GEOTECHNICAL EXPLORATION
GDRTA BUILDING 705
DAYTON, OHIO

Prepared for:

CHAMPLIN ARCHITECTURE
CINCINNATI, OHIO

Prepared by:

GEOTECHNOLOGY, LLC
CINCINNATI, OHIO

Date:

JULY 29, 2021

Geotechnology Project No.:

J038716.01

SAFETY
QUALITY
INTEGRITY
PARTNERSHIP
OPPORTUNITY
RESPONSIVENESS

July 29, 2021

Mr. Jay Derenthal
Champlin Architecture
720 E. Pete Rose Way, Suite 140
Cincinnati, Ohio 45202

Re: Geotechnical Exploration
GDRTA Building 705
Dayton, Ohio
Geotechnology Project No. J038716.01

Dear Mr. Derenthal:

Presented in this report are the results of our geotechnical exploration completed for the GDRTA Building 705 project in Dayton, Ohio. Our services were performed in general accordance with our Proposal P038716.01, which was dated April 13, 2021, and signed for authorization on April 22, 2021.

We appreciate the opportunity to provide the geotechnical services for this project. If you have any questions regarding this report, or if we may be of any additional service to you, please do not hesitate to contact us.

Respectfully submitted,
GEOTECHNOLOGY, LLC

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DAYTON, OHIO
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1.0 INTRODUCTION

Geotechnology, LLC. (Geotechnology) prepared this geotechnical exploration report on behalf of Champlin Architecture (Champlin) for the GDRTA Building 705 project located at the address of 705 Longworth Street in Dayton, Ohio.[1] Our services documented in this report were provided in general accordance with the terms and scope of services described in our Proposal P038716.01, which was dated April 13, 2021, and signed for authorization on April 22, 2021.

The purposes of the geotechnical exploration were: to evaluate the general subsurface profile at the site and the engineering properties of the soils and to develop recommendations for the geotechnical aspects of the design and construction of the project, as defined in our proposal. Our scope of services included a site reconnaissance, geotechnical borings, laboratory testing, engineering analyses, and preparation of this report.

2.0 PROJECT INFORMATION

The following project information was derived from:

- Undated Site Plan, titled “GDRTA Existing Conditions”, which was prepared by LJB, Inc. (LJB), and received electronically on May 4, 2021;
- Memorandum titled “705 Longworth St. Geotechnical Investigation Information for RFP”, which was prepared by Schaefer Structural Engineers (Schaefer) and dated March 31, 2021, and
- Correspondence with Champlin.

We understand that this project will include the construction of canopy structures and below ground tanks for a fueling station north of existing Building 705; a parking canopy for paratransit buses in the existing parking lot west of Building 705; demolition of the northern part of Building 705; a structural repair/replacement of the north bearing wall of the part of Building 705 to remain; and construction of surface parking in the area of the demolished building. Based on the above-mentioned memorandum, it is understood that the new canopies will be cantilever steel structures

[1] GDRTA refers to the Greater Dayton Regional Transit Authority.

with central columns supported on concrete drilled shafts. A layout of the canopy structures along with their foundation loads were not available at the time of this report.

A grading plan has not yet been prepared at the time of this report, but the grading is expected to be relatively minimal based on our understanding of the project and the existing grades.

3.0 SITE CONDITIONS

The site location and regional topography of the area are shown on the Site Location Plan, which is shown as Sheet No. 1 in Appendix B.

The existing property comprises an approximate 3.8-acre tract of land situated within the floodplain of the Great Miami River in an urban business district of Dayton bounded by Veterans Parkway to the west, Longworth Street to the east, an existing GDRTA facility to the north (601 Longworth), and separate isolated GDRTA buildings to the south. The existing structure at 705 Longworth is an irregularly-shaped, 1-story, brick and CMU-block building that encompasses roughly 34,000 square feet in plan area positioned within the southeast quadrant of the subject tract of land. The east, south, and north exterior wall lines were composed of brick, while the west wall line exhibited larger CMU block units. Existing asphalt-paved parking exists to the west and north of the building. A wide concrete-paved service drive extends from Longworth Street to a loading area that exists along the northern wing of the existing building.

Much of the existing property is dominated by relatively flat paved impervious surfaces with limited grass-covered landscaping surrounding the northern parking lot. Given its close proximity to the Great Miami River some 300 to 400 feet away, the site drains to the west, which then empties directly into the Great Miami River. The asphalt parking lots have exhibited significant distress with evidence of numerous alligator cracking, occasional potholes, and overall weathered asphalt that had reduced to sand and gravel-sized fragments in localized areas. The concrete-paved loading zone appeared to be in fair condition with less observed distress. Minor faulting of the concrete slab was noted in a few areas, but the distress was not as great or prevalent as the asphalt pavement. Relative to the building, our observations indicated several areas of stair-stepped cracks in the exterior block walls, which is consistent with the exterior observations noted in the Schaefer memorandum. Our site reconnaissance was generally focused on the existing property west of Longworth Street, though our site observations and review of aerial imagery indicates a relatively large single-story, enclosed brick building on the east side of Longworth Street, which appears to be for the maintenance of GDRTA equipment.

Site conditions were noted and photographed during our reconnaissance. Photographs shown below as Figure 1 and Figure 2 document the condition of the existing paved surfaces while Figure 3 provides a representation of the observed distress along the east exterior wall of the building.

Figure 1. Photograph looking south at the loading dock area of the existing building.

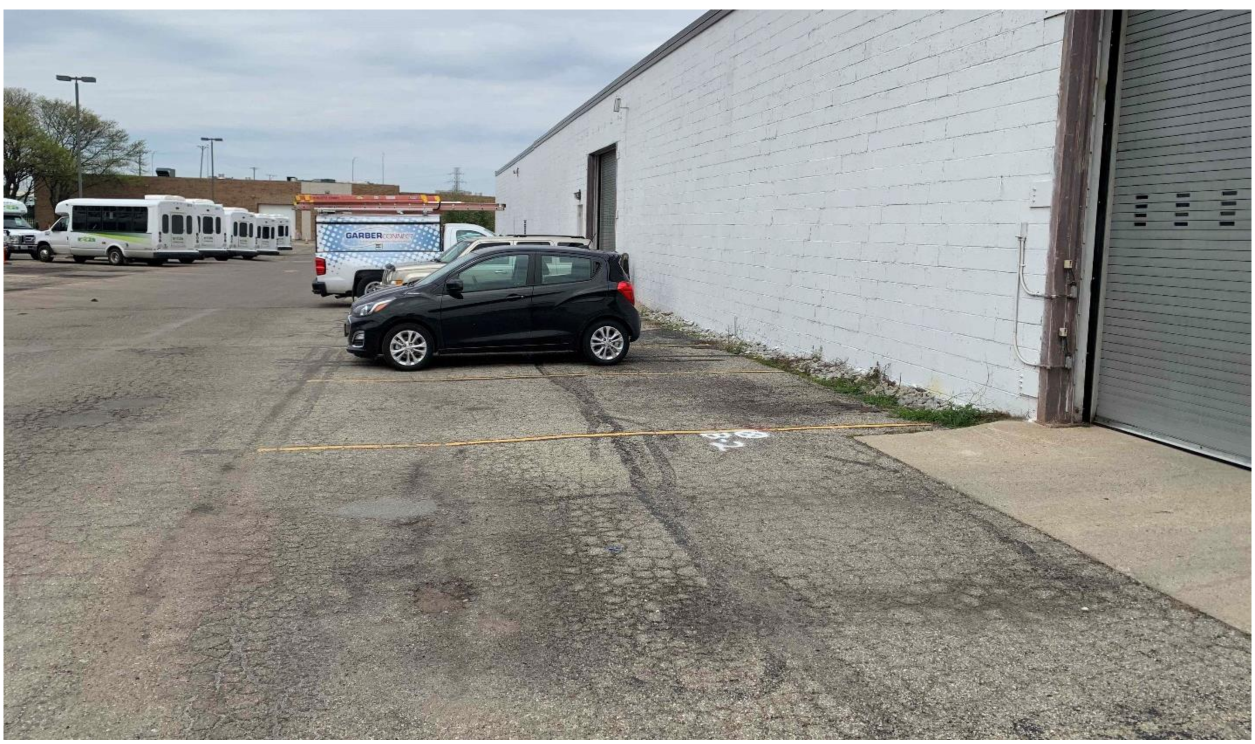

Figure 2. Photograph looking north along the west exterior wall of existing building.

Figure 3. Photograph looking east toward the existing building.

4.0 PROJECT RESEARCH

4.1 Historic Information

The following list of readily available historic information was reviewed for this project:

- USGS Topographic Maps of the Dayton South Quadrangle (1953, 2019), Montgomery County, Ohio; and
- USGS Topographic Map of the Waynesville Quadrangle (1913), Dayton, Ohio

4.2 Previous Geotechnical Explorations

A previous exploration was completed and a geotechnical exploration report prepared by Terracon Consultants, Inc. in 2018 for the GDRTA Building Renovations, located on the east side of Longworth Street. Given its relatively close proximity to the project site, this report was made available to Geotechnology for our use on this GDRTA Building 705 Renovations project.

5.0 SUBSURFACE EXPLORATION

The subsurface exploration initially consisted of six borings (numbered B-1 through B-6), though one boring planned to be drilled along the east side of the existing building (Boring B-6) was omitted prior to our mobilization due to congestion of existing overhead and underground utilities. The boring locations were selected by us and were staked in the field by an LJB survey crew relative to their survey control and benchmark elevation. The locations of the borings are shown on our Boring Plan, which is shown as Sheet No. 2 in Appendix B.

The borings were drilled between April 27 and 30, 2021 with a track-mounted drill rig advancing continuous flight hollow-stem augers, as indicated on the boring logs presented in Appendix C. Sampling of the overburden soils was accomplished ahead of the augers at the depths indicated on the boring logs with a 2-inch-outside-diameter (O.D.) split-barrel sampler in general accordance with the procedures outlined by ASTM D1586. Standard Penetration Tests (SPTs) were performed with the split-barrel sampler to obtain the standard penetration resistance or N-value[2] of the sampled material. Observations for groundwater were made in the borings during and upon completion of drilling.

As each boring was advanced, the Drilling Foreman prepared a field log of the subsurface profile noting the soil and bedrock types and stratifications, groundwater, SPT results, and other pertinent data.

Representative portions of the split-barrel samples were placed in glass jars with lids to preserve the in-situ moisture contents of the samples. The glass jars were marked and labeled in the field for identification when returned to our laboratory.

6.0 LABORATORY REVIEW AND TESTING

Upon completion of the fieldwork, the samples recovered from the borings were transported to our Soil Mechanics Laboratory, where they were visually reviewed and classified by the Project Geotechnical Engineer.

Laboratory testing was performed on selected soil samples to estimate engineering and index properties. Laboratory testing of the selected soil samples included various combinations of the following tests: moisture content, Atterberg limits, and gradation (particle-size) analyses. The results of these tests are summarized in the Tabulation of Laboratory Tests in Appendix D, along with the particle-size analysis forms.

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[2] The standard penetration resistance, or N-value, is defined as the number of blows required to drive the split-barrel sampler 12 inches with a 140-pound hammer falling 30 inches. Since the split-barrel sampler is driven 18 inches or until refusal, the blows for the first 6 inches are for seating the sampler, and the number of blows for the final 12 inches is the N-value, which is reported as blows per foot (or bpf). Additionally, “refusal” of the split-barrel sampler occurs when the sampler is driven less than 6 inches with 50 blows of the hammer.

The boring logs, which are included in Appendix C, were prepared by the Project Geotechnical Engineer on the basis of the field logs, the visual classification of the soil samples in the laboratory, and the laboratory test results. A Soil Classification Sheet is also included in Appendix C, which describes the terms and symbols used on the boring logs. The dashed lines on the boring logs indicate an approximate change in strata as estimated between samples, whereas a solid line indicates that the change in strata occurred within a sample where a more precise measurement could be made. Furthermore, the transition between strata can be abrupt or gradual.

7.0 SUBSURFACE CONDITIONS

7.1 Stratification

Generally, the ground surface was underlain by existing undocumented cohesive and cohesionless fill over native cohesive and cohesionless alluvial soils followed by glacial outwash interbedded with lacustrine soils to the explored depths of the borings. More specific descriptions of the subsurface strata are provided below, and the boring logs containing detailed material descriptions are located in Appendix C.

7.1.1 Pavement

Pavement was encountered in each of the borings drilled for this project. With the exception of Boring B-5, the existing pavement was comprised of 3 to 10 inches of asphaltic concrete over approximately 2 to 3 inches of aggregate base, as summarized below in Table 1. The total pavement section thickness encountered in the borings ranged from 6 to 12 inches with an average thickness just over 7 inches.

Table 1. Summary of pavement thicknesses

Boring	Thickness (inches)			
	AC Concrete (in.)[a]	PC Concrete (in.)[b]	Aggregate Base (in.)	Total (in.)
B-1	10		2	12
B-2	3		3	6
B-3	3		3	6
B-4	6			6
B-5		6		6

[a] AC = Asphaltic Concrete.
[b] PC = Portland Cement.

7.1.2 Fill

Existing undocumented fill was encountered beneath the ground surface in each of the borings and extended to depths ranging from 7 feet (Borings B-1 and B-2) to 12 feet (Borings B-3 through B-5) below existing grade. The existing fill consisted of both cohesive and granular soils. The cohesive fill encountered at the site was described as brown to dark brown, moist stiff to very stiff lean clay with various proportions of gravel, cinders, and other miscellaneous debris. The granular fill was described as brown and dark brown with occasional gray, moist to wet, very loose to medium dense, fine-grained silts and sands and coarse-grained sands and gravels with occasional cinders and brick fragments. It should be noted that the fill in general was highly variable in composition, density, color, and moisture. Two natural moisture content tests obtained from samples of the fine-grained granular fill were 14.9 and 19.1 percent. Uncorrected SPT N-values of the cohesive fill ranged from 3 blows per foot (bpf) to 15 bpf and averaged approximately 10 bpf, while the SPT N-values in the cohesionless fill ranged from 2 bpf to 19 bpf and averaged 8 bpf. Two hand penetrometer (HP) readings were obtained on the cohesive fill with values ranging from 2.0 to 2.5 tons per square foot (tsf) indicating a stiff to very stiff consistency.

7.1.3 Alluvium

Native alluvial soils were encountered beneath the existing fill in each of the borings, which then extended to depth of approximately 19 feet. Alluvial soils (or alluvium) are sedimentary soils that are deposited by fluvial or flowing water systems (e.g., streams, rivers, etc.). Similar to the existing fill, the alluvial soils encountered at the site included cohesive soils interbedded with granular soils. The cohesive alluvium was described as brown to dark brown and occasional grayish-brown, moist medium-stiff to very stiff silty to lean clay and one fat clay seam. The thickness of the cohesive alluvium ranged from 2.5 feet in Borings B-1, B-3, and B-5 to 10 feet in Boring B-2. The granular alluvium was described as brown to gray, moist to very moist, very loose to medium dense, fine-grained clayey sands and coarse-grained sands. Uncorrected SPT N-values of the cohesive alluvium ranged from 5 bpf to 9 bpf with an average of about 7 bpf, while the SPT N-values in the granular alluvium ranged from 2 bpf to 14 bpf and averaged 6 bpf. Several HP readings were obtained on the cohesive alluvium with values ranging from 1.0 tsf to 2.5 tsf and averaging about 1.75 tsf, indicating a stiff consistency on average. Natural moisture content tests were performed on representative samples of the cohesive alluvium with results ranging from 25.0 to 38.7 percent and an average of 33.3 percent. Where subjected to classification testing, the cohesive alluvium classified as lean clay (CL) and fat clay (CH) according to the USCS. The results of these classification tests are tabulated below in Table 2 and in Appendix D.

Table 2. Summary of Atterberg limits test results of the alluvium.

	Liquid Limit (%)	Plastic Limit (%)	Plasticity Index (%)
Minimum	44	23	21
Maximum	55	29	26
Average	50	26	24

7.1.4 Glacial Outwash

Native glacial soils, and occasional interbedded lacustrine soils (discussed below), were encountered beneath the native alluvium and extended to the explored depths of the borings. Glacial soils are soils that have been deposited, transported, or reworked in place by the advancement or retreat of a glacier across the area. In general, the native glacial profile consisted of cohesionless glacial outwash. Glacial outwash is generally associated with the retreat of glaciers. As glacial ice melts, fine sediments, sand, and gravel trapped in the ice are released and settle out as the water flows away from the glacier. The outwash deposits also are derived from the erosion of the meltwater streams.

The glacial outwash consisted of brown to gray, medium to very dense, fine to coarse sands and gravels with various proportions of silt and clay. With the exception of three SPT samples that encountered sampler refusal conditions, uncorrected SPT N-values recorded in the glacial outwash ranged from 23 bpf to 76 bpf with an average of about 40 bpf, indicative of a dense condition on average. One natural moisture test was performed on a representative sample of the glacial outwash with a result of 23.4 percent. The glacial outwash was classified in the laboratory as either well-graded gravel with silt (GW-GM) or poorly-graded sand with silt (SP-SM). The classification test results of the glacial soils are tabulated below in Table 3 and in Appendix D.

Table 3. Glacial soil classification test results

Boring	Sample	Gradation Analysis (%)				USCS Classification[a]
		Gravel	Sand	Silt	Clay	
B-2	SS-9	46.8	44.2	9.0		GW-GM
	SS-13	0.4	89.6	10.0		SP-SM

[a] On samples where Atterberg limits tests were not performed, the proportion of fines were assumed to primarily consist of silt in determining the USCS classification.

7.1.5 Lacustrine Soils

Lacustrine soils were encountered as two separate layers interbedded with the glacial outwash in Boring B-2, one at a depth of 53 feet and another that extended from a depth of 73 feet to the explored depth of the boring. Lacustrine (or lakebed) soils are sedimentary soils that are deposited in quiescent lakes, which produce fine horizontal laminations that are characteristic of these soils. Where fully penetrated, the thickness of this layer was approximately 5 feet. The lacustrine soils in this boring were described as grayish-brown, very moist, stiff to very stiff lean to silty clay with occasional laminations of sand and silt based on visual observations and/or laboratory test results as tabulated below in Table 4. Uncorrected SPT N-values of the lacustrine soils ranged from 53 bpf to 89 bpf with an average of about 71 bpf. Three HP readings were obtained in this layer with values ranging from 2.0 to 4.5 tsf indicating a stiff to hard consistency

Table 4. Lacustrine classification test results

Boring	Samples	Atterberg Limits (%)			Gradation Analysis (%)				USCS Class
		LL	PL	PI	Gravel	Sand	Silt	Clay	
B-2	20 – 21	23	19	4	11.3	16.4	72.3		CL-ML

7.2 Groundwater Conditions

As mentioned in Section 5.0, groundwater observations were made in the borings during and upon completion of drilling. These measurements are documented on the boring logs in Appendix C and are summarized below in Table 5.

Table 5. Summary of groundwater observations.

Boring	Elevation (feet)			
	Boring		Water Level	
	Top	Bottom	During Drilling[a]	Upon Completion[a]
B-1	741.3	704.8	711.3	704.8
B-2	740.2	658.7	710.2	NE[b]
B-3	739.6	703.1	714.6	
B-4	739.2	702.7	709.2	702.7
B-5	736.6	700.1	706.6	700.6

[a] Abbreviation: NE = not encountered.
[b] Note that drilling mud was added down the hollow stem augers to prevent heaving of sands after encountering groundwater at 30.0 feet in Boring B-2.

Based on the groundwater observations and our local experience, groundwater seepage is anticipated along the fill/native soil interface and in the saturated zones of fill or native soils that are either within perched groundwater zones or below the groundwater table. Locally concentrated flow may occur due to saturated layers of fill or native soils (particularly the native alluvial and glacial silts, sands, or gravels). Groundwater levels are anticipated to fluctuate with rises and falls in the Great Miami River due to the close proximity of this river and the cohesionless soils encountered in the borings; however, these rises and falls may have a delayed response to the river levels. Additionally, groundwater levels and seepage amounts are expected to vary with time, location, season of the year, and amounts of precipitation.

8.0 CONCLUSIONS AND RECOMMENDATIONS

Based on our engineering reconnaissance of the site, the borings, the visual examination of the recovered samples, the laboratory test results, our understanding of the proposed project, our engineering analyses, and our experience as Geotechnical Engineers in the Greater Dayton Area, the following conclusions and recommendations are presented.

8.1 Excavation Support

Excavation support should be the responsibility of the Contractor. Excavation support should be designed and implemented such that excavations are adequately ventilated and braced, shored, and/or sloped in order to protect and ensure the safety of workers within and near the excavations and to protect adjacent ground, slopes, structures, and infrastructure. Federal, state, and local safety regulations should be satisfied. The analyses, discussions, conclusions, and recommendations throughout this report are not to be interpreted as pre-engineering compliance with any safety regulation.

8.2 Site Preparation and Earthwork

As stated in Section 2.0, earthwork for this project will be minimal and likely limited to the removal and/or rehabilitation of the existing pavements.

Where applicable, the initial preparation of the site for grading should include the removal of vegetation, heavy root systems, and topsoil from the proposed cut, fill, pavement, and structure areas. The topsoil may be stockpiled for future use on the completed cut and fill slopes or in landscaped areas, if permitted by specification, whereas the vegetation, including the heavy root systems, should be disposed of off-site in accordance with applicable regulations.

Existing structures and pavements within the grading and proposed structure limits should be demolished, and the foundations removed. Concrete, asphalt, rubble, and debris associated with those structures and pavements should be disposed of off-site, unless there are provisions in the specifications for on-site reclamation of these materials. We should be retained to review these provisions to evaluate their impact on the recommendations of this report. Pavements outside of the footprints of the proposed structures may temporarily be left in place prior to removal and/or replacement to provide a stable base for construction equipment. The existing undocumented fill is highly variable in composition, density, and moisture. As such, it is generally considered unsuitable for the support of pavements, new fill, and structures in its current condition. However, since the undocumented fill extends to depths ranging from 7 to 12 feet deep, it may not be cost effective to remove it in its entirety when the foundation support for the proposed structures can be addressed separately. In this case, the undocumented fill may remain in place provided that the existing subgrade is deemed acceptable upon completion of a successful proofroll as defined below and provided that the foundations for the proposed load-bearing structures are installed in accordance with the recommendations presented in Section 8.4.

After the above operations and making required excavations in cut areas, the exposed subgrade should be thoroughly proofrolled using a loaded tandem-axle dump truck weighing at least 40,000 pounds under the review of the Project Geotechnical Engineer, or a representative thereof. Soft or yielding soils observed during the proofrolling should be undercut to stiff, non-yielding, cohesive soils or medium dense to dense, well-graded, cohesionless soils; the depth of undercut below proposed subgrade may be limited to 4 feet.

Where undercuts are performed, the excavations should be backfilled with new compacted fill satisfying the material and compaction requirements presented in this section. The undercut soils may be reused provided that they conform to the recommendations contained in this report regarding acceptable fill materials. We recommend that the Contract Documents include a bid item for the recommended undercutting, as deemed necessary, and their replacement with new compacted and tested fill on a “per cubic yard of in-place compacted fill” basis.

If soft or yielding soils are encountered at the maximum undercut depth specified above and the compaction requirements of the undercut backfill cannot be achieved at the bottom of the undercut, the subgrade may be stabilized at those depths using an approved biaxial or triaxial geogrid (e.g., Tensar BX-1200 or TriAx TX160) and an 8-inch lift of compacted crushed stone. The remainder of the undercut should be backfilled with dense-graded aggregate or clayey soils satisfying the material and compaction requirements presented in this section. If clayey soils are used, an approved separation geotextile fabric should be provided at the interface between the crushed stone and the clayey soils.

Fill materials should consist of approved on-site, non-organic, clayey soils, bedrock, or approved borrow material that are relatively free of topsoil, vegetation, trash, construction or demolition debris, frozen materials, particles over 6 inches in maximum dimension, or other deleterious materials.

The fill should be placed in shallow level lifts (or layers), 6 to 8 inches in loose thickness. Each lift should be moisture-conditioned to within the acceptable moisture content range provided in Table 6, and compacted with a sheepsfoot roller or self-propelled compactor to at least the minimum percent compaction indicated in the same table. Moisture-conditioning may include: aeration and drying of wetter soils; wetting of drier soils; and/or thoroughly mixing wetter and drier soils into a uniform mixture.

Table 6. Percent compaction and moisture-conditioning requirements for fill and backfill.

Area	Minimum Percent Compaction[a,b]	Acceptable Moisture Content Range[c]
Structural[d]	98% of SPMDD	-2% to +3% of OMC
Non-structural	95% of SPMDD	±3% of OMC
Floor slab subgrade	98% of SPMDD	0% to +3% of OMC
Pavement subgrade: ≤ 8 inches below subgrade	98% of SPMDD	±2% of OMC

a SPMDD = standard Proctor maximum dry density determined from ASTM D698.
b For granular soils that do not exhibit a well-defined moisture-density relationship, refer to Table 11 for minimum relative density requirements.
c OMC = optimum moisture content determined from ASTM D698.
d Structural fill and backfill for foundations are defined as fill and backfill located within the zones of influence of structures. The zone of influence of a structure is defined as the area below the footprint of the structure and 2H:1V outward and downward projections from the bearing elevation of the structure.

Groundwater is not expected to have a significant adverse effect on the proposed construction; however, the Contractor must be prepared to remove seepage that accumulates in excavations, on fill surfaces, or at subgrade levels.

Maintaining the moisture content of bearing and subgrade soils within the acceptable range provided in Table 6 is very important during and after construction for the proposed structures. The clayey bearing and subgrade soils should not be allowed to become excessively wet or dried during or after construction, and measures should be taken to prevent water from ponding on these soils and to prevent these soils from desiccating during dry weather.

Positive drainage should be established around the proposed structures to promote the rapid drainage of surface water away from these structures and to prevent the ponding of water adjacent to these structures. Finish grading in grass and landscaped areas should be sloped down and away from the structures at 10 percent for at least 10 feet, and then at a gradient of at least 2 percent beyond the initial 10 feet from the structures. Proposed pavements should drain away from the structures at a minimum of 2 percent. The final grades should direct the surface water to storm water collection systems.

We recommend that the earthwork operations be carried out during the drier season of the year and that a sufficient gradient be maintained at the ground surface to prevent ponding of surface water. In our experience, the weather conditions are historically more favorable for earthwork during the months of May through October in the Greater Dayton Area. Regardless of the time of year, asphalt, concrete, or fill should not be placed over frozen or saturated soils, and frozen or saturated soils should not be used as compacted fill or backfill.

Best management practices (BMPs) should be implemented to reduce the effects of erosion and the siltation of adjacent properties. Upon completion of earthwork, disturbed areas should be stabilized.

8.3 Seismic Site Classification

Based on the borings and our interpretation of the International Building Code, it is our opinion that Site Class D is applicable for this project site.

8.4 Foundation Design and Construction

Given the relatively heavy foundation loads anticipated in combination with the presence of a relatively thick layer of undocumented fill materials over loose native alluvial sands, both layers of which are considered moderately to highly compressible, we recommend that the proposed new canopy structures and the reconfigured north wall of the building be supported on a series of deep foundation elements that penetrate through the existing fill and native alluvial soils and into the underlying medium dense to very dense cohesionless glacial outwash. Given its proximity to the existing building of unknown foundation support, we recommend that the proposed load-bearing structures be supported by non-displacement, drilled augercast piles or helical piles. Such deep foundations pose less risk of negatively affecting the existing structures in comparison to a driven system, which is a louder installation technique and capable of densifying the existing loose fill and alluvial soils thus increasing the risk of settlement of shallow foundations. Given the presence of groundwater within the underlying alluvial and outwash sands, augercast piles also provide an advantage over drilled shaft foundations, as they are routinely installed under pressure in a variety of soil and groundwater conditions with minimal deviations in installation whereas drilled shaft foundations on this project would require either casing or slurry to stabilize the shaft walls, which would add a significant expense. Helical piles would be advantageous in that it is a drilled system where the pile itself remains in the ground to serve as foundation support and hence not as affected by the influence of groundwater within a borehole. Discussions of the recommended deep foundation options are presented below.

8.4.1 Augercast Piles

Augered cast-in-place piles (also known as augercast piles or ACIP piles) have been evaluated for support of the proposed new canopy structures and new exterior load-bearing wall of the reconfigured building. Table 7 provides allowable axial capacities in compression and tension for individual 16-inch and 18-inch-diameter augercast piles. The allowable capacities in this table are based on providing a factor of safety (FS) of 2.5 for compression and 3.0 for tension, using the groundwater elevations assumed near the top of glacial outwash.

Table 7. Allowable axial augercast pile capacities.

Pile Length (ft.)[a]	Allowable Axial Pile Capacity (k)[b,c]			
	16-inch-diameter Augercast Pile		18-inch-diameter Augercast Pile	
	Compression (FS = 2.5)	Tension (FS = 3)	Compression (FS = 2.5)	Tension (FS = 3)
30	90	45	110	50
35	115	65	135	75
40	140	85	165	95
45	165	105	195	120
50	195	130	220	145

a Pile length measured from bottom of pile cap.
b Groundwater elevation assumed at El. 720.
c Capacities are to be limited to the smaller of the capacities provided in this table and the maximum allowable by code based on the compressive strength of the grout.

The Project Structural Engineer should verify the structural capacity of the piles based on the unfactored service loads and the requirements of the applicable building code, including, but not limited to, the minimum seismic reinforcement requirements for augercast piles, which vary with the seismic design category. If different augercast pile sizes are to be used, or if higher load capacities are required, then Geotechnology should be retained to reevaluate the augercast pile analyses.

8.4.1.1 Group Effects

Because conventional ACIP piles are a drilled foundation system subject to disturbance of the surrounding soil mass during installation, the design of the piles should consider group effects and its impact to the axial capacity of the group. In modeling the response of the entire group of piles, we recommend that the center-to-center spacing should not be less than three pile diameters. Considering the group effects from neighboring piles, the ultimate capacity will be controlled by the presence of cohesionless soils at or near the anticipated bottom of pile cap extending to the anticipated pile tip elevations. Recognizing this condition and based on guidance presented in the *AASHTO LRFD Bridge Design Specifications* (AASHTO 2020), the ultimate capacity of pile groups can be designed using an efficiency factor (η) of 0.8 whether the pile cap will be in firm contact with the ground or not. Therefore, we recommend that the axial capacity of each pile in a group be reduced by applying an efficiency factor (η) of 0.8 when computing the total axial capacity of all piles in a group. This efficiency factor assumes a pile spacing of three pile diameters, a pile arrangement comprised of more than one row, and a predominately cohesionless soil profile along the length of the piles. Consideration could be given to using an efficiency factor of 1.0 where a single pile or single row of piles will be used, or where the pile spacing is greater than 4 times the pile diameter.

Uplift may be checked on the basis of guidance presented in AASHTO 2020. Considering cohesionless soils were predominately encountered below the ground surface, the allowable uplift capacity of a group of piles may be computed based on the lesser of: (1) the sum of individual pile capacities in a group, considering the efficiency factor and factor of safety noted above, or (2) the uplift capacity of a block of soil defined by a 1H:4V projection up from the base of the pile group to the elevation of the bottom of pile cap all around. The uplift capacity of a pile group embedded in cohesionless soils is based on the weight of the block of soil itself considering buoyant unit weights below the static groundwater level. A factor of safety of 2.0 should be applied to the ultimate block weight. Refer to Table 8 and Figure 4 for the computation of the soil block in resisting uplift in a pile group.

Table 8. Recommended unit weights in computing the uplift of a soil block.

Elevation	Unit Weight (pcf)
Above 727.5	120
727.5 – 720	110
Below 720	62.6

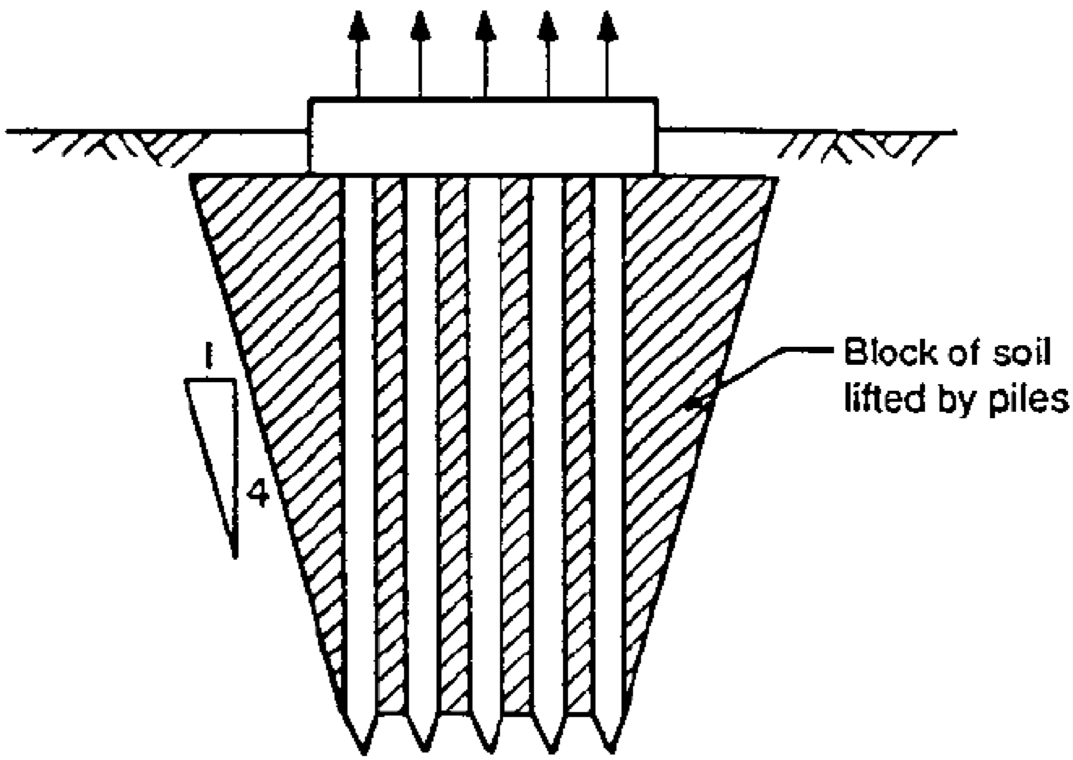


Figure 4. Definition of soil block in computing uplift capacity (Figure 10.7.3.11-1 from AASHTO 2020).

8.4.1.2 Lateral Pile Capacity

Where piles will be supporting lateral loads, we recommend that the response of the piles to these lateral forces be evaluated using p-y soil-structure interaction theory (LPILE) in accordance with the recommended parameters provided in Table 9.

Table 9. Recommended design parameters for p-y analyses of laterally loaded shafts

Elevation (ft.)	Layer	p-y Curve Model	γ (pcf)	s_u (psf)	ε_{50}	ϕ (°)	k (pci)
Above 727.5	FILL	Soft Clay	120	375	0.02		
727.5 – 720	ALLUVIUM	Reese Sand	110			28	10
Below 720	OUTWASH		62.6			40	60

[a] Definitions:
γ = unit weight, s_u = undrained shear strength, ε_{50} = strain at 50% stress
ϕ = angle of internal friction, k = initial horizontal modulus of subgrade reaction

When conducting lateral analyses, please note that where the spacing of laterally loaded deep foundations will be close enough that their areas of resistance overlap, we recommend that an appropriate p-multiplier be applied in the analyses to account for the overlap and reduction in lateral resistance. For piles spaced closer than 3.75 times the pile diameter and where the direction of pile spacing will be perpendicular to the load direction, we recommend that the p-multiplier (p_m) be defined by the empirical relationship presented in Reese et al. (2006):

$$p_m = 0.64(S/D)^{0.34} \leq 1.0$$

where S is the pile spacing and D is the pile diameter or width. For piles where the direction of pile spacing will be parallel to the load direction, the p-multipliers should be per Table 10.7.2.4-1 from the *AASHTO LRFD Bridge Design Specifications* (AASHTO 2020).

8.4.1.3 Augercast Pile Construction and Monitoring Considerations

The augercast piles should be installed by a prequalified Contractor experienced in the installation of augercast piles. To apply for prequalification, the Contractor should submit a list of augercast pile projects completed in the past 5 years. At least three of the listed augercast pile projects should be of equivalent difficulty and/or scope as this project. No Contractor should be considered acceptable without a minimum of 5 years of experience in this type of pile installation.

It is recommended that the Contract Documents for the augercast piles be lump sum based upon the total length estimated from the data contained in this report. Additional add and deduct items per foot of pile should also be included in the contract for deviations from the total estimated lengths of piles.

The augercast piles should be installed with proper, well-maintained equipment capable of drilling straight and plumb to the necessary depths, and then maintaining continuous high grout pressure during the withdrawal of the auger to prevent any “necking” of the grout column. The auger should be slowly rotated during withdrawal and grouting. The installation should be sequenced so that no pile is drilled within 6 pile diameters (center-to-center) of a nearby pile filled with grout less than 24 hours old.

Experience indicates that high pressure grouting through side discharge portals at the tip of the hollow stem auger tends to scour soils along the sides of the holes. The scoured soils then tend to swirl back into and contaminate the grout, which compromises the structural integrity of the pile. This contamination can cause structural failure of the pile at a load less than the design load.

For this reason, it is strongly recommended that the piles be installed using a bottom center discharge auger to reduce the possibility of grout contamination due to side scour. The specifications should include an item to this effect.

Undocumented fill is known to exist on site. In the event that auger refusal is encountered within 10 feet of the proposed bottom of pile cap elevation, the obstruction should be removed with an excavator, the excavation should be restored with compacted structural fill in accordance with the Project Specifications, and then the pile restarted. If an obstruction is encountered more than 10 feet below the bottom of pile cap elevation, but above the design tip elevation, the pile should be abandoned and grouted from the refusal point to 1 foot below the bottom of the pile cap elevation. It is recommended that the Contractor be paid for that footage at the same unit rate as the production piles. Where refusal is encountered, the Project Geotechnical Engineer and Structural Engineer should be advised, and replacement piles and/or reconfigured pile caps should be designed. The intention is to pay the Contractor for unforeseen conditions when constructing the piles at the design location.

We recommend that production piles not be accepted if any of the following conditions occur:

1. The design pile reinforcement cannot be placed manually in the pile following the completion of grouting.
2. The trap door at the bottom discharge outlet fails to open completely, effectively creating a side discharge condition.
3. Loss of grout head occurs for any reason during pile installation.
4. There is more than a 20-minute delay during grouting of any individual pile.
5. There is a drop of grout level after completion of the pile, which exceeds the average of the other pile installations by 2 feet or more.
6. There is a rise in the grout level of any amount.
7. The volume of grout installed in any pile is less than 125 percent of the theoretical volume of the augered hole.

In the event that any of these conditions occur, it will be necessary to redrill and regrout the individual pile for the pile to be considered acceptable as a production pile. The redrilling and the regrouting should be included in the original cost of the pile installation and should not be considered an extra.

The installation of the augercast piles should be reviewed by the Project Geotechnical Engineer, or a representative thereof, in order to confirm that the installation of the piles is consistent with the intent of the Project Specifications. The review should include confirmation of pile lengths, grouting pressures, grout volume, rate of auger withdrawal, changes in levels of completed grout columns, and installation of design reinforcement.

8.4.2 Helical Piles

Helical piles are also considered to be an appropriate foundation system for the existing site and subsurface conditions. Such piles consist of a steel shaft with round helix plates that provide foundation support. When the pile is rotated into the ground, the helix plates generate an axial thrust causing the pile to advance into the ground much like a screw into wood.

We anticipate that helical piles can be designed for allowable axial compression and uplift capacities of 45 and 20 kips, respectively, assuming a dual 10-inch diameter helix followed by a 12-inch diameter helix that penetrates into the medium dense to dense outwash. We recommend that the uppermost helix extend at least 2 feet into the outwash (i.e., at least 22 feet below existing grades). However, a Specialty Contractor should be consulted for the actual arrangement of the helices and determining the required torque to meet the project capacity and the minimum embedment into the outwash. The Specialty Contractor should coordinate with the Structural Engineer on the tolerable settlement. Additionally, the Specialty Contractor should coordinate with the Structural Engineer with regards to eccentric loads on the helical piles and the connections to the existing foundations.

Underpin excavations for the connection of the helical piles to the existing foundations should be limited in length to maintain stability and avoid undermining of the existing foundations.

We recommend that one static axial compression pile load test be performed for this project prior to the installation of the production piles near Boring B-2. The pile should be tested to a minimum of 2 times the maximum allowable design load.

The axial compression pile load test should be accomplished in accordance with ASTM D1143-07, Paragraph 8.1.2, "Procedure A: Quick Test". The reaction frame should be capable of a safe compression test load equal to at least 3 times the selected design load. The load test should be performed by the Specialty Contractor and observed during testing by the Project Geotechnical Engineer, or representative thereof.

We recommend that the helical pile design be reviewed by Geotechnology prior to implementation to evaluate conformance with the conclusions and recommendations of this report.

8.5 Buoyancy

Based on a 100-year flood elevation of El. 736, roughly 5 feet or less beneath existing grades, in combination with a predominately granular soil profile, we recommend that the proposed below-grade tanks associated with the fueling station be designed to resist buoyant uplift forces. For design against uplift, the groundwater level should be assumed at the ground surface surrounding the structure, and the floodwater level should be assumed at the design flood elevation.

Resistance to buoyant uplift may be provided by a combination of the dead weight of the tank, the buoyant unit weight of the backfill over the tank, and the soil friction around the tank.

For these analyses, the buoyant unit weight of the soil should be 47.6 pcf. Additionally, for the soil friction, an ultimate coefficient of static friction of 0.49 should be used. The normal force for these analyses should be determined from at-rest pressures based on submerged equivalent fluid weight of 27 pcf. A minimum factor of safety of 1.5 should be applied to the resistance to buoyant uplift using the above mechanism.

8.6 Lateral Earth Pressures

Where foundation and retaining walls for this project will be subjected to unbalanced lateral earth pressures, we recommend that the lateral earth pressures be computed on the basis of equivalent fluid weights of the backfill, plus surcharges for foundation loads, pavement loads, sloping backfill, etc. Table 10 provides the recommended equivalent fluid weights for soil for both drained and undrained conditions, and also the recommended earth pressure coefficients for proposed surcharges. Unless a site-specific analysis is performed, we recommend that surcharges be modeled as a uniform horizontal pressure equal to the vertical intensity of the surcharge multiplied by the recommended lateral earth pressure coefficient.

Table 10. Lateral earth pressures for level (horizontal) ground surfaces.

	Active[a]	At-Rest[a]	Passive[a,b]
Lateral earth pressure coefficient, K	0.39	0.56	2.56
Drained equivalent fluid weight, EFW (pcf)	47	67	307
Undrained equivalent fluid weight, EFW_u (pcf)[c]	85	95	210

[a] Parameters are based on level ground surfaces, a soil unit weight (γ) of 120 pcf, and a soil internal angle of friction (ϕ) of 26 degrees.

[b] Passive resistance may be considered where concrete is cast against free-standing vertical faces of soil; however, passive resistance should be ignored in the upper 30 inches below proposed grade due to seasonal variations in moisture and frost penetration. If the ground is sloping down and away from the foundation in the area of passive resistance, we should be contacted to provide site-specific recommendations.

[c] Includes hydrostatic pressure of 62.4 pcf.

The values provided in Table 10 assume that the ground surface adjacent to the wall is level and not sloping toward the wall. For ground sloping toward the wall on its active or at-rest side, we recommend that it be accounted for as a surcharge on the wall, as discussed above, unless site-specific equivalent fluid weights are computed on the basis of the backfill slope.

The decision to use active or at-rest earth pressures should be based upon the ability of the wall or structure to deflect as a result of the lateral earth pressures. In cohesionless granular backfill, active earth pressures are assumed to be applicable if the top of the wall is able to deflect a minimum of 0.002 times the height of the wall. In cohesive clayey backfill, the minimum deflection at the top of the wall for active earth pressures to develop is 0.02 times the height of the wall. If these minimum horizontal deflections at the top of the wall are restrained from occurring or unacceptable to the structure, at-rest earth pressures are applicable.

Undrained equivalent fluid weights should be used in computing the lateral loads on the wall wherever the backfill is unable to be drained by a drainage system (discussed below). For the drained equivalent fluid weights to be applicable, a drainage system should be incorporated along the backfilled face of the wall (i.e., the high side of the wall) consisting of either a prefabricated drainage board or an approximately 18-inch width of free-draining gravel with less than 3 percent fines wrapped with a non-woven drainage geotextile. At the base of the drainage board or free-draining gravel should be a minimum 12-inch-thick by 12-inch-wide, free-draining gravel zone wrapped with a non-woven drainage geotextile. Within the wrapped gravel at its base should be a 4-inch-diameter rigid perforated plastic pipe. The plastic pipe should be connected to a suitable gravity outlet (e.g., the proposed storm sewer system). The granular backfill should be compacted to at least 75 percent relative density per ASTM D4253 and D4254. We recommend that the drainage system extend to subgrade elevation beneath pavements or floor slabs; otherwise, the drainage system should extend to within 2 feet of finished grade and be capped with at least 2 feet of compacted clayey soils to reduce the infiltration of surface water behind the wall. Clayey backfill should be compacted per the requirements presented in Table 6. The drainage system should not connect to interior drainage systems below floor slabs. These interior drainage systems should have separate, independent outlets.

8.7 Utility Construction

We anticipate that select granular backfill will be used as pipe bedding and pipe zone backfill for the utilities. We recommend that the granular backfill be limited to the pipe bedding and minimum required pipe/utility cover. The remainder of the utility trenches should be backfilled with flowable fill or compacted clayey soils up to design subgrade elevation to reduce the potential for water collecting in these trenches and being absorbed by the surrounding clays, causing heave of foundations, slabs, pavement, etc.

Granular bedding and backfill that exhibits a well-defined moisture-density relationship should be compacted and moisture-conditioned per the requirements presented in Table 6; otherwise, the granular material should be compacted to at least the minimum relative densities indicated in Table 11.

Table 11. Relative density compaction requirements for granular fill and backfill.

Area	Minimum Relative Density[a,b]
Structural[c]	80%
Non-structural	75%
Floor slab and pavement subbase	80%

[a] Relative density evaluated on the basis of the maximum and minimum index densities determined from ASTM D4253 and D4254, respectively.

[b] For granular soils that exhibit a well-defined moisture-density relationship, refer to Table 6 on page 11 for minimum percent compaction and moisture-conditioning requirements.

[c] Structural fill and backfill for foundations are defined as fill and backfill located within the zones of influence of structures. The zone of influence of a structure is defined as the area below the footprint of the structure and 2H:1V outward and downward projections from the bearing elevation of the structure.

Utility trench backfill should be placed in 6- to 8-inch-thick lifts with each lift compacted to at least the specified degree of compaction. Under no circumstances should the backfill be flushed in an attempt to obtain compaction.

If flowable fill is used, it should have a design strength of at least 30 psi for stability and not greater than 100 psi for future excavatability.

Prior to placing the bedding and utilities within the utility trench, soft, saturated, and compressible material should be removed from the bottom of the trench, exposing moist stiff soils or undisturbed bedrock.

8.8 Pavement Design and Construction

Pavements for this project should be designed in accordance with expected axle loads, frequency of loading, and the properties of the subgrade. A California Bearing Ratio (CBR) value of **3** should be assumed in the pavement design for subgrade prepared per the recommendations in this report.

As previously mentioned in Section 8.2, proposed pavement subgrades should be proofrolled with loaded tandem-axle dump truck weighing at least 40,000 pounds under the review of the Project Geotechnical Engineer, or representative thereof. Soft or yielding soils observed during the proofroll should be undercut to stiff, non-yielding soils; however, the depth of undercut below subgrade may be limited to 3 feet in light-duty traffic areas and 4 feet in heavy-duty traffic areas. The undercut should be backfilled with new compacted fill satisfying the material and compaction requirements presented in Section 8.2. We recommend that the Contract Documents include an item for undercutting unsuitable soils and replacing them with new compacted and tested fill on a "per cubic yard of compacted replacement fill" basis.

If soft or yielding soils are encountered at the maximum undercut depths specified above (i.e., 3 feet for light-duty traffic and 4 feet for heavy-duty traffic) and the compaction requirements of the undercut backfill cannot be achieved at the bottom of the undercut, the subgrade may be stabilized at those depths using a biaxial or triaxial geogrid (e.g., Tensar BX-1200 or TriAx TX160 or equivalent) and an 8-inch lift of compacted crushed stone. The remainder of the undercut should be backfilled with dense-graded aggregate or clayey soils satisfying the material and compaction requirements presented in Section 8.2. If clayey soils are used, a separation geotextile should be provided at the interface between the crushed stone and the clayey soils.

In lieu of undercutting soft or yielding soils to the maximum undercut depths specified above (i.e., 3 feet for light-duty traffic and 4 feet for heavy-duty traffic), the subgrade may be stabilized using a biaxial or triaxial geogrid (e.g., Tensar BX-1200 or TriAx TX160 or equivalent) and at least 12 inches of compacted crushed stone. We recommend that the thickness of undercut and compacted crushed stone be field-evaluated based on the conditions encountered during construction and using a test section. This alternative should also be considered if weather, other site conditions, or the project schedule make earthwork activities with clayey soils impractical.

Prior to the placement of pavement or aggregate base, where provided, we recommend that the top 8 inches of clayey subgrade be scarified and recompacted per the requirements presented in Table 6.

If the proposed pavement section includes an aggregate base, we recommend that caution be exercised so that the proposed aggregate base does not become saturated during or after construction. Water trapped in the aggregate base is capable of freezing, causing it to expand within the voids it occupies. Consequently, ice lenses may form and potentially heave the pavement. Furthermore, the thawing process can soften underlying cohesive subgrades, which reduces the pavement support provided by the subgrade, giving rise to "pumping" of the pavements under loads. Preferably, the aggregate base should be a free-draining material with provisions for draining the base through a system of underdrains.

Surface drainage should be directed away from the edges of proposed or existing pavements so that water does not pond next to pavements or flow onto pavements from unpaved areas. Such ponding or flow can cause deterioration of pavement subgrades and premature failure of pavements. In those areas where exterior grades do not fully slope away from the edges of the proposed pavement, we recommend that edge drains be installed along the perimeter of the pavement.

If dumpsters are utilized at the project site, we recommend that the dumpster be supported on concrete slabs and that the slabs be sized to accommodate the loading wheels of the dumpster truck. The access lane to the dumpster should also be designed for the heavier wheel loads associated with dumpster trucks.

9.0 RECOMMENDED ADDITIONAL SERVICES

The conclusions and recommendations given in this report are based on: Geotechnology's understanding of the proposed design and construction, as outlined in this report; site observations; interpretation of the exploration data; and our experience. Since the intent of the design recommendations is best understood by Geotechnology, we recommend that Geotechnology be included in the final design and construction process, and be retained to review the project plans and specifications to confirm that the recommendations given in this report have been correctly implemented. We recommend that Geotechnology be retained to participate in prebid and preconstruction conferences to reduce the risk of misinterpretation of the conclusions and recommendations in this report relative to the proposed construction of the subject project.

Since actual subsurface conditions between boring locations may vary from those encountered in the borings, our design recommendations are subject to adjustment in the field based on the subsurface conditions encountered during construction. Therefore, we recommend that Geotechnology be retained to provide construction observation services as a continuation of the design process to confirm the recommendations in this report and to revise them accordingly to accommodate differing subsurface conditions. Construction observation is intended to enhance compliance with project plans and specifications. It is not insurance, nor does it constitute a warranty or guarantee of any type. Regardless of construction observation, contractors, suppliers, and others are solely responsible for the quality of their work and for adhering to plans and specifications.

10.0 LIMITATIONS

This report has been prepared on behalf of, and for the exclusive use of, Champlin Architecture for specific application to the named project as described herein. If this report is provided to other parties, it should be provided in its entirety with all supplementary information. In addition, Champlin Architecture should make it clear that the information is provided for factual data only, and not as a warranty of subsurface conditions presented in this report.

Geotechnology has attempted to conduct the services reported herein in a manner consistent with that level of care and skill ordinarily exercised by members of the profession currently practicing in the same locality and under similar conditions. The recommendations and conclusions contained in this report are professional opinions. The report is not a bidding document and should not be used for that purpose.

Our scope for this phase of the project did not include any environmental assessment or investigation for the presence or absence of wetlands or hazardous or toxic materials in the soil, surface water, groundwater, or air, on or below or around this site. Any statements in this report or on the boring logs regarding odors noted or unusual or suspicious items or conditions observed are strictly for the information of our client. Our scope did not include an assessment of the effects of flooding and erosion of creeks or rivers adjacent to or on the project site.

Our scope did not include: any services to investigate or detect the presence of mold or any other biological contaminants (such as spores, fungus, bacteria, viruses, and the by-products of such organisms) on and around the site; or any services, designed or intended, to prevent or lower the risk of the occurrence of an infestation of mold or other biological contaminants.

The analyses, conclusions, and recommendations contained in this report are based on the data obtained from the subsurface exploration. The field exploration methods used indicate subsurface conditions only at the specific locations where samples were obtained, only at the time they were obtained, and only to the depths penetrated. Consequently, subsurface conditions may vary gradually, abruptly, and/or nonlinearly between sample locations and/or intervals.

The conclusions or recommendations presented in this report should not be used without Geotechnology's review and assessment if the nature, design, or location of the facilities is changed, if there is a substantial lapse in time between the submittal of this report and the start of work at the site, or if there is a substantial interruption or delay during work at the site. If changes are contemplated or delays occur, Geotechnology must be allowed to review them to assess their impact on the findings, conclusions, and/or design recommendations given in this report. Geotechnology will not be responsible for any claims, damages, or liability associated with any other party's interpretations of the subsurface data or with reuse of the subsurface data or engineering analyses in this report.

The recommendations included in this report have been based in part on assumptions about variations in site stratigraphy that may be evaluated further during earthwork and foundation construction. Geotechnology should be retained to perform construction observation and continue its geotechnical engineering service using observational methods. Geotechnology cannot assume liability for the adequacy of its recommendations when they are used in the field without Geotechnology being retained to observe construction.

A copy of "Important Information about This Geotechnical-Engineering Report" that is published by the Geotechnical Business Council (GBC) of the Geoprofessional Business Association (GBA) is included in Appendix A for your review. The publication discusses some other limitations, as well as ways to manage risk associated with subsurface conditions.

REFERENCES

American Association of State Highway and Transportation Officials (2020). *AASHTO LRFD Bridge Design Specifications, 9th Edition.* Washington, D.C.

Reese, L.C., Isenhower, W.M., and Wang, S.T. (2006). *Analysis and Design of Shallow and Deep Foundations*, John Wiley & Sons, Inc., Hoboken, New Jersey.

APPENDIX A – IMPORTANT INFORMATION ABOUT THIS GEOTECHNICAL-ENGINEERING REPORT

Important Information about This

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 constructor — 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 this geotechnical-engineering report without first conferring with the geotechnical engineer who prepared it. *And no one — not even you* — should apply this 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.

Geotechnical Engineers Base Each Report on a Unique Set of Project-Specific Factors

Geotechnical engineers consider many 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;
- the elevation, configuration, location, orientation, or weight of the proposed structure;
- the 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 geotechnical engineer performed the study. *Do not rely on a geotechnical-engineering report whose adequacy may have been affected by*: the passage of time; man-made events, such as construction on or adjacent to the site; or natural events, such as floods, droughts, earthquakes, or groundwater fluctuations. *Contact the geotechnical engineer before applying this 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 geotechnical-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 confirmation-dependent recommendations included in your report. *Confirmation-dependent 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 confirmation-dependent recommendations if that engineer does not perform the geotechnical-construction observation required to confirm the recommendations' applicability.*

A Geotechnical-Engineering Report Is Subject to Misinterpretation

Other design-team members' misinterpretation of geotechnical-engineering reports has resulted in costly problems. Confront 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. Constructors can also misinterpret a geotechnical-engineering report. Confront that risk by having your geotechnical engineer participate in prebid and preconstruction conferences, and by providing geotechnical 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 Constructors a Complete Report and Guidance

Some owners and design professionals mistakenly believe they can make constructors liable for unanticipated subsurface conditions by limiting what they provide for bid preparation. To help prevent costly problems, give constructors the complete geotechnical-engineering report, *but* preface it with a clearly written letter of transmittal. In that letter, advise constructors 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 constructors have sufficient time* to perform additional study. Only then might you be in a position to give constructors 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 constructors fail to 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.

Environmental Concerns Are Not Covered

The equipment, techniques, and personnel used to perform an *environmental* study differ significantly from those used to perform a *geotechnical* study. For that reason, a geotechnical-engineering report does not usually relate any environmental 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 environmental 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, many 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 GBC-Member Geotechnical Engineer for Additional Assistance

Membership in the Geotechnical Business Council of the Geoprofessional Business Association exposes geotechnical engineers to a wide array of risk-confrontation techniques that can be of genuine benefit for everyone involved with a construction project. Confer with you GBC-Member geotechnical engineer for more information.

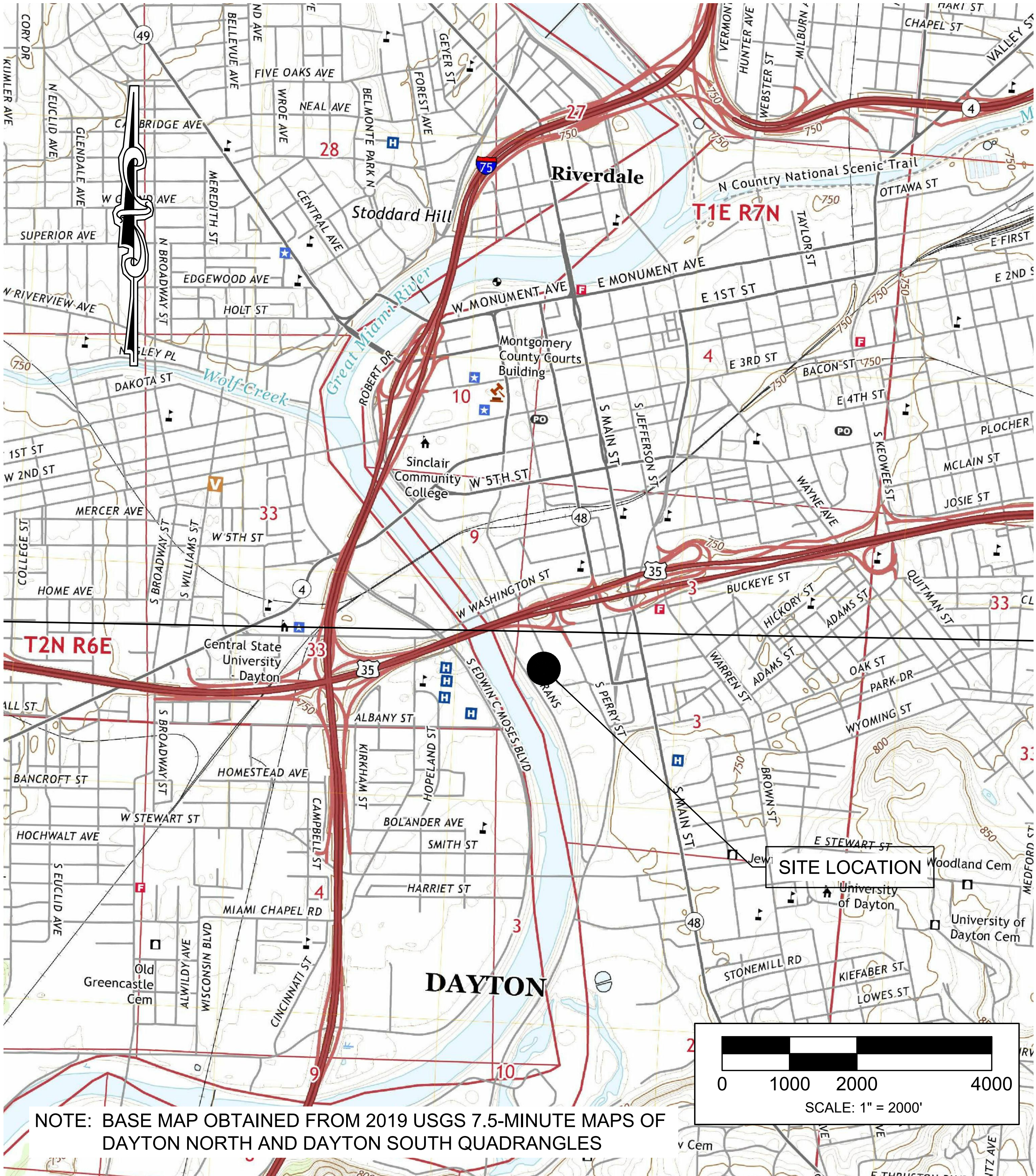
GEOTECHNICAL BUSINESS COUNCIL
of the Geoprofessional Business Association

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

APPENDIX B – PLANS

Site Location Plan, Sheet No. 1

Boring Plan, Sheet No. 2



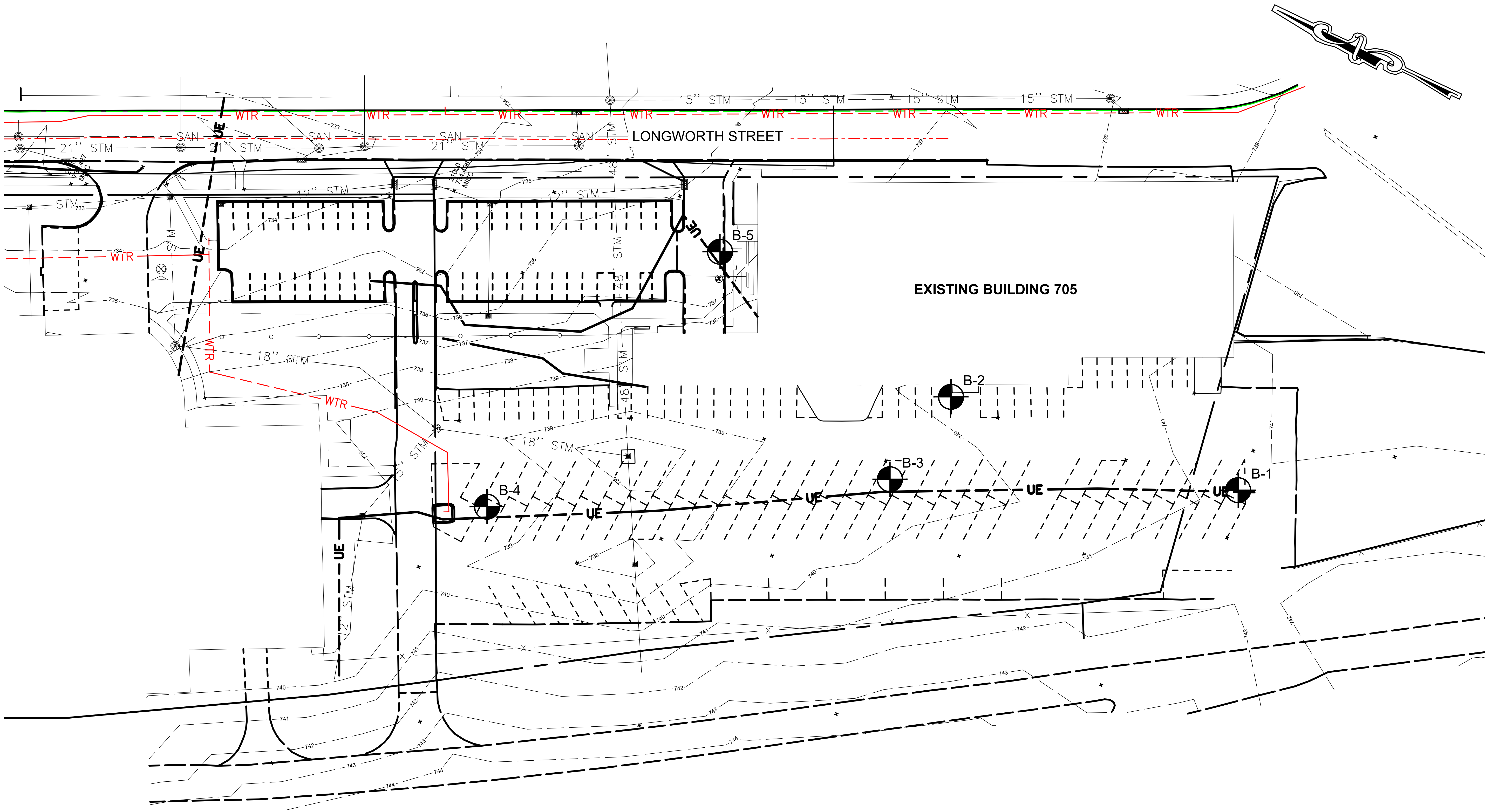
NOTE: BASE MAP OBTAINED FROM 2019 USGS 7.5-MINUTE MAPS OF DAYTON NORTH AND DAYTON SOUTH QUADRANGLES

GEOTECHNOLOGY

SITE LOCATION PLAN

Client:	CHAMPLIN ARCHITECTURE
Project:	GDRTA BUILDING 705
Location:	DAYTON, OH
Date: 7/29/2021	Project No.: J038716.01	Sheet No.: 1

Date Printed: 7/29/2021 10:01 AM Path: \\10.0.12.42\erl\data\projects\J038\J038716.01-GDRTA Building 705\Drawing\Boring Plan-J038716.01.dwg



INDICATES TEST BORING LOCATIONS

0 30 60 120

SCALE: 1" = 60'

NOTE: BASE MAP AUTOCAD DRAWING RECEIVED ON 4/26/2021 FROM LJB, INC.

GEOTECHNOLOGY

Project: GDRTA BUILDING 705

Location: 705 LONGWORTH STREET DAYTON, OHIO

Title: BORING PLAN

Client: CHAMPLIN ARCHITECTURE

Date: 7/29/2021

Project No.: J038716.01

Sheet No.: 2

APPENDIX C – BORING INFORMATION

Boring Logs

Soil Classification Sheet

GEOTECHNOLOGY
A Universal Engineering Sciences Company

LOG OF TEST BORING

CLIENT: Champlin Architecture
BORING #: B-1
PROJECT: GDRTA Building 705
Dayton, Ohio
PROJECT #: J038716.01
PAGE #: 1 of 1
LOCATION OF BORING: As shown on Boring Plan, Drawing Sheet No. 2

ELEV.	COLOR, MOISTURE, DENSITY, PLASTICITY, SIZE, PROPORTIONS DESCRIPTION	Strata Depth (feet)	Depth Scale (feet)	Sample Condition	Sample Number	Sample Type	SPT* Blows/6" Rock Core RQD (%)	Recovery (in.)	Recovery (%)	HP (tsf)
741.3	Ground Surface	0.0	0							
740.5	ASPHALT (10 inches)	0.8								
740.3	GRAVEL BASE (2 inches)	1.0		D	1	SS	7-8-11	10	56	
736.8	Dark brown and black moist medium dense SAND and GRAVEL, and CINDERS with brick fragments, glass fragments, carpet, and clay (fill).	4.5		D	2	SS	10-8-7	12	67	
734.3	Dark brown and black moist very loose clayey SAND with cinders and brick fragments (fill).	7.0	5	D	3	SS	2-2-1	17	94	
	Brown moist to very moist very loose to loose clayey SAND with trace gravel, trace shells (alluvium).			I	4	SS	0-0-2	16	89	1.0
			10	I	5	SS	4-3-2	14	78	1.25
726.8		14.5		I	6	SS	2-2-2	14	78	1.0
724.3	Dark brown moist stiff LEAN CLAY with oxide stains (alluvium).	17.0	15	I	7	SS	2-2-3	11	61	1.0
721.8	Orange brown moist loose fine to medium SAND with trace silt and clay (alluvium).	19.5		D	8	SS	3-2-4	14	78	
	Brown slightly moist medium dense to dense silty GRAVEL and fine to coarse SAND (outwash).		20	D	9	SS	7-10-16	14	78	
713.3		28.0	25	D	10	SS	13-14-16	14	78	
	Brown slightly wet medium dense to dense silty GRAVEL and fine to coarse SAND (outwash).		30	D	11	SS	11-12-17	16	89	
704.8		36.5	35	D	12	SS	22-20-23	15	83	
	Bottom of test boring at 36.5 feet.		40							
			45							

Datum: NAVD 88
Hammer Weight: 140 lb.
Hole Diameter: 8 in.
Drill Rig: TD-6
Surface Elevation: 741.3 ft.
Hammer Drop: 30 in.
Rock Core Diameter: --
Foreman: LRK
Date Started: 4/27/2021
Pipe Size: 2 in. O.D.
Boring Method: HSA-3.25
Engineer: Joseph D. Hauber
Date Completed: 4/27/2021

BORING METHOD
HSA = Hollow Stem Augers
CFA = Continuous Flight Augers
DC = Driving Casing
MD = Mud Drilling

SAMPLE TYPE
PC = Pavement Core
CA = Continuous Flight Auger
SS = Split-Spoon Sample
ST = Shelby Tube
RC = Rock Core

SAMPLE CONDITIONS
D = Disintegrated
I = Intact
U = Undisturbed
L = Lost

GROUNDWATER DEPTH
First Noted 30.0 ft.
At Completion 36.5 ft.
After --
Backfilled Immediately

* SPT = Standard Penetration Test - Driving 2" O.D. Sampler 18" with 140-Pound Hammer Falling 30"; Count Made at 6" Intervals

GEOTECHNOLOGY
A Universal Engineering Sciences Company

LOG OF TEST BORING

CLIENT: Champlin Architecture **BORING #:** B-2
PROJECT: GDRTA Building 705 **PROJECT #:** J038716.01
Dayton, Ohio **PAGE #:** 1 of 2
LOCATION OF BORING: As shown on Boring Plan, Drawing Sheet No. 2

ELEV.	COLOR, MOISTURE, DENSITY, PLASTICITY, SIZE, PROPORTIONS DESCRIPTION	Strata Depth (feet)	Depth Scale (feet)	Sample Condition	Sample Number	Sample Type	SPT* Blows/6" / Rock Core RQD (%)	Recovery (in.)	Recovery (%)	HP (tsf)
740.2	Ground Surface	0.0	0							
739.9	ASPHALT (3 inches)	0.3								
739.7	SAND and BRICK BASE (3 inches)	0.5		I	1	SS	3-6-9	6	33	
737.7		2.5								
735.7	Brown and gray slightly moist medium dense silty GRAVEL and SAND with brick fragments and cinders (fill).	4.5		I/D	2	SS	6-6-7	8	44	
733.2	Brown moist stiff sandy LEAN CLAY with gravel (fill).	7.0	5	D	3	SS	8-5-5	8	44	
	Brown slightly moist medium dense silty SAND with gravel, trace clay (fill).			I	4	SS	3-3-6	7	39	1.5
728.2	Dark brown moist stiff to very stiff LEAN CLAY with little sand (alluvium).	12.0	10	I	5	SS	3-4-5	10	56	2.5
725.7	Dark gray moist stiff FAT CLAY with organic odor (alluvium) (CH).	14.5		I	6	SS	3-4-5	10	56	2.0
723.2	Brown to dark brown moist stiff LEAN CLAY with sand and oxide stains (alluvium) (CL).	17.0	15	I	7	SS	4-4-4	12	67	
720.7	Brown moist loose fine to coarse SAND with trace silt and clay (alluvium).	19.5		D	8	SS	2-3-4	13	72	
	Brown slightly most dense to medium dense well-graded GRAVEL and SAND with silt (outwash) (GW-GM).		20	D	9	SS	15-17-17	14	78	
712.2		28.0	25	D	10	SS	5-6-17	17	94	
702.2	Brown wet dense to very dense silty GRAVEL and SAND (outwash).	38.0	30	D	11	SS	17-17-14	13	72	
			35	D	12	SS	26-38-30	15	83	
697.2	Brown wet dense to very dense poorly graded fine SAND with trace gravel and fines (outwash) (SP-SM).	43.0	40	D	13	SS	22-24-25	18	100	
	Brown wet very dense silty GRAVEL and SAND (outwash).		45							

Datum: NAVD 88 | Hammer Weight: 140 lb. | Hole Diameter: 8 in. | Drill Rig: TD-6
Surface Elevation: 740.2 ft. | Hammer Drop: 30 in. | Rock Core Diameter: -- | Foreman: LRK
Date Started: 4/28/2021 | Pipe Size: 2 in. O.D. | Boring Method: HSA-3.25 | Engineer: Joseph D. Hauber
Date Completed: 4/28/2021

BORING METHOD
HSA = Hollow Stem Augers
CFA = Continuous Flight Augers
DC = Driving Casing
MD = Mud Drilling

SAMPLE TYPE
PC = Pavement Core
CA = Continuous Flight Auger
SS = Split-Spoon Sample
ST = Shelby Tube
RC = Rock Core

SAMPLE CONDITIONS
D = Disintegrated
I = Intact
U = Undisturbed
L = Lost

GROUNDWATER DEPTH
First Noted 30.0 ft.
At Completion See Note
After --
Backfilled Immediately

* SPT = Standard Penetration Test - Driving 2" O.D. Sampler 18" with 140-Pound Hammer Falling 30"; Count Made at 6" Intervals

GEOTECHNOLOGY
A Universal Engineering Sciences Company

LOG OF TEST BORING

CLIENT: Champlin Architecture
BORING #: B-2
PROJECT: GDRTA Building 705
Dayton, Ohio
PROJECT #: J038716.01
PAGE #: 2 of 2
LOCATION OF BORING: As shown on Boring Plan, Drawing Sheet No. 2

ELEV.	COLOR, MOISTURE, DENSITY, PLASTICITY, SIZE, PROPORTIONS DESCRIPTION	Strata Depth (feet)	Depth Scale (feet)	Sample Condition	Sample Number	Sample Type	SPT* Blows/6" / Rock Core RQD (%)	Recovery (in.)	Recovery (%)	HP (tsf)
			45	D	14	SS	18-36-50/1"	18	100	
692.2	Brown wet very dense silty GRAVEL and SAND (outwash).	48.0								
	Brown wet medium dense silty GRAVEL and SAND (outwash).		50	D	15	SS	11-13-15	16	89	
687.2		53.0								
	Grayish brown very moist stiff LEAN CLAY, laminated with sand and gravel seams (lacustrine/outwash).		55	I/D	16	SS	19-25-28	15	83	2.0
682.2		58.0								
	Brown and gray wet very dense clayey SAND and GRAVEL with limestone and shale fragments (outwash).		60	D	17	SS	50/3"	3	100	
			65	I/D	18	SS	50/4"	4	100	
			70	D	19	SS	38-29-47	12	67	
667.2		73.0								
	Grayish brown very moist very stiff SILTY CLAY, partly laminated, with wet silt seams, sand, and gravel dropstones (lacustrine) (CL-ML).		75	I	20	SS	36-23-47	14	78	4.0+
			80	I	21	SS	38-40-49	12	67	4.0+
658.7		81.5								
	Bottom of test boring at 81.5 feet.									
	Note: Drilling mud added down hollow stems to prevent heaving of sands after encountering groundwater at 30.0 ft.		85							
			90							

Datum: NAVD 88
Hammer Weight: 140 lb.
Hole Diameter: 8 in.
Drill Rig: TD-6
Surface Elevation: 740.2 ft.
Hammer Drop: 30 in.
Rock Core Diameter: --
Foreman: LRK
Date Started: 4/28/2021
Pipe Size: 2 in. O.D.
Boring Method: HSA-3.25
Engineer: Joseph D. Hauber
Date Completed: 4/28/2021

BORING METHOD
HSA = Hollow Stem Augers
CFA = Continuous Flight Augers
DC = Driving Casing
MD = Mud Drilling

SAMPLE TYPE
PC = Pavement Core
CA = Continuous Flight Auger
SS = Split-Spoon Sample
ST = Shelby Tube
RC = Rock Core

SAMPLE CONDITIONS
D = Disintegrated
I = Intact
U = Undisturbed
L = Lost

GROUNDWATER DEPTH
First Noted 30.0 ft.
At Completion See Note
After --
Backfilled Immediately

* SPT = Standard Penetration Test - Driving 2" O.D. Sampler 18" with 140-Pound Hammer Falling 30"; Count Made at 6" Intervals

GEOTECHNOLOGY

A Universal Engineering Sciences Company

LOG OF TEST BORING

CLIENT: Champlin Architecture

BORING #: B-3

PROJECT: GDRTA Building 705

Dayton, Ohio

PROJECT #: J038716.01

PAGE #: 1 of 1

LOCATION OF BORING: As shown on Boring Plan, Drawing Sheet No. 2

ELEV.	COLOR, MOISTURE, DENSITY, PLASTICITY, SIZE, PROPORTIONS DESCRIPTION	Strata Depth (feet)	Depth Scale (feet)	Sample Condition	Sample Number	Sample Type	SPT* Blows/6" / Rock Core RQD (%)	Recovery (in.)	Recovery (%)	HP (tsf)
739.6	Ground Surface	0.0	0							
739.3	ASPHALT (3 inches)	0.3								
739.1	GRAVEL BASE (3 inches)	0.5		I	1	SS	3-7-8	8	44	2.0
737.1	Dark brown and black moist stiff to very stiff sandy LEAN CLAY with gravel and organic odor (fill).	2.5		I	2	SS	2-2-3	10	56	
	Brown moist loose clayey SAND with gravel, brick fragments, cinders, and lean clay layers/pockets (fill).		5	D	3	SS	2-3-2	6	33	
				D	4	SS	2-3-3	10	56	
			10	D	5	SS	2-3-4	6	33	
727.6		12.0								
	Dark brown moist stiff LEAN CLAY, trace sand (alluvium).			I	6	SS	3-4-5	18	100	
725.1		14.5	15							
	Brown, trace gray moist loose clayey SAND with trace shells (alluvium).			I	7	SS	3-2-4	18	100	1.5
722.6		17.0								
	Brown and orange brown moist medium dense fine to coarse SAND with little gravel (alluvium).			D	8	SS	4-7-7	16	89	
720.1		19.5	20							
	Brown slightly moist dense silty GRAVEL and SAND (outwash).			D	9	SS	12-20-15	11	61	
			25	D	10	SS	15-17-19	6	33	
711.6		28.0								
	Brown wet dense silty GRAVEL and SAND (outwash).		30	D	11	SS	11-16-17	10	56	
			35	D	12	SS	10-15-16	16	89	
703.1		36.5								
	Bottom of test boring at 36.5 feet.		40							
			45							

Datum: NAVD 88	Hammer Weight: 140 lb.	Hole Diameter: 8 in.	Drill Rig: TD-6
Surface Elevation: 739.6 ft.	Hammer Drop: 30 in.	Rock Core Diameter: --	Foreman: LRK
Date Started: 4/28/2021	Pipe Size: 2 in. O.D.	Boring Method: HSA-3.25	Engineer: Joseph D. Hauber
Date Completed: 4/28/2021			

BORING METHOD
HSA = Hollow Stem Augers
CFA = Continuous Flight Augers
DC = Driving Casing
MD = Mud Drilling

SAMPLE TYPE
PC = Pavement Core
CA = Continuous Flight Auger
SS = Split-Spoon Sample
ST = Shelby Tube
RC = Rock Core

SAMPLE CONDITIONS
D = Disintegrated
I = Intact
U = Undisturbed
L = Lost

GROUNDWATER DEPTH
First Noted 25.0 ft.
At Completion 25.0 ft.
After --
Backfilled Immediately

* SPT = Standard Penetration Test - Driving 2" O.D. Sampler 18" with 140-Pound Hammer Falling 30"; Count Made at 6" Intervals

GEOTECHNOLOGY
A Universal Engineering Sciences Company

LOG OF TEST BORING

CLIENT: Champlin Architecture　　**BORING #:** B-4

PROJECT: GDRTA Building 705　　**PROJECT #:** J038716.01

Dayton, Ohio　　**PAGE #:** 1 of 1

LOCATION OF BORING: As shown on Boring Plan, Drawing Sheet No. 2

ELEV.	COLOR, MOISTURE, DENSITY, PLASTICITY, SIZE, PROPORTIONS DESCRIPTION	Strata Depth (feet)	Depth Scale (feet)	Sample Condition	Sample Number	Sample Type	SPT* Blows/6" / Rock Core RQD (%)	Recovery (in.)	Recovery (%)	HP (tsf)
739.2	Ground Surface	0.0	0							
738.7	ASPHALT (6 inches)	0.5								
738.2	BRICK and SAND (fill)	1.0								
736.7	Dark brown and brown moist very stiff sandy LEAN CLAY with gravel, trace cinders (fill).	2.5		I	1	SS	4-5-8	12	67	2.5
				D	2	SS	5-7-7	14	78	
727.2	Dark brown, black, and brown very moist very loose to loose CINDERS, SAND, GRAVEL, and BRICK FRAGMENTS with lean clay (fill).	12.0	5	D	3	SS	3-2-2	14	78	
				D	4	SS	2-2-2	16	89	
			10	D	5	SS	2-3-6	18	100	
724.7	Dark brown moist very stiff LEAN CLAY with trace shells (alluvium).	14.5		I	6	SS	2-1-4	18	100	2.5
722.2	Dark brown and black moist stiff LEAN CLAY with organics and organic odor, trace shells (alluvium).	17.0	15	I	7	SS	2-2-3	18	100	
719.7	Brown slightly moist loose fine to medium SAND with organic seams (alluvium).	19.5		D	8	SS	2-3-5	14	78	
711.2	Brown slightly moist dense to very dense silty GRAVEL and SAND (outwash).	28.0	20	D	9	SS	9-11-21	16	89	
			25	D	10	SS	27-28-43	18	100	
702.7	Brown wet dense to very dense silty GRAVEL and SAND (outwash).	36.5	30	D	11	SS	42-39-16	18	100	
			35	D	12	SS	14-20-24	18	100	
	Bottom of test boring at 36.5 feet.		40							
			45							

Datum: NAVD 88	Hammer Weight: 140 lb.	Hole Diameter: 8 in.	Drill Rig: TD-6
Surface Elevation: 739.2 ft.	Hammer Drop: 30 in.	Rock Core Diameter: --	Foreman: LRK
Date Started: 4/27/2021	Pipe Size: 2 in. O.D.	Boring Method: HSA-3.25	Engineer: Joseph D. Hauber
Date Completed: 4/27/2021			

BORING METHOD
- HSA = Hollow Stem Augers
- CFA = Continuous Flight Augers
- DC = Driving Casing
- MD = Mud Drilling

SAMPLE TYPE
- PC = Pavement Core
- CA = Continuous Flight Auger
- SS = Split-Spoon Sample
- ST = Shelby Tube
- RC = Rock Core

SAMPLE CONDITIONS
- D = Disintegrated
- I = Intact
- U = Undisturbed
- L = Lost

GROUNDWATER DEPTH
- First Noted: 30.0 ft.
- At Completion: 36.5 ft.
- After: --
- Backfilled: Immediately

* SPT = Standard Penetration Test - Driving 2" O.D. Sampler 18" with 140-Pound Hammer Falling 30"; Count Made at 6" Intervals

GEOTECHNOLOGY

A Universal Engineering Sciences Company

LOG OF TEST BORING

CLIENT: Champlin Architecture **BORING #:** B-5

PROJECT: GDRTA Building 705 **PROJECT #:** J038716.01

Dayton, Ohio **PAGE #:** 1 of 1

LOCATION OF BORING: As shown on Boring Plan, Drawing Sheet No. 2

ELEV.	COLOR, MOISTURE, DENSITY, PLASTICITY, SIZE, PROPORTIONS DESCRIPTION	Strata Depth (feet)	Sample Condition	Sample Number	Sample Type	SPT* Blows/6" / Rock Core RQD (%)	Recovery (in.)	Recovery (%)	HP (tsf)
736.6	Ground Surface	0.0							
736.1	CONCRETE (6 inches)	0.5							
734.1	Gray dry medium dense silty SAND and GRAVEL (fill).	2.5	D	1	SS	3-5-7	3	17	
	Brown, little black moist stiff sandy LEAN CLAY with gravel, cinders, organics, glass, plastic, and carpet (fill).		D	2	SS	3-2-2	10	56	
729.6		7.0	I/D	3	SS	2-1-2	6	33	
727.1	Brown very moist very loose clayey SAND with gravel and oxide stains (fill).	9.5	I	4	SS	2-1-1	8	44	0.5
724.6	Dark brown and black very moist to wet very loose clayey SAND with gravel and cinders (fill).	12.0	I	5	SS	8-2-2	8	44	
722.1	Grayish brown very moist medium stiff to stiff LEAN CLAY with sand, trace shells, organics and organic odor (alluvium).	14.5	I	6	SS	2-2-3	16	89	1.0
719.6	Grayish brown very moist very loose clayey SAND with trace shells, organics, organic odor, and apparent unnatural odor (alluvium).	17.0	I	7	SS	2-1-1	16	89	
717.1	Dark gray moist loose silty fine to medium SAND with trace shells (alluvium).	19.5	D	8	SS	2-2-3	14	78	
	Brown and gray slightly moist dense silty GRAVEL and SAND (outwash).		D	9	SS	11-15-17	16	89	
708.6		28.0	D	10	SS	23-16-15	16	89	
	Brown and gray wet dense silty GRAVEL and SAND (outwash).		D	11	SS	17-21-22	18	100	
700.1		36.5	D	12	SS	26-21-23	18	100	
	Bottom of test boring at 36.5 feet.								

Datum: NAVD 88 Hammer Weight: 140 lb. Hole Diameter: 8 in. Drill Rig: TD-6

Surface Elevation: 736.6 ft. Hammer Drop: 30 in. Rock Core Diameter: -- Foreman: LRK

Date Started: 4/30/2021 Pipe Size: 2 in. O.D. Boring Method: HSA-3.25 Engineer: Joseph D. Hauber

Date Completed: 4/30/2021

BORING METHOD
- HSA = Hollow Stem Augers
- CFA = Continuous Flight Augers
- DC = Driving Casing
- MD = Mud Drilling

SAMPLE TYPE
- PC = Pavement Core
- CA = Continuous Flight Auger
- SS = Split-Spoon Sample
- ST = Shelby Tube
- RC = Rock Core

SAMPLE CONDITIONS
- D = Disintegrated
- I = Intact
- U = Undisturbed
- L = Lost

GROUNDWATER DEPTH
- First Noted 30.0 FT.
- At Completion 36.0 FT.
- After --
- Backfilled Immediately

* SPT = Standard Penetration Test - Driving 2" O.D. Sampler 18" with 140-Pound 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	- 4 blows/ft. or less
Loose	- 5 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.

APPENDIX D – LABORATORY TEST DATA

Tabulation of Laboratory Tests

Particle-Size Analysis Test Forms

GEOTECHNOLOGY
A Universal Engineering Sciences Company

GDRTA BUILDING 705
705 LONGWORTH STREET
DAYTON, KENTUCKY
J038716.01

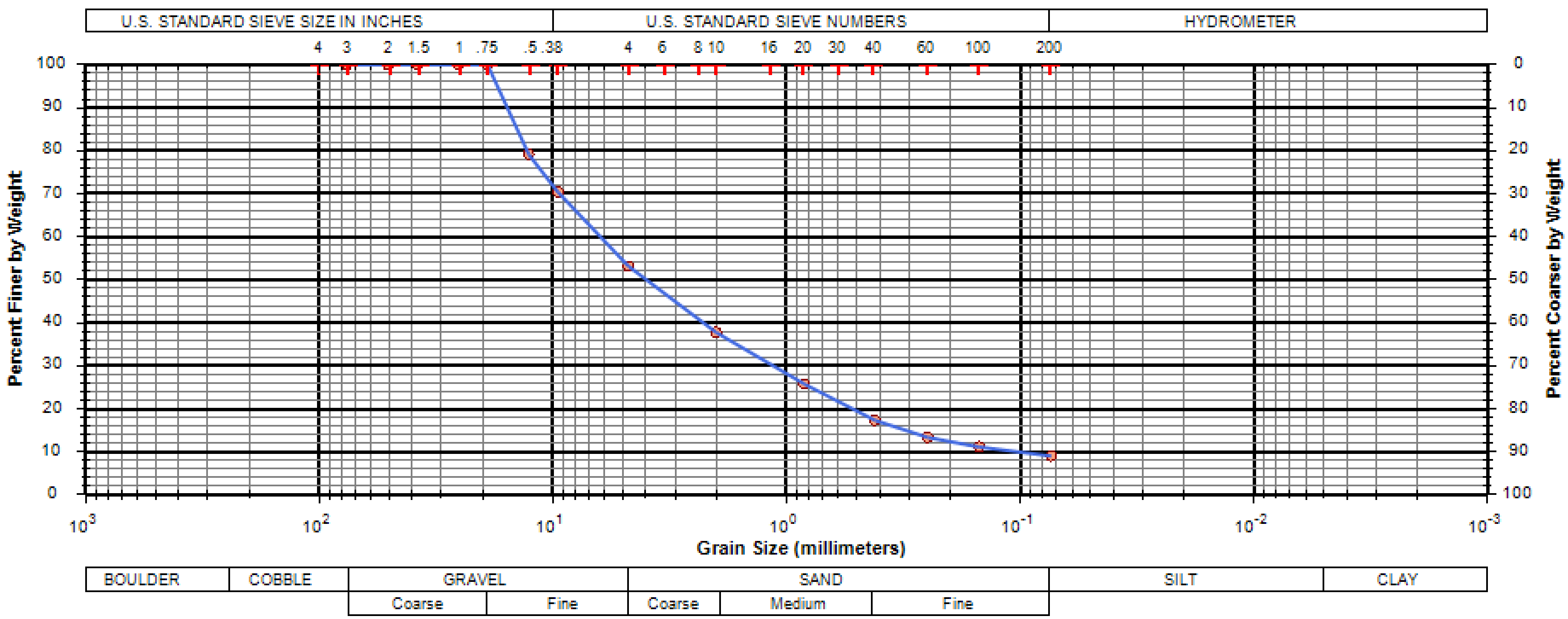
TABULATION OF LABORATORY TESTS

Boring No.	Sample No.	Depth (ft.)		Moisture Content (%)	Atterberg Limits (%)			Gradation Analysis (%)				USCS Classification
		From	To		LL	PL	PI	Gravel	Sand	Silt	Clay	
B-1	4	7.5	9.0	24.5								
B-1	7	15.0	16.5	35.1								
B-2	4	7.5	9.0	25.0								
B-2	6	12.5	14.0	36.7	55	29	26					CH
B-2	7	15.0	16.5	31.5	44	23	21					CL
B-2	9	20.0	21.5					46.8	44.2	9.0		GW-GM
B-2	13	40.0	41.5					0.4	89.6	10.0		SP-SM
B-2	16	55.0	65.5	23.4								
B-2	20&21	75.0	81.5	18.8	23	19	4	11.3	16.4	72.3		CL-ML
B-3	5	10.0	11.5	14.9								
B-3	6	12.5	14.0	33.4								
B-3	7	15.0	16.5	26.1								
B-4	6	12.5	14.0	33.8								
B-4	7	15.0	16.5	38.7								
B-5	5	10.0	11.5	19.1								
B-5	6	12.5	14.0	32.4								
B-5	7	15.0	16.5	31.6								

GEOTECHNOLOGY
A Universal Engineering Sciences Company

PARTICLE-SIZE ANALYSIS OF SOILS ASTM D-422

Client:	Champlin Architecture								**Project No.:**	J038716.01
Project:	Geotechnical Exploration, GDRTA Building 705, Dayton, OH								**Date:**	05/07/2021
Boring No.:	B-2	**Sample No.:**	9	**Depth (ft.):**	20.0 - 21.5	**Gravel (%)**	**Sand (%)**	**Silt & Clay (%)**		**USCS**
Sample Description:	Brown well-graded GRAVEL and SAND with silt (outwash)					46.8	44.2	9.0		GW-GM
						LL	**PL**	**PI**	**Group Index**	**WC (%)**



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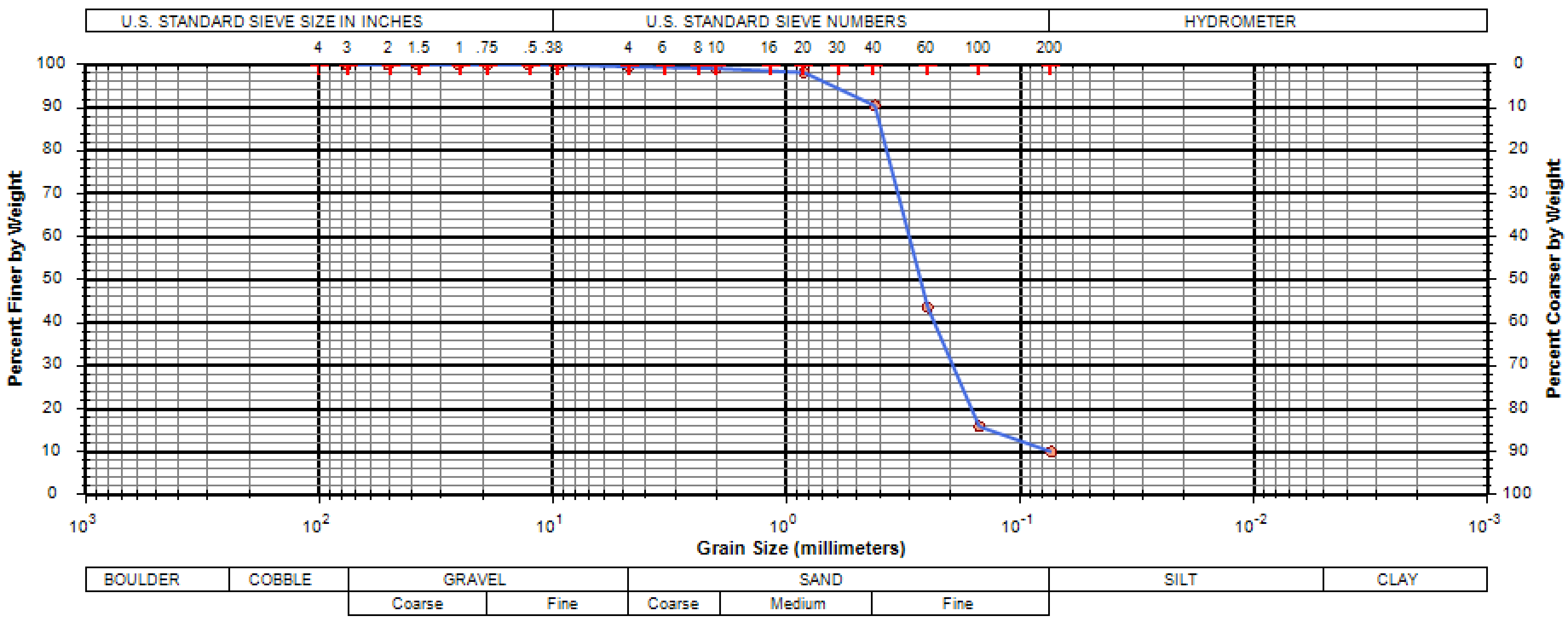
PARTICLE-SIZE ANALYSIS OF SOILS ASTM D-422

Client:	Champlin Architecture						**Project No.:**	J038716.01
Project:	Geotechnical Exploration, GDRTA Building 705, Dayton, OH						**Date:**	05/07/2021
Boring No.:	B-2	**Sample No.:** 13	**Depth (ft.):** 40.0 - 41.5	**Gravel (%)**	**Sand (%)**	**Silt & Clay (%)**		**USCS**
Sample Description:	Brown poorly graded fine SAND with trace gravel and fines (alluvium)			0.4	89.6	10.0		SP-SM
				LL	**PL**	**PI**	**Group Index**	**WC (%)**



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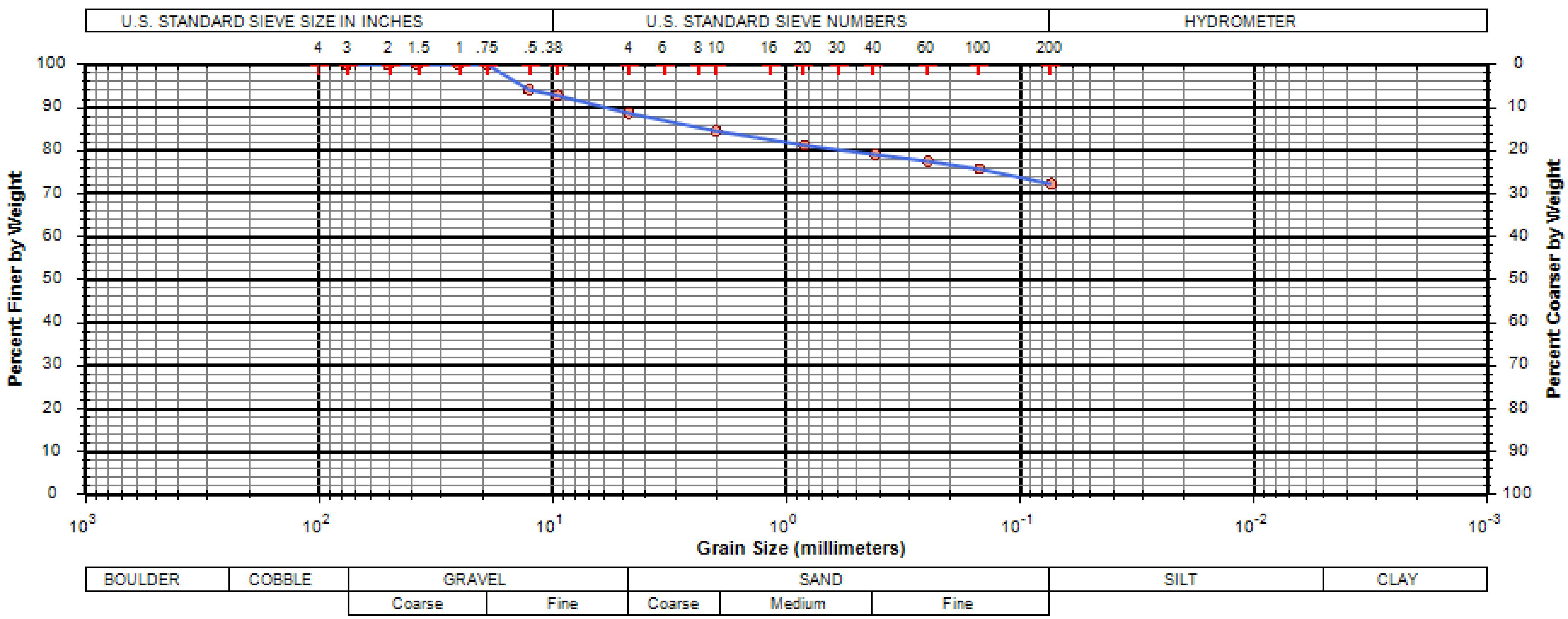
PARTICLE-SIZE ANALYSIS OF SOILS ASTM D-422

Client:	Champlin Architecture						**Project No.:**	J038716.01
Project:	Geotechnical Exploration, GDRTA Building 705, Dayton, OH						**Date:**	05/07/2021
Boring No.:	B-2	**Sample No.:** 20 & 21	**Depth (ft.):** 75.0 - 81.5	**Gravel (%)**	**Sand (%)**	**Silt & Clay (%)**		**USCS**
Sample Description:	Grayish brown SILTY CLAY, partly laminated, with sand and gravel dropstones (lacustrine)			11.3	16.4	72.3		CL-ML
				LL	**PL**	**PI**	**Group Index**	**WC (%)**
				23	19	4		18.8



October 16, 2024

Mr. David Henners, AIA, RIBA
Champlin Architecture
720 E. Pete Rose Way
Cincinati, Ohio 45202

Re: Geotechnical Exploration Report-Addendum No. 1
GDRTA Building 705
Longworth Street, Dayton, Ohio
UES Project No. J038716.02

Dear Mr. Henners:

Geotechnology LLC (dba UES) prepared this Addendum No. 1 to our July 29, 2021, geotechnical report, titled "Geotechnical Exploration, GDRTA Building 705, Dayton, Ohio" (July 2021-Geotechnical Report). This addendum is supplemental to, and to be used in conjunction with, our July 2021 Geotechnical Report.

The project includes the construction of two separate buildings, one for bus garage and another for bus wash and fueling station purposes. The existing GDRTA (Greater Dayton Regional Transit Authority) Building at 705 Longworth Street (Building 705) will be completely demolished as part of the current project plan and the new garage building will be constructed in its place. The existing Building 705 is an irregularly shaped, 1 story, brick and CMU-block building that encompasses roughly 34,000 square feet in plan area. The new garage building will have approximately double plinth area than that of the existing Building 705.

Geotechnology drilled six (6) test borings in 2021 outside the existing building footprint. Due to cost constraints, boring(s) inside the existing building were not performed. Hence, the recommendations presented in the original report assumed that similar subsurface conditions to the exterior borings exist within the footprint of the existing building. No additional borings and laboratory testing were performed to prepare recommendations presented in this addendum.

1.0 PROJECT INFORMATION

The following project information was derived from the following information:

- Correspondence via phone conversation and email with Mr. David Henners (Champlin) since September 17, 2024;
- Email correspondence with Mr. Chris Buckreus, PE, SE (Schaefer) from September 23, 2024.

- Existing conditions Exhibit (Sheet C100 and C101) prepared by LJB dated October 30, 2023.

- Site Plan Exhibit (Sheet C300 and C301) prepared by LJB dated October 30, 2023.

Based on a review of Schaefer's document dated September 23, 2024, the following information is summarized:

- Maximum column and wall load for Bus Wash (slab-on-grade, one story building over footprint of approximately 62 feet by 34 feet) and Fueling Station (canopy and roof structure) are 60 kips and 2.65 kips per lineal feet (klf) respectively.

- The dimensions of GDRTA Paratransit Bus Garage (Garage) are approximately 208'-6" along north-south orientation, and the dimensions along the east-west orientation range from approximately 291 feet along the northern perimeter to 345 feet along the southern perimeter. The wall along the western perimeter is inclined at an angle of 75.6º to horizontal. The Garage is slab-on-grade addition (no basement) at finished floor elevation near El. 739 feet. The maximum interior column load is 235 kips and that for exterior (perimeter) column load is 120 kips under service case. The proposed bus garage building will provide 75 spaces for parking vehicles.

- The Garage building columns are planned to be supported by group of auger-cast piles with grade beam spanning between the columns. However, the aggregate pier foundation option (presented below) is also being considered for cost-comparison purposes.

- The roof framing plan of GDRTA Garage Building indicates presence of truss structures.

- Floor slab loads on the GDRTA Building and fuel station are in the range of 250 pounds per square foot (psf), and that on the bus wash is around 100 psf.

A grading plan has not been yet prepared at the time of this report addendum, but the grading is expected to be relatively minimal based on our understanding of the project, finished floor elevation of building(s), and the existing grades.

Based on LJB Plan Sheet C301, the proposed construction also involves a new parking lot, and a heavy-duty lane dedicated for fuel truck loop to the east side of the existing building at 601 Longworth Street. Additional borings, as part of supplemental study, will be required to provide design recommendations for pavement subgrade over that area.

2.0 DEMOLITION CONSIDERATIONS

As mentioned above, the project includes the complete demolition of the existing Building 705. The building demolition should include the complete removal of below-grade foundations and floor slabs. At this time, a foundation plan for the existing building has not been made available for review as part of the preparation of this addendum. If foundation plans and construction record for the existing building do not exist, we recommend that additional exploration (test pit or geophysical testing) be performed after the removal of superstructure to explore the type, size, and bearing depth of existing foundation. If the building demolition encounters deep foundations beneath the existing building, we should be contacted to provide recommendations for dealing with these obstructions. The pavement layers on parking/driveway areas should also be completely removed.

The LJB Plan showing existing conditions (Sheet C100 dated October 30, 2023) indicates the presence of several utilities that include underground electric, storm sewers, etc. running across the proposed construction area. All existing utilities within the footprint of the proposed building(s) footprint should be removed or relocated at least 5 feet beyond the perimeter of the proposed structure(s). The resulting excavations from the removal of utilities should be backfilled with structural fill, placed and compacted in accordance with the recommendations provided in our July 2021 Geotechnical Report. Alternatively, the utility excavations can be backfilled with controlled low-strength mortar or flowable fill.

Stripped asphalt, concrete pavement, construction rubbles and demolition debris are not suitable for reuse as structural fill. The proposed construction area for the new building and pavement subgrade should be thoroughly cleaned of any demolition debris or concrete remnants or any other deleterious materials before placing any new structural fill to bring the site to desired subgrade elevation.

We strongly recommend performing a pre-demolition and pre-construction survey of existing structures within 200 feet of the proposed building which could potentially be impacted during excavation/demolition and construction activities. Please note that the existing brick-façade building to the east side of the proposed garage building is less than 100 feet away from the proposed construction. Periodic monitoring of the existing structures should be performed during excavation and construction of the new structures. UES can assist with preconstruction surveys and periodic monitoring as part of additional scope.

Also, existing storm sewers run on the north and south sides of the proposed building at 705 Longworth Street. The existing storm sewers are about 15 to 17 feet below the existing ground. A camera inspection for existing sewer is recommended during pre-construction and post-construction survey for the documentation of existing conditions.

All exposed subgrades in the construction area should be proof rolled after performing required undercut and prior to placing any fill. Please refer to **Site Preparation and Earthwork Recommendations** for further information on proof-rolling and earthwork considerations.

3.0 AGGREGATE PIERS

With removal of the existing building and size of the proposed structure, it is our opinion that aggregate piers would also provide a suitable option to support the structure. The subsurface conditions generally consist of existing undocumented/uncontrolled fill and marginally competent alluvium in the upper 19.5 feet of boring. The upper 19.5 feet of soil were deemed to be unsuitable for direct support of foundations and floor slabs and hence recommended to be penetrated by foundation systems (auger-cast piles or helical piles) in our July 2021-Geotechnical Report. We recommend that shallow foundations and the slab supported over an aggregate pier system be considered for value engineering purposes.

Vibration during the installation of aggregate piers could be a concern for surrounding structures and existing utilities in the vicinity. We recommend a pre-construction survey and periodic monitoring of adjacent structures be performed as discussed above. Any concerns by the aggregate pier contractor related to vibrational disturbance/damage (such as to the storm sewer pipes) should be accounted for in the overall design layout. Alternative ground improvement methods in the vicinity of concern may be necessary.

Aggregate piers are a ground improvement technique that strengthens and stiffens the subsurface soils to support higher footing and floor slab bearing pressures that can be installed by either vertical compaction (Rammed Aggregate Piers) or vibration (Vibratory Piers). The rammed aggregate piers are typically constructed by first auguring 24- to 36-inch-diameter holes to pre-determined depths (typically in the range of 10 to 25 feet below grade) below the proposed foundation bearing elevations, and then by backfilling the holes with aggregate compacted in thin lifts. Compaction is achieved using high-frequency impact hammers that deliver a vertical ramming energy that densifies the aggregate and forces it laterally into the sidewalls of the hole. This action increases the lateral stress in the surrounding soil, thereby further stiffening the stabilized composite soil mass to increase the bearing capacity. For vibratory piers, a hollow mandrel, charged with crushed stone and the mandrel is vibrated into the ground and the stone is placed at the design depth in subsequent layers to create a similar lateral improvement as described above.

Aggregate pier construction may reduce time for foundation placement as compared to other deep foundation systems since conventional spread footings are placed directly on the reinforced soil mass, and there is no set up time for the aggregate pier elements.

The design and construction of this ground improvement technique is typically proprietary and should be performed by a qualified Design/Build Contractor using the subsurface information included in this report. Schaefer requested Geopier's local representative to review the subsurface conditions for the project and based upon their initial comments, an allowable bearing pressure of 5,000 psf should be able to be achieved using the aggregate piers; however, complete analysis and design should be performed by the ground improvement contractor. We recommend that aggregate piers for foundations penetrate the existing undocumented/uncontrolled fill and weak (soft to medium stiff) alluvium.

For this ground improvement system, we recommend that the following issues be considered during design prior to construction.

a. Specifications for aggregate pier foundation systems should be prepared by a qualified Design/Build Aggregate Pier Contractor (Aggregate Pier Contractor), including the layout and spacing of the aggregate piers.

b. The Aggregate Pier Contractor should coordinate with the Structural Engineer on the tolerable settlements of the proposed structure and design the aggregate piers to accommodate those tolerable settlements.

c. The selection of the aggregate pier installation method should be determined by the Aggregate Pier Contractor. The Aggregate Pier Contractor should have a plan if unanticipated encumbrances (e.g., boulders, cobbles, construction rubbles, demolition debris, etc.) are encountered that result in premature installation depths of the aggregate piers. If the Aggregate Pier Contractor indicates that complete removal of such obstacles is not required for their installation, we recommend that this material be removed within at least 3 feet of the floor slab subgrade.

d. The site plan should be reviewed for potential conflicts with the aggregate pier locations and the location of existing and proposed utilities, with respect to the influence zone of the reinforced soil.

e. The aggregate pier installations should be conducted under the observation of the Project Geotechnical Engineer, or representative thereof, to verify proper installation procedures and document observed changes in the explored soil conditions.

* * * *

We appreciate the opportunity to be of continuing service on this project. If you have any questions concerning the information contained herein, please do not hesitate to contact us.

Respectfully submitted,
GEOTECHNOLOGY LLC (DBA UES)

Suraj Khadka, PE
Project Manager

Joseph S. Burkhardt, PE
Geotechnical Department Manager

SK/JB:sk/jb

Copies submitted: Mr. David Henners, AIA, RIBA (Champlin Architecture)-email
Mr. Chris Buckreus, PE, SE (Schaefer)-email