Report of Geotechnical Study
Proposed Expansion of General Aviation Services
Nashville, Tennessee

Submitted to:

MNAA Planning, Design and Construction Department

Submitted by:

AMEC Earth & Environmental, Inc.
Nashville, TN

May 26, 2004

AMEC File No. 3-4990-0000-0004

May 26, 2004

Ms. Amani E. Dudley, P.E.
Metropolitan Nashville Airport Authority (MNAA)
Planning, Design and Construction Department
One Terminal Drive
Nashville, Tennessee 37214-4114

Re: Proposal for Subsurface Materials Characterization
Nashville, Tennessee
AMEC Project No. 3-4990-0000-0004

Dear Ms. Dudley:

AMEC Earth & Environmental, Inc. (AMEC) has completed the authorized study and herewith submits the data, our comments and recommendations. Our services were performed in general accordance with the terms of our April 23, 2004, proposal (No. 2004-040, Revision 1) to you. The scope of work includes general subsurface exploration and the development of geotechnical engineering recommendations for site preparation and foundation design.

AMEC appreciates this opportunity to be of service to you. At your convenience, we are available to discuss the details of this report and any questions that you may have.

Sincerely,

AMEC

J. Sam Vance, P.E.
Senior Geotechnical Engineer

Larry Sciple
Senior Geologist

Reviewed By:

Daniel Hurst
Geotechnical Unit Manager

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1.0 INTRODUCTION

AMEC has completed the authorized geotechnical study for this project and herewith submit the data and our conclusions. The scope of work was performed in general accordance with our April 23, 2004 Proposal and Agreement. The objectives of the geotechnical study are to explore the general subsurface conditions across the area of proposed construction and to provide comments and recommendations to help with your planning of future site work. We understand the gist of the exploration is to better understand overburden thickness in planned excavation areas so that grading costs, and rock removal in particular, can be more accurately estimated. Our services are geared toward defining the general character of materials that will be generated within planned excavations, and how to best implement those materials for use in structural fills. The ASFE organization has prepared important information regarding studies of the type performed, and this is attached for your review. Environmental services are beyond the scope of this investigation.

2.0 EXPLORATION AND TESTING

Twelve (12) soil borings were drilled across the site at the approximate locations and to the depths shown on the Plan, Profiles and Logs contained in Appendix 1. Our representative established the points of exploration in the field by taping distances from existing features. Each boring was power augured to the boring termination depths; soil samples were obtained at five-foot intervals in general accordance with ASTM D 1586 (*Penetration Test and Split Barrel Sampling of Soils).* Bulk samples of auger cuttings were obtained from selected borings. A member of our professional staff documented the exploration and logged the samples in the field. Soil samples were field classified with respect to material type and consistency. A hand penetrometer was used to obtain indices of relative shear strength of selected soil samples. Upon completion, each boring was checked for the presence of ground water and was then backfilled with auger cuttings. One soil boring, boring B-12, was continued into bedrock an additional five feet below refusal, to better characterize bedrock conditions.

The soil samples were transported to the AMEC Geotechnical and Construction Materials Laboratory in Nashville, Tennessee. Selected samples were subjected to index testing to assess to soils' index properties including natural moisture content, grain size analysis, moisture

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density relationship (ASTM D 698) and Atterberg limits. The laboratory testing results are summarized in Appendix 1. The individual Logs in Appendix 2 show our interpretations of the subsurface conditions and descriptions of the materials encountered.

3.0 SITE CONDITIONS

The site for the proposed project is adjacent to, and east of, Taxiway Sierra and immediately south of the Murfreesboro Road underpass. The Dell USA campus abuts the east side of the proposed construction area, and undeveloped airport property lies south of the project area. An un-named perimeter road is present between Taxiway Sierra and the site; a wide drainage ditch parralled the perimeter road. As shown on the Plan in Appendix 2, the proposed project site is occupied by a short hill whose topographic high point (approximately 602 MSL) is located just south of Murfreesboro Road. The ground slopes steeply downward from that point to the north and west; those grades were formed by excavation for the Murfreesboro Road underpass and the airport perimeter road. The excavation for the Murfreesboro Road Underpass exposes bedrock in a pre-split, vertical face. From the hilltop, the ground surface slopes gently downward to the south about 1, 000 feet to a natural valley or swale. Existing construction includes a north-south trending chain link fence. We understand, from airport personnel, that this fence is located over 200 feet west of the airport property line. The southernmost approximately 400 feet of hill slope is occupied by scattered to dense trees. Surface drainage is judged to be generally good.

4.0 SUBSURFACE CONDITIONS

Beneath topsoil ranging from approximately six inches to 18 inches, and averaging roughly 12 inches in thickness, the generalized subsurface profile at the locations explored includes residual soil that extends to the refusal surface, which is presumed to be the weathered bedrock surface. The residual soil horizon at the boring locations includes predominately stiff to very stiff, natural, variably silty, variably sandy clay, and loose to very-loose, variably silty, variably clayey sand, which was typically encountered near the refusal surface. The borings drilled for the current study refused at depths ranging from 1.5 feet (Boring B-9) to 28 feet (Boring B-6); the average depth to refusal for all borings was approximately 14 feet.

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None of the borings in this program encountered groundwater. Based on our experience in the area, we expect that the permanent water table is below the depths explored and should not impact the proposed construction, provided the grading work is performed during a dry season.

Published geological literature indicates that the site is underlain by Ordovician-age limestone of the Bigby Formation above an approximate elevation of 595 MSL, and Ordovician-Age limestone of the Hermitage Formation below that elevation. Weathering patterns in the Bigby Formation may be highly erratic, and form bedrock pinnacles with the elevation of the weathered bedrock surface varying significantly over short horizontal distances. The Hermitage Formation has thin beds of a coarse-grained limestone, similar in lithology to the Bigby Formation, at the Bigby-Hermitage contact. The coarse-grained beds of the Hermitage Formation may also weather erratically, producing pinnacles. The beds of the Hermitage Formation underlying the thin coarse-grained limestone consist of thin-bedded sandy, silty limestone separated by thin shaley beds. The Hermitage formation weathers to a variably sandy, variably silty clay or silty sand. Soils derived from the Hermitage Formation are typically thick (15 feet or more), with sand content frequently increasing with depth. Our review of the core specimens indicate the core was likely recovered from the Hermitage Formation.

5.0 DESIGN CONSIDERATIONS

According to information provided by the project team, we understand the project includes the construction of an approximately 1,000 feet long by 350 feet wide concrete-paved area that will accommodate multiple small aircraft hangers. In addition, the project will include an access road and parking area, which will be built east of the hangar area. The access road will extend eastward and connect to the Dell AM1 building entrance drive. We understand the tentative finished grade across the hangar pad is approximately 580.5 MSL. Excavation up to at least 20 feet in depth and fill in excess of 20 feet thick will be required to achieve finished subgrade elevations.

The comments and recommendations that follow are predicated upon our experience in similar geologic settings, the assumed design criteria stated above and the data obtained during this study and earlier studies performed in relative close proximity to the project site. If the assumed

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design criteria differ significantly from actual, we must have the opportunity to review our recommendations in light of the differences and offer appropriate revisions, as warranted.

6.0 COMMENTS AND RECOMMENDATIONS

6.1 Geologic Hazards

Because this site is underlain by carbonate rock there is a risk of sinkhole development within the subject property. During our site reconnaissance, we noted no visible evidence of sinkholes within the footprint of the proposed construction. In any event, we believe the potential for sinkhole development at the subject site is no greater than for other sites within this geologic setting. However, because the present state-of-the-art of geotechnical engineering does not permit accurate prediction of where or when sinkholes will occur, the Owner should realize that the possibility for post-construction sinkhole development cannot be completely eliminated and that construction on this property, or essentially any other site within this geologic setting, carries with it some risk that future sinkholes may occur. The associated risk can be reduced by careful attention to the details of site preparation presented later in this report. Site grading should be established to provide positive drainage, both during and after construction, so as to minimize the potential for future sinkhole development. During construction, the grading contractor should be alert to any indication of possible incipient sinkholes within the subsurface. Any sinkhole features encountered during the site grading, or during later stages of construction, should be repaired under the direction of the geotechnical engineer.

6.2 General

Based on our review of the data collected to date, it is our opinion that the site is generally suitable for the proposed project, and the overburden should be acceptable, with some caveats, for use as engineered soil fill. As shown by the appended Laboratory Testing Summary, select portions of the clay soils are designated CH (USC Classification); such soils may be difficult to compact if the natural moisture at time of grading varies above optimum. Further, CH soils are typically sensitive to changes in moisture (shrink/swell). For this reason, CH soils should be excluded from the upper few feet of structural fill beneath pavements and foundations. Similarly, it may be difficult to obtain adequate compaction of overly silty or phosphatic soils,

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which are common in this geologic setting, if the soil moisture varies appreciably from optimum. In particular, the soil interval just above the bedrock surface will likely be silty, sandy and phosphatic, and is likely to be problematic in terms of achieving subgrade stability or in reusing as fill.

The subsurface data suggest that rock pinnacles will be encountered above finished subgrade elevation near the north end of the proposed hangar pad, as shown by the appended Profiles. The overburden depth elsewhere supports our opinion that the rock excavation/removal requirements may decrease if the construction footprint is shifted to the south. We note that the top of rock can be highly irregular in this geologic setting and rock pinnacles could be encountered between the locations explored.

6.3 Site Preparation

Initially, all trees, topsoil, 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 should approximate 12 inches but depth of excavation to remove topsoil will exceed that value at selected locations. Stripped topsoil can be stockpiled on site and used for Landscaping purposes, or wasted off-site.

After the site is appropriately stripped, the exposed subgrade within areas to receive fill should be proofrolled with heavy, rubber-tired equipment, such as a loaded, tandem axle dump truck. Any unstable soils detected by the proofrolling activities should be undercut to firm ground and replaced with engineered fill or, if approved by the geotechnical engineer, scarified, moisture conditioned, and recompacted 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, undercutting requirements could be significant. After the subgrade has been stabilized, engineered fill can be placed upon the uniformly stable subgrade. Cut areas should be proofrolled and repaired in a similar fashion after reaching required subgrade elevations.

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Organic-free, clayey soil derived from on-site excavation may be used as engineered fill. In general, engineered soil fill should consist of low to medium plasticity (PI < 30) clay designated CL by the Unified Soil Classification System. Higher plasticity clays can be used, if restricted to the lower portions of fills for pavement and landscape areas. Proposed borrow sources should be tested and approved by the geotechnical engineer before being used as engineered fill. Engineered soil fill placed within building areas should be placed in lifts not exceeding eight inches in loose thickness and densified to at least 98% of the standard Proctor (ASTM D 698) maximum dry density. Fill placed in pavement areas should be densified to within 95% of that same index, except the upper one foot of fill, which 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 the engineered fill should be controlled to within ± 2% of the standard *Proctor* optimum moisture content.

Bedrock will be encountered during site grading, if the footprint remains as oriented on the appended Plan, and mixed rock/soil and shot rock will be generated from the cut at the hilltop at the north end of the site. Mixed rock/soil fill should not be considered as select fill but may be restricted to initial lifts within deep fills in pavement areas, only. Details for mixed fill use regarding particle size, lift thickness, compactive equipment and effort, etc., will be provided in the field by the geotechnical engineer based on the project needs at specific locations, and based on visual review of the mixed fill material. We do not expect that a significant amount of clean shot rock fill will be produced, based on the proposed limits of construction and the relatively low number of borings used for this study. However, if produced by systematic, controlled blasting, shot rock fill will be acceptable for use as engineered fill. Engineered shot rock fill should consist of hard, durable limestone fragments. The material should include well-graded particles ranging in size from a maximum of18 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.

Select portions of the exposed sugrade near the north end of the project may expose dissimilar materials at the finished cut elevation. We anticipate that soil should comprise much of the cut subgrade. If encountered, rock pinnacles should be removed to at least two feet below finished

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subgrade, and the undercut area replaced with engineered soil fill. In addition, highly sandy soil may be encountered at/near the rock surface in the deeper cuts. As discussed earlier, those soil types can be problematic and it may be more efficient to undercut and replace those soils with select fill.

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 necessitating overexcavation 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 perched ground water will be encountered in some excavations and near the bedrock surface during construction especially following periods of wet weather. 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. Therefore, the work should be scheduled to coincide with dryer periods of the construction season, if possible.

7.0 CONSTRUCTION QUALITY CONTROL

Grading at this site could be difficult if the sitework proceeds during wet seasons. The satisfactory, long-term performance of the construction will be highly dependent upon the adherence to the recommendations presented herein and to the project specifications. We strongly recommend that a qualified soils technician, responsible to the geotechnical engineer and the Owner, monitor all aspects of site grading. In particular, the proofrolling operations and any remedial treatment of the subgrade, as well as placement and compaction of engineered fill, should be continuously observed and periodically tested. Prior to concrete being placed, the geotechnical engineer or his representative should review the integrity of all foundation bearing surfaces. The recommendations contained in this report assume that we will be retained to provide construction monitoring services so that we can confirm that the subsurface conditions are as anticipated, or adjust our recommendations accordingly. Our forces will be available to further assist you by providing these and other normally specified quality control services.

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8.0 CLOSURE

AMEC appreciates this opportunity to be of continuing service to MNAA. At your convenience, we are available to discuss the details of this report and any questions you may have.

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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

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APPENDIX 1

SUMMARY OF LABORATORY TEST RESULTS

AMEC GEOTECHNICAL AND CONSTRUCTION MATERIALS LABORATORY

5211 Linbar Drive, Suite 513, Nashville, Tennessee 37211

Telephone: 615/831-9202 Fax: 615/831-9516

SUMMARY OF LABORATORY TEST RESULTS

PROJECT: MNAA General Aviation Expansion
PROJECT NO.: 3-4990-0000-0004
DATE: May 21, 2004

					PROCTOR		Atterberg Limits				
Boring No.	Sample No.	Sample Type *	Depth (Ft.)	Natural Moisture (%)	MOIST. (%)	Dry DENSITY (PCF)	Liquid Limit	Plasticity Index	Unified Soil Classification	Other Test **	SOIL DESCRIPTION
COMP-1	S-1	B	3.5-5.0	26.9	21.2	102.9	42	21	CL	S	CLAY, silty, brown to reddish brown.
B-1	1	SS	3.5-5.0	27.0						PP=3.5	CLAY, silty, brown w/trace gravel.
B-2	1	SS	3.5-5.0	26.5			46	24	CL	PP=2.0	CLAY, silty, mottled brown and tan w/trace gravel .
B-3	1	SS	3.5-5.0	37.0						PP=3.0	CLAY. silty, brown and tan, w/ some gravel.
B-5	1	SS	3.5-5.0	31.5						PP=3.5	CLAY, silty, brown, w/trace gravel.
B-6	1	SS	3.5-5.0	30.0			46	20	CL	PP=3.5	CLAY, silty, brown.
B-6	2	SS	8.5-10.0	35.2						PP=2.0	CLAY, silty, tan.
B-8	1	SS	3.5-5.0	36.5			67	35	CH	PP=2.0	CLAY, silty, brown.
B-10	1	SS	3.5-5.0	24.9						PP=3.5	CLAY, silty, brown.
B-10	2	SS	8.5-10.0	30.9						PP=3.5	CLAY. Silty, tan and brown.

* ST-SHELBY TUBE, SS-SPLIT SPOON / SPLIT-BARREL SAMPLER, B-BAG / BULK, C-CORE

**C- Consolidation Test
S-Sieve or Grain Size Analysis
U-Unconfined Compression Test
R-Relative Density
SL-Shrinkage Limits

P-Permeability
D-Direct Shear
T-Triaxial Compression Test
CBR-California Bearing Ratio
H-Hydrometer

G_s-Specific Gravity
PP-Pocket Penetrometer Reading (TSF)

DATA CHECKED BY LS

amec

AMEC GEOTECHNICAL AND CONSTRUCTION MATERIALS LABORATORY

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Telephone: 615/831-9202 Fax: 615/831-9516

MOISTURE DETERMINATIONS

CLIENT: METRO NASHVILLE AIRPORT AUTHORITY **PROJECT NO.:** 3-4990-0000-0004

PROJECT NAME: GENERAL AVIATION EXPANSION **DATE:** May 20, 2004

DATE SAMPLE(S) RECEIVED: May 12, 2004

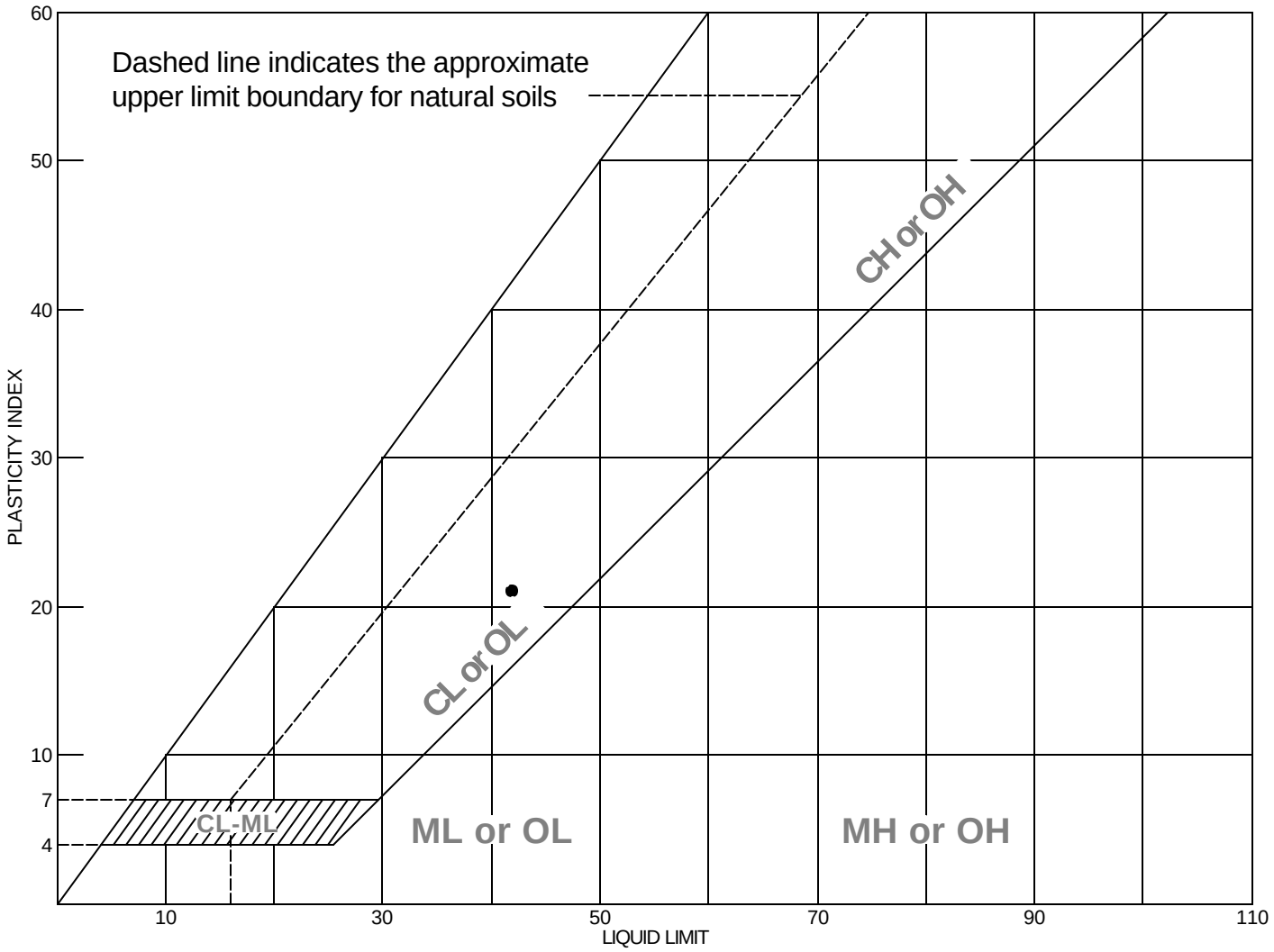
Hole No.	COMP-1	B-1	B-2	B-3	B-5	B-6	B-6	B-8	B-10
Sample No.	S-1	3.5-5.0	3.5-5.0	3.5-5.0	3.5-5.0	3.5-5.0	8.5-10.0	3.5-5.0	3.5-5.0
Container No.	16	21	13	22	23	3	24	D	25
Weight of Container	0.00	0.42	0.00	0.42	0.42	0.00	0.42	0.00	0.42
Container + Wet Soil	730.50	76.52	732.15	84.01	100.21	425.77	74.85	811.97	95.69
Container + Dry Soil	575.59	60.33	578.92	61.42	76.29	327.39	55.47	594.76	76.71
Weight of Water	154.91	16.19	153.23	22.59	23.92	98.38	19.38	217.21	18.98
Container + Dry Soil	575.59	60.33	578.92	61.42	76.29	327.39	55.47	594.76	76.71
Weight of Dry Soil	575.59	59.91	578.92	61.00	75.87	327.39	55.05	594.76	76.29
Percent Water	26.9%	27.0%	26.5%	37.0%	31.5%	30.0%	35.2%	36.5%	24.9%

Hole No.	B-10								
Sample No.	8.5-10.0								
Container No.	26								
Weight of Container	0.42								
Container + Wet Soil	90.44								
Container + Dry Soil	69.20								
Weight of Water	21.24								
Container + Dry Soil	69.20								
Weight of Dry Soil	68.78								
Percent Water	30.9%								

NOTE: Test results shown were derived from tests performed in accordance with the applicable test method(s), unless otherwise noted

JAYE RICHARDSON
LABORATORY SUPERVISOR

LIQUID AND PLASTIC LIMITS TEST REPORT



	MATERIAL DESCRIPTION	LL	PL	PI	%<#40	%<#200	USCS
●	CLAY, silty, brown to reddish brown	42	21	21	95.4	78.3	CL

Project No. 3-4990-0000- **Client:** METRO NASHVILLE AIRPORT AUTHORITY

Project: GENERAL AVIATION EXPANSION

● **Source:** COMP 1 **Elev./Depth:** COMPOSITE

Remarks:

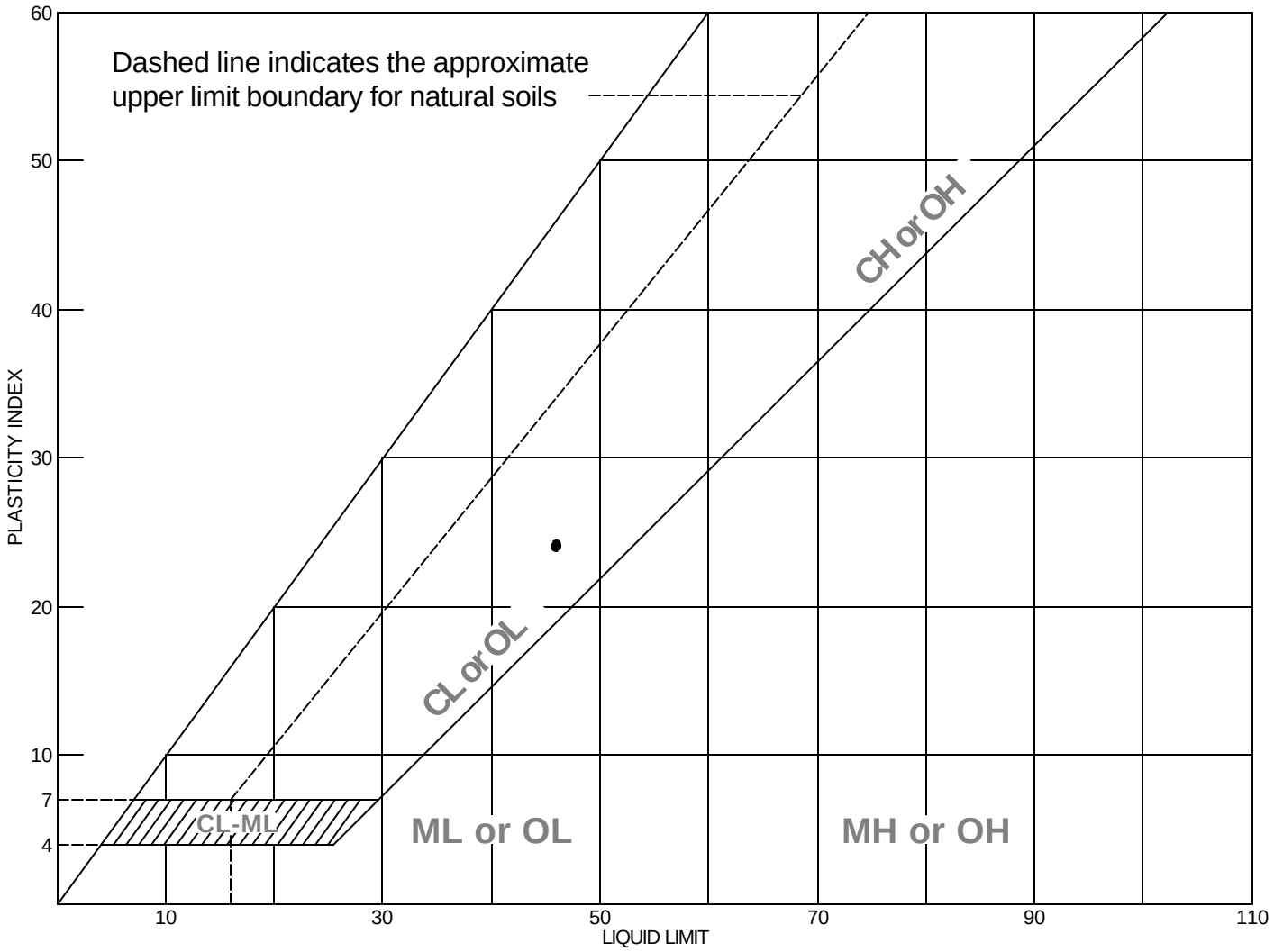
●

LIQUID AND PLASTIC LIMITS TEST REPORT

AMEC GEOTECHNICAL AND CONSTRUCTION MATERIALS LABORATORY

Figure no. COMP-1

LIQUID AND PLASTIC LIMITS TEST REPORT



	MATERIAL DESCRIPTION	LL	PL	PI	%<#40	%<#200	USCS
●	CLAY, silty, mottled brown and tan w/ trace gravel	46	22	24			

Project No. 3-4990-0000- **Client:** METRO NASHVILLE AIRPORT AUTHORITY

Project: GENERAL AVIATION EXPANSION

● **Source:** BORING 2 **Elev./Depth:** 3.5'-5.0'

Remarks:

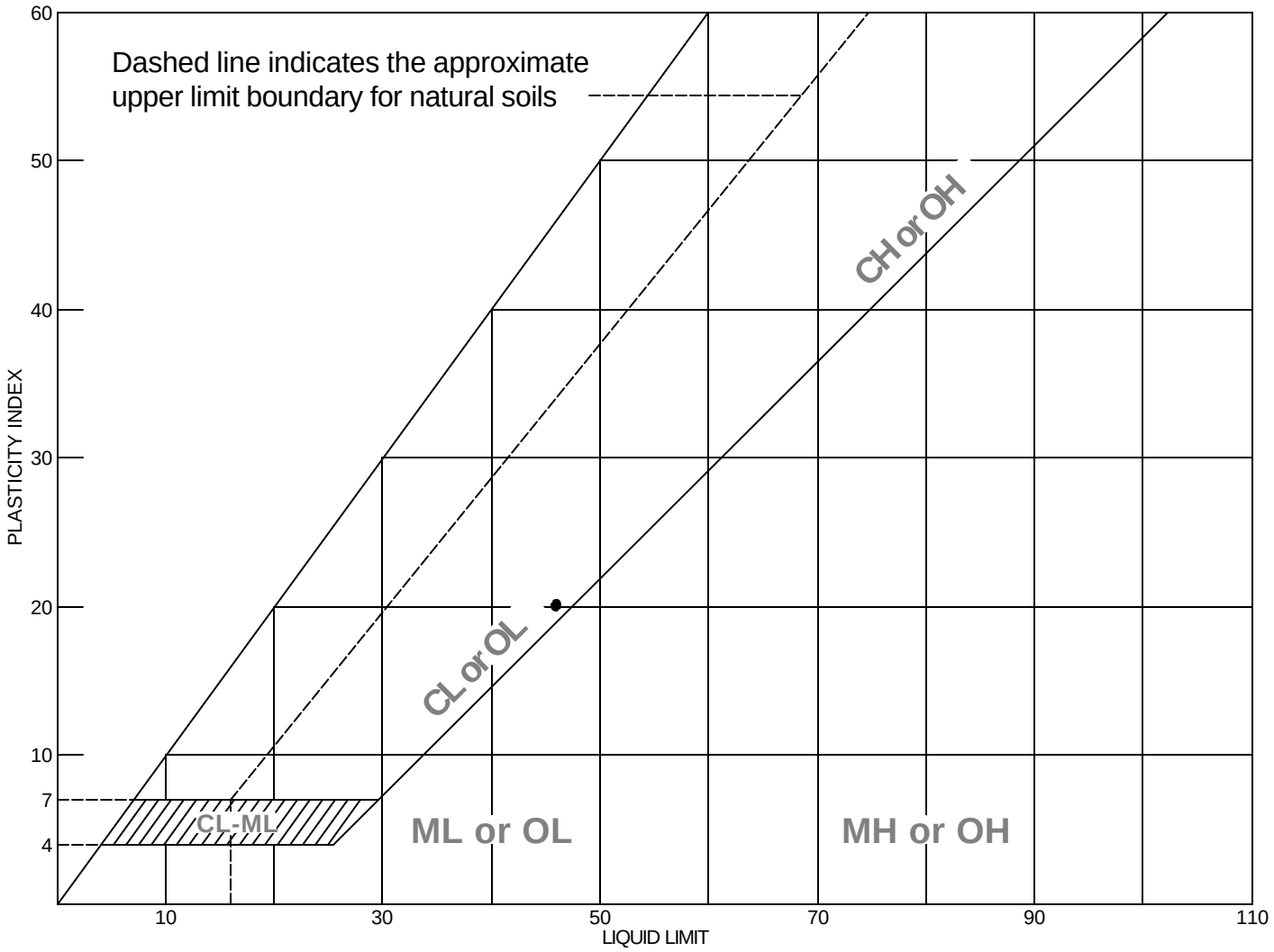
● Casagrande Classification of "CL"

LIQUID AND PLASTIC LIMITS TEST REPORT

AMEC GEOTECHNICAL AND CONSTRUCTION MATERIALS LABORATORY

Figure no. B-2

LIQUID AND PLASTIC LIMITS TEST REPORT



	MATERIAL DESCRIPTION	LL	PL	PI	%<#40	%<#200	USCS
●	CLAY, silty, brown	46	26	20			

Project No. 3-4990-0000- **Client:** METRO NASHVILLE AIRPORT AUTHORITY

Project: GENERAL AVIATION EXPANSION

● **Source:** BORING 6 **Elev./Depth:** 3.5'-5.0'

Remarks:

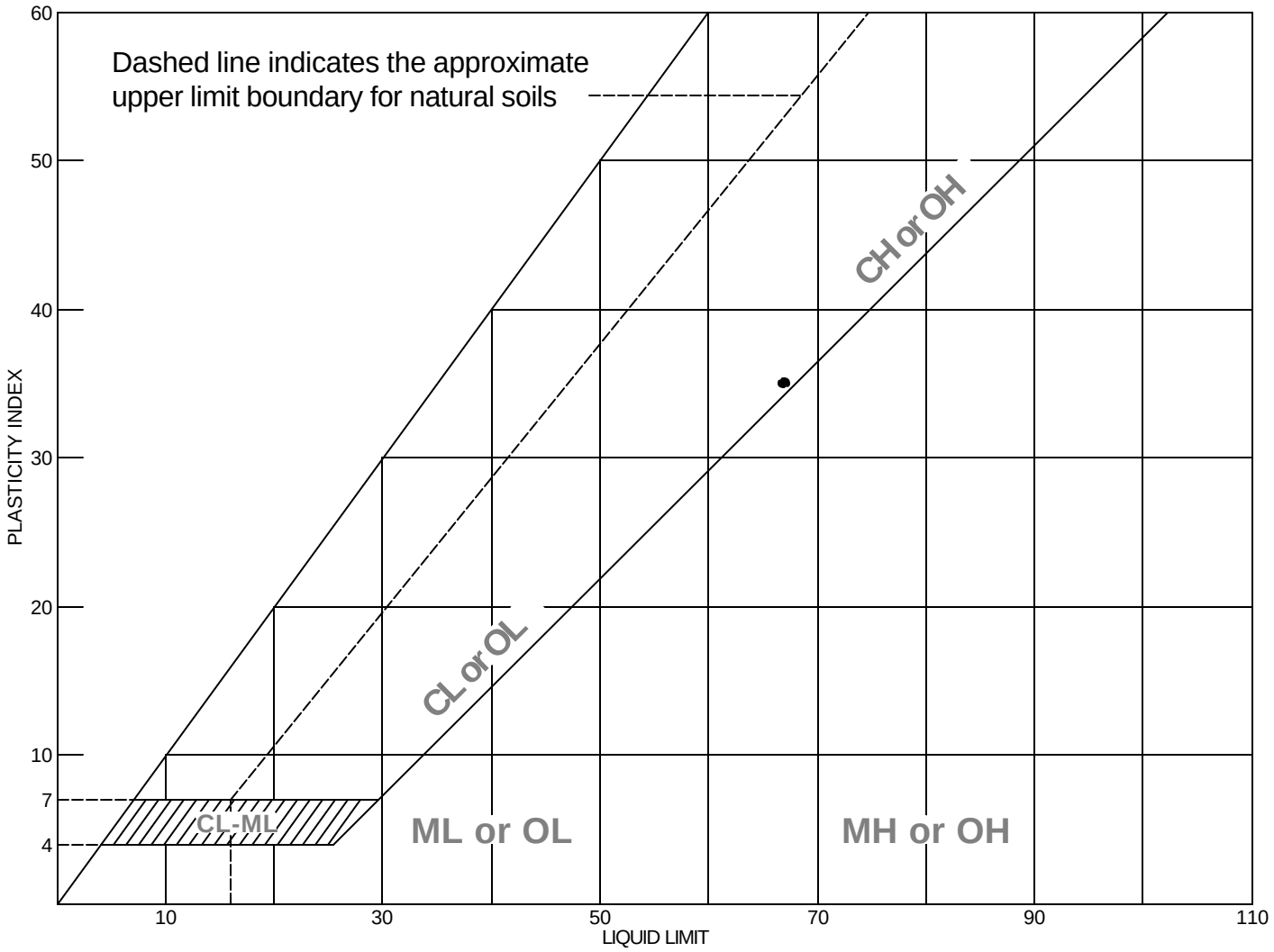
● Casagrande Classification of "CL"

LIQUID AND PLASTIC LIMITS TEST REPORT

AMEC GEOTECHNICAL AND CONSTRUCTION MATERIALS LABORATORY

Figure no. B-6

LIQUID AND PLASTIC LIMITS TEST REPORT



	MATERIAL DESCRIPTION	LL	PL	PI	%<#40	%<#200	USCS
●	CLAY, silty, brown	67	32	35			

Project No. 3-4990-0000- **Client:** METRO NASHVILLE AIRPORT AUTHORITY

Project: GENERAL AVIATION EXPANSION

● **Source:** BORING 8 **Elev./Depth:** 3.5'-5.0'

Remarks:

● Casagrande Classification of "CH"

LIQUID AND PLASTIC LIMITS TEST REPORT

AMEC GEOTECHNICAL AND CONSTRUCTION MATERIALS LABORATORY

Figure no. B-8

Particle Size Distribution Report



	% + 3"	% GRAVEL		% SAND			% FINES	
		CRS.	FINE	CRS.	MEDIUM	FINE	SILT	CLAY
○	0.0	0.0	0.7	0.6	3.3	17.1	78.3	

	LL	PL	D_{85}	D_{60}	D_{50}	D_{30}	D_{15}	D_{10}	C_c	C_u
○	42	21	0.133							

MATERIAL DESCRIPTION	USCS	AASHTO
○ CLAY, silty, brown to reddish brown	CL	A-7-6(16)

Project No. 3-4990-0000- **Client:** METRO NASHVILLE AIRPORT AUTHORITY
Project: GENERAL AVIATION EXPANSION

○ **Source:** COMP 1 **Elev./Depth:** COMPOSITE

Remarks:
○

Particle Size Distribution Report
AMEC GEOTECHNICAL AND CONSTRUCTION MATERIALS LABORATORY

Figure no. COMP-1

APPENDIX 2

PLANS, PROFILES AND BORING LOGS

Relocated
Plane Ports (7)
42 Units Total

ADG 1 Ramp

Access Road

Runway 2C/20C

Taxiway Sierra

Connector
Taxiway

Automobile Parking
50 Spaces

Nested 12 Unit
T Hangars (47' x 41')
24 Units Total

Dell Computer Campus

Relocated
Perimeter Road

B-8 (15.0/586.0)
B-12 (16.0/585.0)
B-11 (13.0/580.0)
B-9 (1.5/590.0)
B-123 (17.5/577.5)
B-7 (24.0/571.5)
B-122 (20.0/575.0)
B-122A (31.0/564.0)
B-10 (14.0/581.0)
B-123A (17.5/578.0)
B-121 (26.3/559.7)
B-6 (28.0/558.3)
B-1 (11.3/561.7)
B-105 (12.4/575.6)
B-106 (13.2/568.8)
B-5 (10.0/564.3)
B-115 (21.7/553.3)
B-4 (6.8/566.3)
B-116 (9.7/558.3)
B-2 (12.0/542.0)
B-3 (10.0/557.5)

GRAPHIC SCALE
100' 0 100' 200'

NOTES

1. PLAN ADAPTED FROM DRAWING PROVIDED BY MNAA.
2. BORING LOCATIONS AND LAYOUT BY MNAA & AMEC.
3. BORINGS DRILLED WEEK OF MAY 4, 2004.
4. BORINGS DESIGNATED B-105 THROUGH B-123A WERE TAKEN FROM OGDEN TECHNICAL REPORT 9-4267-0300, AM-1 BUILDING, DATED 11/11/99.

LEGEND

- AUGERED HOLE
- DRIVE-SAMPLED HOLE
- AUGERED & CORED HOLE

(13.0/580.0) DEPTH TO REFUSAL/APPROXIMATE REFUSAL ELEVATION

NO.	REVISIONS:	BY	DATE

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GEOTECHNICAL STUDY
PROPOSED EXPANSION OF GENERAL AVIATION SERVICES
NASHVILLE, TENNESSEE

PREPARED FOR:
MNAA PLANNING, DESIGN
AND CONSTRUCTION DEPARTMENT

3900 EZELL ROAD, SUITE 100
NASHVILLE, TENNESSEE 37211
(615) 333-0630 fax (615) 781-0655

amec

SCALE:
GRAPHIC

PLAN

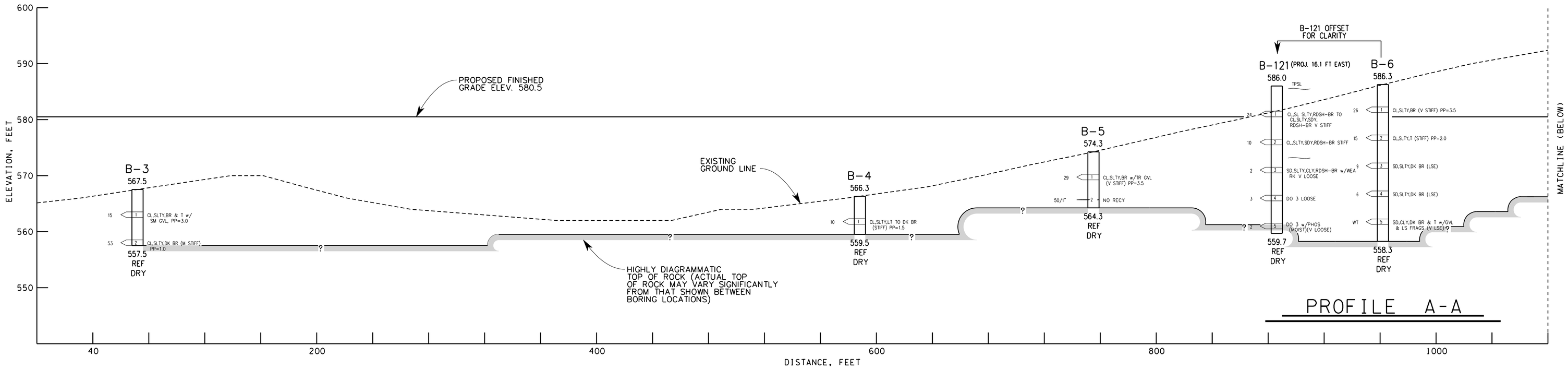
FILE:
3-4990-0000-0004

DR	CHK	REV
JMR		

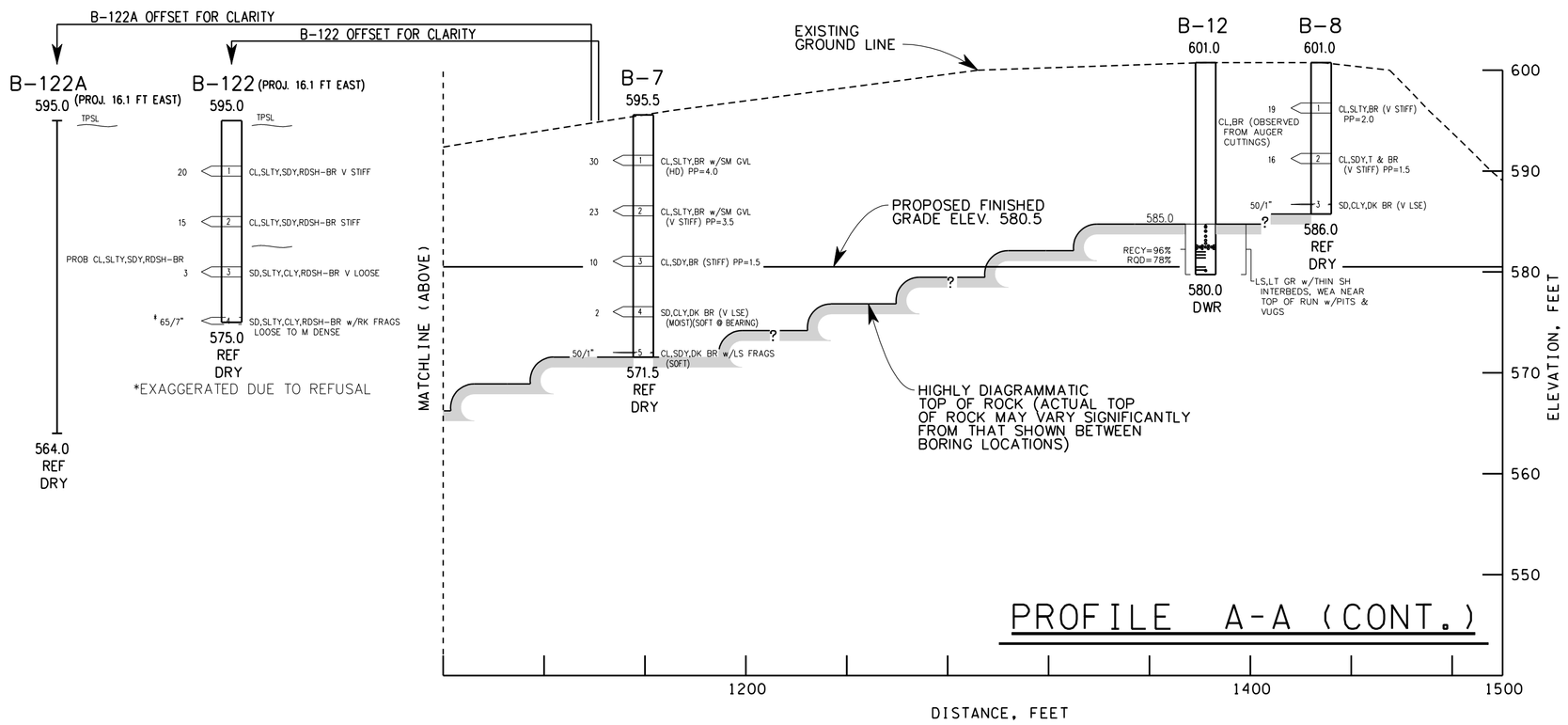
DATE:
5/26/04

SHEET
1 OF 3

GEOPLAN.DGN



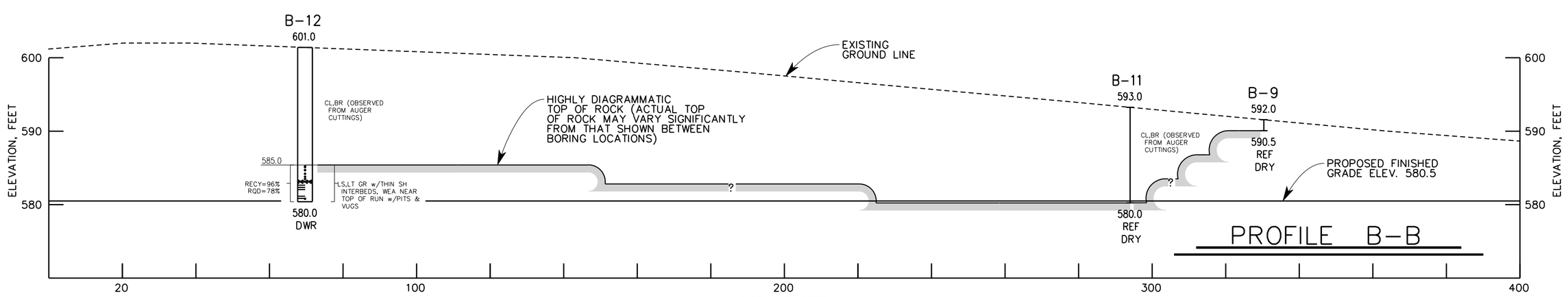
NOTE: VERTICAL EXAGGERATION ×4



NOTE: VERTICAL EXAGGERATION ×4

GENERALIZED BEDROCK DESCRIPTION

(Oh) THIN TO MEDIUM BEDDED, SILICEOUS LIMESTONE LIGHT GRAY WHEN FRESH, WEATHERS TO LIGHT BROWN
HERMITAGE FORMATION



NOTE: VERTICAL EXAGGERATION ×2

SEE SHEET 3 OF 3 FOR LEGEND AND ABBREVIATIONS

PROFILES.DGN

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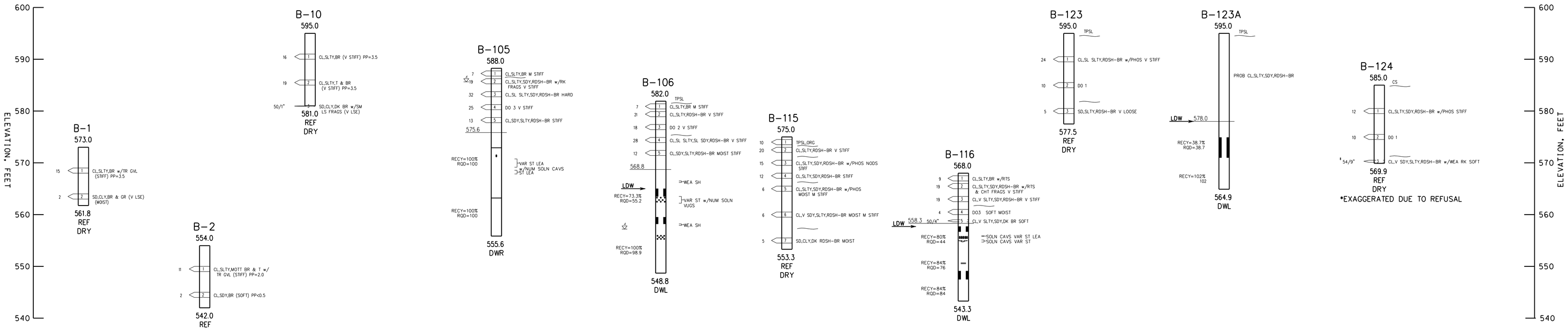
PROFILES

FILE: 3-4990-0000-0004

DR	CHK	REV
JMR		

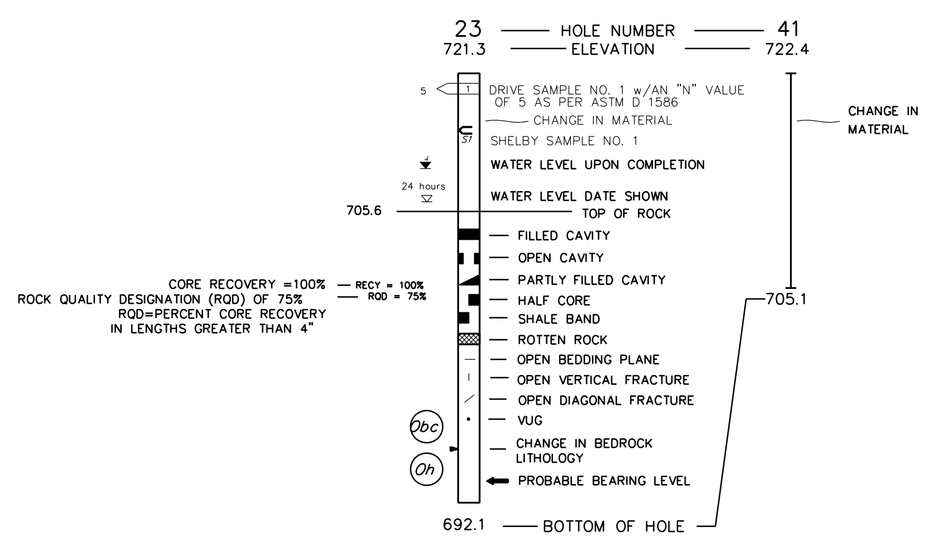
DATE: 5/26/04

SHEET 2 OF 3



LOGS OF BORINGS NOT SHOWN IN PROFILE

LEGEND



SYMBOLS REFER TO INDIVIDUAL BEDROCK WEAKNESSES UNLESS OTHERWISE NOTED.

BEDROCK IS UNWEATHERED UNLESS OTHERWISE NOTED.

ABBREVIATIONS

ABUND –ABUNDANT
APPROX–APPROXIMATE
ARG –ARGILLACEOUS
ARTH –ARTHROPOD
AX –CORE DRILL SIZE
BD/S –BAND/S
BENT –BENTONITE
BGS –BELOW GROUND SURFACE
BL –BLUE
BLDR/S–BOULDER/S
BLK –BLACK
BLSH –BLUISH
BP/S –BEDDING PLANE/S
BR –BROWN
BRCC –BRECCIATED
BRK/S –BREC/A
BRNSH –BROWNISH
BX/BO –CORE DRILL SIZE
C –COARSE
CALC –CALCAREOUS
CAV/Y –CAVITY/CAVITIES
CBLS –COBBLES
CEM –CEMEN/CEMENTED
CHT/Y –CHERT/CHERTY
CL/Y –CLAY/CLAYEY
CNDS –CINDERS
CONC –CONCRETE
CS –CRUSHED STONE
CSG –CASING
D –DENSE/DENSELY
DEB –DEBRIS
DEC –DECOMPOSED
DF/S –DIAGONAL FRACTURE/S
DGA –DENSE GRADED AGGREGATE
DISPL –DISPLACEMENT
DK –DARK
DLLG –DRILLING
DO –DITTO
DOL –DOLOMITE
DWR –DRILL WATER RETAINED
ELEV –ELEVATION
EST –ESTIMATED
F –FINE/LY
FRM –FIRM
FE –FERROUS
FM –FORMATION
FOSS –FOSSILIFEROUS
FRAC/S–FRACTURE/S
FRAG/S–FRAGMENT/S
GD –GROUND
GPM –GALLONS PER MINUTE
GR –GRAY
GRN –GREEN
GRND –GROUND
GRNSH –GREENISH
GRSH –GRAYISH
GVL/Y –GRAVEL/LY
GYP –GYPSUM
HD –HARD
HYCB –HONEYCOMBED
I –INITIAL WATER LEVEL DURING DRILLING
JTD –JOINTED
K'SF –KIPS PER SQUARE FOOT
LAM –LAMINATED
LOW –LOST DRILL WATER
LEA –LEACHED
LG –LARGE
LGT –LIGHT
LL –LIQUID LIMIT
LS –LIMESTONE
LSE –LOOSE
L'TL –LITTLE
LYS –LAYERS
LZ –LENSES
M –MEDIUM
MATL –MATERIAL
MAX –MAXIMUM
MIC –MICACEOUS
MIN –MINIMUM
MNL –MINERAL
MOD –MODERATELY
MOTT –MOTTLED
NX –CORE DRILL SIZE
NODS –NODULES
NUM –NUMEROUS
OCC –OCCASIONAL/OCCASIONALLY
ORG –ORGANIC
OSBP/S –OPEN,STAINED BEDDING PLANE/S
OSDF/S –OPEN,STAINED DIAGONAL FRACTURE/S
OSLBP/S –OPEN,STAINED/LEACHED BEDDING PLANE/S
OSLVF/S –OPEN,STAINED/LEACHED VERTICAL FRACTURE/S
OSVF/S –OPEN,STAINED VERTICAL FRACTURE/S
OVB –OVERBURDEN
PF –PARTLY FILLED
PHOS –PHOSPHATIC
PI –PLASTICITY INDEX
POSS –POSSIBLE/POSSIBLY
PL –PLASTIC LIMIT
PP –POCKET PENETROMETER (TSF)
PROB –PROBABLE/PROBABLY
PROJ –PROJECTED
PTG –PARTING
PVD –PAVED
PTD –PITTED
PP –PETROMETER
RDSH –REDDISH
RECY –RECOVERY
REF –REFUSAL
RK –ROCK
RTN –ROTTEN
RTS –ROOTS
SAT –SATURATED
SCATT –SCATTERED
SD/Y –SAND/Y
SVRLY –SEVERELY
SEV –SEVERAL
SH/Y –SHALE/Y
SL –SLIGHTLY
SLSD –SLICKENSIDED
SLT/Y –SILT/SILTY
SM –SOME
SML –SMALL
SOFT –SOFT
SOFN –SOFTENED
SOLN –SOLUTION
SPLE –SAMPLE
SS –SANDSTONE
ST –STAINED
STA –STATION
STKS –STREAKS
STRAT–STRATIFIED
SURF –SURFACE
T –TAN
TOR –TOP OF ROCK
TR/S –TRACE/STRACES
TSF –TONS PER SQUARE FOOT
USC –UNIFIED SOIL CLASSIFICATION
V –VERY
VAR –VARIABLE/VARIABLY
VEG –VEGETATION
VF/S –VERTICAL FRACTURE/S
W –'N NATURAL MOISTURE CONTENT
w/ –WITH
WB –WASHBOARD
WD –WOOD
WEA –WEATHERED
WHT –WHITE
WL –WATER
WTR –WATER
XCPT –EXCEPT
XLN –CRYSTALLINE
YEL –YELLOW
YELSH –YELLOWISH
ZN/S –ZONE/S
γd –DRY UNIT WEIGHT IN PCF
qu –UNCONFINED COMPRESSIVE STRENGTH IN KSF

THE BORING LOGS SHOWN DEPICT SUBSURFACE CONDITIONS ONLY AT THE SPECIFIC LOCATIONS DRILLED AND AT THE PARTICULAR TIMES WHICH MAY BE DESIGNATED ON THE LOGS. SUBSURFACE CONDITIONS AT OTHER LOCATIONS MAY DIFFER FROM CONDITIONS OCCURRING AT THESE BORING LOCATIONS. ALSO, THE PASSAGE OF TIME MAY RESULT IN A CHANGE IN THE SUBSURFACE CONDITIONS AT THE BORING LOCATIONS DRILLED.

NO.	REVISIONS:	BY	DATE

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GEOTECHNICAL STUDY
PROPOSED EXPANSION OF GENERAL AVIATION SERVICES
NASHVILLE, TENNESSEE

PREPARED FOR:
MNAA PLANNING, DESIGN AND CONSTRUCTION DEPARTMENT

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NASHVILLE, TENNESSEE 37211
(615) 333-0630 fax (615) 781-0655

SCALE: AS SHOWN

BORING LOGS

FILE: 3-4990-0000-0004

DR	CHK	REV
JMR		

DATE: 5/26/04

SHEET 3 OF 3

PROFILES.DGN



REPORT OF GEOTECHNICAL EXPLORATION

General Aviation Area South of Murfreesboro Road

Nashville, Tennessee

Prepared For:

RW Armstrong & Associates, Inc.
1600 Westgate Circle, Suite 225
Brentwood, Tennessee 37027

Prepared By:

K. S. Ware and Associates, L.L.C.
54 Lindsley Avenue
Nashville, Tennessee 37210

KSWA Project No. 100-06-0008

August 14, 2007

August 13, 2007

K.S. Ware & Associates, L.L.C.
Engineering, Environmental & Information Services
54 Lindsley Avenue
Nashville, Tennessee 37210
te: 615-255-9702
fx: 615-256-5873

Ms. Kim McClintock, P.E.
RW Armstrong & Associates, Inc.
1600 Westgate Circle, Suite 225
Brentwood, Tennessee 37027

Subject: **Report of Geotechnical Exploration**
General Aviation Area South of Murfreesboro Road
Nashville, Tennessee
KSWA Project No. 100-06-0008

Dear Ms. McClintock:

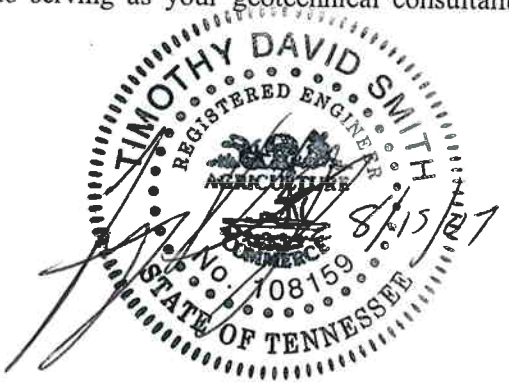
K. S. Ware and Associates, L.L.C. (KSWA) is pleased to submit this report which details the results of our geotechnical exploration for the referenced project. Our services were provided in accordance with our revised Proposal No. 100-06-0008 dated June 11, 2007.

The attached report reviews the project information provided to us, describes the site and subsurface conditions encountered, and details our geotechnical recommendations for the project. The Appendices contain a description of our field-testing procedures, and our field and laboratory test results.

We appreciate this opportunity to be of service to you on this project. Please contact us if you have any questions regarding this report. We look forward to serving as your geotechnical consultant on the remainder of this project and on future projects.

Sincerely,

K. S. Ware and Associates, L.L.C.



Mark Luskin, P.G., E.I.
Project Manager

Timothy D. Smith, P.E.
Senior Project Engineer

Enclosures: Report of Geotechnical Exploration
Distribution: Addressee (2)
File (1)

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1.0 INTRODUCTION

1.1 PROJECT INFORMATION

Project information was provided in the form of RFP #05-110 provided by The Metropolitan Government of Nashville and Davidson County and a Geotechnical Study of the proposed General Aviation Service by AMEC, dated May 26, 2004. We understand that the project includes the possible development of hangars and a general aviation aircraft apron for airplane design groups 1 and 2 at (BNA) Nashville, Tennessee. An access road will connect this area to Murfreesboro road.

The proposed development area will be east of Taxiway Sierra and immediately south of the Murfreesboro Road underpass, adjacent to Dell Campus. The AMEC report indicates that the proposed finished grade will be 580.5 feet above MSL. Based on this proposed grade, cuts and fills of up to 25 feet would be required.

1.2 PURPOSE AND SCOPE OF EXPLORATION

The geotechnical exploration included four borings, each to a depth of ten feet or auger refusal. The borings were spaced to effectively represent the subsurface conditions along the access road easement of the proposed development area. The borings were located in the field by a KSWA representative using known landmarks. Split spoon samples were collected from each boring. Laboratory testing of the material included natural moisture contents, Atterberg limits tests, and hand penetrometer tests. Borings were backfilled with auger cuttings.

The purpose of this geotechnical exploration was to determine the general subsurface profile at the project site and to relate the engineering properties of the soils that is, their classification, strength and suitability as foundation bearing material for the proposed access road. Field and laboratory information obtained together with our interpretation of the findings, are presented herein.

2.0 EXPLORATION FINDINGS

2.1 SURFACE CONDITIONS

Mr. Mark Luskin, a representative of KSWA, visited the subject site during this study to observe the site surface conditions. The subject site is an irregularly shaped parcel located east of Taxiway Sierra and immediately south of the Murfreesboro Road underpass, adjacent to Dell Campus.

The ground surface along the access road easement area of the property gradually slopes upward in an easterly direction with grass cover. Total relief across the subject area is approximately 25 feet.

Rock outcrops were observed near the proposed access road. Additionally a rock cut face was observed along Dell's truck entrance. The rock cut face indicated shallow rock features in this area.

2.2 SITE GEOLOGY

The site lies in the Central Basin Physiographic Province of Middle Tennessee. The Central (or Nashville) Basin is an oval-shaped topographic depression approximately located in the geographic center of the state. The terrain is gently rolling to hilly with some nearly level areas. Elevations range from 500 to 700 ft. The USGS Topographic Quadrangle, Antioch, Tennessee (dated 1979) indicates that the site is approximately 590 feet above MSL. The Central Basin was formed as a result of erosion of the gently-arched Nashville Dome and is underlain by essentially flat-lying limestones and shales of Ordovician age.

The USGS Geologic Map of the Antioch Quadrangle indicates this particular site is underlain by the Hermitage Formation. The Hermitage Formation is typically a medium light-gray to brownish-gray medium-bedded limestone. The formation weathers to produce residual soil layers that are typically clays, silts, and sands. The residual soils often grade with increasing depth from silts and clays to sands. The lower sandy soils often contain thin zones of highly plastic and very wet clays. These materials are termed "phosphates" by local contractors.

Overburden thicknesses typically range from 5 to 20 feet. Shallower rock depths may be encountered at some locations when "floating" boulders are contained within the soil overburden. The bedrock surface may also be irregular due to cutters (slots filled with unconsolidated materials) and pinnacles and the soil-rock interface. Calcarenite (sandstone rock composed of weathered limestone) zones may also occur above the more competent limestone bedrock mass.

2.3 SUBSURFACE CONDITIONS

The field work phase of our exploration was carried out on July 13, 2007. Four test borings, B-1 through B-4, were drilled along the proposed new access road easement for the development area. The boring locations were spaced evenly to incorporate the entire length of the access road. A Boring Location Map with actual boring locations is included in the Appendix of this report.

The field testing and sample classification procedures are included in the Appendix. The subsurface materials encountered were classified using the visual Unified Soil Classification System (USCS) as a guide. Test Boring Logs which detail the subsurface conditions encountered at each boring location are also included in the Appendix.

The proposed development area and access road easement location are shown on the boring location plan in the Appendix of this report.

2.3.1 Topsoil

Three to six inches of topsoil was encountered at the four boring locations.

2.3.2 Subsurface Soil

The subsurface soils encountered typically contained a brown silty lean clay underlain by fractured limestone. The visual field classification by the Unified Soil Classification System (USCS) classified the soil at the borings as having material properties of a CL classified soil. The top 1.5 feet of each of the borings appeared to be fill material. That is, the materials did included non-deleterious material indicating that the soils have been disturbed in past.

Laboratory samples indicated that the material was lean clay (CL) with the exception for boring B-3, SS-3 approximately 4.5 to 7.5 feet. This soil was classified in the laboratory as a CH; that is a fat clay with a liquid limit of over 50.

2.3.3 Auger Refusal

The borings were drilled to a depth of 10 feet or auger refusal. Auger refusal was encountered at depths of 2.5 feet at B-1, 8.5 feet at B-2 and 1.2 feet at B-4. Auger refusal was not encountered at boring B-3.

2.3.4 Groundwater Conditions

Groundwater was not encountered in the borings at the time of drilling.

2.4 LABORATORY RESULTS

2.4.1 Moisture Content Tests

Moisture content tests were performed on each of the split spoon samples. The average moisture content was found to be 20.5 percent with a minimum of 11 percent and a maximum of 35 percent. The average moisture content, when compared to the Atterberg Limits tests, indicated that the soils in their present condition were near or below their plastic limit (dry).

2.4.2 Atterberg Limits and Penetrometer Testing

Nine Atterberg limits tests were performed, one from test boring B-1, three from B-2, four from B-3, and one from B-4. In addition, pocket penetrometer tests were performed. The results are shown on Table 1 below:

Table 1: Atterberg Limits and Penetrometer Test Results

Boring	Sample No.	Liquid Limit	Plastic Limit	Plasticity Index	Pocket Penetrometer (tsf)	Classification
B-1	SS-1	38	20	18	>4.5	CL
B-2	SS-1	36	20	16	>4.5	CL
B-2	SS-2	34	19	15	3.5	CL
B-2	SS-3	42	26	16	1.5	CL
B-3	SS-1	42	23	19	2.25	CL
B-3	SS-2	38	22	16	2.75	CL
B-3	SS-3	56	26	30	3.25	CH
B-3	SS-4	43	26	17	1	CL
B-4	SS-1	36	19	17	>4.5	CL

Notes: Classifications determined by USCS

The Atterberg Limits tests indicate that the soils are typically lean clays. Fat, high plasticity clays with liquid limit over 50 was identified at sample SS-3 of boring B-3.. This sample is also considered to be a high plasticity clay with a plasticity index of 30.

The pocket penetrometer readings of the samples indicated an average bearing strength of approximately 3 tons per square foot.

3.0 GEOTECHNICAL EVALUATION

3.1 GENERAL

Based upon our engineering reconnaissance of the site, the test borings, a visual examination of the samples, the laboratory tests, our understanding of the proposed construction, and our experience as Geotechnical Engineers, we have reached the following conclusions and make the following recommendations.

The conclusions and recommendations of this report have been derived by relating the general principles of the discipline of Geotechnical Engineering to the proposed construction outlined by the Project Information section of this report. Because changes in surface, subsurface and climatic conditions can occur, we recommend that the use of this report be restricted to this specific project.

Our understanding of the proposed design and construction is based on the documents provided to us at the time this report was prepared and which is referenced in the Project Information section of this report. We recommend that we review the final design documents, plans and specifications to insure that the conclusions and recommendations of this Geotechnical Report have been interpreted correctly. Any changes or modifications which are made in the field during the construction phase which alter site grading, structure locations, infrastructure or other related site work should also be reviewed by our office.

If conditions are encountered in the field during construction which varies from the facts of this report, we recommend that the Geotechnical Engineer of record be contacted immediately to review the changed conditions in the field and make appropriate recommendations. The stratifications illustrated on the attached test boring logs represent the conditions only at the actual test boring location. Differences in soil consistencies and moisture contents should be expected at other areas across the site.

3.2 SOIL SUITABILITY

Based on the project information provided and the available subsurface data, it is our opinion that the proposed site is suitable for the planned construction, provided the site preparation recommendations are followed as intended. The plasticity index for the majority of the soils was approximately 17 indicating moderate potential for swelling/shrinking. The existing lean clay soil materials have been used in the past for a pavement sub base. With appropriate moisture conditioning and compaction, the lean clay soils will be acceptable with the implementation of the pavement recommendations.

The Atterberg limits of the soils did indicate the presence of highly plastic soils (i.e. soils with a liquid limit over 50), commonly referred to as fat clays in the sample collected from soil boring B-3 at an

approximate interval of 6.0 to 7.5 feet. The plasticity index for the isolated fat clay was approximately 30 indicating a high potential for swelling/shrinking. Fat clays, when encountered during construction, should not be used alone under roadways or foundations. However, several alternatives are available to effectively utilize this material. During excavation, if the material is found is relatively small quantities, the fat clays can be easily mixed with leaner clays to reduce the potential for shrinkage/swelling. Additionally, the fat clays can be used for non-structural or landscaping purposes.

Based on the project specifications provided, the proposed finished grade will be approximately 580.5 feet above MSL. The approximate surface elevation of soil boring B-3 is 601 feet above MSL and is presumed to be in a cut area of approximately 20 feet. It is additionally anticipated that blasting of underlying rock will occur along the length of the access road to achieve the final grade requirements.

3.3 EXISTING FILL MATERIALS

Possible fill material was identified at various areas across the site. Fill materials were encountered in test borings B-1 through B-4 beneath the ground surface to a depth of about 1.5 feet. The evidence of fill in this area included limestone fragments and additional building debris. Existing fill materials may also be present in other areas not investigated, outside of our test boring locations. The samples obtained appeared relatively free of deleterious material. However, information pertaining to the age, placement and compaction of the fill was not available. As is the case with any fill placed without technical observations, the possibility exists that the fill may contain concentrated amounts of deleterious material and soft compressible zones not disclosed by our borings. Based on our understanding of the proposed road grades, the fill material will be excavated and will not be left in place.

3.4 SHALLOW BEDROCK

Shallow bedrock was encountered at depths of about 1.2 to 8.5 feet in the test borings located across the site. Based on the provided site grading information, cuts for the new road will be below the level of the bedrock. Depending on final grades, blasting is expected to be required for utility lines and general site grading.

3.5 SINKHOLE HAZARDS

Our site reconnaissance did not identify depressions at the site or in the immediate vicinity of the proposed building. Depressions were not observed on the topographic map. The Nashville Tennessee area is considered a low risk for the development of sinkholes.

The present state-of-the-art of geotechnical engineering does not permit accurate prediction of where or when sinkholes will occur. Site grading should be established to provide positive drainage both during and after construction so as to minimize the potential for future sinkhole development. During

construction the grading contractor should be alert to any indication of possible sinkhole activity. Any sink features encountered during the site grading should be repaired under the direction of the geotechnical engineer.

4.0 GEOTECHNICAL RECOMMENDATIONS

The construction recommendations contained in this report section were developed in consideration of the project information detailed in Section 1.1 of this report. If the information contained in Section 1.1 has been revised, we recommend KSWA be contacted to confirm that our foundation design and construction recommendations are appropriate in consideration of the new available information.

4.1 PAVEMENT RECOMMENDATIONS

We understand that site pavements will be categorized into heavy duty asphalt pavements. We have not been provided with specific traffic frequency and loading. However, we have been requested to provide expected standard pavement loading for the design of heavy duty pavement sections. Based on the required grade elevations along the development area access road easement, it is anticipated that blasting of the underlying rock will be required across the length of the road. Therefore, we expect the entire length of the access road to have a blasted rock subbase. We recommend, and have estimated for design purposes, a California Bearing Ratio (CBR) value of 40 be used. If the access road elevation changes such that the road subbase will not be expected to be bearing on bed rock, we recommend that a sample of the subgrade material be collected to test for CBR properties.

For the purpose of pavement thickness design, a layered analysis has been used. Standard traffic counts for the anticipated loading and other presumed design criteria is shown in table 2 below:

Table 2: Pavement Design Criteria and Thickness Recommendations

PAVEMENT TYPE:	HEAVY DUTY
Design Life:	20 yrs
Initial Serviceability:	4.2
Terminal Serviceability:	2.0
Reliability:	95
Subgrade CBR:	40
Wearing Course Layer Coefficients:	0.42
Base Course Layer Coefficients:	0.38
Design Estimated ESALs	100,000 (Traffic Class III)
Asphalt Wearing Course Thickness	1.5
Asphalt Base Course Thickness	1.5
Aggregate Base Course Thickness	5.0

Notes: Heavy Duty is defined as the drive lanes of the Site. The design criteria are typical data taken from "Asphalt Thickness Design" by the Asphalt Institute 1981 edition and KSWA's experience with similar projects.
Thickness designs are based on a well-graded gravel subgrade with a CBR of 40.

Subgrade properties should be determined either by field CBR tests, Dynamic Cone Penetrometer (DCP) testing, field plate load tests or the correlation between fill density tests and laboratory CBR tests.

The pavement can be designed for a CBR value of 40, provided that the subgrade is prepared in accordance with the Construction Considerations section of this report. Pavement may be placed after the subgrade has been properly compacted, fine graded and proofrolled. We also recommend that the top eight inches of the pavement subgrade be compacted within the range of maximum dry density and optimum moisture content immediately prior to paving. The activity should be accomplished in accordance with proper construction specifications. Actual pavement section thickness should be determined by the design civil engineer or geotechnical engineer based on traffic loads, volume, and the owner's design life requirements.

Water should not be allowed to pond behind curbs and saturate the pavement basestone. In down grade areas basestone should extend through the slope to allow any water entering the basestone a path to exit.

5.0 CONSTRUCTION CONSIDERATIONS

This report section details our general earthwork recommendations including site preparation and construction of compacted fills. These are general recommendations and the recommendations detailed in the Geotechnical Evaluation Section of this report should be implemented in addition to the following recommendations. We recommend that all earthwork activities be observed and documented by a representative of the geotechnical engineer. As your geotechnical consultant on this project, KSWA would be happy to discuss our capabilities for providing these services.

5.1 SITE PREPARATION

Based on the provided site information and field observations, it is anticipated that blasting will occur to remove the underlying rock to the proposed site grades. The access road has been assumed to be bearing on rock. Any potential soil seams within the limestone should be excavated to a depth of approximately three feet below grade and replaced with well-graded gravel and properly compacted. At the completion of these activities, the site should then be prepared as follows:

- Proofrolling the exposed subgrade using suitable heavy equipment under the direction of a qualified geotechnical engineer.
- Undercutting soft, organic, or other unsuitable materials encountered during the proofrolling operations under the direction of a qualified geotechnical engineer. Particular attention should be given to planned drive areas or areas with existing grade elevations, which are at or near final grade elevations.
- Proofrolling the exposed subgrade in cut areas prior to pavement construction.

5.2 COMPACTED FILL RECOMMENDATIONS

Once the subgrade has been properly prepared, compacted engineered fill may be placed to attain final desired construction elevations. Based on the anticipated elevations, we do not anticipate soil fill to be used under the roadway. Where soil fill is used, recommend that fill placement and compaction operations be observed and documented by a representative of a qualified geotechnical engineer.

Soil fill materials should consist of soil with the following characteristics:

- Liquid Limit less than 50.
- Plasticity Index less than 25.

- Free of large rock fragments (greater than three inches in diameter) and organic materials (less than five percent by weight).

- The amount of rock fragments retained on a 3/4-inch sieve should be less than 30 percent.

The results of the limited laboratory testing and our visual-manual classifications indicate that the on-site soils meet the aforementioned criteria with the exception of the fat clay interval identified in test boring B-3. Depending on the time of year and weather conditions, the on-site soils may be above the optimum moisture content and require drying before use. KSWA recommends that representative samples of the proposed fill materials be collected and tested to confirm that they meet the above criteria prior to placement as compacted fill. We recommend that suitable materials be placed and compacted using the following criteria:

- Excavation for the proposed construction will produce some rock. This material, with acceptable gradation, can be used as structural fill. Shotrock utilized as structural fill. Shotrock utilized as structural fill should be well graded with a maximum rock size of 18 inches and be placed in lifts not to exceed 24 inches thick. Shotrock fill should have adequate fines to effectively "choke" the larger rock pieces, filling all voids or open spaces. The larger rock pieces should lie flat and not overlap each other. The percentage of soil in the fill should be limited to a maximum of 10 percent. Shotrock fill should be compacted using 6 to 8 complete passes of a D-8 class crawler tractor. A pass is defined as a complete coverage of the surface with the D-8 track overlapping 50 percent. Half of the passes should be in each perpendicular direction. Shotrock fill placement should be accomplished under the full time observation of representative of the geotechnical engineer.

- Soil fill should be placed in lifts of uniform thickness. The lift thickness should not exceed that which can be properly compacted throughout its entire depth with appropriate equipment; usually no more than eight inches for clay soils and no more than 10 inches for granular soils for area fill. In confined areas such as utility trenches, lifts of three to four inches may be required to achieve the specified degree of compaction.

- All fill should be properly keyed into stripped and scarified subgrades. The upper one-foot of materials in planned cut areas or in areas which do not receive more than one-foot of new fill should be scarified and recompacted using the guidelines outlined in this report section.

- Fill should not be placed on frozen or saturated subgrades.

- Fills placed and compacted in structural areas should be compacted within the range provided in section 2.3.1 of the maximum dry density. Additionally, the compacted fill should be stable under the moving load of a loaded tandem-axle dump truck.

- Density tests should be performed at a frequency of no less than one test per 5,000 to 7,500 square feet for pavement areas for each one-foot thick fill layer placed. A minimum of at least two tests per lift is recommended. For utility trenches, density tests should taken every 50 linear feet for each one-foot thick fill layer placed. Any areas not meeting the compaction specifications should be recompacted to achieve compliance.

- The soils should be placed within the range provided in section 2.3.1 for the optimum moisture content. Aeration is often necessary to bring fill materials to the required moisture condition during wet and rainy periods. During dry periods, water may need to be added to bring the soils up to the proper moisture content for compaction. Clay soils may require aeration prior to compaction even during dry periods.

- All backfill for use in utility trenches for this project should be placed in thin lifts and should be compacted with an appropriate type of compaction equipment. Granular backfill should be compacted to at least 75 percent relative density (ASTM D4253 and ASTM D4254). Under no conditions should any backfill be flushed to obtain compaction.

- All excavations should be constructed in accordance with applicable Occupational Safety and Health Administration (OSHA) regulations

5.3 PAVEMENT SUBGRADE RECOMMENDATIONS

Pavement subgrade preparation prior to aggregate and asphalt placement will help to ensure a properly functioning and long lasting pavement system. If fill material is required below pavements, follow the recommendations in the compacted fill section of this report. If native soil material is used for the subgrade, the following recommendations apply:

- The CBR value will be significantly lower than expected CBR value of rock. CBR values should be determined for any soils used as fill and the pavement section be redesigned to reflect to the new CBR values.

- Native subgrade that has been properly compacted, fine graded and proofrolled can be used. We recommend that the top eight inches of the pavement subgrade be compacted within the ranges mentioned above in section 2.3.1 for both dry density and optimum moisture contents immediately prior to paving.

- Density tests should be performed at a frequency of no less than one test per 5,000 to 7,500 square feet for pavement areas over native subgrade. Any areas not meeting the compaction specifications should be recompacted to achieve compliance.

- Native soil should be protected from rain and surface water run-off prior to placement of aggregates and pavements.

- If materials other than native soils are encountered during preparation, the materials should either, removed and replaced with properly compacted fill or KSWA should be contacted for an evaluation of the material.

5.4 GENERAL EARTHWORK CONSIDERATIONS

During all earthwork operations, positive surface drainage should be maintained to prevent water from ponding on the surface. The exposed subgrade may be rolled with a rubber-tired or steel drummed roller

to improve surface run-off if precipitation is expected. A qualified geotechnical engineer should be consulted if the subgrade soils become excessively wet, dry, or frozen.

5.5 GROUNDWATER CONTROL RECOMMENDATIONS

Groundwater was not encountered during drilling operations. However, groundwater levels may fluctuate with season changes. Water conditions which typically affect shallow excavations for construction projects in the Nashville, Tennessee area are related to trapped or "perched water" which often occurs in irregular, discontinuous locations within the native soil, at the soil/rock interface, or at discontinuous locations within the rock unit. If these water-bearing strata are exposed in excavations, the magnitude and duration of seepage varies. These water sources are often not linked to the more continuous relatively stable ground water table that typically occurs at greater depths. We anticipate that in most cases, depending on seasonal conditions, any seepage encountered can be handled by conventional dewatering methods (i.e., pumping from small sumps located near the source or in collector areas). If larger quantities of groundwater are encountered or silting of the sump occurs, the geotechnical engineer should be contacted.

6.0 QUALIFICATIONS OF RECOMMENDATIONS

Our recommendations for this report were developed using the subsurface information obtained from the soil test borings that were advanced at the site. Soil test borings only depict the soil conditions at the specific location and time at which they were made. The soil conditions at other locations on the site may differ from those occurring at the boring locations.

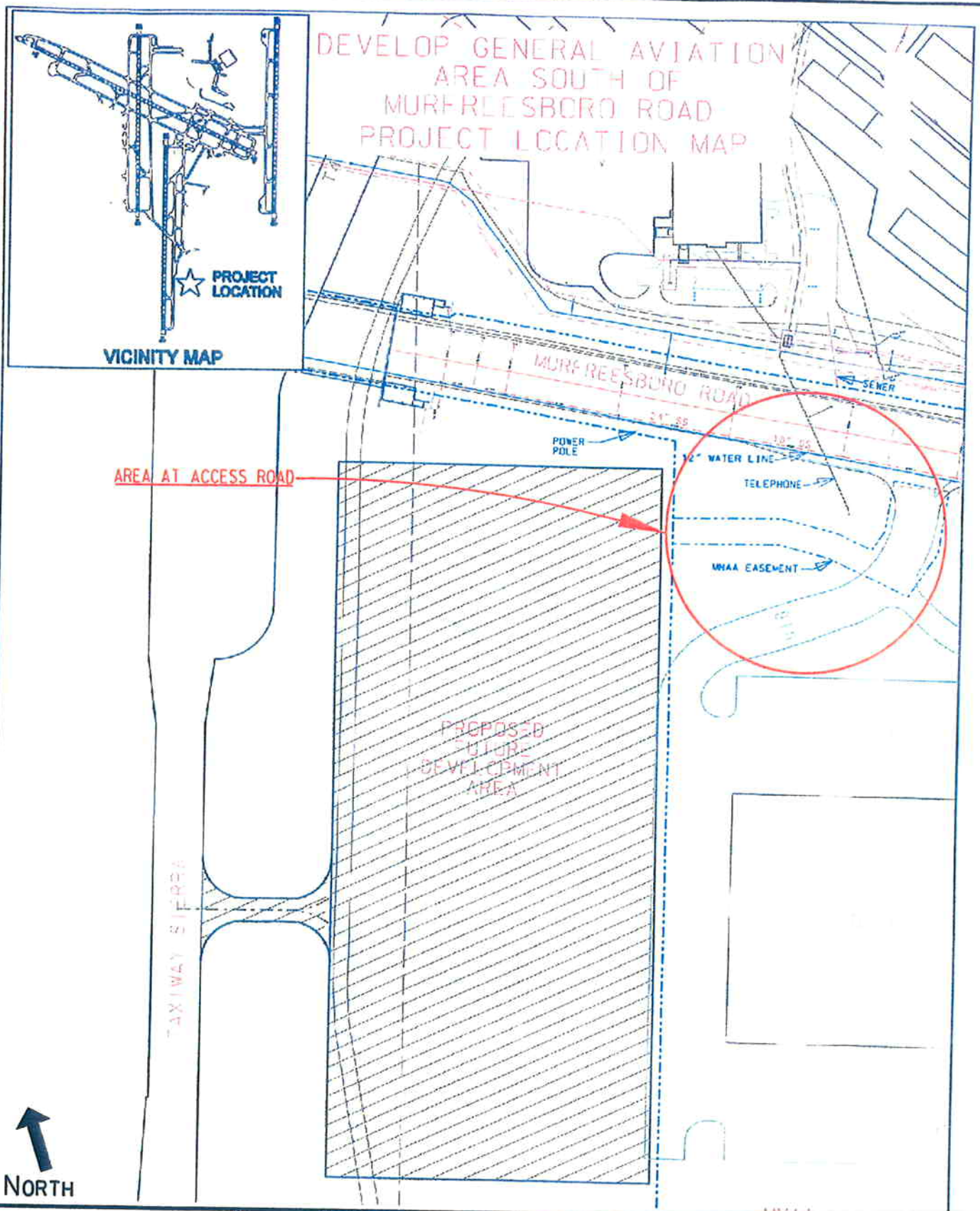
The conclusions and recommendations contained in this report were based on the available subsurface information, the project information provided, and the assumptions previously stated. Revisions in the plans for the proposed construction from those anticipated in this report should be brought to the attention of a qualified geotechnical engineer to determine whether any changes in the foundation recommendations are necessary. If deviations from the noted foundation conditions are encountered during the construction, they should also be brought to the attention of the soils engineer.

The scope of our geotechnical services did not include assessment or investigation for the presence or absence of hazardous or toxic materials in the soil, groundwater or surface water within or beyond the site studied. Any statements in this report or indicated on the test boring logs regarding odors, staining of soils or other unusual conditions observed are strictly for the information of our client.

Our professional services were performed, our findings obtained, and our recommendations prepared in accordance with generally accepted geotechnical engineering principles and practices. KSWA is not responsible for the conclusions, opinions, or recommendations made by others based upon the data included herein.

Our services include retaining the soil samples obtained during this study for 30 days after report submittal. Further storage or transfer of the samples can be made at the Client's expense upon a written request.

FIGURES



SITE LOCATION PLAN

General Aviation Area Project # 0475

Nashville, Tennessee

Prepared for:

Metropolitan Nashville Airport Authority

Nashville, Tn

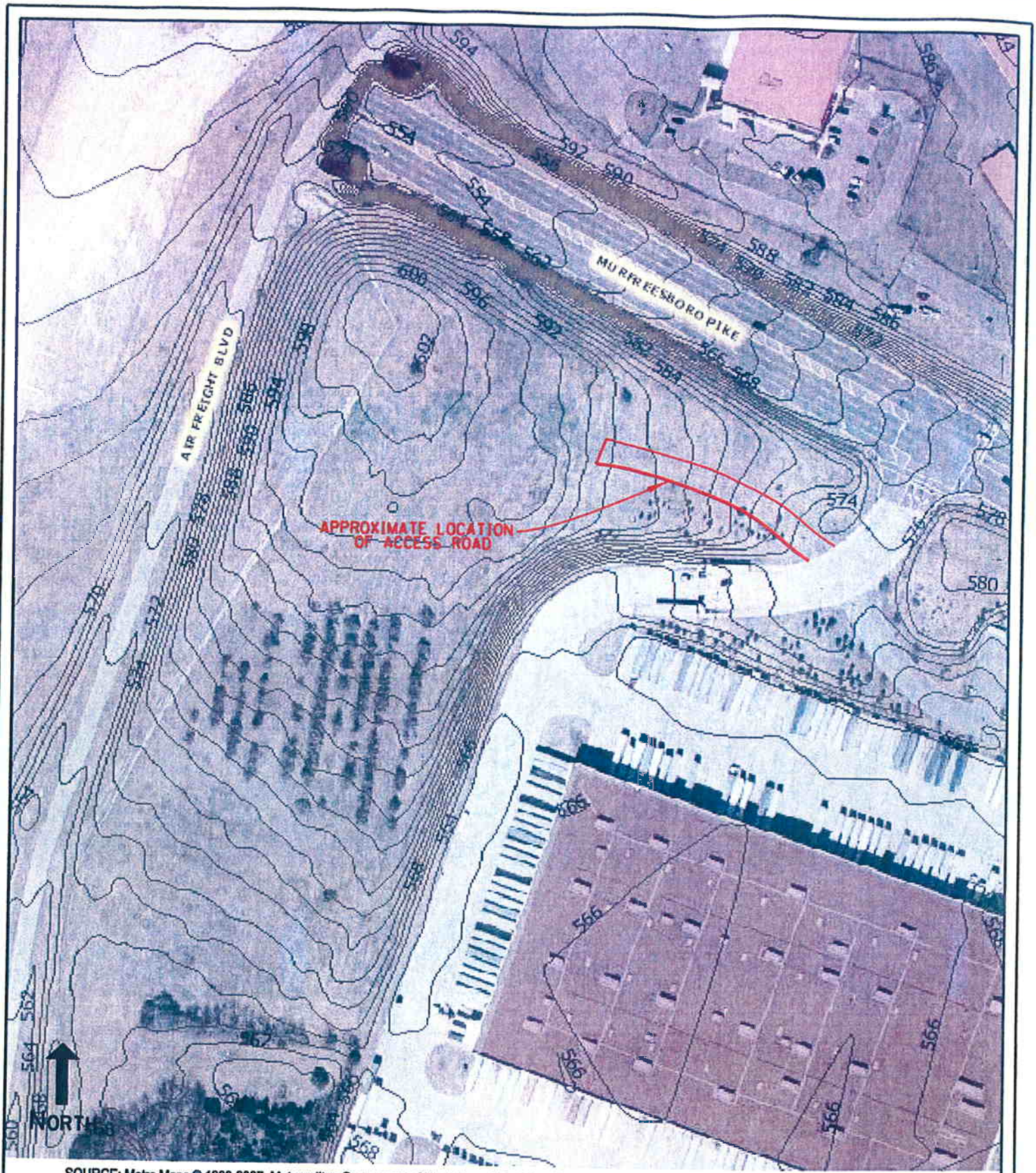
KSWA

K.S. Ware & Associates, L.L.C.

Engineering, Environmental & Information Services

Designed by: RW Armstrong	
Date: 7/31/2007	
Drawn by: ND	
Date: 7/31/2007	
Checked by: JDG	
Date: 7/31/2007	
Project Number:	100-06-0008

Figure 1



SOURCE: Metro Maps © 1999-2007, Metropolitan Government of Nashville and Davidson County, Tennessee www.nashville.gov/mpc/property 7/31/2007

KSWA

K.S. Ware & Associates, L.L.C.
Engineering, Environmental & Information Services

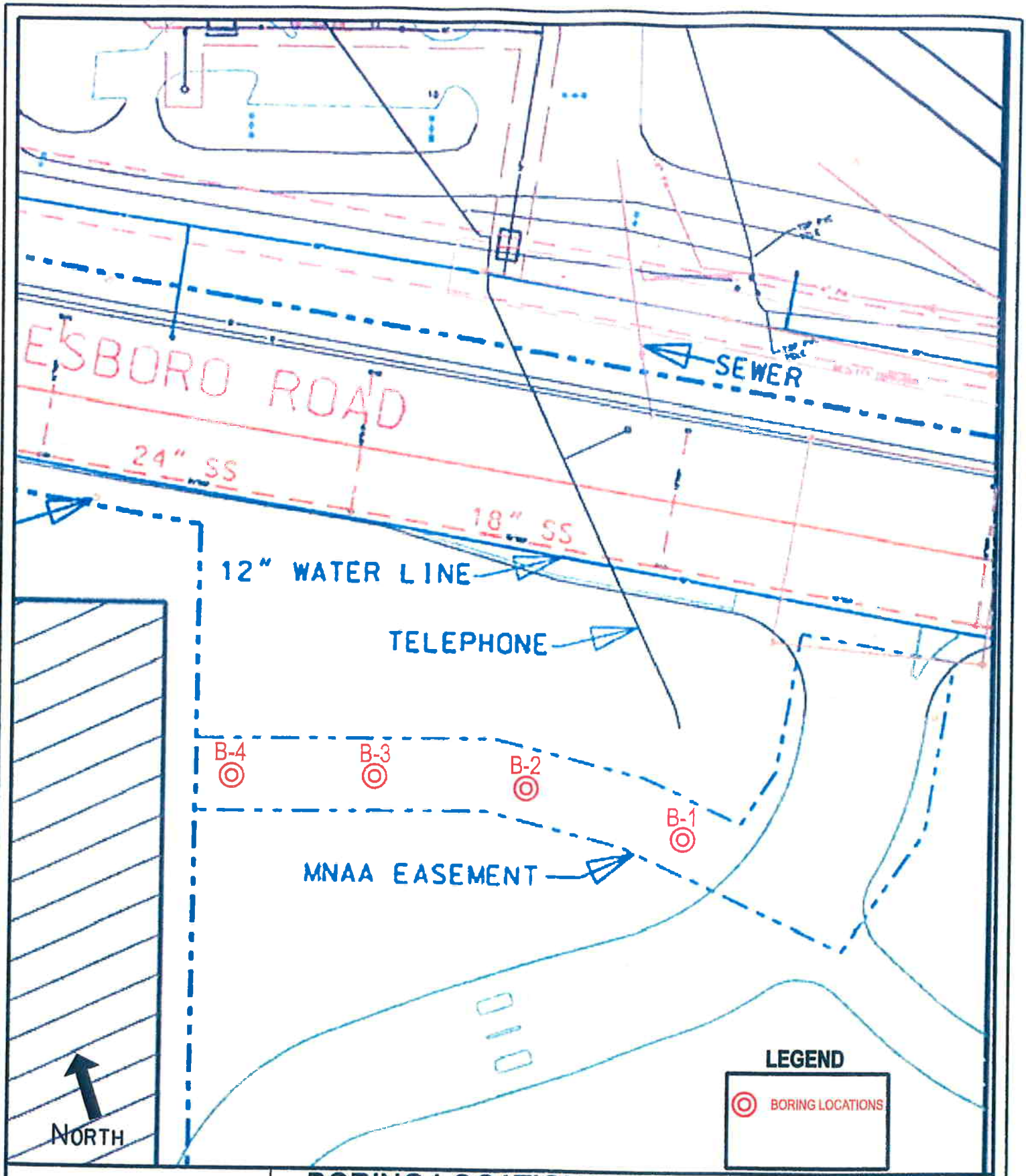
TOPOGRAPHIC SITE LOCATION PLAN

General Aviation Area Project # 0475
Nashville, Tennessee

Prepared for:
Metropolitan Nashville Airport Authority
Nashville, Tn

Designed by: RW Armstrong	
Date: 7/31/2007	
Drawn by: ND	
Date: 7/31/2007	
Checked by: JDG	
Date: 7/31/2007	
Project Number:	100-06-0008

Figure 2



KSWA
K.S. Ware & Associates, L.L.C.
Engineering, Environmental & Information Services

BORING LOCATION PLAN

General Aviation Area Project # 0475

Nashville, Tennessee

Prepared for:
Metro Nashville Airport Authority
Nashville, Tennessee

Designed by: RW Armstrong	
Date: 7/30/2007	
Drawn by: ND	
Date: 7/30/2007	
Checked by: JDG	
Date: 7/30/2007	
Project Number:	100-06-0008

Figure 3

APPENDIX A

FIELD TESTING PROCEDURES

FIELD TESTING PROCEDURES

Soil Test Borings

A total of four (4) soil test borings were advanced at the site. The boring locations and depths were selected by KSWA. The actual field boring locations were determined by our personnel who paced distances in the field relative to on-site features.

Soil sampling and penetration testing was performed in general accordance with ASTM Method D-1586, "Penetration Test and Split-Barrel Sampling of Soils". The borings were advanced with continuous flights of powered hollow stem augers. At predetermined intervals, soil samples were obtained with a split-tube sampler. The sampler was first seated six inches to penetrate any loose cuttings, then driven an additional foot using an 140-pound hammer falling 30-inches. The number of hammer blows required to drive the sampler the final foot was recorded and is designated the "standard penetration resistance" (N-value).

A representative soil sample obtained from each sample interval was placed into glass jars and subsequently classified by our field personal prior to being transported to our laboratory. A geotechnical engineer verified the soil classifications once in the laboratory.

Refusal Material

Refusal is a description of materials, which either have an N-value exceeding 50 blows per foot or cannot be penetrated by soil drilling procedures. Refusal may result from soft weathered bedrock, boulders, thin rock seams, or the upper surface of sound continuous bedrock. Sampling of refusal materials using core-drilling procedures was beyond the scope of our services.

Test Boring Logs

Our interpretation of the subsurface materials encountered at each boring location is indicated on the Test Boring Logs (Logs) in the Appendix. These Logs were prepared by our engineer after reviewing the field logs, visually-manually classifying the soils, and reviewing laboratory classification test results. The samples were classified using the Unified Soil Classification System (USCS) as a guide. The USCS designations indicated on the Logs are based on visual-manual evaluation of the samples unless otherwise defined by laboratory testing.

The Logs indicate estimated interfaces between soil strata. The interfaces indicated represent the approximate interface location. The actual transition between strata may be gradual. Water levels indicated on the Logs represent the conditions only at the time each measurement was taken.

APPENDIX B

TEST BORING LOGS

K.S. Ware & Associates, L.L.C.
Engineering & Testing Services

LOG OF BORING NO. B-1

Project Name: **GA South of Murfreesboro Rd.**
Location: **Nashville, Tennessee**
Number: **100-06-0008**

Surface El.: **591.0**
Location: **Development Area Entrance Road Easement - East End**

Depth, feet	Graphic Log	MATERIAL DESCRIPTION	Samples	Recovery %	Penetration Blows/Foot	Water Content, %	Liquid Limit	Plastic Limit	Plasticity Index
		TOPSOIL 0.2							
		SILTY CLAY (CL), trace gravel, brown, dry, firm		100	10-12-18	11.5	38	20	18
		2.5							
		Auger Refusal @ 2.5 feet BGS (probable limestone).							
5									
10									

Completion Depth: **2.5**
Date Started: **7/13/07**
Date Completed: **7/13/07**
Drilled By: **R. S. Drilling**
Logged By: **Mark Luskin**

Remarks: **Boring completed with track-mounted HSA drill rig. Water table was not encountered. Elevations are based on site topographic map and are approximate.**

LOG_OF_BORING2 100-06-0008 BORING LOGS.GPJ KSWARE.GDT 8/14/07

K.S. Ware & Associates, L.L.C.
Engineering & Testing Services

LOG OF BORING NO. B-2

Project Name: **GA South of Murfreesboro Rd.**

Location: **Nashville, Tennessee**

Number: **100-06-0008**

Surface El.: **596.0**

Location: **Development Area Entrance Road Easement - East Center**

Depth, feet	Graphic Log	MATERIAL DESCRIPTION	Samples	Recovery %	Penetration Blows/Foot	Water Content, %	Liquid Limit	Plastic Limit	Plasticity Index
		TOPSOIL 0.2							
		SILTY CLAY (CL), some organics, brown, dry, slightly plastic, firm		100	7-16-16	14.5	36	20	16
		1.5							
		SILTY CLAY (CL), trace sand, brown, moist, slightly plastic, firm - soft at depth		67	3-8-10	23.7	34	19	15
5									
		7.3		89	3-4-5	35.9	42	26	16
		FRACTURED LIMESTONE, fine-medium grained sand from 7.3-7.4' BGS 7.5							
		SILTY/SANDY CLAY (CL), brown, very moist, slightly plastic, soft							
		8.5							
10		Auger Refusal @ 8.5 feet BGS (probable limestone).							

Completion Depth: **8.5**
Date Started: **7/13/07**
Date Completed: **7/13/07**
Drilled By: **R. S. Drilling**
Logged By: **Mark Luskin**

Remarks: **Boring completed with track-mounted HSA drill rig. Water table was not encountered. Elevations are based on site topographic map and are approximate.**

LOG OF BORING2 100-06-0008 BORING LOGS.GPJ KSWARE.GDT 8/14/07

K.S. Ware & Associates, L.L.C.
Engineering & Testing Services

LOG OF BORING NO. B-3

Project Name: **GA South of Murfreesboro Rd.**
Location: **Nashville, Tennessee**
Number: **100-06-0008**

Surface El.: **601.0**
Location: **Development Area Entrance Road Easement - West Center**

Depth, feet	Graphic Log	MATERIAL DESCRIPTION	Samples	Recovery %	Penetration Blows/Foot	Water Content, %	Liquid Limit	Plastic Limit	Plasticity Index
		TOPSOIL 0.2							
		SILTY CLAY (CL), some organics and debris (brick/gravel), brown, dry, slightly plastic, stiff		89	4-11-19	15.2	42	23	19
		1.5 SILTY CLAY (CL), trace sand and gravel at top, brown, moist, plastic, firm		100	7-11-13	20.6	38	22	16
5		4.5 CLAY (CH), trace silt and sand, brown, moist-very moist, plastic, firm-soft at depth (thin sand layer at 7.2' BGS, fine-medium grained)		89	2-5-5	26	56	26	30
		7.5 SILTY CLAY (CL), some silt and sand, brown, very moist, plastic, firm-soft (thin sand layers at 10.0' and 10.3' BGS, fine-medium grained)		100	4-6-9	25.9	43	26	17
10		10.5 Boring Terminated @ 10.5 feet BGS - No Refusal.							

Completion Depth: **10.5**
Date Started: **7/13/07**
Date Completed: **7/13/07**
Drilled By: **R. S. Drilling**
Logged By: **Mark Luskin**

Remarks: **Boring completed with track-mounted HSA drill rig. Water table was not encountered. Elevations are based on site topographic map and are approximate.**

LOG OF BORING2 100-06-0008 BORING LOGS.GPJ KSWARE.GDT 8/14/07

K.S. Ware & Associates, L.L.C.
Engineering & Testing Services

LOG OF BORING NO. B-4

Project Name: **GA South of Murfreesboro Rd.**
Location: **Nashville, Tennessee**
Number: **100-06-0008**

Surface El.: **606.0**
Location: **Development Area Entrance Road Easement - West End**

Depth, feet	Graphic Log	MATERIAL DESCRIPTION	Samples	Recovery %	Penetration Blows/Foot	Water Content, %	Liquid Limit	Plastic Limit	Plasticity Index
		TOPSOIL 0.2							
		SILTY CLAY (CL), some organics and gravel at top, brown, dry, slightly plastic, hard (fractured limestone at 1.2' BGS		100	11-18-50(ref)	11	36	19	17
		1.2							
		Auger Refusal @ 1.2 feet BGS (limestone).							
5									
10									

Completion Depth: **1.2**
Date Started: **7/13/07**
Date Completed: **7/13/07**
Drilled By: **R. S. Drilling**
Logged By: **Mark Luskin**

Remarks: **Boring completed with track-mounted HSA drill rig. Water table was not encountered. Elevations are based on site topographic map and are approximate.**

LOG_OF_BORING2 100-06-0008 BORING LOGS.GPJ KSWARE.GDT 8/14/07

APPENDIX C

FIELD CLASSIFICATION SYSTEM

FIELD CLASSIFICATION SYSTEM

Sands and Gravels

No. of Blows	Relative Consistency
0-4	Very Loose
5-10	Loose
11-20	Firm
21-30	Very Firm
31-50	Dense
Over 50	Very Dense

Silts and Clays

No. of Blows	Relative Consistency
0-2	Very Soft
2-4	Soft
5-9	Firm
10-15	Stiff
16-30	Very Stiff
Over 31	Hard

Particle Size Identification

Boulders:	8-inch diameter or more
Cobbles:	3- to 8-inch diameter
Gravel:	
Coarse:	1- to 3-inch
Medium:	.50- to 1-inch
Fine:	.25- to .50-inch
Sand:	
Coarse:	2.00-mm to .25-inch (diameter of pencil lead)
Medium:	0.42-mm to 2.00-mm (diameter of broom straw)
Fine:	0.074-mm to 0.042-mm (diameter of human hair)
Silt:	0.042-mm to 0.002-mm (Cannot see particles)
Clay:	<0.002-mm

Relative Proportions

Descriptive Term	Percent
Trace	1-10
Little	11-20
Some	21-35
And	36-50

Relative Quality of Rock Cores

Quality	RQD
Very Poor	0-25%
Poor	25-50%
Fair	50-75%
Good	75-90%
Excellent	90-100%

$$RQD = \frac{\text{Total core, counting only pieces over 4" long}}{\text{Length of core run}} \times 100\%$$

Rock Hardness

Very Soft — Rock disintegrates or easily compressed to touch; can be hard to very hard soil.

Soft — Rock is coherent but breaks easily to thumb pressure at sharp edges and crumbles with firm hand pressure.

Moderately Hard — Small pieces can be broken off along sharp edges by considerable hard thumb pressure; can be broken by light hammer blows.

Hard — Rock cannot be broken by thumb pressure, but can be broken by moderate hammer blows.

Very Hard — Rock can be broken by heavy hammer blows.

Note: Taken from various USCS documents

APPENDIX D

SOIL CLASSIFICATION CHART

K.S. Ware & Associates, L.L.C.
Engineering, Environmental & Information Services

UNIFIED SOIL CLASSIFICATION SYSTEM

UNIFIED SOIL CLASSIFICATION AND SYMBOL CHART

COARSE-GRAINED SOILS (more than 50% of material is larger than No. 200 sieve size.)			
GRAVELS More than 50% of coarse fraction larger than No. 4 sieve size	Clean Gravels (Less than 5% fines)	GW	Well-graded gravels, gravel-sand mixtures, little or no fines
		GP	Poorly-graded gravels, gravel-sand mixtures, little or no fines
	Gravels with fines (More than 12% fines)	GM	Silty gravels, gravel-sand-silt mixtures
		GC	Clayey gravels, gravel-sand-clay mixtures
SANDS 50% or more of coarse fraction smaller than No. 4 sieve size	Clean Sands (Less than 5% fines)	SW	Well-graded sands, gravelly sands, little or no fines
		SP	Poorly graded sands, gravelly sands, little or no fines
	Sands with fines (More than 12% fines)	SM	Silty sands, sand-silt mixtures
		SC	Clayey sands, sand-clay mixtures
FINE-GRAINED SOILS (50% or more of material is smaller than No. 200 sieve size.)			
SILTS AND CLAYS Liquid limit less than 50%		ML	Inorganic silts and very fine sands, rock flour, silty of clayey fine sands or clayey silts with slight plasticity
		CL	Inorganic clays of low to medium plasticity, gravelly clays, sandy clays, silty clays, lean clays
		OL	Organic silts and organic silty clays of low plasticity
SILTS AND CLAYS Liquid limit 50% or greater		MH	Inorganic silts, micaceous or diatomaceous fine sandy or silty soils, elastic silts
		CH	Inorganic clays of high plasticity, fat clays
		OH	Organic clays of medium to high plasticity, organic silts
HIGHLY ORGANIC SOILS		PT	Peat and other highly organic soils

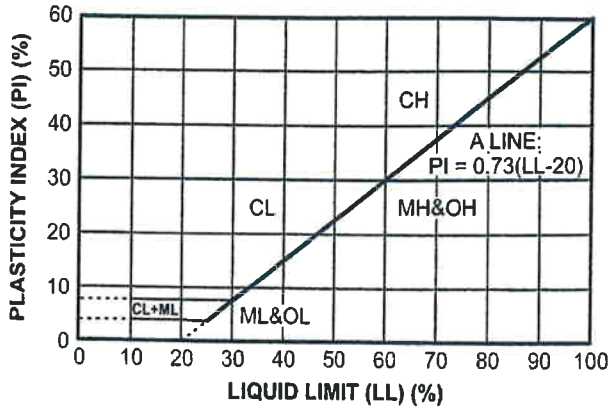
LABORATORY CLASSIFICATION CRITERIA

GW	$C_u = \frac{D_{60}}{D_{10}}$ greater than 4; $C_c = \frac{D_{30}}{D_{10} \times D_{60}}$ between 1 and 3	
GP	Not meeting all gradation requirements for GW	
GM	Atterberg limits below "A" line or P.I. less than 4	Above "A" line with P.I. between 4 and 7 are borderline cases requiring use of dual symbols
GC	Atterberg limits above "A" line with P.I. greater than 7	
SW	$C_u = \frac{D_{60}}{D_{10}}$ greater than 4; $C_c = \frac{D_{30}}{D_{10} \times D_{60}}$ between 1 and 3	
SP	Not meeting all gradation requirements for GW	
SM	Atterberg limits below "A" line or P.I. less than 4	Limits plotting in shaded zone with P.I. between 4 and 7 are borderline cases requiring use of dual symbols.
SC	Atterberg limits above "A" line with P.I. greater than 7	

Determine percentages of sand and gravel from grain-size curve. Depending on percentage of fines (fraction smaller than No. 200 sieve size), coarse-grained soils are classified as follows:

Less than 5 percent ..... GW, GP, SW, SP
More than 12 percent ..... GM, GC, SM, SC
5 to 12 percent ..... Borderline cases requiring dual symbols

PLASTICITY CHART



APPENDIX E

LABORATORY DATA

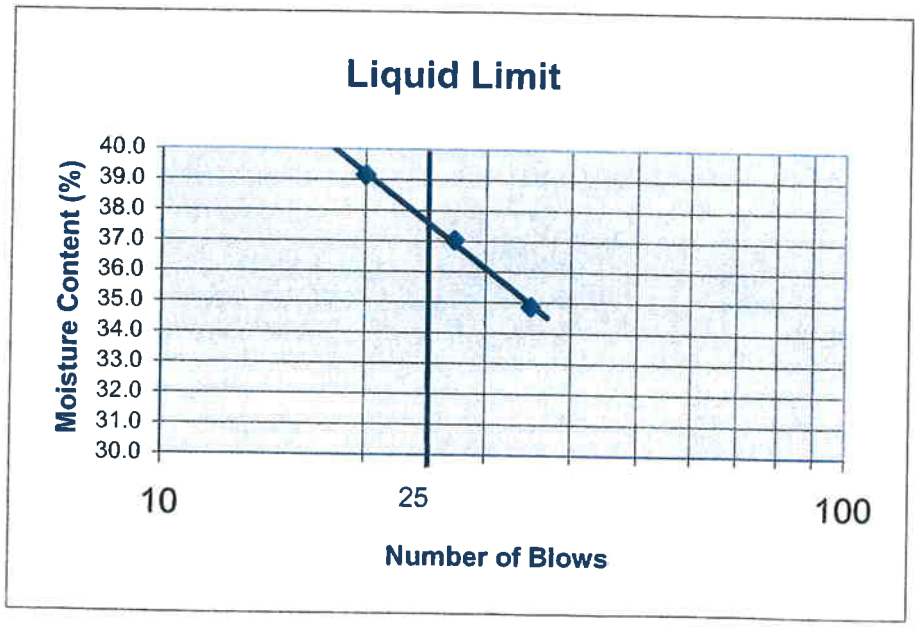
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Engineering, Environmental & Information Services

Report of Liquid Limit, Plastic Limit & Plasticity Index ASTM D4318

Project Name: GA South - Murfreesboro Rd
Project Number: 100-06-0008
Sample ID: B-1 SS-1
Test Date: 7/17/2007
Equipment Used: Liquid Limit Device #2, Oven, Ohaus 3kg Balance

Sample Description: Silty Clay, brown, dry, firm

	Liquid Limit				Plastic Content		
Tare No	LL	G4	KK		I	AAA	
Wet Soil and Tare	25.11	21.53	27.87		35.34	35.98	
Dry Soil and Tare	21.40	18.89	23.38		34.31	34.79	
Wt. of Water	3.71	2.64	4.49		1.03	1.19	
Tare Wt.	10.76	11.76	11.91		29.17	28.6	
Dry Soil	10.64	7.13	11.47		5.14	6.19	
Moisture content%	34.9	37.0	39.1		20.0	19.2	
No. Of blows	35	27	20		Average:	20	
Required Blows	***25-35***	***20-30***	***15-25***				



Liquid Limit: 38

Plastic Limit: 20

Plasticity Index: 18

USCS CLASSIFICATION:

CL

Tested By: A. Biney Date: 7/18/2007
Reviewed By: MBL Date: 8/8/07

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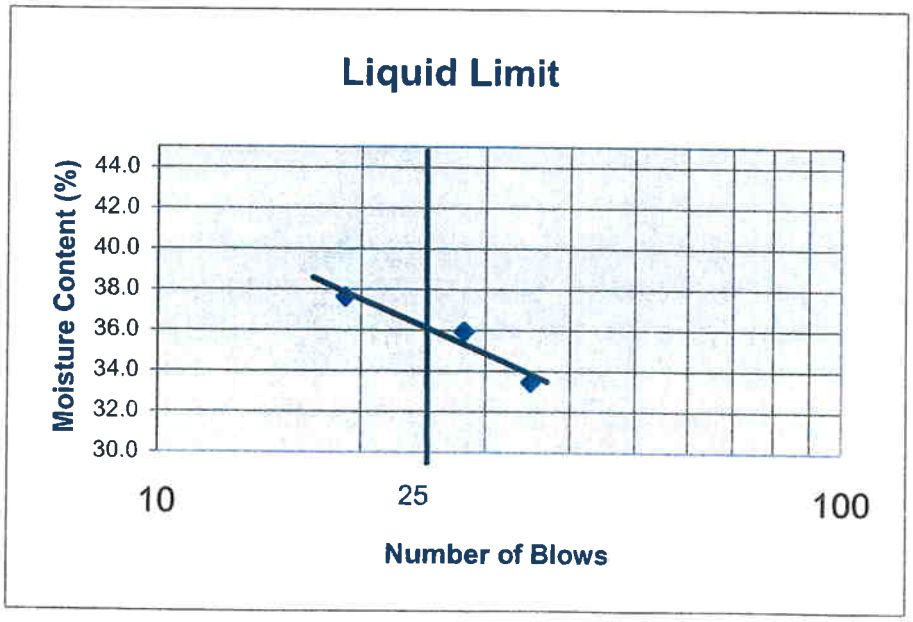
K.S. Ware & Associates, L.L.C.
Engineering, Environmental & Information Services

Report of Liquid Limit, Plastic Limit & Plasticity Index ASTM D4318

Project Name: GA South - Murfreesboro Rd
Project Number: 100-06-0008
Equipment Used: Liquid Limit Device #2, Oven, Ohaus 3kg Balance
Sample ID: B-2 SS-1 (0-1.5')
Test Date: 7/16/2007

Sample Description: Silty Clay, brown, dry, firm

	Liquid Limit				Plastic Content		
Tare No	AA6	1G	KK		G-4	YY	
Wet Soil and Tare	23.28	23.86	25.56		19.05	17.05	
Dry Soil and Tare	20.36	20.67	21.81		17.86	16.18	
Wt. of Water	2.92	3.19	3.75		1.19	0.87	
Tare Wt.	11.63	11.79	11.84		11.76	11.83	
Dry Soil	8.73	8.88	9.97		6.10	4.35	
Moisture content%	33.4	35.9	37.6		19.5	20.0	
No. Of blows	35	28	19		Average:	20	
Required Blows	***25-35***	***20-30***	***15-25***				



Liquid Limit: 36

Plastic Limit: 20

Plasticity Index: 16

USCS CLASSIFICATION:

CL

Tested By: M. Wolfe Date: 7/17/2007
Reviewed By: MW Date: 8/8/07

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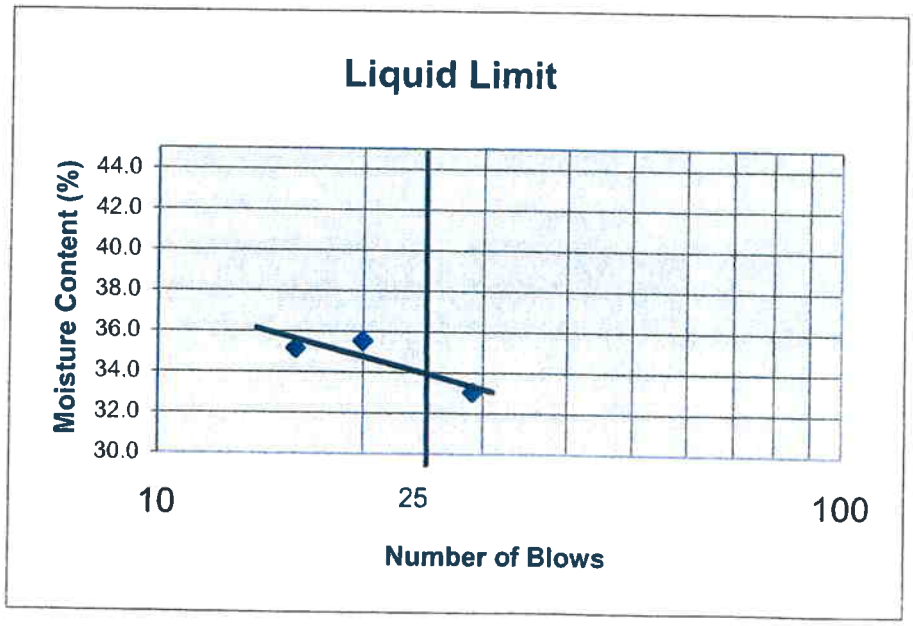
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Engineering, Environmental & Information Services

Report of Liquid Limit, Plastic Limit & Plasticity Index ASTM D4318

Project Name: GA South - Murfreesboro Rd
Project Number: 100-06-0008
Equipment Used: Liquid Limit Device #2, Oven, Ohaus 3kg Balance
Sample ID: B-2 SS-2 (3.0-4.5')
Test Date: 7/16/2007

Sample Description: Silty Clay, brown, moist, firm

	Liquid Limit				Plastic Content		
Tare No	AA	FF	G-5		LL	Z3	
Wet Soil and Tare	23.85	27.99	25.46		17.15	17.67	
Dry Soil and Tare	20.59	23.44	21.90		16.12	16.71	
Wt. of Water	3.26	4.55	3.56		1.03	0.96	
Tare Wt.	10.72	10.63	11.77		10.76	11.81	
Dry Soil	9.87	12.81	10.13		5.36	4.90	
Moisture content%	33.0	35.5	35.1		19.2	19.6	
No. Of blows	29	20	16		Average:	19	
Required Blows	***25-35***	***20-30***	***15-25***				



Liquid Limit: 34

Plastic Limit: 19

Plasticity Index: 15

USCS CLASSIFICATION:

CL

Tested By: M. Wolfe Date: 7/17/2007
Reviewed By: MDL Date: 8/8/07

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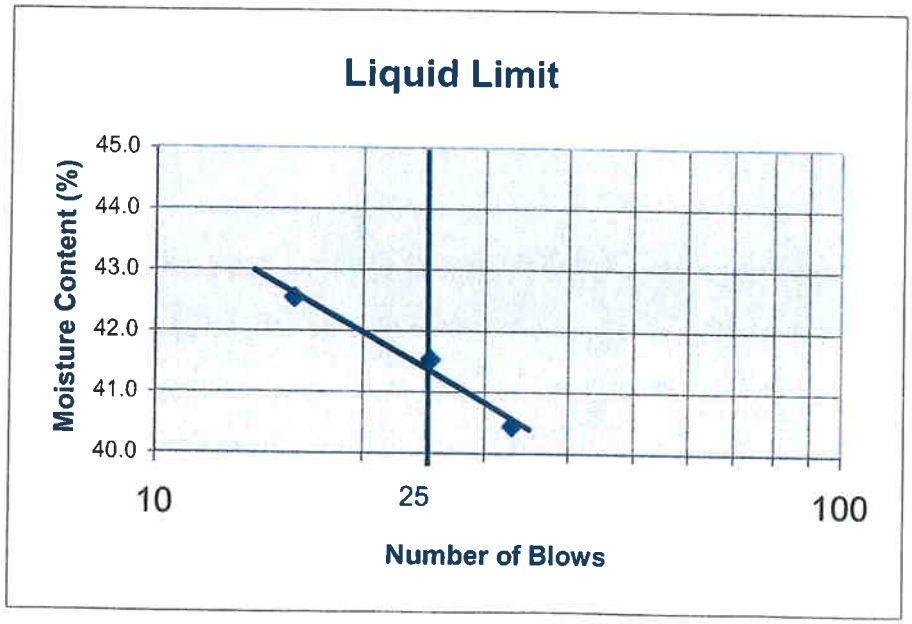
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Engineering, Environmental & Information Services

Report of Liquid Limit, Plastic Limit & Plasticity Index ASTM D4318

Project Name:	GA South - Murfreesboro Rd	Sample ID:	B-2 SS-3
Project Number:	100-06-0008	Test Date:	7/17/2007
Equipment Used:	Liquid Limit Device #2, Oven, Ohaus 3kg Balance		

Sample Description: Silty Clay, trace sand, brown, moist, firm

	Liquid Limit				Plastic Content		
Tare No	A3	2	A17		Z	B	
Wet Soil and Tare	33.90	30.12	36.93		27.69	26.18	
Dry Soil and Tare	29.20	25.34	31.23		25.21	24.15	
Wt. of Water	4.70	4.78	5.70		2.48	2.03	
Tare Wt.	17.58	13.83	17.83		16.11	16.18	
Dry Soil	11.62	11.51	13.40		9.10	7.97	
Moisture content%	40.4	41.5	42.5		27.3	25.5	
No. Of blows	33	25	16		Average:	26	
Required Blows	***25-35***	***20-30***	***15-25***				



Liquid Limit: 42

Plastic Limit: 26

Plasticity Index: 16

USCS CLASSIFICATION:

CL

Tested By: A. Biney Date: 7/18/2007

Reviewed By: MDL Date: 8/8/07

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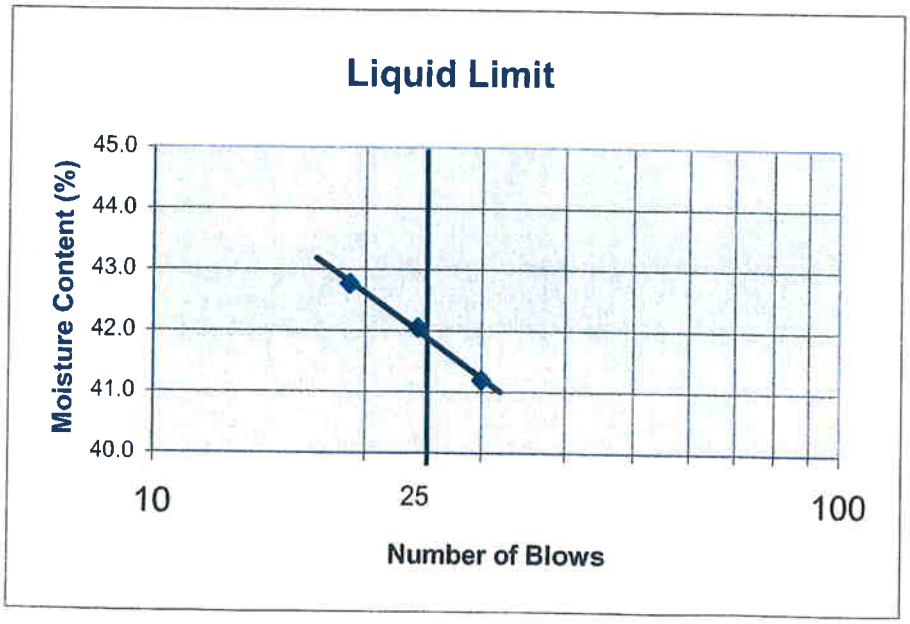
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Report of Liquid Limit, Plastic Limit & Plasticity Index ASTM D4318

Project Name:	GA South - Murfreesboro Rd	Sample ID:	B-3 SS-1
Project Number:	100-06-0008	Test Date:	7/17/2007
Equipment Used:	Liquid Limit Device #2, Oven, Ohaus 3kg Balance		

Sample Description: Silty Clay, brown, dry, stiff

	Liquid Limit				Plastic Content		
Tare No	Z3	CC	33		51	JJ	
Wet Soil and Tare	21.52	21.93	24.25		23.30	19.7	
Dry Soil and Tare	18.69	18.60	20.17		21.08	18.03	
Wt. of Water	2.83	3.33	4.08		2.22	1.67	
Tare Wt.	11.82	10.68	10.63		11.76	10.6	
Dry Soil	6.87	7.92	9.54		9.32	7.43	
Moisture content%	41.2	42.0	42.8		23.8	22.5	
No. Of blows	30	24	19		Average:	23	
Required Blows	***25-35***	***20-30***	***15-25***				



Liquid Limit: 42

Plastic Limit: 23

Plasticity Index: 19

USCS CLASSIFICATION:

CL

Tested By: A. Biney Date: 7/18/2007

Reviewed By: MDL Date: 8/8/07

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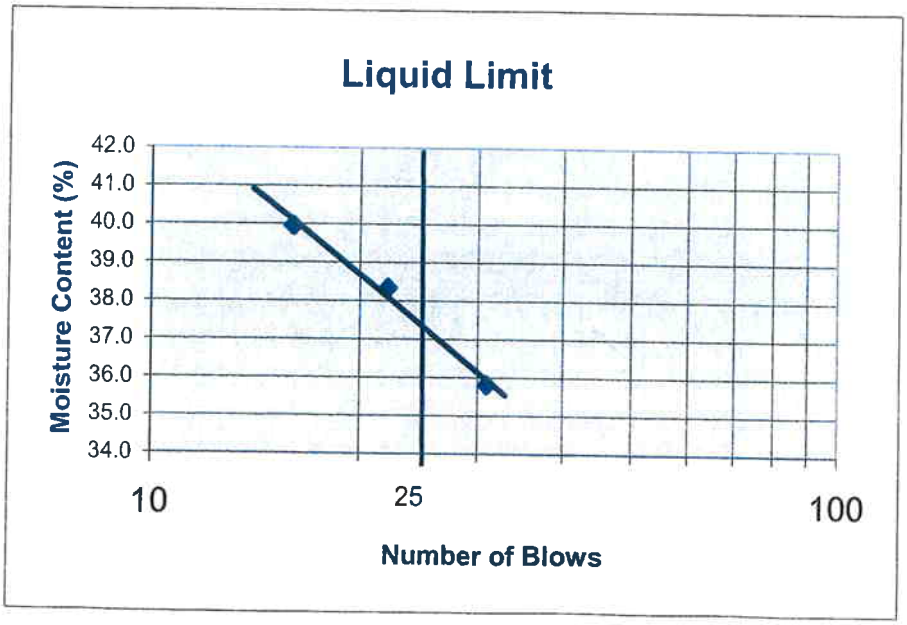
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Report of Liquid Limit, Plastic Limit & Plasticity Index ASTM D4318

Project Name:	GA South - Murfreesboro Rd	Sample ID:	B-3 SS-2
Project Number:	100-06-0008	Test Date:	7/17/2007
Equipment Used:	Liquid Limit Device #2, Oven, Ohaus 3kg Balance		

Sample Description: Silty Clay, trace sand, brown, moist, firm

	Liquid Limit				Plastic Content		
Tare No	A4	28	A21		A23	A	
Wet Soil and Tare	34.96	33.07	40.56		28.86	25.72	
Dry Soil and Tare	30.57	27.70	34.02		26.93	23.93	
Wt. of Water	4.39	5.37	6.54		1.93	1.79	
Tare Wt.	18.31	13.69	17.65		18.03	16.05	
Dry Soil	12.26	14.01	16.37		8.90	7.88	
Moisture content%	35.8	38.3	40.0		21.7	22.7	
No. Of blows	31	22	16		Average:	22	
Required Blows	***25-35***	***20-30***	***15-25***				



Liquid Limit: 38

Plastic Limit: 22

Plasticity Index: 16

USCS CLASSIFICATION:

CL

Tested By: A. Biney Date: 7/18/2007

Reviewed By: MDL Date: 8/8/07

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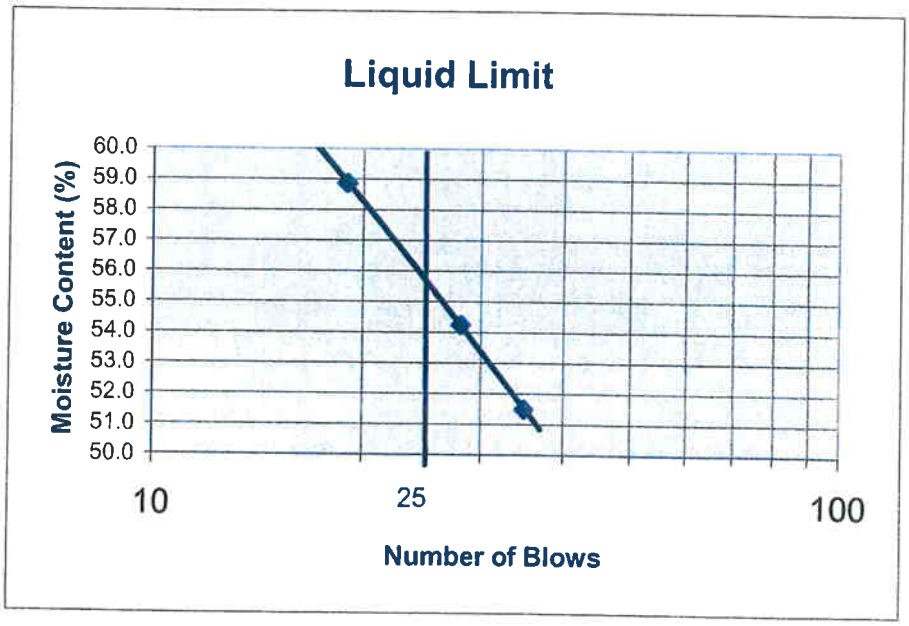
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Report of Liquid Limit, Plastic Limit & Plasticity Index ASTM D4318

Project Name:	GA South - Murfreesboro Rd	Sample ID:	B-3 SS-3
Project Number:	100-06-0008	Test Date:	7/17/2007
Equipment Used:	Liquid Limit Device #2, Oven, Ohaus 3kg Balance		

Sample Description: Clay, brown, moist, soft-firm

	Liquid Limit				Plastic Content		
Tare No	JK	K-5	YY		ZZ	X6	
Wet Soil and Tare	22.88	27.61	30.12		15.99	16.21	
Dry Soil and Tare	18.72	22.04	23.35		14.88	15.32	
Wt. of Water	4.16	5.57	6.77		1.11	0.89	
Tare Wt.	10.64	11.77	11.85		10.59	11.78	
Dry Soil	8.08	10.27	11.50		4.29	3.54	
Moisture content%	51.5	54.2	58.9		25.9	25.1	
No. Of blows	35	28	19		Average:	26	
Required Blows	***25-35***	***20-30***	***15-25***				



Liquid Limit: 56

Plastic Limit: 26

Plasticity Index: 30

USCS CLASSIFICATION:

CH

Tested By: A. Biney Date: 7/18/2007

Reviewed By: MAL Date: 8/8/07

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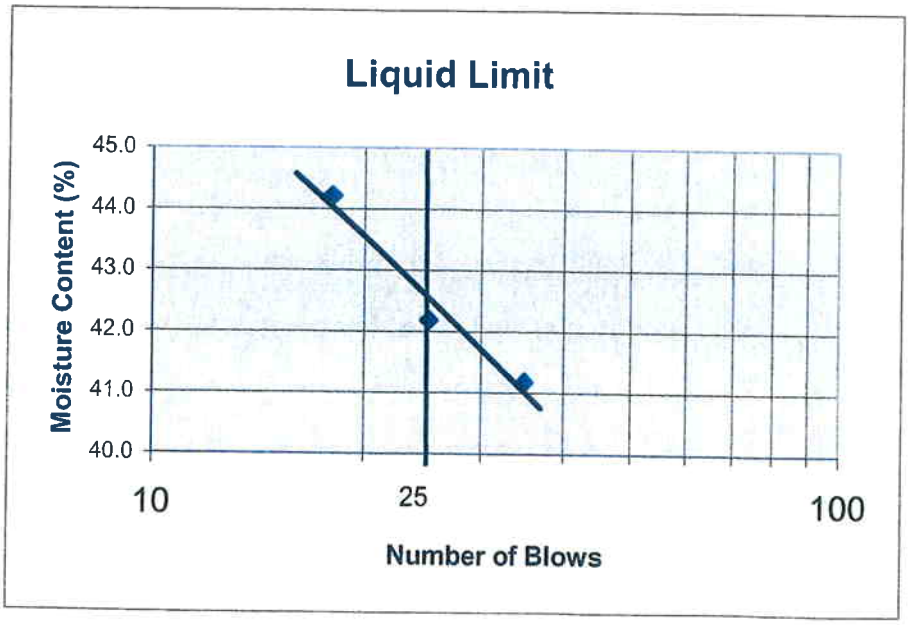
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Report of Liquid Limit, Plastic Limit & Plasticity Index ASTM D4318

Project Name: GA South - Murfreesboro Rd
Project Number: 100-06-0008
Sample ID: B-3 SS-4
Test Date: 7/17/2007
Equipment Used: Liquid Limit Device #2, Oven, Ohaus 3kg Balance

Sample Description: Silty Clay, trace sand, brown, very moist, firm

	Liquid Limit				Plastic Content		
Tare No	FF	AA	AA6		G-5	1G	
Wet Soil and Tare	24.08	26.76	27.34		22.53	28.77	
Dry Soil and Tare	20.16	22.01	22.53		20.23	25.31	
Wt. of Water	3.92	4.75	4.81		2.30	3.46	
Tare Wt.	10.64	10.75	11.65		11.80	11.79	
Dry Soil	9.52	11.26	10.88		8.43	13.52	
Moisture content%	41.2	42.2	44.2		27.3	25.6	
No. Of blows	35	25	18		Average:	26	
Required Blows	***25-35***	***20-30***	***15-25***				



Liquid Limit: 43

Plastic Limit: 26

Plasticity Index: 17

USCS CLASSIFICATION:

CL

Tested By: A. Biney Date: 7/18/2007
Reviewed By: MBR Date: 8/8/07

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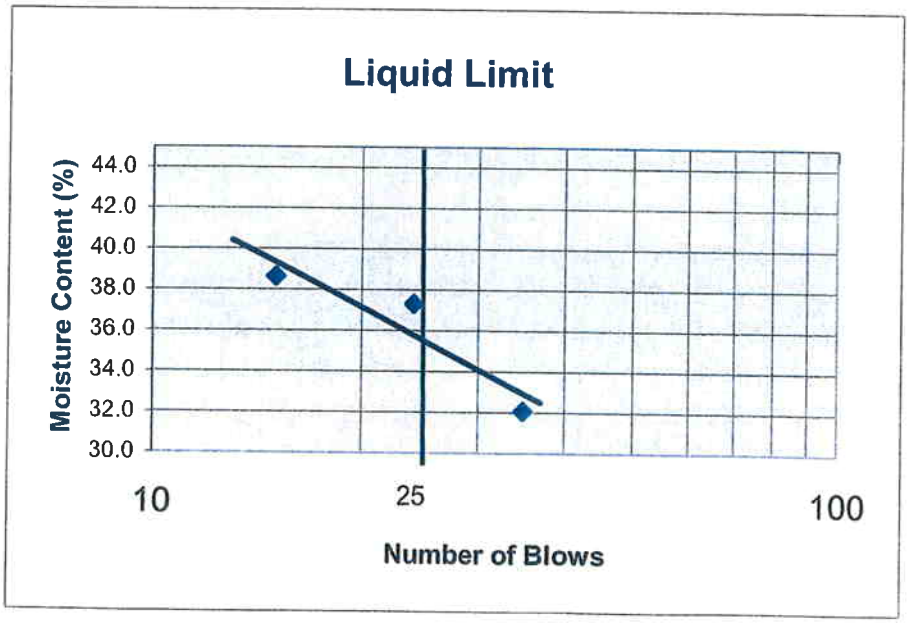
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Report of Liquid Limit, Plastic Limit & Plasticity Index ASTM D4318

Project Name:	GA South - Murfreesboro Rd	Sample ID:	B-4 SS-1
Project Number:	100-06-0008	Test Date:	7/17/2007
Equipment Used:	Liquid Limit Device #2, Oven, Ohaus 3kg Balance		

Sample Description: Silty Clay, brown, dry, hard

	Liquid Limit				Plastic Content		
Tare No	HH	Q	A18		50	A6	
Wet Soil and Tare	39.01	36.91	35.33		24.65	28.7	
Dry Soil and Tare	33.42	31.21	30.39		22.88	26.99	
Wt. of Water	5.59	5.70	4.94		1.77	1.71	
Tare Wt.	15.98	15.93	17.60		13.97	17.97	
Dry Soil	17.44	15.28	12.79		8.91	9.02	
Moisture content%	32.1	37.3	38.6		19.9	19.0	
No. Of blows	35	24	15		Average:	19	
Required Blows	***25-35***	***20-30***	***15-25***				



Liquid Limit: 36

Plastic Limit: 19

Plasticity Index: 17

USCS CLASSIFICATION:

CL

Tested By: A. Biney Date: 7/18/2007

Reviewed By: MDL Date: 8/8/07

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KSWA

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REPORT OF NATURAL MOISTURE CONTENT (ASTM D 2216)

Project Name: GA South - Murfreesboro Road
Project Number: 100-06-0008
Equipment Used: Metal Tares, Oven, Ohaus 3kg Balance, Pocket Penetrometer
Remarks: Testing performed three days after receiving samples in lab.

Sample ID	Sample Depth	Tare Number	Tare Weight (grams)	Tare and Wet Soil Weight (grams)	Tare and Dry Soil Weight (grams)	Water Weight (grams)	Percent Moisture	Pocket Pen. (tsf)
B-1	0-1.5	51	11.75	48.08	44.33	3.75	11.5	>4.5
B-2	0-1.5	JK	10.64	36.22	32.98	3.24	14.5	>4.5
B-2	3.0-4.5	K-5	11.76	34.02	29.76	4.26	23.7	3.50
B-2	6.0-7.5	X6	11.79	50.29	40.11	10.18	35.9	1.50
B-3	0-1.5	X2	11.81	57.62	51.58	6.04	15.2	2.25
B-3	3.0-4.5	33	10.63	51.81	44.79	7.02	20.6	2.75
B-3	6.0-7.5	ZZ	10.58	59.57	49.46	10.11	26.0	3.25
B-3	9.0-10.5	CC	10.67	42.37	35.85	6.52	25.9	1.00
B-4	0-1.2	JJ	10.59	58.87	54.07	4.80	11.0	>4.5

Tested By: M. Wolfe Date: 7/16/2007

Reviewed By: MBL Date: 8/8/07

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