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GEOTECHNICAL EXPLORATION

PROPOSED RETAINING WALL

NORTHERN KENTUCKY WATER DISTRICT

MEMORIAL PARKWAY TREATMENT PLANT

FT. THOMAS, KENTUCKY

Prepared for: **CH2M Hill**

Thelen Project No.: **080977E**

THELEN ASSOCIATES, INC.
www.thelenassoc.com
Geotechnical • Testing Engineers
• 1398 Cox Avenue, Erlanger, Kentucky 41018-1002 / 859-746-9400 / Fax 859-746-9408
Offices
Erlanger, Kentucky
Lexington, Kentucky
Cincinnati, Ohio
Dayton, Ohio

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November 11, 2009

CH2M Hill
300 E-Business Way
Suite 400
Cincinnati, Ohio 45241

Attention: Mr. Nicholas Winnike

Re: Geotechnical Exploration
Proposed Retaining Wall
Northern Kentucky Water District
Memorial Parkway Treatment Plant
Ft. Thomas, Kentucky

Gentlemen:

Presented in this report are the results of the geotechnical exploration made for the proposed retaining wall to be located at the Northern Kentucky Water District (NKWD) Memorial Parkway Treatment Plant (MPTP) in Ft. Thomas, Kentucky. The geotechnical work included test borings and engineering services performed in general accordance with the scope outlined in our Proposal-Agreement K29160, dated August 5, 2009.

1.0 SCOPE

The main purpose of this exploration was to determine the general subsurface profile at the site and to relate the engineering properties of the soils and bedrock, that is their classification, strength and compressibility characteristics, to the proposed retaining wall's foundation design and construction.

2.0 SITE CONDITIONS AND PROJECT CHARACTERISTICS

Proposed for construction is a reinforced concrete retaining wall. For the purposes of this report, it was assumed that the entrance drive to the Memorial Parkway Treatment Plant is traversing in a north-south direction and that the existing Filter Building is facing towards the west. The proposed retaining wall will be located parallel to and approximately 20 feet west of the western side of the entrance drive and the length of the retaining wall will be approximately 120 feet. The northern end of the retaining wall will be located approximately 17 feet south of the southern end of an existing concrete retaining wall.

The writer reviewed the Site Plan that was posted on the CH2M Hill's SharePoint website on September 2, 2009, in the Civil Design section, and then visited the site on September 8, 2009. Currently, the ground surface in the vicinity of the proposed retaining wall area slopes downward from east to west with a slope gradient of roughly 2.5 horizontal to 1 vertical (2.5H:1V) to 3H:1V. At the top of the slope, the ground surface that is covered by asphalt pavement that gradually slopes downward from north to south, from approximately elevation 749 (El. 749) to approximately El. 744. The toe of the slope meets the water's edge of the North Reservoir at approximately El. 720. During the site reconnaissance, the writer did not observe any tension cracks in the ground surface or other signs of slope movement at the site in the general vicinity of the proposed retaining wall.

To the north of the proposed retaining wall, there is an existing reinforced concrete retaining wall that is approximately 120 feet long and has a dog-leg shape in plan view. The retaining wall is roughly 7.0 feet tall at its maximum height from the top of the wall to the ground surface on the downslope side of the wall. However, the top of the wall extends approximately 2.0 feet above the ground surface on the upslope side of the wall, so the wall is retaining a maximum of approximately 5.0 vertical feet of soil at the wall location. No as-built drawings, design drawings or details of this existing retaining wall were available for review as part of this study.

The ground surface sloped down and away below the existing retaining wall and up and away above the existing wall. Below the existing retaining wall and also below a projection of the southern end of the wall to the northernmost concrete flume, and then extending westward to the water's edge of the North Reservoir, soil-impregnated rip-pap was observed at the ground surface. The writer did not notice the existing retaining wall to be tilting downward, bowing or excessively cracking during the site reconnaissance. The writer did observe that a portion of the existing retaining wall, within the middle region of the wall and roughly 20 to 25 feet long in plan dimensions, was apparently constructed with a slight curve in plan view.

The proposed retaining wall will be a maximum of 8.5 feet high from the top of the wall to the ground surface on the downslope side of the wall, extending from El. 746.0 to El. 737.5. The ground surface will slope up and away from the wall at a slope gradient of 2.5 to 3.0H:1V on the upslope side of the wall. The top of the wall will extend approximately 2.0 to 2.5 feet above the final ground surface on the upslope side of the wall, so the wall will be retaining a maximum of approximately 6.0 vertical feet of soil at the wall location. The proposed retaining wall will have its maximum height in the central region of the wall and will decrease in height to both the north and south ends as the existing ground surface on the downslope side of the wall rises in these areas. At both ends of the proposed retaining wall, the top of the wall, which will be at El. 746.0, will extend approximately 2.0 to 2.5 feet above the final ground surface on the upslope side of the wall and final grade will be at El. 743.0 on the downslope side of the wall, so the wall will be retaining approximately 1.0 vertical foot of soil or less at the wall location.

The previously referenced Site Plan indicated that the proposed retaining wall will be constructed over an existing 24-inch diameter concrete wash water pipe and an existing 24-inch diameter concrete drain pipe. The plan indicated that the existing pipes are located near the southern third point of the proposed retaining wall and that the locations of the existing pipes are skewed to the alignment of the proposed wall, in plan view. No information was provided regarding the depth of the two existing pipes.

A previous site plan labeled *Site Layout and Grading Plan* that was drawn by J. Stephen Watkins, Consulting Engineers and dated May 31, 1961 with revisions dated September 10, 1963 to include as-built conditions, was obtained from our in-house archived files and reviewed. After this site plan was drawn, it was discovered that the ground surface elevations are roughly 5.4 feet higher than the United States Geological Survey (USGS) datum. Therefore, when comparing the ground surface elevations from this plan to plans that utilize Mean Sea Level (MSL) as the 0 datum, 5.4 vertical feet should be subtracted from the elevations shown on the *Site Layout and Grading Plan* to obtain elevations that correlate to MSL.

The 1963 revised *Site Layout and Grading Plan* indicated that fill was placed within the vicinity of the proposed retaining wall to achieve the 1963 ground surface elevations. At the Boring 201 and Boring 202 locations, approximately 16.4 vertical feet and 9.4 vertical feet of soil was placed to achieve the 1963 ground surface elevations. The 1963 *Site Layout and Grading Plan* indicated that no fill was placed at the B-203 location. Then, since 1963, approximately 6.4 vertical feet of fill was placed in the vicinity of Boring 203 to achieve the current ground surface elevation, based upon the current Site Plan previously referenced in this report section. A review of the ground surface elevations shown on the current Site Plan indicated that no fill had been placed at the Boring 201 and 202 locations since 1963. No specifications or testing records indicating the manner nor degree to which the existing fill was compacted were available for review as part of this study.

It is probable that the fill that was placed in or around 1963 was placed and compacted in a controlled manner, since the fill that formed the fill slope within that area was utilized to ultimately support the paved drive area at the top of the slope. Since 1963, the southern portion of a previous retaining wall, located in the vicinity of the proposed retaining wall, was removed. The retaining wall that was removed was located approximately 10 feet west of and roughly parallel to the northern three-quarters of the proposed retaining wall, in plan view. It is possible that the retaining wall was removed when the northern portion of the North Reservoir was filled-in, the slope extending to the reservoir was flattened, and rip-rap was placed that is currently visible at the ground surface.

3.0 SUBSURFACE EXPLORATION

The test borings were drilled on August 31, 2009. Three (3) test borings, numbered 201 through 203, were drilled at the locations shown on the Boring Plan, Drawing 080977E-

2, in the Appendix to this report. The Site Plan that was posted on CH2M Hill's Share Point website on September 2, 2009, which was previously referenced in this report, was utilized as a Base Map for our Boring Plan. In our Geotechnical Report dated May 27, 2009 for the Advanced Treatment Building to be constructed at the Northern Kentucky Water Department Memorial Parkway Plant, Drawing 080977E-1 was utilized as the Boring Plan. The test boring locations were established by us based upon the Site Plan referenced in Section 2.0, *Site Condition and Project Characteristics*, of this report and the proposed retaining wall location indicated to us at that time. The test boring locations were staked in the field by us and the ground surface elevations of the test borings were surveyed based on a grate inlet elevation of El. 743.60 for a storm sewer catch basin located along the western side of the entrance drive to the Memorial Parkway Water Treatment Plant, approximately 52 feet north of the northern side of the existing Wash Water Pump Station.

The test borings were made with a track-mounted drill rig advancing hollow stem augers. Standard split spoon sampling was accomplished ahead of the augers following the procedures outlined in ASTM D1586. Observations for groundwater were made in the borings during drilling and after their completion.

As each test boring was advanced, the Drilling Technician kept a log of the subsurface profile noting the soil and bedrock types and stratifications, groundwater, standard penetration test results and other pertinent data.

4.0 REVIEW OF SOIL AND BEDROCK SAMPLES

Final test boring logs were prepared by the Project Geotechnical Engineer on the basis of the visual classification in the laboratory and the field logs kept by the Drilling Technician. Copies of the final test boring logs are included in the Appendix with a Soil Classification Sheet which describes the terms and symbols used on the boring logs.

5.0 SUBSURFACE CONDITIONS

Borings 201 and 202 encountered 3 inches and 1 inch of topsoil at the surface, respectively. In Boring 203, rip-rap was encountered at the ground surface. All three

(3) borings encountered fill materials to various depths, as listed in the subsequent table.

BORING NUMBER	SURFACE ELEVATION, FEET	APPROXIMATE DEPTH OF FILL, FEET
201	743.0	at least 21.0
202	739.0	12.0
203	724.9	9.5

In Boring 201, the fill that was encountered below the surficial topsoil layer consisted of mottled brown and gray moist medium stiff silty clay with rock fragments. The results of the Standard Penetration Tests (N-values) typically ranged from 6 to 10 blows per foot (bpf) in the fill. In Boring 202, the fill that was encountered below the topsoil layer also consisted of mottled brown and gray moist medium stiff silty clay with rock fragments. The N-values were a little lower in Boring 202 than in Boring 201, ranging from 5 to 7 blows per foot (bpf) in the fill. In Boring 203, rip-rap was encountered at the ground surface that extended to a depth of approximately 4.5 feet. Below the rip-rap, mottled dark brown and dark gray or brown moist soft to medium stiff silty clay fill with rock fragments was disclosed to a depth of approximately 9.5 feet. The N-values were 6 and 9 bpf in the clayey fill encountered in Boring 203.

In Boring 201, auger refusal was encountered at a depth of 21.0 feet. During the drilling operations, it could not be determined whether auger refusal was encountered on the underlying weathered bedrock or due to a large rock fragment within the fill. In Borings 202 and 203, native brown moist stiff to very stiff silty clay soils with rock fragments were encountered below the fill materials, extending to depths of approximately 19.5 feet and approximately 12.0 feet, respectively. The N-values were usually in the range of 15 to 20 bpf in the native soils, but were also in excess of 100 bpf due to rock fragments contained within the clayey matrix.

Below the native silty clay soils in Boring 203, interbedded brown moist soft highly weathered shale and gray hard limestone bedrock was disclosed to a depth of approximately 14.5 feet. Below the interbedded brown highly weathered shale and gray limestone bedrock in Boring 203 and below the previously mentioned native silty clay in

Boring 202, interbedded olive brown and gray moist soft weathered shale and gray hard limestone was encountered to the termination depths of Borings 203 and 202, which were 16.3 feet and 21.5 feet, respectively.

Groundwater was encountered only in Boring 203 for this study. In Boring 203, groundwater was encountered at a depth of 5.0 feet during the drilling operations, which was at or near the interface of the overlying rip-rap and the underlying clayey fill soils. At completion, the groundwater level was at a depth of 3.5 feet, and 3.5 hours after Boring 203 was completed, the groundwater level was at a depth of 3.5 feet. Groundwater was not encountered in Borings 201 and 202 at any times.

Based on our local experience, periodic groundwater seepage can occur at a granular material/clayey material interface, fill soil/native soil interface, the native soil/bedrock interface, and along limestone layers within the bedrock. The groundwater level can be expected to fluctuate with local precipitation events and well as seasonal effects.

6.0 CONCLUSIONS AND RECOMMENDATIONS

6.1 General

Based upon our engineering reconnaissance of the site, the test borings, a visual examination of the samples, our understanding of the proposed construction, and our experience as Consulting Soil and Foundation Engineers in the Northern Kentucky 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 construction outlined in Section 2.0, *Site Conditions and Project Characteristics,* 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 and information provided to us at the time this report was prepared and which are referenced in Section 2.0, *Site Conditions and Project Characteristics*, 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, structure location, 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.

We have performed the test borings 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 at the site with the Owner's representative, the Design Civil Engineer, the Structural Engineer, the General Contractor, the Excavating Contractor, the Geotechnical Engineer (us) and any other interested parties to review the scope and schedule of the proposed earthwork and foundation installation.

6.2 Slope Stability Analyses

Slope stability analyses were performed for the highest section of the proposed reinforced concrete retaining wall, which is located approximately 20 to 30 horizontal feet south of Boring 202. The subsurface conditions encountered in Borings 202 and 203 were utilized to develop a model of the embankment for the slope stability analyses. The description of the sloping ground surface in the vicinity of the proposed retaining wall is outlined in Section 2.0, *Site Conditions and Project Characteristics.* The following estimated soil parameters were used for the slope stability analyses at the retaining wall location. The soil strength parameters (effective cohesion and effective internal friction angle) were estimated from: a) Thelen's data base of triaxial test results on similar soils in the project area; b) Thelen's experience with analyzing stability of the clay soil types in the project area; and c) local published strength parameters for compacted clay fill soils.

Depth, feet (elevation)	Material Type	Effective Cohesion, psf	Effective Internal Friction Angle, degrees
0 to 6.0 (744.0 to 738.0)	Proposed Stiff Silty Clay Fill	100	26
6.0 to 18.0 (738.0 to 726.0)	Existing Medium Stiff Silty Clay Fill	75	23
18.0 to 25.0 (726 to 719.0)	Native Stiff Silty Clay	100	26
Below 25.0 (Below 719.0)	Weathered Bedrock	10,000	50

For the long-term slope stability analyses, it was assumed that the groundwater level was located below the surface of the weathered bedrock, since no groundwater was encountered in Borings 201 and 202, and since the ground surface above the crest of the slope will be paved. In addition, no surcharge loads such as traffic loading were included at the crest of the slope, since it was assumed that parking would be prohibited along the western edge of the paved drive and only transient loads due to moving traffic would be experienced in the drive area.

Utilizing computer-aided slope stability solutions in accordance with the simplified Bishop circular failure surface methods of analysis and Janbu non-circular (wedge) methods of analysis, slope stability factors of safety were computed. XSTABL 5.2, which is a computer software program produced by Interactive Software Designs, Inc., was utilized to compute the slope stability factors of safety in our analyses. The circular failure surfaces were found to be more critical (i.e., to have lower factors of safety) than the wedge failure surfaces. The long-term, effective stress factor of safety for the existing slope condition in the area of the proposed retaining wall was computed to be 1.82. The lowest (most critical) long-term effective stress slope stability factor of safety in the computer-aided solution for the proposed retaining wall and backfill condition was 1.42, assuming that the bottom of retaining wall footing would be buried at a depth of 2.5 feet below the ground surface on the downslope side of the wall. Traditionally, a minimum factor of safety of 1.50 is desired for long-term slope stability.

The factor of safety for the short-term end of construction condition for the proposed retaining wall and backfill was computed to be 2.67. For this analysis, the assumed short-term undrained shear strengths for the soils were 750 psf for the existing medium stiff clay fill, 2,000 psf for the proposed stiff compacted fill, and 1,500 psf for the stiff native clay soils.

Several additional computer-aided slope stability analyses were performed for an assumed lowering of the proposed retaining wall footing through the existing fill soils that are present in the upper elevations of the slope. In order to achieve an increase in the long-term factor of safety from 1.42 to 1.50, the footing for the retaining wall would have to be lowered 6.5 feet below the minimum depth of the footing for frost protection, which would be 9.0 feet below the ground surface on the downslope side of the wall.

<u>6.3 Retaining Wall Foundation</u>

It is our opinion that the medium stiff clayey fill encountered in the test borings could be suitable for support of a shallow footing type foundation for the proposed retaining wall, if the allowable bearing pressure is limited. The foundation for the retaining wall, which could bear within the existing medium stiff clayey fill, can be proportioned based on a maximum allowable toe pressure of 2,000 pounds per square foot (psf), full dead and full live load. The footing bottom should be placed at least 3.0 feet below the downslope finished grades due to the existing slope conditions, which is slightly greater than the acceptable depth for frost protection of 2.5 feet in the Northern Kentucky Area. A "lean" concrete mud-mat (minimum 500 pounds per square inch at 28 days) should be placed over the prepared bearing soils if the footing excavation must remain open for an extended period of time. We estimate settlement of the retaining wall foundation designed and constructed in accordance with the recommendations in this report will be ½-inch or less.

It is noted that the on-site clayey soils are somewhat plastic. These soils can change in volume with changes in moisture content. The moderately plastic clays at this site will tend to swell when wetted and shrink when dried. Therefore, we recommend that efforts be made so that the natural moisture contents of these materials are maintained both during and after construction. Moisture contents may most effectively be controlled by the placement of footing concrete as soon as possible after bearing surface and subgrade preparation. Good drainage should be maintained to prevent the ponding of water in or around footing excavations.

If the proposed bearing elevation of the new retaining wall footing is greater than 4 vertical feet above the crown of either of the two existing pipelines that cross the proposed wall alignment, then the presence of the pipelines can be ignored. If the crowns of the pipelines are within 4 vertical feet of the bottom of the proposed retaining wall footing, it is recommended that the bottom of the retaining wall footing be lowered to below the pipeline. A relationship of 2 horizontal to 1 vertical upward from the invert of the pipeline should be used in setting the bearing elevation of the proposed retaining wall. We recommend that, if required, footing steps be a maximum height of 2 feet with a corresponding minimum length of 4 feet. Reinforcing steel and concrete should remain continuous through the footing steps. All footing excavations should be made to neat lines and grades so that concrete can be placed directly against the banks of excavations without forming. It is also important that good drainage be maintained during and after construction to prevent water from ponding in and around footing excavations. Loose soil, debris, excess surface water, and/or soils disturbed by exposure should be removed from the bearing surface prior to concrete placement.

The Contractor should be responsible for the stability and safety of all excavations and should exercise all necessary cautions to shore, slope or otherwise maintain stable trench excavations to protect workers. All trenches should be made and maintained in accordance with all federal, state and local regulations, as well as in accordance with OSHA requirements.

It is recommended that all footing excavations be reviewed by our Project Geotechnical Engineer or his/her representative prior to placing concrete to determine that bearing soils and surfaces are consistent with the recommendations contained herein.

6.4 Lateral Earth Pressures

Lateral earth pressures will be exerted on the proposed reinforced concrete retaining wall. Since the surface of the backfill soils will be sloping upwards away from the proposed wall as described in Section 2.0, *Site Conditions and Project Characteristics,* lateral earth pressures will be exerted on the retaining wall that are higher than those generated for level backfill conditions. Since the top of the proposed retaining wall will not be restrained, active earth pressures will develop on the uphill face of the wall.

Due to the proposed upward backfill slope of 2.5H:1V and the effective internal friction angles of the on-site clay soils of 23 to 26 degrees, a coefficient of active earth pressure of $K_a = 0.6$ is recommended for the retaining wall design. Since the moist unit weight of the compacted backfill soils is anticipated to be 125 pounds per cubic feet (pcf), it is recommended that an equivalent fluid pressure of 75 pounds per square foot, per vertical foot of wall (psf/ft) be utilized to design the reinforced concrete retaining wall for the clayey on-site soils.

Furthermore, it is recommended that free-draining granular materials, which have a minimum horizontal thickness of 2.0 feet, be utilized as backfill immediately adjacent to the upslope side of the retaining wall. The free-draining granular layer should extend to within 2 vertical feet of the surface and be capped with compacted clayey soils to reduce surface water infiltration.

A foundation drain should be constructed on top of the footing to remove any excessive hydrostatic pressure and should discharge to day light over the slope by gravity.

To resist sliding, there will be frictional resistance between the base of the retaining wall footing and the underlying clayey fill materials. It is recommended that a coefficient of friction of no greater than 0.30 be utilized for the retaining wall footing against the underlying existing clay fill materials.

If additional resistance to sliding for the retaining wall is required, then a reinforced concrete key-way could be attached to the base of the retaining wall foundation. An ultimate passive pressure of 1,500 psf could be utilized for the existing clayey fill soils in contact with the downslope face of the key-way and a suitable factor of safety should be applied in the retaining wall design to resist sliding. It is recommended that a key-way be located near the heel of the proposed retaining wall footing, not near the toe.

6.5 Site Preparation

Any areas where fill will be placed should be cleared of any vegetation and existing pavement. After general site stripping, all surfaces upon which fill will be placed should be proofrolled with a heavily loaded piece of equipment under the review of the Project Geotechnical Engineer or his/her representative. Any soft or yielding soils detected during the proofroll should be undercut to firm non-yielding soils. Prior to placing compacted fill on the existing slope, relatively level benches should be excavated into the existing fill to provide an adequate bond between the new fill and existing fill. The approved proofrolled surfaces after any undercutting should then be compacted to 95 percent of the standard Proctor maximum dry density, ASTM D698, prior to filling.

It is recommended that proposed fill slopes not exceed 2.5H:1V in steepness. Due to the steepness to the proposed slopes it will be difficult to compact soil on the face of the slope. Therefore, it is recommended that the proposed fill slopes be overfilled when compacted and then trimmed-back to the design slope.

We recommend that all new fill soils consist of clean on-site clayey soils or approved borrow relatively free of topsoil, vegetation, trash, construction or demolition debris, frozen materials, particles over 6 inches in maximum thickness or other deleterious materials. The new fill should be placed on the prepared surfaces in shallow horizontal layers, 6 to 8 inches in loose thickness. The fill should be compacted to achieve densities of at least 95 percent of the maximum dry density (MDD) determined by the standard Proctor moisture-density test, ASTM D698, within the proposed slope area and paved area, except within the upper 8 inches of the subgrade for the paved area where the degree of compaction should be increased to a minimum of 100 percent of the MDD and tested immediately prior to paving. The moisture content of the fill at the time of compaction should be maintained within 2 percent below to 3 percent above the optimum moisture content.

It is our opinion that the on-site existing clayey fill soils, encountered within the borings at this site, should be suitable for reuse as new compacted and tested fill, provided they are moisture conditioned to within the criteria listed above. It is advisable that the earthwork operations involving the on-site soils be carried out during favorable seasonal conditions and that a sufficient gradient be maintained at the ground surface to prevent ponding of surface runoff water. Experience has found that the optimum season of the year for earthwork in the Northern Kentucky Area is during the months of May through October because of the historically more favorable weather conditions during that period. The on-site soils, consisting of moderately plastic silty clays, are somewhat susceptible to shrink/swell during and following periods of precipitation.

If any portions of construction are undertaken during the winter or spring months of the year, we recommend that no concrete, asphalt or fill be placed over frozen or saturated soils. In addition, frozen soils should not be used as compacted fill or backfill.

6.6 Drainage and Erosion

We recommend that surface water falling on pavement areas that are located above the slope, where the proposed retaining wall will be constructed, be collected by an internal storm sewer system and curbing, and not allowed to discharge over the face of the slope above the proposed retaining wall. Excessive water that infiltrates the relatively steep slope could trigger slope failures as a result of the build-up of excessive pore water pressures that reduce the effective strength of the resisting soils as well as create a loss of strength of the resisting soils due to the softening action related to the excess water.

During construction, straw bales and/or silt fences should be staked on the slope below the work area to minimize the amount of soils carried from the construction site. Scarified areas should be seeded and strawed, paved, sodded, or otherwise protected from erosion as soon as possible after final grading is completed.

7.0 CLOSURE

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 be part of the Design Team for this project. Should you have any questions regarding this report, please do not hesitate to contact us. We look forward to following through with you on this project by providing the necessary construction review and testing services.

Respectfully submitted,
THELEN ASSOCIATES, INC.

Theodore W. Vogelpohl, P.E.
Principal Geotechnical Engineer

TWV:tmk
080977E

STATE OF KENTUCKY
THEODORE W. VOGELPOHL
14509
LICENSED
PROFESSIONAL ENGINEER

Copies submitted: 5 - CH2M Hill
1 - HDR Engineering, Inc.
1 - Freeland Harris Consulting Engineers
1 - Northern Kentucky Water District

APPENDIX

ASFE Report Information

Test Boring Logs (3)

Soil Classification Sheet

Boring Plan, Drawing 080977E-2 (In pocket)

Important Information about Your

Geotechnical Engineering Report

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

While you cannot eliminate all such risks, you can manage them. The following information is provided to help.

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
THE BEST PEOPLE ON EARTH

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

THELEN ASSOCIATES, INC.
Geotechnical • Testing Engineers
• 1398 Cox Avenue, Erlanger, Kentucky 41018-1002 / 859-746-9400 / Fax 859-746-9408
www.thelenassoc.com
Offices
Erlanger, Kentucky
Cincinnati, Ohio
Dayton, Ohio

LOG OF TEST BORING

CLIENT: CH2M Hill BORING # 201
PROJECT: Geotechnical Exploration, Retaining Wall, Memorial Parkway Treatment Plant, Ft. Thomas, KY JOB # 080977E
LOCATION OF BORING: As shown on Boring Plan, Drawing 080977E-2

ELEV.	SOIL DESCRIPTION COLOR, MOISTURE, DENSITY, PLASTICITY, SIZE, PROPORTIONS	STRATA DEPTH feet	DEPTH SCALE feet	SAMPLE Cond	Blows/6"	No.	Type	Rec. Inches
743.0	SURFACE	0.0						
742.7	TOPSOIL	0.3		I	3/4/5	1A 1B	DS	18
	Mottled brown and gray moist medium stiff FILL, silty clay with rock fragments.			I	5/5/4	2	DS	18
			5	I	12/26/30	3	DS	4
				I	3/2/4	4	DS	3
			10	I	5/5/4	5	DS	12
				I	10/5/5	6	DS	18
			15	I	3/4/4	7	DS	18
				I	3/2/4	8	DS	1
722.0		21.0	20	I	100/3"	9	DS	1
	Auger refusal and bottom of test boring at 21.0 feet.		25					

Datum MSL Hammer Wt. 140 lbs. Hole Diameter 8 in. Foreman LW
Surf. Elev. 743.0 ft. Hammer Drop 30 in. Rock Core Dia. -- in. Engineer JPK
Date Started 8/31/09 Pipe Size O.D. 2 in. Boring Method HSA Date Completed 8/31/09

SAMPLE CONDITIONS
D - DISINTEGRATED
I - INTACT
U - UNDISTURBED
L - LOST

SAMPLE TYPE
DS - DRIVEN SPLIT SPOON
PT - PRESSED SHELBY TUBE
CA - CONTINUOUS FLIGHT AUGER
RC - ROCK CORE

GROUND WATER DEPTH
FIRST NOTED None ft.
AT COMPLETION Dry ft.
AFTER - hrs. - ft.
BACKFILLED - hrs.

BORING METHOD
HSA - HOLLOW STEM AUGERS
CFA - CONTINUOUS FLIGHT AUGERS
DC - DRIVING CASING
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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• 1398 Cox Avenue, Erlanger, Kentucky 41018-1002 / 859-746-9400 / Fax 859-746-9408

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Erlanger, Kentucky
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LOG OF TEST BORING

CLIENT: CH2M Hill BORING # 202

PROJECT: Geotechnical Exploration, Retaining Wall, Memorial Parkway Treatment Plant, Ft. Thomas, KY JOB # 080977E

LOCATION OF BORING: As shown on Boring Plan, Drawing 080977E-2

ELEV.	SOIL DESCRIPTION COLOR, MOISTURE, DENSITY, PLASTICITY, SIZE, PROPORTIONS	STRATA DEPTH feet	DEPTH SCALE feet	SAMPLE Cond	Blows/6"	No.	Type	Rec. inches
739.0	SURFACE	0.0						
738.9	TOPSOIL	0.1		I	3/4/3	1A 1B	DS	18
	Mottled brown and gray moist medium stiff FILL, silty clay with rock fragments.			I	3/4/3	2	DS	18
			5	I	2/3/2	3	DS	18
				I	3/4/2	4	DS	18
			10	I	4/3/2	5	DS	12
727.0		12.0						
	Brown moist stiff SILTY CLAY with rock fragments.			I	8/18/100/2"	6	DS	12
			15	I	100/3"	7	DS	1
				I	20/14/7	8	DS	12
719.5		19.5	20					
	Interbedded olive brown and gray moist soft weathered SHALE and gray hard LIMESTONE (bedrock).	21.5		I	18/31/49	9	DS	18
717.5								
	Bottom of test boring at 21.0 feet.		25					

Datum MSL Hammer Wt. 140 lbs. Hole Diameter 8 in. Foreman LW

Surf. Elev. 739.0 ft. Hammer Drop 30 in. Rock Core Dia. – in. Engineer JPK

Date Started 8/3/09 Pipe Size O.D. 2 in. Boring Method HSA Date Completed 8/31/09

SAMPLE CONDITIONS
D - DISINTEGRATED
I - INTACT
U - UNDISTURBED
L - LOST

SAMPLE TYPE
DS - DRIVEN SPLIT SPOON
PT - PRESSED SHELBY TUBE
CA - CONTINUOUS FLIGHT AUGER
RC - ROCK CORE

GROUND WATER DEPTH
FIRST NOTED None ft.
AT COMPLETION Dry ft.
AFTER - hrs. - ft.
BACKFILLED Immed. hrs.

BORING METHOD
HSA - HOLLOW STEM AUGERS
CFA - CONTINUOUS FLIGHT AUGERS
DC - DRIVING CASING
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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www.thelenassoc.com
Offices
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Cincinnati, Ohio
Dayton, Ohio

LOG OF TEST BORING

CLIENT: CH2M Hill BORING # 203
PROJECT: Geotechnical Exploration, Retaining Wall, Memorial Parkway Treatment Plant, Ft. Thomas, KY JOB # 080977E
LOCATION OF BORING: As shown on Boring Plan, Drawing 080977E-2

ELEV.	SOIL DESCRIPTION COLOR, MOISTURE, DENSITY, PLASTICITY, SIZE, PROPORTIONS	STRATA DEPTH feet	DEPTH SCALE feet	SAMPLE Cond	Blows/6"	No.	Type	Rec. inches
724.9	SURFACE	0.0						
	DUMPED ROCK FILL (RIP-RAP)			D	10/9/14	1	DS	6
				D	5/8/9	2	DS	6
720.4		4.5						
	Mottled dark brown and dark gray moist soft FILL, silty clay with rock fragments.		5	I	6/5/4	3	DS	12
718.9		6.0						
	Mottled brown moist medium stiff FILL, silty clay.			I	3/3/3	4	DS	18
715.4		9.5						
			10	I	20/7/8	5	DS	18
712.9	Brown moist very stiff SILTY CLAY.	12.0						
	Interbedded brown moist soft highly weathered SHALE and gray hard LIMESTONE (bedrock).			I	60/50/2"	6	DS	6
710.4		14.5	15					
	Interbedded olive brown and gray moist soft weathered SHALE and gray hard LIMESTONE (bedrock).	16.3		I	41/50/50/3"	7	DS	12
708.6								
	Bottom of test boring at 16.3 feet.		20					
			25					

Datum	MSL	Hammer Wt.	140 lbs.	Hole Diameter	8 in.	Foreman	LW
Surf. Elev.	724.9 ft.	Hammer Drop	30 in.	Rock Core Dia.	-- in.	Engineer	JPK
Date Started	8/31/09	Pipe Size	O.D. 2 in.	Boring Method	HSA	Date Completed	8/31/09

SAMPLE CONDITIONS
D - DISINTEGRATED
I - INTACT
U - UNDISTURBED
L - LOST

SAMPLE TYPE
DS - DRIVEN SPLIT SPOON
PT - PRESSED SHELBY TUBE
CA - CONTINUOUS FLIGHT AUGER
RC - ROCK CORE

GROUND WATER DEPTH
FIRST NOTED 5.0 ft.
AT COMPLETION 3.5 ft.
AFTER 3.5 hrs. 3.5 ft.
BACKFILLED 3.5 hrs.

BORING METHOD
HSA - HOLLOW STEM AUGERS
CFA - CONTINUOUS FLIGHT AUGERS
DC - DRIVING CASING
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)

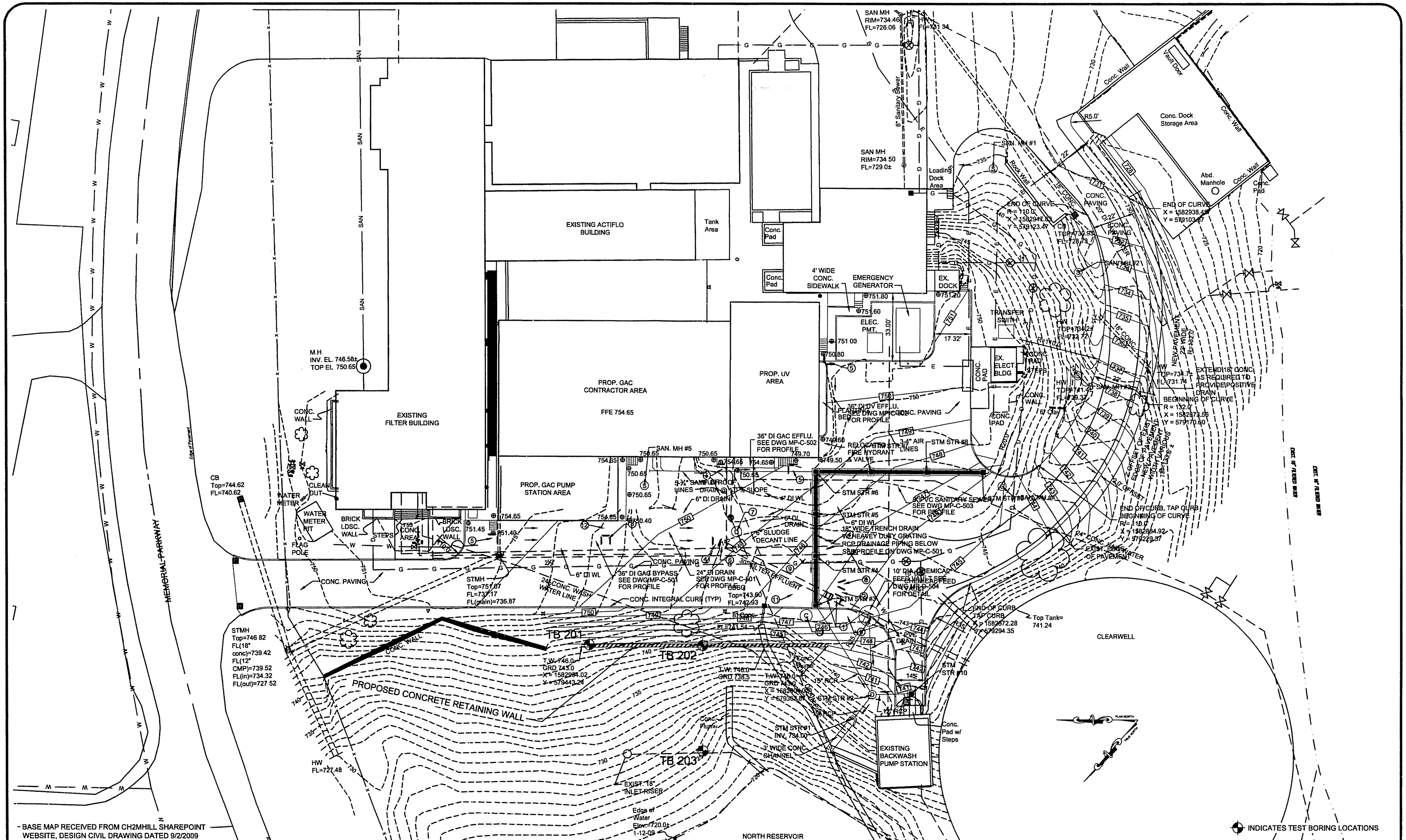
Consistency	Field Identification	Unconfined Compressive Strength (tons/sq. ft.)
Very Soft	Easily penetrated several inches by fist	Less than 0.25
Soft	Easily penetrated several inches by thumb	0.25 – 0.5
Medium Stiff	Can be penetrated several inches by thumb with moderate effort	0.5 – 1.0
Stiff	Readily 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.



Drawing Revisions	
Date:	Description:

THELEN ASSOCIATES, INC.
Geotechnical • Testing Engineers
1398 Cox Avenue / Erlanger, Kentucky 41018 / 859-746-9400
1780 Carillon Boulevard / Cincinnati, Ohio 45240 / 513-825-4350

Title: BORING PLAN

Client: CH2M Hill

Project: Geotechnical Exploration
Retaining Wall
NKWD Memorial Parkway Treatment Plant

Location: Ft. Thomas, Kentucky

Scale: 1' = 20'

Date: 9/28/2009

Drawing No.: 080977E-2