2010-00038

GEOTECHNICAL EXPLORATION

ADVANCED TREATMENT BUILDING

NKWD MEMORIAL PARKWAY PLANT

FT. THOMAS, KENTUCKY

Prepared for: **CH2M Hill**

Thelen Project No.: **080977E**

THELEN ASSOCIATES, INC.

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May 27, 2009

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

Attn: Mr. Nicholas Winnike, P.E.

Re: Geotechnical Exploration
Advanced Treatment Building
NKWD Memorial Parkway Plant
Ft. Thomas, Kentucky

Ladies and Gentlemen:

This letter is the report of our geotechnical exploration for the proposed Advanced Treatment Building to be constructed at the Northern Kentucky Water District's (NKWD) Memorial Parkway Treatment Plant (MPTP), located on Memorial Parkway in Ft. Thomas, Kentucky. Our services were authorized by verbal acceptance of our September 23, 2008 Proposal-Agreement K28212 by Mr. Nicholas Winnike, P.E. of CH2M Hill.

SCOPE

The scope of our services included three new test borings within old Sedimentation Basins 5 and 6; two new test borings within the old North Flocculation Basin; two test borings outside of the sedimentation basins; review of logs of previously-drilled borings; review of 30 percent design plan and profile sheets posted on CH2M Hill's SharePoint website on December 15 and 30, 2008; laboratory testing; engineering evaluation of the accumulated data, including recommendations for foundation design; and preparation of this geotechnical report.

PROJECT DESCRIPTION

Project plan and profile sheets for the Advanced Treatment Building were posted on CH2M Hill's SharePoint website on December 15 and 30, 2008. Following posting of these documents, Viox & Viox, Inc. (Viox) completed a site topographic survey that included the rim and invert elevation of an existing 24-inch-diameter storm drain. The survey results revealed that actual site elevations on the property and in the existing Filter Building were roughly 5.4 feet lower than had been assumed for this project, i.e., the MPTP datum used by CH2M Hill and HDR was found to be roughly 5.4 feet higher than the United States Geological Survey (USGS) datum. CH2M Hill requested that all elevations noted on the SharePoint documents used by Thelen be lowered by 5.4 feet for use on this project. All elevations noted in this report reflect this difference.

NKWD has conducted an engineering evaluation of the Memorial Parkway Treatment Plant in order to evaluate options for upgrading granular activated carbon (GAC) and ultraviolet disinfection (UV) treatment. The area including existing Sedimentation Basins 5 and 6 and the existing North Flocculation Basin is to be used for the new Advanced Treatment Building. The two concrete sedimentation basins and the concrete flocculation basin were built in 1961, and are no longer used. The sedimentation basin floors slope from north to south, from El. 741.5 feet (mean sea level [MSL] datum) to 740.5 feet. The floor elevation in the flocculation basin is 740.6 feet. Based on Sheets G-2 and G-3 of the May 31, 1961 plans prepared by J. Stephen Watkins Consulting Engineers, mass grading for the basins required up to about 20 feet of cut that exposed the interbedded shale and limestone bedrock over most of the project area.

The new Advanced Treatment Building will include four components: GAC contactors, influent and effluent pipe galleries, a GAC pump station, and a UV facility. The building will be about 158 feet by 65 feet in plan dimensions, in addition to a 54-foot by 28-foot extension for the pump station. The GAC contactor section will be located in the existing Sedimentation Basin 5 and 6 area, and will include six GAC contactor basins. Each contactor basin will share common walls with the neighboring basins. Each wall will have a 3-foot-wide walkway at the top at El. 768.6 feet. The maximum water elevation in the contactor basins will be 763.2 feet, yielding a maximum water depth of just under 20 feet. The contactor basins will contain roughly 10 feet of GAC media over a 3-inch sand course, all supported over an HDPE underdrain and an underdrain gullet. The GAC contactor section of the building will bear roughly at El. 738.1 feet on a mat foundation.

Pipe galleries will be located between the GAC contactor basins and the GAC pump station. The galleries will basically be located on two levels, with an operating floor above the second level at El. 767.1 feet. The gallery section will contain a drain trough along its length, and will bear at El. 730.6 feet on a mat foundation. The SharePoint site drawings indicate that the gallery section will bear 7.5 feet lower than the adjacent GAC Contactor section, and 9.67 feet below the lower finish floor elevation of the existing Filter Building.

The top of the ceiling slab over the GAC contactor and pipe gallery sections will be roughly at El. 778.6 feet.

The GAC pump station section will be located in the corner formed by the existing Filter Building and the new pipe gallery section of the Advanced Treatment Building. The pump station will consist of a pump well (also referred to as the wet well) on the lower level and an operating floor on the upper level. The upper floor will be at El. 754.6 feet, while the lower level will bear on a mat foundation at El. 728.6 feet, which is 11.67 feet below the lower-level floor elevation of the existing Filter Building. The existing 30-inch filter effluent pipe will be disconnected from the existing Filtered Water Basin and will discharge into the two chambers (Chamber A and Chamber B) of the pump well. The existing 24-inch drain will be rerouted to pass between the pump station and the pipe galleries.

The SharePoint drawings indicate that the west wall of the gallery section will bear higher than both the gallery floor and the pump station floor. This is because the 24-

inch storm drain will run beneath the west wall and between the gallery and pump station, and will have a crown elevation of about 735.6 feet where it passes the pump station. The west wall of the gallery section (i.e., the east wall of the pump station section) will thus bear about 9 feet above the gallery floor and about 7 feet above the pump station floor.

A cross section through the GAC contactor, pipe gallery, and pump station sections of the new building on Sheet 95189-M16 (posted on SharePoint under the filename DEST2031.pdf on December 15, 2008) shows the existing 24-inch drain bearing at El. 733.6 feet, and the new pump well floor at El. 729.6 feet. A survey of the 30-inch and 24-inch pipes exposed in test pits excavated by NKWD indicated crown elevations of 733.9 and 733.4 feet (respectively) for these pipes, meaning that they bear at about El. 731.4 feet. The locations are shown on Viox's "Topographical Survey of a Portion of Memorial Parkway Treatment Plant" dated January 9, 2009, which was used as the base plan for Thelen's Boring Plan, Drawing 080977E-1, in the report Appendix.

The UV section of the new building will be located in the existing North Flocculation Basin area. The UV section will have two floors at Els. 754.7 feet and 740.35 feet. The lower floor will contain piping and UV disinfection equipment; the upper level will be a control mezzanine. A stairwell will connect the two levels. The ceiling of the UV section's upper level will be at about El. 764.6 feet. An existing raw water tunnel and waste flume (located between the sedimentation and flocculation basins) are to remain in place between the contactor/pipe gallery and UV sections.

Foundation and floor loads for the new facilities are unavailable from the designers at this time. We have estimated that the water and the saturated GAC media will generate new distributed pressures of 1,500 to 1,800 psf on the new structure mat foundations. The 2-foot-thick concrete wall to extend between Els. 738.1 and 778.6 feet on the southeast wall of the GAC contactor section will generate a continuous load of 12,150 pounds per lineal foot (plf) on its own, without factoring in roof and suspended floor loads. The new Advanced Treatment Building will be heavily loaded, and will have adjacent portions of the structure bearing at different elevations.

Sedimentation Basins 5 and 6 and the North Flocculation Basin will be removed for construction of the new facilities. An existing concrete retaining wall and the existing lower-level access ramp will be removed for construction of the new GAC pump station. Two existing chemical storage tanks located in the GAC pump station area will also be removed.

Sedimentation Basins 5 and 6 and the North Flocculation Basin are reinforced concrete structures that have not been used for many years. To our knowledge, these structures did not have performance issues related to water leakage, slab heave, differential settlement, or other geotechnical concerns while they were in use at the treatment plant. The structural condition of the raw water tunnel and waste flume, which are to remain, is unknown to Thelen; these structures have been reviewed by the Project Structural Engineer.

In addition, existing site grades will be raised as high as El. 751.6 feet in front of the new building to allow for first-floor-level parking and entry, and the existing ramp area outside of the new GAC pump station will be filled in. This will involve placement of up to 10 feet of new fill. We understand that the existing pavement between Memorial Parkway and the existing backwash pump station will be replaced with concrete and asphalt pavements, and that the new pavement will be concrete where semis will need to park and turn in front of the GAC contactors. No grading plan has been provided, so it is not certain how much the existing pavement grades will be altered by cutting or filling.

SEISMIC REQUIREMENTS OF KENTUCKY BUILDING CODE

All commercial building project plans and specifications are required to meet the seismic requirements of KBC 2007, which defines the Maximum Credible Earthquake as that earthquake event having a 2 percent probability of exceedance in any 50-year period as the basis for seismic design. KBC 2007 also requires that local site geology, including overburden soils above the bedrock, be factored into the determination of seismic parameters to be used in structural design.

In some cases, the higher seismic standard of KBC 2007 has an impact on structural design in the Northern Kentucky Area. The effects of regional seismicity (as mandated by KBC 2007) have been considered in this study and will be addressed later in this report.

SUBSURFACE EXPLORATION

Seven new test borings were drilled on the property on December 3, 4, and 18, 2008 at the locations shown on the Boring Plan, Drawing 080977E-1, in the Appendix of this report. This plan is based on the January 9, 2009 Viox Site Plan. Thelen marked the test boring locations in the field. The boring locations and elevations were surveyed by Viox. In addition, the log of Boring 2A, previously drilled in Sedimentation Basin 5 in 2005, was used for the present study. The Boring 2A location is still visible in Sedimentation Basin 5, and was surveyed by Viox. Elevations on the log have been adjusted per the Viox survey.

Eight of the original 1961 test borings were also reviewed for the present study. Their locations are also shown on the Boring Plan. Their locations have not been adjusted per the Viox survey, since we do not know the datum that was used to establish their elevations. Detailed logs of these borings are not available; general subsurface information obtained from these borings was found on Sheet G-3 of the 1961 construction drawings.

Boring 103 was made with a truck-mounted drill rig using continuous flight augers, and by sampling ahead of the augers with a 2-inch OD split spoon driven with a 140-pound weight falling 30 inches. This procedure is described as the standard drive sample method and results in the standard penetration test (SPT) as per ASTM D1586. Borings 101, 102, 103A, 104, and 105 were made using hand equipment following coring of the existing basin slabs with a concrete coring barrel. Samples were obtained using a 2-inch OD split spoon driven with a 35-pound weight falling 30 inches. No attempt was made to correlate the N-values (i.e., blows required to drive the sampler the last 12 inches) obtained with the 35-pound hammer to those that would likely have been obtained by the standard penetration test. Boring 103B was made with a truck-mounted drill rig using continuous flight augers to advance the hole to the surface of the gray, unweathered bedrock, and by coring the rock with an NQ-sized, diamond-tipped core barrel.

As each test boring was advanced, the Drilling Technician kept a log of the subsurface profile encountered, noting soil and bedrock types and stratifications, SPT results, groundwater, and other pertinent data. In addition, a representative portion of each split spoon sample was placed in a glass jar. The jars were sealed and marked for proper identification.

Borings 101 and 104 were drilled through 2 inches of surface water, and were backfilled with hydrated bentonite chips. Borings 102, 103A, and 105 were backfilled with bentonite chips and concrete surface patches. Borings 103 and 103B were backfilled with the drill cuttings and an asphalt surface patch. Only Boring 103 encountered natural, free subsurface water at a depth of 12.0 feet (El. 729.6 feet), during and up to 3 hours after completion of drilling.

In addition to the test borings, two test pits were made by NKWD in the access ramp area in order to locate and determine the crown elevations of the existing 30-inch effluent filter pipe and the 24-inch drain where they run beneath the ramp. The exposed pipe crowns were surveyed for both location and elevation by Viox & Viox, Inc.

The eight 1961 test borings reviewed for this study yielded the following bedrock surface elevations.

BORING	BEDROCK SURFACE ELEVATION (FT., MSL)
2A	752
2B	744
3A	721 (refusal)
4	747.5
4A	754
5	753
13	739
13A	722 (refusal)

No free subsurface water was indicated for these borings on Sheet G-3 of the 1961 construction drawings.

LABORATORY TESTING

The samples from the test borings were returned to our Soil Mechanics Laboratory, where they were reviewed and classified by the Project Geotechnical Engineer. Representative soil samples were selected for natural moisture content and Atterberg limits testing. Representative bedrock samples were selected for Atterberg limits and unconfined compressive strength testing. A tabulation of the test results is included in the Appendix to this report.

On the basis of visual examination of the samples, the laboratory test results, and the field logs kept by our Drilling Technician, final test boring logs were prepared. Copies of the final logs are included in the Appendix, together with a Soil Classification Sheet that describes the terms and symbols used on the logs.

SITE AND SUBSURFACE CONDITIONS

As discussed previously, the new GAC contactor/pipe gallery structure is to be constructed within the area of existing Sedimentation Basins 5 and 6. The new UV facility is to be constructed within the area of the existing North Flocculation Basin. The new GAC pump station is to be constructed within the area outside the existing west wall of Sedimentation Basin 6.

Subsurface Geology

In general, the borings encountered medium stiff to stiff, silty clay and dense pea gravel fill over the interbedded, shale and limestone bedrock of the Upper Ordovician Fairview Formation. The shale and limestone bedrock in the Northern Kentucky Area generally occurs in three zones, distinguished by degree of weathering of the shale. In general, the uppermost zone of the shale is brown to olive brown, highly weathered, and has moisture contents in the middle to upper teens; the shale in the next zone is gray and brown, weathered, and has moisture contents in the lower teens; the shale in the lowermost zone is the gray, slightly weathered to unweathered, parent bedrock, and has moisture contents below 10 percent. Published geologic mapping indicates that overall, the Fairview Formation contains about 50 percent limestone. In our experience, karst development is generally not an issue in the Fairview Formation because of the presence of interbedded shales and siltstones in the bedrock.

Boring 101 was drilled through 2 inches of water standing in the North Flocculation Basin. The boring encountered 9½ inches of concrete and 8½ inches of a multicolored, wet, medium to coarse sand and fine gravel (i.e., pea gravel) subbase. The boring encountered refusal at a depth of 18 inches below the top of the slab, possibly on a limestone layer within the bedrock, and was terminated there. We are assuming that the split spoon sampler refused on the bedrock surface; this can only be confirmed during excavation for the new structure. The boring was backfilled with hydrated bentonite chips. A concrete patch could not be placed because of the standing water.

Boring 102 encountered 6 inches of concrete; 4 inches of pea gravel subbase; and 5 inches of interbedded, brown, moist, very soft, very highly weathered shale and gray hard limestone. The shale exhibited a moisture content of 17.9 percent. A measured liquid limit of 33 percent corresponded to a plasticity index of 12 percent, classifying the tested sample as a CL material under the Unified Soil Classification System (USCS), and confirming its low plasticity. The boring was terminated at a depth of 13 inches below the top of the slab, and was backfilled with hydrated bentonite chips and a concrete patch.

Boring 103 encountered 5 inches of asphalt underlain by 7.6 feet of brown and gray, moist to very moist, medium stiff to stiff, silty clay fill containing traces of shale and limestone fragments, and then by the shale and limestone bedrock. Measured moisture contents in the fill ranged from 21.2 to 26.7 percent. The base of the fill was at El. 733.6 feet. The highly weathered bedrock zone was 1.5 feet thick; the shale portion exhibited a moisture content of 14.8 percent. The weathered zone was 3.0 feet thick. A measured liquid limit of 28 percent corresponded to a plasticity index of 12 percent. These Atterberg limits classify the weathered shale as a CL material under the USCS, and confirm its low plasticity. A measured moisture content in the gray, unweathered shale was 10.2 percent. The boring was terminated in the gray, unweathered zone at a depth of 13.5 feet (El. 728.1 feet).

Boring 103A encountered 6½ inches of concrete and 5 inches of interbedded, brown, moist, very soft, very highly weathered shale and gray hard limestone. The shale component exhibited a moisture content of 18.1 percent. The boring was terminated at a depth of 12.5 inches below the top of the slab, and was backfilled with hydrated bentonite chips and a concrete patch.

Boring 103B was drilled adjacent to Boring 103 after a limestone floater caused deflection of the drill string in Boring 103. Boring 103B was augered to a depth of 12.0 feet (El. 729.6 feet), after which the bedrock was cored to a depth of 27.0 feet (El. 714.6 feet) in three, 5-foot-long core runs. From top to bottom, recoveries in the three core runs were 83, 100, and 98 percent; rock quality designations (RQD) were 33, 50, and 71 percent; and limestone percentages were 43, 33, and 14 percent. The measured limestone percentages were less than expected, based on published geologic mapping. The limestone bed thicknesses encountered in the recovered core ranged from 0.5 to 7 inches. Measured unconfined compressive strengths of recovered limestone samples of 1,335.5 and 1,930.9 ksf (9,274.3 and 13,409.0 psi) corresponded to respective measured dry densities of 168.6 and 169.5 pounds per cubic foot (pcf) and moisture contents of 0.2 percent. Measured unconfined compressive strengths of recovered shale samples of 32.0 and 45.9 ksf (222.2 and 318.8 psi) corresponded to respective measured dry densities of 140.9 and 146.3 pcf and moisture contents of 8.1 and 6.2 percent. A measured shale liquid limit of 38 percent corresponded to a plasticity index of 15 percent, classifying the tested sample as a CL material under the USCS, and confirming its low plasticity. The boring was terminated at a depth of 27.0 feet.

Boring 104 was also drilled through 2 inches of water standing in the North Flocculation Basin. The boring encountered 10¼ inches of concrete; ¼ inch of pea gravel subbase; and 6 inches of interbedded, gray, moist to very moist, very soft shale and gray hard limestone. The boring was terminated at a depth of 17 inches below the top of the slab, and was backfilled with hydrated bentonite chips. A concrete patch could not be placed because of the standing water.

Boring 105 encountered 5½ inches of concrete; 1½ inches of a multicolored, medium to coarse sand and fine gravel (i.e., pea gravel) subbase; and 2 inches of interbedded, gray, very moist, very soft shale and gray hard limestone. The shale component exhibited a moisture content of 28.3 percent, likely due to exposure to water in the pea gravel since 1961. The boring was terminated at a depth of 9 inches below the top of the slab, and was backfilled with hydrated bentonite chips and a concrete patch.

Boring 2A (drilled in 2005) encountered 5¾ inches of concrete; 4 inches of a multicolored, very moist, coarse sand and fine gravel (i.e., pea gravel) subbase; 4 inches of interbedded, brown to olive brown, very soft, weathered shale and gray hard limestone; and two inches of interbedded, gray, soft, unweathered shale and gray hard limestone. The weathered shale exhibited a moisture content of 9.7 percent. The gray, unweathered shale exhibited a moisture content of 12.6 percent. The boring was terminated at a depth of about 16 inches below the top of the slab, and was backfilled with hydrated bentonite chips and a concrete patch.

In addition to the borings, a large test pit was excavated adjacent to the southwest end of the existing parking area retaining wall. The soils exposed in the large test pit were observed by the writer. The excavated soils generally consisted of brown and gray, moist, stiff fill of shale origin. The test pit was extended to about El. 732 feet.

Groundwater

The borings drilled in the concrete basins were backfilled immediately upon completion. Apart from Borings 101 and 104, which were drilled through standing water, and Boring 103B, which was drilled using coring water, only Boring 103 encountered free subsurface water, at a depth of 12.0 feet (El. 729.6 feet), during and up to 3 hours after completion of drilling. Based on our experience, excavations made below the bedrock surface can be expected to generate groundwater seepage via limestone beds exposed in the excavated sidewalls. The seepage per unit exposed area is usually minor, but when considered over the entire exposed excavation face, can be significant. Usually, this seepage can be controlled with sumps and pumps; consideration must be given to the height through which the seepage must be pumped in order to discharge it from the excavation.

An issue with groundwater seepage is its effect on shale subgrades. The gray, slightly weathered shale component of the bedrock exists in situ at low moisture contents, typically less than 8 percent. Upon exposure to water, these shales typically absorb water and begin to slake. If water absorption and subsequent moisture content increase occur slowly, the shale will swell, and can generate significant swell pressures and cause slab heave. These issues are typically handled in design and construction by requiring the Contractor to control seepage in excavations, and to provide adequate drainage via proper grading and the use of foundation drains. Placement of thin, concrete mud mats over prepared shale subgrades is also an effective method of isolating exposed shale subgrades from groundwater seepage, and allows the

Contractor to place slab and footing reinforcing steel without ruining a slaking shale subgrade via foot traffic.

A possibility also exists that groundwater seepage will occur through overburden fill and native soils exposed in the GAC Pump Station excavation to be made alongside the south wall of the existing Filter Building. The presence and character of such soils is unknown at this time, as existing conditions there will not be exposed until demolition of the existing construction begins.

Existing Effluent Pipe and Storm Sewer

A Memorial Parkway Treatment Plant Site Plan posted on Share Point on December 15, 2008 indicates a centerline elevation of the 30-inch filter effluent pipe at 732.6 feet (after the 5.4-foot adjustment) both where it exits the Filter Building and where it enters the Filtered Water Basin. The 30-inch pipe elevation was surveyed at three locations in the two test pits, including one location at a low-angle pipe elbow; the three elevations obtained varied from 733.86 to 734.28 feet. (The Viox survey indicates that the crown elevation of 734.28 feet may include some thickness of concrete encasement around the pipe.) The crown elevations obtained by Viox indicate that the 30-inch pipe proceeds slightly upgradient from the direction of the existing filter building to the low-angle elbow, at a slope of 2.2 percent, and then downgradient from the elbow to the southwest at a slope of 6.8 percent. The 30-inch pipe was seen to pass beneath the southwest end of the parking area retaining wall.

The Memorial Parkway Treatment Plant Site Plan indicates an invert elevation of the 24-inch drain at 733.60 feet (after the 5.4-foot adjustment) where it exits the Filter Building. The 24-inch pipe was surveyed by Viox at one location only, and had a reported crown elevation of 733.39 feet.

CONCLUSIONS AND 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

Site Preparation

6. Cuts for this project are expected to extend as deep as El. 728.6 feet (at the GAC pump station location). The excavated materials will include the interbedded shale and limestone bedrock of the Fairview Formation. Fifteen feet of continuous bedrock core was obtained from the Fairview Formation in Boring 103B, which consisted of up to 43 percent limestone in beds as thick as 7 inches. The difficulty of excavation can be expected to increase with depth into the bedrock.

7. The Contractor will need to be prepared to control groundwater seepage in excavations made below the bedrock surface.

8. We are assuming that the GAC pump station excavation will be made beyond the lateral limits of the existing retaining wall backfill and 30-inch effluent filter pipe backfill, and that the excavation sidewalls will expose the interbedded shale and limestone bedrock. We are also assuming that the excavation for the GAC pump station and contactor sections will expose the interbedded shale and limestone bedrock beneath the existing Filter Building. We recommend that the Contractor shore or slope the pump station and contactor excavations in compliance with all applicable Federal, State, and local excavation regulations; the shoring design for the parts of the excavations made adjacent to the existing Filter Building should be designed and constructed in a manner that will provide support for the exposed portion of the south Filter Building wall during construction of the Advanced Treatment Building. The Contractor should be made responsible for design of the shoring system by a Kentucky-licensed Civil Engineer and for excavation safety during the project. While we expect the interbedded shale and limestone bedrock to stand at steep angles for the duration of construction, the Contractor should be aware that as the cut angle steepens, the frequency of rockfalls and the intensity of shale slaking into the excavation will also increase.

9. Prior to raising grades for the new pavements, and prior to replacement of the existing pavement, we recommend that all existing topsoil and surficial vegetation be stripped from proposed cut and fill areas and be wasted off-site or be stockpiled for use as topsoil following completion of the project. Demolition debris from the existing pavement should be wasted off-site.

10. The nature and condition of the existing pavement subgrade materials is unknown. Any site fill, soft to medium stiff native soils, or water-softened shale exposed on the stripped subgrades, within and up to 3 feet outside of the new pavement limits, should be undercut from the proposed cut and fill areas to expose stiff soils or bedrock, on which a compacted and tested fill may be started.

11. It is possible that the new pavement subgrades may expose gray, unweathered shale. Because the gray shale exists at very low moisture contents (typically 2 to 8 percent), there is potential for the surface to hydrate and break down in the long term after the pavement is completed. We recommend that any gray shale exposed on new pavement subgrades be undercut at least 12 inches and be replaced with clayey, compacted and tested fill.

12. If soft or yielding soils are still exposed on heavy-duty pavement subgrades 4 feet below the planned final subgrade elevation, or on light-duty pavement subgrades 3 feet below the planned final subgrade elevation, the soft or yielding areas should be bridged with a biaxial geogrid (such as Tenser BX1200 or approved equivalent) and 12 inches of No. 57 crushed limestone. The geogrid should be pulled taut and even prior to crushed stone placement, since it develops its bridging strength in tension.

13. Following overexcavation to suitable soils or bridging with geogrid and No. 57 stone, final pavement subgrade elevations should then be established with compacted and tested fill.

14. Any existing, poorly compacted utility backfill will not provide adequate pavement support. Any such backfill should be removed (to the depth of the utility if necessary) and be replaced. As an alternative, narrow zones of poorly compacted utility backfill can be bridged with a biaxial geogrid (such as Tensar BX1200 or approved equivalent) pulled taut and extending to at least 5 feet to either side of the trench edges in conjunction with 12 inches of compacted, No. 57 crushed stone.

15. We recommend that two unit cost items be included in the contract documents: one to provide compensation to the Earthwork Contractor for undercuts on a "cost per cubic yard of compacted replacement fill basis" with compacted fill, and the other for removal and replacement with No. 57 crushed limestone and geogrid, whichever is applicable.

16. New clayey fill and backfill should be placed on prepared surfaces in shallow level layers, 6 to 8 inches in loose thickness, and should be compacted with appropriate equipment to a density not less than 98 percent of the maximum dry density determined by the standard Proctor moisture-density test, ASTM D698. The moisture content of the fill at the time of the compaction should be within minus 2 to plus 3 percent of the optimum moisture content. The fill soils should consist of clean, clayey overburden soils from the on-site excavated areas or approved borrow soils. Fill soils should be free of topsoil, vegetation, trash, construction or demolition debris, frozen materials, or other deleterious materials. Moderate to highly plastic soils may only be used within 2 feet of final slab subgrade elevations if thoroughly mixed with low plastic soils such that the resulting material does not have a plasticity index exceeding 22 percent. Any gray shale to be used as fill should first be pulverized to a soil-like consistency and then be moisture conditioned; the Contractor should be aware that moisture conditioning of gray shale may require enough water to raise the natural moisture content by 10 percent or more.

17. Clean, granular fill and backfill should be placed on prepared surfaces in shallow level layers, 4 to 6 inches in loose thickness, and should be compacted with appropriate equipment to at least 85 percent relative density as determined by ASTM D4253 and D4254. (The ASTM D4253 and D4254 criteria only apply to granular soils that are clean enough so as not to exhibit a well-defined moisture density curve as per the standard Proctor [ASTM D698] Test Method). As with clayey fill, granular fill and backfill soils should be free of topsoil, vegetation, trash, construction or demolition debris, frozen materials, or other deleterious materials. The exception would be the use of demolished and crushed structural concrete obtained from on site demolition activities, which in our opinion would be acceptable if a) reinforcing steel and other demolition debris are excluded, and b) it is crushed to a maximum size of 2 inches in maximum dimension.

18. The use of limestone floaters and slabs should be minimized in the fills, and should not be allowed in the upper two feet of pavement subgrades. Some limestone can be incorporated into the fill if necessary, provided that it is broken up to less than 6 inches in maximum dimension and dispersed, and in the opinion of the Geotechnical Engineer or his representative, it does not nest or retard compaction.

19. We recommend that straw bales or silt fences be staked in areas of concentrated runoff to prevent siltation of adjacent properties during the work. Exposed surfaces should be seeded and mulched or paved as soon as possible after the earthwork is completed.

<u>Foundations</u>

20. Comparison of proposed mat foundation bearing elevations with the subsurface conditions encountered in the borings indicates that foundations in the GAC contactor and pump station areas bearing at the proposed elevations will bear on the interbedded shale and limestone bedrock, which will be suitable for support of the heavy structure loads. Portions of the mat foundation for the new UV

facility will need to be lowered several inches to bear on competent, unweathered bedrock. If site fill, native residual soils, or water-softened bedrock are encountered on any subgrades at proposed mat or footing bearing elevations, we recommend that the subgrades be lowered as necessary to bear on competent bedrock. Allowable bearing capacities in the shale and limestone bedrock are 6,000 psf in the brown to olive brown, highly weathered zone; 10,000 psf in the gray and brown, weathered zone; and 30,000 psf in the gray, unweathered zone (full dead plus full live load). We note that in four of the seven test borings, the surficial bedrock was either highly weathered or weathered. The foundation bearing subgrades may need to be lowered to attain higher design bearing capacities.

21. Any new continuous footings should be a minimum of 16 inches wide. Any new individual column footings should be at least 24 inches square.

22. All loose, soft, wetted, or dried materials should be skimmed from mat or footing excavated subgrades before reinforcing steel and concrete are placed. The concrete should be placed on bedrock that is moist, not wet or dry. If bearing surfaces become excessively wet or dry before the concrete is placed, they should be skimmed to expose moist, stiff shale. We recommend that foundation concrete be placed on mat subgrades or in footing trenches the same day that they are excavated and prepared to prevent ponding of water on the subgrades. Foundation construction should be scheduled during favorable weather, and good drainage should be maintained during construction to prevent water from ponding in or around any new footing or mat foundation excavations.

23. While the gray, unweathered shale provides an excellent bearing surface while it is kept dry, it softens and slakes quickly upon exposure to water. In our experience, gray shale subgrades can be quickly ruined by seepage water and foot traffic in much less time than it takes for laborers to set footing or mat reinforcing steel over the subgrades. The subgrades are then very difficult to

clean of sloughed and remolded materials with the reinforcing steel in place. Based on our experience with water seepage on shale subgrades, we recommend that the Contractor be required to clean the exposed shale subgrade and then to immediately place a 2-inch-thick, concrete mud mat on the subgrade to protect it from seepage and foot traffic. The Contractor should let the mud mat set up at least overnight, and then can set reinforcing steel on the mud mat the following day.

24. The SharePoint Drawings show several instances where foundations for adjacent parts of the new and existing construction will bear at different elevations. The drawings also show that the new GAC contactor floor will tie into the wall of the existing tunnel. In these cases, we recommend that stepped foundations be used, or that foundations of adjacent structures bear at the same elevation, to avoid lateral surcharging of lower walls by adjacent higher foundations, unless these lateral surcharges can be accommodated in design.

25. All new mat foundation and footing excavations and subgrades should be examined by the Project Geotechnical Engineer or his representative before reinforcing steel and concrete are placed to confirm that the design recommendations have been properly interpreted and followed by the Contractor.

26. Assuming that our foundation recommendations are followed, we expect total and differential settlements to be less than ¼ inch.

<u>Pavements</u>

27. We recommend that compacted and tested, clayey pavement subgrades be sloped to drain towards catch basins or to collector pipes in order to drain any granular subbase materials used in the pavement sections. The collector pipes should consist of 4-inch-diameter, perforated (perforations down), Schedule 40 PVC pipes sloped to drain to gravity outlets. Undrained granular subbase materials will collect water that can cause subgrade deterioration.

28. We recommend that the top 8 inches of clayey pavement subgrades be scarified, moisture conditioned, and recompacted to at least 100 percent of the maximum dry density obtained using the ASTM D698 test method immediately prior to paving, so that the subgrade will be moist and dense at the time of pavement construction.

Earth Pressures and Retaining Walls

29. Below-grade walls should be designed to resist lateral earth pressures due to differential backfill heights. If rotation of the wall tops will occur after backfilling, we recommend that active earth pressures be approximated using a drained equivalent fluid weight of 51 pcf plus any appropriate surcharge due to pavements, floor slabs, sloping backfill, etc. If rotation of the wall tops will not occur after backfilling, we recommend that at-rest earth pressures be approximated using a drained equivalent fluid weight of 73 pcf plus any appropriate surcharge due to pavements, floor slabs, sloping backfill, etc. These earth pressures assume that drained backfill conditions are provided as per Item 31 below.

30. Resistance to sliding may be provided by friction between the footing or mat foundation concrete and the foundation soils. An ultimate friction coefficient of 0.35 may be used to estimate sliding resistance in combination with an appropriate safety factor.

31. Exterior footing drains should be installed behind all below-grade, backfilled walls to prevent buildup of hydrostatic pressures. Each drain should consist of a perforated, Schedule 40 PVC pipe (perforations down) placed at the base of the wall and connected to a gravity outlet. The drain should be connected to a minimum 24-inch width of free-draining granular backfill extending up to about 2 feet below proposed exterior grades. A manufactured drainage mat can be substituted for the minimum 24-inch width of free-draining granular backfill. All wall backfill should be compacted as per Items 16 and 17.

32. The upper 2 feet of the wall backfill should consist of compacted and tested clayey fill to inhibit infiltration of surface water. All wall backfill should meet the requirements for compacted and tested fill outlined in the "Site Preparation" section.

Utilities

33. We recommend that all utility excavations for this project be backfilled with approved on-site soils or approved borrow placed in uniform level layers where they run beneath pavement surfaces. Each clayey layer should be 6 to 8 inches in loose lift thickness, and should be compacted within minus 2 to plus 3 percent of the optimum moisture content with an appropriate type of compaction equipment to at least 98 percent of maximum density as determined by the standard Proctor moisture-density test, ASTM D698. Where clean, granular fill is used that does not exhibit a well-defined moisture-density curve per ASTM D698, granular layers should be 4 to 6 inches in loose lift thickness, and should be compacted to a minimum of 85 percent relative density as per ASTM D4253 and D4254. Under no conditions should any backfill be flushed to obtain compaction.

34. The Viox survey results suggest that the 30-inch filter effluent pipe is sloped slightly upgradient upon leaving the existing filter effluent building. If the pipe does trend upgradient as it leaves the building, a decision will need to be made as to whether or not the pipe section to remain needs to be rebuilt with a downward gradient towards the new GAC pump station.

Drainage

35. Final grades should be set to promote drainage away from the new building and pavements. We recommend a minimum slope of 10 percent away from the building in the first 10 feet, and a minimum 2 percent slope thereafter in both paved and landscaped areas. Water should not be permitted to pond around the building or at the edges of pavements.

Winter/Spring Construction

36. Care should be taken during winter and early spring construction so that no concrete, asphalt, or new fill are placed over frozen or saturated soils. Frozen or saturated soils should not be used for compacted fill or backfill. Historically, the optimum time for earthwork construction in the Northern Kentucky Area is mid-May through October because of the historically more favorable weather conditions during that period.

CLOSURE

We have included with this letter 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 have appreciated the opportunity to provide these geotechnical recommendations to you for this project. If there are any questions concerning the information contained in this report, or if we may be of further service to you, please do not hesitate to contact us.

Respectfully submitted,
THELEN ASSOCIATES, INC.

John S. Nealon, P.E., P.G.
Senior Geotechnical Engineer/Geologist

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

JSN/TWV:tmk
080977E

Copies submitted: 3 – Client
1 – HDR Engineering, Inc.

APPENDIX

ASFE Report Information

Tabulation Of Laboratory Tests

Laboratory Test Forms

Test Boring Log, Thelen Project 050916E

Test Boring Logs, Thelen Project 080977E

Soil Classification Sheet

Boring Plan, Drawing No. 080977E-1 (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

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1398 COX AVENUE
ERLANGER, KENTUCKY 41018-1002

GEOTECHNICAL EXPLORATION
ADVANCED TREATMENT BUILDING
MEMORIAL PARKWAY TREATMENT PLANT
FT. THOMAS, KENTUCKY
080977E

TABULATION OF LABORATORY TESTS

Boring Number	Sample Number	Depth, ft.		Moisture Content, %	Atterberg Limits, %			Natural Dry Density, pcf	Uncofined Compressive Strength		USCS Classification
		From	To		LL	PL	PI		ksf	psi	
102	2	0.6	1.1	17.9	33	21	12				CL
103	1	0.4	1.7	21.2							
	2	2.5	4.0	26.7							
	3	5.0	6.5	25.0							
	5	8.0	8.2	14.8							
	6	10.0	11.0	16.2	28	16	12				CL
	7	13.0	13.5	10.2							
103A	2	0.5	1.0	18.1							
103B	RC-1A	12.4	12.7	0.2				168.6	1335.5	9274.3	
	RC-2A	18.8	19.2	0.2				169.5	1930.9	13409.0	
	RC-2B	20.5	21.0	8.1	38	23	15	140.9	32.0	222.2	CL
	RC-3A	26.6	27.0	6.2				146.3	45.9	318.8	
105	2	0.6	0.8	28.3							
2A	2A	0.8	1.1	9.7							
	2B	1.1	1.3	12.6							

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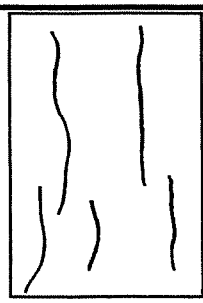
UNCONFINED COMPRESSIVE STRENGTH OF INTACT ROCK CORE, ASTM - D2938
UNIT WEIGHT AND NATURAL MOISTURE

CLIENT : CH2M Hill
PROJECT: Geotechnical Exploration, Advanced Treatment Building, MPTP
LOCATION: Ft. Thomas, Kentucky

PROJECT NUMBER: 080977E
BORING NO.: 103B SAMPLE NUMBER: RC-1A DEPTH (ft.): 12.4-12.7
SAMPLE DESCRIPTION: White to light gray hard biospartic LIMESTONE
BEDROCK FORMATION: Fairview Formation DATE: 12/22/2008
SAMPLE OBTAINED BY: Rock Core CONDITION: Undisturbed LOAD DIRECTION 90° TO LITHOLOGY

NATURAL UNIT WEIGHT	
AVERAGE DIAMETER (in.)	1.86
HEIGHT (in.)	4.06
HEIGHT TO DIAMETER RATIO	2.18
AVERAGE AREA (sq. ft.)	0.0189
VOLUME (cu. ft.)	0.0064
WET WEIGHT (lbs.)	1.08
DRY WEIGHT (lbs.)	1.08
DRY DENSITY (pcf)	**168.6**

FAILURE SHAPE

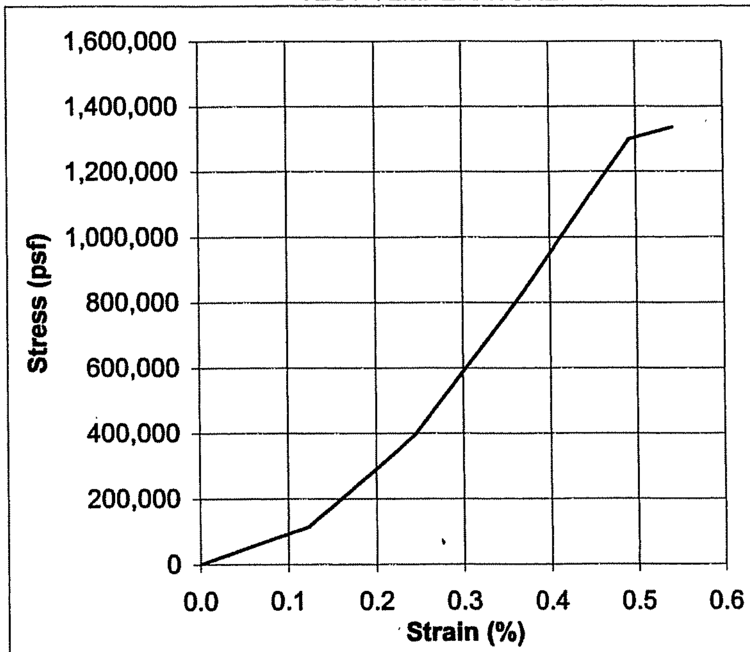


WATER CONTENT AFTER SHEAR	
CAN NUMBER	T23
WET WEIGHT + CAN (lbs.)	1.97
DRY WEIGHT + CAN (lbs.)	1.96
WEIGHT WATER (lbs.)	0.00
WEIGHT CAN (lbs.)	0.89
WEIGHT SOLID (lbs.)	1.07
MOISTURE (%)	**0.2**

PROVING RING NO.: QC200

DEFORM DIAL (0.001 in.)	LOAD DIAL (0.001 in.)	LOAD (lbs.)	STRAIN (%)	STRESS (psf)
0	0	0	0.0	0
5	2,140	2,140	0.1	113,412
10	7,470	7,470	0.2	395,883
15	15,765	15,765	0.4	835,489
20	24,515	24,515	0.5	1,299,208
22	25,200	25,200	0.5	1,335,510

TEST TEMPERATURE: 70°F



AVERAGE RATE OF STRAIN TO FAILURE (%/min.)	**0.2**
STRAIN AT FAILURE (%)	**0.5**
UNCONFINED COMPRESSIVE STRENGTH (ksf)	**1,335.5**
SHEAR STRENGTH (ksf)	**667.8**

REMARKS :

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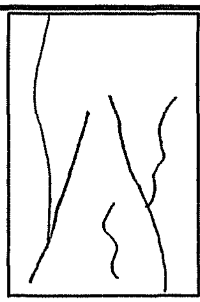
UNCONFINED COMPRESSIVE STRENGTH OF INTACT ROCK CORE, ASTM - D2938
UNIT WEIGHT AND NATURAL MOISTURE

CLIENT : CH2M Hill
PROJECT: Geotechnical Exploration, Advanced Treatment Building, MPTP
LOCATION: Ft. Thomas, Kentucky

PROJECT NUMBER: 080977E
BORING NO.: 103B SAMPLE NUMBER: RC-2A DEPTH (ft.): 18.8-19.2
SAMPLE DESCRIPTION: White to light gray hard biospartic LIMESTONE
BEDROCK FORMATION: Fairview Formation DATE: 12/22/2008
SAMPLE OBTAINED BY: Rock Core CONDITION: Undisturbed LOAD DIRECTION 90° TO LITHOLOGY

NATURAL UNIT WEIGHT	
AVERAGE DIAMETER (in.)	1.86
HEIGHT (in.)	4.45
HEIGHT TO DIAMETER RATIO	2.39
AVERAGE AREA (sq. ft.)	0.0189
VOLUME (cu. ft.)	0.0070
WET WEIGHT (lbs.)	1.19
DRY WEIGHT (lbs.)	1.19
DRY DENSITY (pcf)	**169.5**

FAILURE SHAPE

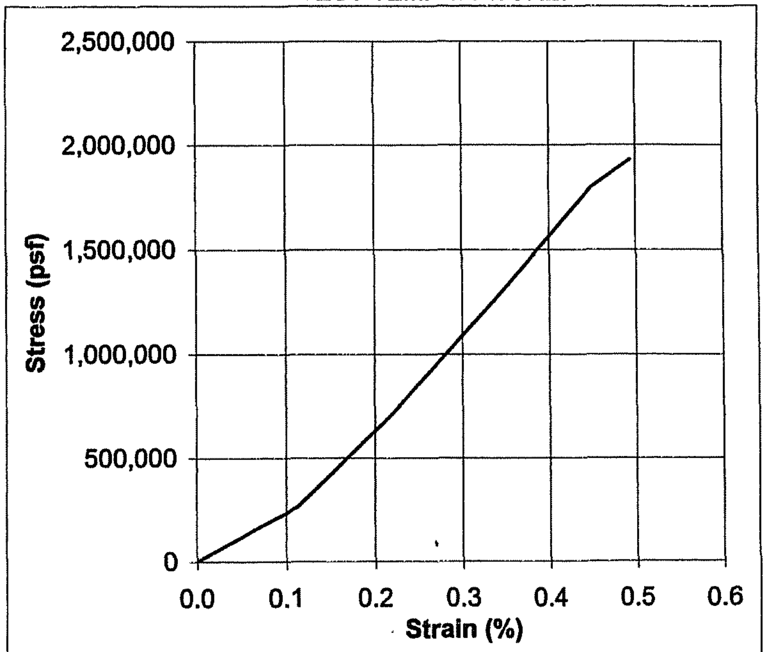


WATER CONTENT AFTER SHEAR	
CAN NUMBER	T29
WET WEIGHT + CAN (lbs.)	1.86
DRY WEIGHT + CAN (lbs.)	1.86
WEIGHT WATER (lbs.)	0.00
WEIGHT CAN (lbs.)	0.90
WEIGHT SOLID (lbs.)	0.96
MOISTURE (%)	**0.2**

PROVING RING NO.: QC200

TEST TEMPERATURE: 70°F

DEFORM DIAL (0.001 in.)	LOAD DIAL (0.001 in.)	LOAD (lbs.)	STRAIN (%)	STRESS (psf)
0	0	0	0.0	0
5	4,950	4,950	0.1	262,332
10	13,895	13,895	0.2	736,386
15	23,845	23,845	0.3	1,263,700
20	33,915	33,915	0.4	1,797,374
22	36,435	36,435	0.5	1,930,925



AVERAGE RATE OF STRAIN TO FAILURE (%/min.)	**0.1**
STRAIN AT FAILURE (%)	**0.5**
UNCONFINED COMPRESSIVE STRENGTH (ksf)	**1,930.9**
SHEAR STRENGTH (ksf)	**965.5**

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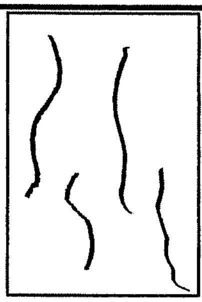
UNCONFINED COMPRESSIVE STRENGTH OF INTACT ROCK CORE, ASTM - D2938
UNIT WEIGHT AND NATURAL MOISTURE

CLIENT : CH2M Hill
PROJECT: Geotechnical Exploration, Advanced Treatment Building, MPTP
LOCATION: Ft. Thomas, Kentucky

PROJECT NUMBER: 080977E
BORING NO.: 103B SAMPLE NUMBER: RC-2B DEPTH (ft.): 20.5-21.0
SAMPLE DESCRIPTION: Gray soft thinly laminated SHALE
BEDROCK FORMATION: Fairview Formation DATE: 12/22/2008
SAMPLE OBTAINED BY: Rock Core CONDITION: Undisturbed LOAD DIRECTION 90° TO LITHOLOGY

NATURAL UNIT WEIGHT	
AVERAGE DIAMETER (in.)	1.84
HEIGHT (in.)	4.29
HEIGHT TO DIAMETER RATIO	2.33
AVERAGE AREA (sq. ft.)	0.0185
VOLUME (cu. ft.)	0.0066
WET WEIGHT (lbs.)	1.00
DRY WEIGHT (lbs.)	0.93
DRY DENSITY (pcf)	**140.9**

FAILURE SHAPE

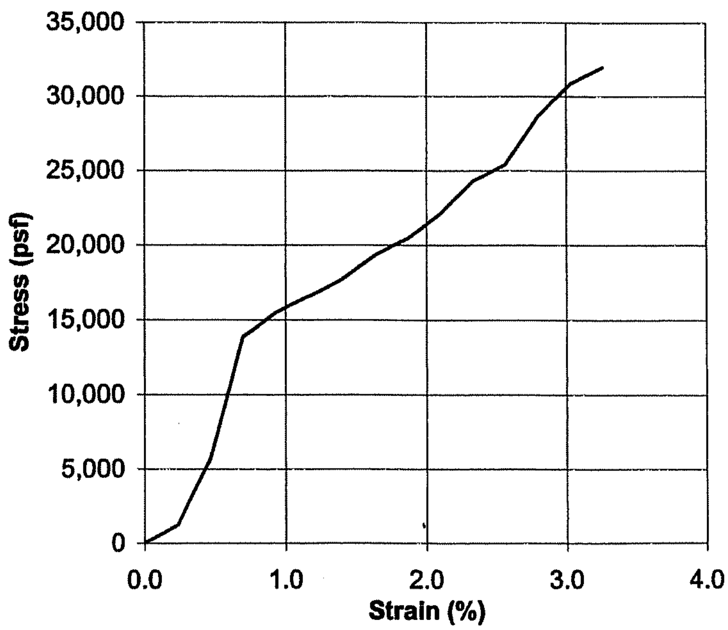


WATER CONTENT AFTER SHEAR	
CAN NUMBER	KP8
WET WEIGHT + CAN (lbs.)	1.91
DRY WEIGHT + CAN (lbs.)	1.84
WEIGHT WATER (lbs.)	0.07
WEIGHT CAN (lbs.)	0.93
WEIGHT SOLID (lbs.)	0.90
MOISTURE (%)	**8.1**

PROVING RING NO.: 78-0860

DEFORM DIAL (0.001 in.)	LOAD DIAL (0.001 in.)	LOAD (lbs.)	STRAIN (%)	STRESS (psf)
0	0	0	0.0	0
10	3	23	0.2	1,223
20	11	104	0.5	5,617
30	26	256	0.7	13,855
40	29	286	0.9	15,502
50	31	307	1.2	16,601
60	33	327	1.4	17,699
70	36	357	1.6	19,347
80	38	378	1.9	20,445
90	41	408	2.1	22,093
100	45	449	2.3	24,290
110	47	469	2.6	25,388
120	53	530	2.8	28,684
130	57	570	3.0	30,880
140	59	591	3.3	31,979

TEST TEMPERATURE: 70°F



AVERAGE RATE OF STRAIN TO FAILURE (%/min.)	**0.9**
STRAIN AT FAILURE (%)	**3.3**
UNCONFINED COMPRESSIVE STRENGTH (ksf)	**32.0**
SHEAR STRENGTH (ksf)	**16.0**

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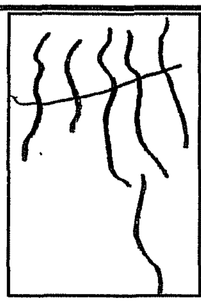
UNCONFINED COMPRESSIVE STRENGTH OF INTACT ROCK CORE, ASTM - D2938
UNIT WEIGHT AND NATURAL MOISTURE

CLIENT : CH2M Hill
PROJECT: Geotechnical Exploration, Advanced Treatment Building, MPTP
LOCATION: Ft. Thomas, Kentucky

PROJECT NUMBER: 080977E
BORING NO.: 103B SAMPLE NUMBER: RC-3A DEPTH (ft.): 26.6-27.0
SAMPLE DESCRIPTION: Gray soft thinly laminated SHALE
BEDROCK FORMATION: Fairview Formation DATE: 12/26/2008
SAMPLE OBTAINED BY: Rock Core CONDITION: Undisturbed LOAD DIRECTION 90° TO LITHOLOGY

NATURAL UNIT WEIGHT	
AVERAGE DIAMETER (in.)	1.83
HEIGHT (in.)	4.21
HEIGHT TO DIAMETER RATIO	2.31
AVERAGE AREA (sq. ft.)	0.0182
VOLUME (cu. ft.)	0.0064
WET WEIGHT (lbs.)	0.99
DRY WEIGHT (lbs.)	0.93
DRY DENSITY (pcf)	**146.3**

FAILURE SHAPE

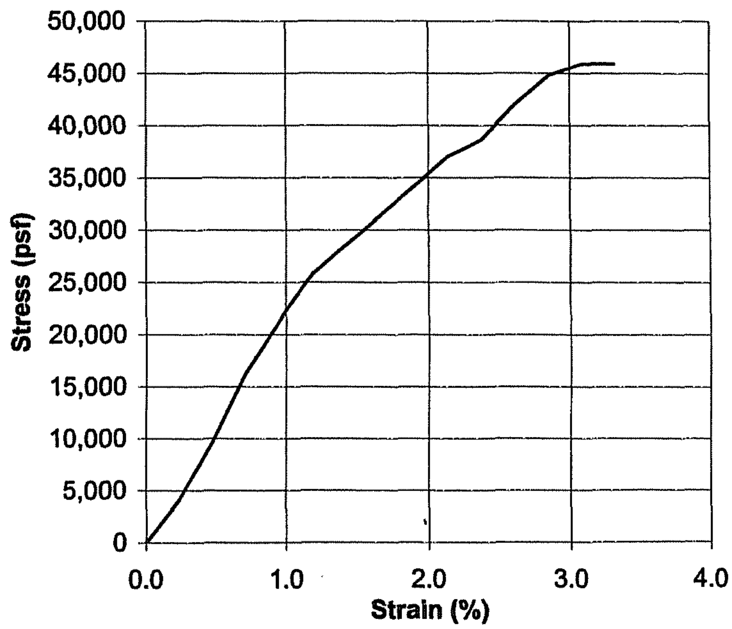


WATER CONTENT AFTER SHEAR	
CAN NUMBER	T5
WET WEIGHT + CAN (lbs.)	1.88
DRY WEIGHT + CAN (lbs.)	1.82
WEIGHT WATER (lbs.)	0.06
WEIGHT CAN (lbs.)	0.90
WEIGHT SOLID (lbs.)	0.92
MOISTURE (%)	**6.2**

PROVING RING NO.: 78-0860

TEST TEMPERATURE: 70°F

DEFORM DIAL (0.001 in.)	LOAD DIAL (0.001 in.)	LOAD (lbs.)	STRAIN (%)	STRESS (psf)
0	0	0	0.0	0
10	8	73	0.2	4,035
20	18	175	0.5	9,617
30	30	296	0.7	16,317
40	39	388	0.9	21,341
50	47	469	1.2	25,807
60	52	520	1.4	28,599
70	57	570	1.7	31,390
80	62	621	1.9	34,181
90	67	672	2.1	36,973
100	70	702	2.4	38,648
110	76	763	2.6	41,997
120	81	814	2.8	44,789
130	83	834	3.1	45,905
140	83	834	3.3	45,905



AVERAGE RATE OF STRAIN TO FAILURE (%/min.)	**1.1**
STRAIN AT FAILURE (%)	**3.1**
UNCONFINED COMPRESSIVE STRENGTH (ksf)	**45.9**
SHEAR STRENGTH (ksf)	**23.0**

REMARKS :

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LOG OF TEST BORING

CLIENT: Jordan, Jones & Goulding, Inc. BORING # 2A
PROJECT: Geotechnical Exploration, NKWD Chemical Storage and Feed Systems Improvements JOB # 050916E
LOCATION OF BORING: As shown on Boring Plan, Drawing 050916E-1 Ft. Thomas, Kentucky

ELEV.	SOIL DESCRIPTION COLOR, MOISTURE, DENSITY, PLASTICITY, SIZE, PROPORTIONS	STRATA DEPTH feet	DEPTH SCALE feet	SAMPLE Cond	Blows/6"	No.	Type	Rec. Inches
740.6	SURFACE	0.0						
740.1	CONCRETE – 5 3/4"	0.5						
739.8	Multicolored wet FILL, coarse sand and fine gravel (pea gravel base).	0.8		D		1	CA	4
739.5	Interbedded brown to olive brown moist soft weathered SHALE and gray hard LIMESTONE (bedrock).	1.1	1	I	50/6"	2A 2B	DS	5
739.3	Interbedded gray moist soft SHALE and gray hard LIMESTONE (bedrock).	1.3						
	Bottom of test boring at 1.3 feet.		2					
	Note: Bentonite backfill and concrete patch.		3					
			4					
			5					
	NOTE: 7							

Datum MSL Hammer Wt. 35 lbs. Hole Diameter 4 in. Foreman BR/GB
Surf. Elev. 740.6 ft. Hammer Drop 30 in. Rock Core Dia. ____ in. Engineer JSN
Date Started 9/27/05 Pipe Size O.D. 2 in. Boring Method HAND Date Completed 9/27/05

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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LOG OF TEST BORING

CLIENT: CH2M Hill **BORING #:** 101

PROJECT: Geotechnical Exploration, Advanced Treatment Building, Memorial Parkway Treatment Plant, **JOB #:** 080977E

LOCATION OF BORING: As shown on Boring Plan, Drawing 080977E-1 / Ft. Thomas, Kentucky

ELEV. (feet)	SOIL DESCRIPTION COLOR, MOISTURE, DENSITY, PLASTICITY, SIZE, PROPORTIONS	STRATA DEPTH (inches)	DEPTH SCALE (inches)	SAMPLE Cond	Blows/6"	No.	Type	Rec. (Inches)
740.5	SURFACE	0.0						
	CONCRETE					1	PC	9.5
739.7		9.5						
	PEA GRAVEL (multicolored, wet, dense, medium to coarse sand and fine gravel, rounded to subangular).		1	D	8/50/3"	2	DS	8
739.0		18.0						
	Split spoon refusal and bottom of test boring at 18 inches.		2					
			3					
			4					
	NOTE:							

Datum MSL Hammer Wt. 35 lbs. Hole Diameter 4 in. Foreman GB

Surf. Elev. 740.5 ft. Hammer Drop 30 in. Rock Core Dia. – in. Engineer JSN

Date Started 12/4/08 Pipe Size O.D. 2 in. Boring Method Hand Date Completed 12/4/08

SAMPLE CONDITIONS	SAMPLE TYPE	GROUNDWATER DEPTH	BORING METHOD
D - DISINTEGRATED	DS - DRIVEN SPLIT SPOON	FIRST NOTED +2 in.	HSA - HOLLOW STEM AUGERS
I - INTACT	PT - PRESSED SHELBY TUBE	AT COMPLETION +2 in.	CFA - CONTINUOUS FLIGHT AUGERS
U - UNDISTURBED	CA - CONTINUOUS FLIGHT AUGER	AFTER – hrs. – ft.	DC - DRIVING CASING
L - LOST	PC - PAVEMENT 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: CH2M Hill **BORING #:** 102

PROJECT: Geotechnical Exploration, Advanced Treatment Building, Memorial Parkway Treatment Plant, **JOB #:** 080977E

LOCATION OF BORING: As shown on Boring Plan, Drawing 080977E-1 / Ft. Thomas, Kentucky

ELEV. (feet)	SOIL DESCRIPTION COLOR, MOISTURE, DENSITY, PLASTICITY, SIZE, PROPORTIONS	STRATA DEPTH (Inches)	DEPTH SCALE (Inches)	SAMPLE Cond	Blows/6"	No.	Type	Rec. (Inches)
740.6	SURFACE	0.0						
	CONCRETE					1	PC	6
740.1		6.0						
740.0	PEA GRAVEL	7.0						
	Interbedded brown moist very soft very highly weathered SHALE (CL) and gray hard LIMESTONE (bedrock).			D/I	50/6"	2	DS	4
739.5		13.0	1					
	Split spoon refusal and bottom of test boring at 13 inches.							
			2					
			3					
			4					

NOTE: 7

Datum MSL Hammer Wt. 35 lbs. Hole Diameter 4 in. Foreman GB

Surf. Elev. 740.6 ft. Hammer Drop 30 in. Rock Core Dia. – in. Engineer JSN

Date Started 12/4/08 Pipe Size O.D. 2 in. Boring Method Hand Date Completed 12/4/08

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

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

GROUNDWATER 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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LOG OF TEST BORING

CLIENT: CH2M Hill **BORING #:** 103

PROJECT: Geotechnical Exploration, Advanced Treatment Building, Memorial Parkway Treatment Plant. **JOB #:** 080977E

LOCATION OF BORING: As shown on Boring Plan, Drawing 080977E-1 / Ft. Thomas, Kentucky

ELEV. (feet)	SOIL DESCRIPTION COLOR, MOISTURE, DENSITY, PLASTICITY, SIZE, PROPORTIONS	STRATA DEPTH (feet)	DEPTH SCALE (feet)	SAMPLE Cond	Blows/6"	No.	Type	Rec. (Inches)
741.6	SURFACE							
741.2	ASPHALT (5 inches)	0.4		I	3/3/3	1	DS	12
739.6	Mixed gray and brown moist stiff FILL, silty clay with limestone fragments.	2.0		I	3/3/4	2	DS	12
737.1	Mixed brown moist to very moist medium stiff FILL, silty clay with trace limestone fragments.	4.5	5	I	2/2/6	3	DS	18
733.6	Mixed brown moist stiff FILL, silty clay with trace shale fragments.	8.0		U		PT	4	0/6
732.1	Interbedded brown moist soft highly weathered SHALE and gray hard LIMESTONE (bedrock).	9.5	10	I	50/3"	5	DS	1
				I	14/50/6"	6	DS	6
729.1	Interbedded brown and gray moist soft weathered SHALE (CL) and gray hard LIMESTONE (bedrock).	12.5						
728.1	Interbedded gray with brown slightly moist soft SHALE and gray hard LIMESTONE (bedrock).	13.5	15	I	50/6"	7	DS	6
	Split spoon refusal and bottom of test boring at 13.5 feet.		20					
	NOTE: Bedrock cored in offset hole, Boring 103B.		25					

Datum MSL | Hammer Wt. 140 lbs. | Hole Diameter 5 in. | Foreman BR
Surf. Elev. 741.6 ft. | Hammer Drop 30 in. | Rock Core Dia. – in. | Engineer JSN
Date Started 12/18/08 | Pipe Size O.D. 2 in. | Boring Method CFA | Date Completed 12/18/08

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

GROUNDWATER DEPTH
FIRST NOTED 12.0 ft.
AT COMPLETION 12.0 ft.
AFTER 3.0 hrs. 12.0 ft.
BACKFILLED 3.0 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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LOG OF TEST BORING

CLIENT: CH2M Hill **BORING #:** 103A
PROJECT: Geotechnical Exploration, Advanced Treatment Building, Memorial Parkway Treatment Plant, **JOB #:** 080977E
LOCATION OF BORING: As shown on Boring Plan, Drawing 080977E-1 / Ft. Thomas, Kentucky

ELEV. (feet)	SOIL DESCRIPTION COLOR, MOISTURE, DENSITY, PLASTICITY, SIZE, PROPORTIONS	STRATA DEPTH (inches)	DEPTH SCALE (inches)	SAMPLE				
				Cond	Blows/6"	No.	Type	Rec. (inches)
741.4	SURFACE	0.0						
740.9	CONCRETE	6.5				1	PC	6.5
740.4	Interbedded brown moist very soft very highly weathered SHALE and gray hard LIMESTONE (bedrock).	12.5	1	I	50/6"	2	DS	6
	Split spoon refusal and bottom of test boring at 12.5 inches.		2, 3, 4					

Datum MSL	Hammer Wt. 140 lbs.	Hole Diameter 4 in.	Foreman GB
Surf. Elev. 741.4 ft.	Hammer Drop 30 in.	Rock Core Dia. – in.	Engineer JSN
Date Started 12/4/08	Pipe Size O.D. 2 in.	Boring Method Hand	Date Completed 12/4/08

SAMPLE CONDITIONS	SAMPLE TYPE	GROUNDWATER 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	PC - PAVEMENT 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: CH2M Hill **BORING #:** 103B

PROJECT: Geotechnical Exploration, Advanced Treatment Building, Memorial Parkway Treatment Plant, **JOB #:** 080977E

LOCATION OF BORING: As shown on Boring Plan, Drawing 080977E-1 / Ft. Thomas, Kentucky

ELEV. (feet)	SOIL DESCRIPTION COLOR, MOISTURE, DENSITY, PLASTICITY, SIZE, PROPORTIONS	STRATA DEPTH (feet)	SAMPLE Cond	Blows/6"	No.	Type	Rec. (Inches)
741.6	SURFACE	0.0					
729.6	Augered to 12.0 feet without sampling. Pea gravel noted at 7.5 - 9.5 feet.	12.0					
724.6	Interbedded SHALE and LIMESTONE. Shale is gray, moist, soft, thin to medium bedded and thinly laminated. Limestone is white to light gray, slightly moist, hard, thin to medium bedded, and biosparitic, with traces of shale-like mudstone in matrix. Limestone occurs in 0.5- to 7-inch beds and comprises 43 percent of the cored interval, assuming the unrecovered core to be shale (bedrock).	17.0		RQD = 33%	1	RC	49.5/60
719.6	Interbedded SHALE and LIMESTONE. Shale is gray, moist, soft, medium bedded with trace thin bedded, thinly laminated, and low plastic (CL). Limestone is white to light gray, slightly moist, hard, medium bedded with trace thin bedded, and biosparitic, with traces of shale-like mudstone in matrix. Limestone occurs in 0.5- to 6-inch beds and comprises 33 percent of the cored interval (bedrock).	22.0		RQD = 50%	2	RC	60/60
714.6	SHALE with a little interbedded LIMESTONE. Shale is gray, moist, soft, thick bedded with trace thin bedded, and thinly laminated. Limestone is white to light gray, slightly moist, hard, thin bedded, and biosparitic, with traces of shale-like mudstone in matrix. Limestone occurs in 0.5- to 2-3/4-inch beds and comprises 14 percent of the cored interval, assuming the unrecovered core to be shale.	27.0		RQD = 71%	3	RC	59/60
	Bottom of test boring at 27.0 feet. *Boring drilled with water.						

Datum: MSL	Hammer Wt.: 140 lbs.	Hole Diameter: 5 in.	Foreman: BR
Surf. Elev.: 741.6 ft.	Hammer Drop: 30 in.	Rock Core Dia.: – in.	Engineer: JSN
Date Started: 12/18/08	Pipe Size: O.D. 2 in.	Boring Method: CFA	Date Completed: 12/18/08

SAMPLE CONDITIONS	SAMPLE TYPE	GROUNDWATER DEPTH	BORING METHOD
D - DISINTEGRATED	DS - DRIVEN SPLIT SPOON	FIRST NOTED * ft.	HSA - HOLLOW STEM AUGERS
I - INTACT	PT - PRESSED SHELBY TUBE	AT COMPLETION * 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: CH2M Hill **BORING #:** 104

PROJECT: Geotechnical Exploration, Advanced Treatment Building, Memorial Parkway Treatment Plant, **JOB #:** 080977E

LOCATION OF BORING: As shown on Boring Plan, Drawing 080977E-1 / Ft. Thomas, Kentucky

ELEV. (feet)	SOIL DESCRIPTION COLOR, MOISTURE, DENSITY, PLASTICITY, SIZE, PROPORTIONS	STRATA DEPTH (inches)	DEPTH SCALE (inches)	SAMPLE Cond	Blows/6"	No.	Type	Rec. (inches)
740.6	SURFACE	0.0						
	CONCRETE					1	PC	10
739.8		10.2						
739.7	PEA GRAVEL	11.0	1	I	50/6"	2	DS	6
739.2	Interbedded gray moist to very moist very soft SHALE and gray hard LIMESTONE (bedrock).	17.0						
	Split spoon refusal and bottom of test boring at 17 inches.		2					
			3					
			4					

Datum: MSL	Hammer Wt.: 140 lbs.	Hole Diameter: 4 in.	Foreman: GB
Surf. Elev.: 740.6 ft.	Hammer Drop: 30 in.	Rock Core Dia.: – in.	Engineer: JSN
Date Started: 12/4/08	Pipe Size: O.D. 2 in.	Boring Method: Hand	Date Completed: 12/4/08

SAMPLE CONDITIONS	SAMPLE TYPE	GROUNDWATER DEPTH	BORING METHOD
D - DISINTEGRATED	DS - DRIVEN SPLIT SPOON	FIRST NOTED +2 in.	HSA - HOLLOW STEM AUGERS
I - INTACT	PT - PRESSED SHELBY TUBE	AT COMPLETION +2 in.	CFA - CONTINUOUS FLIGHT AUGERS
U - UNDISTURBED	CA - CONTINUOUS FLIGHT AUGER	AFTER – hrs. – ft.	DC - DRIVING CASING
L - LOST	PC - PAVEMENT 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: CH2M Hill **BORING #:** 105

PROJECT: Geotechnical Exploration, Advanced Treatment Building, Memorial Parkway Treatment Plant, **JOB #:** 080977E

LOCATION OF BORING: As shown on Boring Plan, Drawing 080977E-1 / Ft. Thomas, Kentucky

ELEV. (feet)	SOIL DESCRIPTION COLOR, MOISTURE, DENSITY, PLASTICITY, SIZE, PROPORTIONS	STRATA DEPTH (inches)	DEPTH SCALE (inches)	SAMPLE Cond	Blows/6"	No.	Type	Rec. (inches)
741.1	SURFACE	0.0						
740.6	CONCRETE	5.5		X		1	PC	5.5
740.5	PEA GRAVEL (multi-colored, medium to coarse sand and fine gravel, rounded, trace subangular)	7.0		I	50/2"	2	DS	2
740.4	Interbedded gray very moist soft SHALE and gray hard LIMESTONE (bedrock).	9.0	1					
	Split spoon refusal and bottom of test boring at 9 inches.		2					
			3					
	NOTE:		4					

Datum	MSL		Hammer Wt.	35	lbs.	Hole Diameter	4	in.	Foreman	GB	
Surf. Elev.	741.1	ft.	Hammer Drop	30	in.	Rock Core Dia.	–	in.	Engineer	JSN	
Date Started	12/3/08		Pipe Size	O.D. 2	in.	Boring Method	Hand		Date Completed	12/3/08	

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

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

GROUNDWATER 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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Dayton, Ohio

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.