CGC, Inc.
Construction • Geotechnical
Consulting Engineering/Testing

June 2, 2017
C16051-14

Mr. Dave Schaller
City of Madison
Department of Public Works
Engineering Division - Facilities and Sustainability
City-County Building, Room 115
210 Martin Luther King, Jr. Blvd.
Madison, WI 53703-3342

Re: Supplemental Geotechnical Exploration Report
Proposed Judge Doyle Square – Block 88
Between E. Doty Street, E. Wilson Street, S. Pinckney St. & Madison Municipal Building
Madison, Wisconsin

Dear Mr. Schaller:

As requested, Construction • Geotechnical Consultants, Inc. (CGC) has completed the updated geotechnical report for the proposed development, which is based on in-progress construction documents for Block 88 and previously-completed geotechnical exploration programs from 2010 and 2015. The purpose of this geotechnical report is to compile the subsurface information available from previous explorations to evaluate the subsurface conditions at the site from a geotechnical engineering viewpoint and to provide updated recommendations regarding site preparation, foundation, floor slab, seismic site class and pavement design and construction. An electronic copy of this report is submitted for your use, and we can provide a paper copy upon request.

PROJECT DESCRIPTION

We understand the Judge Doyle Square – Block 88 project will include the construction of a private 12-story building above a public parking garage consisting of five below-grade parking levels. The project will extend from just east of the Madison Municipal Building (MMB), with the below-grade parking extending below S. Pinckney Street, which will be temporarily closed during construction and reopened upon completion of construction. (We have assumed project north points from E. Wilson Street towards E. Doty Street, as shown on the project documents.) We understand the second phase of the Judge Doyle Square project will involve the demolition of the Government East Parking Ramp on Block 105 and construction of a private development consisting of one to three levels of below-grade parking garage with thirteen story hotel and apartment buildings above. The geotechnical recommendations in this report are focused on Block 88, and some of the recommendations may require adjustment for the Block 105 project with fewer below-grade levels when that project information becomes available.

The lowest parking level (U4) on the Block 88 ramp will be established at EL 849.5 ft (with low point at 848.5 ft), but the U4 Ramp level will slope down to EL 845.7 ft in the northeast footprint. Level 1 (at E. Wilson St.) will be established at EL 900.0 ft, while Level 2 (at E. Doty St.) will be established at EL 907.0 ft. Top of footing is generally 1 ft below top of slab grade, and footings will be 1.2 to 3.2-ft thick, placing footings about 2 to 4 ft below slab grade. Elevator and stairwell footings will be 3-ft thick and will likely be a few feet deeper than column pads and wall footings. Maximum column loads are expected to be 2,000 kips.

EXISTING SITE CONDITIONS

Block 88 consists of a surface parking lot that slopes down from north to south. A former addition to the MMB was recently demolished, but the MMB, which includes one basement level, will remain. Multi-story masonry buildings, which we understand include one basement level, exist along the east side of the Block 105 site. Existing grades along East Doty Street range from about EL 905.5 to 910.5 ft, and site grades along East Wilson range from about EL 894.5 to 901.5 ft. Site grades drop moderately to the south toward Lake Monona and drop more gently to the east across most of the building footprint. For reference, the first floor elevation of the MMB is at EL 913.9 ft, and the water level of nearby Lake Monona is controlled to have a target summer minimum level of EL 844.7 and target maximum summer level of EL 845.2 ft.

SUBSURFACE CONDITIONS

Subsurface conditions on this site have been explored by CGC in 2010 and 2015 for previous development proposals under consideration by the City. The 2010 preliminary exploration program (see CGC report C10041-5, dated August 31, 2010) involved seven Standard Penetration Test (SPT) soil borings (B-1 through B-7) drilled to depths of 90 to 100 ft below existing site grades. A monitoring well (MW-1) was installed about 5 ft north of Boring 5. A second drilling phase in 2010 (see CGC report C10041-5, dated September 27, 2010) involved pressuremeter testing in Borings 4P and 5P, which were offset about 4.5 to 11 ft north of Borings 4 and 5. Pressuremeter testing measures the *in situ* strength and deformation properties of the soil, which allows for more accurate determination of the allowable bearing pressure. In 2015, two additional Standard Penetration Test (SPT) soil borings (Borings 2A and 4A) were completed to depths of 35 to 55 ft below existing site grades in order to conduct additional pressuremeter testing (see report C15237, dated November 5, 2015). A second groundwater monitoring well (MW-2) was installed in Boring 2A in 2015. More information regarding the drilling programs is included in Appendix A of this report, with the boring locations presented on a Soil Boring Location Exhibit found in Appendix B.

The subsurface conditions encountered across the site are fairly uniform and consist of medium dense to very dense ***silty sand strata*** with moderate to significant gravel content and scattered cobbles and boulders. The silty sand strata is below about 3 to 12.5 ft of miscellaneous sand and clay ***fill*** with scattered brick and concrete debris (including an apparent 1-ft thick concrete layer/slab from about 11 to 12 ft in B-3), gravel and cobbles/boulders, etc. and/or layers native ***lean clay***. In general, very dense silty sand strata with standard penetration resistances (N-values on the boring logs) greater than 50 blows/ft are present from about 10 to 20 ft below existing grade and extends to the maximum depths explored. The conditions described above are typical for the glacial tills found in the Capitol Hill area. Exceptions to the generalized profile, which are not uncommon in this area, include the following:

- Very dense *sandy silt* layers were encountered near 27.5 ft in Boring 3 and near 86 ft in Boring 6, and a *sandy clayey silt* layer was encountered in the lower few feet of Boring 7.

- Occasional *sand layers with lower silt content* (denoted as SP/SP-SM on the soil boring logs) were encountered near 22.5 ft in Boring 3, near 87.5 ft in Boring 5, near 32.5 ft in Boring 6 and near 78 ft in Boring 7. These erratic layers are notable in that these relatively clean sand are generally much more permeable than the surrounding silty sand and are often water bearing. (Although not encountered in the borings, gravel layers can also be encountered in the Capitol Hill area that can also be water bearing.) Therefore, water can collect in these layers as perched lenses or pockets, and water-bearing clean sand layers may have implications for the excavation and earth retention activities, which will be discussed later in this report.

- A hard *lean clay* was encountered near 91 ft in Boring 6.

- A *possible petroleum odor* was encountered near 50 ft in Boring 1, which may be the result of a possible spill or leak from a previous gas station at this site with underground fuel storage tanks, or from other unknown source. We understand the tanks were previously removed. Environmental issues are not covered in this report. We understand they will be or have been addressed by others.

Based on the very dense nature of the silty sand till and use of drilling mud to advance the boreholes, the groundwater elevation could not be accurately determined in the borings themselves, and where water levels are indicated on the boring logs, these depths should be considered very approximate. Therefore, monitoring well MW-1 was installed in 2010 near Boring 5, and MW-2 was installed in Boring 2A in 2015 to obtain more accurate long-term water level readings. Water levels have been periodically recorded since 2010, with the last reading on June 1, 2017, and are summarized in Table 1. As groundwater generally follows topography, the higher water level within MW-2 on the higher end of the site would generally be expected. Our experience indicates that a groundwater mound above lake level such as at this location is frequently seen in the Capitol Hill area. Groundwater levels typically drop as the ground surface falls off towards Lake Monona to approximate lake level, which has a target maximum summer level near EL 845.2 ft. The historic high water level of Lake Monona is 847.9 ft recorded in June 2008.

In our opinion, an apparent perched groundwater seam was located near 32.5 to 35 ft below existing site grades (approx. EL 870.1 to 872.6 ft) in Boring 6 in a “cleaner” sand layer between silty sand layers. A similar clean sand layer was encountered near 22.5 to 25 ft below existing site grades (approx. EL 881.6 to 884.1 ft) in Boring 3 that also appeared to contain perched water. Such water-bearing, perched clean sand layers have been encountered on other projects in the Capitol Hill area. Fluctuations in the groundwater level should be expected in response to seasonal variations in precipitation, infiltration, lake levels, pumping rates from nearby wells and other factors.

Table 1 - Summary of Groundwater Readings

Date	Approximate Water Level Elevation	
	MW-1 (Near B-5)	MW-2 (Boring 2A)
8/16/2010	863.0	-
8/17/2010	862.7	-
8/19/2010	862.9	-
8/24/2010	863.0	-
9/4/2010	863.0	-
12/23/2011	859.7	-
6/2/2015	857.1	-
8/20/2015	858.3	-
10/20/2015	859.8	864.3
4/20/2017	860.3	866.6
6/01/2017	861.2	866.3

More detailed information regarding soil and groundwater conditions at the site are presented in the Boring Logs found in Appendix B. Grain size distribution test reports on soil boring samples are also included in Appendix B.

DISCUSSION AND RECOMMENDATIONS

Subject to the limitations discussed below and based on the subsurface explorations, it is our opinion that the this site is suitable for the proposed project, and the building can be supported on spread footing foundations proportioned to take advantage of the high bearing pressure supported by pressuremeter testing. However, since the parking garage will extend well below the water table, with U4 ramping down near the level of Lake Monona (including foundation excavations below lake level), the following items will need to be carefully considered during the design and construction phases:

- Dewatering will be an important component of the excavation process to reduce foundation and floor slab subgrade disturbance.
- Protection of footing subgrade soils with lean mix concrete “mud mats” will be required to reduce the risk of subgrade disturbance.
- Special earth retention considerations will also be required due to groundwater, and dewatering should be considered integral in the design of the earth retention system.

- Provisions will need to be made to permanently dewater below the building extending below the water table, as currently planned. Alternatively, the below-grade portion of the building can be designed as a watertight system.

Specific recommendations regarding the geotechnical aspects of design and construction follow. Additional information regarding this report is discussed in Appendix C.

1. Site Preparation

a. General Excavation

Based on the depth of excavation required for five below-grade parking levels, we anticipate that any remnants of former structures that were previously demolished will be removed in their entirety. Any remaining demolition debris should be taken to an appropriate waste disposal site. Except for minor amounts of on-site granular soils reused for backfill, we anticipate that the bulk of the soils in the excavation will be hauled off site. *As noted earlier, a possible petroleum odor was noted near 50 ft in B-1, and miscellaneous fill was encountered in the upper part of several borings. It is not uncommon for cinders or other waste material to be located in shallow fill in the downtown Madison area that will require disposed of in a licensed landfill.* An environmental consultant should be retained during excavation to determine the extent of the impacted soils, if any, and develop a materials management plan for disposal purposes.

b. Dewatering Evaluation

The deeper parking garage planned for the Block 88 project will extend significantly deeper below the water table compared to what was envisioned when the previous reports were prepared for other proposed development concepts. Therefore, the dewatering evaluation and recommendations are significantly different from our earlier reports. Based on a lowest level (U4) slab grade near EL 845.7 to 849.5 ft, with footing excavations near EL 840 ft, it appears that the footing excavations will extend about 20 to 27 ft below the groundwater level measured in the monitoring wells. Groundwater may also be higher due to seasonal fluctuations, including higher lake levels. Additionally, perched water lenses in cleaner sand and gravel layers could be encountered above the static water level that will need to be drained as construction advances deeper. The presence of water within the silty sand soils can result in unstable footing subgrades and excavation instability if not handled properly. The very dense silty sand soils are generally expected to have a relatively low permeability and will be difficult to dewater.

Since floor slab grade will be about 15 to 20 ft below the groundwater table and footing subgrade will be 20 to 27 ft below the groundwater table, dewatering will be an important component of the excavation process. However, since the very dense silty sands at and below slab and footing grade have a very low permeability, dewatering will be a slow and difficult process. We recommend that the dewatering process begin well in advance of excavating below the water table, and groundwater should be lowered at least 2 ft below footing grade prior to beginning footing excavation. Groundwater levels should be monitored in wells located throughout the site to determine when sufficient groundwater drawdown has been achieved and the excavation can be extended deeper or foundation excavation can begin.

Appropriate dewatering system/implementation should be provided by an experienced dewatering contractor. This can be accomplished by presenting the relevant data in this report to the dewatering contractor, which they can in turn use along with their experience to develop a means and methods dewatering system, which is the dewatering contractor's responsibility. Based on the grain size distribution curves developed from samples collected from the soil borings (attached), we expect that a vacuum well-point system with regular, closely-spaced well points throughout the excavation area, including around the perimeter and within the excavation will be required to effectively dewater the soils. Additional well points may be required around deeper excavations (elevator, etc.). *Modifications to the dewatering, excavation and earth retention program may be required to deal with zones of water-bearing highly permeable sand and gravel that will need to be drained before excavation can continue. Such layers were encountered in Borings 3, 5, 6 and 7, and have been encountered on previous projects in the Capitol Hill area. The higher permeable zones are erratic and the elevation and location of these zone are very difficult to predict, but prospective contractors should be aware of the potential to encounter these layers and have contingency plans in place to react to their presence.* The well point screen and filter pack around the screen should be properly sized to prevent loss of soil through dewatering. The water from dewatering system should be monitored for evidence of soil loss in a sedimentation basin prior to discharge. If there is evidence of significant soil loss, the screen size and/or filter pack may require adjustment.

Even with an effective dewatering system, the low permeability silty sand soils will be difficult to completely dewater, and because of this likelihood, along with the high soil bearing pressure determined through pressuremeter testing, *we recommend that thin concrete "mud mats" be used to protect footing subgrades from disturbance immediately after excavation.* Supplemental localized dewatering using pumps in filtered sump pits may be required to dry up the subgrade in between well points. The risk associated with ineffective dewatering in advance of excavation is that the subgrade integrity may be compromised. Additionally, water-bearing seams of more permeable sands or gravels may be encountered resulting in subgrade instability and potentially earth retention stability issues.

In general, dewatering has the potential for causing settlement of nearby structures. However, we expect minimal (if any) settlement attributed to dewatering at this site due to the predominance of medium to very dense silty sand underlying the site. Although with appropriate dewatering and retention systems the amount of settlement is not expected to be detrimental, we recommend that a precondition survey and monitoring program be implemented prior to installation of the earth retention and dewatering systems as a means of determining if settlement takes place. Note that some minor cracking of the existing structures (especially masonry structures) may occur as a result of various construction activities (e.g., vibrations, excavations, etc.). If significant settlement or cracking is noted, the dewatering and/or earth retention system will require re-evaluation.

c. Earth Retention Recommendations

As space is limited due to adjacent buildings and streets, excavation retention systems are expected to be required on all sides. If space allows in some locations, the upper soils could possibly be sloped back to a stable angle per OSHA slope requirements and protected from erosion with shotcrete or reinforced plastic, but the bulk of the excavation volume will require a robust retention system. Retention systems should be designed by an appropriately qualified registered professional engineer. The earth retention system should be designed in conjunction with the dewatering system, with appropriate lateral soil pressures and hydrostatic pressures included in the earth retention design.

In our opinion, the predominant soil type on site, a silty sand glacial till, is generally suitable for the soil nailing method and may prove to be the most cost-effective earth retention system for this project along Wilson and Doty Streets. Based on recent project experience soil nailing may also be feasible, with appropriate modification, along the east and west sides of the excavation where the earth retention will need to support buildings. Soil nailing is a method of earth retention that is based on in-situ reinforcement of the earth mass adjacent to the excavation. The reinforcement is accomplished by inserting steel bars (“nails”) into the soil in a grid pattern, spaced typically about 5 to 6 ft in both horizontal and vertical directions. The depth to which the reinforcement extends beyond the excavation face is usually in the range of 60 to 80 percent of the depth of excavation. Nails typically consist of rebar grouted in 4-in. (minimum) diameter predrilled holes. Both nail spacing and length can be readily modified to accommodate different soil and loading conditions. To retain the soil at the face of the excavation, shotcrete about 4 to 6-in. thick and reinforced with wire mesh is applied to the face and anchored to the nails. A thicker shotcrete face and/or pre-stress nails may be required where heavier loads need to be supported. The excavation, installation of nails and shotcrete application proceed in “lifts” equal to one vertical grid space.

Special attention must be provided during both design and installation of the retention system to adequately support the adjacent roadways and avoid damage to buried utilities. More challenging installation, including the use of hollow core nails, should be anticipated in deeper portions of the excavation that extend near or below the water table or in zones containing clean sands/gravels that may include perched water. Three-dimensional drainage board behind the shotcrete face that drains through the shotcrete face using weep holes (wrapped in non-woven geotextile filter fabric) will likely be required in some locations to adequately drain perched water lenses. Where the excavation extends below the water table, additional drainage provisions may be required behind the shotcrete face, potentially including inclined or horizontal well points, or the retention system may need to switch from soil nailing to a soldier pile/wood lagging system (discussed below), where seepage can occur through the face of the lagging. Non-woven geotextile fabric (e.g., Mirafi 160N or equivalent) may be required behind the lagging to minimize soil migrating through the lagging, and three-dimensional drainage board may be required to drain water behind the lagging and prevent the build-up of hydrostatic pressures.

Where retention systems are required next to existing structures where excavation will extend below footing grade, a stiffer retention system will be required to accommodate the heavier loads, depending on the excavation depth, separation distance between the buildings and the relative foundation grades. Where slopes cut back to stable angles would encroach on the zone of influence of the existing foundations, a denser soil nail grid, pre-stressed soil nails or a soldier pile/wood lagging system may be required to reduce potential movement.

For a soldier pile/wood lagging system, H-piles (soldier piles) are installed in vertical pre-drilled holes that are backfilled with concrete. The H-piles will likely require intermediate tie-back anchoring based on the excavation depth, pending site details and loading conditions. Provisions will need to be included in the wood lagging face, such as the use of three-dimensional drainage board or filter fabric, to accommodate water seepage behind the wall while preventing soil erosion. Draining water will be especially critical where the lagging will extend below the water table.

In some cases, bracket piles have been used to essentially underpin the shallower foundations of adjacent buildings and transmit those loads below the base of the excavation. If the foundation loads are very high, underpinning with micropiles will likely be required. Appropriate protection should be provided below foundations of structures (and potentially utilities) around the perimeter of the excavation to prevent the soil freezing below the foundations, which could result in unacceptable movement due to freeze-thaw action.

If the earth retention system will be considered a permanent support system of below-grade lateral pressures, appropriate corrosion protection measures should be included to satisfy the project design life.

We recommend that a precondition survey be performed on adjacent properties prior to the start of excavation. It is also important that a monitoring program be established and implemented until the permanent, below-grade walls are backfilled. The program should check for visual signs of wall and/or adjacent structure movement on a regular basis during the excavation stage of construction. Regular monitoring should continue until the lower level walls are backfilled or a sufficient amount of data is generated to draw a conclusion on the retention system's performance.

d. Underground Utilities

Numerous utilities exist along Wilson, Doty and Pinckney Streets. Where possible, utilities should be disconnected a significant distance away from the excavation limits (i.e., typically 5 ft or more) and abandoned or rerouted. Abandoned utility lines should be capped or plugged to eliminate potential unexpected seepage into the excavations. Utilities in the vicinity of the excavation that cannot be rerouted or disconnected may require special temporary support measures beyond soil nailing. On past projects, active gas lines, electric cables, and fiber optic lines have sometimes required special protection and support structures.

e. Floor Slab Subgrades

The subgrade soils present at the lowest parking level elevation are expected to consist of natural silty sands. As the excavation base will be about 15 to 20 ft below the groundwater table and will be exposed to typical construction traffic, precautions should be taken to protect the subgrade from disturbance until floor slab and footing construction is to proceed. The recommended procedure for floor subgrade protection is as follows:

- Excavate a minimum of 2 ft below proposed floor slab subgrade elevation with a backhoe equipped with a smooth-edged bucket. At the bottom of the excavation, a woven geotextile fabric (e.g., Mirafi 600X or equivalent) should be placed on the subgrade, which should be overlapped at least 2 ft between adjacent sheets. Above the fabric layer, 12-in. of 3-in. nominal diameter crushed stone containing minimal "fines" (P200) content should be placed and compacted. If groundwater seepage occurs that destabilizes the subgrade, the subgrade may need to be exposed in small sections followed by prompt placement of fabric and stone layers. If isolated subgrade disturbance persists, deeper undercutting and thicker stone placement may be necessary. *Note that careful consideration should be given to installation of utilities that will be located below the fabric and stone stabilization layer prior to fabric placement. In these situations, it may be advantageous to excavate and backfill foundations prior to installing the stabilization layer.* Supplemental construction dewatering can potentially be achieved, where needed, by pumping from sumps within the stone layer.

- After footing excavation is complete (see below), a second fabric and stone layer will be installed throughout the building footprint to permanently dewater below the building footprint, which is discussed in more detail in the Floor Slab section of this report. In general, the process will include installation of a heavy duty non-woven geotextile fabric (e.g, Mirafi 160N or equivalent) on top of the stone stabilization layer in accordance with manufacturer's guidelines. The primary purpose of the non-woven geotextile fabric is to allow water infiltration and to a lesser degree provide soil separation. The fabric should be overlapped per manufacturer specifications (typically a 2-ft. minimum). To prevent piping of fines from the subgrade soils, extra care should be taken to completely cover the subgrade with geotextile. Particular attention is required at the interfaces between the fabric and the structure and pipes. If perimeter walls will be backfilled with clear stone, the fabric should also be wrapped up the sides of the earth retention.

- Place and compact 12-inches of WDOT No. 1 crushed stone *WDOT Standard Specifications* (Section 501.2.5.4.4) to complete the slab subgrade surface. This layer will serve as the drainage medium below the slab in which drain tile will be installed (discussed in the Floor Slab section).

2. Foundation Design

Based on the anticipated footing grade of EL 840 to 847 ft within the Block 88 footprint, foundations for the structure are anticipated to bear within natural very dense silty sand soils. Based on pressuremeter testing, it is our opinion that the proposed structure may be supported by conventional spread foundations proportioned using a relatively high bearing pressure. *This opinion is based on an effective dewatering system lowering the water table at least 2 ft below footing grade and placement of concrete "mud mats" such that undisturbed soils existing at footing subgrades.* A backhoe with a smooth-edged bucket should be used to excavate to footing grade within the structure limits. The footing subgrades should then be observed by CGC using a dynamic cone penetrometer (DCP) to check for loose or soft pockets that will require removal. To restore footing grade in undercut excavations and to protect subgrades from disturbance, we recommend that a ***minimum 4-in. thick "mud mat"*** be placed as soon as possible after excavation over the footing subgrades. The mud mat should be a "lean mix" concrete capable of developing a compressive strength of 1,000 psi after 28 days. (Normal footing mix can also be used for the mud mat.)

As mentioned above and discussed in the Site Preparation section of this report, footing excavations are expected to extend 20 ft to 27 ft below the water table and approximately 5 to 6 ft below the level of Lake Monona, such that lowering the groundwater at least 2 ft below footing grade with an effective dewatering system will be a critical component of the foundation excavation process. We anticipate a vacuum well point system will likely be required as primary dewatering system, although supplemental dewatering of the relatively low permeability silty sands may be required using pumps operating in shallow sump pits or trenches.

Provided the foundations are installed on the very dense, natural sand silty strata and protected with a lean mix mud mat in accordance with the preceding recommendations such that minimal subgrade disturbance occurs, the following parameters should be used for foundation design:

• Maximum net allowable bearing pressure:	30,000 psf
• Minimum foundation widths:	
-- Continuous wall footings:	18 in.
-- Column pad footings:	36 in.
• Minimum footing depths:	greater of one quarter of the foundation width or 4 ft for frost protection (where required)

We recommend that CGC be retained during construction to document that the soil conditions are suitable for the design bearing pressure or recommend corrective measures, if required. A smooth-edged backhoe bucket should be used for footing excavations. Additionally, the granular soils exposed at footing grade should be hand trimmed to remove loose or disturbed soil prior to concrete mud mat placement. The foundation soils should be checked using a dynamic cone penetrometer (DCP), where the DCP blow count should be equivalent to a minimum SPT blow count of 63 blows/ft. Note that this technique has been successfully used on numerous projects in the Madison area where relatively high bearing pressures were determined with pressuremeter testing. The silty sand subgrade soils will be susceptible to disturbance from groundwater if not effectively dewatered and protected *immediately* after excavation, so we recommend that the subgrades be protected from disturbance by placing a 4-in. thick lean mix concrete layer *immediately* after excavation and DCP evaluation. The mud mat is recommended for all footings designed using the 30,000 psf bearing pressure, even if above the water table. This operation will require special attention and coordination between the general contractor, the excavator and CGC in the field.

Undercutting below footing grade will be required if native clays (if any) or loose/disturbed sands are observed at or slightly below footing grade. Where undercutting is required, the base of the undercut excavation should be widened beyond the footing edges at least 0.5 ft beyond the edge of the footing and extended vertically to the required bearing stratum. (Undercuts deeper than 4 ft will require proper slopes or bracing to meet OSHA guidelines, if workers need to enter the excavation. Undercuts adjacent to the earth retention system should be reviewed by the earth retention design engineer and/or contractor.) Footing grade should be re-established using “lean mix” concrete or regular footing mix concrete. Alternatively, the foundations can be lowered to bear on the suitable bearing stratum.

Provided the foundation design/construction recommendations discussed above are followed, we estimate that total and differential settlements should be on the order of 1.0 and 0.5 in., respectively.

Note that if footings are planned above the lowest parking level grade (e.g., for canopies extending beyond the basement level) such they will bear on wall backfill, careful consideration will be needed so that these footings are properly supported. The shallow undocumented fill soils extending approximately 12.5 ft below existing site grade in some borings (and potentially deeper around parts of the site) are considered unacceptable for foundation support. Shallow foundations in areas of thicker undocumented fill may require support with helical piers or micropiles unless the fill is removed and replaced with engineered granular backfill. Shallow native soils and engineered wall backfill are <u>not</u> suitable for the 30,000 psf bearing pressure recommended on deeper sand strata, as discussed below.

Footings bearing on newly-placed engineered backfill above suitable natural soils will require careful and systematic wall backfilling to reduce the chance of excessive post-construction settlement. *If clear stone will be used as below-grade wall backfill, it is imperative that the clear stone be placed in controlled lifts of approximately 12 in. and compacted with a vibratory compactor until deflection ceases.* Granular backfill supporting footings should be placed in 8 in. to 12 in. loose lifts and compacted to a minimum of 95% compaction based on modified Proctor methods (ASTM D 1557). Footings that bear on well compacted clear stone or granular backfill compacted to a minimum of 95% can be designed using an allowable bearing pressure of 3,000 psf. As an alternative, lightly loaded shallow structures could be designed to cantilever off the below-grade wall.

3. <u>Seismic Design</u>

It is our opinion that the average soil properties in the upper 100 ft based on SPT blow counts (N-values) exceed 50 blows/ft, on average, can be characterized as very dense soil. This characterization would place the site in Site Class C for seismic design according to the International Building Code (Table 1613.5.2).

4. **Floor Slab**

Although building below the water table can be accomplished, *the owner should understand that there are additional risks and costs associated with such plans.* Where below-grade levels will extend below the water table on a permanent basis, it is our opinion that two typical strategies can be used to deal with the water table (other alternatives may also be applicable):

1. Install a subfloor drainage system that *permanently* lowers the water table below slab level for the life of the structure, or

2. Design and construct the levels below the water table as a watertight (i.e., “bath tub”) structure capable of resisting hydrostatic uplift pressures below the slab and along the walls.

On past projects with groundwater drawdowns of similar magnitude to this project in low permeability silty sand soils, the subfloor drainage alternative has typically been chosen based on economics. Based on 90% construction drawings provided to CGC, we understand the subfloor drainage alternative is planned for this project. We understand that there are higher up-front costs associated with constructing a water tight structure, but it should be recognized that there will be higher long-term operation and maintenance (O & M) costs and greater risk associated with permanently lowering the water table below the floor slab during the lifetime of the building.

a. Subfloor Drainage System

The U4/U4 Ramp slab elevation is EL 845.7 to 849.5 ft, which is approximately 15 to 20 ft below the water table based on well readings. *Accordingly, provisions should be made to effectively drain the water from below the slab on a full time basis for the life of the building through the incorporation of a subfloor drainage system.* For this portion of the building, we expect that the floor slab subgrades will need to be stabilized upon excavation to prevent degradation and develop a firm subgrade, as discussed in a previous section. Above the stabilization layer we expect the subfloor dewatering system to consist of the following components:

- A geotextile (e.g., Mirafi 160N or approved equivalent) should be carefully placed and positioned above the stone stabilization layer prior to stone placement to separate the drainage blanket from the subgrade soils. A minimum 2-ft overlap is recommended between adjoining geotextile sheets and the fabric should be wrapped up the sides of foundations, walls and columns a minimum of 2 ft. Careful attention is required so that the fabric is also sealed around vertical pipe penetrations. If perimeter walls will be backfilled with clear stone, the fabric should also be wrapped up the sides of the earth retention.

- The drainage blanket below the floor slab should be a minimum 12-in. thick layer of Size No. 1 washed stone (WDOT Specification Section 501.2.5.4.4) or an equivalent open-graded crushed clear stone.

- Drain lines should be spaced approximately 20- to 25-ft in the longitudinal direction. A slightly wider spacing may be acceptable if the plumbing designer determines that wider-spaced drain tile can adequately remove the water. The drain lines should be bedded in trenches that extend slightly below the drainage blanket, and the drain lines should be sloped towards either a header/collector pipe or the sump crocks. Note that we assume that the subgrade will be sloped towards the sump crocks and the drain lines will follow the general slope of the subgrade. The geotextile should be draped inside the shallow trenches before installing the bedding stone and pipe. The maximum drain slot size should be equal to 0.25 inches.

- Schedule 40 PVC drain pipe is recommended for the main/central drainage pipes. However, if flexible, corrugated ABS pipe can be effectively cleaned/jetted without damage through cleanouts extending through the slab, this type of material can be substituted for the PVC pipe.

- Pressure relief ports should be included in the slab design to prevent slab uplift in the event of a system shutdown. Note that the pressure relief ports are included to allow the lowest level to flood in the event that the subfloor drainage system is inoperable (e.g., during a power outage, etc.) during unlikely high water events. Although flooding of the lowest level may occur, damage to the slab is prevented. Pressure relief ports can be as simple as vertical pipes extending through the slab with a conventional floor drain as a cover. They should be installed at high points in the slab to prevent snow melt from collecting in the subfloor drainage system.

- Appropriate connections between the drainage system behind the earth retention system, drainage system behind the permanent basement walls and sub-floor drainage system should be provided to adequately drain water behind the below-grade walls and prevent the build-up of hydrostatic pressures (unless the below-grade walls are designed to accommodate such increased lateral pressures).

- Exterior basement walls below approximately EL 870 ft should be waterproofed with a waterproofing membrane.

- A qualified civil or mechanical engineering consultant experienced in the design of permanent drainage systems should be included on the design team for the project to detail the system required on this project. The drainage system should be designed so that it is continuously connected to an interior perimeter drain line which discharges to one or more sump pits. Details such as sump locations/sizes, pump selection, backup generator, pumps and alarm systems, final pipe sizes and locations should be completed by a plumbing designer and are not addressed in this report. We recommend that redundancy be built into the system, such as duplicate sumps, pumps and backup generator, in the event of pump break-down or loss of primary power. If possible, critical electrical and mechanical equipment should <u>not</u> be located in the U4/U4 Ramp levels to avoid potential damage in the event of subfloor drainage system shutdown and subsequent flooding.

Note that there is considerable flexibility in the details of the drainage system, and we can work with the design team to develop a system suitable for the project. We can also provide additional details regarding estimated long-term dewatering rates. The *in situ* hydraulic conductivity can be estimated by conducting drawdown and recovery (slug) tests in the monitoring wells, which can in turn be used to provide *preliminary* sub-floor dewatering rates based on the building footprint and groundwater drawdown. Sub-floor dewatering rates should be re-evaluated during construction to reflect actual construction dewatering rates, which will more accurately reflect anticipated long-term dewatering rates.

In our opinion, the lower level floor slab for the building will be supported on the compacted ¾-in. stone drainage layer over the native silty sand soils, and a subgrade modulus of 150 pci may be used in design. Prior to slab construction, the subgrades should be recompacted to densify soils that may become disturbed or loosened during construction activities. The design subgrade modulus is based on a recompacted subgrade such that non-yielding conditions are developed. The floor slab should be isolated from the building walls and columns with compressible filler, and the design should include an adequate number of isolation and contraction joints.

b. Water Tight Structure

A water tight structure would likely involve a reinforced concrete base slab or mat foundation underlain by a waterproof system significantly thicker and more robust than a typical below-grade wall damp-proofing system. For the base slab system, a reinforced concrete slab would form the bottom of the water tight system and conventional footings and floor slab would be constructed above the base slab. Alternatively, a hybrid mat foundation system could be designed such that thicker foundations and thinner floor slab are poured monolithically in a waffle-like configuration thereby potentially reducing concrete volume. The waterproofing system would extend up the walls several feet above the water table to approximately EL 870 ft. Careful detailing would be required to prevent seepage through construction joints between the base slab/mat and perimeter foundation walls, and the below-grade walls would need to be able to withstand both soil and hydrostatic loads. Although long-term dewatering would not be required, the construction dewatering system would need to remain operational until the dead weight of the structure exceeds the hydrostatic pressures pushing up on the bottom of the base slab or mat foundation. We can provide additional details if this alternative will be considered.

5. <u>Below-Grade Walls</u>

a. Below-Grade Parking Levels – Conventional Two-Sided Formed Walls and Backfilling

We anticipate that below-grade walls will be restrained by the lower-level floor slab, as well as lower level and ground level framing. These walls should be designed for *at-rest lateral earth pressures*. To minimize the development of lateral pressures on the walls, high quality backfill should be placed within 3 to 5 ft of the walls that is hydraulically connected with the below-grade wall drainage system. The backfill should consist of free-draining clear stone or sand with less than 5 percent passing the No. 200 U.S. standard sieve up to EL 870 ft to get well above the water table. Above the zone of free-draining backfill, the backfill material should consist of well-graded sand or gravel having no more than 12 percent passing the No. 200 U.S. Standard Sieve. The on-site granular soils are too silty for this purpose and should not be used, unless a three dimensional drainage board is used directly against the below-grade walls which is hydraulically connected to the perimeter drain system. Imported pea gravel or 3/4-in. clear stone is commonly used to backfill the space between the new walls and the temporary retention system because it is more easily compacted in tight spaces. Note that the gravel/stone backfill should be separated on all sides from the native sands or other fill materials by a non-woven geotextile (e.g., Mirafi 160N or equivalent) to prevent migration of fines into the void spaces of the stone.

A perimeter drainage system should be installed to prevent hydrostatic pressure from developing against the walls (refer to Appendix E for conceptual details). The exterior perimeter drains should be integrated with the underfloor drainage system. The granular backfill placed behind the walls should be hydraulically connected to the underlying natural granular soils and perimeter drainage system. To impede the inflow of surface moisture, the final 2 ft of backfill should consist of a clayey fill cap or pavement. The cap should be graded in a manner that promotes positive drainage away from the walls.

Prior to placing backfill, the exterior of the walls below EL 870 ft should be water-proofed with heavy-duty water-proofing membrane, and walls above EL 870 ft should be damp-proofed with a spray-applied or mopped-on rubber or bituminous sealer. Compaction of the backfill within 3 to 5 ft of the walls should be performed with hand-operated compaction equipment to prevent excess lateral earth pressures. The backfill should be compacted to at least 92% compaction (ASTM D1557), but 95% compaction is required if footings will bear on the backfill. Clear stone or pea gravel should be placed in loose lifts of 12 in. and compacted with a vibratory compactor until deflection ceases.

Below-grade walls constructed in accordance with the above recommendations may be designed for an equivalent fluid pressure of 55 psf per ft of depth. The wall design should take into account surcharge or hydrostatic effects that could be applied either during or after construction. Note that water stop should be included in portions of the walls that will be below the water table to reduce seepage through the walls.

b. Below-Grade Parking Levels – Single-Sided Formed Walls With Braced Excavation Assumption

Assuming a temporary earth retention system will surround the excavation and basement walls will be constructed in direct contact with the earth retention system, separated only by drainage and waterproofing layers, and parking level slabs will be designed to provide lateral support of the below-grade walls, it is our opinion that the below-grade walls could be modeled as a braced excavation. Under such a scenario, and assuming lateral movements are to be minimized, the lateral earth pressure (σ_h) would approximately equal $0.5*K_0*\gamma*H$, where K_0 is the at-rest lateral earth pressure coefficient, γ is the unit weight of soil, and H is the wall height. If the wall extends below the water table, hydrostatic pressures should be applied, as appropriate based on drainage conditions. We recommend using the following soil parameters to calculate the lateral earth pressures:

- Natural very dense silty sand: friction angle (ϕ) of 38 degrees (K_0 = 0.38); moist unit weight (γ) of 140 pcf, and wet unit weight of 143 pcf. This condition is generally present below about 10 to 20 ft in the borings.

- Natural or fill medium dense sand: friction angle (ϕ) of 32 degrees (K_0 = 0.47), and moist unit weight (γ) of 130 pcf. This condition is present within portions of the upper approximately 15 ft of several borings.

- Natural or clay fill and very loose sand fill: long-term (drained) friction angle (ϕ) of 26 degrees (K_0 = 0.56), and moist unit weight (γ) of 120 pcf. These soil conditions were present in the upper approximately 3 to 12.5 ft of the borings.

c. Near-Surface Retaining/Loading Dock Walls

If retaining and loading dock walls (if any) will not be laterally restrained, these walls should be designed for *active earth pressures* behind the walls and *passive pressures* in front of the walls. Lateral pressures behind the retaining walls can be minimized by backfilling as described in Section 5A. In addition, weepholes should be placed near the base of these walls on 10-ft centers to provide adequate drainage of the wall backfill. The weepholes should be hydraulically connected with the backfill and should be protected with a geotextile fabric to minimize soil loss through the weepholes.

Retaining walls constructed in accordance with the above recommendations may be designed for an active equivalent fluid pressure of 35 psf per foot of depth. Passive equivalent fluid pressures are expected to be on the order of 200 psf per ft. The passive pressure includes a safety factor of 2 to prevent excessive wall deflection. The retaining wall design should also take into account any surcharge or hydrostatic effects which could be applied during or after construction.

6. Pavement Design

We anticipate that only minor sections of pavement will be constructed, primarily the entrance ramp to the parking levels. Assuming that this will be rigid, Portland cement concrete pavement, we recommend that it be designed for a subgrade modulus of 150 pci, based on a subgrade comprised of sand backfill, with a minimum of 6 in. of dense graded base (DGB) course below the concrete slab. Undercutting/stabilization of existing fill soils or natural clays may be required to develop a suitable subgrade. Likewise, pavement areas subjected to concentrated wheel loads (i.e., loading docks, dumpster pads, etc.) should be constructed of Portland cement concrete. The concrete pavement should be a minimum of 6-in. thick, should be underlain by at least 6 in. of 1-¼ DGB, and should contain reinforcement for crack control.

CONSTRUCTION CONSIDERATIONS

Due to variations in weather, construction methods and other factors, specific construction problems are difficult to predict. Soil related difficulties that could be encountered on the site are discussed below:

- We recommend that final site grading activities be completed during dry weather, if possible. Earthwork construction during the early spring or late fall could be complicated as a result of wet weather and freezing temperatures.
- During cold weather, exposed subgrades should be protected from freezing before and after footing construction. Fill should never be placed while frozen or on frozen ground.
- Excavations extending greater than 4 ft in depth below the existing ground surface should be sloped in accordance with current OSHA guidelines. The native granular soils generally appear to be classified as Type B per OSHA and therefore, excavation slopes of 1H:1V or flatter are expected to be temporarily stable. Flatter side slopes will be required if perched water seams or when excavating near or below the water table. Flatter side slopes may also be required in the shallow fill soils, including loose sands and softer clays. The excavation side slopes should be evaluated in the field by a competent person. Excavations 20 ft or greater in depth and earth retention systems should be designed by an appropriately qualified licensed professional engineer.
- As noted, dewatering will be an important component of the excavation and earth retention process and will be critical to maintaining the integrity of the foundation and floor slab subgrades. We expect that dewatering with vacuum well points will be required as a primary dewatering system, with supplemental dewatering using pumps in shallow sump pits.
- The special subgrade preparation procedures (i.e., concrete mud mats) discussed in the text are critical to the success of the foundations at this site.
- If abandoned cisterns and/or wells are encountered on-site that coincide with footing locations, special procedures will be required, such as plugging the holes with concrete.
- When excavating adjacent to existing structures, exercise care to prevent undermining of their foundations. These issues should be addressed in advance during design of the earth retention and underpinning systems.

RECOMMENDED CONSTRUCTION MONITORING

The quality of the foundation and floor slab subgrades will be largely determined by the level of care exercised during site development. To check that earthwork and foundation construction proceeds in accordance with our recommendations, the following operations should be monitored by CGC:

- Floor slab subgrade checking within the construction areas;
- Earth retention system construction. We also recommend that the contractor monitor vertical and lateral movement of the earth retention system and surrounding structures via optical survey. Reading frequency can be established when appropriate. The optical survey points should have an accuracy of 0.005 ft or better. Data should be provided to CGC and others on the project team for review on a regular basis;
- Footing subgrade preparation/protection with lean mix concrete mud mats;
- Concrete placement; and
- Backfill placement and compaction (including sub-floor drainage system).

* * * * *

It has been a pleasure to serve you on this project. If you have any questions or need additional consultation, please contact us.

Sincerely,

CGC, Inc.

David A. Staab, P.E., LEED AP
Senior Consulting Professional

William W. Wuellner, P.E.
Senior Geotechnical Engineer

Encl:	Appendix A -	Summary of Field Explorations
	Appendix B -	2010 Soil Boring Location Exhibit, Logs of Test Borings (7), Pressuremeter Test Borings (2) & Monitoring Well (1) 2015 Soil Boring Location Exhibit Logs of Pressuremeter Test Borings (2) and Monitoring Well (1) Particle Size Distribution Test Reports (9) Log of Test Boring-General Notes Unified Soil Classification System
	Appendix C -	Document Qualifications
	Appendix D -	Recommended Compacted Fill Specifications
	Appendix E -	Typical Perimeter Drain Details
	Appendix F -	WKG^2 Pressuremeter Test Reports (2010 & 2015)

APPENDIX A

SUMMARY OF FIELD EXPLORATION

APPENDIX A

FIELD EXPLORATIONS

2010 Field Explorations

A total of seven Standard Penetration Test (SPT) soil borings (B-1 through B-7) were drilled to depths of 90 to 100 ft below existing site grades. Pressuremeter testing was completed in supplemental soil borings 4P and 5P, which were drilled to 61.5 to 62 ft below existing site grades near the locations of Borings 4 and 5. The borings were completed by Badger State Drilling (under subcontract to CGC) on August 6 through 16, 2010 using truck-mounted rotary D-120 drill rigs equipped with hollow-stem augers and rotary wash equipment. Pressuremeter testing was completed on September 8 and 9, 2010 under the direction of Wagner Komurka Geotechnical Group (WKG^2 - also under subcontract to CGC). The boring locations were selected by JSD Professional Services and CGC, and located in the field by CGC. Elevations are referenced to Madison City Datum (MCD), with the pavement at the northwest corner of the Municipal Building used as a benchmark at EL 65.15 ft (USGS EL 910.75 ft based on 0 ft MCD equal to 845.6 ft USGS). Ground surface elevations of the supplemental borings were assumed to be the same as the initial borings 4 and 5.

2015 Field Exploration

Two Standard Penetration Test (SPT) soil borings (Borings 2A and 4A) were drilled to depths of 35 to 55 ft below existing site grades. The borings were completed by Badger State Drilling (under subcontract to CGC) on October 5, 2015 using a truck-mounted rotary D-120 drill rig equipped with hollow-stem augers and rotary wash equipment, as well as an automatic SPT hammer. A groundwater monitoring well was installed in Boring 2A. In addition to conventional SPT sampling, pressuremeter testing was performed in these soil borings under the direction of WKG^2.

In Borings 1 through 7, soil samples were obtained at 2.5-ft intervals to 10 ft and at 5-ft intervals thereafter using a drill rig equipped with hollow stem augers and mud rotary equipment. Soil samples were obtained in general accordance with specifications for Standard Penetration Testing, ASTM D1586. Pressuremeter testing was conducted in four of the boreholes under the direction of WKG^2. The specific procedures used for drilling and sampling are described below:

1. Drilling Procedures Between Samples

   The boring was extended downward between samples using a roller bit and circulating drilling mud. Hollow stem augers were also used in the upper reaches of borings.

2. Standard Penetration Test and Split-Barrel Sampling of Soils
   (ASTM Designation: D1586)

   This method consists of driving a 2-inch outside diameter split barrel sampler using a 140-pound weight falling freely through a distance of 30 inches. The sampler is first seated 6 inches into the material to be sampled and them driven 12 inches. The number of blows required to drive the sampler the final 12 inches is recorded on the log of borings and is known as the Standard Penetration Resistance.

3. Pressuremeter Testing

   Pressuremeter testing consists of inflating a flexible probe against the sidewalls of a pre-drilled borehole. The Pressuremeter tests were completed at pre-determined depths by trained field personnel in accordance with established WKG^2 and ASTM procedures. The data obtained from the tests was evaluated following ASTM standards and using guidelines presented in FHWA Publication No. FHWA-1P-89-008. A total of eight tests were conducted. Additional details can be found in Appendix D.

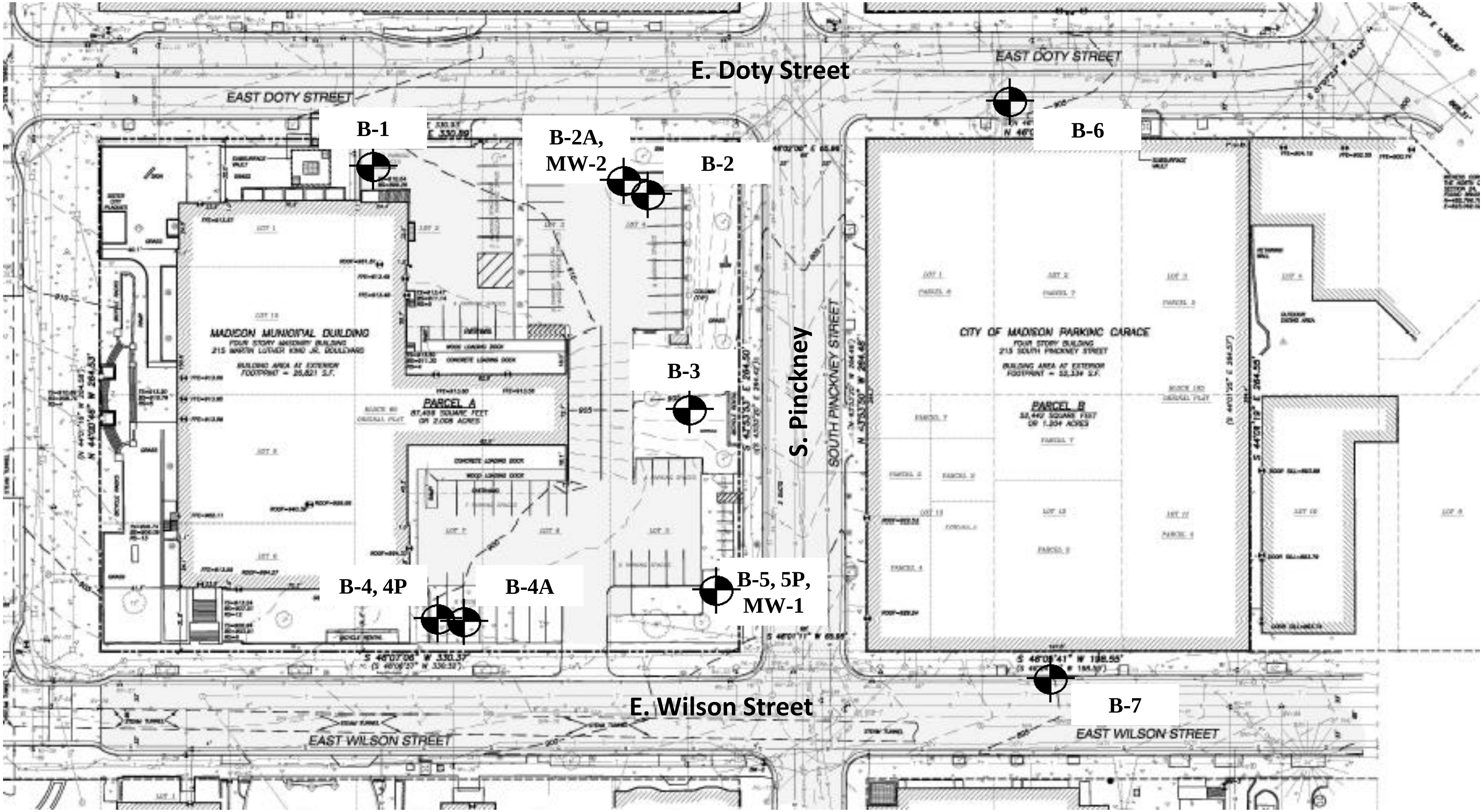
During the field exploration, the driller visually classified the soil and prepared a field log. *Field screening of the samples for possible environmental contaminants was outside CGC's work scope and is not addressed in this report.* Water level observations were made in the hollow stem auger boring during and after drilling and are shown at the bottom of each boring log. Upon completion of drilling, the open boreholes were backfilled with bentonite in accordance with WDNR guidelines with a monitoring well installed in Borings 2A and in a blind-drilled borehole near B-5. The soils were then delivered to our laboratory for visual classification and laboratory testing. The soils were visually classified by a geotechnical engineer using the Unified Soil Classification System. Particle size distribution tests were performed on representative boring samples to aid in classification. The final logs prepared by the engineer and a description of the Unified Soil Classification System are presented in Appendix B.

APPENDIX B

2010 SOIL BORING LOCATION EXHIBIT, LOGS OF TEST BORINGS (7), PRESSUREMETER TEST BORINGS (2) & MONITORING WELL (1)

2015 SOIL BORING LOCATION EXHIBIT LOGS OFPRESSUREMETER TEST BORINGS (2) AND MONITORING WELL (1)

PARTICLE SIZE DISTRIBUTION TEST REPORTS (9)
LOG OF TEST BORING-GENERAL NOTES
UNIFIED SOIL CLASSIFICATION



N
Plan North

Legend

⊕ Denotes Boring Location and Number

Notes: 1. Borings drilled by Badger State Drilling in 2010 and 2015
2. Base map from City of Madison.
3. Boring locations are approximate.

Job No. C16051-14	CGC, Inc.	**SOIL BORING LOCATION EXHIBIT** **Judge Doyle Square Development** **E. Doty Street & S. Pinckney Street** **Madison, WI**
Date: 05/2017		

CGC Inc.

LOG OF TEST BORING

Project **Proposed Parking Ramp**
Between E. Doty St. & E. Wilson St.
Location **Madison, WI**

Boring No. **1**
Surface Elevation (ft) **64.6**
Job No. **C10041-5**
Sheet **1** of **3**

2921 Perry Street, Madison, WI 53713 (608) 288-4100, FAX (608) 288-7887

SAMPLE						VISUAL CLASSIFICATION and Remarks	SOIL PROPERTIES				
No.	TYPE	Rec (in.)	Moist	N	Depth (ft)		qu (qa) (tsf)	W	LL	PL	LI
						5 in. Asphalt Pavement/5 in. Concrete					
1	■	6	M	4		FILL: Medium Stiff, Brown Silty Clay, Scattered Asphalt Debris	(0.75-1.0)				
2	■	12	M	13		Stiff, Brown Lean CLAY, Trace Sand (CL)	(1.7)				
					5	Medium Dense, Brown Fine to Medium SAND, Some Silt, Little Gravel, Trace Clay (SM)					
3	■	12	M	11							
4	■	14	M	30		Dense to Very Dense, Brown Fine to Medium SAND, Some Silt, Little to Some Gravel, Scattered Cobbles/Boulders (SM)					
					10						
5	■	18	M	70							
					15						
6	■	14	M/W	92							
					20						
7	■	14	M/W	93							
					25						
						P200 = 30.9%					
8	■	8	M	50/4"							
					30						
9	■	4	M/W	100/6"							
					35	Numerous Cobbles Near 35 ft					
10	■	16	M	80/9"							
					40						

WATER LEVEL OBSERVATIONS

While Drilling ∇ ________ Upon Completion of Drilling ________
Time After Drilling ________ ________ ________ ________
Depth to Water ________ ________ ________ ________ ▼
Depth to Cave in ________ ________ ________ ________

The stratification lines represent the approximate boundary between soil types and the transition may be gradual.

GENERAL NOTES

Start **8/6/10** End **8/6/10**
Driller **Badger** Chief **AP** Rig **D-120**
Logger **KD** Editor **DAS**
Drill Method **4 1/4" HSA 0-15'; 3-7/8" RB/DM 15'- 100'**

LOG OF TEST BORING

Project Proposed Parking Ramp
Between E. Doty St. & E. Wilson St.
Location Madison, WI

Boring No. 1
Surface Elevation 64.6
Job No. C10041-5
Sheet 2 of 3

2921 PERRY STREET; MADISON, WIS. 53713 (608) 288-4100, FAX (608) 288-7887

SAMPLE						VISUAL CLASSIFICATION and Remarks	SOIL PROPERTIES				
No.	TYPE	Rec (in.)	Moist	N	Depth (ft)		qu (qa) (tsf)	W	LL	PL	LI
11	■	0	M	100/1"	45	Dense to Very Dense, Brown Fine to Medium SAND, Some Silt, Little Gravel, Scattered Cobbles/Boulders (SM) Pushed Stone (No Recovery) Near 45 ft					
12	■	6	M	50/3"	50	*Color Changed to Gray-Brown with Possible Petroleum Odor Near 50 ft*					
13	■	10	M	50/4"	55						
14	■	8	M	50/3"	60						
15	■	10	M/W	50/4"	65						
16	■	10	M/W	50/4"	70						
17	■	8	M	50/3"	75						
18	■	8	M/W	55/4"	80	Fewer Cobbles Near 80 ft					
19	■	18	M	60	85						

CGC Inc.

LOG OF TEST BORING

Project **Proposed Parking Ramp**
Between E. Doty St. & E. Wilson St.
Location **Madison, WI**

Boring No. **1**
Surface Elevation **64.6**
Job No. **C10041-5**
Sheet **3** of **3**

2921 PERRY STREET; MADISON, WIS. 53713 (608) 288-4100, FAX (608) 288-7887

SAMPLE						VISUAL CLASSIFICATION and Remarks	SOIL PROPERTIES				
No.	TYPE	Rec (in.)	Moist	N	Depth (ft)		qu (qa) (tsf)	W	LL	PL	LI
20	■	3	M/W	50/5"		Dense to Very Dense, Brown Fine to Medium SAND, Some Silt, Little Gravel, Scattered Cobbles/Boulders (SM)					
					90						
21	■	5	M	65/6"							
					95						
22	■	3	M	65/6"							
					100	End Boring at 100 ft Borehole backfilled with bentonite slurry, chips and asphalt patch					
					105						
					110						
					115						
					120						
					125						

CGC Inc.

LOG OF TEST BORING

Project **Proposed Parking Ramp**
Between E. Doty St. & E. Wilson St.
Location **Madison, WI**

Boring No. **2**
Surface Elevation (ft) **63.7**
Job No. **C10041-5**
Sheet **1** of **3**

2921 Perry Street, Madison, WI 53713 (608) 288-4100, FAX (608) 288-7887

SAMPLE						VISUAL CLASSIFICATION and Remarks	SOIL PROPERTIES				
No.	TYPE	Rec (in.)	Moist	N	Depth (ft)		qu (qa) (tsf)	W	LL	PL	LI
						5 in. Asphalt Pavement/5 in. Concrete					
1	■	8	M	13		FILL: Stiff, Brown/Dark Brown Lean Clay, Little Sand and Gravel	(1.5-2.0)				
2	■	6	M	17		FILL: Very Loose to Medium Dense, Brown Silty Fine to Medium Sand, Little to Some Gravel, Intermixed with Brick, Cobbles, Boulders, etc. Possible Brick Layer Near 5 ft					
					5						
3	■	3	M	3							
4	■	8	M	24							
					10						
5	■	12	M	50/5"		Very Dense, Brown Fine to Medium SAND, Some Silt, Little to Some Gravel, Scattered Cobbles/Boulders (SM)					
					15						
6	■	8	M/W	50/5"							
					20						
7	■	0	M/W	50/2"							
					25	Pushed Stone (No Recovery) at 25 ft					
8	■	2	M/W	50/3"							
					30						
9	■	3	M/W	50/3"							
					35						
10	■	3	M	50/4"							
					40						

WATER LEVEL OBSERVATIONS

While Drilling ▽ ________ Upon Completion of Drilling ________
Time After Drilling ________ ________ ________ ________
Depth to Water ________ ________ ________ ________ ▼
Depth to Cave in ________ ________ ________ ________

The stratification lines represent the approximate boundary between soil types and the transition may be gradual.

GENERAL NOTES

Start **8/6/10** End **8/6/10**
Driller **Badger** Chief **AP** Rig **D-120**
Logger **KD** Editor **DAS**
Drill Method **4 1/4" HSA 0-15'; 3-7/8" RB/DM 15'- 100'**

CGC Inc.

LOG OF TEST BORING

Project: **Proposed Parking Ramp**
Between E. Doty St. & E. Wilson St.
Location: **Madison, WI**

Boring No. **2**
Surface Elevation **63.7**
Job No. **C10041-5**
Sheet **2** of **3**

2921 PERRY STREET; MADISON, WIS. 53713 (608) 288-4100, FAX (608) 288-7887

SAMPLE						VISUAL CLASSIFICATION and Remarks	SOIL PROPERTIES				
No.	TYPE	Rec (in.)	Moist	N	Depth (ft)		qu (qa) (tsf)	W	LL	PL	LI
11	■	0	M	50/3"	45	Dense to Very Dense, Brown Fine to Medium SAND, Some Silt, Little to Some Gravel, Scattered Cobbles/Boulders (SM) Pushed Stone (No Recovery) at 45 ft					
12	■	10	M	50/5"	50						
13	■	5	M	50/5"	55						
14	■	3	M/W	50/3"	60						
						P200 = 30.7%					
15	■	4	M/W	50/5"	65						
16	■	4	M/W	50/4"	70						
17	■	4	M/W	50/4"	75						
18	■	4	M/W	50/5"	80	Fewer Cobbles Near 80 ft					
19	■	3	M/W	50/3"	85						

LOG OF TEST BORING

Project **Proposed Parking Ramp**
Between E. Doty St. & E. Wilson St.
Location **Madison, WI**

Boring No. **2**
Surface Elevation **63.7**
Job No. **C10041-5**
Sheet **3** of **3**

2921 PERRY STREET; MADISON, WIS. 53713 (608) 288-4100, FAX (608) 288-7887

SAMPLE						VISUAL CLASSIFICATION and Remarks	SOIL PROPERTIES				
No.	TYPE	Rec (in.)	Moist	N	Depth (ft)		qu (qa) (tsf)	W	LL	PL	LI
						Dense to Very Dense, Brown Fine to Medium SAND, Some Silt, Little to Some Gravel, Scattered Cobbles/Boulders (SM)					
20	■	3	M/W	50/3"	90						
21	■	4	M/W	50/4"	95						
22	■	9	M	50/3"	100	Color Changed to Gray Near 100 ft					
						End Boring at 100 ft Borehole backfilled with bentonite slurry, chips and asphalt patch					
					105						
					110						
					115						
					120						
					125						

CGC Inc.

LOG OF TEST BORING

Project Proposed Parking Ramp
Between E. Doty St. & E. Wilson St.
Location Madison, WI

Boring No. 3
Surface Elevation (ft) 60.9
Job No. C10041-5
Sheet 1 of 3

2921 Perry Street, Madison, WI 53713 (608) 288-4100, FAX (608) 288-7887

SAMPLE						VISUAL CLASSIFICATION and Remarks	SOIL PROPERTIES				
No.	TYPE	Rec (in.)	Moist	N	Depth (ft)		qu (qa) (tsf)	W	LL	PL	LI
						12 in. Black Silty Clay TOPSOIL FILL (OL)					
1	■	12	M	8		FILL: Stiff to Very Stiff, Brown to Dark Brown Silty to Lean Clay, Little Sand and Gravel	(2.2)				
2	■	12	M	12	5		(2.5)				
3	■	12	M	14			(1.7)				
4	■	12	M	15	10		(1.7)				
						Possible Concrete Layer at 11 to 12 ft					
						Dense to Very Dense, Brown Fine to Medium SAND, Some Silt, Little to Some Gravel, Scattered Cobbles/Boulders (SM)					
5	■	16	M	34	15						
6	■	6	M/W	50/2"	20						
						Very Dense, Light Brown Fine to Medium SAND, Trace to Little Silt (SP/SP-SM) (Lost 175 gallons of mud after augers advanced to 25 ft - possibly in above concrete layer)					
7	■	10	M/W	50/5"	25						
						Very Dense, Brown Sandy SILT, Trace Gravel (ML)					
8	■	2	M/W	50/3"	30						
						Dense to Very Dense, Brown Fine to Medium SAND, Some Silt, Little to Some Gravel, Scattered Cobbles/Boulders (SM)					
9	■	3	M/W	50/3"	35						
10	■	2	M/W	50/2"	40						

WATER LEVEL OBSERVATIONS

While Drilling ▽ 25'± Upon Completion of Drilling
Time After Drilling
Depth to Water ▼
Depth to Cave in

The stratification lines represent the approximate boundary between soil types and the transition may be gradual.

GENERAL NOTES

Start 8/9/10 End 8/9/10
Driller Badger Chief AP Rig D-120
Logger KD Editor DAS
Drill Method 4 1/4" HSA 0-15'; 3-7/8" RB/DM 15'- 95'

CGC Inc.

LOG OF TEST BORING

Project Proposed Parking Ramp
Between E. Doty St. & E. Wilson St.
Location Madison, WI

Boring No. 3
Surface Elevation 60.9
Job No. C10041-5
Sheet 2 of 3

2921 PERRY STREET; MADISON, WIS. 53713 (608) 288-4100, FAX (608) 288-7887

SAMPLE						VISUAL CLASSIFICATION and Remarks	SOIL PROPERTIES				
No.	TYPE	Rec (in.)	Moist	N	Depth (ft)		qu (qa) (tsf)	W	LL	PL	LI
						Dense to Very Dense, Brown Fine to Medium SAND, Some Silt, Little to Some Gravel, Scattered Cobbles/Boulders (SM)					
11	■	12	M	50/3"							
					45						
12	■	0	M	50/4"							
					50	Pushed Stone (No Recovery) at 50 ft					
13	■	4	M	50/5"							
					55						
14	■	4	W	50/5"		P200 = 34.6%					
					60						
15	■	3	M	50/3"							
					65						
16	■	4	M	50/4"							
					70						
17	■	5	M	50/5"							
					75						
18	■	5	M	50/5"							
					80						
19	■	3	M	50/3"							
					85						

LOG OF TEST BORING

CGC Inc.

Project **Proposed Parking Ramp**
Between E. Doty St. & E. Wilson St.
Location **Madison, WI**

Boring No. **3**
Surface Elevation **60.9**
Job No. **C10041-5**
Sheet **3** of **3**

2921 PERRY STREET; MADISON, WIS. 53713 (608) 288-4100, FAX (608) 288-7887

SAMPLE						VISUAL CLASSIFICATION and Remarks		SOIL PROPERTIES				
No.	TYPE	Rec (in.)	Moist	N	Depth (ft)			qu (qa) (tsf)	W	LL	PL	LI
						▨	Dense to Very Dense, Brown Fine to Medium SAND, Some Silt, Little to Some Gravel, Scattered Cobbles/Boulders (SM)					
20	■	4	W	50/4"		▨						
					90	▨						
21	■	2	W	50/3"		▨						
					95	▨						
					100		End Boring at 95 ft Borehole backfilled with bentonite slurry and chips					
					105							
					110							
					115							
					120							
					125							

CGC Inc.

LOG OF TEST BORING

Project **Proposed Parking Ramp**
Between E. Doty St. & E. Wilson St.
Location **Madison, WI**

Boring No. **4**
Surface Elevation (ft) **55.9**
Job No. **C10041-5**
Sheet **1** of **3**

2921 Perry Street, Madison, WI 53713 (608) 288-4100, FAX (608) 288-7887

SAMPLE						VISUAL CLASSIFICATION and Remarks	SOIL PROPERTIES				
No.	TYPE	Rec (in.)	Moist	N	Depth (ft)		qu (qa) (tsf)	W	LL	PL	LI
						5 in. Concrete/6 in. Base Course					
1	■	10	M	12		FILL: Stiff, Brown Lean Clay, Intermixed with Sand and Gravel	(1.2)				
2	■	12	M	26		Medium Dense to Very Dense, Brown Fine to Medium SAND, Some Silt, Little to Some Gravel, Scattered Cobbles/Boulders (SM) Possible Fill in Upper Few Feet of Layer					
					5						
3	■	16	M	44							
4	■	18	M	77							
					10						
5	■	16	M/W	71							
					15						
6	■	16	M/W	74							
					20						
7	■	8	M	50/3"							
					25						
8	■	6	M	50/3"							
					30						
9	■	16	M	97							
					35						
10	■	10	M/W	50/4"		P200 = 31.2%					
					40						

WATER LEVEL OBSERVATIONS

While Drilling ∇ **40'±** Upon Completion of Drilling ________
Time After Drilling ________ ________ ________ ________
Depth to Water ________ ________ ________ ________ ▼
Depth to Cave in ________ ________ ________ ________

The stratification lines represent the approximate boundary between soil types and the transition may be gradual.

GENERAL NOTES

Start **8/10/10** End **8/10/10**
Driller **Badger** Chief **AP** Rig **D-120**
Logger **KD** Editor **DAS**
Drill Method **4 1/4" HSA 0-10'; 3-7/8"**
RB/DM 10'- 90' Rope & Cathead Hammer

CGC Inc.

LOG OF TEST BORING

Project **Proposed Parking Ramp**
Between E. Doty St. & E. Wilson St.
Location **Madison, WI**

Boring No. **4**
Surface Elevation **55.9**
Job No. **C10041-5**
Sheet **2** of **3**

2921 PERRY STREET; MADISON, WIS. 53713 (608) 288-4100, FAX (608) 288-7887

SAMPLE						VISUAL CLASSIFICATION and Remarks	SOIL PROPERTIES				
No.	TYPE	Rec (in.)	Moist	N	Depth (ft)		qu (qa) (tsf)	W	LL	PL	LI
11	■	4	M/W	50/5"		Medium Dense to Very Dense, Brown Fine to Medium SAND, Some Silt, Little to Some Gravel, Scattered Cobbles/Boulders (SM)					
					45						
12	■	4	W	50/2"							
					50						
13	■	5	M	50/5"							
					55						
14	■	14	M	90							
					60						
15	■	3	M	50/5"							
					65						
16	■	4	M/W	50/4"							
					70						
17	■	3	M	50/5"							
					75						
18	■	3	M/W	50/3"							
					80						
19	■	4	M	50/5"		Color Changes to Gray-Brown Near 85 ft					
					85						

CGC Inc.

LOG OF TEST BORING

Project **Proposed Parking Ramp**
Between E. Doty St. & E. Wilson St.
Location **Madison, WI**

Boring No. **4**
Surface Elevation **55.9**
Job No. **C10041-5**
Sheet **3** of **3**

2921 PERRY STREET; MADISON, WIS. 53713 (608) 288-4100, FAX (608) 288-7887

SAMPLE						VISUAL CLASSIFICATION and Remarks	SOIL PROPERTIES				
No.	TYPE	Rec (in.)	Moist	N	Depth (ft)		qu (qa) (tsf)	W	LL	PL	LI
20	■	4	M	50/4"	90	Medium Dense to Very Dense, Brown Fine to Medium SAND, Some Silt, Little to Some Gravel, Scattered Cobbles/Boulders (SM)					
						End Boring at 90 ft Borehole backfilled with bentonite slurry and concrete patch					
					95						
					100						
					105						
					110						
					115						
					120						
					125						

LOG OF TEST BORING

CGC Inc.

Project: Proposed Parking Ramp
Between E. Doty St. & E. Wilson St.
Location: Madison, WI

Boring No. 5
Surface Elevation (ft) 56.4
Job No. C10041-5
Sheet 1 of 3

2921 Perry Street, Madison, WI 53713 (608) 288-4100, FAX (608) 288-7887

SAMPLE / SOIL PROPERTIES

No.	TYPE	Rec (in.)	Moist	N	qu (qa) (tsf)	W	LL	PL	LI
1	■	6	M	13	(1.5-2.0)				
2	■	4	M	24					
3	■	14	M	20					
4	■	8	M	50/4"					
5	■	12	M/W	89					
6	■	14	M	96					
7	■	2	M/W	50/5"					
8	■	3	M/W	50/3"					
9	■	10	M/W	50/4"					
10	■	10	M	50/4"					

Depth (ft): 5, 10, 15, 20, 25, 30, 35, 40

VISUAL CLASSIFICATION and Remarks

12 in. Dark Gray Silty Clay TOPSOIL FILL (OL)

FILL: Stiff, Brown to Dark Brown Silty to Lean Clay, Little Sand and Gravel

FILL: Medium Dense, Brown Fine to Medium Sand, Some Gravel, Little to Some Silt, Intermixed with Concrete and Brick Debris
Possible Concrete Layer Near 5 ft

Medium Dense to Very Dense, Brown Fine to Medium SAND, Some Silt, Little to Some Gravel, Scattered Cobbles/Boulders (SM)
Numerous Cobbles/Boulders near 10 ft

WATER LEVEL OBSERVATIONS

While Drilling ∇ MW-1 Upon Completion of Drilling ____

Time After Drilling	8-16-10	8-17-10	8-19-10	8-24-10
Depth to Water	39'	39.3'	39.1	39.0 ▼
Depth to Cave in				

The stratification lines represent the approximate boundary between soil types and the transition may be gradual.

GENERAL NOTES

Start 8/11/10 End 8/11/10
Driller Badger Chief AP Rig D-120
Logger KD Editor DAS
Drill Method 4 1/4" HSA 0-10'; 3-7/8" RB/DM 10'- 90' Rope & Cathead Hammer

CGC Inc.

LOG OF TEST BORING

Project **Proposed Parking Ramp**
Between E. Doty St. & E. Wilson St.
Location **Madison, WI**

Boring No. **5**
Surface Elevation **56.4**
Job No. **C10041-5**
Sheet **2** of **3**

2921 PERRY STREET; MADISON, WIS. 53713 (608) 288-4100, FAX (608) 288-7887

SAMPLE						VISUAL CLASSIFICATION and Remarks	SOIL PROPERTIES				
No.	TYPE	Rec (in.)	Moist	N	Depth (ft)		qu (qa) (tsf)	W	LL	PL	LI
						Medium Dense to Very Dense, Brown Fine to Medium SAND, Some Silt, Little to Some Gravel, Scattered Cobbles/Boulders (SM)					
11	■	16	M	100	45						
12	■	4	M/W	50/5"	50						
13	■	10	M/W	50/4"	55						
14	■	12	M/W	50/5"	60						
15	■	5	M	50/5"	65						
16	■	5	M	50/5"	70						
17	■	5	M/W	50/5"	75						
18	■	5	M/W	50/5"	80						
19	■	4	W	50/4"	85						

CGC Inc.

LOG OF TEST BORING

Project **Proposed Parking Ramp**
Between E. Doty St. & E. Wilson St.
Location **Madison, WI**

Boring No. **5**
Surface Elevation **56.4**
Job No. **C10041-5**
Sheet **3** of **3**

2921 PERRY STREET; MADISON, WIS. 53713 (608) 288-4100, FAX (608) 288-7887

SAMPLE						VISUAL CLASSIFICATION and Remarks	SOIL PROPERTIES				
No.	TYPE	Rec (in.)	Moist	N	Depth (ft)		qu (qa) (tsf)	W	LL	PL	LI
						Very Dense, Gray Fine SAND, Little Silt (SP-SM)					
20	■	3	M	50/3"	90						
						End Boring at 90 ft Borehole backfilled with bentonite slurry and chips					
					95						
					100						
					105						
					110						
					115						
					120						
					125						

CGC Inc.

LOG OF TEST BORING

Project **Proposed Parking Ramp**
Between E. Doty St. & E. Wilson St.
Location **Madison, WI**

Boring No. **6**
Surface Elevation (ft) **59.6**
Job No. **C10041-5**
Sheet **1** of **3**

2921 Perry Street, Madison, WI 53713 (608) 288-4100, FAX (608) 288-7887

SAMPLE						VISUAL CLASSIFICATION and Remarks	SOIL PROPERTIES				
No.	TYPE	Rec (in.)	Moist	N	Depth (ft)		qu (qa) (tsf)	W	LL	PL	LI
						Air Knife (No Sampling) to 4 ft					
1	■	6	M	20	5	FILL: Medium Dense to Dense, Brown Fine to Medium Sand, Some Silt, Little to Some Gravel, Intermixed with Clay Pockets, Cobbles and Boulders					
2	■	6	M	40							
3	■	16	M	54	10	Very Dense, Brown Fine to Medium SAND, Some Silt, Little to Some Gravel, Scattered Cobbles/Boulders (SM)					
4	■	2	M	50/5"	15						
5	■	3	M/W	50/5"	20						
6	■	3	M/W	50/5"	25						
7	■	2	M/W	50/2"	30						
8	■	3	W	50/5"	35	Very Dense, Light Brown Fine to Medium SAND, Little Silt (SP-SM)					
9	■	3	M	50/5"	40	Very Dense, Brown Fine to Medium SAND, Some Silt, Little to Some Gravel, Scattered					

WATER LEVEL OBSERVATIONS

While Drilling ▽ **35'±** Upon Completion of Drilling ______
Time After Drilling ______ ______ ______ ______
Depth to Water ______ ______ ______ ______ ▼
Depth to Cave in ______ ______ ______ ______

The stratification lines represent the approximate boundary between soil types and the transition may be gradual.

GENERAL NOTES

Start **8/16/10** End **8/16/10**
Driller **Badger** Chief **AP** Rig **D-120**
Logger **KD** Editor **DAS**
Drill Method **4 1/4" HSA 4-10'; 3-7/8" RB/DM 10'- 100'**

CGC Inc.

LOG OF TEST BORING

Project Proposed Parking Ramp
Between E. Doty St. & E. Wilson St.
Location Madison, WI

Boring No. 6
Surface Elevation 59.6
Job No. C10041-5
Sheet 2 of 3

2921 PERRY STREET; MADISON, WIS. 53713 (608) 288-4100, FAX (608) 288-7887

SAMPLE						VISUAL CLASSIFICATION and Remarks	SOIL PROPERTIES				
No.	TYPE	Rec (in.)	Moist	N	Depth (ft)		qu (qa) (tsf)	W	LL	PL	LI
						Cobbles/Boulders (SM)					
10	■	10	M	50/4"		Very Dense, Brown Fine to Medium SAND, Some Silt, Little to Some Gravel, Scattered Cobbles/Boulders (SM)					
					45						
11	■	1	M/W	50/2"							
					50						
12	■	4	M	50/5"							
					55						
13	■	10	M/W	50/5"							
					60						
14	■	8	M	50/3"							
					65						
15	■	4	M/W	50/5"							
					70						
						Very Dense, Gray Silty Fine to Medium SAND, Little Gravel, Trace Clay (SM)					
16	■	18	M	62							
					75						
17	■	0		50/0"		Drove Stone (No Recovery) at 80 ft					
					80						
						Very Dense, Brown Fine to Medium SAND, Some Silt, Little to Some Gravel, Scattered Cobbles/Boulders (SM)					
18	■	3	W	50/3"							
					85						

CGC Inc.

LOG OF TEST BORING

Project **Proposed Parking Ramp**
Between E. Doty St. & E. Wilson St.
Location **Madison, WI**

Boring No. **6**
Surface Elevation **59.6**
Job No. **C10041-5**
Sheet **3** of **3**

2921 PERRY STREET; MADISON, WIS. 53713 (608) 288-4100, FAX (608) 288-7887

SAMPLE						VISUAL CLASSIFICATION and Remarks	SOIL PROPERTIES				
No.	TYPE	Rec (in.)	Moist	N	Depth (ft)		qu (qa) (tsf)	W	LL	PL	LI
						Very Dense, Gray Sandy SILT, Laminated with Fine Sand and Clay Seams (ML)					
19	▮	3	M	50/3"	90	Very Dense, Gray Sandy SILT, Laminated with Fine Sand and Clay Seams (ML)					
						Hard, Gray Lean CLAY, Some Sand, Little to Some Gravel (CL)					
20	▮	1	M	50/3"	95		(-)				
21	▮	1	M/W	50/1"	100		(-)				
					105 110 115 120 125	End Boring at 100 ft Borehole backfilled with bentonite slurry, chips, and concrete patch					

CGC Inc.

LOG OF TEST BORING

Project Proposed Parking Ramp
Between E. Doty St. & E. Wilson St.
Location Madison, WI

Boring No. 7
Surface Elevation (ft) 49.3
Job No. C10041-5
Sheet 1 of 3

2921 Perry Street, Madison, WI 53713 (608) 288-4100, FAX (608) 288-7887

SAMPLE						VISUAL CLASSIFICATION and Remarks	SOIL PROPERTIES				
No.	TYPE	Rec (in.)	Moist	N	Depth (ft)		qu (qa) (tsf)	W	LL	PL	LI
					5	Air Knife (No Sampling) from 0 to 8 ft					
1	■	16	M	23	10	Medium Dense to Very Dense, Brown Fine to Medium SAND, Some Silt, Little to Some Gravel, Scattered Cobbles/Boulders (SM)					
2	■	18	M	77	15						
3	■	16	M	96	20						
4	■	16	M/W	92	25						
5	■	10	M	50/5"	30						
6	■	10	M	50/5"	35						
7	■	5	M/W	50/5"	40						

WATER LEVEL OBSERVATIONS

While Drilling ∇ ________ Upon Completion of Drilling ________
Time After Drilling ________ ________ ________ ________
Depth to Water ________ ________ ________ ________ ▼
Depth to Cave in ________ ________ ________ ________

The stratification lines represent the approximate boundary between soil types and the transition may be gradual.

GENERAL NOTES

Start 8/13/10 End 8/13/10
Driller Badger Chief AP Rig D-120
Logger KD Editor DAS
Drill Method 4 1/4" HSA 8-15'; 3-7/8" RB/DM 15'- 90'

CGC Inc.

LOG OF TEST BORING

Project Proposed Parking Ramp
Between E. Doty St. & E. Wilson St.
Location Madison, WI

Boring No. 7
Surface Elevation 49.3
Job No. C10041-5
Sheet 2 of 3

2921 PERRY STREET; MADISON, WIS. 53713 (608) 288-4100, FAX (608) 288-7887

SAMPLE						VISUAL CLASSIFICATION and Remarks	SOIL PROPERTIES				
No.	TYPE	Rec (in.)	Moist	N	Depth (ft)		qu (qa) (tsf)	W	LL	PL	LI
8	■	10	M	50/4"							
					45	P200 = 34.3% Medium Dense to Very Dense, Brown Fine to Medium SAND, Some Silt, Little Gravel, Scattered Cobbles/Boulders (SM)					
9	■	10	M	50/3"							
					50						
10	■	18	M	84							
					55						
11	■	10	M/W	50/4"							
					60						
12	■	4	M/W	50/5"							
					65						
13	■	4	W	50/4"							
					70						
14	■	3	W	50/5"							
					75						
15	■	2	M/W	50/2"		Very Dense, Gray-Brown Fine to Medium SAND, Trace to Little Silt and Gravel (SP/SP-SM)					
					80						
16	■	0		50/2"							
					85						

CGC Inc.

LOG OF TEST BORING

Project **Proposed Parking Ramp**
Between E. Doty St. & E. Wilson St.
Location **Madison, WI**

Boring No. **7**
Surface Elevation **49.3**
Job No. **C10041-5**
Sheet **3** of **3**

2921 PERRY STREET; MADISON, WIS. 53713 (608) 288-4100, FAX (608) 288-7887

SAMPLE						VISUAL CLASSIFICATION and Remarks	SOIL PROPERTIES				
No.	TYPE	Rec (in.)	Moist	N	Depth (ft)		qu (qa) (tsf)	W	LL	PL	LI
						Pushed Stone (No Recovery) at 85 ft					
17	▮	5	M	50/5"		Very Dense, Gray Sandy Clayey SILT, Little Gravel (ML)					
					90	End Boring at 90 ft Borehole backfilled with bentonite slurry, chips and asphalt patch					
					95						
					100						
					105						
					110						
					115						
					120						
					125						

CGC Inc.

LOG OF TEST BORING

Project **Proposed Parking Ramp**
Between E. Doty St. & E. Wilson St.
Location **Madison, WI**

Boring No. **4P**
Surface Elevation (ft) **56±**
Job No. **C10041-5**
Sheet **1** of **2**

2921 Perry Street, Madison, WI 53713 (608) 288-4100, FAX (608) 288-7887

SAMPLE						VISUAL CLASSIFICATION and Remarks	SOIL PROPERTIES				
No.	TYPE	Rec (in.)	Moist	N	Depth (ft)		qu (qa) (tsf)	W	LL	PL	LI
					5 10 15 20 25 30	Boring 4A offset 11 ft north from Boring 4 and blind drilled without sampling to 35 ft (see Boring 4 for soil descriptions).					
1	■	1	W	50/1"	35	Very Dense, Brown Fine to Medium SAND, Some Silt, Little to Some Gravel, Scattered Cobbles/Boulders (SM) PMT #1 - 35.5 ft to 38 ft					
					40						

WATER LEVEL OBSERVATIONS

While Drilling ∇ ______ Upon Completion of Drilling ______
Time After Drilling
Depth to Water ▼
Depth to Cave in

The stratification lines represent the approximate boundary between soil types and the transition may be gradual.

GENERAL NOTES

Start **9/8/10** End **9/8/10**
Driller **Badger** Chief **AP** Rig **D-120**
Logger **KD** Editor **DAS**
Drill Method **4 1/4" HSA 0-10'; 3-7/8" RB/DM 10'- 61.5'; Rope & Cathead Hammer**

CGC Inc.

LOG OF TEST BORING

Project: **Proposed Parking Ramp**
Between E. Doty St. & E. Wilson St.
Location: **Madison, WI**

Boring No. **4P**
Surface Elevation **56±**
Job No. **C10041-5**
Sheet **2** of **2**

2921 PERRY STREET; MADISON, WIS. 53713 (608) 288-4100, FAX (608) 288-7887

SAMPLE						VISUAL CLASSIFICATION and Remarks	SOIL PROPERTIES				
No.	TYPE	Rec (in.)	Moist	N	Depth (ft)		qu (qa) (tsf)	W	LL	PL	LI
2	■	5	M	65		PMT #2 - 42 ft to 44.5 ft					
					45			9.2			
3	■	4	M	50/5"	50	PMT #3 - 51 ft to 53.5 ft					
					55			12.1			
4	■	0		50/2"	60	PMT #4 - 59 ft to 61.5 ft					
					65	End Boring at 61.5 ft Borehole backfilled with bentonite chips and asphalt patch					
					70						
					75						
					80						
					85						

CGC Inc.

LOG OF TEST BORING

Project **Proposed Parking Ramp**
Between E. Doty St. & E. Wilson St.
Location **Madison, WI**

Boring No. **5P**
Surface Elevation (ft) **56.5±**
Job No. **C10041-5**
Sheet **1** of **2**

2921 Perry Street, Madison, WI 53713 (608) 288-4100, FAX (608) 288-7887

SAMPLE						VISUAL CLASSIFICATION and Remarks	SOIL PROPERTIES				
No.	TYPE	Rec (in.)	Moist	N	Depth (ft)		qu (qa) (tsf)	W	LL	PL	LI
					5 10 15 20 25 30	Boring 5A offset 4.5 ft north from Boring 5 and blind drilled without sampling to 35 ft (see Boring 5 for soil descriptions).					
1	■	10	M	50/5"	35 40	Very Dense, Brown Fine to Medium SAND, Some Silt, Little to Some Gravel, Scattered Cobbles/Boulders (SM) PMT #1 - 36 ft to 38.5 ft		8.7			

WATER LEVEL OBSERVATIONS

While Drilling ▽ ________ Upon Completion of Drilling ________
Time After Drilling ________ ________ ________ ________
Depth to Water ________ ________ ________ ________ ▼
Depth to Cave in ________ ________ ________ ________

The stratification lines represent the approximate boundary between soil types and the transition may be gradual.

GENERAL NOTES

Start **9/8/10** End **9/8/10**
Driller **Badger** Chief **AP** Rig **D-120**
Logger **KD** Editor **DAS**
Drill Method **4 1/4" HSA 0-10'; 3-7/8" RB/DM 10'- 62'; Rope & Cathead Hammer**

CGC Inc.

LOG OF TEST BORING

Project Proposed Parking Ramp
Between E. Doty St. & E. Wilson St.
Location Madison, WI

Boring No. 5P
Surface Elevation 56.5±
Job No. C10041-5
Sheet 2 of 2

2921 PERRY STREET; MADISON, WIS. 53713 (608) 288-4100, FAX (608) 288-7887

SAMPLE						VISUAL CLASSIFICATION and Remarks	SOIL PROPERTIES				
No.	TYPE	Rec (in.)	Moist	N	Depth (ft)		qu (qa) (tsf)	W	LL	PL	LI
2	■	8	M/W	50/4"		PMT #2 - 42.5 ft to 45 ft		10.8			
					45						
					50						
3	■	4	M/W	50		PMT #3 - 51.5 ft to 54 ft		12.6			
					55						
4	■	4	M	50	60	PMT #4 - 59.5 ft to 62 ft		9.2			
						End Boring at 62 ft Borehole backfilled with bentonite chips					
					65						
					70						
					75						
					80						
					85						

CGC Inc.

LOG OF TEST BORING

Project **Proposed Parking Ramp**
Between E. Doty St. & E. Wilson St.
Location **Madison, WI**

Boring No. **MW-1**
Surface Elevation (ft) **56.4**
Job No. **C10041-5**
Sheet **1** of **2**

2921 Perry Street, Madison, WI 53713 (608) 288-4100, FAX (608) 288-7887

SAMPLE						VISUAL CLASSIFICATION and Remarks	SOIL PROPERTIES				
No.	TYPE	Rec (in.)	Moist	N	Depth (ft)		qu (qa) (tsf)	W	LL	PL	LI
						Monitoring Well No. 1 located approximately 4 ft north of Boring 5 Blind drilled with 4-1/4 in. HSA to 10 ft and then switched to 3-7/8 in. roller bit to 66 ft					
					5						
					10	Set well at 65 ft					
						20 ft of screen (65 ft to 45 ft)					
					15	Filter Sand 66 ft to 40 ft					
						Fine Sand from 40 ft to 39 ft					
						Bentonite chips from 39 ft to 1 ft					
					20	Flush mount top concreted in place					
					25						
					30						
					35						
					40						

WATER LEVEL OBSERVATIONS

While Drilling ∇ ________ Upon Completion of Drilling ________

Time After Drilling	8-16-10	8-17-10	8-19-10	8-24-10
Depth to Water	39'	39.3'	39.1'	39.0' ▼
Depth to Cave in				

The stratification lines represent the approximate boundary between soil types and the transition may be gradual.

GENERAL NOTES

Start **8/12/10** End **8/12/10**
Driller **Badger** Chief **AP** Rig **D-120**
Logger **KD** Editor **DAS**
Drill Method **4 1/4" HSA 0-10'; 3-7/8" RB/DM 10'- 66'**

CGC Inc.

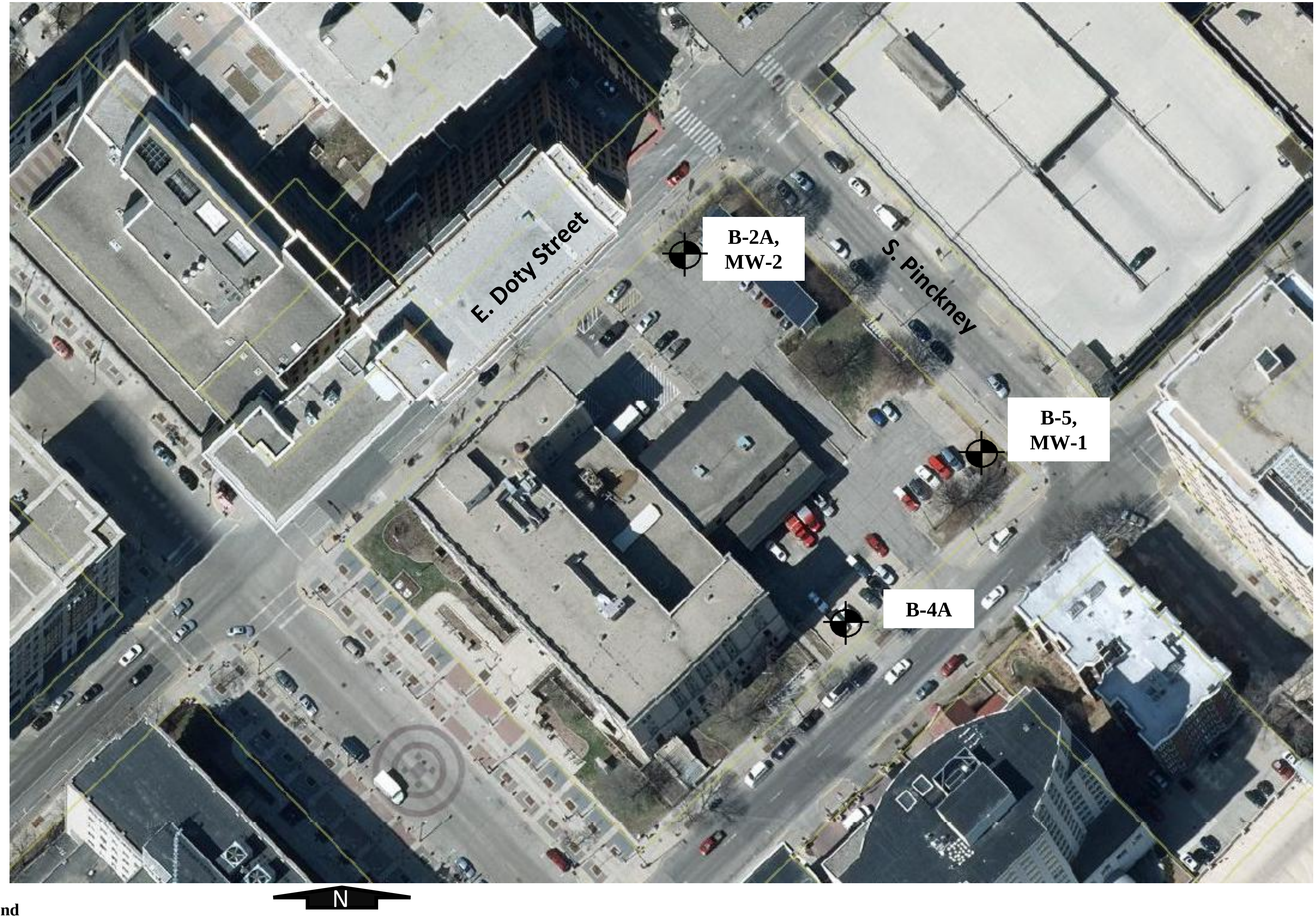
LOG OF TEST BORING

Project **Proposed Parking Ramp**
Between E. Doty St. & E. Wilson St.
Location **Madison, WI**

Boring No. **MW-1**
Surface Elevation **56.4**
Job No. **C10041-5**
Sheet **2** of **2**

2921 PERRY STREET; MADISON, WIS. 53713 (608) 288-4100, FAX (608) 288-7887

SAMPLE						VISUAL CLASSIFICATION and Remarks	SOIL PROPERTIES				
No.	TYPE	Rec (in.)	Moist	N	Depth (ft)		qu (qa) (tsf)	W	LL	PL	LI
					45						
					50						
					55						
					60						
					65						
						End Boring at 66 ft					
					70						
					75						
					80						
					85						



Legend

⊕ Denotes Proposed Boring Location and Number

Notes: 1. Borings drilled by Badger State Drilling on October 5, 2015
2. Base map from Dane County DCiMap.
3. Boring locations are approximate.

Job No.
C15237

Date:
10/2015

CGC, Inc.

SOIL BORING LOCATION EXHIBIT
Judge Doyle Square Development
E. Doty Street & S. Pinckney Street
Madison, WI

CGC Inc.

LOG OF TEST BORING

Project: **Proposed Judge Doyle Square**
E. Doty & S. Pinckney Street
Location: **Madison, Wisconsin**

Boring No. **2A**
Surface Elevation (ft) **910.1**
Job No. **C15237**
Sheet **1** of **2**

2921 Perry Street, Madison, WI 53713 (608) 288-4100, FAX (608) 288-7887

SAMPLE						VISUAL CLASSIFICATION and Remarks	SOIL PROPERTIES				
No.	TYPE	Rec (in.)	Moist	N	Depth (ft)		qu (qa) (tsf)	W	LL	PL	LI
						6 in. ± TOPSOIL FILL (OL)					
					5 10 15 20 25	Blind-Drilled (No Sampling) from 0 to 28.5 ft (See Boring 2)					
1	■	24	M	94	30	Very Dense, Brown Silty Fine to Medium SAND, Some Gravel, Scattered Cobbles/Boulders (SM)					
						PMT #1: 28.5 - 31 ft					
2	■	15	M	88/11"	35	PMT #2: 35 to 37.5 ft P200 (Sample 2): 35.2%		9.5			
					40						

WATER LEVEL OBSERVATIONS

While Drilling ▽ ________ Upon Completion of Drilling ________

Time After Drilling				10/20/15
Depth to Water				45.8 ▼
Depth to Cave in				(864.3)

The stratification lines represent the approximate boundary between soil types and the transition may be gradual.

GENERAL NOTES

Start **10/5/15** End **10/5/15**
Driller ________ Chief **KD/DD** Rig **D-120**
Logger **MC** Editor **DAS**
Drill Method **4.25" HSA 0'-28.5';3-7/8" RB/DM 28.5'-63'; Autohammer**

CGC Inc.

LOG OF TEST BORING

Project Proposed Judge Doyle Square
E. Doty & S. Pinckney Street
Location Madison, Wisconsin

Boring No. 2A
Surface Elevation 910.1
Job No. C15237
Sheet 2 of 2

2921 PERRY STREET, MADISON, WIS. 53713 (608) 288-4100, FAX (608) 288-7887

SAMPLE						VISUAL CLASSIFICATION and Remarks	SOIL PROPERTIES				
No.	TYPE	Rec (in.)	Moist	N	Depth (ft)		qu (qa) (tsf)	W	LL	PL	LI
3	■	12	M/W	85/9"		PMT #3: 41 to 43.5 ft		10.7			
					45 ▼ 50	P200 (Sample 3): 42.2% PMT #4: 52.5 to 55 ft					
4	■	6	M/W	50/5"		P200 (Sample 4): 42.1%		9.3			
					55 60	Blind-Drilled (Witout Sampling) from 54 to 63 ft					
					65 70 75 80 85	End Boring at 63 ft Set 2 in. PVC Monitoring Well with Flush Mount Cover					

LOG OF TEST BORING

Project **Proposed Judge Doyle Square**
E. Doty & S. Pinckney Street
Location **Madison, Wisconsin**

Boring No. **4A**
Surface Elevation (ft) **900.7**
Job No. **C15237**
Sheet **1** of **1**

2921 Perry Street, Madison, WI 53713 (608) 288-4100, FAX (608) 288-7887

SAMPLE						VISUAL CLASSIFICATION and Remarks	SOIL PROPERTIES				
No.	TYPE	Rec (in.)	Moist	N	Depth (ft)		qu (qa) (tsf)	W	LL	PL	LI
						4 in. Concrete Pavement/6 in. Base Course					
						Blind-Drilled (No Sampling) from 0 to 20 ft (See Boring 4)					
1	■	24	M	72	20	Very Dense, Brown Silty Fine to Medium SAND, Some Gravel, Scattered Cobbles/Boulders (SM)					
						PMT #1: 20.5 to 23 ft					
2	■	24	M	78	25	PMT #2: 25.5 to 28.5 ft P200 (Sample 2): 36.9%					
								6.9			
3	■	24	M	62	30	PMT #3: 30 to 32.5 ft					
						End Boring at 32.5 ft Backfilled with Bentonite Chips and Asphalt Patch					

WATER LEVEL OBSERVATIONS

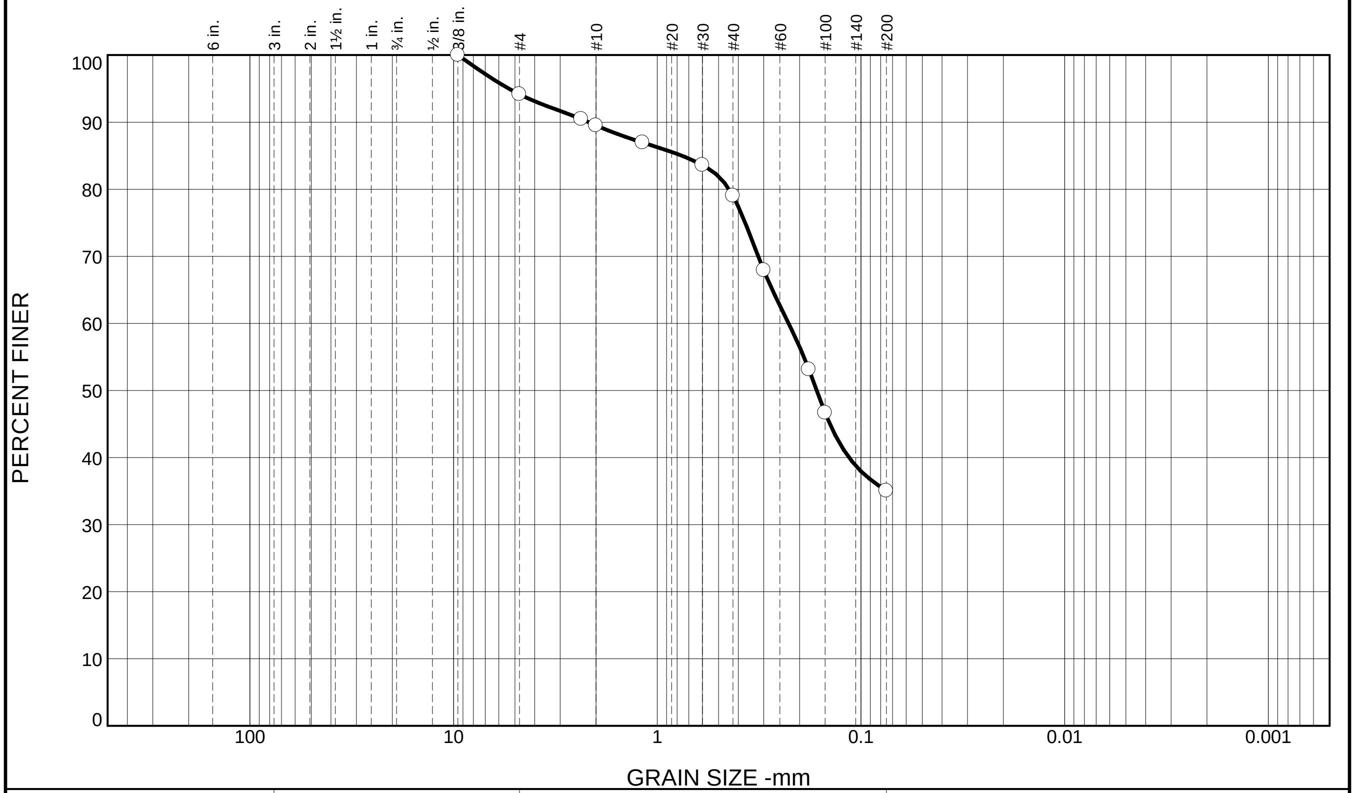
While Drilling ∇ NW Upon Completion of Drilling NW
Time After Drilling
Depth to Water
Depth to Cave in

The stratification lines represent the approximate boundary between soil types and the transition may be gradual.

GENERAL NOTES

Start **10/5/15** End **10/5/15**
Driller **BSD** Chief **KD/DD** Rig **D-120**
Logger **MC** Editor **DAS**
Drill Method **4.25" HSA; Autohammer**

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0.0	0.0	5.9	4.6	10.5	44.0	35.0	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
3/8	100.0		
#4	94.1		
#8	90.5		
#10	89.5		
#16	87.0		
#30	83.6		
#40	79.0		
#50	67.9		
#80	53.1		
#100	46.7		
#200	35.0		

* (no specification provided)

Material Description

Brown Fine to Medium Sand, Some Silt, Little Gravel

Atterberg Limits

PL= LL= PI=

Coefficients

D_{90}= 2.1793 D_{85}= 0.7586 D_{60}= 0.2266
D_{50}= 0.1649 D_{30}= D_{15}=
D_{10}= C_u= C_c=

Classification

USCS= SM AASHTO=

Remarks

Sample Number: B-5A **Depth:** 42-43.5 ft

Date: 9/16/10

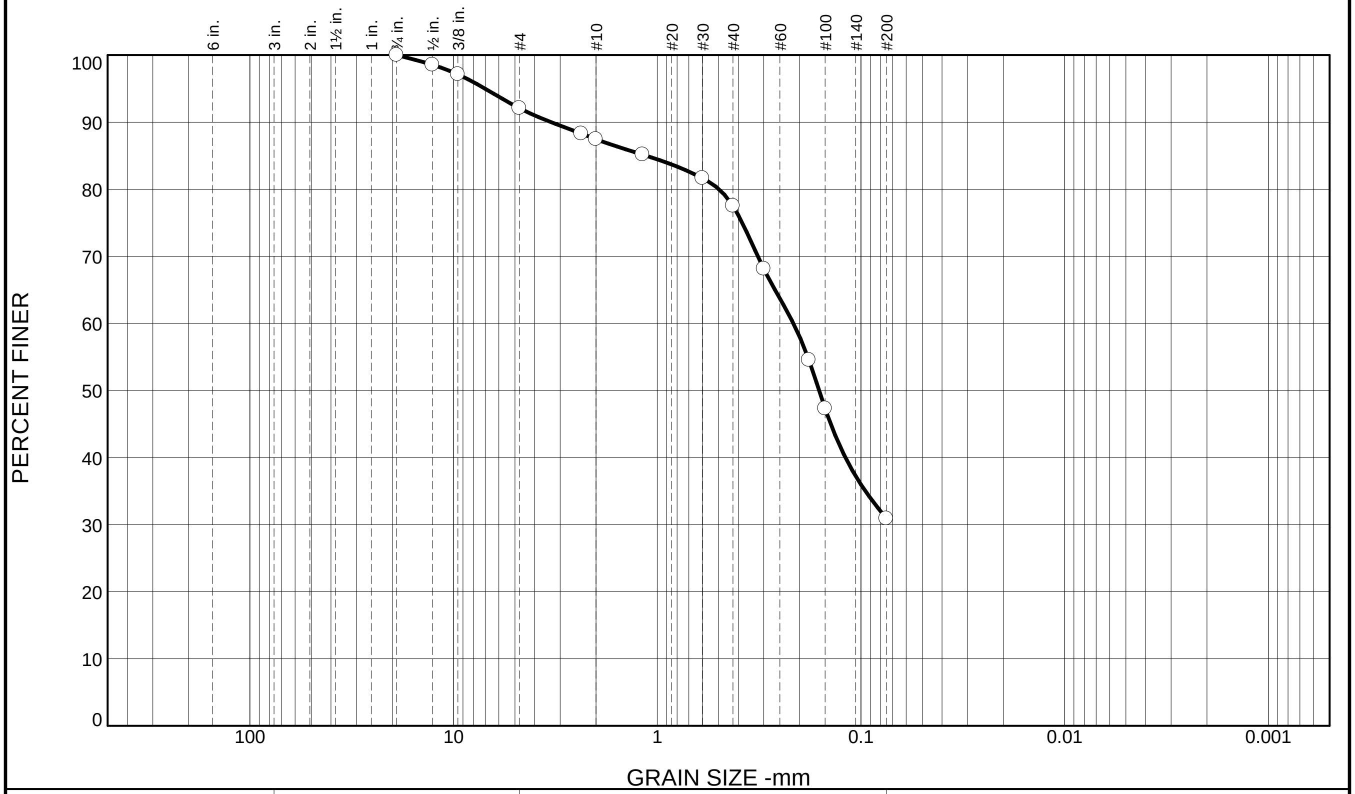
CGC,Inc.

Client: City of Madison
Project: Madison Parking Ramp

Project No: C10041-5 **Figure**

Tested By: JSG **Checked By:** DAS

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0.0	0.0	7.9	4.6	10.0	46.6	30.9	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
3/4	100.0		
1/2	98.5		
3/8	97.1		
#4	92.1		
#8	88.3		
#10	87.5		
#16	85.2		
#30	81.6		
#40	77.5		
#50	68.1		
#80	54.5		
#100	47.3		
#200	30.9		

* (no specification provided)

Material Description

Brown Fine to Medium Sand, Some Silt, Little Gravel

Atterberg Limits

PL= LL= PI=

Coefficients

D_{90}= 3.3185 D_{85}= 1.1351 D_{60}= 0.2147

D_{50}= 0.1605 D_{30}= D_{15}=

D_{10}= C_u= C_c=

Classification

USCS= SM AASHTO=

Remarks

Sample Number: B1-S8 (30 ft)

Date: 8/16/10

CGC,Inc.

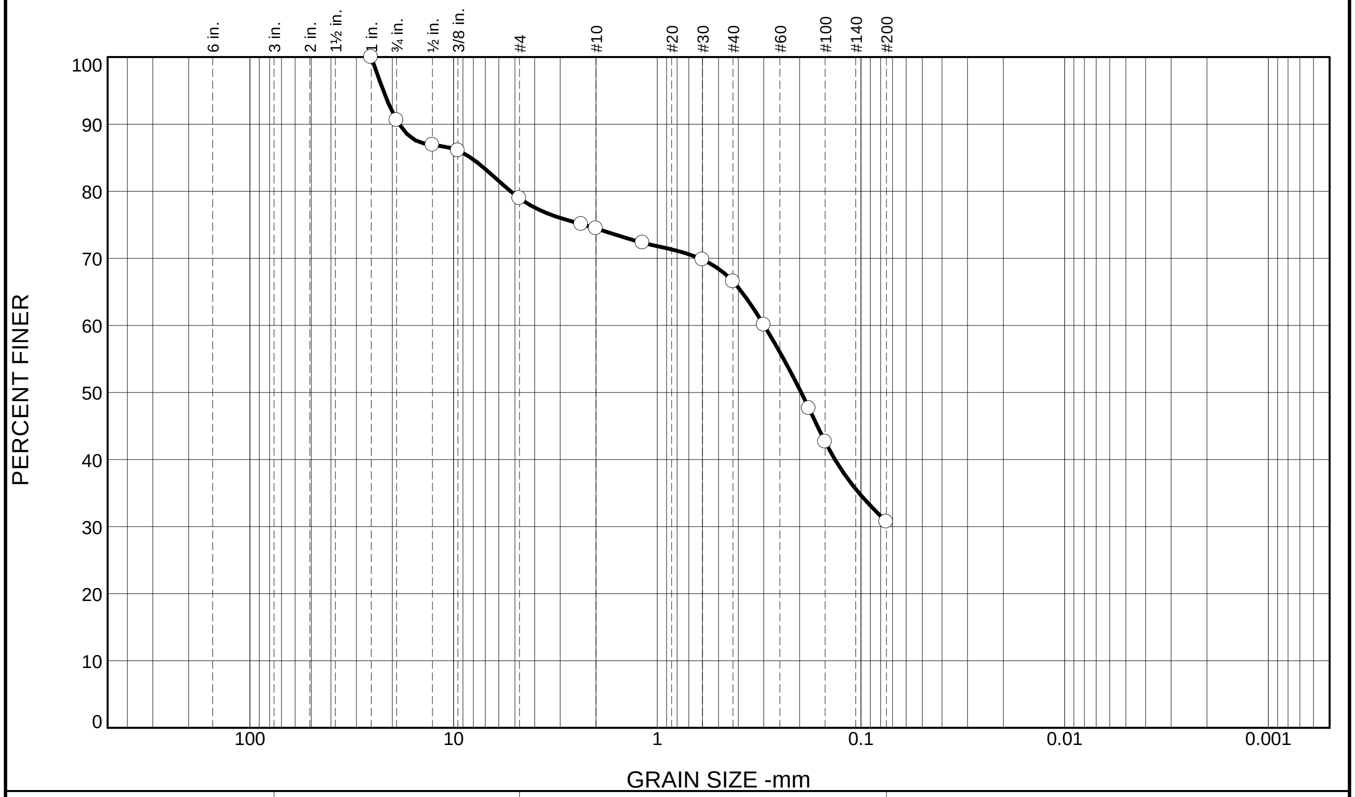
Client: City of Madison

Project: Madison Parking Ramp

Project No: C10041-5 **Figure**

Tested By: JSG **Checked By:** DAS

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0.0	9.4	11.6	4.6	7.8	35.9	30.7	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
1	100.0		
3/4	90.6		
1/2	86.9		
3/8	86.1		
#4	79.0		
#8	75.1		
#10	74.4		
#16	72.3		
#30	69.8		
#40	66.6		
#50	60.1		
#80	47.6		
#100	42.6		
#200	30.7		

* (no specification provided)

Material Description

Brown Fine to Medium Sand, Some Silt and Gravel

Atterberg Limits

PL= LL= PI=

Coefficients

D_{90}= 18.5421 D_{85}= 8.2372 D_{60}= 0.2988
D_{50}= 0.1965 D_{30}= D_{15}=
D_{10}= C_u= C_c=

Classification

USCS= SM AASHTO=

Remarks

Sample Number: B2-S15 (65 ft)

Date: 8/16/10

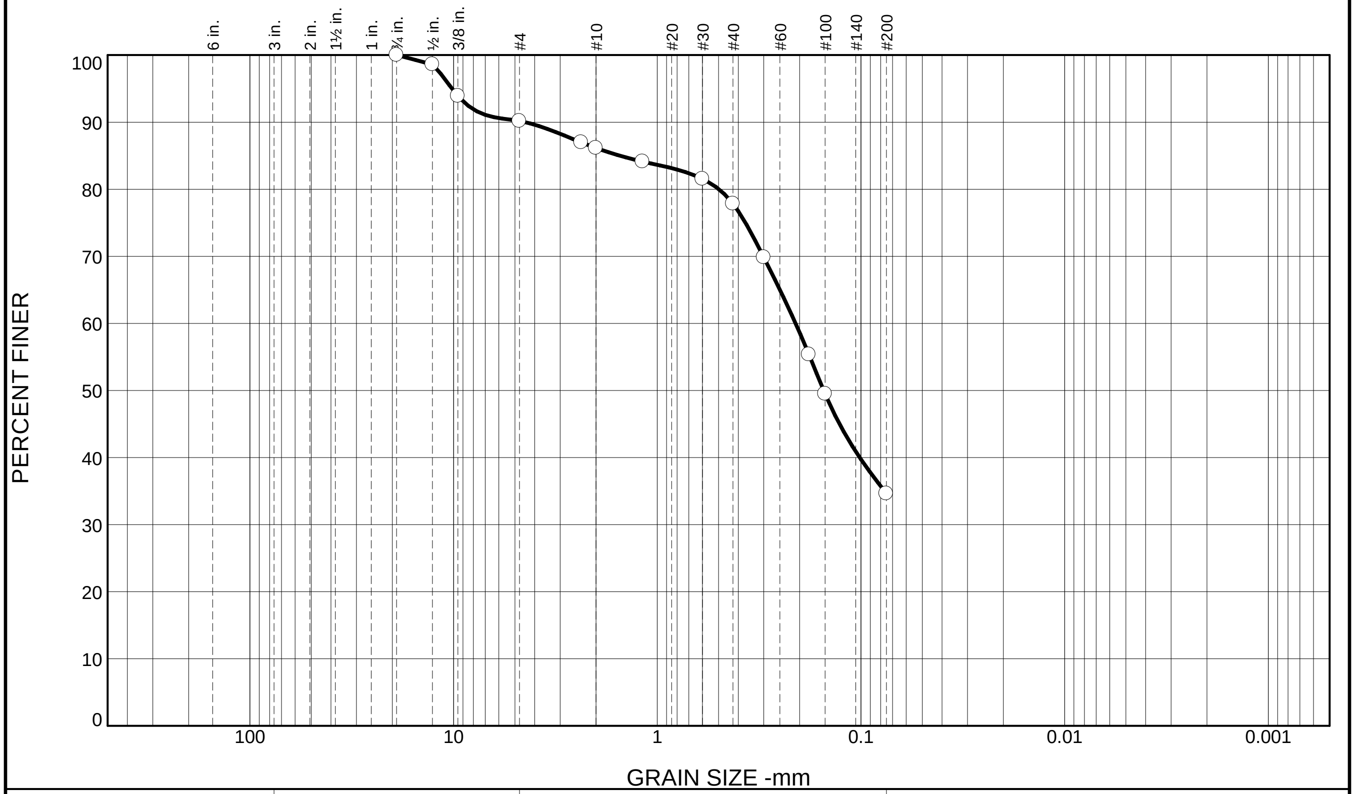
CGC,Inc.

Client: City of Madison
Project: Madison Parking Ramp

Project No: C10041-5 **Figure**

Tested By: JSG **Checked By:** DAS

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0.0	0.0	9.8	4.1	8.3	43.2	34.6	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
3/4	100.0		
1/2	98.6		
3/8	93.9		
#4	90.2		
#8	87.0		
#10	86.1		
#16	84.1		
#30	81.5		
#40	77.8		
#50	69.9		
#80	55.4		
#100	49.5		
#200	34.6		

* (no specification provided)

Material Description

Brown Fine to Medium Sand, Some Silt, Little Gravel

Atterberg Limits

PL= LL= PI=

Coefficients

D_{90}= 4.4649 D_{85}= 1.5285 D_{60}= 0.2094

D_{50}= 0.1525 D_{30}= D_{15}=

D_{10}= C_u= C_c=

Classification

USCS= SM AASHTO=

Remarks

Sample Number: B3-S14 (60 ft)

Date: 8/16/10

CGC,Inc.

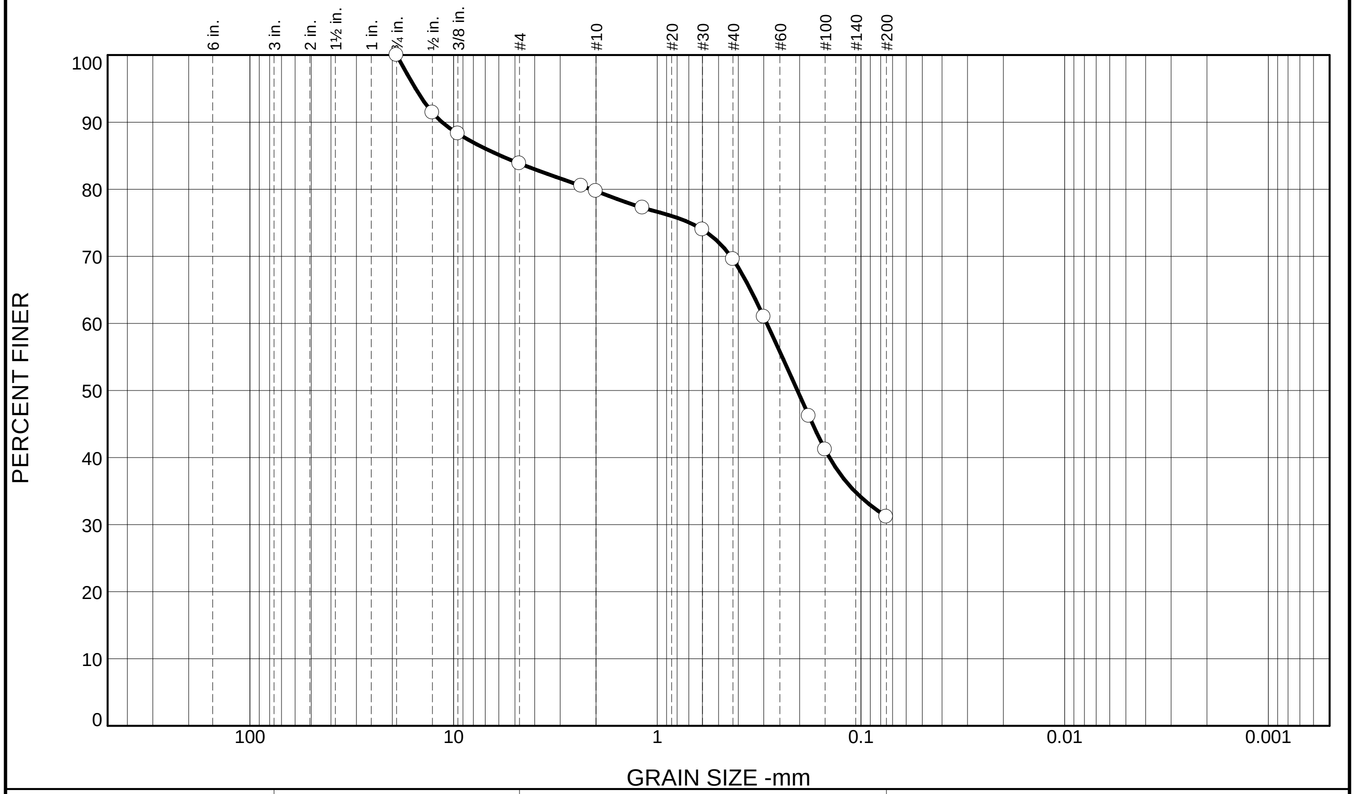
Client: City of Madison

Project: Madison Parking Ramp

Project No: C10041-5 **Figure**

Tested By: JSG **Checked By:** DAS

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0.0	0.0	16.2	4.1	10.1	38.4	31.2	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
3/4	100.0		
1/2	91.4		
3/8	88.3		
#4	83.8		
#8	80.5		
#10	79.7		
#16	77.2		
#30	74.0		
#40	69.6		
#50	61.0		
#80	46.2		
#100	41.2		
#200	31.2		

* (no specification provided)

Material Description

Brown Fine to Medium Sand, Some Silt and Gravel

Atterberg Limits

PL= LL= PI=

Coefficients

D_{90}= 11.4115 D_{85}= 5.8752 D_{60}= 0.2899

D_{50}= 0.2051 D_{30}= D_{15}=

D_{10}= C_u= C_c=

Classification

USCS= SM AASHTO=

Remarks

Sample Number: B4-S10 (40 ft)

Date: 8/16/10

CGC,Inc.

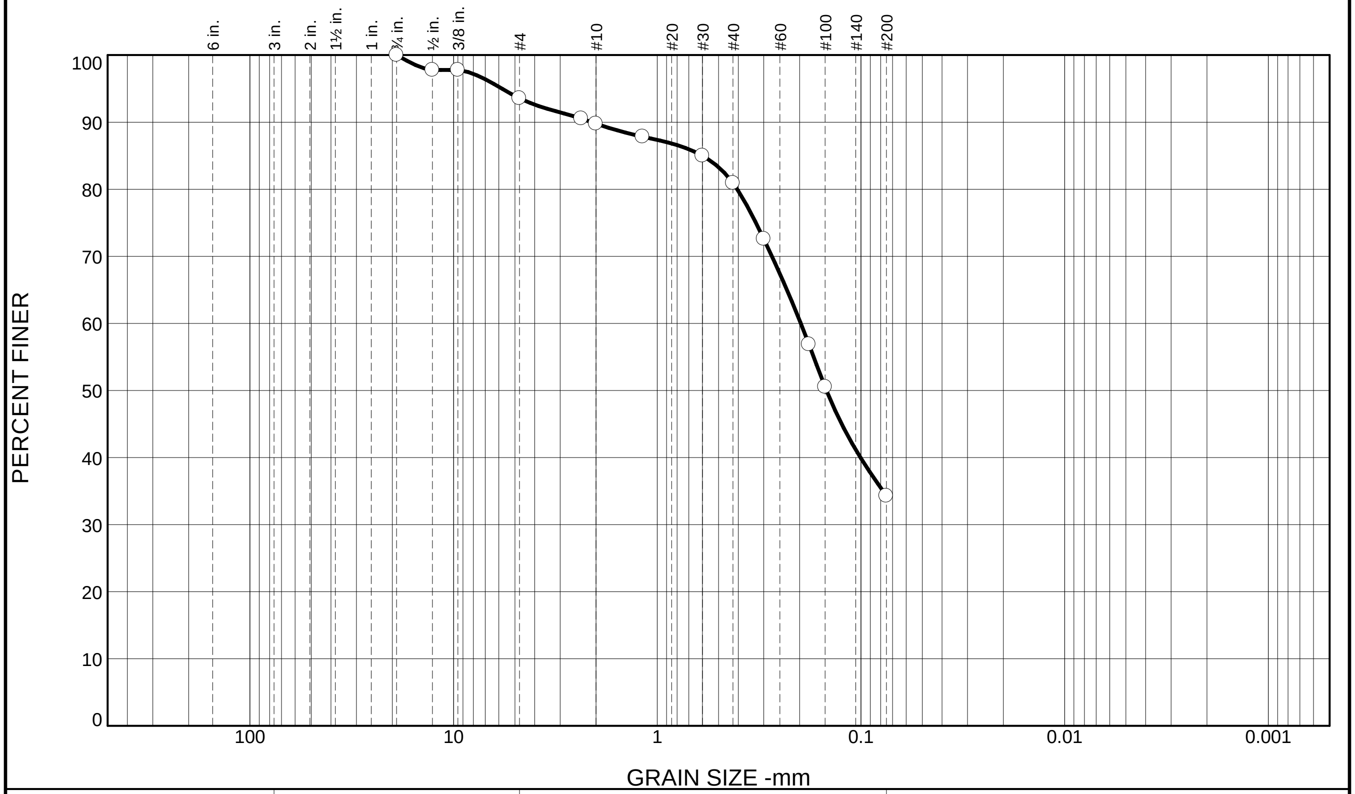
Client: City of Madison

Project: Madison Parking Ramp

Project No: C10041-5 **Figure**

Tested By: JSG **Checked By:** DAS

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0.0	0.0	6.4	3.8	8.9	46.6	34.3	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
3/4	100.0		
1/2	97.8		
3/8	97.8		
#4	93.6		
#8	90.5		
#10	89.8		
#16	87.8		
#30	85.0		
#40	80.9		
#50	72.6		
#80	56.9		
#100	50.5		
#200	34.3		

* (no specification provided)

Material Description

Brown Fine to Medium Sand, Some Silt, Little Gravel

Atterberg Limits

PL= LL= PI=

Coefficients

D_{90}= 2.1012 D_{85}= 0.6008 D_{60}= 0.1974
D_{50}= 0.1476 D_{30}= D_{15}=
D_{10}= C_u= C_c=

Classification

USCS= SM AASHTO=

Remarks

Sample Number: B7-S8 (45 ft)

Date: 8/16/10

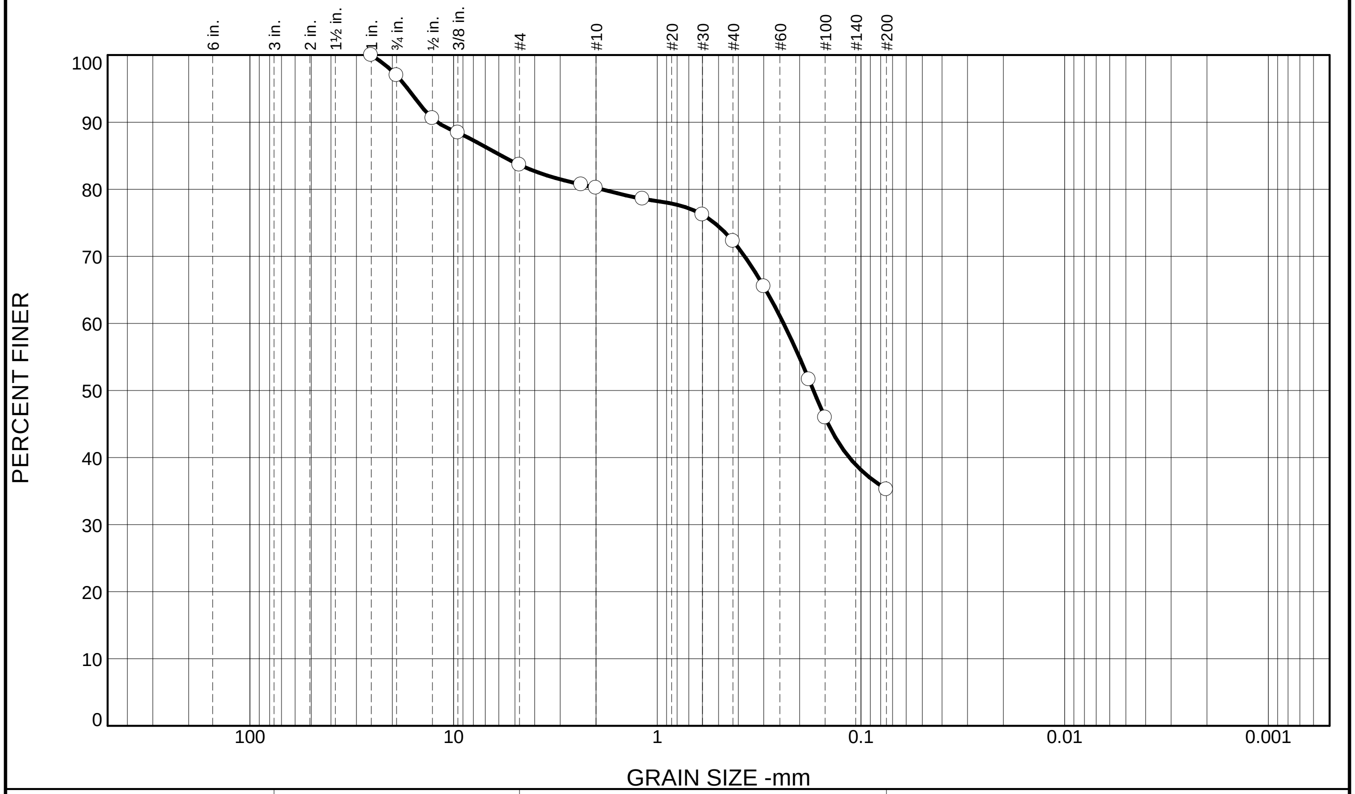
CGC,Inc.

Client: City of Madison
Project: Madison Parking Ramp

Project No: C10041-5 **Figure**

Tested By: JSG **Checked By:** DAS

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0.0	3.0	13.4	3.4	7.9	37.1	35.2	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
1	100.0		
3/4	97.0		
1/2	90.6		
3/8	88.4		
#4	83.6		
#8	80.7		
#10	80.2		
#16	78.6		
#30	76.2		
#40	72.3		
#50	65.5		
#80	51.6		
#100	45.9		
#200	35.2		

* (no specification provided)

Material Description

Brown Silty Fine to Medium Sand, Some Gravel

Atterberg Limits

PL= LL= PI=

Coefficients

D_{90}= 12.0191	D_{85}= 5.8306	D_{60}= 0.2399
D_{50}= 0.1711	D_{30}=	D_{15}=
D_{10}=	C_u=	C_c=

Classification

USCS= SM AASHTO=

Remarks

Natural Moisture = 9.5%

Sample Number: B-2A/2A+2B+2C

Date: 10/7/15

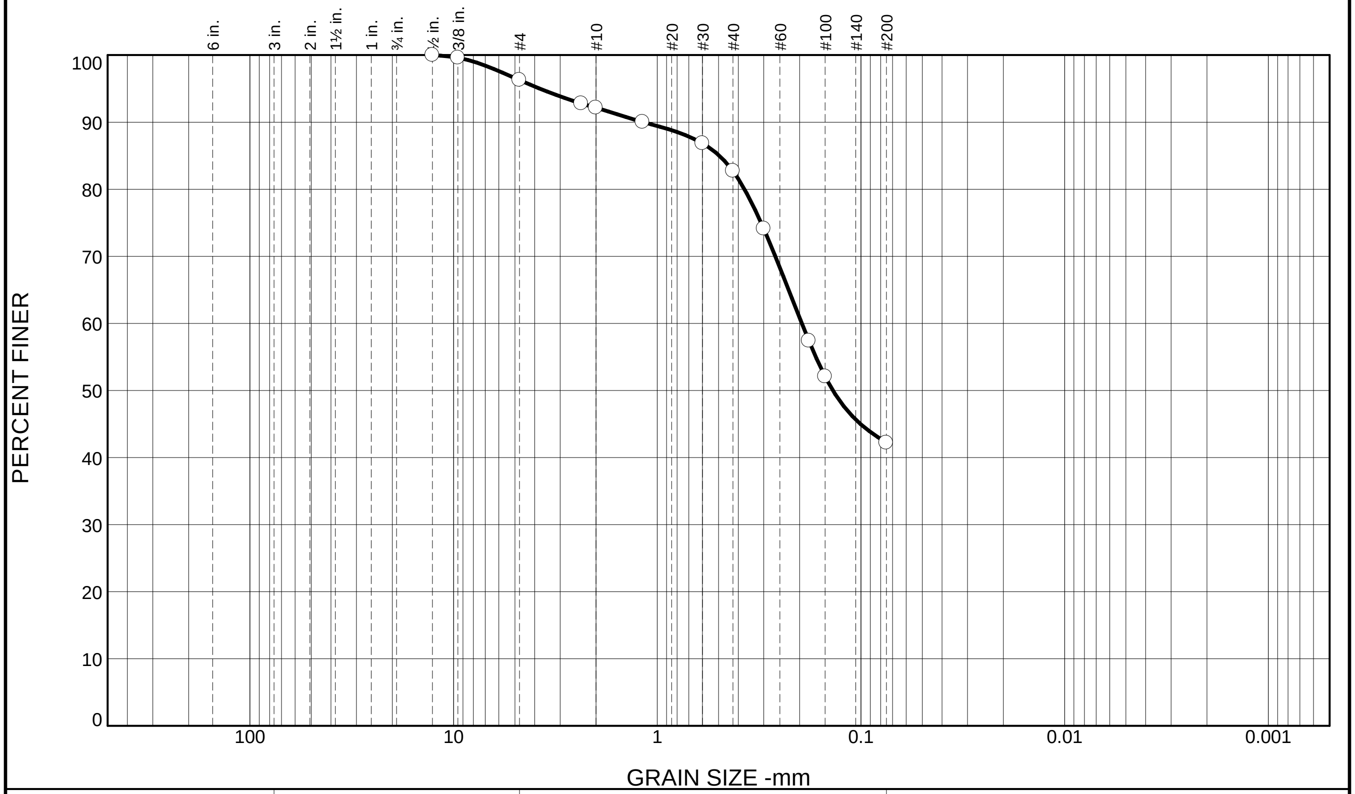
CGC,Inc.

Client: Hammes Co. Sports Dev.
Project: JDS Supplemental PMTs

Project No: C15237 **Figure**

Tested By: DRW **Checked By:** DAS

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0.0	0.0	3.7	4.1	9.5	40.5	42.2	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
1/2	100.0		
3/8	99.6		
#4	96.3		
#8	92.8		
#10	92.2		
#16	90.0		
#30	86.8		
#40	82.7		
#50	74.1		
#80	57.4		
#100	52.1		
#200	42.2		

* (no specification provided)

Material Description

Brown Silty Fine to Medium Sand, Trace Gravel

Atterberg Limits

PL= LL= PI=

Coefficients

D_{90}= 1.1730 D_{85}= 0.4955 D_{60}= 0.1951
D_{50}= 0.1374 D_{30}= D_{15}=
D_{10}= C_u= C_c=

Classification

USCS= SM AASHTO=

Remarks

Natural Moisture = 10.7%

Sample Number: B-2A/3+3B

Date: 10/7/15

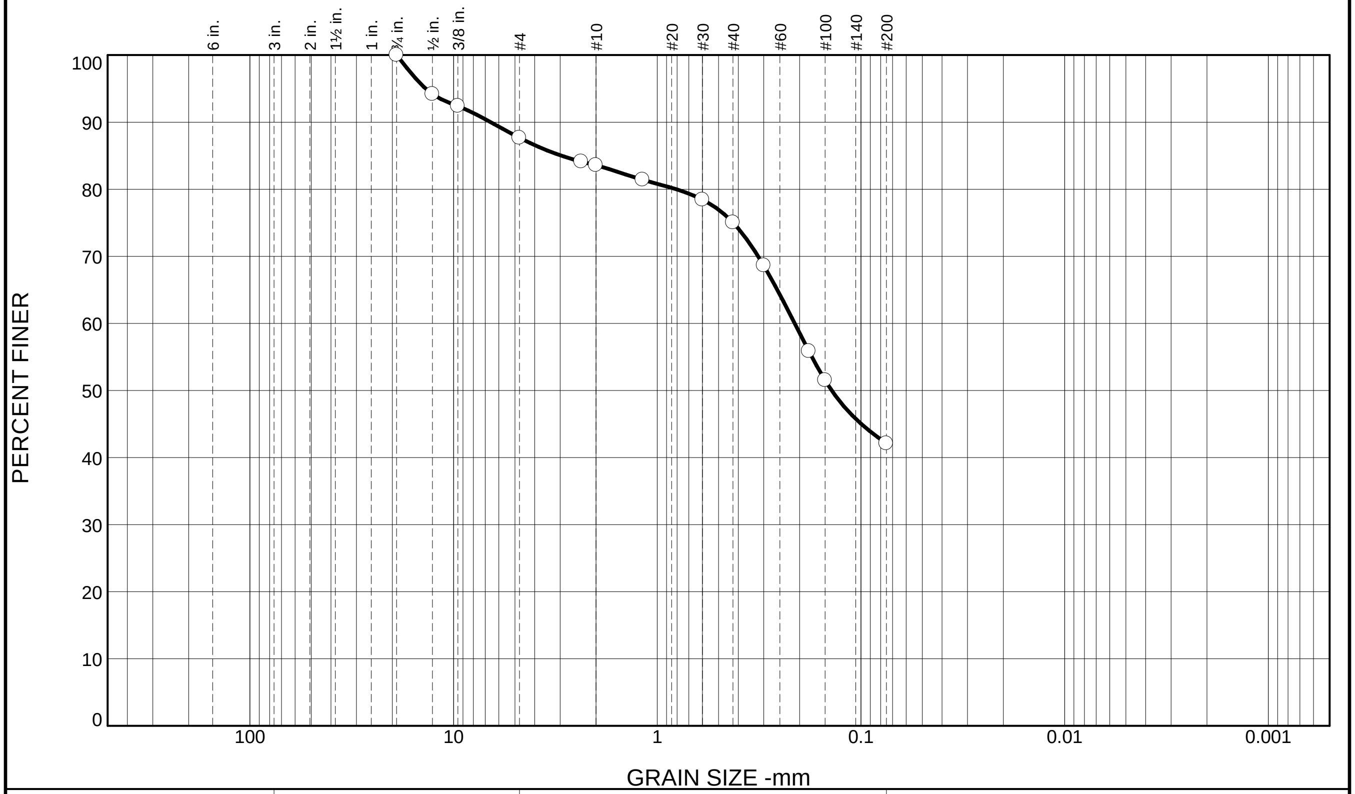
CGC,Inc.

Client: Hammes Co. Sports Dev.
Project: JDS Supplemental PMTs

Project No: C15237 **Figure**

Tested By: DRW **Checked By:** DAS

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0.0	0.0	12.3	4.1	8.6	32.9	42.1	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
3/4	100.0		
1/2	94.2		
3/8	92.4		
#4	87.7		
#8	84.1		
#10	83.6		
#16	81.4		
#30	78.4		
#40	75.0		
#50	68.6		
#80	55.8		
#100	51.5		
#200	42.1		

* (no specification provided)

Material Description

Brown Silty Fine to Medium Sand, Some Gravel

Atterberg Limits

PL= LL= PI=

Coefficients

D_{90}= 6.5746 D_{85}= 2.9557 D_{60}= 0.2115
D_{50}= 0.1393 D_{30}= D_{15}=
D_{10}= C_u= C_c=

Classification

USCS= SM AASHTO=

Remarks

Natural Moisture = 9.3%

Sample Number: B-2A/4

Date: 10/7/15

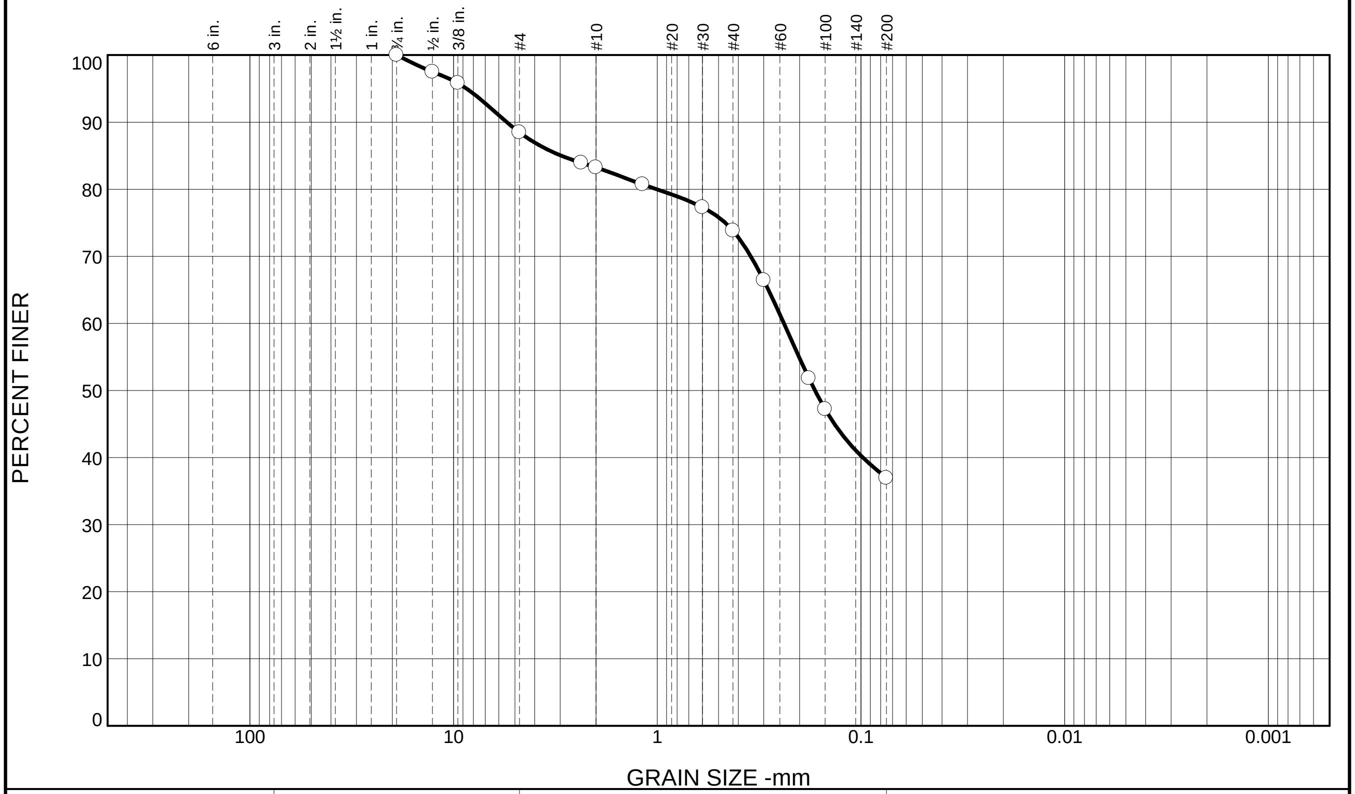
CGC, Inc.

Client: Hammes Co. Sports Dev.
Project: JDS Supplemental PMTs

Project No: C15237 **Figure**

Tested By: DRW **Checked By:** DAS

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0.0	0.0	11.5	5.3	9.4	36.9	36.9	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
3/4	100.0		
1/2	97.5		
3/8	95.8		
#4	88.5		
#8	83.9		
#10	83.2		
#16	80.7		
#30	77.3		
#40	73.8		
#50	66.4		
#80	51.8		
#100	47.2		
#200	36.9		

* (no specification provided)

Material Description

Brown Silty Fine to Coarse Sand, Little Gravel

Atterberg Limits

PL= LL= PI=

Coefficients

D_{90}= 5.4835 D_{85}= 2.9586 D_{60}= 0.2388
D_{50}= 0.1682 D_{30}= D_{15}=
D_{10}= C_u= C_c=

Classification

USCS= SM AASHTO=

Remarks

Natural Moisture = 6.9%

Sample Number: B-4A/2

Date: 10/7/15

CGC,Inc.

Client: Hammes Co. Sports Dev.
Project: JDS Supplemental PMTs

Project No: C15237 **Figure**

Tested By: DRW **Checked By:** DAS

CGC, Inc.

LOG OF TEST BORING

General Notes

DESCRIPTIVE SOIL CLASSIFICATION

Grain Size Terminology

Soil Fraction		Particle Size	U.S. Standard Sieve Size
Boulders		Larger than 12”	Larger than 12”
Cobbles		3” to 12”	3” to 12”
Gravel:	Coarse	¾” to 3”	¾” to 3”
	Fine	4.76 mm to ¾”	#4 to ¾”
Sand:	Coarse	2.00 mm to 4.76 mm	#10 to #4
	Medium	0.42 to mm to 2.00 mm	#40 to #10
	Fine	0.074 mm to 0.42 mm	#200 to #40
Silt		0.005 mm to 0.074 mm	Smaller than #200
Clay		Smaller than 0.005 mm	Smaller than #200

Plasticity characteristics differentiate between silt and clay.

General Terminology

Physical Characteristics
  Color, moisture, grain shape, fineness, etc.
Major Constituents
  Clay, silt, sand, gravel
Structure
  Laminated, varved, fibrous, stratified, cemented, fissured, etc.
Geologic Origin
  Glacial, alluvial, eolian, residual, etc.

Relative Density

Term	“N” Value
Very Loose	0 - 4
Loose	4 - 10
Medium Dense	10 - 30
Dense	30 - 50
Very Dense	Over 50

Relative Proportions Of Cohesionless Soils

Proportional Term	Defining Range by Percentage of Weight
Trace	0% - 5%
Little	5% - 12%
Some	12% - 35%
And	35% - 50%

Consistency

Term	q_u-tons/sq. ft
Very Soft	0.0 to 0.25
Soft	0.25 to 0.50
Medium	0.50 to 1.0
Stiff	1.0 to 2.0
Very Stiff	2.0 to 4.0
Hard	Over 4.0

Organic Content by Combustion Method

Soil Description	Loss on Ignition
Non Organic	Less than 4%
Organic Silt/Clay	4 – 12%
Sedimentary Peat	12% - 50%
Fibrous and Woody Peat	More than 50%

Plasticity

Term	Plastic Index
None to Slight	0 - 4
Slight	5 - 7
Medium	8 - 22
High to Very High	Over 22

The penetration resistance, N, is the summation of the number of blows required to effect two successive 6” penetrations of the 2” split-barrel sampler. The sampler is driven with a 140 lb. weight falling 30” and is seated to a depth of 6” before commencing the standard penetration test.

SYMBOLS

Drilling and Sampling

CS – Continuous Sampling
RC – Rock Coring: Size AW, BW, NW, 2”W
RQD – Rock Quality Designation
RB – Rock Bit/Roller Bit
FT – Fish Tail
DC – Drove Casing
C – Casing: Size 2 ½”, NW, 4”, HW
CW – Clear Water
DM – Drilling Mud
HSA – Hollow Stem Auger
FA – Flight Auger
HA – Hand Auger
COA – Clean-Out Auger
SS - 2” Dia. Split-Barrel Sample
2ST – 2” Dia. Thin-Walled Tube Sample
3ST – 3” Dia. Thin-Walled Tube Sample
PT – 3” Dia. Piston Tube Sample
AS – Auger Sample
WS – Wash Sample
PTS – Peat Sample
PS – Pitcher Sample
NR – No Recovery
S – Sounding
PMT – Borehole Pressuremeter Test
VS – Vane Shear Test
WPT – Water Pressure Test

Laboratory Tests

q_a – Penetrometer Reading, tons/sq ft
q_a – Unconfined Strength, tons/sq ft
W – Moisture Content, %
LL – Liquid Limit, %
PL – Plastic Limit, %
SL – Shrinkage Limit, %
LI – Loss on Ignition
D – Dry Unit Weight, lbs/cu ft
pH – Measure of Soil Alkalinity or Acidity
FS – Free Swell, %

Water Level Measurement

▽- Water Level at Time Shown
NW – No Water Encountered
WD – While Drilling
BCR – Before Casing Removal
ACR – After Casing Removal
CW – Cave and Wet
CM – Caved and Moist

Note: Water level measurements shown on the boring logs represent conditions at the time indicated and may not reflect static levels, especially in cohesive soils.

CGC, Inc.

Madison - Milwaukee

Unified Soil Classification System

UNIFIED SOIL CLASSIFICATION AND SYMBOL CHART

COARSE-GRAINED SOILS

(more than 50% of material is larger than No. 200 sieve size)

	Clean Gravels (Less than 5% fines)		
GRAVELS More than 50% of coarse fraction larger than No. 4 sieve size	▨	GW	Well-graded gravels, gravel-sand mixtures, little or no fines
	▨	GP	Poorly-graded gravels, gravel-sand mixtures, little or no fines
	Gravels with fines (More than 12% fines)		
	▨	GM	Silty gravels, gravel-sand-silt mixtures
	▨	GC	Clayey gravels, gravel-sand-clay mixtures
	Clean Sands (Less than 5% fines)		
SANDS 50% or more of coarse fraction smaller than No. 4 sieve size	▨	SW	Well-graded sands, gravelly sands, little or no fines
	▨	SP	Poorly graded sands, gravelly sands, little or no fines
	Sands with fines (More than 12% fines)		
	▨	SM	Silty sands, sand-silt mixtures
	▨	SC	Clayey sands, sand-clay mixtures

FINE-GRAINED SOILS

(50% or more of material is smaller than No. 200 sieve size.)

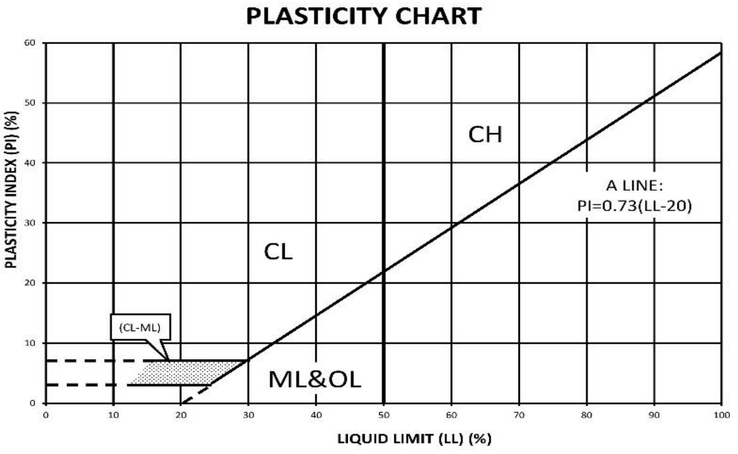
SILTS AND CLAYS Liquid limit less than 50%	▨	ML	Inorganic silts and very fine sands, rock flour, silty or clayey fine sands or clayey silts with slight plasticity
	▨	CL	Inorganic clays of low to medium plasticity, gravelly clays, sandy clays, silty clays, lean clays
	▨	OL	Organic silts and organic silty clays of low plasticity
SILTS AND CLAYS Liquid limit 50% or greater	▨	MH	Inorganic silts, micaceous or diatomaceous fine sandy or silty soils, elastic silts
	▨	CH	Inorganic clays of high plasticity, fat clays
	▨	OH	Organic clays of medium to high plasticity, organic silts
HIGHLY ORGANIC SOILS	▨	PT	Peat and other highly organic soils

LABORATORY CLASSIFICATION CRITERIA

GW	$C_u = \frac{D_{60}}{D_{10}}$ greater than 4; $C_C = \frac{D_{30}}{D_{10} \times D_{60}}$ between 1 and 3	
GP	Not meeting all gradation requirements for GW	
GM	Atterberg limts below "A" line or P.I. less than 4	Above "A" line with P.I. between 4 and 7 are borderline cases requiring use of dual symbols
GC	Atterberg limts above "A" line or P.I. greater than 7	
SW	$C_u = \frac{D_{60}}{D_{10}}$ greater than 4; $C_C = \frac{D_{30}}{D_{10} \times D_{60}}$ between 1 and 3	
SP	Not meeting all gradation requirements for GW	
SM	Atterberg limits below "A" line or P.I. less than 4	Limits plotting in shaded zone with P.I. between 4 and 7 are borderline cases requiring use of dual symbols
SC	Atterberg limits above "A" line with P.I. greater than 7	

Determine percentages of sand and gravel from grain-size curve. Depending on percentage of fines (fraction smaller than No. 200 sieve size), coarse-grained soils are classified as follows:

Less than 5 percent ………………………………………….. GW, GP, SW, SP
More than 12 percent ……………………….………………… GM, GC, SM, SC
5 to 12 percent ……………….…..... Borderline cases requiring dual symbols



APPENDIX C

DOCUMENT QUALIFICATIONS

APPENDIX C
DOCUMENT QUALIFICATIONS

I. GENERAL RECOMMENDATIONS/LIMITATIONS

CGC, Inc. should be provided the opportunity for a general review of the final design and specifications to confirm that earthwork and foundation requirements have been properly interpreted in the design and specifications. CGC should be retained to provide soil engineering services during excavation and subgrade preparation. This will allow us to observe that construction proceeds in compliance with the design concepts, specifications and recommendations, and also will allow design changes to be made in the event that subsurface conditions differ from those anticipated prior to the start of construction. CGC does not assume responsibility for compliance with the recommendations in this report unless we are retained to provide construction testing and observation services.

This report has been prepared in accordance with generally accepted soil and foundation engineering practices and no other warranties are expressed or implied. The opinions and recommendations submitted in this report are based on interpretation of the subsurface information revealed by the test borings indicated on the location plan. The report does not reflect potential variations in subsurface conditions between or beyond these borings. Therefore, variations in soil conditions can be expected between the boring locations and fluctuations of groundwater levels may occur with time. The nature and extent of the variations may not become evident until construction.

II. IMPORTANT INFORMATION
ABOUT YOUR
GEOTECHNICAL ENGINEERING REPORT

Subsurface problems are a principal cause of construction delays, cost overruns, claims, and disputes. While you cannot eliminate all such risks, you can manage them. The following information is provided to help.

Geotechnical engineers structure their services to meet the specific needs of their clients. A geotechnical engineering study conducted for a civil engineer may not fulfill the needs of a construction contractor or even another civil engineer. Because each geotechnical engineering study is unique, each geotechnical engineering report is unique, prepared *solely* for the client. *No one except you* should rely on your geotechnical engineering report without first conferring with the geotechnical engineer who prepared it. *And no one - not even you* - should apply the report for any purpose or project except the one originally contemplated.

READ THE FULL REPORT

Serious problems have occurred because those relying on a geotechnical engineering report did not read it all. Do not rely on an executive summary. Do not read selected elements only.

A GEOTECHNICAL ENGINEERING REPORT IS BASED ON A UNIQUE SET OF PROJECT-SPECIFIC FACTORS

Geotechnical engineers consider a number of unique, project-specific factors when establishing the scope of a study. Typical factors include: the client's goals, objectives, and risk management preferences; the general nature of the structure involved, its size, and configuration; the location of the structure on the site; and other planned or existing site improvements, such as access roads, parking lots, and underground utilities. Unless the geotechnical engineer who conducted the study specifically indicates otherwise, *do not rely on a geotechnical engineering report* that was:

- not prepared for you,
- not prepared for your project,
- not prepared for the specific site explored, or
- completed before important project changes were made.

Typical changes that can erode the reliability of an existing geotechnical report include those that affect:

- the function of the proposed structure, as when it's changed from a parking garage to an office building, or from a light industrial plant to a refrigerated warehouse,
- elevation, configuration, location, orientation, or weight of the proposed structure,
- composition of the design team, or project ownership.

As a general rule, *always* inform your geotechnical engineer of project changes - even minor ones - and request an assessment of their impact. *CGC cannot accept responsibility or liability for problems that occur because our reports do not consider developments of which we were not informed.*

SUBSURFACE CONDITIONS CAN CHANGE

A geotechnical engineering report is based on conditions that existed at the time the geotechnical engineer performed the study. *Do not rely on a geotechnical engineering report* whose adequacy may have been affected by: the passage of time; by man-made events, such as construction on or adjacent to the site; or by natural events, such as floods, earthquakes, or groundwater fluctuations. *Always* contact the geotechnical engineer before applying the report to determine if it is still reliable. A minor amount of additional testing or analysis could prevent major problems.

MOST GEOTECHNICAL FINDINGS ARE PROFESSIONAL OPINION

Site exploration identifies subsurface conditions only at those points where subsurface tests are conducted or samples are taken. Geotechnical engineers review field and laboratory data and then apply their professional judgement to render an opinion about subsurface conditions throughout the site. Actual subsurface conditions may differ - sometimes significantly - from those indicated in your report. Retaining the geotechnical engineer who developed your report to provide construction observation is the most effective method of managing the risks associated with unanticipated conditions.

A REPORT'S RECOMMENDATIONS ARE NOT FINAL

Do not over-rely on the confirmation-dependent recommendations included in your report. *Those confirmation-dependent recommendations are not final*, because geotechnical engineers develop them principally from judgement and opinion. Geotechnical engineers can finalize their recommendations *only* by observing actual subsurface conditions revealed during construction. *CGC cannot assume responsibility or liability for the report's confirmation-dependent recommendations if we do not perform the geotechnical-construction observation required to confirm the recommendations' applicability.*

A GEOTECHNICAL ENGINEERING REPORT IS SUBJECT TO MISINTERPRETATION

Other design team members' misinterpretation of geotechnical engineering reports has resulted in costly problems. Confront that risk by having your geotechnical engineer confer with appropriate members of the design team after submitting the report. Also retain your geotechnical engineer to review pertinent elements of the design team's plans and specifications. Constructors can also misinterpret a geotechnical engineering report. Confront that risk by having CGC participate in prebid and preconstruction conferences, and by providing geotechnical construction observation.

DO NOT REDRAW THE ENGINEER'S LOGS

Geotechnical engineers prepare final boring and testing logs based upon their interpretation of field logs and laboratory data. To prevent errors or omissions, the logs included in a geotechnical engineering report should *never* be redrawn for inclusion in architectural or other design drawings. Only photographic or electronic reproduction is acceptable, *but recognize that separating logs from the report can elevate risk.*

GIVE CONSTRUCTORS A COMPLETE REPORT AND GUIDANCE

Some owners and design professionals mistakenly believe they can make constructors liable for unanticipated subsurface conditions by limiting what they provide for bid preparation. To help prevent costly problems, give constructors the complete geotechnical engineering report, *but* preface it with a clearly written letter of transmittal. In that letter, advise constructors that the report was not prepared for purposes of bid development and that the report's accuracy is limited; encourage them to confer with the geotechnical engineer who prepared the report (a modest fee may be required) and/or to conduct additional study to obtain the specific types of information they need or prefer. A prebid conference can also be valuable. *Be sure constructors have sufficient time* to perform additional study. Only then might you be in a position to give constructors the best information available to you, while requiring them to at least share some of the financial responsibilities stemming from unanticipated conditions.

READ RESPONSIBILITY PROVISIONS CLOSELY

Some clients, design professionals, and constructors do not recognize that geotechnical engineering is far less exact than other engineering disciplines. This lack of understanding has created unrealistic expectations that have led to disappointments, claims, and disputes. To help reduce the risk of such outcomes, geotechnical engineers commonly include a variety of explanatory provisions in their reports. Sometimes labeled "limitations," many of these provisions indicate where geotechnical engineer's responsibilities begin and end, to help others recognize their own responsibilities and risks. *Read these provisions closely*. Ask questions. Your geotechnical engineer should respond fully and frankly.

ENVIRONMENTAL CONCERNS ARE NOT COVERED

The equipment, techniques, and personnel used to perform an *environmental* study differ significantly from those used to perform a *geotechnical* study. For that reason, a geotechnical engineering report does not usually relate any environmental findings, conclusions, or recommendations; e.g., about the likelihood of encountering underground storage tanks or regulated contaminants. *Unanticipated environmental problems have led to numerous project failures.* If you have not yet obtained your own environmental information, ask your geotechnical consultant for risk management guidance. *Do not rely on an environmental report prepared for someone else.*

OBTAIN PROFESSIONAL ASSISTANCE TO DEAL WITH MOLD

Diverse strategies can be applied during building design, construction, operation, and maintenance to prevent significant amounts of mold from growing on indoor surfaces. To be effective, all such strategies should be devised for the *express purpose* of mold prevention, integrated into a comprehensive plan, and executed with diligent oversight by a professional mold prevention consultant. Because just a small amount of water or moisture can lead to the development of severe mold infestations, many mold prevention strategies focus on keeping building surfaces dry. While groundwater, water infiltration, and similar issues may have been addressed as part of the geotechnical engineering study whose findings are conveyed in this report, the geotechnical engineer in charge of this project is not a mold prevention consultant; ***none of the services performed in connection with the geotechnical engineer's study were designed or conducted for the purpose of mold prevention. Proper implementation of the recommendations conveyed in this report will not of itself be sufficient to prevent mold from growing in or on the structure involved.***

RELY ON YOUR GEOTECHNICAL ENGINEER FOR ADDITIONAL ASSISTANCE

Membership in the Geotechnical Business Council (GBC) of Geoprofessional Business Association exposes geotechnical engineers to a wide array of risk confrontation techniques that can be of genuine benefit for everyone involved with a construction project. Confer with CGC, a member of GBC, for more information.

Modified and reprinted with permission from:

Geotechnical Business Council
of the Geoprofessional Business Association
8811 Colesville Road, Suite G 106
Silver Spring, MD 20910

APPENDIX D

RECOMMENDED COMPACTED FILL SPECIFICATIONS

APPENDIX D

CGC, INC.

RECOMMENDED COMPACTED FILL SPECIFICATIONS

General Fill Materials

Proposed fill shall contain no vegetation, roots, topsoil, peat, ash, wood or any other non-soil material which by decomposition might cause settlement. Also, fill shall never be placed while frozen or on frozen surfaces. Rock, stone or broken concrete greater than 6 in. in the largest dimension shall not be placed within 10 ft of the building area. Fill used greater than 10 ft beyond the building limits shall not contain rock, boulders or concrete pieces greater than a 2 sq ft area and shall not be placed within the final 2 ft of finish subgrade or in designated utility construction areas. Fill containing rock, boulders or concrete pieces should include sufficient finer material to fill voids among the larger fragments.

Special Fill Materials

In certain cases, special fill materials may be required for specific purposes, such as stabilizing subgrades, backfilling undercut excavations or filling behind retaining walls. For reference, WisDOT gradation specifications for various types of granular fill are attached in Table 1.

Placement Method

The approved fill shall be placed, spread and leveled in layers generally not exceeding 10 in. in thickness before compaction. The fill shall be placed at moisture content capable of achieving the desired compaction level. For clay soils or granular soils containing an appreciable amount of cohesive fines, moisture conditioning will likely be required.

It is the Contractor's responsibility to provide all necessary compaction equipment and other grading equipment that may be required to attain the specified compaction. Hand-guided vibratory or tamping compactors will be required whenever fill is placed adjacent to walls, footings, columns or in confined areas.

Compaction Specifications

Maximum dry density and optimum moisture content of the fill soil shall be determined in accordance with modified Proctor methods (ASTM D1557). The recommended field compaction as a percentage of the maximum dry density is shown in Table 2. Note that these compaction guidelines would generally not apply to coarse gravel/stone fill. Instead, a method specification would apply (e.g., compact in thin lifts with a vibratory compactor until no further consolidation is evident).

Testing Procedures

Representative samples of proposed fill shall be submitted to CGC, Inc. for optimum moisture-maximum density determination (ASTM D1557) prior to the start of fill placement. The sample size should be approximately 50 lb.

CGC, Inc. shall be retained to perform field density tests to determine the level of compaction being achieved in the fill. The tests shall generally be conducted on each lift at the beginning of fill placement and at a frequency mutually agreed upon by the project team for the remainder of the project.

Table 1
Gradation of Special Fill Materials

Material	WisDOT Section 311	WisDOT Section 312	WisDOT Section 305			WisDOT Section 209		WisDOT Section 210
	Breaker Run	Select Crushed Material	3-in. Dense Graded Base	1 1/4-in. Dense Graded Base	3/4-in. Dense Graded Base	Grade 1 Granular Backfill	Grade 2 Granular Backfill	Structure Backfill
Sieve Size	Percent Passing by Weight							
6 in.	100							
5 in.		90-100						
3 in.			90-100					100
1 1/2 in.		20-50	60-85					
1 1/4 in.				95-100				
1 in.					100			
3/4 in.			40-65	70-93	95-100			
3/8 in.				42-80	50-90			
No. 4			15-40	25-63	35-70	100 (2)	100 (2)	25-100
No. 10		0-10	10-30	16-48	15-55			
No. 40			5-20	8-28	10-35	75 (2)		
No. 100						15 (2)	30 (2)	
No. 200			2-12	2-12	5-15	8 (2)	15 (2)	15 (2)

Notes:

1. Reference: Wisconsin Department of Transportation *Standard Specifications for Highway and Structure Construction.*
2. Percentage applies to the material passing the No. 4 sieve, not the entire sample.
3. Per WisDOT specifications, both breaker run and select crushed material can include concrete that is 'substantially free of steel, building materials and other deleterious material'.

Table 2
Compaction Guidelines

	Percent Compaction (1)	
Area	Clay/Silt	Sand/Gravel
Within 10 ft of building lines		
Footing bearing soils	93 - 95	95
Under floors, steps and walks		
- Lightly loaded floor slab	90	90
- Heavily loaded floor slab and thicker fill zones	92	95
Beyond 10 ft of building lines		
Under walks and pavements		
- Less than 2 ft below subgrade	92	95
- Greater than 2 ft below subgrade	90	90
Landscaping	85	90

Notes:

1. Based on Modified Proctor Dry Density (ASTM D 1557)

APPENDIX E

PERIMETER DRAIN DETAILS

General Notes

1. This system's primary function is to intercept infiltrating surface water. These alternates are not appropriate for use in situations of high groundwater (i.e., cases where the water table approaches floor slab elevation).

2. Grade surface cap to slope away from structure.

3. Exterior surface of walls below grade should be damp-proofed.

4. A plastic vapor barrier should be installed below the slab.

5. Recommended types of drain pipes:

Specification	Description
ASTM D2729	Polyvinyl Chloride (PVC) Drain Pipe
ASTM F405	Corrugated Polyethylene Drain Pipe
ASTM D2852	Styrene-Rubber Plastic Drain Pipe
AASHTO M1366	Corrugated Metal Underdrain Pipe

6. Minimum slope of drain pipes should be 2 in. per 100 lin ft.

7. Place drain pipe below basement floor level and orient the perforations toward the bottom.

8. Clean-outs should be provided to service the pipe.

9. Collected field water should be discharged to a sump, storm sewer or drainage field.

10. The geotextile for Alternative Nos. 2 and 3 may be eliminated if filter requirements are satisfied between the wall and pipe backfill, as well as between backfill materials and natural soils.

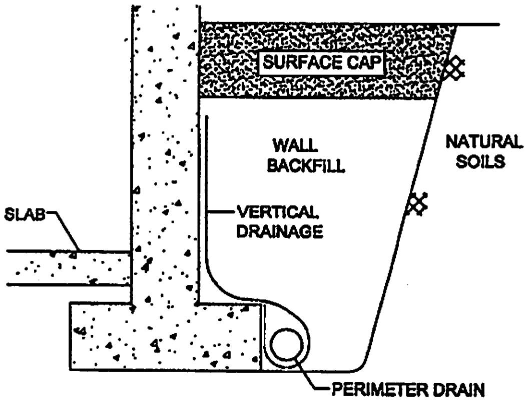
11. Pipe backfill materials should satisfy filter requirements for the slot width or hole diameter of the perforated pipe.

12. Care should be taken during backfilling not to damage the integrity of the system. For compaction requirements, refer to geotechnical report.

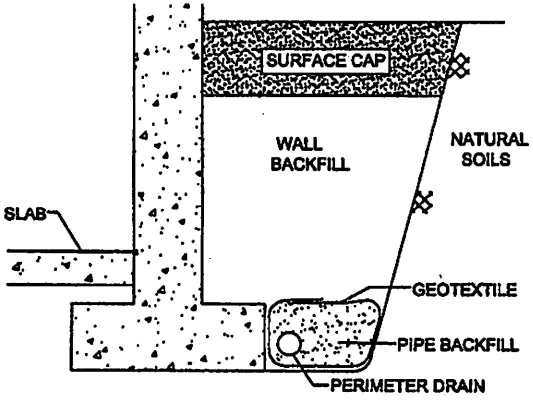
13. Pipe, geotextile, and geocomposite should be installed according to manufacturer specifications.

CGC, Inc.

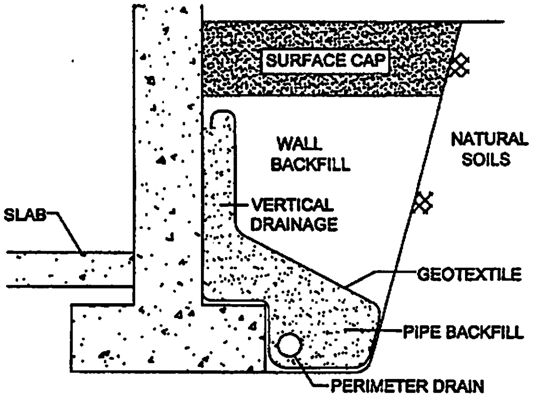
Typical Perimeter Drain Detail
General Notes



ALTERNATE NO. 1



ALTERNATE NO. 2



ALTERNATE NO. 3

DRAINAGE SYSTEM COMPONENTS

Component	Alternate No. 1	Alternate No. 2	Alternate No. 3
Surface Cap	1 to 2 ft of clayey soils. Minimum 1 ft thick if overlain by pavement	Refer to Alternate No. 1	Refer to Alternate No. 1
Vertical Drainage	3-dimensional drainage geocomposite hydraulically connected to perimeter drain.	Relatively free-draining granular soils with P200 (% fines) ≤12%.	Minimum 6-in. wide zone of free-draining granular soils with P200 ≤ 5% hydraulically connected to perimeter drain. Provide geotextile as required (see note 10.
Perimeter Drain	Perforated pipe encapsulated in geocomposite.	Perforated pipe surrounded by free-draining granular pipe backfill with P200 ≤ 5%. Provide geotextile as required (See Note 10).	Refer to Alternate No. 2
Wall Backfill	Excavation spoils or imported materials (granular soils preferred).	Relatively free-draining granular soils with P200 ≤ 12%.	Refer to Alternate No. 1

CGC, Inc.

Typical Perimeter Drain Detail

APPENDIX F

WKG^2 PRESSUREMETER TEST REPORTS (2010 & 2015)

September 23, 2010

Mr. David Staab, P.E.
CGC, Inc.
3011 Perry Street
Madison, Wisconsin 53713

Re: City of Madison Parking Ramp – Madison, Wisconsin
WKG^2 Project No. 10020

Dear Mr. Staab:

At your request, Wagner Komurka Geotechnical Group, Inc. (“WKG^2”) arranged for in-situ pressuremeter tests to be performed in two borings drilled for the City of Madison Parking Ramp project in Madison, Wisconsin. The pressuremeter tests were performed to evaluate soil strength and compressibility to optimize allowable foundation bearing stress. Pressuremeter test results, and the results of our analysis using the pressuremeter test data, are presented in the attached report.

If you have any questions regarding this report, please feel free to contact us. We appreciate this opportunity to provide geotechnical engineering services to CGC, Inc.

Sincerely,

WAGNER KOMURKA
GEOTECHNICAL GROUP, INC.

Van E. Komurka

Van E. Komurka, P.E.
President

Janine L. Grauvogl-Graham

Janine L. Grauvogl-Graham, P.E.
Geotechnical Engineer

WAGNER KOMURKA GEOTECHNICAL GROUP, INC. • W67 N222 Evergreen Boulevard • Suite 100 • Cedarburg, WI 53012
(262) 376-2001 • FAX: (262) 376-2002 • E-MAIL: geotek@wkg2.com

PRESSUREMETER ENGINEERING REPORT

CITY OF MADISON PARKING RAMP

Madison, Wisconsin

September 23, 2010

Prepared for:

CGC, Inc.

by:

Wagner Komurka Geotechnical Group, Inc.

CITY OF MADISON PARKING RAMP
MADISON, WISCONSIN
PRESSUREMETER ENGINEERING REPORT

PROJECT DESCRIPTION

The project involves construction of a proposed parking ramp located between East Doty Street and East Wilson Street, Northeast of Martin Luther King Jr. Boulevard in Madison, Wisconsin. The building will include 5 levels below grade, and up to 12 levels above grade. The foundation is expected to bear at Elevation 15 or 20[1]. Existing site grades range from approximate Elevations 49 to 65. Maximum column loads are estimated at 2,500 kips.

PRESSUREMETER TESTING PROGRAM

The scope of Wagner Komurka Geotechnical Group, Inc.'s ("WKG^2's") services is limited to foundation-related design (bearing capacity assessment and settlement estimation). This was accomplished using data from soil borings drilled prior to the pressuremeter testing program for CGC, Inc. ("CGC") of Madison, Wisconsin, and pressuremeter tests arranged by, and performed for, WKG^2. The pressuremeter tests were performed by AECOM of Vernon Hills, Illinois. The pressuremeter tests were performed in two blind-drilled borings (Boring Nos. B-4P, and B-5P) drilled by Badger State Drilling Company, Inc. ("Badger") of Stoughton, Wisconsin. WKG^2 selected the borings for pressuremeter testing, the number of tests in each boring, and the test depths (elevations). Pressuremeter test zones were prepared using a split-barrel sampler and specially sized roller bits.

In a pressuremeter test, a cylindrical probe is inserted to the (then current) borehole bottom, and hydraulically expanded radially against the borehole sides. Probe volume versus pressure is recorded. A more-detailed description of pressuremeter testing is provided on the sheet titled "Pressuremeter Procedures" in the Appendix. Plots of probe volume versus pressure for each individual test are also included in the Appendix.

SOIL CONDITIONS

A review of CGC's boring logs indicates that the site soils are predominately granular. The specific conditions encountered below anticipated spread-footing bearing elevations are summarized in the following paragraphs. More-detailed descriptions of the subsurface conditions are presented on CGC's logs in their report.

---

[1] Unless indicated otherwise, elevations are positive, have units of feet, and with respect to Madison City Datum.

Native Deposits

Below anticipated spread-footing bearing elevations, the borings encountered native predominately granular deposits typically consisting of fine to medium sand, with some silt and little to some gravel. The granular soils' relative densities ranged from dense to very dense (with density generally increasing with depth).

Groundwater Conditions

Water level observations in the boreholes while drilling indicated groundwater at depths ranging from 32.0 to 47.0 feet below existing ground surface. Long-term groundwater level observations, from Monitoring Well No. 1 located 4 feet north of Boring B-5, indicated groundwater at a depth of approximately 39 feet below the ground surface.

Fluctuations in the water table should be expected with variations in precipitation, evapotranspiration, surface runoff, etc., and the water elevations in nearby surface waters. Development of perched groundwater may occur above the primary groundwater table, especially following precipitation events.

PRESSUREMETER TEST RESULTS

Pressuremeter test results are presented in Table 1.

The at-rest pressure, P_o , represents the pressure at which the probe has expanded into firm contact with the borehole sides, and the pressure at which the plot of probe volume versus pressure becomes linear. The creep pressure, P_f , represents the pressure at which the plot ceases to be linear (i.e., the pressure at which deformations increase for a given incremental pressure increase). The limit pressure, P_l , is the pressure at which complete soil failure has occurred (i.e., the plot is vertical). The deformation modulus, E_d, is the slope of the initial linear portion of the plot. The rebound modulus, E^+, is the slope of the linear reload portion of the plot. The ratio E_d/E^+ (commonly referred to as the α parameter) is used, along with the deformation modulus, to estimate settlement.

TABLE 1

Pressuremeter Test Results Summary

Boring Number	Elevation (ft)	P_o (tsf)	P_f (tsf)	P_l (tsf)	E_d (tsf)	E^+ (tsf)	E_d/E^+
B-4P	17.9 to 20.4	4.0	25.0	50.0[A]	945	2477	0.38
	11.4 to 13.9	4.0	33.0	66.0[A]	1554	2515	0.62
	2.4 to 4.9	4.0	33.0	66.0[A,B]	778	3481	0.22
	-5.6 to -3.1	5.0	33.0	66.0[A]	795	2511	0.32
B-5P	18.4 to 20.9	4.0	[c]	--	1013	2147	0.47
	11.4 to 13.9	4.0	28.0	56.0[A]	1066	2694	0.40
	2.4 to 4.9	5.0	33.0	66.0[A]	1081	2367	0.46
	-5.6 to -3.1	5.0	28.0	56.0[A]	820	2365	0.35
						Average:	0.40

Notes: [A] Limit pressure (P_l) not reached during test. Limit pressure assumed as twice the creep pressure.

[B] No cobbles or gravel detected while preparing the test area.

[C] Numerous cobbles or gravel detected while preparing the test area, pressurized the test area to 27 tsf and did not reach the P_f.

ANALYSES AND RECOMMENDATIONS

Foundation Recommendations

Allowable Bearing Stress

Based strictly on lower-bound pressuremeter test program results, the calculated maximum net allowable bearing stress for foundations bearing on native very dense granular soil at either Elevation 15 or 20 without consideration of other factors (discussed subsequently) would be 43,000 pounds per square foot (“psf”). The recommended maximum net allowable bearing stress is the stress transmitted by the foundation to the soil in excess of the minimum final adjacent overburden stress.

The above-reported calculated design bearing stress is relatively high. It is likely that other factors, such as structural design issues, practical limitations on strength of concrete used in the foundations, etc. may control foundation sizing. Furthermore, there may be some disturbance of the subsoils resulting from excavation, especially since the excavation will extend below the groundwater table. Due to these factors, we recommend that the maximum net allowable bearing stress be limited to 30,000 psf for this project. Further reductions in design foundation bearing stresses could be required depending on methods employed for overall excavation, foundation subgrade preparation, and especially groundwater control. The recommended design bearing stress may be increased (potentially to 36,000 psf) if an additional pressuremeter test is performed after excavation to confirm soil response to removal of the overburden soil.

The recommended maximum net allowable bearing stress is predicated on a minimum foundation embedment of one-quarter the foundation width (e.g., a 16-foot-square foundation would require a minimum embedment of 4 feet). Foundations with shallower embedment must be designed for a lower allowable bearing stress. Conversely, foundations with greater embedment could possibly be designed for a higher allowable bearing stress. The recommended maximum net allowable bearing stress incorporates a minimum safety factor of 3.0 against bearing capacity failure.

For stability considerations, continuous wall and isolated column foundations should have minimum plan dimensions of 18 and 36 inches, respectively. This minimum width requirement may control the size of certain lightly loaded foundations. In this event, the actual soil/foundation contact stress will be less than the recommended maximum net allowable.

Settlement Estimate

Based on lower-bound pressuremeter testing soil deformation moduli, and the maximum net allowable bearing stress of 30,000 psf presented above, estimated foundation settlement for a 2,500-kip column load bearing at elevations of either 15 or 20 is ½ inch. Smaller, or more-lightly loaded, foundations are expected to experience proportionately less settlement. It is our opinion that differential settlement between similarly sized foundations will be on the order of half these foundations' total settlement.

Construction Considerations

We recommend that CGC provide observation and testing at the base of each foundation excavation to confirm that the soils are the same type as encountered by the subsurface exploration and pressuremeter testing program, and meet minimum relative density criteria associated with the maximum net allowable bearing stresses presented above. Granular subgrade soils should exhibit a minimum equivalent Standard Penetration Test ("SPT") "N" value of 63 blows per foot. This testing should be performed using a device such as a dynamic cone penetrometer ("DCP"). DCP testing should extend a minimum of 2 feet below foundation subgrade, unless DCP refusal (greater than 50 blows per 6 inches) is encountered shallower.

It is anticipated that foundation excavations will terminate below the primary groundwater table. To allow use of the allowable bearing stresses presented herein, it is imperative that foundation subgrades be properly dewatered and remain undisturbed by groundwater inflow or construction activities. Excavating below the groundwater table in granular soils likely will result in at minimum running sands from the sidewalls of excavations, and more likely, development of a "quick" condition at the base of the excavation. Dewatering by pumping from the base of the excavation will likely exacerbate soil disturbance. Accordingly, WKG^2 recommends that any required dewatering be accomplished by a specialty contractor able to demonstrate successful dewatering of similar soils on a minimum of five previous projects.

If the bearing subsoils become disturbed for any reason (surface water inflow, worker traffic during reinforcing steel placement, etc.), the disturbed soils should be removed, and the base of the excavation stabilized by the placement of a lean concrete mud mat.

If soils of the anticipated type or exhibiting the minimum required relative density criteria are not found at the base of foundation excavations, it will be necessary to extend excavations deeper, or the affected foundations will have to be re-designed for a lower bearing stress. We recommend that any foundation excavations that must be extended below their design bearing elevation be backfilled to that elevation with cementitious backfill. The cementitious backfill could consist of lean concrete or controlled low-strength material ("CLSM"). The cementitious backfill should have a minimum 28-day compressive strength of 300 pounds per square inch (psi), and the backfill should be at least as wide as the foundation it supports.

All excavations deeper than 4 feet that will be entered by workers should have sloped or braced sidewalls that are consistent with OSHA guidelines for excavation safety.

GENERAL QUALIFICATIONS

The services provided by WKG^2 on this project were performed with the degree of skill and care typically performed by other members of our profession practicing in this locale at this time. No other warranty, expressed or implied, is given.

APPENDIX

- General Report Qualifications
- Pressuremeter Procedures
- Pressuremeter Test Results

General Report Qualifications

This geotechnical engineering report was prepared as part of the evaluation of the specific area covered by the soil borings, specifically for the project described in the report. The description of the project represents our understanding of the project. Should there be any changes in the concept of the project, its location, orientation, or elevation, we request that we be notified so that we may assess any impacts of the changes on our recommendations. The drawings and specifications for the project shall be submitted to WKG^2 for review of conformance with the recommendations contained in the report. Failure to submit the plans and specifications for this review relieves WKG^2 from any liability for failure to comply with our recommendations.

The recommendations presented in this report have been based on subsurface information obtained from soil samples at intervals in the soil borings which were drilled at the locations shown on the soil boring location diagram. The number of borings and the sampling intervals used are considered to be consistent with standards of the industry.

It should be recognized that variations in subsurface conditions can occur both between soil samples in a given boring, and between soil borings. Further, groundwater conditions should be expected to vary with time. The extent of the variations in subsurface conditions may not become apparent until construction begins. If variations in subsurface conditions become apparent, we request that we be notified so that we can observe the site conditions and evaluate how our recommendations may be affected.

We strongly recommend that all construction work related to geotechnical issues be monitored by an experienced geotechnical engineer or technician to determine if the subsurface conditions are as anticipated, and if the intent of our recommendations is met. We are available to provide the monitoring and testing services required during construction on this project.

Due to possible variation in subsurface conditions, we recommend that the Standard General Conditions of the construction contract prepared by the Engineers Joint Contract Documents Committee (1910-8-(Latest Edition)) be included in the contract with the general contractor and any subcontractors who will be involved in geotechnical issues on this project. We also advise incorporating a dispute resolution clause in the contract, based on non-binding mediation, to resolve any disputes among the parties involved with geotechnical issues on the project.

The services provided by WKG^2 on this project were performed with the degree of skill and care typically performed by other members of our profession practicing in this locale at this time. No other warranty, expressed or implied, is given.

Introduction

The pressuremeter is a soil testing device which measures stress-strain characteristics of soils in-situ. It is a portable piece of equipment consisting of three main components:

1. a cylindrical expanding probe which is inserted into a bore hole.
2. a pressure source for expanding the probe, and
3. a metering system.

A schematic drawing showing these components is shown in Figure 1.

Pressuremeter Test

The test consists of inserting the probe into the bore hole and expanding the probe against the side of the hole at measured intervals of time until failure of the soil is reached.

The pressurermeter can be used to test nearly all soil types, from loose sand or silt to hard cohesive or dense granular soils and soft rock. Tests can be performed in a drilled bore hole or hand augered hole at depths normally achieved by these methods of drilling. Tests can be performed above or below the water table. Special procedures or techniques including the use of a bore hole shaver have been developed to prepare the bore hole in squeezing or caving soils so that reliable test parameters are measured.

Using correlations with routine or special laboratory tests, a pressuremeter is a very useful geotechnical tool.

General Uses

The following is a summary of some of the applications of the pressuremeter investigation.

1. Determination of bearing capacity of pile or cassion type foundations,
2. Determination of bearing capacity for shallow foundations,
3. Estimates of foundation settlement.
4. Determination of soil shear strength.
5. Determination of horizontal subgrade modulus to predict horizontal movement under lateral loads for piles, sheet pile walls, cast-in-place concrete walls, and drilled piers.
6. Determination of the modulus of vertical subgrade reaction, and
7. Determining the improvement in soil properties following site densification.

Apparatus

The probe measures 2.5 inches in diameter, is 2 ft. 2 inches long, fits inside of a BX size casing, with the length of the center expanding cell of the probe measuring 7 inches. A liquid (water in summer and glycerin in winter) is used to expand the center cell of the probe and gas pressure, usually carbon dioxide, is used to expand the two end cells of the probe. When the probe is inserted into the soil and the cells are expanded, the top and bottom portions of the probe tend to seal off the bore hole while the volume change in the center portion is measured. By this method, a nearly plane stress, plane strain condition is set up on the soil. Volume changes in the center portion of the probe are measured versus the pressure increment. Six to fourteen load increments are used per test, each increment being applied to the soil for a 1 minute period. Readings are to be at 30 seconds and 60 seconds after head increment.

Interpretation of Test Results

The results of the pressuremeter tests are generally plotted as pressure versus volume change at 60 seconds for each pressure increment. A typical curve is shown in Figure 2. The interpretation of the test results is generally in conformance with Menard's Theory. The soil behavior generally follows two zones, pseudo-elastic and plastic. The elastic zone, in which strains are completely recoverable, is generally not noticed due to the bore hole disturbance. The lower limit of this elastic zone is defined as P_O. As pressures above P_O, the solid behaves as a pseudo-elastic material which is indicated as a straight line on the pressure verses probe volume curve. The strains occurring within this zone are not completely recoverable.

The upper limit of the pseudo-elastic zone is defined as P_F. At pressures greater than the value of P_F, creep deformation of the soil particles occurs as the pressure increases and eventually causes failure of the soil. The pressure at which the failure occurs is called the limit pressure, P_L and is related to the ultimate bearing capacity of the soil.

The pressuremeter modulus is calculated for the pseudo-elastic zone portion of the test. From classical soil mechanics principles in which soil anisotropy is often assumed, the vertical modulus may be significantly different from the horizontal modulus and one might expect erratic predictions of vertical settlement of footings. However, recent theoretical as well as full scale experimental studies have shown that in many situations this test still permits a much better prediction of foundation settlements predictions based on pressuremeter test results are presently the most reliable for granular materials and preconsolidated glacial tills.

General Equations

The analysis of the pressuremeter test is based upon the principles of theoretical soil mechanics. The parameters obtained from these tests have been correlated to parameters obtained from laboratory tests. The general equation for bearing capacity and settlement have been modified by and confirmed with numerous field tests including full scale load tests.

The bearing capacity of a foundation is derived from the following general equation:

$$q = P_V + k(P_L - P_O)$$

where q = Ultimate bearing capacity

P_O = Lateral pressure at rest of the soil at the elevation of the foundation element

P_L = Limit pressure of the soil

k = A coefficient depending upon soil type, geometric shape of the foundation, and depth of embedment.

P_V = Overburden pressure at foundation level

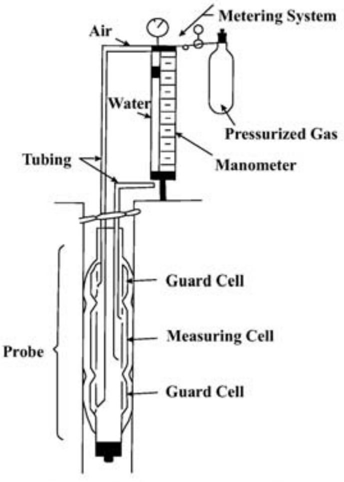
The calculations of settlements for a foundation are based upon the following formula:

$$w = \frac{1.33\,p(\lambda_2 R)^{\alpha}}{3E_B} + \frac{\alpha p \lambda_3 R}{4.5E_A}$$

Where P equals pressure transmitted to the soil by the foundation, E is the weighted pressuremeter modulus, R is the radius of the foundation, λ_2 and λ_3 are shape coefficients and α is the rheologic coefficient depending upon the type of soil.

The above equations are generally used in soil evaluation and interpretation, depending upon loading conditions, shape and size of the foundation, weaker compressible layers and other factors associated with the soil conditions.

This is intended to be a summary of the test interpretation procedures and references are included for details for these procedures.



Schematic Diagram Showing Pressuremeter in a Borehole

Figure 1

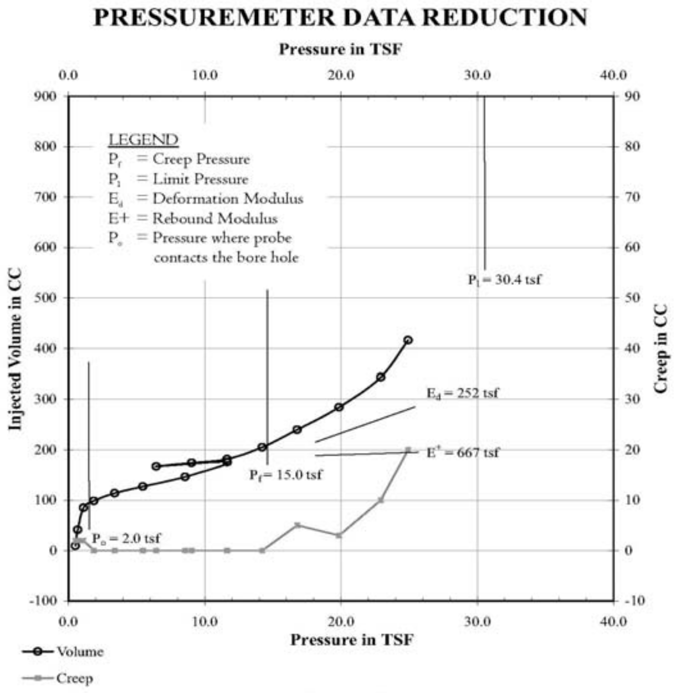
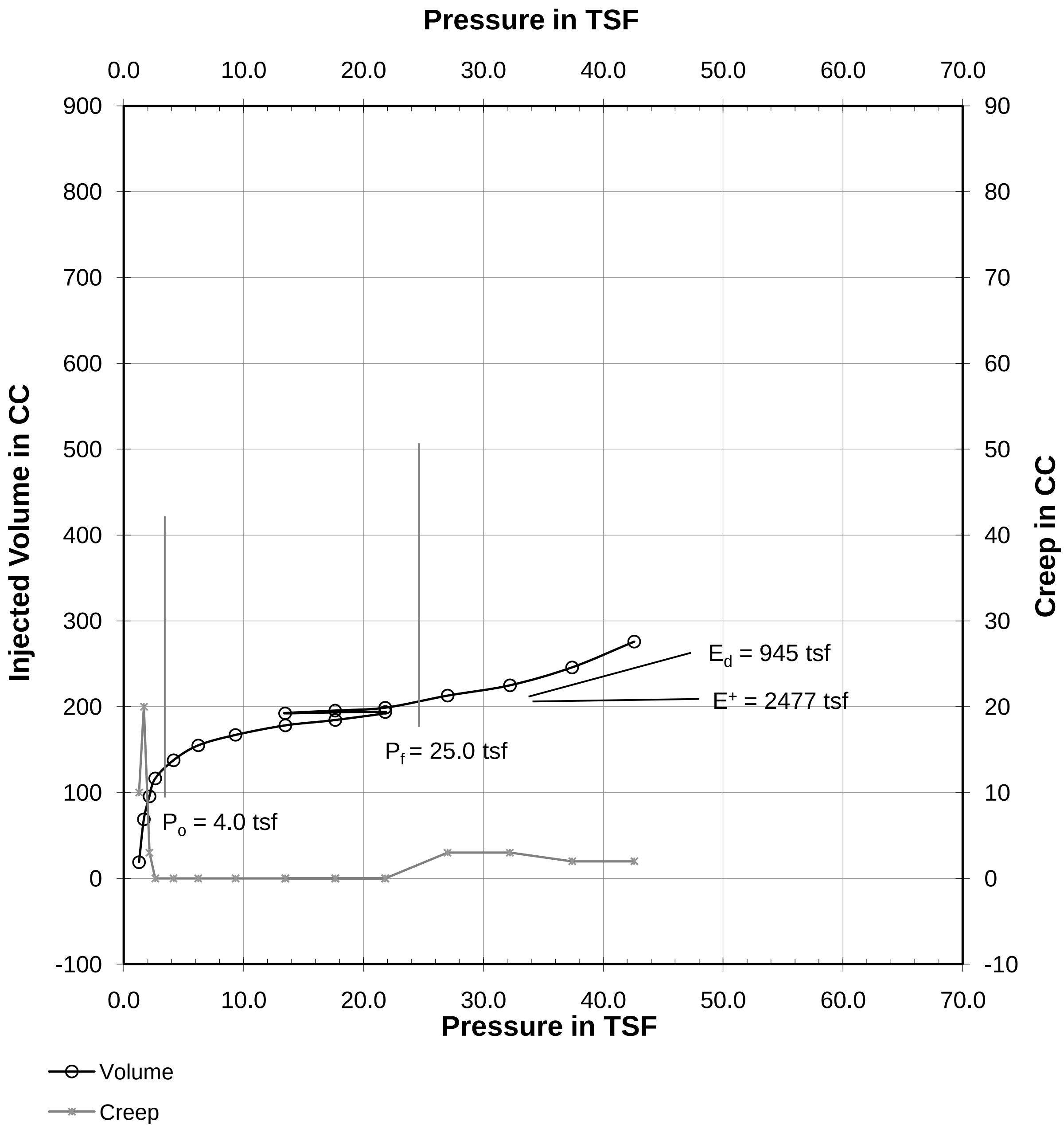


Figure 2

<u>List of References</u>

1. The Menard Pressuremeter – Investigation and Application of Pressuremeter Test Results – Sol – Soils 26, 1975.
2. Gibson, R.E. and Anderson, W.F., "In-Situ Measurement of Soil Properties with the Pressure-Meter," <u>Civil Engineering And Public Works Review</u>, London, May, 1981.
3. Goodman, R.E., Van, T.K. and Heuze, F.E., "The Measurement of Rock Deformability in Bore Holes," <u>10th Symposium on Rock Mechanics</u>.
4. Higgins, C.M., "Pressuremeter Correlation Study," <u>Highway Research Record No. 284</u>, Highway Research Board, 1969.
5. Menard, L., "The Application of the Pressuremeter for Investigation of Rock Masses," presented at the <u>1965 Colloquium of the International Society for Rock Mechanics</u> in Salzburg, Austria.
6. <u>Canadian Manual on Foundation Engineering</u>, (Draft for Public Comment), Issued by the Associate Committee on the National Building Code, National Research Council of Canada Ottawa, 1975.
7. Lukas, Robert G. and DeBussy, Bruno, "Pressuremeter and Laboratory Test Correlations for Clays," <u>ASCE Geotechnical Division, GTS</u>, September 1976.
8. Baguelin F., Jezequel J.F., Shields, D.H., "The pressuremeter and Foundation Engineering," <u>Trans Tech Publications</u>, 1978.
9. Lukas, Robert G. and Seiler, Norman H., "Experience with Menard Pressuremeter Testing," Engineering Foundation Conference, <u>Updating Subsurface Sampling of Soils and Rocks and Their In-Situ Testing</u>, January 1983.

AECOM Job Number: MadPkgRamp   Date: 09-09-10
Boring No.: 4
Test Depth: 35.5-38.0 Feet



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Boring No.: 4
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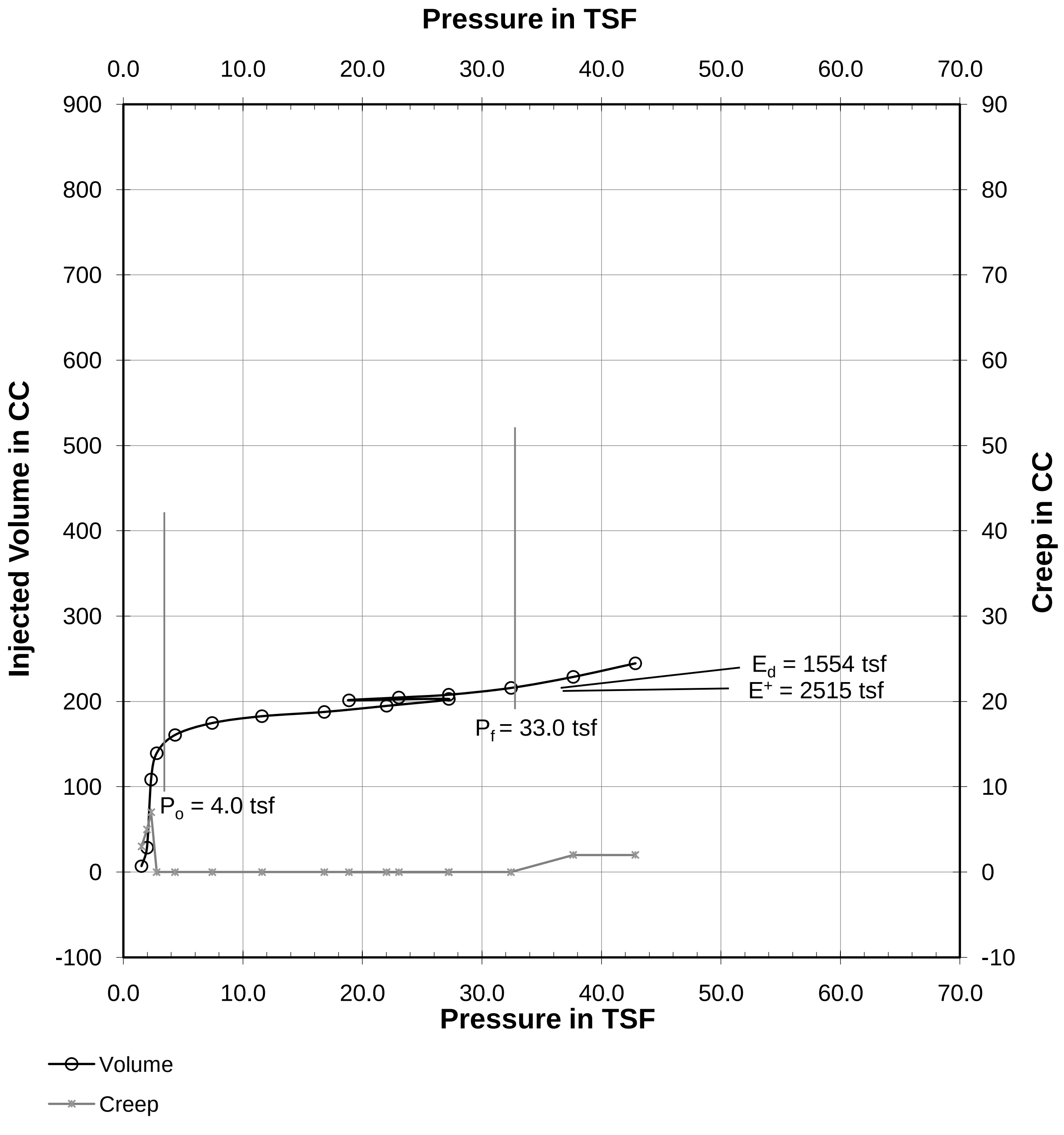

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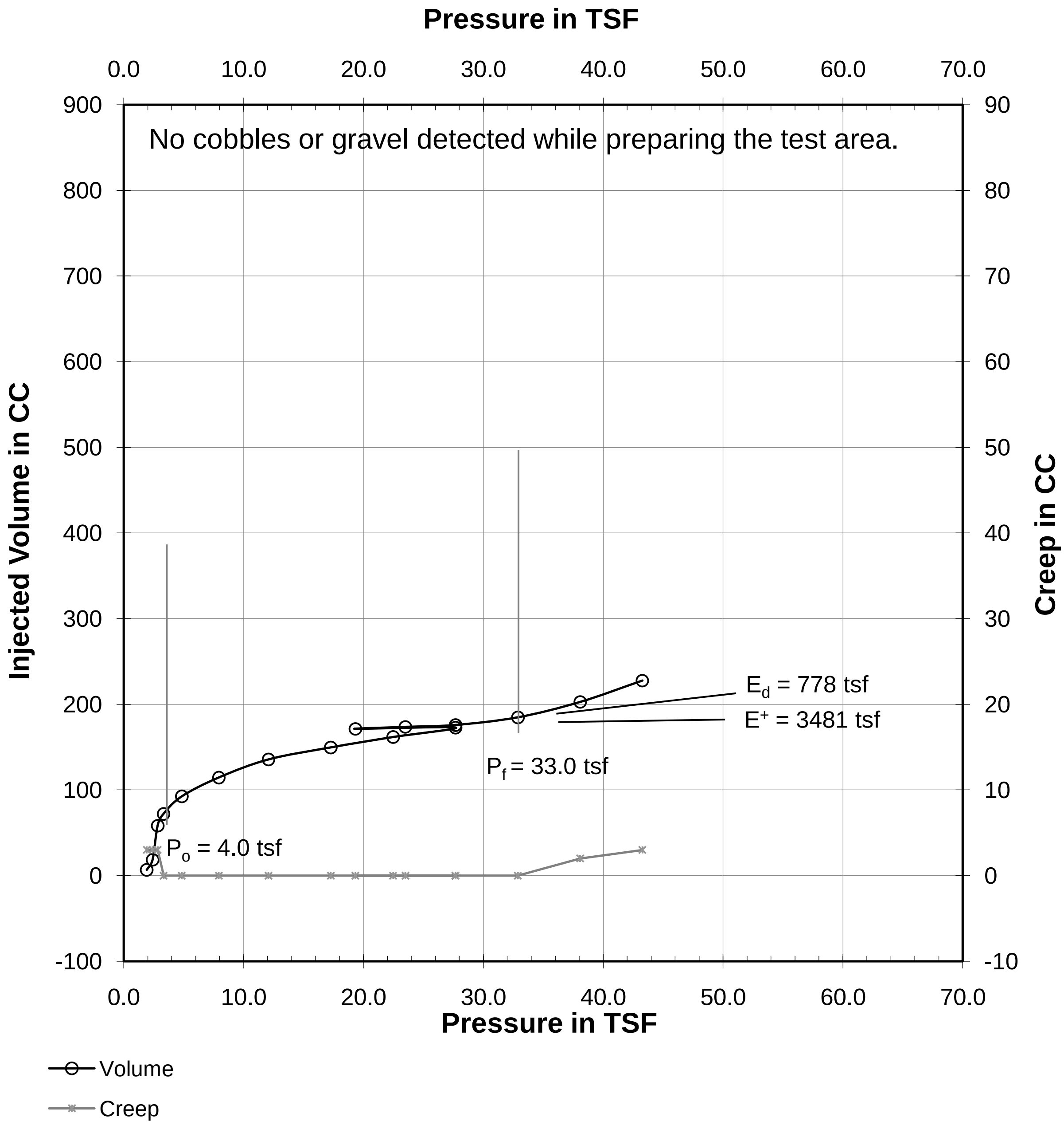
Test Depth: 51.0-53.5 Feet



AECOM

Pressuremeter Data Reduction (BX)

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Boring No.: 4
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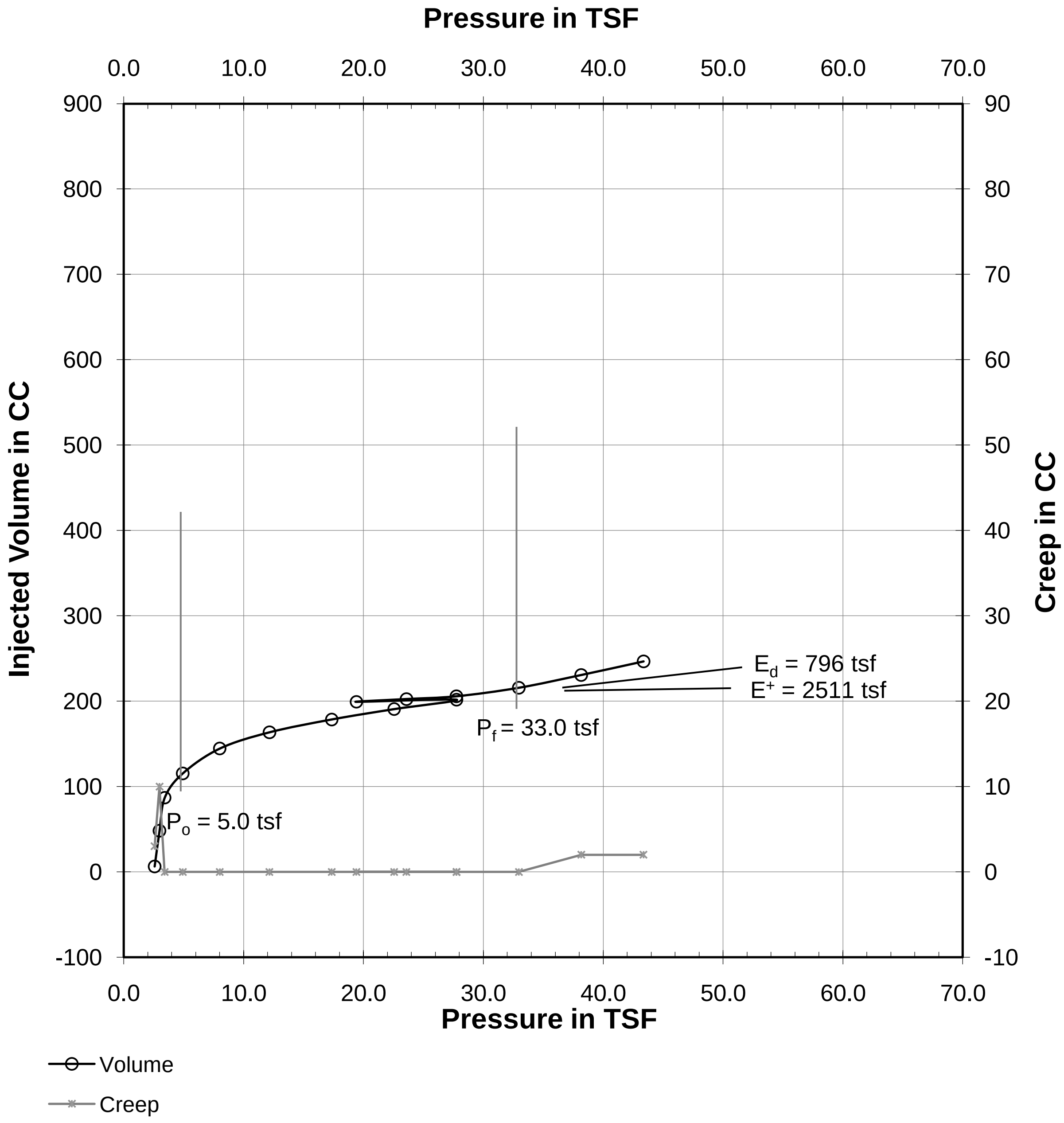

Pressuremeter Data Reduction (BX)

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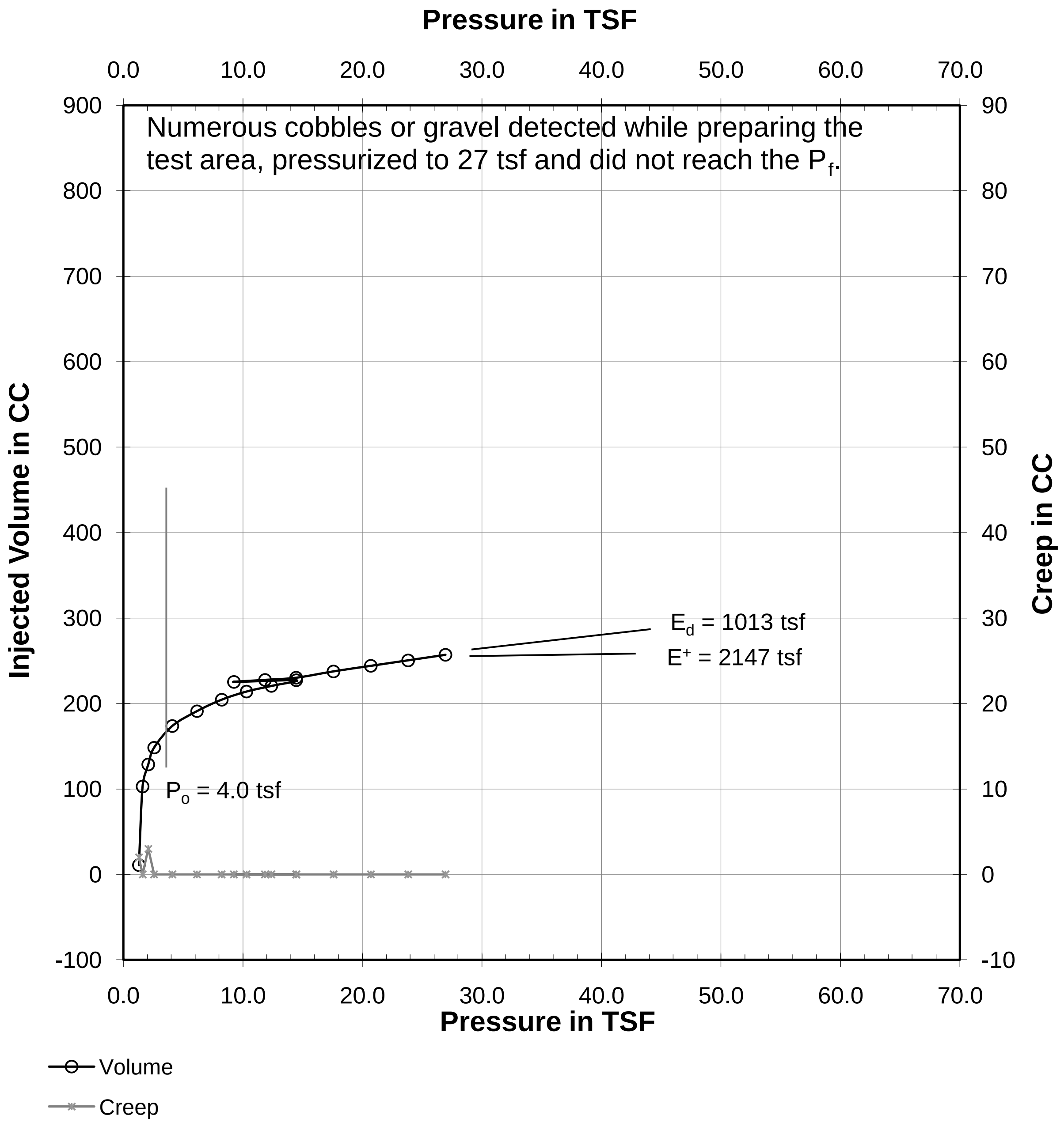
Boring No.: 5
Test Depth: 35.5-38.0 Feet



Pressuremeter Data Reduction (BX)

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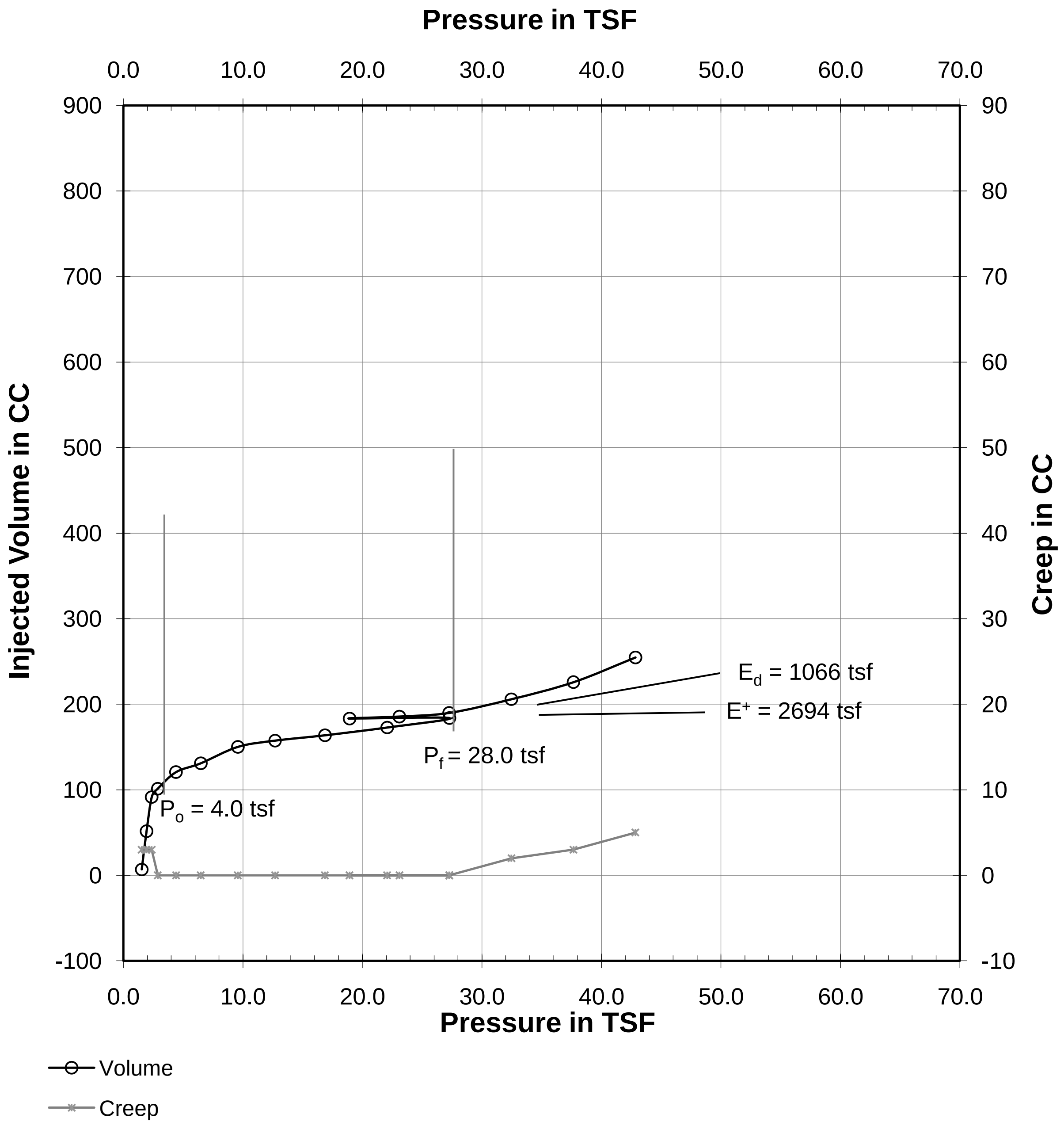
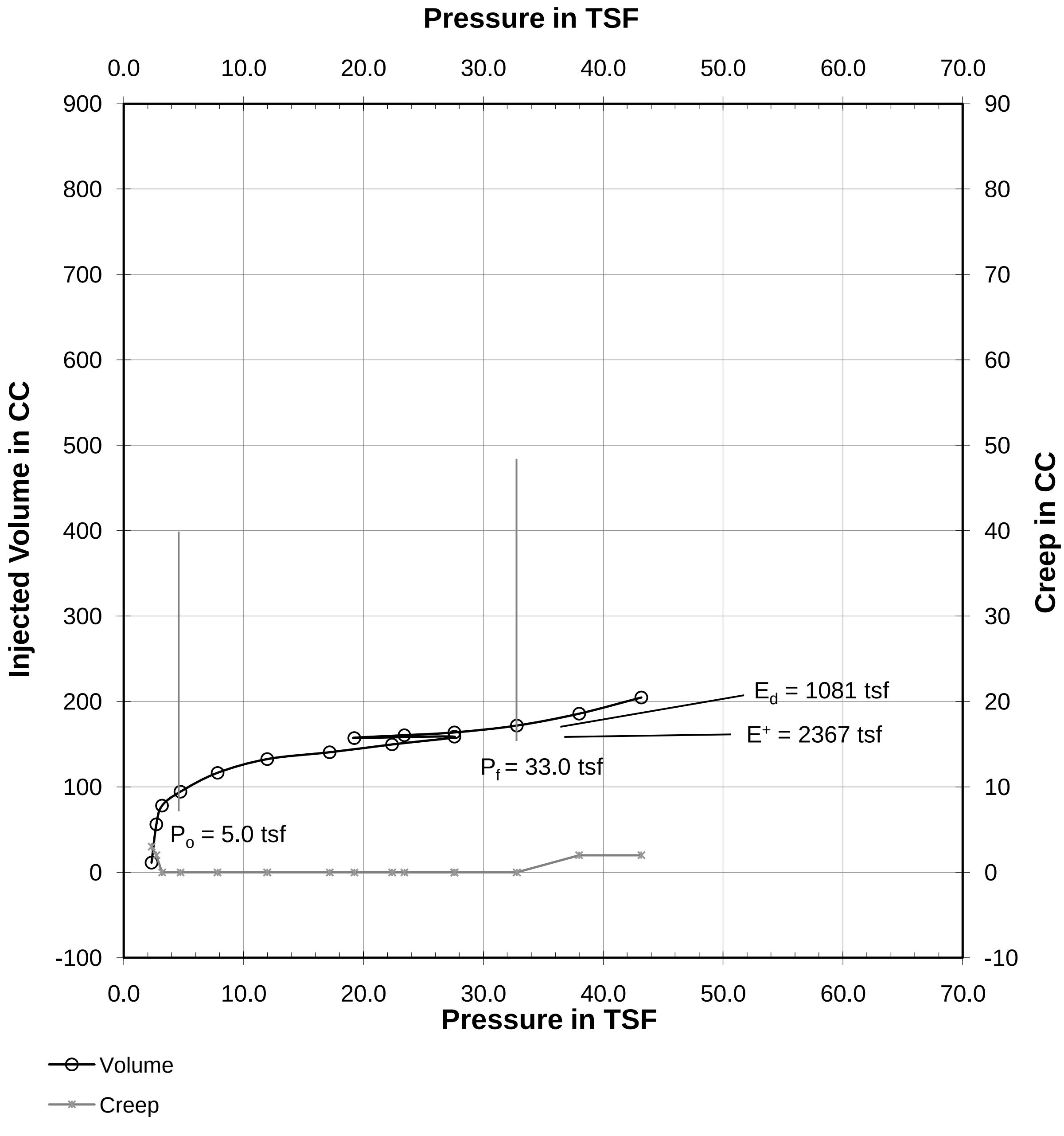
Boring No.: 5
Test Depth: 42.5-45.0 Feet



Pressuremeter Data Reduction (BX)

AECOM Job Number: MadPkgRamp Date: 09-08-10
Boring No.: 5
Test Depth: 51.5-54.0 Feet



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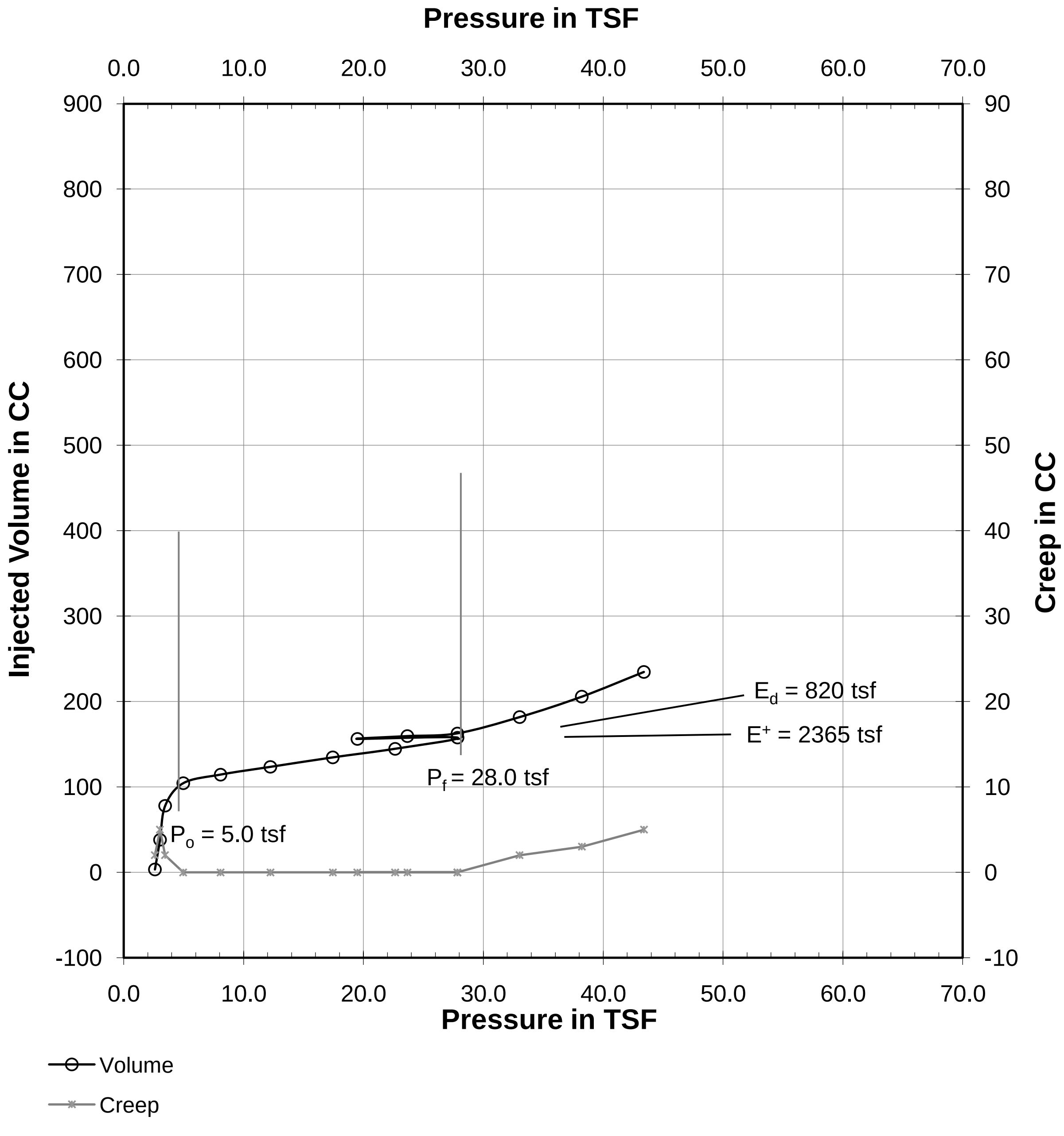
Boring No.: 5
Test Depth: 59.5-62.0 Feet



October 20, 2015

Mr. David A. Staab, P.E., LEED AP
CGC, Inc.
2921 Perry Street
Madison, Wisconsin 53713

Re: Judge Doyle Square Pressuremeter Testing – Madison, Wisconsin
WKG^2 Project No. 15024

Dear Mr. Staab:

At your request, Wagner Komurka Geotechnical Group, Inc. (“WKG^2”) arranged for in-situ pressuremeter tests to be performed for The Judge Doyle Square project in Madison, Wisconsin. The pressuremeter tests were performed to evaluate soil strength and compressibility to optimize allowable foundation bearing stress. The pressuremeter test results, and the results of our analysis using the pressuremeter test data, are presented in the attached report.

If you have any questions regarding this report, please feel free to contact us. We appreciate this opportunity to provide geotechnical engineering services to CGC, Inc.

Sincerely,

WAGNER KOMURKA
GEOTECHNICAL GROUP, INC.

Janine L. Grauvogl-Graham　　　Van E. Komurka

Janine L. Grauvogl-Graham, P.E.
Geotechnical Engineer

Van E. Komurka, P.E., D.GE, F.ASCE
President

WAGNER KOMURKA GEOTECHNICAL GROUP, INC. • W67 N222 Evergreen Boulevard • Suite 100 • Cedarburg, WI 53012
(262) 376-2001 • FAX: (262) 376-2002 • E-MAIL: geotek@wkg2.com

PRESSUREMETER ENGINEERING REPORT

JUDGE DOYLE SQUARE

MADISON, WISCONSIN

October 20, 2015

Prepared for:

CGC, Inc.

by:

Wagner Komurka Geotechnical Group, Inc.

WKG^2 Project No. 15024

JUDGE DOYLE SQUARE
MADISON, WISCONSIN
PRESSUREMETER ENGINEERING REPORT

PROJECT DESCRIPTION

The project involves construction of a new 8- to 10-story building in the block bordered by E. Doty Street, S. Pinckney Street, E. Wilson, and Martin Luther King Junior Boulevard in Madison, Wisconsin. The structure will have two to five below-grade levels. This report addresses the western portion of the building which will have two below-grade levels with column footings bearing at approximately Elevation 880[1]. We understand that the maximum column load is 1,800 kips.

A pressuremeter testing program was previously completed for the portion of the building with five below-grade levels and column footings bearing at approximately 855. The results of that pressuremeter program were presented in our previous report[2].

Existing site grades range from approximately Elevation 900 to 910, resulting in anticipated depths to footing subgrades, for this portion of the building, ranging from approximately 20 to 30feet.

PRESSUREMETER TESTING PROGRAM

The scope of Wagner Komurka Geotechnical Group, Inc.'s ("WKG^2's") services is limited to foundation-related design (bearing capacity assessment and settlement estimation). This was accomplished using data from soil borings drilled by Badger State Drilling, Inc. ("Badger") of Stoughton, Wisconsin, the logs of which were provided by CGC. Pressuremeter tests were arranged by WKG^2, and performed for CGC, Inc., by AECOM of Vernon Hills, Illinois, in Borings 2A and 4A. The borings for pressuremeter testing, and number of tests and test depths (elevations) in each boring, were selected by WKG^2. Pressuremeter test zones were prepared using a split-barrel sampler and specially sized roller bits and hand augers.

In a pressuremeter test, a cylindrical probe is inserted to the (then current) borehole bottom, and hydraulically expanded radially against the borehole sides. Probe volume versus pressure is recorded. A more-detailed description of pressuremeter testing is provided on the sheet titled "Pressuremeter Procedures" in the Appendix. Plots of probe volume versus pressure for each individual test are also included in the Appendix.

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[1] Unless indicated otherwise, elevations are positive, have units of feet, and are with respect to USGS Datum.

[2] Wagner Komurka Geotechnical Group, Inc., "Pressuremeter Engineering Report – City of Madison Parking Ramp – Madison, Wisconsin," WKG^2 Project No. 10020, prepared for Mr. David A. Staab, P.E. of CGC, Inc., September 23, 2010.

SOIL CONDITIONS

A review of CGC's boring logs indicates that the site is covered with either 5 inches of asphalt pavement over 5 inches of concrete, 5 inches of concrete pavement over 6 inches of basecourse, or 12 inches of topsoil fill. Fill, consisting of silty clay or silty sand, was present below the pavement section or surficial topsoil fill, and extended to depths ranging from 2.5 to 12.5 feet. Dense to very dense granular soils were typically encountered below the fill, and extended to the maximum depth explored (90 to 100 feet). Hard clay was encountered in Boring 6 at a depth of 91 feet, and extended to the maximum depth explored (100 feet).

The conditions encountered below anticipated spread-footing bearing elevations consist of very dense sand. More-detailed descriptions of the subsurface conditions are presented on CGC's logs in their report.

GROUNDWATER CONDITIONS

CGC measured the groundwater level in a monitoring well at approximately Elevation 863. CGC also noted perched groundwater seams at approximately Elevations 882 and 873. The upper perched groundwater seam is above the footing bearing elevation for this portion of the project. The groundwater level was not observed in the all of the borings due to the use of drilling mud to extend the borings.

Fluctuations in the water table should be expected with variations in precipitation, evapotranspiration, surface runoff, etc., and the water elevations in nearby surface waters. Development of perched groundwater may occur above the primary groundwater table, especially following precipitation events.

PRESSUREMETER TEST RESULTS

Pressuremeter test results for the proposed structure are presented in Table 1.

The at-rest pressure, P_o, represents the pressure at which the probe has expanded into firm contact with the borehole sides, and the pressure at which the plot of probe volume versus pressure becomes linear. The creep pressure, P_f, represents the pressure at which the plot ceases to be linear (i.e., the pressure at which deformations increase for a given incremental pressure increase). The limit pressure, P_l, is the pressure at which complete soil failure has occurred (i.e., the plot is vertical). The deformation modulus, E_d, is the slope of the initial linear portion of the plot. The rebound modulus, E^+, is the slope of the linear reload portion of the plot. The ratio E_d/E^+ (commonly referred to as the α parameter) is used, along with the deformation modulus, to estimate settlement.

TABLE 1

Pressuremeter Test Results Summary

Boring Number	Elevation, ft.	P_o, tsf	P_f, tsf	P_l, tsf	E_d, tsf	E^+, tsf	E_d/E^+
2A	879.1-881.6	2.0	--^{A}	--	1636	4454	0.37
2A	872.6-875.1	3.0	--^{A}	--	1401	2473	0.57
2A	866.6-869.1	3.0	--^{B}	--	1532	3147	0.49
2A	855.1-857.6	4.0	35.0	--^{C}	861	1923	0.45
4A	877.7-880.2	2.0	--^{A}	--	1493	5279	0.28
4A	872.7-875.2	2.0	--^{A}	--	1410	2639	0.53
4A	868.2-870.7	2.0	--^{D}	--	1342	3027	0.44
						Average	0.45

Notes: A: Pressurized test area to 42 tsf, P_f not reached.
B: Pressurized test area to 41 tsf, P_f not reached.
C: Pressurized test area to 41 tsf, P_l not reached.
D: Pressurized test area to 45 tsf, P_f not reached.

ANALYSES AND RECOMMENDATIONS

Foundation Recommendations

Allowable Bearing Stress

Based on lower-bound pressuremeter test program results, the maximum net allowable bearing stress for foundations bearing on native very dense sand is 50,000 psf. We anticipate that foundation performance will likely be affected by potential subgrade disturbance, and potential intersection of perched groundwater seams. Based on these factors and discussions with CGC, the recommended maximum net allowable bearing stress to be used for design is 30,000 psf. The recommended maximum net allowable bearing stress is the stress transmitted by the foundation to the soil in excess of the minimum final adjacent overburden stress. The recommended maximum net allowable bearing stress incorporates a minimum safety factor of 3.0 against bearing capacity failure.

The recommended maximum net allowable bearing stress is predicated on a square foundation with minimum foundation embedment depth of ¼ the footing width. Foundations with shallower embedment must be designed for a lower allowable bearing stress. Conversely, foundations with greater embedment could possibly be designed for a higher allowable bearing stress. For stability considerations, continuous wall and isolated column foundations should have minimum plan dimensions of 18 and 36 inches, respectively. This minimum width requirement may control the size of certain lightly loaded foundations. In this event, the actual soil/foundation contact stress will be less than the recommended maximum net allowable.

Settlement Estimates

Based on lower-bound pressuremeter testing soil deformation moduli, and the recommended maximum net allowable bearing stresses presented above, estimated foundation settlement is on the order of 1/3 to 1/2 inch for foundations bearing on native very dense sand.

Smaller, or more-lightly loaded, foundations are expected to experience proportionately less settlement. It is our opinion that differential settlement between similarly sized foundations will be on the order of half these foundations' total settlement.

Construction Considerations

We recommend that CGC provide observation and testing at the base of each foundation excavation to confirm that the soils are the same type as encountered by the subsurface exploration and pressuremeter testing program, and that the soils meet minimum strength criteria associated with the maximum net allowable bearing stresses presented above. Given the relatively high allowable bearing stress, the soil type, appropriate subgrade observation and testing is considered especially important. The subgrade soils should exhibit a minimum equivalent Standard Penetration Test ("SPT") blow count ("N" value) (comparable to the SPT hammer used) of 62 blows per foot.

Testing in granular soil should be performed using a device such as a dynamic cone penetrometer ("DCP"). DCP testing should extend a minimum of 2 feet below foundation subgrade, unless DCP refusal (greater than 50 blows per 6 inches) is encountered shallower.

If soils of the anticipated type or exhibiting the minimum required strength criteria are not found at the base of foundation excavations, it will be necessary to extend excavations deeper, or the affected foundations will have to be re-designed for a lower bearing stress. We recommend that any foundation excavations that must be extended below their design bearing elevation be backfilled to that elevation with cementitious backfill. The cementitious backfill could consist of lean concrete or controlled low-strength material ("CLSM"). The cementitious backfill should have a minimum 28-day compressive strength of 500 pounds per square inch (psi), and the backfill should be at least as wide as the foundation it supports.

Positive steps should be taken to limit subgrade disturbance from construction activities, groundwater inflows, precipitation, runoff, etc. Use of a lean mix concrete layer may be necessary to protect the subgrade soils from disturbance. Footing excavation should be completed with a flat-blade bucket to minimize subgrade soil disturbance. Precipitation should not be allowed to pond on subgrade soils. Subgrade soils which become disturbed or softened should be removed to suitable material and replaced with properly compacted engineered granular fill or CLSM.

All excavations deeper than 4 feet that will be entered by workers should have sloped or braced sidewalls that are consistent with OSHA guidelines for excavation safety.

GENERAL QUALIFICATIONS

The services provided by WKG^2 on this project were performed with the degree of skill and care typically performed by other members of our profession practicing in this locale at this time. No other warranty, expressed or implied, is given.

APPENDIX

- General Report Qualifications
- Pressuremeter Procedures
- Pressuremeter Test Results

General Report Qualifications

This geotechnical engineering report was prepared as part of the evaluation of the specific area covered by the soil borings, specifically for the project described in the report. The description of the project represents our understanding of the project. Should there be any changes in the concept of the project, its location, orientation, or elevation, we request that we be notified so that we may assess any impacts of the changes on our recommendations. The drawings and specifications for the project shall be submitted to WKG^2 for review of conformance with the recommendations contained in the report. Failure to submit the plans and specifications for this review relieves WKG^2 from any liability for failure to comply with our recommendations.

The recommendations presented in this report have been based on subsurface information obtained from soil samples at intervals in the soil borings which were drilled at the locations shown on the soil boring location diagram. The number of borings and the sampling intervals used are considered to be consistent with standards of the industry.

It should be recognized that variations in subsurface conditions can occur both between soil samples in a given boring, and between soil borings. Further, groundwater conditions should be expected to vary with time. The extent of the variations in subsurface conditions may not become apparent until construction begins. If variations in subsurface conditions become apparent, we request that we be notified so that we can observe the site conditions and evaluate how our recommendations may be affected.

We strongly recommend that all construction work related to geotechnical issues be monitored by an experienced geotechnical engineer or technician to determine if the subsurface conditions are as anticipated, and if the intent of our recommendations is met. We are available to provide the monitoring and testing services required during construction on this project.

Due to possible variation in subsurface conditions, we recommend that the Standard General Conditions of the construction contract prepared by the Engineers Joint Contract Documents Committee (1910-8-(Latest Edition)) be included in the contract with the general contractor and any subcontractors who will be involved in geotechnical issues on this project. We also advise incorporating a dispute resolution clause in the contract, based on non-binding mediation, to resolve any disputes among the parties involved with geotechnical issues on the project.

The services provided by WKG^2 on this project were performed with the degree of skill and care typically performed by other members of our profession practicing in this locale at this time. No other warranty, expressed or implied, is given.

Introduction

The pressuremeter is a soil testing device which measures stress-strain characteristics of soils in-situ. It is a portable piece of equipment consisting of three main components:

1. a cylindrical expanding probe which is inserted into a bore hole.
2. a pressure source for expanding the probe, and
3. a metering system.

A schematic drawing showing these components is shown in Figure 1.

Pressuremeter Test

The test consists of inserting the probe into the bore hole and expanding the probe against the side of the hole at measured intervals of time until failure of the soil is reached.

The pressurermeter can be used to test nearly all soil types, from loose sand or silt to hard cohesive or dense granular soils and soft rock. Tests can be performed in a drilled bore hole or hand augered hole at depths normally achieved by these methods of drilling. Tests can be performed above or below the water table. Special procedures or techniques including the use of a bore hole shaver have been developed to prepare the bore hole in squeezing or caving soils so that reliable test parameters are measured.

Using correlations with routine or special laboratory tests, a pressuremeter is a very useful geotechnical tool.

General Uses

The following is a summary of some of the applications of the pressuremeter investigation.

1. Determination of bearing capacity of pile or cassion type foundations,
2. Determination of bearing capacity for shallow foundations,
3. Estimates of foundation settlement.
4. Determination of soil shear strength.
5. Determination of horizontal subgrade modulus to predict horizontal movement under lateral loads for piles, sheet pile walls, cast-in-place concrete walls, and drilled piers.
6. Determination of the modulus of vertical subgrade reaction, and
7. Determining the improvement in soil properties following site densification.

Apparatus

The probe measures 2.5 inches in diameter, is 2 ft. 2 inches long, fits inside of a BX size casing, with the length of the center expanding cell of the probe measuring 7 inches. A liquid (water in summer and glycerin in winter) is used to expand the center cell of the probe and gas pressure, usually carbon dioxide, is used to expand the two end cells of the probe. When the probe is inserted into the soil and the cells are expanded, the top and bottom portions of the probe tend to seal off the bore hole while the volume change in the center portion is measured. By this method, a nearly plane stress, plane strain condition is set up on the soil. Volume changes in the center portion of the probe are measured versus the pressure increment. Six to fourteen load increments are used per test, each increment being applied to the soil for a 1 minute period. Readings are to be at 30 seconds and 60 seconds after head increment.

Interpretation of Test Results

The results of the pressuremeter tests are generally plotted as pressure versus volume change at 60 seconds for each pressure increment. A typical curve is shown in Figure 2. The interpretation of the test results is generally in conformance with Menard's Theory. The soil behavior generally follows two zones, pseudo-elastic and plastic. The elastic zone, in which strains are completely recoverable, is generally not noticed due to the bore hole disturbance. The lower limit of this elastic zone is defined as P_O. As pressures above P_O, the solid behaves as a pseudo-elastic material which is indicated as a straight line on the pressure verses probe volume curve. The strains occurring within this zone are not completely recoverable.

The upper limit of the pseudo-elastic zone is defined as P_F. At pressures greater than the value of P_F, creep deformation of the soil particles occurs as the pressure increases and eventually causes failure of the soil. The pressure at which the failure occurs is called the limit pressure, P_L and is related to the ultimate bearing capacity of the soil.

The pressuremeter modulus is calculated for the pseudo-elastic zone portion of the test. From classical soil mechanics principles in which soil anisotropy is often assumed, the vertical modulus may be significantly different from the horizontal modulus and one might expect erratic predictions of vertical settlement of footings. However, recent theoretical as well as full scale experimental studies have shown that in many situations this test still permits a much better prediction of foundation settlements predictions based on pressuremeter test results are presently the most reliable for granular materials and preconsolidated glacial tills.

General Equations

The analysis of the pressuremeter test is based upon the principles of theoretical soil mechanics. The parameters obtained from these tests have been correlated to parameters obtained from laboratory tests. The general equation for bearing capacity and settlement have been modified by and confirmed with numerous field tests including full scale load tests.

The bearing capacity of a foundation is derived from the following general equation:

$$q = P_V + k(P_L - P_O)$$

where q = Ultimate bearing capacity

P_O = Lateral pressure at rest of the soil at the elevation of the foundation element

P_L = Limit pressure of the soil

k = A coefficient depending upon soil type, geometric shape of the foundation, and depth of embedment.

P_V = Overburden pressure at foundation level

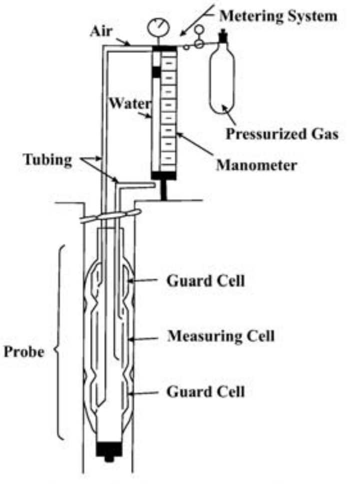
The calculations of settlements for a foundation are based upon the following formula:

$$w = \frac{1.33\, p(\lambda_2 R)^{\alpha}}{3E_B} + \frac{\alpha p \lambda_3 R}{4.5E_A}$$

Where P equals pressure transmitted to the soil by the foundation, E is the weighted pressuremeter modulus, R is the radius of the foundation, λ_2 and λ_3 are shape coefficients and α is the rheologic coefficient depending upon the type of soil.

The above equations are generally used in soil evaluation and interpretation, depending upon loading conditions, shape and size of the foundation, weaker compressible layers and other factors associated with the soil conditions.

This is intended to be a summary of the test interpretation procedures and references are included for details for these procedures.



Schematic Diagram Showing Pressuremeter in a Borehole

Figure 1

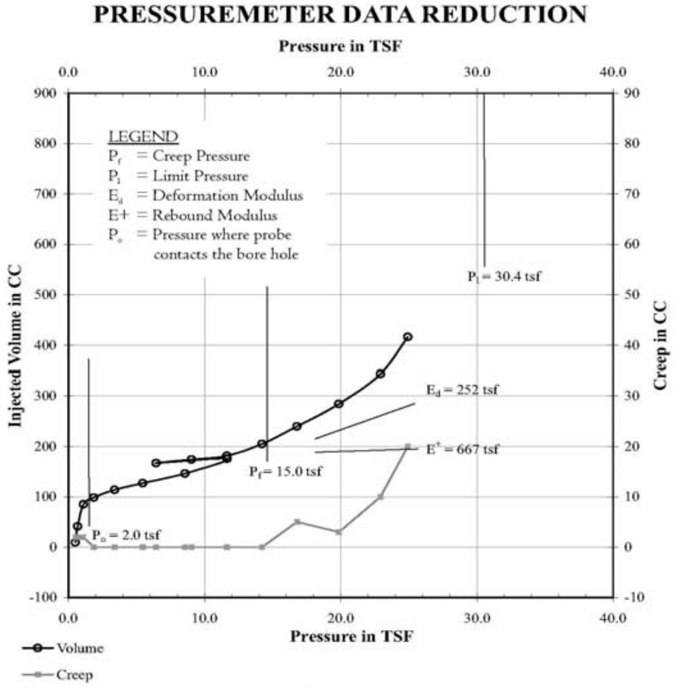


Figure 2

List of References

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2. Gibson, R.E. and Anderson, W.F., “In-Situ Measurement of Soil Properties with the Pressure-Meter,” Civil Engineering And Public Works Review, London, May, 1981.
3. Goodman, R.E., Van, T.K. and Heuze, F.E., “The Measurement of Rock Deformability in Bore Holes,” 10th Symposium on Rock Mechanics.
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5. Menard, L., “The Application of the Pressuremeter for Investigation of Rock Masses,” presented at the 1965 Colloquium of the International Society for Rock Mechanics in Salzburg, Austria.
6. Canadian Manual on Foundation Engineering, (Draft for Public Comment), Issued by the Associate Committee on the National Building Code, National Research Council of Canada Ottawa, 1975.
7. Lukas, Robert G. and DeBussy, Bruno, “Pressuremeter and Laboratory Test Correlations for Clays,” ASCE Geotechnical Division, GTS, September 1976.
8. Baguelin F., Jezequel J.F., Shields, D.H., “The pressuremeter and Foundation Engineering,” Trans Tech Publications, 1978.
9. Lukas, Robert G. and Seiler, Norman H., “Experience with Menard Pressuremeter Testing,” Engineering Foundation Conference, Updating Subsurface Sampling of Soils and Rocks and Their In-Situ Testing, January 1983.

Pressuremeter Data Reduction (BX)

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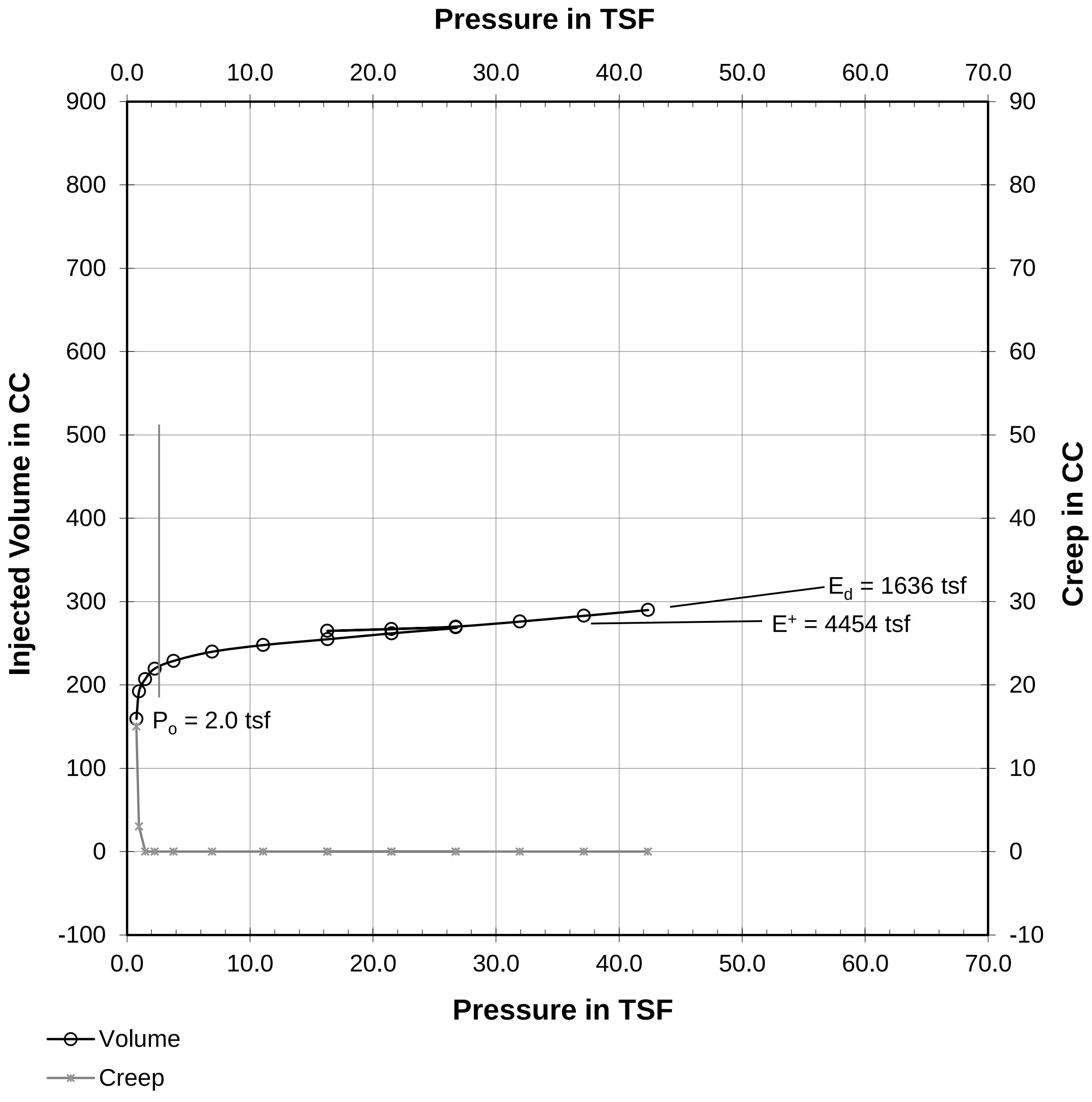
Boring No.: 2A
Test Depth: 28.5-31.0 Feet



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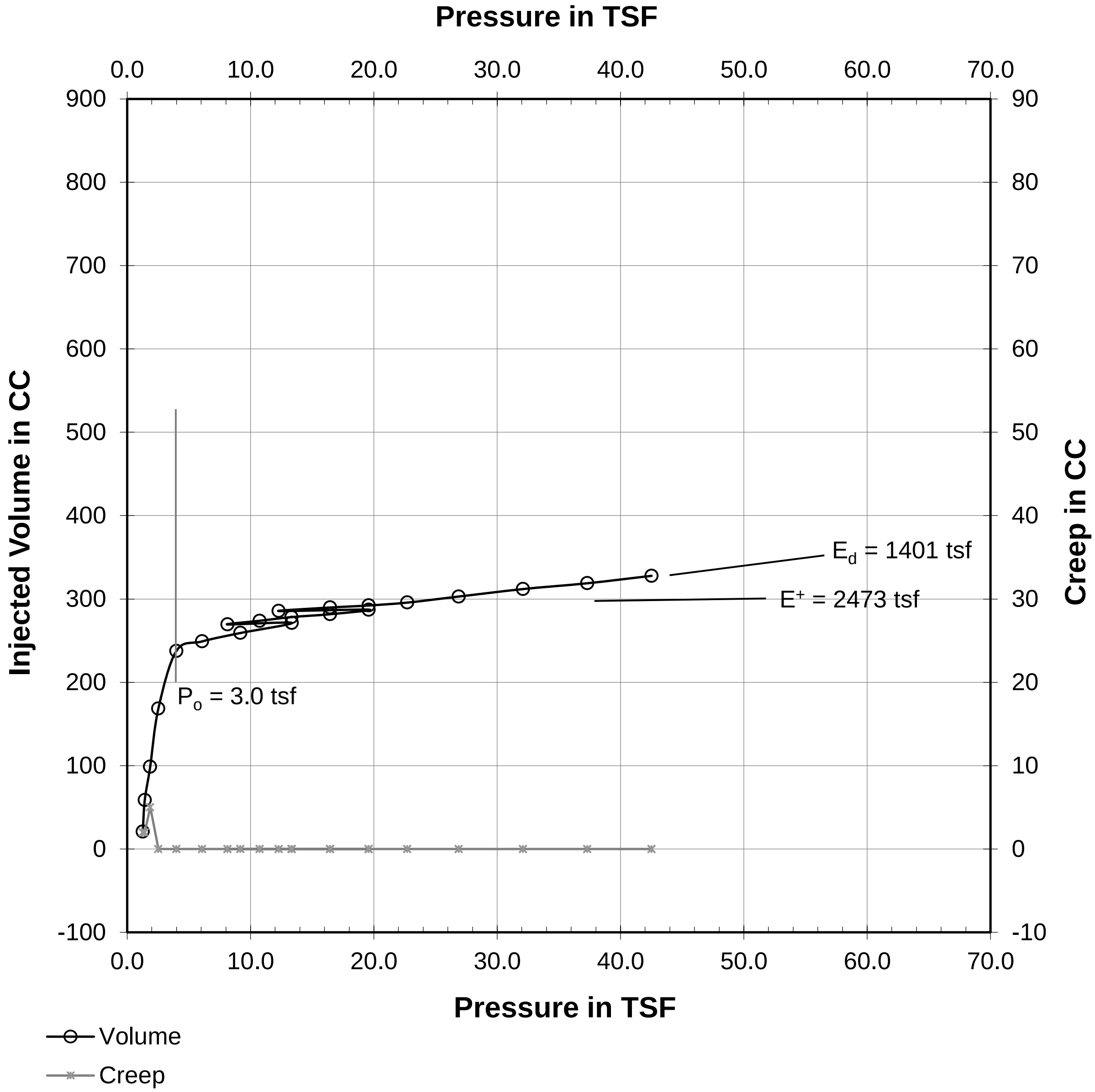

AECOM

Pressuremeter Data Reduction (BX)

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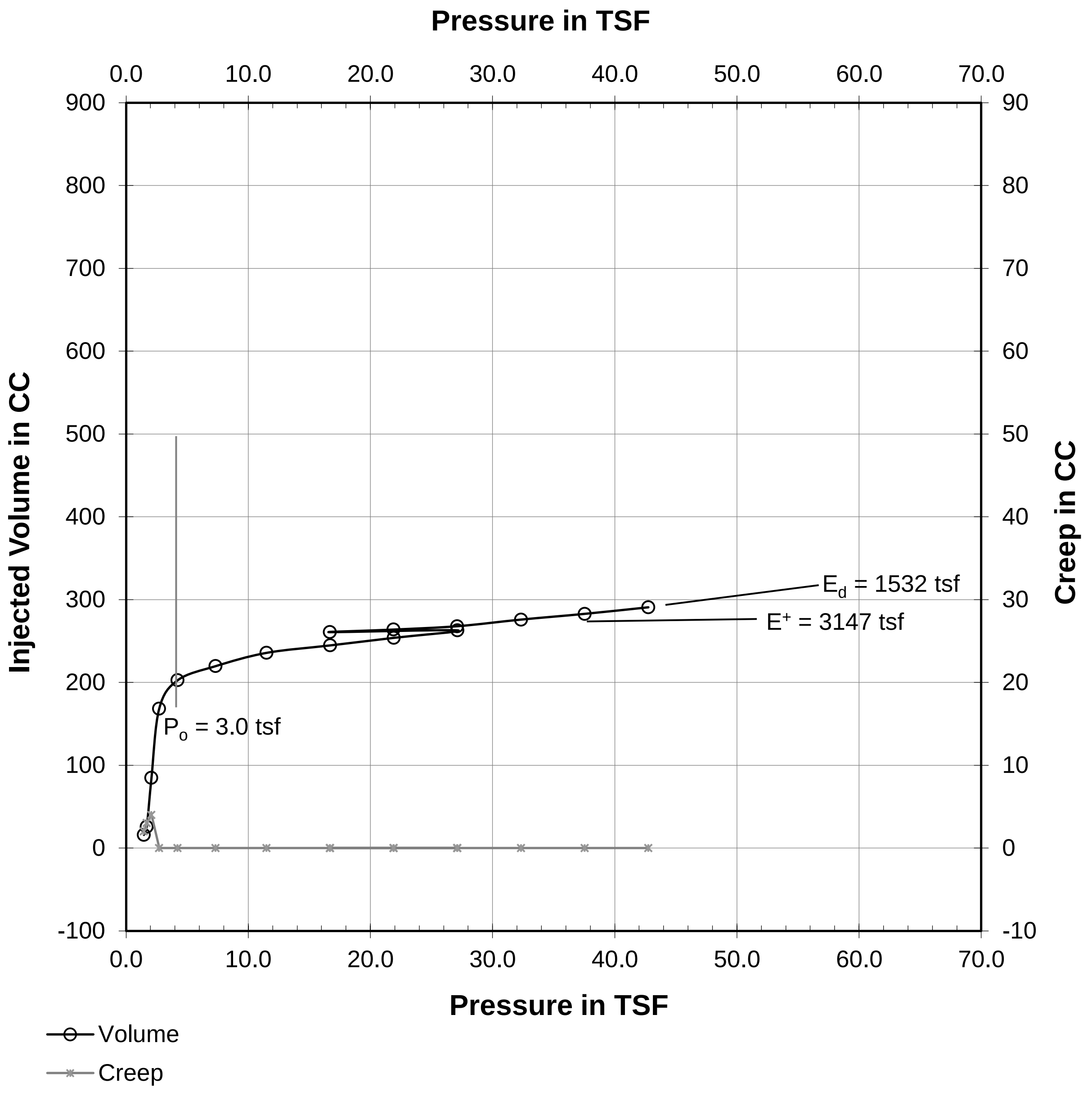
Boring No.: 2A
Test Depth: 41.0-43.5 Feet



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Pressuremeter Data Reduction (BX)

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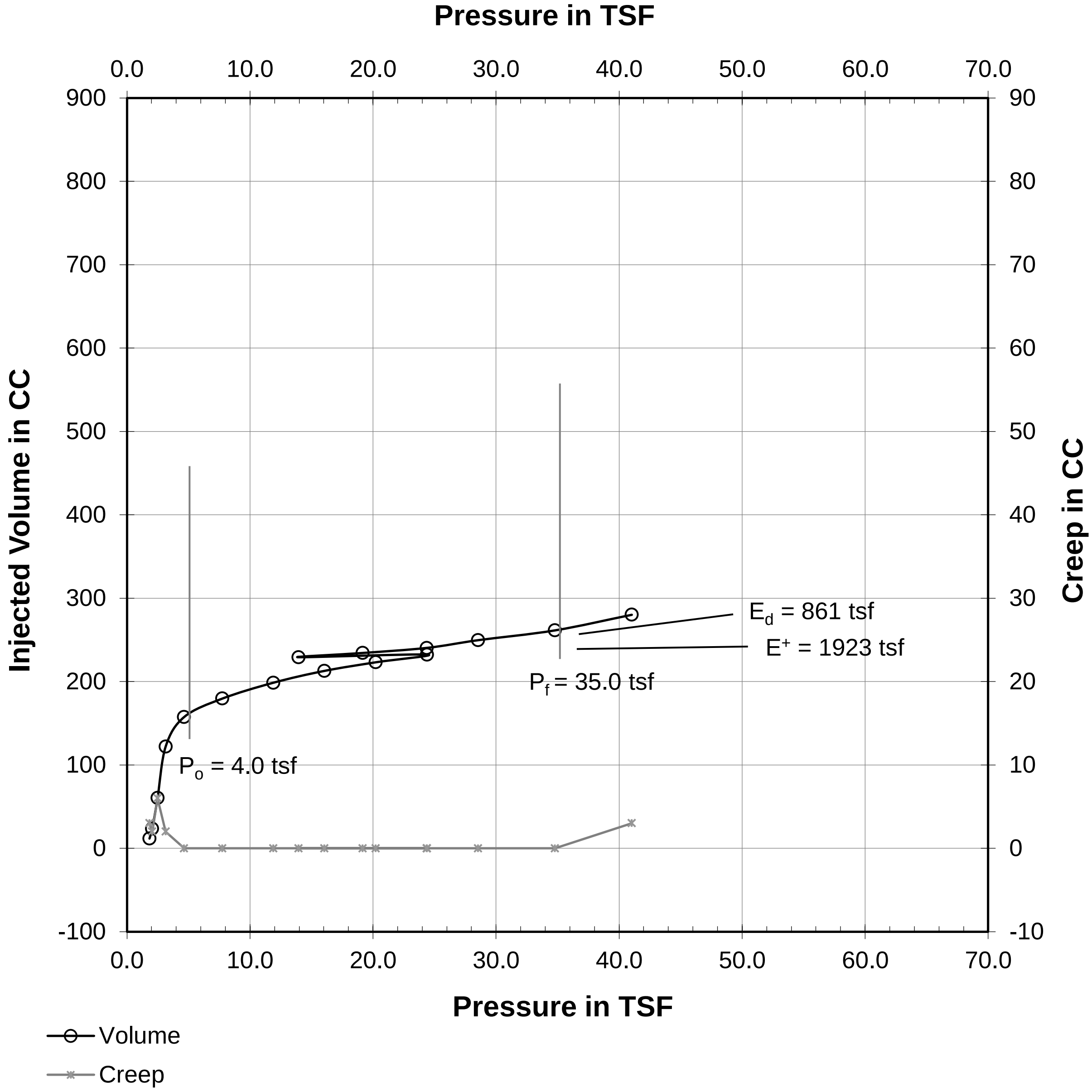
Test Depth: 52.5-55.0 Feet



Pressuremeter Data Reduction (BX)

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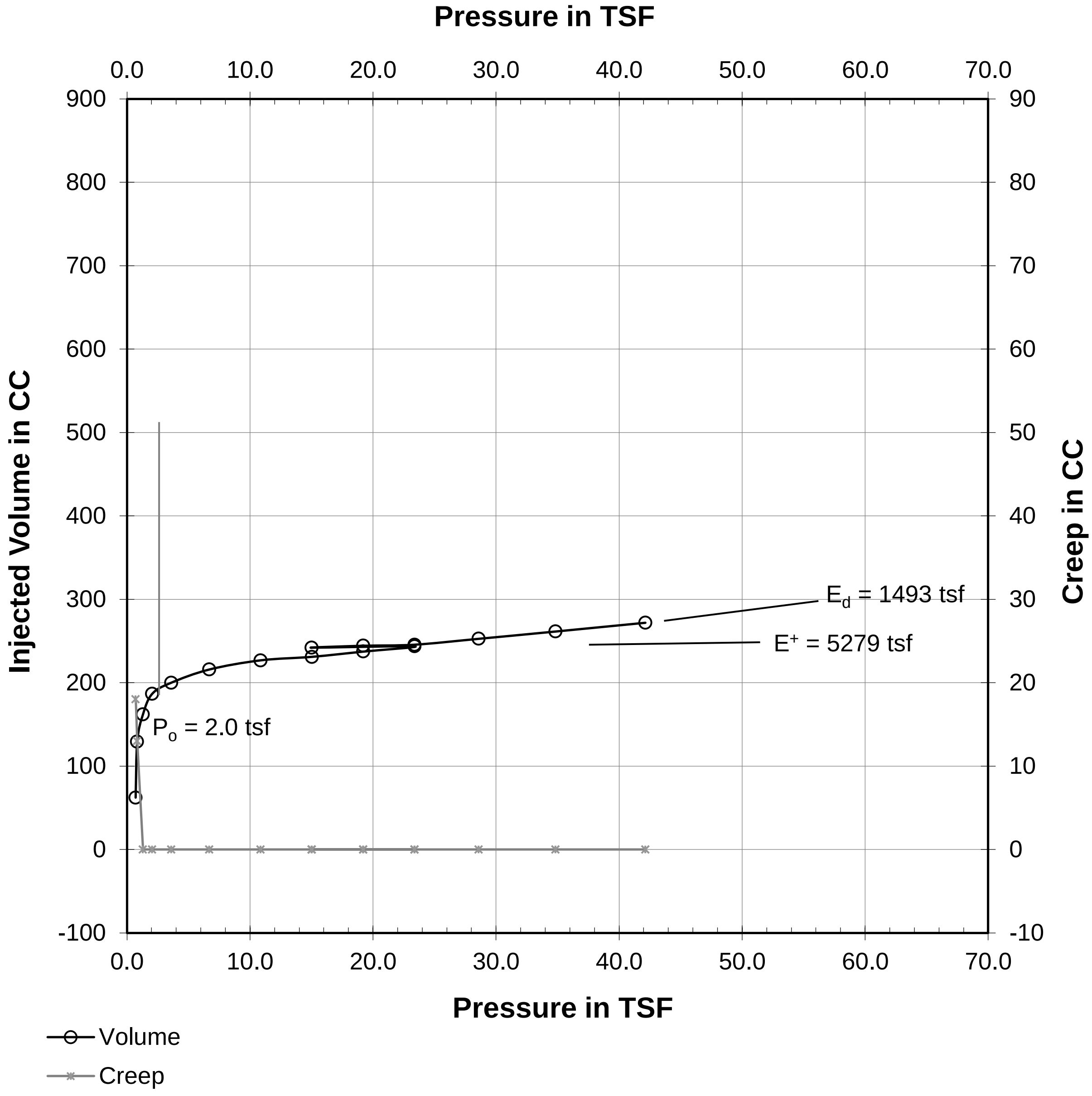
Boring No.: 4A
Test Depth: 20.5-23.0 Feet



AECOM

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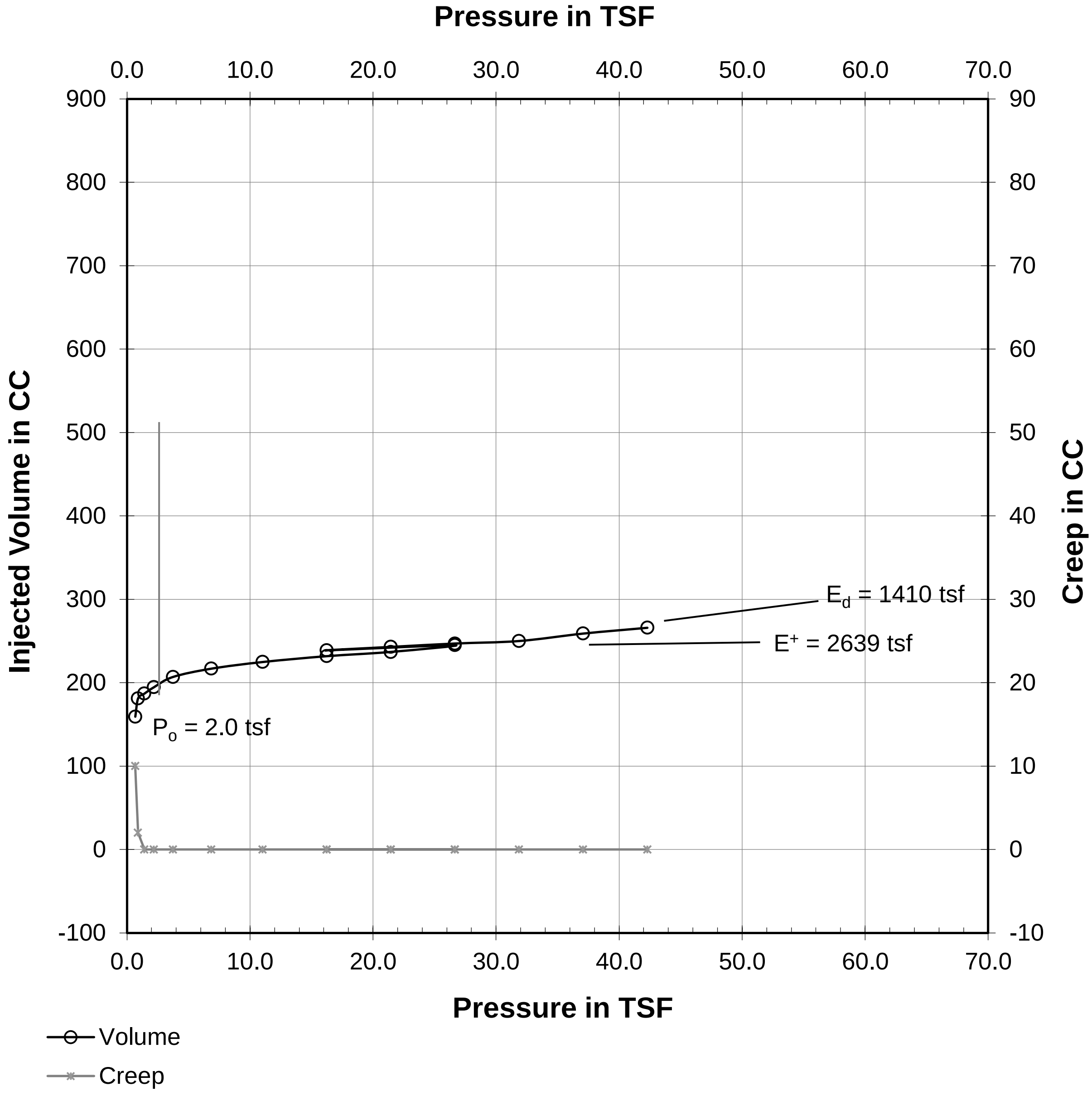
Boring No.: 4A
Test Depth: 25.5-28.0 Feet



Pressuremeter Data Reduction (BX)

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Date: 10-05-15
Boring No.: 4A
Test Depth: 30.0-32.5 Feet