GEOTECHNICAL ENGINEERING STUDY

FOR

BRUSHY CREEK MUD PARKS IMPROVEMENT
PEPPER ROCK PARK, COMMUNITY PARK
& CAT HOLLOW PARK
ROUND ROCK & AUSTIN, TEXAS

RABA KISTNER CONSULTANTS

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Project No. AAA14-007-00
April 4, 2014

Mr. Noah Shaffer, P.E.
Halff & Associates, Inc.
Two Sierra Way, Suite 105
Georgetown, Texas 78626-7574

RE: Geotechnical Engineering Study
Brushy Creek MUD Parks Improvement
Pepper Rock Park, Community Park, & Cat Hollow Park
Round Rock & Austin, Texas

Dear Mr. Shaffer:

RABA KISTNER Consultants, Inc. (RKCI) is pleased to submit the report of our Geotechnical Engineering Study for the above-referenced project. This study was performed in accordance with RKCI Proposal No. PAA13-119-00, Revision No. 1 dated November 7, 2013. Authorization for this study was received by our firm on February 18, 2014. The purpose of this study was to drill borings within the proposed lighting installation, to perform laboratory testing to classify and characterize subsurface conditions, and to prepare an engineering report presenting foundation design and construction recommendations for the proposed lighting Installation, as well as to provide pavement design and construction guidelines for Pepper Rock Park.

The following report contains our design recommendations and considerations based on our current understanding of finished floor elevations, design tolerances and structural loads. There may be alternatives for value engineering of the foundation and pavement systems, and RKCI recommends that a meeting be held with the Owner and design team to evaluate these alternatives.

We appreciate the opportunity to be of service to you on this project. Should you have any questions about the information presented in this report, or if we may be of additional assistance with value engineering or on the materials testing-quality control program during construction, please call.

Very truly yours,

RABA KISTNER CONSULTANTS, INC.

Matthew J. Robbins, E.I.T.
Graduate Engineer

Gabriel Ornelas, Jr., P.E., PMP
Vice President

4/4/2014

MJR/GO: tlc

Attachments
Copies Submitted: Above (1-electronic, 1-bound)

GEOTECHNICAL ENGINEERING STUDY

For

BRUSHY CREEK MUD PARKS IMPROVEMENT
PEPPER ROCK PARK, COMMUNITY PARK, & CAT HOLLOW PARK
ROUND ROCK & AUSTIN, TEXAS

Prepared for

HALFF & ASSOCIATES, INC.
Georgetown, Texas

Prepared by

RABA KISTNER CONSULTANTS, INC.
Austin, Texas

PROJECT NO. AAA14-007-00

April 4, 2014

RABAKISTNER

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INTRODUCTION

RABA KISTNER Consultants, Inc. (RKCI) has completed the authorized subsurface exploration and foundation analysis for the proposed lighting installations located at Pepper Rock Park, Community Park, and Cat Hollow Park in Round Rock and Austin, Texas. This report briefly describes the procedures utilized during this study and presents our findings along with our recommendations for foundation design and construction considerations, as well as for pavement design and construction guidelines for Pepper Rock Park.

PROJECT DESCRIPTION

The facilities being considered in this study include lighting installation located at Pepper Rock Park, Community Park, and Cat Hollow Park in Round Rock and Austin, Texas. The proposed structures are anticipated to create relatively light loads which will be carried by the foundation systems. It is our understanding that at the time of this study, site grading plans and proposed structural loads were not yet available. Also included in this report are recommendations for ancillary driveway and parking area pavements for Pepper Rock Park.

LIMITATIONS

This engineering report has been prepared in accordance with accepted Geotechnical Engineering practices in the region of central Texas and for the use of Halff & Associates, Inc. (CLIENT) and its representatives for design purposes. This report may not contain sufficient information for purposes of other parties or other uses. This report is not intended for use in determining construction means and methods.

The recommendations submitted in this report are based on the data obtained from eight (8) borings drilled at the sites, our understanding of the project information provided to us, and the assumption that site grading will result in only minor changes in the existing topography. If the project information described in this report is incorrect, is altered, or if new information is available, we should be retained to review and modify our recommendations.

This report may not reflect the actual variations of the subsurface conditions across the site. The nature and extent of variations across the site may not become evident until construction commences. The construction process itself may also alter subsurface conditions. If variations appear evident at the time of construction, it may be necessary to reevaluate our recommendations after performing on-site observations and tests to establish the engineering impact of the variations.

The scope of our Geotechnical Engineering Study does not include an environmental assessment of the air, soil, rock, or water conditions either on or adjacent to the site. No environmental opinions are presented in this report.

If final grade elevations are significantly different from existing grades (more than plus or minus 1 ft), our office should be informed about these changes. If needed and/or if desired, we will reexamine our analyses and make supplemental recommendations.

BORINGS AND LABORATORY TESTS

Subsurface conditions at the site were evaluated by eight (8) borings drilled at the locations shown on the Boring Location Map, Figure 1. These locations are approximate and distances were measured using tape, angles, pacing, etc. The borings were drilled to depths of approximately 15 ft below the existing ground surface using a truck-mounted drilling rig. During drilling operations, the following samples were collected:

Type of Sample	Number Collected
Split-Spoon (with Standard Penetration Test)	19
Undisturbed Shelby Tube	3
Nx Rock Core	78.5 ft

Each sample was visually classified in the laboratory by a member of our Geotechnical Engineering staff. The geotechnical engineering properties of the strata were evaluated by the following tests:

Type of Test	Number Conducted
Natural Moisture Content	22
Atterberg Limits	4
Unconfined Compression (Rock)	5

The results of all laboratory tests are presented in graphical or numerical form on the boring logs illustrated on Figures 2 through 9. A key to classification terms and symbols used on the logs is presented on Figure 10. The results of the laboratory and field testing are also tabulated on Figure 11 for ease of reference.

Standard Penetration Test results are noted as “blows per ft” on the boring logs and Figure 11, where “blows per ft” refers to the number of blows by a falling hammer required for 1 ft of penetration into the soil/weak rock (N-value). Where hard or dense materials were encountered, the tests were terminated at 50 blows even if one foot of penetration had not been achieved. When all 50 blows fall within the first 6 in. (seating blows), refusal “ref” for 6 in. or less will be noted on the boring logs and on Figure 11.

Samples will be retained in our laboratory for 30 days after submittal of this report. Other arrangements may be provided at the request of the Client.

GENERAL SITE CONDITIONS

GEOLOGY

A review of the *Geologic Atlas of Texas, Austin Sheet*, indicates that this site is naturally underlain with the soils/rock (limestone) of the Edwards Group. Edwards limestone is generally considered hard in induration and typically contains harder zones/seams of chert and dolomite. Edwards limestone also typically contains karstic features in the form of open and/or clay-filled vugs, voids, and/or solution cavities that form as a result of solution movement through fractures in the rock mass. Key geotechnical engineering considerations for development supported on this formation will be the depth to rock, the expansive nature of the overlying clays, the condition of the rock, and the presence/absence of karstic features.

SEISMIC COEFFICIENTS

Based upon a review of Section 1613 *Earthquake Loads – Site Ground Motion* of the 2012 International Building Code, the following information has been summarized for seismic considerations associated with this site.

- Site Class Definition (Chapter 20 of ASCE 7): **Class C**. Based on the soil borings conducted for this investigation, the upper 100 feet of soil may be characterized as very dense soil and soft rock.
- Risk-Targeted Maximum Considered Earthquake Ground Motion Response Accelerations for the Conterminous United States of 0.2-Second Spectral Response Acceleration (5% Of Critical Damping) (Figure 1613.3.1(1)): $\mathbf{S_s = 0.063g}$. Note that the value taken from Figure 1613.3.1(1) is based on Site Class B and is adjusted per 1613.3.3.
- Risk-Targeted Maximum Considered Earthquake Ground Motion Response Accelerations for the Conterminous United States of 1-Second Spectral Response Acceleration (5% Of Critical Damping) (Figure 1613.3.1(2)): $\mathbf{S_1 = 0.034g}$. Note that the value taken from Figure 1613.3.1(2) is based on Site Class B and is adjusted per 1613.3.3.
- Values of Site Coefficient (Table 1613.3.3(1)): $\mathbf{F_a = 1.2}$
- Values of Site Coefficient (Table 1613.3.3(2)): $\mathbf{F_v = 1.7}$
- Where g is the acceleration due to gravity.

The Maximum Considered Earthquake Spectral Response Accelerations are as follows:

- 0.2 sec, adjusted based on equation 16-37: $\mathbf{S_{ms} = 0.075g}$
- 1 sec, adjusted based on equation 16-38: $\mathbf{S_{m1} = 0.058g}$

The Design Spectral Response Acceleration Parameters are as follows:

- 0.2 sec, based on equation 16-39: $\mathbf{S_{DS} = 0.050g}$
- 1 sec, based on equation 16-40: $\mathbf{S_{D1} = 0.039g}$

Based on the parameters listed above, Tables 1613.3.5(1) and 1613.3.5(2), and calculations performed using the United States Geological Survey (USGS) website, the Seismic Design Category for both short period and 1 second response accelerations is **A**. As part of the assumptions required to complete the calculations, a Risk Category of “I or II or III” was selected.

STRATIGRAPHY

The subsurface stratigraphy at this site can be described as 1 to 8 ft of lean clay overlying limestone. Since the borings are on three separate properties we have elected to keep this section as a general stratigraphy. Each stratum has been designated by grouping soils that possess similar physical and engineering characteristics. The boring logs should be consulted for more specific stratigraphic information. The lines designating the interfaces between strata on the boring logs represent approximate boundaries. Transitions between strata may be gradual.

GROUNDWATER

Groundwater was not observed in the borings either during or immediately before the Nx coring was started. Nx coring drilling, which involves the introduction of water to facilitate drilling operations, was utilized. We were unable to determine if and when groundwater was encountered within the rock. During coring of the rock, drilling fluid was loss indicating voids, fissures, or other karst features. It is possible for groundwater to exist beneath this site at shallow depths on a transient basis, particularly in weathered layers and following periods of precipitation. Fluctuations in groundwater levels occur due to variation in rainfall and surface water run-off. The construction process itself may also cause variations in the groundwater level.

FOUNDATION ANALYSIS

EXPANSIVE SOIL-RELATED MOVEMENTS

The anticipated ground movements due to swelling of the underlying soils at the site were estimated for slab-on-grade construction using the empirical procedure, Texas Department of Transportation (TxDOT) Tex-124-E, Method for Determining the Potential Vertical Rise (PVR). PVR values ranging from on the order of 1 in. or less to 1-1/4 in. were estimated for the stratigraphic conditions encountered in our borings. A surcharge load of 1 psi (concrete slab and sand cushion), an active zone varying from 1 to 8 ft (to top of limestone), and dry moisture conditions were assumed in estimating the above PVR values. Due to the structures proposed for this project, we do not feel that any PVR reduction options are necessary.

The TxDOT method of estimating expansive soil-related movements is based on empirical correlations utilizing the measured plasticity indices and assuming typical seasonal fluctuations in moisture content. If desired, other methods of estimating expansive soil-related movements are available, such as estimations based on swell tests and/or soil-suction analyses. However, the performance of these tests and the detailed analysis of expansive soil-related movements were beyond the scope of the current study. It should also be noted that actual movements can exceed the calculated PVR values due to isolated changes in moisture content (such as due to leaks, landscape watering....) or if water seeps into the soils to greater depths than the assumed active zone depth due to deep trenching or excavations.

FOUNDATION RECOMMENDATIONS

SITE GRADING

Site grading plans can result in changes in almost all aspects of foundation recommendations. We have prepared all foundation recommendations based on the existing ground surface and the stratigraphic conditions encountered at the time of our study. If site grading plans differ from existing grade by more than plus or minus 1 ft, RKCI must be retained to review the site grading plans prior to bidding the project for construction. This will enable RKCI to provide input for any changes in our original recommendations that may be required as a result of site grading operations or other considerations.

DRILLED, STRAIGHT-SHAFT PIERS

Drilled, straight-shaft piers may be considered to support the proposed lighting installations. Straight-shaft piers should be designed as friction units using an allowable side shear resistance of 2 ksf for the portion of the shaft extending into the intact, native limestone. Side shear resistance shall be neglected within the upper 3 ft of the pier shaft extending into the limestone. Based on the 15-ft maximum depth of exploration, pier depths should not exceed a depth of 15 ft below the ground surface existing at the time of our study without consulting RKCI.

If clay seams/and or voids are encountered within the limestone formation during drilled shaft excavations, the shafts must be extended to develop the required side shear resistance. Representatives from RKCI must be present at the time of construction to verify that conditions are similar to those encountered in our borings and that sufficient penetration into the limestone is achieved. For bid purposes, the owner should anticipate that deeper piers will be required in some areas. Consequently, contractors bidding on the job should include unit costs for various depths of additional pier embedment. Unit costs should include those for both greater and lesser depth in both rock and soil.

Pier Shafts

The pier shafts will be subject to potential uplift forces if the surrounding expansive soils within the active zone are subjected to alternate drying and wetting conditions. The maximum potential uplift force acting on the shaft may be estimated by:

$$Fu = 35*D$$

where:

Fu = uplift force in kips; and
D = diameter of the shaft in feet.

Allowable Uplift Resistance

Resistance to uplift forces exerted on the drilled, straight-shaft piers will be provided by the sustained compressive axial force (dead load) plus the allowable uplift resistance provided by the soil. The resistance provided by the soil depends on the shear strength of the soils adjacent to the pier shaft and below the depth of the active zone. The allowable uplift resistance provided by the soils at this site may be estimated using 1.3 ksf for the portion of the shaft extending a minimum of 5 ft into intact, native limestone. This value was evaluated using a factor of safety of 2.

Reinforcing steel will be required in each pier shaft to withstand a net force equal to the uplift force minus the sustained compressive load carried by that pier. We recommend that each pier be reinforced to withstand this net force or an amount equal to 1 percent of the cross-sectional area of the shaft, whichever is greater.

LATERAL RESISTANCE

Resistance to lateral loads and the expected pier behavior under the applied loading conditions will depend not only on subsurface conditions, but also on loading conditions, the pier size, and the engineering properties of the pier. Once pier sizes, concrete strength, and reinforcement are finalized, piers should be analyzed to determine the resulting lateral deflection, maximum bending moment, and ultimate bending moment. This type of analysis is typically performed utilizing a computer analysis program and usually requires a trial and error procedure to appropriately size the piers and meet project tolerances.

To assist the design engineer in this procedure, we are providing the following soil parameters for use in analysis. These parameters are in accordance with the input requirements of one of the more commonly used computer programs for laterally loaded piles, the LPile program. If a different program is used for analysis, different parameters and limitations may be required than what were assumed in selecting the parameters given below. Thus, if a program other than LPile is used, RKCI must be notified of the analysis method, so that we can review and revise our recommendations if required. Evaluating the lateral resistance on different pier sizes is outside our scope of work at this time.

The soil-related parameters required for input into the LPile program are summarized in the tables below:

Assumed Behavior for Analysis	Depth* (ft)	c (tsf)	k_s (pci)	k_c (pci)	ε_{50}	γ (pcf)	qu (psi)
Stiff Clay without free water (Reese)	0 - 8[(1)]	0.50	100	-	0.01	120	-
Strong Rock (Vuggy Limestone)	8[(1)] - 15	-	-	-	-	140	2,800

*Depth below the existing ground surface at the time of our study
[(1)] Or to top of limestone.

Where: c = undrained cohesion
k_s = p-y modulus (static)
k_c = p-y modulus (cyclic)
ε_{50} = strain factor
γ = effective unit weight

The parameters presented in the above table do not include factors of safety. Per the general procedures of Section 1810.3.3.2 of the IBC 2012 edition, the allowable lateral capacity shall not exceed one-half of the lateral load that produces a lateral movement of 1-inch at the ground surface.

It should be noted that where piers are spaced closer than three shaft diameters center to center, a modification factor should be applied to the p-y curves to account for a group effect. We recommend the following p-Multipliers for the corresponding center to center pier spacings.

Spacing (in shaft diameters)	p-Multiplier
3	1.0
2	0.75
1	0.50

PIER SPACING

Where possible, we recommend that the piers be spaced at a center to center distance of at least three shaft diameters on-center for straight-shaft piers. Such spacing will not require a reduction in the load carrying capacity of the individual piers.

If design and/or construction restraints require that piers be spaced closer than the recommended three shaft diameters, RKCI must re-evaluate the allowable bearing capacities presented above for the individual piers. Reductions in load carrying capacities may be required depending upon individual loading and spacing conditions.

AREA FLATWORK

It should be noted that ground-supported flatwork such as walkways, courtyards, etc. will be subject to the same magnitude of potential soil-related movements as discussed previously (see *Expansive Soil-Related Movement* section). Thus, where these types of elements abut rigid building foundations or light pole installations, differential movements should be anticipated. As a minimum, we recommend that flexible joints be provided where such elements abut the structure to allow for differential movement at these locations. Where the potential for differential movement is objectionable, it may be beneficial to consider methods of reducing anticipated movements.

FOUNDATION CONSTRUCTION CONSIDERATIONS

TEMPORARY CASING AND SLURRY TECHNIQUES

Groundwater seepage was observed in our boring at the time of our subsurface exploration. Some of the soils encountered in our boring were gravelly, and water may be encountered in these seams. Groundwater seepage and/or side sloughing may be encountered at the time of construction, depending on climatic conditions prevalent at the time of construction. Therefore, we recommend that the bid documents require the foundation contractor to specify unit costs for different lengths of casing and unit costs for slurry drilling techniques that may be required.

DRILLED PIERS

Each drilled pier excavation should be examined by a geotechnical engineer who is familiar with the geotechnical aspects of the subsurface stratigraphy, the structural configuration, foundation design details and assumptions, prior to placing concrete. This is to observe that:

- The shaft has been excavated to the specified dimensions at the correct depth established by the previously mentioned criteria;
- Sufficient depth/penetration has been achieved by the pier shafts;
- The shaft has been drilled plumb within specified tolerances along its total length;
- Excessive cuttings, buildup and soft, compressible materials have been removed from the bottom of the excavation.

REINFORCEMENT AND CONCRETE PLACEMENT

Reinforcing steel should be checked for size and placement prior to concrete placement. Placement of concrete should be accomplished as soon as possible after excavation to reduce changes in the moisture content or the state of stress of the foundation materials. No foundation element should be left open overnight without concreting.

PAVEMENT RECOMMENDATIONS

Recommendations for both flexible and rigid pavements are presented in this report. The Owner and/or design team may select either pavement type depending on the performance criteria established for the project. In general, flexible pavement systems have a lower initial construction cost as compared to rigid pavements. However, maintenance requirements over the life of the pavement are typically much greater for flexible pavements. This typically requires regularly scheduled observation and repair, as well as overlays and/or other pavement rehabilitation at approximately one-half to two-thirds of the design life. Rigid pavements are generally more "forgiving", and therefore tend to be more durable and require less maintenance after construction.

For either pavement type, drainage conditions will have a significant impact on long term performance, particularly where permeable base materials are utilized in the pavement section. Drainage considerations are discussed in more detail in a subsequent section of this report.

SUBGRADE CONDITIONS

We have assumed the subgrade in pavement areas of Pepper Rock Park will consist of either the surficial clay subgrades which we anticipate to be less than about 1 ft thick, recompacted on-site clays, placed and compacted as recommended in the *On-Site Clay Fill* section of this report, and/or the tan limestone rock assuming the surficial clays are stripped and removed. Based on our experience with similar subgrade soils, we have assigned a California Bearing Ratio (CBR) value of 3.0 and 10.0 for use in pavement thickness design analyses for the clay and limestone subgrade conditions, respectively. We strongly recommend removing the surficial soils and exposing the limestone subgrade and/or constructing roadway embankments following removal of the surficial clays with limestone millings placed to finished grades.

DESIGN INFORMATION

The following recommendations were prepared using the DARWin 3.1 software program which utilizes a procedure based on the 1993 “Guide for the Design of Pavement Structures” by the American

Association of State Highway and Transportation Officials (AASHTO). The following recommendations were prepared assuming a 20-yr design life and Equivalent Single Axle Loads (ESAL's) of 20,000 for light duty pavements. This traffic frequency is approximately equivalent to 500 two way daily traffic with approximately 2 percent of the traffic comprised of trucks for a design period of 20 years. This is the same type of traffic conditions utilized for City of Austin Street Classifications designated as Local Residential. **The Project Civil Engineer should review anticipated traffic loading and frequencies to verify that the assumed traffic loading and frequency is appropriate for the intended use of the facility.**

FLEXIBLE PAVEMENT

Flexible pavement sections recommended for this site are as listed in the table below:

	Flexible Pavement Components	
Subgrade Type	**Flexible Base (in.)**	**Surface Course (in.)**
Light Duty Traffic (Limestone Subgrade)	8	2
Light Duty Traffic (Clay Subgrade)	9	2

Garbage Dumpsters

Where flexible pavements are constructed at any site, we recommend that reinforced concrete pads be provided in front of and beneath trash receptacles. The dumpster trucks should be parked on the rigid pavement when the receptacles are lifted.

It is suggested that such pads also be provided in drives where the dumpster trucks make turns with small radii to access the receptacles. The concrete pads at this site should be a minimum of 6 in. thick and reinforced with conventional steel reinforcing bars or welded wire mats.

RIGID PAVEMENT

. We recommend that rigid pavement sections at this site consist of the following:

Traffic Type	Portland Cement Concrete
Light Duty Traffic	5 in.

We also recommend that rigid pavements be considered in areas of channelized traffic, particularly in areas where truck or bus traffic is planned, and particularly where such traffic will make frequent turns, such as for garbage dumpster areas.

We recommend that the concrete pavements be reinforced with bar mats. As a minimum, the bar mats should be No. 3 reinforcing bars spaced 18 in. on center in both directions. The concrete reinforcing should be placed approximately 1/3 the slab thickness below the surface of the slab, but not less than 2 in. The reinforcing should not extend across expansion joints.

Joints in concrete pavements aid in the construction and control the location and magnitude of cracks. Where practical, lay out the construction, expansion, control and sawed joints to form square panels, but not to exceed ACI 302.69 Code recommendations. The ratio of slab length-to-width should not exceed 1.25. Recommended maximum joint spacings are 12 ft longitudinal and 12 ft transverse.

All control joints should be formed or sawed to a depth of at least 1/4 the thickness of the concrete slab. Sawing of control joints should begin as soon as the concrete will not ravel, generally the day after placement. Control joints may be hand formed or formed by using a premolded filler. We recommend that all longitudinal and transverse construction joints be dowelled to promote load transfer. Isolation joints are needed to separate the concrete slab from fixed objects such as drop inlets, light standards and buildings. Expansion joint spacings are not to exceed a maximum of 75 ft and no expansion or construction joints should be located in a swale or drainage collection locations.

If possible, the pavement should develop a minimum slope of 0.015 ft/ft to provide surface drainage. Reinforced concrete pavement should cure a minimum of 3 and 7 days before allowing automobile and truck traffic, respectively.

PAVEMENT CONSTRUCTION CONSIDERATIONS

SUBGRADE PREPARATION

Areas to support pavements should be stripped of all vegetation and organic topsoil and the exposed subgrade should be proofrolled in accordance with the recommendations in the *Site Preparation* section under *Foundation Construction Considerations*.

In areas where clay will remain in place, the exposed subgrade should be moisture conditioned. This should be done after completion of the proofrolling operations and just prior to flexible base placement. Moisture conditioning is done by scarifying to a minimum depth of 6 in. and recompacting to a minimum of 95 percent of the maximum density determined from the Texas Department of Transportation Compaction Test (TxDOT, Tex-114-E). The moisture content of the subgrade should be maintained within the range of optimum moisture content to 3 percentage points above optimum until permanently covered.

DRAINAGE CONSIDERATIONS

As with any soil-supported structure, the satisfactory performance of a pavement system is contingent on the provision of adequate surface and subsurface drainage. Insufficient drainage which allows saturation of the pavement subgrade and/or the supporting granular pavement materials will greatly reduce the performance and service life of the pavement systems.

Surface and subsurface drainage considerations crucial to the performance of pavements at this site include (but are not limited to) the following:

1) Any known natural or man-made subsurface seepage at the site which may occur at sufficiently shallow depths as to influence moisture contents within the subgrade should be intercepted by drainage ditches or below grade French drains.
2) Final site grading should eliminate isolated depressions adjacent to curbs which may allow surface water to pond and infiltrate into the underlying soils. **Curbs should completely penetrate base materials and should be installed to sufficient depth to reduce infiltration of water beneath the curbs**.
3) Pavement surfaces should be maintained to help minimize surface ponding and to provide rapid sealing of any developing cracks. These measures will help reduce infiltration of surface water downward through the pavement section.

ON-SITE CLAY FILL

As discussed previously, the pavement recommendations for the clay subgrade conditions presented in this report were prepared assuming that on-site soils will be used for fill grading in proposed pavement areas. If used, we recommend that on-site soils be placed in loose lifts not exceeding 8 in. in thickness and compacted to at least 95 percent of the maximum density as determined by TxDOT, Tex-114-E. The moisture content of the fill should be maintained within the range of optimum water content to 3 percentage points above the optimum water content until permanently covered. We recommend that fill materials be free of roots and other organic or degradable material. We also recommend that the maximum particle size not exceed 4 in. or one half the lift thickness, whichever is smaller.

FLEXIBLE BASE COURSE

The flexible base course should be crushed limestone conforming to TxDOT Standard Specifications, Item 247, Type A, Grades 1 or 2. Base course should be placed in lifts with a maximum thickness of 8 in. and compacted to a minimum of 100 percent of the maximum density at a moisture content within the range of 2 percentage points below to 2 percentage points above the optimum moisture content as determined by Tex-113-E.

ASPHALTIC CONCRETE SURFACE COURSE

The asphaltic concrete surface course should conform to TxDOT Standard Specifications, Item 340, Type D. The asphaltic concrete should be compacted to a minimum of 92 percent of the maximum theoretical specific gravity (Rice) of the mixture determined according to Test Method Tex-227-F. Pavement specimens, which shall be either cores or sections of asphaltic pavement, will be tested according to Test Method Tex-207-F. The nuclear-density gauge or other methods which correlate satisfactorily with results obtained from project roadway specimens may be used when approved by the Engineer. Unless otherwise shown on the plans, the Contractor shall be responsible for obtaining the required roadway specimens at their expense and in a manner and at locations selected by the Engineer.

PORTLAND CEMENT CONCRETE

The Portland cement concrete should be air entrained to result in a 4 percent plus/minus 1 percent air, should have a maximum slump of 5 inches, and should have a minimum 28-day compressive strength of

4,000 psi. A liquid membrane-forming curing compound should be applied as soon as practical after broom finishing the concrete surface. The curing compound will help reduce the loss of water from the concrete. The reduction in the rapid loss in water will help reduce shrinkage cracking of the concrete.

CONSTRUCTION RELATED SERVICES

CONSTRUCTION MATERIALS TESTING AND OBSERVATION SERVICES

As presented in the attachment to this report, *Important Information About Your Geotechnical Engineering Report*, subsurface conditions can vary across a project site. The conditions described in this report are based on interpolations derived from a limited number of data points. Variations will be encountered during construction, and only the geotechnical design engineer will be able to determine if these conditions are different than those assumed for design.

Construction problems resulting from variations or anomalies in subsurface conditions are among the most prevalent on construction projects and often lead to delays, changes, cost overruns, and disputes. These variations and anomalies can best be addressed if the geotechnical engineer of record, RKCI is retained to perform construction observation and testing services during the construction of the project. This is because:

- RKCI has an intimate understanding of the geotechnical engineering report's findings and recommendations. RKCI understands how the report should be interpreted and can provide such interpretations on site, on the client's behalf.
- RKCI knows what subsurface conditions are anticipated at the site.
- RKCI is familiar with the goals of the owner and project design professionals, having worked with them in the development of the geotechnical workscope. This enables RKCI to suggest remedial measures (when needed) which help meet the owner's and the design teams' requirements.
- RKCI has a vested interest in client satisfaction, and thus assigns qualified personnel whose principal concern is client satisfaction. This concern is exhibited by the manner in which contractors' work is tested, evaluated and reported, and in selection of alternative approaches when such may become necessary.
- RKCI cannot be held accountable for problems which result due to misinterpretation of our findings or recommendations when we are not on hand to provide the interpretation which is required.

BUDGETING FOR CONSTRUCTION TESTING

Appropriate budgets need to be developed for the required construction testing and observation activities. At the appropriate time before construction, we advise that RKCI and the project designers meet and jointly develop the testing budgets, as well as review the testing specifications as it pertains to this project.

Once the construction testing budget and scope of work are finalized, we encourage a preconstruction meeting with the selected contractor to review the scope of work to make sure it is consistent with the construction means and methods proposed by the contractor. RKCI looks forward to the opportunity to