GEOTECHNICAL EXPLORATION

BLUE ROCK ROAD & CHEVIOT ROAD

NORTH INTERSECTION IMPROVEMENTS

HAMILTON COUNTY, OHIO

Prepared for: **County of Hamilton**
Hamilton County Engineer
Thelen Project No.: **060547NE**

THELEN ASSOCIATES, INC.
Geotechnical • Testing Engineers

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August 29, 2006

County of Hamilton
Hamilton County Engineer
10480 Burlington Road
Cincinnati, Ohio 45231

Attention: Mr. Steven N. Reutelshofer

Re: Geotechnical Exploration
Blue Rock Road & Cheviot Road
North Intersection Improvements
Hamilton County, Ohio

Ladies and Gentlemen:

Contained herein are the results of our geotechnical exploration for the proposed improvements to the north intersection of Blue Rock Road and Cheviot Road in Colerain Township, Hamilton County, Ohio. Our services were performed in accordance with our Professional Services Contract with the County of Hamilton. Our estimate of services was provided on our Proposal-Agreement N26108 dated May 22, 2006. Our services were authorized by Mr. Steven N. Reutelshofer, Hamilton County Engineer, in a meeting with our Ms. Nancy M. Goins on June 1, 2006.

We are enclosing with this report a reprint of "Important Information About Your Geotechnical Engineering Report" published by ASFE, Professional Firms Practicing in the Geosciences, which our firm would like to introduce to you at this time.

We appreciate the opportunity to provide the geotechnical exploration for this project. Should you have any questions concerning the information, conclusions or recommendations contained in this report, please do not hesitate to contact us.

Respectfully submitted,
THELEN ASSOCIATES, INC.

Craig M. Davis, P.E.
Staff Geotechnical Engineer

Arthur T. Sturbaum, P.E.
Senior Geotechnical Engineer

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Copies submitted: 2 - Client

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1.0 INTRODUCTION

This report contains the results of our geotechnical exploration performed at the north intersection of Blue Rock Road and Cheviot Road in Colerain Township, Hamilton County, Ohio. The improvements to the intersection will include a new alignment of Blue Rock Road and a partial widening of Cheviot Road, thereby relocating the intersection to the north of its current location. The new pavement will consist of asphalt concrete over an asphalt-treated aggregate base.

2.0 SCOPE

The purposes of this geotechnical exploration were to determine the shallow subsurface soil profile along the new alignment of Blue Rock Road and Cheviot Road, determine the engineering properties of the subsurface soils, that is their strength, classification and compressibility characteristics and to provide our recommendations for the design and installation of the new alignment. Our services included an engineering reconnaissance of the site, the performance of seven (7) test borings, laboratory testing on recovered soil samples, engineering analysis of the data and the preparation of this report.

3.0 PROJECT CHARACTERISTICS

The details regarding the proposed intersection improvements were provided to us on plan and profile drawings. These drawings also included typical sections of the pavement. These drawings show that the realignment of Blue Rock Road will begin at Project Station 12+65.00 and intersect Cheviot Road at Station 20+00.00. The realigned section of Blue Rock Road will extend through a former residential property. Proposed grades will vary from 1 to 3 feet from the existing grades. The grade changes will primarily be cuts. Thin fill will be required in small, isolated areas. The realigned section of Blue Rock Road will taper outward from its current 2 lane width to a width of 50 feet at its new intersection with Cheviot Road, presumably 4 lanes. The new Blue Rock Road alignment will be constructed along a new storm sewer system and catch basins at the north curb. The new pavement section will be constructed with surface slopes at a maximum inclination of 4 horizontal to 1 vertical.

Cheviot Road will be widened at the new intersection beginning at Station 1+13.00 and ending at Station 15+30.00. This section will widened from its current width of roughly 35 feet to a width of 50 feet at the new intersection. The widening will also include an additional turning lane from southbound Cheviot Road to westbound Blue Rock Road. Storm sewers will be relocated along the widened section. Existing pavement grades will be maintained along the centerline, however, at the new pavement edge, grade changes of 1 to 3 feet will be required. Slopes from curbs will be at a maximum inclination of 3 horizontal to 1 vertical.

The proposed pavement section will consist of asphalt concrete over an asphalt-treated aggregate base course. Edge drains are not depicted on the typical sections, however a subpavement drainage systems, such as finger drains from catch basins or perforated drain lines beneath the curbs is anticipated.

4.0 FIELD EXPLORATION

To determine the subsurface profile along the new roadway alignments, seven (7) test borings were performed. The test borings were originally located and staked by the Hamilton County Engineering department. However, due to overhead and underground utility conflicts and traffic maintenance concerns, the test borings within and along Cheviot Road were offset from their original staked location. The ground surface elevation at each test boring was interpolated from the grades depicted on the profile drawings. It is assumed that the elevations shown on the individual test boring logs are within 6 inches of their actual elevations. Therefore, the elevations shown on the test boring logs should be considered to be approximate. Elevations are related the vertical datum of Mean Sea Level (MSL). The location of the as-drilled test borings are shown on our Test Boring Plan, Drawing 060547NE-1, attached to this report. The plan view/schematic of the alignment serves as the base map for the Test Boring Plan.

The test borings were performed with a truck-mounted drill rig and continuous flight augers. Two-inch outer-diameter split-spoon samples were obtained ahead of the augers in accordance with the procedures of ASTM D1586. Three (3) inch out-diameter Shelby tube samples were obtained in accordance with ASTM D1587 at depths selected by the Project Geotechnical Engineer. Representative portions of the split-spoon samples were placed in glass jars and sealed. The Shelby tube samples were capped and taped to maintain the specimens at their in situ moisture contents. All samples were labeled in the field for proper identification.

Concurrent with the drilling operation, the Drilling Technician prepared field test boring logs of the subsurface profile noting sample types and depths, soil and bedrock stratifications, standard penetration test resistances (N-values), groundwater levels or the lack thereof and other pertinent data.

5.0 LABORATORY REVIEW

Following completion of the test borings, the samples were returned to our Soil Mechanics Laboratory where they were reviewed and visually classified by the Project Geotechnical Engineer. Representative samples were selected to determine natural moisture content, Atterberg limits, particle size distribution, unconfined compressive strength, natural dry density, California Bearing Ratio (CBR) and standard Proctor moisture-density relationship. A tabulation of the laboratory test results is included in the Appendix along with the appropriate test forms.

Based on the Drilling Technician's field logs, the results of the laboratory tests and the Engineer's visual classification of the samples, the final test boring logs were prepared. Copies of these logs are included in the Appendix along with a Soil Classification Sheet describing the terms and symbols used in their preparation.

The dashed lines on the test boring logs identify the changes between soil or bedrock types that were determined by interpolation between the samples and should be considered to be approximate. Only changes which occur within samples can be precisely determined and are indicated by solid lines on the logs. The transition between soil and bedrock types may be abrupt or gradual.

6.0 SUBSURFACE PROFILE

The general subsurface profile in the area of the new alignment of Blue Rock Road consists of 7 inches of topsoil and thin fill underlain by shallow, weathered surface soils consisting lean silty clays. Beneath the weathered surface soils, stiff native glacial and residual clays of high plasticity were encountered over the bedrock. The depth to bedrock along the new alignment of the Blue Rock Road ranged from 4.5 to 9.5 feet below the ground surface.

The general subsurface profile beneath Cheviot Road consists of 4.0 to 4.5 inches of asphalt concrete underlain by 10 inches of a cement concrete and a soil profile consisting of stiff clay fill and very stiff residual silty clays. The grass-covered area west of the western edge of Cheviot Road consists of a thin topsoil layer underlain by stiff to very stiff residual clays, then the bedrock. Bedrock in this area ranged in depth from 2.5 to 7.0 feet below the existing ground surface.

The cement concrete pavement within Cheviot Road was disintegrated in the pavement cores. The asphalt concrete appeared to consist of two (2) layers (or courses) as described on the logs of Test Borings 5 and 7.

The thin fill encountered in Test Borings 1 and 4 consisted of a silty clay and likely represents grading operations performed around the former residential structures on that parcel. The fill encountered in Test Boring 5 consists of a very stiff sandy clay and is likely a reconditioned soil subgrade prior to the paving of Cheviot Road.

The native soil profile initially consisted of a lean silty clay in Test Borings 1 through 3. These lean silty clays are derived by the near-surface weathering of denser parent soils. This weathering is caused by wet/dry and freeze/thaw cycles and the permeation of roots from vegetation. These soils are occasionally dark brown in color yet not significantly organic. The upper 3 feet of the soil profile in Test Borings 1 through 3 was desiccated or fractured due to these weathering processes. Lean silty clays are generally stiff in consistency when dry but become medium stiff in consistency when wet, such as during the seasons of winter and spring. These weathered silty clays constitute poor subgrade soils as a result of their low-density and internal fracturing. These soils are also susceptible to softening, pumping and yielding under repeated traffic applications, particularly at moisture contents above the optimum compaction levels. These soils typically degrade in strength under traffic loading conditions, resulting in a reduced subgrade soil modulus.

Beneath the weathered surface soils in Test Borings 1 through 3, native soils of glacially-derived clays were encountered. Atterberg limits classification tests performed on samples of these clays identified liquid limits ranging from 45 to 80 percent, with plasticity indices (liquid limit minus plastic limit) ranging from 25 to 51 percent. These soils classify as lean to fat clays of medium to high plasticity, CL to CH, according to the Unified Soil Classification System (USCS). Natural moisture contents ranged from 20 to 28 percent, typically at or slightly above the plastic limit. Natural dry densities ranged from 93 to 103 pounds per cubic foot (pcf). Unconfined compressive strengths ranged from 1,100 to 2,900 pounds per square foot (psf). The unconfined compressive strengths were lower than anticipated based upon the visual consistency of the soil. This is due to failure along internal soil fractures within the samples, as discussed above.

Residual silty clays and clays were encountered directly above the bedrock. Residual soils are derived by the physical and chemical weathering of the surface of the bedrock resulting in a soil-like consistency. Residual soils are identified by trace bedding planes and shale and limestone fragments indicative of the parent bedrock material. A sample from Test Boring 4 yielded a liquid limit of 62 percent and a plasticity index of 37 percent. This classifies the soil as a highly plastic clay, CH (USCS). The natural dry density of the sample was 102.3 pcf and yielded an unconfined compressive strength of 1,760 psf. This sample tested at a strength lower than its visual consistency. The natural moisture content of the residual clays was typically in the middle teens to lower twenties.

Beneath the residual clays, the bedrock was encountered. The bedrock consists of a Ordovician Age system of interbedded shale and limestone. The bedrock encountered at the site is typical for this area and weathers from the surface downward, classifying it into three (3) commonly-accepted zones in the Greater Cincinnati Area. The uppermost zone is labeled as interbedded brown, highly weathered shale and gray hard limestone where the shale portion of the bedrock has nearly weathered to a residual clay, yet still possesses the horizontally-aligned bedding planes and intact limestone layers. The intermediate zone is described as interbedded olive brown, weathered shale and gray hard limestone and is characterized by a shale component which is tougher and at lower moisture contents than the highly weathered zone. The upper and intermediate zones have weathered from the interbedded unweathered, parent gray shale and limestone.

The weathered zones of the shale bedrock were encountered at moisture contents ranging from 12 to 18 percent.

The limestone component of the bedrock consists of horizontal beds which are gray, crystalline, fossiliferous, occasionally marled and hard relative to the shale. The beds are randomly fractured and occur in layers which typically vary between ¼ inch and 8 inches in individual layer thickness, based upon experience. Thicker or layers or concentrations of layers may be encountered. The refusal of the sampling equipment, which is defined as 50 blows or more of the sampling hammer for less than 6 inches of sampler penetration, generally indicates a limestone layer greater than 2 inches in thickness when encountered in the weathered zones. Refusal is typically encountered in the unweathered, parent bedrock. Refusal was encountered at or above the bottom of the test borings. Refusal of the sampling equipment generally indicates that the bedrock is difficult to excavate with conventional bulk earthmoving equipment.

Standard Proctor moisture-density tests, ASTM D698, were performed on four (4) bag samples of auger cuttings obtained from Test Borings 1, 3, 4 and 6. Maximum dry densities ranged from 103.8 to 120.3 pcf at optimum moisture contents ranging from 18.4 to 13.2 percent, respectively. The samples obtained from Test Borings 1, 3 and 4 classified as highly plastic clays, CH (USCS). The sample obtained from Test Boring 6 consisted of degraded highly weathered shale and occasional limestone fragments. This sample classified as a lean clay, CL (USCS). California Bearing Ratio (CBR) tests were performed on the bag samples from Test Borings 1 and 4. These tests were performed on remolded samples that were compacted to roughly 100 percent of the maximum standard dry density of the material to replicate the degree of compaction that will be required during construction. California Bearing Ratios (CBR) of 6.6 and 5.2 were determined for the individual specimens at 0.2 inches of penetration.

Groundwater readings were taken during drilling and immediately upon completion of the test borings. The test borings were backfilled immediately upon completion and paved sections were patched with asphalt concrete to maintain traffic. Groundwater was only encountered in Test Boring 4 in a trace amount at a depth of 10.5 feet. This depth corresponds to a limestone bed in the highly weathered bedrock. A groundwater table, per se, does not exist at the site, however, seepage is occasionally observed at the bedrock surface and along limestone layers within the bedrock system. Groundwater is not anticipated to be an impediment to the roadway or utility installations.

7.0 CONCLUSIONS AND RECOMMENDATIONS

7.1 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 Consulting Geotechnical and Civil Engineers in the Greater Cincinnati Area, 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 roadway alignment outlined in the Project Characteristics section of this report. Because changes in surface, subsurface, climatic and economic conditions can occur with time and location, we recommend for our mutual interest 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 are referenced in the Project Characteristics section of this report. We recommend that our office be retained to review the final design documents, plans and specifications, to assess any impact changes, additions or revisions in these documents may have on the conclusions and recommendations of this geotechnical report. Any changes or modifications which are made in the field during the construction phase which alter site grading, infrastructure or other related site work should also be reviewed by our office prior to their implementation.

If conditions are encountered in the field during construction which vary from the facts of this report, we recommend that our office be contacted immediately to review the changed conditions in the field and make appropriate recommendations.

The scope of our services did not include any environmental assessment or investigation for the presence or absence of wetlands or hazardous or toxic materials in the soil, bedrock, surface water, groundwater or air, on or below or around this site.

It is our understanding that the time frame for beginning and completing the roadway improvements and site work for this project will be continuous without interruption or delay. Should interruptions or delays occur, our office should be kept appraised to determine what recommendations must be modified accordingly.

We have performed the test borings and laboratory tests for our evaluation of the site conditions and for the formulation of the conclusions and recommendations of this report. We assume no responsibility for the interpretation or extrapolation of the data by others.

The earthwork recommendations of this report presume that the earthwork will be monitored continuously by an Engineering Technician under the direction of a Registered Professional Geotechnical Engineer. We recommend that these services be contracted directly with Thelen Associates, Inc.

We recommend that a preconstruction meeting be held with the Design Civil Engineer, the General Contractor, the Excavating Contractor, the Geotechnical Engineer and any other interested parties to review the scope and schedule of the proposed earthwork and pavement subgrade preparation.

7.2 Earthwork and Utility Backfill

We presume that all fill will consist of cuts from the site or granular borrow. In general, the required earthwork will consist of 1 to 3 feet of cut and fill to provide the proposed subgrade levels along the new location of Blue Rock Road and the new edges of Cheviot Road. Subsequent to stripping topsoil and pavements in proposed fill areas and completing cuts to design subgrade levels, the exposed soils should be proofrolled with a heavily loaded tandem-axle dump truck. If yielding soils are exposed at the design depth, undercuts will be required. Deep undercuts will not likely be feasible along Cheviot Road due to the depth of existing utilities. If undercuts are limited by the presence of existing utilities, a geogrid-reinforced granular backfill or other subgrade soil improvements may be required. Undercuts, where feasible, should be extended to expose firm, unyielding soils prior to refilling.

Any new fill that is required to alter site grading, refill undercut areas or for use as utility backfill should consist of approved soil from the cuts or approved inorganic borrow with a liquid limit less than 60 percent and a plasticity index less than 35. Soils from Test Borings 3 and 4 yielded liquid limits and plasticity indices greater than the mentioned above. These soils should not be placed as a single lift but rather mixed with lean soils to create a soil mix which will meet this specification.

All fill should be placed in shallow, level layers, 6 to 8 inches in thickness, and should be compacted using appropriate equipment, such as a sheepsfoot roller or self-propelled compactor for clayey soils. If granular fill is to be used in any location it should be compacted with vibratory equipment and permanently drained to the storm sewer system.

Limestone fragments may be used as fill only if degraded to a maximum dimension of 8 inches. Limestone fragments should not be placed in a manner that impedes compaction or so that the fragments are nested together, thereby creating voids. Limestone fragments should be excluded from the final lift of fill to prevent interferences with fine grading.

We recommend that the granular backfill of new storm sewers be restricted to 1 foot above the top of pipe, and should consist of free-draining granular material (less than 3 percent fines). Granular backfill should placed in lifts of maximum of 12 inches in loose thickness and should be compacted to a minimum of 95 percent (ASTM D698). Following the placement of the granular backfill, the excavation may be backfilled to original grades using the clean clayey soils, excavated to reach the inverts and compacted to the project requirements. Under no circumstances should flushing be used to obtain compaction.

All fill should be placed at a moisture content between 2 percent below and 3 percent above the optimum moisture content as determined by the standard Proctor moisture-density test, ASTM D698 (AASHTO T99). All bulk fill for support of pavements should be compacted in accordance with Hamilton County Earthwork Regulations. These regulations are based upon Ohio Department of Transportation (ODOT) guidelines and specify a degree of compaction that is related to the maximum dry density of the soil.

ODOT Item 203 requires a degree of compaction between 98 and 102 percent for embankment fill soils. The Contractor should be aware of all governing specifications prior to beginning site work.

We recommend that edge drains be installed to collect water that infiltrates the pavement section. The soil subgrade should be crowned toward the drains to promote rapid drainage. Water trapped below pavements is a significant cause of reduced pavement serviceability and life span.

Under no circumstances should any pavement or fill be placed over frozen or saturated soils. In addition, frozen soils should not be used as compacted fill or backfill.

During construction straw bales or silt fences should be staked at points of concentrated runoff. Following completion of the development, disturbed areas which may exist beyond the pavements should be seeded and strawed or sodded for erosion protection.

All utility trenches should be shored or laid back as outlined by federal, state and local codes and in accordance to OSHA requirements to protect workers.

7.3 Pavements

We recommend that the pavements for the project be designed based on the anticipated axle loads and the frequency of loading, life cycle, reliability of design and the properties of the subgrade soils. The subgrade properties for use in formal pavement designs should be determined from a correlation between field density tests and laboratory CBR tests, or field CBR tests. Based on the laboratory CBR tests, we recommend that the pavement design be based on a subgrade CBR value of 6. We recommend that the design CBR value be verified for any imported borrow material. This can be accomplished by the performance of field CBR tests or by laboratory CBR tests performed on the borrow material prior to importation.

To prepare soil subgrades for paving following completion of bulk fill, the upper 8 inches of the subgrade soil should be manipulated as needed to bring the moisture content to within 2 percent of optimum. The subgrade should be recompacted the specified degree, a minimum of 100 percent of maximum dry density, ASTM D698. This compaction should be performed immediately prior to the placement of the asphalt base course to reduce the potential of subgrade softening due to exposure to the weather.

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APPENDIX

ASFE Report Information

Tabulation of Laboratory Tests

Unconfined Compression Test Forms

Gradation Analysis Test Forms

CBR Test Forms

Test Boring Plan, Drawing 060547NE-1

Test Boring Logs

Soil Classification Sheet

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 the risk of such outcomes, 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.*

Obtain Professional Assistance To Deal with Mold

Diverse strategies can be applied during building design, construction, operation, and maintenance to prevent significant amounts of mold from growing on indoor surfaces. To be effective, all such strategies should be devised for the *express purpose* of mold prevention, integrated into a comprehensive plan, and executed with diligent oversight by a professional mold prevention consultant. Because just a small amount of water or moisture can lead to the development of severe mold infestations, a number of mold prevention strategies focus on keeping building surfaces dry. While groundwater, water infiltration, and similar issues may have been addressed as part of the geotechnical engineering study whose findings are conveyed in this report, the geotechnical engineer in charge of this project is not a mold prevention consultant; ***none of the services performed in connection with the geotechnical engineer's study were designed or conducted for the purpose of mold prevention. Proper implementation of the recommendations conveyed in this report will not of itself be sufficient to prevent mold from growing in or on the structure involved.***

Rely, on Your ASFE-Member Geotechncial Engineer for Additional Assistance

Membership in ASFE/The Best People on Earth 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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COUNTY OF HAMILTON COUNTY
HAMILTON COUNTY ENGINEER
GEOTECHNICAL SERVICES
BLUE ROCK & CHEVIOT ROAD
NORTH INTERSECTION IMPROVEMENTS
HAMILTON COUNTY, OHIO
060547NE

TABULATION OF LABORATORY TESTS

Boring No.	Sample No.	Depth, ft. From	Depth, ft. To	Moisture Content, %	Atterberg Limits % LL	PL	PI	Natural Dry Density, pcf	Unconfined Compressive Strength, psf	USCS Class.
1	PT-7B	2.3	2.8	27.2	56	23	33	93.6	1140	CH
	2	2.5	4.0	17.3						
	3	5.0	6.5	21.1						
2	PT-2	2.5	3.0	20.7	45	20	25	103.0	2850	CL
	3	4.0	5.5	26.6						
3	PT-2	3.4	3.9	28.1	80	29	51	93.6	1360	CH
4	1	0.0	1.5	19.5						
	PT-2	3.3	3.8	22.0	62	25	37	102.3	1760	CH
	3	4.5	6.0	18.3						
5	1	1.0	2.5	30.4						
	2	2.5	4.0	26.2						
	3	5.0	6.5	19.3						
6	1B	0.2	1.5	13.7						CH
	3	2.5	4.0	16.1						CH
	4	5.0	5.5	12.0						
	5	7.5	8.5	17.5						
	6	10.0	10.5	14.3						CL
7	1	1.4	2.5	17.6						
	2	2.5	4.0	16.8						CH

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COUNTY OF HAMILTON COUNTY
HAMILTON COUNTY ENGINEER
GEOTECHNICAL SERVICES
BLUE ROCK & CHEVIOT ROAD
NORTH INTERSECTION IMPROVEMENTS
HAMILTON COUNTY, OHIO
060547NE

TABULATION OF LABORATORY TESTS

Boring No.	Sample No.	Depth, ft.		Moisture Content, %	Standard Proctor		Atterberg Limits %			Gradation Analysis, %				CBR @ 0.2" Penetration	USCS Class.
		From	To		Max. Dry Density, pcf	Optimum Moisture, %	LL	PL	PI	Gravel	Sand	Silt	Clay		
1	Bag	1.0	3.0	30.5	103.8	18.4	52	28	24	2	6	27	65	6.6	CH
															CH
3	Bag	5.0	7.0	31.8	109.4	17.0	52	25	27	11	14	75			CH
4	Bag	3.0	5.0	19.6	113.1	15.1	51	22	29	6	17	25	52	5.2	CH
6	Bag	3.0	5.0	9.3	120.3	13.2	37	17	20	13	22	65			CL

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UNCONFINED COMPRESSIVE STRENGTH OF COHESIVE SOIL, ASTM - D2166
UNIT WEIGHT AND NATURAL MOISTURE

CLIENT : **Hamilton County Engineer**
PROJECT : **G.S., Blue Rock & Cheviot Road Improvements**
LOCATION : **Hamilton County, Ohio**

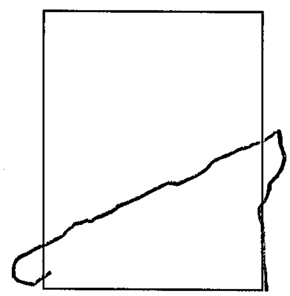
PROJECT NUMBER : **060547NE**
LAB NUMBER :
BORING NUMBER : **1** SAMPLE NUMBER : **PT-7B**
DEPTH (FT.): **2.3** to **2.8**
SAMPLE DESCRIPTION : **Brown moist medium stiff CLAY, trace sand with iron oxide stains**

SAMPLE OBTAINED BY : **SHELBY TUBE** CONDITION **UNTRIMMED** DATE : **07/14/06**

NATURAL UNIT WEIGHT

AVERAGE DIAMETER (in.)	2.85
HEIGHT (in.)	5.58
HEIGHT TO DIAMETER RATIO	1.96
AVERAGE AREA (sq. ft.)	0.0443
VOLUME (cu. ft.)	0.0206
WET WEIGHT (lbs.)	2.45
DRY WEIGHT (lbs.)	1.93
DRY DENSITY (pcf)	**93.6**

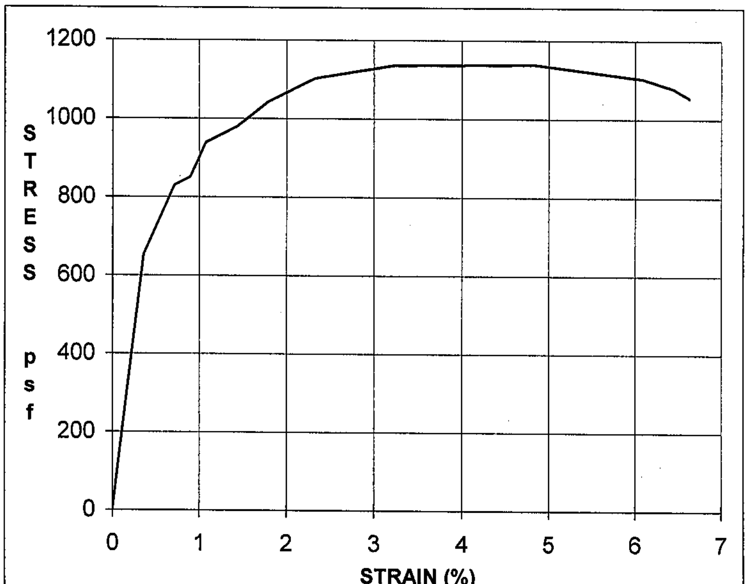
FAILURE SHAPE



WATER CONTENT AFTER SHEAR

CAN NUMBER	B1
WET WEIGHT + CAN (lbs.)	2.33
DRY WEIGHT + CAN (lbs.)	1.92
WEIGHT WATER (lbs.)	0.41
WEIGHT CAN (lbs.)	0.42
WEIGHT SOLID (lbs.)	1.50
MOISTURE (%)	**27.2**
LOAD CELL NUMBER	CELL

DEFORM DIAL .001 IN.	LOAD CELL	LOAD LBS.	STRAIN %	CORR. AREA SQ. FT.	STRESS PSF
0	0	0	0	0.0443	0
20	29.0	29.0	0.4	0.0445	652
40	37.0	37.0	0.7	0.0446	829
50	38.0	38.0	0.9	0.0447	850
55	40.0	40.0	1.0	0.0448	894
60	42.0	42.0	1.1	0.0448	938
70	43.0	43.0	1.3	0.0449	958
80	44.0	44.0	1.4	0.0450	979
100	47.0	47.0	1.8	0.0451	1042
130	50.0	50.0	2.3	0.0454	1102
180	52.0	52.0	3.2	0.0458	1136
270	53.0	53.0	4.8	0.0466	1138
340	52.0	52.0	6.1	0.0472	1102
360	51.0	51.0	6.5	0.0474	1077
370	50.0	50.0	6.6	0.0475	1053



AVERAGE RATE OF STRAIN TO FAILURE (% per minute)	**1.1**
STRAIN AT FAILURE (%)	**4.8**
UNCONFINED COMPRESSIVE STRENGTH (psf)	**1,140**
SHEAR STRENGTH (psf)	**570**

REMARKS : The sample failed along an existing fracture. Tested strength was lower than its visual consistency.

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UNCONFINED COMPRESSIVE STRENGTH OF COHESIVE SOIL, ASTM - D2166
UNIT WEIGHT AND NATURAL MOISTURE

CLIENT : **Hamilton County Engineer**
PROJECT : **G.S., Blue Rock & Cheviot Road Improvements**
LOCATION : **Hamilton County, Ohio**

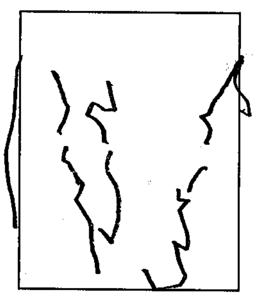
PROJECT NUMBER : **060547NE** LAB NUMBER :
BORING NUMBER : **2** SAMPLE NUMBER : **PT-2** DEPTH (FT.): **2.5** to **3.0**
SAMPLE DESCRIPTION : **Mottled brown, trace gray stiff SILTY CLAY with clay seams and iron oxide stains**

SAMPLE OBTAINED BY : **SHELBY TUBE** CONDITION **UNTRIMMED** DATE : **07/14/06**

NATURAL UNIT WEIGHT

AVERAGE DIAMETER (in.)	2.85
HEIGHT (in.)	5.58
HEIGHT TO DIAMETER RATIO	1.96
AVERAGE AREA (sq. ft.)	0.0443
VOLUME (cu. ft.)	0.0206
WET WEIGHT (lbs.)	2.56
DRY WEIGHT (lbs.)	2.12
DRY DENSITY (pcf)	**103.0**

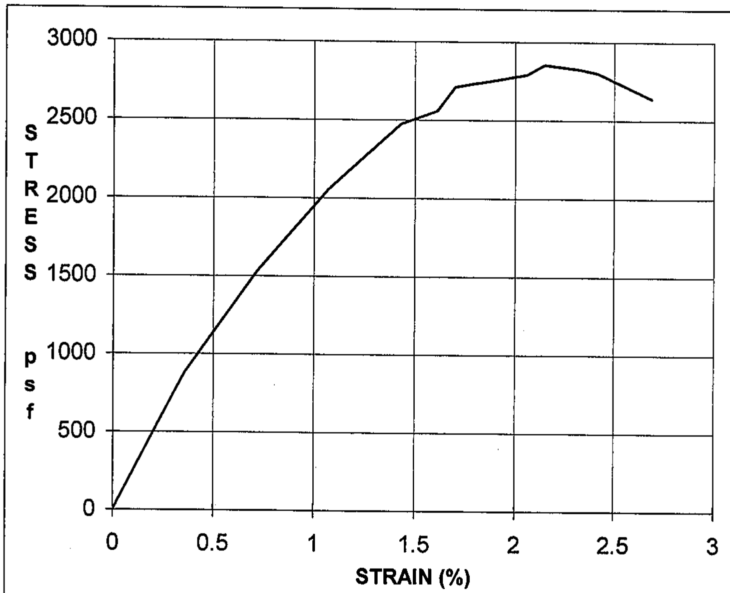
FAILURE SHAPE



WATER CONTENT AFTER SHEAR

CAN NUMBER	OH5
WET WEIGHT + CAN (lbs.)	3.02
DRY WEIGHT + CAN (lbs.)	2.58
WEIGHT WATER (lbs.)	0.44
WEIGHT CAN (lbs.)	0.48
WEIGHT SOLID (lbs.)	2.10
MOISTURE (%)	**20.7**
LOAD CELL NUMBER	CELL

DEFORM DIAL .001 IN.	LOAD CELL	LOAD LBS.	STRAIN %	CORR. AREA SQ. FT.	STRESS PSF
0	0	0	0	0.0443	0
20	39.0	39.0	0.4	0.0444	878
40	68.0	68.0	0.7	0.0446	1526
60	92.0	92.0	1.1	0.0447	2057
80	111.0	111.0	1.4	0.0449	2472
90	115.0	115.0	1.6	0.0450	2557
95	122.0	122.0	1.7	0.0450	2710
100	123.0	123.0	1.8	0.0451	2730
105	124.0	124.0	1.9	0.0451	2749
110	125.0	125.0	2.0	0.0451	2769
115	126.0	126.0	2.1	0.0452	2789
120	129.0	129.0	2.1	0.0452	2852
130	128.0	128.0	2.3	0.0453	2825
135	127.0	127.0	2.4	0.0453	2800
150	120.0	120.0	2.7	0.0455	2639



AVERAGE RATE OF STRAIN TO FAILURE (% per minute)	**1.1**
STRAIN AT FAILURE (%)	**2.1**
UNCONFINED COMPRESSIVE STRENGTH (psf)	**2,850**
SHEAR STRENGTH (psf)	**1425**

REMARKS :

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UNCONFINED COMPRESSIVE STRENGTH OF COHESIVE SOIL, ASTM - D2166
UNIT WEIGHT AND NATURAL MOISTURE

CLIENT : **Hamilton County Engineer**
PROJECT : **G.S., Blue Rock & Cheviot Road Improvements**
LOCATION : **Hamilton County, Ohio**

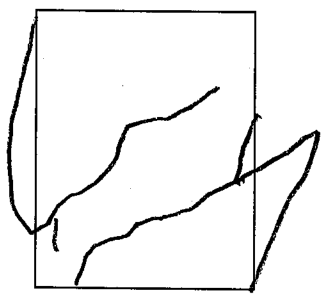
PROJECT NUMBER : **060547NE**
LAB NUMBER :
BORING NUMBER : **3**
SAMPLE NUMBER : **PT-2**
DEPTH (FT.): **3.4** to **3.9**
SAMPLE DESCRIPTION : **Brown moist medium stiff CLAY with roots and iron oxide stains**

SAMPLE OBTAINED BY : **SHELBY TUBE**
CONDITION **UNTRIMMED**
DATE : **07/13/06**

NATURAL UNIT WEIGHT

AVERAGE DIAMETER (in.)	2.85
HEIGHT (in.)	5.58
HEIGHT TO DIAMETER RATIO	1.96
AVERAGE AREA (sq. ft.)	0.0443
VOLUME (cu. ft.)	0.0206
WET WEIGHT (lbs.)	2.47
DRY WEIGHT (lbs.)	1.93
DRY DENSITY (pcf)	**93.6**

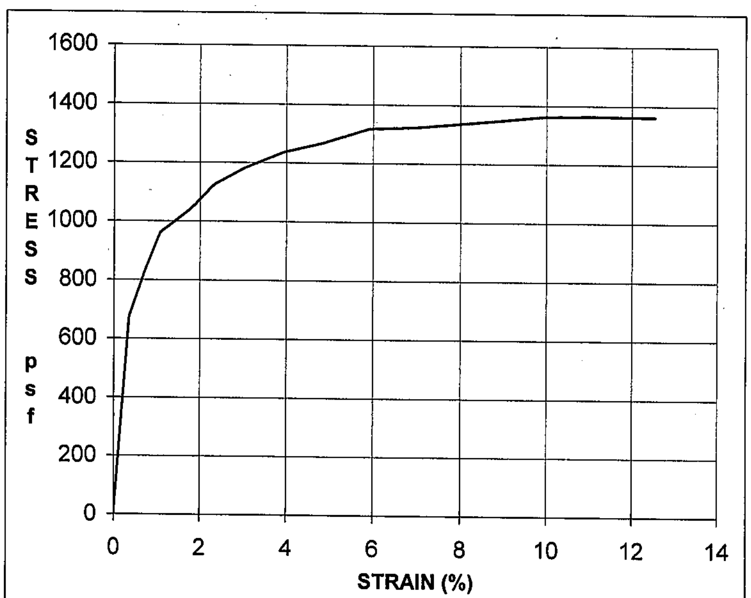
FAILURE SHAPE



WATER CONTENT AFTER SHEAR

CAN NUMBER	ky15
WET WEIGHT + CAN (lbs.)	2.50
DRY WEIGHT + CAN (lbs.)	2.06
WEIGHT WATER (lbs.)	0.44
WEIGHT CAN (lbs.)	0.50
WEIGHT SOLID (lbs.)	1.56
MOISTURE (%)	**28.1**
LOAD CELL NUMBER	CELL

DEFORM DIAL .001 IN.	LOAD CELL	LOAD LBS.	STRAIN %	CORR. AREA SQ. FT.	STRESS PSF
0	0	0	0	0.0443	0
20	30.0	30.0	0.4	0.0445	675
40	37.0	37.0	0.7	0.0446	829
60	43.0	43.0	1.1	0.0448	960
100	47.0	47.0	1.8	0.0451	1042
130	51.0	51.0	2.3	0.0454	1124
170	54.0	54.0	3.0	0.0457	1181
220	57.0	57.0	3.9	0.0461	1236
270	59.0	59.0	4.8	0.0466	1267
330	62.0	62.0	5.9	0.0471	1316
390	63.0	63.0	7.0	0.0476	1322
475	65.0	65.0	8.5	0.0484	1342
555	67.0	67.0	9.9	0.0492	1361
620	68.0	68.0	11.1	0.0499	1364
700	69.0	69.0	12.5	0.0507	1362



AVERAGE RATE OF STRAIN TO FAILURE (% per minute)	**1.1**
STRAIN AT FAILURE (%)	**11.1**
UNCONFINED COMPRESSIVE STRENGTH (psf)	**1,360**
SHEAR STRENGTH (psf)	**680**

REMARKS : The sample failed along an existing fracture. Tested strength was lower than its visual consistency.

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UNCONFINED COMPRESSIVE STRENGTH OF COHESIVE SOIL, ASTM - D2166
UNIT WEIGHT AND NATURAL MOISTURE

CLIENT : **Hamilton County Engineer**
PROJECT : **G.S., Blue Rock & Cheviot Road Improvements**
LOCATION : **Hamilton County, Ohio**

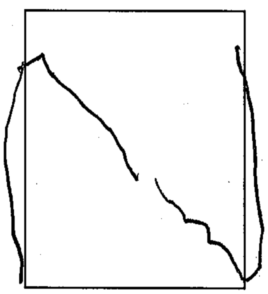
PROJECT NUMBER : **060547NE** LAB NUMBER :
BORING NUMBER : **4** SAMPLE NUMBER : **PT-2** DEPTH (FT.): **3.3** to **3.8**
SAMPLE DESCRIPTION : **Yellowish brown, trace gray moist medium stiff CLAY with bedding planes and iron oxide stains (residual)**

SAMPLE OBTAINED BY : **SHELBY TUBE** CONDITION **UNTRIMMED** DATE : **07/14/06**

NATURAL UNIT WEIGHT

AVERAGE DIAMETER (in.)	2.86
HEIGHT (in.)	5.58
HEIGHT TO DIAMETER RATIO	1.95
AVERAGE AREA (sq. ft.)	0.0447
VOLUME (cu. ft.)	0.0208
WET WEIGHT (lbs.)	2.59
DRY WEIGHT (lbs.)	2.13
DRY DENSITY (pcf)	**102.3**

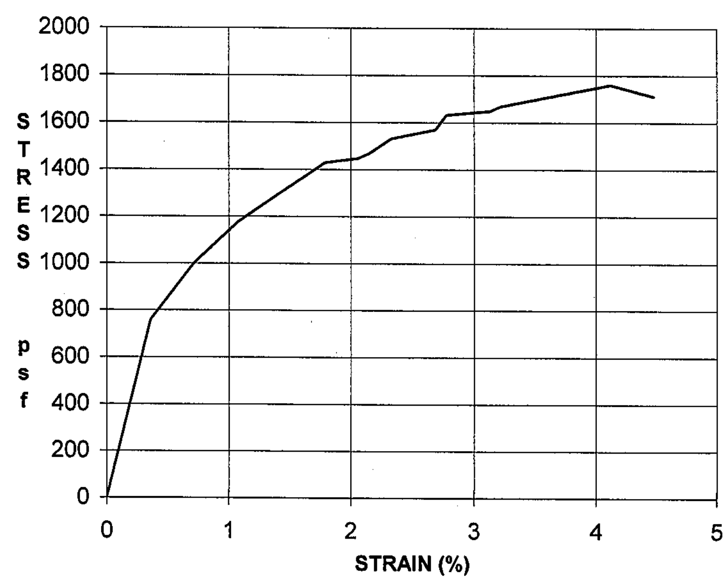
FAILURE SHAPE



WATER CONTENT AFTER SHEAR

CAN NUMBER	X3
WET WEIGHT + CAN (lbs.)	2.72
DRY WEIGHT + CAN (lbs.)	2.32
WEIGHT WATER (lbs.)	0.40
WEIGHT CAN (lbs.)	0.49
WEIGHT SOLID (lbs.)	1.83
MOISTURE (%)	**22.0**
LOAD CELL NUMBER	CELL

DEFORM DIAL .001 IN.	LOAD CELL	LOAD LBS.	STRAIN %	CORR. AREA SQ. FT.	STRESS PSF
0	0	0	0	0.0447	0
20	34.0	34.0	0.4	0.0448	758
40	45.0	45.0	0.7	0.0450	1000
60	53.0	53.0	1.1	0.0452	1173
80	59.0	59.0	1.4	0.0453	1301
100	65.0	65.0	1.8	0.0455	1428
115	66.0	66.0	2.1	0.0456	1446
120	67.0	67.0	2.2	0.0457	1467
130	70.0	70.0	2.3	0.0458	1530
150	72.0	72.0	2.7	0.0459	1568
155	75.0	75.0	2.8	0.0460	1632
175	76.0	76.0	3.1	0.0461	1647
180	77.0	77.0	3.2	0.0462	1667
230	82.0	82.0	4.1	0.0466	1759
250	80.0	80.0	4.5	0.0468	1710



AVERAGE RATE OF STRAIN TO FAILURE (% per minute)	**1.1**
STRAIN AT FAILURE (%)	**4.1**
UNCONFINED COMPRESSIVE STRENGTH (psf)	**1,760**
SHEAR STRENGTH (psf)	**880**

REMARKS : The sample failed along an existing fracture. Tested strength was lower than its visual consistency.

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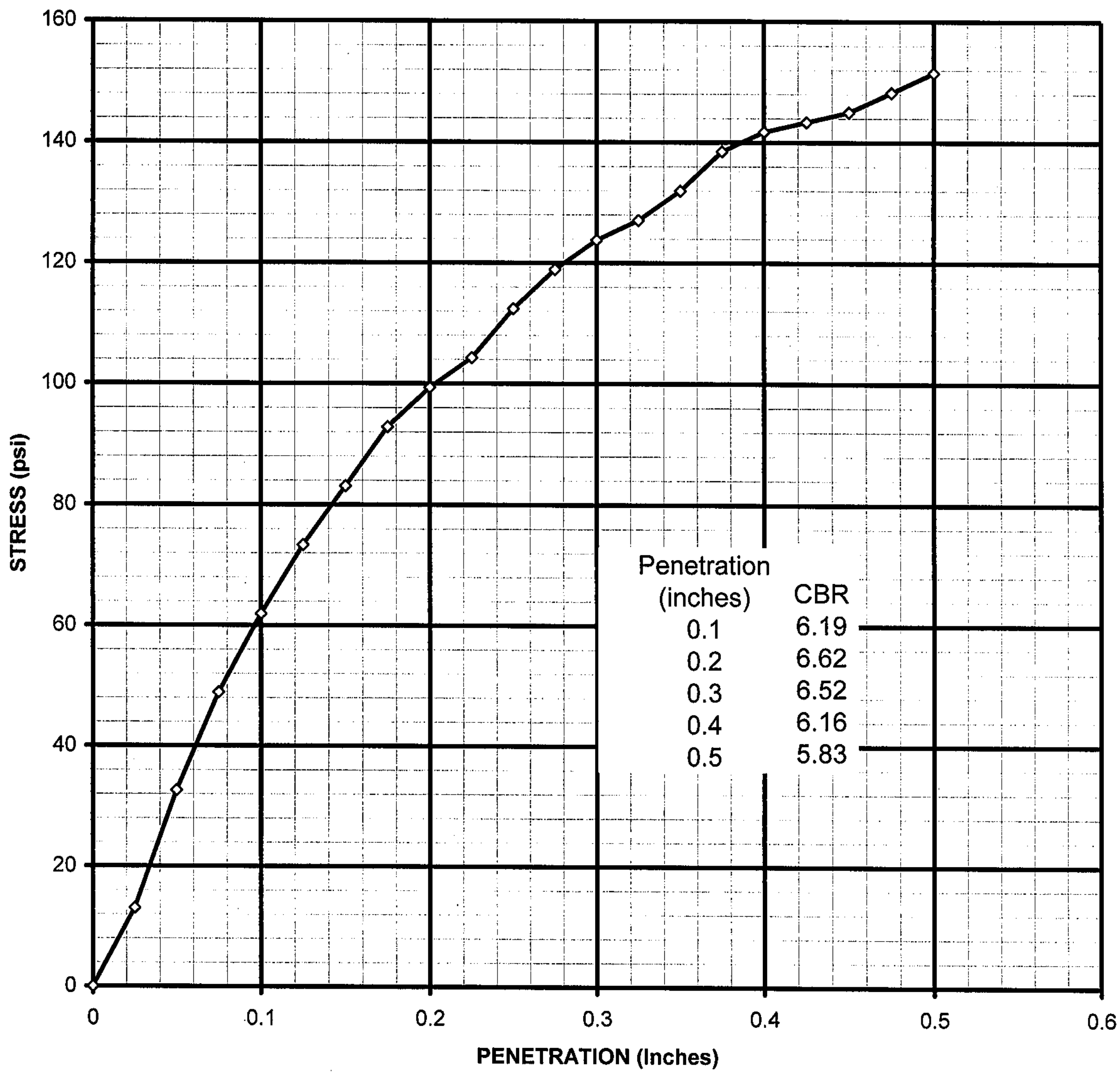
LABORATORY CBR TEST RESULT

Client :	Hamilton County Engineer

Date :	07/18/06
Project No. :	060547NE
Project :	Geotechnical Services
	Blue Rock & Cheviot Road
	Improvement
	Hamilton County, Ohio

Sample Obtained From :	**Test Boring 1, Bag Sample**	Depth :	**1.0' - 3.0'**
Sample Description :	**Brown CLAY, trace fine to coarse sand and fine gravel**		
USCS Classification :	**CH**	In Situ Moisture Content :	**30.5%**
LL = 52 PL = 28 PI = 24			
Maximum Dry Density :	**103.8 P.C.F.**	Optimum Moisture Content :	**18.4 %**
Test Type :	**Standard Proctor, ASTM D698**		
Percent compaction during CBR test = 104.0% at a dry density of 107.9 P.C.F. and 18.0% moisture			
Moisture content of top inch =	**24.4 %**		



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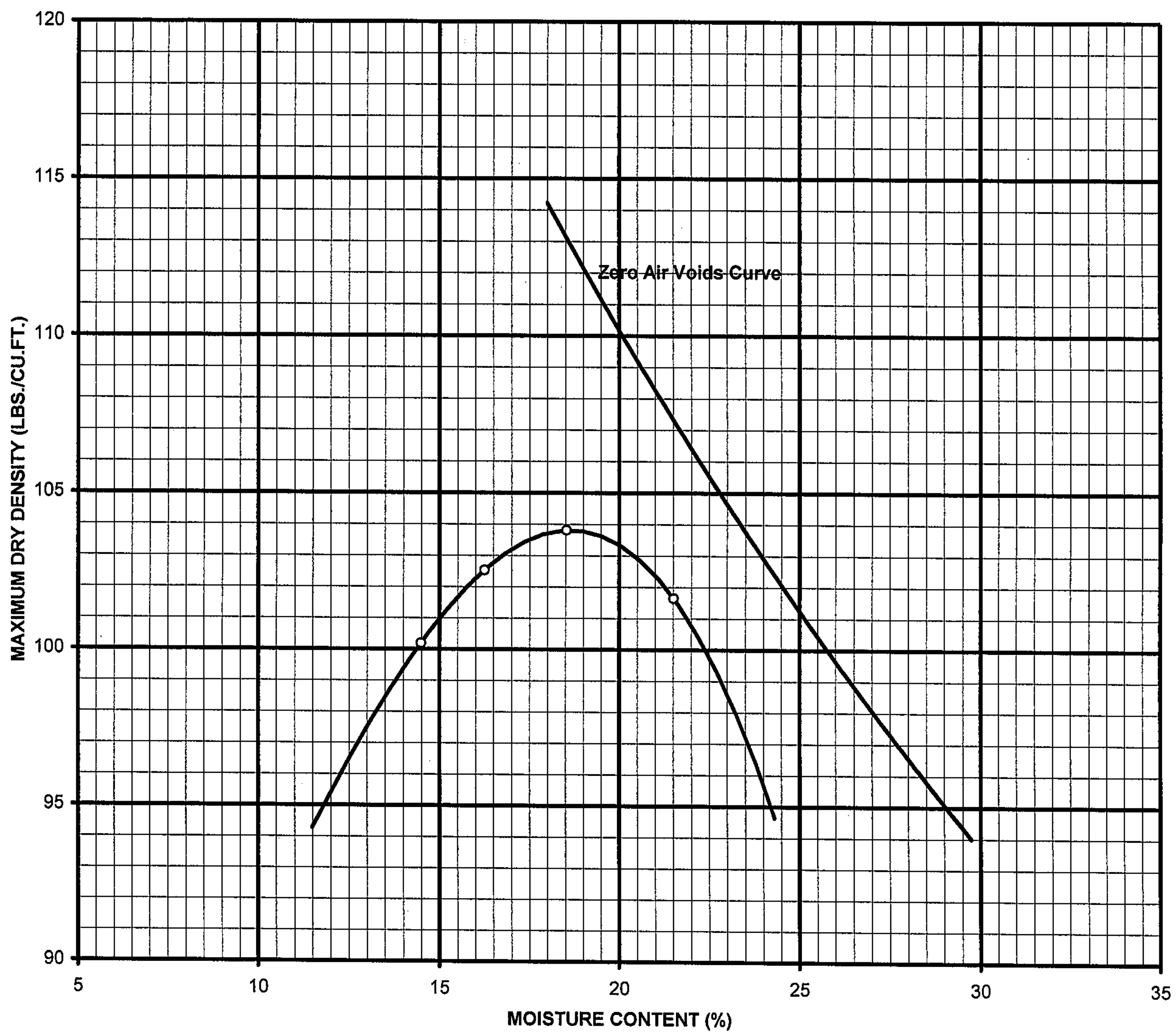
MOISTURE DENSITY TEST

Client:	Hamilton County Engineer

Date:	07/12/06
Project No.:	060547NE
Project:	Geotechnical Services
	Blue Rock & Cheviot Road
	Improvement
	Hamilton County, Ohio

Sample Obtained From:	Test Boring 1, Bag Sample	Depth:	1.0' - 3.0'
Sample Description:	Brown CLAY, trace fine to coarse sand and fine gravel		
USCS Classification:	CH	In Situ Moisture Content:	30.5%
LL = 38 PL = 24	PI = 14		
Maximum Dry Density:	103.8 P.C.F.	Optimum Moisture Content:	18.4 %
Test Type:	Standard Proctor, ASTM D698		
Method:	Method A		



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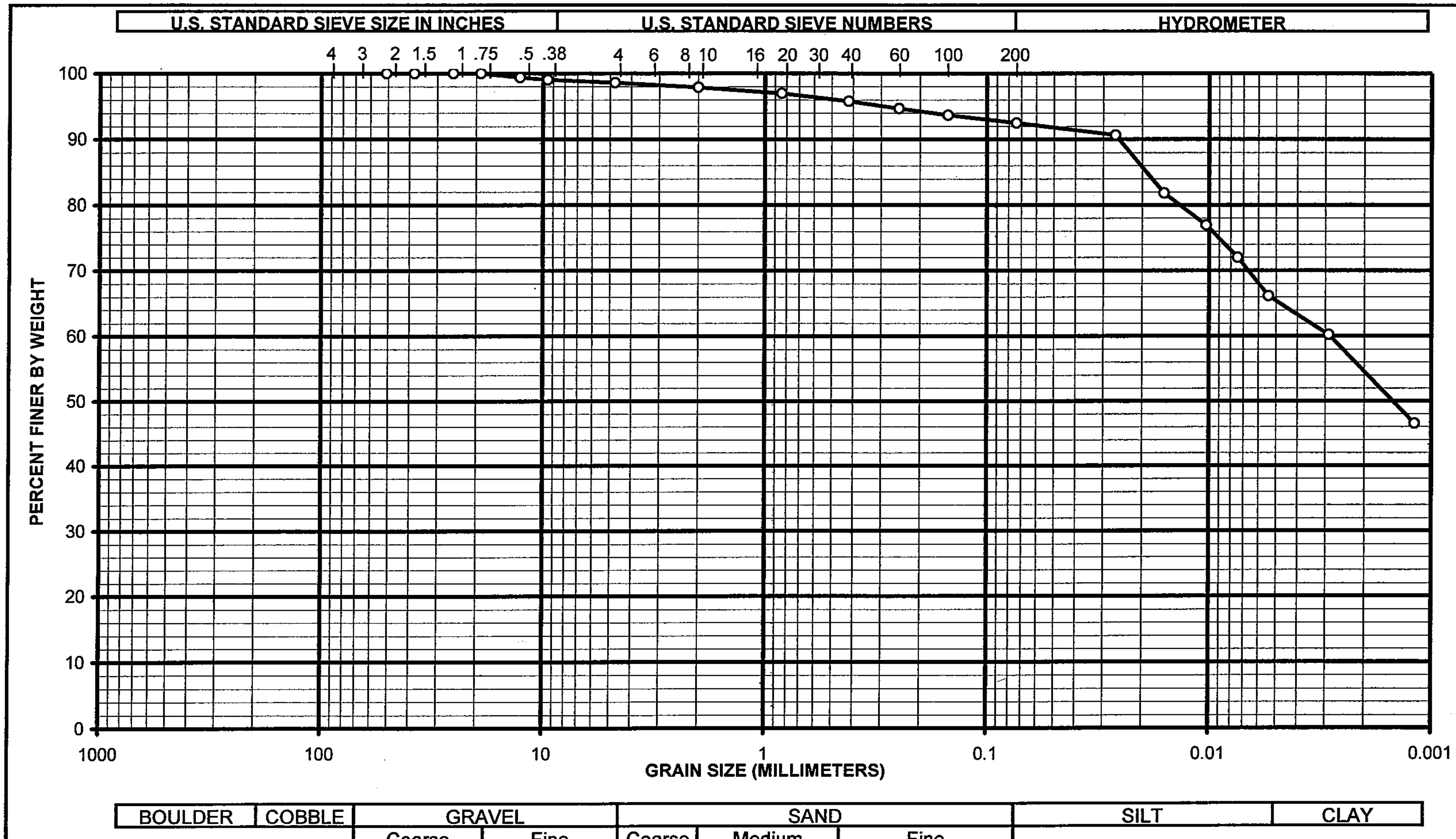
SIEVE ANALYSIS OF FINE AND COARSE AGGREGATES ASTM C-136



Boring No.	Sample No.	Elev. or Depth	Description	Nat. w%	LL	PL	PI	Client :	Hamilton County Engineer
1	Bag Sample	1.0' - 3.0'	Brown CLAY, trace fine to coarse sand and fine gravel	30.5	52	28	24	Project :	Blue Rock & Cheviot Road Improvement
									Hamilton County, Ohio
GRAVEL	SAND	SILT & CLAY							
2%	6%	27% 65%	USCS CLASSIFICATION IS CH					Project No.:	060547NE
Specification								Date :	07/14/06

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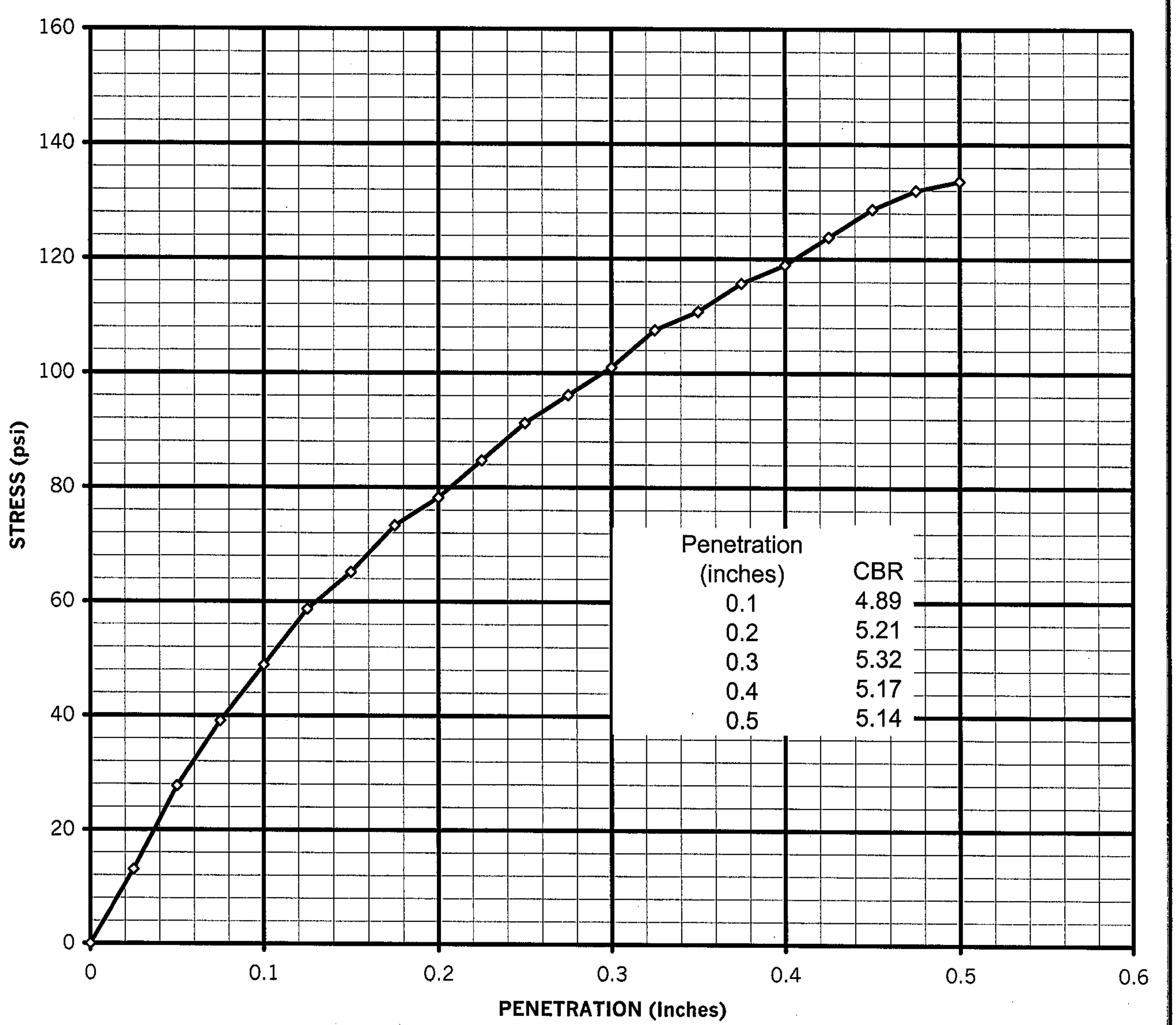
LABORATORY CBR TEST RESULT

Client :	**Hamilton County Engineer**

Date :	**07/18/06**
Project No. :	**060547NE**
Project :	**Geotechnical Services**
	Blue Rock & Cheviot Road
	Improvement
	Hamilton County, Ohio

Sample Obtained From :	**Test Boring 4, Bag Sample**	Depth :	**3.0' - 5.0'**
Sample Description :	**Brown CLAY, little fine to coarse sand, trace fine gravel**		
USCS Classification :	**CH**	In Situ Moisture Content :	**19.6%**
LL = 51 PL = 22 PI = 29			
Maximum Dry Density :	**113.1 P.C.F.**	Optimum Moisture Content :	**15.1 %**
Test Type :	**Standard Proctor, ASTM D698**		
Percent compaction during CBR test = 100.7% at a dry density of 113.8 P.C.F. and 15.3% moisture			
Moisture content of top inch =	**21.6 %**		



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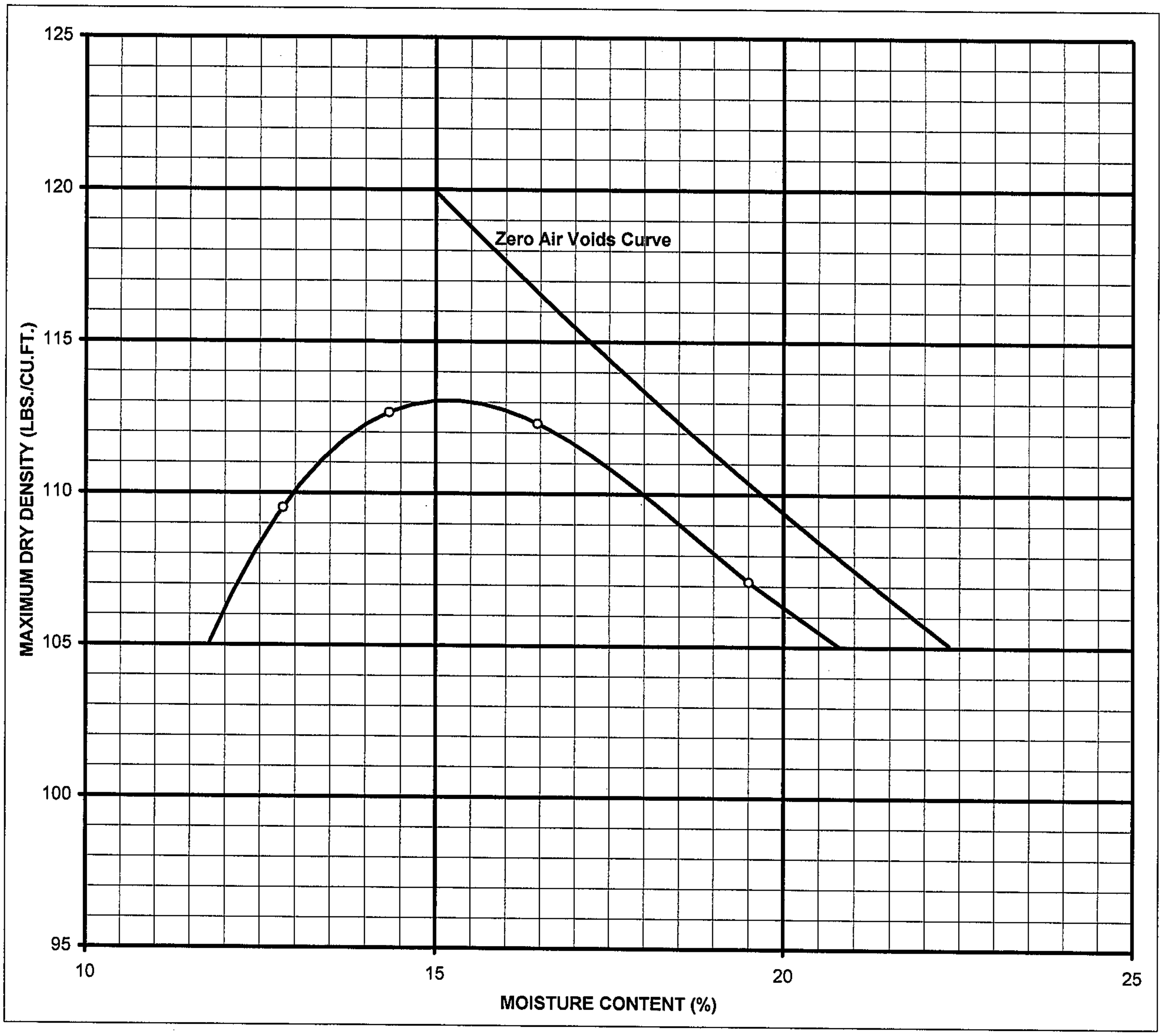
MOISTURE DENSITY TEST

Client:	Hamilton County Engineer

Date:	07/12/06
Project No.:	060547NE
Lab No:	Geotechnical Services
Project:	Blue Rock & Cheviot Road Improvement Hamilton County, Ohio

Sample Obtained From:	Test Boring 4, Bag Sample	Depth:	3.0' - 5.0'
Sample Description:	Brown CLAY, little fine to coarse sand, trace fine gravel		
USCS Classification:	CH	In Situ Moisture Content:	19.6%
LL = 38 PL = 24	PI = 14		
Maximum Dry Density:	113.1 P.C.F.	Optimum Moisture Content:	15.3 %
Test Type:	Standard Proctor, ASTM D698		
Method:	Method A		



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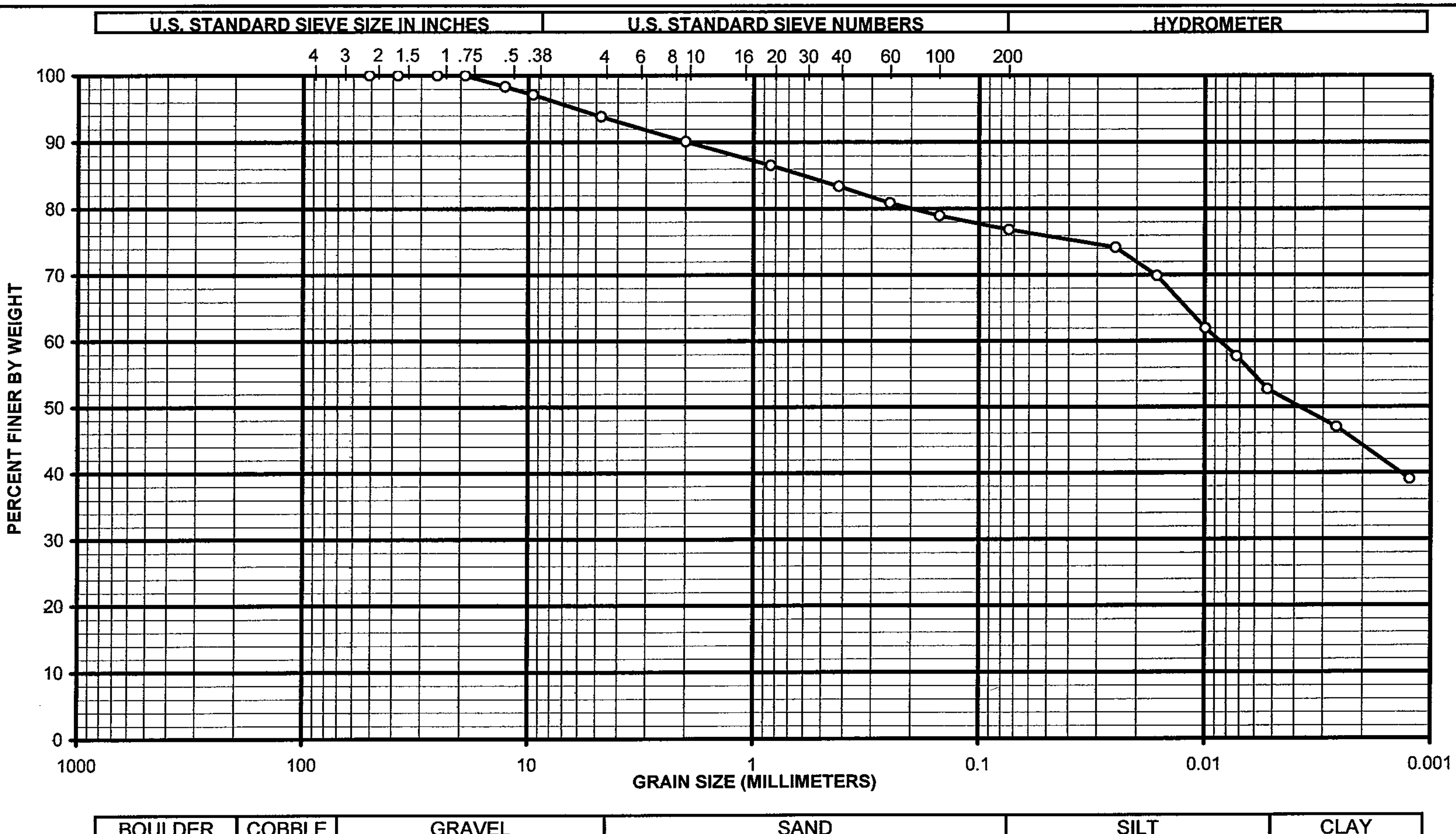
SIEVE ANALYSIS OF FINE AND COARSE AGGREGATES ASTM C-136



Boring No.	Sample No.	Elev. or Depth	Description	Nat. w%	LL	PL	PI
4	Bag Sample	3.0' - 5.0'	Brown CLAY, little fine to coarse sand, trace fine gravel		51	22	29

GRAVEL	SAND	SILT & CLAY	
6%	17%	25%	52%

USCS CLASSIFICATION IS CH

Specification

Client :	Hamilton County Engineer
Project :	Geotechnical Services
	Blue Rock & Cheviot Rds. Improvemer
	Hamilton County, Ohio
Project No.:	060547NE
Date :	07/18/06

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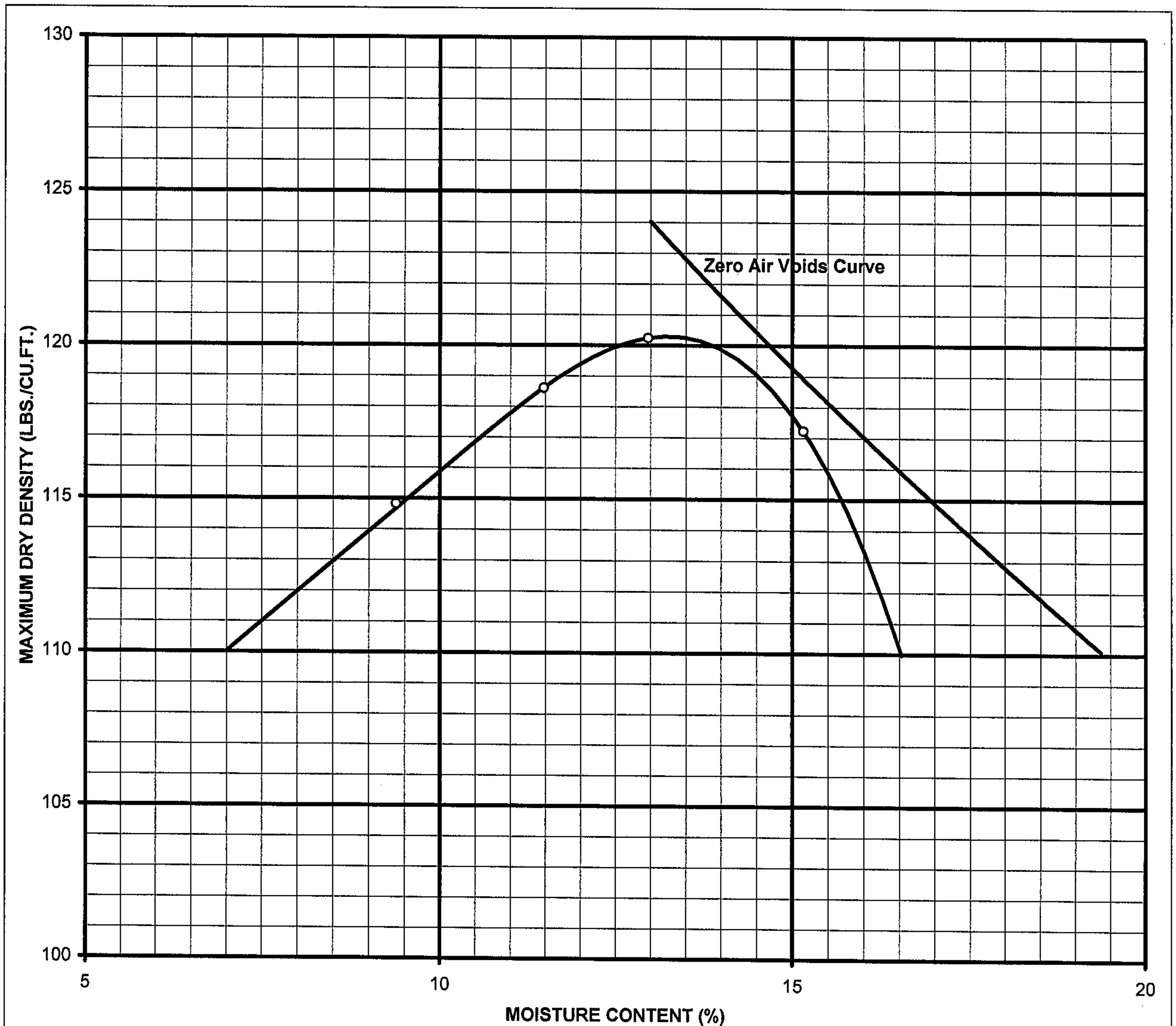
MOISTURE DENSITY TEST

Client:	Hamilton County Engineer

Date:	07/12/06
Project No.:	060547NE
Project:	Geotechnical Services
	Blue Rock & Cheviot Road
	Improvement
	Hamilton County, Ohio

Sample Obtained From:	Test Boring 6, Bag Sample	Depth:	3.0' - 5.0'
Sample Description:	Brown highly weathered SHALE with limestone fragments		
USCS Classification:	CL	In Situ Moisture Content:	9.3%
LL = 37 PL = 17	PI = 20		
Maximum Dry Density:	120.3 P.C.F.	Optimum Moisture Content:	13.2 %
Test Type:	Standard Proctor, ASTM D698		
Method:	Method A		



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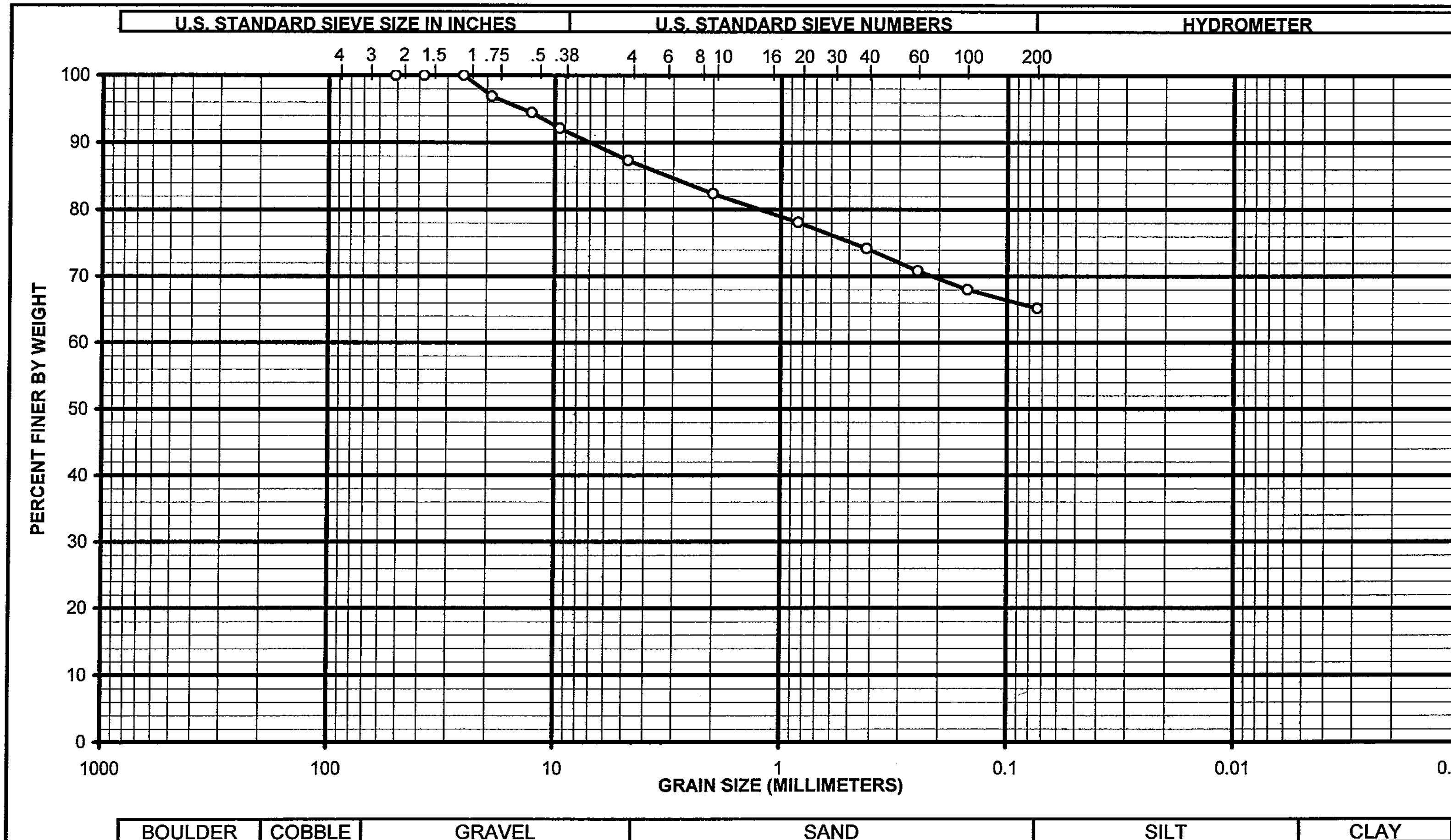
SIEVE ANALYSIS OF FINE AND COARSE AGGREGATES ASTM C-136



Boring No.	Sample No.	Elev. or Depth	Description	Nat. w%	LL	PL	PI
6	Bag Sample	3.0' - 5.0'	Brown highly weathered SHALE with limestone fragments	9.3	37	17	20

GRAVEL	SAND	SILT & CLAY	
13%	22%	65%	USCS CLASSIFICATION IS CL

Specification

Client :	Hamilton County Engineer
Project :	Blue Rock & Cheviot Road
	Improvement
	Hamilton County, Ohio
Project No.:	060547NE
Date :	07/13/06

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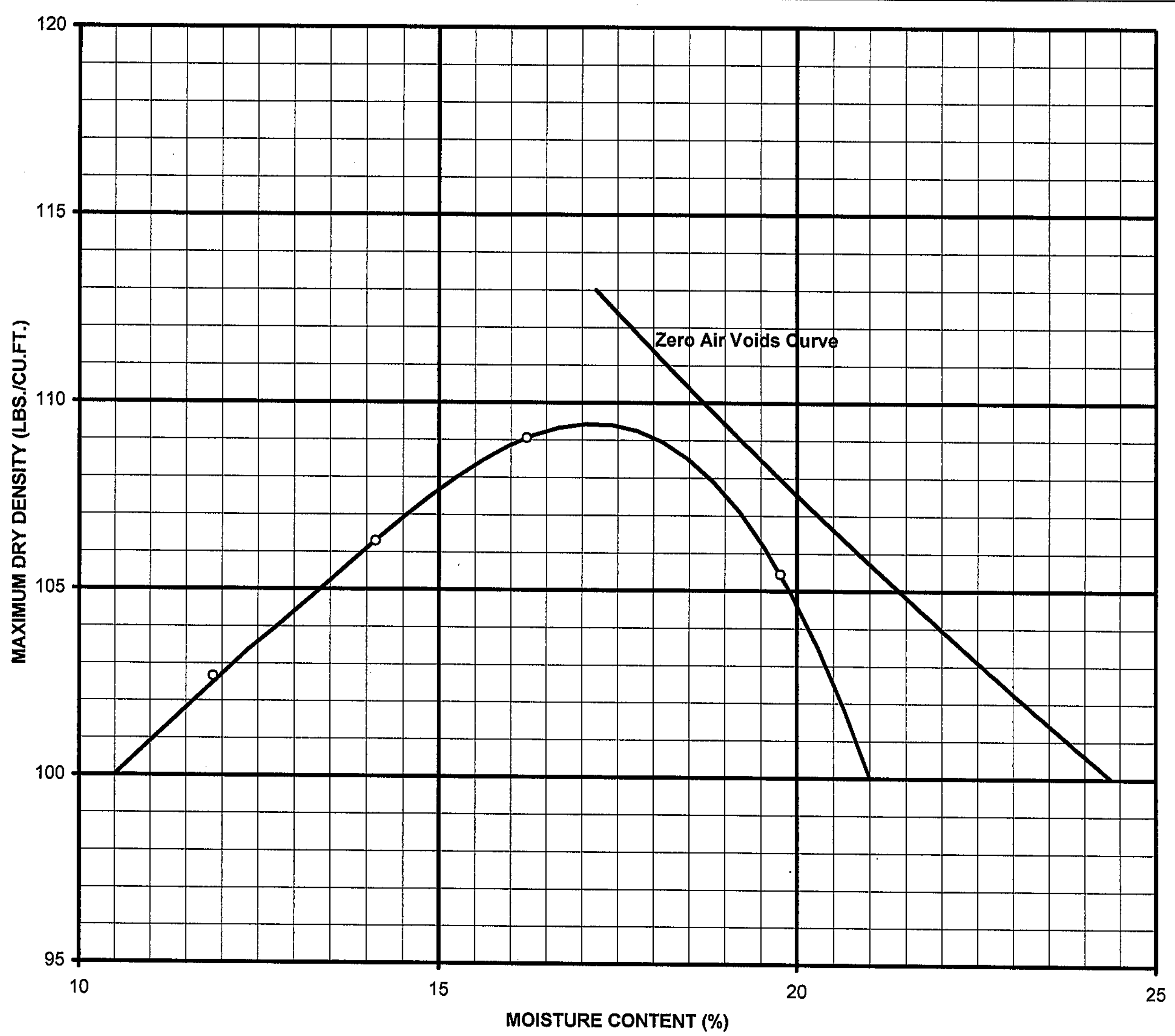
MOISTURE DENSITY TEST

Client:	Hamilton County Engineer

Date:	07/12/06
Project No.:	060547NE
Project:	Geotechnical Services
	Blue Rock & Cheviot Road
	Improvement
	Hamilton County, Ohio

Sample Obtained From:	Test Boring 3, Bag Sample	Depth:	5.0' - 7.0'
Sample Description:	Brown CLAY, little sand and gravel with roots		
USCS Classification:	CH	In Situ Moisture Content:	31.8%
LL = 52 PL = 25	PI = 27		
Maximum Dry Density:	109.4 P.C.F.	Optimum Moisture Content:	17.0 %
Test Type:	Standard Proctor, ASTM D698		
Method:	Method A		



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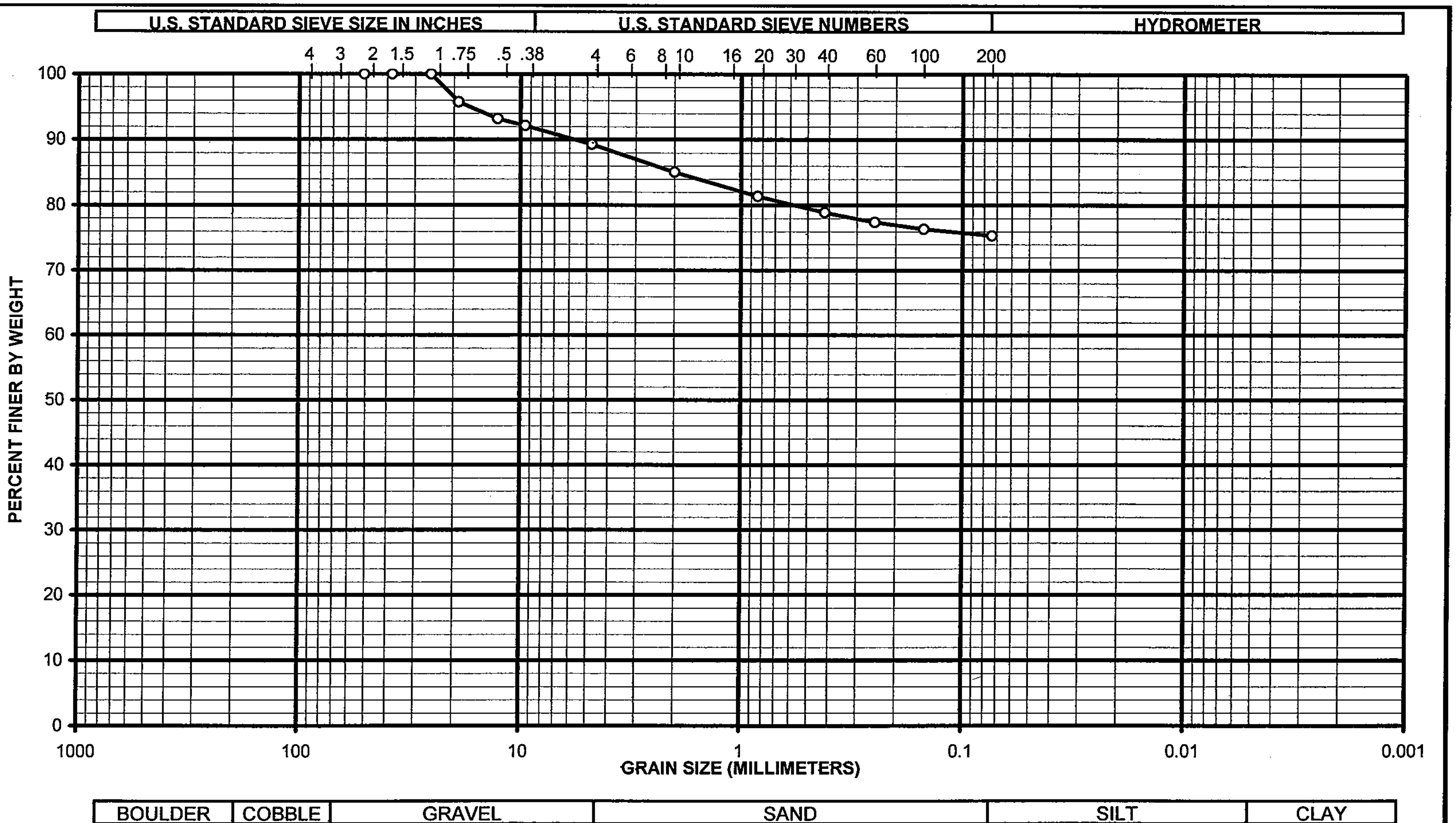
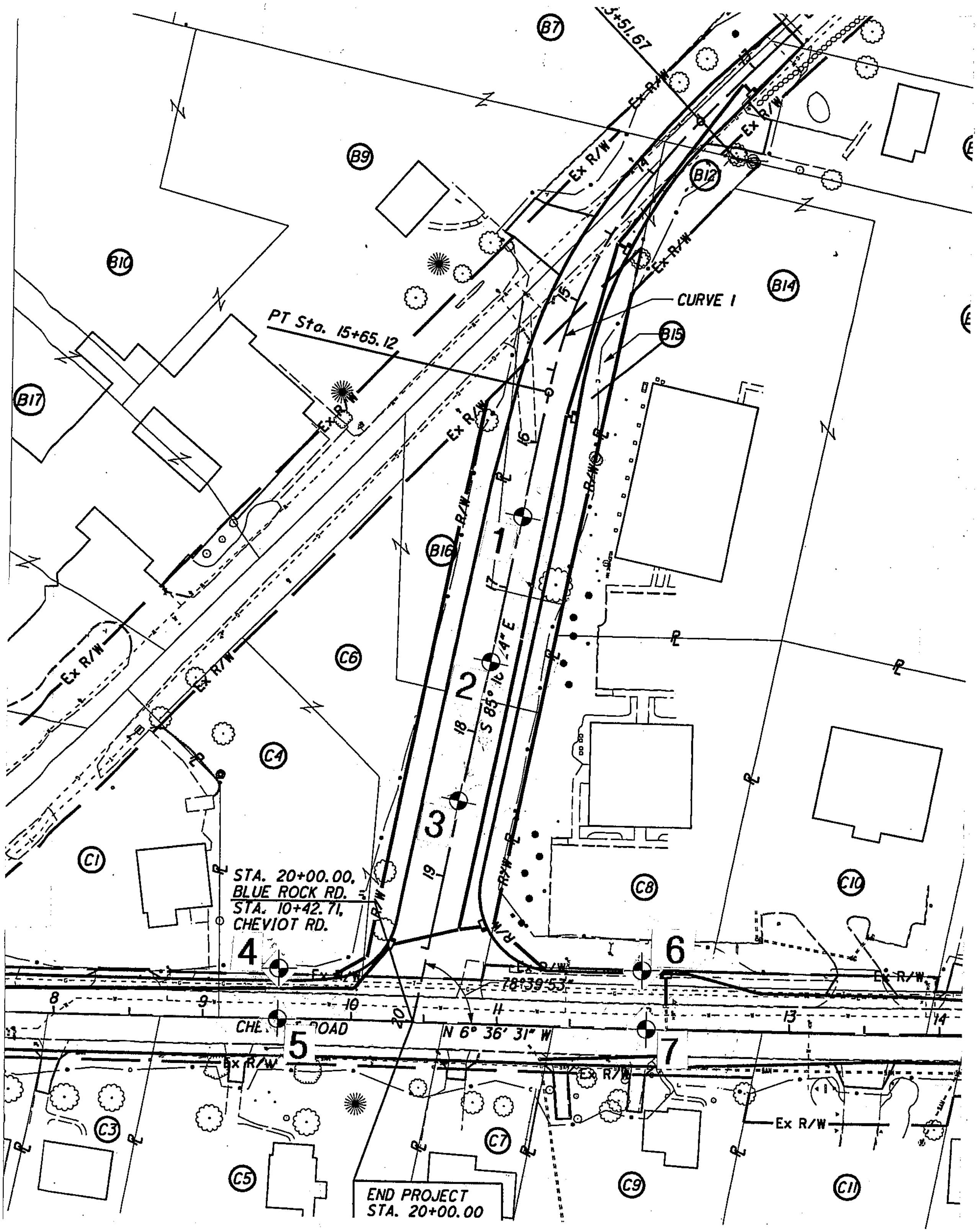
SIEVE ANALYSIS OF FINE AND COARSE AGGREGATES ASTM C-136



Boring No.	Sample No.	Elev. or Depth	Description	Nat. w%	LL	PL	PI	Client :	Hamilton County Engineer
3	Bag Sample	5.0' - 7.0'	Brown CLAY, little sand and gravel with roots	31.8	52	25	27	Project :	Blue Rock & Cheviot Road Improvement
GRAVEL	SAND	SILT & CLAY							Hamilton County, Ohio
11%	14%	75%	USCS CLASSIFICATION IS CH					Project No.:	060547NE
Specification								Date :	07/13/06



INDICATES TEST BORING LOCATION

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TEST BORING PLAN

Client:	County of Hamilton, Hamilton County Engineer
Project:	Geotechnical Exploration
	Blue Rock & Cheviot Road North Intersection Improvements
Location:	Colerain Township, Hamilton County, Ohio
Scale: 1" = 50'	Date: 8/24/06 — Drawing No.: 060547NE-1

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LOG OF TEST BORING

CLIENT: County of Hamilton, Hamilton County Engineer BORING # 1

PROJECT: Geotechnical Services, Blue Rock & Cheviot Road North Improvement, Hamilton County, Ohio JOB # 060547NE

LOCATION OF BORING: As shown on Test Boring Plan, Drawing 060547NE-1

ELEV.	SOIL DESCRIPTION COLOR, MOISTURE, DENSITY, PLASTICITY, SIZE, PROPORTIONS	STRATA DEPTH (ft.)	DEPTH SCALE (ft.)	Cond	Blows/6"	No.	Type	Rec. (in.)
926.4		0.0						
925.4	Mixed light brown slightly moist stiff FILL, lean silty clay with hairlike roots and limestone fragments, trace topsoil.	1.0		I	3/3/7	1A 1B	DS	18
924.9	Dark brown slightly moist stiff lean SILTY CLAY with roots and iron oixde stains.	1.5						
924.1	Brown moist medium stiff SILTY CLAY with iron oxide stains and roots.	2.3		I	3/4/8	2	DS	18
921.9	Brown moist stiff CLAY, trace sand with iron oxide stains (CH).	4.5	5	I	4/16/30	3	DS	18
				I	32/50/6"	4	DS	12
915.9	Interbedded brown moist very soft highly weathered SHALE and gray hard LIMESTONE (bedrock).	10.5	10	I	50/6"	5	DS	6
	Split spoon refusal and bottom of test boring at 10.5 feet.							
	Note: A Shelby tube sample (PT-6) was obtained in an offset hole from 1.0 to 3.0 feet. Recovery was 23 inches. A bag sampe of auger cuttings was obtained from 1.0 to 3.0 feet.		15					
			20					
			25					

Datum MSL — Hammer Wt. 140 lb — Hole Diameter 5 in. — Foreman BR

Surf. Elev. 926.4± — Hammer Drop 30 in. — Rock Core Dia. ____ — Engineer CMD

Date Started 6-28-06 — Pipe Size 2 in. O.D. — Boring Method CFA — Date Completed 6-28-06

SAMPLE CONDITIONS	SAMPLE TYPE	GROUND WATER DEPTH	BORING METHOD
D – DISINTEGRATED	DS – DRIVEN SPLIT SPOON	FIRST NOTED None ft.	HSA– HOLLOW STEM AUGERS
I – INTACT	PT – PRESSED SHELBY TUBE	AT COMPLETION Dry ft.	CFA– CONTINUOUS FLIGHT AUGERS
U – UNDISTURBED	CA – CONTINUOUS FLIGHT AUGER	AFTER ____ hrs. ____ ft.	DC – DRIVING CASING
L – LOST	RC – ROCK CORE	BACKFILLED Immed. hrs.	MD – MUD DRILLING

* STANDARD PENETRATION TEST – DRIVING 2" O.D. SAMPLER 1' WITH 140# HAMMER FALLING 30"; COUNT MADE AT 6" INTERVALS

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LOG OF TEST BORING

CLIENT: County of Hamilton, Hamilton County Engineer BORING # 2

PROJECT: Geotechnical Services, Blue Rock & Cheviot Road North Improvement, Hamilton County, Ohio JOB # 060547NE

LOCATION OF BORING: As shown on Test Boring Plan, Drawing 060547NE-1

ELEV.	SOIL DESCRIPTION COLOR, MOISTURE, DENSITY, PLASTICITY, SIZE, PROPORTIONS	STRATA DEPTH (ft.)	DEPTH SCALE (ft.)	SAMPLE Cond	Blows/6"	No.	Type	Rec. (in.)
928.3		0.0						
927.7	TOPSOIL (7")	0.6		I	3/3/3	1A 1B	DS	18
926.8	Light brown slightly moist stiff lean SILTY CLAY with hairlike roots and iron oxide stains.	1.5						
924.3	Mottled brown, trace gray moist stiff SILTY CLAY with clay seams and iron oxide stains (CL).	4.0		U		2	PT	24"/24"
921.3	Yellowish brown moist stiff CLAY with iron oxide stains and concretions.	7.0	5	I	4/6/14	3	DS	18
				I	7/50/3"	4	DS	8
917.3	Interbedded brown, trace gray moist very soft highly weathered SHALE and gray hard LIMESTONE (bedrock).	11.0	10	I	12/50/6"	5	DS	12
	Split spoon refusal and bottom of test boring at 11.0 feet.		15 20 25					

Datum MSL | Hammer Wt. 140 lb | Hole Diameter 5 in. | Foreman BR

Surf. Elev. 928.3± | Hammer Drop 30 in. | Rock Core Dia. ______ | Engineer CMD

Date Started 6-28-06 | Pipe Size 2 in. O.D. | Boring Method CFA | Date Completed 6-28-06

SAMPLE CONDITIONS	SAMPLE TYPE	GROUND WATER DEPTH	BORING METHOD
D – DISINTEGRATED	DS – DRIVEN SPLIT SPOON	FIRST NOTED None ft.	HSA – HOLLOW STEM AUGERS
I – INTACT	PT – PRESSED SHELBY TUBE	AT COMPLETION Dry ft.	CFA – CONTINUOUS FLIGHT AUGERS
U – UNDISTURBED	CA – CONTINUOUS FLIGHT AUGER	AFTER ____ hrs. ____ ft.	DC – DRIVING CASING
L – LOST	RC – ROCK CORE	BACKFILLED Immed. hrs.	MD – MUD DRILLING

* STANDARD PENETRATION TEST – DRIVING 2" O.D. SAMPLER 1' WITH 140# HAMMER FALLING 30"; COUNT MADE AT 6" INTERVALS

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LOG OF TEST BORING

CLIENT: County of Hamilton, Hamilton County Engineer BORING # 3

PROJECT: Geotechnical Services, Blue Rock & Cheviot Road North Improvement, Hamilton County, Ohio JOB # 060547NE

LOCATION OF BORING: As shown on Test Boring Plan, Drawing 060547NE-1

ELEV.	SOIL DESCRIPTION COLOR, MOISTURE, DENSITY, PLASTICITY, SIZE, PROPORTIONS	STRATA DEPTH (ft.)	DEPTH SCALE (ft.)	SAMPLE Cond	Blows/6"	No.	Type	Rec. (in.)
926.7		0.0						
926.2	TOPSOIL (7")	0.5		I	3/5/7	1A 1B	DS	18
924.2	Brown moist stiff lean SILTY CLAY with iron oxide stains and roots (desiccated).	2.5						
				U		2	PT	9.5"/12"
			5	I	13/18/37	3	DS	18
919.7	Brown moist stiff CLAY, little sand and gravel with roots and iron oxide stains (desiccated) (CH).	7.0						
				I	11/50/6"	4	DS	12
916.2	Interbedded brown moist very soft highly weathered SHALE and gray hard LIMESTONE (bedrock).	10.5	10	I	50/6"	5	DS	6
	Split spoon refusal and bottom of test boring at 10.5 feet. Note: A bag sample of auger cuttings was obtained from 5.0 to 7.0 feet.		15, 20, 25					

Datum MSL | Hammer Wt. 140 lb | Hole Diameter 5 in. | Foreman BR

Surf. Elev. 926.7± | Hammer Drop 30 in. | Rock Core Dia. ____ | Engineer CMD

Date Started 6-28-06 | Pipe Size 2 in. O.D. | Boring Method CFA | Date Completed 6-28-06

SAMPLE CONDITIONS	SAMPLE TYPE	GROUND WATER DEPTH	BORING METHOD
D – DISINTEGRATED	DS – DRIVEN SPLIT SPOON	FIRST NOTED None ft.	HSA– HOLLOW STEM AUGERS
I – INTACT	PT – PRESSED SHELBY TUBE	AT COMPLETION Dry ft.	CFA– CONTINUOUS FLIGHT AUGERS
U – UNDISTURBED	CA – CONTINUOUS FLIGHT AUGER	AFTER ____ hrs. ____ ft.	DC – DRIVING CASING
L – LOST	RC – ROCK CORE	BACKFILLED Immed. hrs.	MD – MUD DRILLING

* STANDARD PENETRATION TEST – DRIVING 2" O.D. SAMPLER 1' WITH 140# HAMMER FALLING 30"; COUNT MADE AT 6" INTERVALS

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LOG OF TEST BORING

CLIENT: County of Hamilton, Hamilton County Engineer BORING # 4
PROJECT: Geotechnical Services, Blue Rock & Cheviot Road North Improvement, Hamilton County, Ohio JOB # 060547NE
LOCATION OF BORING: As shown on Test Boring Plan, Drawing 060547NE-1

ELEV.	SOIL DESCRIPTION COLOR, MOISTURE, DENSITY, PLASTICITY, SIZE, PROPORTIONS	STRATA DEPTH (ft.)	DEPTH SCALE (ft.)	SAMPLE Cond	Blows/6"	No.	Type	Rec. (in.)
925.0		0.0						
923.0	Mixed brown moist stiff FILL, lean silty clay, trace topsoil surface.	2.0		I	4/4/4	1	DS	18
				U		2	PT	11"/18"
			5	I	32/18/18	3	DS	18
918.0	Brown moist stiff CLAY with shale fragments, iron oxide stains and bedding planes (residual) (CH).	7.0						
				I	27/50/6"	4	DS	12
914.5	Interbedded brown moist very soft highly weathered SHALE and gray hard LIMESTONE (bedrock).	10.5	10	I	50/6"	5	DS	6
	Split spoon refusal and bottom of test boring at 10.5 feet. Note: A bag sample of auger cuttings was obtained from 3.0 to 5.0 feet.		15					
			20					
			25					

Datum MSL	Hammer Wt. 140 lb	Hole Diameter 5 in.	Foreman BR
Surf. Elev. 925.0±	Hammer Drop 30 in.	Rock Core Dia.	Engineer CMD
Date Started 6-28-06	Pipe Size 2 in. O.D.	Boring Method CFA	Date Completed 6-28-06

SAMPLE CONDITIONS	SAMPLE TYPE	GROUND WATER DEPTH	BORING METHOD
D – DISINTEGRATED	DS – DRIVEN SPLIT SPOON	FIRST NOTED Trace @ 10.5 ft.	HSA– HOLLOW STEM AUGERS
I – INTACT	PT – PRESSED SHELBY TUBE	AT COMPLETION Dry ft.	CFA– CONTINUOUS FLIGHT AUGERS
U – UNDISTURBED	CA – CONTINUOUS FLIGHT AUGER	AFTER ____ hrs. ____ ft.	DC – DRIVING CASING
L – LOST	RC – ROCK CORE	BACKFILLED Immed. hrs.	MD – MUD DRILLING

* STANDARD PENETRATION TEST – DRIVING 2" O.D. SAMPLER 1' WITH 140# HAMMER FALLING 30"; COUNT MADE AT 6" INTERVALS

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LOG OF TEST BORING

CLIENT: County of Hamilton, Hamilton County Engineer BORING # 5

PROJECT: Geotechnical Services, Blue Rock & Cheviot Road North Improvement, Hamilton County, Ohio JOB # 060547NE

LOCATION OF BORING: As shown on Test Boring Plan, Drawing 060547NE-1

ELEV.	SOIL DESCRIPTION COLOR, MOISTURE, DENSITY, PLASTICITY, SIZE, PROPORTIONS	STRATA DEPTH (ft.)	DEPTH SCALE (ft.)	SAMPLE Cond	Blows/6"	No.	Type	Rec. (in.)
923.5		0.0						
923.2	ASPHALT CONCRETE (4"), Surface AC = 2"	0.3						
922.3	CEMENT CONCRETE (10") (disintegrated)	1.2						
				I	4/7/6	1	DS	18
919.5	Mixed brown very moist stiff FILL, sandy clay with iron oxide stains and limestone fragments.	4.5		I	5/9/10	2	DS	18
917.5	Yellowish brown moist very stiff SILTY CLAY with shale and limestone fragments and bedding planes (residual).	6.5	5	I	10/16/18	3	DS	18
	Bottom of test boring at 6.5 feet.							
			10					
			15					
			20					
			25					

Datum MSL Hammer Wt. 140 lb Hole Diameter 5 in. Foreman BR

Surf. Elev. 923.5± Hammer Drop 30 in. Rock Core Dia. ____ Engineer CMD

Date Started 6-27-06 Pipe Size 2 in. O.D. Boring Method CFA Date Completed 6-27-06

SAMPLE CONDITIONS	SAMPLE TYPE	GROUND WATER DEPTH	BORING METHOD
D – DISINTEGRATED	DS – DRIVEN SPLIT SPOON	FIRST NOTED None ft.	HSA– HOLLOW STEM AUGERS
I – INTACT	PT – PRESSED SHELBY TUBE	AT COMPLETION Dry ft.	CFA– CONTINUOUS FLIGHT AUGERS
U – UNDISTURBED	CA – CONTINUOUS FLIGHT AUGER	AFTER ____ hrs. ____ ft.	DC – DRIVING CASING
L – LOST	RC – ROCK CORE	BACKFILLED Immed. hrs.	MD – MUD DRILLING

* STANDARD PENETRATION TEST – DRIVING 2" O.D. SAMPLER 1' WITH 140# HAMMER FALLING 30"; COUNT MADE AT 6" INTERVALS

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LOG OF TEST BORING

CLIENT: County of Hamilton, Hamilton County Engineer BORING # 6

PROJECT: Geotechnical Services, Blue Rock & Cheviot Road North Improvement, Hamilton County, Ohio JOB # 060547NE

LOCATION OF BORING: As shown on Test Boring Plan, Drawing 060547NE-1

ELEV.	SOIL DESCRIPTION COLOR, MOISTURE, DENSITY, PLASTICITY, SIZE, PROPORTIONS	STRATA DEPTH (ft.)	DEPTH SCALE (ft.)	SAMPLE Cond	Blows/6"	No.	Type	Rec. (in.)
920.0		0.0						
919.8	TOPSOIL (3")	0.2		I	6/11/13	1A 1B	DS	18
917.5	Brown slightly moist very stiff SILTY CLAY with hairlike roots, shale and limestone fragments and bedding planes (residual).	2.5		U		2	PT	4"/6"
				I	35/32/50	3	DS	12
			5	I	50/6"	4	DS	6
910.5	Interbedded brown moist very soft highly weathered SHALE and gray hard LIMESTONE (bedrock).	9.5		I	11/50/6"	5	DS	12
909.5	Interbedded olive brown moist very soft weathered SHALE and gray hard LIMESTONE (bedrock).	10.5	10	I	50/6"	6	DS	6

Split spoon refusal and bottom of test boring at 10.5 feet.

Note: A bag sample of auger cuttings was obtained from 3.0 to 5.0 feet.

Datum MSL | Hammer Wt. 140 lb | Hole Diameter 5 in. | Foreman BR

Surf. Elev. 920.0± | Hammer Drop 30 in. | Rock Core Dia. ______ | Engineer CMD

Date Started 6-28-06 | Pipe Size 2 in. O.D. | Boring Method CFA | Date Completed 6-28-06

SAMPLE CONDITIONS	SAMPLE TYPE	GROUND WATER DEPTH	BORING METHOD
D – DISINTEGRATED	DS – DRIVEN SPLIT SPOON	FIRST NOTED None ft.	HSA– HOLLOW STEM AUGERS
I – INTACT	PT – PRESSED SHELBY TUBE	AT COMPLETION Dry ft.	CFA– CONTINUOUS FLIGHT AUGERS
U – UNDISTURBED	CA – CONTINUOUS FLIGHT AUGER	AFTER ____ hrs. ____ ft.	DC – DRIVING CASING
L – LOST	RC – ROCK CORE	BACKFILLED Immed. hrs.	MD – MUD DRILLING

* STANDARD PENETRATION TEST – DRIVING 2" O.D. SAMPLER 1' WITH 140# HAMMER FALLING 30"; COUNT MADE AT 6" INTERVALS

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LOG OF TEST BORING

CLIENT: County of Hamilton, Hamilton County Engineer BORING # 7

PROJECT: Geotechnical Services, Blue Rock & Cheviot Road North Improvement, Hamilton County, Ohio JOB # 060547NE

LOCATION OF BORING: As shown on Test Boring Plan, Drawing 060547NE–1

ELEV.	SOIL DESCRIPTION COLOR, MOISTURE, DENSITY, PLASTICITY, SIZE, PROPORTIONS	STRATA DEPTH (ft.)	DEPTH SCALE (ft.)	SAMPLE Cond	Blows/6"	No.	Type	Rec. (in.)
918.6		0.0						
918.3	ASPHALT CONCRETE (4.5"), Surface AC = 2"	0.3						
917.4	CEMENT CONCRETE (10") (disintegrated)	1.2						
				I	3/5/7	1	DS	18
				I	13/18/37	2	DS	18
912.1	Yellowish brown slightly moist very stiff SILTY CLAY with iron oxide stains and limestone fragments (residual).	6.5	5	I	20/23/25	3	DS	18
	Bottom of test boring at 6.5 feet.		10					
			15					
			20					
			25					

Datum MSL Hammer Wt. 140 lb Hole Diameter 5 in. Foreman BR

Surf. Elev. 918.6± Hammer Drop 30 in. Rock Core Dia. ___ Engineer CMD

Date Started 6–27–06 Pipe Size 2 in. O.D. Boring Method CFA Date Completed 6–27–06

SAMPLE CONDITIONS	SAMPLE TYPE	GROUND WATER DEPTH	BORING METHOD
D – DISINTEGRATED	DS – DRIVEN SPLIT SPOON	FIRST NOTED None ft.	HSA– HOLLOW STEM AUGERS
I – INTACT	PT – PRESSED SHELBY TUBE	AT COMPLETION Dry ft.	CFA– CONTINUOUS FLIGHT AUGERS
U – UNDISTURBED	CA – CONTINUOUS FLIGHT AUGER	AFTER ___ hrs. ___ ft.	DC – DRIVING CASING
L – LOST	RC – ROCK CORE	BACKFILLED Immed. hrs.	MD – MUD DRILLING

* STANDARD PENETRATION TEST – DRIVING 2" O.D. SAMPLER 1' WITH 140# HAMMER FALLING 30"; COUNT MADE AT 6" INTERVALS

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SOIL CLASSIFICATION SHEET

NON COHESIVE SOILS
(Silt, Sand, Gravel and Combinations)

Density

Very Loose	- 5 blows/ft. or less
Loose	- 6 to 10 blows/ft.
Medium Dense	- 11 to 30 blows/ft.
Dense	- 31 to 50 blows/ft.
Very Dense	- 51 blows/ft. or more

Relative Properties

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

Particle Size Identification

Boulders	- 8 inch diameter or more	
Cobbles	- 3 to 8 inch diameter	
Gravel	- Coarse	- 3/4 to 3 inches
	- Fine	- 3/16 to 3/4 inches
Sand	- Coarse	- 2mm to 5mm (dia. of pencil lead)
	- Medium	- 0.45mm to 2mm (dia. of broom straw)
	- Fine	- 0.075mm to 0.45mm (dia. of human hair)
Silt		- 0.005mm to 0.075mm (Cannot see particles)

COHESIVE SOILS
(Clay, Silt and Combinations)

Consistency	Field Identification	Unconfined Compressive Strength (tons/sq. ft.)
Very Soft	Easily penetrated several inches by fist	Less than 0.25
Soft	Easily penetrated several inches by thumb	0.25 – 0.5
Medium Stiff	Can be penetrated several inches by thumb with moderate effort	0.5 – 1.0
Stiff	Readily indented by thumb but penetrated only with great effort	1.0 – 2.0
Very Stiff	Readily indented by thumbnail	2.0 – 4.0
Hard	Indented with difficulty by thumbnail	Over 4.0

Classification on logs are made by visual inspection.

Standard Penetration Test – Driving a 2.0" O.D., 1 3/8" I.D., sampler a distance of 1.0 foot into undisturbed soil with a 140 pound hammer free falling a distance of 30 inches. It is customary to drive the spoon 6 inches to seat into undisturbed soil, then perform the test. The number of hammer blows for seating the spoon and making the tests are recorded for each 6 inches of penetration on the drill log (Example – 6/8/9). The standard penetration test results can be obtained by adding the last two figures (i.e. 8+9=17 blows/ft.). Refusal is defined as greater than 50 blows for 6 inches or less penetration.

Strata Changes – In the column "Soil Descriptions" on the drill log, the horizontal lines represent strata changes. A solid line (————) represents an actually observed change; a dashed line (— — — —) represents an estimated change.

Groundwater observations were made at the times indicated. Porosity of soil strata, weather conditions, site topography, etc., may cause changes in the water levels indicated on the logs.