TIRZ 17 REDEVELOPMENT AUTHORITY

APPENDIX H

GEOTECHNICAL INVESTIGATION

GEOTECHNICAL STUDY
KIMBERLEY LANE IMPROVEMENTS PROJECT
HOUSTON, TEXAS

Prepared for

Lockwood, Andrews & Newnam, Inc.
Houston, Texas

January 2010

Prepared by

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Project No. 09.13.106

January 8, 2010
TWEI Project No.: 09.13.106

Lockwood, Andrews & Newnam, Inc.
2929 Briarpark Drive, Suite 400
Houston, Texas 77042-3720

Attn: Ms. Tara G. Godwin, P.E.

GEOTECHNICAL STUDY
KIMBERLEY LANE IMPROVEMENTS PROJECT
HOUSTON, TEXAS

Dear Ms. Godwin:

We are pleased to submit our report covering the geotechnical study for the above referenced project. Should you have any questions or comments concerning the information contained herein, please contact us.

We appreciate the opportunity to serve you during this phase of the project, and look forward to continue our services during the construction phase and on future projects.

Sincerely,

Tolunay-Wong Engineers, Inc.
TBPE Registration Number F-124

Zeki A. Tolunay, P.E.

STATE OF TEXAS
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EXECUTIVE SUMMARY

This geotechnical study addresses the proposed Kimberley Lane Improvements Project from east of the West Sam Houston Parkway to east of West Bough Lane in Houston, Texas. The principal findings and conclusions of the geotechnical study are summarized as follows:

The generalized subsurface soil stratigraphy beneath the pavement, inferred from project borings B-1 through B-3, consists of mostly medium to high plasticity cohesive lean clays, fat clays with sand, and fat clays to the 20-ft termination depth in borings B-2 and B-3, and to 13-ft depth in boring B-1, followed by semi-cohesive clayey sands to 23.5-ft depth, and cohesionless sands with silt to the 25-ft termination depth. Substructures observed in the recovered cohesive soil samples typically consisted of sand pockets/seams, claystones, ferrous nodules, and calcareous nodules/deposits.

Groundwater observations were made during drilling. Boring B-2 was converted to a standpipe piezometer upon completion of sampling. Based on the groundwater level readings summarized in Table 3-1 and in Appendix B, it appears that the static groundwater level at the site at the beginning of December 2009 was at about 9.5-ft depth below existing grade.

Excavation for the storm sewer in the general area of borings B-2 and B-3 is anticipated to encounter cohesive soils, based on limited subsurface information. Water seepage or surface runoff within cohesive soils can probably be handled by pumping from sumps, as defined in ASTM D 2321. Excavation in the general area of boring B-1 is anticipated to encounter water bearing semi-cohesive clayey sand soils below the 13-ft depth. Mechanical dewatering may be necessary if excavation exposes such soils. The piezometer should be monitored prior to construction.

Bedding and backfill for storm sewers should be constructed in accordance with the City of Houston Department of Public Works and Engineering Standards dated October 2002, or an equivalent standard. In accordance with the current OSHA regulations, the observed soils in the project borings would be classified as "Type C".

Based on the results of our field investigation and laboratory testing, we recommend the subgrade soils within the pavement areas to be stabilized with a lime-fly ash mixture. For preliminary planning purposes, the subgrade soils may be stabilized with 4% lime and 8% fly ash by dry weight. We recommend establishing a separate line item for stabilizer since the actual stabilization requirements should be verified in the field by trial. Lime-fly ash stabilization procedures should be performed in accordance with City of Houston Specification Item 02337, *Lime-flyash Stabilized Subgrade*. The subgrade should be stabilized to a minimum 6-in. depth.

The City of Houston Specifications require that compaction should begin immediately after final mixing. The stabilized soil should be compacted to at least 95% of the standard Proctor maximum dry density (ASTM D 698). The compacted moisture content should be at a moisture content of optimum to three percent above optimum.

1. INTRODUCTION

Lockwood, Andrews & Newnam, Inc. (LAN) retained Tolunay-Wong Engineers, Inc. (TWEI) to perform a geotechnical study for the Kimberley Lane Improvements Project from east of the West Sam Houston Parkway to east of West Bough Lane in Houston, Texas (Key Map 489 G/H). The work was performed in general accordance with TWEI Proposal No. P09-G087, dated September 21, 2009, that was authorized with the "Agreement Between Engineer And Geotechnical Engineer For Professional Services" dated October 7, 2009 and executed by Mr. Rafael Ortega of LAN. Ms. Tara G. Godwin, P.E., of LAN provided project details via e-mail transmittal on April 24, 2009.

The proposed improvements include a full concrete curb and gutter pavement section and storm sewer construction. The improved section length will be approximately 1,300 lineal ft. The project layout is shown in Figure 1.

2. SCOPE OF STUDY

The study included field exploration to obtain subsurface information and to secure representative soil samples, laboratory testing to measure selected soil engineering properties, and geotechnical analyses to develop design criteria to assess utility line excavation and bedding and backfill requirements and pavement recommendations. Information developed as part of the study includes:

- Soil stratigraphy and groundwater condition;
- Existing pavement thickness and composition;
- Evaluation of subsurface soils for use as bedding and backfill material;
- Soil classifications for OSHA trenching and shoring recommendations;
- Developing guidelines for utility line excavation and bedding and backfill based on City of Houston specifications;
- Pavement subgrade preparation and stabilization recommendations; and
- Rigid pavement design in accordance with AASHTO guidelines.

Environmental assessment, recommendations for areas not covered by the boring layout, and site-specific fault studies were outside the scope of work for this study.

3. FIELD EXPLORATION AND LABORATORY TESTING

3.1 Test Borings

The fieldwork was performed on October 14, 2009. The field exploration consisted of drilling, logging and sampling three (3) soil borings (B-1 through B-3). The borings were drilled to 25-ft depth below existing grade. TWEI personnel used approximate methods to locate the test borings in the field. TWEI personnel drilled and sampled the borings and converted boring B-2 into a standpipe piezometer after completion of sampling. A TWEI representative logged the boreholes, and measured the groundwater levels. The approximate boring locations are shown on Figure 1.

3.2 Drilling Methods

The field operations were performed in general accordance with *Standard Practice for Soil Investigation and Sampling by Auger Borings* [American Society for Testing and Materials (ASTM) D 1452]. The soil borings were drilled with truck mounted equipment, after coring the existing pavement, and were dry augered to completion depth in order to evaluate the presence of either perched or free water. Core-Ect cored the existing pavement. Soil samples from the borings were generally taken at continual 2-ft intervals to 12-ft depth, at the 13 to 15-ft, and 18 to 20-ft depth intervals, and from the 23-ft depth to the 25-ft termination depth. The open boreholes B-1 and B-3 were grouted after completion of sampling. Boring B-2 was converted to standpipe piezometer PZ-2 upon completion of sampling.

3.3 Soil Sampling

Cohesive soil and soil inferred to be cohesive during drilling were obtained by hydraulically pushing a 3-inch diameter, thin-walled tube a distance of about 24 inches. The field sampling procedure was conducted in accordance with the *Standard Practice for Thin-Walled Tube Sampling of Soils* (ASTM D 1587). The field technician extruded the soil samples in the field, visually classified the recovered soils, and obtained a penetration resistance measurement of the recovered cohesive soils using a calibrated pocket penetrometer. The pocket penetrometer readings are presented on the boring logs in Appendix A. Based on experience with local soils, a factor of 0.67 was applied to penetrometer reading to estimate soil consistency. Representative portions of the soil samples extruded in the field were wrapped in foil, placed into plastic bags, and transported to the laboratory.

Cohesionless soil and soil interpreted to be granular during drilling in boring B-1 were obtained by driving a 2-inch-diameter, split barrel sampler. The sampler was driven 18 inches by a 140-pound hammer falling about 30 inches in general accordance with the *Standard Method for Penetration Test and Split-Barrel Sampling of Soils* (ASTM D 1586). The field technician recorded the number of blows required to drive the sampler through three consecutive 6-inch sampling intervals. The sum of the blows required to penetrate the final 12 inches is the Standard Penetration Test (SPT) "N" value. The samples obtained from the split-barrel sampler were visually classified and sealed in plastic bags for transport to our laboratory. Relative density of granular soil and consistency of cohesive soil were inferred from the N_{60} value (i.e., SPT "N" blowcount value corrected for field procedure to an average energy ratio of 60%). The SPT data is presented on the boring log in Appendix A.

The soil immediately beneath the concrete pavement was sampled with the auger. The recovered samples were secured in plastic bags and delivered to the laboratory.

3.4 Boring Logs

Our interpretations of general soil and groundwater conditions at the boring locations are included on the boring logs. The interpretations of the soil types throughout the boring depth and the locations of strata change were based on visual classifications during field sampling, and laboratory testing in accordance with *Standard Practice for Classification of Soils For Engineering Purposes* (ASTM D 2487) and *Standard Practice for Description and Identification of Soils (*ASTM D 2488). The boring logs include the type and interval depth for each sample along with the corresponding penetration resistance of soils. The project boring logs and a key to the terms and symbols used on the logs are presented in Appendix A.

3.5 Water-Level Measurements

Groundwater level observations were made during drilling. We grouted borings B-1 and B-3 upon completion of sampling. The open borehole B-2 was converted to 1-in. diameter standpipe piezometer upon completion of sampling. The piezometer comprised of a PVC screen with 0.010-in. slots and a PVC riser tube extending to the ground surface. We filled the annulus with 20/40 silica sand to 8-ft depth, and placed bentonite chips to grade. Detailed information concerning installation and groundwater level measurement is presented on the Piezometer Installation Report in Appendix B.

3.6 Laboratory Testing

Laboratory tests were performed on selected soil samples to measure physical and engineering properties. A brief description of the tests is presented below.

- ***Laboratory Determination of Water (Moisture) Content of Soil and Rock -ASTM D 2216.*** The water content of the soil/rock, expressed as a percentage, is defined at the ratio of the mass of fluid to the mass of soil solids. The moisture content may provide an indication of cohesive soil shear strength and compressibility when compared to Atterberg Limits.

- ***Liquid Limit, Plastic Limit and Plasticity Index of Soils -ASTM D 4318.*** These tests, also known as Atterberg Limits, are used for soil classification and provide an indication of volume change potential when considered in conjunction with the natural moisture content. The liquid limit and plastic limit establish the boundaries of the consistency states of plastic soils. The Plasticity Index (PI) is the difference between the liquid limit and the plastic limit.

- ***Amount of Material in Soils Finer Than the No. 200 (75-μm) Sieve – ASTM D 1140.*** This test measures the total amount of material in soils finer than the No. 200 sieve. The test result is presented as the percentage of silt and clay sizes by weight in the sample and may provide an indication of the soil hydraulic conductivity (permeability).

- ***Unconfined Compressive Strength of Cohesive Soil - ASTM D 2166.*** This test measures the unconfined compressive strength of cohesive soils in undisturbed or remolded condition, using strain-controlled deformation under load application. The undrained shear strength of a cohesive soil sample is one-half of the unconfined compressive strength.

- ***Density of Soil – ASTM D 2937.*** Total unit weight of the soil aggregate is defined as the weight of the aggregate (soil plus water) per unit volume. Knowing the total unit weight and moisture content, dry unit weight can be computed.

The results of the laboratory tests are included on the boring logs in Appendix A.

4. SUBSURFACE CONDITIONS

4.1 General

Our interpretations of soil and groundwater conditions at the site are based on information obtained at the soil boring locations only. The project boring logs are presented in Appendix A. This information has been used as the basis for our conclusions and recommendations. Subsurface conditions may vary between soil boring locations. Significant variations at areas not explored by the project borings will require re-evaluation of our recommendations.

4.2 Regional Geology

The site is located in an area identified with the Beaumont Formation. Beaumont Formation includes mainly stream channel, point-bar, natural levee, backswamp, and to a lesser extent coastal marsh and mud-flat deposits consisting of mostly clay, silt, and sand. Concretions of calcium carbonate (calcareous nodules) and concentrations of iron oxide and iron-manganese oxides (ferrous nodules) are commonly found in the zone of weathering. The surface is almost featureless, characterized by relict river channels shown by meander patterns and pimple mounds on meanderbelt ridges, separated by areas of low, relatively smooth, featureless backswamp deposits without pimple mounds. The thickness of the formation is about 100 ft.

In the general site area Beaumont Formation is dominantly clay and mud of low permeability, high water-holding capacity, high compressibility, high to very high shrink/swell potential, poor drainage, low shear strength and high plasticity. Geologic units include interdistributary muds, abandoned channel-fill muds, and overbank fluvial muds.

4.3 Existing Pavement Thicknesses

The existing pavement at the boring locations were drilled through prior to accessing the underlying soils. Pavement section thicknesses and composition are noted on each boring log. A summary of the existing pavement section thickness and composition at the borehole locations is presented in the following table.

Table 4-1
Existing Pavement Thicknesses

Boring	Measured Section Thickness and Composition
B-1	2.5" Asphalt + 21.5" Concrete + Stabilized Clayey Sand "Fill"
B-2	12" Concrete + Stabilized Clayey Sand/Lean Clay "Fill"
B-3	9.5" Concrete + Stabilized Silty Sand "Fill"

4.4 Soil Stratigraphy

The generalized subsurface soil stratigraphy beneath the pavement, inferred from project borings B-1 through B-3, consists of mostly medium to high plasticity cohesive [lean clay (CL), fat clay with sand (CH), fat clay (CH)] soils to the 25-ft termination depth in borings B-2 and B-3, and to about 13-ft depth in boring B-1, underlain by semi-cohesive [clayey sand (SC)] soils to about 23.5-ft depth, followed by cohesionless [poorly graded sand with silt (SP-SM)] soils to the 25-ft termination depth. Substructures observed in the recovered cohesive soil samples typically consisted of sand pockets/seams, claystones, ferrous nodules, and calcareous nodules/deposits.

The soil immediately beneath the concrete pavement to about 6-ft depth in borings B-1 and B-2 was inferred to be clayey sand, lean clay fill. We observed about 4.5 in. thick silty sand fill layer immediately beneath the concrete pavement in boring B-3. The fill was stabilized.

It should be stressed that it is relatively difficult in practice to accurately delineate fill from similar natural soils. Fill classifications are made based upon visual observations and require considerable judgment. The interpreted fill depths may vary somewhat from actual conditions.

Detailed descriptions of the soils encountered are given on the boring logs in Appendix A.

4.5 Soil Properties

We measured liquid limits of 48% and 45%, with corresponding plasticity indices of 28 and 25, and fines contents of 71% and 50% on respective lean clay with sand and sandy lean clay fill samples recovered from the upper 6-ft depth in boring B-2, indicative of moderately high shrink/swell potential with moisture variation. In situ moisture contents of the samples were three percentage points less than and equal to their corresponding plastic limit. We measured liquid limits ranging from 51% to 68%, with corresponding plasticity indices of 31 to 45, on selected four fat clay samples recovered from the upper 10-ft depth in borings B-1 through B-3, indicative of a high shrink/swell potential with moisture variation. In situ moisture contents of the samples were between equal to and four percentage points more than their corresponding plastic limit. Measured fines contents of the samples ranged from 75% to 86%.

Unconfined compression tests on the two lean clay fill samples recovered from the 3 to 6-ft depth range in boring B-1, and the fat clay with sand sample recovered from the 6 to 8-ft depth range in boring B-3 measured undrained shear strengths ranging from 1,490 psf to 2,010 psf. Total unit weights of selected ten cohesive samples ranged from 125 pcf to 135 pcf. We obtained SPT "N" value of 31 blows per foot within the lean clay stratum between 8-ft and 9.5-ft depths in boring B-1. Apparent shear strengths of the cohesive samples recovered from the project borings, based on pocket penetrometer readings, ranged from about 170 psf to more than 3000 psf. Based on calibrated pocket penetrometer readings, the SPT data, and the undrained shear strengths, the cohesive soils recovered from the project borings were inferred to have very soft to very stiff-hard, but mostly stiff to very stiff consistencies. Very soft to firm consistency samples were recovered from the upper 6-ft depth in boring B-2 (fill) and between 8 and 10-ft depths in boring B-3.

We measured respective SPT "N" values of more than 50 and 39 blows per foot within semi-cohesive clayey sand and cohesionless poorly graded sand with silt strata at the 18.5-ft to 25-ft depth range in boring B-1, indicative of very dense and dense relative density. The clayey sand sample recovered from the 18.5-ft depth yielded liquid limit of 25% and plasticity index of 9. Fines contents of the recovered two clayey sand samples were 30% and 35%. The cohesionless sample recovered from the 23.5-ft depth had fines content of 12%.

4.6 Groundwater Observations

Based on the groundwater measurements obtained in piezometer PZ-2 presented in Appendix B, the static groundwater level ranged from 8.5-ft to 9.5-ft depth below existing grade between mid October and early December 2009.

Fluctuations in groundwater levels may occur with changes in seasonal and climatic conditions. Depending on when the project is developed, the groundwater condition observed in early December 2009 may not be representative during construction. We recommend that the water level be verified prior to construction.

5. GEOTECHNICAL RECOMMENDATIONS

5.1 Utility Recommendations

The proposed storm sewer lines likely will be installed within the upper 20-ft depth based on the requested 25-ft depth boring program. The recommendations presented in this report are based on an assessment of the observed subsurface conditions at widely spaced borings. Excavation retention and construction dewatering are the responsibility of the contractor. The contractor should collect additional subsurface information as necessary to determine if the conditions reported herein are representative.

5.1.1 Trenching and Shoring

All vertical excavations deeper than 5-ft must be provided with a trench safety system in accordance with the current Occupational Safety and Health Administration (OSHA) standards (29 CFR, Part 1926, Subpart P). The OSHA standards include provisions for the design of sloping and/or benched trench excavations in single or multiple layer soil stratigraphies less than 20-ft deep, in lieu of bracing and shoring. Sloping or benching for excavations greater than 20-ft deep must be designed by a registered professional engineer. The regulations specify maximum slope declivities contingent on soil type. The cohesive/semi-cohesive soils encountered in the project borings may be classified as "Type C". A trench shield (box), if used, should be designed to withstand lateral loads imposed by specific site soil conditions.

It should be noted that the soil stratigraphy and groundwater conditions encountered during excavations may vary from those observed in the project borings or characterized herein. The contractor should collect additional subsurface data as he deems necessary to determine if the conditions described in this report are representative on a station-by-station basis. It is also mandatory that all excavations and retaining structures be monitored on a continuous basis by experienced personnel who can make evaluations as to the appropriateness of the retention system being used.

5.1.2 Utility Bedding and Backfill Criteria

Bedding and backfill for storm sewer lines may be constructed using the City of Houston Department of Public Works and Engineering *Standard Construction Specifications for Wastewater Collection Systems, Water Lines, Storm Drainage, and Street Paving*, dated October 2002, or an equivalent standard. In accordance with these specifications, the backfill requirements should conform to Section 02317 – "Excavation and Backfill for Utilities," and Section 02320 – "Utility Backfill Material."

Storm Sewers. Bedding recommendations outlined on Drawing No. 02317-03 dated October 1, 2002 are anticipated to be applicable for storm sewer lines bedded within stable soils.

Backfill Placement. Backfill placement should be in accordance with the City of Houston Standard Construction Specifications. Trench zone backfill placement and compaction requirements are provided in Section 02317, *Excavation and Backfill for Utilities*, and are summarized as follows.

Table 5-1
Utility Backfill Recommendations

Soil Type	Maximum Lift Thickness (compacted)	Minimum Compacted Density (ASTM D 698)	Compaction Moisture Content (ASTM D 698)
Bank Run Sand	9 inches	95%	±3 points
Cement-Stabilized Sand	12 inches	95% [1]	Less than optimum [1]
Select Fill	6 inches	95%	± 2 points
Random Fill [2]	9 inches (clay) 12 inches (sand)	90%	None

[1] As determined by ASTM D 558
[2] Random fill is to be used outside pavement areas.

We recommend a minimum 95% relative compaction for random fill. Backfill material specifications for bank run sand, select backfill and random backfill are provided in the Specification Section 02320, *Utility Backfill Materials*. The City of Houston Standard Construction Specifications preclude the use of silt, organic clay, and peat as utility backfill materials. Cement-stabilized sand material specifications are provided in Section 02321, *Cement Stabilized Sand.*

The City of Houston Standard Construction Specifications require in-place density testing of pipe embedment and trench zone backfill at a minimum frequency of one test per 40 linear feet (embedment) and two tests per 40 linear feet (backfill), with a minimum of three density tests for each shift of work (Section 02317).

5.1.3 Groundwater Control

Construction dewatering, if required, and excavation retention are the contractor's responsibilities. We provide excavation planning comments and suggestions for informational purposes only. These comments may be used to review the contractor's proposed excavation procedures.

Excavations in the general area of borings B-2 and B-3 are anticipated to encounter cohesive soils, based on limited subsurface information. Water seepage or surface runoff within cohesive soils can probably be handled by pumping from sumps, as defined in ASTM D 2321.

We encountered semi-cohesive clayey sand (SC) soils below the 13-ft depth, followed by cohesionless sands with silt (SP-SM) in boring B-1. The water level in piezometer PZ-2 was measured at the 9.5-ft depth at the beginning of December 2009. Mechanical dewatering may be necessary if excavation exposes water bearing semi-cohesive soils.

The contractor is responsible for assessing the need for groundwater control at the site. The condition of the bearing surface should be carefully monitored during construction to check for possible bottom heave or other instabilities. Undercutting may be employed to achieve competent bearing conditions. In such cases, grade adjustments can be made by placing lean concrete, or backfilling to grade with cement-stabilized sand (one bag of cement per cubic yard of sand). If precast sections are used, we recommend using cement-stabilized sand for the bedding.

It should be noted that the soil stratigraphy and groundwater conditions encountered during excavations may vary from those observed in the project borings or characterized herein. The contractor should collect additional subsurface information, as he deems necessary, to determine if the existing conditions are representative of those described in this report. It is also mandatory that all excavations and retaining structures be monitored on a continuous basis by experienced personnel who can make evaluations as to the appropriateness of the retention system being used.

5.2 Pavement Subgrade Preparation

Pavement subgrade preparation including stripping, proof-rolling, subgrade stabilization, and fill placement may be required. These considerations are addressed in the following paragraphs.

5.2.1 Site Stripping and Proof Rolling

We understand that areas within the project alignment will be stripped of the existing pavement and base material. After stripping, the exposed pavement subgrade should be proof-rolled to detect zones of soft or wet soils. If encountered, soft or wet soils should be undercut and replaced with material of similar physical and moisture characteristics. Particular care should be given to limiting excessive subgrade drying and desiccation as a means of reducing subsequent subgrade swelling after construction where medium to high plasticity cohesive soils are exposed.

The stripping and proof-rolling should be witnessed by the Geotechnical Engineer or a representative. Accomplish proof-rolling by making a minimum of 2 complete passes with a heavy rubber-tire vehicle such as a pneumatic-tire roller or a fully-loaded tandem-axle dump truck with a loaded weight of 20 tons, or approved equal, under supervision and direction of the independent testing laboratory. Excavate and recompact areas of failure as specified herein.

5.2.2 Grading

The roadway alignment should be graded such that positive surface drainage away from the work areas is established and maintained at all times. Water should not be allowed to pond on the surface during construction. Failure to achieve good drainage could result in significant construction delays during periods of inclement weather. The site is relatively flat and natural drainage may not be adequate for construction operations during wet weather conditions. The establishment of temporary drainage swales may be necessary to expedite construction during periods of extended rainfall.

5.2.3 Fill Placement for Roadway

Fill required for grading at roadway should preferably be cohesive soil, and should be free of organic matter and excessive silt. All fill should be placed in lifts not exceeding 8-in. loose measure, and compacted to at least 95% of the standard Proctor test maximum dry density (ASTM D 698) at a moisture content within two percentage points of the optimum moisture content. Fill placement should be tested and documented by the Geotechnical Engineer or a representative.

Fill required for grading not under proposed paved roadway should be earth fill having a plasticity index similar to the in situ cohesive soils and should be free of organic matter and excessive silt. Fill not placed under proposed roadway pavement should be compacted to at least 90% relative compaction and at a moisture content within five percentage points of the optimum moisture content (ASTM D 698).

5.2.4 Subgrade Stabilization

The near surface soils along the proposed project alignment consist primarily of medium to high plasticity lean clays and fat clays. Roadway subgrade stabilization should be considered because stabilization will help prevent construction delays due to inclement weather, reduce shrink/swell potential (high PI clays), and increase the modulus of subgrade reaction and thus, the pavement life.

Based on the results of our field investigation and laboratory testing, we recommend the subgrade soils within the pavement areas to be stabilized with a lime-fly ash mixture. For preliminary planning purposes, the subgrade soils may be stabilized with 4% lime and 8% fly ash by dry weight. We recommend establishing a separate line item for stabilizer since the actual stabilization requirements should be verified in the field by trial. Lime-fly ash stabilization procedures should be performed in accordance with City of Houston Specification Item 02337, *Lime-flyash Stabilized Subgrade*. The subgrade should be stabilized to a minimum 6-in. depth.

The City of Houston Specifications require that compaction should begin immediately after final mixing. The stabilized soils should be compacted to at least 95% of the standard Proctor maximum dry density (ASTM D 698). The compacted moisture content should be at a moisture content of optimum to three percent above optimum.

5.3 Pavement Construction Design Recommendations

Current plans call for replacement of the existing asphalt topped concrete roadway. The data presented in this report have been used for analysis of pavement design requirements in accordance with the "*AASHTO Guide for Design of Pavement Structures – 1993*" prepared by the American Association of State Highway and Transportation Officials (AASHTO) and HCPID-AED "*Regulations of Harris County, Texas For The Approval And Acceptance Of Infrastructure*", dated May 15, 2002. The design approach includes certain modifications to the "AASHTO Interim Guide for Design of Pavement Structures, 1981" which was developed as a result of the AASHTO Road Test program and based on road user definition of failure. The primary basis for the AASHTO pavement prediction method is cumulative heavy axle load applications. A mixed traffic stream of different axle loads and configurations is converted into an equivalent number of heavy load applications, termed 18-kip Equivalent Single Axle Loads (18-kip ESAL), using load equivalency factors determined at the AASHTO Road Test. The general methodology in the *AASHTO Guide for Design of Pavement Structures, 1993* (AASHTO Design Guide) relates the total number of 18-kip ESAL's to the service life of the pavement structure. The proposed roadways are collectors.

We performed rigid pavement design analyses that included selection of design parameters, in accordance with the AASHTO Design Guide and the HCPID-AED May 15, 2002 regulations, Section 7. The following parameters were used in our analyses.

5.3.1 Design Parameters

The AASHTO pavement prediction method requires the definition of four categories of parameters. The categories include design variables, performance criteria, material properties for structural design, and structural characteristics. The following paragraphs describe the parameters used in our pavement design analysis. The selected parameters are in general accordance with the current AASHTO Design Guide, Section III, "*Paving Design Requirements*" of the HCPID-AED, and Section 7 of the HCPID-AED "*Regulations of Harris County, Texas For The Approval And Acceptance of Infrastructure*", dated May 15, 2002.

Time Constraints. Selection of performance analysis period inputs will affect the pavement design from the dimension of time. The performance period refers to the period of time that an initial pavement structure will last before rehabilitation. The analysis period refers to the period of time for which the analysis is conducted. We used a 20-year performance period as a design basis for this project based on the August 1988 Guidelines, Section III, Paving Design Requirements of the HCPID-ED

Traffic. The design procedure is based on cumulative expected 18-kip Equivalent Single Axle Load (ESAL) applications during the analysis period. The AASHTO Design Guide presents a procedure for converting a mixed traffic stream of different axle loads and axle configurations into a design traffic number. Each expected axle load is converted into an equivalent number of

18-kip single axle loads and these loads are summed over the performance period. We used the HCPID-ED recommended 10 million 18-kip ESAL's over a 20-year period.

Reliability. Reliability is defined, as the probability that a pavement section designed using the AASHTO procedures will perform satisfactorily for the design period, given the assumed traffic and environmental conditions. Application of the reliability concept requires definition of the functional classification of the facility, selection of reliability level, and selection of a standard deviation that is representative of local conditions.

For this project, we used a reliability of 95% as required by the May 15, 2002, Section 7, *Paving* requirements of the HCPID-ED. Based on the performance prediction error that was developed at the original AASHTO Road Test, a standard deviate (Z_R) value of -1.645 corresponding to the 95% level of reliability selected was used in the rigid pavement design. An overall standard deviation (S_o) of 0.35 for the projection of future 18-kip ESAL traffic was used in the rigid pavement design.

Environmental Effects. Loss of riding quality and serviceability can result from temperature and moisture changes affecting the strength, durability, and load carrying capacity of the pavement and roadbed materials, as well as roadbed swelling due to expansive potential of the subgrade soils. Providing proper drainage and surficial sealing can control moisture effects. Effects of subgrade treatment and drainage on the pavement design are included in such parameters as the loss of support factor and the drainage coefficient, which are discussed in the following paragraphs.

Serviceability. The serviceability of a pavement is defined as its ability to serve the types of traffic that use the facility. The primary measure of serviceability is the Present Serviceability Index (PSI) which ranges from 0 (impossible road) to 5 (perfect road). The basic design philosophy of the AASHTO Guide is the serviceability/performance concept. This concept provides a means of designing a pavement based on a specific total traffic volume and a minimum level of serviceability desired at the end of the performance period (Terminal Serviceability Index, p_t). Selection of p_t is based on the lowest index that will be tolerated before rehabilitation, resurfacing, or reconstruction becomes necessary. The AASHTO Design Guide suggests a p_t of 2.5 (used to indicate pavement in fair condition) or higher for design of major highways, and 2.0 (used to indicate pavement in poor condition) for highways with lesser traffic volumes. Original or Initial Serviceability Index (p_o) of 4.5 for rigid pavements which was observed at the AASHTO Road Test, was used in our analysis. Once p_o and p_t are established, the following equation is applied to define the total change in the serviceability index:

$$\Delta PSI = p_o - p_t$$

We used a p_o of 4.5 and a p_t of 2.5 for the rigid pavement analysis based on the AASHTO Design Guide and the May 15, 2002, Section 7, *Paving* requirements of the HCPID-ED.

Modulus of Subgrade Reaction (k). The strength of the subgrade for design of rigid pavements is characterized by the modulus of subgrade reaction (k). The value of k depends on the modulus of elasticity of the subgrade soils. Stabilization and compaction of the subgrade soils will typically result in a composite k value of about 350 pci.

Modulus of Rupture. The modulus of rupture (M_R) for the concrete pavement as required by the design procedure is the mean value determined after 28 days using third-point loading (AASHTO T97). Because of the treatment of reliability in the AASHTO Design Guide, it is strongly recommended that the normal construction specification for modulus of rupture (flexural strength) not be used as input, since it represents a value below which only a small percent of the distribution may lie. If it is desirable to use the construction specification, then some adjustment should be applied, based on the standard deviation of modulus of rupture and the percent of the strength distribution that normally falls below the specification. The May 15, 2002 Section 7, requirements of the HCPID-ED requires a 28 day concrete compressive strength of 3000 psi using a minimum of 5.5 sacks of cement per cubic yard, which is equivalent to a Modulus of Rupture, S'c = 570 psi.

Loss of Support. This factor accounts for the potential loss of support for rigid pavements arising from subbase erosion and/or differential vertical soil movement. It is treated in the actual design procedure by diminishing the effective modulus of subgrade reaction values based on the size of the void that may develop beneath the slab. Recommended loss of support factors (L.S.) in the AASHTO Design Guide range from 1.0 to 3.0 for stabilized soils. Recommended L.S. in the May 15, 2002, Section 7, *Paving* requirements of the HCPID-ED is 1.0. We used a L.S. of 1.0 to reduce the k value for the stabilized subgrade from 350 pci to 110 pci.

Drainage. The expected level of drainage for a rigid pavement is incorporated into the performance equation through the use of a drainage coefficient, C_d. The coefficient will depend on the quality of drainage and the percent of time during the year the pavement structure would normally be exposed to moisture levels approaching saturation. Drainage coefficients ranging from 1.25 (excellent drainage, less than 1% exposure) to 0.70 (very poor drainage, greater than 25% exposure) are given in the AASHTO Design Guide. A C_d value of 1.2 is recommended by the May 15, 2002, Section 7 *Paving* requirements of the HCPID-ED and was used in our analysis.

Load Transfer. The load transfer coefficient, J, is a factor used in rigid pavement design to account for the ability of a concrete pavement structure to distribute load across discontinuities such as cracks or joints. The recommended values for J in the AASHTO Design Guide, for different conditions developed from experience and mechanistic stress analysis, range from 2.3 to 4.4. A load transfer coefficient of 3.2 is specified in the May 15, 2002, Section 7 *Paving* requirements of the HCPID-ED and was used.

5.3.2 Rigid Pavement Thickness

Based on the discussed design parameters and using the equation presented in AASHTO 1993, Figure 3.7, page II-46, the following design pavement thicknesses were computed for a stabilized subgrade and a concrete flexural strength (modulus of rupture) of 570 psi.

Table 5-2
Rigid Pavement Thickness

18-kip ESAL Applications	Rigid Pavement Thickness (in.) (5.5 Sacks of Cement Per Cubic Yard of Concrete)			Subgrade
	7-Day Flexural Break Strength by the Third Point Loading Method	28-day Compressive Strength	Concrete Modulus of Rupture, S'c	
	500 psi [1]	3000 psi	570 psi	
10 million	10.0			6 in. Lime-Fly Ash Stabilized

[1] The May 2002 Regulations, Section 7, Paving of the HCPID-AED.

5.3.3 Rigid Pavement Reinforcement

Reinforcing steel should consist of reinforcing bars running in both directions. The amount of reinforcing steel may be determined from the following equation:

$$A_s = WFL/2f_s$$

Where:

A_s = required steel area per ft of width
W = weight of the slab (psf)
F = coefficient of resistance between slab and subgrade (generally assumed to be 1.8)
f_s = allowable tensile stress in the steel (psi)
L = length of slab (ft)

The above equation also applies to the design of transverse steel reinforcement in continuously reinforced concrete pavements. Longitudinal reinforcing steel requirements in continuously reinforced concrete pavements are presented in the AASHTO Design Guide.

Assuming that expansion joints are spaced 80-ft longitudinally and 25-ft transversally, allowable tensile stress in the steel f_s = 0.75 x 40000 psi = 30000 psi, and concrete pavement thickness is 10-in. the longitudinal steel reinforcement is calculated as follows:

$$A_s = \frac{(10\text{-in}/12\text{-in/ft}) \times 150 \text{ pcf} \times 1.8 \times 80\text{-ft}}{2 \times 30000 \text{ psi}} = 0.30 \text{ in}^2/\text{ft}$$

Using #5 deformed steel reinforcing bars [cross-sectional area = $\pi(5/8/2)^2$ = 0.307 in^2]the longitudinal steel spacing is equal to [0.307 in^2/(0.30 in^2/ft) x 12 in/ft] = 12.3 in.

Transverse steel reinforcement for non-continuously or continuous reinforced concrete pavements is calculated as follows:

$$A_s = \frac{(10\text{-in}/12\text{-in/ft}) \times 150 \text{ pcf} \times 1.8 \times 25\text{-ft}}{2 \times 30000 \text{ psi}} = 0.094 \text{ in}^2/\text{ft}$$

Using #5 deformed steel reinforcing bars the transverse steel spacing is equal to:

$$[0.307 \text{ in}^2/(0.094 \text{ in}^2/\text{ft}) \times 12 \text{ in/ft}] = 39.2\text{-in.}$$

Based on the above assumptions for expansion joint spacing, steel allowable tensile stress, and pavement thickness, the rigid pavement may be reinforced with #5 deformed steel reinforcing bars spaced at a maximum of 12-in. center to center longitudinally, and 36-in. center to center transversally.

The steel reinforcement recommendations provided in this report are based on the assumptions presented above for pavement thickness, joints spacing (longitudinally and transversally), and steel allowable tensile stress. If any of the assumptions used to calculate the steel reinforcement is modified then the steel reinforcement should be modified accordingly.

All longitudinal, transverse and construction joints should be keyed and doweled. As a minimum, dowels should consist of No. 5 bars, 18-in. long on 18-in. centers as specified in HCPID-AED Specification Item No. 360.

Dummy groove joints are recommended to control cracking in spans in excess of 30-ft in length. Dummy joints may be sawed to a depth equal to ¼ the thickness of the slab. The HCPID-ED May 15, 2002, *Regulations of Harris County, Texas For The Approval And Acceptance of Infrastructure*, should be consulted in planning the reinforcing and joint details for the pavement and should govern if found to conflict with the above recommendations.

5.4 Pavement Maintenance

It is essential to maintain the pavement to prevent infiltration of water into the subgrade soils. Allowing water into the subgrade will accelerate pavement failure and maintenance requirements. Periodic maintenance must be performed on pavement sections to seal any surface cracks and prevent infiltration of water.

6. CLOSURE

6.1 Limitations

This report has been prepared for the exclusive use of Lockwood, Andrews & Newnam, Inc. for specific application to the referenced project at the aforementioned location in Houston, Texas. Our report has been prepared in accordance with the generally accepted geotechnical engineering practice common to the local area. No other warranty, express or implied, is made.

The analyses and recommendations contained in this report are based on the data obtained from the referenced subsurface exploration. The borings indicated subsurface conditions only at the specific locations and time, and only to the depths penetrated. The borings do not necessarily reflect strata variations that may exist between boring locations. The validity of the recommendations is based in part on assumptions about the stratigraphy made by the Geotechnical Engineer. Such assumptions may be confirmed only during earthwork and pavement construction. Our recommendations presented in this report must be re-evaluated if subsurface conditions during construction are different from those described in this report.

If any changes in the nature, design, or location of the project are planned, the conclusions and recommendations contained in this report should not be considered valid unless the changes are reviewed, and the conclusions are modified or verified in writing by TWEI. TWEI is not responsible for any claims, damages, or liability associated with interpretation or reuse of the subsurface data or engineering analyses without the expressed written authorization of TWEI.

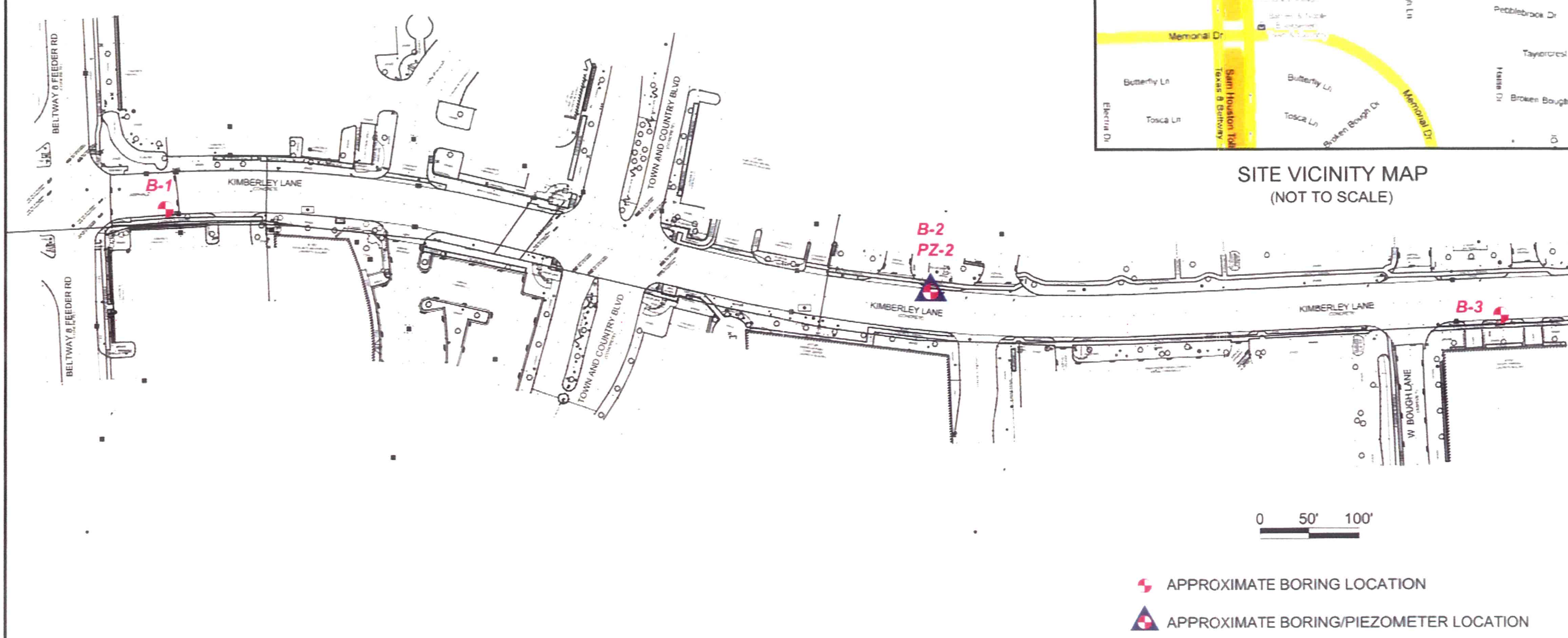
6.2 Design Review

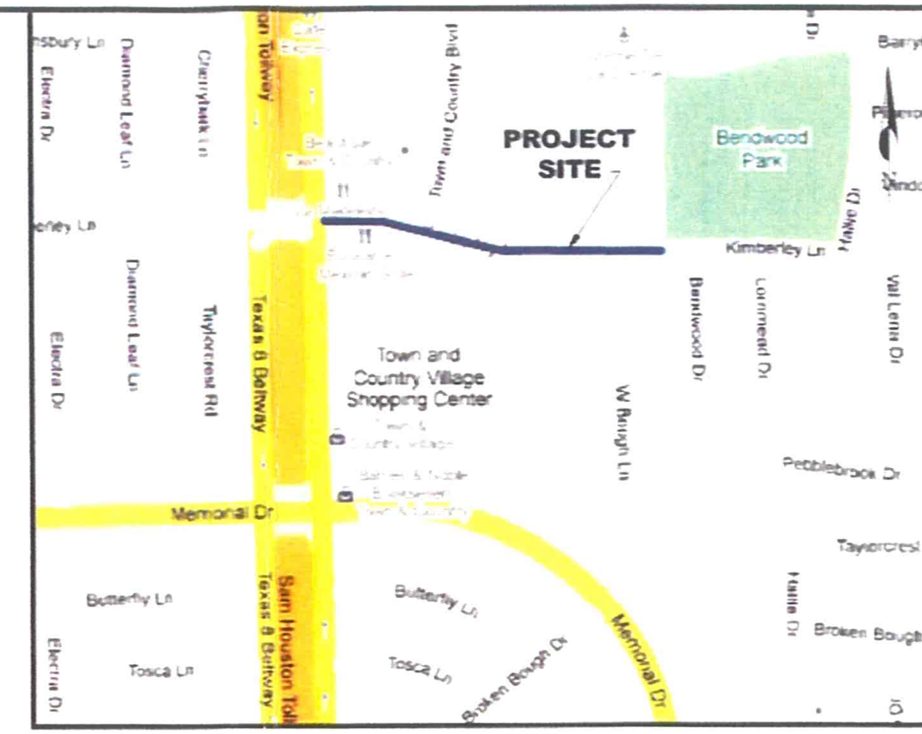
Review of the design and construction drawings as well as the specifications should be performed by TWEI before release. The review is aimed at determining if the geotechnical design and construction recommendations contained in this report have been properly interpreted. Design review is not within the authorized scope of work for this study. Should you elect to retain TWEI to perform a design review, additional fees would be applicable.

6.3 Construction Monitoring

Site work and pavement construction monitoring is recommended and has been assumed in preparing our report. These field services are required to check for changes in conditions that may result in modifications to our recommendations.

ILLUSTRATION





SITE VICINITY MAP
(NOT TO SCALE)

KIMBERLEY LANE IMPROVEMENTS PROJECT HOUSTON, TEXAS	Tolunay-Wong Engineers, Inc. Houston, Texas	PLAN OF BORINGS
LOCKWOOD, ANDREWS & NEWNAM, INC. HOUSTON, TEXAS		PROJECT NO.: 09.13.106 — FIGURE 1

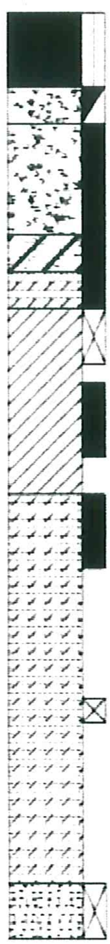
APPENDIX A
LOGS OF BORINGS

LOG OF BORING B-1

Project: Kimberley Lane Improvements Project
Houston, Texas
Client: Lockwood, Andrews & Newnam, Inc.
Houston, Texas

Project No.: 09.13.106
Date: 10.14.2009
Elevation:

Dry Augered: 0 **to** 25 **ft** **Free Water During Drilling at:** Dry (1)
Washed Bored: **to** **ft** **Water at:** **Caving at:**



ELEVATION DEPTH	POCKET PEN. (tsf) or SPT	DESCRIPTION	Wc (%)	Dens (pcf)	Qu or UU (tsf)	Str. (%)	LL	PI	Pass #200 (%)
0		2.5" Asphalt & 21.5" Concrete pavement							
		Gray CLAYEY SAND "FILL"	24						
	1.25	stabilized	17	111	1.49	10			
	4.00	Stiff gray LEAN CLAY "FILL" w/ sand seams & pockets, ferrous nodules	17	113					
5	2.50	-very stiff @ 4'-6' -gray & tan @ 5'-6'	17	112	2.01	7			
	1.75	Stiff gray & tan FAT CLAY w/ SAND (CH)	24				65	43	75
	2.50	w/ calcareous nodules	16						
	8/6" 14/6" 17/6"	Gray & tan CLAYEY SAND (SC)	14						
		Very stiff gray & tan LEAN CLAY (CL) w/ sand seams & ferrous nodules							
10	2.00	-stiff @ 10'-12'							
	4.50	Gray & tan CLAYEY SAND (SC) w/ sand seams	14						35
15									
	50/6" 50/2"	-very dense, w/ calcareous nodules @ 18.5'-20'	14				25	9	30
20									
	13/6" 15/6" 21/6"	Dense tan POORLY GRADED SAND w/ SILT (SP-SM)	14						12
25		Boring terminated @ 25 ft							
30									

Note(s): (1) Open borehole was grouted upon completion of sampling.

TOLUNAY-WONG ENGINEERS, INC.

LOG OF BORING B-2

Project: Kimberley Lane Improvements Project
Houston, Texas
Client: Lockwood, Andrews & Newnam, Inc.
Houston, Texas

Project No.: 09.13.106
Date: 10.14.2009
Elevation:

Dry Augered: 0 **to** 25 **ft** **Free Water During Drilling at:** 23 ft
Washed Bored: **to** **ft** **Water at:** (1) **Caving at:**

ELEVATION DEPTH	POCKET PEN (tsf) or SPT	DESCRIPTION	Wc (%)	Dens (pcf)	Qu or UU (tsf)	Str (%)	LL	PI	Pass #200 (%)
0		12" Concrete pavement							
		Gray CLAYEY SAND "FILL"	20						
	1.50	w/ gravel, stabilized	25				48	28	71
		Gray SANDY LEAN CLAY "FILL"	23						
	0.25	stabilized	27						
	1.00	Stiff gray & tan LEAN CLAY w/ SAND "FILL" stabilized	20				45	25	50
5	0.75	Very soft gray SANDY LEAN CLAY "FILL"	25						
	2.75	w/ gravel	19	111					
	1.75	-w/ lime @ 3'-4' -firm, tan & gray below 4'	26				68	45	86
	4.50	Stiff reddish brown & tan FAT CLAY (CH) w/ claystones & calcareous deposits -very stiff-hard @ 8'-10'	20						
10	1.50	Stiff light gray LEAN CLAY (CL) -w/ ferrous nodules @ 10'-12'	17	112					
	3.00	-very stiff, w/ sand seams & ferrous stains @ 13'-15'	15						
15									
	4.50	Very stiff-hard light gray FAT CLAY (CH) -w/ sand seams @ 18'-20'	15						
20									
	4.50	-reddish brown & light gray, w/ claystones @ 23'-25'	21	103					
25		Boring terminated @ 25 ft							
30									

Note(s): (1) Open borehole was converted to standpipe piezometer PZ-2 upon completion of sampling.

TOLUNAY-WONG ENGINEERS, INC.

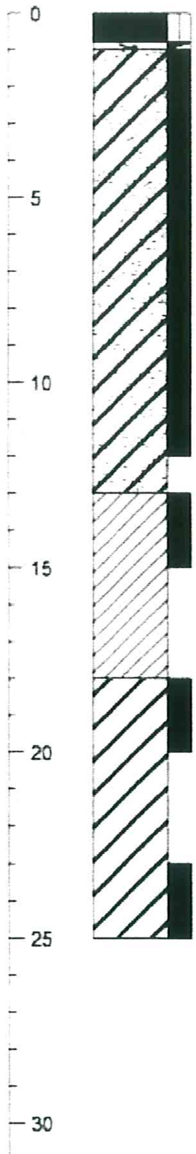
LOG OF BORING B-3

Project: Kimberley Lane Improvements Project
Houston, Texas
Client: Lockwood, Andrews & Newnam, Inc.
Houston, Texas

Project No.: 09.13.106
Date: 10.14.2009
Elevation:

Dry Augered: 0 **to** 25 **ft**
Washed Bored: **to** **ft**
Free Water During Drilling at: Dry (1)
Water at:
Caving at:

ELEVATION/ DEPTH — SOIL/SAMPLER SYMBOLS & FIELD DATA



POCKET PEN. (tsf) or SPT	DESCRIPTION	Wc (%)	Dens (pcf)	Qu or UU (tsf)	Str (%)	LL	PI	Pass #200 (%)
	9.5" Concrete pavement							
	Gray SILTY SAND "FILL" w/ gravel, stabilized	29 16						
1.50	Dark gray FAT CLAY w/ SAND (CH)	20				51	31	77
2.25	-stiff @ 2'-4' -w/ ferrous & calcareous nodules below 3'	20						
3.50	-very stiff @ 4'-6' -stiff, light gray, tan & red @ 6'-8'	17	114					
2.00		27	101	1.57	6			
1.25	-firm, red & light gray @ 8'-10'	26				64	42	81
3.75	-very stiff, light gray & tan @ 10'-12'	17	115					
4.25	Very stiff light gray & tan LEAN CLAY (CL) w/ sand seams & ferrous nodules	16						
4.50	Very stiff-hard light gray & reddish brown FAT CLAY (CH) -w/ sand pockets & calcareous nodules @ 18'-20'	19	107					
3.75	-very stiff, slickensided w/ calcareous deposits @ 23'-25'	22						
	Boring terminated @ 25 ft							

Note(s): (1) Open borehole was grouted upon completion of sampling.

TOLUNAY-WONG ENGINEERS, INC.

SYMBOLS AND TERMS USED ON BORING LOGS

Most Common Unified Soil Classifications System Symbols

Fill	Silt w/ Sand (ML)
Pavement	Well Graded Sand (SW)
Lean Clay (CL)	Well Graded Sand w/ Gravel (SW-GM)
Lean Clay w/ Sand (CL)	Poorly Graded Sand (SP)
Sandy Lean Clay (CL)	Poorly Graded Sand w/ Silt (SP-SM)
Fat Clay (CH)	Silt (ML)
Fat Clay w/ Sand (CH)	Elastic Silt (MH)
Sandy Fat Clay (CH)	Elastic Silt w/ Sand (MH-SP)
Silty Clay (CL)	Silty Gravel (GM)
Sandy Silty Clay (CL-ML)	Clayey Gravel (GC)
Silty Clayey Sand (SC-SM)	Well Graded Gravel (GW)
Clayey Sand (SC)	Well Graded Gravel w/ Sand (SP-GM)
Sandy Silt (ML)	Poorly Graded Gravel (GP)
Silty Sand (SM)	Peat

Sampler Symbols	Meaning
	Pavement core
	Thin - walled tube sample
	Standard Penetration Test (SPT)
	Auger sample
	Sampling attempt with no recovery
	TxDOT Cone Penetrometer Test

Field Test Data

2.50	Pocket penetrometer reading in tons per square foot
8/6"	Blow count per 6 - in. interval of the Standard Penetration Test
	Observed free water during drilling
	Observed static water level

Laboratory Test Data

Wc (%)	Moisture content in percent
Dens. (pcf)	Dry unit weight in pounds per cubic foot
Qu (tsf)	Unconfined compressive strength in tons per square foot
UU (tsf)	Compressive strength under confining pressure in tons per square foot
Str. (%)	Strain at failure in percent
LL	Liquid Limit in percent
PI	Plasticity Index
#200 (%)	Percent passing the No. 200 mesh sieve
( )	Confining pressure in pounds per square inch
*	Slickensided failure
**	Did not fail @ 15% strain

RELATIVE DENSITY OF COHESIONLESS & SEMI-COHESIONLESS SOILS

The following descriptive terms for relative density apply to cohesionless soils such as gravels, silty sands, and sands as well as semi-cohesive and semi-cohesionless soils such as sandy silts, and clayey sands.

Relative Density	Typical N_{60} Value Range*
Very Loose	0-4
Loose	5-10
Medium Dense	11-30
Dense	31-50
Very Dense	Over 50

* N_{60} is the number of blows from a 140-lb weight having a free fall of 30-in. required to penetrate the final 12-in. of an 18-in. sample interval, corrected for field procedure to an average energy ratio of 60% (Terzaghi, Peck, and Mesri, 1996).

CONSISTENCY OF COHESIVE SOILS

The following descriptive terms for consistency apply to cohesive soils such as clays, sandy clays, and silty clays.

Pocket Penetrometer (tsf)	Typical Compressive Strength (tsf)	Consistency	Typical SPT "N_{60}" Value Range**
pp < 0.50	qu < 0.25	Very soft	≤ 2
0.50 ≤ pp < 0.75	0.25 ≤ qu < 0.50	Soft	3-4
0.75 ≤ pp < 1.50	0.50 ≤ qu < 1.00	Firm	5-8
1.50 ≤ pp < 3.00	1.00 ≤ qu < 2.00	Stiff	9-15
3.00 ≤ pp < 4.50	2.00 ≤ qu < 4.00	Very Stiff	16-30
pp ≥ 4.50	qu ≥ 4.00	Hard	≥ 31

** An "N_{60}" value of 31 or greater corresponds to a hard consistency. The correlation of consistency with a typical SPT "N_{60}" value range is approximate.

Tolunay-Wong Engineers, Inc.

REVISION DATE 2-13-07

APPENDIX B
PIEZOMETER INSTALLATION REPORT

PIEZOMETER COMPLETION

Date: 10/14/09

Dry Augered: 0.0 ft to 25.0 ft

Wash Bored: to

Drilling Fluid: None

PIEZOMETER DEVELOPMENT

Date: 11/5/2009

Method: Air Lift

WATER LEVEL READINGS

Free Water at: 23.0 ft

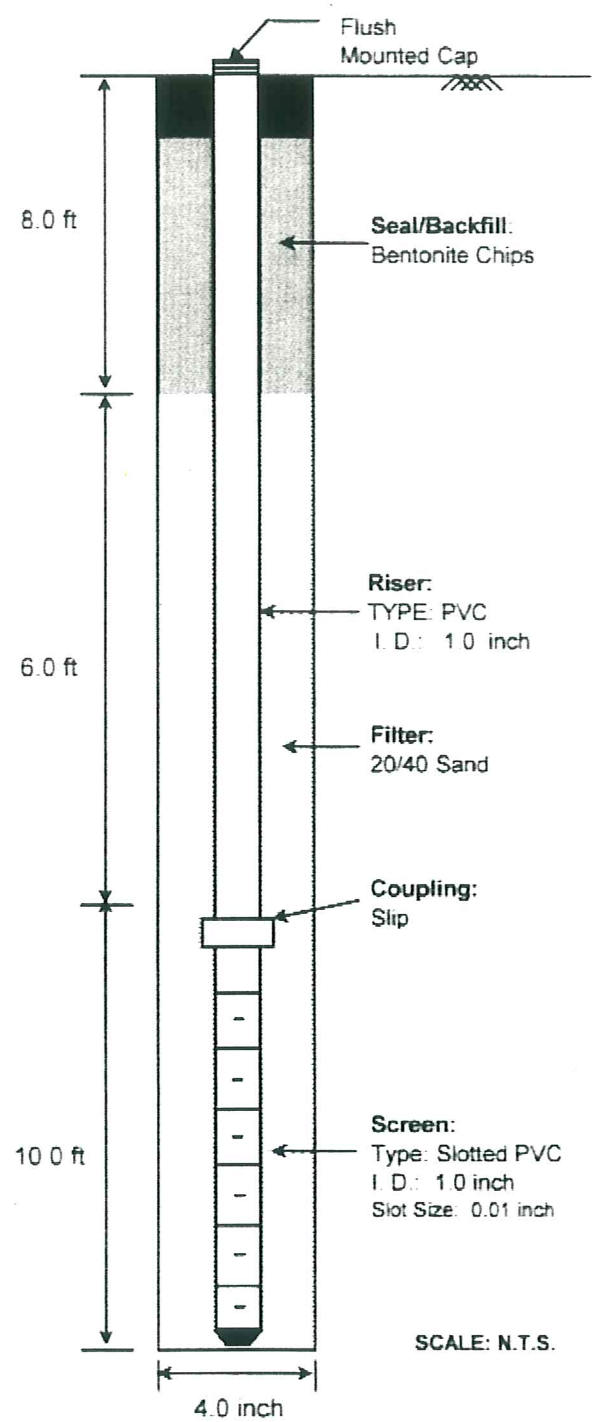
Date	Depth, ft*
10/16/09	8.5
11/05/09	9.3
12/01/09	9.5

*Depths are measured below existing grade

REMARKS

1. Standpipe piezometer installed in open borehole B-2.

Depth (ft)



Project: Kimberley Lane Improvements Project, Houston, Texas	Tolunay-Wong Engineers, Inc. Houston, Texas	**Project Number:** 09.13.106
Client: Lockwood, Andrews & Newnam, Inc. Houston, Texas	**Piezometer Installation Report:** PZ-2	**Figure**