GEOTECHNICAL EXPLORATION

KRUM ISD – HIGH SCHOOL BAND HALL AND CTE ADDITIONS
700 Bobcat Boulevard
Krum, Texas
UES Report No. W240767-rev1
June 12, 2024

Prepared for:

KRUM ISD
1200 Bobcat Boulevard
Krum, Texas 76249
Attention: Dr. Jason Cochron

Prepared By:

UES™

June 12, 2024

Krum ISD
1200 Bobcat Boulevard
Krum, Texas 76249
Attention: Dr. Jason Cochron

Re: Geotechnical Exploration
Krum ISD – High School Band Hall and CTE Additions
700 Bobcat Boulevard
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Attached is the report of the geotechnical exploration performed for the project referenced above. This study was authorized by Dr. Jason Cochran on February 16, 2024 and performed in accordance with accordance with UES Professional Solutions 44, LLC (hereinafter UES) Proposal No. 102619-rev1, dated February 8, 2024.

The purpose of this revision is to include slab foundation recommendations for the CTE building additions at the request of the Structural Engineer.

This report contains results of field explorations and laboratory testing and an engineering interpretation of these with respect to available project characteristics. The results and analyses were used to develop recommendations to aid design and construction of foundations and pavement.

UES Professional Solutions 44, LLC 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.

Sincerely,
UES PROFESSIONAL SOLUTIONS 44, LLC

STATE OF TEXAS
GREGORY S. FAGAN
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LICENSED
PROFESSIONAL ENGINEER
June 12, 2024

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TABLE OF CONTENTS

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1.0 PURPOSE AND SCOPE

The purpose of this geotechnical exploration is for UES PROFESSIONAL SOLUTIONS 44, LLC (UES) to evaluate for Krum ISD (Client) some of the physical and engineering properties of subsurface materials at selected locations on the subject site with respect to formulation of appropriate geotechnical design parameters for the proposed construction. The field exploration was accomplished by securing subsurface samples from widely spaced test borings performed across the expanse of the site. Engineering analyses were performed from results of the field exploration and results of laboratory tests performed on representative samples.

Also included are general comments pertaining to reasonably anticipated construction problems and recommendations concerning earthwork and quality control testing during construction. This information can be used to evaluate subsurface conditions and to aid in ascertaining construction meets project specifications.

Recommendations provided in this report were developed from information obtained in test borings depicting subsurface conditions only at the specific boring locations and at the particular time designated on the logs. Subsurface conditions at other locations may differ from those observed at the boring locations, and subsurface conditions at boring locations may vary at different times of the year. The scope of work may not fully define the variability of subsurface materials and conditions that are present on the site.

The nature and extent of variations between borings may not become evident until construction. If significant variations then appear evident, our office should be contacted to re-evaluate our recommendations after performing on-site observations and possibly other tests.

2.0 PROJECT CHARACTERISTICS

It is proposed to construct a single-story band hall building with a plan area of 17,500 SF and two (2) additions to the existing pre-engineered metal welding building (CTE), each with a respective plan area of about 3,100 SF and 4,800 SF. The site is located at the existing Krum High School located at 700 Bobcat Boulevard in Krum, Texas. A site plan illustrating the general outline of the property is provided as Figures 1A and 1B, the Boring Location Plan, in the Appendix.

At the time the field exploration was performed, the site consisted of an existing high school and associated parking and drives. The site in the vicinity of Borings 1 and 2 consisted of a grassy area north of the existing school. Borings 3 and 4 were located on grassy and gravel paved areas adjacent to the existing building. Historical aerial images available from Google Earth® indicate grading and clearing previously occurred in the project areas. No information regarding previous development in the project area was provided to us. Cursory visual observations and review of topographical maps provided by www.dfwmaps.com indicate the band hall project area generally slopes down towards the southeast about 4 ft (Appx. Elev. 724 ft to Appx. Elev. 720 ft) and the CTE metal welding building additions project area is generally level (Appx Elev. 716 ft).

We understand the buildings will be designed for about 1 inch of post-construction seasonal movement. Structural loading information was not available for this study. We have assumed maximum column loads of 250 kips. No below grade slabs are planned. Pavement for the project will consist of portland cement concrete (PCC). Grading plans were not provided for this study. For the purpose of our analysis, we have assumed maximum cuts and fills of 2 ft to achieve final grades.

3.0 FIELD EXPLORATION

Subsurface conditions on the site were explored by drilling a total of four (4) test borings. Table A contains a summary of borings, depths and associated structures.

TABLE A Boring Locations and Depth Summary		
Structure	**Boring Nos.**	**Boring Depths (ft)**
Future Band Hall	1 and 2	45
CTE Building Additions	3 and 4	45

The test borings were performed in general accordance with ASTM D 420 using standard rotary drilling equipment. The approximate location of each test boring is shown on the Boring Location Plan, Figures 1A and 1B, enclosed in the Appendix. Details of drilling and sampling operations are briefly summarized in Methods of Field Exploration, Section A-1 of the Appendix.

Subsurface types encountered during the field exploration are presented on the Log of Boring sheets (boring logs) included in the Appendix. The boring logs contain our Field Technician's and Engineer's interpretation of conditions believed to exist between actual samples retrieved. Therefore, the boring logs contain both factual and interpretive information. Lines delineating subsurface strata on the boring logs are approximate and the actual transition between strata may be gradual.

4.0 LABORATORY TESTS

Selected samples of the subsurface materials were tested in the laboratory to evaluate their engineering properties as a basis in providing recommendations for foundation design and earthwork construction. A brief description of testing procedures used in the laboratory can be found in Methods of Laboratory Testing, Section B-1 of the Appendix. Individual test results are presented on the Log of Boring sheets enclosed in the Appendix.

5.0 GENERAL SUBSURFACE CONDITIONS

Based on geological maps available from the Bureau of Economic Geology, published by The University of Texas at Austin, the project site lies within the undivided Pawpaw, Weno Limestone, and Denton Clay formation. This undivided formation generally consist of alternating layers of limestone and marl (limey shale). Residual overburden soils associated with these undivided formations generally consist of clay soils characterized by moderate to high shrink-swell potential.

Subsurface conditions encountered in the borings generally consisted of clay and shaly clay to depths of about 19 ft to 21 ft below the ground surface underlain by limestone or shale extending to the 45 ft termination depth of the borings. The upper 4 ft and 6 ft of clay in Borings 1 and 4, respectively, was visually classified as fill material. More detailed stratigraphic information is presented on the Log of Boring sheets.

Most of the materials encountered in the borings are considered relatively impermeable and are expected to have a relatively slow response to water movement. Therefore, several days of observation would be required to evaluate actual groundwater levels within the depths explored. The groundwater level at the site is anticipated to fluctuate seasonally depending on the amount of rainfall, prevailing weather conditions and subsurface drainage characteristics.

Free groundwater was not encountered in the borings. However, it is common to encounter seasonal groundwater in fill materials, from natural fractures within the clayey matrix, at the soil/rock (limestone and/or shale) interface or from fractures in the rock (limestone and/or shale), particularly during or after periods of precipitation. If more detailed groundwater information is required, monitoring wells or piezometers can be installed.

Further details concerning subsurface materials and conditions encountered can be obtained from the Boring Logs provided in the Appendix.

6.0 DESIGN RECOMMENDATIONS

The following design recommendations were developed on the basis of the previously described Project Characteristics (Section 2.0) and General Subsurface Conditions (Section 5.0). Should the project criteria change, our office should conduct a review to determine if modifications to the recommendations are required.

The following design criteria was developed assuming grading will consist of maximum cuts and fills of 2 ft. Cutting or filling on the site more than 2 ft can alter the recommended design parameters. Therefore, it is recommended our office be provided with a copy of final grading plans to verify appropriate design parameters are utilized for final design.

6.1 Existing Fill and Differential Movements

Existing Fill

Existing fill was encountered to depths of about 4 ft and 6 ft below the ground surface in Borings 1 and 4, respectively. If compaction records for this fill cannot be obtained, the fill should be treated as uncontrolled fill. Uncontrolled fill is generally not suitable for support of foundations, floor slabs, synthetic turf surfaces, or other grade supported structures sensitive to settlement due to the risk of under-compacted zones resulting in failures of weak soil and/or indeterminate levels of settlement. Any existing uncontrolled fill should be removed from below areas sensitive to settlement and replaced with engineered fill as recommended in Section 6.4 or Section 7.3 as applicable. The excavated materials may be suitable for reuse as engineered fill provided they are free of organics, boulders, rubble, and other debris.

The lateral extent, depth and nature of the fill is not known. Test pits could be performed prior to construction to verify the lateral extent, depth, and nature of the fill. ALPHA would be pleased to provide this service if desired. It is generally not required to remove and replace uncontrolled fill below structurally suspended floor slabs.

Differential Movements

Plans were not provided to us for the existing building foundations. However, we understand the existing building is supported on a slab foundation. Differential movements can occur between the existing building and the proposed additions even if the additions are constructed with a similar foundation as the existing building. Methods should be implemented to allow for possible differential movement between the foundation system of the existing building and the new additions. Further, preventative measures should be taken to avoid damaging or adversely affecting the integrity of the existing foundation system during construction activities.

6.2 Drilled, Straight-Shaft Piers

Our findings indicate the structural frame and walls of the buildings/additions could be supported using a system of drilled, straight-shaft piers bearing at least 3 ft into gray shale or limestone. Gray shale or limestone was encountered at depths of about 19 ft to 21 ft below the ground surface in Borings 1 through 4. Deeper penetrations will be required to develop sufficient skin friction and/or uplift resistance. Allowable end bearing and skin friction parameters are provided in Table B.

TABLE B Allowable End Bearing and Skin Friction Parameters			
Bearing Stratum	Allowable End Bearing (ksf)	Skin Friction in Compression (ksf)[1]	Skin Friction in Uplift Resistance (ksf)[1]
At least 3 ft into Gray Shale or Limestone	20	3.0	2.6
Gray Shale or Limestone (at least 35 ft below existing grade)	30	4.5	3.8
[1]Skin friction should be neglected in the upper 3 ft of gray shale or limestone and above the bottom of temporary casing.			

At least two (2) pier shaft diameters should remain between the bottom of the pier and the termination depth of our deepest boring (about 45 ft below existing grade) to use the allowable end bearing parameters. If the minimum clearance between the bottom of the pier and the deepest boring is not provided, piers should be designed as friction piers, neglecting end bearing. In no case should piers bear deeper than the deepest boring (about 45 ft below existing grade). Deeper borings will be required to verify the bearing stratum below about 45 ft below existing grade or if deeper piers are planned.

The minimum clear spacing between piers should be at least two (2) pier shaft diameters, based on the larger pier, to develop the full load carrying capacity from skin friction. The allowable skin friction should be reduced by 50 percent for piers with adjacent touching edges. The allowable skin friction can be interpolated between 100 percent and 50 percent for piers spaced between two (2) pier shaft diameters and piers with adjacent touching edges.

The allowable bearing pressures in Table B have a factor of safety of at least 3 and the skin friction values have a factor of safety of at least 2. Normal elastic settlement of piers under loading is estimated at less than about 1 inch.

Each pier should be sufficiently embedded into the bearing stratum and should be designed with full length reinforcing steel 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 the average uplift adhesion will not exceed about 2.2 kips per sq ft. This soil adhesion is approximated to act uniformly over the portion of the pier shaft in contact with clay soils to a maximum depth of 12 ft below the ground surface. A reduced uplift adhesion of 1.0 kips per sq ft can be used for the pier shaft bearing against moisture improved soil as discussed in Section 6.4. Uplift adhesion can be neglected over any portion of the pier shaft in contact with non-expansive fill.

Table C contains L-PILE design parameters for design of lateral resistance of drilled piers. Lateral resistance should be neglected within 6 ft of final grade due to potential soil shrinkage and/or disturbance.

TABLE C Design Parameters for L-PILE			
Material	**Clay Soils Deeper than 6 ft Below Final Grade**	**Gray Shale or Limestone**	**Gray Shale or Limestone (at least 35 ft below existing grade)**
L-Pile p-y Model	Stiff clay	Weak rock	Weak rock
Effective Unit Weight (γ), pci	0.069	0.078	0.078
Undrained Cohesion (c), psi	5.0	-	-
Rock Uniaxial Compressive Strength (qu), psi	-	160	250
Rock Mass Modulus (Er), psi	-	16,000	25,000
Rock Quality Designation (RQD)[1], %	-	50-70	60-80
Rock Strain Factor (krm)	-	0.0001	0.0005
[1] Rock Quality Designation (RQD) is based on our area experience and the results of the field exploration.			

All grade beams connecting piers should be formed and not cast in earthen trenches. Grade beams should be formed with a nominal 16 inch and 12 inch void at the bottom of the band hall and CTE building additions, respectively. A reduced void space of 6 inches can be used if the subgrade is improved as discussed in Section 6.4. Commercially available cardboard box forms (cartons) are made for this purpose. The cardboard cartons should extend the full length and width of the grade beams. Prior to concrete placement, cartons should be inspected to verify they are firm, properly placed, and capable of supporting wet concrete. Some type of permanent soil retainer, such as pre-cast concrete panels, must be provided to prevent soils adjacent to grade beams from sloughing into the void space at the bottom of the grade beams. Additionally, backfill soils placed adjacent to grade beams must be compacted as outlined in Section 7.3.

6.3 Potential Seasonal Movements and Floor Slabs with Drilled Piers

Our findings indicate soil-related potential seasonal movements of the floor slab constructed within 2 ft of could be:

- about 8 inches in the band hall (Borings 1 and 2)
- about 6 inches in the CTE building additions (Borings 3 and 4)

This estimate of potential movement is based on the assumption that any fill used to raise the grade consists of onsite or similar soils with a plasticity index of 45 or less. Use of fill material with a higher plasticity index could result in potential movements exceeding our estimates.

Potential seasonal movements were estimated in general accordance with methods outlined by Texas Department of Transportation (TxDOT) Test Method Tex-124-E, from results of absorption swell tests and engineering judgment and experience. The 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 methods outlined in Texas Department of Transportation Test Method 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 herein could occur if the existing soils are exposed to an extended dry period, 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 migration from off-site locations.

In view of these potential seasonal movements, the most positive floor system for the structure supported on piers is a slab suspended completely above the existing expansive soils on drilled piers as discussed in Section 6.4.1.

As an alternative, movements could be reduced to about 1 inch by placing a minimum 2 ft cap of non-expansive fill between the bottom of the floor slab and the top surface of moisture conditioned, water pressure injected or chemical injected soil extending to a depth of 10 ft. Moisture conditioning is described in Section 6.4.2, water pressure injection is described in Section 6.4.3 and chemical injection is described in Section 6.4.4. Non-expansive fill could consist of select fill or flexible base material as described in Section 7.3. In choosing this method of foundation movement reduction, the Owner is accepting some post construction seasonal movement of the foundation (about 1 inch).

If a soil-supported floor slab is utilized for the planned building, consideration should be given to a "floating" (fully ground supported, and not structurally connected to walls or foundations) floor slab. This can reduce the risk of cracking and displacement of the floor slab due to differential movements between the slab and foundations. A floor slab doweled into perimeter grade beams can develop a plastic hinge (crack) parallel to and approximately 5 to 10 ft inside the building perimeter. Differential movements can still occur between the grade beam and a “floating” floor slab. The structural engineer should determine the need for connections between the slab and structural elements and determine if control joints to limit cracking are needed. A properly designed and constructed moisture barrier should be placed between the slab and subgrade soils to retard moisture migration through the slab.

6.3.1 Structurally Suspended Floor Slab

In view of these potential seasonal movements, the most positive floor system for the buildings supported on piers is a slab suspended completely above the existing expansive soils. At least 16 inches of void space should be provided between the bottom of the floor slab, or lowest suspended fixture, and top surface of the underlying expansive clays. Provisions should be made for (a) adequate drainage of the under-floor space should be provided in crawl spaces and (b) differential movement of utility lines, including areas where the utility penetrates through the grade beam and/or where the utility penetrates below grade areas.

6.3.2 Subgrade Improvement Using Moisture Conditioning

Potential movements of the floor slab could be reduced to about 1 inch by placing at least 2 ft of non-expansive fill between the bottom of the floor slab and top surface of moisture conditioned soils extending to about 10 ft below the ground surface. Non-expansive fill used with moisture conditioning could consist of select fill or flexible base material as described in Section 7.3.

Installation of moisture conditioned soils will require relatively deep excavations adjacent to existing structures and pavement. Care must be taken to shore the existing structures or otherwise prevent them from being undermined during subgrade improvement as described herein.

Moisture-conditioning consists of over-excavating the site soils, then processing and compacting the specified minimum thickness of soil at a “target” moisture content approximated to be at least 5 percent above the material’s optimum moisture content as determined by the standard Proctor method (ASTM D 698). The moisture-conditioned soil, free of debris and any rock fragment greater than 4 inches, should be placed in about 8-inch thick loose lifts and compacted to a dry density of 93 to 97 percent of standard Proctor maximum dry density. Moisture conditioning of the on-site soil should extend throughout the entire building area, below any adjacent flatwork and at least 5 ft beyond the area of floor slab. In entrance areas and at outward swinging doors, moisture conditioning should extend at least 10 ft beyond the perimeter of the work area. However, non-expansive material should not extend beyond the building limits. If flatwork or paving is not planned adjacent to the structure (i.e. above the moisture-conditioned soils), a moisture barrier consisting of a minimum of 10 mil plastic sheeting with 8 to12 inches of soil cover should be provided above the moisture-conditioned soils. Moisture-conditioned soils should be maintained in a moist condition prior to placement of the required thickness of non-expansive material, flatwork or plastic sheeting.

The resulting estimated potential seasonal movement (about 1 inch) was calculated assuming the moisture content of the moisture-conditioned soil varies between the “target” moisture content and the “wet” condition while the deeper undisturbed in-situ soil within the normal zone of seasonal moisture content change varies between the "dry" condition and the "wet" condition as defined by methods outlined in TxDOT Test Method Tex-124-E.

The purpose of moisture-conditioning is to reduce the free swell potential of the moisture-conditioned soil to 1 percent or less. Additional laboratory tests (i.e., standard Proctors, absorption swell tests, etc.) should be conducted during construction to verify the “target” moisture content for moisture-conditioning (estimated at 5 percentage points above the material’s optimum moisture content as defined by ASTM D 698) is sufficient to reduce the free swell potential of the processed soil to 1 percent or less. In addition, it is recommended samples of the moisture-conditioned material be routinely obtained during construction to verify the free swell of the improved material is 1 percent or less.

Installation of moisture-conditioned clays should be monitored and tested on a full-time basis by a representative of ALPHA to verify the soils tested were placed with the proper lift thickness, moisture content, and degree of compaction.

Moisture conditioning should be monitored and tested on a full-time basis by ALPHA to verify materials tested are placed with the proper degree of moisture and compaction as presented in this report. Field density tests should be performed for each lift of fill placed in each building pad area.

6.3.3 Subgrade Improvement Using Water Pressure Injection (WPI)

Water pressure injection (WPI) is an alternative method to pre-swell the subgrade soils to reduce potential seasonal movements. *It is difficult to control the extent of swelling, therefore WPI should not be performed within 15 ft of existing structures.*

As an alternative to moisture conditioning, installation of 10 ft of water pressure injection (WPI) in conjunction with a 2-ft cap of non-expansive fill could reduce potential ground movements to about 1 inch. Non-expansive material could consist of select fill or flexible base material as discussed in Section 7.3 of this report. Complete removal of uncontrolled fill as described in Section 6.1 is required prior to WPI.

Water pressure injection improvement procedures:

- Following removal of the necessary thickness of on-site expansive soils to allow for placement of at least 2 ft of non-expansive fill, the exposed subgrade should be water pressure injected (WPI) to a depth of 10 ft below the bottom of the non-expansive fill. The water pressure injection should extend throughout the entire building area, at least 5 ft beyond the perimeter of the building and below any adjacent flatwork for which it is desired to reduce potential movements. At building entrances and outward swinging doors, WPI should extend at least 10 ft beyond the building perimeter. However, the non-expansive fill should not extend beyond the building limits. Recommended specifications for WPI are attached to this report in the appendix.

- If flatwork or paving is not planned adjacent to the structure (i.e. above the injected soils), a moisture barrier consisting of a minimum of 10 mil plastic sheeting with 8 to 12 inches of soil cover should be provided above the injected soils. Injected soils should be maintained in a moist condition prior to placement of the required thickness of select, non-expansive material or flatwork.

Performance of post-injection swell testing and moisture content determinations should be employed as final acceptance criteria in engineering analysis to examine accomplishment of intended objectives of the injection treatment. Maximum benefit of these movement reduction procedures can be achieved by employing ALPHA to observe, monitor and test the entire process. Construction specifications for the water pressure injection process are provided in the Appendix.

The purpose of water pressure injection is to pre-swell the existing soils. Satisfactory completion of the injection process is achieved when the desired moisture content and abatement of swell in the injected subgrade clay soils are reached. Acceptance criteria for water pressure injection should be based upon obtaining an average free swell of 1 percent or less in the injected zone. Performance of post-injection swell testing and moisture content determinations should be employed as final acceptance criteria in engineering analysis to verify accomplishment of intended objectives of the injection treatment.

The resulting estimated potential seasonal movement (about 1 inch) was calculated assuming the average free swell of the injected soils does not exceed 1 percent. Further, it is assumed the moisture content of the soil below the injected zone and within the normal zone of seasonal moisture content change varies between a "dry" condition and a "wet" condition as defined by TxDOT Test Method Tex-124-E.

6.3.4 Subgrade Improvement Using Chemical Injection

Potential movements of floor slabs constructed adjacent to existing structures could be reduced to about 1 inch by placing a minimum 2-ft cap of non-expansive material between the bottom of floor slab and the top surface of 10 ft of chemical injected soil. Non-expansive fill could consist of select fill or flexible base material as described in Section 7.3. Complete removal of uncontrolled fill as described in Section 6.1 is required prior to chemical injection. The lateral extents of chemical injection, non-expansive fill and plastic sheeting should match that recommended for moisture conditioning, respectively. However, the chemical injection contractor should confirm whether plastic sheeting is required for long term performance of the chemical injection.

Chemical injection consists of injecting the clayey soils with a proprietary chemical specifically formulated for long-term reduction of shrink-swell capacity in expansive clayey soils. The Client should obtain appropriate documentation from the manufacturer indicating the chemical is environmentally safe and long lasting (effective for 10 years or more). Verification that the chemical solution will not heave adjacent structures as a result of the injection process should also be obtained. All references should be obtained and verified. Chemical injection proposals should only be considered from contractors whose chemicals and processes have been studied and shown to be effective by a major U.S. research university.

Satisfactory completion of the injection process will have been achieved when the desired allowable percent free swell has been achieved in the injected soils. In order to reduce overall building pad movements to about 1 inch, the resulting measured free swell of the injected material should not exceed 1 percent. Multiple passes with chemical injection may be required to meet this design requirement. The performance of post-injection free swell testing by UES should be employed as acceptance criteria in engineering analysis to examine accomplishment of the intended objectives of the injection treatment.

Construction specifications as related to the chemical injection process should be provided by the contractor due to the proprietary nature of the chemicals used during the injection process. This includes acceptance criteria and any warranty.

Maximum benefits of this procedure can best be achieved provided the entire process is carefully observed and monitored by UES.

6.4 Slab-on-Grade Foundations (CTE Building Additions)

As an alternative, the CTE building additions could be supported with a slab on grade foundation. A slab foundation will be subject to similar potential movements as a grade supported floor slab as discussed in section 6.3 (about 6 inches). Subgrade improvement as discussed in section 6.3 will be required to reduce potential movements of the slab foundation to about 1 inch. **It should be understood that the risk of movement is higher for a building supported on a slab foundation as compared to a building supported on drilled piers.**

The slab foundations should be designed with exterior and interior grade beams adequate to provide sufficient rigidity to the foundation system. A net allowable soil bearing pressure of 1.5 kips per sq ft may be used for design of grade beams bearing on improved subgrade soils as discussed in Section 6.3. Grade beams should bear a minimum depth of 12 inches below final grade and should have a minimum width of 10 inches for bearing capacity considerations.

To reduce cracking as normal movements, occur in foundation soils, all grade beams and the floor slab should be adequately reinforced with steel (conventional reinforcing steel). It is common to experience some minor cosmetic distress to structures with slab-on-grade foundation systems due to normal ground movements. A properly designed and constructed moisture barrier should be placed between the slab and subgrade soils to retard moisture migration through the slab.

6.4.1 Wire Reinforcement Institute Design Criteria

The foundation could be designed using the Design of Slab-On-Ground Foundations published by the Wire Reinforcement Institute, Inc. (Aug., 1981). WRI parameters are provided in Table D.

Table D
WRI Design Criteria
Potential Seasonal Movement = 1 inch
(After respective subgrade improvement as discussed in Section 6.3)

Design Method	WRI
Climatic Rating (Cw)	18
Effective Plasticity Index	44
Soil Compressive Strength (tsf)	0.5

The structural engineer should verify beam spacing is adequate for support of structural loads and applicable building codes.

6.5 Exterior Flatwork

Flatwork, pavement, and any other soil-supported structural elements will be subjected to the same level of movement discussed in Section 6.4 (about 6 to 8 inches). If this level of movement is not acceptable, flatwork could be suspended on drilled piers as described in Section 6.2. As an alternative, subgrade improvements as discussed in Section 6.4 could be considered for reduction in soil movements in any areas where post-construction movements are critical.

6.6 Seismic Considerations

The Site Class for seismic design is based on several factors that include soil profile (soil or rock), shear wave velocity, and strength, averaged over a depth of 100 ft. Since our borings did not extend to 100-foot depths, we based our determinations on the assumption that the subsurface materials below the bottom of the borings were similar to those encountered at the termination depth of the borings. Based on Section 1613.3.2 of the 2012 International Building Code and

Table 20.3-1 in the 2010 ASCE-7, we recommend using Site Class C (very dense soil and soft rock) for seismic design at this site.

6.7 Area Pavement

To permit correlation between information from test borings and actual subgrade conditions exposed during construction, a qualified Geotechnical Engineer should be retained to provide subgrade monitoring and testing during construction. If there is any change in project criteria, the recommendations contained in this report should be reviewed by our office.

Calculations used to determine the required pavement thickness are based only on the physical and engineering properties of the materials used and conventional thickness determination procedures. Pavement joining buildings should be constructed with a curb and the joint between the building and curb should be sealed. Related civil design factors such as subgrade drainage, shoulder support, cross-sectional configurations, surface elevations, reinforcing steel, joint design and environmental factors will significantly affect the service life and must be included in preparation of the construction drawings and specifications, but all were not included in the scope of this study. Normal periodic maintenance will be required for all pavement to achieve the design life of the pavement system.

Please note, the recommended pavement sections are considered the minimum necessary to provide satisfactory performance based on the expected traffic loading. In some cases, City minimum standards for pavement section construction may exceed those recommended.

6.7.1 Pavement Subgrade Preparation

Based on the soil profile encountered in the borings, we would expect the pavement subgrade could consist of clay. In general, clay with a plasticity index of 15 or greater should be lime stabilized.

The exposed surface of the pavement subgrade soil should be scarified to a depth of 6 inches and mixed with a minimum 7 percent hydrated lime (by dry soil weight) in conformance with TxDOT Standard Specification Item 260. Assuming an in-place unit weight of 100 pcf for the pavement subgrade soils, this percentage of lime equates to about 32 lbs of lime per sq yard of treated subgrade. The actual amount of lime required should be confirmed by additional laboratory tests (ASTM C 977 Appendix XI) prior to construction. The soil-lime mixture should be compacted to at least 95 percent of standard Proctor maximum dry density (ASTM D 698) and within the range of 0 to 4 percentage points above the mixture's optimum moisture content. In all areas where hydrated lime is used to stabilize subgrade soil, routine Atterberg-limit tests should be performed to verify the resulting plasticity index of the soil-lime mixture is at/or below 15.

We recommend subgrade improvement procedures extend at least 1 ft beyond the edge of the pavement to reduce effects of seasonal shrinking and swelling upon the extreme edges of pavement.

Lime stabilization of the pavement subgrade soil will not prevent normal seasonal movement of the underlying untreated materials. Pavement and other flatwork will have the same potential for movement as slabs constructed directly on the existing undisturbed soils. Good perimeter surface drainage with a minimum slope of 2 percent away from the pavement is recommended. Normal maintenance of pavement should be expected over the life of the structures.

6.7.2 Portland Cement Concrete (PCC) Pavement

Following subgrade improvement as recommended in Section 6.7.1, PCC (reinforced) pavement sections are recommended in Table E.

TABLE E
Recommended PCC Pavement Sections

Paving Areas and/or Type	Subgrade Thickness, Inches	PCC Thickness, Inches
Parking Areas Subjected Exclusively to Passenger Vehicle Traffic,	Lime Modified, 6	5
Drive Lanes, Fire Lanes, Bus Lanes, Areas Subject to Light Volume Truck Traffic	Lime Modified, 6	6
Dumpster Traffic Areas, Areas subject to Moderate Volume Truck Traffic,	Lime Modified, 6	7

PCC should have a minimum compressive strength of 3,000 psi at 28 days in parking areas subjected exclusively to passenger vehicle traffic. We recommend a minimum compressive strength of 3,500 psi at 28 days for the drive lanes, bus lanes, fire lanes, and truck areas. Concrete should be designed with 4.5+1.5 percent entrained air. Joints in concrete paving should not exceed 15 ft. Reinforcing steel should consist of No. 3 bars placed at 18 inches on-center in two directions.

6.8 Drainage and Other Considerations

Adequate drainage should be provided to reduce seasonal variations in the moisture content of foundation soils. All pavement and sidewalks within 10 ft of the structure should be sloped away from the building to prevent ponding of water around the foundation. Final grades within 10 ft of the structure should be adjusted to slope away from the structure at a minimum slope of 2 percent. **Maintaining positive surface drainage throughout the life of the structure is essential.**

In areas with pavement or sidewalks adjacent to the new structure, a positive seal must be maintained between the structure and the pavement or sidewalk to minimize seepage of water into the underlying supporting soils. Post-construction movement of pavement and flat-work is common. Normal maintenance should include inspection of all joints in paving and sidewalks, etc. as well as resealing where necessary.

Several factors relate to civil and architectural design and/or maintenance, which can significantly affect future movements of the foundation and floor slab system:

- Preferably, a complete system of gutters and downspouts should carry runoff water a minimum of 5 feet from the completed structure.

- Large trees and shrubs should not be allowed closer to the foundation than a horizontal distance equal to roughly one-half of their mature height due to their significant moisture demand upon maturing.

- Moisture conditions should be maintained constant around the edge of grade slabs. Ponding of water in planters, in unpaved areas, and around joints in paving and sidewalks can cause slab movements beyond those predicted in this report.

  Planter box structures placed adjacent to the building should be provided with a means to assure concentrations of water are not available to the subsoil stratigraphy.

Trench backfill for utilities should be properly placed and compacted as outlined in Section 7.4 and in accordance with requirements of local City standards. Since granular bedding backfill is used for most utility lines, the backfilled trench should not become a conduit and allow access for surface or subsurface water to travel toward the new structures. Concrete cut-off collars or clay plugs should be provided where utility lines cross building lines to prevent water from traveling in the trench backfill and entering beneath the structures.

7.0 GENERAL CONSTRUCTION PROCEDURES AND GUIDELINES

Variations in subsurface conditions could be encountered during construction. To permit correlation between test boring data and actual subsurface conditions encountered during construction, it is recommended a registered Professional Engineering firm be retained to observe construction procedures and materials.

Some construction problems, particularly degree or magnitude, cannot be anticipated until the course of construction. The recommendations offered in the following paragraphs are intended not to limit or preclude other conceivable solutions, but rather to provide our observations based on our experience and understanding of the project characteristics and subsurface conditions encountered in the borings.

7.1 Site Preparation and Grading

Existing fill was encountered to depths of about 4 ft and 6 ft below the ground surface in Borings 1 and 4, respectively. Although not encountered in the borings, existing fill could contain organics, boulders, rubble and other debris which could be encountered during site grading and general excavation. The earthwork and excavation contracts should contain provision for removal of unsuitable materials in the existing fill. Test pit excavations performed prior to construction can be used to evaluate the depth, lateral extent and composition of uncontrolled fill at this site. ALPHA would be pleased to provide this service if desired.

All areas supporting floor slabs, pavement, flatwork or areas to receive new fill should be properly prepared.

- After completion of the necessary stripping, clearing, and excavating, and prior to placing any required fill, the exposed soil subgrade should be carefully evaluated by probing and testing. Any undesirable material (organic material, wet, soft, or loose soil) still in place should be removed.

- The exposed soil subgrade should be further evaluated by proof-rolling with a heavy pneumatic-tired roller, loaded dump truck or similar equipment weighing approximately 20 tons to check for pockets of soft or loose material hidden beneath a thin crust of possibly better soil. Proof-rolling procedures should be observed routinely by a Professional Engineer or his designated representative. Any undesirable material (organic material, wet, soft, or loose soil) exposed during proof-rolling should be removed and replaced with well-compacted material as outlined in Section 7.3.

- Prior to placement of any fill, the exposed soil subgrade should then be scarified to a minimum depth of 6 inches and re-compacted as outlined in Section 7.3.

If fill is to be placed on existing slopes (natural or constructed) steeper than six horizontal to one vertical (6:1), the fill materials should be benched into the existing slopes in such a manner as to provide a minimum bench width of five (5) feet. This should provide a good contact between the existing soils and new fill materials, reduce potential sliding planes, and allow relatively horizontal lift placements.

Even if fill is properly compacted as described in Section 7.3, 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 planning or placing deep fills.

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. 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.

Due to the nature of the clay soils found near the surface at the borings, traffic of heavy equipment (including heavy compaction equipment) may create pumping and general deterioration of shallow soils. Therefore, some construction difficulties should be anticipated during periods when these soils are saturated.

7.2 Foundation Excavations

All foundation excavations should be properly monitored to verify loose, soft or otherwise undesirable materials are removed and foundations will bear on satisfactory material. Soil exposed in the base of all foundation (pier/grade beam) excavations should be protected against detrimental change in condition, such as surface sloughing or side disturbance, rain or excessive drying. Drilled pier foundations should be completed in one day.

Prolonged exposure of the bearing surface to air or water will result in changes in strength and compressibility of the bearing stratum. Therefore, if delays occur, straight shaft pier excavations should be slightly widened and deepened, or a new (deeper) design penetration made to provide a fresh bearing surface.

All pier shafts should be at least 1/30th of the pier length or 1.5 ft in diameter, whichever is greater, for pier stability considerations and to facilitate clean-out of the base and for proper monitoring. Concrete placed in pier holes should be directed through a tremie, hopper, or equivalent. Placement of concrete should be vertical through the center of the shaft without hitting the sides of the pier or reinforcement to reduce the possibility of segregation of aggregates. Concrete placed in piers should have a minimum slump of 6 inches (but not greater than 8 inches) to avoid potential honey-combing.

Observations during pier drilling should include, but not necessarily be limited to, the following items:

- Verification of proper bearing strata and consistency of subsurface stratification with regard to boring logs,

- Confirmation the minimum required penetration into the bearing strata is achieved,
- Complete removal of cuttings from bottom of pier holes,
- Proper handling of any observed water seepage and sloughing of subsurface materials,
- No more than 2 inches of standing water should be permitted in the bottom of pier holes prior to placing concrete, and
- Verification of pier diameter size and steel reinforcement.

Free groundwater was not encountered in the borings. However, the risk of encountering groundwater during pier drilling is increased during or after periods of precipitation. Where submersible pumps or bailing cannot control groundwater, temporary casing may be required to control seepage. The casing should be properly seated below the depth of seepage, and all groundwater and soil should be removed prior to beginning the design penetration. As casing is extracted, care should be taken to maintain a positive head of plastic concrete and minimize the potential for intrusion of water seepage. A separate bid item should be provided for casing on the contractor bid schedule.

7.3 Fill Compaction

Select fill used as non-expansive material in the building pad should have a liquid limit less than 35, a plasticity index (PI) not less than about 4 nor greater than 15 and contain no more than 0.5 percent fibrous organic materials, by weight. All select material should contain no deleterious material and should be compacted to a dry density of at least 95 percent standard Proctor maximum dry density (ASTM D 698) and within the range of 1 percentage point below to 3 percentage points above the material's optimum moisture content. The plasticity index and liquid limit of material used as select non-expansive material should be routinely verified during placement using laboratory tests. Visual observation and classification should not be relied upon to confirm the material to be used as select, non-expansive material satisfies the Atterberg-limit criteria.

Flexible base used as non-expansive material in the building pad should consist of material meeting the requirements of TxDOT Standard Specifications Item 247, Type A or D, Grade 1-2 or 3. The flexible base should be compacted to at least 95 percent of standard Proctor maximum dry density (ASTM D 698) and within the range of 2 percentage points below to 2 percentage points above the material's optimum moisture content. Flexible base material placed deeper than 8 ft below the building floor slab should be compacted to at least 100 percent standard Proctor maximum dry density.

The following recommendations pertain to fill soils placed for general site grading as follows:

- *Outside* the designated flatwork and utility areas if moisture conditioning will be used as the method for subgrade improvement. Where moisture conditioning is utilized for subgrade improvement, all fill within the designated areas and associated adjacent areas should meet the requirements of Section 6.4.2.

- For general grading *including* building areas below the select fill requirement *if* water pressure injection or chemical injection will be used as the method for subgrade improvement.

Clay with a plasticity index equal to or greater than 25 should be compacted to a dry density between 93 and 98 percent of standard Proctor maximum dry density (ASTM D 698). The compacted moisture content of the clays during placement should be within the range of 2 to 6 percentage points above optimum.

Clayey materials used as fill should be processed such that the largest particle or clod is less than 6 inches prior to compaction.

Where mass fills are deeper than 10 ft, 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 percentage points of the material's optimum moisture content. The portion of the fill/backfill shallower than 10 ft should be compacted as outlined herein.

Compaction should be accomplished by placing fill in about 8-inch thick loose lifts and compacting each lift to at least the specified minimum dry density. Field density and moisture content tests should be performed on each lift.

In general site grading areas where final fill slopes will be four horizontal to one vertical (4:1) or steeper and greater than 5 ft in height, field density and moisture content tests should be performed on each lift.

7.4 Utilities

Where utility lines are deeper than 10 ft, 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 pavement over utility lines and/or other areas with deep 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.

7.5 Groundwater

Groundwater was not encountered in the borings. However, from our experience, shallow groundwater seepage could be encountered in excavations for foundations, utilities and other general excavations at this site. The risk of seepage increases with depth of excavation and during or after periods of precipitation. Standard sump pits and pumping may be adequate to control seepage on a local basis.

In any areas where cuts are made, attention should be given to possible seasonal water seepage that could occur through natural cracks and fissures in the newly exposed stratigraphy. In these areas subsurface drains may be required to intercept seasonal groundwater seepage. The need for these or other dewatering devices should be carefully addressed during construction. Our office could be contacted to visually observe final grades to evaluate the need for such drains.

8.0 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 2.0. 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 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

SOIL MODIFICATION
WATER PRESSURE INJECTION (WPI)
GUIDELINE SPECIFICATIONS

Purpose

The purpose of this specification is to provide a procedural basis for using water pressure injection as a method to obtain a relatively uniform, moist, pre-swelled zone of soil beneath the floor slab. Specifically, the intent of this procedure is to reduce the average free swell potential of soils within the injected zone to 1 percent or less.

Material

1. Only potable water shall be used during the entire injection process.

2. A non-ionic surfactant (wetting agent) will be added to the water according to manufacturer's recommendations, but, in no case will proportions be less than one part (undiluted) per 3,500 gallons of water.

Application

1. The water pressure injection work shall be accomplished after the site has been brought to near final subgrade elevation and prior to installation of any plumbing, trenches and utilities.

2. The injection vehicle will have a minimum gross weight of 5 tons and shall be capable of making straight vertical penetrations to minimize pressure loss around the injector rods to at least 10 ft.

3. Injections will be continued to "REFUSAL" (until the maximum reasonable quantity of water has been injected into the soil and water is flowing freely at the surface, either out of previous injection holes or from areas where the surface soils have fractured. The amount of water flowing from the areas described above will be approximately equivalent to the volume of water being pumped into the soil. As a minimum, injections should be at least 30 seconds at each injection interval unless altered by the Geotechnical Engineer).

   Note: Loss of water or blow-back around injector pipes does not constitute refusal. Continued loss of water in this manner may indicate inadequate injection equipment or techniques, or in some instances, surficial soils that will not form an adequate seal to contain the water. In either instance, the owner's representative should be contacted and an on-site observation made to determine appropriate steps to achieve adequate injection.

After completion of water injection, the injection contractor will submit records which reflect the total quantity of water used. The injection contractor will be totally responsible for determining the means and methods of injecting the on-site soils such that the average free swell of soils within the injected zone does not exceed 1 percent.

4. Injection pipe(s) will penetrate the soil in approximately 12 to 18-inch intervals, injecting to refusal at each interval for a total depth of 10 ft or impenetrable material, whichever occurs first. If a seemingly impenetrable layer is encountered, ALPHA must be contacted to evaluate the significance of the lack of penetration with the injector tubes or provide alternate recommendations. A minimum of seven (7) injection intervals will be provided for the 12-ft injection depth. The lower portion of the injection pipe will consist of a hole pattern that will uniformly disperse water throughout the entire depth.

5. Spacing for the injections will not exceed 5 feet on-center each way. Subsequent injections will be offset laterally at one-half the distance in both directions between the original injection centers.

6. Injection pressures should be adjusted to inject the greatest quantity of water possible within a pressure range of 50 - 200 psi pump pressure.

7. After a minimum curing time of 48 hours, the water injected pad shall be tested for moisture content and swell abatement to determine if additional injections with water are necessary. Subsequent water injections will be 5 feet on-center each way and spaced 2½ feet offset in two orthogonal directions from the initial injection.

8. Upon completion of the final water pressure injection, the top surface of the injected pad should be scarified to a depth of at least 6 inches and re-compacted to between 93 and 98 percent of the optimum density, at a moisture content between 2 and 4 percentage points above the optimum values, as defined by ASTM D-698. Compaction tests should be performed at a frequency of one (1) test per 5,000 sq ft with a minimum of two (2) tests.

9. The moisture content of the injected soils will be maintained until the floor slab is placed. Loss of moisture from the surface or sides of the building pad must be prevented by watering or use of a membrane. Any open trenches should be sealed or kept wet to prevent loss of moisture. All trenches should be backfilled with the excavated material. The moisture content of the backfill should be maintained in the range of 2 to 4 percentage points above optimum.

Special Considerations

Several water injections may be required to achieve the desired final moisture content and corresponding soil swell abatement. A minimum 24 hour waiting period should be implemented between water injection passes. Due to variations in the subsurface soils, the number of injection passes required to reduce the swell potential of the injected soils to 1 percent or less is unknown. Hence, the Client should allow for extra construction time on the site considering the time frame required to achieve the desired reduction in swell potential is unknown. Further, the contract with the Injection Contractor should address the situation where more injection passes than predicted are required to achieve the desired result.

Between the time the subgrade is water pressure injected and either the select fill material or plastic sheeting is placed, the upper surface of the injected soil should not be allowed to dry. To allow for adequate pre-swelling of the soils from the injection procedure, concrete for slabs should not be placed above injected areas until at least two (2) weeks following the final water injection. During this two-week period, the surface of the injected soil must be kept moist or covered with plastic sheeting to prevent moisture loss. About 3 to 5 inches of heave can be expected in building pads during and shortly after completion of the injection process.

Additionally, experience indicates injection adjacent to existing structures (such as, but not limited to, buildings, pavements, grade slabs, and buried utility conduits) can result in swelling of soil in the injected zone as well as those beneath existing nearby structures. Swelling of soil supporting existing structures can result in distress (movement) to existing structures. Therefore, if an existing structure or property line is located within 30 ft of the proposed water injection area, it is recommended a temporary vertical moisture barrier be installed longitudinally between the existing structure and the injected pad to prevent injected water from entering the subgrade of the existing structure. The moisture barrier could consist of a 1 ft wide trench, extending 2 ft deeper than the injection depth, backfilled with lean concrete or other suitable relatively impermeable material.

Monitoring

A full-time ALPHA technician should be retained and present throughout the injection operations. Moisture content and free swell samples should be taken at 1-foot intervals to the total depth injected from a minimum of one test boring per each 4,000 sq feet of injected area. The moisture content and shear strength (using a pocket-penetrometer) will be determined for each sample. One-dimension free swell tests (ASTM D 4546-85 Method B) will be performed on selected samples at a frequency of at least three (3) free swell tests per test boring. The free swell tests will be performed with a surcharge equal to the overburden pressure anticipated upon completion of the new structure. Based upon the test results, the current swell potential of the injected soils should be determined by the project Geotechnical Engineer. Acceptance criteria for water pressure injection will be based upon achieving the potential movements indicated in the Geotechnical Exploration. As a guide, an average free swell of 1 percent or less in the injected zone could be used for planning. However, due to variations in the soils across the site, an average free swell of more than 1 percent may be allowable in some areas. Acceptance of soils with average free swells of more than 1 percent should be evaluated by ALPHA. Depending upon the moisture content and the potential swell remaining in the existing injected soils, additional injections with water containing surfactant may be required until these requirements are met.

Wet and soft surface conditions resulting from the water injection procedures will require the contractor to provide access to drilling equipment used to obtain the soil samples which verify the injection process. Special track equipment may be required to provide the required access. The contractor will be responsible for providing and operating suitable equipment to permit sampling of the injected soils (test borings) with a standard truck-mounted drilling rig.

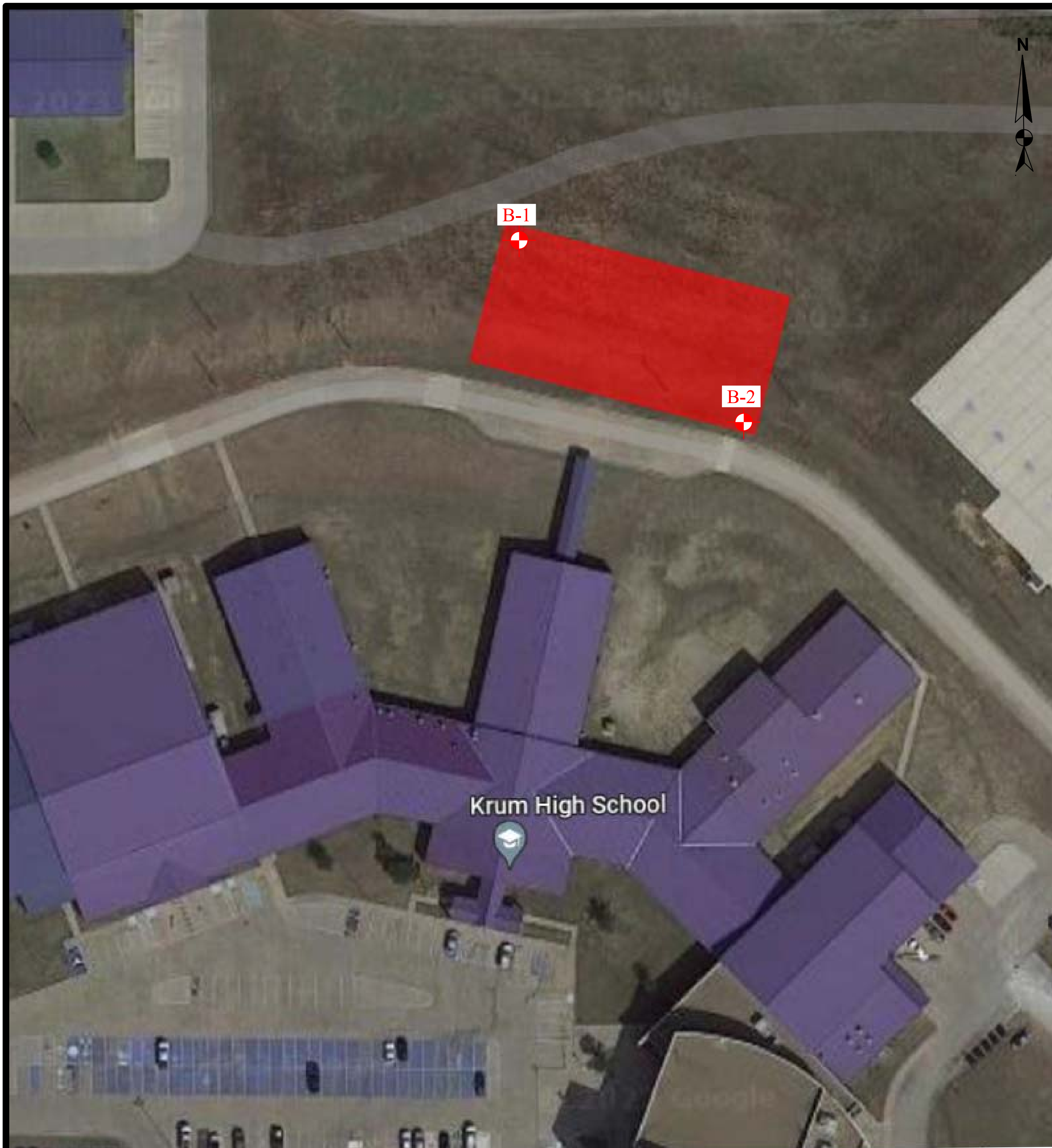
A-1 METHODS OF FIELD EXPLORATION

Using standard rotary drilling equipment, a total of four (4) test borings were performed for this geotechnical exploration. The approximate locations of the borings are shown on the Boring Location Plan, Figures 1A and 1B. The test boring locations were staked by either pacing or taping and estimating right angles from landmarks which could be identified in the field and as shown on the site plan provided during this study. The locations of test borings shown on the Boring Location Plan are considered accurate only to the degree implied by the methods used to define them.

Relatively undisturbed samples of the cohesive subsurface materials were obtained by hydraulically pressing 3-inch O.D. thin-wall sampling tubes into the underlying soils at selected depths (ASTM D 1587). These samples were removed from the sampling tubes in the field and evaluated visually. One representative portion of each sample was sealed in a plastic bag for use in future visual evaluation and possible testing in the laboratory.

Texas Department of Transportation Texas Cone Penetration (TCP) tests were completed in the field to determine the apparent in-place strength characteristics of the rock type materials. A 3-inch diameter steel cone driven by a 170-pound hammer dropped 24 inches is the basis for TxDOT strength correlations. Depending on the resistance (strength) of the materials, either the number of blows of the hammer required to provide 12 inches of penetration, or the inches of penetration of the cone due to 100 blows of the hammer are recorded on the field logs and are shown on the Log of Boring sheets as “TX Cone” (reference: TxDOT Test Method TEX 132-E).

Logs of the borings are included in the Appendix. The logs show visual descriptions of subsurface strata encountered using the Unified Soil Classification System. Sampling information, pertinent field data, and field observations are also included. Samples not consumed by testing will be retained in our laboratory for at least 14 days and then discarded unless the Client requests otherwise.



GEOTECHNICAL EXPLORATION
KRUM - HIGH SCHOOL BAND HALL AND CTE ADDITIONS
700 BOBCAT BOULEVARD
KRUM, TEXAS
UES PROJECT NO. W240767

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FIGURE 1A

BORING LOCATION PLAN

APPROXIMATE BORING LOCATION

GEOTECHNICAL EXPLORATION
KRUM ISD - HIGH SCHOOL BAND HALL AND CTE ADDITIONS
700 BOBCAT BOULEVARD
KRUM, TEXAS
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FIGURE 1B

BORING LOCATION PLAN

APPROXIMATE BORING LOCATION

B-1 METHODS OF LABORATORY TESTING

Representative samples were evaluated and classified by a qualified member of the Geotechnical Division and the boring logs were edited as necessary. To aid in classifying the subsurface materials and to determine the general engineering characteristics, natural moisture content tests (ASTM D 2216), Atterberg-limit tests (ASTM D 4318), percent material finer than the No. 200 sieve tests (ASTM D 1140), and dry unit weight determinations were conducted on selected samples. In addition, unconfined compressive strength tests (ASMT D 2166) and pocket-penetrometer tests were conducted on selected soil samples to evaluate the soil shear strength. Results of these laboratory tests are provided on the Log of Boring sheets.

In addition to the Atterberg-limit tests, the expansive properties of the clayey soils were further analyzed by absorption swell tests. The swell test is performed by placing a selected sample in a consolidation machine and applying either the approximate current or expected overburden pressure and then allowing the sample to absorb water. When the sample exhibits very little tendency for further expansion, the height increase is recorded and the percent free swell and total moisture gain calculated. Results of the absorption swell tests are provided on the attached Log of Boring sheets.

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BORING NO.: 1
Sheet 1 of 1

PROJECT NO.: W240767

Client: Krum ISD
Project: Krum ISD - High School Band Hall and CTE Additions
Start Date: 4/1/2024 **End Date:** 4/1/2024
Drilling Method: CONTINUOUS FLIGHT AUGER

Location: Krum, Texas
Surface Elevation:
West:
North:
Hammer Drop (lbs / in): 170 / 24

GROUND WATER OBSERVATIONS
- On Rods (ft): NONE
- After Drilling (ft): DRY
- After ____ Hours (ft):

Depth, feet	Graphic Log	MATERIAL DESCRIPTION	Sample Type	Recovery % RQD	TX Cone or Std. Pen. (blows/ft, in)	Pocket Penetrometer (tsf)	Unconfined Comp. Strength (tsf)	UU Shear Strength (tsf)	% Passing No. 200 Sieve	Unit Dry Weight (pcf)	Water Content, %	Liquid Limit	Plastic Limit	Plasticity Index	Swell, %
		Light Brown CLAY with gravel - FILL				4.5+					16	66	21	45	
		4.0				4.5+					15	62	20	42	
5		Dark Brown CLAY with limestone fragments and gravel				4.5+					15				
						4.5+					17				
10		10.0				4.5+					16	72	23	49	6.5
15		Light Brown SHALY CLAY with calcareous deposits				4.5					18				
		19.0													
20		Gray LIMESTONE			100/ 0"										
		22.0													
25		Gray SHALE with limestone seams and layers			100/ 3.50"										
30					100/ 3"										
35					100/ 2.50"										
40					100/ 1.50"										
45		45.0			100/ 1"										
		TEST BORING TERMINATED AT 45 FT													
50															

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BORING NO.: 2

PROJECT NO.: W240767

Client: Krum ISD
Project: Krum ISD - High School Band Hall and CTE Additions
Start Date: 4/1/2024 **End Date:** 4/1/2024
Drilling Method: CONTINUOUS FLIGHT AUGER

Location: Krum, Texas
Surface Elevation:
West:
North:
Hammer Drop (lbs / in): 170 / 24

GROUND WATER OBSERVATIONS
- On Rods (ft): NONE
- After Drilling (ft): DRY
- After ____ Hours (ft):

Depth, feet	Graphic Log	MATERIAL DESCRIPTION	Bottom Depth (ft)	Sample Type	Recovery % RQD	TX Cone or Std. Pen. (blows/ft, in)	Pocket Penetrometer (tsf)	Unconfined Comp. Strength (tsf)	UU Shear Strength (tsf)	% Passing No. 200 Sieve	Unit Dry Weight (pcf)	Water Content, %	Liquid Limit	Plastic Limit	Plasticity Index	Swell, %
0		Dark Brown CLAY	4.0				2.75					17				
							4.5+					16				
5		Brown CLAY	6.0				4.0					21	65	21	44	
		Light Brown SHALY CLAY with calcareous deposits and nodules	14.0				4.5+					21	61	21	40	5.3
10							4.5+					20				
15		Light Brown SHALY CLAY with iron deposits and seams	19.0				4.5+	3.6			106	18				
20		Brown CLAY	21.0				4.5					17				
		Gray LIMESTONE	23.0													
25		Gray SHALE with limestone seams and layers	45.0			100/ 2"										
30						100/ 1.50"										
35						100/ 2.50"										
40						100/ 2"										
45						100/ 1.50"										
		TEST BORING TERMINATED AT 45 FT														
50																

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BORING NO.: 3
Sheet 1 of 1

PROJECT NO.: W240767

Client: Krum ISD
Project: Krum ISD - High School Band Hall and CTE Additions
Start Date: 3/20/2024 **End Date:** 3/20/2024
Drilling Method: CONTINUOUS FLIGHT AUGER

Location: Krum, Texas
Surface Elevation:
West:
North:
Hammer Drop (lbs / in): 170 / 24

GROUND WATER OBSERVATIONS
On Rods (ft): NONE
After Drilling (ft): DRY
After ____ Hours (ft):

Depth, feet	Graphic Log	MATERIAL DESCRIPTION	Sample Type	Recovery % RQD	TX Cone or Std. Pen. (blows/ft, in)	Pocket Penetrometer (tsf)	Unconfined Comp. Strength (tsf)	UU Shear Strength (tsf)	% Passing No. 200 Sieve	Unit Dry Weight (pcf)	Water Content, %	Liquid Limit	Plastic Limit	Plasticity Index	Swell, %
0 – 2.0		Dark Brown CLAY				2.0					23				
2.0 – 20.0		Light Brown CLAY with limestone fragments				1.0					28				
5						4.5					20	65	21	44	4.0
						4.5					21				
						4.5					21	53	20	33	
15						4.5+					17				
20						4.5+					14				
20.0 – 45.0		Gray LIMESTONE with shale seams and layers													
25					100/ 0.50"										
30					100/ 1"										
35					100/ 0.50"										
40					100/ 0.50"										
45					100/ 0.50"										
45.0		TEST BORING TERMINATED AT 45 FT													

UES
Formerly Alpha Testing

5058 Brush Creek Rd.
Fort Worth, Texas 76119
Phone: 817-496-5600
Fax: 817-496-5608
www.alphatesting.com

BORING NO.: 4

PROJECT NO.: W240767

Client: Krum ISD
Project: Krum ISD - High School Band Hall and CTE Additions
Start Date: 3/20/2024 **End Date:** 3/20/2024
Drilling Method: CONTINUOUS FLIGHT AUGER

Location: Krum, Texas
Surface Elevation:
West:
North:
Hammer Drop (lbs / in): 170 / 24

GROUND WATER OBSERVATIONS
On Rods (ft): NONE
After Drilling (ft): DRY
After ____ Hours (ft):

Depth, feet	Graphic Log	MATERIAL DESCRIPTION	Sample Type	Recovery % RQD	TX Cone or Std. Pen. (blows/ft, in)	Pocket Penetrometer (tsf)	Unconfined Comp. Strength (tsf)	UU Shear Strength (tsf)	% Passing No. 200 Sieve	Unit Dry Weight (pcf)	Water Content, %	Liquid Limit	Plastic Limit	Plasticity Index	Swell, %
		Dark BrownGRAVELLY CLAY - FILL				4.5+					7				
		3.0													
		Light Brown CLAY with limestone fragments - FILL													
5		6.0				4.25					23				
		Light Brown CLAY with limestone fragments				4.5					20				
10						4.5					21				
15						4.5	4.2			110	19	53	20	33	
20		20.0				4.5+					16				
		Gray LIMESTONE with shale seams and layers													
25					100/ 0.50"										
30					100/ 0.50"										
35					100/ 0.50"										
40					100/ 0.50"										
45		45.0			100/ 0.50"										
		TEST BORING TERMINATED AT 45 FT													
50															

UES
Formerly Alpha Testing

KEY TO SOIL SYMBOLS AND CLASSIFICATIONS

SOIL & ROCK SYMBOLS

- (CH), High Plasticity CLAY
- (CL), Low Plasticity CLAY
- (SC), CLAYEY SAND
- (SP), Poorly Graded SAND
- (SW), Well Graded SAND
- (SM), SILTY SAND
- (ML), SILT
- (MH), Elastic SILT
- LIMESTONE
- SHALE / MARL
- SANDSTONE
- (GP), Poorly Graded GRAVEL
- (GW), Well Graded GRAVEL
- (GC), CLAYEY GRAVEL
- (GM), SILTY GRAVEL
- (OL), ORGANIC SILT
- (OH), ORGANIC CLAY
- FILL

SAMPLING SYMBOLS

- SHELBY TUBE (3" OD except where noted otherwise)
- SPLIT SPOON (2" OD except where noted otherwise)
- AUGER SAMPLE
- TEXAS CONE PENETRATION
- ROCK CORE (2" ID except where noted otherwise)

RELATIVE DENSITY OF COHESIONLESS SOILS (blows/ft)

VERY LOOSE	0 TO 4
LOOSE	5 TO 10
MEDIUM	11 TO 30
DENSE	31 TO 50
VERY DENSE	OVER 50

SHEAR STRENGTH OF COHESIVE SOILS (tsf)

VERY SOFT	LESS THAN 0.25
SOFT	0.25 TO 0.50
FIRM	0.50 TO 1.00
STIFF	1.00 TO 2.00
VERY STIFF	2.00 TO 4.00
HARD	OVER 4.00

RELATIVE DEGREE OF PLASTICITY (PI)

LOW	4 TO 15
MEDIUM	16 TO 25
HIGH	26 TO 35
VERY HIGH	OVER 35

RELATIVE PROPORTIONS (%)

TRACE	1 TO 10
LITTLE	11 TO 20
SOME	21 TO 35
AND	36 TO 50

PARTICLE SIZE IDENTIFICATION (DIAMETER)

BOULDERS	8.0" OR LARGER
COBBLES	3.0" TO 8.0"
COARSE GRAVEL	0.75" TO 3.0"
FINE GRAVEL	5.0 mm TO 3.0"
COURSE SAND	2.0 mm TO 5.0 mm
MEDIUM SAND	0.4 mm TO 5.0 mm
FINE SAND	0.07 mm TO 0.4 mm
SILT	0.002 mm TO 0.07 mm
CLAY	LESS THAN 0.002 mm