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ADDENDUM NO. 02
June 27, 2025

To Drawings and Specifications dated June 10, 2025.

PKG 3A – GPHS New Fieldhouse

Prepared by: PBK
11 Greenway Plaza, 22nd Floor
Houston, TX 77046-1104
PBK Project No: 240539

Notice to Bidders

A. Receipt of this Addendum shall be acknowledged on the Bid Form.
B. This Addendum forms part of the Contract documents for the above referenced project and shall be incorporated integrally therewith.
C. Each bidder shall make necessary adjustments and submit his proposal with full knowledge of all modifications, clarifications, and supplemental data included therein. Where provisions of the following supplemental data differ from those of the original Contract Documents, this Addendum shall govern.

GENERAL

Item No. 01 Pre-proposal Questions

Question 01: The plumbing plans show a fire pump and tank, but the configuration is missing. Do you have a plan showing how much GPM and PSI fire pump is needed? Though the utility shows a 6-inch water line for the fire sprinkler system, do you still need a fire pump and a break tank? Please clarify.

i. Response: Pump schedule is shown on P-501. Reference sheet P-602 for break tank details.

Question 02: Please confirm all wall finishes shown on the DIRTT drawings will be part of the DIRTT scope

i. Response: Yes, wall finishes shown in DIRTT drawings will be part of DIRTT scope.

Question 03: Please provide the thickness of the foundation slab.

i. Response: The slab thickness is 6”, as per note 2.2 in sheet S-101 Foundation Plan.

Question 04: Please provide Geotech report. Please specify thickness and overbuild of select fill under the building.

i. Response: Please see the attached Geotech report reference civil engineering drawings as required.

Question 05: A901 - Pump Room 1903 - Flooring details not given

i. Response: A-901 - Level 1 - Overall Finish Plan - Room tag has been added to pump room with floor finish being 03 35 00.CSI Sealed Concrete

Question 06: 09 30 00 - Tile type T-3, T-4 - none seen on plans. Please clarify where do they occur in this bldg.

i. Response: Tile Type T-3, T-4 are no longer applicable since wall now falls within DIRTT scope. This scope will be required if alternate 05 scope is accepted.

Question 07: Per sheet A-921, it states for room 1915A the East wall is to receive Paint P6 in lieu of T5, T6, & 17. Room 1916A states the north Wall is to receive no finish. Shouldn't both walls in question receive wall tile T5, T6 & T7?18. Where does 09 51 00.XCT6 Axiom Ceiling Clouds occur? Per RCP, there are hexagonal shapes on both 1st and 2nd floors, however per the material legend, it states they are electrical light fixtures. Please advise. 19. Where are corner guards located? None appear to be indicated on the drawings. Please let us know if this is applicable or not

ii. Response:

a. In 1915A the east wall will receive tiles T5, 6 & 7 - North wall is part of the DIRTT scope. In 1916A North wall is part of the DIRTT scope.
b. Hexagon shapes on first and second floor are light fixtures. Axiom Ceiling Clouds are no longer applicable.
c. We do not need corner guards on DIRTT walls, nor do we need them on CMU walls. If alternate 05 scope is accepted corner guard scope will be coordinated with the client.

Question 08: There is a metal shelving spec. Rooms 1908, 1963 and 1967 could have metal shelving, but there are millwork notes also. Which if any of these are metal shelves?

i. .Response: Details 10 + 15/A-951 and 7/A-971 are referencing specific casework elevations as required by the district. The remaining items as shown in these rooms are intended to be metal shelving.

<u>SPECIFICATIONS</u>

Item No. 1 **00 31 32 GEOTECHNICAL DATA**

A. Specification has been added in its entirety. See attached.

Item No. 2 **00 31 32.1 GALENA PARK HIGH SCHOOL BORING LOGS**

A. Specification has been removed in its entirety.

END OF ADDENDUM NO. 02

06/27/2025

SECTION 00 31 32 - GEOTECHNICAL DATA

PART 1 - GENERAL

1.1 GEOTECHNICAL REPORT

A. Geotechnical Report: A report of a geotechnical investigation entitled Geotechnical Engineering Report Galena Park High School – Phase 3A, 1000 Keene Street, Galena Park, Texas 77547, project number H251673-1, dated June 5, 2025, has been prepared for Galena Park Independent School District , Harris County, Texas by the Geotechnical Consultant, UES Professional Solutions 44, LLC, Houston, Texas (713) 360-0460, based on soil boring samples obtained at the Project site on April 3, 2025 through April 19, 2025.

B. Boring Logs: Excerpts from the Geotechnical Report, including a Boring Plan, Boring Logs describing strata for each test hole, and results of laboratory tests, are bound herein, or if not bound herein, will be made available to Offerors by the Owner upon request.

C. The Drawings and Specifications govern the construction of the Project. Boring Logs and the Geotechnical Report are made available for the information and convenience of Offerors. The findings and recommendations are the responsibility of the preparer, and are not part of the Contract Documents.

1.2 SUBSURFACE CONDITIONS

A. Subsurface conditions indicated in the report were found to exist at the locations shown on the dates the samples were taken and the tests performed. Since subsurface conditions, including but not limited to the presence of groundwater, may vary significantly from time to time, no representation or warranty is made that the conditions described in the Geotechnical Report describe the actual conditions that will be extant during the performance of the Work of This Contract.

B. Offerors shall visit the site and become fully acquainted with the conditions affecting the Work of This Contract.

PART 2 - PRODUCTS *(Not Used)*

PART 3 - EXECUTION *(Not Used)*

END OF SECTION 00 31 32

GEOTECHNICAL ENGINEERING REPORT

GALENA PARK HIGH SCHOOL – PHASE 3A
1000 Keene Street
Galena Park, Texas 77547
UES Project No. H251673-1-Final
June 5, 2025

Prepared for:

GALENA PARK ISD
14705 Woodforest Boulevard
Houston, Texas, 77015
Attention: Ed Martir

Prepared by:

UES™

UES

Environmental
Geotechnical Engineering
Materials Testing
Field Inspections & Code Compliance
Geophysical Technologies

June 5, 2025

Ed Martir
Galena Park ISD
14705 Woodforest Boulevard
Houston, Texas, 77015

Re: GEOTECHNICAL ENGINEERING REPORT
Galena Park High School – Phase 3A
Galena Park, Texas
UES Project No.H251673-1-Final

Dear Mr. Martir:

UES Professional Solutions 44, LLC (hereinafter "UES"), is pleased to submit this Geotechnical Engineering Report for the referenced project. The results of this exploration, together with our recommendations, are presented in the accompanying report, an electronic copy of which is being transmitted herewith. This geotechnical study was authorized by Michael McKay with Galena Park ISD via a Geotechnical Testing & Reporting Services Agreement, and performed in accordance with UES Proposal No. 111602-Rev2, dated March 24, 2025.

UES appreciates the opportunity to be of service on this project. If we can be of further assistance, such as providing materials testing services during construction, please contact our office.

Respectfully submitted,

UES Professional Solutions 44, LLC

Victor Guevara Jr., E.I.T.
Staff Geotechnical Engineer

STATE OF TEXAS
DURAISAMY S. SARAVANATHIIBAN
123650
LICENSED
PROFESSIONAL ENGINEER
06/05/2025

Duraisamy S. (Roy) Saravanathiiban, Ph.D., P.E.
West Houston Geotechnical Department Manager

VG/RS/vg

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1.0 INTRODUCTION

Purpose and Scope. The purpose of this geotechnical study was to evaluate some of the physical and engineering properties of subsurface materials at selected locations on the subject site to develop geotechnical engineering design parameters and recommendations for the proposed project. To accomplish this, the scope of this study included field exploration consisting of drilling test borings and collecting samples of the subsurface materials, performing laboratory testing on selected samples obtained during the field exploration, performing engineering analysis and evaluation of the subsurface conditions with respect to the project characteristics, and development of foundation and pavement recommendations suitable for the proposed project. The scope of services did not include an environmental assessment of the site.

Project Location. The project is located at 1000 Keene Street, in Galena Park, Texas. The general location and orientation of the site are provided in Appendix A - Project Location Diagrams.

Project Description. The project consists of a proposed two-story athletic field house building (approximately 20,800 SF).

Loading Information. Based on our discussion with the structural engineer, we understand that the maximum foundation loads at main columns for slab-on-grade are anticipated to be up to 200 kips. Sustained loads may be taken as 75 percent of the column loads provided. Perimeter grade beam loads may be assumed as approximately 2 to 4 klf for the slab on grade. *Any change in the structural loads should be brought to our attention to review the design and assess the suitability of the recommendations provided.*

Site Grading Plan. The site grading plan was unavailable at the time this report was prepared. Our recommendations provided herein are on the basis that cuts and fills 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 site grading plan to assess and modify our recommendations, as necessary.

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. UES, by necessity, has assumed the user of this document possesses the technical acumen to understand and properly utilize the information and recommendations provided herein. UES 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 UES for clarification. Clarification will be provided verbally and/or issued by UES 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 UES.

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 EXPLORATION

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-05	25 to 60	04/03-19/2025	Proposed Building Area

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 soil 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 with onsite soil cuttings from the top and plugged at the surface.

3.0 LABORATORY TESTING

UES performs visual classification and any of several 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 D1140	Standard Test Methods for Amount of Material in Soils Finer than the No. 200 (75-µm) Sieve
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 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 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

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	Structures were noted at the project site.
1953	Previously noted structures were demolished and removed
1978	A new building was noted at the project site.
1995	Added additions to the building and portable buildings to the site.
2002	Asphalt pavement was noted at the project site and portable buildings were removed.
2004	No visible changes were noted.
2009	No visible changes were noted.
2014	No visible changes were noted.
2019	No visible changes were noted.
2025	No visible changes were noted.

Site Fills Based on Aerial photographs. The aerial photographs reviewed reveal that the site was previously developed with multiple building structures and pavement. Therefore, we would expect a surficial disturbance of site soil. Our review revealed obvious areas of significant fill on-site. **Existing fill recommendations are provided in Section 5.8.**

Potential Existing Foundations. It is not known whether the foundations supporting the former structures at the site were removed and backfilled or abandoned in-place. 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 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.

4.2 Geology

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

4.3 Soil Conditions

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 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-05)[1]			
Nominal Depth, feet bgs (Except as Noted)		General Description	Detailed Description of Soils/Materials Encountered
Top of Layer	Bottom of Layer		
0	**0.21 to 0.58**	**ASPHALT/CONCRETE**	**Asphalt/Concrete, 2.5 to 7 inches thick.**
0.21 to 0.58	**0.5 to 1.17**	**BASE MATERIAL**	**Soil Base Material, 6 and 7 inches thick, crushed concrete.**
0.5 to 1.17	**2**	**FILL**	**Stiff to hard SANDY LEAN CLAY (CL) FILL and SILTY SAND (SM) FILL.**
2	25 to 60	PREDOMINANTLY FAT CLAY, LEAN CLAY and SILTY SAND SOME SANDY SILTY CLAY and CLAYEY SAND	Firm to very stiff SANDY/FAT CLAT (CH), Soft to very stiff SANDY/LEAN/LEAN CLAY WITH SAND (CL), Loose to medium dense SILTY SAND (SM), Firm to hard SANDY SILTY CLAY (CL-ML), and Loose CLAYEY SAND (SC).
Note: 1. Boring Termination Depth = 25 to 60 feet bgs.			

Swell Tests. Swell tests were performed on selected clay soil samples. Swell test details are provided in "Appendix C - Boring Logs and Laboratory Results".

4.4 Groundwater

Groundwater Levels. The test borings were advanced using continuous flight augers and air-rotary drilling methods, with intermittent sampling methods. These dry drilling techniques enable observation of potential groundwater seepage levels. Groundwater levels encountered in the borings during this study are identified in the table below. Depths referenced in this report and in the table below are measured from the existing ground surface at the respective boring location at time of the field exploration.

Boring No.	Depth Groundwater Initially Encountered (feet, bgs)	Groundwater Depth after 15 Minutes (feet, bgs)
B-01	12.6	11.1
B-02	11.0	10.5
B-03	13.0	12.0
B-04	12.0	9.0
B-05	15.0	13.0

Long-term Groundwater Monitoring. These groundwater observations are indicative of the groundwater conditions present at the time the borings were drilled. The amount of water in an open borehole largely depends on the permeability of the soil encountered at the boring location. In relatively impervious soils, such as clayey soils, a suitable estimate of the groundwater depth may not be possible, even after several days of observation. Long-term monitoring of groundwater conditions via piezometers or groundwater monitoring wells was not performed during this study and was beyond the scope of this study. Long-term monitoring can reveal groundwater levels materially different than those encountered during measurements taken while drilling the borings.

Groundwater Fluctuations. It is difficult to accurately predict the magnitude of subsurface water fluctuations that might occur based upon short-term observations. Future construction activities may alter the surface and subsurface drainage characteristics of this site. Seasonal variations, temperature, land-use, proximity to water bodies, and recent rainfall conditions may influence the depth to the groundwater. With these considerations UES recommends that the contractor verifies the groundwater elevation before construction starts.

5.0 ANALYSIS AND RECOMMENDATIONS

5.1 Seismic Site Classification

The Site Class assigned for seismic design considers various factors, such as the soil profile (whether it's soil or rock), shear wave velocity, and strength, averaged over a depth of 100 feet. As our borings didn't reach depths of 100 feet, we made determinations under the assumption that the subsurface materials beneath the borehole bottoms resembled those encountered at the termination depth. Following the guidelines outlined in Section 1613.3.2 of the 2018 International Building Code and Table 20.3-1 in the 2010 ASCE-7, we recommend utilizing Site Class D for seismic design purposes at this location.

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 clay soil to swell is 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 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 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 causes 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 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.

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.

Maintaining 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. Please see our recommendations in "Section 5.5.4 - Grading and Drainage" for water control and limit the extreme wetting or drying of the subsurface soils.

Calculated PVR. Considering the subsurface conditions encountered at this site and methods used to estimate the potential vertical rise of the soil, floor slabs and other soil-supported elements could experience soil-related movements of up to about 3 inches if constructed at the grades discussed in Section 1.0.

These potential seasonal movements were estimated in general accordance with methods outlined by Texas Department of Transportation (TxDOT) Test Method Tex-124-E, the results of swell tests, a Volflo analysis and engineering judgment and experience. Estimated movements were calculated assuming the moisture content of the in-situ soil within the normal zone of seasonal moisture content change varies between a "dry" condition and a "wet" condition as defined by Tex-124-E. Also, it was assumed a 1 psi surcharge load from the floor slab acts on the subgrade soils. Movements exceeding those predicted could occur if positive drainage of surface water is not maintained or if soils are subject to an outside water source, such as leakage from a utility line or subsurface moisture migration from off-site locations.

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 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

The contractor is responsible for designing any excavation slopes, temporary sheeting or shoring. Design of these structures should include any imposed surface surcharges. Construction site safety is the sole responsibility of the contractor, who shall also be solely responsible for the means, methods and sequencing of construction operations. The contractor should also be aware that slope height, slope inclination or excavation depths (including utility trench excavations) should in no case exceed those specified in local, state and/or federal safety regulations, such as OSHA Health and Safety Standard for Excavations, 29 CFR Part 1926, or successor regulations.

Preventative measures should be taken to avoid damaging or adversely affecting the integrity of the existing foundation system during construction activities. Temporary shoring may be required when excavating adjacent to the existing structure to install non-expansive fill material.

Stockpiles should be placed well away from the edge of the excavation and their heights should be controlled so they do not surcharge the sides of the excavation. Surface drainage should be carefully controlled to prevent flow of water over the slopes and/or into the excavations. Construction slopes should be closely observed for signs of mass movement, including tension cracks near the crest or bulging at the toe. If potential stability problems are observed, a geotechnical engineer should be contacted immediately. Shoring, bracing or underpinning required for the project (if any) should be designed by a professional engineer registered in the State of Texas.

5.4 Groundwater Control

Groundwater was initially encountered at depths as shallow as 11 feet bgs in borings during drilling and rose to depths as shallow as 9 feet within 15 minutes. 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 Construction Considerations

Surface Sandier/Siltier 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. As such, construction difficulties should be anticipated, especially during the wet season or immediately after rain events. Although having a thin layer of non-plastic or low plasticity soils overlying cohesive soils is typical of this geologic region, our experience suggests that the local contractors find these materials troublesome and can often be the source of change orders, construction delays, and budget over runs. Soils of this type are especially prone to requiring the implementation of wet weather/soft subgrade recommendations provided in this report.

Maintenance of Subgrade during Construction. While the exposed subgrade is expected to remain relatively stable initially, unstable conditions may arise during general construction activities, particularly if the soil is exposed to wet weather conditions and repetitive construction traffic. The use of lighter construction equipment can help minimize disturbance to the subgrade. In the event of unstable conditions, stabilization measures will be necessary. After grading is completed, it's crucial to maintain the moisture content of the subgrade before proceeding with pavement/building slab construction. Minimizing construction traffic over the finished subgrade is advisable. If the subgrade becomes frozen, desiccated, saturated, or disturbed, the affected material should either be removed or treated by scarification, moisture conditioning, and recompaction before pavement/building slab construction begins. UES should be retained to observe earthwork and to perform necessary tests and observations during subgrade preparation.

5.5.4 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 soil 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 soil to become too dry or too wet.

5.5.5 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 soil 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. UES should be contacted for additional recommendations if chemical treatment is planned due to wet and/or soft soils during construction. The treatment depth and chemical reagent type and application rate depend on the site condition during construction.

5.5.6 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 of less than 40 and a Plasticity Index between 8 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 3 percent above optimum moisture content. The subgrade to receive select fill should be scarified to a depth of 6 inches and compacted to 93 to 96 percent of the material's maximum standard Proctor dry density (ASTM D-698) at a workable moisture level at least 3 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 8-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.

Utilities and Deep Fills. In cases where utility lines and/or mass fills are more than 10 ft deep, the fill/backfill below 10 ft should be compacted to at least 100 percent of standard Proctor maximum dry density (ASTM D 698) and within –2 to +2 percentage points of the material's optimum moisture content. The portion of the fill/backfill shallower than 10 ft should be compacted as previously outlined. Density tests should be performed on each lift (maximum 12-inch thick) and should be performed as the trench is being backfilled.

Even if fill is properly compacted, fills in excess of about 10 ft are still subject to settlements over time of up to about 1 to 2 percent of the total fill thickness. This should be considered when designing pavements and other structures over utility lines or adjacent to retaining walls with deep fill, or any other structure in deep fill areas. To reduce the risk of fill settlement, the portion of the fill below a depth of 10 ft below final grade should be compacted to a minimum of 100 percent of the material's maximum standard Proctor dry density (ASTM D-698). This procedure will reduce (but not eliminate) the risk of fill settlement. If this risk of subgrade settlement is not acceptable, consideration could be given to backfilling portions or all of the excavation with flexible base material, cement-stabilized sand, or flowable fill.

If utility trenches or other excavations extend to or beyond a depth of 5 ft below construction grade, the contractor or others shall be required to develop an excavation safety plan to protect personnel entering the excavation or excavation vicinity. The collection of specific geotechnical data and the development of such a plan, which could include designs for sloping and benching or various types of temporary shoring, is beyond the scope of this study. Any such designs and safety plans shall be developed in accordance with current OSHA guidelines and other applicable industry standards.

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.

5.5.7 Testing

Required Testing and Inspections. Field compaction and classification tests should be performed by UES. 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, UES 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. UES 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.

<u>Existing Utilities</u>. 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.

<u>Backfill</u>. Excavations resulting from the excavation of existing foundations and utilities should be backfilled in accordance with Section 5.5.6.

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

5.7 Existing Fill

Our subsurface study indicates existing fill on site. Existing fill was encountered in all boring locations B-01 through B-05. Existing fill extended to a depth of up to about 2-feet bgs. It is worth noting that existing fill may also be present, potentially at greater depths, in other parts of the site. Accurately delineating fill soils, especially those resembling native soils, based on discrete test boreholes is challenging. As such, the recorded fill depths should be considered as estimates and may slightly deviate from the actual fill depths. Although not encountered in the borings for this project, uncontrolled fills may contain trash, debris, concrete rubble, construction debris, boulders, and other unsuitable materials.

Considering the depth of excavation required for subgrade improvement to reduce movements due to shrinking and swelling of active clays (see Section 5.8), we anticipate most or all of the existing fill will be removed from the building area. Any remaining uncontrolled fill after excavation for subgrade improvement should be removed to expose firm native soils. The resulting excavation should be properly backfilled to the bottom of the subgrade improvement depth with controlled fill as described in Section 5.5.6. Any excavated materials proposed for re-use as controlled fill in the building pad area should have a plasticity index of 20 or less, and should be free or organics, debris, or other unsuitable materials.

In pavement areas, the existing fill at the pavement subgrade level should be proof-rolled with a heavy roller to detect possible weak areas. Any weak soils identified as part of the

proof-rolling process should be removed and replaced with well-compacted soil as outlined in Section 5.5.6 of this report.

5.8 Slab-on-Grade and Subgrade Improvement

Potential Vertical Slab Movements. Based on the information gathered during this study, a slab constructed on-grade will be subject to potential vertical slab movements of up to about 3-inches.

Subgrade Treatment Using Select Fill. The depth of subgrade treatment is dependent on desired post-construction PVR. The following table presents recommended depth of subgrade treatment for various allowable post-construction PVR levels (as determined by Structural Engineer).

Subgrade Treatment - Select Fill Option		
Required PVR (inches)	Minimum Thickness of Select Fill Soil (feet, bgs) [1]	Thickness of Compacted Subgrade below Select Fill (inches) [2]
¾	5½	6
1	4½	6

Notes:
1. Depth measured below bottom of the slab-on-grade.
2. The subgrade to receive select fill soil should be scarified to a depth indicated above. The scarified subgrade should be compacted to 93 to 96 percent of the material's maximum standard Proctor dry density (ASTM D-698) at a workable moisture level at least 3 percentage points above optimum.

Subgrade treatment should extend at least 5-feet horizontally beyond the perimeter of the building.

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. Sidewalks tied to pavements and other flatworks that extend beyond the subgrades treated for PVR reduction may be subjected to movements similar to those experienced for untreated subgrades.

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).

Fill Related Slab Settlement. Fill will settle under its own weight. A properly constructed fill will generally settle up to 2% of 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 type, pore water, and drainage path conditions and therefore can vary widely. As a result, fill-related settlement should be expected before AND after construction of the slab. Slab movement related to settling fill can be reduced by allowing as much time as possible between the time the fill is placed and construction of the slab. Furthermore, we recommend survey monitoring of constructed fills be performed to verify the rate and magnitude of settlement has been reduced to an acceptable level prior to construction of slabs on the fill.

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. 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.

5.9 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 slab/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. Slab/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.

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 our geotechnical recommendations.

5.9.1 Shallow Footings

General Requirement. Shallow strip and spread footing foundations may be used for support of the proposed structure if recommendations in the sections 5.7 "Existing Fill" and 5.8 "Slab-on-Grade and Subgrade Improvement" 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. Shallow strip and spread footings should not bear on moisture conditioned soil.

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 up to about ¾ or 1 inch. We should be allowed to review foundations larger than 10 feet to assess their settlement. Our settlement estimate assumes that proper construction practices are followed and there are no overlapping stresses due to adjacent footings. To mitigate any overlapping stresses due to adjacent footings, we recommend a minimum clear spacing of one footing width (width of larger footing) between adjacent footings.

Lateral Resistance. Resistance to lateral loads may be provided by the soil adjacent to the footings. We recommend using an equivalent fluid weight of 180 pcf for lateral resistance. A coefficient of sliding friction of 0.25 between the concrete footings and underlying soil may be combined with the passive resistance. Appropriate safety factors should be utilized by the structural engineer for lateral stability of the shallow footings.

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.

5.9.2 Underreamed Drilled Piers

General. Underreamed drilled pier foundations bearing in native soil may be utilized at this site for the proposed structure provided that recommendations in the sections 5.7 "Existing Fill" and 5.8 "Slab-on-Grade and Subgrade Improvement" are followed.

Foundation Depth. We recommend that underreamed piers should bear in native soil at a depth of 10-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.

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, 8-feet in diameter or less, constructed as described above should be up to about ¾ or 1 inch.

We should be allowed to review piers greater than 8-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	Clay	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. Each pier should contain full length reinforcing steel and should be designed to resist the uplift pressure (soil-to-pier adhesion) due to potential soil swell along the shaft from post-construction heave and other uplift forces applied by structural loadings. The magnitude of uplift adhesion due to soil swell along the pier shaft cannot be defined accurately and can vary according to the actual in-place moisture content of the soils during construction. It is estimated this uplift adhesion will not exceed about 1,600 psf. This soil adhesion is approximated to act uniformly over the upper 8 ft of the pier shaft in contact with clayey soils.

Uplift Resistance. The uplift force due to swelling of active clays should be resisted by the underreamed portion of the pier. The underreamed portion should be at least two (2) and not exceeding 3 times the diameter of the shaft. The minimum clear spacing between edges of adjacent piers should be at least one (1) underream diameter, based on the larger underream.

Shaft/Diameter Ratio. The piers should be provided with an underream diameter to shaft diameter ratio of not less than 2 to 1 and not greater than 3 to 1. **There is an inherent risk of bell collapse during construction due to the presence of isolated sand and silt pockets/seams which can cause significant loss of tensile strength resulting in bell collapse. Therefore, UES recommends test piers with underreams be constructed prior to finalizing**

the foundation design to assess the risk of bell collapse. As a minimum, UES recommend four (4) test piers with the largest bell size be constructed near each building corner and monitored for at least four hours. The test piers should be backfilled with excavatable flowable fill to a depth of 24-inches below the final grade. Excavated soil can be used to backfill upper 24-inches. If bell collapse happens at locations during construction phase, *drilled, straight-shaft piers with a straight-shaft diameter equal to the planned underream pier diameter need to be substituted in lieu of underreamed piers. Field observations by UES during construction will be required to determine areas where underreaming is not possible.*

Some field adjustments in the depth of the underreamed piers may still be required in some areas to maintain the bottom of the piers above any possible groundwater seepage and caving soils encountered near the bearing depth. Adjustments in the depths of the piers should be approved and observed in the field by UES personnel.

<u>Grade Beams</u>. Grade beams may be used to support loads by spanning the drilled-and-underreamed piers. Grade beams should be designed to transfer loads to the piers as a simply supported beam, ignoring any support from the soil between the piers. The depth of exterior and interior grade beams can be varied according to the structural requirements of the floor slab. However, we recommend that exterior grade beams extend at least 12 inches below the lowest adjacent grade. Additionally, backfill soils placed adjacent to grade beams must be compacted as outlined in Section 5.5.6 of this report.

In general, where the subgrade is improved and the floor slab is supported on-grade, we do not recommend the use of void boxes below grade beams and caps because of the potential to collect free water within the void space, especially if replacing the excavated subgrade soils with relatively pervious select fill materials.

<u>Construction Observation</u>. 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.

<u>Groundwater</u>. Groundwater was encountered in the borings at depths of about 9 to 15 feet below the existing ground surface during drilling and immediately after completion of drilling. Groundwater was not encountered in the remaining shallow borings. 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.

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

<u>Concrete Placement</u>. 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.

6.0 LIMITATIONS

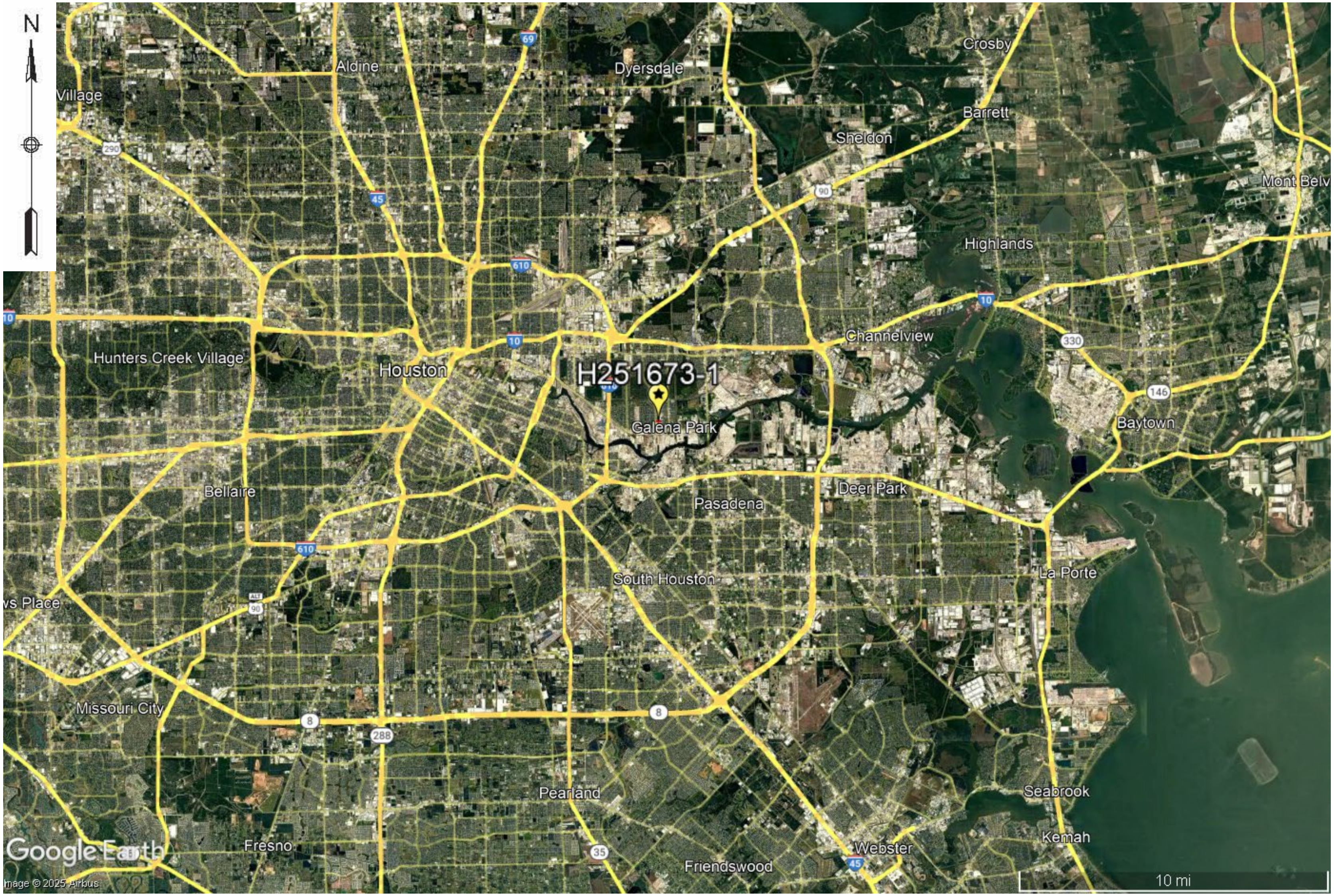
Professional services provided in this geotechnical exploration were performed, findings obtained, and recommendations prepared in accordance with generally accepted geotechnical engineering principles and practices. The scope of services provided herein does not include an environmental assessment of the site or investigation for the presence or absence of hazardous materials in the soil, surface water or groundwater. UES, upon written request, can be retained to provide these services.

UES is not responsible for conclusions, opinions or recommendations made by others based on this data. Information contained in this report is intended for the exclusive use of the Client (and their designated design representatives) and is related solely to design of the specific structures outlined in Section 1.0. No party other than the Client (and their designated design representatives) shall use or rely upon this report in any manner whatsoever unless such party shall have obtained UES's written acceptance of such intended use. Any such third party using this report after obtaining UES's written acceptance shall be bound by the limitations and limitations of liability contained herein, including UES's liability being limited to the fee paid to it for this report. Recommendations presented in this report should not be used for design of any other structures except those specifically described in this report. In all areas of this report in which UES may provide additional services if requested to do so in writing, it is presumed that such requests have not been made if not evidenced by a written document accepted by UES. Further, subsurface conditions can change with passage of time. Recommendations contained herein are not considered applicable for an extended period of time after the completion date of this report. It is recommended our office be contacted for a review of the contents of this report for construction commencing more than one (1) year after completion of this report. Non-compliance with any of these requirements by the Client or anyone else shall release UES from any liability resulting from the use of, or reliance upon, this report.

Recommendations provided in this report are based on our understanding of information provided by the Client about characteristics of the project. If the Client notes any deviation from the facts about project characteristics, our office should be contacted immediately since this may materially alter the recommendations. Further, UES is not responsible for damages resulting from the workmanship of designers or contractors. It is recommended the Owner retain qualified personnel, such as a Geotechnical Engineering firm, to verify construction is performed in accordance with plans and specifications.

Appendix A - Project Location Diagrams

PROJECT LOCATION DIAGRAM - GENERAL



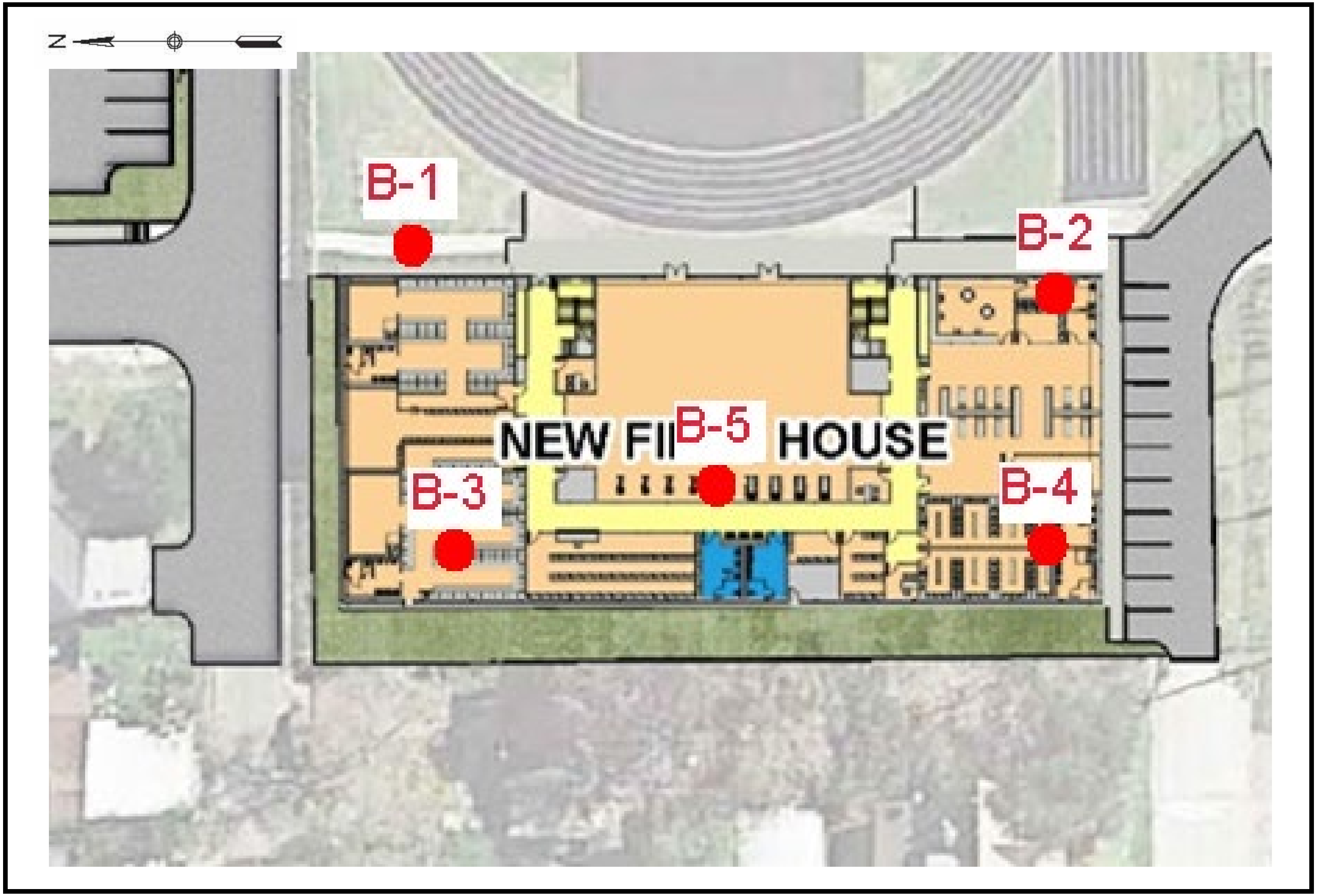
PROJECT LOCATION DIAGRAM - LOCAL

UES Project No. H251673-1

Galena Park High School – Phase 3A

Appendix B - Boring Location Diagram

BORING LOCATION DIAGRAM



Appendix C - Boring Logs and Laboratory Results

UES Professional Solutions 44, LLC
15811 Tuckerton Road, Houston, TX 77095
Telephone: 713-360-0460; Fax: 713-360-0481

BORING NUMBER B-01

CLIENT Galena Park ISD
PROJECT NAME Galena Park High School - Phase 3A
PROJECT NUMBER H251673-1
PROJECT LOCATION Galena Park, TX 77547
DATE STARTED 4/19/25 **COMPLETED** 4/19/25
GROUND ELEVATION **NORTHING**
CONTRACTOR UES
GROUND WATER LEVELS: **EASTING**
METHOD Auger 0 - 25 feet
∇ **INITIALLY ENCOUNTERED** 12.6 ft
LOGGED BY JA **CHECKED BY** V.G.
▼ **AFTER 15 MIN.** 11.1 ft
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		SANDY LEAN CLAY (CL) FILL - Stiff, dark brown, with root fibers and sand seams.	ST			2.00					24	39	21	18	55
		FAT CLAY (CH) - Stiff, dark gray, gray.	ST			2.00					23				
5		With sand seams from 4 to 6 feet.	ST			2.00					26				
		With gravel from 6 to 8 feet.	ST			2.00					22	60	20	40	
10		Light gray, brown, light brown from 8 to 13 feet.	ST			2.50		1.7		100	24				
15		CLAYEY SAND (SC) - Stiff, light gray, light brown.	ST			2.00					20				
20			ST			2.00					20				
25		SANDY FAT CLAY (CH) - Very stiff, reddish brown, light gray, with sand seams.	ST			4.00					22				

Bottom of hole at 25.0 feet.

UES Professional Solutions 44, LLC
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BORING NUMBER B-02

CLIENT Galena Park ISD
PROJECT NAME Galena Park High School - Phase 3A
PROJECT NUMBER H251673-1
PROJECT LOCATION Galena Park, TX 77547
DATE STARTED 4/11/25 **COMPLETED** 4/11/25
GROUND ELEVATION ____ **NORTHING** ____
CONTRACTOR UES
GROUND WATER LEVELS: **EASTING** ____
METHOD ____
▽ **INITIALLY ENCOUNTERED** 11.0 ft
LOGGED BY JA **CHECKED BY** V.G.
▼ **AFTER 15 MIN.** 10.5 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	PLASTIC LIMIT	PLASTICITY INDEX	FINES CONTENT (%)
0	PAVEMENT - 2.5 inch thick concrete.	RC												
	SOIL BASE MATERIAL - 6 inches thick, with crushed concrete.	AU								19				
		AU												
	SANDY LEAN CLAY (CL) FILL - Dark gray, gray, with gravel.	ST			1.00		1.2		105	21				
5	LEAN CLAY (CL) / SANDY LEAN CLAY (CL) - Soft to stiff, Gray, brown, with sand seams. With gravel 2 to 8 feet.	ST			1.50					19	44	16	27	85
		ST			1.00					19				
	Reddish brown, light gray from 8 to 13 feet.	ST			3.00					15				
10														
15	SILTY SAND (SM) - Medium dense, light brown, gray, with clay pockets.	SS		7-10-8 (18)						20				
20		SS		5-7-12 (19)						22	NP	NP	NP	13
25		SS		8-9-12 (21)						23				
30	FAT CLAY (CH) - Stiff, light gray, reddish brown, with sand seams.	SS		3-8-10 (18)						24				
35	SANDY SILTY CLAY (CL-ML) - Firm to very stiff, light gray, reddish brown.	ST			1.50					18				
40		SS		18-16-20 (36)						19				
	Bottom of hole at 40.0 feet.													

UES Professional Solutions 44, LLC
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BORING NUMBER B-03

CLIENT Galena Park ISD
PROJECT NAME Galena Park High School - Phase 3A
PROJECT NUMBER H251673-1
PROJECT LOCATION Galena Park, TX 77547
DATE STARTED 4/14/25 **COMPLETED** 4/14/25
GROUND ELEVATION ______ **NORTHING** ______
CONTRACTOR UES
GROUND WATER LEVELS: **EASTING** ______
METHOD ______
▽ **INITIALLY ENCOUNTERED** 13.0 ft
LOGGED BY JA **CHECKED BY** V.G.
▼ **AFTER 15 MIN.** 12.0 ft
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		SANDY LEAN CLAY (CL) FILL - Hard, dark brown, gray, with root fibers and gravel.	ST			4.50+					9				
		FAT CLAY (CH) - Stiff, gray, reddish brown. With sand seams from 2 to 4 feet.	ST			2.00					23	64	20	44	
5		Brownish yellow, light gray from 4 to 6 feet.	ST			2.50					20				
		SANDY LEAN CLAY (CL) - Stiff, light brown, light gray, with sand seams.	ST			2.50					17				
10		CLAYEY SAND (SC) - Firm, light brown.	ST			1.50					17	26	15	11	49
15		LEAN CLAY WITH SAND (CL) - Very stiff, brownish yellow, light gray, with sand seams.	ST			3.50					19				
20		FAT CLAY (CH) - Stiff to very stiff, reddish brown, light gray, with sand seams.	SS		4-6-10 (16)						24				
25			ST			3.50					23				
30		Light gray from 28 to 33 feet.	ST			3.00		1.5		110	20				
35		SANDY SILTY CLAY (CL-ML) - Stiff, light gray.	SS		7-6-6 (12)						18				
40			SS		7-10-8 (18)						27				
		Bottom of hole at 40.0 feet.													

UES Professional Solutions 44, LLC
15811 Tuckerton Road, Houston, TX 77095
Telephone: 713-360-0460; Fax: 713-360-0481

BORING NUMBER B-04

CLIENT Galena Park ISD
PROJECT NAME Galena Park High School - Phase 3A
PROJECT NUMBER H251673-1
PROJECT LOCATION Galena Park, TX 77547
DATE STARTED 4/3/25 **COMPLETED** 4/3/25
GROUND ELEVATION **NORTHING**
CONTRACTOR UES
GROUND WATER LEVELS: **EASTING**
METHOD
INITIALLY ENCOUNTERED 12.0 ft
LOGGED BY JA **CHECKED BY** V.G.
AFTER 15 MIN. 9.0 ft
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		PAVEMENT - 7 inch thick Ashphalt.	RC												
		SOIL BASE MATERIAL - 7 inches thick.	AU												
		SANDY LEAN CLAY (CL) FILL - Dark brown, with sand seams.	AU								15				
		FAT CLAY (CH) - Firm, gray, reddish brown, with sand seams.	SS		3-5-3 (8)						21				
5		LEAN CLAY (CL) - Soft to stiff, brownish yellow, with sand seams.	ST			1.00					19	49	16	33	
			ST			1.00					17				
			ST			3.00					17	39	18	21	
10															
15		SILTY SAND (SM) - Loose to midum dense, brown, light brown.	SS		4-4-5 (9)						17				
20			SS		5-7-7 (14)						20				
25			SS		4-6-5 (11)						20				
		Bottom of hole at 25.0 feet.													

UES Professional Solutions 44, LLC
15811 Tuckerton Road, Houston, TX 77095
Telephone: 713-360-0460; Fax: 713-360-0481

BORING NUMBER B-05

CLIENT Galena Park ISD
PROJECT NAME Galena Park High School - Phase 3A
PROJECT NUMBER H251673-1
PROJECT LOCATION Galena Park, TX 77547
DATE STARTED 4/11/25 **COMPLETED** 4/11/25
GROUND ELEVATION **NORTHING**
CONTRACTOR UES
GROUND WATER LEVELS: **EASTING**
METHOD
INITIALLY ENCOUNTERED 15.0 ft
LOGGED BY JA **CHECKED BY** V.G.
AFTER 15 MIN. 13.0 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	PAVEMENT - 4.5 inch thick Ashphalt.	RC												
	SILTY SAND (SM) FILL - Dark brown, with root fibers.	AU								12	19	16	3	38
		ST			1.50					21				
	LEAN CLAY (CL) - Firm, gray, reddish brown, with sand seams.	ST			1.50					22				
	FAT CLAY (CH) - Firm to stiff, gray, dark gray.	ST			1.50					20	53	17	36	
10	Light gray, reddish brown from 8 to 18 feet.	ST			2.00					24				
		ST			2.00		2.8		106	20				
20	Reddish brown, light gray, with sand seams from 18 to 28 feet.	ST			3.00					23				
		ST			3.00					28				
30	LEAN CLAY WITH SAND (CL) - Stiff, light gray, reddish brown, with sand seams and layers.	ST			2.00					17				
	With calcareous nodules from 33 to 38 feet.	ST			2.50					19				
40	CLAYEY SAND (SC) - Loose, light brown, reddish brown.	SS		3-4-6 (10)						26				
	SILTY SAND (SM) - Medium dense, reddish brown.	SS		7-7-11 (18)						24				
50	SANDY SILTY CLAY (CL-ML) - Hard, reddish brown, light brown.	SS		20-42-27 (69)						27	23	17	6	51
	SANDY FAT CLAY (CH) - Very stiff, reddish brown.	SS		17-20-23 (43)						30				
										28				
60	Reddish brown, light gray from 58 to 60 feet.	SS		17-19-20 (39)						28				
	Bottom of hole at 60.0 feet.													

ABSORPTION SWELL TEST (ASTM D4546) RESULTS						
Boring No.	B-01	B-03	B-04			
Average Sample Depth (ft)	7	3	9			
Sample Height (in)	1	1	1			
Sample Diameter (in)	2.5	2.5	2.5			
Initial Sample Volume (cu in)	4.91	4.91	4.91			
Initial Sample Weight (gr)	155.2	153.4	166.2			
Initial Moisture (%)	22	27	17			
Final Moisture (%)	22	29	17			
Initial Wet Unit Weight (pcf)	120	119	129			
Initial Dry Unit Weight (pcf)	99	94	110			
Applied Over Burden (psi)	6.1	2.6	7.8			
Initial Dial Reading (in)	0.0000	0.0000	0.0000			
Final Dial Reading (in)	0.0000	0.0040	0.0030			
Swell (%)	0.00	0.40	0.30			

Appendix D - Aerial Photographs

AERIAL PHOTOGRAPH - 1944

AERIAL PHOTOGRAPH - 1953

AERIAL PHOTOGRAPH - 1978

AERIAL PHOTOGRAPH - 1995

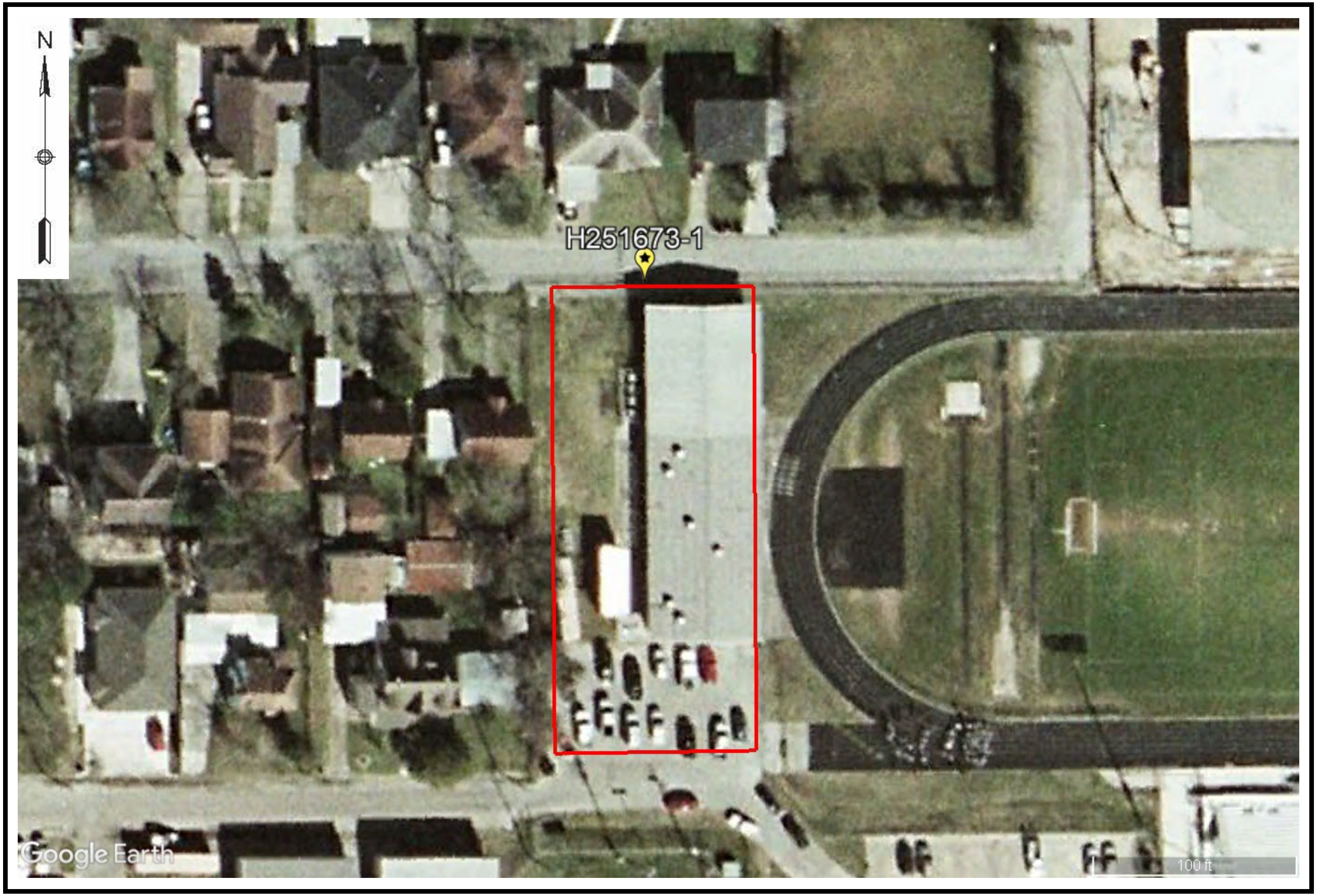
UES Project No. H251673-1

Galena Park High School – Phase 3A

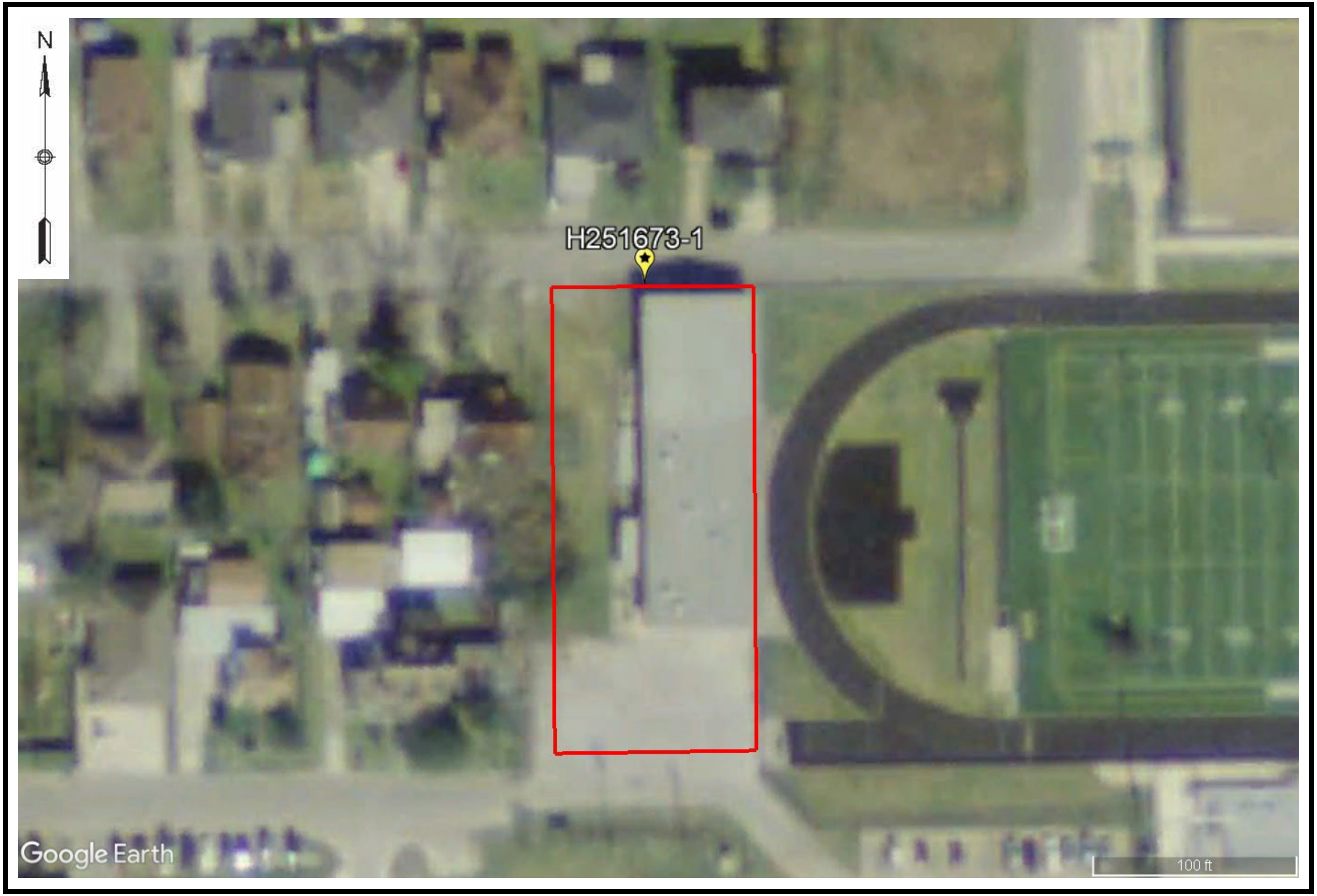
UES

AERIAL PHOTOGRAPH - 2002

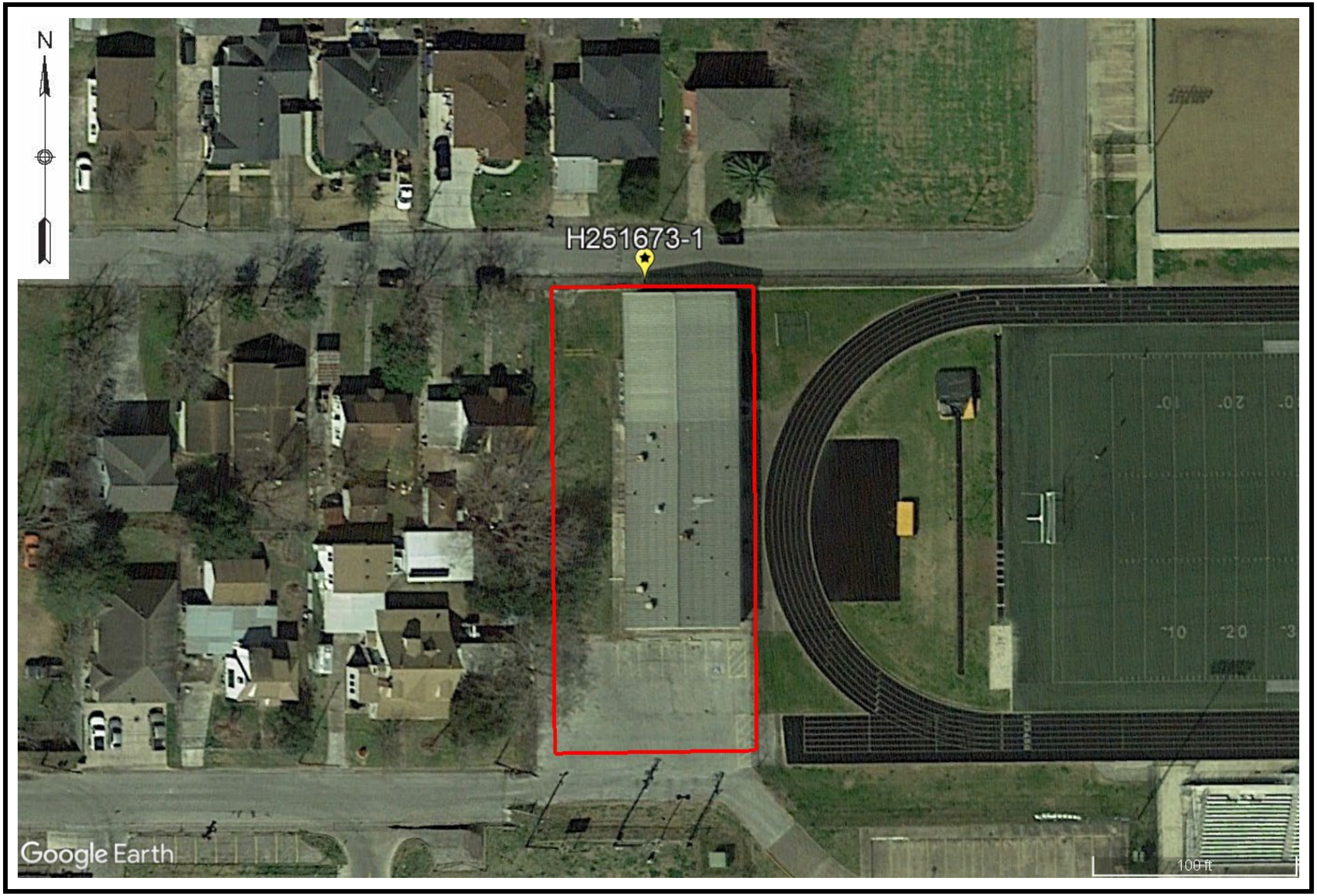
AERIAL PHOTOGRAPH - 2004



AERIAL PHOTOGRAPH - 2009

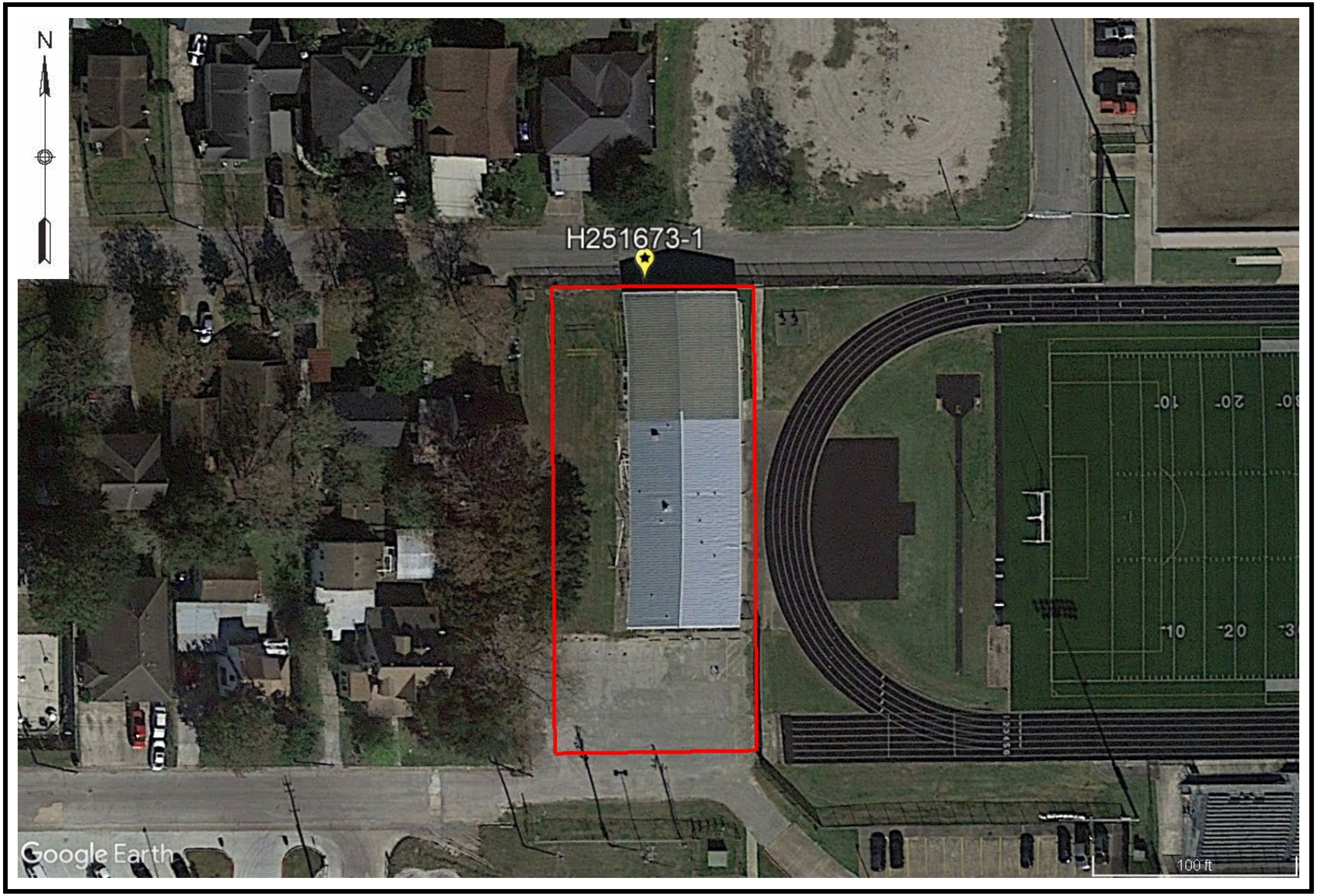


AERIAL PHOTOGRAPH - 2014



UES Project No. H251673-1 Galena Park High School – Phase 3A

AERIAL PHOTOGRAPH - 2019

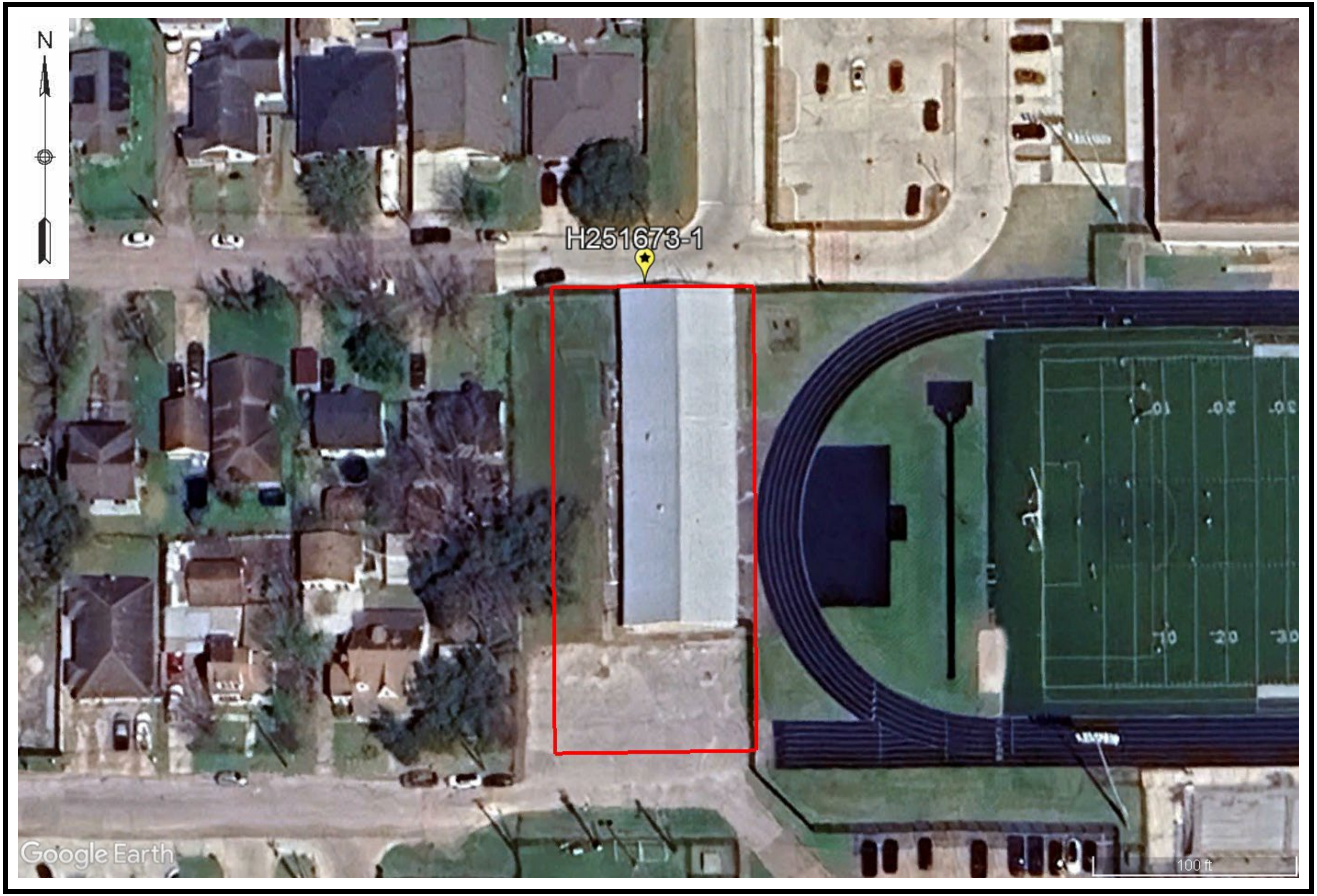


UES Project No. H251673-1

Galena Park High School – Phase 3A

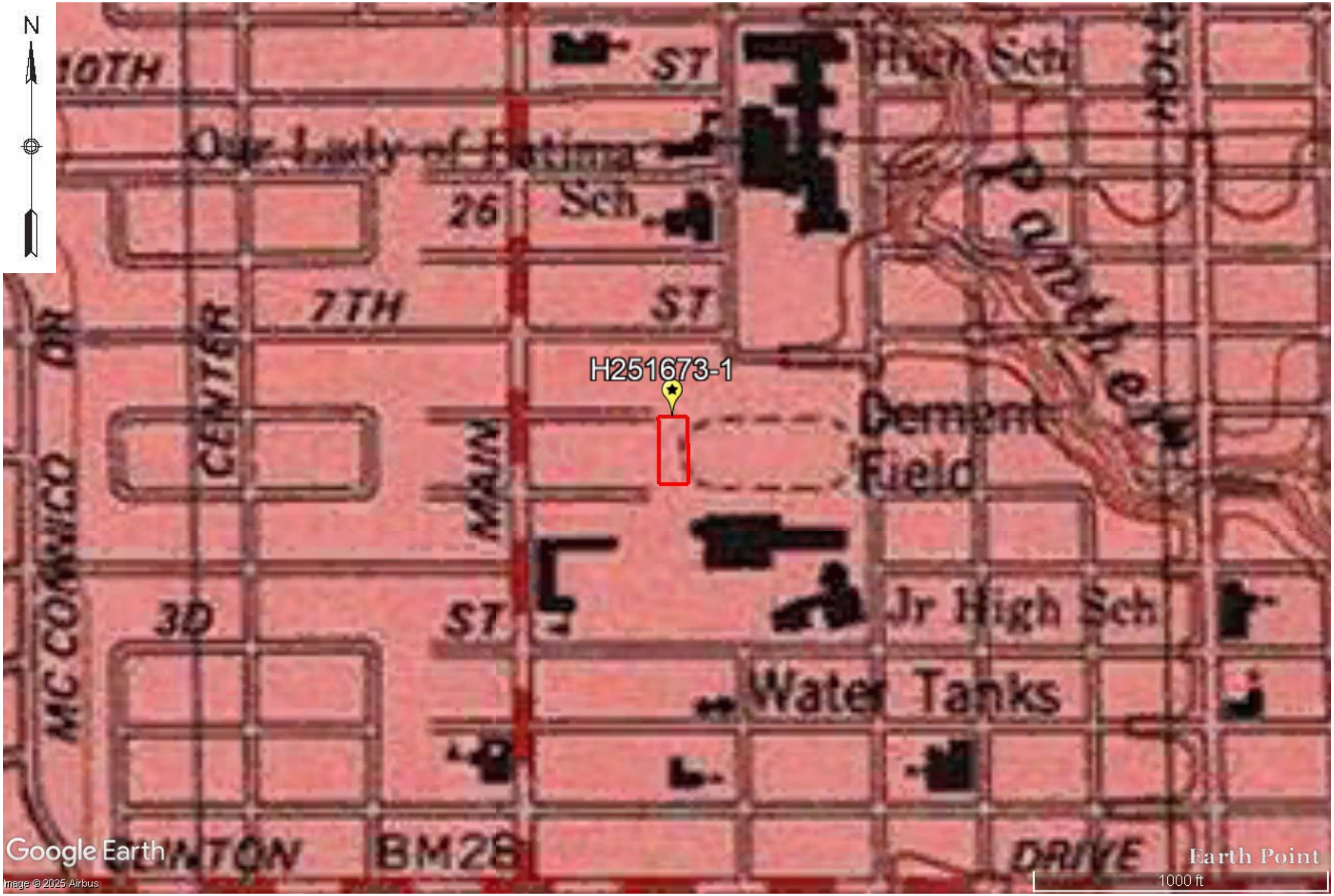
UES

AERIAL PHOTOGRAPH - 2025



Appendix E - USGS Topographic Map

USGS TOPOGRAPHIC MAP



Appendix F - Site Photographs

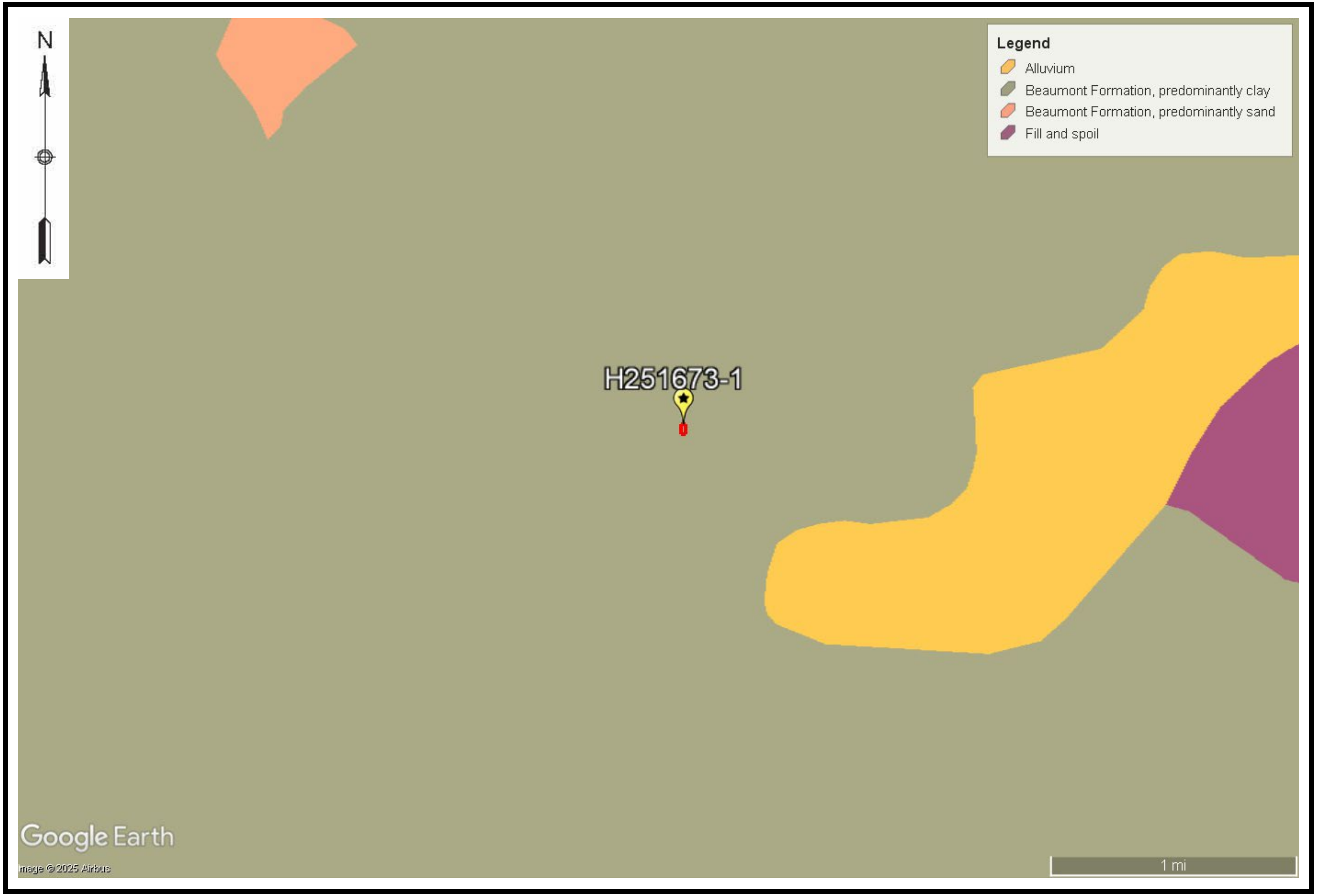
SITE PHOTOGRAPHS

Facing North at Boring B-01

Facing South at Boring B-03

Appendix G - Geologic Information

GEOLOGIC ATLAS



Mineral Resources On-Line Spatial Data

Beaumont Formation, areas predominantly clay

Beaumont Formation, areas predominantly clay

State	Texas
Name	Beaumont Formation, areas predominantly clay
Geologic age	Phanerozoic | Cenozoic | Quaternary | Pleistocene-Late
Original map label	Qbc
Comments	On McAllen-Brownsville Sheet (1976) dominantly clay and mud of low permeability. (from Moore and Wermund, 1993a, 1993b): Light- to dark-gray and bluish- to greenish-gray clay and silt, intermixed and interbedded; contains beds and lenses of fine sand, decayed organic matter, and many buried organic-rich, oxidized soil(?) zones that contain calcareous and ferruginous nodules. Very lt. gray to v. lt. yell-gray sediment cemented by calcium carbonate present in varied forms, veins, laminar zones, burrows, root casts, nodules. Locally, small gypsum crystals present. Includes plastic and compressible clay and mud deposited in flood basins, coastal lakes, and former stream channels on a deltaic plain. Disconformably overlies Lissie Fm. Thickness 5-10 m along north edge of outcrop; thickens southward in subsurface to more than 100 m.
Primary rock type	clay or mud
Secondary rock type	silt
Other rock types	
Lithologic constituents	Major Unconsolidated > Fine-detrital > Silt *(Bed)* Unconsolidated > Fine-detrital > Clay *(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)]

Bureau of Economic Geology, 1976, McAllen-Brownsville Sheet, Geologic Atlas of Texas, Bureau of Economic Geology, University of Texas at Austin, 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 Aransas - Austin - Bee - Brazoria - Calhoun - Cameron - Chambers - Colorado - Fort Bend - Galveston - Hardin - Harris - Hidalgo - Jackson - Jasper - Jefferson - Jim Wells - Kenedy - Kleberg - Liberty - Live Oak - Matagorda - Newton - Nueces - Orange - Refugio - San Patricio - Victoria - Waller - Wharton - Willacy

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

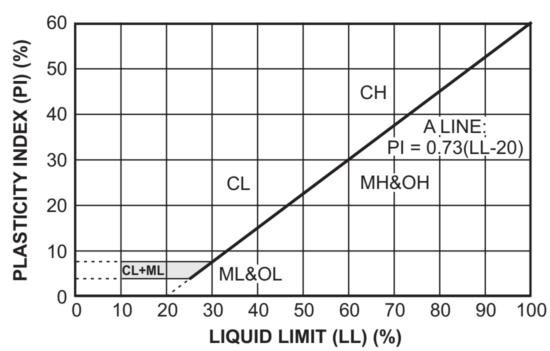
LABORATORY CLASSIFICATION CRITERIA

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%