CROUCH ENGINEERING, P.C.

Geotechnical Report

Hitchin' Post Commercial Center
Spring Hill, Tennessee

Prepared for:

Harvey & Harvey Associates, LLC
5159 Columbia Pike
Spring Hill, TN 37174

CEPC Project No. 5092

Prepared by:

CROUCH ENGINEERING®, P.C.

428 Wilson Pike Circle, P. O. Box 1186, Brentwood, Tennessee 37024-1186
Phone: (615) 791-0630 Fax: (615) 791-8451
Website: www.crouchengineering.com

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APPENDICES

1.0 INTRODUCTION

Crouch Engineering, P.C. (CEPC) has completed the authorized geotechnical study for this project and submit the data and our conclusions below. The purposes of the geotechnical study were to explore the general subsurface conditions across the area of proposed construction and to provide comments and recommendations for site work, criteria for the design of foundations and slabs, and other issues that may have an impact on site development.

The Association of Engineering Firms Practicing in the Geosciences (ASFE) has prepared important information regarding studies of the type performed, and this is attached for your review.

Specifically excluded from the scope of study was any assessment related to environmental aspects of the property.

2.0 PROJECT DESCRIPTION

The development is not yet finalized, but will likely consist of single-story commercial or retail buildings fronting Highway 31, similar to those on adjacent properties to the north, and will potentially cover approximately 10± acres.

3.0 SITE DESCRIPTION

The site is located along Highway 31 in Spring Hill, Tennessee and is situated just north of the City of Spring Hill. The property is generally rectangular in shape, extending approximately 2,000 feet in a north-south manner, and about 400 feet in width. The property is bounded on the east by Highway 31 (Columbia Pike), on the west by McCutcheon Creek, on the north by the Early property, and terminates at a proposed County Road to the south. The location of the site is shown on a portion of the United States Geological Survey topographic map in Figure 1, and the street map in Figure 2, below. The site layout is shown on the Boring Location Plan in Appendix 1.

The area is generally a level, open field with a heavily wooded area along the western boundary that follows McCutcheon Creek. An access road transects the site in an east-west orientation near the middle of the property and serves a private residence.

Existing improvements within the property include fencing on the eastern property line along Highway 31, an existing, small one-story office building, and overhead power and lighting lines. Site Reconnaissance indicates past improvements include a septic field, concrete pad, propane gas tank, and small shed or spring house.

The higher portions of the property are generally along Highway 31 and the northeastern corner, where the elevation is 725± Mean Sea Level (MSL); the site transitions to topographically lower areas to the west toward McCutcheon Creek, to an elevation about 715± (MSL).

Based on observations at the site, there were no sinkholes or other depressional features observed.

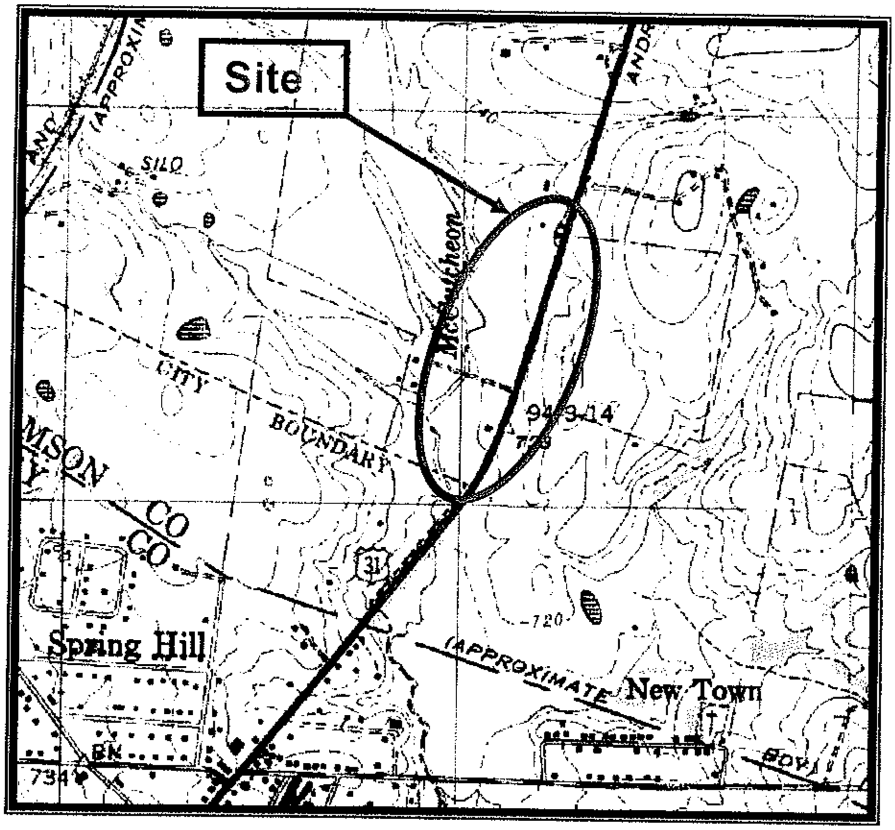


Figure 1. United States Geological Survey topographic map showing approximate site location; Spring Hill Quadrangle.

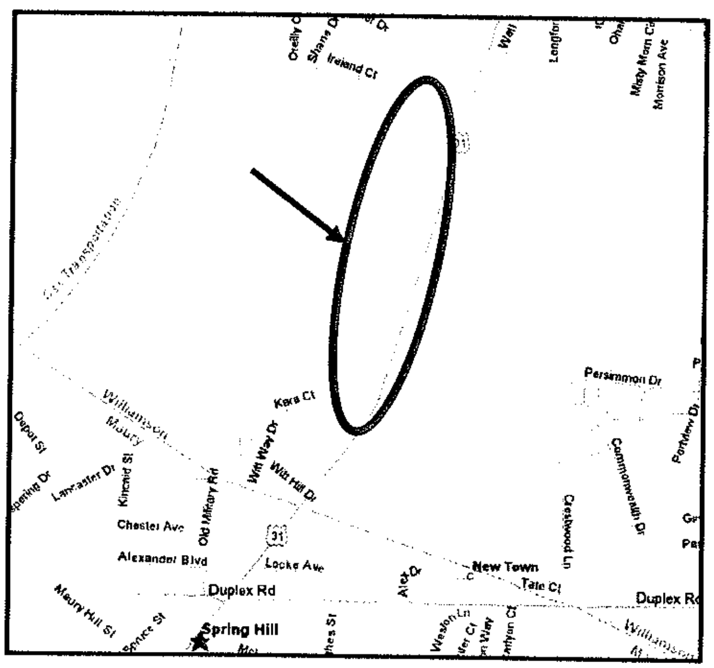


Figure 2. Street map showing approximate site location.

4.0 GEOLOGIC SETTING

Review of published geologic literature indicates the site is situated in the Heritage Formation of Ordovician age. The unit is described as laminated, argillaceous limestone; sandy, medium-gray to dark-gray, weathering to pale and darkish-yellow, very-fine grained, thin-bedded to laminated with thin shale partings; Thickness between 55 to 70 feet. (*Pre-Chattanooga Stratigraphy in Central Tennessee; Division of Geology; Charles W. Wilson, Jr.; 1990*).

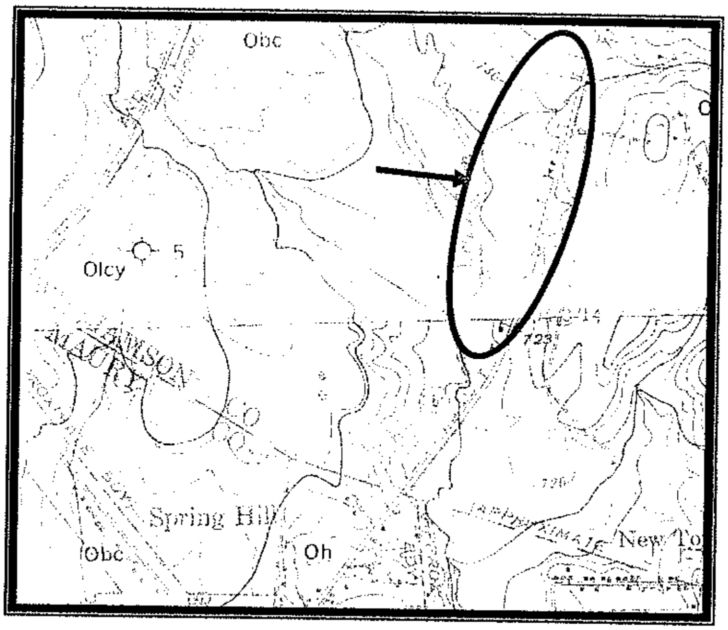


Figure 3. Spring Hill Geologic quadrangle showing site situated in the Heritage Formation of Ordovician Age.

5.0 SUBSURFACE EXPLORATION

5.1 General

The exploration included 11 soil test borings that were conducted using an Adverse-Terrain Vehicle (ATV) drilling rig at the approximate locations shown on the Boring Location Plan in Appendix 1 and to the depths shown on the Logs in Appendix 2. Drilling was conducted by our subcontractor, Professional Services Industries, Inc., and an engineer with our staff was on site to document and direct the drilling activities.

Soil test borings shown on the Boring Location Plan in Appendix 1 were located in the field by measuring from prominent features and should therefore be considered approximate. Borings B-2, B-3, and B-8 were eliminated from the drilling program based on the relatively uniform conditions encountered at nearby borings.

At all borings, the soil overburden was augured, and at predetermined intervals, split spoon samples were obtained in general accordance with ASTM D1586; *Penetration Test and Spilt*

Barrel Sampling of Soils. At these boring locations, samples were obtained continuously within the first ten (10) feet and then at five-foot intervals thereafter. Additionally, the Standard Penetration Test Value (SPT 'N') was logged for each sample retrieved.

All soil samples obtained in the investigation were classified in the field in accordance with ASTM D 653 and then supplemented with the Uniform Soil Classification System, where laboratory data is available.

All borings were advanced until auger refusal and/or spoon refusal was achieved. Auger refusal is the depth at which the borehole can no longer be advanced by standard auger drilling techniques. On some sites, auger refusal indicates the top of hard rock; however, the rock at the subject site becomes gradually harder with depth and, therefore, refusal occurs gradually and is usually a function of the drill production rate.

Spoon refusal is encountered when the sampling spoon can no longer be advanced under continued blows of the sampling hammer and an SPT 'N' value of 50 blows over four (4) inches is obtained. Spoon refusal was generally encountered at all borings near the soil-bedrock interface.

Upon refusal at Borings B-5 and B-11, rock coring was then conducted employing standard NX/NQ wireline coring techniques. At both borings, a 2-¼" core barrel was advanced to obtain a 10-foot, relatively, undisturbed core sample of the rock underlying the site. The rock cores were classified and evaluated for percent of Recovery and Rock Quality Designator (RQD).

Rock Quality Designator (RQD) is a measure of the quality of a rock mass and is evaluated by summing the intact pieces of core that are greater than four (4) inches in length and dividing by the total length of the core run.

Three potential borrow areas were identified in the vicinity of borings B-1, B-11, and B-13/14. Individual bulk samples were obtained at borings B-1 and B-11, and a combined bulk sample retrieved at borings B-13 and B-14 for Standard Proctor and California Bearing Ratio testing.

Upon completion, each boring was checked for the presence of ground water and was backfilled with auger cuttings.

6.0 LABORATORY TESTING

Soil samples were field-classified as to consistency, material type and probable origin. As noted above, Borings B-5 and B-11 were extended into bedrock in order to obtain core for visual review.

Soil samples were transported to the laboratory at Professional Services Industries, Inc. (PSI) for classification. Selected soil samples were subjected to index testing to assess the soil's natural moisture content, Atterberg Limits, Unconfined Compressive Strength testing, California Bearing Ratio evaluation, and Standard Proctor (ASTM D-698) to establish moisture-density relationships. The bedrock core was logged for physical weaknesses.

The individual Logs and Profiles in Appendix 2 show interpretations of the subsurface conditions, descriptions of the materials encountered, and laboratory testing results.

7.0 SUBSURFACE CONDITIONS

7.1 Topsoil/Near Surface

A layer of organic laden topsoil and root mat was encountered at all boring locations at the site. Our exploration indicated that the topsoil thickness ranged from between 6 and 19 inches. Specific thicknesses are shown on the boring logs. Deeper topsoil and root mat should be anticipated in areas of dense vegetation or mature trees. Some of the large trees near the creek will have root systems that extend more than 3 feet below the ground surface.

7.2 Alluvial Soil

Alluvial soil is generally classified as water-deposited soil that is found within, and flanking drainage features. Alluvial soils are typically distinguished by their mottled appearance and lack of structure. Soil classified as alluvium was encountered at all boring locations and generally consisted of mottled, gray-to-dark gray, brown-to-dark brown, and yellow-to-dark yellow, lean clay with varying amounts of chert fragments and gravel. Blow counts for alluvial soils generally ranged between 10 and 20.

At Borings B-6, B-9, B-10, B-11, and B-12, brown and gray clay of high plasticity were encountered near refusal depths, with blow counts ranging between 11 to refusal of 50 blows over four (4) inches (50/4").

7.3 Rock Coring

Published geological literature indicates, and our review of the cores confirms, that the project lies within the Heritage Formation. The rock cores retrieved at Borings B-5 and B-11 were light-gray to gray, fine-to-medium grained limestone with thin shale partings. Inspection of the rock cores at indicates the bedrock at these locations has experienced only slight weathering, with only slight fracturing. Coring produced recoveries of 95% and 93%, and Rock Quality Designations (RQDs) of 68% and 70%, respectively. Table 1 below lists general guidelines related to RQDs and their description with respect to rock quality.

TABLE 1. Rock Quality Designator

RQD	Description
Greater than 90%	Excellent
75% to 90%	Good
50% to 75%	Fair
25% to 50%	Poor
Less than 25%	Very Poor

* *Foundation Analysis and Design; Joseph E. Boles; Fourth Edition; 1988.*

It should be recognized that even with high recovery and "Fair" RQD values, the limestone formation underlying the site is still susceptible to solution weathering along near-vertical fractures and gently dipping bedding planes, and could lead to the formation of sinkholes since the unit is composed of carbonate limestone. Additionally, even though an indication of extended weathering was not visible in the core sample, it is likely that weathering has occurred over the rock surface.

7.4 Groundwater

Groundwater was only observed only at borings B-5 and B-11, at depths of 10.9' and 4.9', respectively, upon termination of rock coring operations. In general, the presence or absence of water in the boreholes does not necessarily mean that groundwater will not be encountered at other locations or at other times.

It should be understood that our fieldwork was conducted following a relatively dry period of time. Consequently, groundwater levels were probably depressed. Groundwater levels should be expected to rise and the occurrence of shallow, perched or trapped water will likely increase after significant rainfall events. Additionally, seasonal variations will likely cause fluctuations in groundwater levels and influence the presence of water in the upper soils.

In any event, shallow foundation construction at the site may require cuts several feet deeper than the existing ground surface where perched or trapped water may be encountered. The Contractor should expect to provide for such conditions. Moreover, the Contractor should expect that even in modest excavations, water, rainfall, and runoff will accumulate and could affect overall construction.

8.0 COMMENTS AND RECOMMENDATIONS

8.1 General

At the time of this report, a plan for the site showing building type, layout or configuration, structural data, foundation loading data, or planned use had not been developed. However, based on our conversations with the Owner, we are assuming that development will generally be single-story retail and/or commercial buildings, similar to other developments in the area. Additionally, based on the projected limits of the 100-year flood plain throughout the site, we are assuming proposed development will necessitate construction of a shot rock or engineered fill building pad.

The comments and recommendations contained herein are predicated upon our experience in similar geologic settings, the assumed design criteria stated above, and the data obtained during this study. We request that when development plans are finalized, we have the opportunity to review our recommendations in light of the differences and offer appropriate revisions, as warranted.

With respect to the investigation, the Owner should recognize that it would be impractical and costly to conduct an intensive geotechnical study that identifies subsurface conditions and potential hazards encompassing the entire site. Accordingly, our recommendations are based on a limited number of observations and tests and reflect assumptions for the site based on characteristics revealed at boring locations. Finally, the comments and recommendations that follow are based on data that has been developed during this study as well as on our experience with similar projects in similar geologic settings.

Our comments and recommendations consider the locations of improvements as currently proposed and do not consider the effects of moving any proposed facilities to other, alternate locations not specifically noted herein. In the event that the proposed improvements are moved significantly from the currently proposed locations, it is likely that differing subsurface conditions will be encountered, thereby necessitating a review of the data and possible revision in our conclusions and recommendations. Relocating any proposed facilities could also necessitate additional exploration.

8.2 Hazards Associated with Sinkholes and Karst Terrain

Because this site is underlain by carbonate rock there is a risk of sinkhole development within the subject property. Based on our field review of the site and U.S.G.S. topographic map, no sinkholes or depressional features were discerned. In any event, we believe the potential for sinkholes or other karst-type features, such as voids, seams, and other solutional features to develop at the subject site is no greater than for other sites within this geologic setting.

It should be understood that present state-of-the-art of geotechnical engineering does not permit accurate prediction of where or when sinkholes or other karst-type features will occur and the Owner should realize that the possibility for post-construction sinkhole development cannot be completely eliminated. Accordingly, construction on this property, or essentially any other site within a carbonate bedrock setting, carries with it some risk that future sinkholes may occur.

During construction, the grading contractor should be alert to any indication of possible incipient sinkholes within the subsurface. Any sinkhole, karst, or depressional features encountered during the site grading, or during later stages of construction, should be brought to the Owner's attention and repaired under the direction of the Owner's geotechnical engineer.

8.3 Pinnacle Conditions

Typically, limestone bedrock pinnacles display relatively high relief and project vertically upward from narrow bedrock lows. Limestone pinnacles are commonly bordered by "cutters," or mud-filled seams and can even consist of large limestone blocks or boulders "floating" in a clay matrix.

Based on subsurface conditions revealed at explored locations, refusal depths across the site were relatively consistent, and as such, judge that pinnacle-type conditions will probably not be encountered during excavation and grading.

8.4 Unclassified Fill

Based on our site reconnaissance and review of drawings, several structures from prior development including the existing office building (formerly the Hitchin' Post), concrete pad, septic tank and field, shed, and propane tank are located near the southeast portion of the site. Additionally, at boring B-5, a gravel layer approximately 1-foot thick was encountered.

Based on these conditions, it is likely that unclassified fill was used during previous development in the vicinity of the existing office building and surrounding areas and was not placed and compacted in accordance with standards considered to be acceptable for structural fill. Consequently, there is a geotechnical risk associated with construction upon this or any fill placed without technical oversight, and the possibility exists that the fill contains poorly compacted zones or deleterious materials not detected by the exploration.

The geotechnical risk is related to the potential for poor subgrade reaction and objectionable settlement resulting from the unknown aspects of the fill. In order to eliminate the risk, any existing fill should be completely removed and replaced with engineered fill.

8.5 General Site Preparation

Initially, all trees, bushes, shrubs, topsoil, unclassified fill, and other deleterious materials should be stripped from the area proposed for construction. Where possible, stripping operations should extend a minimum of ten feet beyond the perimeter of the proposed structures and at least five feet beyond the edge of planned pavement.

Based on the exploration data, the average depth of stripping to remove topsoil could extend 20 inches or more. Stripped topsoil should be stockpiled on site and used for landscaping purposes, or wasted off-site. All wasted material from excavation including asphalt, concrete, metal, rubble, building materials, and boulders should be completely removed from the site and disposed of in a proper, safe, and legal manner.

Although most of the soils exhibited a high consistency during our fieldwork, exposure to inclement weather prior to construction will result in some deterioration of fine-grained soils at the site. Therefore, the subgrade soils may rut and pump when wet and exposed to construction traffic and additional effort on the contractor's part may be necessary to maintain an acceptable subgrade.

8.6 Plastic Soils

Based on laboratory testing results, relatively higher-plasticity cohesive soil samples were encountered at Borings B-4, B-5, B-9, B-11, and B-12. Generally, a Plasticity Index (PI) of 25 is used as a 'benchmark' to determine if treatment of plastic soils at a particular site will be required. Plasticity Indices of higher-plasticity soils ranged between 20 and 28, and were generally encountered at deeper sampling intervals at the boring locations mentioned.

In general, there are two main concerns with highly-plastic soils. First, when exposed to moisture content changes, plastic clay soil will exhibit a high shrink/swell potential that could result in distress of foundations and floor slabs. Second, from a construction standpoint, highly-plastic soils are easily disturbed and erodible when exposed to moisture and may require additional effort during excavation and grading.

Based on our understanding, proposed construction at the site will likely include construction of a shot rock building pad. As a result, we do not anticipate cuts into, or construction within higher-plasticity soils.

In any event, if plastic soils are encountered within the footprint and immediate vicinity of proposed construction, these soils should be undercut a minimum of 24 inches below the designated bearing surface and replaced with suitable fill. Additionally, excavation, undercut and fill placement is typically extended outside of the building footprint to a distance of typically, 5 feet. The Owner's geotechnical engineer should be afforded the opportunity to evaluate exposed soils and subgrades prior to construction in areas where higher-plasticity soils have been encountered and make additional recommendations, if necessary.

8.7 Subgrade Preparation

In general, **all** areas that are at final subgrade elevation that are to receive fill should be evaluated by the Owner's geotechnical engineer. In areas where residual soil (alluvium) is exposed, such an evaluation may include proofrolling or other heavy, pneumatic tire-mounted, construction equipment in order to reveal pockets of soft or loose soil.

Subsequent to proofrolling, the Owner's geotechnical engineer can then determine the amount of undercutting or stabilization, if any, that will be necessary to prepare a suitable subgrade. Any unstable soils detected by the proofrolling activities should be undercut to firm ground and bridged with shot rock fill, engineered fill, stone, or, if approved by the Owner's geotechnical engineer, scarified, moisture conditioned soil, re-compacted to 98% of the soil's maximum dry density as determined by the Standard Proctor test (ASTM D 698).

We expect that the potential for undercut will depend upon the prevailing weather and seasonal conditions at the time grading occurs. If conditions are generally wet at the time of construction, undercutting requirements could be significant.

After the subgrade has been stabilized, shot rock or engineered fill (as specified) can be placed upon the uniformly stable subgrade. Cut areas should be proofrolled and repaired in a similar fashion after reaching required subgrade elevations.

8.8 On-Site Soil for Use as Fill

Organic-free soil derived from on-site excavations in the vicinity of borings B-1, B-11, B-13, and B-14 will be suitable for use as engineered fill in proposed building and parking areas, provided they are properly moisture conditioned and densified in accordance with the laboratory testing results in Appendix 3, and the tolerances for Engineered Fill discussed in Section 8.9.

8.9 Engineered Fill

"Engineered fill" refers to on-site soil obtained from the designated areas in Section 8.8 above, or other approved, off-site locations. Should the Contractor elect to use fill obtained from an off-site source, the source should be tested, evaluated, and approved by the Owner's geotechnical engineer before being used as engineered fill.

Engineered fill should consist of organic-free, clayey soil derived from an approved borrow source. In general, engineered soil fill should consist of low to medium plasticity (PI < 25) clay designated "CL" by the Unified Soil Classification System.

Engineered soil fill placed within the proposed building areas should be densified to at least 98% of the soil's maximum dry density in accordance with ASTM D 698 (Standard Proctor) and placed in lifts not exceeding eight (8) inches in uncompacted thickness.

Engineered soil fill placed within the proposed parking areas or utility trenches should be placed in lifts not exceeding eight inches in loose thickness and densified to at least 95% of the Standard Proctor (ASTM D 698) maximum dry density. The upper one foot of fill should be compacted to 98% of maximum dry density.

In order to reduce the potential for volume change in response to changes in moisture, the moisture content of all engineered fill should be controlled to within ± 2% of the Standard Proctor optimum moisture content.

It should be recognized that should the Contractor choose to use engineered fill, a considerable amount of additional and repeated effort may be required to properly moisture condition the material used for engineered fill in order to obtain adequate stability and density during compaction. Additionally, the Contractor should be responsible for establishing regular and systematic moisture-density testing and must be able to provide records to the Owner and Owner's geotechnical engineer demonstrating that engineered fill was placed in accordance with recommendations discussed above.

8.10 Shot Rock Fill

Based on projects at nearby sites and typical construction practices in the area, the Owner may elect to use shot rock fill for building pad construction. Engineered shot rock fill should consist of hard, durable limestone fragments. The material should include well-graded particles ranging in size from 18 inches to fines. Shot rock fill should be placed in loose, horizontal lifts no thicker than 24 inches and compacted until stable, based on technical observation, by repeated passes with heavy, steel-tracked equipment no lighter than a D-8 bulldozer. Placement of shot rock fill should be monitored by the Owner's geotechnical engineer.

8.11 Slopes

Outslopes of soil fill should be permanently inclined no steeper than 2.5:1; an outslope inclination of 3:1 or flatter should be incorporated if those areas are to be accessed with mowers or other landscaping equipment. Excavations in stable soil may be permanently laid back at inclinations no steeper than 2.5H:1V. Shot rock fill may inclined to grades not steeper than 1.5H:1V.

8.12 Groundwater and Runoff Control

The site should be maintained in a well-drained condition, both during and after construction, to prohibit water from ponding on soil subgrades. Ponding of water could lead to the deterioration of the subgrade where residual soil is exposed and may necessitate over-excavation of the softened soil. In addition, the amount of subgrade repair that may be required will vary based on weather conditions during the construction period.

We expect that limited quantities of ground water will be encountered in some excavations during construction. In any event, our experience has been that site preparation and foundation work in this geologic setting is most easily accomplished during periods of dry weather.

8.13 Trenches for Excavation Work

The sidewalls of trenches or other temporary excavations should in no case exceed the maximum safe inclination as specified by OSHA (OSHA 29 CFR Part 1926). If workers are to enter trenches or excavations greater than four feet in depth or work areas adjacent to excavated slopes that do not have sidewalls laid back to maximum safe inclinations mandated by OSHA, an OSHA-approved trench box or a shoring/sheeting system designed by a registered engineer must be utilized to protect work crews.

8.14 Utility Trenches

Backfilling of storm drains and utility trenches is often accomplished in an uncontrolled manner leading to subsequent settlement of the fill and cracking of floor slabs and pavements. Additionally, backfilling around manholes and other confined excavations has been a problem on many projects due to poor filling and compaction practices. Consequently, utility trench backfill should consist of free-draining, uniformly sized stone, such as ASTM D 448 size No. 57 or engineered fill. Stone fill should be compacted with vibratory sled compactors and be placed in lift thickness not exceeding 12 inches. Engineered fill, if used, should be placed and compacted in accordance with Section 8.9.

8.15 Foundations - General

The Owner's geotechnical engineer or his representative should examine all footing excavations immediately prior to being cast with concrete in order to observe the bearing surface and to document that conditions are as anticipated, as well as observe the placement of engineered soil or shot rock fill in the building area and all footing excavations.

During foundation installation in soil, isolated soft zones may be encountered at the bearing elevation. If soft zones are encountered, the footing subgrade in that area should be undercut to a firm stratum and backfilled with engineered fill so that the foundation element bears on a uniformly stable subgrade.

When founded in accordance with our recommendations, both gross and differential settlements for the buildings are expected to be within limits normally considered tolerable for structures of the type proposed.

Settlement of footings founded upon bedrock is expected to be negligible, while settlement of foundations bearing on shot rock or engineered fill constructed on residual soil may be expected to settle up to 1 inch; differential settlement could approach 50% of that value. Measurable settlement occurs in all building foundations; however, if the guidelines above are adhered to, settlement should be negligible. Additionally, the owner can help minimize the effects of settlement in the structure itself by, for example, employing liberally spaced, vertical control joints in masonry walls to help minimize cosmetic cracking.

Lateral loads exerted against the spread foundation systems can be resisted by the passive earth pressure developed against the vertical face of the footing and by the friction acting between the base of the footing and the weathered bedrock subgrade. Provided that the concrete for the footing is adequately formed or cast neat against the sides of an excavation, the passive earth pressure can be computed based on an equivalent fluid pressure of 300 PCF. The coefficient of friction between the base of the foundations and the bedrock subgrade is estimated at 0.65; a friction coefficient of 0.4 may be assumed for foundations bearing in soil. A factor of safety of at least 1.5 should be used when calculating resistance to lateral loads. In order to accommodate minor uplift loads, the designers may take into account the weight of the footing and the weight of backfill above the footing element. For backfill compacted to 98% of Proctor density, the unit weight of the backfill above the foundation (vertical projection of the footing limits) may be taken as 100 PCF. For random backfill placed thereon, the unit weight above the footing should be taken as 90 PCF.

8.16 Foundations - Design

Provided that the site is prepared in accordance with the recommendations stated previously, the proposed structure can be supported by means of a conventionally designed, shallow foundation system bearing on engineered or shot rock fill or stable, natural soil. Based on a safety factor of at least 3 with respect to general shear failure, we assess the allowable load bearing capability of both the stable, natural soil and properly compacted, engineered or shot rock fill at 3,000 and 2,500 pounds-per-square-foot for loads as applied by individual and continuous footings, respectively.

A minimum footing width of 30 inches should be specified for all foundations, regardless of loading, in order to accommodate minor subgrade inconsistency and to resist punching failure. Further, perimeter footings should be designed to bear at least 24 inches below exterior, finished grades to provide frost protection, adequate confinement and to provide a bearing level that is below the depth of significant seasonal moisture change.

8.17 Slabs-on-Grade

Concrete slabs-on-grade are expected to perform satisfactorily if founded on a properly prepared subgrade consisting of shot rock or engineered fill. A free-draining, well-compacted granular base at four (4) inches thick and a vapor barrier should be incorporated into the slab design. Slab thickness and reinforcing requirements can be designed based on an estimated subgrade reaction modulus of 150 pounds-per-cubic-inch (PCI). An appropriate number of control joints should be included in the slab design to accommodate minor differential settlement that may occur.

8.18 Pavement Design

At the time of this report, traffic and anticipated loading data were not available. However, based on laboratory testing results, we estimate that the soil subgrade will develop minimum support characteristics at least equal to a California Bearing Ratio (**CBR**) of **2**.

All elements of pavement construction should conform to the latest requirements of the Tennessee Department of Transportation's Standard Specifications for Road and Bridge Construction, except that the aggregate base course should contain no more than 12% particles passing the No. 200 sieve as determined by the wet method.

Immediately prior to the installation of the mineral aggregate base course, the pavement subgrade should be proofrolled in order to detect unstable areas; any unstable areas should be repaired as previously described. We recommend that subgrades be graded to provide positive drainage away from the paved areas to prevent the aggregate base course from being saturated and thereby reducing the support capabilities of the subgrade. In addition, we recommend that the base course be daylighted at the edges of the pavement, if possible. During construction of the aggregate base, in-place density tests and thickness checks should be performed to evaluate compliance with project specifications. If a significant delay occurs between installation of the aggregate base and the bituminous elements above, the mineral aggregate should be proofrolled in order to confirm that no loss in stability has occurred. Ultimately, it is essential that the bituminous pavement elements only be installed on a uniformly stable aggregate base. In addition, landscaped areas should be constructed so that surface water infiltration is not permitted to migrate laterally to the base stone or subgrade beneath the adjoining pavements.

As plans for development and construction are finalized, we are available to assist with specific pavement design to fit the proposed development.

9.0 CONSTRUCTION QUALITY CONTROL

The satisfactory, long-term performance of the proposed project will be dependent upon the quality of the geotechnical aspects of construction. The Owner should recognize that unanticipated or changed conditions might be encountered during any site grading and/or foundation installation effort.

The recommendations contained in this report assume that CEPC will be retained to provide construction monitoring services so that we can confirm that the subsurface conditions are generally as anticipated, or adjust our recommendations accordingly. Additionally, our forces will be available to further assist you by providing these and other normally specified quality control and testing services, should you so desire.

10.0 REPORT LIMITATIONS

The recommendations submitted in this report based on the available subsurface information obtained during the investigation. The geotechnical engineer warrants that the findings, recommendations, specifications, or professional advice contained herein have been made in accordance with generally accepted professional geotechnical engineering practices in the local area. No other warranties are implied or expressed.

After the Plans and Specifications are complete, the geotechnical engineer should be provided the opportunity to review final design Plans and Specifications to check that our engineering recommendations have been properly incorporated into the design documents. At that time, it may be necessary to submit supplementary recommendations.

This report has been prepared for the specific application to the proposed development by Harvey & Harvey Associates, LLC.

11.0 CLOSURE

Crouch Engineering, P.C. appreciates this opportunity to be of service to Harvey & Harvey Associates, LLC. At your convenience, we are available to discuss the details of this report and any questions you may have.

Respectfully,
Crouch Engineering, P.C.

William J. Cedzich

William J. Cedzich, P.E., R.G.
Project Manager

Important Information About Your

Geotechnical Engineering Report

Subsurface problems are a principal cause of construction delays, cost overruns, claims, and disputes.

The following information is provided to help you manage your risks.

Geotechnical Services Are Performed for Specific Purposes, Persons, and Projects

Geotechnical engineers structure their services to meet the specific needs of their clients. A geotechnical engineering study conducted for a civil engineer may not fulfill the needs of a construction contractor or even another civil engineer. Because each geotechnical engineering study is unique, each geotechnical engineering report is unique, prepared *solely* for the client. *No one except you* should rely on your geotechnical engineering report without first conferring with the geotechnical engineer who prepared it. *And no one—not even you*—should apply the report for any purpose or project except the one originally contemplated.

Read the Full Report

Serious problems have occurred because those relying on a geotechnical engineering report did not read it all. Do not rely on an executive summary. Do not read selected elements only.

A Geotechnical Engineering Report Is Based on A Unique Set of Project-Specific Factors

Geotechnical engineers consider a number of unique, project-specific factors when establishing the scope of a study. Typical factors include: the client's goals, objectives, and risk management preferences; the general nature of the structure involved, its size, and configuration; the location of the structure on the site; and other planned or existing site improvements, such as access roads, parking lots, and underground utilities. Unless the geotechnical engineer who conducted the study specifically indicates otherwise, *do not rely on a geotechnical engineering report* that was:

- not prepared for you,
- not prepared for your project,
- not prepared for the specific site explored, or
- completed before important project changes were made.

Typical changes that can erode the reliability of an existing geotechnical engineering report include those that affect:

- the function of the proposed structure, as when it's changed from a parking garage to an office building, or from a light industrial plant to a refrigerated warehouse,
- elevation, configuration, location, orientation, or weight of the proposed structure,
- composition of the design team, or
- project ownership.

As a general rule, *always* inform your geotechnical engineer of project changes—even minor ones—and request an assessment of their impact. *Geotechnical engineers cannot accept responsibility or liability for problems that occur because their reports do not consider developments of which they were not informed.*

Subsurface Conditions Can Change

A geotechnical engineering report is based on conditions that existed at the time the study was performed. *Do not rely on a geotechnical engineering report* whose adequacy may have been affected by: the passage of time; by man-made events, such as construction on or adjacent to the site; or by natural events, such as floods, earthquakes, or groundwater fluctuations. *Always* contact the geotechnical engineer before applying the report to determine if it is still reliable. A minor amount of additional testing or analysis could prevent major problems.

Most Geotechnical Findings Are Professional Opinions

Site exploration identifies subsurface conditions *only* at those points where subsurface tests are conducted or samples are taken. Geotechnical engineers review field and laboratory data and then apply their professional judgment to render an *opinion* about subsurface conditions throughout the site. Actual subsurface conditions may differ—sometimes significantly—from those indicated in your report. Retaining the geotechnical engineer who developed your report to provide construction observation is the most effective method of managing the risks associated with unanticipated conditions.

A Report's Recommendations Are *Not* Final

Do not overrely on the construction recommendations included in your report. *Those recommendations are not final,* because geotechnical engineers develop them principally from judgment and opinion. Geotechnical engineers can finalize their recommendations only by observing actual subsurface conditions revealed during construction. *The geotechnical engineer who developed your report cannot assume responsibility or liability for the report's recommendations if that engineer does not perform construction observation.*

A Geotechnical Engineering Report Is Subject To Misinterpretation

Other design team members' misinterpretation of geotechnical engineering reports has resulted in costly problems. Lower that risk by having your geotechnical engineer confer with appropriate members of the design team after submitting the report. Also retain your geotechnical engineer to review pertinent elements of the design team's plans and specifications. Contractors can also misinterpret a geotechnical engineering report. Reduce that risk by having your geotechnical engineer participate in prebid and preconstruction conferences, and by providing construction observation.

Do Not Redraw the Engineer's Logs

Geotechnical engineers prepare final boring and testing logs based upon their interpretation of field logs and laboratory data. To prevent errors or omissions, the logs included in a geotechnical engineering report should *never* be redrawn for inclusion in architectural or other design drawings. Only photographic or electronic reproduction is acceptable, *but recognize that separating logs from the report can elevate risk.*

Give Contractors a Complete Report and Guidance

Some owners and design professionals mistakenly believe they can make contractors liable for unanticipated subsurface conditions by limiting what they provide for bid preparation. To help prevent costly problems, give contractors the complete geotechnical engineering report, *but* preface it with a clearly written letter of transmittal. In that letter, advise contractors that the report was not prepared for purposes of bid development and that the report's accuracy is limited; encourage them to confer with the geotechnical engineer who prepared the report (a modest fee may be required) and/or to conduct additional study to obtain the specific types of information they need or prefer. A prebid conference can also be valuable. *Be sure contractors have sufficient time* to perform additional study. Only then might you be in a position to give contractors the best information available to you, while requiring them to at least share some of the financial responsibilities stemming from unanticipated conditions.

Read Responsibility Provisions Closely

Some clients, design professionals, and contractors do not recognize that geotechnical engineering is far less exact than other engineering disciplines. This lack of understanding has created unrealistic expectations that have led to disappointments, claims, and disputes. To help reduce such risks, geotechnical engineers commonly include a variety of explanatory provisions in their reports. Sometimes labeled "limitations", many of these provisions indicate where geotechnical engineers responsibilities begin and end, to help others recognize their own responsibilities and risks. *Read these provisions closely.* Ask questions. Your geotechnical engineer should respond fully and frankly.

Geoenvironmental Concerns Are Not Covered

The equipment, techniques, and personnel used to perform a *geoenvironmental* study differ significantly from those used to perform a *geotechnical* study. For that reason, a geotechnical engineering report does not usually relate any geoenvironmental findings, conclusions, or recommendations; e.g., about the likelihood of encountering underground storage tanks or regulated contaminants. *Unanticipated environmental problems have led to numerous project failures.* If you have not yet obtained your own geoenvironmental information, ask your geotechnical consultant for risk management guidance. *Do not rely on an environmental report prepared for someone else.*

Rely on Your Geotechnical Engineer for Additional Assistance

Membership in ASFE exposes geotechnical engineers to a wide array of risk management techniques that can be of genuine benefit for everyone involved with a construction project. Confer with your ASFE-member geotechnical engineer for more information.

ASFE

8811 Colesville Road Suite G106 Silver Spring, MD 20910
Telephone: 301-565-2733 Facsimile: 301-589-2017
email: info@asfe.org www.asfe.org

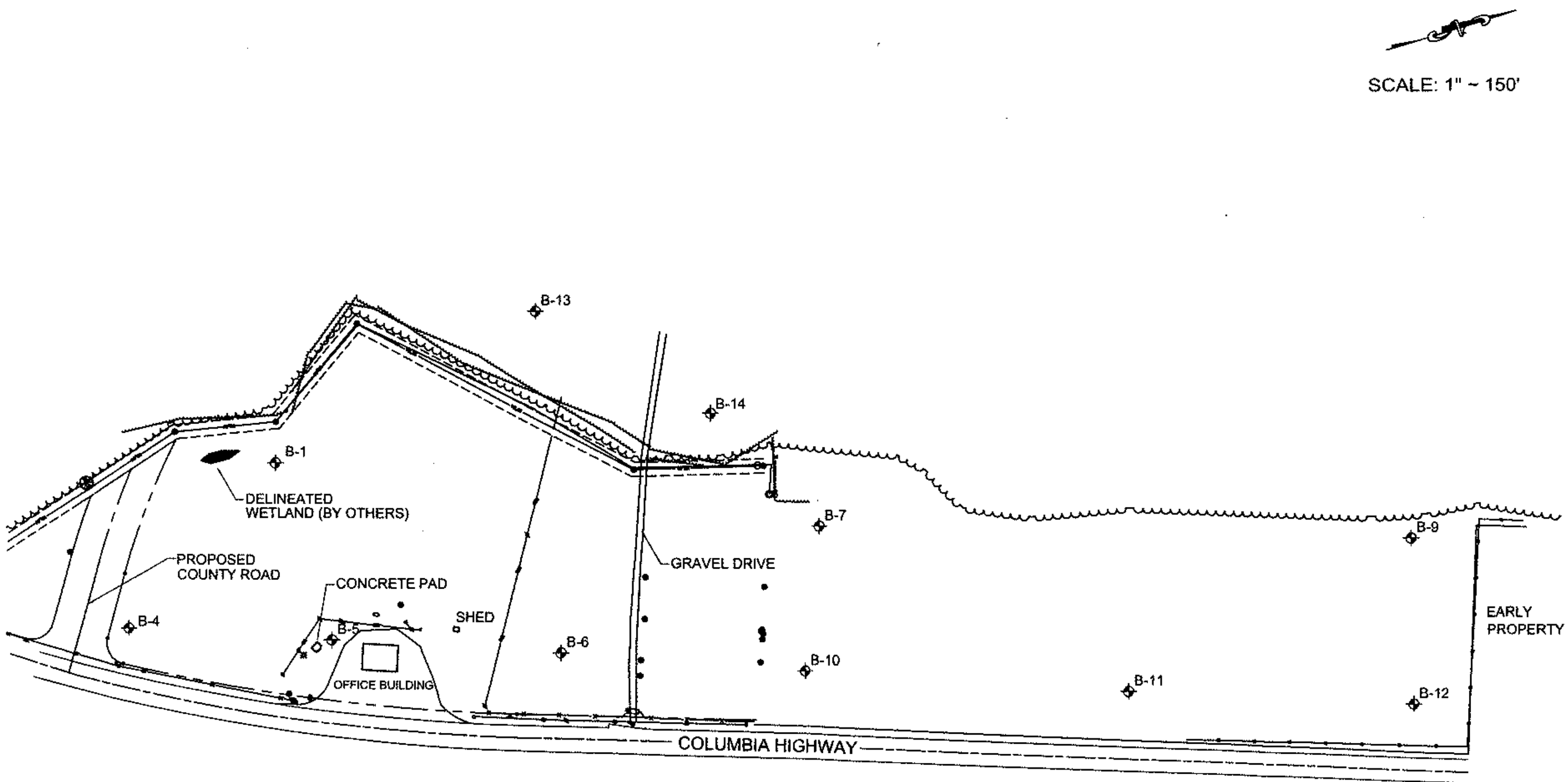
IIGER1000.10M

Hitchin' Post Commercial Center
for
Harvey and Harvey Associates, LLC
Spring Hill, Tennessee

Appendix 1
Boring Location Plan

CROUCH ENGINEERING®, P.C.

428 Wilson Pike Circle, P. O. Box 1186, Brentwood, Tennessee 37024-1186
Phone: (615) 791-0630 Fax: (615) 791-8451
Website: www.crouchengineering.com



NOTES

B-11 DRIVE SAMPLED BORING

- BORINGS B-2, B-3, AND B-8 WERE ELIMINATED FROM THE DRILLING PROGRAM.
- BORINGS WERE LOCATED IN THE FIELD BY MEASURING FROM PROMINENT FEATURES; LOCATIONS SHOULD BE CONSIDERED APPROXIMATE.
- BORINGS PERFORMED BY PROFESSIONAL SERVICES INDUSTRIES, INC. AUGUST, 2005.

BORING LOCATION PLAN

HITCHIN' POST COMMERCIAL CENTER
SPRING HILL, TENNESSEE

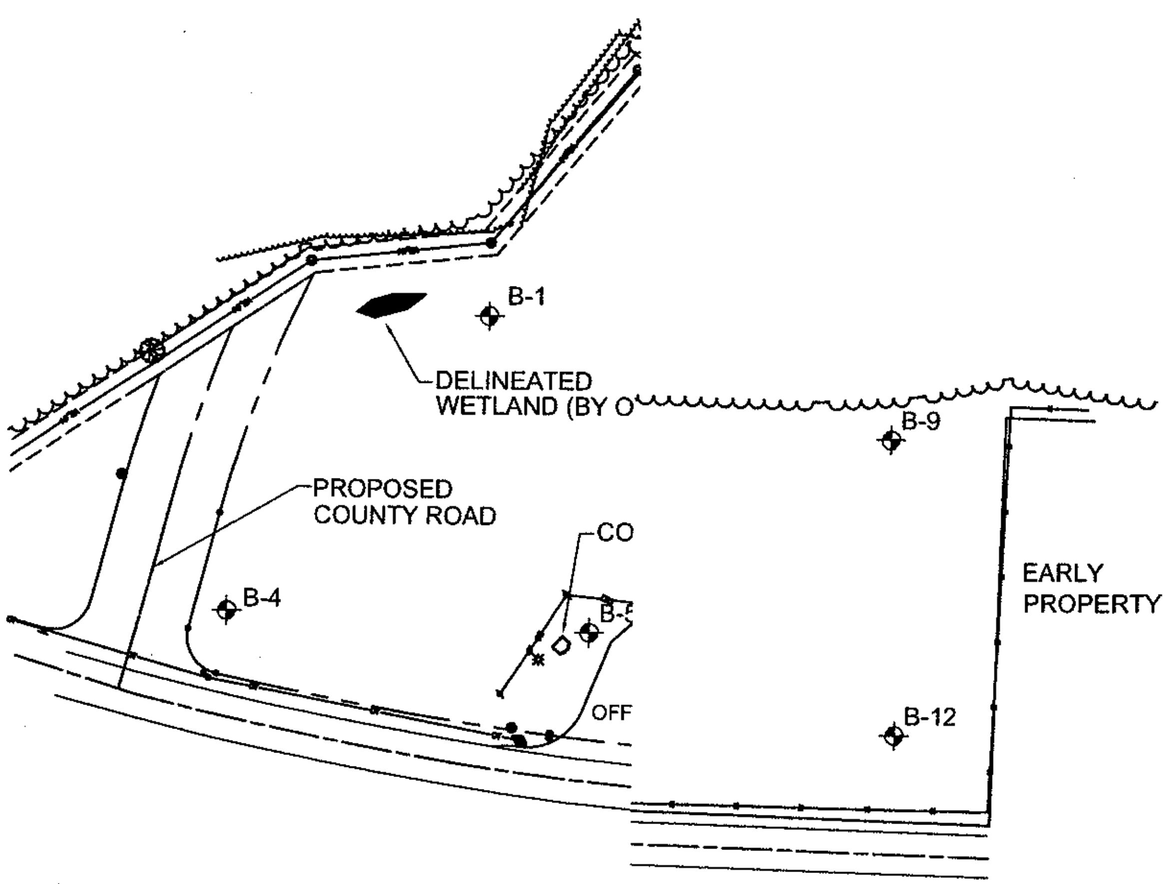
HITCHIN' POST COMMERCIAL CENTER
HARVEY & HARVEY ASSOCIATES, LLC
HIGHWAY 31
SPRING HILL, TENNESSEE

CROUCH ENGINEERING P.C.
428 WILSON PIKE CIRCLE
BRENTWOOD, TN 37027
PHONE NO. (615) 791-0630

PROJECT NO: 5063
DATE: 10/10/2005

SHEET NUMBER
B-1
1 of 1

SCALE: 1" ~ 150'



NOTES

B-11 DRIVE SAMPLED BORING

* BORINGS B-2, B-3, AND B-8 WERE ELIMIN/
FROM THE DRILLING PROGRAM.

* BORINGS WERE LOCATED IN THE FIELD E
PROMINENT FEATURES; LOCATIONS SHC
CONSIDERED APPROXIMATE.

* BORINGS PERFORMED BY PROFESSION/
AUGUST, 2005.

OCATION PLAN
COMMERCIAL CENTER
L, TENNESSEE

HITCHIN' POST
COMMERCIAL CENTER
HARVEY & HARVEY ASSOCAITES, LLC
HIGHWAY 31
SPRING HILL, TENNESSEE

CROUCH ENGINEERING® P.C.
428 WILSON PIKE CIRCLE
BRENTWOOD, TN 37027
PHONE NO. (615) 791-0630

PROJECT NO: — 5092
DATE: — 10/10/2005
DRAWN BY: — WJC
CHECKED BY: — KAG
REVISIONS:

SHEET NUMBER

B-1

1 of 1

Hitchin' Post Commercial Center
for
Harvey and Harvey Associates, LLC
Spring Hill, Tennessee

Appendix 2
Boring Logs

CROUCH ENGINEERING®, P.C.

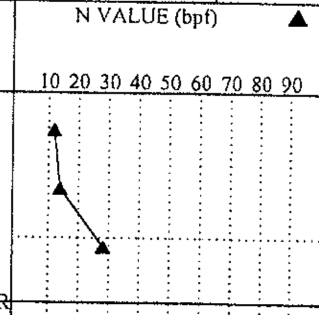
428 Wilson Pike Circle, P. O. Box 1186, Brentwood, Tennessee 37024-1186
Phone: (615) 791-0630 Fax: (615) 791-8451
Website: www.crouchengineering.com

BORING LOG

Information To Build On

Project: **Highway 31**		PSI No.: **358-55176**	Date: **8/23/05**
		Location: **Spring Hill, TN**	
Boring No.: **B-1**	Total Depth **7.2'**	Elev:	Water at Completion of Drilling: **Not Encountered**
Boring Method: **Hollow Stem Auger**	Drill Type: **CME-55**	Driller: **RC**	

Elevation (MSL)	Depth (feet)	Sample	DESCRIPTION OF MATERIALS	N	N VALUE (bpf)	% M	LL	PI	Qp	CBR	% RET. #200
	1.6		19"TOPSOIL	12		11				2	
	2.0		Gray Brown Lean CLAY with roots, sand and silt, moist, stiff. (CL)	14		15					
	4.0		Gray Brown to Brown Lean CLAY with mineral deposits, roots, sand and silt, moist, stiff. (CL)	28		43	36	15			47
	6.5		Gray Brown Lean CLAY with limestone pieces and sand, moist, very stiff. (CL)	50/PR		10					
	7.2		Dark Gray SILT with limestone layers, moist. (ML)								
			Auger Refusal 7.2 Feet								



BORING LOG

PSI Information To Build On

Project: Highway 31		PSI No.: 358-55176	Date: 8/22/05
		Location: Spring Hill, TN	
Boring No.: B-4	Total Depth 9.2'	Elev:	Water at Completion of Drilling: Not Encountered
Boring Method: Hollow Stem Auger	Drill Type: CME-55		Driller: RC

Elevation (MSL)	Depth (feet)	Sample	DESCRIPTION OF MATERIALS	N	N VALUE (bpf) 10 20 30 40 50 60 70 80 90	% M	LL	PI	Qp	Qu	% RET. #200
	0.2		2"TOPSOIL								
	2.0	X	Gray Brown Lean CLAY with roots, sand and silt, moist, firm. (CL)	8		10					
	4.0	X	Gray Brown to Dark Gray Lean CLAY with chert pieces, sand and silt, moist, firm. (CL)	7		15					
	6.5	X	Gray to Dark Yellow Brown Lean CLAY with mineral deposits, sand and silt, moist, stiff. (CL)	10		23	43	26	1.5		
	9.0	X	Dark Yellow Brown SILT with limesotne pieces and sand, very stiff. (ML)	25		13			2.0		46
	9.2		Gray to Dark Yellow Brown Lean CLAY with sand and silt, moist. (CL)	50/PR		21					

Auger Reufsal 9.2 Feet

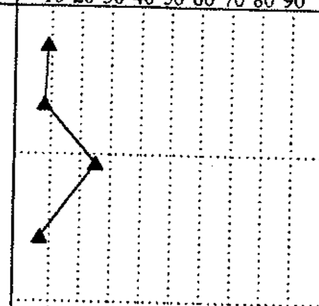
NVI 3585176.GPJ 9/16/05

BORING LOG

psi *Information To Build On*

Project: **Highway 31**		PSI No.: **358-55176**	Date: **8/22/05**
		Location: **Spring Hill, TN**	
Boring No.: **B-5**	Total Depth **19.0'**	Elev:	Water at Completion of Soil Drilling: **Not Encountered**
Boring Method: **Hollow Stem Auger**	Drill Type: **CME-55**	Driller: **RC**	

Elevation (MSL)	Depth (feet)	Sample	DESCRIPTION OF MATERIALS	N	N VALUE (bpf) 10 20 30 40 50 60 70 80 90	% M	LL	PI	Qp	Qu	PID
	0.5		6"TOPSOIL								
	2.0		Dark Gray Brown Lean CLAY with roots, sand and silt, moist, stiff. (CL)	9		12					
	4.0		Brown Lean CLAY with roots, sand, silt, moist, firm. (CL)	8		18			4.0		
	6.5		Gray to Dark Yellow Brown Lean CLAY with weathered chert, sand and silt, moist, very stiff. (CL)	25		18			3.5		43
	9.0		Brown to Gray Brown Lean CLAY with mineral deposits, sand and trace chert layers, moist, firm. (CL)	7		30	44	24	1.5		
			Auger Refusal 9.0 Feet; Begin Coring:								
			Light Gray to Gray, Fine to Medium Grained LIMESTONE with shale partings and trace clay seams, slightly fractured.								
			REC = 95%; RQD = 68%								
	19.0		UCS (upper rock core) = 3,302 psi								
			Coring Terminated at 19.0 Feet								
			Water Level After Coring at 10.9 feet.								



BORING LOG

Information To Build On

Project: **Highway 31**		PSI No.: **358-55176**	Date: **8/22/05**
		Location: **Spring Hill, TN**	
Boring No.: **B-6**	Total Depth **6.6'**	Elev:	Water at Completion of Drilling: **Not Encountered**
Boring Method: **Hollow Stem Auger**	Drill Type: **CME-55**	Driller: **RC**	

Elevation (MSL)	Depth (feet)	Sample	DESCRIPTION OF MATERIALS	N	N VALUE (bpf) 10 20 30 40 50 60 70 80 90	% M	LL	PI	Qp	Qu	% RET. #200
	0.7		8"TOPSOIL								
	2.0		Gray Brown Lean CLAY with roots and sand, moist, firm. (CL)	8		12					
	4.0		Light Brown to Brown Lean CLAY with chert pieces, mineral deposits and sand, moist, very stiff. (CL)	21		14					
	6.5		Dark Yellow Brown to Gray Fat CLAY with chert pieces and mineral deposits, moist, stiff. (CH)	11		24			4.0		
	6.6		Dark Brown to Brown Fat CLAY with mineral deposits, trace chert pieces and limestone pieces, moist. (CH)	50/PR		25					
			Auger Refusal 6.6 Feet								

NV1 35855176.GPJ 9/16/05

BORING LOG

psi Information To Build On

Project: **Highway 31**		PSI No.: **358-55176**	Date: **8/23/05**
		Location: **Spring Hill, TN**	
Boring No.: **B-7**	Total Depth **8.0'**	Elev:	Water at Completion of Drilling: **Not Encountered**
Boring Method: **Hollow Stem Auger**	Drill Type: **CME-55**	Driller: **RC**	

Elevation (MSL)	Depth (feet)	Sample	DESCRIPTION OF MATERIALS	N	N VALUE (bpf) 10 20 30 40 50 60 70 80 90	% M	LL	PI	Qp	Qu	% RET. #200
	1.0		12"TOPSOIL								
	2.0		Light Brown to Brown Lean CLAY with sand and silt, moist, very stiff. (CL)	16		9					
	4.0		Gray to Dark Yellow Brown Lean CLAY with chert pieces, mineral deposits and sand, moist, hard. (CL)	48		11					
	6.5		Gray to Dark Yellow Brown Lean CLAY with chert pieces, sand and silt, dry to moist, very stiff. (CL)	20		13			4.5		
	8.0		Brown Lean CLAY with chert layers, limestone layers, mineral deposits and sand, dry to moist. (CL)	50/PR		17					

Auger Refusal 8.0 Feet

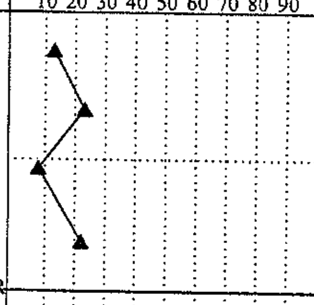
NV1 35855176.GPJ 9/16/05

BORING LOG

Information To Build On

Project: **Highway 31**		PSI No.: **358-55176**	Date: **8/23/05**
		Location: **Spring Hill, TN**	
Boring No.: **B-9**	Total Depth **9.5'**	Elev:	Water at Completion of Drilling: **Not Encountered**
Boring Method: **Hollow Stem Auger**	Drill Type: **CME-55**	Driller: **RC**	

Elevation (MSL)	Depth (feet)	Sample	DESCRIPTION OF MATERIALS	N	N VALUE (bpf) 10 20 30 40 50 60 70 80 90	% M	LL	PI	Qp	Qu	% RET. #200
	1.0		12"TOPSOIL								
	2.0		Dark Gray to Brown Lean CLAY with roots, sand, silt and trace chert pieces, dry to moist, stiff. (CL)	13		12					
	4.0			23		12					
	6.5		Light Gray to Brown Lean CLAY with weathered chert pieces, sand and silt, moist, very stiff. (CL)	8		26	43	23			
	9.0		Gray to Dark Yellow Brown Lean CLAY with chert pieces, mineral deposits, sand and silt, moist, firm. (CL)	22		21			2.0		46
	9.5		Brown Fat CLAY with limestone pieces, sand and silt, moist, very stiff. (CH)	50/PR		26					
			Brown to Dark Gray Fat CLAY with limestone layers and silt, moist. (CH)								
			Auger Refusal 9.5 Feet								



BORING LOG

psi Information To Build On

Project: Highway 31		PSI No.: 358-55176	Date: 8/22/05
		Location: Spring Hill, TN	
Boring No.: B-10	Total Depth 9.5'	Elev:	Water at Completion of Drilling: Not Encountered
Boring Method: Hollow Stem Auger	Drill Type: CME-55	Driller: RC	

Elevation (MSL)	Depth (feet)	DESCRIPTION OF MATERIALS	N	% M	LL	PI	Qp	Qu	% RET. #200
	0.8	10"TOPSOIL							
	2.0	Light Brown Lean CLAY with sand and trace chert pieces, moist, firm. (CL)	8	16			4.0		
	4.0	Light Brown to Brown Lean CLAY with sand, silt, trace chert pieces and mineral deposits, moist, firm. (CL)	6	18			4.0		
	6.5	Light Brown to Brown to Dark Brown Lean CLAY with weathered chert pieces, mineral deposits and sand, moist, stiff. (CL)	11	22	39	18	4.5		
	9.0	Gray Brown to Dark Yellow Brown Fat CLAY with mineral deposits, sand and trace chert pieces, moist, very stiff. (CL)	19	19			4.5		25
	9.5	Brown Fat CLAY with chert layers and sand, moist. (CH)	50/PR	19					

Auger Refusal 9.5 Feet

N VALUE (bpf): 10 20 30 40 50 60 70 80 90

BORING LOG

Information To Build On

Project: Highway 31	PSI No.: 358-55176	Date: 8/22/05
Boring No.: B-11	Total Depth 18.0'	Elev:
Location: Spring Hill, TN	Water at Completion of Drilling: Not Encountered	
Boring Method: Hollow Stem Auger	Drill Type: CME-55	Driller: RC

Elevation (MSL)	Depth (feet)	Sample	DESCRIPTION OF MATERIALS	N	N VALUE (bpf) 10 20 30 40 50 60 70 80 90	% M	LL	PI	Qp	CBR	% RET. #200
	0.8		10"TOPSOIL								
	2.0		Gray to Dark Yellow Brown Lean CLAY with weathered chert pieces, sand and silt, moist, stiff. (CL)	10		14				2	
	4.0		Dark Yellow Brown to Gray Sandy CLAY with chert pieces, roots and silt, moist, stiff. (SC)	9		17					50
	6.5		SHELBY TUBE Sample.			--					
	8.0		Brown to Dark Gray Brown SILT with mineral deposits and sand, moist. (ML)	50/PR		36	33	5			
			Auger Refusal 7.8 Feet; Begin Coring:								
			Light Gray to Gray, Fine to Medium Grained LIMESTONE with shale partings, slightly fractured.								
			REC = 93%; RQD = 70%								
	18.0		UCS (upper rock core) = 9,282 psi								

Coring Terminated at 18.0 Feet

Water Level After Coring at 4.9 feet.

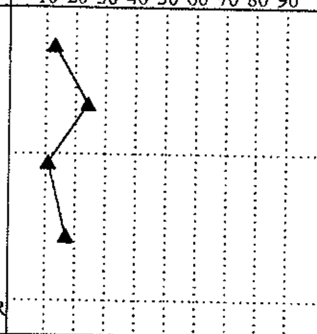
NV1 35855176.GPJ 10/3/05

BORING LOG

psi Information To Build On

Project: **Highway 31**		PSI No.: **358-55176**	Date: **8/22/05**
		Location: **Spring Hill, TN**	
Boring No.: **B-12**	Total Depth **11.2'**	Elev:	Water at Completion of Drilling: **Not Encountered**
Boring Method: **Hollow Stem Auger**	Drill Type: **CME-55**	Driller: **RC**	

Elevation (MSL)	Depth (feet)	Sample	DESCRIPTION OF MATERIALS	N	N VALUE (bpf) 10 20 30 40 50 60 70 80 90	% M	LL	PI	Qp	Qu	% RET. #200
	0.8		10"TOPSOIL								
	2.0		Light Brown Lean CLAY with roots, sand and silt, moist, very stiff. (CL)	13		15					
	4.0		Dark Yellow Brown Lean CLAY with weathered chert pieces, roots, sand and silt, moist, very stiff. (CL)	24		13					
	6.5		Dark Yellow Brown Fat CLAY with chert pieces, mineral deposits and sand, moist, stiff. (CH)	11		25	53	30	4.25		
	9.0		Dark Yellow Brown Fat CLAY with chert pieces, mineral deposits and sand, moist, very stiff. (CH)	17		21			4.5		14
	11.2		Brown Fat CLAY with mineral deposits sand sand, moist. (CH)	50/PR		28			1.5		



Auger Refusal 11.2 Feet

NV1 35855176.GPJ 9/16/05

BORING LOG

PSI Information To Build On

Project: **Highway 31**		PSI No.: **358-55176**	Date: **8/23/05**
		Location: **Spring Hill, TN**	
Boring No.: **B-13**	Total Depth **6.5'**	Elev:	Water at Completion of Drilling: **Not Encountered**
Boring Method: **Hollow Stem Auger**	Drill Type: **CME-55**	Driller: **RC**	

Elevation (MSL)	Depth (feet)	Sample	DESCRIPTION OF MATERIALS	N	N VALUE (bpf) 10 20 30 40 50 60 70 80 90	% M	LL	PI	Qp	Qu	% RET. #200
	0.8		10"TOPSOIL								
	2.0		Light Brown Lean CLAY with chert pieces, roots and sand, moist, stiff. (CL)	14		8					
	4.0		Brown to Dark Gray Lean CLAY with mineral deposits, roots, sand, silt and trace chert pieces, moist, stiff. (CL)	14		17			4.5		
	6.5		Dark Brown to Dark Gray to Dark Yellow Brown Fat CLAY with weathered chert layers and sand, moist. (CH)	50/PR		23					
			Auger Refusal 6.5 Feet								

BORING LOG

psi Information To Build On

Project: Highway 31	PSI No.: 358-55176	Date: 8/23/04
	Location: Spring Hill, TN	

Boring No.: B-13/14	Total Depth 1.0'	Elev:	Water at Completion of Drilling: Not Encountered
Boring Method: Hollow Stem Auger	Drill Type: CME-55		Driller: RC

Elevation (MSL)	Depth (feet)	Sample	DESCRIPTION OF MATERIALS	N	N VALUE (bpf) 10 20 30 40 50 60 70 80 90	% M	LL	PI	Qp	CBR	% RET. #200
	1.0		Brown Lean CLAY with silt. (CL)								
		B								3	

NV1 35855176.GPJ 10/3/05

BORING LOG

psi Information To Build On

Project: Highway 31		PSI No.: 358-55176	Date: 8/23/05
		Location: Spring Hill, TN	
Boring No.: B-14	Total Depth 7.0'	Elev:	Water at Completion of Drilling: Not Encountered
Boring Method: Hollow Stem Auger	Drill Type: CME-55	Driller: RC	

Elevation (MSL)	Depth (feet)	Sample	DESCRIPTION OF MATERIALS	N	N VALUE (bpf) 10 20 30 40 50 60 70 80 90	% M	LL	PI	Qp	Qu	% RET. #200
	1.0		12"TOPSOIL								
	2.0		Dark Gray to Brown SILTY CLAY with chert pieces, mineral deposits and sand, moist, stiff. (CL-ML)	11		14					
	4.0		Weathered CHERT pieces with brown lean clay and mineral deposits.	12							
	6.5		Brown to Dark Gray Lean CLAY with mineral deposits, sand and silt, moist, stiff. (CL)	9		28	43	19			19
	7.0		Weathered LIMESTONE layers with brown lean clay and silt.	50/PR							
			Auger Refusal 7.0 Feet								

NV1 35855176.GPJ 9/16/05

Hitchin' Post Commercial Center
for
Harvey and Harvey Associates, LLC
Spring Hill, Tennessee

Appendix 3
Laboratory Testing Results

CROUCH ENGINEERING®, P.C.

428 Wilson Pike Circle, P. O. Box 1186, Brentwood, Tennessee 37024-1186
Phone: (615) 791-0630 Fax: (615) 791-8451
Website: www.crouchengineering.com

SUMMARY OF LABORATORY TEST RESULTS

						ATTERBERG LIMITS			PROCTOR					
Hole No.	Sample No.	Sample Type*	Depth (ft.)	Natural Moisture (%)	PPqu	Liquid Limit (%)	Liquid Limit (%)	Plasticity Index (%)	Max Dry Density	Optimum Water Content (%)	CBR	Percent Retained #200 Sieve	UCS	Unified Soil Classification; Notes
B-1	1	SS	0.0-1.5'	11%		39	23	16	106.3 PCF	17.4%	2			CI
B-1	2	SS	2.0'–3.5'	15%										CI
B-1	3	SS	4.0'-5.5'	43%		36	21	15				47		CL
B-1	4	SS	6.5'-8.0'	10%										ML
B-4	1	SS	0.0-1.5'	10%										CL
B-4	2	SS	2.0'–3.5'	15%										CL
B-4	3	SS	4.0'-5.5'	23%	1.5	43	17	26						CL
B-4	4	SS	6.5'-8.0'	13%	2.0							46		ML
B-4	5	SS	9.0'-10.5'	21%										CL
B-5	1	SS	0.0-1.5'	12%										CL
B-5	2	SS	2.0'–3.5'	18%	4.0									CL
B-5	3	SS	4.0'-5.5'	18%	3.5							43		CL
B-5	4	SS	6.5'-8.0'	30%	1.5	44	20	24						CL
B-6	1	SS	0.0-1.5'	12%										CL
B-6	2	SS	2.0'–3.5'	14%										CL

ST - SHELBY TUBE SAMPLE, SS - SPLIT SPOON SAMPLE, B - BAG SAMPLE

TEST RESULTS REPORTED ON OTHER SHEETS:

C - CONSOLIDATION
S - SIEVE OR GRAIN SIZE ANALYSIS
UCS - UNCONFINED COMPRESSION TEST
CBR - CALIFORNIA BEARING RATIO
P - PROCTOR TEST
D - DIRECT SHEAR TEST
T - TRIAXIAL TEST
PPqu – POCKET PENETROMETER

DATA CHECKED BY ____________

SUMMARY OF LABORATORY TEST RESULTS

						ATTERBERG LIMITS			PROCTOR					
Hole No.	Sample No.	Sample Type*	Depth (ft.)	Natural Moisture (%)	PPqu	Liquid Limit (%)	Liquid Limit (%)	Plasticity Index (%)	Max Dry Density	Optimum Water Content (%)	CBR	Percent Retained #200 Sieve	UCS	Unified Soil Classification; Notes
B-6	3	SS	4.0'-5.5'	24%	4.0									CH
B-6	4	SS	6.5'-8.0'	25%										CH
B-7	1	SS	0.0-1.5'	9%										CL
B-7	2	SS	2.0'–3.5'	11%										CL
B-7	3	SS	4.0'-5.5'	13%	4.5									CL
B-7	4	SS	6.5'-8.0'	17%										CL
B-9	1	SS	0.0-1.5'	12%										CL
B-9	2	SS	2.0'–3.5'	12%										CL
B-9	3	SS	4.0'-5.5'	26%		43	20	24						CL
B-9	4	SS	6.5'-8.0'	21%	2.0							46		CH
B-9	5	SS	9.0'-10.5'	26%										CH
B-10	1	SS	0.0-1.5'	16%	4.0									CL
B-10	2	SS	2.0'–3.5'	18%	4.0									CL
B-10	3	SS	4.0'-5.5'	22%	4.5	39	21	18						CL
B-10	4	SS	6.5'-8.0'	19%	4.5							25		CL

ST - SHELBY TUBE SAMPLE, SS - SPLIT SPOON SAMPLE, B - BAG SAMPLE

TEST RESULTS REPORTED ON OTHER SHEETS:

C - CONSOLIDATION
S - SIEVE OR GRAIN SIZE ANALYSIS
UCS - UNCONFINED COMPRESSION TEST
CBR - CALIFORNIA BEARING RATIO
P - PROCTOR TEST
D - DIRECT SHEAR TEST
T - TRIAXIAL TEST
PPqu – POCKET PENETROMETER

DATA CHECKED BY ____________

SUMMARY OF LABORATORY TEST RESULTS

						ATTERBERG LIMITS			PROCTOR					
Hole No.	Sample No.	Sample Type*	Depth (ft.)	Natural Moisture (%)	PPqu	Liquid Limit (%)	Liquid Limit (%)	Plasticity Index (%)	Max Dry Density	Optimum Water Content (%)	CBR	Percent Retained #200 Sieve	UCS	Unified Soil Classification; Notes
B-10	5	SS	9.0'-10.5'	19%										CH
B-11	1	SS	0.0-1.5'	14%		39	19	20	108.2 PCF	17.6%	2			CL
B-11	2	SS	2.0'–3.5'	17%								50		SC
B-11	3	**ST**	**4.0'-6.0'**	26%									X	CL
B-11	4	SS	6.5'-8.0'	36%		33	28	5						ML
B-12	1	SS	0.0-1.5'	15										CL
B-12	2	SS	2.0'–3.5'	13										CL
B-12	3	SS	4.0'-5.5'	25	4.25	53	23	30						CH
B-12	4	SS	6.5'-8.0'	21	4.5									CH
B-12	5	SS	9.0'-10.5'	28	1.5							14		CH
B-13	1	SS	0.0-1.5'	8										CL
B-13	2	SS	2.0'–3.5'	17	4.5									CL
B-13	3	SS	4.0'-5.5'	23										CH
B-14	1	SS	0.0-1.5'	14										CL-ML
B-14	2	SS	2.0'–3.5'	-										

ST - SHELBY TUBE SAMPLE, SS - SPLIT SPOON SAMPLE, B - BAG SAMPLE

TEST RESULTS REPORTED ON OTHER SHEETS:
C - CONSOLIDATION
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DATA CHECKED BY ______________

SUMMARY OF LABORATORY TEST RESULTS

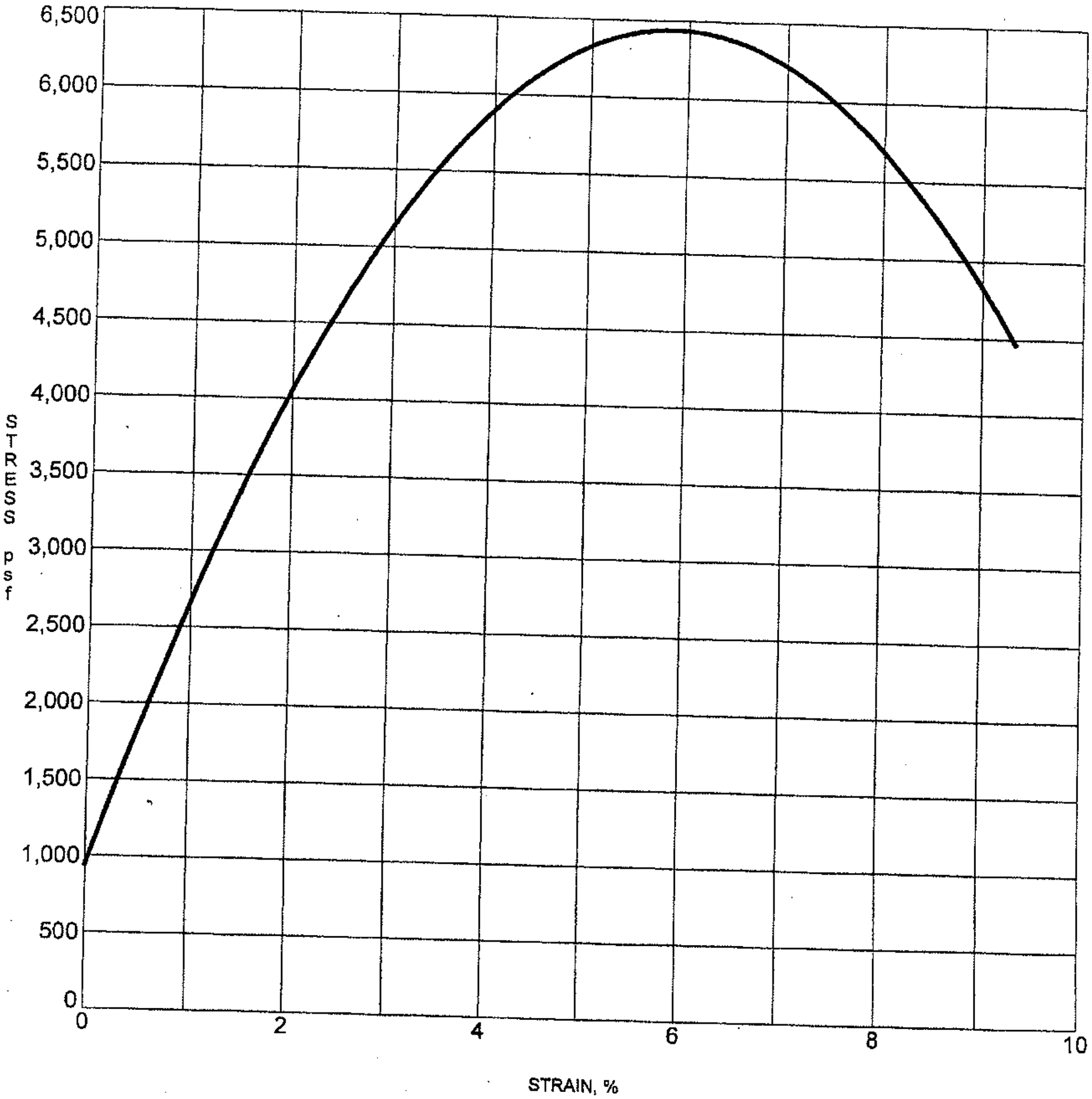
						ATTERBERG LIMITS			PROCTOR					
Hole No.	Sample No.	Sample Type*	Depth (ft.)	Natural Moisture (%)	PPqu	Liquid Limit (%)	Liquid Limit (%)	Plasticity Index (%)	Max Dry Density	Optimum Water Content (%)	CBR	Percent Retained #200 Sieve	UCS	Unified Soil Classification; Notes
B-14	3	SS	4.5'-5.5'	-		43	24	19				19		CL
B-14	4	SS	6.5'-8.0'	28										
B-13/14	1	Bulk	1.0'			38	23	15	107.9 PCF	17.7%	3			

ST - SHELBY TUBE SAMPLE, SS - SPLIT SPOON SAMPLE, B - BAG SAMPLE

TEST RESULTS REPORTED ON OTHER SHEETS:

C - CONSOLIDATION
S - SIEVE OR GRAIN SIZE ANALYSIS
UCS - UNCONFINED COMPRESSION TEST
CBR - CALIFORNIA BEARING RATIO
P - PROCTOR TEST
D - DIRECT SHEAR TEST
T - TRIAXIAL TEST
PPqu – POCKET PENETROMETER

DATA CHECKED BY ____________



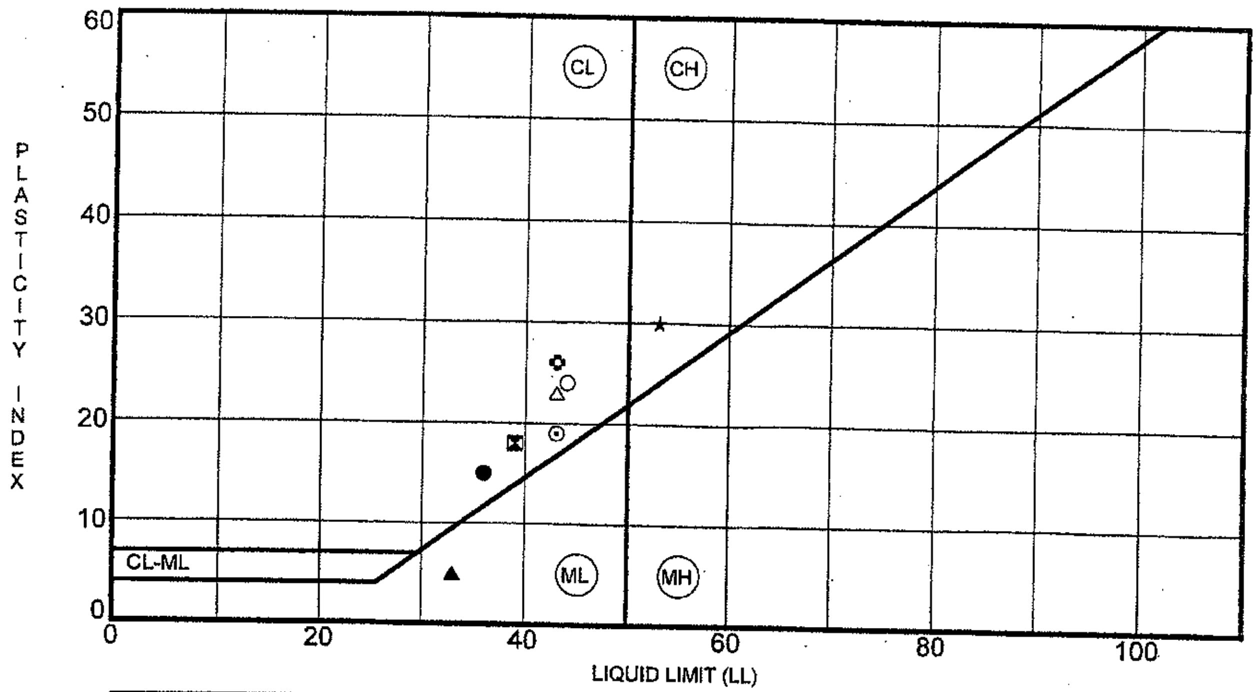
Specimen Identification		Classification	DD	MC%
B-11	5.0	**Light Brown to Brown Lean CLAY. (CL)**		26

PROJECT **Highway 31 - Spring Hill, TN**

JOB NO. **358-55176**
DATE **9/16/05**

UNCONFINED COMPRESSION TEST

PSI



	Specimen Identification		LL	PL	PI	Fines	Classification
●	B-1	5.0	36	21	15		Gray Brown Lean CLAY. (CL)
⧗	B-10	5.0	39	21	18		Light Brown to Brown Lean CLAY. (CL)
▲	B-11	7.5	33	28	5		Brown to Dark Gray Brown SILT. (ML)
★	B-12	5.0	53	23	30		Dark Yellow Brown Fat CLAY. (CH)
⊙	B-14	5.0	43	24	19		Gray to Dark Yellow Brown Lean CLAY. (CL)
✪	B-4	5.0	43	17	26		Brown to Gray Brown Lean CLAY. (CL)
○	B-5	7.5	44	20	24		Gray to Dark Yellow Brown Lean CLAY. (CL)
△	B-9	5.0	43	20	23		Gray to Dark Yellow Brown Lean CLAY. (CL)

PROJECT#: 358-55176

DATE: Sep 05

ATTERBERG LIMITS' RESULTS

Highway 31

Spring Hill, TN

psi *Information To Build On*

Job No. 358-55176 Date 9/16/05
Project Highway 31 - Spring Hill, TN

Source of Material B-11 1.0
Description of Material
Test Method 698A

TEST RESULTS

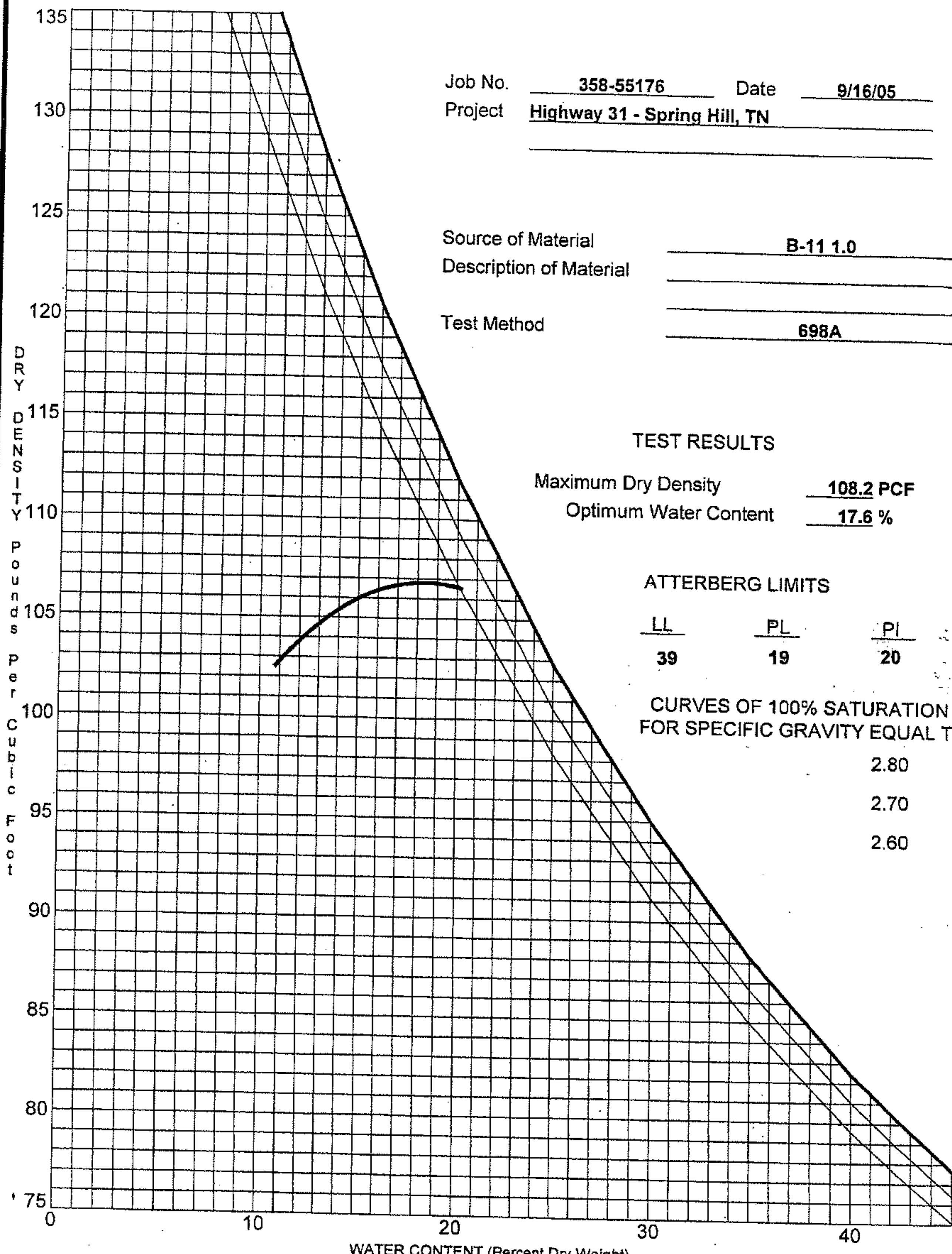
Maximum Dry Density 108.2 PCF
Optimum Water Content 17.6 %

ATTERBERG LIMITS

LL	PL	PI
39	19	20

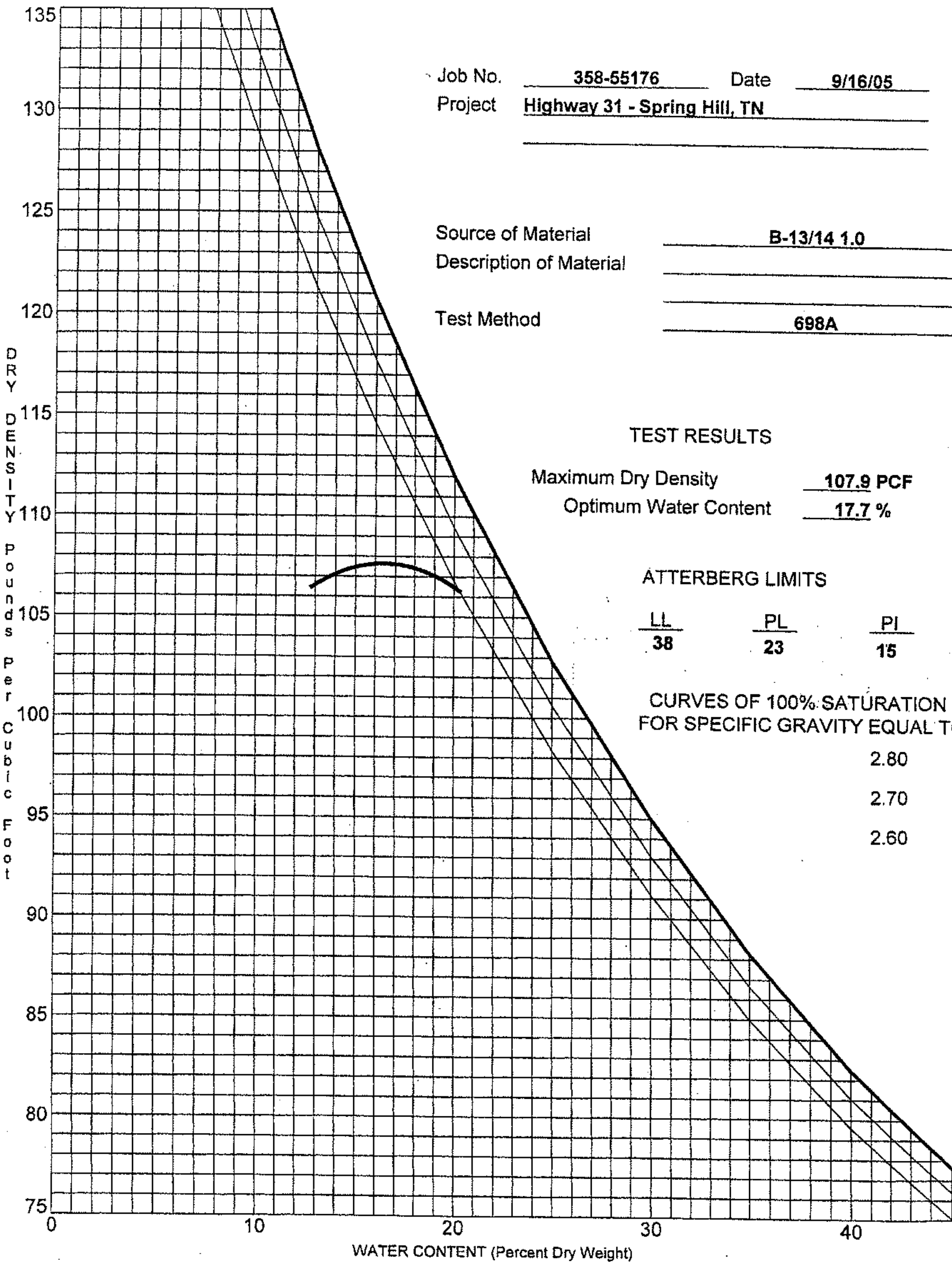
CURVES OF 100% SATURATION FOR SPECIFIC GRAVITY EQUAL TO:

2.80
2.70
2.60



MOISTURE-DENSITY RELATIONSHIP

PSI



Job No. 358-55176 Date 9/16/05

Project Highway 31 - Spring Hill, TN

Source of Material B-13/14 1.0

Description of Material

Test Method 698A

TEST RESULTS

Maximum Dry Density 107.9 PCF

Optimum Water Content 17.7 %

ATTERBERG LIMITS

LL	PL	PI
38	23	15

MOISTURE-DENSITY RELATIONSHIP

PSI

Job No. 358-55176 Date 9/16/05

Project Highway 31 - Spring Hill, TN

Source of Material B-1 1.0

Description of Material

Test Method 698A

TEST RESULTS

Maximum Dry Density 106.3 PCF

Optimum Water Content 17.5 %

ATTERBERG LIMITS

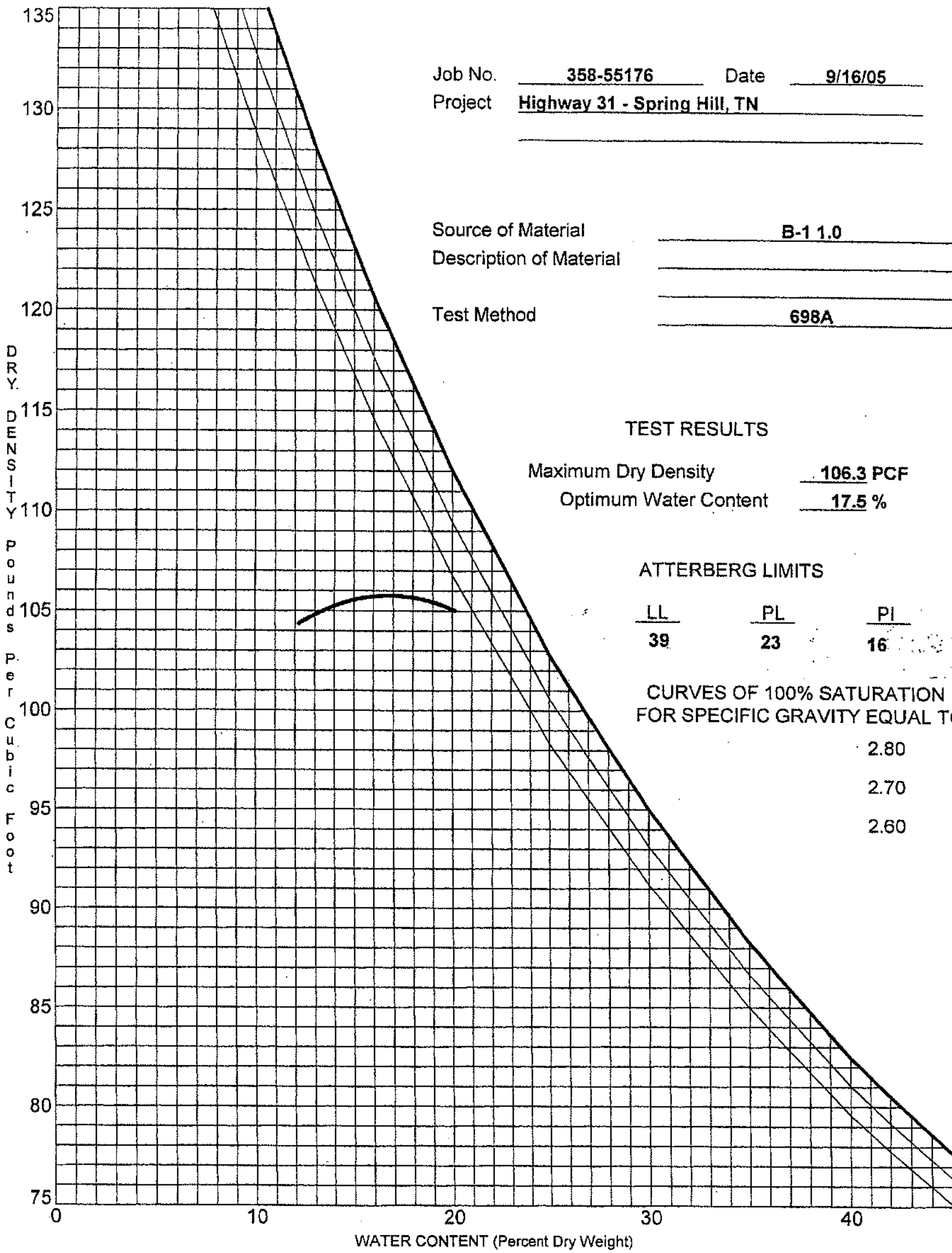
LL	PL	PI
39	23	16

CURVES OF 100% SATURATION FOR SPECIFIC GRAVITY EQUAL TO:

2.80

2.70

2.60



MOISTURE-DENSITY RELATIONSHIP

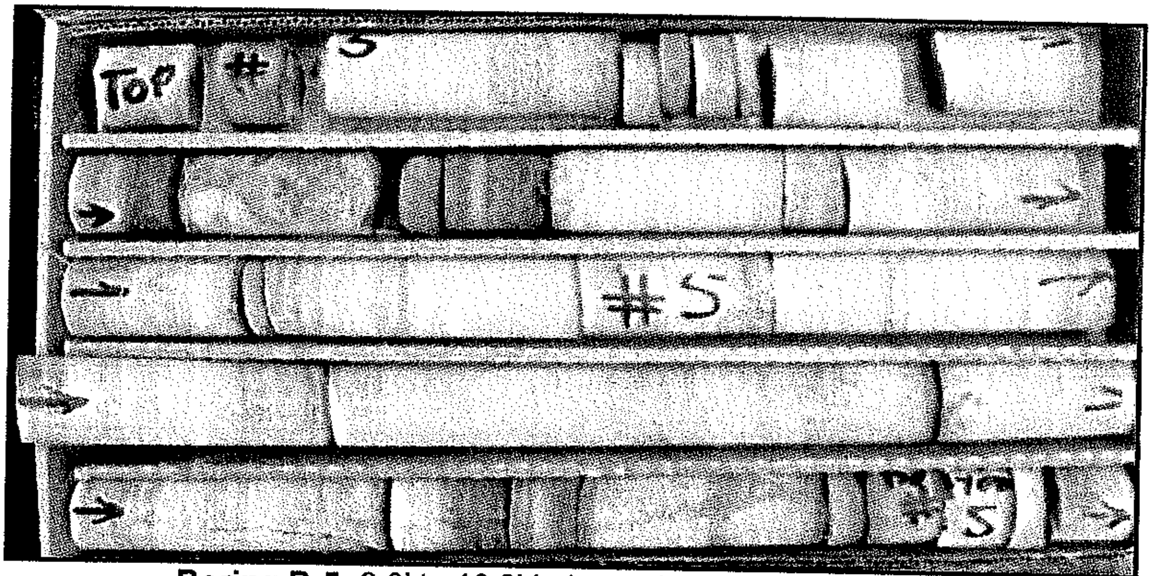
PSI

Hitchin' Post Commercial Center
for
Harvey and Harvey Associates, LLC
Spring Hill, Tennessee

Appendix 4
Rock Core Photographs

CROUCH ENGINEERING®, P.C.

428 Wilson Pike Circle, P. O. Box 1186, Brentwood, Tennessee 37024-1186
Phone: (615) 791-0630 Fax: (615) 791-8451
Website: www.crouchengineering.com



Boring B-5: 9.0' to 19.0' below existing ground surface.
Recovery = 95%; RQD = 68%.

Boring B-11: 8.0' to 18.0' below existing ground surface.
Recovery = 93%; RQD = 70%.

Hitchin' Post Commercial Center

for

Harvey and Harvey Associates, LLC

Spring Hill, Tennessee

Appendix 5
Summary of Field and Laboratory Testing Procedures

CROUCH ENGINEERING®, P.C.

428 Wilson Pike Circle, P. O. Box 1186, Brentwood, Tennessee 37024-1186
Phone: (615) 791-0630 Fax: (615) 791-8451
Website: www.crouchengineering.com

FIELD TESTING PROCEDURES

Soil Test Borings (ASTM D-1586)

Soil drilling and sampling operations were conducted in general accordance with ASTM D-1586. The soil test borings were advanced by mechanically twisting continuous hollow stem auger flights into the ground. Samples were obtained with a standard 1.4 inch I.D., 2-inch O.D., split-tube sampler. The sampler was first seated six inches to penetrate any loose cuttings, and then driven an additional foot with blows of a 140 pound hammer falling 30 inches. The number of hammer blows required to drive the sampler the final foot of penetration was recorded and designated the "Standard Penetration Test N-Value". The penetration resistance, when properly evaluated, is an index to the soil's strength, density, and behavior under applied loads.

Representative portions of the samples were placed in containers and transported to the Professional Service Laboratories, Inc. (PSI), where they were examined by an engineer to verify the driller's field classification. The soil descriptions and penetration for each soil test boring are shown on the Boring Logs in Appendix 2.

Rock Coring

The coring was performed in accordance with ASTM Specification D-2113-70. This drilling procedure consists of boring into the material with a diamond-studded bit fastened to the end of a hollow double-cored barrel. This device is rotated at high speeds and is capable of cutting hard rock. Core samples of the materials are protected and retained in the swivel-mounted inner tube. Upon completion of each drill run, the core barrel is brought to the surface and the samples removed and placed in boxes.

The samples were then returned to Professional Service Laboratories, Inc. (PSI) where the rock was identified and the "recovery" and "rock quality designation" (RQD) was determined by a geologist. The recovery is the ratio of the sample obtained to the depth drilled expressed as a percent. The RQD is the percentage of the length of the core run which has rock segments of moderately hard or harder rock four inches or greater in length, compared to the total length of the run. The percent recovery and RQD are related to rock soundness and continuity. Generalized rock descriptions, percent recover, and RQD values are shown on the Boring Logs in Appendix 2.

SUMMARY OF LABORATORY TESTING

In addition to the field exploration, a limited laboratory testing program was conducted to ascertain additional engineering characteristics of potential foundation materials. To supplement the visual classification of the soil samples, the following tests were performed.

Description of Soils (Visual-Manual Procedure) (ASTM D-2488)

The soil samples were visually examined by our engineer and soil descriptions were provided. Representative samples were then selected and tested to determine soil classification as described above. This data was used to correlate our visual descriptions with the Unified Soil Classification.

Natural Moisture Content (ASTM D-2216)

Natural moisture contents (M%) were determined on selected samples. The natural moisture content is the ratio, expressed as a percentage, of the weight of water in a given amount of soil to the weight of solid particles. The results are indicated for selected samples on the Boring Logs in Appendix 2.

Atterberg Limits

Atterberg Limits tests were performed to evaluate the soil's plasticity characteristics. The soil Plasticity Index (PI) is representative of this characteristic and is bracketed by the Liquid Limit (LL) and the Plastic Limit (PL). The Liquid Limit is the moisture content at which the soil will flow as a heavy viscous fluid. The Plastic Limit is the moisture content at which the soil is between "plastic" and the semi-soil stage. The results of these tests are presented for selected samples in the Boring Logs in Appendix 2, and Laboratory Test Results in Appendix 3. The Plasticity Index (PI = LL - PL) is a frequently used indicator for a soil's potential for volume change. Typically, volume change potential increases with higher Plasticity Indices.

Particle Analysis of Soils

Grain-size tests were performed to determine the soil particle size distribution. The distribution of particle sizes larger than 75μm (retained on the #200 sieve) is determined by sieving. The separation of particles sizes is used to classify soil samples in accordance with the Unified Soil Classification System. The results of these tests are presented for selected samples in the Boring Logs in Appendix 2.

Moisture Density Relationships, Standard Proctor (ASTM D-698)

The compaction characteristics of two representative samples of the on-site soils were determined in accordance with ASTM D-698. The compaction test determines the maximum dry density and optimum moisture content of a particular soil for a given compactive effort. The results of the tests are shown on the "Report of Moisture-Density Relationship" sheets in terms of moisture content versus dry density.

Laboratory California Bearing Ratio (CBR) TESTS (ASTM D-1883)

The California Bearing Ratio, usually abbreviated as CBR, is a punching shear test. The CBR value is a semi-empirical index of the soil's strength and deflection characteristics, and has been correlated with pavement performance to establish design curves for pavement thickness. The test was performed on a six inch diameter, five inch thick disk of compacted soil, confined in a steel cylinder. The specimens were then soaked for 96 hours prior to testing. A piston approximately two inches in diameter was then forced into the soil at a standard rate to determine the resistance to penetration. A CBR is the ratio, expressed as a percentage, of the actual load required to produce a 0.1-inch deflection to achieve the same deflection in crushed stone.

Unconfined Compressive Strength of Cohesive Soils (ASTM D-2166)

This test method determines the strength and stress-strain relationships of a cylindrical specimen of an undisturbed cohesive soil sample. Specimens are compressed to failure with load and deformation data periodically recorded to obtain a stress-strain curve. The peak of the curve may be taken as the undrained shear strength ($s_u = q_u/2$ = ***cohesion***).

Pocket Penetrometer

Pocket Penetrometer (PPqu) tests were performed on cohesive soil samples. The pocket penetrometer provides a consistency classification and an indication of the soil's unconfined compressive strength. The pocket penetrometer data are presented on the Boring Logs in Appendix 2.