• GEOTECHNICAL ENGINEERING
• CONSTRUCTION MATERIALS
ENGINEERING & TESTING
• SOILS • ASPHALT • CONCRETE

June 13, 2008

Karnes City United Methodist Church
201 North Esplanade Street
Karnes City, Texas 78118

Attention: Reverend Ken Houston

SUBJECT: **SUBSURFACE INVESTIGATION, LABORATORY TESTING PROGRAM AND FOUNDATION RECOMMENDATIONS FOR THE PROPOSED KARNES CITY UNITED METHODIST CHURCH ADDITIONS**
201 North Esplanade Street
Karnes City, Texas
RETL Job No.: G108399

Dear Reverend Houston,

In accordance with our agreement, we have conducted a subsurface exploration and foundation evaluation for the above referenced project. The results of this investigation, together with our recommendations, are to be found in the accompanying report, two copies of which are being transmitted herewith. Additionally, one copy each is being forwarded to Mr. David E. Lewis, AIA and Mr. Homer Castillo, P.E.

Often, because of design and construction details that occur on a project, questions arise concerning soil conditions and Rock Engineering and Testing Laboratory, Inc. (RETL), would be pleased to continue its role as the Geotechnical Engineer during project implementation.

RETL also has great interest in providing materials testing and observation services during the construction phase of this project. If you will advise us of the appropriate time to discuss these engineering services, we will be pleased to meet with you at your convenience.

Sincerely,

Christopher A. Rock, P.E.
Branch Manager

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SUBSURFACE INVESTIGATION, LABORATORY TESTING PROGRAM AND FOUNDATION RECOMMENDATIONS FOR THE PROPOSED KARNES CITY UNITED METHODIST CHURCH ADDITIONS
201 NORTH ESPLANADE STREET
KARNES CITY, TEXAS

RETL JOB NUMBER: G108399

PREPARED FOR:
KARNES CITY UNITED METHODIST CHURCH
201 NORTH ESPLANADE STREET
KARNES CITY, TEXAS 78118

JUNE 13, 2008

PREPARED BY:
ROCK ENGINEERING AND TESTING LABORATORY, INC.
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STATE OF TEXAS
CHRISTOPHER A. ROCK
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PROFESSIONAL ENGINEER

Christopher A. Rock, P.E.
Branch Manager

ENGINEERING & TESTING
ROCK
LABORATORY INCORPORATED

TABLE OF CONTENTS

INTRODUCTION

This report presents the results of a soils exploration and foundation analysis for the proposed Karnes City United Methodist Church Additions to be constructed at 201 North Esplanade Street located in Karnes City, Texas.

Authorization

The work for this project was performed in accordance with RETL proposal number P040808A dated April 9, 2008. The proposal was verbally approved and notice to proceed was given by Mr. David E. Lewis, AIA. The proposal was signed by Reverend Kendall Houston on May 13, 2008 and returned to RETL on June 13, 2008 via fax transmission.

Purpose and Scope

The purpose of this exploration was to evaluate the soil and groundwater conditions at the site and to recommend types and depths of foundation systems suitable for the proposed project.

The scope of the exploration and analysis included the subsurface exploration, field and laboratory testing, engineering analysis and evaluation of the subsurface soils, foundation recommendations and preparation of this report.

The scope of services did not include an environmental assessment. Any statements in this report, or on the boring logs, regarding odors, colors, unusual or suspicious items or conditions are strictly for the information of the client.

General

The exploration and analysis of the subsurface conditions reported herein are considered sufficient in detail and scope to form a reasonable basis for the foundation designs. The recommendations submitted for the proposed project are based on the available soil information and the preliminary design details provided by Mr. David E. Lewis, AIA. If the foundation designer requires additional soil parameters to complete the design of the proposed foundations, and the requested information can be determined from the data obtained within the agreed scope of work provided in the proposal, then RETL will provide this information as a supplement to this report.

The Geotechnical Engineer states that the findings, recommendations, specifications or professional advice contained herein have been presented after being prepared in a manner consistent with that level of care and skill ordinarily exercised by reputable members of the Geotechnical Engineer's profession practicing contemporaneously under similar conditions in the locality of the project. RETL operates in general accordance with, "*Standard Practice for Minimum Requirements for Agencies Engaged in the Testing and/or Inspection of Soil and Rock as Used in Engineering Design and Construction*, (ASTM D 3740)." No other representations are expressed or implied, and no warranty or guarantee is included or intended.

This report has been prepared for the exclusive use of Karnes City United Methodist Church and Mr. David E. Lewis for the specific purpose of the proposed Karnes City United Methodist Church Additions to be constructed at 201 North Esplanade Street located in Karnes City, Texas.

DESCRIPTION OF SITE

The project site is located at the existing Karnes City United Methodist Church on 201 North Esplanade Street in Karnes City, Texas. The site of the proposed addition is relatively level and covered with grass and trees. The numerous trees and existing fencing restricted movement around the project site. Prior to the site investigation, clearing operations were performed in order to access the boring locations. Overhead utilities and evidence of underground utilities were observed on and adjacent to the site. At the time of our drilling operations, the ground surface was firm and did not pose any significant difficulties to the drill crew moving their equipment around the site.

FIELD EXPLORATION

Scope

The field exploration, to evaluate the engineering characteristics of the foundation materials, included reconnaissance of the project site, drilling the test borings and recovering disturbed split spoon soil samples and relatively undisturbed Shelby tube soil samples. During the sample recovery operations, the soils encountered were classified and recorded on the boring logs in accordance with, "*Standard Guide for Field Logging of Subsurface Exploration of Soil and Rock*, (ASTM D 5434)."

Two borings were performed at the site to termination depths of 25-feet. RETL determined the number and depth of the borings. Mr. David E. Lewis, AIA determined the location of the borings. Jones Environmental Drilling, Inc., a drilling subcontractor to RETL, performed the boring operations. Upon completion of the drilling operations and obtaining the delayed groundwater observations, the drill holes were backfilled with excavated soil and the site cleaned as required. A Boring Location Plan, which is a reproduction of a drawing provided to RETL by Mr. David E. Lewis, is provided in this report showing the approximate location of the borings with respect to existing predominant site features.

Drilling and Sampling Procedures

The borings were performed using a drilling rig equipped with a rotary head turning flight augers to advance the boreholes. Disturbed soil samples were obtained employing split-barrel sampling procedures in general accordance with the procedures for "*Penetration Test and Split-Barrel Sampling of Soils,* (ASTM D 1586)." Undisturbed soil samples were obtained using thin-wall tube sampling procedures in accordance with "*Thin Walled Tube Sampling of Soils*, (ASTM 1587)." The samples obtained by this procedure were extruded by a hydraulic ram and classified in the field.

All of the samples were placed in plastic bags, marked according to boring number, depth and any other pertinent field data, stored in special containers and delivered to the laboratory for testing.

Field Tests and Observations

Penetration Tests - During the sampling procedures, standard penetration tests (SPT) were performed to obtain the standard penetration value of the soil at selected intervals. The standard penetration value (N) is defined as the number of blows of a 140-pound hammer, falling 30-inches, required to advance the split-barrel sampler 1-foot into the soil. The sampler is lowered to the bottom of the previously cleaned drill hole and advanced by blows from the hammer. The number of blows are recorded for each of three successive 6-inch penetrations. The "N" value is obtained by adding the second and third 6-inch increment number of blows. The results of standard penetration tests indicate the relative density of cohesionless soils and comparative consistency of cohesive soils, thereby providing a basis for estimating the relative strength and compressibility of the soil profile components.

Water Level Observations - Water level observations were obtained during the test boring operations. Water level observations are noted on the boring logs provided in the Appendix. In relatively pervious soils, such as sandy soils, the indicated depths are usually reliable groundwater levels. In relatively impervious soils, a suitable estimate of the groundwater depth may not be possible, even after several days of observation. Seasonal variations, temperature, land-use, proximity to a creek, river or lake and recent rainfall conditions may influence the depth to the groundwater. The amount of water in open boreholes largely depends on the permeability of the soils encountered at the boring locations.

Ground Surface Elevations – The ground surface elevations were not provided at the boring locations. Therefore, all depths referred to in this report are from the actual ground surface elevations at the boring locations during the time of our field investigation.

LABORATORY TESTING PROGRAM

In addition to the field investigation, a laboratory-testing program was conducted to determine additional pertinent engineering characteristics of the subsurface materials necessary in analyzing the behavior of the foundation systems for the proposed project.

The laboratory-testing program included supplementary visual classification (ASTM D 2487) and water content tests (ASTM D 2216) on all samples. In addition, selected samples were subjected to dry unit weight determinations (ASTM D 2937), Atterberg limits tests (ASTM D 4318) and percent material finer than the #200 sieve.

The shear strength of a cohesive soil sample was evaluated from an unconfined compressive strength test (ASTM D 2166). The estimated soil strength was obtained using a hand penetrometer.

All phases of the laboratory-testing program were conducted in general accordance with applicable ASTM Specifications. The results of these tests are to be found on the accompanying boring logs provided in the Appendix.

SUBSURFACE CONDITIONS

General

The types of foundation bearing materials encountered in the test borings have been visually classified and are described in detail on the boring logs. The pocket penetrometer test values and other laboratory tests are presented on the boring logs in numerical form. Representative samples of the soils were placed in polyethylene bags and are now stored in the laboratory for further analysis, if desired. Unless notified to the contrary, all samples will be disposed of three months after issuance of this report.

The stratification of the soil, as shown on the boring logs, represents the soil conditions at the actual boring locations. Variations may occur between, or beyond, the actual boring locations. Lines of demarcation represent the approximate boundary between different soil types, but the transition may be gradual, or not clearly defined.

It should be noted that, whereas the test borings were drilled and sampled by experienced drillers, it is sometimes difficult to record changes in stratification within narrow limits. In the absence of foreign substances, it is also difficult to distinguish between discolored soils and clean soil fill.

Soil Conditions

The generalized soil conditions encountered at the project site have been summarized and soil properties including soil classification, strength and plasticity are provided in the following table:

Soil Profile Table

D	Description	LL	PI	C	ϕ	γ_e	-#200	PP	N
0-8	Clayey **SAND** & Lean/Fat **CLAY**	44-75	26-46	2300	0	120	49-64	4.5+	7-25
8-25	Clayey **SAND &** Fat **CLAY**	49-61	31-41	3200	0	120	46-94	4.5+	34-67

where:

D = Depth in feet below existing grade
LL = Liquid limit (%)
PI = Plasticity index
C = Soil Cohesion, psf (undrained)
ϕ = Angle of Internal Friction, deg. (undrained)
γ_e = Effective soil unit weight, pcf
-#200 = Material passing #200 sieve, %
PP = Pocket penetrometer value range, tsf
N= Standard penetration tests range, blows per foot

Detailed descriptions of the soils encountered at the boring locations are provided on the boring logs included in the Appendix.

Groundwater Observations

Groundwater (GW) was not encountered during drilling operations. Based on our observations made in the field and moisture contents obtained in the laboratory, it appears as if groundwater at this site will be encountered at depths greater than the 25-foot depth, the termination depth of the test borings. It should be noted that water levels in open boreholes may require several hours to several days to stabilize depending on the permeability of the soils and that groundwater levels at this site may be subject to seasonal conditions, recent rainfall, drought or temperature effects.

FOUNDATION DISCUSSION

Project Description

Based on information provided to RETL, the project will include the construction of three, single-story structures with a combined footprint on the order of 3,900 square feet. Estimated eave heights are on the order of 11 to 12-feet. The structural loads were not provided to RETL at the time this report was being prepared, but, based on our experience with similar type structures, it is assumed that the expected concentrated loads will be on the order of 30 to 60 kips and continuous wall loads will be on the order of 1 ½ to 2 kips/lf.

It is understood that the existing Karnes City United Methodist Church is supported by a drilled pier foundation system. It is typically desirable to support additions on the same foundation type as the existing structure in order to minimize differential movements between the structures. Therefore, only recommendations for drilled piers will be provided in this report.

PVR Discussion

The laboratory test results indicate that the subsoils in the active zone at this site are moderate to high in plasticity. **The maximum calculated total potential vertical rise (PVR) at this site is on the order of 2 ¾ to 3-inches.** The PVR was calculated using the Texas Department of Transportation Method TEX-124E and took into account the depth of active zone, estimated to extend to a depth of 15-feet, and the Atterberg limits test results of the soils encountered within the active zone.

The estimated PVR value provided is based on the floor system applying a sustained surcharge load of approximately 1.0 pound per square inch on the subgrade soils. The value represents the vertical rise that can be experienced by dry subsoils if they are subjected to conditions that allow them to become saturated, such as poor drainage. Using dry soil conditions to calculate the PVR is generally considered the worst case scenario. The actual movement of the subsoils is dependent upon their change in moisture content. Differential vertical movements can potentially be equal to the expected total movements. Differential vertical movements associated with the soils at this site may occur over a distance of 15-feet, or approximately the depth of the active zone, within the footprint of a slab-on-grade.

Undercutting the natural expansive soils at this site and replacing them with properly compacted non-expansive select fill soils should reduce the PVR. The resulting reduction in PVR at this site, utilizing undercutting and replacement operations, are included in the following table:

Required Undercut Depth (ft)	Calculated PVR (in)
1	2.1
2	1.8
3	1.4
4	1.2
5	1.0

Based on our calculations, in order to reduce the PVR to approximately 1-inch, it will be necessary to remove the expansive soils to a depth of 5-feet, moisture condition and compact the exposed subgrade soils, and place a minimum of 6-feet of properly compacted non-expansive select fill soils in the excavations. Additional, undercutting and replacement may be required to further reduce the PVR based on architectural or structural considerations.

Drilled Pier Recommendations

Based on the description of the existing structure, the proposed additions, the soils at the project site and the discussion above, RETL recommends the proposed structures be supported by drilled piers. Recommendations for straight shaft drilled piers and underreamed drilled piers to support the proposed structures are provided for use at this site. If an alternative foundation type is considered, RETL should be notified in order to provide a supplemental report.

Underreamed drilled piers should be founded near the 15-foot depth, and can be sized to exert a net allowable unit end bearing pressure of **5,500 psf**. It may be required to adjust the underreamed drilled pier depth at the time the production piers are installed. The net allowable unit end bearing pressure for underreamed drilled piers includes a factor of safety on the order of 3 against bearing failure. If the underreamed drilled piers are designed using the recommended net allowable unit end bearing pressure provided above, the piers should experience total settlements on the order of ½-percent of the diameter of the belled portion of the underreamed drilled pier. The belled portion of the underreamed drilled pier should be limited to a diameter of 2 to 2 ½ times the shaft diameter. The shaft diameter of the underreamed drilled pier should be a minimum of 12-inches.

The structural designer can utilize the allowable unit skin friction values for the range in depths included in the following table for straight shaft drilled piers to resist the axial loads given the strengths of the subsurface soils encountered:

ALLOWABLE UNIT SKIN FRICTION VALUES	
Depth Below Existing Grade (ft)	**Allowable Unit Skin Friction (psf)**
0-5	Neglect
5-8	600
8-23	850

All depths are referenced from the existing ground surface elevations at the boring locations during the time of our field investigation.

The allowable unit skin friction values provided above are based on the average strengths of the in-situ soils and utilize a safety factor of 2 to prevent shear failure. The minimum depth of a straight shaft drilled pier required to resist uplift forces from the expansive soils encountered at this site, assuming a minimum load of 25 kips, is 21-feet and the maximum recommended termination depth for straight shaft drilled piers is 23-feet. If the minimum load on any pier is less than 25-kips, RETL should be given the opportunity to revisit the minimum depth recommendations for straight shaft drilled piers at this site. Resistance to uplift can be calculated by taking 60-percent of the axial capacity of a straight shaft drilled pier.

Underreamed drilled piers should be spaced a minimum distance measured center to center of 2-times the diameter of the belled portion of the largest adjacent underreamed drilled pier. Straight shaft drilled piers should be spaced no closer than three pier diameters apart measured center to center. Drilled piers at this site should be adequately reinforced with a minimum of 1 percent of the cross-sectional area of the pier shaft throughout the depth of the pier to withstand uplift forces.

Structural floor slab systems should be utilized in conjunction with drilled piers. Grade beams spanning between piers should be structurally connected to the piers. It is recommended that the structural floor slab systems and grade beams spanning between the piers be constructed with a minimum 6-inch void space between the slabs and the soil at the site.

CONSTRUCTION CONSIDERATIONS

Drilled, Cast-in-Place, Pier Construction Considerations

Based on observations made in the field, it does not appear as if temporary steel casing will be required to successfully install straight shaft drilled piers at this site. Even though it does not appear as if temporary steel casing will be required, a unit bid price for temporary steel casing should be solicited in the bid documents in the event temporary steel casing is required to successfully install straight shaft drilled piers at this site.

Concrete should be placed as soon as possible after all loose material has been removed, the pier excavation inspected and reinforcing steel installed. A relatively high slump concrete mix (6 to 7-inches) is suggested to minimize aggregate segregation caused by the reinforcing steel. Free fall of concrete into the pier excavation is permitted provided the concrete can be placed into the pier excavation without striking the sides of the excavation or hitting the rebar.

It should be noted that research has shown that free fall concrete, guided at the top of excavations, to avoid contact with the sides of the pier excavation and reinforcing steel, can drop more than 80-feet without any measurable segregation. In addition, the research has shown that as long as the concrete drop is in air, the strength of the concrete was not adversely affected. In situations where it is impossible for the concrete to fall freely without striking the rebar cage or sides of pier excavations, the free fall should be limited to 10-feet, or placed with a tremie. Pier excavations should not be allowed to stay open overnight.

The successful placement of drilled pier foundation systems is dependent on the expertise of the drilled pier foundation contractor. **A test pier excavation should be performed at the site to verify the contractor's construction methods and to identify any potential groundwater infiltration and soil sloughing problems.** The Geotechnical Engineer, or his designated representative, should be present to witness the installation of all the drilled piers, including the test pier excavation.

Utilities

Utilities that project through floors should be designed with either some degree of flexibility, or with sleeves, in order to prevent damage to these lines should vertical movement occur.

Expansion Joints

Expansion joints should be designed and placed in various portions of the structures. Properly planned placement of these joints will assist in controlling the degree and location of material cracking that normally occurs due to material shrinkage, thermal affects, soil movements and other related structural conditions.

Concrete Flatwork Construction Considerations

Concrete site flatwork such as driveways, sidewalks, patios, etc. will be subject to movements when constructed over highly plastic soils. Changes in the moisture content of the supporting highly plastic soils causes volumetric changes, resulting in differential movements of the flatwork. Traditional methods to minimize movements within structures where performance criteria is dictated by the owner or the owner's design professional, is to undercut the highly plastic soils and replace them with properly compacted non-expansive select fill soils to achieve tolerable movements, usually on the order of 1-inch or less, or where a higher level of performance is required, the structures are supported on drilled piers in conjunction with structural slabs. The cost/benefit ratio of these methods to minimize PVR and differential movements are generally not considered cost effective for use on flatwork.

As previously stated, the change in moisture content of the highly plastic soils is the primary mechanism resulting in the volumetric changes of the supporting soils. Provisions in the site development should be made in order to maintain relative uniform moisture contents of the supporting soils. A number of measures may be used to attain a reduction in subsoil moisture content variations, thus reducing the soil's shrink/swell volume change potential. Some of these measures are outlined below:

- During construction, a positive drainage scheme should be implemented to prevent ponding of water on the subgrade.

- Positive drainage should be maintained around the structures and flatwork through a roof/gutter system connected to piping or directed to paved surfaces, transmitting water away from foundation perimeters and flatwork. In addition, positive grades sloping away from foundations and flatwork should be designed and implemented. We recommend that effective site drainage plans be devised by others prior to commencement of construction to provide positive drainage away from the site improvements and off the site, both during, and after construction.

- The top 2-feet of utility trenches should be backfilled with low plasticity clays to assure the trenches do not serve as aqueducts that could transport water beneath the structures and flatwork due to excessive surface water infiltration.

- Vegetation placed in landscape beds that are adjacent to the structures and flatwork should be limited to plants and shrubs that will not exceed a mature height of 3-feet. Large bushes and trees should be planted away from the slab foundations and flatwork at a distance that will exceed their full mature height and canopy width.

- Individual concrete panels of concrete sitework should be dowelled together to minimize trip hazards as a result of differential movements within the flatwork.

- All efforts should be made to avoid having situations where site flatwork panels are partially supported on properly compacted select fill soils and partially supported on natural in-situ highly plastic soils which will result in differential movement and may also result in a negative slope back to the building causing ponding of water next to the structures.

- Pavements should be designed to drain quickly with a minimum positive slope of 1 percent. Planter islands should incorporate a 12-inch clay cap at the surface and the curbs should be designed to prevent moisture from entering the pavement base materials.

- In areas when flatwork is planned for construction and extending 5-feet outside the plan area of the flatwork, all surface organics and deleterious materials shall be removed, the upper 12-inches of exposed subgrade soils should be scarified, moisture conditioned to a minimum of 2-percent above the optimum moisture content and then compacted to at least 90% and not more than 95% of the maximum dry density as determined by the standard Proctor test (ASTM D-698).

- If it is desired to reduce the PVR movements beneath flatwork to 1-inch, the clay soils should be undercut and replaced with select fill as discussed in the "PVR Discussion" section of this report.

All project features beyond the scope of those discussed above should be planned and designed similarly to attain a region of relatively uniform moisture content within the foundations and flatwork areas. Poor drainage schemes are generally the primary cause of foundation and flatwork problems on clay soils.

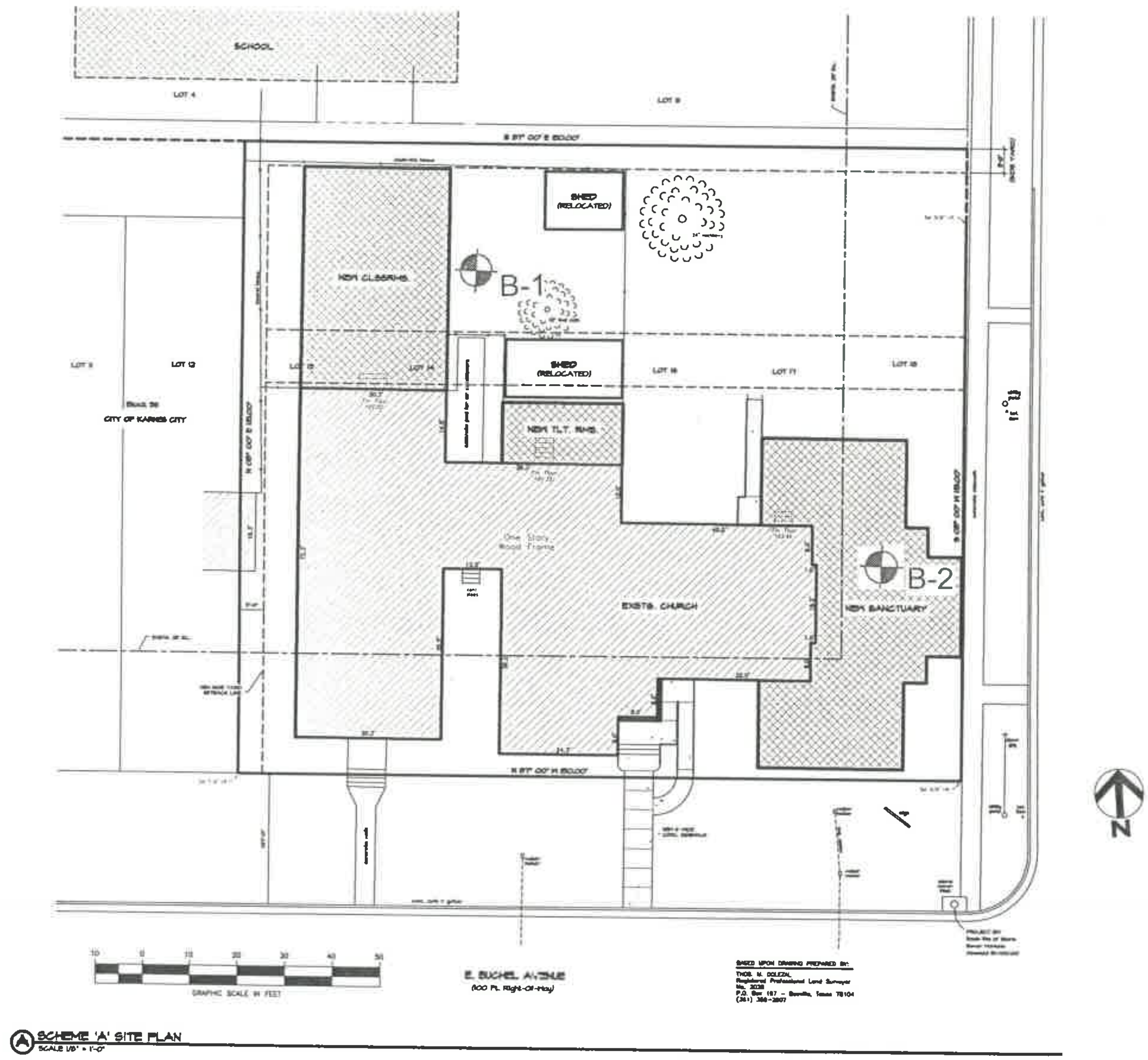
GENERAL COMMENTS

If significant changes are made in the character or location of the proposed project, a consultation should be arranged to review any changes with respect to the prevailing soil conditions. At that time, it may be necessary to submit supplementary recommendations.

It is recommended that the services of RETL be engaged to test and evaluate the soils in drilled pier excavations prior to concreting in order to verify that the bearing soils are consistent with those encountered in the borings. RETL cannot accept any responsibility for any conditions that deviate from those described in this report, nor for the performance of the foundations if not engaged to also provide construction observation and testing for this project. If it is required for RETL to accept any liability, then RETL must agree with the plans and perform such observation during construction as we recommend.

All sheeting, shoring and bracing of trenches, pits and excavations should be made the responsibility of the contractor and should comply with all current and applicable local, state and federal safety codes, regulations and practices, including the Occupational Safety and Health Administration.

APPENDIX



BORING LOCATION PLAN

NO SCALE

June 13, 2008
Attn.: Reverend Ken Houston
RETL Job No.: G108399

PROP. KARNES CITY UNITED METHODIST CHURCH ADDITIONS
201 North Esplanade Street
Karnes City, Texas

ROCK ENGINEERING AND TESTING LABORATORY, INC.
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(361) 883-4555

LOG OF BORING B-1

Rock Engineering & Testing Lab., Inc.
6817 Leopard St.
Corpus Christi, TX 78409
Telephone: (361) 883-4555
Fax: (361) 883-4711

CLIENT: Karnes City United Methodist Church
PROJECT: Proposed Church Additions
LOCATION: 201 N. Esplanade St.; Karnes City, TX
NUMBER: G108399

DATE(S) DRILLED: 05/22/08 - 05/22/08

DRILLING METHOD(S):
Flight Auger

GROUNDWATER INFORMATION:
Groundwater (GW) was not encountered during drilling operations.
Dry and open upon completion of drilling operations.

SURFACE ELEVATION: N/A

DEPTH (FT)	SAMPLE NUMBER	N: BLOWS/FT; P: TONS/SQ FT; T: TONS/SQ FT; PERCENT RECOVERY/ ROCK QUALITY DESIGNATION	MOISTURE CONTENT (%)	LIQUID LIMIT LL	PLASTIC LIMIT PL	PLASTICITY INDEX PI	DRY DENSITY POUNDS/CU.FT	COMPRESSIVE STRENGTH (TONS/SQ FT)	MINUS NO. 200 SIEVE (%)	DESCRIPTION OF STRATUM
	SS S-1	N= 7	8	44	18	26			49	**CLAYEY SAND**, with organic material and calcareous deposits, dark brown, dry, firm. (SC)
5	SS S-2	N= 25	10							Same as above, very stiff.
	SS S-3	N= 24	21							Same as above, moist.
10	SH S-4	P= 4.5+	15	53	19	34			62	**SANDY FAT CLAY**, with calcareous nodules, dark brown and dark gray, dry, very stiff. (CH)
	SS S-5	N= 48	15							**CLAYEY SAND**, with calcareous nodules, dark brown, gray, and light brown, dry, hard.
15	SS S-6	N= 37	12							Same as above, light brown.
20	SH S-7	P= 4.5+	20	61	20	41			92	**FAT CLAY**, with calcareous deposits, light gray, moist, very stiff. (CH)
25	SH S-8	P= 4.5+	25							Same as above, light gray and reddish brown.
										Boring was terminated at a depth of 25-feet.

N - STANDARD PENETRATION TEST RESISTANCE
P - POCKET PENETROMETER RESISTANCE
T - POCKET TORVANE SHEAR STRENGTH

REMARKS:
Boring depth was determined by RETL and boring location was determined by Mr. David Lewis, AIA. Boring operations were performed by JEDI, a drilling sub-contractor to RETL.

LOG OF BORING B-2

Rock Engineering & Testing Lab., Inc.
6817 Leopard St.
Corpus Christi, TX 78409
Telephone: (361) 883-4555
Fax: (361) 883-4711

CLIENT: Karnes City United Methodist Church
PROJECT: Proposed Church Additions
LOCATION: 201 N. Esplanade St.; Karnes City, TX
NUMBER: G108399

DATE(S) DRILLED: 05/22/08 - 05/22/08

DRILLING METHOD(S):
Flight Auger

GROUNDWATER INFORMATION:
Groundwater (GW) was not encountered during drilling operations.
Dry and open upon completion of drilling operations.

SURFACE ELEVATION: N/A

Depth (ft)	Sample Number	N: Blows/ft; P: Tons/sq ft; T: Tons/sq ft; Percent Recovery/Rock Quality Designation	Moisture Content (%)	Liquid Limit (LL)	Plastic Limit (PL)	Plasticity Index (PI)	Dry Density Pounds/cu.ft	Compressive Strength (tons/sq ft)	Minus No. 200 Sieve (%)	Description of Stratum
	SS S-1	N= 18	14							**SANDY LEAN CLAY**, with organic material and calcareous nodules, brown, dry, very stiff.
5	SS S-2	N= 12	17						64	Same as above, brown and light gray, stiff.
	SH S-3	P= 4.5+	18	75	29	46				**FAT CLAY**, with sand and calcareous deposits, dark brown and gray, dry, very stiff. (CH)
10	SS S-4	N= 34	27							**CLAYEY SAND**, with calcareous nodules, light gray, moist, hard.
	SS S-5	N= 44	21							Same as above.
15	SS S-6	N= 67	15	49	18	31			46	Same as above, dry. (SC)
20	SH S-7	P= 4.5+	21				103	3.0		**FAT CLAY**, with calcareous deposits, gray, moist, very stiff, blocky.
25	SH S-8	P= 4.5+	23						94	Same as above, gray and reddish brown.
										Boring was terminated at a depth of 25-feet.

N - STANDARD PENETRATION TEST RESISTANCE
P - POCKET PENETROMETER RESISTANCE
T - POCKET TORVANE SHEAR STRENGTH

REMARKS:
Boring depth was determined by RETL and boring location was determined by Mr. David Lewis, AIA. Boring operations were performed by JEDI, a drilling sub-contractor to RETL.

ROCK Engineering & Testing Laboratory, Inc.

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Office: (361) 883-4555 Fax: (361) 883-4711

KEY TO SOIL CLASSIFICATIONS AND SYMBOLS

UNIFIED SOIL CLASSIFICATION SYSTEM

Major Divisions		Letter	Symbol: Hatching	Symbol: Color	NAME
COARSE GRAINED SOILS	GRAVEL AND GRAVELLY SOILS	GW		RED	Well – graded gravels or gravel – sand mixtures, little or no fines
		GP		RED	Poorly-graded gravels or gravel – sand mixtures, little or no fines
		GM		YELLOW	Silty gravels, gravel – sand – silt mixtures
		GC		YELLOW	Clayey gravels, gravel - sand – clay mixtures
	SAND AND SANDY SOILS	SW		RED	Well - graded sands or gravelly sands, little or no fines
		SP		RED	Poorly – graded sands or gravelly sands, little or no fines
		SM		YELLOW	Silty sands, sand – silt mixtures
		SC		YELLOW	Clayey sands, sand – clay mixtures
FINE GRAINED SOILS	SILTS AND CLAYS LL < 50	ML		GREEN	Inorganic silts and very fine sands, rock flour, silty or clayey fine sands or clayey silts with
		CL		GREEN	Inorganic clays of low to medium plasticity, gravelly clays, sandy clays, silty clays, lean clays
		OL		GREEN	Organic silts and organic silt-clays of low plasticity
	SILTS AND CLAYS LL > 50	MH		BLUE	Inorganic silts, micaceous or diatomaceous fine sandy or silty soils, elastic silts
		CH		BLUE	Inorganic clays of high plasticity, fat clays
		OH		BLUE	Organic clays of medium to high plasticity, organic silts
HIGHLY ORGANIC SOILS		Pt		ORANGE	Peat and other highly organic soils

TERMS CHARACTERIZING SOIL STRUCTURE

SLICKENSIDED – having inclined planes Of weakness that are slick and glossy in appearance

FISSURED – containing shrinkage cracks, frequently filled with fine sand or silt; usually more or less vertical

LAMINATED (VARVED) – composed of thin layers of varying color and texture, usually grading from sand or silt at the bottom to clay at the top.

CRUMBLY – cohesive soils which break into small blocks or crumbs on drying

CALCAREOUS – containing appreciable quantities of calcium carbonate, generally nodular.

WELL GRADED – having wide range in grain sizes and substantial amounts of all intermediate particle sizes.

POORLY GRADED - predominantly of one grain size (uniformly graded) or having a range of sizes with some intermediate size missing (gap or skip graded)

SYMBOLS FOR TEST DATA

M/C = 15 – Natural moisture content in percent.

γ = 95 – Dry unit weight in lbs/cu ft.

Qu = 1.23 – Unconfined compression strength in tons/ sq ft.

51 – 21 – 30 – Liquid limit, Plastic limit, and Plasticity index.

30% FINER – Percent finer than No. 200 mesh sieve

30 B/F – Blows per foot, standard penetration test.

▼ – Ground water table.

TERMS DESCRIBING CONSISTENCY OF SOIL (2)

COARSE GRAINED SOILS: DESCRIPTIVE TERM	COARSE GRAINED SOILS: NO. BLOWS / FT. STANDARD PEN. TEST	FINE GRAINED SOILS: DESCRIPTIVE TERM	FINE GRAINED SOILS: NO. BLOWS / FT. STANDARD PEN. TEST	FINE GRAINED SOILS: UNCONFINED COMPRESSION TONS PER SQ. FT.
Very loose	0 – 4	Very Soft	< 2	< 0.25
Loose	4 – 10	Soft	2 – 4	0.25 – 0.50
Firm (medium)	10 – 30	Plastic (med. Stiff)	4 – 8	0.50- 1.00
Dense	30 – 50	Stiff	8 – 15	1.0 – 2.00
Very Dense	over 50	Very Stiff	15 – 30	2.00 – 4.00
		Hard	over 30	over 4.00

Field classification for "Consistency" is determined with a 0.25" diameter penetrometer.