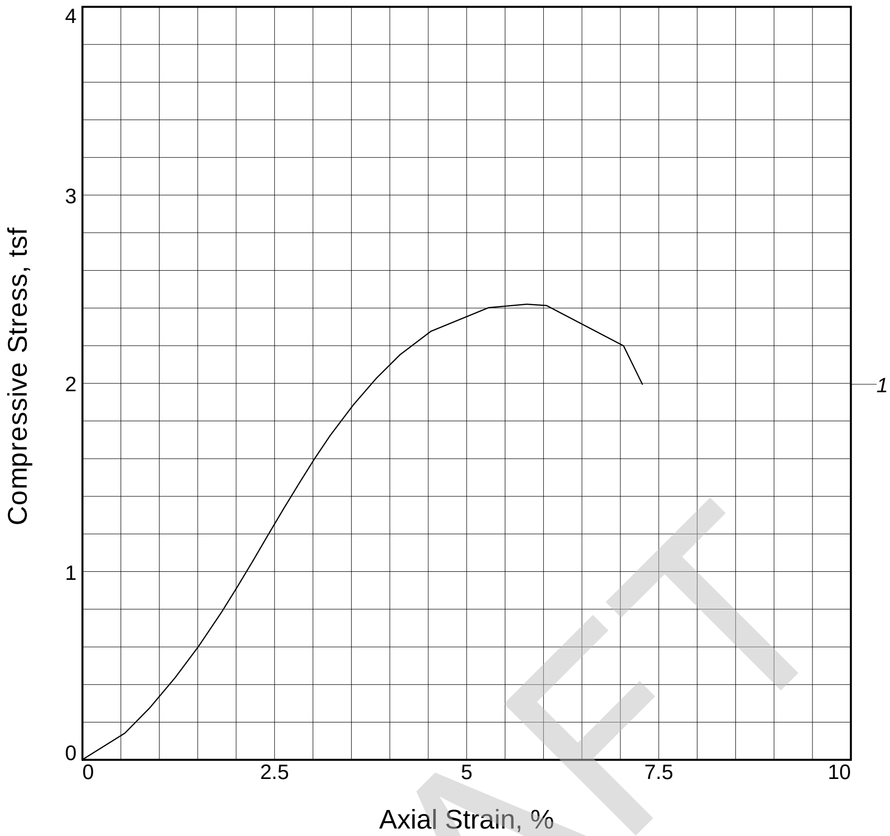
Project: Cypress Rosehill Business Park

Location: Boring B-01
Sample Number: 5 **Depth:** 8'-10'

UNCONFINED COMPRESSION TEST
Riner Engineering, Inc.
Houston, Texas

Tested By: A.C. **Checked By:** A.M.

UNCONFINED COMPRESSION TEST



Sample No.	1			
Unconfined strength, tsf	2.4204			
Undrained shear strength, tsf	1.2102			
Failure strain, %	5.8			
Strain rate, %/min.	1.00			
Water content, %	13.5			
Wet density, pcf	140.9			
Dry density, pcf	124.2			
Saturation, %	99.7			
Void ratio	0.3671			
Specimen diameter, in.	2.74			
Specimen height, in.	5.75			
Height/diameter ratio	2.10			

Description: Light gray, tan, and reddish brown SANDY LEAN CLAY (CL)

LL =	**PL =**	**PI =**	**Assumed GS=** 2.72	**Type:** Shelby Tube

Project No.: 23-0242
Date Sampled: 05/08/2023
Remarks:

Figure _______

Client: Project Montana, LLC

Project: Cypress Rosehill Business Park

Location: Boring B-03
Sample Number: 5 **Depth:** 8'-10'

UNCONFINED COMPRESSION TEST
Riner Engineering, Inc.
Houston, Texas

Tested By: A.C. **Checked By:** A.M.

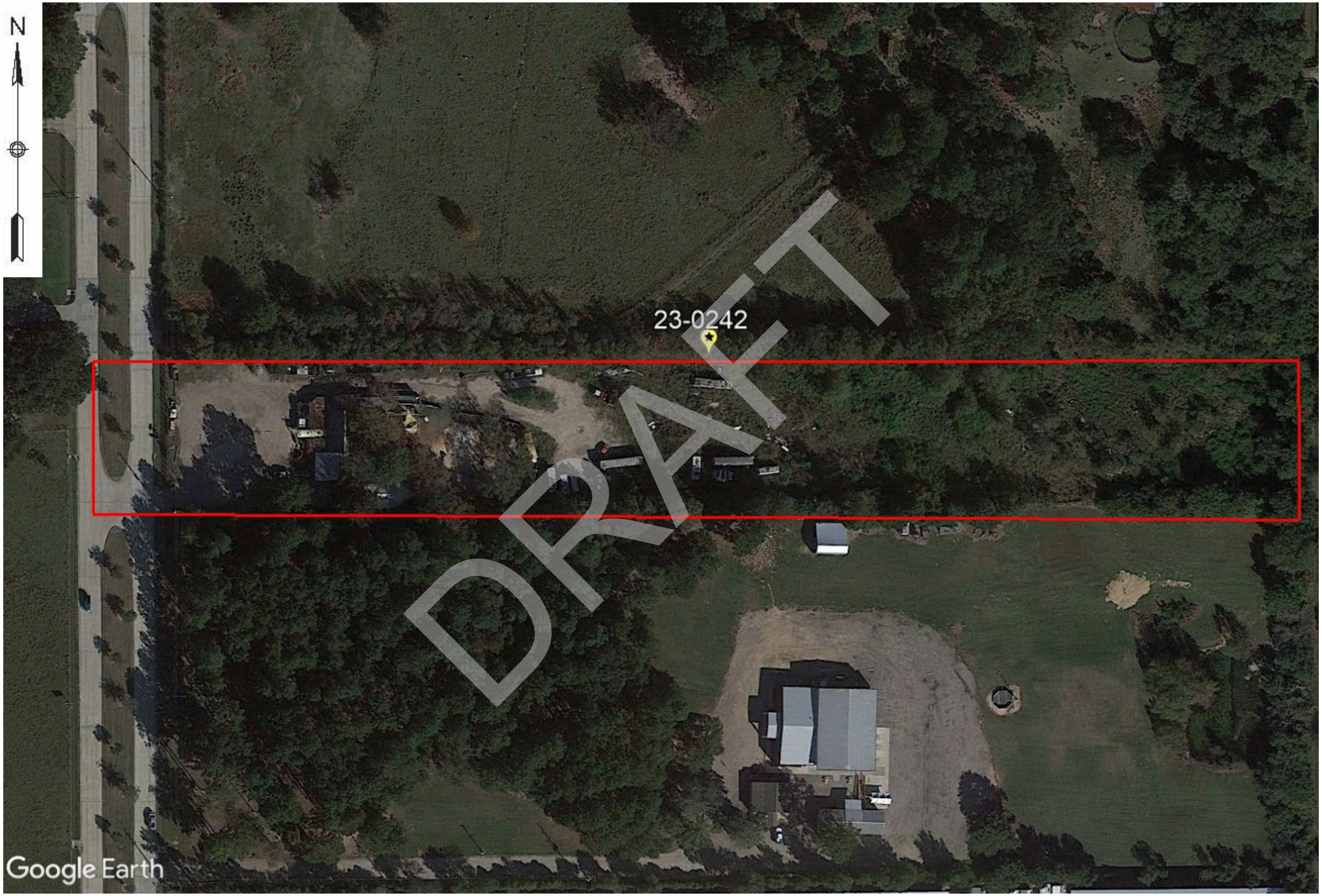
Appendix D - Aerial Photographs

AERIAL PHOTOGRAPH - 2022

RINER Project No. 23-0242

Cypress Rosehill Business Park

AERIAL PHOTOGRAPH - 2020



RINER Project No. 23-0242 Cypress Rosehill Business Park

AERIAL PHOTOGRAPH - 2019

AERIAL PHOTOGRAPH - 2017

AERIAL PHOTOGRAPH - 2016

RINER Project No. 23-0242 Cypress Rosehill Business Park

AERIAL PHOTOGRAPH - 2015

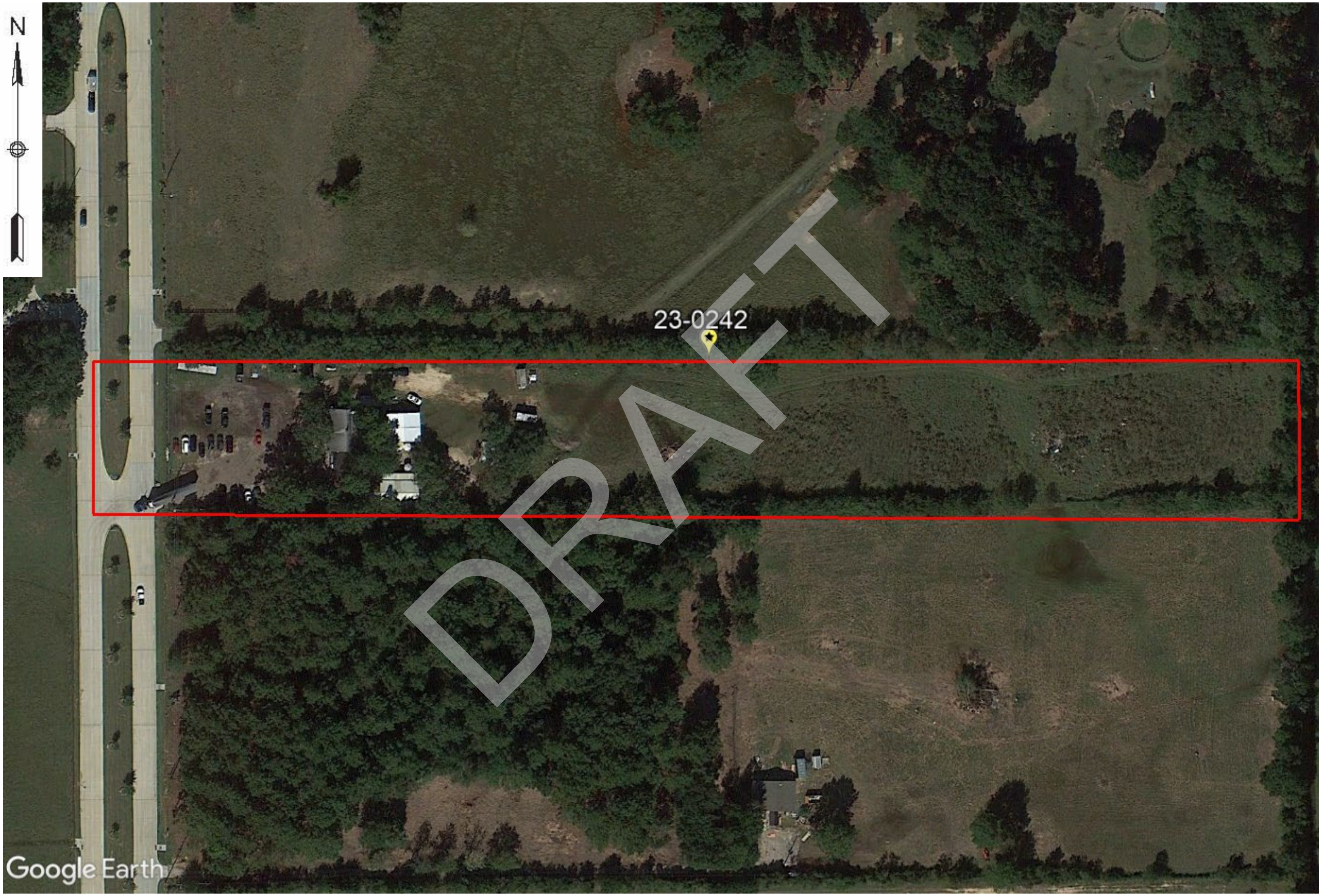
RINER Project No. 23-0242

Cypress Rosehill Business Park

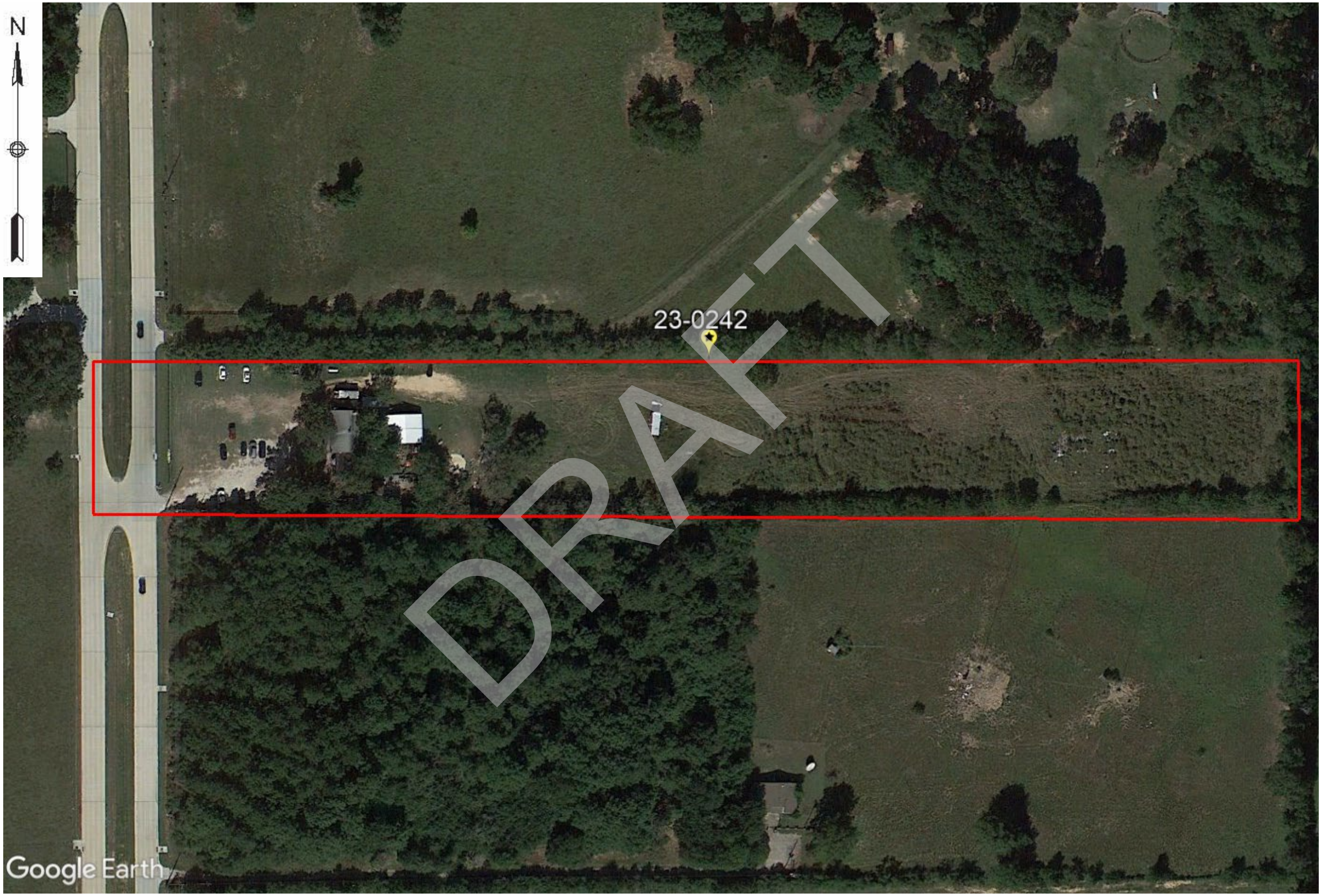
AERIAL PHOTOGRAPH - 2014

RINER Project No. 23-0242 Cypress Rosehill Business Park

AERIAL PHOTOGRAPH - 2013



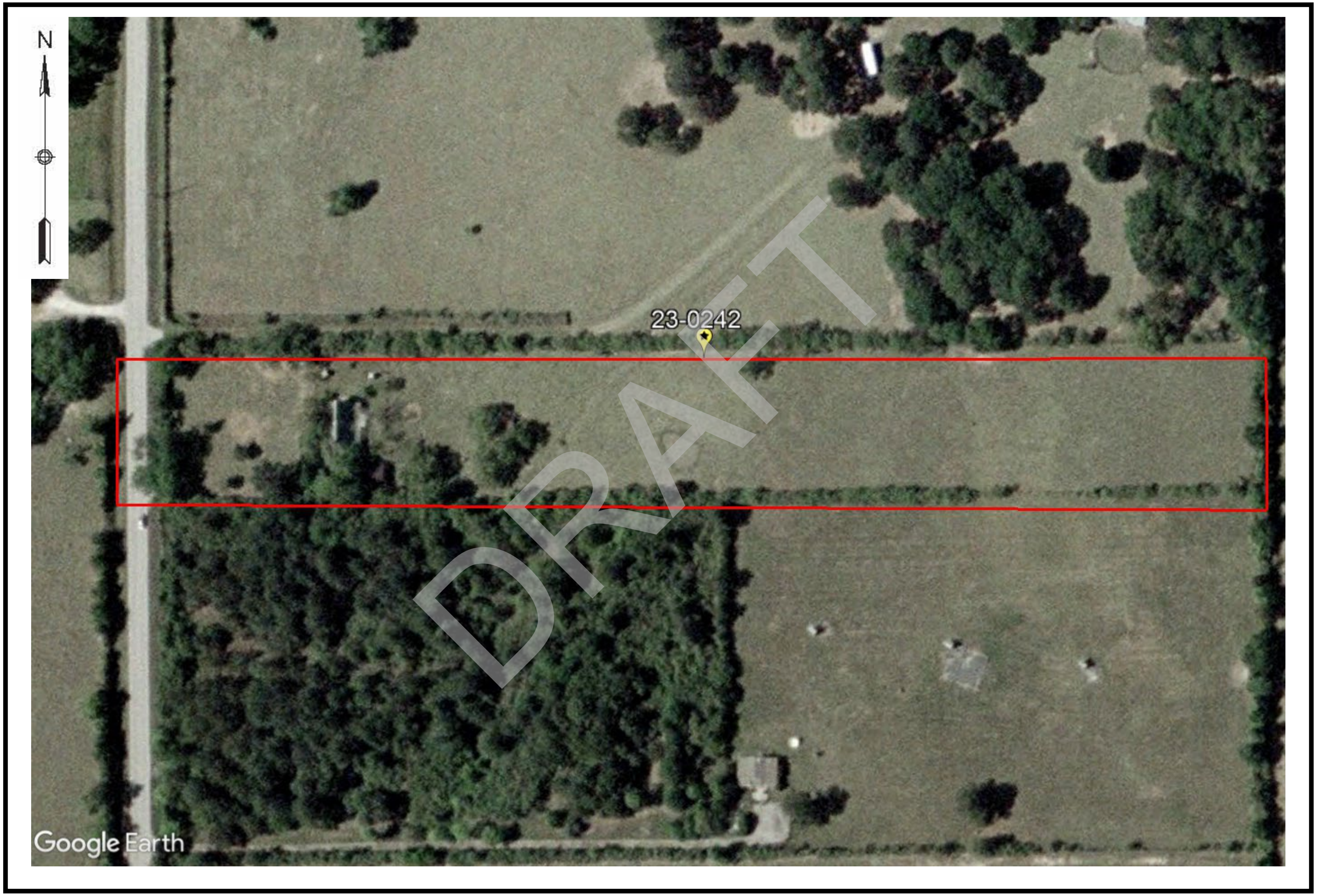
AERIAL PHOTOGRAPH - 2012



AERIAL PHOTOGRAPH - 2011

AERIAL PHOTOGRAPH - 2010

AERIAL PHOTOGRAPH - 2006



RINER Project No. 23-0242 Cypress Rosehill Business Park

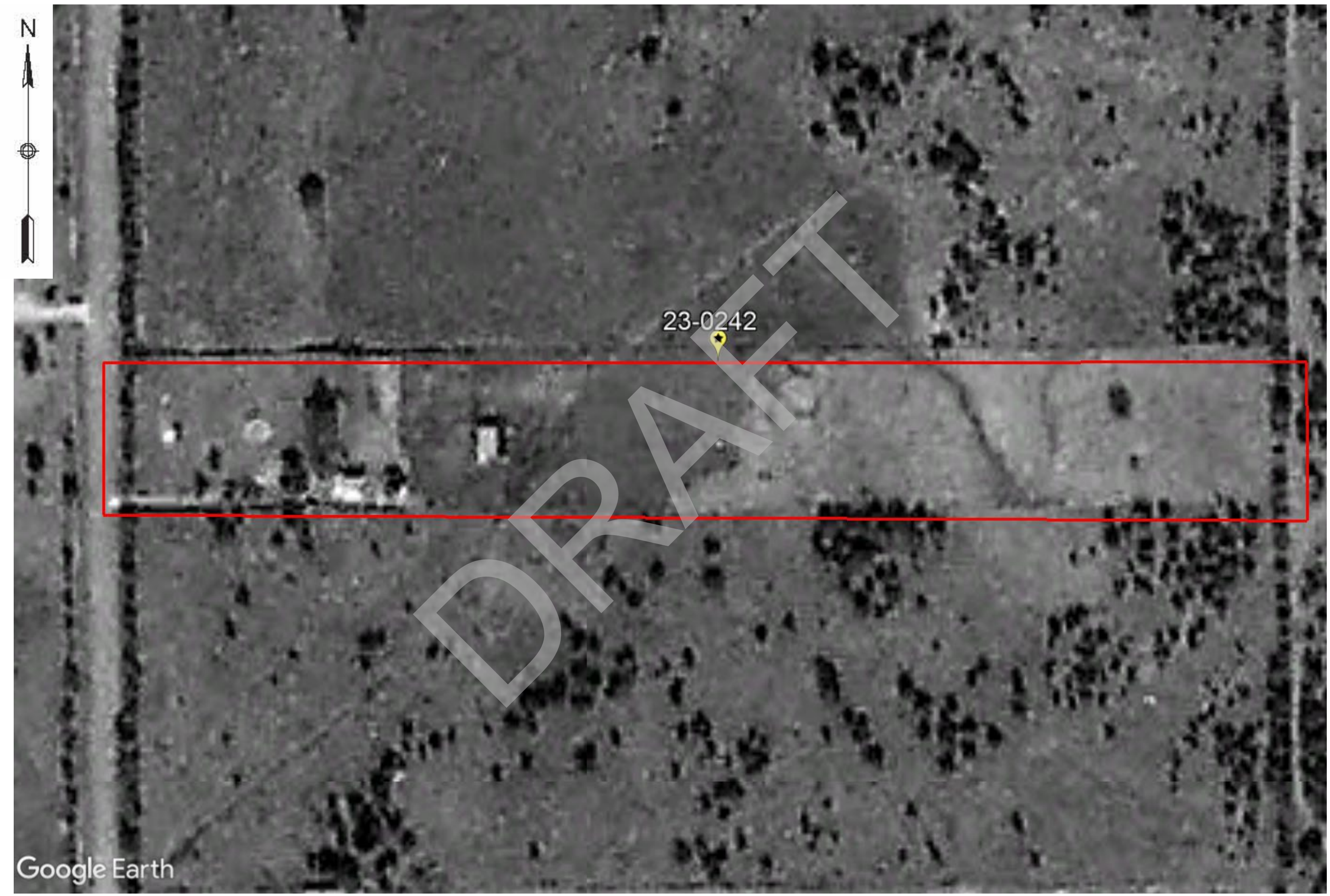
AERIAL PHOTOGRAPH - 2004

AERIAL PHOTOGRAPH - 2002

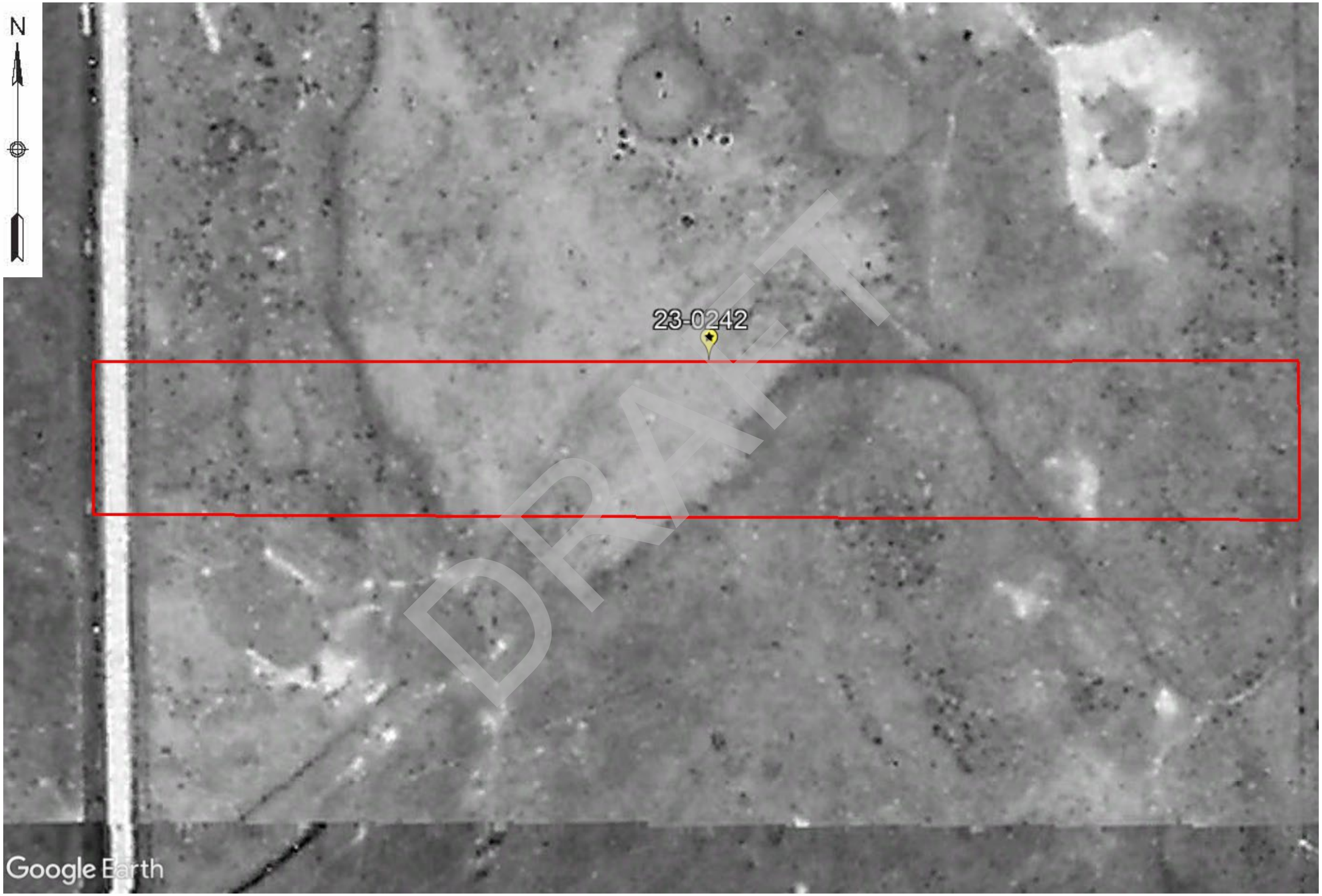
AERIAL PHOTOGRAPH - 1995

AERIAL PHOTOGRAPH - 1989

AERIAL PHOTOGRAPH - 1978



AERIAL PHOTOGRAPH - 1944



RINER Project No. 23-0242 Cypress Rosehill Business Park

Appendix E - USGS Topographic Map

USGS TOPOGRAPHIC MAP

Appendix F - Site Photographs

SITE PHOTOGRAPHS

Facing North at Boring B-01

Facing East at Boring B-02

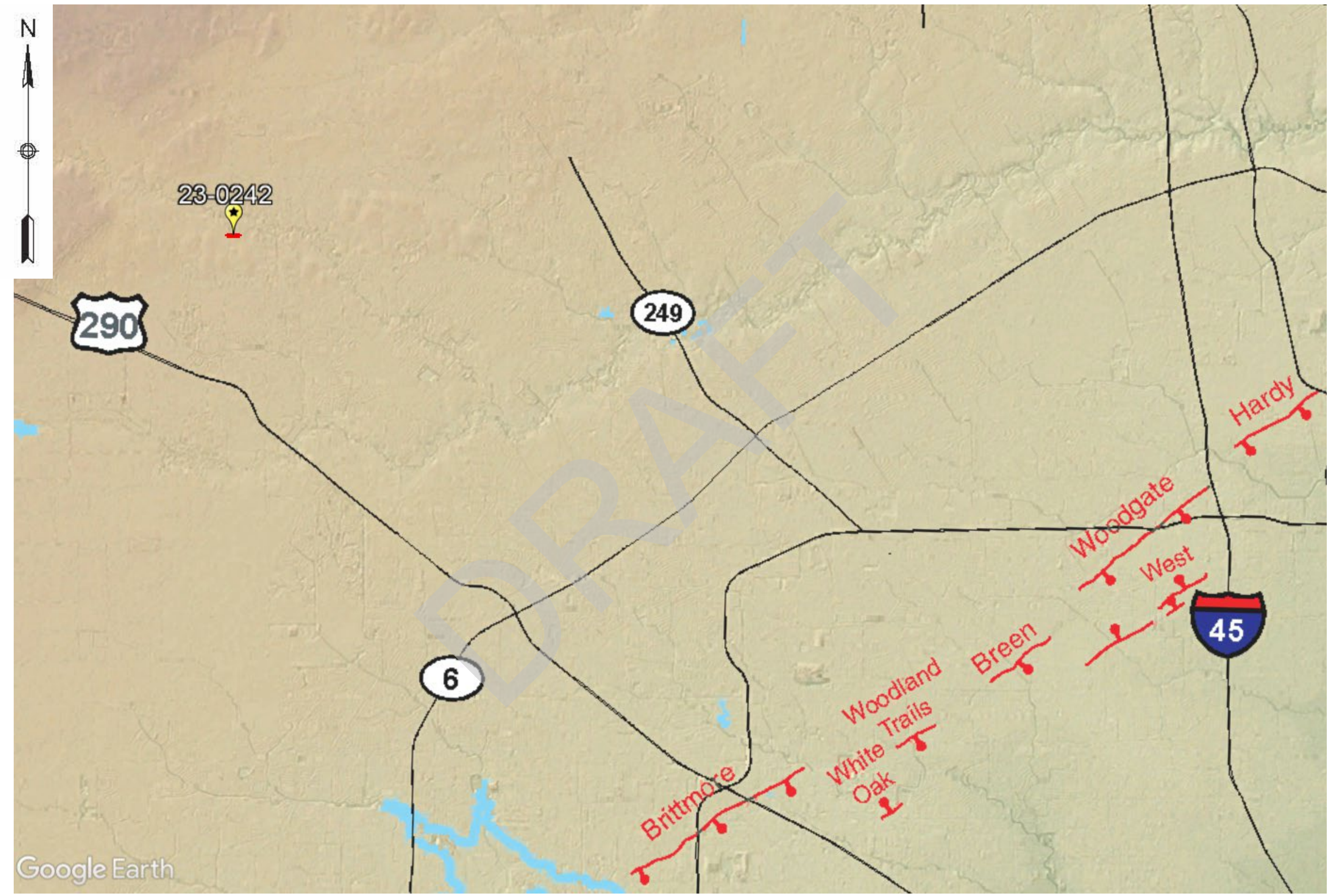
Facing East at Boring B-04

Facing South at Boring B-07

Appendix G - Geologic Information

GEOLOGIC ATLAS

USGS GEOLOGIC FAULT MAP



Mineral Resources On-Line Spatial Data

Lissie Formation

Lissie Formation

State	Texas
Name	Lissie Formation
Geologic age	Phanerozoic | Cenozoic | Quaternary | Pleistocene-Middle
Original map label	Ql
Comments	Sand, silt, clay, and minor amount of gravel. Iron oxide and iron-manganese nodules common in zone of weathering; locally calcareous. Surface fairly flat and featureless except for many shallow depressions and pimple mounds. Moore and Wermund (1993a) mapped three units--(1) alluvium undifferentiated as to texture and origin--includes meander belt, levee, crevasse splay, and distributary sand, and flood-basin mud deposits, about 60 m thick, (2) fine-grained channel facies (alluvial sand, silt, and clay) about 10-25 m thick, thicker seward, and (3) fine-grained overbank facies (alluvial silt and clay) about 55-65 m thick, thicker seaward. Together, these deposits form a deltaic plain that parallels the Gulf Coast. Unit contains Pleistocene vertebrate fauna, dips seaward beneath the Beaumont Fm. and disconformably overlies deposits of the Pliocene and early Pleistocene Willis Formation. The deltaic plain is entrenched as much as 7 m by streams. In Hidalgo County (southernmost part of Texas) the unit underlies a semiarid plain, widely irrigated and cultivated. Unit is locally veneered with thin, discontinuous stabilized eolian sand.
Primary rock type	sand
Secondary rock type	silt
Other rock types	clay or mud
Lithologic constituents	Major Unconsolidated > Fine-detrital > Clay (Bed) Unconsolidated > Coarse-detrital > Sand (Bed) Unconsolidated > Fine-detrital > Silt (Bed)
Map references	Bureau of Economic Geology, 1992, Geologic Map of Texas: University

of Texas at Austin, Virgil E. Barnes, project supervisor, Hartmann, B.M. and Scranton, D.F., cartography, scale 1:500,000

Unit references

Bureau of Economic Geology, 1975, Corpus Christi Sheet, Geologic Atlas of Texas, Bureau of Economic Geology, University of Texas at Austin, scale 1:250,000.

Moore, D.W. and Wermund, E.G., Jr., 1993a, Quaternary geologic map of the Austin 4 x 6 degree quadrangle, United States: U.S. Geological Survey Miscellaneous Investigations Series Map I-1420 (NH-14), scale 1:1,000,000.
[http://pubs.er.usgs.gov/publication/i1420(NH14)]

Moore, D.W. and Wermund, E.G., Jr., 1993b, Quaternary geologic map of the Monterrey 4 x 6 degree quadrangle, United States: U.S. Geological Survey Miscellaneous Investigations Series Map I-1420 (NG-14), scale 1:1,000,000.
[http://pubs.er.usgs.gov/publication/i1420(NG14)]

Bureau of Economic Geology, 1974, Seguin Sheet, Geologic Atlas of Texas, University of Texas, Bureau of Economic Geology, scale 1:250,000.

Bureau of Economic Geology, 1976, Crystal City-Eagle Pass Sheet, Geologic Atlas of Texas, University of Texas, Bureau of Economic Geology, scale 1:250,000.

Bureau of Economic Geology, 1975, Beeville-Bay City Sheet, Geologic Atlas of Texas, Bureau of Economic Geology, University of Texas at Austin, scale 1:250,000.

Bureau of Economic Geology, 1982, Houston Sheet, Geologic Atlas of Texas, Bureau of Economic Geology, University of Texas at Austin, scale 1:250,000.

Geographic coverage Austin - Bee - Calhoun - Colorado - DeWitt - Duval - Fort Bend - Goliad - Grimes - Hardin - Harris - Hidalgo - Jackson - Jasper - Jim Wells - Lavaca - Liberty - Live Oak - Montgomery - Newton - Nueces - Polk - Refugio - San Jacinto - San Patricio - Tyler - Victoria - Waller - Wharton - Willacy

Show this information as [XML] - [JSON]

Mineral Resources On-Line Spatial Data

Willis Formation

Willis Formation

State	Texas
Name	Willis Formation
Geologic age	Phanerozoic | Cenozoic | Tertiary | Pliocene
Original map label	Pow
Comments	Clay, silt, sand, siliceous granule to pebble gravel, some petrified wood; sand coarser than younger units, noncalcareous; deeply weathered, locally cemented by iron oxide; fluvitile; forms north-facing scarp. On Seguin Sheet (1974) thickness 100+- ft. On Austin 4 x 6-degree sheet (Moore and Wermund, 1993) unit consists of 1) channel facies--alluvial pebble gravel and sand, lt. gray to orange-brown, orange, gravelly coarse to fine sand which lenses of red, sandy silt and white to gray clay 10-60 m thick, pebbles mostly quartz, some chert and petrified wood, and 2) overbank facies--alluvial silt and clay made of brown, yellow, orange, fine silt and clay are intermixed and interbedded. 5-50+m thick.
Primary rock type	clay or mud
Secondary rock type	silt
Other rock types	sand; gravel
Lithologic constituents	Major Unconsolidated > Fine-detrital *(Alluvial)* Unconsolidated > Coarse-detrital *(Alluvial)* Minor Unconsolidated > Coarse-detrital > Gravel *(Bed)*
Map references	Bureau of Economic Geology, 1992, Geologic Map of Texas: University of Texas at Austin, Virgil E. Barnes, project supervisor, Hartmann, B.M. and Scranton, D.F., cartography, scale 1:500,000
Unit references	Moore, D.W. and Wermund, E.G., Jr., 1993a, Quaternary geologic map of the Austin 4 x 6 degree quadrangle, United States: U.S. Geological Survey Miscellaneous Investigations Series Map I-1420 (NH-14), scale 1:1,000,000. [http://pubs.er.usgs.gov/publication/i1420(NH14)]

Geographic coverage Austin - Bastrop - Colorado - DeWitt - Fayette - Gonzales - Grimes - Guadalupe - Hays - Jasper - Lavaca - Liberty - Montgomery - Newton - Polk - Sabine - San Jacinto - Tyler - Victoria - Walker - Waller - Washington

Show this information as [XML] - [JSON]

Appendix H - Unified Soil Classification System

UNIFIED SOIL CLASSIFICATION SYSTEM

UNIFIED SOIL CLASSIFICATION AND SYMBOL CHART

COARSE-GRAINED SOILS (more than 50% of material is larger than No. 200 sieve size.)		
GRAVELS More than 50% of coarse fraction larger than No. 4 sieve size		Clean Gravels (Less than 5% fines)
	GW	Well-graded gravels, gravel-sand mixtures, little or no fines
	GP	Poorly-graded gravels, gravel-sand mixtures, little or no fines
		Gravels with fines (More than 12% fines)
	GM	Silty gravels, gravel-sand-silt mixtures
	GC	Clayey gravels, gravel-sand-clay mixtures
SANDS 50% or more of coarse fraction smaller than No. 4 sieve size		Clean Sands (Less than 5% fines)
	SW	Well-graded sands, gravelly sands, little or no fines
	SP	Poorly graded sands, gravelly sands, little or no fines
		Sands with fines (More than 12% fines)
	SM	Silty sands, sand-silt mixtures
	SC	Clayey sands, sand-clay mixtures
FINE-GRAINED SOILS (50% or more of material is smaller than No. 200 sieve size.)		
SILTS AND CLAYS Liquid limit less than 50%	ML	Inorganic silts and very fine sands, rock flour, silty of 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 50% or greater	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

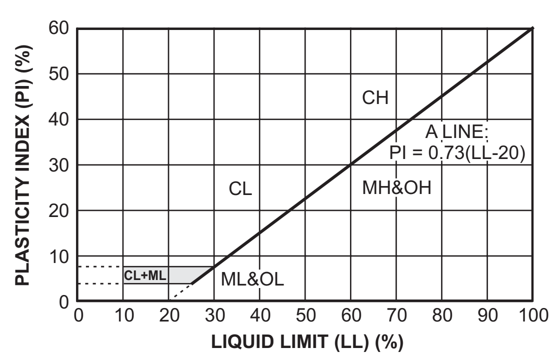
LABORATORY CLASSIFICATION CRITERIA

GW	$C_u = \frac{D_{60}}{D_{10}}$ greater than 4; $C_c = \frac{D_{30}}{D_{10} \times D_{60}}$ between 1 and 3	
GP	Not meeting all gradation requirements for GW	
GM	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
GC	Atterberg limits above "A" line with P.I. greater than 7	
SW	$C_u = \frac{D_{60}}{D_{10}}$ greater than 4; $C_c = \frac{D_{30}}{D_{10} \times D_{60}}$ between 1 and 3	
SP	Not meeting all gradation requirements for GW	
SM	Atterberg limits below "A" line or P.I. less than 4	Limits plotting in shaded zone with P.I. between 4 and 7 are borderline cases requiring use of dual symbols.
SC	Atterberg limits above "A" line with P.I. greater than 7	

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

PLASTICITY CHART



TERMS DESCRIBING SOIL CONSISTENCY

Fine Grained Soils		Coarse Grained Soils		
Description	Penetrometer Reading (tsf)	Penetration Resistance (blows/ft)	Description	Relative Density
Soft	0.0 to 1.0	0 to 4	Very Loose	0 to 20%
Firm	1.0 to 1.5	4 to 10	Loose	20 to 40%
Stiff	1.5 to 3.0	10 to 30	Medium Dense	40 to 70%
Very Stiff	3.0 to 4.5	30 to 50	Dense	70 to 90%
Hard	4.5+	Over 50	Very Dense	90 to 100%