GEOTECHNICAL STUDY
PROPOSED NORMANDY OAKS EXTENSION
NORD DU LAC DEVELOPMENT
COVINGTON, LOUISIANA

Report to:

ALL STATE FINANCIAL COMPANY
c/o WAINER BROTHERS
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ALL STATE FINANCIAL COMPANY
321 Veterans Boulevard, Suite 201
Metairie, Louisiana 70005

Attention: Mr. Bruce Wainer

Geotechnical Study
Proposed Normandy Oaks Extension, Nord du Lac Development
Covington, Louisiana

Mr. Wainer, this report is a design document that presents the results of our geotechnical study for this project. On September 19, 2010, you authorized this study and confirmed our involvement through signed acceptance of our proposal and contract. In general, we performed our services according to our proposal and contract dated August 31, 2010. During this study, we discussed our findings with you and Messrs. Franklin Kyle, P.E., and Scott Gros, P.E., from Kyle Associates, LLC (Kyle). On December 11, 2010, we submitted a draft copy of this report to you for your review, and on January 18, 2011, we received permission to finalize the report. This final report supersedes all previously furnished data and completes our involvement with this phase of the project.

Your Project

Our sources of information about this project include data provided by you, as well as conversations with and information from the project engineer, Mr. Gros. This geotechnical study focuses on undeveloped, wooded land in St. Tammany Parish, Louisiana. The site is on the east side of Louisiana Highway 21 (LA Hwy 21), north of Interstate 12 (I-12). Plate 1 shows a map of the site.

Your proposed project primarily comprises new roadways and a new bridge. The bridge will be located where the roadway crosses a pond on the southeast side of the subdivision. Mr. Gros indicated the primary roadway traffic would be light cars. He also provided us with the following estimated traffic types and volumes for the different roadway classifications planned for the project.

- through route: 350 vehicles per day with 5 percent heavy vehicles
- semi-arterials: 200 vehicles per day with 2 percent heavy vehicles
- cul-de-sacs: 100 vehicles per day with 2 percent heavy vehicles

According to Mr. Gros, the bridge loads are based on HS-20 44 loading, which results in a 120-ton design load for each of the two abutments. He anticipates using tapered-timber piles to support the abutments.

The survey data provided by Kyle indicates existing site grades at the boring locations range between about El 12.9 and 22.3, although in the roadway areas, they range from about El 15.2 to 22.3. In this report, all elevations are in feet and they are referenced to North American Vertical Datum 1988 (NAVD 88). Based on information from Mr. Gros, the roadway elevation will generally follow the existing topography. Therefore, only minor grade changes of about 7 in. or less are planned for most areas. An exception occurs in the area near the proposed bridge where up to about 18 in. of fill may be required. For various site-related reasons, it is possible that more extensive cutting and/or filling activities may be required in some areas to obtain the design grades.

Other civil engineering project features include roadway drainage, utilities, and retention ponds. We understand that subsurface drainage will be provided for the roadways. The large retention ponds will be excavated with side slopes of 1-vertical on 5-horizontal (1-V:5-H) to about 13-ft depth with the bottom 5 ft remaining wet.

Finally, two shallow soil borings were located in previously filled areas that are intended to be residential lots. This information was provided to us after the field and laboratory testing programs were designed and completed. Also, no building or structural details were provided to us. As such, only very general information is presented in this report about conditions at those locations. The information, while useful for assessing some aspects of the fill placement and the shallow site soils, is not suitable for and is not intended to be used for designing foundations for structures.

Purposes and Scope

One purpose of our study was to explore existing soil and groundwater conditions. Another purpose was to develop geotechnical recommendations to guide design and construction of your proposed project. We accomplished these purposes by doing the following:

- drilling ten undisturbed-sample borings to determine soil stratigraphy and to obtain samples for laboratory testing
- having laboratory tests performed on selected samples to evaluate relevant soil properties
- analyzing the field and laboratory data to develop the geotechnical recommendations presented in this report

Report Summary

We conducted a geotechnical study specifically for the Proposed Normandy Oaks Extension, Nord du Lac Development project in Covington, Louisiana. Based on our interpretation of the data collected for this study and our various analyses, we have summarized our findings and conclusions below. Please recognize that we have prepared this summary solely to provide a general overview. Do not rely on this summary for any purpose except that for which it was prepared. Rely **only** on the **full** report for information about findings, recommendations, and other concerns.

- The field and laboratory data showed soil conditions vary somewhat across the site, with the primary variations occurring in the shallow soils. At most locations, the near-surface soils are clayey silts to silty clays that in some areas of the site were "spongy." They also generally contained roots and organic matter.

- Two boring locations in areas that reportedly are residential lots had fill that was apparently placed without engineering control. Additional geotechnical and structural engineering efforts are needed to develop foundation design recommendations for these locations.
- In general, the borings revealed an all-clay profile. Several strata are low to moderate plasticity clays, although highly plastic clays are present, too.
- Considering the International Building Code (IBC) Site Class definitions and our experience in and knowledge of the area geology for the soils between the ground surface and 100-ft penetration, the project site is classified as Site Class D.
- The near-surface soils are moisture-sensitive materials. These soils can be relatively strong when they are dry. However, if they become wet, they can quickly lose their strength and load carrying ability. Also, if these soils become wet they can be slow to dry which can result in significant construction delays. Weak and/or wet surficial soils can make construction and compaction operations difficult. Therefore, for the shallow soil types encountered in this study, we believe that it is essential to:
  - Establish adequate drainage before site development begins.
  - Maintain site drainage throughout construction.
  - Incorporate proper drainage features into the site development plans.
- Site and subgrade preparation should consist of establishing site drainage and removing the vegetation and roots that are present to about 6- to 8- in. depth.
- Using the traffic criteria provided to us and discussed in this report, we have developed flexible pavement section recommendations for this site that include asphaltic concrete placed over an aggregate base that is underlain by cement-stabilized soil.
  - Through routes: 5 in. of asphaltic concrete over 4 in. of aggregate base over 8 in. of cement-stabilized subgrade.
  - Semi-arterials and cul-de-sacs: 4 in. of asphaltic concrete over 4 in. of aggregate base over 6 in. of cement-stabilized subgrade.
- Based on our data interpretation, we believe the planned driven timber pile foundations are suitable for supporting the proposed bridge. The text presents axial pile capacity recommendations for two timber pile sizes and various tip penetrations.
- The proposed side slopes of 1-V:5-H for the large retention ponds should provide adequate long-term stability, and no unusual problems are expected to achieve the planned excavation depth of 13 feet.
- As discussed in the report, some of the shallow on-site soils appear to be suitable for use as fill if they are cement-stabilized. Furthermore, the low to moderate plasticity clays present within certain depth intervals in the upper 10 ft may be used as structural fill. Finally, the highly plastic soils may be used as general fill. However, modification of their properties through the addition of the proper amount of lime would allow them to be used as structural fill.

The remainder of this report contains more detailed descriptions of our findings and recommendations. Therefore, the above summary must be used with the remainder of this report.

Report Format

The initial sections of this report contain details of the field and laboratory phases of our study. Following these sections, we present a description of the site and subsurface conditions. Next, we present our findings and recommendations about design and construction of your proposed project. The illustrations developed for this study follow the text and complete this report.

Limitations

This report is a design document that we have prepared for the specific project and location described here. We have conducted this study using the standard level of care and diligence normally practiced by recognized engineering firms now performing similar services under similar circumstances. Unless specifically stated otherwise, any environmental, biological pollutant (including any issues related to mold), or contaminant assessment efforts are beyond the scope of service on this project. We intend for this report, including all illustrations , to be used in its entirety. The following paragraphs describe other aspects about the use of this report.

Subsurface Conditions. This report describes our findings and conclusions about subsurface conditions at this location. We have based our interpretation of the soil and groundwater conditions on data obtained from the borings drilled for this study. Site exploration identifies subsurface conditions *only* at those points where subsurface tests are conducted or samples are taken. Although we have allowed for minor variations in subsurface conditions, our recommendations may not be appropriate for other soil conditions. Therefore, we recommend advising and retaining us if there are changes in the structure location, layout, loading conditions, usage, or site grading plans. We also recommend informing and retaining us if different soil conditions occur during construction. This will allow us to review our recommendations, assess the influence of the changes, and, if necessary, revise our conclusions.

Use of Data. We have prepared this report for the exclusive use of our Client (Allstate Financial Services Company) and the Civil Engineer (Kyle Associates, LLC). We developed our scope of service and prepared this report based on conversations and the exchange of information between Lourie Consultants, our Client, and the Civil Engineer. This information includes details about current site conditions, the proposed site development, and various site development issues. Because of the importance of these communications, third parties should not rely on the results of this study. Any third parties that do rely on this report do so at their own risk. All parties that use this report will be bound by all of the contract conditions and report limitations that exist between Lourie Consultants and its Client for this project. In addition, third parties are specifically excluded from becoming a third-party beneficiary to Lourie Consultants' contract with its Client. The purpose of this report is to evaluate the design of the project as it relates to our interpretation of the geotechnical aspects discussed here. This complete report should be available to potential contractors for information only and not as a warranty of subsurface conditions or a stand-alone construction specifications document. Contractors must recognize that the report was not prepared for purposes of bid development so they may need to conduct additional study to obtain the specific types of information they need or prefer.

Based on information from Mr. Gros, the roadway elevation will generally follow the existing topography, which in the proposed roadway locations, ranges from about El 15.2 to 22.3 ft NAVD 88. Consequently, only minor grade changes of about 7 in. or less are planned, although up to about 18 in. of fill may required in the area near the proposed bridge. Because of site conditions and/or topographic variations along the roadway alignment, it is possible that more extensive cutting and/or filling activities may be required in some areas to achieve the design grades.

Other civil engineering project features include roadway drainage, utilities, and retention ponds. Subsurface drainage will be provided for the new roadways. The large retention ponds will be excavated with side slopes of 1-vertical on 5-horizontal (1-V:5-H) to about 13-ft depth with the bottom 5 ft remaining wet.

Finally, two borings (B-8 and B-9) were located in previously filled areas that are intended to be residential lots. This information was provided to us after the field and laboratory testing programs were designed and completed. Also, no building or structural details were provided to us. As such, only very general information is presented in this report about conditions at those locations. The information, while useful for assessing some aspects of the fill placement and the shallow site soils, is not suitable for and is not intended to be used for designing foundations for structures.

IBC Site Class. The currently applicable version of the International Building Code (IBC) requires the site to be classified and assigned to a site class defined in Table 1615.1.1 of the IBC. In general, the IBC requires using site-specific data about the soils to 100-ft depth to make the classification. However, the IBC recognizes that site-specific data to 100-ft depth may not be available or known in sufficient detail on every project. Therefore, the IBC permits the registered design professional preparing the soils report to estimate appropriate soil properties based on known geologic conditions. Furthermore, the IBC notes that Site Class D is to be used unless Site Class E or F soil is likely to be present at the site. Based on the IBC's Site Class definitions and our experience in and knowledge of the area geology for the soils between the ground surface and 100-ft penetration, we have classified the project site as Site Class D. We believe this is the appropriate Site Class because the site soils do not meet all of the criteria for Site Class E or F.

Pavement Design Assumptions and Recommendations. We understand a flexible (asphaltic concrete, AC) pavement system is the preferred option for the proposed roadways. We further understand that on other similar roadway projects in St. Tammany Parish, you have constructed the pavements over cement-stabilized soils. One reason for preferring this approach is that it allows you to construct the pavement system in two phases. In the first phase, the required site and subgrade preparation occurs, the base is constructed, and then several inches of AC are placed. At this stage, the pavement system provides access to the site for construction activities. The second phase of pavement construction occurs when site construction is completed. It consists of repairing damaged areas if they exist, followed by the placement of another layer of AC to obtain the total required AC thickness for the design traffic conditions. We understand you have used this approach successfully elsewhere and expect it be successful for this project.

To allow us to perform the pavement design following the AASHTO[1] method, we had to convert the furnished traffic criteria from Kyle to total equivalent 18-kip single-axle load applications (ESALs). Presented below are Kyle's criteria and the equivalent ESALs we computed for flexible pavement systems with a 20-yr design life.

- through route: 350 vehicles per day with 5 percent heavy vehicles; this is equivalent to about 263,000 ESALs, which is characteristic of a roadway subject to medium to medium-heavy traffic with some loaded trucks
- semi-arterials: 200 vehicles per day with 2 percent heavy vehicles; this is equivalent to about 60,000 ESALs, which is characteristic of a roadway subject to subject to light to medium traffic with few loaded trucks
- cul-de-sacs: 100 vehicles per day with 2 percent heavy vehicles; this is equivalent to about 30,000 ESALs, which is characteristic of a roadway subject to light traffic with very few loaded trucks

As expected, the computed ESAL applications are quite sensitive to truck traffic. Even a minor increase in truck traffic can result in significantly greater ESAL applications. Therefore, it is extremely important to prevent or minimize truck traffic on roadways that exceed the criteria used for design.

The surface and near-surface soils of Stratum Ib were present at all boring locations. Stratum Ib comprises clayey silts to silty clays with various amounts of organic matter. At most boring locations, laboratory tests showed these soils to be desiccated, although some areas of the site were somewhat "spongy" based on field observations. The "spongy" soil behavior is often indicative of soft, wet, and weak soils. The Stratum Ib soils are considered to be moisture-sensitive materials that can be relatively strong when they are dry. However, if they become wet, they can quickly lose their strength and load carrying ability. As such, in an untreated state, they can be expected to have a soaked CBR of 3 or less and will provide poor support for pavement structures. However, the Stratum Ib soils may be improved and stabilized with the addition of the proper amount of cement after site stripping and the removal of organics and other deleterious material occurs. In cases where filling is required, the selected fill should be stabilized with cement, too.

For the pavement design presented below, we have assumed the soil-cement base will have a 7-day unconfined compressive strength of at least 250 psi. Based on the laboratory test results and Louisiana Department of Transportation and Development (DOTD)[2] guidelines, we estimate about 10 to 11 percent cement by volume is required for the Stratum Ib soils to achieve this strength value. DOTD guidelines presented in TR 432-82[3] indicate the 7-day unconfined compressive strength should be at least 250 psi for these soils when they are treated with the quantity of cement recommended here. A laboratory-testing program should be conducted to determine the optimum amount of cement before finalizing the pavement design.

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(1) *AASHTO Guide for Design of Pavement Structures 1993*, American Association of State Highway and Transportation Officials, Washington, D.C.

(2) *Louisiana Department of Highways Testing Procedures Manual*, Department of Transportation and Development, Office of Highways, Baton Rouge, Louisiana.

(3) *Testing Procedures Manual, Vol 2*, Department of Transportation and Development, Office of Highways, Baton Rouge, Louisiana.

Table 3 (Continued)
Summary of Test Results
Proposed Normandy Oaks Extension, Nord du Lac Development
Covington, Louisiana

Stratum	Measure	Plasticity and Classification Properties, %				Strength or Consistency Properties		
		Wc	LL	PI	LI	Su (U), ksf	Su (Q), ksf	N, bpf
IIb	No. of Tests:	8	6	6	6	1	0	0
	Range:	19 to 31	50 to 70	20 to 51	-0.9 to 0.2	N/A	N/A	N/A
	Avg:	23	59	37	0	2.55	N/A	N/A
III	No. of Tests:	1	1	1	1	1	1	0
	Range:	N/A	N/A	N/A	N/A	N/A	N/A	N/A
	Avg:	22	32	10	0.1	0.75	0.92	N/A
IV	No. of Tests:	3	2	2	2	0	3	0
	Range:	26 to 42	55 & 57	32 & 34	0.1 & 0.4	N/A	0.88 to 1.83	N/A
	Avg:	35	56	33	0.2	N/A	1.43	N/A

Note: N/A = Not Applicable; PI = Plasticity Index = (LL-PL); LI = Liquidity Index = (Wc-PL)/PI

Our field observations and laboratory test results show soil conditions vary somewhat across the site. Field observations and laboratory tests suggest the soils in Strata Ia and Ib are often desiccated. We believe Stratum Ib represents the site's original ground surface and it often contains roots and other organic material. The clays in Stratum IV often are often slickensided below about 18-ft depth. The individual boring logs contain values of dry unit weight for representative samples of the cohesive soils. Using these values and the corresponding moisture content values, we were able to compute total unit weights for use in our analyses.

Design Considerations, Analyses, and Recommendations

This section of the report presents our recommendations for this project. Where appropriate, we also discuss our methods of analysis and state key assumptions.

Site Development Plans. The subject site, which is undeveloped and wooded, is on the east side of Louisiana Highway 21 (LA Hwy 21), north of Interstate 12 (I-12) in St. Tammany Parish, Louisiana. The proposed roadway system extends the existing roadway network in the Nord du Lac development. Estimates of the traffic types and volumes for the different roadway classifications planned for the project that Mr. Gros provided are summarized below:

- through route: 350 vehicles per day with 5 percent heavy vehicles
- semi-arterials: 200 vehicles per day with 2 percent heavy vehicles
- cul-de-sacs: 100 vehicles per day with 2 percent heavy vehicles

Additionally, a bridge will be located where the roadway crosses the pond on the southeast side of the subdivision. Mr. Gros indicated that bridge loads are based on HS-20 44 loading, which results in a 120-ton design load for each of the two abutments. He anticipates using tapered-timber piles to support the abutments.

Table 2
Generalized Stratigraphy
Proposed Normandy Oaks Extension, Nord du Lac Development
Covington, Louisiana

Stratum Designation and (Boring Occurrence)	Approx. Depth, ft From	To	Generalized Description
Ia (B-8 and B-9)	0	2	Fill: Stiff sandy or silty clay, desiccated
Ib (All Borings)	0	2	Clayey silt to silty clay with roots and organic material; often desiccated
IIa (B-2, B-3, B-4, and B-6)	2	3.5	Soft to stiff low- to medium-plastic clay
IIb (All Borings Except B-3)	3.5	6.5	Firm to very stiff highly plastic clay
III (B-5, B-7, and B-10)	6.5	10	Firm low- to medium-plastic clay
IV (B-10)	10	50+	Stiff to very stiff highly plastic clay, often slickensided

The basis for cohesive soil stiffness descriptions is a combination of field and laboratory strength estimates and measurements. To obtain granular soil density descriptions, we used the driving resistance of the split-barrel sampler ('N' values in blows per foot).

Plasticity, Classification, and Strength Properties. Table 3 presents a summary of the plasticity, classification, and strength properties. We have compiled these values using the results of the field and laboratory testing performed for this study. The individual boring logs contain more detailed data about the properties at each boring location. The stratum numbers correspond to those presented above.

Table 3
Summary of Test Results
Proposed Normandy Oaks Extension, Nord du Lac Development
Covington, Louisiana

		Plasticity and Classification Properties, %				Strength or Consistency Properties		
Stratum	Measure	Wc	LL	PI	LI	Su (U), ksf	Su (Q), ksf	N, bpf
Ia	No. of Tests:	2	2	2	2	0	0	2
	Range:	10 & 11	27 & 32	12 & 13	-0.8 & -0.3	N/A	N/A	15 & 17
	Avg:	10	30	12	-0.5	N/A	N/A	16
Ib	No. of Tests:	9	9	9	9	0	0	7
	Range:	9 to 20	24 to 38	3 to 17	-5.5 to 0.1	N/A	N/A	4 to 19
	Avg:	16	30	8	-1.3	N/A	N/A	10
IIa	No. of Tests:	5	5	5	5	0	0	0
	Range:	14 to 29	35 to 47	14 to 30	-0.1 to 0.3	N/A	N/A	N/A
	Avg:	23	42	22	0.1	N/A	N/A	N/A

Note: N/A = Not Applicable; PI = Plasticity Index = (LL-PL); LI = Liquidity Index = (Wc-PL)/PI

Borehole Sealing. The drilling contractor's personnel sealed the boreholes according to applicable regulations. Several State agencies issue and administer these rules. The drilling personnel backfilled the shallow boreholes with soil cuttings, and they grouted the 50-ft-deep boring using a cement-bentonite grout mixture. The crew placed the grout from the bottom of the borehole up to the ground surface using the pump-down method.

Laboratory Testing Program

We directed the laboratory program toward evaluating necessary soil properties to guide geotechnical design and construction. Our primary concerns were determining the plasticity, classification, and strength of the site soils. Table 1 summarizes the testing program we designed for this project. Plates 2 through 11, the individual boring logs, contain the test results.

Table 1
Summary of Laboratory Testing Program
Proposed Normandy Oaks Extension, Nord du Lac Development
Covington, Louisiana

Type of Laboratory Test	Number of Tests
Liquid limit (LL) and plastic limit (PL) tests that collectively are known as Atterberg limits	25
Water contents (Wc) with visual classifications	28
Unconfined compression (U) tests with unit dry weights	2
Unconsolidated-undrained triaxial compression (Q) tests with unit dry weights	3

Site Conditions

This section of the report describes the site conditions that existed at the time we performed this study. We have used the information provided to us, as well as the data collected in this study to develop the information presented here.

Surficial. The site is undeveloped, vegetated land that contains trees, shrubs, and grasses. Tree limbs, leaves, and other organic materials are present at the ground surface. At the time we drilled the borings, the site was generally dry, although some "spongy" areas were present. At the boring locations, approximate ground surface elevations ranged from El 12.9 to 22.3 ft NAVD 88.

Stratigraphic Model. Table 2 presents the generalized stratigraphy for this site. We have based this model on our interpretation of the field and laboratory data. Unless noted otherwise, the strata lines shown on the boring logs and the depths shown in Table 2 are approximate boundaries of the various strata. The actual transition between materials may be gradual or may occur between recovered samples. The stratification described here and shown on the boring logs is for use in our analyses, and should not be used as the basis of design or construction cost estimates without realizing that variations are possible.

Field Exploration Program

On September 21, 2010, we explored subsurface soil conditions by drilling 10 undisturbed-sample borings, B-1 through B-10. Plate 1 shows the approximate boring locations. Kyle selected the boring locations, provided us with a site plan that showed the boring locations, and assisted the field crew in locating the borings on the site. The field program consisted of the following:

- Roadways – seven borings, B-1 through B-7, each advanced to 8-ft depth
- Previously Filled Area – two borings, B-8 and B-9, each advanced to 8-ft depth
- Bridge Location – one boring, B-10, advanced to 50-ft depth

Because of the wooded site conditions and the potential for wet, soft soils to be present, the drilling contractor used a buggy-mounted drilling rig with rice-and-cane tires to advance the borings. Plates 2 through 11 present detailed descriptions of the soils found in the borings. In general, the boring logs represent conditions that we observed at the time of exploration; however, we have edited them based on the results of the laboratory test data as appropriate. Plate 12 presents a key identifying most of the terms and symbols used on the boring logs. The individual boring logs, Plates 2 through 11, also show the approximate ground surface elevation and the coordinates at each boring location. Kyle provided us with these data on November 2 and 4, 2010.

Sampling Method. At the shallow boring locations, B-1 through B-9, the driller advanced the borings using dry-auger drilling methods while at the deeper boring location, B-10, the driller used a combination of dry-auger and wet-rotary drilling techniques. The crew sampled the soils continuously at about 2-ft intervals to 10-ft depth and at about 5-ft intervals below that to the completion depth. They obtained undisturbed samples of cohesive soils by hydraulically pushing a 3-in.-diameter thin-walled tube sampler into the soil. They usually sampled granular soils using a 2-in.-diameter split-barrel sampler driven with a 140-lb hammer falling about 30 inches.

Sample Handling. After recovery, the geotechnical logger removed the soil samples from the samplers in the field. The logger examined the samples, visually classified them, and preserved representative portions of each sample for laboratory testing. The logger also obtained strength estimates of most cohesive samples using a calibrated hand penetrometer.

Groundwater Conditions. As noted above, the drilling contractor used a combination of drilling techniques to advance the borings to their completion depths. During drilling operations for the 8-ft-deep borings, no water was encountered; however, in the 50-ft-deep boring, we observed water at a depth of about 16 ft below existing grade during dry augering and before converting to wet-rotary drilling methods. After completing the borings, we allowed the boreholes to remain open for up to about 15 minutes. The 8-ft-deep boreholes were dry, while the measured depth to water in the 50-ft-deep boring was about 1.5 feet. The individual boring logs contain more details about our water level observations. These water level observations occurred during drilling operations and within a relatively short time after completing the borings. Therefore, stable water level readings may require additional time. Groundwater levels can fluctuate with seasonal variations in rainfall, surficial runoff, drainage improvements, nearby construction activities, water levels in nearby waterways, and other factors. To obtain information about long-term water levels and its influence on construction operations, the depth to groundwater should be determined using piezometers or other suitable devices by the party responsible for construction before that party begins work.

Although cement-stabilized soils provide good support, they tend to develop cracks as the mixture cures, and when AC pavements are placed directly on soil-cement stabilized soils, reflective cracking can occur in the AC. Therefore, to reduce the potential for reflective cracking to occur in the AC pavement, we recommend using an aggregate layer between the AC and the soil-cement base. Our recommendations for the various pavement system components are presented in later sections of this report.

We have used the soil properties determined in this study, the AASHTO method, anticipated drainage conditions, and our experience to develop the pavement section recommendations for this site. These pavement sections should be evaluated, and the design refined if anticipated traffic volumes and loadings change or differ from those presented above. Table 4 presents our pavement section recommendations for this site.

Table 4
Recommended Flexible Pavement Sections
Proposed Normandy Oaks Extension, Nord du Lac Development
Covington, Louisiana

Pavement System Component	Required Minimum Thickness, in.		
	Through Route	**Semi Arterials**	**Cul-de-sacs**
Asphaltic Concrete	5	4	4
Aggregate Base	4	4	4
Cement-Stabilized Subgrade	8	6	6

Note: The cement-stabilized subgrade must be constructed on stable ground, and the cement-stabilized subgrade may consist of properly prepared Stratum I soils or other suitable soils mixed with an adequate amount of cement. Later sections of this report present recommendations for the aggregate base and the cement-stabilized soils.

Our flexible pavement design assumes a pavement coefficient based on a minimum Marshall stability of 1200 to 1400 pounds. We recommend using an asphaltic concrete mix that satisfies the material and mixture requirements outlined by the DOTD[(4)] for asphaltic concrete wearing course mixtures that are dense and contain a high percentage of voids filled with bitumen. Other asphaltic concrete mixtures may achieve the stability requirements outlined above; however, many wearing course mix designs optimize stability, which is typical for highway pavement applications. For less traveled pavements such as subdivision streets, the intent of our recommendations is to use a mix design that accounts for traffic conditions that are significantly different from highway traffic. Experience shows that parking lot pavements can achieve greater long-term durability when they are constructed using a dense mix and a mix design with a high percentage of voids filled with bitumen.

A roadway is an investment, as is a building or any other capital structure. Just as a building must be maintained to ensure its maximum utility, so must a roadway. Pavements generally tend to deteriorate very slowly during the first few years after construction and very rapidly when they are aged. Proper maintenance will serve to carefully protect and preserve the facility for the many years necessary to justify the initial substantial investment. Therefore, we recommend de-

(4) *Louisiana Standard Specifications for Roads and Bridges, 2006 Edition*, Department of Transportation and Development, Office of Highways, Baton Rouge, Louisiana.

veloping a comprehensive pavement maintenance program. The key to successful maintenance is careful planning and programming of the work to be done. Therefore, the program should include a regular and consistent schedule for inspecting and evaluating all paved surfaces with respect to surface condition, structural strength, and drainage. The next step is to select the appropriate maintenance (joint and crack sealing, surface treatment, patching, overlay, etc.) based on the results of the inspection and evaluation program. In general, there are three types of maintenance:

- **Preventive Maintenance:** Performed to improve or extend the functional life of a pavement. It is a strategy of surface treatments and operations intended to retard progressive failures and reduce the need for routine maintenance and service activities. Preventive maintenance is ***completing the right repair at the right time.***
- **Corrective Maintenance:** Performed after a deficiency occurs in the pavement, such as loss of friction, moderate to severe rutting, extensive cracking, etc. May also be referred to as 'reactive' maintenance. In all cases of pavement distress, it is important to determine the cause(s) of the distress. This will facilitate repairs that will both correct the defect and reduce the likelihood of its recurrence.
- **Emergency Maintenance:** Performed during an emergency situation, such as a blowout or severe pothole that needs repair immediately. This also describes temporary treatments designed to hold the surface together until more permanent repairs can be performed.

Although all types of maintenance are needed in a comprehensive pavement maintenance program, emphasizing preventive maintenance may prevent a pavement from requiring corrective maintenance. A preventive maintenance program is a systematic approach to using a series of preventive maintenance treatments over time. A one-time treatment will improve the quality of the pavement surface and extend the pavement life, but the true benefits of pavement maintenance are realized when there is a consistent schedule for performing the preventive maintenance. An effective pavement preservation program integrates many preventive maintenance strategies and rehabilitation treatments. The goal of such a program is to extend pavement life in a cost-effective and efficient way. Studies show that preventive maintenance is six to ten times more cost-effective than a "do nothing" maintenance strategy.

Site Grading and Vegetation Considerations. Adequate subgrade and pavement system grading and drainage are essential for acceptable pavement performance and to satisfy the design criteria. The frequent and/or or recurring presence of water in the subgrade materials can weaken them and lead to shortened pavement life, a premature need to perform maintenance, pavement distress, or swift pavement failure. Therefore, positive drainage should be provided away from the edges of the roadways, and ditches or other drainage features should direct and carry water well away from the roadways. Where trees and shrubs are desired close to a roadway for aesthetics or other reasons, they should be limited in type and size. Vegetation, especially large, fast-growing deciduous trees near flat work (roadways, sidewalks, etc.) can cause large soil-volume changes to occur because of variations in soil moisture content. Therefore, if vegetation will be present, we recommend selecting slow-growing types that have a relatively small mature size.

General Foundation Design Criteria. Foundation selection for any structure must satisfy two basic, independent criteria. First, the bearing pressure transmitted to the foundation soils should not exceed the allowable bearing pressure. This allowable bearing pressure includes an adequate factor of safety (FS) applied to the soil shear strength. Second, movements of the underlying soils during the operating life of the structure must be within tolerable limits. Movements include settlement due to consolidation of the foundation soils as well as those associated with shrinking/swelling (active) soils. In addition to these technical issues, other factors such as construction schedules, weather conditions, project economics, the owner's performance criteria, and future site development plans can influence final foundation selection.

Overview of Shrink/Swell Considerations. The laboratory test results presented above show that moderately to highly plastic clays are present at this site in the Stratum II and Stratum IV soils. For grade-supported structures and shallow foundations, the Stratum IIb soils are of greater concern than the Stratum IV soils. Moderately to highly plastic clays can shrink and swell (heave) due to changes in soil moisture content. This volume change behavior can cause cracking and movements to occur in building foundations, slabs-on-grade, flat work, and other lightly loaded structures, including most residences. The shrink/swell behavior of natural clay depends on several factors:

- mineralogical composition
- physical state of the soil
- environmental factors

In general, moderately to highly plastic clays typically contain those minerals that are associated with shrinking and swelling (active) clays. When active clays are in a moisture-deficient condition, they have an affinity for water, and they have the ability to experience large increases in moisture content before they reach an equilibrium condition. Increases in moisture content cause swell pressures to develop and expansion to occur. Conversely, if active clay has a high water content, it is susceptible to significant shrinkage caused by moisture loss. In addition to moisture content, soil density has a considerable influence on the shrink/swell behavior of active clays. When active clays have high densities, they are more prone to swell and develop higher swelling pressures than the same soil at a lower density. However, a high-density soil will not shrink as much as a lower density soil upon drying. Environmental factors include climatic conditions, type and amount of vegetation covering the site before and after construction and its proximity to the completed structure, depth to groundwater, surface drainage conditions, etc. The primary factor controlling the shrink/swell behavior of active clay is the moisture content of the soil. In general, **if the soil's *in-situ* moisture content remains constant, then volume changes due to shrinking and swelling will be limited.**

The amount of foundation movement associated with changing soil moisture conditions depends on many complex factors. Therefore, it is difficult to make an accurate estimation of the amount of movement that could occur. Considering the site conditions and the proposed development, we believe that soil swelling may be of more concern than soil shrinkage at this site. This volume change potential can be reduced if the:

- soil moisture contents remain constant
- subgrade soils are properly prepared and moisture conditioned
- footings and grade beams are placed sufficiently far below grade to act as a moisture barrier and to extend below the upper portions of the active zone where most of the total heave is likely to occur
- net footing contact pressures equal or exceed the swell pressure of the foundation soils
- floor slabs are placed over an adequate thickness of inert (non-expansive) clay fill that increases the surcharge on the underlying soils and isolates the floor slabs from the moderately to highly plastic clays; experience indicates 3 ft or more of inert clay fill should be adequate for the site conditions

Residential Foundation Considerations. In general, residential structures often are supported on shallow foundations when suitable soils are present at and immediately below the foundation elements (footings). Generally, they should be placed in strong, competent, and stable soils, and they should bear at least 3 ft below the lowest adjacent grade. In some cases, deep foundation systems such as straight-sided drilled shafts, drilled-and-underreamed footings, or driven piles are required or desired. The soils of Stratum I (the fill and the apparently natural site soils) do not appear suitable for foundation support, although the soils in Stratum II probably are suitable for foundation support.

In addition to building foundations, floor slabs also must be supported adequately to reduce the potential for movements to occur and cracks to develop. Frequently, floor slabs are supported on structural fill, which is selected, placed, compacted, and protected in accordance with applicable engineering guidelines and specifications. The field and laboratory activities performed for this project indicate that the fill at B-8 and B-9 may have been placed with little to no engineering controls. While the soil type appears suitable for use as structural fill, the moisture contents suggest the soil was placed and compacted without moisture-density control. Also, it appears the fill was placed with little to no stripping of the underlying soils. Consequently, the behavioral characteristics of the fill and the underlying soils are unknown; however, improperly placed and compacted soils tend to settle and perform poorly over time.

The comments presented here are not intended to be design recommendations and may not be used as such. We are providing them for general guidance only. Additional geotechnical engineering activities are required to confirm the opinions and assessments presented above. Furthermore, we recommend having a structural engineer work with the geotechnical engineer to develop site- and structure-specific foundation designs.

Bridge Foundation Recommendations. Based on the information developed during this study, we believe the proposed bridge may be supported on driven tapered, treated timber piles as planned. For driven piles, we recommend a tip penetration of at least 30 ft below the existing site grade. Timber piles should conform to the requirements of ASTM D 25, and preservative treatments should comply with all applicable requirements. We believe that all of the piles for a particular structure should terminate at the same depth and in the same soil stratum to reduce the potential for excessive differential settlement of the structure.

To estimate the ultimate axial compressive and tensile capacities for piles installed to various penetrations, we used the static method of analysis. In this method, the ultimate axial capacity is developed through two components, skin friction and end bearing. The ultimate capacity for a given penetration is taken as the sum of the skin friction on the embedded perimeter of the pile and the end bearing on the pile tip.

To compute axial pile capacity at this site, we used the field and laboratory test results to select soil parameters and develop a soil model. For this study, we computed axial pile capacity for two cases:

- Case 1 – timber piles with a 6-in.-diameter tip and an 8-in.-diameter butt
- Case 2 – timber piles with a 7-in.-diameter tip and a 12-in.-diameter butt

Table 5, shows the results of our computations for piles driven to various penetrations and assuming full pile embedment. However, a pile cutoff of 2 ft should have no significant influence on computed pile capacity.

Table 5
Allowable Pile Capacity Values
Proposed Normandy Oaks Extension, Nord du Lac Development
Covington, Louisiana

Tapered Timber Pile Description ASTM D 25	Penetration of Pile Tip Below Existing Grade, ft	Allowable Temporary Tension Capacity, kips	Allowable Sustained Compression Capacity, kips
Case 1A – 6-in.-diameter tip and 8-in.-diameter butt measured 3 ft from the butt end	30	27	28
Case 1B – 6-in.-diameter tip and 8-in.-diameter butt measured 3 ft from the butt end	40	37	39
Case 2A – 7-in.-diameter tip and 12-in.-diameter butt measured 3 ft from the butt end	30	36	37
Case 2B – 7-in.-diameter tip and 12-in.-diameter butt measured 3 ft from the butt end	40	50	52
Case 2C – 7-in.-diameter tip and 12-in.-diameter butt measured 3 ft from the butt end	50	65	67

The values presented in Table 5 are ***allowable*** values. To determine these values, we ***reduced*** our computed ***ultimate*** capacities by applying a factor of safety. For piles in sustained compression or transient (temporary) tension, we used a factor of safety of 2. To determine the allowable capacity values for piles subjected to sustained tension, we recommend using a factor of safety of 3; therefore, multiply the temporary tension values shown in Table 5 by 0.67. These factors of safety apply to the failure of the pile through the soil; the structural capacity of the pile also must be considered in selecting the allowable pile capacity for design.

We recommend using a clear center-to-center spacing between piles of at least three pile diameters (but not less than about 24 to 36 in.) so the piles can be considered individual, isolated pile foundations. We calculated the capacities shown in Table 5 by assuming the final grade at the site ***would not*** be raised more than about 2 feet. If more than 2 ft of fill will be required, we should be advised and retained to evaluate the expected influence on foundation design and performance. We also recommend that any fill placement occur before pile installation and as early during the site development program as practicable.

Pile Foundation Settlement. A detailed settlement analysis was beyond the scope of this study. Experience suggests that with the design loads, some movements are possible. However, we would expect these movements to occur as the dead loads are applied during construction. Therefore, long-term settlements due to consolidation of the foundation soils should be within tolerable limits for foundations designed and constructed according to our recommendations.

Retention Ponds. We understand retention ponds are planned for this project. We further understand the large ones will be excavated to about 13-ft depth with the bottom 5 ft remaining wet. The borings suggest the excavations should be made through an all-clay profile. For these soil conditions, we recommend that the side slopes of the proposed retention ponds be 1-V:3-H or flatter for stability purposes. For maintenance purposes and other reasons, flatter slopes may be desired. Therefore, the proposed side-slope geometry of 1-V:5-H for the large retention ponds satisfies our recommendations and should have adequate long-term stability.

Suitability of On-site Soils for Fill. Based on the laboratory test results and our observations, we believe the on-site soils from Stratum Ib may be used as fill if they are cement-stabilized as discussed elsewhere in this report. Furthermore, the low to moderate plasticity clays in Stratum IIa (about 2- to 3.5-ft depth) and in Stratum III (about 6.5- to 10-ft depth) are suitable for use as structural clay fill. Therefore, they may be used to raise the site grade and they may be placed directly below slabs and other structural elements. The higher plasticity clay soils in Stratum IIb (about 3.5- to 6.5-ft depth) and in Stratum IV (below about 10-ft depth) may be used as general fill for raising grade outside the area of present and future construction; however, they are too plastic to use for structural fill unless their properties are modified through the addition of lime. A later section of this report provides guidelines for both structural and general fill selection and placement. If on-site soils will be stockpiled for future use, we recommend keeping the stockpile neat, well drained, and in a workable condition at all times.

Construction Considerations

Earthwork and site preparation operations should be conducted in the presence of qualified geotechnical personnel. In general, for pile foundations driven to the recommended depths, no unusual problems are expected. Although the shallow borings were dry during the soil boring operations, experience suggests that some seepage of water into shallow excavations, including those for pile caps, can be anticipated, but we believe that it can probably be controlled using open pumping methods (ditches, sumps, and pumps). We recommend allowing the contractor to be responsible for the design of the systems to control and treat groundwater encountered in the required excavations. For certain-sized construction sites, an NPDES (National Pollutant Discharge Elimination System) permit must be obtained from the regulatory authority having jurisdiction over the site for stormwater discharges from the site.

Site Drainage and Surface Water Control. We believe it is essential for the contractor to establish and maintain adequate site drainage. This should reduce access problems and delays, as well as help other earthwork-related activities. All traffic should be minimized during extended periods of wet weather and for some time afterward.

Our on-site observations and our review of the site's existing topography suggest there are localized low areas that can pond water. Furthermore, the surface and near-surface soils of Stratum Ib are moisture-sensitive soils. These soils can be relatively strong when they are dry. However, if they become wet, they can quickly lose their strength and load carrying ability. Instability of the subgrade will be evidenced by sponginess under the passage of light equipment or even foot traffic. Also, if these soils become wet they can be slow to dry which can result in significant construction delays. Weak and/or wet surficial soils can make construction and compaction operations difficult. Therefore, for the shallow soil types encountered in this study, we believe that it is essential to:

- establish adequate drainage before site development begins
- maintain site drainage throughout construction
- incorporate proper drainage features into the site development plans

The intent of both the construction and permanent site drainage programs should be to prevent the accumulation of water on the site and reduce the potential for rainwater infiltration into the subgrade soils. Without a proper construction drainage program, the contractor should limit the extent of the exposed subgrade soils to reduce the potential for water damage to them. A typical construction drainage program could consist of a series of shallow interconnected ditches designed to remove water from the site. Permanent drainage measures can include topographic changes, surface drains, and underdrains.

During construction, we believe the contractor should be responsible for designing, constructing, operating, and maintaining ditches, check dams, dikes, sumps, culverts, and other appropriate measures that may be required to collect surface water from all sources (direct precipitation and flow from adjacent land). In addition, the contractor should be responsible for treating the collected water to reduce suspended solids and other constituents to the levels required by applicable regulations, and discharge the water as appropriate.

Site Clearing. The site is now wooded and some clearing will occur as part of the site development. Therefore, we recommend cleaning out the resulting stump holes and other depressions to remove organic material and water. The required clean-out depth will vary, but the base of the excavation should be in firm to stiff natural soil. The resulting excavations should be backfilled promptly with structural fill. We also recommend having the clearing contractor establish and maintain adequate site drainage during site clearing operations to reduce the potential for deterioration of the near-surface soils due to wet weather.

Site Stripping. The stripping operations within the proposed construction area should consist of removing all shrubs, weeds, vegetation, and other deleterious materials to an estimated depth of about 6 to 8 inches. However, since conditions can vary across a site, it is possible for other unsuitable materials to exist and/or be thicker in some areas. Therefore, it may be beneficial for the owner to include a contingency in both the construction schedule and budget to accommodate this risk. Unless the stripped materials will be used for landscaping purposes, the contractor should remove them from the site and dispose of them in accordance with applicable regulations.

After stripping the site, the earthwork contractor should proofroll the subgrade using a closely spaced rolling pattern. We suggest using a heavily loaded dump truck or other suitable equipment to detect weak or soft zones. The contractor should make at least two complete passes over the construction area. We recommend making the second pass perpendicular to the first pass. Weak or soft materials should be removed and replaced with structural fill. Proofrolling operations should be conducted in the presence of qualified geotechnical personnel. As discussed previously, we believe it is essential for the contractor to establish and maintain adequate site drainage. This should reduce access problems and delays, as well as help other earthwork-related activities. All traffic, including proofrolling, should be minimized during extended periods of wet weather and for some time afterward.

Site Preparation Considerations. To obtain the desired site grades and provide suitable support for the roadway pavement system, some cutting and filling will be required. However, before fill, slabs, or paving can be placed, effective site drainage must be established and site preparation must occur.

The subsurface exploration shows the Stratum Ib soils (clayey silts, very silty clays, and silty clays) extend to about 2-ft depth below current site grade although in some areas they may be somewhat shallower or deeper. These moisture-sensitive soils have low plasticity properties and will absorb and mix with free water readily. When wet, they can quickly lose their strength and load carrying ability and will rapidly deteriorate further with even small increases in moisture content. Also, due to the high percentage of fines, this soil may not drain well by gravity. Therefore, if these soils become wet, they can be slow to dry which can result in significant construction delays. A stable working surface is necessary during construction for:

- moving equipment across the site during construction
- placing fill to obtain design grades
- supporting the design loads from building and pavement structures

In general, various approaches are available for site preparation at this location. In addition to technical issues, the appropriate approach needs to include the consideration of cost factors, weather conditions, and project scheduling requirements. Regardless of the approach, we recommend establishing proper site drainage and stripping the site to remove uncontrolled or debris fill, vegetation, organic material, and other deleterious materials. Previous sections of the report discuss our site drainage and stripping recommendations.

It should be noted that in-place cement-stabilization the Stratum Ib soils is a suitable approach during dry weather conditions; however, if the contractor attempts this during or after wet weather conditions, construction difficulties should be anticipated. Also, if weak, wet, and unstable soils are left in place, pavement performance problems may occur. If these conditions are present, then other methods, including the complete removal and replacement of the problematic soils, may be required.

Areas Containing Unstable or Other Unsuitable Soils. During construction, if locations within the construction area are found to contain wet, weak, unstable, or other types of unsuitable soils, their lateral and vertical extent should be determined and we should be advised. Depending on the nature and extent of the unsuitable material, as well as the consideration of other factors, various options are available. Some of these options include:

- excavating and backfilling with structural fill that is selected, placed, and compacted according to the recommendations presented in this report
- scarifying the area to the required depth, moisture conditioning the material, and recompacting the soils to the required density
- treating the materials chemically with admixtures such as lime, cement, flyash, lime-flyash, or a combination of these admixtures to dry the soils and improve their support characteristics

In some cases, the use of a woven polypropylene geotextile (Amoco 2006, Mirafi 600X, Tenax T–3005, or an equivalent reinforcement geotextile) may be an expedient alternative for addressing wet, weak, or unstable subgrade soils during construction. Extremely weak and/or submerged soils are likely to require stronger reinforcing geotextiles than those recommended here; they also may require the use of geogrids or more extensive modification, in which case other recommendations would be required.

In general, installation details for the geotextile should conform to the manufacturer's recommendations. Furthermore, the geotextile should be composed of synthetic fibers formed into a woven fabric. Fibers used in the manufacturing of the geotextile should be composed of at least 85 percent by weight polyolefins, polyesters, or polyamides. The geotextile should be free of defects or flaws that significantly affect its physical properties. The geotextile also should meet AASHTO M 288 requirements for Class 1 separation and stabilization applications and the requirements of Table 6.

Table 6
Physical Requirements for Woven Reinforcing Geotextiles
Proposed Normandy Oaks Extension, Nord du Lac Development
Covington, Louisiana

Property	Units	Value	Test Method
Machine Grab Tensile Strength	lb	315	ASTM D 4632
Cross-Machine Grab Tensile Strength	lb	315	ASTM D 4632
Machine Grab Tensile Elongation	%	12	ASTM D 4632

Please see the notes that follow the table.

Table 6 (Continued)
Physical Requirements for Woven Reinforcing Geotextiles
Proposed Normandy Oaks Extension, Nord du Lac Development
Covington, Louisiana

Property	Units	Value	Test Method
Cross-Machine Grab Tensile Elongation	%	12	ASTM D 4632
Mullen Burst	psi	500	ASTM D 3786
Puncture	lb	110	ASTM D 4833
Trapezoid Tear	lb	110	ASTM D 4533
UV Resistance (at 500 hr)	%	70	ASTM D 4355
Apparent Opening Size	US Sieve	40 (maximum)	ASTM D 4757
Permittivity	1/sec	0.05	ASTM D 4491
Flow Rate	gal./min./sf	4	ASTM D 4491

Notes for Table 6:

1. Conformance of geotextiles to specification property requirements shall be labeled and determined according to ASTM D 4873 and ASTM D 4759.

2. Contracting entity may require a letter from the manufacturer certifying that its geotextile meets specification requirements.

3. All numerical values represent MARV or minimum average roll values (*i.e.*, average of test results from any sampled roll in a lot shall exceed the average roll values in the table) in weaker principal direction. Lot sampled according to applicable ASTM procedures.

4. Grab tensile elongation shall not exceed a MARV of 35 percent at failure.

5. Geotextile permittivity must be greater than the specified minimum value and result in a geotextile permeability that is greater than the permeability of the subgrade soil.

Trench and Excavation Safety. In 1989(5), the US Department of Labor's Office of Occupational Safety and Health Administration (OSHA) amended its "Construction Standards for Excavations, 29 CFR, Part 1926, Subpart P" to better ensure the safety of workers entering trenches or excavations. This federal regulation mandates constructing all excavations (utility trenches, footing excavations, etc.) according to these guidelines. Failure to comply with these regulations can endanger workers and result in substantial penalties for the owner and contractor. The contractor is solely responsible for designing and constructing stable, temporary excavations and should shore, slope, or bench the sides of the excavations as required to maintain the stability of both the excavation sides and bottom. As part of the contractor's safety procedures, the contractor's *competent person*, as defined in 29 CFR Part 1926, should evaluate the soil exposed in the excavations to identify existing and predictable hazards in the surroundings or working conditions. That person also should take prompt corrective measures to eliminate the hazards. In no case should any slope

(5) Occupational Safety and Health Administration, Department of Labor, (1989), *Occupational Safety and Health Standards–Excavations (Final Rule)*, 29 CFR, Part 1926, Federal Register, Vol 54 No. 209, pp 45894-45958.

height, slope inclination, or excavation depth exceed those specified in local, state, and/or federal safety regulations.

Pile Cap Excavations. The side slopes of shallow, temporary excavations in clay soils will probably stand near vertical for a limited time. We recommend, however, that vertical-sided excavations in clay be limited to a depth of about 4 feet. Excavations in or through fill will probably need to be sloped back to 1-vertical on 3-horizontal. Sides of temporary excavations deeper than about 4 ft should be braced or sloped back to at least 1-vertical on 1-horizontal. Bracing requirements for excavations deeper than 4 ft should conform to applicable federal, state, and local regulations. We recommend establishing positive drainage away from excavations to avoid surface water ponding within excavations and around the completed pile caps.

Pile Installation. Although this section of the report presents pile installation recommendations based on our interpretation of the site conditions, the pile-driving contractor must make an independent interpretation of the boring logs included with this report to determine installation requirements. Usually, the most economical pile installation procedure is by driving alone without resorting to supplemental procedures to aid pile driving. We have based our computed capacities on the assumption that piles will be driven to the desired penetration without supplemental drilling or jetting.

Foundation piles should be driven using a hammer of adequate size in as nearly as continuous operation as is practicable, without interruption, if possible. Pile driving hammers may be diesel, steam, or air operated. The use of a drop hammer, with a light ram and a large stroke, is discouraged since this type of hammer can produce very high and damaging stresses in the pile. The total energy delivered by the hammer is not the sole criterion for effective pile driving. In general, hammers with heavy rams and short strokes are often more effective in driving piles than hammers with light rams and long strokes. In addition, the proper hammer size and limiting blow count depends on the influence of the pile cushion material, the hammer operation, and other factors. We believe the design engineer should prepare detailed pile driving specifications in conjunction with the geotechnical consultant. In part, the specification should cover the requirements for furnishing and installing the piles including the scope of work, necessary submittals, piling details, equipment requirements, installation requirements and tolerances, capacity evaluation, and construction records. The specification should require the contractor to submit a complete package detailing the proposed piling equipment and installation procedures for acceptance before mobilization to the site. Presented below are general guidelines about driving energy and limiting blow counts.

Our experience suggests that timber piles with a tip diameter of 6 in. should be driven with a hammer delivering a driving energy of about 10,000 ft-lb per blow. Timber piles with a tip diameter of 7 in. should be driven with a hammer delivering a driving energy of about 10,000 to 15,000 ft-lb per blow. Timber piles with larger tip diameters should be driven with a hammer delivering a minimum driving energy of 15,000 ft-lb per blow. To reduce the potential for pile damage to occur, we suggest limiting blow counts on small-diameter timber piles to about 30 blows per foot. For large-diameter timber piles, we recommend limiting blow counts to about 40 blows per foot.

It is possible that unfavorable subsurface conditions and/or driving equipment problems can prevent piles from being driven to the desired penetration. The pile-driving contractor should make an independent interpretation of the boring logs and included with this report to determine installation requirements. When techniques other than driving are used to aid pile installation, conditions assumed in computations based on driving alone may not be met and computed capacities may need to be adjusted to fit installation conditions as nearly as possible. If supplemental installation techniques are anticipated or required, we should be retained to evaluate their influence on our computed capacities.

Piles should be examined carefully before driving and piles with structural defects should be rejected. During pile installation, erratic or unusually high or low blow counts may indicate a problem or a damaged pile. If this occurs, the questionable pile should be evaluated. We recommend surveying the production piles to detect possible vertical and/or horizontal movement (often called heave) that can result from soil displacement caused by driving nearby piles. Piles that heave after driving adjacent piles should be redriven to at least their original penetration and final driving resistance.

Pile driving and construction operations can transmit vibrations to nearby structures and underground utilities. Depending on the intensity and proximity, the vibrations may be sufficiently intense to annoy the recipients, or they may have detrimental effects on structures or equipment. Therefore, it is important to maintain the intensity of these transmitted vibrations at or below a level that can present problems or cause damage. It structures or utilities are in close proximity to the pile driving operations, we recommend being retained to help you devleop a vibration monitoring program.

We also recommend retaining qualified geotechnical personnel to observe, monitor, and keep records of the pile driving operations. They should promptly submit the data to us. This will allow us to maintain and review driving records, detect variations in pile installation if they occur, and assess the pile driving operations.

Indicator Piles and Pile Load Tests. If desired, a program of indicator piles and pile load testing can be performed to verify the predicted pile capacities presented in this report and provide information about pile installation and driveability. The results of a properly designed and conducted program can be used to:

- refine the design and define pile lengths more completely
- evaluate the suitability of the planned pile driving equipment and methods
- establish installation criteria

The indicator and test piles should be the same type and size that will be used for construction. These piles should be driven using the same methods, equipment, and techniques as proposed for the production (job) piles. In addition, qualified geotechnical personnel should maintain complete driving logs. During the indicator pile program, the pile showing the least driving resistance should be selected for load testing. After driving the reaction piles around the test pile, there should be a waiting period of at least 10 to 14 days between the driving of the reaction piles and the start of the load test. We recommend loading the selected pile to failure in general accordance with the "Quick Load" method outlined in ASTM D 1143, *Standard Test Method for Piles under Static Axial Compressive Load.*

Earthwork

This section of the report contains recommendations for fill selection, placement, and testing. To achieve a well-constructed system, we recommend having qualified geotechnical personnel onsite to observe and document earthwork-related activities. If a previously prepared surface loses the required stability, density, or finish due to construction traffic, wet weather, extended dry periods, or other causes, the earthwork contractor should remove or rework the material as necessary such that the resulting surface satisfies the intent of the recommendations presented in this report. When fill is required next to a wall or retaining structure, only lightweight construction equipment should be allowed within 5 ft of the wall. Furthermore, fill within 5 ft of walls should be placed in 6-in.-thick loose lifts and compacted with hand-operated compaction equipment.

General Site Access Considerations. We believe it is essential for the contractor to establish and maintain adequate site drainage. This should reduce access problems and delays, as well as help other earthwork-related activities. All construction traffic, including proofrolling, should be minimized during extended periods of wet weather and for some time afterward. Experience has shown that it often is desirable to establish an "all-weather" construction access road for contractor vehicles, material deliveries, etc.

Fill Materials and Stockpiles. For classification purposes, we have designated two classes of fill materials: (1) general fill and (2) structural or engineered fill as discussed below. During construction, it may be necessary to stockpile fill materials for future use on the project. When this is necessary, the stockpiles should be kept neat, well drained, and in a workable condition at all times. To protect a stockpiled material from the elements, it may be beneficial to cover the stockpile with a layer of impermeable plastic sheeting. In general, stockpiles should be kept about 15 ft or more from the edge of excavations, ditches, bayous, etc. If it becomes necessary to stockpile more than one type of material, we recommend clearly identifying each stockpile. To reduce the potential for soil fines to be flushed off a stockpile onto surrounding properties or into waterways, it may be necessary to surround the stockpile with a low earthen berm, a shallow swale, or a post-mounted filter fence.

General Fill. General fill for raising grade outside the area of present and future construction, may consist of locally available clays or granular soils. The earthwork contractor should place the fill in loose lifts that are 9 in. thick or less. The water content of the clays should be close to the optimum moisture content. We recommend compacting the fill to at least 90 percent of the standard Proctor (ASTM D 698) maximum dry density. Clean granular fill should be compacted to at least 70 percent relative density determined using appropriate test procedures (ASTM D 4253 and D 4254). Silty sand fill with no more than about 20 percent passing the No. 200 sieve can be compacted using the clay fill criteria above. Fill placed below or next to soil-supported structures should meet structural fill specifications.

Structural Fill. For fill that is required beneath or next to structures, slabs, or other load bearing units, we recommend using low plasticity clays. Due to the presence of moderately to highly plastic clays at shallow depth, we do not recommend using sand fill for this purpose because sand can provide a conduit for water to enter the subgrade soils. Structural fill should be free of roots, wood, debris, and other deleterious materials. In the flexible (AC) pavement areas, we recommend using a soil-cement subgrade and we recommend using an aggregate base to separate the asphaltic concrete from the soil-cement subgrade. Our recommendations for various types of structural fills are presented below.

Clay Fill. Clay fill should have a liquid limit of less than 40 percent and a plasticity index between 15 and 20 percent. Clay fill should be placed in 8-in.-thick loose lifts and each lift should be uniformly compacted to at least 95 percent of the standard Proctor maximum dry density. The moisture content should range from 1 percentage point dry to 3 percentage points wet of the optimum moisture content during compaction. If wet weather or extended dry periods deteriorate the surface whereby a good bond cannot be formed between successive layers, the earthwork contractor should prepare the surface as necessary. This preparation may include removing or scarifying the top 2 in. of the underlying material before placing the next lift. The moisture content of the fill should be maintained within 1 percentage point dry to 3 percentage points wet of the optimum moisture content until it is permanently covered.

Aggregate Base Course. The aggregate base course material should conform to Sections 302 and 305, and Subsections 1003.03(b) or 1003.03(c) of the DOTD(6) specifications. It should be placed in loose lifts up to about 8 in. thick and uniformly compacted to at least 98 percent of the standard Proctor maximum dry density. The moisture content should be within about 2 percentage points of the optimum moisture content during compaction and it should be maintained within 2 percentage points of the optimum moisture content until it is permanently covered. The suitability of the fill material should be verified during construction by periodic testing of the appropriate soil properties.

Soil-Cement Mixtures. Soil-cement mixtures should conform to Sections 302, 303, 305, and 1001 of the previously referenced DOTD specifications. In general, the soil-cement mixture should be spread, mixed, compacted, and maintained following the DOTD's guidelines. We recommend compacting the mixture to at least 95 percent of the maximum dry density obtained in the laboratory using the equivalent of the standard Proctor compaction effort. During compaction and curing, the moisture content should be maintained to allow hydration of the cement to occur. The moisture content of the cement-treated soil should be maintained within 1 percentage point wet to 3 percentage points wet of the optimum moisture content until the subgrade is permanently covered.

Utility Trench Fill. We recommend placing and compacting all utility trench backfill according to the guidelines presented above for structural fill. In addition, utility lines should be placed on suitable bedding material and adequate fill support should be provided beneath the pipe haunches to help prevent pipe distortion under load. Furthermore, the contractor should place the fill evenly around the sides of the pipe in thin lifts and then uniformly compact it to the specified degree of compaction using hand-operated compaction equipment. Inadequate fill placement and/or compaction can result in the distortion of utility lines and post-construction settlements.

Typically Used Compaction Equipment. A sheepsfoot roller is typically used for compacting clay fill. Vibratory rollers are typically well suited for compacting granular soils, including

(6) *Louisiana Standard Specifications for Roads and Bridges, 2006 Edition*, Department of Transportation and Development, Office of Highways, Baton Rouge, Louisiana.

aggregates. However, the use of vibratory rollers should be limited when the depth to water is shallow, as well as during and immediately after periods of wet weather. For soils containing admixtures (lime, cement, flyash, etc.), the subgrade should be sealed lightly with a pneumatic roller after mixing occurs. This process will help maintain moisture contents and protect the subgrade from damage caused by heavy rains. Sealing also can be effective on compacted clays and granular soils without admixtures for the same reasons. In trenches, near walls and other structures, and in other confined areas, hand-operated (vibratory plates for granular soils and impact rammers for clays) and/or light-duty compaction equipment should be used. Heavy equipment and over-compaction can cause increased lateral loads on walls.

Chemical Analysis of Proposed Fill. We recommend having the earthwork contractor submit a representative sample of all imported (offsite) fill to an analytical laboratory before approving it for use on the project. This is especially applicable to fill obtained from highly industrialized areas as well as fill material that has an unusual color or odor. A possible analytical program could consist initially of testing to determine total petroleum hydrocarbons (TPH) according to US EPA Method 1664 or 8015B. If necessary, additional testing can include metals or other constituents depending on the material's source and the initial test results.

Plan Review and Construction Monitoring

Once the project's other design professionals complete their design and before construction begins, we suggest that you retain us to perform a geotechnical review of their plans relative to their interpretation of this geotechnical report. Typically, our services include reviewing site preparation drawings as well as foundation plans and specifications. We also can be retained to participate in prebid and preconstruction conferences to reduce the likelihood of contractors misinterpreting the geotechnical report and our recommendations.

During construction, we believe that continuous surveillance by qualified geotechnical personnel retained by the Owner is essential and helps to obtain a well-constructed system. We recommend having geotechnical personnel present to monitor site preparation, observe foundation installation, and verify the suitability of the foundation soils.

Concluding Remarks

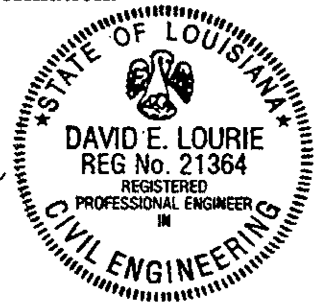
The following illustrations are attached and complete this report:

	Plate(s)
Plan of Borings	1
Log of Borings	2 thru 11
Terms and Symbols Used On Boring Logs	12

Mr. Wainer, we appreciate this opportunity to be of service to you and your civil engineers, Messrs. Kyle and Gros, on this project. We will call you and Mr. Gros in a few days to answer any questions you may have. In the meantime, please call us if you need additional information.

Sincerely,

LOURIE CONSULTANTS

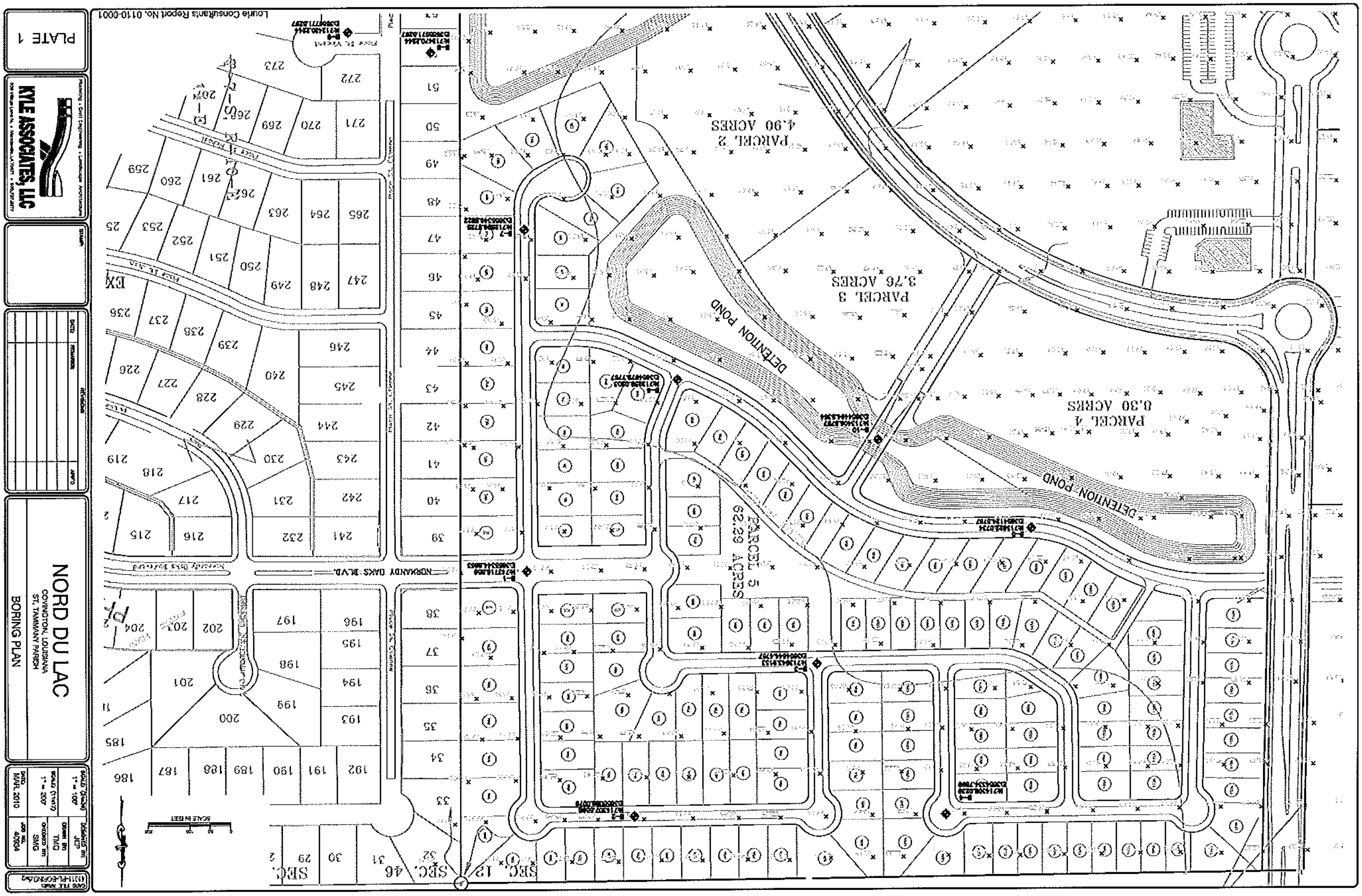


David E. Lourie, P.E., D.GE
Consulting Engineer

DEL/mbs (R10-0001.DOCM)
Copies Submitted:

Mr. Bruce Wainer, Allstate Financial Services Company, Metairie, LA, (1) E-mail and (2) US Mail
Mr. Scott Gros, P.E., Kyle Associates, LLC, Mandeville, LA, (1) E-mail and (1) US Mail

ILLUSTRATIONS



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