Geotechnical Engineering Report

Macon Aquatic Center
1203 Noll Drive
Macon, Missouri
June 22, 2016
Terracon Project No. 09165033

Prepared for:
City of Macon
Macon, Missouri

Prepared by:
Terracon Consultants, Inc.
St. Louis, Missouri

June 22, 2016

City of Macon
106 West Bourke Street
Macon, Missouri 63552

Attn: Ms. Vicky McLeland
P: [660] 385 6421
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Re: Geotechnical Engineering Report
Macon Aquatic Center
1203 Noll Drive
Macon, Missouri
Terracon Project Number: 09165033

Dear Mr. McLeland:

Terracon Consultants, Inc. (Terracon) has completed the geotechnical engineering services for the above-referenced project. This study was performed in general accordance with our proposal number P09165033, dated May 26, 2016. This report presents the findings of the subsurface exploration and provides geotechnical recommendations concerning earthwork and the design and construction of shallow foundations, floor slabs, and swimming pools.

We appreciate the opportunity to be of service to you on this project. If you have any questions concerning this report, or if we may be of further service, please contact us.

Sincerely,
Terracon Consultants, Inc.

for:

P. Benjamin Luetkemeyer, R.G.
Senior Staff Geophysicist

Jonathan D Bruner, P.E.
Senior Geotechnical Engineer
Missouri No. E-2001004566

STATE OF MISSOURI
JONATHAN D. BRUNER
NUMBER
E-2001004566
6-22-2016
REGISTERED PROFESSIONAL ENGINEER

TABLE OF CONTENTS

TABLE OF CONTENTS (continued)

EXECUTIVE SUMMARY

A geotechnical exploration has been performed for the proposed Macon Aquatic Center located at 1203 Noll Drive, in Macon, Missouri. Six (6) borings, designated B-1 through B-6 were extended to depths of approximately 20 to 30 feet below the existing ground surface. Based on the information obtained from our subsurface exploration, the following geotechnical considerations were identified:

- The site is generally comprised of about 5½ to 9 feet of existing undocumented fill over native soils. Based on our field and laboratory results, the existing fill does not appear to be adequate for foundation, pool, or floor slab support. The native soils below the existing fill materials are generally very soft to medium stiff and very loose to loose and will likely present an inadequate bearing surface. Therefore, we recommend that the pool structures be supported using one of the following options:
  - removing and replacing/recompacting the existing undocumented fill;
  - utilizing ground improvement consisting of vibro-stone columns or aggregate piers to improving the site soils by stiffening both the existing fill and the native soils;
  - supporting the structures on helical piers; or
  - deep foundations such as drilled shafts or auger-cast piles could also be considered, but are expected to be cost prohibitive.

- The subject site is located approximately 250 miles from the New Madrid Seismic Zone. Granular soils beneath groundwater level are susceptible to an increase in pore water pressure and subsequent liquefaction due to dynamic loading during earthquake excitation. Liquefaction results in a significant decrease of soil shear strength, causing permanent total and differential settlement. The profile indicates that the clayey silts and sands encountered in our borings are subject to liquefaction. About ten (10) inches of seismically induced settlement was calculated in the upper 30 feet of the subgrade from the design earthquake. Since our analysis indicates that the site soils are susceptible to liquefaction, an IBC Site Class F is appropriate for the site.

- Groundwater levels observed in the borings were as shallow as 4 feet below existing ground surface. We understand the new facility will contain below-grade structures (e.g., swimming pool, lazy river) that could potentially extend below groundwater levels. Dewatering may likely be required to allow for construction of the deeper pool floors and permanent measures may be needed to resist uplift/buoyancy should the groundwater level rise above the pool floor elevation.

- Some of the near-surface soils exhibited high moisture levels indicating they may be easily disturbed by construction activities. During periods of dry weather, these soils may be stable upon initial exposure; however, in their present conditions these soils will likely become unstable under construction traffic. We recommend the owner budget for overexcavation and/or subgrade stabilization. Also, contractors should be prepared to handle potentially wet, soft conditions.

- Grading or site plans were not available at the time this report was written. Very soft to soft and very loose to loose native soils were encountered in many of the borings on this site. These native soils are anticipated to consolidate (i.e., compress) due to the loading of new fill and proposed structures. The settlement of the new fill should be monitored with surface settlement measurement devices. We estimate that the foundation construction should wait about 1 to 3 months after the fill placement is complete to allow the pre-construction settlements to occur.

- Demolition of the existing building, swimming pool, etc. should include removal of all above- and below-grade elements including floor slabs, foundation walls, and footings. All existing utilities should also be properly abandoned and/or relocated. Excavations created by demolition and removal of existing features should be backfilled with engineered fill that is placed and compacted as recommended in this report.

- Close monitoring of the construction operations discussed herein will be critical in achieving the design subgrade support. We therefore recommend that Terracon be retained to monitor this portion of the work.

The professional opinions and recommendations presented in this report are based on evaluation of data developed by testing discrete samples obtained from widely-spaced borings. Site subsurface conditions have been inferred from available data, but actual subsurface conditions will only be revealed by excavation. So that variations in subsurface conditions which may affect the design can be addressed as they are encountered, we recommend that Terracon be retained to observe excavations and perform tests during the site preparation, earthwork and foundation construction phases of the project.

This executive summary should not be separated from or used apart from this report. This report presents fully developed recommendations and opinions based on our understanding of the project at the time the report was prepared. The report limitations are described in the **GENERAL COMMENTS** section of this report.

GEOTECHNICAL ENGINEERING REPORT
MACON AQUATIC CENTER
1203 NOLL DRIVE
MACON, MISSOURI

Terracon Project No. 09165033
June 22, 2016

1.0 INTRODUCTION

A geotechnical exploration has been performed for the proposed Macon Aquatic Center located at 1203 Noll Drive, in Macon, Missouri. Six (6) borings, designated B-1 through B-6 were extended to depths of approximately 20 to 30 feet below the existing ground surface. Logs of the borings along with a Topographic Map and Boring Location Diagram are included in Appendix A of this report.

The purpose of these services is to provide information and geotechnical engineering recommendations relative to:

- subsurface soil conditions
- groundwater conditions
- earthwork
- seismic considerations
- lateral earth pressure recommendations
- foundation design and construction

2.0 PROJECT INFORMATION

2.1 Project Description

Item	Description
Site layout	See Appendix A, Exhibit A-2: Boring Location Diagram
Building construction	Details on the proposed construction were not available at the time of report preparation, but expected to be a single-story, steel- or wood-framed structure with reinforced concrete foundations, and a slab-on-grade floor. An in-ground pool is also planned.
Finished floor elevation (FFE)	Assumed to be within 2 feet of existing grade.
Maximum loads (estimated by Terracon)	Columns: 100 kips Walls: 3 kips/lf Slab: 150 psf
Grading	Proposed grades were not provided; assumed to be less than 2 feet of excavation or fill placement.
Cut and fill slopes	None anticipated
Free-standing retaining walls	None anticipated

Item	Description
Below-grade areas	Swimming pool and lazy river

Should any of the above information be inconsistent with the planned construction, or if construction plans change, please notify us so that we may make any necessary modifications to the recommendations in this report.

2.2 Site Location and Description

Item	Description
Location	The project will be located at 1203 Noll Drive in Macon, Missouri Latitude: 39.747106°N Longitude: 92.494353°W
Existing improvements	Swimming pool, concrete deck, pool house and Macon Lake overflow drainage channel
Current ground cover	Asphalt pavement parking lot and grass
Existing topography	The site is relatively flat with the exception of the earthen dam approximately 100 feet north of the swimming pool

3.0 SUBSURFACE CONDITIONS

3.1 Geology

Based on the 2003 Geologic Map of Missouri, Missouri Department of Natural Resources, bedrock at this site consists of the Pennsylvanian Cherokee and Marmaton Group. Rock types include cyclic deposits of shale, limestone, sandstone and coal beds.

3.2 Typical Profile

Based on the results of the boring, subsurface conditions on the project site can be generalized as follows:

Stratum	Approximate Depth to Bottom of Stratum (feet)	Material Description	Consistency/ Density
1	5½ to 9	Fill: Lean (CL) clay, trace sand and gravel	N/A
2	20	[1]Clayey silt (ML) and Sand (SP) [2]Lean clay (CL)	Very loose to loose Soft to medium stiff

1. Borings B-1, B-2, B-3, B-4, B-5
2. Boring B-6

Conditions encountered at the boring location are indicated on the boring logs in Appendix A of this report. Stratification boundaries on the boring log represent the approximate location of changes in soil types; in-situ, the transition between materials may be gradual.

3.3 Groundwater

The borehole was observed while drilling and after completion for the presence and level of groundwater. Groundwater was encountered in all borings while drilling and at the completion of drilling. The water levels observed are noted in the attached boring logs and are summarized in the table below.

Boring Number	Depth to Groundwater while Drilling, (feet) [1]	Depth to Groundwater at completion of drilling, (feet) [1]
B-1	10	8
B-2	6	4
B-3	10	8
B-4	5	4
B-5	13	10
B-6	8	8

1. Depths are below existing grades

The water levels noted above are not necessarily stable groundwater levels. Groundwater levels may fluctuate due to seasonal variations in the amount of rainfall, runoff, the level of nearby Macon Lake, and other factors not evident at the time the boring was performed. Trapped or "perched" water can occur above lower permeability soil layers. Therefore, groundwater levels during construction or at other times in the life of the structure may be different from the levels indicated on the boring log. Groundwater level fluctuations and perched water should be considered when developing design and construction plans and specifications for the project.

4.0 RECOMMENDATIONS FOR DESIGN AND CONSTRUCTION

4.1 Geotechnical Considerations

Based on the results of the subsurface exploration, laboratory testing, and our analyses, it is our opinion that the proposed structure can be supported on shallow foundations if the site soils are improved. Special geotechnical considerations for this project include:

- Building/pool support,
- Liquefaction susceptibility,
- Shallow groundwater levels,
- Soft subgrades,

- Settlement monitoring, and the
- Demolition of existing structures.

4.1.1 Building/Pool Support

The site is generally comprised of about 5½ to 9 feet of existing undocumented fill over native soils. Based on our field and laboratory results, the existing undocumented fill does not appear to be adequate for foundation, pool, or floor slab support. The native soils below the existing fill materials are generally very soft to medium stiff native silts and clays and very loose to loose native granular soils and will likely present an inadequate bearing surface.

Therefore, we recommend that the pool structures be supported using one of the following options:

- removing and replacing/recompacting the existing undocumented fill;
- utilizing ground improvement consisting of vibro-stone columns or aggregate piers to improving the site soils by stiffening both the existing fill and the native soils;
- supporting the structures on helical piers; or
- deep foundations such as drilled shafts or auger-cast piles could also be considered, but are expected to be cost prohibitive.

If the removal and replacement option is chosen, we will need to determine the required depth of soil removal and replacement once the grading and foundation plans (and loads) are available for evaluation. For preliminary budgeting purposes, we would likely require at least 3 feet of new fill beneath the foundations. Removal of the existing fill materials will expose the sensitive soft and loose native soils and likely require soil stabilization to get the new compacted fill started. Additionally, these excavations may be near the groundwater level and dewatering could be necessary.

The foundation options are discussed further in section **4.3 Foundations**.

4.1.2 Liquefaction susceptibility

The subject site is located approximately 250 miles from the New Madrid Seismic Zone, which is a major seismic zone in the Southern and Midwestern United States. Granular soils beneath groundwater level are susceptible to an increase in pore water pressure and subsequent liquefaction due to dynamic loading during earthquake excitation. Liquefaction results in a significant decrease of soil shear strength, causing permanent total and differential settlement. The profile indicates that the clayey silts and sands encountered in our borings are subject to liquefaction. About ten (10) inches of seismically induced settlement was calculated in the upper 30 feet of the subgrade from the design earthquake. Additional settlement could occur from deeper liquefaction. Since our analysis indicates that the site soils are susceptible to liquefaction, an IBC Site Class F is appropriate for the site. See section **4.4 Seismic Considerations** for details.

One or more of the following practices may be considered to induce the settlement prior to construction or improve the soils to reduce post-construction settlement:

- Perform vibro compaction or dynamic compaction in the building areas to densify the sands prior to construction and reduce post-construction settlement. Dynamic compaction may have limited results since the groundwater level at this site is relatively shallow.
- Install earthquake drains to relieve the pore water pressures induced during an earthquake.
- Install vibro-stone columns in the building footprint to densify the native sands and reduce post-construction settlement.

See section **4.5 Soil Improvement** for additional discussion of soil improvement at this site.

4.1.3 Groundwater Level

Groundwater levels observed in the borings were as shallow as 4 feet below existing ground surface. We understand the new facility will contain below-grade structures (e.g., swimming pool, lazy river) that could potentially extend below groundwater levels. Dewatering may likely be required to allow for construction of the deeper pool floors and permanent measures may be needed to resist uplift/buoyancy should the groundwater level rise above the pool floor elevation.

4.1.4 Soft Subgrade

Relatively soft and moist soils were encountered in the borings and could be exposed in excavations and cuts. These soils tend to become unstable when disturbed. During periods of dry weather, these soils may be stable upon initial exposure; however, if exposed, these soils could become relatively soft and unstable under construction traffic. Further, depending upon site conditions during construction, overexcavation or stabilization of the subgrade and/or base of overexcavations may be needed to achieve a suitable working surface. Accordingly, we recommend that the owner budget for the possibility that overexcavation and/or subgrade stabilization may be required and contractors be prepared to handle potentially unstable and/or soft conditions.

4.1.5 Settlement and Monitoring

Grading or site plans were not available at the time this report was written. However, settlement is anticipated if grade-raise fill is planned during construction. Very soft to soft and very loose to loose native soils were encountered in many of the borings on this site. These native soils are anticipated to consolidate (i.e., compress) due to the loading of new fill and proposed structures.

Due to the potential for settlement, new fill should be placed far in advance of planned construction to allow a significant amount of the anticipated settlement to occur. The settlement of the new fill should be monitored with surface settlement measurement devices. We estimate that the foundation construction should wait about 1 to 3 months after the fill placement is complete to allow the pre-construction settlements to occur. To reduce the lag time between the fill placement and foundation construction, one or more of the following practices may be considered:

- Place fill in the building/pool areas at the start of grading operations.
- Place surcharge fill material for a length of time to accelerate the compression of the native soils.
- Install wick drains in the building footprint to accelerate the consolidation of the native soils.
- Install aggregate piers in the building footprint to stiffen the native soils and/or accelerate their consolidation.

4.1.6 Demolition of Existing Structures

Demolition of the existing building, swimming pool, etc. should include removal of all above- and below-grade elements including floor slabs, foundation walls, and footings. Attention should be given to removing all loose or poorly compacted existing fill materials that are often located adjacent to existing and former foundation and pool walls.

All existing utilities should also be properly abandoned and/or relocated. This should include removal of all poorly compacted trench backfill extending into the proposed building area. In addition, care should be taken by contractors to protect all existing improvements to remain, such as pavements and utilities. Excavations created by demolition and removal of existing features should be backfilled with engineered fill that is placed and compacted as recommended in this report.

4.1.7 General

We recommend that the exposed subgrade be thoroughly evaluated after stripping of any topsoil or unsuitable soil and at the base of all cut areas, but prior to the start of any fill operations. We recommend that the geotechnical engineer be retained to evaluate the bearing material for the foundations and subgrade soils. Subsurface conditions, as identified by the field and laboratory testing programs, have been reviewed and evaluated with respect to the proposed project plans known to us at this time.

4.2 Earthwork

4.2.1 Site Preparation

We anticipate construction will be initiated by demolition of the existing pool, building, and removal of pavements and landscaping, topsoil, or vegetation that may be present. Demolition of the existing structures should include removal of all above and below grade elements including floor slabs, foundation walls, and footings. Attention should be given to removing all loose or poorly compacted existing fill materials that are often located adjacent to foundation walls. All existing utilities should also be properly abandoned or relocated. This should include removal of all poorly compacted trench backfill extending into the proposed building area.

We recommend that the exposed subgrade be thoroughly evaluated by a geotechnical engineer prior placement of new fill. The soils on the site are sensitive to disturbance from construction equipment traffic, particularly during wet periods. Excessively wet or dry material should either be removed or moisture conditioned and recompacted. The exposed subgrade, including areas of existing undocumented fill, should be proof-rolled where possible to aid in locating loose or soft areas. Proof-rolling can be performed with a loaded tandem axle dump truck. If unsuitable areas are observed during construction, subgrade improvement will then be necessary to establish a suitable subgrade support condition. Subgrade stabilization is discussed in section **4.2.2 Soil Stabilization**.

4.2.2 Soil Stabilization

Methods of subgrade improvement could include scarification, moisture conditioning and recompaction, removal of unstable materials and replacement with granular fill (with or without geosynthetics) and chemical stabilization. The appropriate method of improvement, if required, would be dependent on factors such as schedule, weather, the size of the area to be stabilized, and the nature of the instability. More detailed recommendations can be provided during construction as the need for subgrade stabilization occurs. Performing site grading operations during warm seasons and dry periods would help to reduce the amount of subgrade stabilization required.

If the exposed subgrade is unstable during proofrolling operations, it could be stabilized using one of the methods outlined below.

- **Scarification and Recompaction** – It may be feasible to scarify, dry, and recompact the exposed soils. The success of this procedure would depend primarily upon favorable weather and sufficient time to dry the soils. Stable subgrades would likely not be achievable if the thickness of the unstable soil is greater than about 1 foot, if the unstable soil is at or near groundwater levels (level of Deer Creek), or if construction is performed during a period of wet or cool weather when drying is difficult.

- **Crushed Stone** – The use of crushed stone or gravel is the most common procedure to improve subgrade stability. Typical undercut depths would be expected to range from about 6 to 30 inches below finished subgrade elevation with this procedure. The use of high modulus geotextiles (i.e., engineering fabric or geogrid) could also be considered. Equipment should not be operated above the fabric or geogrid until one full lift of crushed stone fill is placed above it. The maximum particle size of granular material placed over geotextile fabric or geogrid should meet the manufacturer's specifications, and generally should not exceed 1½ inches.

- **Chemical Stabilization** – Improvement of subgrades with Portland cement, lime kiln dust, Code L, or Class C fly ash could be considered for improving unstable soils. Chemical modification should be performed by a prequalified contractor having experience with successfully stabilizing subgrades in the project area on similar sized projects with similar soil conditions. Results of chemical analysis of the additive materials should be provided to the geotechnical engineer prior to use. The hazards of chemicals blowing across the site or onto adjacent property should also be considered. Additional testing would be needed to develop specific recommendations to improve subgrade stability by blending chemicals with the site soils. Additional testing could include, but not be limited to, evaluating various stabilizing agents, the optimum amounts required, the presence of sulfates in the soil, and freeze-thaw durability of the subgrade.

Further evaluation of the need and recommendations for subgrade stabilization can be provided during construction as the geotechnical conditions are exposed.

4.2.3 Material Requirements

Compacted structural fill should meet the following material property requirements:

Fill Type [1]	USCS Classification	Acceptable Location for Placement
Moderate to High Plasticity Material [2]	CH or CL (LL≥45 or PI≥25)	Not recommended for import on-site
Granular Material[3]	GM, GC, SM, or SC	All locations and elevations
Low Plasticity (LP) Material[4]	CL (LL<45 & PI<25), Granular Material[3]	All locations and elevations

1. Controlled, compacted fill should consist of approved materials that are free of organic matter and debris. Frozen material should not be used, and fill should not be placed on a frozen subgrade. A sample of each material type should be submitted to the geotechnical engineer for evaluation.
2. Delineation of high plasticity clays should be performed in the field by a qualified geotechnical engineer or their representative, and will likely require additional laboratory testing.
3. Aggregate, limestone screenings, or granular material such as sand, gravel or crushed stone. Fines must be low plasticity.
4. Granular LP materials must contain at least 15% fines, by weight. The recommended moisture content and density of LP material placed within 24 inches of the bottom of floor slabs must be maintained prior to construction of the floor slab. Material should be approved by the geotechnical engineer.

4.2.4 Compaction Requirements

Item	Description
Fill Lift Thickness	9 inches or less in loose thickness when heavy, self-propelled compaction equipment is used 4 to 6 inches in loose thickness when hand-guided equipment (i.e., jumping jack or plate compactor) is used
Compaction Requirements [1]	At least 95% of the material's maximum standard Proctor dry density (ASTM D 698) or 70% relative density (ASTM D4253 and D4254)

Item	Description
Moisture Content – Cohesive Soil	-1 to +3% of the optimum moisture content as determined by the standard Proctor test at the time of placement and compaction
Moisture Content – Granular Material	Workable moisture levels [2]

1. We recommend that compacted structural fill be tested for moisture content and compaction during placement. Should the results of the in-place density tests indicate the specified moisture or compaction limits have not been met, the area represented by the test should be reworked and retested as required until the specified moisture and compaction requirements are achieved.
2. Specifically, moisture levels should be maintained low enough to allow for satisfactory compaction to be achieved without the cohesionless fill material pumping when proofrolled.

4.2.5 Utility Trench Backfill

All trench excavations should be made with sufficient working space to permit construction including backfill placement and compaction. If utility trenches are backfilled with relatively clean granular material, they should be capped with at least 18 inches of cohesive fill in non-pavement areas to reduce the infiltration and conveyance of surface water through the trench backfill.

Utility trenches are a common source of water infiltration and migration. All utility trenches that penetrate beneath buildings should be effectively sealed to restrict water intrusion and flow through the trenches that could migrate below the structure. We recommend constructing an effective clay "trench plug" that extends at least 5 feet out from the face of the structure exterior. The plug material should consist of clay compacted at a water content at or above the soil's optimum water content. The clay fill should be placed to completely surround the utility line and be compacted in accordance with recommendations in this report.

4.2.6 Grading, Drainage and Landscaping

Final grades should slope away from the structures on all sides to prevent ponding of water. Gutters and downspouts should drain water a minimum of 10 feet beyond the footprints of the proposed structures. This can be accomplished through the use of splash-blocks, downspout extensions, and flexible pipes that are designed to attach to the end of the downspout. Flexible pipe should only be used if it is daylighted in such a manner that it gravity-drains collected water. Splash-blocks should also be considered below hose bibs and water spigots.

Trees or other vegetation whose root systems have the ability to remove excessive moisture from the subgrade and foundation soils should not be planted next to the structures. Trees and shrubbery should be kept away from the exterior of the structures a distance at least equal to their expected mature height.

4.2.7 Earthwork Construction Considerations

Due to the sandy nature of some of the soils, excavations may not stand vertically and the foundations/pool structures may need to be formed.

In periods of dry weather, the surficial soils may be of sufficient strength to allow fill construction on the exposed subgrade; however, unstable subgrade conditions could develop during general construction operations, particularly if the soils are wet or subjected to repetitive construction traffic. The use of remotely operated equipment, such as a backhoe, would be beneficial to perform cuts and reduce subgrade disturbance. Should unstable subgrade conditions be encountered, stabilization measures will need to be employed.

The contractor is responsible for selecting and implementing the appropriate dewatering procedures, if required during construction. Groundwater was encountered in the borings at depths expected to affect subgrade structures and foundation excavations. In addition, some surface and/or perched groundwater may enter foundation excavations during construction. The volume of water seepage into shallow isolated excavations may be controllable with an appropriate number of sump pits and pumps; however, more extensive dewatering and/or subgrade stabilization may be required to facilitate construction if larger and/or deeper areas of cut are performed during earthwork operations. Unstable excavations for subgrade structures may require sloping the excavation beyond the blueprint of the structure and construction of free-standing walls.

Upon completion of filling and grading, care should be taken to maintain the subgrade moisture content prior to construction of floor slabs and pavements. Construction traffic over the completed subgrade should be avoided to the extent practical. The site should also be graded to prevent ponding of surface water on the prepared subgrades or in excavations. If the subgrade should become frozen, desiccated, saturated, or disturbed, the affected material should be removed or these materials should be scarified, moisture conditioned, and recompacted prior to foundation construction.

Temporary excavations will be required during construction. As a minimum, excavations should be performed in accordance with OSHA 29 CFR, Part 1926, Subpart P, "Excavations" and its appendices, and in accordance with any applicable local, state, and federal safety regulations. The contractor should be aware that slope height, slope inclination, and excavation depth should in no instance exceed those specified by these safety regulations. Flatter slopes than those dictated by these regulations may be required depending upon the soil conditions encountered and other external factors. These regulations are strictly enforced and if they are not followed, the owner, contractor, and/or earthwork and utility subcontractor could be liable and subject to substantial penalties.

Construction site safety is the sole responsibility of the contractor who controls the means, methods, and sequencing of construction operations. Under no circumstances shall the information provided herein be interpreted to mean that Terracon is assuming any responsibility for construction site safety or the contractor's activities; such responsibility shall neither be implied nor inferred.

Terracon should be retained during the construction phase of the project to observe earthwork and to perform necessary tests and observations during subgrade preparation, proofrolling, placement and compaction of controlled compacted fills, backfilling of excavations into the completed subgrade, and otherwise during construction.

4.3 Foundations

The site is generally comprised of about 5½ to 9 feet of existing undocumented fill over native soils. Based on our field and laboratory results, the existing undocumented fill does not appear to be adequate for foundation, pool, or floor slab support. The native soils below the existing fill materials are generally very soft to medium stiff native silts and clays and very loose to loose native granular soils and will likely present an inadequate bearing surface.

Therefore, we recommend that the pool structures be supported using one of the following options:

- removing and replacing/recompacting the existing undocumented fill;
- utilizing ground improvement consisting of vibro-stone columns or aggregate piers to improving the site soils by stiffening both the existing fill and the native soils;
- supporting the structures on helical piers; or
- deep foundations such as drilled shafts or auger-cast piles could also be considered, but are expected to be cost prohibitive.

4.3.1 Foundation Design Recommendations

Description	Columns	Walls
Suitable bearing materials	Newly placed engineered fill extending to native soils; aggregate pier foundation elements or helical piers	
Net allowable bearing pressure [1]	See Note 1	
Minimum width	30 inches	18 inches
Minimum embedment below finished grade [2]	30 inches	
Estimated total settlement [3]	Approximately 1 inch	
Estimated differential settlement [3]	<¾ inch	
Ultimate passive pressure [4]	300 pcf, equivalent fluid density	
Ultimate coefficient of sliding friction [4]	0.35	

1. If the removal and replacement option is chosen, we will need to determine the required depth of soil removal and replacement once the grading and foundation plans (and loads) are available for evaluation. For preliminary budgeting purposes, we would likely require at least 3 feet of new fill beneath the foundations to result in a net allowable bearing pressure of 2,000 psf. The recommended net allowable bearing pressure, which includes a factor of safety of 3, is the pressure in excess of the minimum surrounding overburden pressure at the footing base elevation, and can be increased by 1/3 for transient loads (e.g., wind or seismic).

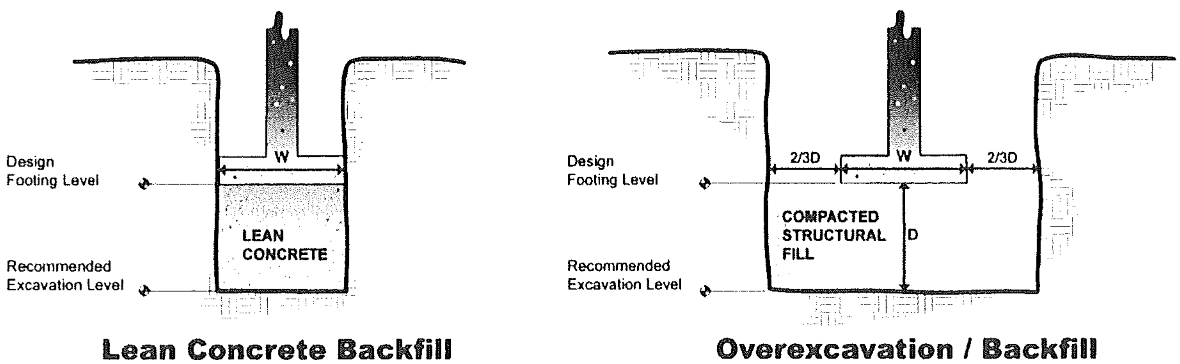
   Aggregate piers and helical piers are typically constructed on a design-build basis. The allowable net bearing capacity will be provided by the designer and they can often be designed for a specified bearing capacity or to allow the use of a higher than typical (for soil) foundation bearing capacity.
2. For frost protection and to reduce the effects of seasonal moisture variations in the subgrade soils. For perimeter footings and footings beneath unheated areas.
3. Assumes any unsuitable existing fill or soft soils, if encountered, will be undercut and replaced with compacted structural fill. Foundation settlement will depend upon variations within the subsurface soil profile, the structural loading conditions, the embedment depth of the footings, the width of the footings, the thickness of compacted fill, and the quality of the earthwork operations. If aggregate pier foundation elements or helical piers are used, then the settlement characteristics will be provided by that designer.
4. The sides of the spread footing foundation excavations must be nearly vertical and the concrete should be placed neat against the vertical faces for the passive earth pressure values to be valid. If the loaded side is sloped or benched, and then backfilled, the allowable passive pressure will be significantly reduced. Passive resistance in the upper 2½ feet of the soil profile should be neglected. If passive resistance is used to resist lateral loads, base friction should be ignored.

4.3.2 Foundation Construction Considerations

The base of all foundation excavations should be free of water and loose soil and rock prior to placing concrete. A lean concrete mud-mat should be placed over the bearing soils if the excavations must remain open for an extended period of time or to allow for placement of reinforcing steel if personnel will enter the foundation excavation.

Groundwater was encountered near the anticipated lower foundation bearing elevations in the borings, and could be encountered during foundation excavation. In addition, some surface and/or perched groundwater may enter foundation excavations during construction. Dewatering may be required to allow for foundation installation. The contractor is responsible for selecting and implementing the appropriate dewatering procedures, if required during construction.

If unsuitable bearing soils are encountered in footing excavations, the excavation could be extended deeper to suitable soils, although difficulty may be encountered with deeper excavations into sand. The footing could then bear directly on these soils at the lower level or on lean concrete backfill placed in the excavations. As an alternative, the footings could also bear on properly compacted structural backfill extending down to suitable soils. Overexcavation for compacted structural fill placement below footings should extend laterally beyond all edges of the footings at least 8 inches per foot of overexcavation depth below footing base elevation. The overexcavation should then be backfilled per the recommendations provided in section **4.2 Earthwork** up to the footing base elevation. The overexcavation and backfill procedure is illustrated in the following figure.



NOTE: Excavations in sketches shown vertical for convenience. Excavations should be sloped as necessary for safety.

4.3.3 Aggregate Pier Foundations

Considering the undocumented fill materials that are on top of lower strength native soils on this site, aggregate pier foundation elements or vibro-stone aggregate columns may be a cost-effective alternative to support the proposed structures.

Aggregate pier foundation elements are usually part of the foundation contractor's design-build system. Therefore, the subsurface exploration information contained in this report should be provided to the foundation contractors for detailed analysis and design and cost information. The allowable net bearing capacity following installation of aggregate piers will be provided by the designer, depending upon the required settlement values. The piers are typically designed based upon allowable settlement.

Aggregate pier elements typically consist of 30-inch diameter drilled holes that are filled in lifts of well-graded aggregate that is densified to form very stiff, high-density aggregate piers. Vibro-stone columns typically consist of a 22- to 24-inch diameter vibroflotted hole that is charged with clean aggregate. The vibroflot compacts this aggregate resulting in very stiff, high density aggregate piers.

The compacted aggregate piers produce high lateral stresses within the surrounding soil matrix, thereby stiffening the reinforced composite soil/aggregate mass. This results in significant strengthening and stiffening of the foundation bearing layer to support footings within the required settlement tolerances.

4.3.4 Helical Pier Foundations

Helical piers could be installed to support the purposed structures. Helical piers typically consist of square or round shafts with a series of auger flights on the bottom end of the deepest section. The capacity is confirmed by the torque measured as the pier is installed. Helical piers are typically a design-build system and, thus, design of the pier capacity is typically performed by the helical pier contractor.

4.4 Floor Slabs

As discussed in **Section 4.1.1**, existing undocumented fill materials are present on the site. Support of floor slabs on or over existing fill materials is not recommended.

4.4.1 Floor Slab Design Recommendations

Item	Description
Floor slab support	Special subgrade preparation per section **4.2 Earthwork**[1]
Modulus of subgrade reaction	150 pounds per square inch per inch (psi/in) for point loading conditions
Aggregate base course/capillary break[3]	At least 4 inches of free draining granular material

1. If the subgrade should become excessively dry or wet prior to construction of floor slabs, the affected material should be removed or the materials scarified, moisture conditioned, and recompacted. Upon completion of grading operations in the building areas, care should be taken to maintain the recommended subgrade moisture content and density until construction of the building floor slabs.
2. The floor slab design should include a capillary break, comprised of free-draining, compacted, granular material, at least 4 inches thick. Free-draining granular material should have less than 5 percent fines (material passing the #200 sieve). Other design considerations such as cold temperatures and condensation development could warrant more extensive design provisions.

Where appropriate, saw-cut control joints should be placed in the slab to help control the location and extent of cracking. For additional recommendations refer to the ACI Design Manual. Joints or any cracks that develop should be sealed with a water-proof, non-extruding compressible compound.

The use of a vapor retarder should be considered beneath concrete slabs on grade that will be covered with wood, tile, carpet or other moisture sensitive or impervious coverings, or when the slab will support equipment sensitive to moisture. When conditions warrant the use of a vapor retarder, the slab designer should refer to ACI 302 and/or ACI 360 for procedures and cautions regarding the use and placement of a vapor retarder.

4.4.2 Floor Slab Construction Considerations

On most projects, site grading is generally accomplished early in the construction phase. However as construction proceeds, the subgrade may be disturbed due to utility excavations, construction traffic, desiccation, rainfall, etc. As a result, the floor slab subgrade may not be suitable for placement of base rock and concrete, and corrective action may be required.

Prior to placement of the base aggregate, we recommend that the floor slab subgrade be rough graded and then thoroughly evaluated for stability, uniformity and moisture condition. If there is no conflict with installed utilities, we recommend the subgrade be proofrolled. Granular soils should be proofrolled and densified by using a heavy, smooth drum, vibratory roller (minimum 10-ton). Proofrolling in clay soils can be performed with a loaded, tandem-axle dump truck. During the evaluations, particular attention should be paid to high traffic areas that were rutted and disturbed earlier and to areas where backfilled trenches are located. Areas where unsuitable conditions are located should be repaired by removing and replacing the affected material with properly compacted fill. All floor slab subgrade areas should be moisture conditioned and properly compacted to the recommendations in this report immediately prior to placement of the aggregate base and concrete.

4.5 Seismic Considerations

Since our analysis indicates that the very loose to loose granular soils encountered at the site are susceptible to liquefaction, an IBC Site Class F is appropriate for the subject site, as discussed below.

Description		Value
2012 International Building Code Site Classification (IBC) [1]		F [2]
Site Latitude		39.7471°N
Site Longitude		92.4944° W
S_{DS} Spectral Acceleration for a Short Period [3]		
Fundamental period of vibration of the structure	**≤ ½ second**	0.199g [3]
	>½ second	Site Specific Evaluation is required
S_{D1} Spectral Acceleration for a 1-Second Period [3]		
Fundamental period of vibration of the structure	**≤ ½ second**	0.176g [3]
	>½ second	Site Specific Evaluation is required

1. In general accordance with the *2012 International Building Code*. IBC Site Class is based on the average characteristics of the upper 100 feet of the subsurface profile. **See discussion below.**
2. The *2012 International Building Code* (IBC) uses a site soil profile determination extending to a depth of 100 feet for seismic site classification. The current scope does not include a 100-foot soil profile determination. Borings were extended to a maximum depth of 30 feet. Additional exploration to deeper depths, or seismic velocity testing would be required to confirm the conditions below the current depth of exploration.
3. These values are only valid if the structure has a fundamental period of vibration of ½-second or less, and were obtained using online seismic design maps and tools provided by the USGS (http://earthquake.usgs.gov/hazards/designmaps/).

Since the site soils are susceptible to liquefaction, the 2012 International Building Code (IBC) requires the site be classified as a Site Class F. As per the IBC, a site class F requires a site-specific evaluation to determine the design ground motion parameters (S_{DS}, S_{D1}). However, there is an exception if the structure has a fundamental period of vibration of ½-second or less, then the design ground motion parameters can be determined using the IBC site class ignoring the liquefaction susceptibility (site class E), and a site-specific evaluation is not required. These parameters are included in the above table. Although the fundamental period of vibration is determined by the structural engineer and needs to be confirmed by the structural engineer, we understand that single-story structures often have fundamental periods of vibration of less than ½-second. Even then, the site would still be susceptible to earthquake liquefaction and the subsequent damaging effects, which are discussed below. Also, the structure still needs to be designed for a Site Class F, but the site-specific evaluation is not needed.

4.5.1 Liquefaction – Design Considerations

Very loose to medium dense sands were encountered in the borings. Due to the ground shaking motions anticipated, some of these sands are prone to densify above the groundwater level and potentially liquefy below the groundwater level. Seismically induced settlement of up to about 10 inches was calculated within the upper 30 feet of subgrade. Additional settlement could occur below this depth.

Foundation and building designs should account for this anticipated settlement and potential damage based on risk-to-benefit factors. Typical failures as a result of earthquakes can range from differential settlement of foundation and floor slab elements, utility line damages or severance, to complete building collapse. Protection from injury or loss of life is achievable through proper design and construction of foundation, building and equipment elements.

Mat foundations help mitigate differential settlement by structurally maintaining a rigid base for equipment and building walls. This is a viable foundation system to help limit major structural damages and loss of life as a result of earthquake events. During and after an earthquake, the structure would generally move as a unit and subside with the surrounding ground. Potential for equipment toppling and building collapse can be reduced or potentially eliminated with adequate structural design. Mat foundation sliding could be a concern during strong ground shaking, but could be mitigated with higher friction capacity bearing soils or passive resistance from below-grade elements below the frost line. These elements could be the mat foundation side walls, mat foundation keyway components, or deep foundation elements. Below-grade utility line damage could be reduced if the structure subsides relatively consistent with the surrounding ground.

Shallow spread footings that do not account for densifiable and liquefiable soils could result in severe differential settlement of individual elements from earthquakes.

4.5.2 Liquefaction Consequences

The consequences of soil liquefaction at this site are ground oscillation during an earthquake, permanent vertical displacements (post-reconsolidation settlement), and a reduction in soil strength. A summary of the principal effects on shallow and deep foundations is provided in the table below:

Liquefaction-Induced Hazard [1]	Effect on Shallow Foundations	Effect on Deep Foundations
Ground Oscillation	Compressive and tensile forces in the foundation	Shear forces and bending moments (kinematic loads) along the length of the foundation
Vertical Displacement (post-reconsolidation settlement)	Total and differential settlement	Downdrag loads and settlement
Reduction in Soil Strength	Total and differential settlement	Reduction in side friction and possibly end bearing capacities
Permanent lateral displacement (lateral spread) due to nearby dam failure	Horizontal forces likely resulting in significant sliding	Shear forces and bending moments along the length of the foundation

Our liquefaction analysis indicates that the magnitude of post-reconsolidation settlement is about 10 inches in the upper 30 feet of subgrade. Additional settlement could occur below this depth. In our opinion, a mat foundation could mitigate the potential consequences of liquefaction to the extent of providing the life safety requirements of the 2012 IBC. Flexible utility connections would reduce utility connection damage.

4.5.3 Earthquake and Liquefaction Settlement Reduction

If the predicted post-earthquake settlement is not acceptable, alternatives for reducing total and/or differential settlement include:

- Aggregate piers or vibro-stone columns,
- Vibro compaction,
- Earthquake drains,
- Soil grouting,
- Deep soil mixing,
- Mat foundation,
- Displacement piles, and
- Deep foundations and a structural slab.

We would be pleased to discuss these options with you, if they are deemed feasible. This report provides foundation recommendations to help mitigate the effects of earthquakes. However, even if these procedures are followed, some movement and cracking in the structures could still occur. The severity of cracking and other damage such as uneven floor slabs and structural distress will likely increase with stronger earthquake events. Eliminating the risk of movement and cosmetic distress may not be feasible, but it may be possible to further reduce the risk of movement if significantly more extensive measures are used during construction.

4.5.4 Soil Improvement

As stated previously, the predicted seismic settlement could be reduced if the sands are improved and/or densified via soil vibro compaction, vibro-stone columns, dynamic compaction, etc. Also, the foundation support could be enhanced using these soil improvement methods.

Vibro compaction consists of densifying the granular soils beneath the proposed building using vibration. A vibroflot is vibrated down into the soil on a grid pattern to specified depths to densify the sands prior to building construction.

Vibro-stone columns typically consist of a 22- to 24-inch diameter vibroflotted hole that is charged with clean aggregate. The vibroflot compacts this aggregate resulting in very stiff, high density aggregate piers. The compacted aggregate piers produce high lateral stresses within the surrounding soil matrix, thereby stiffening the reinforced composite soil/aggregate mass. This results in significant strengthening and stiffening of the foundation bearing layer to support floor slabs and footings within the required settlement tolerances.

Dynamic compaction is where a heavy weight is repeatedly dropped from a specified height on a grid pattern in the building area to densify the sand. The weight, drop height, and grid pattern are customized to achieve the desired improvement. This approach can have limited results if the groundwater level is within a few feet of the ground surface being improved.

These soil improvements are usually part of a foundation contractor's design-build system. Therefore, the subsurface exploration information contained in this report could be provided to the foundation contractors for detailed analysis and design and cost information.

4.6 Buoyancy

Groundwater was encountered as shallow as 4 feet below grade at the time of drilling. Groundwater level fluctuations occur due to seasonal variations, the amount of rainfall and runoff, and other factors. Groundwater levels above the bottom of the pools will create buoyant forces (i.e., uplift conditions) on the pools.

The pools should be designed to resist uplift pressures due to hydrostatic loading. Uplift pressure would be greatest when the pools are empty and a high groundwater level exists on site. The pools should be designed to resist a water pressure of 62.4 psf per foot of embedment below groundwater. The total uplift force could be resisted by the dead weight of the pool, the effective weight of any extensions of the foundations (i.e., tie-down mats or concrete dead men). A backfill total unit weight of 110 pcf could be used above the groundwater table and an effective unit weight of 55 pcf for backfill below the water table.

4.7 Lateral Earth Pressures

Reinforced concrete walls with unbalanced backfill levels on opposite sides, such as foundation or pool walls, should be designed for earth pressures at least equal to those indicated in the following table. Earth pressures will be influenced by structural design of the walls, conditions of wall restraint, methods of construction, and/or compaction and the strength of the materials being restrained. Two wall restraint conditions are shown. Active earth pressure is commonly used for design of free-standing cantilever retaining walls and assumes wall movement. The "at-rest" condition assumes no wall movement. The recommended design lateral earth pressures do not include a factor of safety and do not provide for possible hydrostatic pressure on the walls.

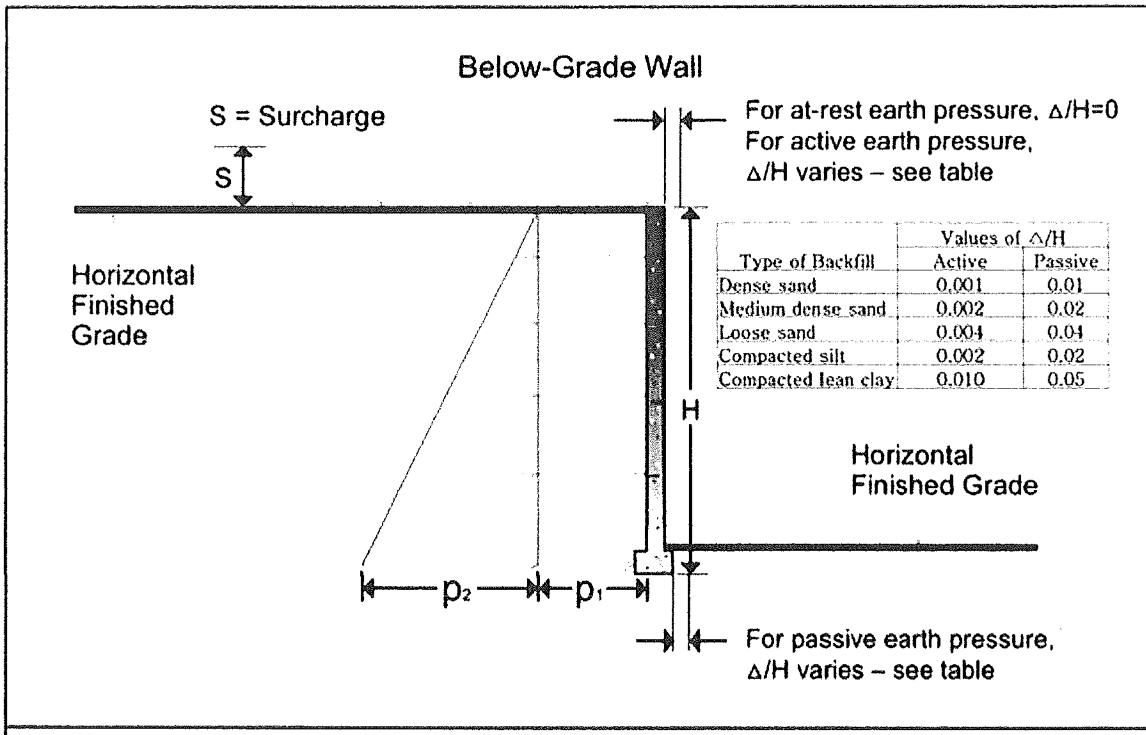


Table from Movements Required to Reach Active or Passive Earth Pressure Conditions (Clough and Duncan, 1991)

Earth Pressure Coefficients

Earth Pressure Conditions	Coefficient for Backfill Type	Equivalent Fluid Density (pcf)	Surcharge Pressure, p_1 (psf)	Earth Pressure, p_2 (psf)
Active (K_a)	Granular - 0.33	40	(0.33)S	(40)H
	Lean Clay - 0.39	50	(0.39)S	(50)H

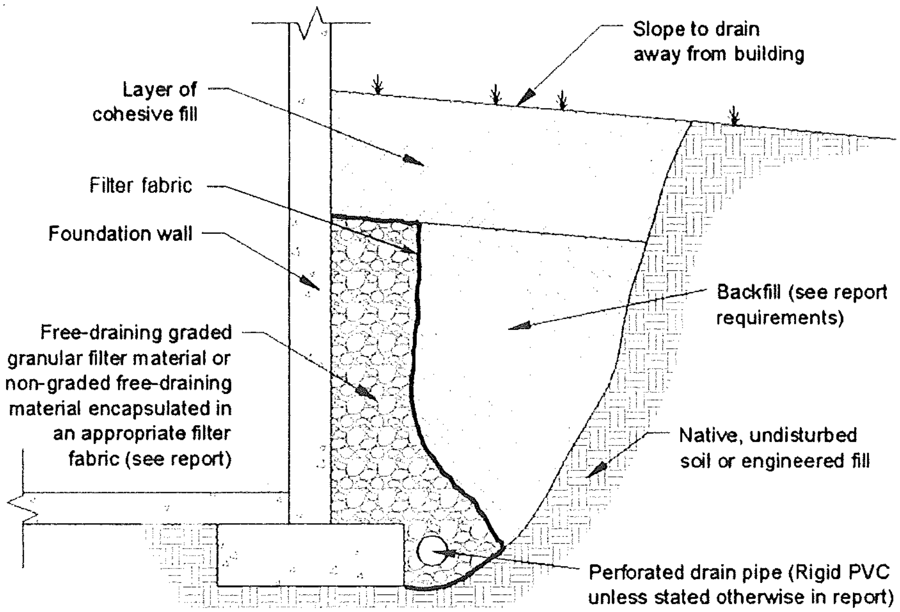
Earth Pressure Conditions	Coefficient for Backfill Type	Equivalent Fluid Density (pcf)	Surcharge Pressure, p_1 (psf)	Earth Pressure, p_2 (psf)
At-Rest (K_o)	Granular - 0.50	60	(0.50)S	(60)H
	Lean Clay - 0.56	70	(0.56)S	(70)H
Passive (K_p)	Granular - 3.0	360	---	---
	Lean Clay - 2.5	300	---	---

Applicable conditions to the above include:

- For active earth pressure, wall must rotate about base, with top lateral movements as indicated in the above tables
- For passive earth pressure to develop, wall must move horizontally to mobilize resistance as indicated in the above tables
- Uniform surcharge, where S is surcharge pressure
- In-situ soil backfill weight a maximum of 120 pcf
- Horizontal backfill, compacted at 95% to 98% of standard its Proctor maximum dry density
- Loading from heavy compaction equipment not included
- No hydrostatic pressures acting on wall
- No dynamic loading
- No safety factor included in soil parameters
- Ignore passive pressure in frost zone

Backfill placed against structures should consist of granular soils or low plasticity cohesive soils (i.e., fat clay is not acceptable backfill material). For the granular values to be valid, the granular backfill must extend out from the base of the wall at an angle of at least 45, 45 and 60 degrees from vertical for the active, at-rest and passive cases, respectively. To calculate the resistance to sliding, a value of 0.35 should be used as the ultimate coefficient of friction between the footing and the underlying soil.

A perforated rigid plastic or metal drain line installed behind the base of walls that extend below adjacent grade is recommended to limit hydrostatic loading on the walls. The invert of a drain line around a below-grade wall should be placed near foundation bearing level. The drain line should be sloped to provide positive gravity drainage or to a sump pit and pump. The drain line should be surrounded by clean, free-draining granular material having less than 5 percent (by weight) passing the No. 200 sieve. The free-draining aggregate should be encapsulated in a filter fabric. The granular fill should extend to within 2 feet of final grade, where it should be capped with compacted cohesive fill to reduce infiltration of surface water into the drain system.



As an alternative to free-draining granular fill, a pre-fabricated drainage composite may be used. A pre-fabricated drainage composite is a plastic drainage core or mesh which is covered with filter fabric to prevent soil intrusion, and fastened to the wall prior to placing backfill.

If controlling hydrostatic pressure behind the wall as described above is not possible, then combined hydrostatic and lateral earth pressures should be calculated for lean clay backfill using an equivalent fluid weighing 90 and 100 pcf for active and at-rest conditions, respectively. For granular backfill, an equivalent fluid weighing 85 and 90 pcf should be used for active and at-rest, respectively. These pressures do not include the influence of surcharge, equipment or pavement loading, which should be added. Heavy equipment should not operate within a distance closer than the exposed height of retaining walls to prevent lateral pressures greater than those provided.

5.0 GENERAL COMMENTS

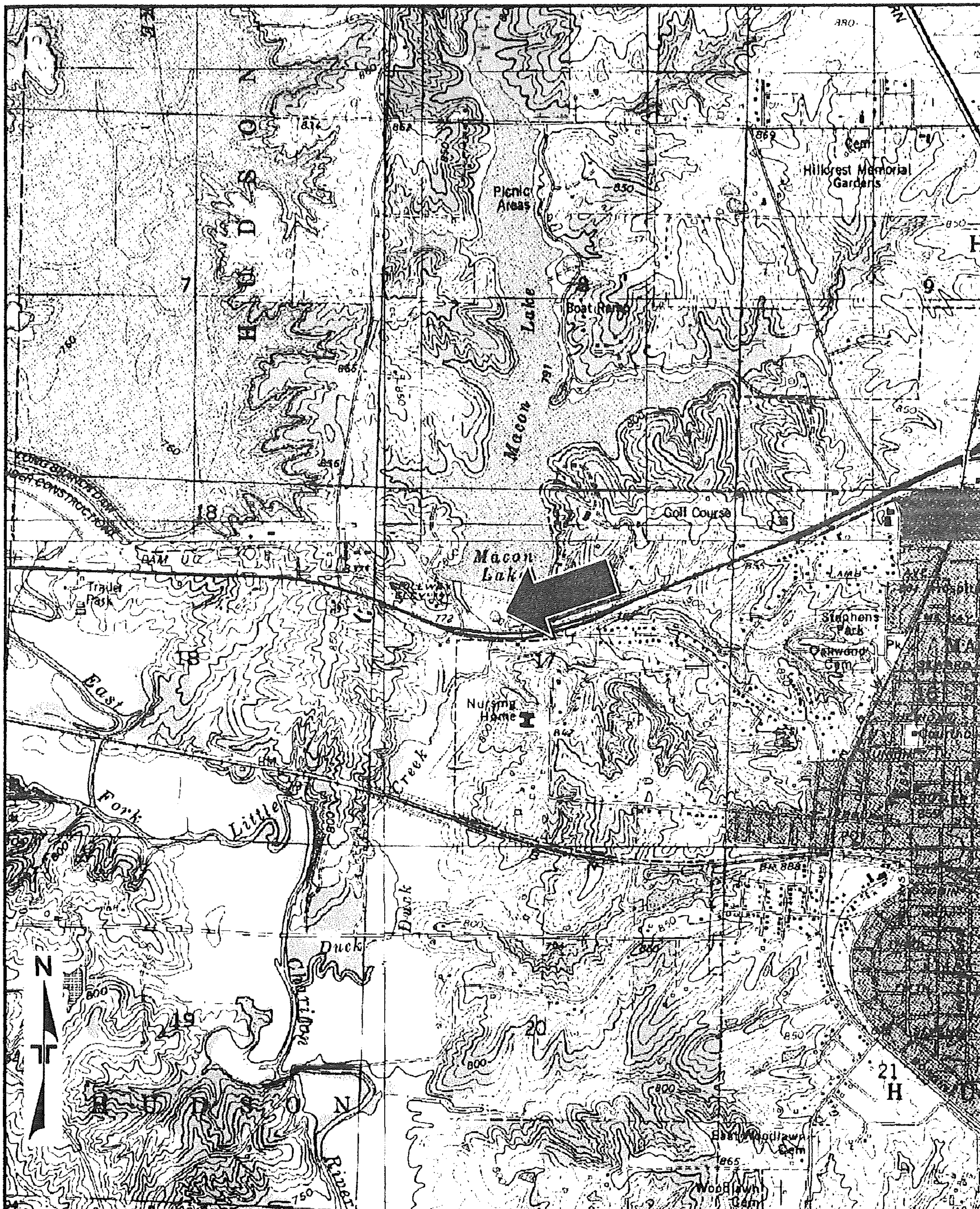
Terracon should be retained to review the final design plans and specifications so comments can be made regarding interpretation and implementation of our geotechnical recommendations in the design and specifications. Terracon should also be retained to provide observation and testing services during grading, excavation, foundation construction and other earth-related construction phases of the project.

The analysis and recommendations presented in this report are based upon the data obtained from the boring performed at the indicated location and from other information discussed in this report. This report does not reflect variations that may occur across the site, or due to the modifying effects of construction or weather. The nature and extent of such variations may not become evident until during or after construction. If variations appear, we should be immediately notified so that further evaluation and supplemental recommendations can be provided.

The scope of services for this project does not include either specifically or by implication any environmental or biological (e.g., mold, fungi, bacteria) assessment of the site or identification or prevention of pollutants, hazardous materials or conditions.

This report has been prepared for the exclusive use of our client for specific application to the project discussed and has been prepared in accordance with generally accepted geotechnical engineering practices. No warranties, either express or implied, are intended or made. Site safety, excavation support, and dewatering requirements are the responsibility of others. In the event that changes in the nature, design, or location of the project as outlined in this report are planned, the conclusions and recommendations contained in this report shall not be considered valid unless Terracon reviews the changes and either verifies or modifies the conclusions of this report in writing.

APPENDIX A
FIELD EXPLORATION



TOPOGRAPHIC MAP IMAGE COURTESY OF THE U.S. GEOLOGICAL SURVEY
QUADRANGLES INCLUDE: BEVIER NORTH, MO (1/1/1979), AXTELL, MO (1/1/1979), BEVIER SOUTH, MO (1/1/1978) and MACON, MO (1/1/1971).

Project Manager: BWR	Project No. 09165033	Terracon	SITE LOCATION MAP	Exhibit
Drawn by: PBL	Scale: 1"=2,000'	3601 Mojave Ct Ste A	Macon Aquatic Center	A-1
Checked by: AGM	File Name: 09165033.A-1-3	Columbia, MO 65202-4043	1203 Noll Dr	
Approved by: AGM	Date: 6/13/16		Macon, Missouri	

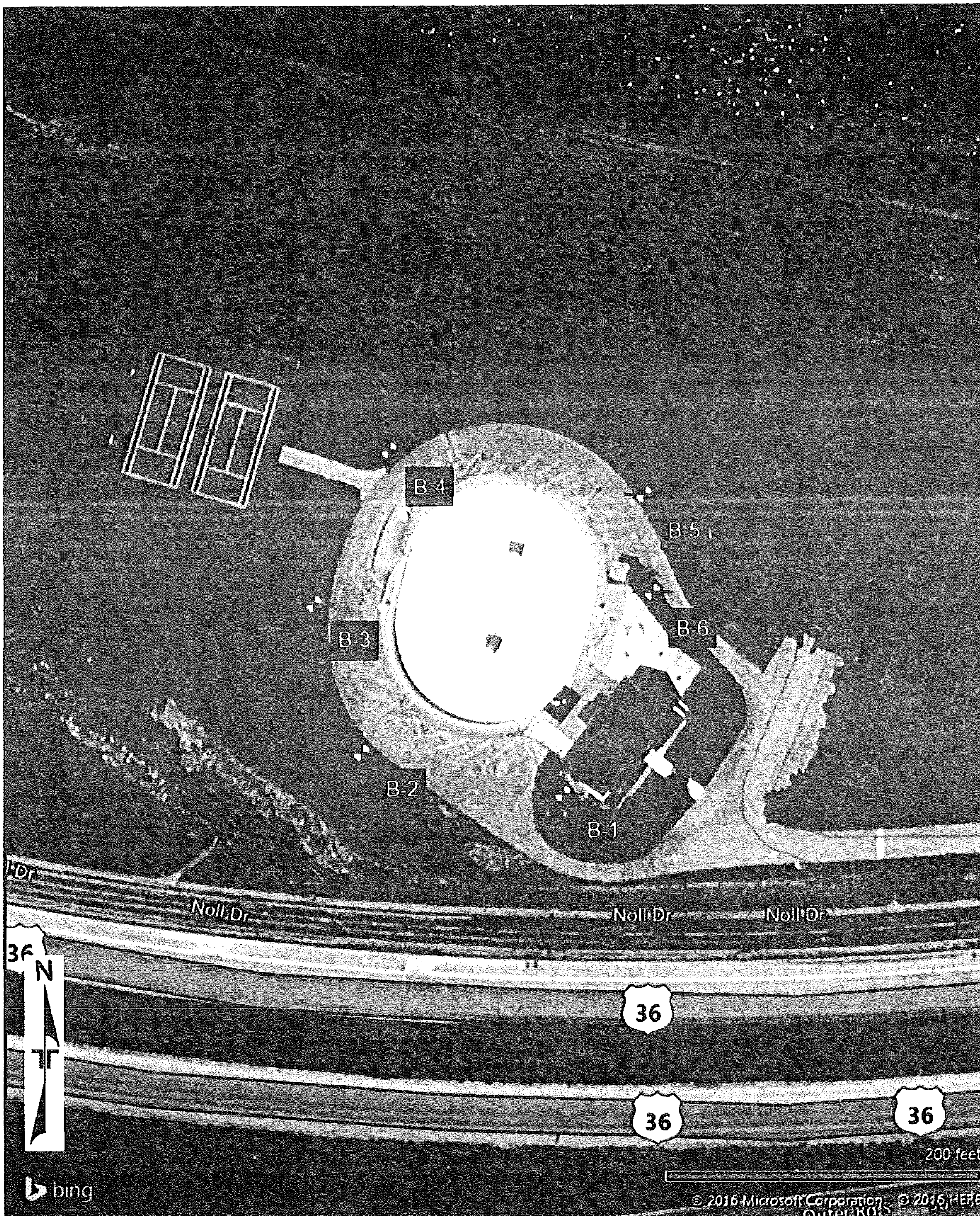


DIAGRAM IS FOR GENERAL LOCATION ONLY, AND IS NOT INTENDED FOR CONSTRUCTION PURPOSES

AERIAL PHOTOGRAPHY PROVIDED BY MICROSOFT BING MAPS

Project Manager: BWR	Project No. 09165033	Terracon	EXPLORATION DIAGRAM	Exhibit
Drawn by: PBL	Scale: AS SHOWN	3601 Mojave Ct Ste A	Macon Aquatic Center	A-2
Checked by: AGM	File Name: 09165033.A-1-3	Columbia, MO 65202-4043	1203 Noll Dr	
Approved by: AGM	Date: 6/13/16		Macon, Missouri	

Field Exploration Description

The proposed boring locations were laid out in the field by the drill crew using a hand-held GPS unit and reference to site features. The ground surface elevations at the boring locations were obtained by the drill crew using an engineer's level and grade rod and is rounded to the nearest ½-foot. The elevation is referenced to the south side of the existing pool which was assigned an elevation of 100.0 feet. The locations and elevations of the borings should be considered accurate only to the degree implied by the means and methods used to define them.

The borings were drilled with a CME-550X, ATV-mounted, rotary drill rig using continuous-flight, solid-stem augers and mud rotary techniques to advance the boreholes. Samples of the soils encountered in the borings were obtained using split-barrel and thin-walled tube sampling procedures.

In the split-barrel sampling procedure, the number of blows required to advance a standard 2-inch O.D. split-barrel sampler the last 12 inches of the typical total 18-inch penetration by means of a 140-pound hammer with a free fall of 30 inches, is the standard penetration resistance (SPT N-value). This value is used to estimate the in-situ relative density of cohesionless soils and the consistency of cohesive soils.

A CME automatic SPT hammer was used to advance the split-barrel sampler in the borings performed on this site. A greater efficiency is achieved with the automatic hammer compared to the conventional safety hammer operated with a cathead and rope. This higher efficiency (90% for drill rig 960) has an appreciable effect on the SPT N-value. The effect of the automatic hammer's efficiency has been considered in the interpretation and analysis of the subsurface information for this report.

In the thin-walled tube sampling procedure, a seamless thin-walled steel tube with a sharpened beveled edge is pushed hydraulically into the cohesive or moderately cohesive soil at a selected depth at the base of the borehole. A relatively undisturbed sample of the soil is retained in the tube, and extracted in the laboratory for further testing.

The samples were tagged for identification, sealed to reduce moisture loss, and taken to our laboratory for further observation, testing, and classification. Information provided on the boring logs attached to this report includes soil descriptions, consistency evaluations, boring depths, sampling intervals, and groundwater conditions. The borings were backfilled with auger cuttings prior to the drill crew leaving the site.

A field log of each boring was prepared by the drill crew. These logs included visual classifications of the materials encountered during drilling as well as the driller's interpretation of the subsurface conditions between samples. Final boring logs included with this report represent the engineer's interpretation of the field logs and include modifications based on laboratory observation and tests of the samples.

BORING LOG NO. B-1

PROJECT: Macon Aquatic Center

CLIENT: City of Macon, Macon, Missouri

SITE: 1203 Noll Drive, Macon, Missouri

LOCATION See Exhibit A-2

Latitude: 39.746771° Longitude: -92.494236°

Approximate Surface Elev: 99.5 (Ft.) +/-

DEPTH	Description	ELEVATION (Ft.)	DEPTH (Ft.)	WATER LEVEL OBSERVATIONS	SAMPLE TYPE	RECOVERY (In.)	FIELD TEST RESULTS	SAMPLE	LABORATORY TORVANE/HP (psf)	UNCONFINED COMPRESSIVE STRENGTH (psf)	WATER CONTENT (%)	DRY UNIT WEIGHT (pcf)	ATTERBERG LIMITS LL-PL-PI
3.0	**FILL - LEAN CLAY**, trace sand, brown	96.5+/-				6	1-2-2 N=4	1	1500 (HP)		23		
5.5	**FILL - LEAN CLAY**, with sand and gravel, reddish brown	94+/-	5			10		2	2000 (HP)		24	102	40-15-25
12.0	**CLAYEY SILT (ML)**, gray, very soft	87.5+/-		▽		14	1-1-1 N=2	3			27		
			10	▽		18	0-0-0 N=0	4			27		
23.0	**SAND (SP)**, fine grained, gray, very loose to loose	76.5+/-	15			12	0-0-1 N=1	5			21		
			20			10	1-2-3 N=5	6			26		
28.0	**CLAYEY SILT (ML)**, trace sand, gray, very soft to soft	71.5+/-	25			12	0-0-1 N=1	7			23		
30.0	**SAND (SP)**, fine grained, gray, very loose	69.5+/-	30			12	0-1-2 N=3	8			21		
	Boring Terminated at 30 Feet												

Stratification lines are approximate. In-situ, the transition may be gradual.

Hammer Type: Automatic SPT Hammer

Advancement Method:
Solid-Stem Auger

Abandonment Method:
Boring backfilled with soil cuttings upon completion.

See Exhibit A-3 for description of field procedures.

See Appendix B for description of laboratory procedures and additional data (if any).

See Appendix C for explanation of symbols and abbreviations.

Elevations were measured in the field using an engineer's level and grade rod.

Notes:

WATER LEVEL OBSERVATIONS

▽ *While sampling*

▽ *At completion of drilling*

Terracon
11600 Lilburn Park Rd
Saint Louis, MO

Boring Started: 6/1/2016	Boring Completed: 6/1/2016
Drill Rig: CME-550	Driller: SB
Project No.: 9165033	Exhibit: A-4

BORING LOG NO. B-2

PROJECT: Macon Aquatic Center

CLIENT: City of Macon, Macon, Missouri

SITE: 1203 Noll Drive, Macon, Missouri

LOCATION See Exhibit A-2

Latitude: 39.746843° Longitude: -92.494692°

Approximate Surface Elev: 98.5 (Ft.) +/-

DEPTH	DESCRIPTION	ELEVATION (Ft.)	DEPTH (Ft.)	WATER LEVEL OBSERVATIONS	SAMPLE TYPE	RECOVERY (In.)	FIELD TEST RESULTS	SAMPLE	LABORATORY TORVANE/HP (psf)	UNCONFINED COMPRESSIVE STRENGTH (psf)	WATER CONTENT (%)	DRY UNIT WEIGHT (pcf)	ATTERBERG LIMITS LL-PL-PI
	FILL - LEAN CLAY, trace gravel, brown				SPT	6	2-2-2 N=4	1	2000 (HP)		19		
3.0		95.5+/-											
	FILL - LEAN CLAY, with sand and gravel, reddish brown			▽ (at completion)	SPT	7	1-2-3 N=5	2	2000 (HP)		25		
5.5		93+/-	5										
	CLAYEY SILT (ML), gray, very soft. Atterberg Limits test indicates sample 3 is not plastic			▽ (while sampling)	Shelby Tube	24		3		402	25	102	
					SPT	14	0-0-0 N=0	4			22		
			10										
13.0		85.5+/-											
	SAND (SP), trace silt, fine grained, gray, very loose				SPT	18	1-1-1 N=2	5			22		
			15										
	with silt				SPT	9	1-1-2 N=3	6			30		
			20										
23.0		75.5+/-											
	CLAYEY SILT (ML), trace sand, gray, very soft				SPT	18	0-0-0 N=0	7			23		
			25										
					SPT		1-1-1 N=2	8			23		
30.0		68.5+/-	30										
	Boring Terminated at 30 Feet												

Stratification lines are approximate. In-situ, the transition may be gradual.

Hammer Type: Automatic SPT Hammer

Advancement Method:
Solid-Stem Auger

Abandonment Method:
Boring backfilled with soil cuttings upon completion.

See Exhibit A-3 for description of field procedures.

See Appendix B for description of laboratory procedures and additional data (if any).

See Appendix C for explanation of symbols and abbreviations.

Elevations were measured in the field using an engineer's level and grade rod.

Notes:

WATER LEVEL OBSERVATIONS

▽ *While sampling*

▽ *At completion of drilling*

Terracon
11600 Lilburn Park Rd
Saint Louis, MO

Boring Started: 6/1/2016	Boring Completed: 6/1/2016
Drill Rig: CME-550	Driller: SB
Project No.: 9165033	Exhibit A-5

BORING LOG NO. B-3

PROJECT: Macon Aquatic Center

CLIENT: City of Macon
Macon, Missouri

SITE: 1203 Noll Drive
Macon, Missouri

LOCATION See Exhibit A-2

Latitude: 39.747102° Longitude: -92.494801°

Approximate Surface Elev: 98.5 (Ft.) +/-

DEPTH	Material Description	ELEVATION (Ft.)	DEPTH (Ft.)	WATER LEVEL OBSERVATIONS	SAMPLE TYPE	RECOVERY (In.)	FIELD TEST RESULTS	SAMPLE	LABORATORY TORVANE/HP (psf)	UNCONFINED COMPRESSIVE STRENGTH (psf)	WATER CONTENT (%)	DRY UNIT WEIGHT (pcf)	ATTERBERG LIMITS LL-PL-PI
	FILL - LEAN CLAY, trace sand and gravel, reddish brown					12	2-2-2 N=4	1	3000 (HP)		17		
						13	3-3-3 N=6	2	1000 (HP)		22		
5.5		93+/-	5										
	LEAN CLAY (CL), gray and brown, medium stiff					18	1-3-2 N=5	3	2000 (HP)		23		
								4	2500 (HP)	1650	22	107	
			10										
13.0		85.5+/-											
	CLAYEY SILT (ML), gray, medium stiff					18	0-2-3 N=5	5			24		
			15										
	trace sand					15	0-2-3 N=5	6			24		
			20										
22.5		76+/-											
	SANDY LEAN CLAY (CL), gray, medium stiff					18	0-2-3 N=5	7	2500 (HP)		26		
			25										
27.0		71.5+/-											
	CLAYEY SILT (ML), gray, medium stiff					18	1-2-3 N=5	8			23		
30.0		68.5+/-	30										
	Boring Terminated at 30 Feet												

Stratification lines are approximate. In-situ, the transition may be gradual.

Hammer Type: Automatic SPT Hammer

Advancement Method:
Solid-Stem Auger

Abandonment Method:
Boring backfilled with soil cuttings upon completion.

See Exhibit A-3 for description of field procedures.

See Appendix B for description of laboratory procedures and additional data (if any).

See Appendix C for explanation of symbols and abbreviations.

Elevations were measured in the field using an engineer's level and grade rod.

Notes:

WATER LEVEL OBSERVATIONS

▽ *While sampling*

▼ *At completion of drilling*

Terracon
11600 Lilburn Park Rd
Saint Louis, MO

Boring Started: 6/1/2016	Boring Completed: 6/1/2016
Drill Rig: CME-550	Driller: SB
Project No.: 9165033	Exhibit: A-6

THIS BORING LOG IS NOT VALID IF SEPARATED FROM ORIGINAL REPORT. GEO SMART LOG-NO WELL 09165033 LOGS.GPJ TERRACON2015.GDT 6/22/16

BORING LOG NO. B-4

PROJECT: Macon Aquatic Center

CLIENT: City of Macon
Macon, Missouri

SITE: 1203 Noll Drive
Macon, Missouri

LOCATION See Exhibit A-2

Latitude: 39.747368° Longitude: -92.494628°

Approximate Surface Elev: 98.5 (Ft.) +/-

DEPTH / Material Description / ELEVATION (Ft.)	DEPTH (Ft.)	WATER LEVEL OBSERVATIONS	SAMPLE TYPE	RECOVERY (In.)	FIELD TEST RESULTS	SAMPLE	LABORATORY TORVANE/HP (psf)	UNCONFINED COMPRESSIVE STRENGTH (psf)	WATER CONTENT (%)	DRY UNIT WEIGHT (pcf)	ATTERBERG LIMITS LL-PL-PI
FILL - LEAN CLAY, trace gravel and sand, reddish brown and gray to gray				12	1-2-2 N=4	1	2000 (HP)		18		
		▽ While sampling; ▽ At completion of drilling		18	1-2-3 N=5	2	1500 (HP)		23		
5.5 — 93+/-	5										
CLAYEY SILT (ML), gray, soft to medium stiff				24		3	500 (HP)	1561	24	103	
				18	0-0-2 N=2	4			25		
	10										
12.0 — 86.5+/-											
CLAYEY SAND (SP), fine grained, gray, very loose to loose				18	1-2-3 N=5	5			24		
	15										
18.0 — 80.5+/-											
LEAN CLAY (CL), gray, medium stiff				18	2-2-3 N=5	6	1500 (HP)		29		
	20										
				18	1-2-3 N=5	7	1500 (HP)		28		
	25										
27.0 — 71.5+/-											
SILT (ML), trace sand, gray, soft				3	0-2-2 N=4	8			25		
30.0 — 68.5+/-	30										
Boring Terminated at 30 Feet											

Stratification lines are approximate. In-situ, the transition may be gradual.

Hammer Type: Automatic SPT Hammer

Advancement Method:
Solid-Stem Auger

Abandonment Method:
Boring backfilled with soil cuttings upon completion.

See Exhibit A-3 for description of field procedures.

See Appendix B for description of laboratory procedures and additional data (if any).

See Appendix C for explanation of symbols and abbreviations.

Elevations were measured in the field using an engineer's level and grade rod.

Notes:

WATER LEVEL OBSERVATIONS

▽ *While sampling*

▽ *At completion of drilling*

Terracon
11600 Lilburn Park Rd
Saint Louis, MO

Boring Started: 6/1/2016	Boring Completed: 6/1/2016
Drill Rig: CME-550	Driller: SB
Project No.: 9165033	Exhibit: A-7

BORING LOG NO. B-5

PROJECT: Macon Aquatic Center

**CLIENT: City of Macon
Macon, Missouri**

**SITE: 1203 Noll Drive
Macon, Missouri**

LOCATION See Exhibit A-2
Latitude: 39.74729° Longitude: -92.494048°
Approximate Surface Elev: 98.5 (Ft.) +/-

GRAPHIC LOG	DEPTH / MATERIAL DESCRIPTION / ELEVATION (Ft.)	DEPTH (Ft.)	WATER LEVEL OBSERVATIONS	SAMPLE TYPE	RECOVERY (In.)	FIELD TEST RESULTS	SAMPLE	LABORATORY TORVANE/HP (psf)	UNCONFINED COMPRESSIVE STRENGTH (psf)	WATER CONTENT (%)	DRY UNIT WEIGHT (pcf)	ATTERBERG LIMITS LL-PL-PI
	FILL - LEAN CLAY, trace gravel, brown and gray				13	1-2-3 N=5	1	2000 (HP)		19		
	3.0 — 95.5+/-											
	FILL - LEAN CLAY, gray and brown				14	3-4-4 N=8	2	2000 (HP)		22		
		5			6	0-1-2 N=3	3	1000 (HP)		24		35-16-19
	9.0 — 89.5+/-											
	CLAYEY SAND (SP), fine grained, gray, very loose	10	▽ (at completion)		6	0-0-0 N=0	4	500 (HP)		29		
			▽ (while sampling)									
	loose	15			7	2-2-2 N=4	5			22		
	trace organics	20			12	0-0-0 N=0	6			25		
	25.0 — 73.5+/-	25			18	1-1-1 N=2	7			26		
	Boring Terminated at 25 Feet											

Stratification lines are approximate. In-situ, the transition may be gradual.

Hammer Type: Automatic SPT Hammer

Advancement Method:
Solid-Stem Auger

Abandonment Method:
Boring backfilled with soil cuttings upon completion.

See Exhibit A-3 for description of field procedures.

See Appendix B for description of laboratory procedures and additional data (if any).

See Appendix C for explanation of symbols and abbreviations.

Elevations were measured in the field using an engineer's level and grade rod.

Notes:

WATER LEVEL OBSERVATIONS

▽ *While sampling*

▽ *At completion of drilling*

Terracon
11600 Lilburn Park Rd
Saint Louis, MO

Boring Started: 6/1/2016	Boring Completed: 6/1/2016
Drill Rig: CME-550	Driller: SB
Project No.: 9165033	Exhibit: A-8

BORING LOG NO. B-6

PROJECT: Macon Aquatic Center

CLIENT: City of Macon
Macon, Missouri

SITE: 1203 Noll Drive
Macon, Missouri

LOCATION See Exhibit A-2

Latitude: 39.747121° Longitude: -92.49403°

Approximate Surface Elev: 98 (Ft.) +/-

GRAPHIC LOG	DEPTH / MATERIAL DESCRIPTION / ELEVATION (Ft.)	DEPTH (Ft.)	WATER LEVEL OBSERVATIONS	SAMPLE TYPE	RECOVERY (In.)	FIELD TEST RESULTS	SAMPLE	LABORATORY TORVANE/HP (psf)	UNCONFINED COMPRESSIVE STRENGTH (psf)	WATER CONTENT (%)	DRY UNIT WEIGHT (pcf)	ATTERBERG LIMITS LL-PL-PI
	FILL - LEAN CLAY, trace sand and gravel, gray and brown				0	3-4-3 N=7	1	2000 (HP)				
		5			9	2-3-4 N=7	2	1000 (HP)		23		
	6.0 — 92+/-											
	LEAN CLAY (CL), gray, soft				18	0-1-2 N=3	3			28		
			∇		24		4		513	29	96	
		10										
		15			9	0-0-3 N=3	5			26		
	16.0 — 82+/-											
	LEAN CLAY (CL), gray, medium stiff				18	0-3-3 N=6	6	2000 (HP)		24		
	20.0 — 78+/-	20										
	Boring Terminated at 20 Feet											

Stratification lines are approximate. In-situ, the transition may be gradual.

Hammer Type: Automatic SPT Hammer

Advancement Method:
Solid-Stem Auger

Abandonment Method:
Boring backfilled with soil cuttings upon completion.

See Exhibit A-3 for description of field procedures.

See Appendix B for description of laboratory procedures and additional data (if any).

See Appendix C for explanation of symbols and abbreviations.

Elevations were measured in the field using an engineer's level and grade rod.

Notes:

WATER LEVEL OBSERVATIONS

∇ *While sampling*

∇ *At completion of drilling*

Terracon
11600 Lilburn Park Rd
Saint Louis, MO

Boring Started: 6/1/2016	Boring Completed: 6/1/2016
Drill Rig: CME-550	Driller: SB
Project No.: 9165033	Exhibit: A-9

APPENDIX B
SUPPORTING INFORMATION

Laboratory Testing

Soil samples were tested in the laboratory to measure their natural water content (ASTM D4959). The test results are provided on the boring logs included in Appendix A. The thin-walled tube samples were tested for dry density and unconfined compressive strength (ASTM D2166). Atterberg limits tests (ASTM D4318) were performed on selected samples.

As part of the testing program, samples were examined in our laboratory and classified in accordance with the General Notes and the Unified Soil Classification System (USCS) based on the material's texture and plasticity (ASTM D2487 and ASTM D2488). The USCS group symbol is shown on the boring logs, and a brief description of the USCS is included with this report in Appendix C.

Procedural standards noted above are for reference to general methodology. In some cases, variations to methods are applied as a result of local practice or professional judgment.

APPENDIX C
SUPPORTING DOCUMENTS

GENERAL NOTES

DESCRIPTION OF SYMBOLS AND ABBREVIATIONS

SAMPLING

- Auger
- Split Spoon
- Shelby Tube
- Macro Core
- Ring Sampler
- Rock Core
- Grab Sample
- No Recovery

WATER LEVEL

- Water Initially Encountered
- Water Level After a Specified Period of Time
- Water Level After a Specified Period of Time

Water levels indicated on the soil boring logs are the levels measured in the borehole at the times indicated. Groundwater level variations will occur over time. In low permeability soils, accurate determination of groundwater levels is not possible with short term water level observations.

FIELD TESTS

(HP)	Hand Penetrometer
(T)	Torvane
(b/f)	Standard Penetration Test (blows per foot)
(PID)	Photo-Ionization Detector
(OVA)	Organic Vapor Analyzer

DESCRIPTIVE SOIL CLASSIFICATION

Soil classification is based on the Unified Soil Classification System. Coarse Grained Soils have more than 50% of their dry weight retained on a #200 sieve; their principal descriptors are: boulders, cobbles, gravel or sand. Fine Grained Soils have less than 50% of their dry weight retained on a #200 sieve; they are principally described as clays if they are plastic, and silts if they are slightly plastic or non-plastic. Major constituents may be added as modifiers and minor constituents may be added according to the relative proportions based on grain size. In addition to gradation, coarse-grained soils are defined on the basis of their in-place relative density and fine-grained soils on the basis of their consistency.

LOCATION AND ELEVATION NOTES

Unless otherwise noted, Latitude and Longitude are approximately determined using a hand-held GPS device. The accuracy of such devices is variable. Surface elevation data annotated with +/- indicates that no actual topographical survey was conducted to confirm the surface elevation. Instead, the surface elevation was approximately determined from topographic maps of the area.

STRENGTH TERMS

RELATIVE DENSITY OF COARSE-GRAINED SOILS (More than 50% retained on No. 200 sieve.) Density determined by Standard Penetration Resistance			CONSISTENCY OF FINE-GRAINED SOILS (50% or more passing the No. 200 sieve.) Consistency determined by laboratory shear strength testing, field visual-manual procedures or standard penetration resistance			
Descriptive Term (Density)	Standard Penetration or N-Value Blows/Ft.	Ring Sampler Blows/Ft.	Descriptive Term (Consistency)	Unconfined Compressive Strength, Qu, psf	Standard Penetration or N-Value Blows/Ft.	Ring Sampler Blows/Ft.
Very Loose	0 - 3	0 - 6	Very Soft	less than 500	0 - 1	< 3
Loose	4 - 9	7 - 18	Soft	500 to 1,000	2 - 4	3 - 4
Medium Dense	10 - 29	19 - 58	Medium-Stiff	1,000 to 2,000	4 - 8	5 - 9
Dense	30 - 50	59 - 98	Stiff	2,000 to 4,000	8 - 15	10 - 18
Very Dense	> 50	≥ 99	Very Stiff	4,000 to 8,000	15 - 30	19 - 42
			Hard	> 8,000	> 30	> 42

RELATIVE PROPORTIONS OF SAND AND GRAVEL

Descriptive Term(s) of other constituents	Percent of Dry Weight
Trace	< 15
With	15 - 29
Modifier	> 30

GRAIN SIZE TERMINOLOGY

Major Component of Sample	Particle Size
Boulders	Over 12 in. (300 mm)
Cobbles	12 in. to 3 in. (300mm to 75mm)
Gravel	3 in. to #4 sieve (75mm to 4.75 mm)
Sand	#4 to #200 sieve (4.75mm to 0.075mm
Silt or Clay	Passing #200 sieve (0.075mm)

RELATIVE PROPORTIONS OF FINES

Descriptive Term(s) of other constituents	Percent of Dry Weight
Trace	< 5
With	5 - 12
Modifier	> 12

PLASTICITY DESCRIPTION

Term	Plasticity Index
Non-plastic	0
Low	1 - 10
Medium	11 - 30
High	> 30

Terracon

UNIFIED SOIL CLASSIFICATION SYSTEM

Criteria for Assigning Group Symbols and Group Names Using Laboratory Tests[A]				Soil Classification: Group Symbol	Soil Classification: Group Name[B]
Coarse Grained Soils: More than 50% retained on No. 200 sieve	**Gravels:** More than 50% of coarse fraction retained on No. 4 sieve	**Clean Gravels:** Less than 5% fines[C]	$Cu \geq 4$ and $1 \leq Cc \leq 3$[E]	GW	Well-graded gravel[F]
			$Cu < 4$ and/or $1 > Cc > 3$[E]	GP	Poorly graded gravel[F]
		Gravels with Fines: More than 12% fines[C]	Fines classify as ML or MH	GM	Silty gravel[F,G,H]
			Fines classify as CL or CH	GC	Clayey gravel[F,G,H]
	Sands: 50% or more of coarse fraction passes No. 4 sieve	**Clean Sands:** Less than 5% fines[D]	$Cu \geq 6$ and $1 \leq Cc \leq 3$[E]	SW	Well-graded sand[I]
			$Cu < 6$ and/or $1 > Cc > 3$[E]	SP	Poorly graded sand[I]
		Sands with Fines: More than 12% fines[D]	Fines classify as ML or MH	SM	Silty sand[G,H,I]
			Fines classify as CL or CH	SC	Clayey sand[G,H,I]
Fine-Grained Soils: 50% or more passes the No. 200 sieve	**Silts and Clays:** Liquid limit less than 50	**Inorganic:**	PI > 7 and plots on or above "A" line[J]	CL	Lean clay[K,L,M]
			PI < 4 or plots below "A" line[J]	ML	Silt[K,L,M]
		Organic:	Liquid limit - oven dried / Liquid limit - not dried < 0.75	OL	Organic clay[K,L,M,N]
					Organic silt[K,L,M,O]
	Silts and Clays: Liquid limit 50 or more	**Inorganic:**	PI plots on or above "A" line	CH	Fat clay[K,L,M]
			PI plots below "A" line	MH	Elastic Silt[K,L,M]
		Organic:	Liquid limit - oven dried / Liquid limit - not dried < 0.75	OH	Organic clay[K,L,M,P]
					Organic silt[K,L,M,Q]
Highly organic soils:	Primarily organic matter, dark in color, and organic odor			PT	Peat

[A] Based on the material passing the 3-inch (75-mm) sieve

[B] If field sample contained cobbles or boulders, or both, add "with cobbles or boulders, or both" to group name.

[C] Gravels with 5 to 12% fines require dual symbols: GW-GM well-graded gravel with silt, GW-GC well-graded gravel with clay, GP-GM poorly graded gravel with silt, GP-GC poorly graded gravel with clay.

[D] Sands with 5 to 12% fines require dual symbols: SW-SM well-graded sand with silt, SW-SC well-graded sand with clay, SP-SM poorly graded sand with silt, SP-SC poorly graded sand with clay

[E] $Cu = D_{60}/D_{10}$ $\quad Cc = \dfrac{(D_{30})^2}{D_{10} \times D_{60}}$

[F] If soil contains ≥ 15% sand, add "with sand" to group name.

[G] If fines classify as CL-ML, use dual symbol GC-GM, or SC-SM.

[H] If fines are organic, add "with organic fines" to group name.

[I] If soil contains ≥ 15% gravel, add "with gravel" to group name.

[J] If Atterberg limits plot in shaded area, soil is a CL-ML, silty clay.

[K] If soil contains 15 to 29% plus No. 200, add "with sand" or "with gravel," whichever is predominant.

[L] If soil contains ≥ 30% plus No. 200 predominantly sand, add "sandy" to group name.

[M] If soil contains ≥ 30% plus No. 200, predominantly gravel, add "gravelly" to group name.

[N] PI ≥ 4 and plots on or above "A" line.

[O] PI < 4 or plots below "A" line.

[P] PI plots on or above "A" line.

[Q] PI plots below "A" line.