ADDENDUM NO. 1 **08/09/2022**

RE: **Paintsville Pool Renovation**
Paintsville, KY
Project No. 21141

FROM: **Brandstetter Carroll Inc.**
2360 Chauvin Drive
Lexington, Kentucky 40517
Phone 859-268-1933
Fax 859-268-3341

TO: **Plan Holders**

This addendum forms a part of the Construction Documents and modifies the original bidding documents dated 07/27/2022. Each bidder shall acknowledge receipt of this addendum in the space provided on the Bid Form. Failure to do so may subject the Bidder to disqualification.

This Addendum consists of thirty-three (33) pages, plus Attachments: Geotechnical Report, ADD1.1, and ADD1.2.

GENERAL:

1. The Geotechnical Report has been included as an attachment.
2. The Prebid meeting and bid opening dates have been revised.

CHANGES TO SPECIFICATIONS:

1. Specification Section 001113 – Advertisement for Bids
   A. Section: 1.2 Bid Submittal and Opening
      1. **Bid Date: August 31st, 2022.**

   B. Section 1.4 Prebid Meeting
      1. The Prebid Meeting will be held on **August 24th, 2022.**
2. Specification Section 102113 – Toilet Compartments
   A. Section 2.1, Scranton Products; Hiny Hiders is an approved equal.
3. Specification Section 102800 – Toilet, Bath and Laundry Accessories
   A. Section 2.1, Saniflow Corp., Speedflow Plus and BabyMedi are approved equals.
4. Specification Section 105114 – Solid High-Density Polyethylene (HDPE) Plastic Lockers
   A. Section 2.1, Summit Lockers; Phenolic Lockers and Plastic Lockers are approved equals.
   B. Section 2.1, Scranton Products; Tufftec Lockers are an approved equal.
5. Specification Section 131660 – Water Features
   A. Section 2.1, Water Odyssey is an approved equal. Contractor to provide shop drawings.

CHANGES TO DRAWINGS:

1. CP-403 – Filter Building Structural & Architectural
    A. Detail C2 - Added powder coated angle braces behind perforated panels to detail
2. CP-404 – Filter Building Elevations
    A. Detail C1 - Added powder coated angle braces behind perforated panels.

ATTACHMENTS:

1. Geotechnical Report
2. ADD1.1
3. ADD1.2

END OF ADDENDUM NO. 1

GEOTECHNICAL ENGINEERING INVESTIGATION

PROPOSED POOL RENOVATIONS

412 MADISON AVENUE, PAINTSVILLE, KENTUCKY

ATLAS PROJECT NO. LOUGE22082

PREPARED FOR:

Brandstetter Carrol Inc
2360 Chauvin Drive
Lexington, KY 40517

PREPARED BY:

Atlas Technical Consultants LLC
2724 River Green Circle
Louisville, KY 40206

August 8, 2022

2724 River Green Circle
Louisville, KY 40206
(502) 722-1401 | oneatlas.com

August 8, 2022

MR. PHIL N. SCHILFFARTH
BRANDSTETTER CARROLL, INC.
2360 CHAUVIN DRIVE
LEXINGTON, KY 40517

Subject: **Geotechnical Engineering Investigation**
Proposed Pool Renovations
412 Madison Avenue, Paintsville, Kentucky
Atlas Project No. LOUGE22082

Dear Mr. Schilffarth:

Submitted herewith is the report of our geotechnical engineering investigation for the referenced project. This report contains the results of our field and laboratory testing program, an engineering interpretation of this data with respect to the available project characteristics, and recommendations to aid design and construction of the foundations and other earth-connected phases of this project. This revision has been issued based on requests by the design team and updated structural loading information.

We wish to remind you that we will store the samples for 30 days after which time they will be discarded unless you request otherwise. We appreciate the opportunity to be of service to you on this project. If we can be of any further assistance, or if you have any questions regarding this report, please do not hesitate to contact either of the undersigned.

Respectfully submitted,

Atlas Technical Consultants LLC

STATE OF KENTUCKY
RYAN CHRISTOPHER ORTIZ
33219
LICENSED PROFESSIONAL ENGINEER

Ryan C. Ortiz, PE
Project Geotechnical Engineer
Licensed Kentucky 33219

Travis Andres, P.E.
Senior Geotechnical Engineer
Licensed Kentucky 29429

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1. PURPOSE AND SCOPE

The purpose of this study was to determine the general subsurface conditions at the project site by excavating three (3) test locations via hand methods to evaluate the pool renovations, foundation concept, and design for the splash pad. This includes an evaluation of the site with respect to potential construction problems and recommendations relating to earthwork and quality control during construction. Hand methods were performed for this study, due to difficult access for typical drilling equipment.

All test locations were performed, where reasonably accessible, per the proposed test location plan provided in the Atlas proposal. The locations of which are noted in the Appendix.

2. PROJECT CHARACTERISTICS

Planned for design and construction is a splash pad at 412 Madison Avenue in Paintsville, Kentucky. The site is bounded by Madison Avenue, 7th Street, 8th Street, and a baseball field. The existing footprint of the proposed improvements contains an existing below ground swimming pool. The general location of the project site is shown on the Site Location Map (Figure 1 in the Appendix).

Based on review of the Category 3 Development Plan, the elevation of the slab adjacent the pool is about 610 feet. At the bottom of the pool, elevations range from 600 to 605 feet. A proposed splash pad is planned to replace the existing pool. The proposed splash pad is expected to be slab-on-grade construction. Foundations may be required for the splash pad or any associated structures. A grading plan was not available at the issuance of this report. We expect the splash pad will range 2 to 3 feet deep. We expect that up to 10 feet of fills may be required following demolition to achieve desired grades. Structural loading is not known at the issuance of this report, but we expect vertical and slab loading will be limited to 10 kips and 200 psf, respectively. The approximate location of the proposed building is shown on the Site Plan (Figure 2 in the Appendix).

Construction plans, structural loading information, and traffic loading information was not available at the issuance of this report. These details are critical for development of the design parameters and information provided in this report. Construction plans, structural loading, and traffic counts should be provided to Atlas once available. Atlas should be allowed to review this information and revise the report, if needed. Traffic counts will be beneficial to determine appropriate pavement section thicknesses

3. GENERAL SUBSURFACE CONDITIONS

3.1 Site Geology

A review of Kentucky Geological Survey (KGS) publicly available mapping service indicates the site is underlain by alluvium apart of the Paintsville Quadrangle. The alluvium deposits consist of sand, silty sand, sandy silt, and minor quantities of coal and gravel. Based on local experience, we expect this formation is underlain by sandstone, shale, and coal of the Pikeville formation.

3.2 Subsurface Soil Conditions

Due to limited access within the proposed project area and existing structures and utilities, the general subsurface conditions were investigated by hand excavating three (3) test locations, with refusal ranging from 2.5 to 7.4 feet below existing grade (BEG) at the approximate locations shown on the Site Plan (Figure 2 in the Appendix).

The subsurface conditions disclosed by the field investigation are summarized in the following paragraphs. Detailed descriptions of the subsurface conditions encountered in each test boring are presented on the Test Boring Logs in the Appendix. The letters in parentheses following the soil descriptions are the soil classifications in accordance with the Unified Soil Classification System. It should be noted that the stratification lines shown on the soil boring logs represent approximate transitions between material types. In-situ stratum changes could occur gradually or at slightly different depths

At the ground surface, test borings encountered concrete surface materials, with thicknesses ranging 3.5 to 7.5 inches. Underlying the surface materials, variable interbedded material types were generally encountered including Lean Clays (CL) with variable amounts of silt and sand or Poorly Graded Sand (SP). Based on DCP and hand penetrometer testing, these materials are interpreted as very soft to medium stiff, with stiff soils encountered only at Boring B-3. Refusal was encountered at depths of 2.5 to 7.4 feet BEG. Refusal may have encountered on bedrock, rock fragments, or other obstructions included in possible fill materials.

The consistencies of the cohesive soils and relatively densities of granular soils as described above and on the boring logs were estimated based on the results of the standard penetration test (ASTM D1586).

3.3 Ground Water

Groundwater level observations were made during drilling operations. Free water was encountered at Boring B-2A at about 2 feet during excavating. The remaining borings did not encounter free water for the short time they were allowed to remain open, and were likely terminated shallower than the groundwater table. Free water seepage or perched water encountered may be encountered wet seasons.

Because of the complexity of the hydrogeology within the cohesive soils that underlies this site, it is not possible to determine an accurate ground water level using the conventional test methods used for this investigation. In order to better define the ground water regime at this site, an extensive ground water monitoring program would be required at numerous locations for an extended period of time. The ground water flow patterns can also be changed as a result of construction and changes in site grading and therefore higher ground water levels could be encountered in the future.

4. DESIGN RECOMMENDATIONS

The following design recommendations have been developed on the basis of the previously described project characteristics (Section 2.0) and subsurface conditions (Section 3.0). If there is any change in these project criteria, including project location on the site, a review should be made by this office.

4.1 General Construction Considerations

Based upon the results of the subsurface investigation performed at this site in conjunction with the assumed finished floor elevations (as discussed in Section 2.0 of this report), the most feasible and economical foundation system for support of the proposed structure appears to be conventional shallow spread footings bearing on stiff natural soils, well-compacted engineered fill materials that are placed over these natural materials. Design recommendations for foundations are provided in Section 4.2.

Careful evaluation of the foundation bearing materials will be required at the time of construction in order to identify uncontrolled fill materials that must be removed from beneath the foundations and replaced with engineered fill. It is important that the observation and evaluation methods outlined in Section 5.3 be implemented and that any soft natural soils, old fill materials, and remnants from previous construction revealed by such observations and evaluations be removed and replaced.

4.1.1 Unsuitable Soils

Unsuitable soils and bearing conditions were observed in several of our borings. These included soft to medium stiff soils (evidenced by SPT N-values) and potential variable fill soils. Due to highly variable soils encountered in this study, they should be expected across the site. Soft to medium stiff soils at the foundation or slab bearing elevations may require undercutting to stiff soils and replaced with engineered fill or lean concrete. Where soft soils were encountered, such as at Borings B-1 and B-2A, undercuts may be required of soft native soils. Soils underneath the slab elevations, or prior to fill placement should pass a proofroll observed by the geotechnical engineer's representative or probing and penetrometer testing, if possible. Additional details for site development with respect to soft soils are provided in 5.1 Site Preparation.

4.1.2 Existing Underground Structures

The site was a previous underground recreational pool, with maximum depth of 10 feet. The existing pool cannot be left in place, without some remediation activity. Options for remediation of the pool are as follows, and should be selected based on owner/operator risk tolerance.

- Full removal of the existing structures and backfill with compacted structural fill
- Full removal of the walls, puncturing of the bottom, and backfill with compacted structural fill
- Partial removal of the walls to 3 feet below bottom of proposed new structures, puncture of the bottom, and backfill with compacted structural fill

To completely alleviate risk of poor performance due to existing structures, along with possible poor past construction practices, the existing pool should be removed. If some risk of poor performance is acceptable to the owner/operator, then the pool walls may be excavated to at least 3 feet below bottom of structures, and the pool bottom should be punctured on a 5 feet grid spacing to facilitate drainage. Backfill materials should consist of cohesive soils or KYTC Dense Graded Aggregate.

4.1.3 Foundation Considerations

For foundation support, based on conditions encountered at the borings, modest bearing capacities can be achieved with shallow remediation effort to improve bearing characteristics and limit settlements to tolerable levels. Shallow remediation efforts may include undercutting and replacement to depths below foundation or slab bearing elevations.

4.2 Foundations

Our findings show that the proposed building can be supported on conventional shallow spread footings or mat foundation bearing on stiff natural soils provided that any unsuitable materials (such as soft natural soil or old fill) are removed, where encountered, and replaced with engineered fill. It will be necessary to remove any unsuitable materials (including pockets of soft natural soils, all old uncontrolled fill materials judged unsuitable or remnants from previous construction) where encountered below the nominal spread footing bearing elevations in some isolated cases and to re-establish the nominal design bearing level using engineered fill, flowable fill, or lean concrete.

Footings bearing on stiff natural soils or on engineered fill that is placed over stiff natural soils can be designed for **a net allowable bearing pressure of 1,500 psf.** Mat foundations bearing on stiff natural soils or on engineered fill that is placed over stiff natural soils can be designed for **a soil modulus of 10 pci.** All unsuitable materials below foundations should be identified, removed and replaced with well-compacted engineered fill or lean concrete as described in Section 5.2. It is important that the soil at the base of and below each spread footing excavation be carefully observed and evaluated as described in Section 5.3 to determine whether the actual bearing materials are consistent with those upon which the recommendations are based. It is recommended that the contract documents include provisions for the removal and replacement of unsuitable materials as determined to be necessary based on field observations.

In using the net allowable bearing pressure, the weight of the footing and backfill over the footing including the weight of the floor slab need not be considered; hence, only loads applied at or above the finished floor need to be used for dimensioning the footings. Wall footings should be at least 1.5 feet wide and column footings should be at least 2.5 feet wide for bearing capacity considerations. The Kentucky Building Code requires 33 inches of foundation embedment below the exterior grade for Pike County.

Provided that the footings are designed as prescribed herein and the footing excavations are observed and evaluated as outlined in Section 5.3, it is estimated that the total and differential foundation settlements should not exceed about 1 and ¾ inches, respectively. Careful field control will contribute substantially to minimizing the settlements.

Uplift forces on the spread footings can be resisted by the weight of the footings and the soil material that is placed over the footings. It is recommended that the soil weight considered to resist uplift loads be limited to that immediately above and within the perimeter of the footings (unless a much higher factor of safety is used). A total soil unit weight of 120 lbs/cu.ft can be used for the backfill material placed above the footings, provided it is compacted as recommended in Section 5.2. It is also recommended that a factor of safety of at least 1.3 be used for calculating uplift resistance from the footings (provided only the weight of the footing and the soil immediately above it are used to resist uplift forces).

Lateral forces on a spread footing can be resisted by the passive lateral earth pressure against the side of the footing and by friction between the soil and the base of the footing. A uniform allowable passive pressure of 350 lbs/sq.ft can be used for that portion of the footing that is below a depth of 2 ft below the final exterior grade (no portion of the footing above this depth should use for lateral resistance). An allowable coefficient of friction between the base of the footing and the underlying soil of 0.2 can be used in conjunction with the minimum downward load on the base of the footing.

Care must be exercised when excavating near the existing streets, utilities, etc. to protect the integrity of the existing foundations, and other features. Bracing or underpinning may be required where it is necessary to excavate below the bottom elevation of the existing streets, utilities, etc

4.3 Floor Slabs

Floor slabs can be supported on stiff, low-plasticity natural soils or on new compacted structural fill. It is recommended that the slabs be supported on a minimum 6-inch thick layer of granular material such as crushed stone. This is to help equalize moisture conditions beneath the slab and provide uniform support of the slab, as well as protect the subgrade through construction. Provided that a minimum of 6 inches of crushed stone is placed beneath the slabs, a modulus of subgrade reaction of 100 pci can be used for design of the floor slabs. Where soft soils or existing fill is encountered, these should be undercut a maximum 3 feet and replaced with new engineered fill.

We have recommended in this report that a minimum 6-inch-thick layer of granular base be used to support construction equipment prior to concrete placement. However, a minimum of 4 inch may be considered to be used provided no construction traffic is allowed on the floor slab base prior to concrete slab placement. Based on our experience, limiting traffic on the building pad stone is difficult to achieve.

4.4 Lateral Earth Pressures

Retaining or subsurface walls can be constructed as flexible walls subject to deflection or as self-supported cantilever walls restrained from movement. Flexible walls and walls that are not restrained from movement (i.e., walls that are free to rotate or deflect sufficiently to develop the active lateral earth pressure condition) can be designed using the active lateral earth pressure case using a coefficient of active lateral earth pressure, Ka. Walls that are restrained against lateral movement by fixed slabs, other walls, or bracing should be designed for the at-rest lateral earth pressure condition using an at-rest lateral earth pressure coefficient, Ko. Should wall backfill be placed before wall bracing is constructed, it may be necessary to provide temporary bracing or the walls should be designed for the active earth pressure condition as self-supported cantilever walls. All earth supporting walls should be designed with sufficient exterior drainage to relieve hydrostatic pressure that may build up on the walls. It is recommended the space between walls and retained earth be backfilled with granular material.

Heavy equipment should not be used for compaction of backfill behind walls. Crushed stone backfill immediately behind walls should be consolidated using a vibratory plate compactor. Recommendations for both crushed stone and cohesive backfill options are presented below. Additionally, below-grade retaining walls should be sealed to prevent migration of free water through cracks, joints into below grade cavities. Three alternative wall backfill cases are presented below.

4.4.1 Granular Backfill Behind Structures

For this case, the backfill material against below-grade walls should consist of consolidated, well-graded, free-draining granular material.

- The granular material should preferably be "SP", "SW", or "GW" as classified by the USCS so that it will be clean (i.e. less than 5% fines and all material passing the 1.5 inch sieve), free draining, and exhibit a shear resistance angle of 34 degrees or more (e.g., KYTC No. 57 crushed limestone)
- To utilize the following granular material earth pressure values, the granular material must occupy a triangular shaped minimum backfill zone. The minimum zone should start at the base of the wall, at the outside face of the footing. At the top of the backfill, the zone should extend from the edge of footing a distance equal to three-fifths of the backfill height.
- The backfill zone should be drained using a perforated pipe placed at the base of the footing and the pipe should drain either by gravity or to a sump system to collect and remove accumulated water. The granular backfill zone should be separated from natural soil by a non-woven, geotextile filter fabric to prevent clogging of the pervious backfill.
- A minimum 2 feet thickness of low plasticity clay on top of the granular wall backfill material should be provided where the backfill material will be exposed to the weather. This low plasticity clay "cap" is recommended to minimize infiltration and accumulation of surface runoff water behind the walls. As such, the cap should be

sloped away from the structure at a minimum 2-percent slope, and grading should be designed that keeps water from ponding over the cap.

- The following table presents granular backfill, lateral earth pressure design parameters for Equivalent Hydrostatic Pressures (EHP) and Earth Pressure coefficients. The values given are based upon four assumptions: 1) the backfill surface is level; 2) the backfill is drained; 3) the zone of backfill conforms to the minimum zone size described above; and 4) no surcharge loading is applied or imparted to the backfill.

Granular Material Equivalent Hydrostatic Pressures (EHP) and Earth Pressure Coefficients

Condition	EHP (pcf)	Coefficients
Active*	36	K_a = 0.28
At-Rest	57	K_o = 0.44

*Assumes that the wall is not braced and is free to rotate or deflect sufficiently to develop the active lateral earth pressure condition

4.4.2 Cohesive Backfill Behind Structures

As an alternative, backfill against below-grade walls may also be constructed using cohesive backfill materials.

- Cohesive fill material should conform to recommendations presented in previous sections of this report. The Plasticity Index of the backfill material, as measured by Atterberg limits testing, should be less than 20.
- Any processed, forcibly weathered bedrock for this project, should be considered as cohesive backfill.
- To provide drainage behind the wall, construct a vertical section of crushed stone at least 18 inches wide behind the wall with a perforated drain pipe located at the foundation level. The granular wall backfill material should be capped with a two-foot layer of low plasticity clay to minimize infiltration of surface water runoff behind below grade walls. As with any drainage system, accumulated water will need to be conveyed from behind the wall through gravity drainage or a sump system as described above.
- Calculations were performed assuming an internal friction angle of 25 degrees and a soil unit weight of 130 pcf for the cohesive soil backfill case.
- The following table presents cohesive backfill earth pressure design parameters for Equivalent Hydrostatic Pressures (EHP) and Earth Pressure Coefficients. The table assumes the backfill surface is level, the backfill is drained and no surcharge load is applied or imparted to the backfill.

In-situ Cohesive Soil Equivalent Hydrostatic Pressures (EHP) and Earth Pressure Coefficients

Condition	EHP (pcf)	Coefficients
Active*	53	$K_a = 0.41$
At-Rest	75	$K_o = 0.55$

* Assumes that the wall is not braced and is free to rotate or deflect sufficiently to develop the active lateral earth pressure condition

4.5 Seismic Site Classification

A seismic site classification was performed and design spectral responses were calculated using USGS Seismic Design Maps. Seismic design parameters were calculated based upon the observed subsurface soil profiles and a maximum approximate boring depth of 30 feet. We have assumed soil conditions at boring termination extends to a depth of 100 feet. Recommended seismic design parameters follow:

Table 1: Seismic Site Design Parameters

Seismic Design Parameter	Parameter Value
Seismic Site Classification	C
Design Spectral Response at Short Periods (SD_S)	0.140
Design Spectral Response at 1-Second Periods (SD_1)	0.092

4.6 Site Grading and Drainage

Proper surface and subgrade drainage should be provided at the site to minimize any increase in moisture content of the foundation soils. Pavement subgrades should be sloped to drain and stone base underlying pavement sections should be daylighted (exposed and draining) where possible at the edge of pavements. The exterior grade should be sloped away from the structures to prevent ponding of water. Any roof drains or down spouts should be channeled or piped well away from the structure.

5. GENERAL CONSTRUCTION PROCEDURES AND RECOMMENDATIONS

Since this investigation identified actual subsurface conditions only at the test boring locations, it was necessary for our geotechnical engineers to extrapolate these conditions in order to characterize the entire project site. Even under the best of circumstances, the conditions encountered during construction should be expected to vary somewhat from the test boring results and may, in the extreme case, differ to the extent that modifications to the foundation recommendations become necessary. Therefore, we recommend that Atlas be retained as geotechnical consultant throughout the earth-related phases of this project to correlate actual soil conditions with test boring data, identify variations, conduct additional tests that may be needed and recommend solutions to earth-related problems that may develop.

5.1 Site Preparation

All areas that will support slabs and pavements should be properly prepared. After rough grade has been established and prior to placement of fill, the exposed subgrade should be carefully observed by the geotechnical engineer, or a qualified soils technician working under the direction of the geotechnical engineer, by probing and testing as needed. Any organic material still in place, frozen, wet, soft or loose soil, uncontrolled fill, existing demolition debris and pavements, foundation remnants, utilities, and other undesirable materials should be removed. The exposed subgrade should be evaluated by proofrolling with suitable equipment to check for pockets of soft material hidden beneath a thin crust of better soil. Any unsuitable materials thus exposed should be removed and replaced with well-compacted, engineered fill as outlined in Section 5.2.

It is important that positive surface drainage be established at the beginning of the earthwork operations and be maintained throughout the project. Surface water must not be allowed to pond. Furthermore, compaction and sealing of the subgrade surface is important when precipitation is expected. The site storm drainage elements (i.e., catch basins, pipes, manholes, etc.) should be installed as early as possible, which will aid in control of surface and ground water.

Care should be exercised during the grading operations at the site. Due to the nature of the near surface soils, the traffic of construction equipment may create pumping and general deterioration of the shallower soils, especially if excess surface water is present. The grading, therefore, should be done during a dry season, if at all possible. Based on our experience on other nearby sites, it is likely that the subgrade soils in some areas will be wet and soft when exposed. The extent to which yielding subgrade may be a problem is difficult to predict beforehand since it is dependent upon several factors including seasonal conditions, precipitation, cut depths, sequencing and scheduling of the earthwork, surface and subsurface drainage measures, the weight and traffic patterns of construction equipment, etc. Therefore, it is suggested that provisions be made in the contract documents for subgrade improvements to be used where determined to be necessary in the field at the time of construction.

It may be possible to improve or stabilize the subgrade soils in the areas that are found to be excessively wet, soft or yielding at the time of construction, by discing, aerating and recompacting. However, this will require a combination of time to allow for working the soils, favorable weather conditions for drying and firmer soils at shallow depth below the yielding soils in order to be successful. If site grading operations are planned through the winter months, subgrade stabilization is expected to be required as part of fill construction to aid in moisture conditioning during fill construction through the seasonably wetter winter months.

If it is not possible to improve the subgrade soils in this manner because of weather conditions, scheduling or other constraints or site conditions (which is most often the case); it is recommended that the subgrade soils be improved or modified using either chemical stabilization (i.e., cement), mechanical stabilization (i.e., a geogrid with additional crushed limestone placed over the subgrade), or removal of the unsuitable soils and replacement with crushed limestone or engineered soil fill. The best method for stabilizing the subgrade should be determined in the field at the time of construction based upon the actual field conditions in conjunction with the specific soil type encountered at the locations requiring stabilization, the size of the areas requiring stabilization and the construction schedule. For soil conditions such as those at this site, chemical stabilization is often the most cost effective subgrade stabilization method particularly when large areas require stabilization. The chemical stabilization is typically performed in a single 14-16 inches thick lift and should be performed by a specialty contractor who has the necessary equipment and experience in the application of chemical stabilization methods. The site soils should be evaluated by the contractor to identify the most suitable approach. Preliminarily, for budgeting purposes, it is recommended that a minimum quantity of cement added by weight of 3.5% be considered.

5.2 Fill Compaction

5.2.1 Earthen Fill

All engineered fill beneath footings, floor slabs, and pavements should be compacted to a dry density of at least 98 percent of the standard Proctor maximum dry density (ASTM D-698). For soil, the compaction should be accomplished by placing the fill in about 8 inches (or less) loose lifts and mechanically compacting each lift to at least the specified minimum dry density.

It is recommended that only well-graded granular material, such as pit-run sand, gravel, or KYTC DGA crushed stone or lean concrete be used to fill undercut excavations beneath footings and other excavations of limited lateral dimensions where proper compaction of cohesive materials is difficult and compaction can only be accomplished with hand-held vibratory equipment.

Soil fill materials should be compacted using a non-vibratory sheeps-foot roller and aggregate fill materials should be compacted using a vibratory smooth-drum roller or as judged acceptable by the geotechnical engineer. Field density tests should be performed on each lift as necessary to insure that adequate moisture conditioning and compaction is being achieved. Both low and highly plastic clay soils are present on the site and are considered suitable as general fill material provided the recommendations provided in Sections 4 and 5 are considered. The need for some aeration or chemical modification of the clayey soils, especially moderate to high plasticity clays should be expected before they can be placed and compacted to the specified density.

Prior to beginning fill construction, we recommend samples of proposed borrow materials be collected for standard Proctor testing. The following criteria are recommended where soil material is utilized for structural fill:

- Soils referred to as ‘low volume change” in this report have a Liquid Limit less than 50 percent.
- Limit maximum particle sizes to 4-inches (in the largest dimension) and less than 3 percent organic material by weight.
- Maintain the moisture content of the fill soils to within ±2 percentage points of the soils' optimum moisture content.

- Perform one in-place density test in every 5,000 square feet for each one-foot-thick fill layer, with a minimum of two tests per lift.
- Retain the geotechnical engineer to observe, document and test fill placement and compaction operations.
- Provide and maintain efficient drainage of building and pavement subgrades both during and after construction to prevent ponding of water and to promote rapid and efficient surface drainage.
- Maintain positive surface drainage to prevent water from ponding on surfaces during all earthwork operations.
- Roll fill surfaces with a rubber-tired or steel-drummed roller prior to precipitation events to improve surface runoff if precipitation is expected.
- Contact the geotechnical engineer should the subgrade soils become excessively wet, dry, or frozen.

5.2.2 Processed Demolition Debris Fill

Demolition debris is expected to be generated during site demolition activities of existing structure foundations and slabs. The use of such materials as new engineered fill is being considered. Materials are expected to consist of mostly reinforced concrete. Properly processed demolition debris may be used as new structural fill, provided the following considerations and recommendations presented below are adhered to.

- The material should be free of materials that may impede placement and compaction, such as reinforcing steel. In any instance of reuse of concrete as new engineered fill, the materials should be processed in such a manner to sufficiently remove any steel reinforcement and deleterious debris.
- All deleterious materials should be removed, including organics, wood, tree limbs, or any other materials. Any materials encountered should be observed and approved by a qualified engineer.
- Soils should not be included the crushed demolition debris fill.
- Processed and approved demolition debris fill should not be placed within 2 feet of proposed finish grade in floor slab and pavement areas.
- The material should be placed and compacted in a uniform manner that minimizes the formation of void spaces around the particles.
- It is likely that a crusher will be required to properly size and segregate the materials.

- The product of processing should produce a densely graded material with sufficient fines and top size not exceeding 4 inches.
- The crushed demolition debris should generate sufficient fines, and no more than 14 percent fines, similarly to KYTC dense grade aggregate or crushed stone base gradations.
- Material placed within the building or equipment pad should extend a minimum 5 feet outside of the planned building extents to ensure adequate compaction of the lift edges and to provide uniform support of overlying and adjacent foundations.
- Each lift of fill should be placed in lifts not to exceed 8 inches. The engineer may allow the thickness of material and the embankment lifts to increase, as necessary and deemed acceptable, due to the nature of the material.
- Each lift should be pushed/distributed and tracked into place thoroughly using a dozer to minimize voids, pockets, and bridging. A vibratory smooth drum roller is recommended for consolidation of each lift.
- Processed demolition debris fill should not be used within 6 inches of planned structures. The recommended mineral stone base section (ie. KYTC DGA or CSB), as designed, should not be replaced with processed demolition debris.
- The operations should be observed by a representative of Atlas to evaluate field conditions and work effort with respect to these recommendations.
- Fill construction using soil or aggregate fill per project specifications may proceed atop the placed crushed debris fill upon acceptance by the geotechnical engineer.

Increased particle sizes and use of demolition debris does carry increased risk, when compared to more common materials such as suitable native soils or KYTC Crushed Stone sizes. Increased particle sizes in the engineered fill may result in increased void spaces, increasing likelihood for migration of overlying soils, resulting in apparent settlement and poor performance. This can be easily managed by proper oversight and control, and ensuring the crushed demolition debris is adequately mixed. Potential risk can be reduced by not including processed demolition debris fill within 2 feet of structures.

5.3 Foundation Excavation Observations

The soil at the base of each foundation excavation should be observed and evaluated by a geotechnical engineer or a qualified soils technician working under the direction of the geotechnical engineer to insure that any remnants from previous construction, old fill material, soft natural soil and any otherwise undesirable material is identified and removed at footing locations and that the footing will bear on satisfactory material. At the time of such inspection, it will be necessary to make hand auger borings or use a hand penetration device in the base of the foundation excavation to determine whether the soils below the base are satisfactory for foundation support. The necessary depth of penetration will be established during inspection.

Where undercutting is required to remove unsuitable materials beneath footings, the proposed footing bearing elevation may be re-established by backfilling after all undesirable materials have been removed. The undercut excavation beneath each footing should extend to suitable bearing soils. The dimensions of the excavation base should be determined by imaginary planes extending downward and outward on a 2 (vertical) to 1 (horizontal) slope from the base perimeter of the footing. The entire excavation should then be refilled with engineered fill. The engineered fill should be limited to low plasticity site soils or well-graded crushed stone (e.g., KYTC DGA) compacted to the minimum dry density recommended in Section 5.2; or lean concrete or cementitious flowable fill may be used. Special care should be exercised to remove any sloughed, loose or soft materials near the base of the excavation slopes. In addition, special care should be taken to "tie-in" the compacted fill with the excavation slopes with benches as necessary. This is to ensure that no pockets of loose or soft materials will be left in place along the excavation slopes below the foundation bearing level.

Soils exposed in the bases of all satisfactory foundation excavations should be protected against any detrimental change in condition such as from disturbance, rain and freezing. Surface run-off water should be drained away from the excavation and not allowed to pond. If possible, all footing concrete should be placed the same day the excavation is made. If this is not practical, the footing excavations should be adequately protected. It is recommended that a concrete “mud mat” be placed at the base of the footing excavations to protect the subgrade soils from deterioration due to seepage of ground water, surface water, etc., and to aid in the proper placement of reinforcing steel.

5.4 Construction Dewatering

Perched water and groundwater was encountered at the site, and this exploration was completed during the dry summer months. Elevated groundwater levels and more widespread perched and groundwater can be anticipated during wet times of year, or following precipitation events. However, depending on the seasonal conditions, some seepage into excavations may be experienced. It is anticipated that such seepage can be handled by conventional dewatering methods such as by pumping from sumps. However, in cases where a saturated layer is encountered in the base or sidewall of the excavation, it will not be possible to pump water directly from the base of the excavation without causing deterioration of the subgrade soil. In this case, it will be necessary to pump from a sump located adjacent to the excavation or to depress the ground water using wells or well points. The best dewatering system for each case must be determined at the time of construction based upon actual field conditions. Dewatering is not expected to be required.

6. FIELD INVESTIGATION

Three (3) borings were advanced at the approximate locations shown on the Site Plan (Figure 2 in the Appendix). The test borings were extended to select depths. Split-barrel samples were obtained within the overburden soils by the Standard Penetration Test procedures (ASTM D-1586) at 2.5 to 5 ft intervals.

Logs of all borings, which show visual descriptions of all soil strata encountered using the Unified Soil Classification System, have been included in numerical order in the Appendix. Ground water observations, sampling information and other pertinent field data and observations are also included. In addition, a "Legend to Classification and Symbols" document defining the terms and symbols used on the logs is provided immediately following the boring logs.

7. LABORATORY INVESTIGATION

The disturbed samples were inspected and classified in accordance with the Unified Soil Classification System and the boring logs were edited as necessary. To aid in classifying the soils and to determine general soil characteristics, select laboratory testing methods were performed on selected samples. The results of these tests are included on the test boring logs and summary sheets in the Appendix.

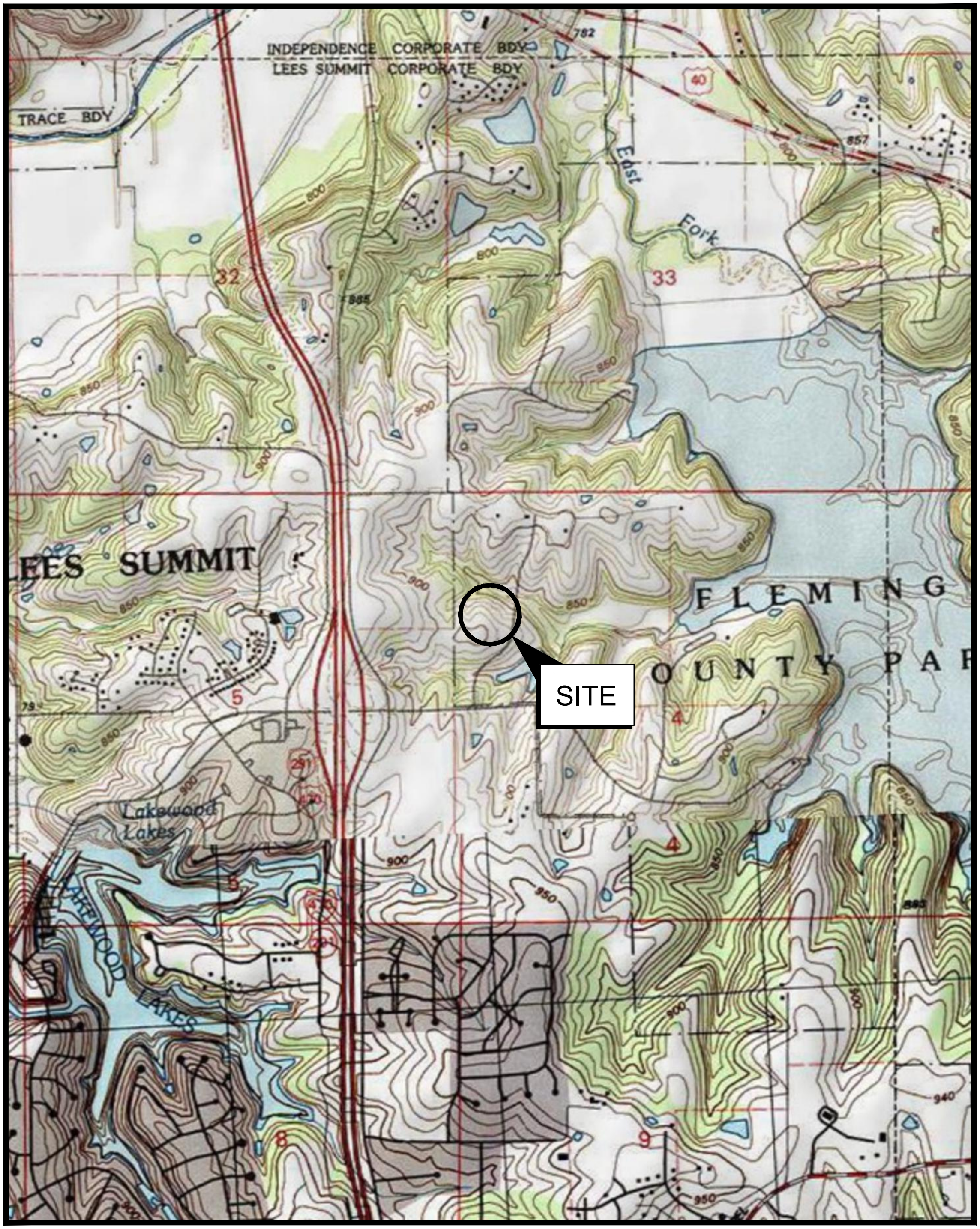
8. LIMITATIONS OF STUDY

An inherent limitation of any geotechnical engineering study is that conclusions must be drawn on the basis of data collected at a limited number of discrete locations. The recommendations provided in this report were developed from the information obtained from the test borings that depict subsurface conditions only at these specific locations and at the particular time designated on the logs. Soil conditions at other locations may differ from conditions occurring at these boring locations. The nature and extent of variations between the borings may not become evident until the course of construction. If variations then appear evident, it will be necessary to re-evaluate the recommendations of this report after performing on-site observations during the excavation period and noting the characteristics of any variation.

Our professional services have been performed, our findings obtained and our recommendations prepared in accordance with generally accepted geotechnical engineering principles and practices. This warranty is in lieu of all other warranties either express or implied. This company is not responsible for the independent conclusions, opinions or recommendations made by others based on the field exploration and laboratory test data presented in this report.

The scope of our services does not include any environmental assessment or investigation for the presence or absence of hazardous or toxic materials in the soil, ground water or surface water within or beyond the site studied.

Atlas assumes no responsibility for any construction procedures, temporary excavations (including utility trenches), temporary dewatering or site safety during or after construction. The contractor will be solely responsible for all construction procedures, construction means and methods, construction sequencing and for safety measures during construction. All applicable federal, state and local laws and regulations regarding construction safety must be followed, including current Occupational Safety and Health Administration (OSHA) Regulations including OSHA 29 CFR Part 1926 “Safety and Health Regulations for Construction”, Subpart P “Excavations”, and/or successor regulations. The Contractor is solely responsible for designing and constructing stable, temporary excavations and should brace, shore, slope, or bench the sides of the excavations as necessary to maintain stability of the excavation sides and bottom.



N

H:\2022\1 OTHER OFFICES\KENTUCKY\PAINTSVILLE POOL\LOUGE22082-VIC.DWG, FIG1

VICINITY MAP

PROPOSED SPLASHED PAD
412 MADISON AVENUE
PAINTSVILLE, KENTUCKY

Project Number: LOUGE22082		Drn. By: DH
Date: 07/28/2022	Scale: 1":2,000'	Ckd. By: TA
ATLAS		1

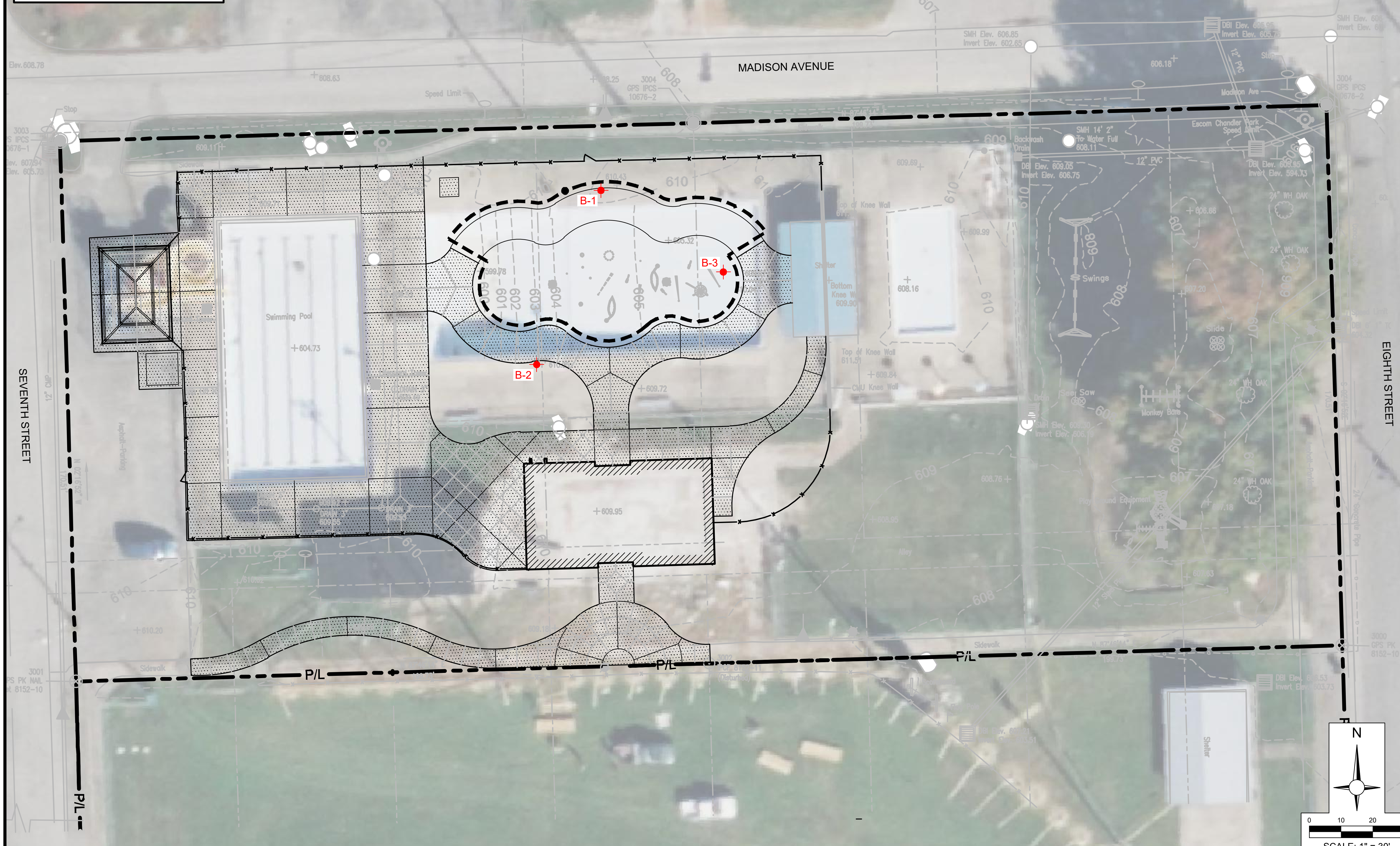
LEGEND:

B-1 PROPOSED TEST BORING

Boring Identification

NOTE:
BORING COORDINATES OBTAINED USING A RECREATIONAL GRADE GPS WITH ACCURACY OF ABOUT 20 FT. ELEVATIONS OBTAINED FROM GRADING PLAN PROVIDED BY BRANDSTETTER CARROLL INC.

Boring ID	Latitude (deg)	Longitude (deg)	Elevation (ft)
B-1	37.809007	-82.804557	610
B-2	37.808858	-82.804633	610
B-3	37.808933	-82.804426	605



ATLAS

TEST LOCATION PLAN

PROPOSED SPLASH PAD
412 MADISON AVENUE
PAINTSVILLE, KENTUCKY

Project Number: LOUGE22082	
Date: 07/28/2022	
Drn. By: DH	Ckd. By: TA
Scale: AS SHOWN	
Figure: 2	

H:\2022\1 OTHER OFFICES\KENTUCKY\PAINTSVILLE POOL\LOUGE22082-SITE.DWG, FIG2

Atlas Technical Consultants
2724 River Green Circle
Louisville, KY 40206
(502) 722-1401
Fax (502) 267-4072

TEST BORING LOG

CLIENT **Brandstetter Carroll INC**
PROJECT NAME **Proposed Paintsville Pool**
PROJECT LOCATION **412 Madison Avenue**
Paintsville, Kentucky

BORING # **B-1**
JOB # **LOUGE22082**
DRAWN BY **Z. Nichols**
APPROVED BY **T. Andres**

DRILLING and SAMPLING INFORMATION

Date Started	**7/6/22**	Hammer Wt.	**140** lbs.
Date Completed	**7/6/22**	Hammer Drop	**30** in.
Drill Foreman	**Z. Nichols**	Spoon Sampler OD	**2** in.
Inspector	**P. Presnell**	Rock Core Dia.	**2** in.
Boring Method	**Hand auger, DCP**	Shelby Tube OD	**3** in.

TEST DATA

SOIL CLASSIFICATION
SURFACE ELEVATION (ft): 610.0
Latitude (deg): 37.809007, Longitude (deg): -82.804557

Soil Classification	Stratum Depth	Depth Scale	Sample No.	Sample Type	Sampler Graphics / Recovery Graphics	Groundwater	Standard Penetration Test N-Value (blows/foot)	Qu-psi Unconfined Compressive Strength	PP-tsf Pocket Penetrometer	Moisture Content %	Liquid Limit (LL)	Plastic Limit (PL)	Percent Passing #200 Sieve	Remarks
CONCRETE 7 inches														
POORLY GRADED SAND (SP), Brown DCP at 0.6 feet: 1-1-1	0.6		1	CU						8.6				
LEAN CLAY (CL), Light brown and Reddish brown - DCP at 2 feet: 2-3-3	1.5		2	CU						24.3				
POORLY GRADED SAND (SP), Brown	2.5													
LEAN CLAY (CL), with sand, trace river gravel, Light reddish brown and Gray - DCP at 3.5 feet: 2-3-4	3.0		3	CU						25.8				
- Pocket Pen at 4.5 feet: 0-0.25 tsf			4	CU						24.6				
Hand Auger refusal on suspected bedrock or concrete foundation at 5 feet	5.0	5												

Sample Type
SPT - Standard Penetration Test
SS - Driven Split Spoon
ST - Pressed Shelby Tube
CA - Continuous Flight Auger
RC - Rock Core
CU - Cuttings
CT - Continuous Tube

Depth to Groundwater
Noted on Drilling Tools -- ft
At Completion (in augers) -- ft
At Completion (open hole) -- ft
After -- hours -- ft
After -- hours -- ft
Cave Depth -- ft

Boring Method
HSA - Hollow Stem Augers
CFA - Continuous Flight Augers
DC - Driving Casing
MD - Mud Drilling

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TEST BORING LOG

CLIENT **Brandstetter Carroll INC**
PROJECT NAME **Proposed Paintsville Pool**
PROJECT LOCATION **412 Madison Avenue**
Paintsville, Kentucky

BORING # **B-2**
JOB # **LOUGE22082**
DRAWN BY **Z. Nichols**
APPROVED BY **T. Andres**

DRILLING and SAMPLING INFORMATION

Date Started **7/6/22**
Date Completed **7/6/22**
Drill Foreman **Z. Nichols**
Inspector **P. Presnell**
Boring Method **Hand auger, DCP**

Hammer Wt. **140** lbs.
Hammer Drop **30** in.
Spoon Sampler OD **2** in.
Rock Core Dia. **2** in.
Shelby Tube OD **3** in.

TEST DATA

SOIL CLASSIFICATION
SURFACE ELEVATION (ft): 610.0
Latitude (deg): 37.808858, Longitude (deg): -82.804633

Soil Classification	Stratum Depth	Depth Scale	Sample No.	Sample Type	Sampler Graphics Recovery Graphics	Groundwater	Standard Penetration Test N-Value (blows/foot)	Qu-psi Unconfined Compressive Strength	PP-tsf Pocket Penetrometer	Moisture Content %	Liquid Limit (LL)	Plastic Limit (PL)	Percent Passing #200 Sieve	Remarks
CONCRETE 3.5 inches	0.3													
POORLY GRADED SAND (SP), Light brown fine to medium grained, DCP at 0.3 feet: 2-1-2	1.0		1	CU						15.4				
Hand Auger refusal on Scrap Metal encountered at 1 feet														

Sample Type
SPT - Standard Penetration Test
SS - Driven Split Spoon
ST - Pressed Shelby Tube
CA - Continuous Flight Auger
RC - Rock Core
CU - Cuttings
CT - Continuous Tube

Depth to Groundwater

Noted on Drilling Tools	-- ft
At Completion (in augers)	-- ft
At Completion (open hole)	-- ft
After -- hours	-- ft
After -- hours	-- ft
Cave Depth	-- ft

Boring Method
HSA - Hollow Stem Augers
CFA - Continuous Flight Augers
DC - Driving Casing
MD - Mud Drilling

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TEST BORING LOG

CLIENT **Brandstetter Carroll INC**
PROJECT NAME **Proposed Paintsville Pool**
PROJECT LOCATION **412 Madison Avenue**
Paintsville, Kentucky

BORING # **B-2A**
JOB # **LOUGE22082**
DRAWN BY **Z. Nichols**
APPROVED BY **T. Andres**

DRILLING and SAMPLING INFORMATION

Date Started **7/6/22**
Date Completed **7/6/22**
Drill Foreman **Z. Nichols**
Inspector **P. Presnell**
Boring Method **Hand auger, DCP**

Hammer Wt. **140** lbs.
Hammer Drop **30** in.
Spoon Sampler OD **2** in.
Rock Core Dia. **2** in.
Shelby Tube OD **3** in.

TEST DATA

SOIL CLASSIFICATION
SURFACE ELEVATION (ft): 610.0
Latitude (deg): 37.808858, Longitude (deg): -82.804633

Soil Classification	Stratum Depth	Depth Scale	Sample No.	Sample Type	Sampler Graphics / Recovery Graphics	Groundwater	Standard Penetration Test N-Value (blows/foot)	Qu-psi Unconfined Compressive Strength	PP-tsf Pocket Penetrometer	Moisture Content %	Liquid Limit (LL)	Plastic Limit (PL)	Percent Passing #200 Sieve	Remarks
CONCRETE 7.5 inches	0.6													
POORLY GRADED SAND (SP), Brown	1.0													
LEAN CLAY (CL), Reddish brown and Gray DCP at 1 feet: 2-2-2 - free water seepage observed at 2 feet	2.0		1	CU		●				28.2	40	21		Offset 2 feet south of B-2
Hand Auger Refusal at 2 feet on obstruction, steel t-probe utilitized to advance hole - T-probe performance indicative of soft soils	7.4	5												
Steel T-Probe refusal at 7.4 feet														

Sample Type
SPT - Standard Penetration Test
SS - Driven Split Spoon
ST - Pressed Shelby Tube
CA - Continuous Flight Auger
RC - Rock Core
CU - Cuttings
CT - Continuous Tube

Depth to Groundwater
Noted on Drilling Tools **2.0** ft
At Completion (in augers) **--** ft
At Completion (open hole) **--** ft
After **--** hours **--** ft
After **--** hours **--** ft
Cave Depth **--** ft

Boring Method
HSA - Hollow Stem Augers
CFA - Continuous Flight Augers
DC - Driving Casing
MD - Mud Drilling

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TEST BORING LOG

CLIENT: **Brandstetter Carroll INC**
PROJECT NAME: **Proposed Paintsville Pool**
PROJECT LOCATION: **412 Madison Avenue**
Paintsville, Kentucky

BORING #: **B-3**
JOB #: **LOUGE22082**
DRAWN BY: **Z. Nichols**
APPROVED BY: **T. Andres**

DRILLING and SAMPLING INFORMATION

Date Started: **7/6/22**
Date Completed: **7/6/22**
Drill Foreman: **Z. Nichols**
Inspector: **P. Presnell**
Boring Method: **Hand auger, DCP**

Hammer Wt.: **140** lbs.
Hammer Drop: **30** in.
Spoon Sampler OD: **2** in.
Rock Core Dia.: **2** in.
Shelby Tube OD: **3** in.

TEST DATA

SOIL CLASSIFICATION
SURFACE ELEVATION (ft): 605.0
Latitude (deg): 37.808933, Longitude (deg): -82.804426

Soil Classification	Stratum Depth	Depth Scale	Sample No.	Sample Type	Sampler Graphics / Recovery Graphics	Groundwater	Standard Penetration Test N-Value (blows/foot)	Qu-psi Unconfined Compressive Strength	PP-tsf Pocket Penetrometer	Moisture Content %	Liquid Limit (LL)	Plastic Limit (PL)	Percent Passing #200 Sieve	Remarks
CONCRETE 6 inches	0.5													
CRUSHED STONE	1.0													
LEAN CLAY (CL), Reddish brown and Gray DCP at 1 feet: 10-9-11			1	CU						22.3				
	2.5		2	CU						24.9				
Hand Auger Refusal at 2.5 feet on hard lean clay														

Sample Type
SPT - Standard Penetration Test
SS - Driven Split Spoon
ST - Pressed Shelby Tube
CA - Continuous Flight Auger
RC - Rock Core
CU - Cuttings
CT - Continuous Tube

Depth to Groundwater
Noted on Drilling Tools -- ft
At Completion (in augers) -- ft
At Completion (open hole) -- ft
After -- hours -- ft
After -- hours -- ft
Cave Depth -- ft

Boring Method
HSA - Hollow Stem Augers
CFA - Continuous Flight Augers
DC - Driving Casing
MD - Mud Drilling

Borehole	Depth	Sample Type	Liquid Limit	Plastic Limit	Plasticity Index	Class-ification	Water Content (%)	Unconfined Compressive Strength (psi)	Dry Density (pcf)	Wet Density (pcf)	Max. Dry Density (pcf)	Opt. Water Content (%)	CBR	Swell (%)	RQD	Percent Recovery	Cc	Cr	pH
B-1	0.6	CU					8.6												
B-1	2.0	CU					24.3												
B-1	3.5	CU					25.8												
B-1	4.5	CU					24.6												
B-2	0.3	CU					15.4												
B-2A	1.0	CU	40	21	19	CL	28.2												
B-3	1.0	CU					22.3												
B-3	2.0	CU					24.9												

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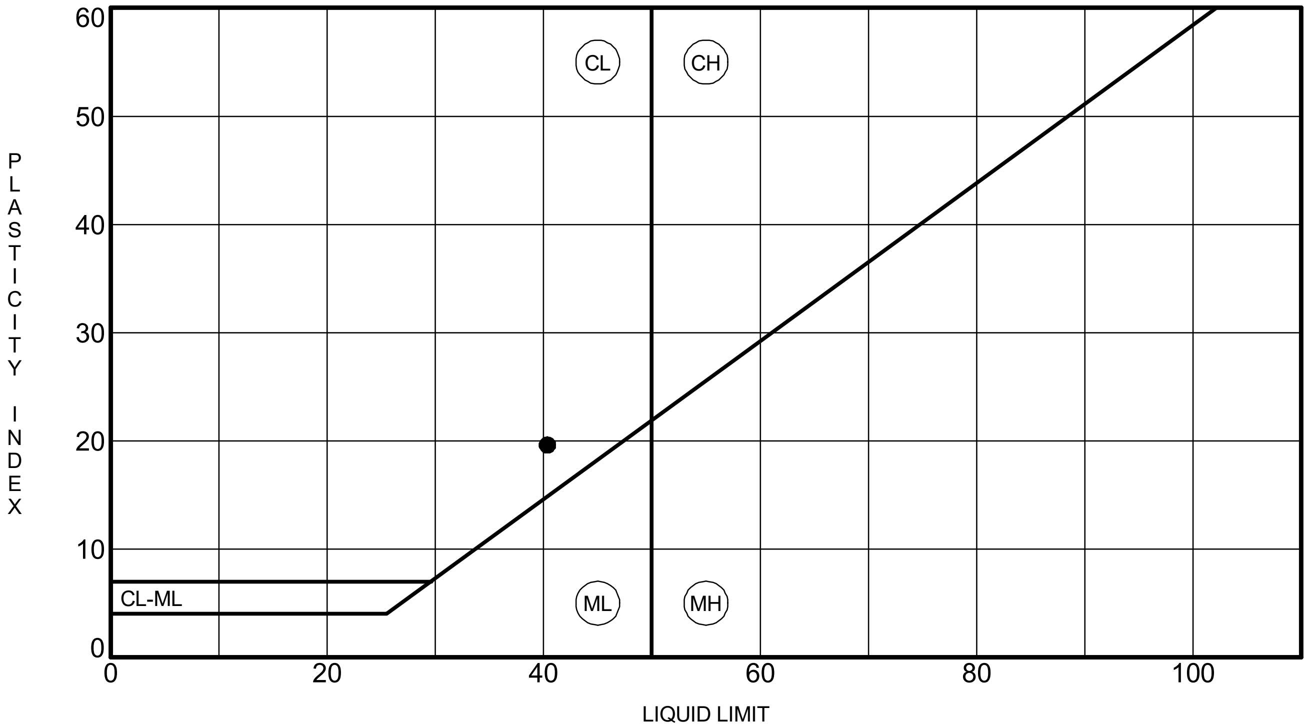
Summary of Laboratory Results

Client: Brandstetter Carroll INC
Project: Proposed Paintsville Pool
Location: 412 Madison Avenue
City, State: Paintsville, Kentucky
Number: LOUGE22082 Date: 7/26/2022



	Specimen Identification		LL	PL	PI	Water Content	Description
●	**B-2A**	**1.0**	**40**	**21**	**19**	**28.2**	**LEAN CLAY (CL), Reddish brown and gray**

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ATTERBERG LIMITS RESULTS

Client: Brandstetter Carroll INC
Project: Proposed Paintsville Pool
Location: 412 Madison Avenue
City, State: Paintsville, Kentucky
Number: LOUGE22082
Date: 7/26/2022

LEGEND TO CLASSIFICATION AND SYMBOLS

SOIL TYPES

(Shown in Graphic Log)

- Fill
- Asphalt
- Topsoil
- Gravel
- Sand
- Silt
- Lean Clay
- Fat Clay
- Silty Sand
- Clayey Sand
- Sandy Silt
- Clayey Silt
- Sandy Clay
- Silty Clay
- Limestone
- Sandstone
- Siltstone
- Shale

SAMPLER TYPES

(Shown in Sampler Column)

- Shelby Tube
- Split Spoon
- Rock Core
- Grab Sample
- No Recovery

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CONSISTENCY OF COHESIVE SOILS

(Automatic Hammer)

SPT "N" VALUE	CONSISTENCY	UNCONFINED COMPRESSIVE STRENGTH (PSF)
<2	Very Soft	<500
2-3	Soft	500-1,000
4-6	Medium Stiff	1,000-2,000
7-12	Stiff	2,000-4,000
13-26	Very Stiff	4,000-8,000
>26	Hard	>8,000

RELATIVE DENSITY OF COHESIONLESS SOILS

SPT "N" VALUE	RELATIVE DENSITY
<5	Very Loose
5 to 10	Loose
11 to 30	Medium Dense
31 to 50	Dense
>50	Very Dense

ESTIMATES RELATIVE MOISTURE CONDITION

(Visual classification relative to assumed optimum moisture content (OMC) of standard proctor)

Dry	-Air dry to dusty
Slightly Moist	-Dusty to approximate -2% OMC
Moist	-Approximate ±2% OMC
Very Moist	-Approximate +2% OMC to saturated
Wet	-Contains free water and/or saturated

RELATIVE HARDNESS OF ROCK

(Automatic Hammer)

Very Soft	-Pieces 1 inch or more in thickness can be broken by finger pressure.
Soft	-May be broken with fingers
Medium	-Corners and edges may be broken with fingers
Moderately Hard	-Moderate blow of hammer required to break sample
Hard	-Hard blow of hammer required to break sample
Very Hard	-Several hard blows of hammer required to break sample

RELATIVE WEATHERING OF ROCK

Fresh	-No visible sign of weathering, slight discoloration
Slightly	-Discoloration and discontinuity surfaces
Moderately	-Less than half disintegrated, significant discoloration
Highly	-More than half disintegrated
Completely	-All rock disintegrated into soil. Rock matrix intact.
Residual Soil	-All rock converted to soil. Rock matrix destroyed.

PARTICLE SIZE IDENTIFICATION

(ASTM D2488)

Boulders	> 12 inches
Cobbles	12 to 3 inches
Gravel	
Coarse	3 to ¾ inches
Fine	¾ to 4.75 mm
Sand [1]	
Coarse	4.75 to 2 mm
Medium	2 to 0.425
Fine	0.425 to 0.075 mm
Silt or Clay [2]	<0.075 mm

1. *No. 4 Sieve to No. 200 Sieve*
2. *Finer than No. 200 Sieve*

PROPORTION OF SAND AND GRAVEL

(By Dry Weight)

Trace	<15%
With	15 to 29%
Modifier	>29%

PROPORTION OF FINES

(By Dry Weight)

Trace	<5%
With	5 to 12%
Modifier	>12%

TERMS

Standard Penetration Test "N" Value (SPT "N" Value)	Number of blows required to drive a 1.4 inch (inside diameter) split spoon sampler 1 foot by a 140 pound hammer falling 30 inches
Recovery (REC)	Total length of rock recovered in the core barrel divided by the total length of the core run
Rock Quality Designation (RQD)	Total length of sound rock segments recovered longer or equal to 4 inches divided by the total length of core run

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.

The Geoprofessional Business Association (GBA) has prepared this advisory to help you – assumedly a client representative – interpret and apply this geotechnical-engineering report as effectively as possible. In that way, you can benefit from a lowered exposure to problems associated with subsurface conditions at project sites and development of them that, for decades, have been a principal cause of construction delays, cost overruns, claims, and disputes. If you have questions or want more information about any of the issues discussed herein, contact your GBA-member geotechnical engineer. Active engagement in GBA exposes geotechnical engineers to a wide array of risk-confrontation techniques that can be of genuine benefit for everyone involved with a construction project.

Understand the Geotechnical-Engineering Services Provided for this Report

Geotechnical-engineering services typically include the planning, collection, interpretation, and analysis of exploratory data from widely spaced borings and/or test pits. Field data are combined with results from laboratory tests of soil and rock samples obtained from field exploration (if applicable), observations made during site reconnaissance, and historical information to form one or more models of the expected subsurface conditions beneath the site. Local geology and alterations of the site surface and subsurface by previous and proposed construction are also important considerations. Geotechnical engineers apply their engineering training, experience, and judgment to adapt the requirements of the prospective project to the subsurface model(s). Estimates are made of the subsurface conditions that will likely be exposed during construction as well as the expected performance of foundations and other structures being planned and/or affected by construction activities.

The culmination of these geotechnical-engineering services is typically a geotechnical-engineering report providing the data obtained, a discussion of the subsurface model(s), the engineering and geologic engineering assessments and analyses made, and the recommendations developed to satisfy the given requirements of the project. These reports may be titled investigations, explorations, studies, assessments, or evaluations. Regardless of the title used, the geotechnical-engineering report is an engineering interpretation of the subsurface conditions within the context of the project and does not represent a close examination, systematic inquiry, or thorough investigation of all site and subsurface conditions.

Geotechnical-Engineering Services are Performed for Specific Purposes, Persons, and Projects, and At Specific Times

Geotechnical engineers structure their services to meet the specific needs, goals, and risk management preferences of their clients. A geotechnical-engineering study conducted for a given civil engineer will not likely meet the needs of a civil-works constructor or even a different civil engineer. Because each geotechnical-engineering study is unique, each geotechnical-engineering report is unique, prepared *solely* for the client.

Likewise, geotechnical-engineering services are performed for a specific project and purpose. For example, it is unlikely that a geotechnical-engineering study for a refrigerated warehouse will be the same as one prepared for a parking garage; and a few borings drilled during a preliminary study to evaluate site feasibility will not be adequate to develop geotechnical design recommendations for the project.

Do not rely on this report if your geotechnical engineer prepared it:

- for a different client;
- for a different project or purpose;
- for a different site (that may or may not include all or a portion of the original site); or
- before important events occurred at the site or adjacent to it; e.g., man-made events like construction or environmental remediation, or natural events like floods, droughts, earthquakes, or groundwater fluctuations.

Note, too, the reliability of a geotechnical-engineering report can be affected by the passage of time, because of factors like changed subsurface conditions; new or modified codes, standards, or regulations; or new techniques or tools. *If you are the least bit uncertain* about the continued reliability of this report, contact your geotechnical engineer before applying the recommendations in it. A minor amount of additional testing or analysis after the passage of time – if any is required at all – could prevent major problems.

Read this Report in Full

Costly problems have occurred because those relying on a geotechnical-engineering report did not read the report in its entirety. Do not rely on an executive summary. Do not read selective elements only. *Read and refer to the report in full.*

You Need to Inform Your Geotechnical Engineer About Change

Your geotechnical engineer considered unique, project-specific factors when developing the scope of study behind this report and developing the confirmation-dependent recommendations the report conveys. Typical changes that could erode the reliability of this report include those that affect:

- the site's size or shape;
- the elevation, configuration, location, orientation, function or weight of the proposed structure and the desired performance criteria;
- the composition of the design team; or
- project ownership.

As a general rule, *always* inform your geotechnical engineer of project or site changes – even minor ones – and request an assessment of their impact. *The geotechnical engineer who prepared this report cannot accept* *responsibility or liability for problems that arise because the geotechnical engineer was not informed about developments the engineer otherwise would have considered.*

Most of the "Findings" Related in This Report Are Professional Opinions

Before construction begins, geotechnical engineers explore a site's subsurface using various sampling and testing procedures. *Geotechnical engineers can observe actual subsurface conditions only at those specific locations where sampling and testing is performed.* The data derived from that sampling and testing were reviewed by your geotechnical engineer, who then applied professional judgement to form opinions about subsurface conditions throughout the site. Actual sitewide-subsurface conditions may differ – maybe significantly – from those indicated in this report. Confront that risk by retaining your geotechnical engineer to serve on the design team through project completion to obtain informed guidance quickly, whenever needed.

This Report's Recommendations Are Confirmation-Dependent

The recommendations included in this report – including any options or alternatives – are confirmation-dependent. In other words, they are <u>not</u> final, because the geotechnical engineer who developed them relied heavily on judgement and opinion to do so. Your geotechnical engineer can finalize the recommendations *only after observing actual subsurface conditions* exposed during construction. If through observation your geotechnical engineer confirms that the conditions assumed to exist actually do exist, the recommendations can be relied upon, assuming no other changes have occurred. *The geotechnical engineer who prepared this report cannot assume responsibility or liability for confirmation-dependent recommendations if you fail to retain that engineer to perform construction observation.*

This Report Could Be Misinterpreted

Other design professionals' misinterpretation of geotechnical-engineering reports has resulted in costly problems. Confront that risk by having your geotechnical engineer serve as a continuing member of the design team, to:

- confer with other design-team members;
- help develop specifications;
- review pertinent elements of other design professionals' plans and specifications; and
- be available whenever geotechnical-engineering guidance is needed.

You should also confront the risk of constructors misinterpreting this report. Do so by retaining your geotechnical engineer to participate in prebid and preconstruction conferences and to perform construction-phase observations.

Give Constructors a Complete Report and Guidance

Some owners and design professionals mistakenly believe they can shift unanticipated-subsurface-conditions liability to constructors by limiting the information they provide for bid preparation. To help prevent the costly, contentious problems this practice has caused, include the complete geotechnical-engineering report, along with any attachments or appendices, with your contract documents, *but be certain to note conspicuously that you've included the material for information purposes only.* To avoid misunderstanding, you may also want to note that "informational purposes" means constructors have no right to rely on the interpretations, opinions, conclusions, or recommendations in the report. Be certain that constructors know they may learn about specific project requirements, including options selected from the report, *only* from the design drawings and specifications. Remind constructors that they may perform their own studies if they want to, and *be sure to allow enough time* to permit them to do so. Only then might you be in a position to give constructors the information available to you, while requiring them to at least share some of the financial responsibilities stemming from unanticipated conditions. Conducting prebid and preconstruction conferences can also be valuable in this respect.

Read Responsibility Provisions Closely

Some client representatives, design professionals, and constructors do not realize that geotechnical engineering is far less exact than other engineering disciplines. This happens in part because soil and rock on project sites are typically heterogeneous and not manufactured materials with well-defined engineering properties like steel and concrete. That lack of understanding has nurtured unrealistic expectations that have resulted in disappointments, delays, cost overruns, claims, and disputes. To confront that risk, geotechnical engineers commonly include explanatory provisions in their reports. Sometimes labeled "limitations," many of these provisions indicate where geotechnical engineers' responsibilities begin and end, to help others recognize their own responsibilities and risks. *Read these provisions closely.* Ask questions. Your geotechnical engineer should respond fully and frankly.

Geoenvironmental Concerns Are <u>Not</u> Covered

The personnel, equipment, and techniques used to perform an environmental study – e.g., a "phase-one" or "phase-two" environmental site assessment – differ significantly from those used to perform a geotechnical-engineering study. For that reason, a geotechnical-engineering report does not usually provide environmental findings, conclusions, or recommendations; e.g., about the likelihood of encountering underground storage tanks or regulated contaminants. *Unanticipated subsurface environmental problems have led to project failures.* If you have not obtained your own environmental information about the project site, ask your geotechnical consultant for a recommendation on how to find environmental risk-management guidance.

Obtain Professional Assistance to Deal with Moisture Infiltration and Mold

While your geotechnical engineer may have addressed groundwater, water infiltration, or similar issues in this report, the engineer's services were not designed, conducted, or intended to prevent migration of moisture – including water vapor – from the soil through building slabs and walls and into the building interior, where it can cause mold growth and material-performance deficiencies. Accordingly, ***proper implementation of the geotechnical engineer's recommendations will <u>not</u> of itself be sufficient to prevent moisture infiltration. Confront the risk of moisture infiltration*** by including building-envelope or mold specialists on the design team. ***Geotechnical engineers are <u>not</u> building-envelope or mold specialists.***

GEOPROFESSIONAL BUSINESS ASSOCIATION

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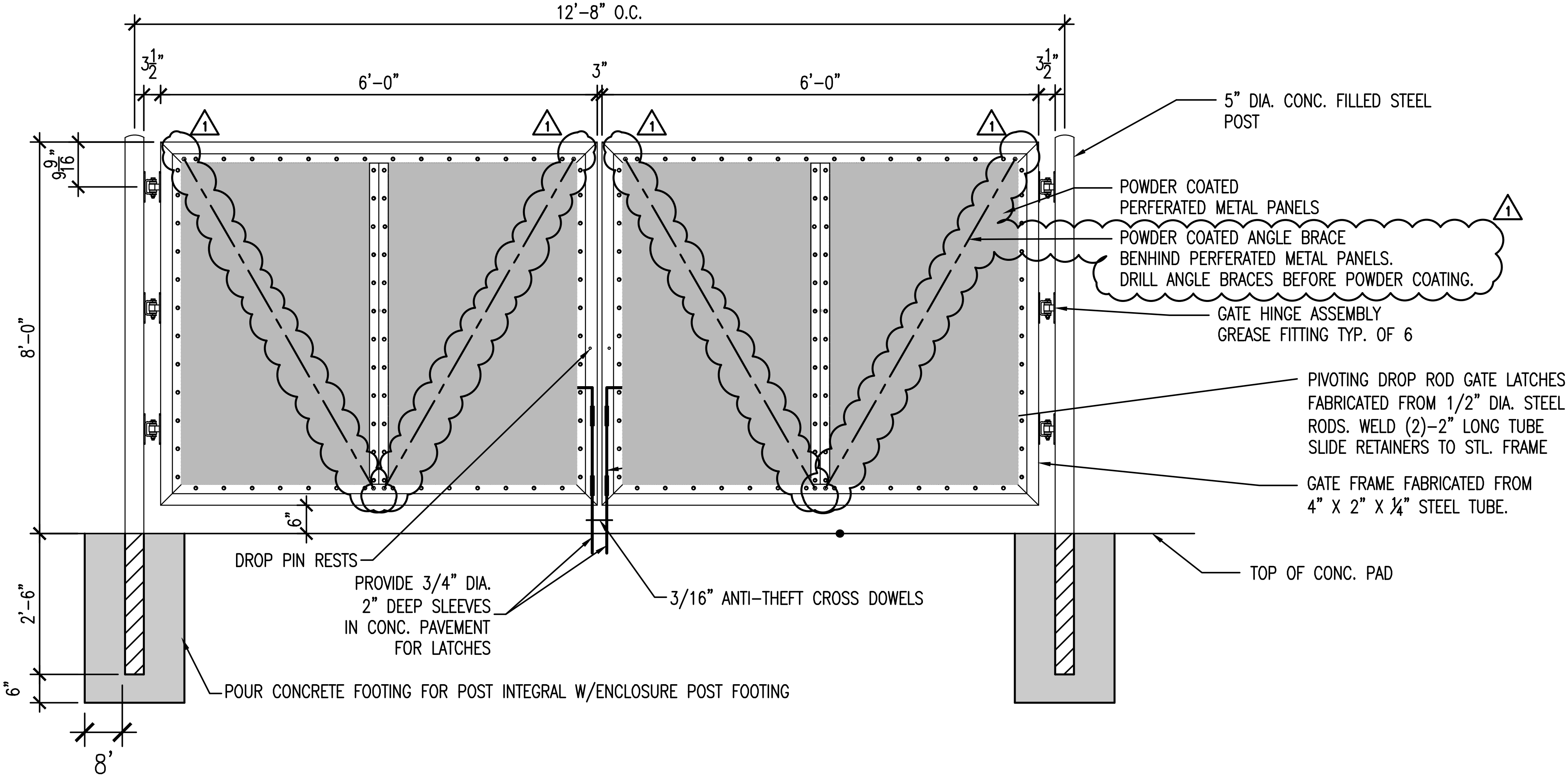


C2

Gate & Post Foundation Details

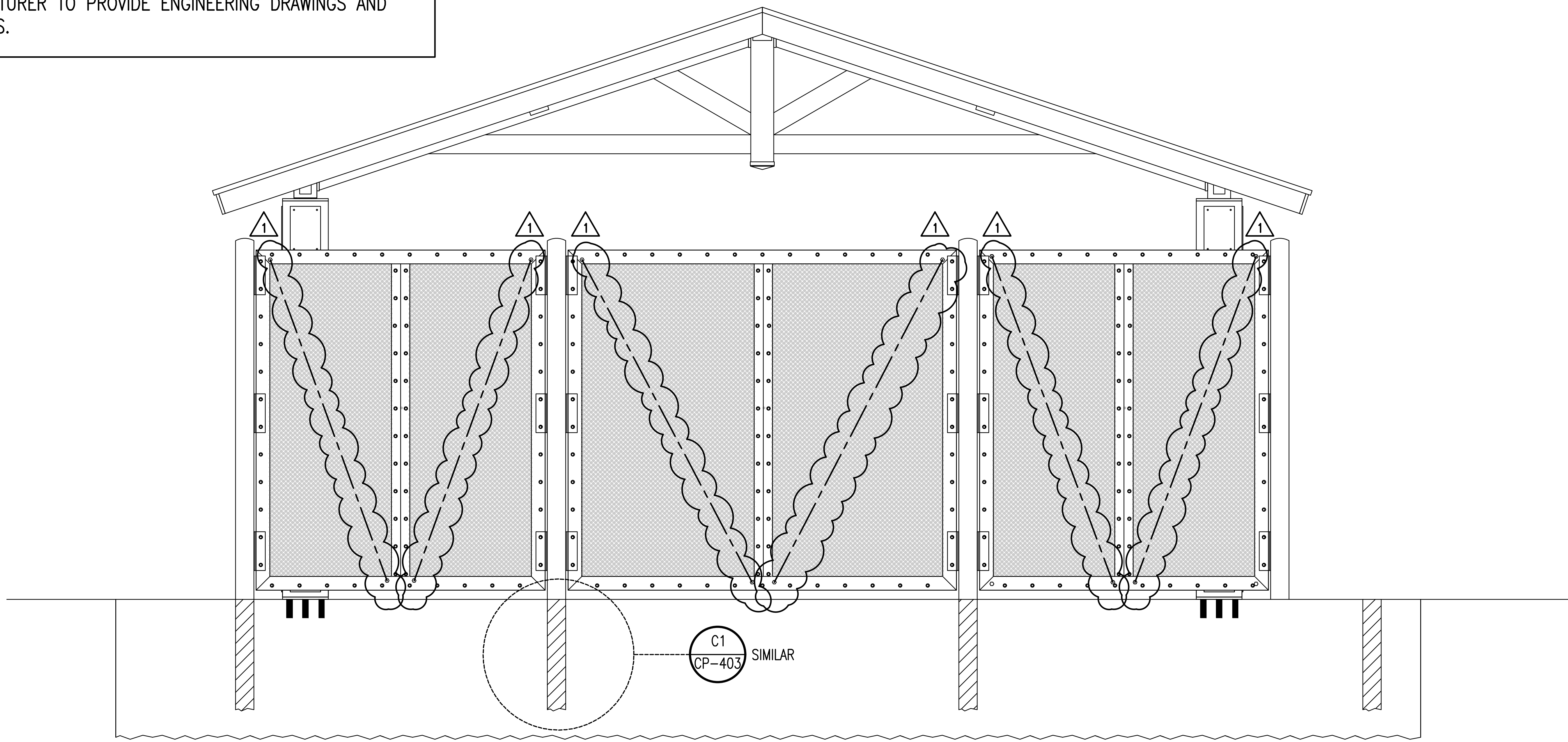
3/8"=1'-0"

ADD1.1

GENERAL NOTE:

THE SHELTER PLANS REFLECT THE BASIS OF DESIGN. THE SHELTER AND ASSOCIATED FOOTINGS ARE A DELEGATED DESIGN. CONTRACTOR TO PROVIDE SHOP DRAWINGS.

SELECTED MANUFACTURER TO PROVIDE ENGINEERING DRAWINGS AND FOUNDATION DETAILS.



C1 Section Thru Building

1/2"=1'-0"